Geotechnical Special Publication No. 226

Geotechnical Engineering State of the Art and Practice

Keynote Lectures from GeoCongress 2012

ASCE

EDITED BY
Kyle Rollins, Ph.D.
Dimitrios Zekkos, Ph.D., P.E.

G-I ASCE GEO-INSTITUTE

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GEOTECHNICAL ENGINEERING STATE OF THE ART AND PRACTICE

KEYNOTE LECTURES FROM GEOCONGRESS 2012

SPONSORED BY
The Geo-Institute of the American Society of Civil Engineers

EDITED BY
Kyle Rollins, Ph.D.
Dimitrios Zekkos, Ph.D., P.E.

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Preface

The Geo-Congress 2012 "State of the Art and Practice in Geotechnical Engineering" conference was held on March 25 – 29 2012 in Oakland, California, and was organized by the Geo-Institute of ASCE.

This event served as the annual Geo-Institute event of the ASCE, but also had a broader scope and different conference program. Geo-Congress 2012 aimed to provide a forum for the interaction among geotechnical engineers and the integration of research and practice. The conference aimed to attract an increased number of geotechnical engineers with emphasis in engineering practice. The program included the honorary Terzaghi Lecture, Seed Lecture, and Peck Lectures, technical sessions, panel sessions as well as a large range of student activities, half-day and full-day short courses, the United States Universities Council on Geotechnical Education and Research (USUCGER) bi-annual workshop as well as a US-Russia Workshop. A major highlight of the conference, and a deviation from previous Geo-Congress programs, was the 16 keynote State of the Art and 14 keynote State of the Practice Lectures. The topics and keynote lecturers were selected primarily based on the recommendations of the Geo-Institute Technical Committees. State of the Art Keynote lecturers were primarily researchers/academicians, while the State of Practice lecturers were primarily practicing engineers. The value of the conference was significantly enhanced by the contribution of time and effort provided by these excellent keynote speakers. During the conference, two keynote lectures were held concurrently on different topics so that each attendee could attend at least half of the lectures.

The conference proceedings include:

A printed volume published as Geotechnical Special Publication #226 with the Keynote State of the Art and State of the Practice Lectures and edited by Prof. Kyle Rollins and Prof. Dimitrios Zekkos. All keynote lecturers submitted a keynote paper. Each paper was anonymously reviewed by at least two reviewers selected by the Geo-Institute Technical Committees.

A cd-rom was published as Geotechnical Special Publication #225, with technical papers spanning the entire range of geotechnical engineering practice and edited by Prof. Roman Hryciw (Chair), Prof. Nazli Yesiller and Prof. Adda Athanasopoulos-Zekkos. These papers were presented orally or in posters during the conference. Each paper published in this ASCE Geotechnical Special Publication (GSP) received positive reviews from at least two peer-reviewers.

It is envisioned that this conference theme and format (with any necessary modification) will be recurrent every decade allowing for improved interaction among geotechnical engineers in practice and research as well as providing an opportunity to summarize the State of the Art and the State of the Practice of our profession.

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Acknowledgments

We express our sincere appreciation to the Geo-Institute Technical Committees who nominated State of the Art and State of the Practice Speakers and topics, then coordinated the review of the papers. We also thank those committee members who conducted the reviews, but whose names are not listed to preserve their anonymity. Lastly, we are indebted to Helen Cook and Robert Schweinfurth for their continual guidance, patience and insight throughout the organization of this congress.

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State of the Art

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Designing Water Balance Covers For Sustainable Waste Containment: Transitioning State-of-the-Art to State-of-the-Practice

By Craig H. Benson[1] and Christopher A. Bareither[2]

[1] Wisconsin Distinguished Professor and Director of Sustainability Research and Education, University of Wisconsin-Madison, Madison, WI USA 53706 USA, chbenson@wisc.edu
[2] Research Associate, Geological Engineering, University of Wisconsin-Madison, Madison, WI USA 53706 USA, bareither@wisc.edu

ABSTRACT

This paper describes key steps in the design process for water balance covers using a case history at a municipal solid waste landfill in Missoula, Montana, USA as an example. The intent is to illustrate how state-of-the-art concepts can be applied in the state-of-the-practice. The process begins by understanding the design objective (including regulatory requirements) and investigating lines of evidence indicating that a water balance cover is likely to function satisfactorily at the design location. Data from two other instrumented water balance covers in the region are used to evaluate efficacy along with historical meteorological data and information contained in a prior unsuccessful submittal for a water balance cover at the Missoula landfill. Site characterization is conducted to define properties of the soil resources and vegetation at the site for preliminary cover sizing and numerical modeling of the water balance. A numerical model is used with various design metrological conditions as input to evaluate whether the cover will meet the design goal under realistic conditions. The final design consists of a monolithic cover comprised of a 1.22-m storage layer overlain by a 0.15-m topsoil layer. A field test with a fully instrumented lysimeter was constructed and monitored to confirm that the cover performs as anticipated in the design.

INTRODUCTION

Final covers for waste containment that rely on principles of variably saturated flow to control percolation into underlying waste have become accepted as a viable methodology for long-term isolation of waste, particularly in semi-arid and arid regions where precipitation is favorably balanced by the energy available for evaporation (Khire et al. 2000, Zornberg et al. 2003, Albright et al. 2004, Malusis and Benson 2006). These covers are referred to using various names, including water balance covers, store-and-release covers, evapotranspirative (or "ET") covers, and alternative covers. The nomenclature "water balance cover" is used by the authors because this term represents the basic principle on which these covers function – the ability to balance storage of water corresponding to an acceptable level of percolation with the ability of plants and the atmosphere to remove stored water and replenish the water storage capacity of the cover profile. The authors specifically do not use the "ET cover" nomenclature because evapotranspiration is the predominant component of the water balance in nearly all cover systems, and thus is not particularly descriptive. The balance between water balance quantities is the feature that makes water balance covers unique.

Monolithic barriers and capillary barriers are the most common forms of water balance covers (Benson 2001, Ogorzalek et al. 2007, Bohnhoff et al. 2009) (Fig. 1). Monolithic barriers consist of a thick layer of fine-textured soil that is engineered and placed with appropriate compaction specifications so that the cover stores infiltrating water with little drainage while unsaturated. Capillary barriers generally consist of a two-layer system comprised of an overlying fine-textured storage layer similar to a monolithic cover that is underlain by a clean coarse-grained layer that provides a contrast in unsaturated hydraulic properties. The "capillary break" formed at the interface of the two layers enhances the storage capacity of the fine-textured layer (Stormont and Morris 1998, Khire et al. 2000), and can also promote lateral diversion of water in the fine-textured layer (Stormont 1995). For both types of covers, thickness of the storage layer is selected to have adequate capacity to store infiltrating water during the wet season while ensuring the cover meets the design percolation rate (Benson 2001). A surface layer of topsoil normally is placed over the storage layer of either type of cover to provide a hospitable environment for the plant community. Storage capacity of the topsoil layer generally is ignored during design.

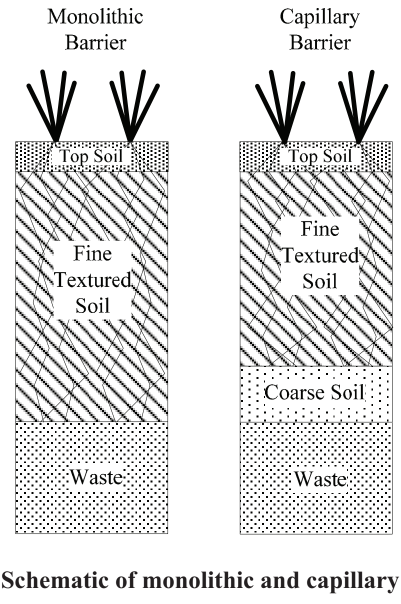

FIG. 1. Schematic of monolithic and capillary barriers.

Water balance covers are designed to be compliant with natural hydrologic conditions and to rely on hydrologic processes comparable to those in the surrounding landscape. Natural hydrologic controls are employed in lieu of engineered hydraulic barriers because, in most cases, final covers must function for decades to centuries, and in some case millennia. A system engineered to be compliant with nature is more likely to function over these long periods compared to a system that employs engineered hydraulic barriers not commonly found in nature. However, because natural hydrologic controls are employed, water balance covers may not be appropriate for all climates (e.g., controlling percolation to minute quantities in a humid region with high precipitation may not be practical). Understanding this limitation is important, and the engineer must resist the temptation to force-fit water balance covers into applications where they are not appropriate.

Sustainability is an intrinsic principle in water balance covers. These covers employ natural hydrologic processes congruent with the surrounding landscape, which reduces long-term maintenance requirements. On-site materials (soils and plants) are employed, which minimizes transportation requirements, and straightforward construction methods are employed that can be implemented by local personnel. As a result, energy consumption and emissions are reduced, natural resource consumption is limited, and local economies are supported. In the current regulatory climate, a tacit assumption is made that the waste being contained is stabilized and has minimal or no value as a resource. This assumption may not be realistic and requires examination in the context of sustainability.

Over the last two decades, the senior author (Benson) has been intimately involved in research focused on exploring the mechanisms controlling performance of water balance covers, developing measurement methods to characterize the engineering properties needed for design (e.g., ASTM D 6093 and D 6836, Suwansawat and Benson 1998, Khire et al. 1995, Meerdink et al. 1995, Albrecht et al. 2003, Wang and Benson 2004, Benson et al. 2007a, 2007b, 2011a, Schlicht et al. 2010), developing methods and models for sizing covers and predicting performance (e.g., Khire et al. 1997, 1999, 2000; Benson and Chen 2003; Shackelford and Benson 2006; Albright et al. 2010; Ogorzalek et al. 2007; Benson 2007, 2010; Bohnhoff et al. 2009; Smesrud et al. 2012), and defining methods to confirm field performance (e.g., Benson et al. 1994, 1999, 2001, 2011b; Kim and Benson 2002; Benson and Wang 2006, Waugh et al. 2008, 2009). Connecting theory and practice as well as coupling bench-scale to field-scale have been threads throughout this research program. This research effort has been sponsored by a broad set of stakeholders concerned with long-term waste containment, including the National Science Foundation, the US Environmental Protection Agency's (USEPA) Alternative Cover Assessment Program (ACAP), the US Department of Energy's (DOE) Environmental Management (EM) and Legacy Management (LM) programs, and the US Nuclear Regulatory Commission. Dr. William Albright (Desert Research Institute) and Dr. William "Jody" Waugh (Stoller Corporation and DOE-LM) have been collaborators in this effort since 1999.

In 2008, USEPA commissioned a guidance document summarizing the knowledge gained from these two decades of research, development, and practice. This document evolved into the book *Water Balance Covers for Waste Containment:*

Principles and Practice, by Albright, Benson, and Waugh, which was published by ASCE Press in 2010 (Albright et al. 2010). The objective of the guidance document and book was to facilitate the transition from state-of-the-art to state-of-the-practice. Practitioners and environmental regulatory agencies in the US and abroad have adopted the principles and strategies described in this book.

A case history is described in this paper where state-of-the-art principles described in the book were employed in the state-of-the-practice to evaluate, design, and demonstrate the viability of a water balance cover for an operating municipal solid waste (MSW) landfill in Missoula, Montana. While this case study applies to MSW containment in a semi-arid climate, the principles are universal and can be (and have been) adapted to design covers for other types of waste and in other climates. The discussion contained herein is brief to meet publication constraints; more detailed discussion of each of the issues is covered in the book. Moreover, the extensive citations common in academic scholarship have been forgone. The book serves as the primary reference, and within the book numerous citations are included.

PROCESS

The procedure for designing and evaluating a water balance cover consists of five steps, which can be summarized as follows (Albright et al. 2010):

1. **Preliminary assessment** – determine the performance goal and seek lines of evidence that a water balance cover may be successful at the proposed location. Understand the expectations of the overseeing regulatory authority and constraints required by the owner.
2. **Site characterization** – characterize the soils and vegetation available for the water balance cover.
3. **Storage assessment** – estimate the required thickness of the water balance cover by determining the amount of water that must be stored and the capacity of the cover to store the water.
4. **Water balance modeling** – predict the performance of the cover identified in the storage assessment for realistic meteorological data using a numerical model that simulates variably saturated flow and root water uptake in a multilayer system with a climatic flux boundary at the surface; refine the cover thickness if necessary.
5. **Performance demonstration** – conduct a performance demonstration to validate that the design meets the performance goal by instrumenting the actual cover or constructing a full-scale test section.

Each of these steps was conducted when evaluating, designing, and demonstrating the water balance cover for the site in Missoula.

PRELIMINARY ASSESSMENT

The preliminary assessment addresses two fundamental questions:

- What is the design goal for the project?
- What evidence exists that the design goal can be achieved with the climate, soils, and vegetation at the site?

Both of these questions need to be addressed during the preliminary assessment. The first question seems obvious, but is often overlooked until the project is far along. The second question is particularly important. If strong evidence is not available indicating a water balance cover will be successful, the engineer must carefully consider whether the design process should continue.

Discussions with the Montana Department of Environmental Quality (MDEQ, the regulatory agency with jurisdiction over the site), and review of MDEQ's *Draft Alternative Final Cover Guidance* (v. 9-11), indicated that the water balance cover for the Missoula landfill must be hydraulically equivalent to the conventional cover required in Montana for MSW landfill cells containing a composite liner. The conventional cover consists of a composite barrier with a compacted soil barrier having a saturated hydraulic conductivity no more than 10^{-5} cm/s overlain by a geomembrane and a vegetated surface layer (Fig. 2).

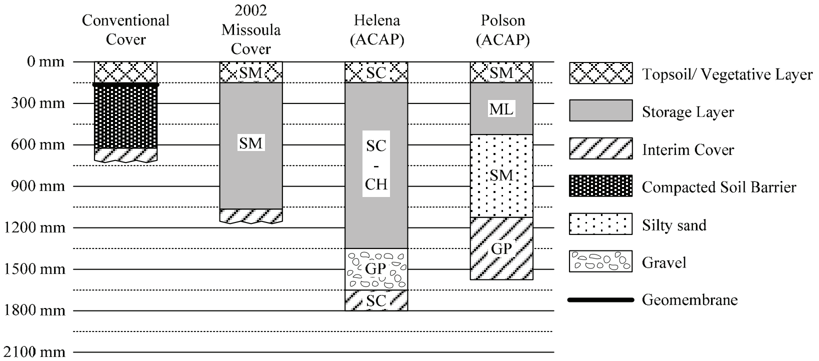

FIG. 2. Cover profiles at Missoula (conventional cover and 2002 Missoula Cover) and water balance cover profiles in Polson and Helena, MT. Soil classification based on Unified Soil Classification System.

MDEQ does not stipulate statewide equivalent percolation rates for conventional covers. Owners are required to propose an equivalent percolation rate for consideration and possible concurrence by MDEQ. For this project, the design percolation rate recommended by ACAP for conventional composite covers (3 mm/yr average percolation rate, Benson 2001) was proposed, and accepted by MDEQ as the design goal. The rate is reported in units of length/time, which corresponds to units of volume/time per area. The equivalency criterion for this project is similar to, but slightly less than the recommendation in Apiwantragoon (2007) for covers with composite barriers (4 mm/yr). North Dakota also stipulates 4 mm/yr for an equivalent percolation rate.

Supporting Evidence

Evidence was sought to determine if a design percolation rate of 3 mm/yr was realistic for the Missoula landfill. Field data from other projects in similar climates and with similar soils and vegetation generally comprise the best evidence. In this case, water balance covers had been evaluated at MSW landfills in Polson and Helena, Montana as part of ACAP (Albright et al. 2004, Apiwantragoon 2007). These sites are 115 km north (Polson) and 185 km east (Helena) of Missoula. Index and hydraulic properties of the cover soils were available for both sites (Benson et al. 2011a) as well as data from an ACAP-style lysimeter (Apiwantragoon 2007) used to characterize the water balance and verify the percolation rate. At both landfills, the water balance cover evaluated with the ACAP lysimeter was deployed as final cover.

Profiles of the water balance covers evaluated in Polson and Helena by ACAP are shown in Fig. 2. Both included a capillary break. At Polson, however, the contrast between the layers was modest because the underlying sand contained fines. At Helena, the storage layer was thick and had relatively high air entry pressure. Consequently, the water balance covers at Polson and Helena functioned like monolithic covers, even though they included a break in soil texture (Apiwantragoon 2007). For this analysis, the covers at Polson and Helena were assumed to function as monolithic covers.

The covers at Polson and Helena functioned remarkably well, with average percolation rates of 0.5 mm/yr (Polson) and 0.0 mm/yr (Helena) during the ACAP monitoring period (2000-04) (Apiwantragoon 2007). Thus, these covers provided a good benchmark for assessing the viability of a water balance cover in Missoula. That is, if the climate in Missoula is sufficiently similar to the climates in Polson and Helena, then a water balance cover for Missoula should function comparable to the covers in Polson and Helena, provided the cover in Missoula has similar available water storage capacity relative to the required storage capacity.

Additional information was available from an application made by the landfill owner in 2002 (Miller 2002) to deploy a water balance cover with a 0.91-m-thick storage layer (Fig. 2). This application was based primarily on findings from a numerical modeling exercise using on-site soil hydraulic properties. The modeling indicated that percolation would be nil for typical meteorological conditions. The application was not approved because the site characterization was limited, wetter than normal conditions were not assessed, and no provision was made to demonstrate performance at full scale. This proposed cover is referred to henceforth as the “2002 Missoula Cover,” although a cover was never constructed based on this design.

The data from Polson and Helena were evaluated in the context of the conditions in Missoula to determine if these sites could be used as analogs, and to determine if the design goal for Missoula was realistic given the meteorological conditions at the site, the soil resources available, and the local vegetation. Soil hydraulic properties reported in Miller (2002) were used for this preliminary assessment.

Climate Assessment

Meteorological data for Missoula were obtained from the National Weather Service. Climate type, average annual precipitation, and average high and low temperatures for Missoula, Polson, and Helena are summarized in Table 1. The data correspond to the period 1952-2010, for which a complete record of precipitation and temperature was available for all three sites.

Missoula and Helena have semiarid climates based on the definitions in UNESCO (1999), whereas Polson is subhumid. Polson has the highest average annual precipitation (380 mm) and the highest ratio of annual precipitation (P) to annual potential evapotranspiration (PET). The ratio P/PET is a measure of the amount of water to be managed (a fraction of P) relative to the energy available to manage water via evapotranspiration. Lower P/PET corresponds to greater aridity and higher confidence in managing precipitation with minimal percolation; i.e., the likelihood of achieving a percolation goal increases as annual P/PET decreases. Annual precipitation at all three locations was close to average during 2000-04, when data were collected from the ACAP-style lysimeters at Polson and Helena (Table 1). Thus, the water balance data from Polson and Helena during this period represent typical conditions.

Table 1. Climatic data for Missoula, Helena, and Polson, Montana.

Site	Climate	Avg. Annual Precip. (mm)	Avg. Annual P/PET	Avg. Annual Precip. During ACAP (mm)	Avg. Air Temperature (°C)	
					High with Month	Low with Month
Missoula	Semi-arid	337	0.40	333	30 (July)	-10 (Jan.)
Helena	Semi-arid	289	0.44	270	29 (July)	-13 (Jan.)
Polson	Sub-humid	380	0.58	362	29 (July)	-7 (Jan.)

Daily average precipitation is shown in Fig. 3a for Missoula, Polson, and Helena. The annual precipitation pattern is similar for all three sites, with the wettest period occurring at the end of spring and beginning of summer, followed by a much drier period in mid summer and a wetter period in late summer and early fall. Fall and winter are the driest seasons. Daily precipitation at Missoula is more similar to precipitation in Polson than Helena. However, Missoula is drier than Polson, particularly in the spring. Missoula is wetter than Helena in the winter and spring, and comparable in the summer (Fig. 3a).

Daily average minimum and maximum air temperatures at all three sites (Fig. 3b) show similar seasonality. Missoula has slightly higher maximum daily air temperatures than Polson and Helena, except in late fall and winter. The daily average minimum air temperature tends to be cooler in Missoula than Polson and

Helena during late spring, summer, and early fall (Fig. 3b), which is indicative of a clearer sky and lower humidity during the summer. Polson exhibits the least seasonal variation in temperature of the three sites, and a slightly smaller difference between daily maximum and minimum temperature, due to buffering provided by Flathead Lake (adjacent to Polson).

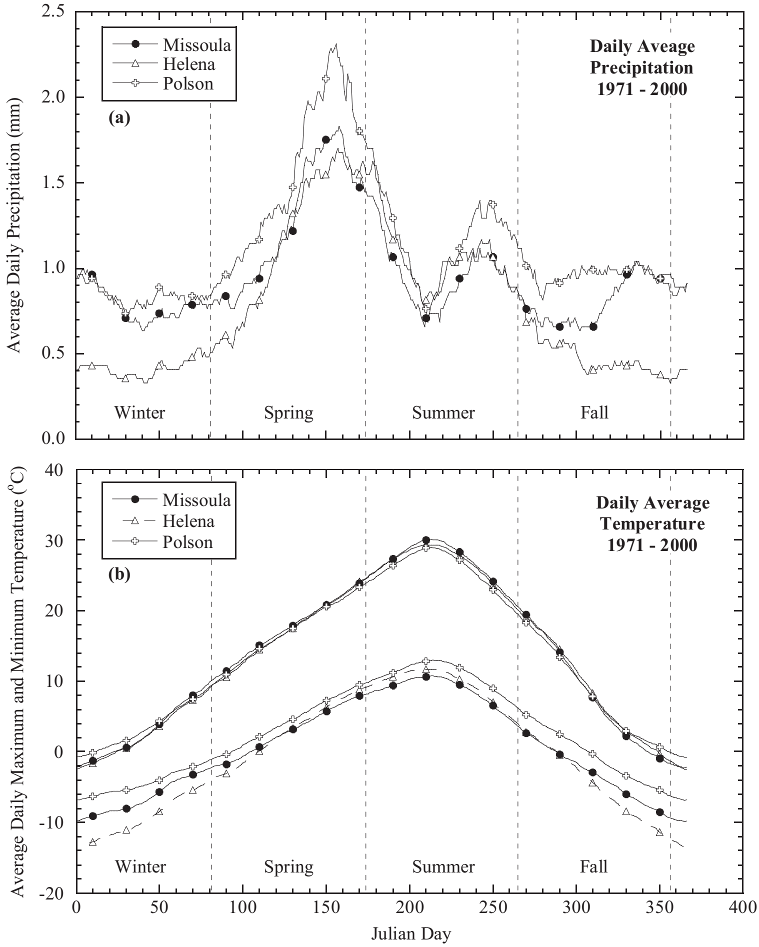

FIG. 3. Daily average precipitation (a) and daily average maximum and minimum temperature (b) between 1971 and 2000 for Missoula, Helena, and Polson.

Annual daily average solar radiation, relative humidity, and wind speed (all affecting ET) for the period 1991-2005 for Missoula, Polson, and Helena (period for which complete data are available for all three sites) are shown in Fig. 4 using box plots. The centerline of the box is the median, the outer boundaries represent the interquartile range (i.e., 25th and 75th percentile), and the upper and lower whiskers represent the 10th and 90th percentiles of the data. Outliers are shown as individual data points above or below the whiskers (e.g., Fig. 4c). Solar radiation at each site is comparable since the three locations are at similar latitude. Polson is the most humid site, due to the proximity of Flathead Lake. Helena is the least humid and windiest site (Fig. 4b and c). Missoula falls between Polson and Helena for all three meteorological parameters.

Soil Resource Assessment

Index properties of the soil proposed by Miller (2002) for the 2002 Missoula Cover are summarized in Table 2 along with the properties of the storage layers in Polson and Helena. The Missoula soil is a broadly graded silty sand with gravel (SM). The storage layer at Polson has an upper layer of lean clay (CL-ML) over a lower layer of silty sand (SM). At Helena, the storage layer is silty clay (SC). The soils at Polson and Helena contain less gravel (≤ 6%) than the Missoula soil reported by Miller (2002) (33% gravel, Table 2).

Table 2. Composition and classification of storage layer soils at Missoula (Miller 2002) and Polson and Helena (Albright et al. 2004).

Site	Unified Soil Classification	Particle Size Distribution (%)				Atterberg Limits	
		Gravel	Sand	Fines	2 μm Clay	Liquid Limit	Plasticity Index
Missoula	SM	33	34	33	NR	NR	NR
Helena	SC	2	54	44	30	67	47
Polson	SM	6	54	42	5	NP	NP
	CL-ML	0.8	6.1	93.2	18	28	7

Notes: NR = not reported; NP = non-plastic as defined in ASTM D 2487; particle sizes based on definitions in the Unified Soil Classification System (ASTM D 2487): gravel > 4.8 mm, 4.8 mm > sand > 0.75 mm, fines < 0.75 mm

The silty clay at Helena has a similar distribution of predominant particle sizes as the silty clay at Polson (Table 2), but the Helena soil has moderately plastic fines and a larger clay fraction (30% vs. 5%). Although Atterberg limits were not reported for the Missoula soil by Miller (2002), the high percentage of sand and gravel and the relatively high saturated hydraulic conductivity (see subsequent discussion), suggests that the Missoula soil probably is non-plastic.

Saturated and unsaturated hydraulic properties reported by Miller (2002) for the Missoula soil are in Table 3 along with properties for covers in Polson and Helena. The hydraulic properties at Polson and Helena were measured during construction as well as 9 yr after the covers had been in service (cited as the "in-service" condition

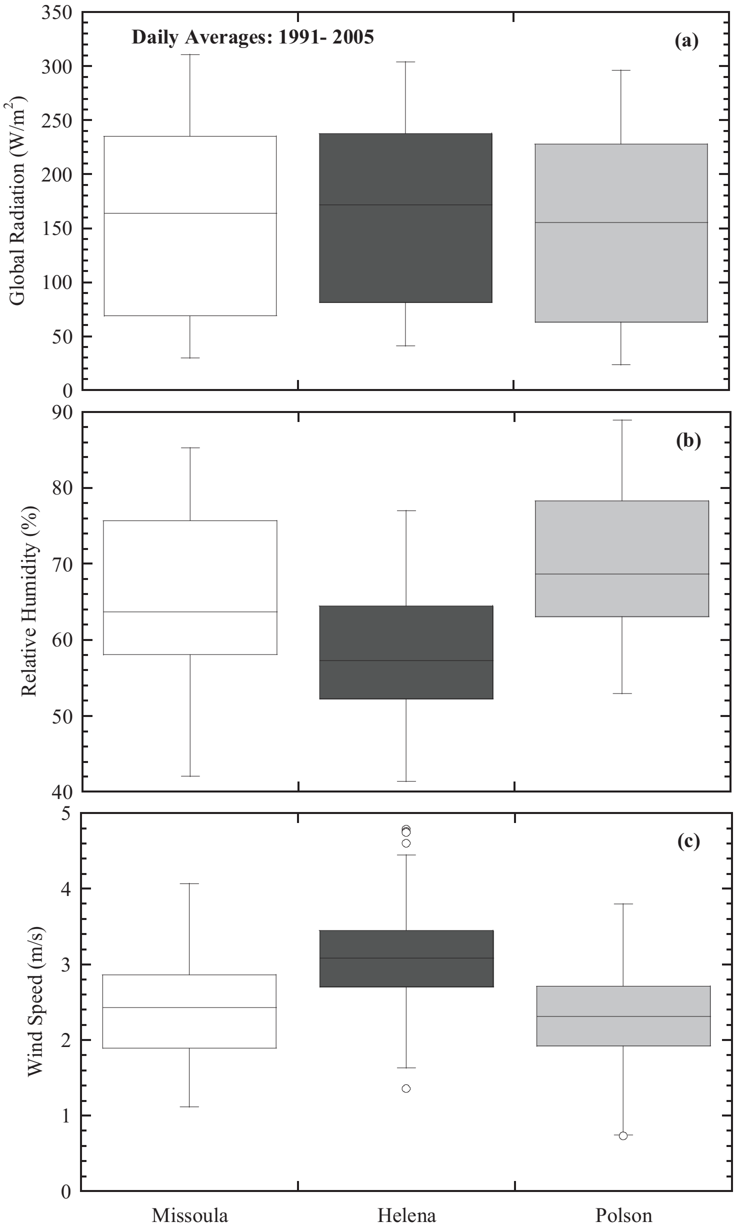

FIG. 4. Box plots of daily average global radiation (a), relative humidity (b), and wind speed (c) from 1991 to 2005 for Missoula, Helena, and Polson.

henceforth). The saturated hydraulic conductivities (K_s) of the Missoula soil and the as-built silty sand at Polson ($4.2x10^{-5}$ to $4.9x10^{-5}$ cm/s) are comparable. The silty clay at Polson and clayey sand at Helena were less permeable when constructed (K_s = $1.5x10^{-7}$ to $4.0x10^{-7}$ cm/s). However, samples collected after 9 yr of service indicated that pedogenesis had altered the soils (K_s = $2x10^{-6}$ to $8x10^{-6}$ cm/s), making them nearly as permeable as the Missoula soil. (Benson et al. 2007, 2011a).

Soil water characteristic curves (SWCC) for the soils at Polson, Helena, and Missoula (as described in Miller 2002) are shown in Fig. 5. van Genuchten's (1980) equation was used to describe the SWCC:

$$\frac{\theta - \theta_r}{\theta_s - \theta_r} = \left\{ \frac{1}{1 + (\alpha\psi)^n} \right\}^m \tag{1}$$

where α and n are fitting parameters, θ_s is the saturated volumetric water content, θ_r is the residual water content, and m = 1-1/n. The fitted parameters are summarized in Table 3 for as-built conditions in Polson and Helena, and laboratory-compacted conditions for the Missoula soil as reported by Miller (2002). The SWCCs in Fig. 5 correspond to the as-built and in-service conditions for Polson and Helena, and the laboratory-compacted Missoula soil reported by Miller (2002).

Table 3. Hydraulic properties of storage layers at Polson and Helena and for 2002 Missoula Cover.

Site	Soil	Hydraulic Properties				
		K_s (cm/s)	α (kPa^{-1})	n	θ_s	θ_r
2002 Missoula Cover	SM	$4.9x10^{-5}$	0.052	1.28	0.34	0.00
Helena	SC	$1.5x10^{-7}$	0.0018	1.19	0.34	0.00
Polson	SM	$4.2x10^{-5}$	0.0010	1.40	0.35	0.00
	CL-ML	$4.0x10^{-7}$	0.0027	1.27	0.30	0.00

The as-built soils for Polson and Helena and the Missoula soil have similar θ_s and θ_r (Table 3), but the α parameter reported by Miller (2002) for the Missoula soil is more than one order of magnitude larger than the as-built α for the soils at Polson and Helena (0.0010-0.0027 1/kPa). However, the in-service α for both Polson and Helena (0.01-0.05 1/kPa, Benson et al. 2011a) after 9 yr of service is similar to α for the Missoula soil reported by Miller (2002). All of the soils have similar n (1.19 to 1.40) in the as-built and in-service conditions (Benson et al. 2011a).

Storage Assessment

Available Soil Water Storage Capacity. Available soil water storage capacity (S_A) was computed for the covers in Polson and Helena along with the profile proposed previously for the 2002 Missoula Cover (Miller 2002) using:

$$S_A = L\,(\theta_{FC} - \theta_{WP}) \tag{2}$$

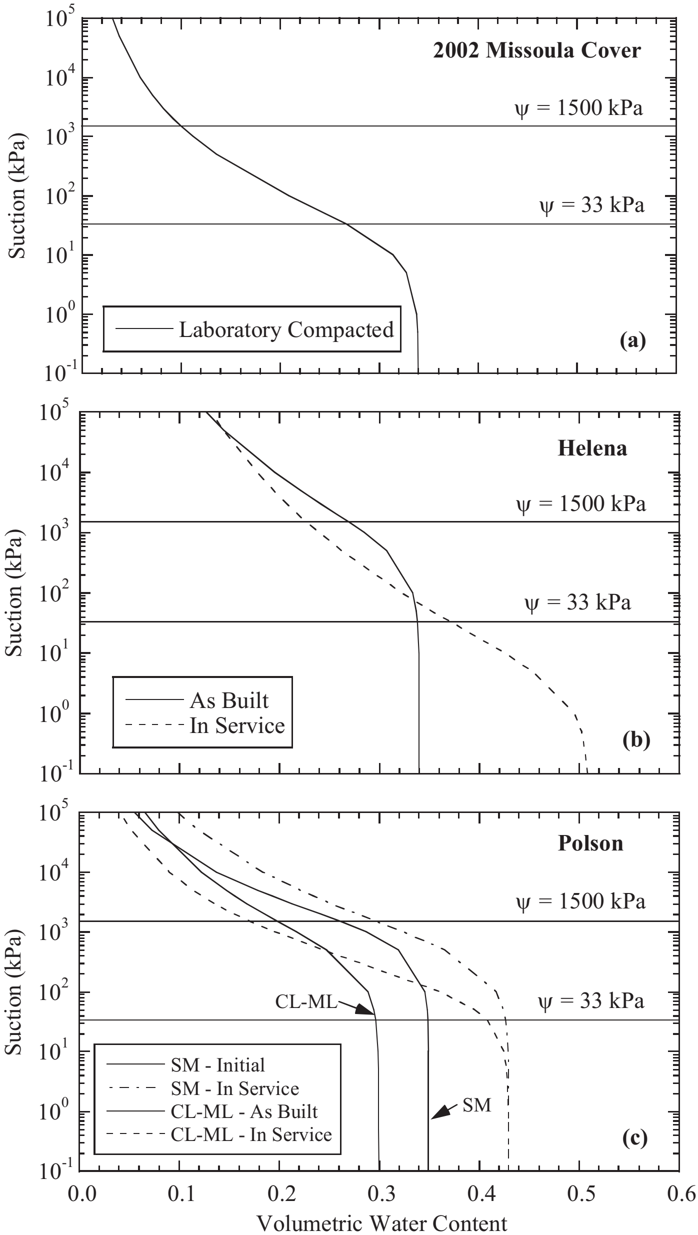

FIG. 5. Soil water characteristic curves (SWCCs) for Missoula (a), Helena (b), and Polson (c).

where L is the thickness of the storage layer, θ_{FC} is the field capacity, and θ_{WP} is the wilting point (Albright et al. 2010). Available soil water storage capacity is reported in units of length (volume per unit area = length). Field capacity (θ_{FC}) is the water content at which drainage becomes negligible under gravity and is estimated from the SWCC as the volumetric water content (θ) at ψ = 33 kPa. The wilting point (θ_{WP}), the water content at which transpiration ceases, was estimated as θ at ψ = 1500 kPa. In semi-arid regions such as western Montana, wilting points can be as high as 5-7 MPa (Apiwantragoon 2007). Thus, defining θ_{WP} based on 1500 kPa underestimates available storage (a conservative estimate).

The following S_A were computed: Polson – 90/166 mm, Helena – 82/164 mm, and 2002 Missoula – 134 mm (the latter S_A for Polson and Helena correspond to in-service conditions). The increase in S_A at Polson and Helena is due to pedogenic changes in the SWCC, which are known to increase θ_s and α (Benson et al. 2007a, 2011a). For these soils, the increase in available storage due to the increase in θ_s was more significant than the reduction in available storage due to the increase in α. When the in-service condition is considered for Polson and Helena, the 2002 Missoula Cover has approximately 30 mm less storage than the covers in Polson and Helena, and therefore could transmit more percolation.

Required Soil Water Storage. Required soil water storage (S_R) was computed for the 2002 Missoula Cover and the covers in Polson and Helena using Eq. 3 and the procedures outlined in Albright et al. (2010):

$$S_r = \sum_{m=1}^{6} \Delta S_{FW,m} + \sum_{m=7}^{12} \Delta S_{SS,m} \tag{3}$$

where $\Delta S_{FW,m}$ is monthly accumulation of soil water storage in fall and winter (m = 1-6 corresponding to October through March):

$$\Delta S_{FW,m} = P_m - \beta_{FW} PET_m - \Lambda_{FW} \tag{4}$$

and $\Delta S_{SS,m}$ is monthly accumulation of soil water storage in spring and summer (m = 7-12, April through September).

$$\Delta S_{SS,m} = P_m - \beta_{SS} PET_m - \Lambda_{SS} \tag{5}$$

In Eqs. 4 and 5, P_m is monthly precipitation and PET_m is monthly PET for the m^{th} month, β_{ij} is the ratio of ET to PET for fall-winter (FW) or spring-summer (SS) conditions, and Λ_{ij} is the water balance residual (runoff, percolation, and internal lateral flow, if any) for fall-winter (FW) or spring-summer (SS) conditions. For sites with snow and frozen ground, β_{FW} = 0.37, β_{SS} = 1.00, Λ_{FW} = 0, and Λ_{SS} = 168 (Albright et al. 2010). PET was computed using methods presented in Allen et al. (1998). For months when P/PET was less than 0.51 (fall and winter months) or 0.32 (spring and summer months), the monthly accumulation was set at zero, as recommended in Albright et al. (2010). In addition, if $\Delta S_{FW,m}$ or $\Delta S_{SS,m}$ computed with Eqs. 4-5 was less than zero for any m, the monthly accumulation was set to zero for that month as recommended in Albright et al. (2010).

Required storage for each site during 2000-04 (period when Polson and Helena were monitored) is summarized in Table 4. The required storage for Missoula during this period (35-156 mm, annual average = 92 mm) is comparable to the required storage in Polson (49-134 mm, annual average = 89 mm), and appreciably more than the required storage in Helena (14-42 mm, annual average = 12 mm).

Table 4. Required and available storage for ACAP monitoring period (2000-04) for 2002 Missoula Cover and covers in Polson and Helena.

Site	Available Storage S_A (mm)	Year	Precip. (mm)	PET (mm)	Required Storage S_R (mm)	S_R/S_A
2002 Missoula Cover	134	2000	314	861	156	1.17
		2001	337	846	95	0.71
		2002	258	783	35	0.26
		2003	370	875	106	0.80
		2004	386	826	66	0.49
Helena	164	2000	213	1038	42	0.26
		2001	273	1105	6	0.04
		2002	319	1004	0	0.00
		2003	238	1093	0	0.00
		2004	308	990	14	0.08
Polson	166	2000	382	822	134	0.81
		2001	341	856	81	0.49
		2002	356	812	49	0.29
		2003	343	898	80	0.48
		2004	386	777	103	0.62

Historical meteorological data for Missoula were used to compute required storage for a typical year (year with annual precipitation closest to long-term average, 1984 – 338 mm) and the more challenging design scenarios recommend in Albright et al. (2010): the wettest year on record (1998, 556 mm) and the 95th percentile precipitation year (1975 – 469 mm). Required storage for these cases is summarized in Table 5.

The required storage for Missoula computed from the historical data varies by a factor of four between the typical year (1984 – S_R = 51 mm) and the 95th percentile precipitation year (1975, S_R = 204 mm). Moreover, the required storage is higher for the 95th percentile precipitation year (S_R = 204 mm), even though the wettest year on record (1998, S_R = 133 mm) received more precipitation. This unexpected difference in S_R reflects differences in the temporal distribution of precipitation. In 1998, large precipitation events were received during summer when ET was high, whereas the large precipitation events in 1975 occurred in late fall and winter, when ET was low and water was accumulating in the cover. A wetter winter also occurred in 1975. Thus, the wettest year is not necessarily the worst-case scenario for Missoula.

Table 5. Required storage and required-available storage ratio for 2002 Missoula Cover for wet-year scenarios cited in Albright et al. (2010).

Quantity	Meteorological Year		
	Wettest	95th Percentile	Typical
Year	1998	1975	1984
Precipitation (mm)	556	469	338
PET (mm)	851	762	953
Required Storage, S_R (mm)	133	204	51
S_R/S_A (S_A = 134 mm)	0.99	1.53	0.38

Relative Storage. Each cover was evaluated using the relative storage ratio S_R/S_A, which describes the required storage relative to the available storage in the cover profile for a given meteorological year. When S_R/S_A is << 1, negligible percolation is anticipated because the cover has adequate capacity to store infiltrating precipitation. Percolation is anticipated when S_R/S_A is ≈ 1 or > 1 because the water to be stored is comparable to or larger relative to the storage capacity available in the cover. For $S_R/S_A > 1$, the annual percolation rate can roughly be estimated as S_R-S_A.

A summary of S_R/S_A is in Table 4 for Polson, Helena, and the 2002 Missoula Cover for the 2000-04. For the covers in Polson and Helena, S_R/S_A is < 1 for each year, which is consistent with the very low percolation rates measured for these covers (< 1 mm/yr). For the 2002 Missoula Cover, S_R/S_A is < 1 for all years except 2000. However, the average S_R/S_A for the 2002 Missoula Cover (0.69) during 2000-04 is larger than for the covers in Polson (0.54) and Helena (0.08).

Ratios of S_R/S_A for the 2002 Missoula Cover computed using historical meteorological data are summarized in Table 5; S_R/S_A = 0.38 for a typical year, 0.99 for the wettest year on record, and 1.53 for the 95th percentile precipitation year.

Overall Assessment

Comparison of the meteorological data from Missoula, Polson, and Helena indicates that a water balance cover in Missoula should function comparably as the water balance cover in Polson, and maybe as well the water balance cover in Helena (both Polson and Helena had very low percolation rates during 2000-04), provided the cover in Missoula has adequate storage capacity and the vegetation at all three sites has similar ability to remove water from the profile. This is supported by the relative magnitudes of the average annual P/PET, which is lower in Missoula than in Polson and Helena. A wheatgrass blend similar to the vegetation at Polson and Helena is present in the grasslands surrounding the Missoula landfill (Table 6). Thus, the vegetation at Missoula should have similar ability to remove stored water as the vegetation at Polson and Helena.

Table 6. Vegetation at Missoula, Polson, and Helena as reported in Miller (2002), Roesler et al. (2002), and Albright et al. (2004).

Site	Vegetation
Missoula	Critana thickspike, sodar streambank, and pryor slender wheatgrass, sheep fescue, yellow sweetclover.
Helena	Bluebunch, slender, and western wheatgrass, sandburg bluegrass, sheep fescue, blue gamma, green needlegrass, needle-and-thread.
Polson	Thickspike, bluebunch, slender, and crested wheatgrass, mountain brome, Idaho fescue, prairie junegrass, needle-and-thread, meadow brome, Canada and Kentucky bluegrasses, yarrow, fringed sagewort, alfalfa, rubber rabbitbrush, prickly rose, arrowleaf, balsamroot, dolted gayfeather, lewis flax, silky lupine, cicer milkvetch.

The 2002 Missoula Cover profile should be adequate under typical conditions, which is consistent with the predictions made by Miller (2002). However, a thicker cover profile is needed to provide sufficient storage capacity for wetter conditions. For example, S_R/S_A exceeds 1 for the Missoula cover when the meteorological data from 2000 are used to compute S_R (Table 4). For this same year, $S_R/S_A = 0.81$ for Polson and 0.26 for Helena. The year 2000 was wetter than the other years during the 2000-04 monitoring period in Polson and Helena, but was not exceptionally wet. Thus, in wetter years, the 2002 Missoula cover has a higher likelihood of transmitting percolation than the covers at Polson or Helena. MDEQ agreed with this assessment, and concurred that a thicker cover was necessary.

Path Forward

Based on the preliminary assessment, MDEQ was satisfied that a properly sized water balance cover could function satisfactorily at Missoula. However, discussions with MDEQ indicated that approval for a water balance cover would require more comprehensive analysis and design, including (i) more comprehensive evaluation of soil resources, (ii) site-specific assessment of vegetation properties, (iii) additional modeling to evaluate wetter conditions, and (iv) a full-scale demonstration to validate that the cover functions as designed. These requirements from MDEQ are consistent with their *Draft Alternative Final Cover Guidance* (v. 9-11) and the approach recommended in Albright et al. (2010). The following steps were conducted to addresses these concerns:

- a soil resource evaluation was conducted and the saturated and unsaturated hydraulic properties were measured for potential cover soils,
- vegetation at the site was sampled and the leaf area index (LAI) and root density function were measured,

- data describing the phenology of the wheatgrass blend in the region were obtained from the literature,
- preliminary design was conducted to estimate the required thickness of the storage layer under wetter conditions (wettest year on record and 95th percentile precipitation year) using soil hydraulic properties from the soil resource evaluation,
- percolation from the cover identified in preliminary design was predicted using a numerical model employing site-specific meteorological data, soil properties, and vegetation properties as input; meteorological data for typical and much wetter conditions were employed,
- a test section was constructed so that the water balance (particularly the percolation rate) could be measured at field scale to confirm the sufficiency of the design.

SOIL RESOURCES

A soil resource evaluation was conducted to determine the suitability of the on-site soils. Ten soil samples were collected from test pits excavated at four sites (Sites 1-4) where soil was available for the cover. Sites 1-3 were soil stockpiles excavated from previous cell construction. Native ground was sampled at Site 4, and at Site 1 in an area adjacent to the soil stockpile at Site 1. Topsoil was sampled at Sites 3 and 4.

The site has an abundance of soil and availability of soil is not an issue. Thus, the characterization focused on identifying soils that were suitable for the cover and not the volume of each soil that was available.

Test pits were excavated with a backhoe at each sampling site. Each test pit was inspected visually to assess homogeneity of the borrow source. Disturbed samples were collected with hand tools and placed in 20-L buckets. All buckets were sealed with plastic lids containing rubber gaskets.

Particle Size Distribution

Particle size analyses were conducted on each sample following ASTM D 422. The particle size distributions are shown in Fig. 6. Atterberg limits were not measured. The average particle size fractions for the stockpile soils (gravel = 39%, sand = 27%, and fines = 32%) are comparable to those for the Missoula soil cited in Miller (2002).

Five soils were selected for additional testing (solid symbols and solid lines in Fig. 6). These soils included three stockpile soils, one at each of Sites 1-3 (S1-XX to S3-XX; XX is an identifier), the topsoil at Site 3 (S3-TS), and the finer-textured native ground from Site 4 (S4-NG-1). Topsoil at Site 3 was selected to define topsoil properties for use in water balance modeling. The topsoil at Site 3 was coarser than the topsoil at Site 4, and was expected to provide a conservative representation of topsoil available for the cover. The three stockpile soils constitute a broad range in particle size distribution (Fig. 6), and were selected to define a range of anticipated hydraulic properties. The fine-grained native ground soil (S4-NG-1 in Fig. 6) was selected for comparison with the stockpile soils.

Compaction Properties

Compaction tests were conducted on the five soils using ASTM D 698 (standard Proctor). All soils were scalped on a 9.5-mm sieve, and a coarse-fraction correction was applied to account for particles larger than 9.5 mm using the procedure in ASTM D 4718. Compaction curves for the three stockpile soils were comparable, with maximum dry unit weight (γ_{dmax}) ranging between 18.3 and 20.1 kN/m^3 and optimum water content (w_{opt}) ranging between 8.2 and 11.4% (Benson and Bareither 2011). The compaction curve reported by Miller (2002) for the 2002 Missoula Cover also falls in this range (γ_{dmax} = 19.9 kN/m^3, w_{opt} = 9.6%).

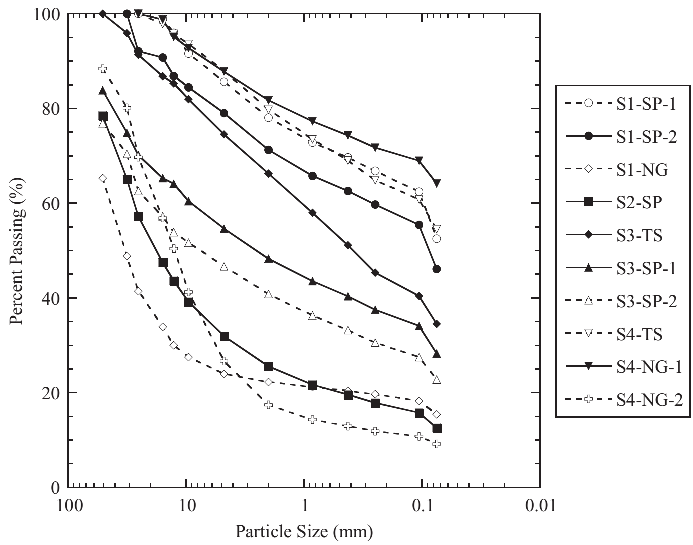

FIG. 6. Particle size distribution curves for 10 soils sampled at Missoula landfill. Solid symbols correspond to soils selected for hydraulic property characterization; SP = stockpile, TS = topsoil, and NG = native ground.

Hydraulic Properties

Saturated hydraulic conductivity was determined on each of the five soils following the procedure in ASTM D 5084. Specimens were compacted to 85% of γ_{dmax} at w_{opt} per ASTM D 698 in 150-mm-diameter molds. Relatively low compaction is used to ensure that the cover soils provide a hospitable environment for root growth (Albright et al. 2010). The effective stress was set at 15 kPa and the hydraulic gradient at 10 to represent conditions existing in a cover.

Soil water characteristic curves (SWCCs) were measured on specimens prepared to the same compaction conditions as specimens for the hydraulic conductivity tests.

Procedures described in ASTM D 6836 were followed. The wet end of each SWCC was measured using a pressure plate extractor and the dry end with a chilled mirror hygrometer. Eq. 1 was fit to the SWCC data using non-linear least-squares optimization (Fig. 7).

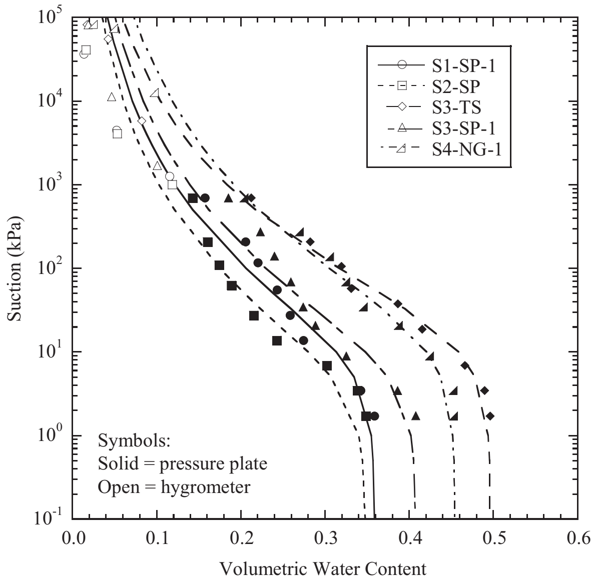

FIG. 7. SWCCs for stockpile soils from Sites 1, 2, and 3, Site 3 topsoil, and Site 4 native ground.

Saturated hydraulic conductivities and van Genuchten parameters for the SWCCs are summarized in Table 7. Saturated hydraulic conductivity of the stockpile soils varies in a narrow range from 3.7×10^{-6} cm/s to 6.0×10^{-5} cm/s. The SWCCs for the stockpile soils are also comparable (Fig. 8), with α ranging from 0.0958 to 0.145 1/kPa, n ranging from 1.27 to 1.28, and θ_s ranging from 0.35 to 0.41. Similar hydraulic properties were reported in Miller (2002) for the storage layer of the 2002 Missoula Cover (Table 3).

VEGETATION

Vegetation samples were collected for measurement of site-specific properties for input to the numerical model. Four locations in the surrounding grassland were selected that had mature vegetation representative of the area surrounding the landfill. A test pit was excavated in each location for root samples and a sampling area was selected for collecting surface biomass. Root samples were collected at 150-mm intervals from the sidewall of each test pit using the modified Weaver-Darland method described in Benson et al. (2007b) and placed in evacuated re-sealable plastic bags. Samples of surface biomass were collected from four 1-m^2 areas by removing all biomass with shears. Surface biomass samples were placed in evacuated plastic bags that were sealed in the field. All of the samples were stored in a refrigerator at 4 ^{o}C prior to analysis.

Table 7. Saturated hydraulic conductivity and unsaturated hydraulic properties for soils sampled from Missoula Landfill.

Sample	Sampling Site	K_S (cm/s)	α (1/kPa)	n	θ_s	θ_r
S1-SP-2	1	6.0x10^{-5}	0.126	1.27	0.36	0.0
S2-SP	2	3.7x10^{-6}	0.0958	1.28	0.35	0.0
S3-TS	3	2.8x10^{-6}	0.0496	1.33	0.50	0.0
S3-SP-1	3	1.2x10^{-5}	0.145	1.28	0.41	0.0
S4-NG-1	4	1.6x10^{-6}	0.115	1.24	0.45	0.0
2002 Miss. Cover	-	4.9x10^{-5}	0.520	1.28	0.34	0.0

Notes: K_S = saturated hydraulic conductivity; α and n = fitting parameters for van Genuchten equation (Eq. 1); θ_s = saturated volumetric water content; θ_r = residual volumetric water content; SP = stockpile; TS = topsoil; NG = native ground.

Leaf area of the clippings was measured using a LI-COR LI-3100C leaf area meter and leaf area index (LAI) was computed as the quotient of the total leaf area and the sampling area (1 m^2). The following LAIs were obtained: 1.46, 1.61, 1.86, and 1.99.

Root densities were measured by soaking each root sample in tap water for 48 h, separating the roots from the soil particles, and air drying the root mass as described in Benson et al. (2007b). Normalized root density profiles for the four pits are shown in Fig. 8. The root profiles in each pit were remarkably similar, and a single root density function was fit to the combined data set from all four pits using least-squares regression.

STORAGE ANALYSIS

A storage analysis was conducted to determine the required thickness of the storage layer (L). This consisted of equating S_R (Eqs. 3-5) and S_A (Eq. 2) and solving for L:

$$L = S_R/(\theta_{FC} - \theta_{WP}) \tag{6}$$

SWCCs corresponding to the combinations of θ_s, θ_r, α, and n yielding the highest and lowest available storage capacity were used to define θ_{FC} and θ_{WP}. Computations were made using two required storage capacities (S_R): (1) S_R = 133 mm for the wettest year on record and (2) S_R = 204 mm for the 95th percentile precipitation year. Accounting for pedogenesis increased S_A for the Polson and Helena sites. Thus, pedogenesis was not included when computing the storage layer thickness with Eq. 4. In addition, pedogenesis is known to make cover soils more similar (Benson et al.

2007a, 2011a), and the hydraulic properties from the soil resource evaluation are similar to the in-service soils for Polson and Helena. Thus, adjusting the hydraulic properties of the Missoula soils obtained from the soil resource evaluation for pedogenesis probably would have been unrealistic.

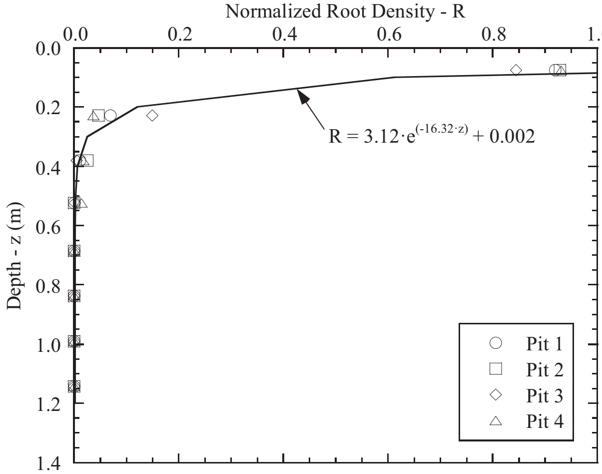

FIG. 8. Normalized root densities from the four test pits at Missoula landfill.

The required storage layer thicknesses (Table 8) ranged from 740-950 mm for S_R = 133 mm (average = 840 mm) and from 1190-1560 mm for S_R = 204 mm (average = 1370 mm), with thicker layers required for the soil with lower storage capacity. Based on this analysis, a preliminary design was selected with a 1.22-m-thick storage layer overlain by a 0.15-m-thick topsoil layer. This cover was expected to provide acceptable storage capacity under most meteorological conditions for typical soils on site. A thicker cover could have been proposed to address worst case conditions, but the assumptions were conservative and increasing the thickness would have raised construction costs an unacceptable amount.

Table 8. Storage layer thicknesses for required storage (S_R) representing wettest year on record (S_R = 133 mm) and 95th percentile precipitation year (S_R = 204 mm).

Storage Layer Capacity	Storage layer thickness (mm)	
	S_R = 133 mm	S_R = 204 mm
Lower Bound Storage Capacity[1]	950	1560
Upper Bound Storage Capacity[2]	740	1190

[1]α = 0.145 1/kPa, n = 1.27, θ_s = 0.35, θ_r = 0.0; [2]α = 0.096 1/kPa, n = 1.28, θ_s = 0.41, θ_r = 0.

WATER BALANCE MODELING

The variably saturated flow model WinUNSAT-H was used to predict the water balance for the proposed water balance cover. WinUNSAT-H (and its DOS counterpart UNSAT-H), is the most widely used numerical model for simulating the hydrology of water balance covers (Benson 2007). When properly parameterized, WinUNSAT-H provides a reliable prediction of the water balance of covers, and over predicts the percolation rate modestly in most cases (Khire et al. 1997, Ogorzalek et al. 2007, Bohnhoff et al. 2009). WinUNSAT-H simulates variably saturated flow, root water uptake, and climatic interactions (Benson 2007, 2010).

Soil Properties

Hydraulic properties used in the design were selected so that the percolation would not be under-predicted, and likely would be over predicted. The topsoil layer was assigned the hydraulic properties associated with Soil S3-TS (Table 7), except the saturated hydraulic conductivity was increased one order of magnitude to account for pedogenesis and to ensure that runoff comprised no more than 10% of the annual water balance, as recommended in Albright et al. (2010). Saturated hydraulic conductivity of the storage layer was set at $6x10^{-5}$ cm/s, the highest of the saturated hydraulic conductivities (Soil S1-SP-2, Table 7) measured during the site characterization. The SWCC was defined using the combination of van Genuchten parameters measured during site characterization yielding the lowest storage capacity.

Vegetation

The vegetation was assigned the minimum LAI (1.46) and the root density function (Fig. 8) obtained from the site characterization. Phenology and water stress parameters described previously (from Roesler et al. 2002) were input.

Meteorological Data

Four meteorological data sets were used for the simulations: the typical year (1984), the wettest year on record, the 95th percentile precipitation year, and the 10-yr period with the highest precipitation (1977-1986). The 10-yr period with the highest precipitation is recommended for design in MDEQ's *Draft Alternative Final Cover Guidance*.

The simulations were conducted in two phases. The first phase consisted of a 5-yr simulation using meteorological data for the typical year for each year of the simulation (i.e., typical year repeated 5 times). This simulation had two purposes: (i) to create a realistic initial condition for the simulations conducted with the 10-yr record with the highest precipitation and (ii) to define a 'typical' percolation rate for the cover, as defined in Albright et al. (2010). The second phase followed immediately after the first phase, and consisted of one of the following: (i) the wettest year on record run 5 times sequentially, (ii) the 95th percentile precipitation year run 5 times sequentially, and (iii) the complete 10-yr record with the highest average precipitation. All three of these scenarios are suggested in Albright et al. (2010).

The simulations with the wettest year and the 95th percentile precipitation year are relatively simple to conduct, and are expected to be very conservative (the likelihood over 5 sequential very wet years is very small). Many engineers believe this design strategy is unrealistic and too conservative. The record for the 10-yr wettest period is realistic, but is more difficult and time consuming to simulate.

Water Balance Predictions

Annual water balances predicted by WinUNSAT-H are summarized in Table 9 for each year of the 10-yr period with highest precipitation along with the average water balance over the 10-yr period, the typical year, the wettest year on record, and the 95th percentile precipitation year. Predictions for the five-year repetitive simulations are for the final year. The maximum annual runoff was 6.6% (95th percentile precipitation year), indicating that nearly all precipitation reaching the surface became infiltration. Thus, the water balance predictions met the runoff criterion (< 10% of annual water balance) suggested in Albright et al. (2010), which applies to arid and humid climates.

Table 9. Predicted water balance quantities for water balance cover with 1.22-m-thick storage layer and 0.15-m-thick topsoil layer.

Year	Annual Water Balance Quantity					
	Cumulative Precip. (mm)	Cumulative Runoff (mm)	Cumulative ET (mm)	Cumulative Percolation (mm)	Avg. Soil Water Storage (mm)	Runoff (% Precip.)
1977	322	0.0	276	1.1	203	0.0
1978	299	0.0	330	7.1	242	0.0
1979	263	0.0	268	4.3	228	0.0
1980	483	16	446	5.7	246	3.3
1981	441	1.5	422	3.0	223	0.3
1982	390	7.8	367	15.9	270	2.0
1983	424	0.0	417	6.6	244	0.0
1984	339	0.1	387	4.2	224	0.0
1985	318	0.3	338	1.5	195	0.1
1986	425	0.0	406	1.5	206	0.0
Avg. (77-86)	370	2.5	366	5.1	228	0.7
Typ. (1984)	338	0.0	366	1.1	197	0.0
Wettest (1998)	556	19.0	522	21.3	285	3.4
95th % (1975)	469	30.8	410	36.4	294	6.6

The annual percolation rate during the wettest 10-yr period ranged from 1.1 to 15.9 mm/yr, with an average of 5.1 mm/yr. The percolation rate was no more than 3.0 mm/yr for four years in the 10-yr period, and no more than 1.5 mm/yr for three years. For the typical year, the percolation rate was 1.1 mm/yr. Much higher percolation rates were obtained from the 5-yr repetitive simulations using the wettest year on record (21.3 mm/yr) and the 95th percentile precipitation year (36.4 mm/yr). The higher percolation rate predicted for the 95th percentile precipitation year is consistent with the findings from the storage assessment.

The water balance graphs for the typical, wettest, and 95th percentile precipitation years are shown in Fig. 9. These graphs illustrate that differences in the total amount of precipitation, as well as the time when precipitation is received, are responsible for the wide range of percolation rates transmitted for these meteorological conditions. During the typical year (Fig. 9a), most of the precipitation occurs in spring and summer (≈ Julian days 100-300), when ET is high. All of the precipitation during this period along with water stored in the cover is returned to the atmosphere, as evinced by a nearly monotonic drop in soil water storage during this period. Storage begins to climb again in mid-fall when ET begins to diminish (> Julian day 300), but the increase in storage is modest because the precipitation is small.

During the wettest year, the majority of the precipitation occurs in spring and early summer (≈ Julian days 90-190) (Fig. 9b). The precipitation between Julian days 130-190 is so heavy at times that soil water storage increases periodically during this period, even though ET is high. In mid-summer to early fall, however, the precipitation rate is low again and the soil water storage diminishes to 223 mm, although not to the very low storage (181 mm) in mid fall of the typical year. Heavy precipitation in late fall raises the soil water storage at the end of the year to 302 mm, 106 mm more than soil water storage at the end of the typical year. This additional storage, when combined with heavy precipitation during the first 25 d of the new year, results in peak storage of 341 mm that persists through spring. At this level of soil water storage, percolation is transmitted and continues through summer.

The 95th percentile precipitation year received less precipitation than the wettest year on record, but more precipitation (26 mm) was received during the fall than occurred in the fall of the wettest year (Fig. 9c). As a result, soil water storage was 325 mm at the end of the year, i.e., higher than at the end of the wettest year on record (302 mm). This high soil water storage, coupled with heavy precipitation during the first 40 d of the new year (100 mm, vs. 50 mm during the same period during the wettest year), resulted in a peak soil water storage of 380 mm at Julian day 49. Percolation began earlier in the year due to the high soil water storage at the end of the previous year, increased significantly as the soil water storage reached its peak, and continued through summer.

Comparison of the water balance graphs in Fig. 9 to the water balance graph in Fig. 10 for the 10-yr period with the most precipitation illustrates that high levels of soil water storage at the end of the previous year, combined with wetter than normal conditions in the winter and spring, consistently give rise to the highest percolation rates. The highest percolation rate predicted in the 10-yr simulation was in 1982 (15.9 mm/yr), which occurred in response to a sustained period of frequent and less intense precipitation beginning in late Fall 1981 and continuing into early Fall 1982. This

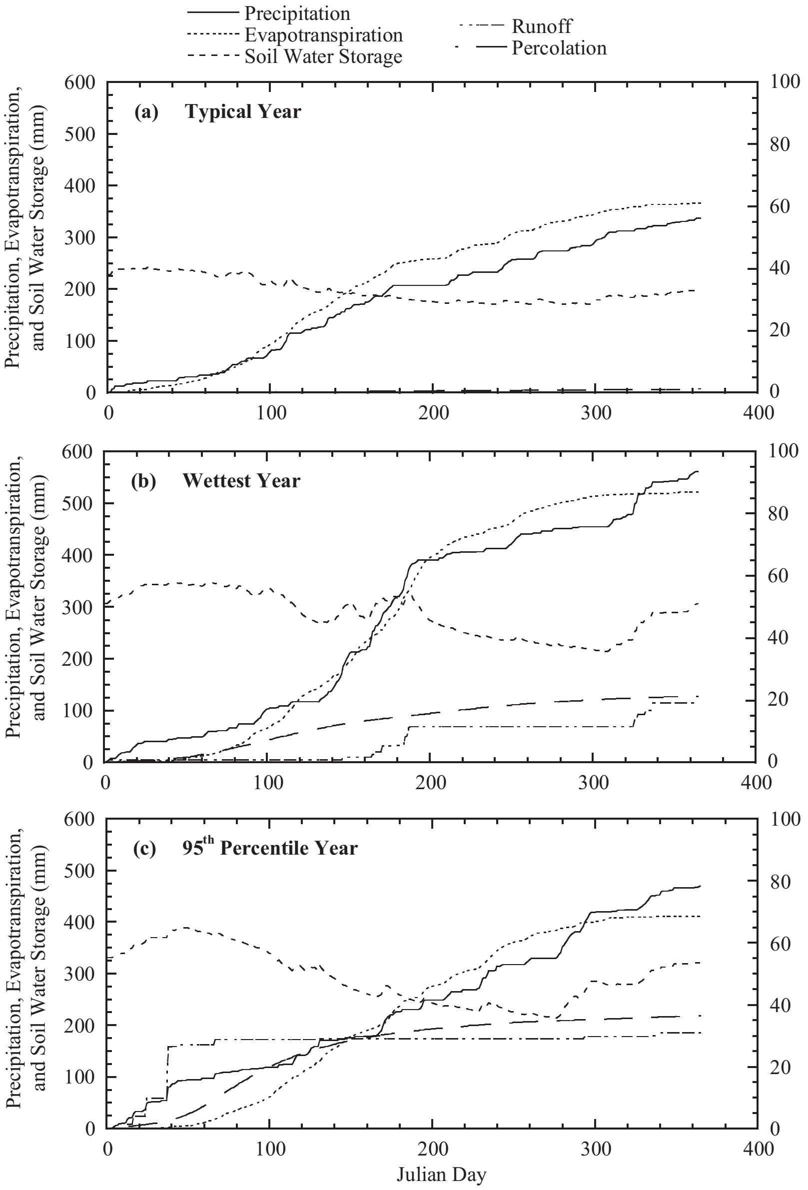

FIG. 9. Predicted water balance quantities for fifth year of 5-yr analysis using hydraulic properties corresponding to lower bound storage.

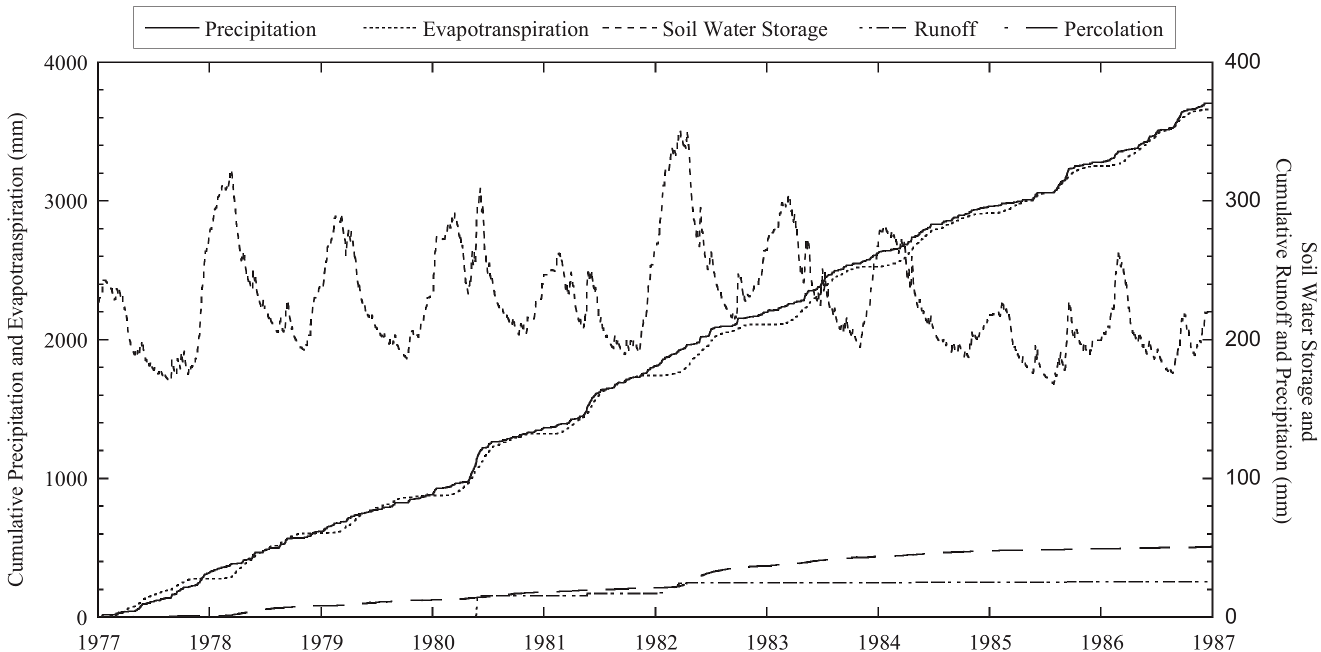

FIG. 10. Water balance predictions for 10-yr period with highest average precipitation made with WinUNSAT-H using hydraulic properties corresponding to lower bound on storage properties.

condition led to a very large increase in storage during the winter of 1982, even though four years during the 10-yr period (1980, 1981, 1983, and 1986) had higher annual precipitation than 1982.

The second highest annual percolation rate (7.1 mm/yr) occurred in 1978, even though this year was drier than average (299 mm precipitation vs. 337 mm, on average) and only one year during the 10-yr record had less annual precipitation (263 mm in 1979). Like 1982, percolation in 1978 occurred in response to a large increase in soil water storage in late Fall 1977 and Winter 1978 that was caused by a sustained period of frequent and less intense precipitation. Thus, the timing of precipitation and the sequencing from one year to the next has a critical impact on the accumulation of storage and the amount of percolation that occurs.

The timing and sequencing of precipitation is represented realistically using actual multi-year time series, whereas repetitive simulations with the wettest year on record or the 95th percentile year probably are unrealistic. As shown in Fig. 10, very wet years generally are not sequential, and the likelihood of five very wet years occurring sequentially is very small.

Implications

None of the modeling predictions confirm that the design objective (3 mm/yr average percolation rate) will be accomplished. The percolation rate for the typical year (1.1. mm/yr) is lower than the design objective, but the average percolation rate over the 10-yr period with highest precipitation (5.1 mm/yr) exceeds the design objective. Percolation rates greatly in excess of the design objective were obtained from the 5-yr repetitive simulations, but these simulations are considered unrealistic.

The long-term percolation rate could be evaluated by simulating the entire 50-yr meteorological record or a very long record generated with a synthetic weather generator based on meteorological statistics for Missoula. However, either simulation would be very time consuming and computationally costly, and probably would not be practical for most projects. This type of simulation was considered impractical for the Missoula landfill.

The average of the percolation rate from the typical year and from the 10-yr simulation is 3.1 mm/yr, which is slightly larger than the design objective. Given that the 10-yr record used in the simulation is the wettest decade on record, and that periods of drought will also occur along with typical conditions, the long-term average percolation rate is likely to be less than 3.1 mm/yr. Thus, the design objective likely will be met, and MDEQ concurred that this conclusion is reasonable.

Based on this assessment, and the favorable monitoring data from the covers in Polson and Helena, the final design consisted of a 1.22-mm-thick storage layer overlain by 0.15 m of topsoil.

TEST SECTION

A field demonstration of the cover is being conducted using an ACAP-style test section constructed following the methods described in Benson et al. (1999). The test section slopes at 3% to simulate the actual top deck slope for the landfill, and faces north to receive the greatest snow accumulation and lowest solar radiation (i.e., worst case orientation). The test section includes a 10 m x 20 m pan-type lysimeter (Fig. 11) for direct measurement of the water balance, including surface runoff, soil water storage, and percolation. The base and sidewalls of the lysimeter are comprised of linear low-density polyethylene geomembrane, and the base is overlain with a geocomposite drainage layer to protect the geomembrane and to transmit percolation to a zero-storage sump. Diversion berms are placed on the surface to prevent run-on and collect run-off.

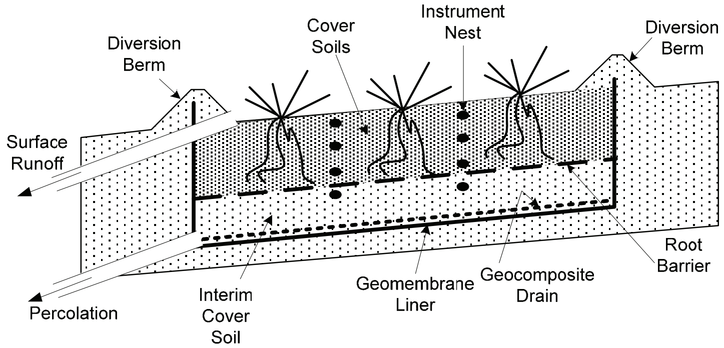

FIG. 11. Schematic of test section for evaluating performance of the water balance cover (not to scale).

Instrumentation

Percolation and surface runoff are routed by pipes to basins equipped with a pressure transducer, tipping bucket, and float switch (triple redundancy) capable of measuring flows with a precision better than 0.1 mm/yr (Benson et al. 2001). Soil water content is measured in three nests located at the quarter points along the centerline. Each nest contains 5 low-frequency (40 MHz) time domain reflectometry (TDR) probes in a vertical stack. Soil water storage is determined by integration of the point measurements of water content. Each probe includes a thermistor for monitoring soil temperature along with water content. Soil-specific calibration of the TDR probes was conducted using the method in Benson and Wang (2006).

Meteorological data (precipitation, air temperature, relative humidity, solar radiation, wind speed, and wind direction) are measured with a weather station mounted on the test section. Data are collected and recorded by a datalogger every 15 min and are stored on one-hour intervals. At times of intense activity (e.g., an intense rain event with high surface runoff), data are stored at time intervals as short as every 15 s. Data from the test section are stored on a server equipped with a screening level quality assurance (QA) algorithm. Detailed QA checks are conducted quarterly. At the time this paper was prepared, the duration of the monitoring period was too short to present the monitoring data in a useful manner.

Placement of Cover Profile

Prior to constructing the cover profile, a layer of soil simulating the existing interim cover was placed on top of the geocomposite drainage layer. The interim cover layer was overlain with a root barrier (thin nonwoven geotextile studded with nodules containing trifluralin, a root inhibitor) to prevent root intrusion into the geocomposite drainage layer and the percolation collection system. Inclusion of the root barrier results in less water being transpired than might occur in an actual cover, where roots can grow through the interim cover and into the waste (Albright et al. 2004). However, the root barrier prevents plants from having access to water retained in the collection and measurement system that would otherwise become deep drainage in an actual application.

The storage layer was constructed with soil from Site 3, which was placed over the root barrier using a bulldozer in three 0.41-m-thick lifts. The dry unit weight was required to be 80-90% of maximum dry unit weight per standard Proctor and the water content was required to be no wetter than optimum water content, as recommended in Albright et al. (2010). Thick lifts were used to minimize the potential for over compaction of the soil, as is common in practice during construction of water balance covers.

Topsoil stripped from Site 3 was placed on the surface and fertilized to stimulate growth. Seed was not added; the natural seed bank within the topsoil serves as the source of seed. This “live haul” approach creates a more realistic and sustainable plant community that is consistent with the surroundings.

SUMMARY AND CONCLUSIONS

The key steps required to design and demonstrate a water balance cover have been illustrated in this paper using a case history where a cover was designed for a MSW landfill in Missoula, Montana. This design process evolved from two decades of research. The state-of-the-art developed through research is now being applied as state-of-the-practice.

The process begins by understanding the design objective (including regulatory requirements) and investigating lines of evidence indicating that a water balance cover is likely to function satisfactorily at the design location. For the Missoula case history, regional data from two instrumented water balance covers were used along with meteorological data to illustrate that a water balance cover could be successful in Missoula if sized properly. Site characterization was then conducted to define properties of the soil resources and vegetation at the site for preliminary cover sizing and numerical modeling of the water balance. A preliminary design was created using semi-empirical analytical methods and the design was evaluated by simulation with a numerical model using several design meteorological conditions as input.

The numerical modeling illustrated that predictions made with an actual multiyear time series (e.g., wettest 10-yr period in the meteorological record) are more realistic than predictions from 5-yr repetitive simulations using a worst case design year, which have been recommended historically and have become common in practice. The multiyear simulations preserve realistic sequencing of seasonal precipitation patterns, which have a strong influence on soil water storage and percolation rate. The drawback is that multiyear simulations are more cumbersome and time consuming to conduct.

The final design for the Missoula landfill is a monolithic water balance cover comprised of a 1.22-m storage layer overlain by a 0.15-m topsoil layer. A field demonstration with a fully instrumented lysimeter was deployed in October 2011 to confirm that the cover performs as anticipated in the design.

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Computational Plasticity for Geotechnical Applications

Ronaldo I. Borja

Professor, Department of Civil and Environmental Engineering, Stanford University, Stanford, CA 94305-4020; borja@stanford.edu

ABSTRACT: Soils and rocks typically exhibit strongly nonlinear, inelastic load-deformation behavior. Capturing their mechanical response in numerical simulations requires elaborate constitutive models and robust numerical algorithms. Computational plasticity strives to strike a balance between complexity of the constitutive model and feasibility of numerical implementation. In this paper, we describe the applications of computational plasticity to four classes of problems in geotechnical engineering: (a) multiscale problems including bridging of different scales; (b) nonlinear contact mechanics; (c) multiphysics and multifield problems; and (d) seismic response of geotechnical structures.

INTRODUCTION

Quantifying the inelastic deformation is a major challenge in performance-based geotechnical engineering. Because soils exhibit a very small range of elastic behavior, nearly all deformations experienced by a geotechnical structure are inelastic. For natural and engineered materials, theory of plasticity is a common platform for material characterization of the inelastic response. This is especially true in geotechnical engineering where very elaborate elastoplastic constitutive models for geomaterials have been developed over the years, and many more are still emerging in the literature. In order to reach the full potential of these constitutive models, they must be implemented efficiently using robust numerical integration algorithms. Computational plasticity strives to strike a balance between model complexity and ease with which these models are implemented into computer codes.

Applications of computational plasticity to geotechnical engineering problems are vast, and for presentation purposes we opt to group them into four broad areas that have enjoyed the most significant research activity in recent years. They include: (a) multiscale problems; (b) nonlinear contact mechanics; (c) multiphysics/multifield problems; and (d) geotechnical earthquake engineering. In multiscale problems we address three levels of material characterization, namely, microscopic, mesoscopic, and macroscopic scales, as well as discuss some relevant aspects for bridging these scales. We also discuss an interesting topic of shear band development, which is a classic example of multiscale problem.

Nonlinear contact mechanics deals with interface problems and unilateral constraints. If the interfaces are well defined throughout the entire solution of the problem, then finite element procedures based on classic nonlinear contact mechanics may be employed. However, there may be problems of interest where the geometry of the interface is not a prior known but instead is calculated as part of the solution. These include crack, fracture, and fault propagation. In this case, enhanced finite element techniques such as the assumed enhanced strain and extended finite element methods may be used to capture an evolving geometry of the discontinuity. Computational plasticity is used to model both the bulk response and slip on interfaces in such problems.

An interesting aspect of geotechnical engineering is the diverse range of topics encountered in this field. Apart from the solid deformation and structural aspects that are common issues in foundation engineering, one also finds different scientific issues such as fluid flow, contaminant transport, chemical/biological reactions, and other multiphysics and multifield processes. Two or more conservation laws must be satisfied at the same time, requiring the solution of a very large system of equations. Scientific topics include thermo-hydro-mechanical and bio-hydro-mechanical processes in geologic media. Irrespective of the multitude of processes involved, the geotechnical engineer almost always faces the need to address the deformation of the soil skeleton, which in turn necessitates an appropriate definition of effective stress. Computational plasticity is used for constitutive modeling of the solid deformation using this effective stress.

The fourth application area covered in this paper is seismic response of geotechnical structures. We discuss how the advances in computational plasticity may be used for nonlinear site response analysis and performance-based seismic design. This requires the calculation of inelastic deformation in addition to the acceleration responses for evaluating seismic performance. We remark that the above grouping is artificial in that an application problem could easily fall under any of the above four categories. For example, the inelastic deformation of unsaturated aggregated soils may be considered as a multiscale (two porosity scales), multiphysics (thermo-hydro-mechanical), and multifield (temperature, air and fluid velocity, and solid deformation) problem. Dynamic soil-pile-structure interaction is a problem in structural dynamics that could also involve nonlinear contact mechanics, and so on.

For the sake of clarity, “computation” is used in this paper to pertain to implicit integration of the rate-constitutive equation within the framework of nonlinear finite element analysis. While explicit integration algorithms for elastoplastic models also have been used in the past, they are not as robust as the implicit algorithms. Furthermore, other computational platforms for elastoplastic analysis also exist, such as those based on the finite difference and meshless methods, but they are not covered in this paper. Finally, we focus the discussions on the applications of computational plasticity to geotechnical engineering problems and not on the numerical algorithms themselves. The latter topic is covered extensively in a recent book (Borja 2012).

MULTISCALE ANALYSIS

Motivated by current imaging technology (see Fig. 1), it is now possible to characterize the inelastic deformation of a soil specimen at different scales. The three scales of interest are the microscopic, mesoscopic, and macroscopic scales. Loosely speaking, the microscopic scale is associated with the particle or pore scale, on the order of tens to hundreds of microns in dimension. Much smaller particles, such as clay particles, are governed by statistical physics and are best modeled by molecular/particle dynamics simulations and not by computational plasticity. The macroscopic scale is usually associated with a specimen deforming homogeneously. The mesoscopic scale is intermediate between the microscopic and macroscopic scales and allows some homogenization to be made over a region that is smaller than the specimen but larger than the particles.

The most common use of multiscale analysis in geotechnical engineering is in the areas of constitutive modeling and localization of deformation. It is generally recognized that the constitutive response of a geomaterial is significantly influenced by the microstructure of the material, particularly in the inelastic and near-failure regimes. Phenomenological plasticity models cannot describe the smaller-scale processes taking place within, for example, a specimen of sand, such as the grain-to-grain interaction and discrete motions of the particles. At sufficiently low stress levels, these smaller-scale processes can be homogenized and scaled upward to a higher level of representation without adverse effects. However, at a certain stress level any further upscaling may no longer be meaningful especially when deformation becomes inhomogeneous. Inhomogeneous deformation could manifest in the form of localized bulging of a sample, or in the form of localized shearing of particles over a very narrow zone called "shear band." With respect to the latter type of localized deformation, homogenization and upscaling are not meaningful when performed over a shear band because the thickness of the band is the same order of magnitude as the dimension of the particles (Mühlhaus and Vardoulakis 1987).

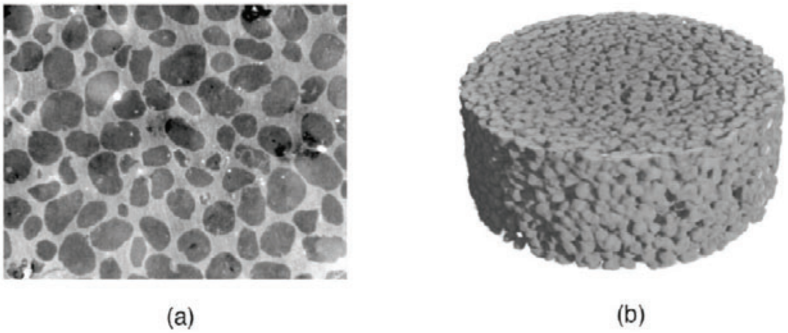

FIG. 1. Imaging at different scales: (a) bright field microscopy image of a uniform 20-30 Ottawa sand, mean grain size = 0.7 mm (image courtesy of E. Kavazanjian, Jr.); (b) computed tomography volume of an aggregated silty clay, diameter = 80 mm, height = 35 mm (Koliji 2008).

In principle, grain-to-grain processes may be upscaled to the continuum scale when the deformation is homogeneous. Inelastic deformation on the continuum level may be attributed to particle slips at contact points, particle rotation, as well as sliding and re-arrangement with neighboring particles. An enduring question is how many particles would be needed to realistically represent a continuum. The question arises because most simulations conducted in practice involve a collection of discrete particles placed inside a box that is subjected to boundary displacements representing an affine deformation (Ogden 1984). However, the boundaries of the box form rigid walls that severely constrain the movement of the particles. This makes the homogenized response strongly dependent on the number of particles placed inside the box. In order to avoid the influence of the boundary constraints, the homogenization should be conducted over a periodic cell that allows particles entering on one side of the cell to be mirrored by particles exiting on the opposite side of the same cell. The cell then becomes a true representative elementary volume (REV). Because particle movements are mirrored on the boundaries of the REV, the configuration is equivalent to an infinite number of particles with the REV defining the microstructure (Borja and Wren 1995).

A widely used criterion for detecting the inception of localized deformation in the form of a shear band is the bifurcation condition of Rudnicki and Rice (1975). The term "localization condition" has become synonymous to this criterion although there are other forms of localized deformation. In a nutshell, this localization condition finds a critical point in the loading history at which the displacement gradient field defines a band of discontinuity. The terms "inside the band" and "outside the band" have been used to distinguish between the displacement gradient fields on each side of the plane of discontinuity, but strictly speaking the localization condition does not define which side is "inside" and which side is "outside." However, the localization condition does define the orientation of the plane of discontinuity, if such plane exists.

Numerical algorithms are often used to detect the onset of localized deformation in realistic boundary-value problem simulations. To this end, the notion of "localization function" is very useful. Essentially, the localization condition is an eigenvalue problem aimed at finding a critical state of stress that allows a non-zero discontinuity, or "jump," in the displacement gradient field. The idea is analogous to buckling of a column under compression where the goal is to find other modes of deformation apart from the column remaining straight. In the case of shear band bifurcation, the jump in the displacement gradient field coincides with a certain coefficient matrix – a rank-two "acoustic" tensor as it is commonly known – becoming singular. The localization function is then the determinant of this acoustic tensor. The localization function is positive in the stable regime and vanishes when a discontinuity is detected.

The acoustic tensor has a product form of the tangent constitutive tensor and the unit normal vector to a potential plane of discontinuity. This tensor is not constant but depends on the unit normal vector chosen to represent any potential plane of discontinuity. To determine the vanishing of the localization function, it is necessary to search for the critical orientation of the plane of discontinuity. This entails searching for the unit normal vector that minimizes the determinant of the acoustic tensor, and the critical stress state at which this minimum value of determinant becomes equal to zero. When the localization function vanishes for the first time, the localization condition also identifies the orientation of the ensuing shear band. For geomaterials the orientation of the shear band depends strongly on the friction and dilatancy angles, both of which should be reflected in some form in the elastoplastic constitutive description.

Detection of a shear band appears easy but the analysis of bifurcation can actually become tricky when more complex constitutive models are used. To elaborate this point, we recall the notion of incremental nonlinearity. As a matter of definition, the material response is incrementally linear if the tangent constitutive tensor is the same irrespective of the direction of the imposed strain increment. For example, the response of an elastic material is incrementally linear (absolutely linear if linearly elastic) because the tangent constitutive tensor is the same irrespective of the strain increment. In contrast, the response of an elastoplastic material is incrementally nonlinear because the constitutive tangent tensor has two branches: a plastic branch when loading and an elastic branch when unloading. This leads to the question, which tangent constitutive tensor should be used when performing a bifurcation analysis, the loading branch or the unloading branch? For elastoplastic materials with two constitutive branches (i.e., loading and unloading branches), Rice and Rudnicki (1980) showed that the loading branch is more critical for shear band bifurcation. For higher-order incrementally nonlinear models the answer is not so obvious because of the multiplicity of the forms for the tangent constitutive tensor. For particulate materials where the overall response is obtained from a homogenization of the discrete particle responses, the constitutive tangent tensor can have well more than two branches.

In a recent paper, Borja et al. (2011) presented a combined experimental imaging-finite element modeling of shear band development in a sand specimen with imposed density heterogeneity. The shear band was induced by a strong density contrast imposed on a plane strain sample of sand (Fig. 2). They quantified the density variation using Computed Tomography (CT) imaging and Digital Image Processing (DIP), and input the results into a nonlinear finite element code for subsequent numerical simulation and shear band analysis. The finite element mesh and quantified density variation for the sand are shown in Fig. 2.

The mesh shown in Fig. 2 consists of constant strain triangular (CST) elements with a characteristic dimension of about 2.5 mm. To model the material response, Borja et al. (2011) used a variant of Nor-Sand model (Jefferies 1993) formulated in a finite deformation setting (Borja and Andrade 2006; Andrade and Borja 2006). Their simulation of the specimen response as a boundary-value problem is an example of mesoscopic modeling: it recognizes the inhomogeneous deformation of the soil sample and therefore treats the sample response as a structural response and not an element response. By closely following the imposed laboratory boundary conditions, the simulations have correctly predicted the location and orientation of the shear band, as shown in Fig. 3. An important conclusion emanating from their paper is that the mesoscopic simulation can resolve the true shear band even with a biased finite element mesh provided the density contrast is strong enough. This is illustrated in Fig. 3c where a biased finite element mesh favoring the development of the conjugate shear band still predicted the true (opposite) shear band.

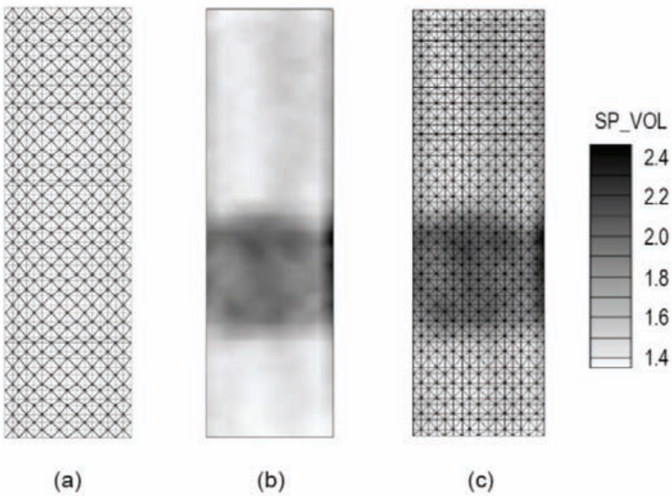

FIG. 2. Plane strain compression of a sand (137 mm tall by 39.5 mm wide by 79.7 m deep): (a) finite element mesh; (b) spatial distribution of specific volume; (c) finite element mesh with spatial distribution of specific volume. After Borja et al. (2011).

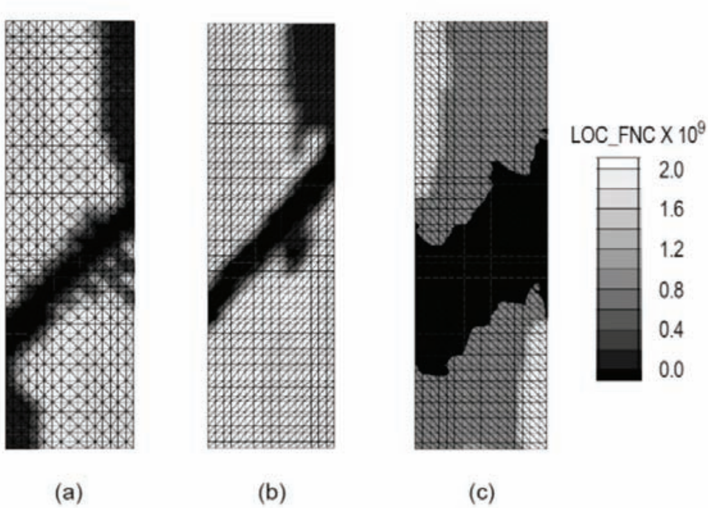

FIG. 3. Plane strain compression of a sand with strong density contrast: (a) unbiased finite element mesh; (b) biased mesh favoring the development of the true shear band; (c) biased mesh favoring the development of the conjugate shear band. After Borja et al. (2011).

FAILURE MODES AND CONTACT PROBLEM

In the context of nonlinear finite element analysis, modeling the post-localization response requires enhancing the finite element interpolation to resolve the intense deformation over the zone of discontinuity. To this end, it is useful to describe different kinematical styles of a discontinuity because, ideally, the finite element enrichment to the displacement field must reflect these kinematical styles. Shear band has been addressed prominently in the literature, although it is only a special case of a more general deformation band, which is a narrow zone of intense shear, compaction, and/or dilation (Borja and Aydin 2004; Borja 2004a). On a macroscopic scale the displacement field in a deformation band is continuous but the strain field is intense. Faults are highly damaged gouge zones where granulized particles roll and slide past each other even as the material outside this zone remains relatively undamaged (Scholz 1990). Fractures or cracks are much narrower zones of intense deformation, approaching a discontinuous displacement field where two surfaces either separate or slide past each other (Broek 1982).

Because of nearly overlapping definitions qualitative descriptions of failure modes are quite artificial, and actual mechanism of deformation could involve combinations of several far more complex processes. However, we distinguish between two mechanisms of localized deformation: a continuum mode in which the two sides of damage zone are in direct physical contact, and a separation mode characterized by a pair of traction-free surfaces. Deformation bands, faults, and frictional cracks are examples of continuum strain localization, opening mode fractures are examples of a separation mode. With this distinction in mind, two prominent finite element enhancement techniques can be mentioned: the assumed enhanced strain and the extended finite element methods. Both methods accommodate a zone of discontinuity to pass through the interior of finite elements (Borja 2008).

The assumed enhanced strain method has been developed primarily for continuum strain localization problems and not for opening mode fractures. In this method the slip on the band is assumed to be constant in each finite element (piecewise constant), which means that it is discontinuous across element boundaries. The tip of the band is usually a fracture process zone that the method smears throughout the element volume containing the band tip. Plasticity models are often used to represent the material behavior inside the band. Static condensation is employed to eliminate the slip degree of freedom prior to global assembly (Borja and Regueiro 2001; Regueiro and Borja 2001; Borja 2000). The assumed enhanced strain method is an example of a multiscale representation where the complex smaller-scale processes taking place inside the band are lumped and brought upwards to the scale of the finite elements.

The extended finite element method also embeds a discontinuity into the finite elements, but it does so one level higher: it takes a continuous distribution of slip across element boundaries. In contrast to the assumed enhanced strain method, which eliminates the slip degrees of freedom on the element level prior to global assembly, the extended finite element method determines the slip degrees of freedom globally, requiring the solution of a larger system of equations. The trade-off to higher computing costs is versatility: the method can accommodate not only contact conditions but also opening mode fractures. In addition, like the assumed enhanced strain method, the extended finite element method also has been used to accommodate frictional crack propagation with a variable coefficient of friction (Foster et al. 2007, Liu and Borja 2008, 2009).

As noted earlier, the assumed enhanced strain and extended finite element methods allow a discontinuity to pass through the interior of finite elements. This is a very important feature because in many problems, such as crack and shear band propagation, the zone of discontinuity is not a priori defined but is determined as the solution progresses. Therefore, it would not be possible to a priori align the sides of the finite elements to an unknown discontinuity. As an aside, adaptive mesh refinement has many problems in accommodating an evolving discontinuity, especially in plasticity, because the Gauss point information may have to be mapped from an old mesh to a new mesh, which is not a simple task. But if the zone of discontinuity is defined beforehand, then the sides of the finite elements can always be aligned to it and classic nonlinear contact mechanics algorithms (e.g. slave node-master segment) may be employed (Sanz et al. 2007; Wriggers 2002). In fact, there are many geotechnical problems where classic nonlinear contact mechanics algorithms alone would be sufficient, such as in the analysis of soil-pile interaction and soil-wall interaction where the interface between the two material groups is clearly defined.

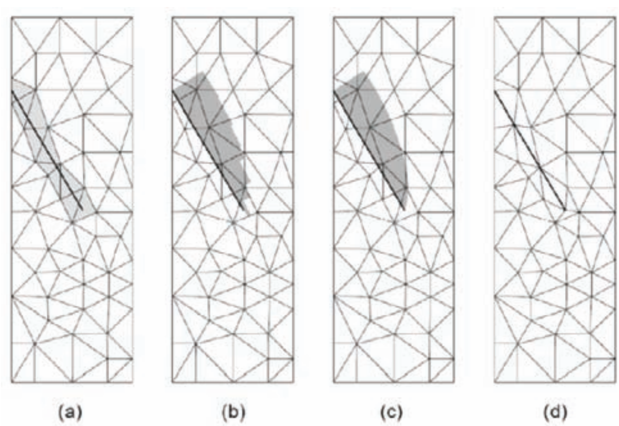

FIG. 4. Solid with an edge crack (heavy line): (a) shaded finite elements are enriched with slip degrees of freedom; (b) assumed enhanced strain calculates piecewise constant slips; (c) extended finite element method calculates piecewise linear slips; (d) classic nonlinear contact mechanics aligns element sides to the surface of discontinuity.

Figure 4 illustrates the underlying idea behind the enriched finite element techniques with embedded discontinuity. Figure 4a shows a crack passing through the interior of constant strain triangular elements (shaded elements). To enable the "localizing" elements to resolve the discontinuity, the displacement interpolation within the shaded finite elements is enriched. Figure 4b demonstrates the results of the enrichment provided by the assumed enhanced strain method: the calculated tangential slips are piecewise constant (the height of the shaded region is a measure of tangential slip). Figure 4c shows the piecewise linear slips calculated by the extended finite element method (because of the resolution, the slope appears continuous). Figure 4d illustrates the idea behind the classic nonlinear contact mechanics approach where the element sides are simply aligned to the surface of discontinuity (assuming the discontinuity is defined at the beginning of the solution).

Regardless of the numerical technique used for resolving the discontinuity, one must deal with the unilateral constraints imposed by the contact condition in the finite element solution. There exists a large body of literature addressing the computational aspects of contact problem using the finite element method. A challenging aspect concerns the enforcement of the contact condition, which inhibits interpenetration of the contact faces as well as requires that the contact pressure be nonnegative. If λ denotes the contact pressure and h is the gap function (i.e., the normal distance between two potentially contacting surfaces), then the contact constraints are given by the relations

$$\lambda \geq 0, \qquad -h \leq 0, \qquad \lambda h = 0. \tag{1}$$

The first two relations state that the contact pressure and gap function cannot be less than zero, while the third relation states that if $\lambda > 0$ then $h = 0$, and if $-h < 0$ then $\lambda = 0$. The above constraints are the classic Karush-Kuhn-Tucker (KKT) conditions in nonlinear programming (Borja 2012).

We now assume frictional contact (i.e., $h = 0$ and $\lambda > 0$) and denote the resolved tangential stress on the surface of discontinuity by t. The friction law then states that

$$f = t - \mu\lambda \leq 0, \tag{2}$$

where μ is the coefficient of friction. Equation (2) defines the yield function for the contact problem. Stick-slip conditions on sliding interfaces are then given by the compact expressions

$$\zeta \geq 0, \qquad f \leq 0, \qquad \zeta f = 0, \tag{3}$$

where ζ is the slip rate. We have a stick condition when $\zeta = 0$ and $f < 0$, and a slip condition when $\zeta > 0$ and $f = 0$. Note that Equation (3) is another set of KKT conditions, so frictional contact can be viewed as nothing else but two KKT conditions in nested form.

Three popular techniques are commonly used for enforcing the nested KKT conditions for contact problems. They are:

- Lagrange multipliers method
- penalty method
- augmented Lagrangian method

In the Lagrange multipliers method the contact pressure λ is taken as the Lagrange multiplier, and the solution satisfies the contact conditions exactly. The method is relatively easy to implement for smooth contact ($\mu = 0$), but is difficult to use for frictional contact ($\mu > 0$), see Liu and Borja (2008) for further discussions on this topic. In the penalty method the contact faces are allowed to penetrate each other by a small amount depending on the value of the penalty parameter. The penalty parameter is equivalent to an elastic spring between the contacting faces: the larger is the penalty parameter, the smaller is the interpenetration distance between the contacting faces. The penalty method is easy to implement for frictional contact, but does not satisfy the contact conditions exactly. The penalty parameter can be increased to reduce the interpenetration distance between the contacting surfaces, but this typically causes ill-conditioning of the coefficient matrix. Finally, the augmented Lagrangian method (Simo and Laursen 1992) provides a compromise between the Lagrange multipliers and penalty methods: it mimics the Lagrange multipliers method iteratively with an additional penalty term (the augmentation). The method does not require very large values of the penalty parameter, but the iterative nature of this approach makes it undesirable to use.

In most geotechnical problems it is usually sufficient to assume a constant coefficient of friction, whether the interface is a shear band consisting of granulized material or simply two bare rock surfaces rubbing against each other. However, there are problems in geophysics, particularly in seismology, that require consideration of a variable coefficient of friction. It is known that the coefficient of friction is influenced by the slip speed and maturity of contacts. It is also well known that the coefficient of friction during fault rupture could drop dramatically to very small values (Di Toro et al. 2004). Computational plasticity has matured to the point where these interesting features can now be incorporated into the numerical modeling of frictional contact.

The Dieterich-Ruina law (Dieterich 1978; Ruina 1983) is a widely used friction law characterizing a velocity- and state-dependent coefficient of friction. Figure 5 depicts the important features of this friction law. From a constant sliding velocity V_1, the slip velocity is increased instantaneously to V_2. The coefficient of friction μ increases instantaneously and then gradually decays to a lower value at this higher velocity. When the sliding speed is reduced instantaneously the coefficient of friction drops instantaneously, but over the course of time it picks up to a higher steady-state value. As noted in this figure, the Dieterich-Ruina friction law is expressed in terms of logarithmic functions of the two velocities.

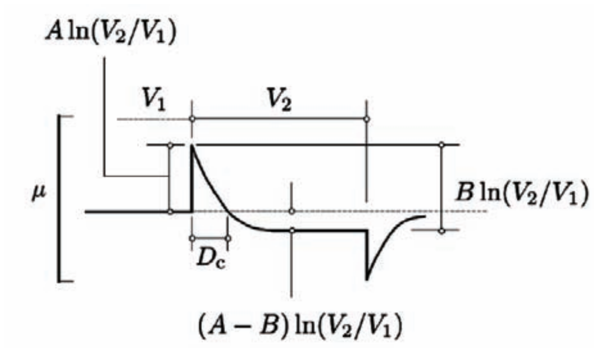

FIG. 5. Variation of coefficient of friction in a velocity stepping test. Note: $V_2 > V_1$.

As an example, we consider a plane strain compression testing of a rectangular block of San Marcos gabbro. This example was meant to recreate a two-dimensional version of the triaxial test described in Wong (1986). In two separate simulations, the specimen at first behaves elastically, and then it undergoes plastic yielding. Once a sufficiently high stress level has been reached the specimen experiences shear band bifurcation, at which point a surface of discontinuity is inserted into the volume to enhance the interpolation capability of finite elements. Slip weakening (Borja and Foster (2007) provides a transition mechanism from intact state to failed state. This is a good example where classic nonlinear contact mechanics cannot be used because the orientation and position of the surface of discontinuity are not initially known. Results of the simulations are shown in Fig. 6 using the assumed enhanced strain method (Foster et al. 2007) and the extended finite element method (Liu and Borja 2009). In the former method the background grid is made up of four-node quadrilateral elements enriched with piecewise constant slip degrees of freedom. In the latter method the background grid is made up of constant strain triangular elements (not shown) enriched with piecewise linear slip degrees of freedom. In both examples the interface is modeled with a variable coefficient of friction following the Dieterich-Ruina law. The extended finite element simulation figure shows bulk plasticity forming in the neighborhood of the slip surface, in addition to interface plasticity (by penalty method) due to sliding on the failure surface.

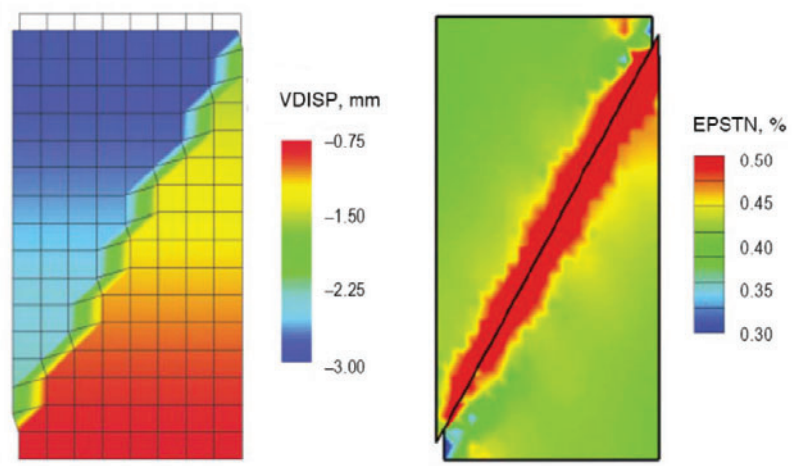

FIG. 6. Simulations of plane strain compression of San Marcos gabbro recreating a two-dimensional version of the triaxial test by Wong (1986). Left: assumed enhanced strain simulation by Foster et al. (2007); right: extended finite element simulation by Liu and Borja (2009).

MULTIPHYSICS PROCESSES

We next describe the application of computational plasticity to multiphysics problems, which include mechanical, hydrological, thermal, and biological processes, among others. In addition to the solid displacement that must be calculated in such problems, the formulation also consists of other field variables such as fluid pressure, temperature, and transport quantities. The field variables are linked by the conservation equations of mass, heat, momentum, and energy. To close the formulation of the problem, equations of state and constitutive laws are specified.

The field variables in a multiphysics formulation are often solved simultaneously. This is a big challenge in computational mechanics because the size of the problem grows very fast as more field variables are included in the formulation. Hence, it is important to distinguish between strong coupling, where the field variables cannot be uncoupled and must be solved simultaneously, and weak coupling, where the field variables can be uncoupled in some way, at least to reduce the burden of simultaneous equation solving.

Hydro-mechanical problems are an example where the coupling between the displacement and pore pressure fields is usually strong, and therefore the two field equations (balance of momentum and balance of mass) must be solved simultaneously. There have been several attempts to uncouple the displacement and pore pressure fields in a hydro-mechanical formulation through sequence of drained and undrained calculations (Kim 2010). However, this approach could potentially suffer from non-convergence depending on the sequence of calculations. In general, solving the displacement and pore pressure fields simultaneously is still the most reliable approach. Direct coupling between the displacement and pore pressure fields enters into the formulation through the effective stress equation. In some cases, such as in unsaturated soils, the pore pressure field could also enter into the constitutive formulation through the suction stress (Borja 2004b).

An "effective stress" is needed for the calculation of inelastic solid deformation in a hydro-mechanical continuum simulation. Note that the effective stress is not the same as the partial stress used in mixture theory. The first law of thermodynamics provides a definition for the effective stress: it is the stress that is energy-conjugate to the rate of deformation for the solid matrix. Borja and Koliji (2009) used this definition and developed the following expression for the effective stress in a mixture of solid-water-air exhibiting two porosity scales:

$$\sigma_{ij} = \sigma'_{ij} - B\overline{p}\delta_{ij}, \tag{4}$$

where σ_{ij} and σ'_{ij} are the total and effective Cauchy stress tensors, respectively (continuum mechanics convention), B is the Biot coefficient, δ_{ij} is the Kronecker delta, and $\overline{p}$ is an overall mean pore pressure determined from the expression

$$\overline{p} = \psi\overline{p}_{\mathrm{mic}} + (1-\psi)\overline{p}_{\mathrm{mac}}, \tag{5}$$

where ψ is the micro-pore fraction, i.e., the fraction of the total pores occupied by the micropores, and $\bar{p}_{\text{mic}}$ and $\bar{p}_{\text{mac}}$ are the mean pore pressures in the micropores and macropores, respectively, which in turn are determined from the local pore air and pore water pressures weighted according to local degrees of saturation. This expression for the effective stress reduces to more familiar forms under suitable assumptions. For example, one recovers Terzaghi's effective stress for a one-porosity material when the degree of saturation is 100% and the Biot coefficient is $B = 1$.

In terms of the effective stress, a general constitutive equation for the stress-strain behavior of the solid skeleton accounting for all relevant multiphysics processes can be written in differential form as

$$d\sigma'_{ij} = c^{\text{e}}_{ijkl}(d\varepsilon_{kl} - d\varepsilon^{\text{p}}_{kl} - d\varepsilon^{\text{th}}_{kl} - d\varepsilon^{\text{tr}}_{kl} - d\varepsilon^{\text{b}}_{kl} - \ldots), \tag{6}$$

where c^{e}_{ijkl} is the tensor of elastic moduli, $d\varepsilon_{kl}$ is the total strain differential, and superscripts p, th, tr, b, etc., denote the differential strains attributed to plastic, thermal, phase change transition, biological, etc., effects. The above constitutive equation is similar to those presented by Liu and Yu (2011) and Laloui and Fauriel (2011) for problems with thermal and biological contributions, respectively. We remark that even in the presence of thermal, phase change transition, biological, and other factors, the general expression for the stress increment is given by the incremental elastic constitutive equation.

As noted earlier, multiphysics problems are challenged by the large number of unknown degrees of freedom to solve, even for simple problems. Consider, for example, a simple solid deformation-fluid diffusion problem. A commonly used approach for solving this problem is by mixed finite element formulation requiring that the finite element nodes contain both solid displacement and fluid pressure degrees of freedom (so-called u/p formulation). However, a simple quadrilateral element with equal (bilinear) interpolation for the displacement and pressure degrees of freedom is known to be unstable in the undrained limit (incompressible and nearly incompressible regimes). By unstable elements we mean that extreme pore pressure oscillations may be noted in the undrained limit, accompanied by mesh locking. Elements that are known to be stable typically require a displacement interpolation that is one level higher (biquadratic) than the pressure interpolation (bilinear). In 3D analysis, one "stable" hexahedral element alone, with 27 displacement nodes and 8 pressure nodes, results in 89 degrees of freedom. It is no wonder that a three-dimensional coupled hydro-mechanical analysis is still a distant option for many modelers because of the enormous cost of equation solving.

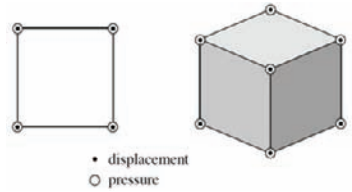

FIG. 7. Stabilized low-order finite elements for coupled solid-deformation/fluid-diffusion. Left: 4-node quadrilateral; right: eight-node hexahedral. After White and Borja (2008).

Low-order mixed finite elements with equal-order interpolation (bilinear) for displacement and pore pressure fields, such as those shown in Fig. 7, are desirable provided they are "stabilized" in the incompressible and nearly incompressible regimes, i.e., the pore pressure oscillations are arrested and the mesh does not exhibit locking tendencies. To elaborate the advantages of low-order elements further, consider a finite element mesh employing continuous biquadratic-displacement/ bilinear-pressure quads (Q9P4), and a mesh with equal-order interpolation quads (Q4P4). The first element possesses 22 degrees of freedom and is known to be stable, while the second element has 12 degrees of freedom and is known to be unstable – unless a stabilized formulation is employed. The two elements are comparable in the sense that they produce the same order of pressure interpolation. The Q9P4, however, leads to algebraic problems with many more degrees of freedom. As the number of elements in each mesh grows, a simple argument shows that the total number of unknowns in the two meshes quickly approaches a ratio of 3:1. If we consider the equivalent three-dimensional situation, this ratio approaches 614:1. The bandwidth of the coefficient matrix will grow similarly. Further computational savings can also be associated with the quadrature rule employed: the Q9P4 element typically requires 3×3 Gauss-quadrature, while the Q4P4 element only needs 2×2 quadrature. Clearly, lower-order elements can significantly alleviate the computational burden in a multiphysics finite element analysis.

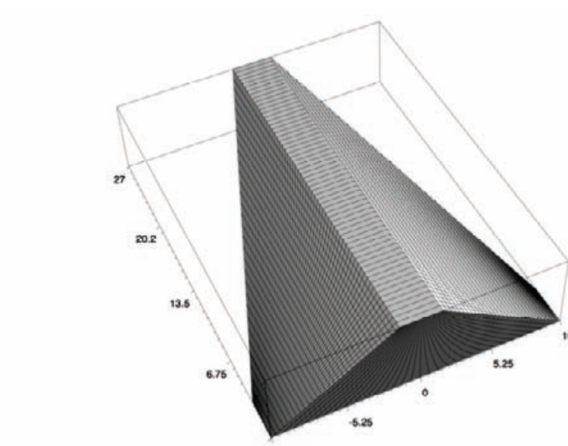

FIG. 8. Finite element mesh (looking from top) for hydro-mechanical simulation of 3D embankment response to reservoir filling and rapid drawdown: the mesh has a total of 54K low-order hexahedral elements for a total of 235K degrees of freedom. The base is a valley and the embankment has a plane of symmetry. After Borja and White (2010b).

Recent studies have shown that low-order mixed finite elements are amenable to numerical stabilization, making them suitable for multiphysics/multifield analysis. White and Borja (2008) proposed stabilized low-order finite elements, such as those shown in Fig. 7, for fully coupled flow and geomechanics. The stabilization is based on polynomial pressure projection technique that quantifies the inherent deficiency of the equal-order bilinear interpolation of the displacement and pressure degrees of freedom. These low-order elements perform just as well as the more expensive higher-order elements, but with added efficiency. An example of a high-resolution mesh for conducting 3D hydro-mechanical finite element analysis is shown in Fig. 8. Borja and White (2010a,b) successfully used the stabilized elements to simulate the response of an unsaturated embankment to reservoir filling and rapid drawdown, and to predict the failure of a slope due to rainfall infiltration.

SEISMIC RESPONSE OF GEOTECHNICAL STRUCTURES

The predictive capability of a mechanical model is traditionally assessed by how well it can reproduce the acceleration-time history response of a seismically loaded structure for purposes of estimating the amount of overstress in this structure. However, it is well known that even an equivalent linear model can reproduce the acceleration history response of a soil deposit reasonably well, even though the soil may be responding well beyond its elastic range. This suggests that the acceleration response is a weak test of the predictive capability of a mechanical model. A stronger test, which entails predicting the inelastic deformation of a structure for performance-based engineering, may be warranted in some cases.

Calculating ground displacements is an important aspect of seismic design and is one of the most challenging applications of computational plasticity. Elastic models and equivalent linear soil models predict zero permanent deformation, so they are not appropriate for the analysis of the performance of a geotechnical structure based on deformation. From a historical perspective, Newmark (1965) proposed to model a slope as a rigid sliding block and evaluated the permanent displacements induced by an earthquake from the sliding displacements of this block. In his procedure, accelerations induced by earthquake shaking produce a destabilizing force leading to sliding episodes and the accumulation of permanent sliding displacements. The original Newmark procedure calculates the permanent displacements using two parameters: a yield acceleration and the acceleration-time history of the rigid mass. Sliding episodes begin when the actual acceleration exceeds the yield acceleration and continues until the velocity of the sliding mass and foundation coincide.

The original rigid sliding block procedure contains a number of features that may not be consistent with reality, including the assumptions that (a) the sliding mass responds in a rigid manner; (b) the soil is rigid-perfectly plastic; and (c) inelastic deformation is concentrated on a discrete failure plane. Of these three assumptions, the third one is the most suspect because it is known that even without a discrete failure plane inelastic distributed (or continuum) deformation could accumulate in soils. On the other hand, theory of plasticity is appropriate for modeling distributed inelastic deformation. The question is whether a particular plasticity model would be robust enough to accommodate the important mechanisms triggered by seismic shaking, and whether it is simple enough to code as well as to use. Simplicity includes not only the computer implementation of the model per se, but also the identification of parameters needed for material characterization.

There have been numerous hysteretic plasticity models for earthquake response simulations of non-liquefiable soils. In this paper, we shall focus on a model developed by the author that has the following desirable features: (1) it is simple enough to be implemented into computer codes, (2) its parameters can be determined easily in practice, and (3) it is predictive. The model is the bounding surface plasticity theory with no elastic region (Borja and Amies 1994) formulated to accommodate 3D distributed deformation. The formulation assumes an additive decomposition of the stress tensor into inviscid and viscous parts. The bounding surface formulation is used for the inviscid part, while a damping matrix that varies linearly with the elastic component of the stiffness matrix is used for the viscous part. The model has been implemented into a nonlinear finite element code SPECTRA and requires the same set of material parameters as the widely used equivalent linear code SHAKE. The model also has been shown to be at least as accurate as SHAKE for predicting the acceleration responses of horizontally layered soil deposits (Borja et al. 1999; 2000). Because the model is fully nonlinear, it can be used to calculate the distributed plastic deformation of 3D soil structures.

Downhole arrays, such as the ones that existed until the 1990's in Lotung, Taiwan (Borja et al. 1999;2000), are useful for assessing the predictive capability of a dynamic site response model. By dynamic response we mean the acceleration of a soil deposit to bedrock excitation. Downhole arrays consist of accelerometers installed vertically to measure the signatures of vertically propagating seismic waves. In principle, the acceleration-time response can be integrated twice to obtain the displacement-time history. However, accelerograms are not pure and contain noise and baseline offsets that produce unrealistic displacement-time history responses when numerically integrated. Algorithms for baseline corrections are available, but they do not guarantee that the calculated strains in the soil deposit are the "correct" strains.

Borja and Sun (2007; 2008) showed that despite the impurities in the recorded accelerograms, an elastoplastic finite element solution is still a viable approach for calculating earthquake-induced deformations in soil deposits, provided that the analyst focuses on the relative deformations and not the absolute ones. For example, they showed that the permanent displacement of the ground surface relative to the bedrock is not influenced by noise or baseline offsets in the imposed bedrock motions. Since the performance of a structure is determined primarily by the inelastic strains in the structure and not by the absolute displacement of the structure per se, one may argue that even in the presence of impurities in the ground motion, elastoplastic finite element solutions can still be used for performance-based seismic design.

There has been much progress in the development of a continuum model for post-liquefaction deformation of fully saturated soils. Most of the models use bounding surface plasticity theory based on the effective stress, combined with Biot's dynamic theory for fully saturated porous media. Because phase transition processes, including liquefaction, flow, and reconsolidation, dominate the mechanisms of fluid-saturated soil deformation, not one model is sufficient for the entire sequence of deformation, and several models are typically necessary to cover the entire range of soil deformation. Plasticity theory is generally limited to the solidified state, and viscoplasticity is often used for the liquefied state. The body of literature pertaining to cyclic plasticity modeling of liquefaction phenomena is enormous, and space limitation does not allow that they be covered extensively in this paper. We simply refer the readers to a recent survey by Zhang and Wang (2011) on the constitutive modeling of post-liquefaction deformation of soils.

As soils liquefy, they flow like fluid until they reach a stable configuration where the particles settle out and reconsolidate. Reconsolidation is the least investigated phase transition process, which explains why mechanical models for this phenomenon are scarce. In a recent article (Moriguchi 2009), a Bingham fluid constitutive formulation has been proposed to represent both solid-like and fluid-like soil behaviors, suggesting their potential for modeling the phase transition process. A notable feature of the formulation is the single form of the stress tensor that is valid for the solid and fluidized flow regimes. In a nutshell, the Bingham fluid is a viscoplastic non-Newtonian fluid triggered when the stress point goes outside a yield surface. Solidification is reached in the limit as the viscosity coefficient reaches infinity. In a frictional material, one can recover the correct angle of repose of the re-solidified soil according to the yield criterion of the model. Tuefelsbauer et al. (2011) demonstrated that the Bingham fluid model is consistent with granular flow mechanics from the standpoint of impact force dynamics.

CONCLUDING REMARKS

Computational plasticity is now commonly used for assessing the performance of geotechnical structures. Its impact in geotechnical engineering has been more significant than in other areas because soils and rocks have very limited elastic range, and nearly all deformations experienced by these materials are inelastic. This paper discussed the applications of computational plasticity to four different but overlapping research topics that have enjoyed considerable activity in recent years. It must be noted, however, that classical theory of plasticity is not the only platform for modeling inelastic deformation. In particular, theory of hypoplasticity is well accepted in other parts of the world, and its applications to geomaterials have also been significant. More advanced models such as gradient, Cosserat, and non-local models have also been used for geomaterials with some success. It may be argued, however, that the mathematical theory of plasticity has enjoyed unparalleled popularity in geotechnical engineering. The development of advanced imaging and sensing technology, computational hardware, and powerful numerical algorithms will only make this theory more popular and useful for analyzing difficult geotechnical engineering problems, whether it is in research or in practice.

ACKNOWLEDGMENTS

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Column-Supported Embankments: Settlement and Load Transfer

George Filz[1], F. ASCE, P.E., Joel Sloan[2], M. ASCE, P.E., Michael P. McGuire[3], M. ASCE, P.E., James Collin[4], F. ASCE, and P.E., Miriam Smith[5], M. ASCE, P.E.

[1]Professor, Civil and Environmental Engineering, Virginia Tech, Blacksburg, VA 24061; filz@vt.edu
[2]Major, US Air Force, Kunsan Air Base, Republic of Korea; sloanja@vt.edu
[3]Research Associate, Civil and Environmental Engineering, Virginia Tech, Blacksburg, VA 24061; mcguirem@vt.edu
[4]The Collin Group Ltd., Bethesda, MD 20814; jim@thecollingroup.com
[5]Principal, AMEC E&I, Inc., Reno; mismit17@vt.edu

ABSTRACT: Column-supported embankments (CSEs) can reduce settlements, improve stability, and prevent damage to adjacent facilities when embankments are constructed on ground that would otherwise be too weak or compressible to support the new load. Geosynthetic reinforcement is often used to help transfer the embankment loads to the columns in CSEs. This paper addresses three important design issues for CSEs: (1) the critical height above which differential settlements at the base of the embankment do not produce measurable differential settlements at the embankment surface, (2) the net vertical load on the geosynthetic reinforcement in the load transfer platform at the base of the embankment, and (3) the tension that develops in the geosynthetic reinforcement. Based on bench-scale tests, field-scale tests, and case history data, the critical height was found to be a linear function of the column spacing and the column diameter. The net vertical load that acts down on the geosynthetic reinforcement can be determined using the load-displacement compatibility method, including determination of the limiting stress distribution at the base of the embankment by a generalized form of the Adapted Terzaghi Method, which accommodates any column or pile cap shape, any repetitive column arrangement, and different soil types in the load transfer platform and the overlying embankment fill. The tension in the geosynthetic can be calculated using a generalized form of the parabolic method, which incorporates stress-strain compatibility and which accommodates rectangular and triangular column arrangements and biaxial and radially isotropic geogrids.

INTRODUCTION

When embankments are constructed on ground that is too weak or compressible to adequately support the load, columns of strong material, e.g., driven piles, vibro-concrete columns, stone columns, deep-mixed columns, and other types of columns, can be installed in the soft ground to provide the necessary support. Often, a load transfer platform consisting of layers of compacted coarse-grained soil and geosynthetic reinforcement is used to help transfer loads from the overlying embankment, traffic, and/or structure to the columns. Advantages of column-supported embankments (CSEs) include rapid construction, small vertical and lateral deformations, and low impact on adjacent facilities, which could otherwise experience undesirable deformations due to the new embankment load. Potential disadvantages of CSEs include relatively initial high cost and uncertainties about design procedures. Alternatives to CSEs include excavation and replacement, staged construction with prefabricated vertical drains, and lightweight fill. CSEs can be a good choice when rapid construction, strict control of total and differential settlements, and/or protection of adjacent facilities are important design objectives.

A schematic cross-section of a CSE is shown in Figure 1. If the load transfer platform includes geosynthetic reinforcement, the technology can be referred to as a geosynthetic-reinforced column-supported embankment (GRCSE). In this paper, GRCSEs and CSEs are both referred to as CSEs.

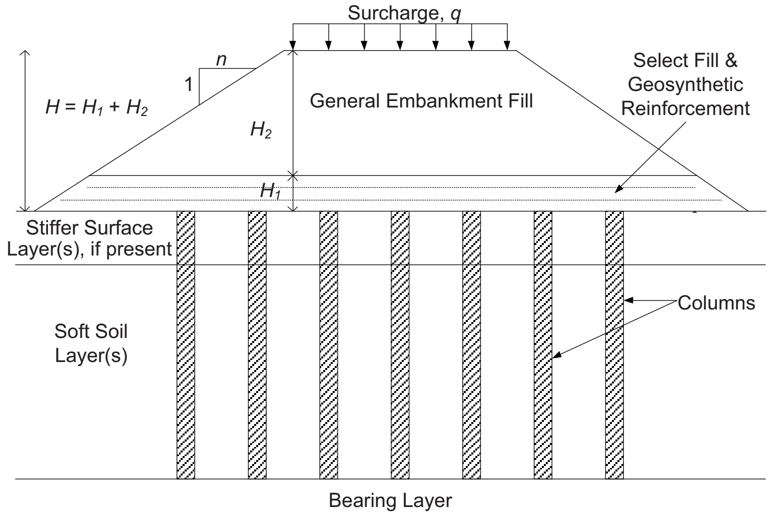

FIG. 1. Schematic diagram of CSE cross-section.

Designing a CSE includes design of the columns, design of the load transfer platform, and design of the embankment. Important considerations for design of the CSE components and system include the load carrying capacity of the columns, the magnitude of the net vertical load acting on the geosynthetic, the resulting tension in the geosynthetic, the total and differential settlements that can occur at the embankment surface, and the potential for lateral spreading and instability of the embankment. Analysis and design procedures for some of these aspects of CSE performance are relatively well established. However, published literature includes a wide range of methods for estimating the vertical load acting on the geosynthetic and the tension that develops in the geosynthetics, e.g., see the comparisons by McGuire and Filz (2008). In addition, consensus has not yet been reached on the critical embankment heights that are necessary to limit differential settlements at the surface of the embankment.

The purpose of this paper is to summarize key findings and developments from recent research on CSEs, particularly the following three investigations, which were undertaken to address the design uncertainties mentioned above:

- Smith (2005) used numerical analyses to investigate load transfer mechanisms at the base of CSEs. This led to development of the load-displacement compatibility method to calculate the distribution of load among the columns, the soil between columns, the geosynthetic reinforcement, and the base of the embankment above the columns and between the columns.
- McGuire (2011) used bench-scale laboratory tests, case history data, and numerical analyses to investigate the critical height that limits differential settlement at the embankment surface.
- Sloan (2011) used instrumented field-scale tests to investigate critical height, vertical load acting on the geosynthetic reinforcement, and tension in the geosynthetic reinforcement.

In this paper, the term "critical height" is defined as the embankment height above which differential settlements at the base of the CSE do not produce measurable differential settlement at the embankment surface. This definition is similar to Naughton's (2007) use of critical height to refer to the vertical distance from the top of the pile caps to the plane of equal settlement in the embankment. Other authors use critical height in other ways, e.g., Horgan and Sarsby (2002) and Chen et al. (2008) use critical height to refer to the height above which all additional loads due to fill and surcharge are distributed completely to the pile caps.

This paper is organized in the following sections: Bench-Scale and Field-Scale Experiments, Critical Embankment Height, Load-Displacement Compatibility Method, Generalized Adapted Terzaghi Method for the limiting load distribution on the base of the embankment, Generalized Parabolic Method for tension in the geosynthetic, and Summary and Conclusions. The findings presented in this paper are incorporated in a comprehensive design procedure described by Sloan et al. (2012). Due to page limitations, a review of the CSE literature is not provided here. CSE literature reviews are provided by Smith (2005), McGuire (2011), and Sloan (2011).

BENCH-SCALE AND FIELD-SCALE EXPERIMENTS

This section describes recent bench-scale and field scale-experiments by McGuire (2011) and Sloan (2011).

Bench-Scale Tests

The bench-scale experiments were conducted in a 2-ft diameter tank with columns that can be moved up into a soil layer inside the tank. Photographs of the test equipment are in Figures 2, 3, and 4.

The bench-scale tests were performed using a dry, clean, uniform, medium-grained, subangular sand. The sand was pluviated into the sample tank, zero readings were taken, and then the columns were penetrated up into the sand with periodic readings of the displaced shape of the sand surface. Parameters that were varied during the bench-scale tests include:

- Column diameter ranging from 0.75 in. to 3.0 in.
- Column center-to-center spacing, ranging from 3.5 in. to 7.0 in., for columns in a square array.
- Relative density of the sand, ranging from about 70% to 100%.
- Height of the sand layer, ranging from about 1.2 in. to 10.3 in.
- Layers of model geosynthetic reinforcement, ranging from zero to two.
- Vacuum pressure in the sand, ranging from zero to 3 psi.

FIG. 2. Bench-scale test apparatus showing soil tank above the bench and 5 by 5 array of columns below the bench.

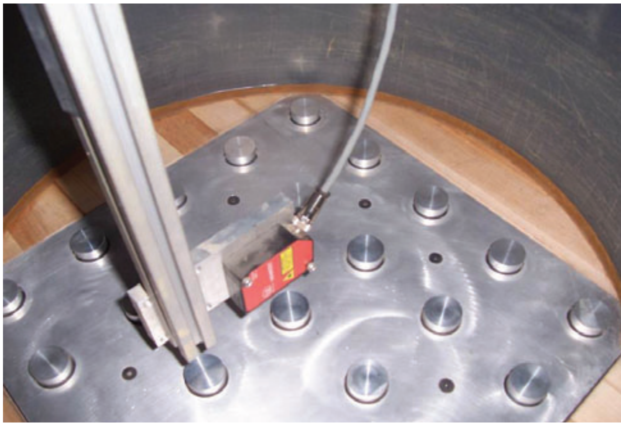

FIG. 3. View inside soil tank of bench-scale test apparatus showing columns and laser profilometer.

FIG. 4. View inside soil tank of bench-scale test apparatus showing differential settlement of sand above columns.

For a given column diameter and spacing, a series tests was performed with increasing height of the sand layer. The results permitted determining the critical height above which differential movement at the base of the sand layer did not produce any measurable differential movement of the sand surface. Altogether, 120 column-array tests were conducted to determine the critical heights for five combinations of column diameter and column spacing. In addition, 63 single column tests were conducted for detailed investigations of the column load and deformed shape of the sand surface for a wide range of parameter values. The test equipment, materials, procedures, data acquisition, data reduction, parameter variations, and test results are described in detail by McGuire (2011).

Key findings from the bench-scale tests include:

- The critical height depends on the spacing and diameter of the columns.
- Over the range of relative densities investigated from about 70% to 100%, the relative density of the sand did not measurably affect the critical height.
- Geosynthetic reinforcement reduced the magnitude of differential settlements for sand layers below the critical height. Geosynthetic reinforcement did not measurably affect the critical height.
- Application of vacuum pressures up to 3 psi did not measurably affect the deformed shape of the soil surface.

McGuire (2011) proposed that the critical height, H_{crit}, is a function of the column diameter, d, and the distance, s', from the edge of the circular column to the farthest point from the center of a column-centered unit cell. The parameters d and s' are shown in Figure 5 for a variety of CSE configurations. McGuire's (2011) experimental data are shown in Figure 6. The trend line through the data, which is also consistent with McGuire's (2011) 3-D numerical analyses, is given by:

$$H_{crit} = 1.15s' + 1.44d \tag{1}$$

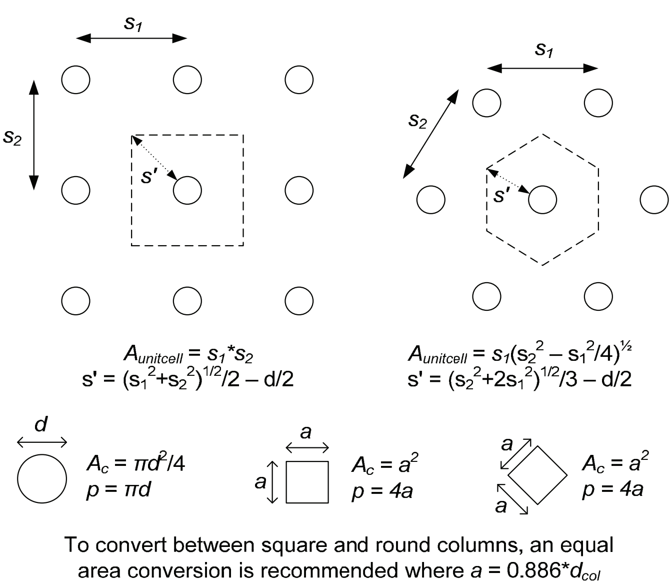

FIG. 5. Definition sketch for inputs to critical height determination and Adapted Terzaghi Method

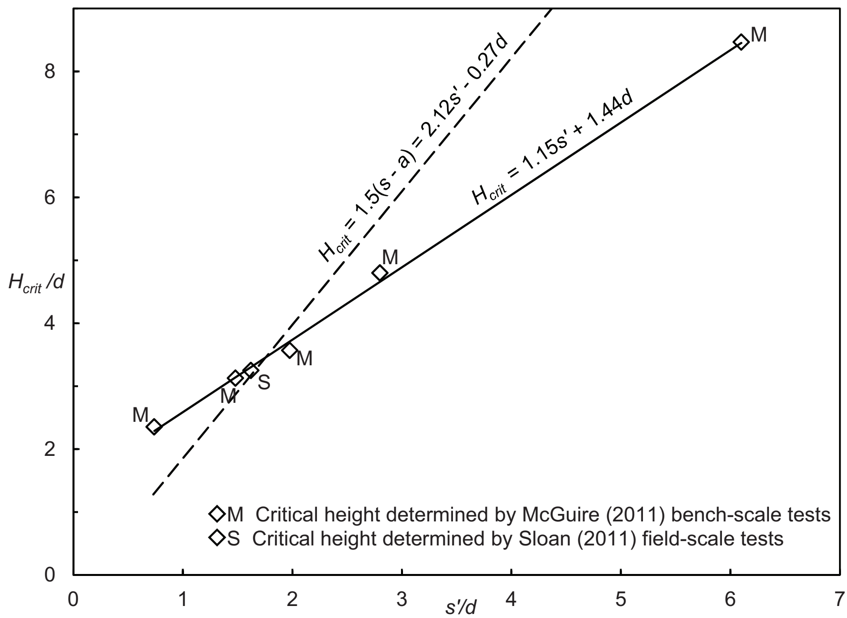

FIG. 6. Critical height from tests by McGuire (2011) and Sloan (2011).

For comparison, the conventional approach of relating H_{crit} to the clear spacing between square pile caps in a square array is also shown in Figure 6, with a typical proportionality factor of 1.5 such that $H_{crit} = 1.5(s - a)$, where s = the center-to-center column spacing and a = the side length of a square pile cap. Substituting the relationships shown in Figure 5, the conventional approach for pile caps in a square array can be expressed as $H_{crit} = 2.12s' - 0.27d$. Figure 6 shows that the conventional approach is unconservative for low values of s'/d and conservative for high values of s'/d, according to the results of the bench-scale experiments.

Field-Scale Tests

The field-scale CSE tests were conducted by placing short concrete columns on a concrete slab, surrounding the columns with geofoam, constructing the load transfer platform above the columns and geofoam, placing the remainder of the embankment above the load transfer platform, and then dissolving the geofoam to remove support between the columns. Photographs of the field-scale tests are shown in Figures 7 through 11.

FIG. 7. CSE test facility with 9-column array.

FIG. 8. CSE test facility with geofoam and geofoam dissolver delivery system.

FIG. 9. Reinforced fill placement.

FIG. 10. Loader and vibrating roller used for fill placement and compaction.

FIG. 11. Differential settlement of CSE Test #5 after dissolving geofoam and applying traffic load.

Lifts of well-graded gravel consisting of crushed rock were placed and compacted for both the load transfer platform and for the overlying embankment fill. Three to five layers of biaxial polypropylene geogrid were used in the load transfer platforms. The surface of the embankment was surveyed several times, including immediately after placing and compacting the final lift of embankment fill, after dissolving the geofoam, and after applying traffic loading using a small rubber-tired loader. One test was performed using 2-ft diameter columns in a 10-ft center-to-center square arrangement with about 4 ft of fill above the top of the columns. For this test, the embankment surface experienced dramatic differential settlements upon dissolving the geofoam, which demonstrated that this embankment was well below the critical height. Then a series of four tests were performed with fill heights ranging from 4 ft to 7.5 ft to determine the critical height for 2-ft diameter columns in a 6-ft center-to-center square arrangement.

Instrumentation and monitoring for the field-scale tests included load cells on the center column, strain gages and lead wire extensometers on the geosynthetic reinforcement, pressure cells in the load transfer platform, settlement profiler to measure the settlement of horizontal tubes embedded in the load transfer platform, and LIDAR and total station surveys of the embankment surface. The test facility, equipment, materials, procedures, data acquisition, data reduction, parameter variations, and test results are described in detail by Sloan (2011).

Key findings from the field-scale tests include:

- The critical height before trafficking for the 2-ft diameter columns in a 6-ft center-to-center square arrangement was 6.5 ft. The critical height for this arrangement after trafficking with the rubber-tired loader was 7.5 ft.
- The Adapted Terzaghi Method (Russell and Pierpoint 1997 and Russell et al. 2003) for determining the limiting vertical load acting on the geosynthetic reinforcement is consistent with the results from the field-scale tests.
- The parabolic method modified to incorporate stress-strain compatibility (Filz and Smith 2006) for determining tension in the geosynthetic reinforcement is consistent with the results from the field-scale tests.
- The load-displacement compatibility method is consistent with the results from the field-scale tests before dissolving the geofoam, when the load is shared between columns and geofoam.

The critical height from the field-scale tests, which is shown in Figure 6, is in good agreement with the trend from the bench-scale tests by McGuire (2011). However, the field scale test was conducted at an s'/d ratio where McGuire's (2011) trend line intersects the conventional approach with $H_{crit} = 1.5(s - a)$, so the field-scale test does not discriminate between McGuire's (2011) trend line and the conventional approach. It would be useful to perform additional field scale tests at different s'/d ratios to determine which relationship applies at field scale. However, additional information from other investigations can be brought to bear on the question, as discussed in the next section.

CRITICAL EMBANKMENT HEIGHT

Avoiding differential settlement at the surface of a CSE is often important, for example, to provide good ride quality and to prevent distress to overlying structures. Factors that influence differential surface settlements include column spacing, column diameter, embankment height, quality of subgrade support relative to column stiffness, and loading acting on the embankment surface. For example, differential surface settlement is likely for a relatively low embankment with wide column spacing and poor subgrade support. Differential surface settlement is unlikely for a high embankment with close column spacing and good subgrade support.

For CSEs without subgrade support, McGuire (2011) found that the critical embankment height depends on the column diameter and spacing, and it is not significantly affected by the relative density of the embankment fill or the use of geosynthetic reinforcement in the load transfer platform. The critical height from Sloan's (2011) field-scale tests is in good agreement with McGuire's (2011) findings. In addition, substantial information is also available from the published literature for other laboratory-scale experiments, other field-scale tests, centrifuge experiments, and full-scale field case histories. In some cases, systematic experiments were done to determine the critical height. In other cases, the critical height was not determined, but differential surface settlements were either reported or not reported. This information is presented in Figure 12, and the key for the data sources in Figure 12 is in Table 1.

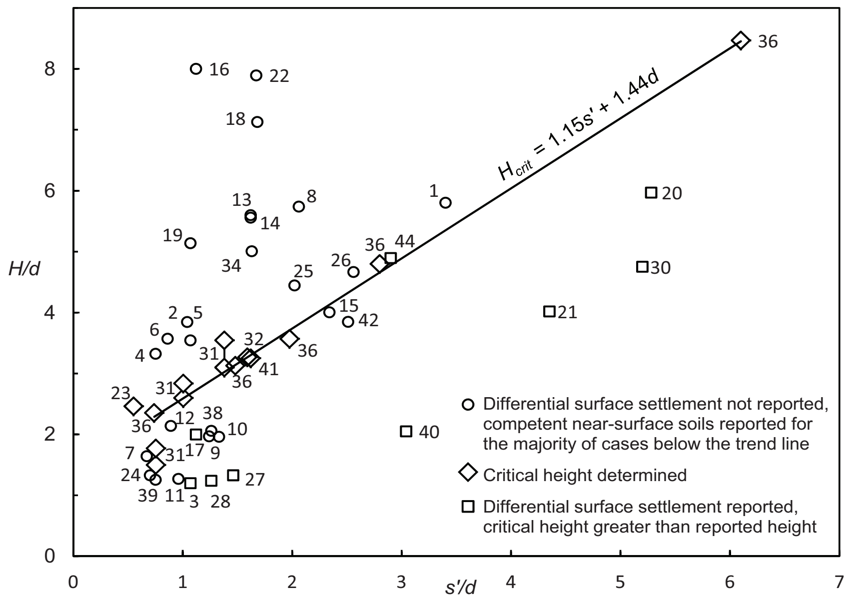

FIG. 12. Impact of CSE geometry on differential surface settlement.

Table 1. Sources for Data Points in Figure 12

Source Number	Reference	Nature of Study and Other Key Descriptors
1	Alexiew (2000, 1996)	Case history
2	Alexiew (2000)	Case history, triangular column array
3	Camp and Siegel (2006) and S&ME (2004)	Case history, triangular column array
4,5	Chen et al. (2010)	Case histories
6	Chen et al. (2010)	Case history, triangular column array
7	Chin (1985)	Case history, triangular column array, no reinforcement
8	Collin et al. (2005)	Case history, triangular column array
9	Gwede and Horgan (2008)	Case history
10	Habib et al. (2002)	Case history, triangular column array
11	Hite and Hoppe (2006)	Case history
12	Jones et al. (1990)	Case history
13	Liu et al. (2007)	Case history
14	Livesey et al. (2008)	Case history
15	Maddison et al. (1996)	Case history, triangular column array

16,17	Miki (1997)	Case history, no reinforcement
18	Pearlman and Porbaha (2006)	Case histories
19	Russell and Pierpoint (1997)	Case history, no reinforcement
20	Ryan et al. (2004)	Case history
21	Ryan et al. (2004)	Case history
22	Smith (2005), Stewart et al. (2004)	Case history, triangular column array, no reinforcement
23	Ting et al. (1994)	Case history, triangular column array, no reinforcement
24	Ting et al. (1994)	Case history, no reinforcement
25	Wood et al. (2004)	Case history
26	Abdullah and Edil (2007)	Field-scale experiments
27,28	Almeida et al. (2008, 2007)	Field-scale experiments, no subgrade support
30	Chew et al. (2004)	Field-scale experiments, no subgrade support
31	Demerdash (1996)	Laboratory-scale experiments, no subgrade support
32	Ellis and Aslam (2009a,b)	Centrifuge experiments, no reinforcement
34	Hossain and Rao (2006)	Field-scale experiments, no reinforcement
36	McGuire (2011)	Laboratory bench-scale experiments with and without reinforcement
38	Quigley et al. (2003)	Field-scale experiments, triangular column array
39	Rogbeck et al. (1998)	Field-scale experiments
40,41	Sloan (2011)	Field-scale experiments, no subgrade support
42	Van Eekelen et al (2008)	Field-scale experiments, no reinforcement
44	Villard et al. (2004)	Field-scale experiments, triangular column array, no subgrade support

Note: Unless otherwise indicated, the sources in Table 1 are for CSEs with geosynthetic reinforcement near the base of the embankment, the columns or pile caps are in a square or rectangular array, and some level of subgrade support is present.

The open diamond symbols in Figure 12 are from experiments and a case history for which the critical height could be determined (source numbers 23, 31, 32, 36, and 41). Some of these sources include more than one CSE geometry, so that a total of ten CSE geometries are represented in Figure 12. The results from source 31 (Demerdash 1996) produce a range of critical heights for each CSE geometry, as represented by a pair of open diamonds connected by a vertical line in Figure 12. Nine of these ten cases are in good agreement with the critical height trend line shown in Figure 12 and provided in Equation (1). One exception is an experiment by Demerdash (1996) for which the pile caps were close to the walls of the test device, and this may have produced a relatively low critical height. Most of the open diamond symbols represent cases without subgrade support and without traffic loading. Subgrade support tends to reduce differential surface settlements and traffic loads tend to increase differential surface settlements.

Figure 12 also shows results from studies whose purpose was not to determine critical height, but for which the authors either did or did not indicate that differential surface settlements occurred. The majority of these cases did include some level of subgrade support and traffic. It can be seen that all of the cases for which differential surface settlements occurred fall on or below the critical height trend line proposed by McGuire (2011). The vast majority of cases for which differential surface settlements were not reported have embankment heights greater than the critical height trend line.

Figure 12 includes some cases for which embankments are lower than the critical height trend line and for which differential surface settlements were not reported. In the majority of these cases, strong soils were reported at the subgrade level near the top of the columns. Other cases in this category may have had strong soil support at subgrade level, even though it was not reported. Strong soil at this location can reduce the potential for large differential surface settlements, even for relatively low height embankments, due to arching in the strong layer immediately below the embankment.

Although the preponderance of data in Figure 12 is consistent with the critical height trend line, it would useful to perform additional field-scale tests to more fully investigate the influences of subgrade support and traffic under controlled conditions. Sloan's (2011) field-scale tests found that traffic loading on a CSE without subgrade support increased the critical height by about 15% for the geometry he investigated (2 ft diameter columns at 6 ft center-to-center spacing in a square array). McGuire's (2011) analysis of the case history reported by Teng et al. (1994), which included traffic loading, concluded that the critical height is about 20% higher than the value from Equation (1).

LOAD-DISPLACEMENT COMPATIBILITY METHOD

Smith (2005) performed numerical analyses of instrumented case histories and pilot-scale laboratory experiments done by others. Using the validated numerical procedures, Smith (2005) demonstrated the importance of subgrade support and CSE geometry on the net vertical load that acts down on the geosynthetic reinforcement in a CSE. Based on the load transfer mechanisms disclosed by the numerical analyses, Smith (2005) and Filz and Smith (2006, 2007) developed a load-displacement compatibility method for analyzing the net vertical load that acts on the geosynthetic reinforcement. Essential features of the load-displacement compatibility method include:

- Vertical load equilibrium and displacement compatibility are assumed at the level of the geosynthetic reinforcement to calculate the load distribution among the columns, the soft soil between columns, the geosynthetic, and the base of the embankment above columns and between columns.
- An axisymmetric approximation of a unit cell is employed for calculating the

vertical load acting on the geosynthetic reinforcement, as also employed by Han and Gabr (2002) and others.
- A 3D representation of the geosynthetics-reinforced CSE system and a parabolic deformation pattern of the geogrid between adjacent columns is assumed for the purpose of calculating the tension in the geogrid, as also employed by BS8006 (1995) and others.
- The load-displacement compatibility method was developed for round columns or square pile caps in a square array.
- Nonlinear response of the embankment is incorporated by providing linear response up to a limit state, at which state additional differential base settlement produces no further load concentration on the columns. The limit state is determined using the Adapted Terzaghi Method described below.
- Linear stress-strain response of the geosynthetic is assumed, but because large displacements of the geosynthetic are involved, the load-displacement relationship for the geosynthetic deformation is nonlinear. Iterations can be performed to approximate nonlinear response of the geosynthetic material.
- Nonlinear compressibility of clay soil between columns is represented using the compression ratio, recompression ratio, and preconsolidation pressure.
- Slippage is allowed between the soil and the column when the interface shear strength is exceeded.

Figure 13 shows an exploded profile view of a unit cell, including the vertical stresses at the contacts above and below the geosynthetic reinforcement. Vertical equilibrium of the system shown in Figure 13 is satisfied when

$$\gamma H + q = a_s \sigma_{col,geotop} + (1 - a_s)\sigma_{soil,geotop} = a_s \sigma_{col,geobot} + (1 - a_s)\sigma_{soil,geobot} \tag{2}$$

where γ = unit weight of the embankment soil, H = height of the embankment, q = surcharge pressure, a_s = area replacement ratio = $A_c/A_{unitcell}$ (defined in Figure 5), $\sigma_{col,geotop}$ = average vertical stress acting down on the top of the geosynthetic in the area underlain by the column, $\sigma_{soil,geotop}$ = average vertical stress acting down on the top of the geosynthetic in the area underlain by the soil foundation, $\sigma_{col,geobot}$ = average vertical stress acting up on the bottom of the geosynthetic in the area underlain by the column, and $\sigma_{soil,geobot}$ = average vertical stress acting up on the bottom of the geosynthetic in the area underlain by the soil foundation.

Load-deflection relationships were developed for (1) the embankment settling down around the column or pile cap, (2) the geosynthetic reinforcement deflecting down under the net vertical load acting on the area underlain by soil, and (3) the soil settling down between the columns. Due to space limitations, the relationships are only described in conceptual terms here. Supporting equations and additional details are presented by Filz and Smith (2006). The composite foundation system consisting of the columns and the soil between the columns is discretized, and the simultaneous nonlinear equations can be solved numerically using a spreadsheet program.

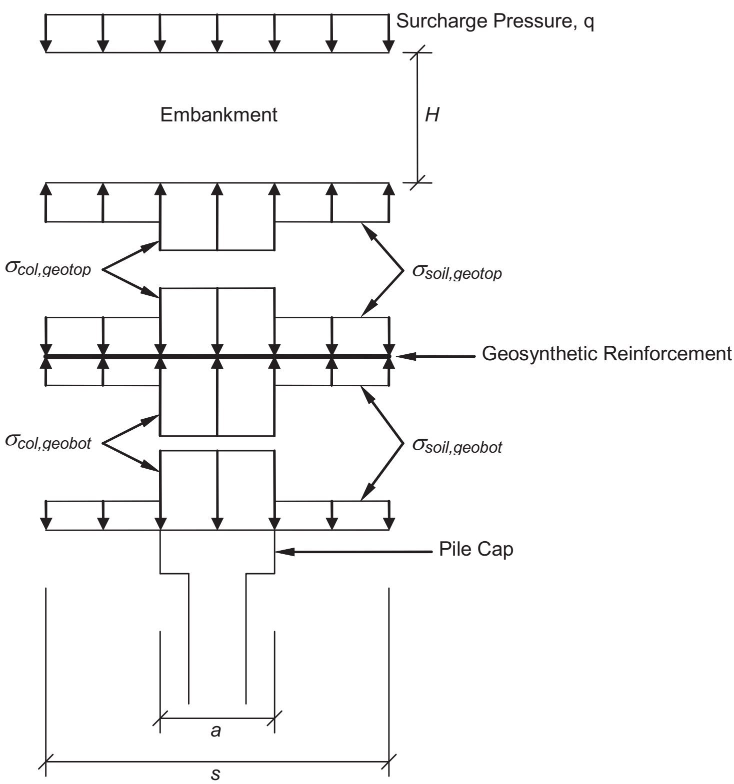

FIG. 13. Definition sketch for load-displacement compatibility method.

The load-deflection relationship for the embankment settling down around the column or pile cap is assumed to be linear up to the maximum load condition. The linear part is approximated using a linear solution for displacement of a circular loaded area on a semi-infinite mass (Poulos and Davis 1974). As indicated previously, square pile caps of width, a, can be approximated as circular pile caps with diameter, d, such that the piles cap areas are the same ($a = 0.866d$). The limiting stress condition in the embankment above the geosynthetic reinforcement is established using the Adapted Terzaghi Method (Russell and Pierpoint 1997) with a lateral earth pressure coefficient, K, of 0.75, which is between the values of 1.0 used by Russell and Pierpoint (1997) and 0.5 used by Russell et al. (2003). Other realistic methods for determining the limiting condition, such as the Hewlett and Randolph (1998) Method or the Kempfert et al. (2004a,b) Method could also be used to establish the limiting condition for settlement of the embankment down around the columns or pile caps.

The geosynthetic deflects down under the net vertical load applied over the area underlain by soil. The geosynthetic load-deflection relationship was developed based on analyses of a uniformly loaded annulus of linear elastic membrane material with the inner boundary pinned, which represents the support provided by the column, and with the outer boundary free to move vertically but not laterally, which represents the axisymmetric approximation of lines of symmetry in the actual three-dimensional configuration of a column-supported embankment. The details of the analyses and the results are presented by Smith (2005) and Filz and Smith (2006).

The settlements of the column and the subgrade soil are determined based on the vertical stress applied to the top of the column or pile, $\sigma_{col,geobot}$, and the vertical stress applied to the subgrade soil, $\sigma_{soil,geobot}$. The column compression is calculated based on a constant value of the column modulus. One-dimensional compression of clay soil located between columns is calculated using the compression ratio, re-compression ratio, and preconsolidation pressure of the soil. If an upper layer of sand is located between the columns, the sand compression is calculated using a constant value of modulus for the sand.

As the compressible soil settles down with respect to the stiffer column, the soil sheds load to the column through shear stresses at the contact between the soil and the column along the column perimeter. The magnitude of the shear stress is determined using an effective stress analysis and a value of the interface friction angle between the soil and the column. The vertical stress increment in the soil from the embankment and surcharge loading decreases with depth due to the load shedding process until the depth at which the column settlement and soil settlement are equal. An important detail is that the settlement profile of the subgrade soil at the level of the top of the columns is likely to be dish-shaped between columns. The difference between the column compression and the average soil compression is the average differential settlement at subgrade level. To account for the dish-shaped settlement profile between columns, the suggestion by Russell et al. (2003) that the maximum differential settlement at subgrade level may be as much as twice the average differential settlement was adopted. The test results by Demerdash (1996), McGuire (2011), and Sloan (2011) indicated that this is a conservative approximation, and refinement of this approximation may be warranted.

The computational method described above is solved by satisfying vertical equilibrium using Equation (2) and requiring that the calculated values of the differential settlement at subgrade level must be the same for the base of the embankment, the geosynthetic, and the underlying foundation soil. The simultaneous nonlinear equations that describe this computational method have been implemented in a spreadsheet (Filz and Smith 2006), which has the following features:

- Two different types of embankment fill are allowed so that lower quality fill can be used above the bridging layer.
- Analyses without geosynthetic reinforcement can be performed by setting the value of the geosynthetic stiffness, J, equal to zero.
- The column area and properties can vary with depth so that embankments supported on piles with pile caps can be analyzed.
- The subsurface profile can include two upper sand layers and two underlying clay layers. The preconsolidation pressure for the clay can vary linearly within each clay layer.
- The simultaneous nonlinear equations are solved automatically, and the input and output are arranged so that design alternatives can be evaluated easily.

The load-displacement compatibility method was validated by comparison with numerical analyses that were previously validated by comparison with instrumented case histories and pilot-scale experiments performed by others. In addition, the overall method was validated by direct comparison with instrumented case histories described by Cao et al. (2006) and Almeida et al. (2007). The comparisons are presented by Filz and Smith (2007) and McGuire et al. (2009).

Other researchers have developed similar approaches (e.g., Cao et al. 2006; Chen et al. 2008, 2009; Kempfert et al. 2004a,b), although they generally do not incorporate nonlinear response of the embankment, nonlinear compressibility of the soft soil underlying the embankment, and slippage between the soil and the columns. Some approaches do not incorporate geosynthetic reinforcement.

GENERALIZED ADAPTED TERZAGHI METHOD

The Adapted Terzaghi Method for determining the limiting distribution of stresses acting up on the base of the embankment has several advantages, including that it is in reasonable agreement with: (1) results of numerical analyses and field case histories (e.g., Russell and Pierpoint 1997, Filz and Smith 2006), (2) other rational methods (e.g., Hewlett and Randolph 1998 or Kempfert et al. 2004a,b, as shown by McGuire and Filz 2008), and (3) field tests by Sloan (2011). In addition, it is relatively simple.

The Adapted Terzaghi Method, as presented by Russell and Pierpoint (1997) and Russell et al. (2003) applies to a square arrangement of square columns and only one type of fill material in the embankment. This section presents a generalized version of the Adapted Terzaghi Method to accommodate the following:

- Any column arrangement and any pile cap cross-section area. Examples are shown in Figure 5.
- Up to two layers of embankment fill so that a higher quality fill in a load transfer platform and a lower quality fill overlying the load transfer platform can both be represented. This includes differences in unit weight, friction angle, and lateral earth pressure coefficient.
- Limitation of the vertical shearing in the embankment to the portion below the critical height, with treatment of the embankment weight above this level as a surcharge.

The first and second items in the list above are described by Filz and Smith (2006) and Sloan et al. (2011).

In the generalized formulation, the two layers of embankment fill are characterized by: $H_{1,2}$ = layer thicknesses as shown in Figure 1, $\gamma_{1,2}$ = layer unit weights, $K_{1,2}$ = layer lateral earth pressure coefficients, and $\varphi_{1,2}$ = layer friction angles. The embankment may have a surcharge, q. As indicated in Figure 5, p = the perimeter of the column or pile cap, $A_{unitcell}$ = the area of the unit cell around a column, and A_c = the area of the column or pile cap. The area within a unit cell underlain by soil is A_{soil} = $A_{unitcell}$ - A_c. Several of these inputs can be combined in the parameter $\alpha_{1,2}$ for each layer:

$$\alpha_{1,2} = \frac{pK_{1,2} \tan \varphi_{1,2}}{A_{soil}} \tag{3}$$

The average stress acting up on the base of the embankment in the area underlain by soil, which is $\sigma_{soil,geotop}$ in Figure 13 and which can be expressed as σ_{soil} for a CSE without geosynthetic reinforcement, is given by Equation (4a) for $H_1 + H_2 \leq H_{crit}$, by Equation (4b) for $H_1 \leq H_{crit} \leq H_1 + H_2$, and by Equation (4c) for $H_{crit} \leq H_1$.

$$\sigma_{soil,geotop} \text{ or } \sigma_{soil} = \frac{\gamma_1}{\alpha_1}\left(1 - e^{-\alpha_1 H_1}\right) + \frac{\gamma_2}{\alpha_2} e^{-\alpha_1 H_1}\left(1 - e^{-\alpha_2 H_2}\right) + q e^{-\alpha_1 H_1} e^{-\alpha_2 H_2} \tag{4a}$$

$$\begin{aligned}\sigma_{soil,geotop} \text{ or } \sigma_{soil} = {} & \frac{\gamma_1}{\alpha_1}\left(1 - e^{-\alpha_1 H_1}\right) + \frac{\gamma_2}{\alpha_2} e^{-\alpha_1 H_1}\left(1 - e^{-\alpha_2 (H_{crit} - H_1)}\right) \\ & + \left[q + \left(H_1 + H_2 - H_{crit}\right)\gamma_2\right] e^{-\alpha_1 H_1} e^{-\alpha_2 (H_{crit} - H_1)}\end{aligned} \tag{4b}$$

$$\sigma_{soil,geotop} \text{ or } \sigma_{soil} = \frac{\gamma_1}{\alpha_1}\left(1 - e^{-\alpha_1 H_{crit}}\right) + \left[q + \left(H_1 - H_{crit}\right)\gamma_1 + H_2 \gamma_2\right] e^{-\alpha_1 H_1} \tag{4c}$$

GENERALIZED PARABOLIC METHOD

There are at least three methods for calculating tension in the geosynthetic reinforcement in a CSE: the parabolic method (BS80006 1995), the tensioned membrane method (Collin 2004, 2007), and the embedded membrane method (Kempfert et al. (2004a,b). The parabolic method shows good agreement with numerical analyses (Filz and Plaut 2009) and with the field-scale tests by Sloan (2011). As presented in BS8006 (1995), the parabolic method applies to square pile caps in a square array, and it does not incorporate stress-strain compatibility.

Filz and Smith (2006) presented a solution of the parabolic method with stress-strain compatibility, and Sloan (2011) adapted the method to the geometries shown in Figure 5. The solution for biaxial geogrids placed in alignment with a rectangular array of columns is

$$6T^3 - 6T\left(\frac{\sigma_{net} A_{soil}}{p}\right)^2 - J\left(\frac{\sigma_{net} A_{soil}}{p}\right)^2 = 0 \tag{5}$$

where T = the tension in the geogrid, $\sigma_{net} = \sigma_{soil,geotop} - \sigma_{soil,geobot}$ = the net vertical stress acting on the geogrid, A_{soil} is the area of geogrid in a unit cell underlain by soil, p = the column or pile cap perimeter, and J = the sum of the stiffnesses of the geogrid layers. Typically, two to four geogrid layers are used, with the direction of each successive geogrid layer rotated by 90 degrees, so use of an average value of J is justified, even if the values of J are slightly different in the two principal directions of a biaxial geogrid. Equation (5) can be solved for the tension T, and the strain in the geosynthetic is given by $\varepsilon = T/J$. Equation (5) is recommended for rectangular column arrays with $0.5 \leq s_1/s_2 \leq 2$, including squares for $s_1 = s_2$, where s_1 and s_2 are defined in Figure 5.

Equation (5) also applies for radially isotropic geogrids, which have relatively uniform stiffness, J, in all directions within the plane of the geogrid, over columns in rectangular or triangular arrays with $0.5 \le s_1/s_2 \le 2$.

For the case of biaxial geogrids aligned over a triangular array of columns, the solution is based on the assumptions shown in Figure 14. The solution for this case is

$$\frac{2aT_1}{\sqrt{1+\dfrac{J_1}{6T_1}}} + \frac{2aT_2}{\sqrt{1+\dfrac{J_2}{6T_2}}} - \sigma_{net} A_{soil} = 0 \quad \text{and} \quad \frac{T_1 c_1^2}{J_1} = \frac{T_2 c_2^2}{J_2} \tag{6}$$

which can be solved simultaneously for T_1 and T_2, which can then be used to determine the strains according to $\varepsilon_1 = T_1/J_1$ and $\varepsilon_2 = T_2/J_2$.

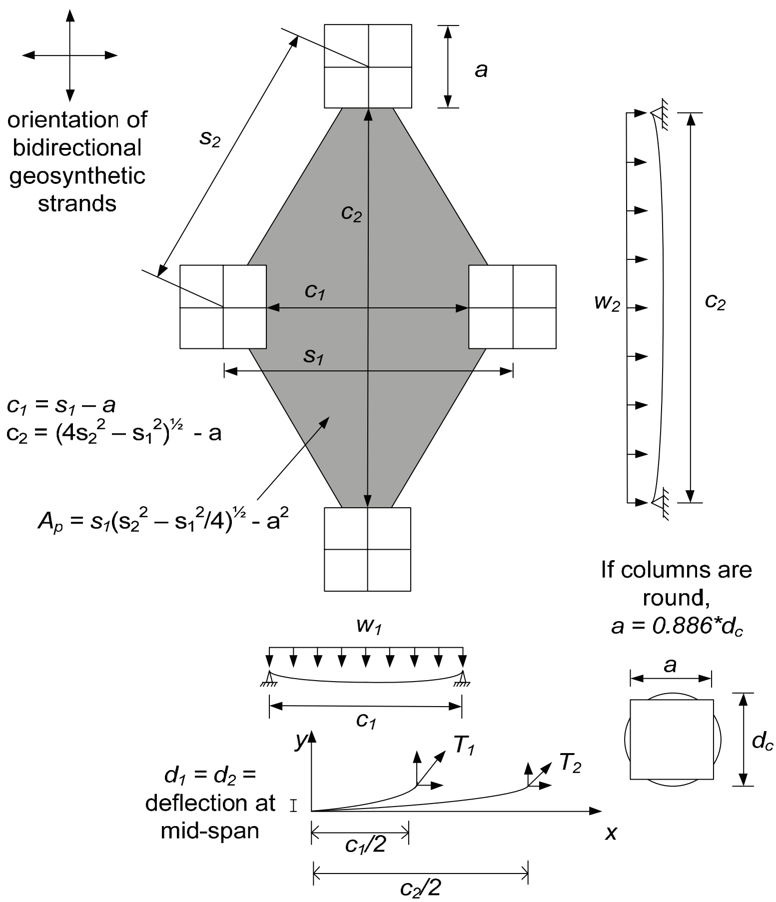

FIG. 14. Triangular column arrangements with biaxial geogrid.

SUMMARY AND CONCLUSIONS

Column-supported embankments can provide for rapid construction, good performance, and protection of adjacent facilities from deformations that would otherwise be induced by the embankment load. One of the impediments to more widespread use of CSEs is uncertainty regarding three design issues: (1) critical embankment height, above which differential settlement at the base of the embankment is not reflected as differential settlement at the embankment surface, (2) vertical load acting on geosynthetic reinforcement near the base of the CSE, and (3) tension in the geosynthetic reinforcement. The purpose of this paper is to present findings from recent research by Smith (2005), McGuire (2011), and Sloan (2011) that address these issues.

Based on bench-scale tests, field-scale tests, and case history data, the critical height is related to the column spacing and the column diameter according to Equation (1). The data used to establish this relationship was based primarily on bench-scale and field-scale CSE tests without subgrade support and without traffic loads. Subgrade support tends to reduce differential surface settlements and traffic tends to increase differential surface settlements, so these effects tend to offset each other. The case history data, which generally includes the effects of traffic and subgrade support, is consistent with Equation (1). If highly compressible soil like normally consolidated clay or peat extends close to the ground surface, then it would be reasonable to increase the critical height above that provided by Equation (1) to account for the effects of traffic. McGuire (2011) concluded that increasing the critical height by 20% above the value from Equation (1) is safe for typical roadway traffic loading when highly compressible soils extend all the way to the top of the columns or pile caps.

For a typical design problem, the embankment height is an input parameter, and the column spacing and diameter are outcomes of the design. For a given embankment height, Equation (1) can be used as part of the process to establish a suitable combination of column spacing and diameter to prevent surface deformations. The capacity of the column to carry the embankment, traffic, and/or overlying structure load also influences the column spacing.

The net vertical load acting down on the geosynthetic reinforcement in a load transfer platform can be calculated using the load and displacement compatibility method, which takes into account the stiffness of the embankment, geosynthetic reinforcement, columns, and soil between columns. For the embankment, the limiting stress condition can be represented by the generalization of the Adapted Terzaghi Method in Equations (4). This generalization accounts for any shape of pile cap and any repetitive arrangement of columns, it allows for two different soil types in the embankment, and it limits shearing in the embankment to the critical height.

The tension in geosynthetic reinforcement in CSEs can be calculated using the generalized parabolic method expressed by Equations (5) and (6). This generalization imposes stress-strain compatibility in the geosynthetic, and it allows for round or square pile caps, rectangular or triangular column arrays, and biaxial or radially isotropic geogrids.

The findings described in this paper are incorporated in a comprehensive design procedure for CSEs by Sloan et al. (2012).

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Risk Assessment in Geotechnical Engineering: Stability Analysis of Highly Variable Soils

D.V. Griffiths[1] ,F ASCE, Jinsong Huang[2] ,M ASCE and Gordon A. Fenton[3] ,M ASCE

[1]Professor, Colorado School of Mines, 1610 Illinois Street, Golden, CO 80401; d.v.griffiths@mines.edu
[2] Research Associate, ARC Center of Excellence for Geotechnical Science and Engineering, University of Newcastle, Callaghan, NSW 2308, Australia; jinsong.huang@newcastle.edu.au
[3] Professor, Dalhousie University, Halifax, NS, Canada, B3J 1Y9; gordon.fenton@dal.ca

ABSTRACT: The paper will review the state-of-the-art in the use of finite element methods for modeling geotechnical engineering problems involving non-typical geometries and highly variable soil properties. Examples will focus on slope stability analyses in which traditional limit equilibrium methods, and even well-established probabilistic methodologies may lead to misleading results.

Keywords: Finite element method, Variable soils, Probability of failure, Random Fields, Risk assessment.

INTRODUCTION

Classical limit equilibrium methods of slope stability analysis have remained essentially unchanged for decades. The finite element method offers a powerful alternative with the following main advantages:

- No assumption needs to be made in advance about the shape or location of the failure surface. The failure mechanism “seeks out” the weakest path through the soil.
- Since there is no concept of slices in the finite element approach there is no need for assumptions about slice side forces. The finite element method preserves global equilibrium until “failure” is reached.
- If realistic soil compressibility data is available, the finite element solutions will give information about deformations at working stress levels.
- The finite element method is able to monitor progressive failure up to and including overall shear failure.

Finite element slope stability analysis can hardly be considered a new technique. The first paper to tackle the subject by Smith & Hobbs (1974) is over 35 years old followed by an important paper on the topic by Zienkiewicz et al. (1975). Both of these papers had a very significant influence on the first author's finite element slope stability software developments over the years. Early publications date back to Griffiths (1980) and the first ever published source code for finite element slope stability appeared in the second edition of the text by Smith & Griffiths (1988, 2004). Readers are also referred to Griffiths & Lane (1999) for a thorough review of how the methodology works.

This paper will focus initially on demonstrating the use of the finite element method as applied to slope examples that would not necessarily be amenable to traditional limit equilibrium methods (LEM). The paper will then go on to discuss risk assessment methods in geotechnical engineering, particularly for slope stability, including the most recent developments that combine random fields with finite element methods in the Random Finite Element Method (RFEM). Examples will be given of slope reliability analysis, where traditional methods may deliver quite misleading results.

LONG SLOPES

How long is "infinite"?

It has been noted previously (e.g. Duncan & Wright 2005) that the infinite slope assumptions can be expected to lead to conservative estimates of the factor of safety. This is primarily due to support provided at the ends of a finite slope that is not accounted for in the infinite slope model. Here we present some finite element slope stability analyses on "long slopes" with uphill and downhill boundary conditions, to assess the range of validity and conservatism of the "infinite slope" assumptions. The main question to be addressed is; Howlong must a slope be for it to be considered "infinite"? A typical finite element mesh of 8-noded quadrilateral elements is shown in Figure 1. Note that H and L are respectively vertical and horizontal measures of the slope geometry.

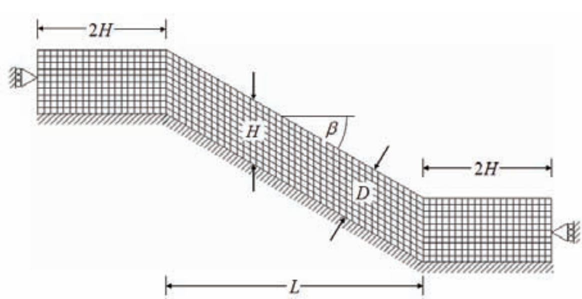

Fig. 1. Mesh of 8-node quadrilateral element for "long slope" analysis.

For simplicity, we have considered an undrained clay slope for which the infinite slope equation would give the following factor of safety

$$FS = \frac{c_u}{\gamma_{sat} H \cos\beta \sin\beta} \tag{1}$$

In this example, the properties shown in the caption of Figure 2 were held constant while L/H was gradually increased. As shown, the computed factor of safety converged on the infinite slope solution of $FS = 1.15$ from equation (1) for L/H greater than about 16. As expected, the infinite slope solution is always conservative.

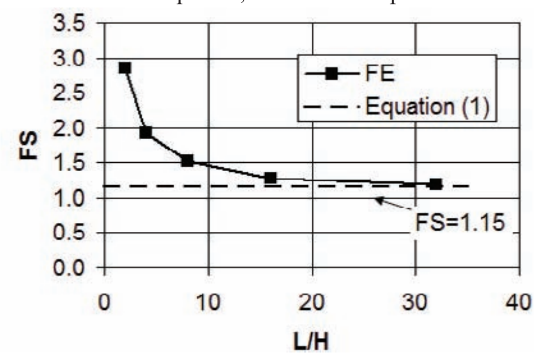

Fig. 2. Influence of length ratio on the computed factor of safety for a slope with $H = 2.5\,\text{m}$, $\beta = 30^\circ$, $c_u = 25\,\text{kN/m}^2$ and $\gamma_{sat} = 20\,\text{kN/m}^3$.

For example, with $L/H = 2$, the computed factor of safety was $FS = 2.86$; more than twice the infinite slope value. A typical failure mechanism for a steeper slope is shown in Figure 3. The figure indicates that as the slope gets longer, the infinite slope mechanism starts to dominate and the "toe" failure at the downhill end becomes less important.

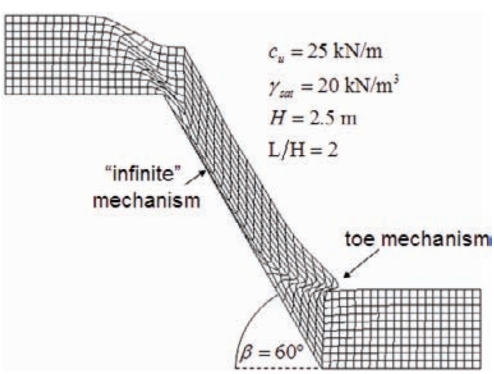

Fig. 3. Deformed mesh at failure corresponding to a slope with $L/H = 2$ and $\beta = 60°$ indicating a toe mechanism with $FS = 1.58$.

Influence of slope angle

A curiosity of the infinite slope equation (1) as shown in Figure 4, is that for constant H, γ_{sat} and c_u, the factor of safety starts to increase as the slope steepens in the range $\beta > 45°$. This result seems counter intuitive since our experience of finite slopes is that the factor of safety always falls as a slope gets steeper.

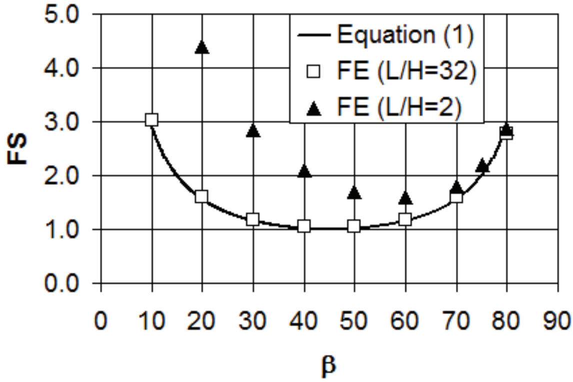

Fig 4. Influence slope angle on the factor of safety for an undrained clay slope with $H = 2.5\,\mathrm{m}$, $c_u = 25\,\mathrm{kN/m^2}$ and $\gamma_{sat} = 20\,\mathrm{kN/m^3}$.

An explanation of this effect for infinite slopes comes from the fact that as the slope becomes steeper, the length of the potential failure surface available to resist sliding is increasing at a faster rate than the down-slope component of soil weight trying to cause sliding. Even a short slope analysis with $L/H = 2$ demonstrates this effect as shown in Fig. 4 (Griffiths et al. 2011a).

STRATIFIED SLOPES

James Bay Dike

The James Bay Dike slope shown in Figure 5 has a terraced cross-section with four different soil types consisting of cohesionless soil in the embankment and undrained clays in the foundation. This profile has attracted considerable interest (see e.g., El Ramly et al. 2002, Duncan et al. 2003) because published LEM solutions that assumed circular failure mechanisms (e.g. Bishop's method), led to unconservative estimates of the factor of safety. Although limit equilibrium procedures are available for estimating the factor of safety associated with non-circular surfaces, it is still hard to guarantee that the critical surface corresponding to the minimum factor of safety has been found.

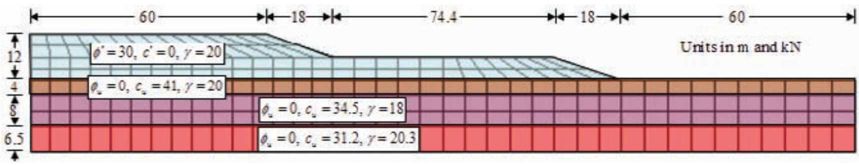

Fig. 5. FE geometry and soil properties assigned to the James Bay dike.

The benefits of the FE slope stability approach are even more striking in an example such as this in which the factor of safety can be accurately estimated, and the corresponding failure mechanism observed. The sudden displacement increase shown in Figure 6 indicates that $FS \approx 1.27$ and the deformed mesh at failure given in Figure 7 clearly shows the anticipated non-circular critical failure mechanism.

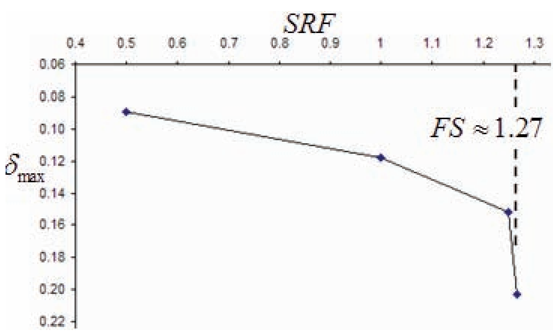

Fig. 6. FE solution of James Bay Dike by strength reduction indicating $FS = 1.27$.

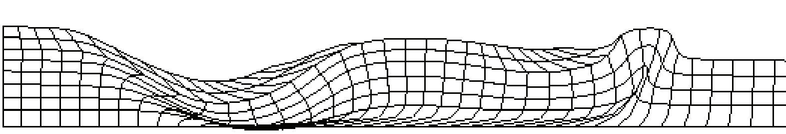

Fig. 7. Deformed mesh at failure demonstrating a non-circular failure mechanism.

Multiple failure mechanisms

As mentioned in the Introduction, the finite element method "seeks out" the most critical failure path through the soil, and unlike many LEM approaches, does not require the user to anticipate in advance where the critical failure mechanism might lie. The example shown in Figures 8 makes this point quite clearly by demonstrating multiple mechanisms, which all have the same factor of safety of $FS = 1.38$. A traditional approach could easily miss one or more of these surfaces, which could lead to an unsafe design if the goal of the analysis, for example, was to identify locations for possible soil reinforcement.

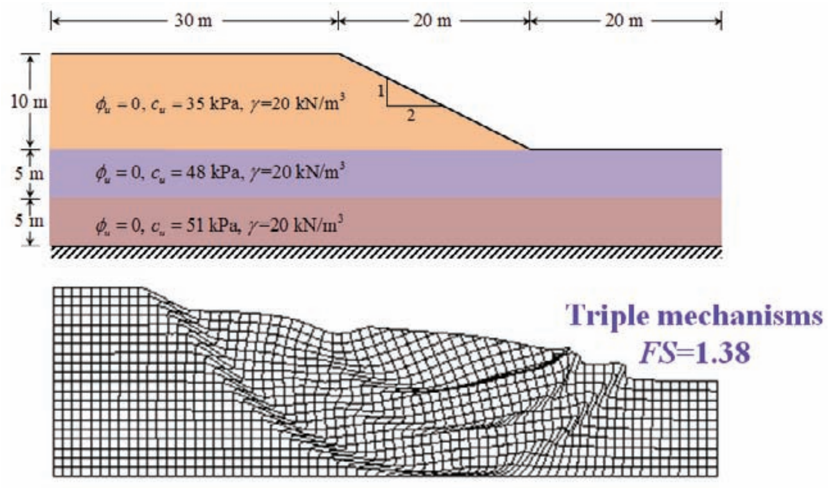

Fig. 8. Multiple failure mechanisms of an undrained slope.

Checkerboard slope stability analysis.

Soils and rocks are the most variable of all engineering materials, so when an engineer chooses "characteristic values" of the soil shear strength for a slope analysis, it is very likely that some parts of slope consist of soil that is stronger than the characteristic values, and other parts that are weaker.

In this section we take a simple 2D undrained clay slope and assign the slope two different properties arranged in a checkerboard pattern as shown in Figure 9.

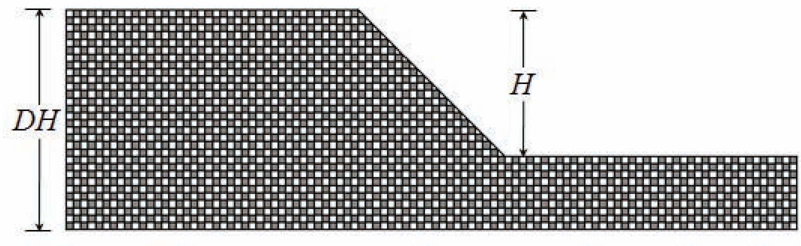

Fig. 9. Slope stability analysis with checkerboard strength pattern. The darker zones are stronger.

The 45° undrained clay slope has a height of $H = 10$ m a unit weight of $\gamma_{sat} = 20$ kN/m^3 and a foundation depth ratio of $D = 1.5$. The mean strength of $c_u = 50$ kPa was held constant, while the stronger soil was made stronger and the weaker soil was made weaker. The results of the factor of safety analysis by strength reduction are shown in Figure 10. Clearly the weaker soil "wins"! This trend will be repeated when wider ranges of strength values are incorporated into an analysis, such as later in this paper when we discuss random field modeling of soils.

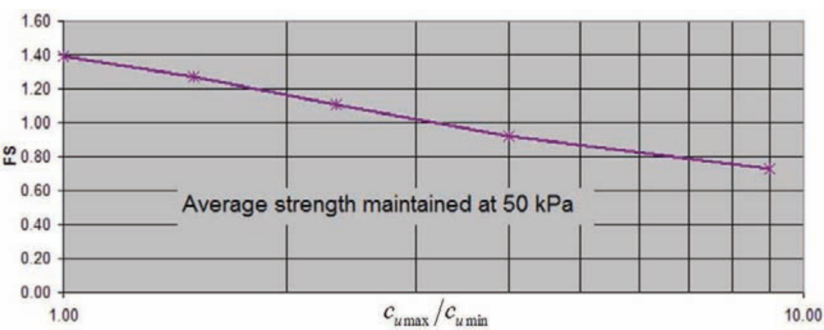

Fig. 10. Influence of strength ratio in checkerboard slope analysis.

3D SLOPE STABILITY ANALYSIS

3D slope stability analysis has received considerable attention in the literature (e.g. Hungr 1987, Seed et al. 1990, Duncan 1996, Stark and Eid 1998, Chen et al. 2005, Griffiths and Marquez 2007, Michalowski 2010), yet the vast majority of slope stability analyses in research and practice, are still performed in 2D under the assumption of plane strain conditions. Even when 2D conditions are not appropriate, 3D analysis is rarely performed. There are a number of reasons for this. The majority of work on this subject has shown that the 2D factor of safety is conservative (e.g. lower than the "true" 3D factor of safety), and existing methods of 3D slope stability analysis are often complex, and not well established in practice. A further disadvantage of some 3D LEM approaches, is that being based on extrapolations of 2D "methods of slices" to 3D "methods of columns", they are complex, and not readily modified to account for realistic boundary conditions in the third dimension. The advantages of FE slope stability methods become even more attractive in 3D. Here we demonstrate when 3D may be justified, and also show that great care must be taken in subscribing to the received wisdom that "2D is always conservative".

When is plane strain a reasonable approximation?

The first issue addressed for a homogeneous slope, is to consider the question "how long does a slope need to be in the third dimension for a 2D analysis to be justified?" Figure 11 shows a simple mesh that might be used for a 3D slope analysis.

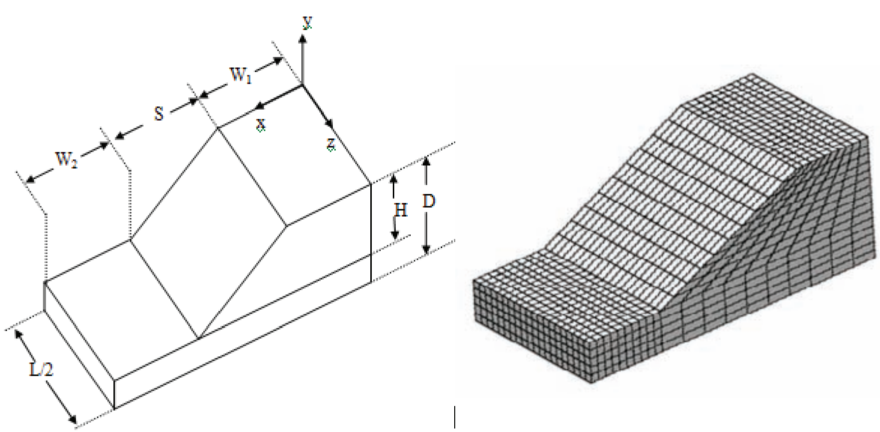

Fig. 11. FE mesh for 3D slope stability analysis using 20-node hexahedral elements.

The boundary conditions are such that one side $(z=0)$ is fully fixed and the other $(z=L/2)$ allows vertical movement only implying a plane of symmetry. The bottom $(y=D)$ of the slope is fully fixed, while the back $(x=0)$ and front-side $(x=W_1+W_2+S)$ of the slope allow vertical movement only. The results from a series of FE analyses with different depth ratios (L/H) while keeping all other parameters constant are shown in Figure 12. It can be seen that the factor of safety in

3D is always higher than in 2D, but tends to the plane strain solution of $FS = 1.25$ for depth ratios of the order $L/H > 10$. It is shown that results of the same slope with a coarser mesh gave slightly higher values of FS.

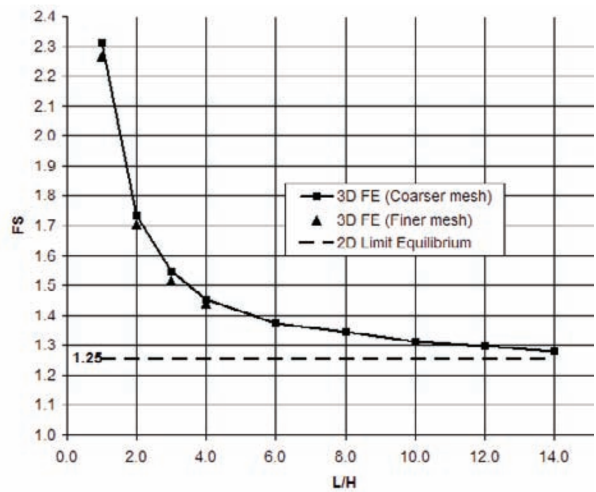

Figure 12: Comparison of 3D and 2D solutions for a $\phi_u = 0^o$ slope with $c_u / (\gamma H) = 0.20$.

Is plane strain conservative?

The assumption that 2D analyses lead to conservative factors of safety needs some qualification. Firstly, a conservative result will only be obtained if the "most pessimistic" section in the 3D problem is selected for 2D analysis (see e.g., Duncan 1996). In a slope that contains layering and strength variability in the third dimension, this conservative 2D section may not be intuitively obvious. Secondly, the corollary of a conservative 2D slope stability analysis is that back analysis of a failed slope will lead to an unconservative overestimation of the soil shear strength (e.g. Arellano & Stark 2000). Bromhead & Martin (2004) argued that some landslide configurations with highly variable cross-sections could lead to failure modes in which the 3D mechanism was the most critical. Other investigators have also indicated situations where more critical 3D factors of safety were observed (e.g., Chen & Chameau 1982 and Seed et al. 1990).

Finite element slope stability analysis offers us the opportunity to perform objective comparisons in which 2D and 3D factors of safety are compared for variable soil conditions. This point is highlighted in the 3D example shown in Figure 13 which represents a 2:1 slope of height 10 m, foundation depth 5 m and a length in the out-of-plane direction of $60\ \text{m}$ with smooth boundary conditions.

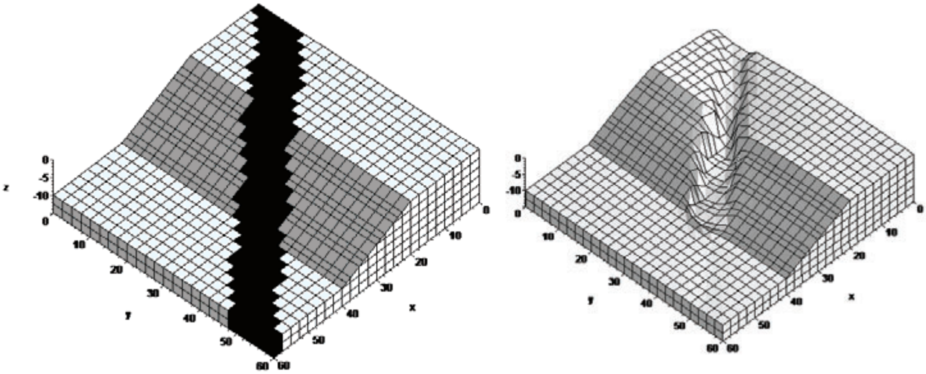

Fig. 13. Three-dimensional slope mesh and at failure including an oblique layer of weak soil.

An oblique zone of weak soil (shaded black) with undrained strength $c_u = 20\ \text{kN/m}^2$ has been introduced into the slope with the surrounding soil four times stronger with $c_u = 80\ \text{kN/m}^2$. The 3D factor of safety was found to be approximately 1.5 and the mechanism clearly follows the weak zone as also shown in Figure 13.

When 2D stability analyses are then performed on successive slices in the $x-z$ plane moving from $y = 0\ \text{m}$ to $y = 60\ \text{m}$, the result shown in Figure 14 is obtained. As a check, the 2D analyses were performed both by finite elements and by a standard LEM. It can be seen that towards the boundaries of the 3D slope ($y < 21\ \text{m}$ and $y > 34\ \text{m}$) where the majority of soil in the sections is strong, the 2D results led to higher and therefore unconservative estimates of the factor of safety. On the other hand, at sections towards the middle of the slope ($21\ \text{m} < y < 34\ \text{m}$) where there is a greater volume of weak soil, the 2D results led to lower, and therefore conservative estimates of the factors of safety. The 2D factor of safety closely approached unity at $y = 29\ \text{m}$. An even more critical 2D plane however, is the oblique one that runs right down the middle of the weak soil. This 2D plane has a 2.5:1 slope and is flatter than the $x-z$ planes considered previously. A 2D slope stability analysis on this plane gives an even lower factor of safety of about 0.7. This result, also shown on Figure 14, is less than half of the factor of safety given by the 3D analysis, and would be considered excessively conservative, even by geotechnical design standards.

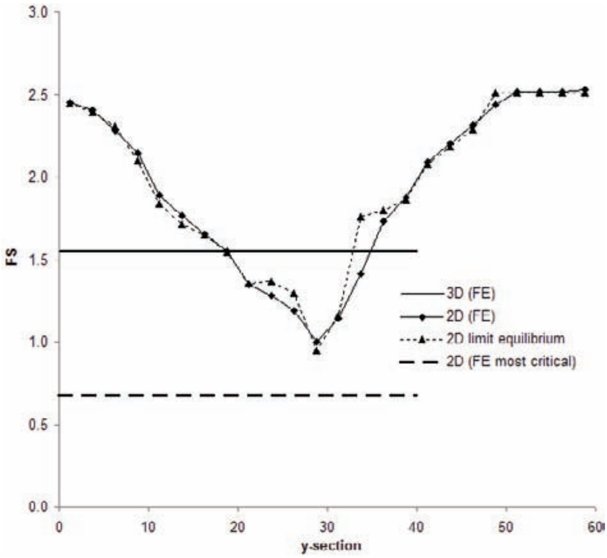

Figure 14: Factors of safety from 3D analysis and various 2D sections.

Even in the rather simple problem considered here, the results have shown a quite complex relationship between 2D and 3D factors of safety. The results confirm that 2D analysis will deliver conservative results, but only if the most pessimistic plane in the 3D problem is selected. Even so, this result may lie well below the "true" 3D factor of safety. More importantly however, it has also been shown that selection of the "wrong" 2D plane could lead to an unconservative result.

RISK ASSESSMENT IN GEOTECHNICAL ENGINEERING

Soils and rocks are the most variable of all engineering materials, yet this is often coupled with inadequate site data. These factors combine to make geotechnical engineering one of the most appropriate areas for the application of probabilistic tools.

Risk assessment and probabilistic analyses in geotechnical engineering are rapidly growing areas of importance and activity for practitioners and academics (e.g. Baecher and Christian 2003, Fenton and Griffiths 2008). At a recent G-I specialty conference called *Georisk 2011* for example, several important state of practice papers were presented (e.g. Christian and Baecher 2011, Lacasse and Nadim 2011, Scott 2011) and in this *GeoCongress 2012*, Lacasse et al. 2012 have presented a comprehensive review of the state of risk assessment and mitigation in geo-practice. It is now commonplace for major geotechnical conferences to include sessions on risk assessment in geotechnical engineering.

Of all areas of geotechnical engineering, slope stability analysis has received greater attention using risk assessment tools than any other, since the concept of replacing a "factor of safety" by a "probability of failure" is immediately appealing to many engineers (see e.g. Alonso 1976, Catalan and Cornell 1976, Li and Lumb 1987, Oka and Wu 1990, Chowdhury and Xu 1992, Mostyn and Soo 1992, Juang et al. 1992, Mostyn and Li 1993, Lacasse 1994, Lacasse and Nadim 1996, Liang et al 1999, Malkawi et al. 2000, Griffiths and Fenton 2000,2004, Duncan 2000, El Ramly et al. 2002, Bhattacharya et al. 2003, Babu and Mukesh 2004, Jiminez-Rodriguez et al. 2006, Low and Tang 2007, Hong and Roh 2009, Griffiths et al. 2009a, Huang et al. 2010, Ching et al. 2010, Mbarka et al. 2010, Wang et al. 2011).

The Random Finite Element Method (RFEM)

The goal of a probabilistic slope stability analysis is to estimate the probability of slope failure as opposed to the ubiquitous factor of safety used in conventional analysis. Several relatively simple tools exist for performing this calculation that include the First Order Second Moment (FOSM) method and the First Order Reliability Method (FORM). The FORM method in particular has now been developed to a quite significant level of sophistication to tackle correlation and system slope reliability (e.g. Low et al. 2007, Low et al. 2011).

A legitimate criticism of these first order methods however, is that they are unable to properly account for spatial correlation in the 2D or 3D random materials, and are inextricably linking with "old fashioned" slope stability methods that involve simple shapes for the failure surfaces (typically circular).

To overcome these deficiencies, a method called the Random Finite Element Method (RFEM) that combines random field theory with deterministic finite element analysis was developed by the authors in the early 1990's and has been applied to a wide range of geotechnical applications (e.g. Griffiths and Fenton 2007, Fenton and Griffiths 2008). In a stability analysis, input to RFEM is provided in the form of the mean, standard deviation and spatial correlation length of the soil strength parameters which may consist of several layers with different statistical input parameters. In the absence of site specific information, there is an increasing number of publications presenting typical ranges for the standard deviation of familiar soil properties (e.g. Lee et al. 1983).

In RFEM, local averaging is fully accounted for at the element level indicating that the mean and standard deviation of the soil properties are statistically consistent with the mesh density. Since the finite element method of slope stability allows mechanisms to "seek out" the most critical path through the soil, the method offers great promise for more realistic reliability assessment of slopes and other geotechnical applications. The flow chart for a typical RFEM slope stability analysis is shown in Figure 15.

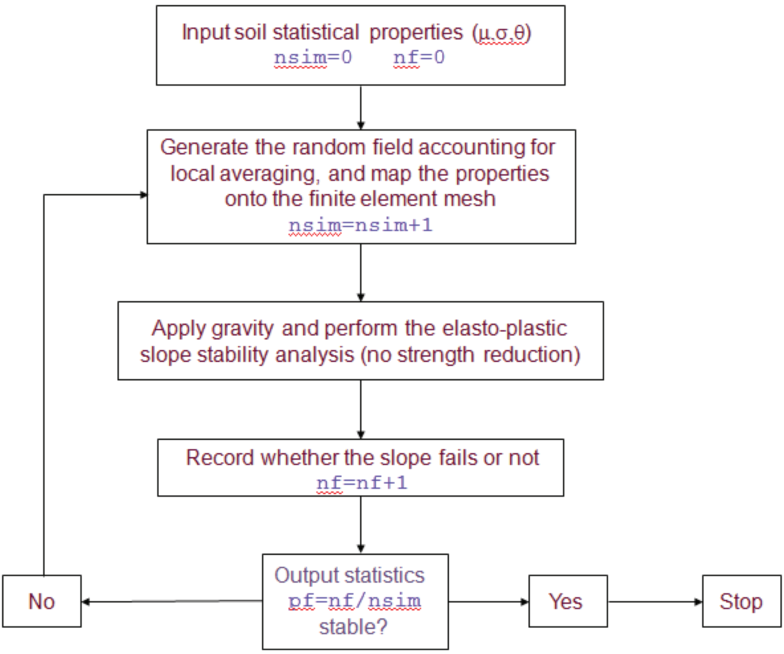

Fig. 15. Flow chart for a typical RFEM slope stability analysis.

The RFEM codes developed by Griffiths and Fenton for a range of geotechnical applications are freely available in source code from the authors' web site at `www.mines.edu/~vgriffit/rfem`. The 2D slope stability program is called `rslope2d`. A couple of failure mechanisms computed using this program for slopes with quite different spatial correlation lengths but with the same mean and standard deviation of strength parameters are shown in Figure 16. The spatial correlation length is expressed in dimensionless form relative to the height of the embankment, e.g. $\Theta_C = 0.5$ means the spatial correlation length is $0.5H$ etc. It is seen that the slope with the higher spatial correlation length in the lower figure gives a quite smooth failure mechanism more like the classical "mid-point" circle. The soil with a lower spatial correlation length in the upper figure however, displays a quite complex system of interacting mechanisms which would defy analysis by any traditional LEM.

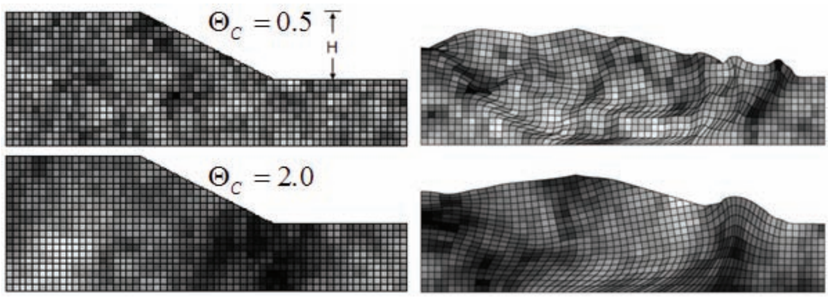

Fig. 16. Typical failure mechanisms from an RFEM analysis with two different spatial correlation lengths.

Following the results of Griffiths and Fenton (2004), the RFEM results for an undrained clay slope with a spatially random, lognormally distributed dimensionless undrained strength given by $C = c_u / (\gamma_{sat} H)$ is shown in Figure 17. The computed probability of failure by RFEM (p_f) is given as a function of the spatial correlation length $(\Theta_C = \theta_{\ln C} / H)$ and the coefficient of variation $(V_C = \sigma_C / \mu_C)$. It can be seen that an increasing correlation length may either increase or decrease the slope failure probability depending on the input coefficient of variation V_C.

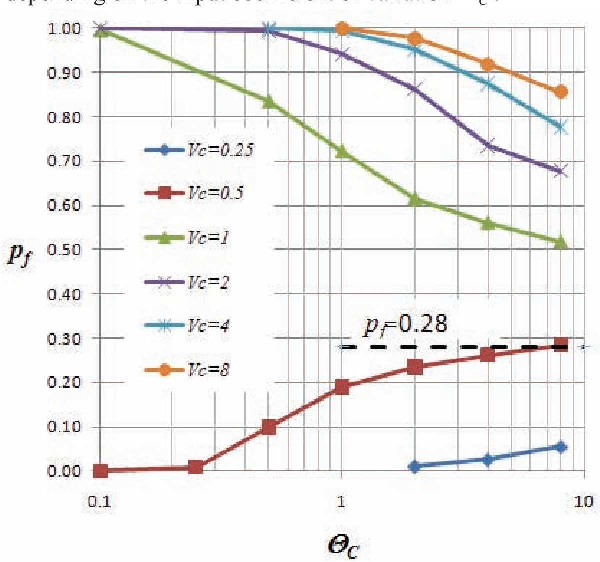

Fig. 17. Influence of the spatial correlation length and coefficient of variation on the probability of failure of an undrained slope $(\mu_C = 0.25)$.

In order to interpret these results, a couple of key deterministic solutions considering a homogeneous soil, should be kept in mind. (i) if $C=0.25$, $FS=1.47$ and (ii) if $C=0.17$, $FS=1.0$. The diverging results from the probabilistic studies shown in Figure 17 can then be explained by considering the limiting cases of $\Theta_C \to 0$ and $\Theta_C \to \infty$. As $\Theta_C \to 0$, the slope becomes essentially homogeneous at each simulation, with a constant strength given by its median. If the median falls below 0.17, all simulations fail and $p_f \to 1$, but if the median is greater than 0.17, none of the simulations fail and $p_f \to 0$. On the other hand, as $\Theta_C \to \infty$, each simulation involves a homogeneous soil with the property varying from one simulation to the next, so $p_f \to \mathrm{P}\left[C < 0.17\right]$.

For example, in the case of $\mu_C = 0.25$, $V_C = 0.5$, the parameters of the underlying normal distribution of $\ln C$ are given as

$$\begin{aligned} \mu_{\ln C} &= \ln \mu_X - \frac{1}{2}\ln\left\{1+V_C^{\,2}\right\} = -1.498 \\ \sigma_{\ln C} &= \sqrt{\ln\left\{1+V_C^{\,2}\right\}} = 0.472 \end{aligned} \tag{2}$$

hence

$$p_f = \Phi\left(\frac{\ln 0.17 - \mu_{\ln C}}{\sigma_{\ln C}}\right) = 0.28 \tag{3}$$

which is shown as the asymptotic trend of the line corresponding to $V_C = 0.5$ as $\Theta_C \to \infty$ in Figure 17.

On the other hand, as $\Theta_C \to 0$, the median of the shear strength is given by

$$\mathrm{Median}_C = \exp(\mu_{\ln C}) = \exp(-1.498) = 0.22 > 0.17 \tag{4}$$

hence $p_f \to 0$.

First order methods and single random variable Monte-Carlo methodologies that treat each simulation as a homogeneous material can be considered special cases of RFEM with $\Theta_C \to \infty$ but cannot be guaranteed to deliver conservative results.

Influence of Mesh Refinement

A commonly asked question of any finite element analysis, including RFEM, is the extent to which mesh refinement and discretization errors affect the results. As mentioned previously, the statistics of the random field mapped onto the finite element mesh are adjusted in a consistent way to account for element size.

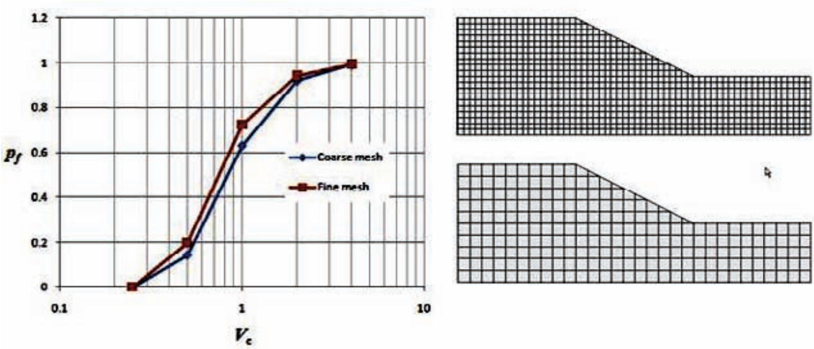

Fig. 18. Influence of mesh density on p_f for an undrained slope.

This is an integral part of the Local Average Subdivision method (Fenton and Vanmarcke 2000). As for the overall discretization issue, Figure 18 shows the influence of mesh refinement for two different cases with $\Theta_C = 1$ and $\mu_C = 0.25$. It can be seen that the finer mesh gives somewhat higher values of p_f, which is to be expected, since more paths are available for failure to occur.

IMPORTANCE OF SPATIAL VARIABILITY

In the following section we present two short examples that emphasize the importance of proper modeling of spatial variability in slope risk assessment.

Infinite Slope Example

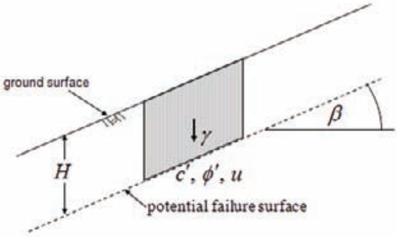

Fig. 19. Geometry and parameters of an infinite slope

This is one of the oldest and simplest types of slope problem in which the failure mechanism is assumed to be purely translational with the failure plane at the base of the layer. In the absence of pore pressures $(u=0)$, the factor of safety can be expressed explicitly by the equation

$$FS=\frac{c'}{\gamma H\sin\beta\cos\beta}+\frac{\tan\phi'}{\tan\beta} \tag{2}$$

In this example (Griffiths et al. 2011b) the cohesion is defined by $\mu_{c'}=10\,\mathrm{kN/m^2}$ and $\sigma_{c'}=3.0\,\mathrm{kN/m^2}$ and the tangent of the friction angle by $\mu_{\tan\phi'}=0.5774$ and $\sigma_{\tan\phi'}=0.1732$. The remaining parameters are assumed to be deterministic with values given by $H=5.0$ m, $\beta=30°$, and $\gamma=17.0$ kN/m^3. Substitution of these deterministic parameters and the mean values of the random variables into Eq. (2) leads to a deterministic factor of safety of $FS=1.27$.

From Eq. (2), and assuming c' and $\tan\phi'$ are uncorrelated, we can estimate the mean and standard deviation of FS by the FOSM as

$$\mu_{FS}\approx\frac{\mu_{c'}}{\gamma H\sin\beta\cos\beta}+\frac{\mu_{\tan\phi'}}{\tan\beta} \tag{3}$$

$$\sigma_{FS}\approx\sqrt{\left(\frac{1}{\gamma H\sin\beta\cos\beta}\right)^2\sigma_{c'}^2+\left(\frac{1}{\tan\beta}\right)^2\sigma_{\tan\phi'}^2} \tag{4}$$

which gives $\mu_{FS}=1.27$ and $\sigma_{FS}=0.311$

Assuming that FS is lognormal, the probability of failure is then given by

$$p_f=\mathrm{P}\left[FS<1\right]=\mathrm{P}\left[\ln(FS)<\ln(1)\right]=\Phi\left[-\frac{\mu_{\ln FS}}{\sigma_{\ln FS}}\right] \tag{5}$$

where the mean and standard deviation of the underlying normal distribution of $\ln(FS)$ are given by $\mu_{\ln(\mathrm{FS})}=0.2113$ and $\sigma_{\ln(\mathrm{FS})}=0.2409$. After substitution

$$p_f=\Phi\left[-\frac{0.2113}{0.2409}\right]=\Phi\left[-0.8772\right]=1-\Phi\left[0.8772\right]=1-0.810=0.19 \tag{6}$$

hence the probability of failure is approximately 19.0%. It should be noted that this result, being based on the deterministic Eq.(2), assumes failure always occurs at the base of the layer.

The same problem was then solved using RFEM by including lognormal and uncorrelated c' and $\tan\phi'$ and a range of spatial correlation lengths defined in dimensionless form as $\Theta=\theta/H$ (assumed in this example to be the same for both

c' and $\tan\phi'$) . The results shown in Figure 20 indicate that the FOSM results are consistently unconservative, but less so as $\Theta \to \infty$. This is because in RFEM, failure takes place along the weakest path, which doesn't necessarily occur at the base of the layer. For shorter values of Θ, the critical plane is more likely to occur above the base and p_f is higher. The figure also shows a typical random field and failure plane from the RFEM Monte-Carlo analyses.

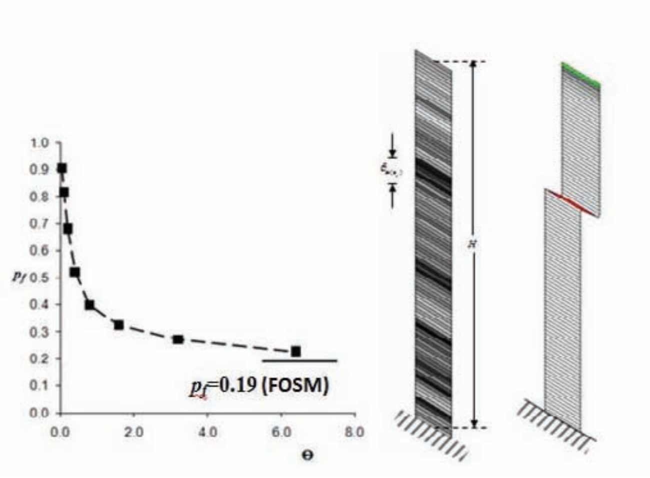

Fig. 20. Comparison of RFEM and FOSM results for an infinite slope analysis.

Three dimensional slope reliability

Since the 2D factor of safety is generally considered to be conservative, practitioners are reluctant to invest in the more time-consuming 3D approaches. A key question to be addressed is, under what circumstances will the probability of failure of a slope predicted by a full 3D analysis be higher than that obtained from an equivalent 2D analysis?

In all the RFEM analyses that follow (Griffiths et al. 2009b), the bottom of the mesh $(y = -H)$ is fully fixed and the back of the mesh $(x = 0)$ is allowed to move only in a vertical plane. Both "rough" and "smooth" boundary conditions have been considered at the ends in the out-of-plane direction $(z = 0 \text{ and } L)$. In the rough cases the ends are fully fixed and in the smooth case, they are allowed to move only in a vertical plane. It is noted that unlike the deterministic study shown previously, there is no symmetry in the RFEM analyses due to the spatial varying soil properties. In this study, it was determined that 2000 realizations of the Monte-Carlo process for each parametric group, was sufficient to give reliable and reproducible estimates of the probability of failure p_f.

The undrained clay slope shown in Figure 21 demonstrates an important characteristic in 3D slope analysis called the "preferred" failure mechanism width W. This is the width of the failure mechanism in the $z-$direction that the finite element analysis "seeks out". Over a suite of Monte-Carlo simulations the average preferred failure mechanism width is called W_{crit}. It will be shown that this dimension has a significant influence on 3D slope reliability depending on whether the length of the slope L is greater than or less than W_{crit}.

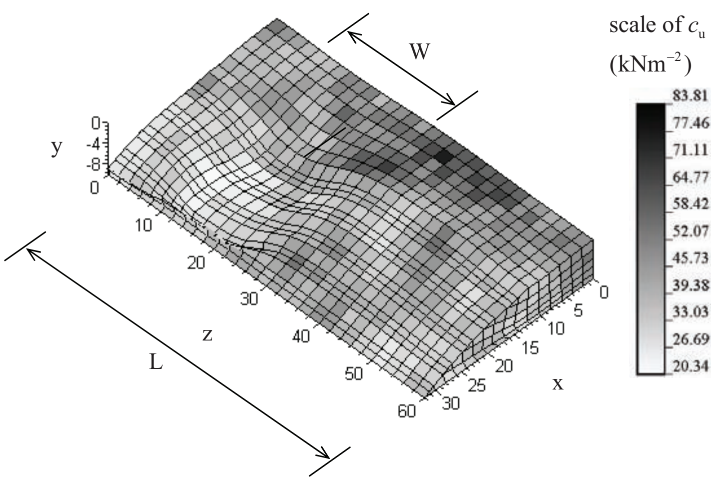

Fig. 21. Slope failure with (isotropic) $\Theta_C = 2.0$ and rough boundary condition

With the same definition of spatial correlation used earlier in the paper for 2D slope analysis $\left(\Theta_C = \theta_{\ln C}/H\right)$, the length ratio was varied in the range $0.2 < L/H < 16$ to investigate the influence of three-dimensionality, with results presented in Figure 22.

In the case of smooth boundary conditions, the p_f of one slice $\left(L/H = 0.2\right)$ in the 3-d analysis is equivalent to that given by a 2D RFEM analysis since the 3D analysis is essentially replicating plane strain.

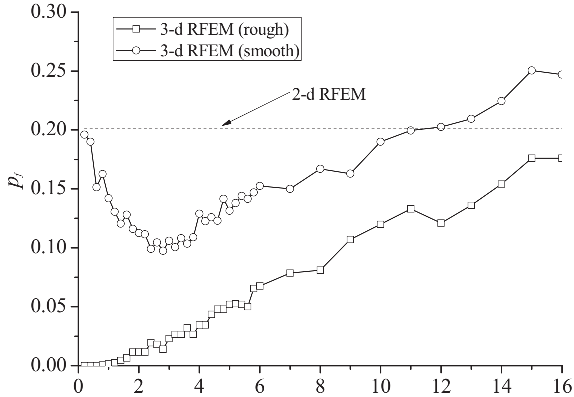

Fig. 22. Probability of failure versus slope length ratio

$\left(V_{c_u} = 0.5, \Theta_C = 1.0, \ FS = 1.39, \text{slope angle 2h : 1v}\right)$

It is also shown in the smooth case that as L/H is increased, p_f initially decreases, reaching a minimum before rising to eventually exceed the 2D value. In the rough case, p_f is close to zero for a narrow slice and increases steadily as L/H is increased due to a gradual reduction in the supporting influence of the rough boundaries in the 3D case. As the length ratio is increased in both the rough and smooth cases, the 3-d p_f eventually exceeds the 2D value, indicating that 2D analysis will be always give unconservative results if the slope is long enough. It may also be speculated that $p_f \to 1$ as $L/H \to \infty$ regardless of boundary conditions.

For the case of smooth boundary conditions, let us define the critical slope length L_{crit} and the critical slope length ratio $\left(L/H\right)_{crit}$ as being that value of L/H for which the slope is safest and its probability of failure p_f a minimum. It will be shown that this minimum probability of failure in the smooth case occurs when $L_{crit} \approx W_{crit}$. If we reduce the slope length ratio below this critical value $\left(L < L_{crit}\right)$, the slope finds it easier to form a global mechanism spanning the entire width of the mesh with smooth end conditions, so the value of p_f increases, tending eventually to the plane strain value. However, if we increase the slope length ratio above this critical value $\left(L > L_{crit}\right)$, the slope finds it easier to form a local mechanism. Since $L > W_{crit}$ the mechanism has more opportunities to develop somewhere in the $z-$direction hence p_f again increases.

CONCLUDING REMARKS

The paper has demonstrated the power and advantages of the finite element method for both deterministic and probabilistic slope stability analysis in highly variable soils. Results were presented indicating the limitations of 2D analysis in infinite slope and 3D slope analysis. It was shown that 2D slope analysis is only conservative if the most pessimistic plane in the 3D geometry is chosen. Even then, the result may be excessively conservative. More seriously however, poor selection of the 2D plane for analysis could lead to unconservative results.

Examples of slope risk analysis were presented using the random finite element method (RFEM) developed by the authors. It was shown that single random variable approaches can give unconservative results compared with RFEM using 2D random fields. The key benefit of RFEM is that it does not require any *a priori* assumptions related to the shape or location of the failure mechanism. In an RFEM analysis, the failure mechanism has freedom to "seek out" the weakest path through the random soil, which generally leads to more simulations reaching failure. The importance of spatial variability was further demonstrated in two examples involving an infinite slope and a 3D slope. In both cases, failure to account for spatial variability could lead to unconservative results.

Acknowledgement

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Reliability-Based Design of Foundations - A Modern View

Fred H. Kulhawy[1], P.E., G.E., Dist.M.ASCE,
Kok Kwang Phoon[2], P.E., F.ASCE, and Yu Wang[3], M.ASCE

[1]Professor Emeritus, School of Civil & Environmental Engineering, Hollister Hall, Cornell University, Ithaca, NY 14853-3501; fhk1@cornell.edu
[2]Professor, Department of Civil & Environmental Engineering, National University of Singapore, Block E1A 07-03, 1 Engineering Drive 2, Singapore 117576; kkphoon@nus.edu.sg
[3]Assistant Professor, Department of Civil & Architectural Engineering, City University of Hong Kong, Tat Chee Avenue, Kowloon, Hong Kong; yuwang@cityu.edu.hk

ABSTRACT: Basic concepts in the transition from allowable stress design (ASD) to reliability-based design (RBD) are reviewed. Critical issues related to RBD are noted, including their strengths and weaknesses. It is stressed that current simplified RBD methods need to be improved and expanded. Recommendations then are made for improving factor calibrations, addressing serviceability and economic limit states, and optimizing the foundation design process.

INTRODUCTION

Foundations have been designed successfully for millennia. Up through the 19th century, even into the early 20th century, foundation design was wholly empirical and was based on precedence, rules of thumb, and local experience, which collectively were expressed in texts and building codes as presumptive bearing stresses. With the development of soil mechanics in the early 20th century, and subsequently more rational methods for analyzing stability and movements, the design process evolved as well into allowable stress design (ASD). Late in the 20th century, largely following the lead of structural design practice, a process was initiated to transition from ASD to reliability-based design (RBD) for foundations. This process is not yet complete.

In this paper, some basic comparisons first are made between ASD and RBD. Then some key aspects of RBD are noted, stressing their strengths and weaknesses. Lastly, some issues are discussed that can improve the RBD process and generalize it so that more optimal foundation designs can evolve.

BASIC ASD AND RBD

The purpose of design is to develop a component or system that performs satisfactorily within its design life. Uncertainties are a part of this process, and they have been fully appreciated for a long time (e.g., ENR 1963, Casagrande 1965). In traditional ASD for the ultimate limit state (ULS), a global factor of safety (FS) is introduced to address these uncertainties and to mitigate against potential undesirable outcomes. For foundation design, this FS often is applied to the geotechnical capacity as:

$$F_n \leq Q_n / FS \tag{1}$$

in which F_n = nominal (unfactored) load and Q_n = nominal capacity. Factors of safety from 2 to 3 typically are considered to be adequate in routine foundation design (e.g., Focht and O'Neill 1985). The selection of an appropriate FS is largely subjective, requiring only a broad appreciation of the shortcomings of this method against a background of previous experience. These shortcomings are discussed elsewhere (e.g., Kulhawy and Phoon 2006, 2009; Kulhawy 2010).

The problems associated with ASD can be resolved conceptually by rendering broad, general concepts, such as uncertainties and risks, into precise mathematical terms that can be assessed consistently. This approach fundamentally forms the basis of RBD. Uncertain engineering quantities (e.g., loads and capacities) are modeled by random variables, while design risk is quantified by the probability of failure. Note that risk typically involves the probability and consequence of failure. For foundation RBD, current practice is to account for different consequences of failure indirectly by prescribing a different target probability of failure [e.g., "Reliability Class" in BS EN 1990:2002 (British Standards Institute 2002)]. Note also that "failure" is synonymous with "adverse performance" and does not necessarily refer to a collapse event. Another extremely important part of evaluating design risk, which is included in structural code calibrations, is that, regardless of the range in the input parameters or their variability (within reason), the resulting design risk or probability of failure will be essentially constant. The evolution of geotechnical RBD is discussed elsewhere (e.g., Kulhawy and Phoon 2002).

For simplified RBD, the basic approach is given in Fig. 1. The design equation is:

$$\eta F_n \leq \psi Q_n \tag{2}$$

in which η = load factor (≥ 1) and ψ = resistance factor (≤ 1), and therefore the name "load-and-resistance-factor-design" (LRFD).

Conceptually, the RBD process is relatively simple and straightforward, as follows: (1) establish all components of the load distribution F, (2) establish all components of the capacity distribution Q, (3) select the nominal values for these F and Q distributions to be used in the design equation, (4) select a target probability of failure that would establish the design risk, which essentially sets the separation distance between the F and Q distributions and therefore their overlap, and (5) perform calibration analyses that incorporate steps 1 through 4 to determine values of η and ψ that achieve a consistent design risk across the range in the input parameters and their variability. However, the details can be a bit complicated.

In this process, the probability of failure (p_f) = Prob(Q < F) = Prob(Q - F < 0), in which Prob(·) = probability of an event. The p_f is cumbersome to use when it becomes very small, and it carries the negative connotation of "failure". A more convenient alternative measure of design risk is the reliability index (β), which is defined

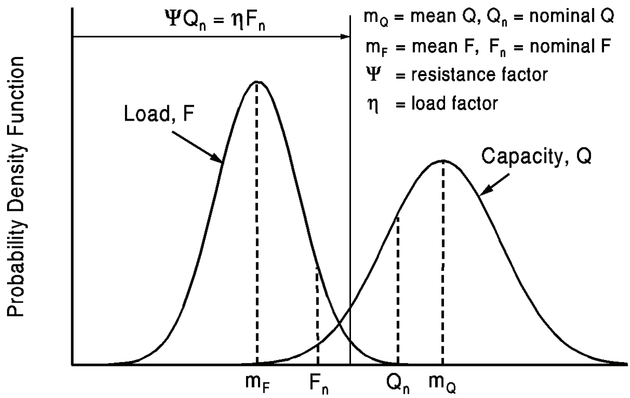

Fig. 1. Simplified reliability-based design

as $\beta = -\Phi^{-1}(p_f)$ in which $\Phi^{-1}(\cdot)$ = inverse standard normal cumulative function given in standard texts on probability. Note that β is not a new measure of design risk; it is an alternative to p_f on a more convenient scale. The reliability indices for most structural and geotechnical components lie between 1 and 4, corresponding to p_f ranging from about 16% to 0.003%, as shown in Table 1. Note that p_f decreases nonlinearly as β increases. For general comparison, the Canadian Building Code uses a target β = 3.5 for the superstructure and the foundations. AASHTO uses a target β = 3.5 for the superstructure and target β values from 2.0 to 3.5 for the foundations.

CALIBRATION PROCESS

The heart of RBD is the calibration process by which values of η and ψ are developed for use in the basic design equation (Eq. 2). For all practical purposes, it is wise and prudent to use the η values developed by our structural colleagues, who have invested a great deal of time and effort to establish these values for the design of the su-

Table 1. Relationship between reliability index and probability of failure (U. S. Army Corps of Engineers 1997)

Reliability Index, β	Probability of Failure, $p_f = \Phi(-\beta)$	Expected Performance Level
1.0	0.16	Hazardous
1.5	0.07	Unsatisfactory
2.0	0.023	Poor
2.5	0.006	Below average
3.0	0.001	Above average
4.0	0.00003	Good
5.0	0.0000003	High

Note: $\Phi(\cdot)$ = standard normal probability distribution perstructure. Use of these values also begins to ensure some degree of compatibility between the design of the superstructure and the foundation.

As described in some detail by Kulhawy and Phoon (2006), calibrations can be, and have been, done by four basic approaches: (1) judgment, (2) fitting, (3) simplified reliability theory, and (4) generalized reliability theory. The first relies on judgment, experience, and performance records, while the second "fits" the ψ values to existing traditional ASD practice ($\psi = \eta$ / FS). Neither of these are RBD.

The third uses simplified reliability theory, in which the load and capacity are modeled as lumped parameters with either normal distributions or lognormal distributions. With these assumptions, simple closed-form solutions are available to assess the ψ values, which are of the functional form:

$$\psi = f\,(\eta, F_n, m_F, COV_F, \beta_T, Q_n, m_Q, COV_Q) \tag{3}$$

in which COV_F = coefficient of variation (standard deviation divided by mean) of the load, COV_Q = coefficient of variation of the capacity, β_T = target reliability index, and all other terms were defined previously. Ideally, the first four (load) terms would be provided by our structural colleagues. Eq. (4) is one such expression that is used often for two lognormal distributions:

$$\psi = \frac{\eta\left(F_n/m_F\right)\sqrt{\left(1+COV_F^{\,2}\right)/\left(1+COV_Q^{\,2}\right)}}{\left(Q_n/m_Q\right)\exp\left\{\beta_T\sqrt{\ln\left[\left(1+COV_F^{\,2}\right)\left(1+COV_Q^{\,2}\right)\right]}\right\}} \tag{4}$$

The fourth way is to use a more generalized reliability theory (e.g., Phoon et al. 1995, 2003 a,b). With this approach, load distributions that better model observed phenomena can be incorporated, and multiple types of distributions can be superimposed. For example, the Gumbel model perhaps best represents wind loading, while a normal distribution might be best for dead loads, and lognormal might be best for some soil parameters. To model these types of variables, more sophisticated approaches are necessary, such as the first-order reliability method (FORM), which involves detailed numerical procedures. Although the procedures are a bit tedious, they are needed to model the distributions and their components most accurately. This methodology and a detailed example are given by Phoon et al. (2003 a,b). As with the simplified reliability theory, the resulting ψ values will be a function of the same parameters as given in Eq. 3, plus any specifics for the different types of distributions, plus any further problem generalities introduced in the numerical solution. As should be expected, there are no simple closed-form equations for the results.

Both the simplified and generalized theories have been used extensively in simplified RBD, as discussed below.

SIMPLIFIED RELIABILITY-BASED DESIGN

Simplified RBD equations are popular because engineers can design for a target probability of failure (p_T) or target reliability index (β_T), albeit approximately, while performing only one check per trial design. No tedious Monte Carlo simulations or other equivalent probabilistic analyses are needed to employ a design equation such as Eq. 2. To the best of our knowledge, this simplified RBD approach is adopted in all geotechnical RBD codes to date. The practical challenge is to calibrate a set of resistance factors that will produce designs that satisfy β_T, at least approximately, over a range of representative design scenarios. The ideal situation would result in the smallest possible set of factors that cover the widest possible design scenarios and result in the least deviation from β_T.

Basic Load and Resistance Factor Design (LRFD)

Perhaps the most popular simplified RBD format in North America is LRFD, as given in Eq. 2. In the earlier code developments, the LRFD calibrations were relatively simple, relying heavily on Eq. 4 and using typical values of COV that were available at the time. In addition, because the knowledge base also was rather limited, judgment and fitting necessarily were a part of the calibration process.

More recent calibrations typically have been done by assuming that the mean actual or measured capacity (m_Q) can be modeled as the product of a mean bias factor (b_Q) and the nominal or calculated capacity (Q_n) (e.g. Paikowsky, 2004; Paikowsky et al. 2010). The bias factor is considered as a lognormal random variable, and the statistics of the bias factor are estimated from a load test database. This bias factor is essentially a "lumped" parameter that includes both systematic bias from the calculation model and random effects from the parametric and model uncertainties. As a lumped parameter, the statistics of the bias factor are a function of the design parameters (e.g., geometrical and soil). In the ideal case, these statistics are completely insensitive to the design parameters, so they can be applied to all possible problem geometries, geologic formations, soil properties, etc. In this ideal case, statistics estimated from a load test database are robust and can be applied confidently to the full range of design scenarios encountered in practice. In the worst case, the statistics are very sensitive to all of the design parameters. For example, the statistics for short piles will differ from those for long piles because of physical reasons (e.g., side resistance dominates total resistance in long piles) or statistical reasons (e.g., spatial averaging of soil strength is more significant in long piles). In this case, it is highly debatable that the statistics derived from a load test database are applicable to circumstances not covered by the database. Phoon et al (1995) and Kulhawy and Phoon (2002) have highlighted this potential problem. Paikowsky (2002) has provided statistics that demonstrate clearly that the statistics of bias factors generally are dependent on some design parameters.

If a particular design parameter, such as pile depth to diameter ratio (D/B), is influential, then it likely will be important to divide the range of the parameter into two or more segments and estimate the different statistics within the different segments. This segmentation procedure is a reasonable and practical solution to the dependency problem. However, there is a more subtle, but rarely appreciated, problem in estimating sensitive statistics from a load test database. The problem is that if a statistic is sensitive, it is important to ensure that the calibration examples are fairly uniformly distributed over any one segment of the parameter range. This issue may be difficult to accomplish in a load test database because the examples are collected from the literature rather than from a single comprehensive research program. For example, in a database segment defined for D/B > 10, it could be that D/B actually is largely between 30 and 50, which would not be representative.

Although these issues are well-known, codes largely ignore them. Subsequently, geotechnical resistance factors are commonly presented as single values, such as in the Table 2 excerpt from AASHTO for drilled shafts in undrained uplift loading. The ψ_u value is to be applied for the α method given by O'Neill and Reese (1999).

Extended LRFD and Multiple Resistance Factor Design (MRFD)

Because of the various limitations in the basic LRFD approach, Phoon et al. (1995) proposed alternatives to this methodology that relied more on basic geotechnical issues than conformance with established LRFD procedures that evolved from structural engineering practice. The proposed approach simply allowed the "best" geotechnical calculation models to be used directly in the reliability calibration process, rather than artificially simplifying the capacity into a single lognormal random variable to fit the requirement of a closed-form reliability formula. The available calculation models were examined and compared with available load test data from the field and laboratory, as well as numerical simulations. These analyses led to a "best" calculation model (most accurate, least variability, essentially no bias) that was used in the calibrations.

As part of this process, the key design parameters, such as the effective stress friction angle, the undrained shear strength, and the coefficient of horizontal soil stress, are modeled directly as random variables. A major advantage of this approach is that the range of an influential design parameter, and its variability, can be segmented and calibration points within each segment or domain can be selected to ensure uniform coverage of the variables during the calibration process (Fig. 2). The disadvantages are: (1) the closed-form reliability formula for lognormal random variables can not be applied, and a more involved reliability calculation method such as the First-Order Reliability Method (FORM) is needed, and (2) it is necessary to adjust the resistance factor over each segment by a tedious optimization procedure to minimize the deviation from the target reliability at each calibration point. However, once the calibration process is complete, the user never has to perform any reliability calculations or do any factor optimization, as described below.

Based on extensive studies of the optimization process, and detailed evaluation of typical ranges in the key design parameters and the COV ranges that should be encountered for them, it was found that a 3×3 segmenting was sufficient for practical

Table 2. Undrained ultimate uplift resistance factor for drilled shafts designed using AASHTO (2010)

Soil	Ψ_u
Clay	0.35^a

Note: Target reliability index = 2.5-3.5 (nominal 3.0)

a - reduce by 20% if a single shaft (equivalent to $\beta_T = 3.5$)

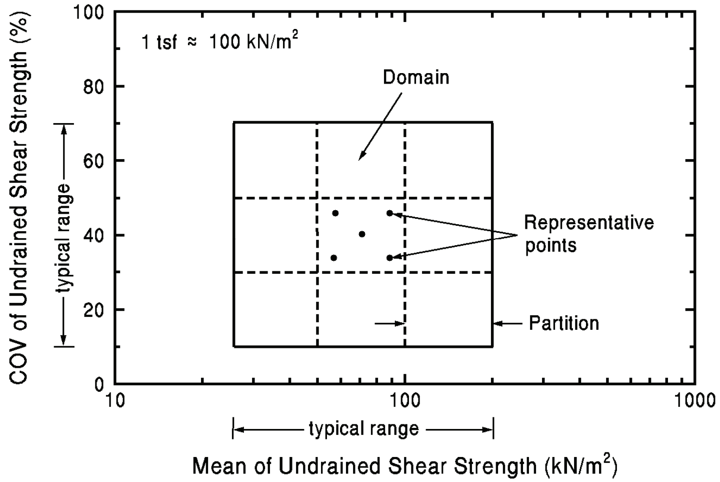

Fig. 2. Partitioning of parameter space for calibration of resistance factors

calibration (Phoon et al. 1995). The typical ranges of COV to be expected are shown in Table 3. As can be seen, low variability corresponds to good quality direct lab or field measurements, medium is typical of most indirect correlations, while high represents strictly empirical correlations.

Once the calibration is complete, Eq. 2 is used directly with resistance factors such

Table 3. Ranges of soil property variability for reliability calibration (Phoon et al. 1995, updated Phoon and Kulhawy 2008)

Geotechnical parameter	Property variability	COV (%)
Undrained shear strength	Low[a]	10 - 30
	Medium[b]	30 - 50
	High[c]	50 - 70
Effective stress friction angle	Low[a]	5 - 10
	Medium[b]	10 - 15
	High[c]	15 - 20
Horizontal stress coefficient	Low[a]	30 - 50
	Medium[b]	50 - 70
	High[c]	70 - 90

a - typical of good quality direct lab or field measurements
b - typical of indirect correlations with good field data, except for the standard penetration test (SPT)
c - typical of indirect correlations with SPT field data and with strictly empirical correlations

as given in the first three columns of Table 4. Use of this table is straightforward with an appropriate site investigation. First, the mean trend line of the key design parameter (s_u for undrained capacity problems) is evaluated with depth. Second, the variability (COV) of that parameter about the mean trend line is determined either by direct measurements or estimation using Table 3. Third, the overall mean strength of the clay deposit, within the foundation zone of influence, is evaluated and placed into one of three broad groupings. Fourth, the range of COV in the key design parameter is assessed, again within three broad categories. Fifth, the design ψ is selected.

As noted previously, in LRFD (basic or extended), the ψ value is applied to the total geotechnical capacity, which is composed of distinctly different components. For example, the uplift capacity of a drilled shaft during undrained loading is composed of the side resistance, tip suction, and self-weight, all of which are displacement-dependent. These components, in turn, generally are nonlinear functions of more fundamental design parameters, such as the foundation depth, diameter, and weight and the soil undrained shear strength. The relative contribution of each component to the overall capacity is not constant, and the degrees of uncertainty associated with each component are different. For example, the shaft weight is almost deterministic in comparison to the undrained side resistance, because the COV of the unit weight of concrete is significantly smaller than the COV of the undrained shear strength.

This issue has been examined in detail (e.g., Phoon et al 1995, 2003b), and it was found that a more consistent result in achieving a constant target reliability index was obtained using the following equation:

$$\eta\ F_n \leq \psi_{su}\ Q_{sun} + \psi_{tu}\ Q_{tun} + \psi_w\ W \tag{5}$$

in which the ψ values are calibrated for each distinctive term in the geotechnical capacity equation, as given in columns 4 through 6 of Table 4 for illustration. Eq. 5 is defined as "multiple-load-and-resistance-factor-design" (MRFD).

Table 4. Undrained ultimate uplift resistance factors for drilled shafts designed by $F_{50} = \Psi_u Q_{un}$ or $F_{50} = \Psi_{su} Q_{sun} + \Psi_{tu} Q_{tun} + \Psi_w W$ (Phoon et al. 1995)

Clay	COV of s_u (%)	Ψ_u	Ψ_{su}	Ψ_{tu}	Ψ_w
Medium	10 - 30	0.44	0.44	0.28	0.50
(mean s_u = 25 - 50 kN/m^2)	30 - 50	0.43	0.41	0.31	0.52
	50 - 70	0.42	0.38	0.33	0.53
Stiff	10 - 30	0.43	0.40	0.35	0.56
(mean s_u = 50 - 100 kN/m^2)	30 - 50	0.41	0.36	0.37	0.59
	50 - 70	0.39	0.32	0.40	0.62
Very Stiff	10 - 30	0.40	0.35	0.42	0.66
(mean s_u = 100 - 200 kN/m^2)	30 - 50	0.37	0.31	0.48	0.68
	50 - 70	0.34	0.26	0.51	0.72

Note: Target reliability index = 3.2

The LRFD and MRFD formats were compared by evaluating how closely the actual achieved values of β were to the target value β_T. For the extended LRFD format, $(\beta - \beta_T) / \beta_T$ was about plus or minus 5 to 10%. However, for the MRFD format, the $(\beta - \beta_T) / \beta_T$ values were only about 1/2 to 2/3 those from the extended LRFD format, indicating significant improvement over the LRFD format when each distinctive term is assigned a resistance factor. For the basic LRFD, there is only one value (e.g., Table 2), and it is not specified or known whether the calibration was done for low, medium, or high variability. Assuming a median value, β would be greater than β_T for COV less than the median and would be less than β_T for COV greater than the median, by amounts exceeding 10% at the limits.

From these results, it should be clear that the MRFD format should be preferred. Not only is the β_T more closely achieved, it is being done using proper geotechnical design equations where the relative weighting of each term is being addressed explicitly. And there is direct recognition of data quality in assessing the property variability. With these tools, an experienced engineer should have no trouble in selecting the appropriate ψ values for an improved design.

It must be remembered that all ψ values strictly are applicable only for the calibration conditions used, such as the load model, calculation model, model variabilities, and how these models and their subcomponents are to be used. Any changes require re-calibration.

SERVICEABILITY LIMIT STATE

The serviceability limit state (SLS) is the second limit state that is evaluated in foundation design. It often is the governing design criterion, particularly for large-diameter shafts and shallow foundations. Unfortunately, foundation movements are difficult to predict accurately, so reliability-based assessments of the SLS are not common. Ideally, the ULS and the SLS should be checked using the same reliability-based design principle. However, the magnitude of uncertainties and the target reliability level for SLS are different from those of ULS, but these differences can be assessed consistently using reliability-calibrated deformation factors (analog of resistance factors).

Single Foundation Behavior

Phoon et al. (1995) first examined this issue by employing large databases of foundation load-displacement data that could be normalized and evaluated. It was found that most databases could be best characterized by a two-parameter hyperbolic model, as illustrated in Fig. 3 for drilled shafts in uplift loading and as given below:

$$F / Q_u = y / (a + b\,y) \tag{6}$$

in which F = load, Q_u = uplift capacity, y = displacement, and a and b are the curve-fitting parameters.

Recently, Phoon and Kulhawy (2008) summarized developments in SLS and noted that this model was most appropriate for the following foundation types: spread foun-

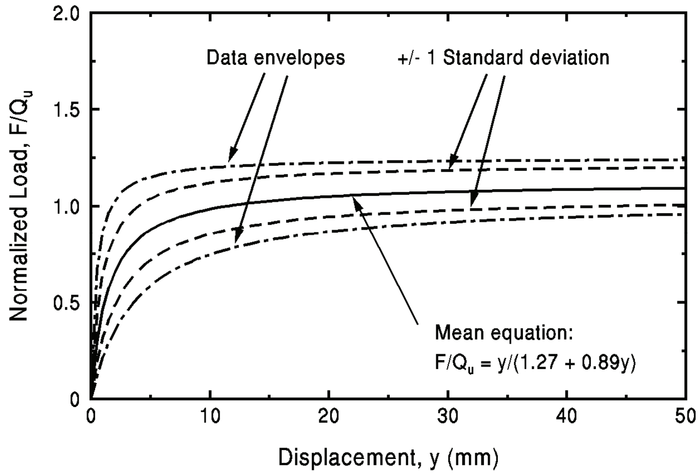

Fig. 3. Load-displacement curves for drilled shafts in uplift (Phoon et al 1995)

dations in uplift (drained and undrained), drilled shafts in uplift and lateral-moment (drained and undrained), drilled shafts in compression (undrained), augered cast-in-place (ACIP) piles in compression (drained), and pressure-injected footings in uplift (drained). Drilled shafts in compression (drained) were fitted best by an exponential model. More recently, Akbas and Kulhawy (2009a) also showed that the hyperbolic model was appropriate for spread foundations in compression (drained).

The reliability of a foundation at the ULS is given by the probability of the capacity being less than the applied load. It is logical to follow the same approach for the SLS, where the capacity is replaced by an allowable capacity that depends on the allowable displacement (Phoon et al. 1995, Phoon and Kulhawy 2008). The nonlinearity of the load-displacement curve is captured by the two-parameter hyperbolic curve-fitting equation. The uncertainty in the entire load-displacement curve is represented by a relatively simple bivariate random vector containing the hyperbolic parameters as its components, and the allowable displacement is introduced as a random variable for reliability analysis. The resulting design equation is given below:

$$F_n = \Psi_u Q_{uan} = \Psi_u [Q_{un} y_a / (m_a + m_b y_a)] \quad (7)$$

in which Ψ_u = uplift deformation factor given in Table 5, Q_{uan} = nominal allowable uplift capacity, Q_{un} = nominal uplift capacity, y_a = allowable displacement, and m_a and m_b = mean values of a and b. Note that the deformation factors are calibrated for a smaller β_T than for the ULS.

Differential Settlement of Footings

In foundation design, the SLS for individual foundations is important, and it can be addressed as above. As long as the ground conditions are reasonably consistent, the

Table 5. Undrained uplift deformation factors for drilled shafts designed using $F_{50} = \Psi_u Q_{uan}$ (Phoon et al 1995)

Clay	COV of s_u (%)	Ψ_u
Medium (mean s_u = 25 - 50 kN/m^2)	10 - 30	0.65
	30 - 50	0.63
	50 - 70	0.62
Stiff (mean s_u = 50 - 100 kN/m^2)	10 - 30	0.64
	30 - 50	0.61
	50 - 70	0.58
Very Stiff (mean s_u = 100 - 200 kN/m^2)	10 - 30	0.61
	30 - 50	0.57
	50 - 70	0.52

Note: Target reliability index = 2.6

differential settlements are likely to be minimal. However, for certain types of soil-foundation systems, such as spread footings on granular soils, the question of differential settlement can be very important. Conventional practices are empirical and commonly assume that the differential settlement is just some fixed percentage of the total computed settlement, typically ranging from 50 to 100%.

Akbas and Kulhawy (2009b) suggested a probabilistic approach to this problem to provide a more rational method of assessment. For illustration, they used the Burland and Burbridge (1985) settlement estimation method in the following form for estimating differential settlements:

$$\rho_{m1} - \rho_{m2} = (1 / M)\, f_s\, f_l\, q\, B^{0.7}\, (I_{c1} - I_{c2}) \qquad (8)$$

in which ρ_{m1} and ρ_{m2} = measured settlements for neighboring footings 1 and 2, M = model factor (ratio of calculated-to-measured settlement), f_s = shape factor, f_l = depth of influence correction factor, q = net increase in effective stress at foundation level, B = footing width, and I_{c1} and I_{c2} = compressibility index values for neighboring footings 1 and 2 (= 1.71 / $N_{60}^{1.4}$), with N_{60} = standard penetration test N value corrected to an average energy ratio of 60%. The stress, model uncertainty, and geotechnical parameters were treated as random variables, including the I_c values that are correlated as a function of distance between the footings.

The results of the study are presented in the following form:

$$q_d = \Psi_D^{SLS}\, q_n = \Psi_D^{SLS}\, [\rho_a / (f_s\, f_l\, B^{0.7}\, I_{cn})] \qquad (9)$$

in which Ψ_D^{SLS} = deformation factor for differential settlement, q_n = nominal value of foundation applied stress, ρ_a = allowable settlement limit, q_d = revised design value of q_n, and I_c = nominal I_c calculated using mean N_{60}. The Ψ_D^{SLS} values are given in a lengthy table and are a function of the allowable angular distortion (1/150, 1/300, 1/500), the COV of N (25 to 55%), and the center-to-center footing distance (3 to

9m). For most parametric combinations, the deformation factors were less than 1.0, which is in contrast to some current practices for SLS. These practices may be unconservative.

ECONOMICALLY- OPTIMIZED LIMIT STATE

During the design process, either in ASD or RBD, a number of feasible designs will result that satisfy the ULS and SLS criteria. Construction issues, such as standard sizes of piles or augers and similar, and equipment availability for the site, will of course be part of the evaluation process. Even after these issues have been addressed, there will be a number of designs that satisfy the criteria.

At this time, economic issues need to be addressed, leading to the third limit state, the economically-optimized limit state (EOLS). The EOLS should be adopted to finalize the design, which will be the one with the minimum construction cost. Wang and Kulhawy (2008) outlined a straightforward optimization process that allows the incorporation of ULS and SLS designs with construction costs to select the most cost-effective foundation of those being considered. The foundation construction costs were estimated using published, annually-updated, unit cost data, such as Means Building Construction Cost Data (Means 2007). For their example, Table 6 summarizes the U. S. national average unit cost for constructing drilled shafts with diameters equal to of 0.9, 1.2, and 1.5m. The costs for constructing a unit depth (i.e., 0.3m) of these drilled shafts are USD77.5, 116, and 157, respectively (USD = U.S. dollars). The construction costs for all feasible designs would be calculated as the product of their unit costs and shaft depths, and the final design would be determined by comparing their construction costs. The final design would satisfy the ULS, SLS, and EOLS, including their reliability requirements.

EXPANDED RELIABILITY-BASED DESIGN APPROACH

The current RBD methodologies tend to be as time-consuming as the prior ASD methods, because one design is considered at a time. This approach is not conducive to optimal design. To address this limitation, a more general RBD approach, known as expanded RBD (RBD^E), was developed recently (Wang et al. 2011). The expanded RBD formulates the foundation design process as an expanded reliability problem in which a single run of Monte Carlo simulations (MCS) is used to address, explicitly and simultaneously, the ULS, SLS, EOLS, and reliability requirements of the design. An expanded reliability problem, as described herein, refers to a reliability analysis of

Table 6. Summary of drilled shaft unit construction costs (after Means 2007)

Shaft diameter, B (m)	National average unit construction cost (for unit shaft depth D = 0.3 m)
0.9	USD77.5
1.2	USD116.0
1.5	USD157.0

a system in which a set of system design parameters are considered artificially as uncertain with probability distributions specified by the user for design exploration purposes. Design parameters, such as pile depth D and diameter B, are treated artificially as discrete uniform random variables (basically a more general version of the representative points shown in Fig. 2), and the design process is considered as a process of finding failure probabilities for designs with various combinations of B and D [i.e., conditional probability p(Failure|B,D)] and comparing them with a target probability of failure p_T, which could be a ULS or SLS requirement. A single run of MCS with a total sample number n is performed to evaluate p(Failure|B,D), as illustrated in Fig. 4. The traditional ULS and SLS deterministic design calculations are repeated n times in the MCS, and it is equivalent to a systematic sensitivity study that contains n different design cases with various input parameters and/or design parameters. Then, the conditional failure probability p(Failure|B,D) is calculated from the MCS results as (Wang et al. 2011):

$$p(\text{Failure}|B,D) = \frac{p(B,D|\text{Failure})}{p(B,D)}\, p_f = \frac{n_B\ n_D\ n_1}{n} \tag{10}$$

in which n_B and n_D = number of possible discrete values for B and D, and n_1 = number of MCS samples where failure and a specific set of B and D values occur simultaneously. The p(Failure|B,D) therefore is obtained by counting the failure sample number n_1 for various combinations of B and D and using Eq. 10.

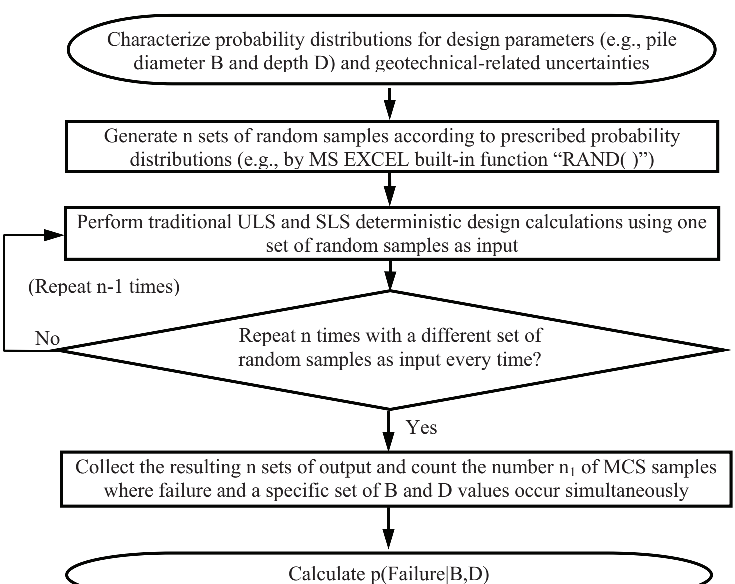

Fig. 4. Flow chart for Monte Carlo Simulation in expanded RBD approach

Fig. 5 shows an illustration of the p(Failure|B,D) obtained from MCS. Note that the relationships given in Fig. 5 are variations of p_f as a function of the design parameters B and D that represent different designs. From this perspective, these are the results of a sensitivity study on p_f versus the design parameters. Feasible designs can be inferred directly from the figure, and they are those with p(Failure|B,D) $\leq p_T$. The feasible designs satisfy the ULS, SLS, and reliability requirements. Then, the EOLS requirement can be considered to select the final design from the feasible ones, as described previously.

When compared with the current RBD approach, the RBD^E approach is perhaps more transparent and "visible" to designers, and it has the following advantages: (1) the ULS and SLS calculation models are established the same as in ASD, and designers have the flexibility to make appropriate design assumptions and modifications that best suit the design situation, (2) the uncertainties are modeled explicitly and directly, and designers have the flexibility to include uncertainties deemed appropriate, and (3) it gives designers the ability to adjust p_T, without additional calculations, to accommodate specific project needs, and it provides insight into how the expected performance level changes as the design parameters change.

Finally, the MCS in RBD^E is conceptually and mathematically simple (i.e., it is just a repetitive computer execution of the ULS and SLS deterministic design calculations), and it decouples the assessment of reliability from the traditional deterministic design calculations. This approach effectively removes the reliability algorithm, and it

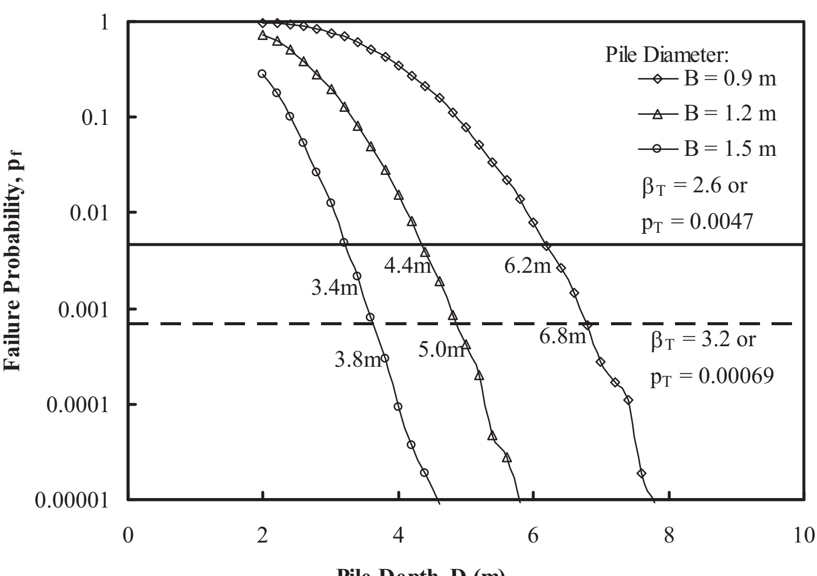

Fig. 5. An illustration of conditional failure probability from MCS (after Wang et al. 2011)

can be implemented easily in a spreadsheet environment (e.g., MS Excel), which is very convenient for engineers who frequently develop designs using spreadsheets. With modern computer technology, thousands of MCS samples can be calculated for conventional foundation designs within seconds.

UNIFORM QUANTILE CODE CALIBRATION METHOD

The MRFD approach was proposed to achieve a closer agreement with β_T for a realistic range of design scenarios. One key component of MRFD is to partition the influential design parameters into domains (Fig. 2) and to calibrate different sets of resistance factors for each domain, as illustrated by Tables 4 and 5. It is possible to envision situations, especially if there are two or more highly influential design parameters, in which the number of sets of resistance factors could proliferate to such an extent that MRFD becomes unwieldy. Accordingly, it is worth examining whether the resistance factors in the simplified RBD format can be replaced by another factor that is less sensitive to the design parameters.

Recently, Ching and Phoon (2011) postulated that a single quantile [point on the cumulative distribution function (CDF) of a random variable] can cover a wider range of design scenarios than a single resistance factor if it is applied within the following uniform quantile RBD format:

$$F_{1-\varepsilon} \leq Q_\varepsilon \tag{11}$$

in which $F_{1-\varepsilon}$ = (1-ε)% upper quantile load and Q_ε = ε% lower quantile capacity. The idea of using a 5% lower quantile (or 5% exclusion limit) as a nominal/characteristic design value for capacity is well-established in existing design codes. The key difference here is that it is possible to find an appropriate value of ε so that Eq. 11 achieves the p_T. In fact, the relationship between ε and p_T can be determined using MCS from the following (Ching and Phoon 2011):

$$p\,[(Q / F) / (Q_\varepsilon / F_{1-\varepsilon}) < 1] = p_T \tag{12}$$

in which Q/F = random version of the factor of safety and $Q_\varepsilon / F_{1-\varepsilon}$ = a deterministic factor of safety evaluated at appropriate quantiles. Despite this difference, both numerator (Q / F) and denominator ($Q_\varepsilon / F_{1-\varepsilon}$) in Eq. 12 are algebraically identical. Therefore, deterministic design parameters such as foundation dimensions and mean soil strength would be normalized and, in some cases, completely cancelled out. If Q and F are independent lognormal random variables, it can be shown that the ε-β_T relationship is given by:

$$\varepsilon \approx \Phi(-0.75\,\beta_T) \tag{13}$$

in which Φ(·) = cumulative distribution function for the standard normal random variable. To be precise, the coefficient in Eq. 13 weakly depends on COV_Q / COV_F. Its average value is 0.75 for $1/3 < COV_Q / COV_F < 3$. For comparison, the ψ-β_T relationship is given approximately by $\psi = \exp(-0.75\,\beta_T\,COV_Q)$, assuming $m_Q = Q_n$

(Ravindra and Galambos 1978). Therefore, for $\beta_T = 3$, $\psi = 0.64$, 0.41, and 0.26 for $COV_Q = 0.2$, 0.4, and 0.6, respectively. Clearly, the current practice of recommending a single resistance factor of, say $\psi = 0.4$, carries an implied $COV_Q \approx 0.4$. This ψ is not applicable for lower or higher COV_Q. However, for $\beta_T = 3$, $\varepsilon = 0.012$ can be applied for simplified RBD using Eq. 11, regardless of COV_Q.

It is worth emphasizing that the uniform quantile method has no theoretical resemblance to the application of quantiles for characteristic/nominal values in some RBD codes such as Eurocode and the ACI and AISC structural codes. The latter quantile is prescribed by design codes without reference to p_T. For example, a quantile between 5% and 10% typically is prescribed for the concrete compressive strength, f_{cu}, in structural design codes. The main purpose of this definition is to give a suitably conservative compressive strength that varies consistently with the COV of f_{cu}. This same quantile is applied to different performance functions, for example the moment/ shear capacity of a beam or compression capacity of a column. The quantile in the Ching and Phoon (2011) method is fundamentally different. It is calibrated, rather than prescribed, to achieve a specific p_T, as illustrated in Eq. 13 for the simple case of two independent lognormal random variables.

Note that once $\varepsilon = 0.012$ is determined using Eq. 13 for a prescribed $\beta_T = 3$, simplified RBD that is based on lognormality of Q and F can be carried out using Eq. 11 using the following common equations:

$$F_{1-\varepsilon} \approx m_F \exp[\Phi^{-1}(1-\varepsilon)\, COV_F] \tag{14a}$$

$$Q_{\varepsilon} \approx m_Q \exp[\Phi^{-1}(\varepsilon)\, COV_Q] \tag{14b}$$

in which $\Phi^{-1}(\cdot)$ = inverse cumulative distribution function for the standard normal random variable. Eq. 11 can be converted to the more familiar mean factor of safety (m_Q / m_F) as follows:

$$Q_{\varepsilon} \geq F_{1-\varepsilon} \;\Rightarrow\; m_Q / m_F \geq \exp[\Phi^{-1}(1-\varepsilon)\, COV_F] / \exp[\Phi^{-1}(\varepsilon)\, COV_Q] \tag{15}$$

Assuming $COV_F = 0.2$, $COV_Q = 0.4$, and $\varepsilon = 0.012$, Eq. 15 requires a mean factor of safety larger than 1.6 / 0.4 = 4 to achieve $\beta_T = 3$.

To illustrate the potential value of the uniform quantile method, Ching and Phoon (2011) presented three examples for the side resistance of a pile installed in: (1) homogeneous clay, (2) homogeneous sand, and (3) a clay layer overlying a sand layer. The first was analyzed using the α-method, the second used an empirical correlation between SPT-N and the side resistance, and the third is a composite method; details are given in the paper. Simplified RBD equations were calibrated using the uniform quantile method, the FORM design point method, and the MRFD method. The FORM design point method is used for calibration of partial factors in Eurocode, as given in Annex C of BS EN 1990:2002 (British Standards Institute 2002). Much earlier it was used for steel design in the AISC code (Ravindra and Galambos 1978). Only one calibration pile can be considered in the FORM. For direct comparison, the MRFD is applied without partitioning the calibration domain, since FORM is calibrated using only 1 pile and quantile does not involve partitioning. Therefore the

MRFD results are the worst possible for this method. A "normal" calibration would fare better.

Once the calibrations were completed, a large number of piles (540 for examples 1 and 2 and 14,580 for example 3), different from those used in the calibrations, were selected to verify if the designs arising from the simplified RBD formats would achieve the $\beta_T = 3.0$ consistently. The reliability indices achieved by these verification piles are summarized in Table 7 and include the mean, COV, highest, and lowest values. Note that a $\beta > 4.75$ (corresponding to $p_f < 10^{-6}$) is an error flag indicating that p_f is too small and can not be estimated using the Monte Carlo simulation sample size adopted (10^6). An "ideal" simplified RBD format would produce mean $\beta = \beta_T = 3.0$, COV $\beta = 0$, highest $\beta = 3.0$, and lowest $\beta = 3.0$.

Some interesting observations follow from these results. First, FORM can produce $\beta \approx 3.0$ in homogeneous clay or sand even though it is calibrated using a single pile. However, both FORM and MRFD result in more scatter (larger COV β) when the degree of uncertainty is large (model uncertainty with N is larger than with α). Table 7b shows the results of piles in homogeneous sand designed using a simple correlation between SPT-N and the side resistance. In this table, the lowest β values from FORM and MRFD are significantly smaller than $\beta_T = 3.0$, meaning that the design of some piles is very unconservative. Table 7c shows the results of piles in one layered

Table 7. Verification examples (modified from Ching and Phoon 2011)

a. homogeneous clay

Calibration methods	Uniform quantile		FORM	MRFD
No. of calibration scenarios	1 pile	2^3 piles	1 pile	2^3 piles
Mean β	3.02	3.00	2.96	2.98
COV β	0.03	0.03	0.03	0.03
Highest β	3.27	3.15	3.20	3.15
Lowest β	2.79	2.82	2.71	2.78

b. homogeneous sand

Calibration methods	Uniform quantile		FORM	MRFD
No. of calibration scenarios	1 pile	2^3 piles	1 pile	2^3 piles
Mean β	3.00	3.01	2.96	2.93
COV β	0.03	0.02	0.24	0.19
Highest β	3.19	3.13	>4.75	>4.75
Lowest β	2.77	2.80	1.68	1.97

c. clay layer overlying sand layer

Calibration methods	Uniform quantile		FORM	MRFD
No. of calibration scenarios	1 pile	2^6 piles	1 pile	2^6 piles
Mean β	3.01	2.99	2.59	2.92
COV β	0.14	0.10	0.26	0.17
Highest β	4.10	3.46	>4.75	4.19
Lowest β	1.99	2.17	0.18	1.70

soil. It is clear that all three methods perform worst for this case, and FORM gives the lowest $\beta = 0.18$! This layered soil problem is discussed in greater detail by Phoon et. al. (2011). Clearly, layered soils pose a challenge to all existing RBD calibration methods.

As noted previously, the concept of LRFD originated in structural design, where a single ψ is adequate, mainly because the COVs of structural materials are relatively small and lie within a narrow band between 5% and 20%. For the wider range of COVs shown in Table 3 that are typical for foundations, it is not possible to adopt a single ψ and achieve a consistent β_T - at the same time! However, this goal may be possible if the resistance factors are replaced by quantiles. In principle, the uniform quantile method can be applied to any limit state, and quantiles can be defined at the level of soil parameters, capacity components, or total capacity. For more complicated performance functions requiring finite elements (or alternatives) as a solution method, the quantiles would be applied at the level of input soil parameters. Regardless, layered soil profiles clearly present a challenge for all methods at this time. Further developments in the uniform quantile method have the potential to develop more robust simplified RBD formats for foundation engineering.

SUMMARY AND CONCLUSIONS

The basic concepts in allowable stress design (ASD) and reliability-based design (RBD) of foundations were reviewed. Key issues in the transition from ASD to RBD were discussed, stressing their strengths and weaknesses. It is shown that current simplified RBD methods need to be improved and expanded. Recommendations are made for improving calibrations and design use by incorporating the ultimate, serviceability, and economic limit states and optimizing the foundation design process. Lastly, some issues are discussed that may improve the RBD process so that more optimal foundation designs can evolve.

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Assessment of Slope Stability

Serge Leroueil[1] and Luciano Picarelli[2]

[1]Department of Civil and Water Engineering, Laval University, Quebec City, PQ, Canada, Serge.Leroueil@gci.ulaval.ca
[2]Department of Civil Engineering, Secunda Università di Napoli, Naples, Italy

ABSTRACT: Assessing the stability of slopes is a difficult task that requires a rigorous approach. This paper starts by presenting a geotechnical characterization of slope movements that helps in classifying the information related to slopes. The paper then shows the tremendous progresses that have recently been made for monitoring slopes, understanding slope behavior, and for numerically simulating coupled hydro-mechanical problems in saturated/unsaturated soils. A methodology for assessing the stability of slopes is presented. It includes qualitative and quantitative approaches and, separately, the characterization of the post-failure stage. Finally, there is some discussion on the use of factor of safety, use of numerical models and consideration of risk.

INTRODUCTION

Assessing the stability of existing slopes is probably the most difficult task of the geotechnical engineer. Literature generally refers to landslides that have occurred but engineer's reality is most of the time to assess the stability of existing, still standing slopes. Stating on the stability of a given slope is also an important responsibility for the engineer as excessive movements may have important safety and economic consequences.

It is clear that the first and essential condition for assessing the stability of a slope is the understanding of the mechanical processes that lead or may lead to movements or failure. After general remarks on the organization of the information on slopes, recent developments, in particular on methods of survey and monitoring, understanding of slope behavior and numerical modeling, are described. Finally the paper proposes a methodology for the assessment of slope stability in practice. A priori, the approach presented concerns natural slopes, but could be adapted to engineered slopes. Also, even if some aspects are of general use, this paper concerns mostly slopes in soils.

Due to the limited space allowed, this paper is not a State-of-the-Art on the assessment of slope stability, but rather a set of remarks that seem important to the authors. For the same reason, the number of citations has been limited and the reader is encouraged to examine them to find additional references. It is worth mentioning that in a recent book, Cornforth (2005) covers many aspects of slope engineering and that other books on landslides are in various stages of preparation.

ORGANIZATION OF THE INFORMATION ON SLOPES

To help in better understanding and analyzing slope movements, and organizing the knowledge on slope behavior, Vaunat et al. (1994) and Leroueil et al. (1996) proposed a "Geotechnical characterization of slope movements" that can be schematized as a 3-D matrix (Fig. 1) with the three axes corresponding to: types of movement, types of material involved, and stages of movement. The types of movement are those proposed by Cruden et al. (1994) and Cruden and Varnes (1996), i.e. falls, topples, slides, lateral spreads and flows. For materials, Varnes (1978) considered 3 main classes, i.e. rock, debris and earth. They are not sufficient to describe and take into account the mechanical behavior of the geomaterials, and more detailed classifications have been proposed (e.g. Fig. 2). Even such classification is not sufficient and it may be important to specify if the sand is loose or dense (contractant or dilatant),if the soil is saturated or unsaturated, etc. It is also suggested dividing slope movements in four stages that are associated with their own controlling laws and parameters (Fig. 3): the pre-failure stage, including all the deformation processes that may occur before failure, even if this latter never happens; the onset of failure, characterized by the formation of a continuous shear surface (slip surface) through the entire soil mass as this is the fundamental mechanism of failure in soils (it is different in the case of rock masses); the post-failure stage, which includes movement of the soil mass involved in the landslide, from just after failure until it essentially stops (flows are thus post-failure movements); the reactivation (or active) stage, when a soil mass slides along one or several pre-existing shear surfaces. At an active or reactivated stage, the residual strength is mobilized along the slip surface, which is different than for a first-time failure. The International Committee on Landslides (JTC1) is presently working on a revision of the classification proposed by Varnes (1978) and Cruden and Varnes (1996).

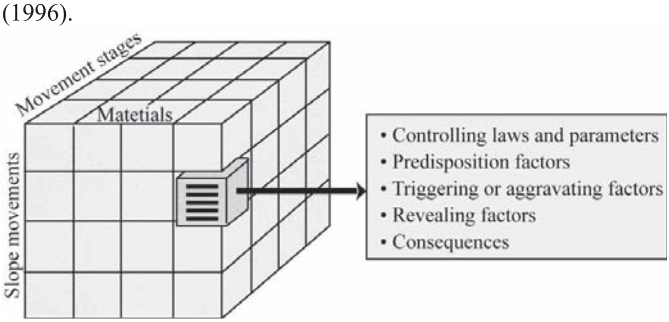

Fig. 1. Schematic slope movement characterization (from Leroueil et al., 1996)

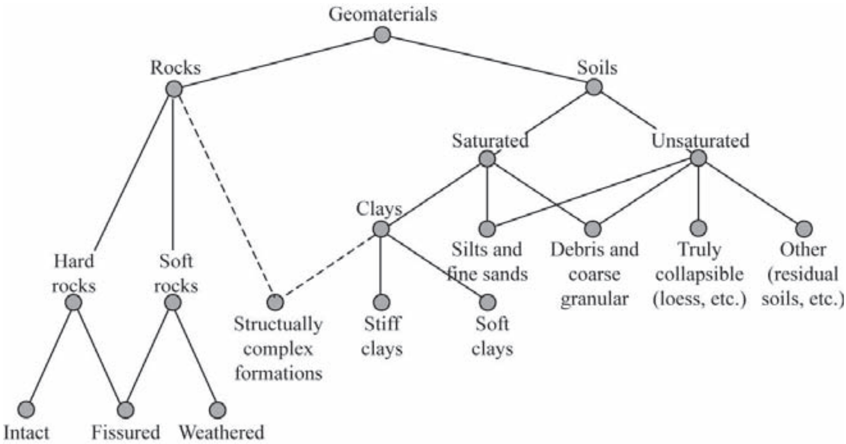

Fig. 2. Material types considered in the slope movement characterization (from Leroueil, 2001)

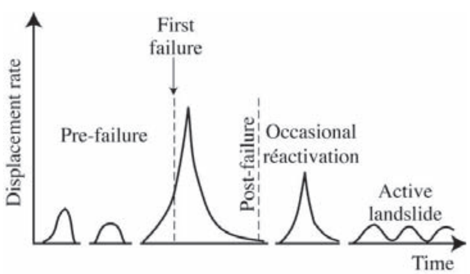

Fig. 3. Different stages of slope movements (from Leroueil et al., 1996)

For each relevant element of the characterization matrix, the information can be put in a characterization sheet (Fig. 1) with:

- The laws and parameters controlling the phenomenon; it could be the strength parameters at the failure and reactivation stages.
- The predisposition factors (in mechanical terms, they are essentially the initial conditions) that give information about the present situation and determine the slope response following the occurrence of a triggering factor (e.g. presence of a loose sand layer in a seismic area).
- The triggering factors that lead to failure, or aggravating factors that produce a significant modification of stability conditions or of the rate of movement. They can be temporary (e.g. heavy rainfall or rapid drawdown) or progressive (e.g. erosion or weathering), (see Table 1).
- The revealing factors that provide evidence of slope movement but generally do not participate to the process (e.g. presence of cracks).
- The possible, direct or indirect, consequences of the movement.

Table 1. Common triggering or aggravating factors (Leroueil, 2004)

	Terrestrial	Submarine
Increase in shear stress		
Erosion and excavation at the toe	x	x
Surcharging at the crest, sedimentation (and possible under-consolidation)	x	x
Rapid drawdown of water level adjacent to slopes	x	
Earthquake	x	x
Volcanic activity	x	x
Fall of rock	x	x
Decrease in strength		
Infiltration due to rainfall, snow melt, irrigation, water leakage from utilities	x	
Weathering	x	
Physico-chemical changes	x	
Gas hydrate dissociation		x
Pile driving	x	x
Fatigue due to cyclic loading and creep	x	x
Thawing of frozen soils	x	
Possible increase in shear stress and decrease in strength		
Vibrations and earthquake shaking that can generate excess pore pressures	x	x
Swinging of trees due to wind gusts	x	
Storm waves and changes in sea level		x

Another concept that helps in analyzing engineering problems and that is increasingly used is the concept of risk. Important works have been performed in recent years for its applicability to slope engineering, and specialty conferences on "Landslide Risk Assessment" and "Landslide Risk Management" were respectively held in Honolulu in 1997 and in Vancouver in 2005.

The total risk, R_T, is defined by Varnes et al. (1984) as the set of damages resulting from the occurrence of a phenomenon. It can be described as follows:

$$R_T = \Sigma H R_i V_i \qquad [1]$$

in which H is the hazard or the phenomenon occurrence probability; R_i (for i = 1 to n) are the elements at risk (persons, buildings, infrastructures, etc.) potentially damaged by the phenomenon; and V_i is the vulnerability of each element R_i, represented by a damage degree between 0 (no loss) and 1.0 (total loss). In the context of the geotechnical characterization of slope movements (Fig. 1), the elements at risk and their vulnerability should be, directly or indirectly, in "Movement consequences" whereas the hazard, probability that the triggering factor may reach a given value that would cause a failure, should be in "Triggering or aggravating factors". Also, in the context of slopes, the hazard has to be subdivided into two parts: the hazard associated with the possibility of having a failure, H_f; and the hazard associated with the possibility that the post-failure stage presents specific characteristics (e.g. debris reaching a given distance), $H_{post\text{-}f}$; $H = H_f \times H_{post\text{-}f}$.

RECENT DEVELOPMENTS

Important recent developments can be divided in three classes: technologies for surveying and monitoring slopes; achievements about the understanding of slope behavior; and the development of procedures for numerical modeling of slope behavior. The technologies for improving the safety of slopes could be added, but are not considered here.

Methods of survey and monitoring

Table 2 lists the main instruments and technologies that can be used for surveying and monitoring slopes.

Table 2. Main instruments and technologies for surveying and monitoring slopes

Displacements and deformations Crackmeters; extensometers Tiltmeters Inclinometers SAA (shape-accel-array) Photogrammetry GPS and dGPS Laser ranging or Electron. Dist. Meas. (EDM) InSAR (Interferometric Synth. Apert. Radar) Optic fibers	*Topography* Aerial photogrammetry LIDAR Multibeam echosounding (marine environ.) *Environmental conditions* Weather station Thermistors
Acoustic emission/micro seismic monitoring system (AE/MS)	*Pore water content and pressure* TDR (Time Domain Reflectometry) Piezometers Tensiometers Psychrometers

Some remarks can be made in relation with new systems:

- Very important developments have occurred in recent years in terms of technologies, possibilities to take, send and store readings, and interpretation methods. Most of the instruments can now be monitored automatically, almost continuously if necessary, and the data can be sent via wireless links to an interpretation and analysis center. Some of these developments are specified below.
- Most of the instruments or systems listed in Table 2 have been used on one of the sites indicated below and the reader can refer to the references for details.
- The information automatically obtained (rainfall intensity, pore pressures, rate of displacement, etc.) can be used for landslide triggering prediction and in early warning systems (Picarelli et al., 2009).
- Inclinometers: The most common procedure is to install an inclinometer casing and take readings at different times with a probe. It is however possible to leave one or several inclinometer segments in-place at selected depths and take regular readings.
- SAA (for Shape-Accelerometer-Array; see Rollins et al., 2009, and Cloutier et

al., 2011): It is a rope-like array of sensors and microprocessors that fits in a small casing. It is made of small segments that are connected by flexible joints and contain three accelerometers. SAA measures deformations (and also vibrations) and can be used vertically (as an in-place inclinometer) or horizontally (Cloutier et al., 2011).

- GPS and dGPS (for Differential Global Positioning System): The accuracy of dGPS is of a few millimeters.
- EDM (for Electronic Distance Measurement) or Laser ranging: The system uses laser beams and mirrors from a large distance to the target (possibly a few kilometers) and, according to the web site of Turtle Mountain (www.ags.gov.ab.ca/geohazards/turtle_mountain), the accuracy would be of one part per million, i.e. one millimeter over a distance of one kilometer.
- InSAR (for Interferometric Synthetic Aperture Radar) and DInSAR: This technology uses radar waves and can be ground-based or satellite-based (Singhroy and Molch, 2004; Couture et al., 2011). In the case of DInSAR, the accuracy is of a few millimeters. Interestingly, it is possible to obtain images back to the early 90s, and thus to establish movement history for the last 20 years or so (T.R.E., www.treuropa.com).
- Optic fibers: Originally used to measure strains in structural elements, the use of optical fibers for monitoring strains in geotechnical engineering is spreading (Shi et al., 2008; Picarelli and Zeni, 2009). Monitoring is performed by stimulated Brillouin scattering (SBS) which enables to obtain temperature and strain variations along a single-mode optical fiber through Brillouin shift measurements (Olivares et al., 2009). The advantage of optical fibers is twofold: the possibility to measure changes in temperature or strains everywhere along a fiber, even over long distances, and their extremely low cost.
- AE/SM (for Acoustic Emission and Micro Seismic Monitoring System): Used in rocks and soils to detect AE generated by inter-particle friction, fracture propagation and displacements along discontinuities (Dixon et al., 2003; Dixon and Spriggs, 2007; Eberhardt et al., 2004; Amitrano et al., 2010). However, as in most soils AE level is low and attenuation is high, Dixon et al. (2003) proposed to use waveguides (tubes in a preformed borehole with “noisy” material (sand or gravel) around the tube) in order to amplify the noises. Dixon and Spriggs (2007) show that the AE signal is linked to the deformation rate.
- LiDAR (for Light Detection and Ranging): Can be airborne or on the ground, and can be used on large areas. It penetrates vegetation and can provide a high-resolution topography of the ground surface that can be used to generate a 3-D terrain model. The accuracy is in the order of 0.15 m.
- Weather station: It may include the measurement of temperature, relative humidity, rainfall, wind speed and solar radiation. These parameters may allow the determination of evapo-transpiration potential.
- TDR (for Time Domain Reflectometry): It is generally used for determining the average water content around a probe. The experimental device consists of an electromagnetic pulse generator connected, through a coaxial cable, to a metallic probe a few decimeters long, which is buried in the soil. An

electromagnetic pulse is sent through the soil, and the reflected signal is acquired, providing the soil volumetric water content. Greco (2006) developed an interesting inverse procedure to retrieve the water content profile along the probe and not only an average value. Figure 4 shows an example from infiltration tests carried out on a model slope subjected to artificial infiltration until failure (Greco et al., 2010).

- Piezometers (for positive pore pressure measurements): The selection of the type of piezometer should be decided mostly on the basis of their time lag and the monitoring method (manual or remote).
- Tensiometers: Tensiometers and other probes such as psychrometers are used to monitor negative pore pressures in saturated and unsaturated soils, in particular granular soils subjected to precipitations. Associated with water content measurements, tensiometer readings can be used to define the in situ Water Retention Curve of the soil (Comegna et al., 2011).

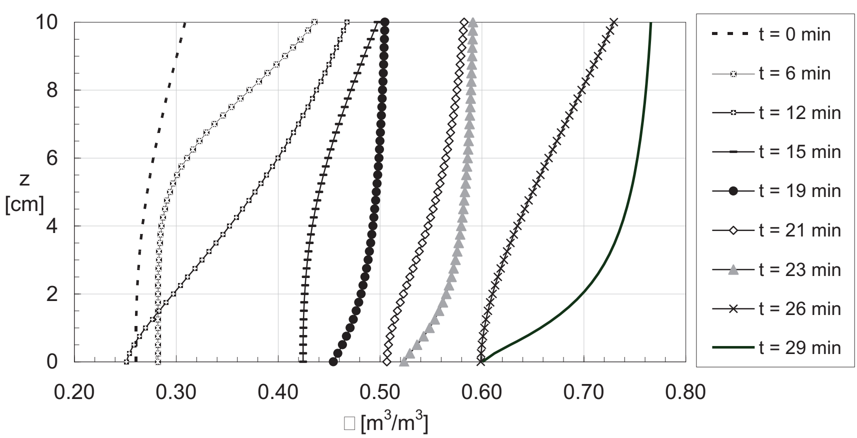

Fig. 4. Volumetric water content profiles obtained from TDR readings (modified after Greco et al., 2010).

The examples given below illustrate the use that can be made of monitoring systems. Cloutier et al. (2011) and Couture et al. (2011) present instrumentation and monitoring data at the Gascons rockslide, Quebec. The monitoring system includes crackmeters, extensometers, tiltmeters, piezometers, inclinometers and SAA system installed horizontally, with most of these instruments remotely monitored. Surface displacements are monitored by satellite-based InSAR technology, using a special technique (PTA-InSAR technique) to improve the accuracy. Figure 5 shows the site with examples of monitoring data. Other examples of very well instrumented sites in rock are the Turtle Mountain Monitoring Project and Field Laboratory in Alberta, Canada, (Froese and Moreno, 2007), the Randa site, Switzerland, and the Aknes site, Norway (see Eberhardt et al., 2008).

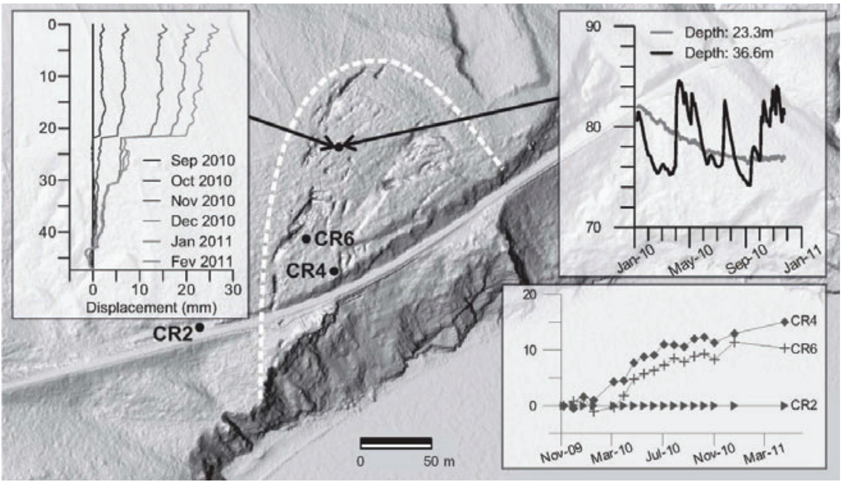

Fig. 5. The Gascons rockslide with displacement profiles obtained from SAA, pore pressures versus time, and displacements of corner reflectors obtained by InSAR technique versus time (modified after Cloutier et al. (2011) and Couture et al. (2011)).

Since 2002 a steep calcareous slope mantled by unsaturated pyroclastic soils subjected to a rapid killer flow-slide (1999) at Cervinara, Italy, has been monitored to investigate the slope-atmosphere interaction (effects of precipitations, and of air temperature and humidity on infiltration and evapo-transpiration, thus on water content and suction values in the soil). The monitoring system includes a weather station, a number of tensiometers and a number of TDRs, with some couples of TDRs and tensiometers at the same locations and depths in order to correlate values of the volumetric water content and suction. Figure 6 shows the layout of a part of the instrumentation, which is powered by solar panels for automatic data acquisition every 60 minutes, and some results (Comegna et al., 2011). The instrumentation allows the estimate at any time of the safety factor of the slope.

Understanding of slope behavior

Understanding the mechanisms of slope movement is essential for evaluating the stability conditions of slopes, the post-failure stage, the possible consequences of a failure, and selecting mitigation methods. So, these processes need to be investigated. The following outcome can be noted.

Infiltrations and evapo-transpiration. The pore water pressures and the physical state of soil (water content, degree of saturation, etc.) generally vary with alternating rainy and dry periods. The processes involved are complex but progresses have recently been made due to a better understanding of the hydraulic and mechanical behavior of unsaturated soils, and also to the development of numerical hydraulic and hydro-mechanical models. The analysis of infiltration in an unsaturated soil requires the soil water retention curve (relationship between degree of saturation and matric suction), the hydraulic conductivity versus suction relationship and, if the geomaterial is not assumed perfectly rigid, a compressibility law. Collins and Znidarcic (2004) examined the case of infiltration in an infinite slope consisting of 4 m of an unsaturated soil deposit over a drained boundary with zero suction. Figure 7 shows the water pressure head profiles at different times for a fine grain soil and a coarse grain soil characterized by typical permeability functions. Due to higher hydraulic conductivity in this latter case, infiltration progresses more rapidly. However, in both cases, it takes a significant amount of time before the water front reaches the bottom of the soil layer. Collins and Znidarcic (2004) also examined the stability of slopes, considering suction-dependent shear strength, in particular for defining the critical depth. Similar studies have been performed by other researchers (e.g. Ng and Shi, 1998; Cai and Ugai, 2004; Lu and Godt, 2008).

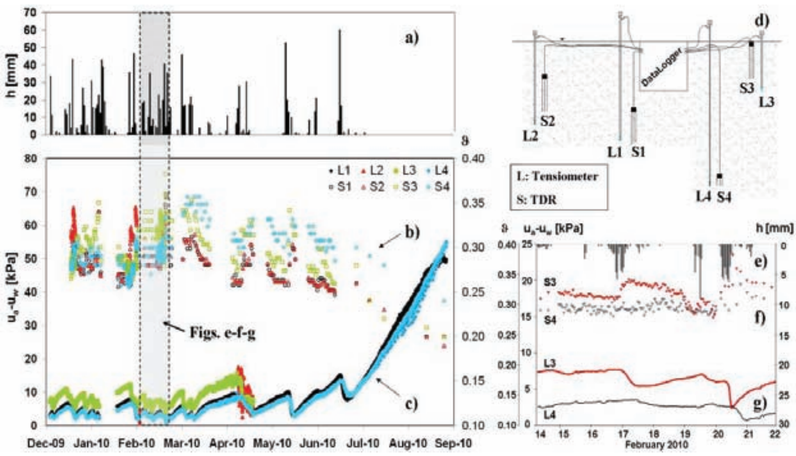

Fig. 6. Measurement of rainfall (a and e), suction (Tensiometers; c and g) and volumetric water content (TDRs; b and f) at Cervinara (modified after Comegna et al., 2011).

It is also worth noting that several natural slopes have been recently instrumented and monitored (Springman et al., 2003; Ochiai et al., 2004; Rahardjo et al., 2005; Papa et al., 2009; Damiano et al., 2012). These studies give useful information regarding the infiltration process, the influence of antecedent rainfall, the influence of heterogeneities, the percentage of rainfall contributing to infiltration, the evapo-transpiration, etc., providing interesting data sets for testing numerical models.

Pore pressure changes in soil deposits. Even in saturated conditions, because of the compressibility of soils, a change in pore water pressure at the boundaries is not reflected instantaneously to the entire soil deposit. Changes require a consolidation/swelling process, controlled by the coefficient of consolidation/swelling of the soil. This can be illustrated by observations made at Wabi Creek, Ontario, in a soft clay deposit (Kenney and Lau, 1984; Fig. 8). At some distance from the slope the annual variation of the water table level is in excess of 2 m, but the variation in pore pressure at depths larger than 10 m or so is less than 0.5 m; it corresponds to a coefficient of consolidation/swelling in the order of 10^{-6} m^2/s. Similar observations were made by Demers et al. (1999). According to Vaughan (1994), the seasonal fluctuations in pore pressure in stiff clays from UK only extend down to about 4 m. So, in most clayey deposits, pore pressures are continuously varying and are not hydrostatic (Leroueil, 2001). This also suggests that weather-induced landslides are generally shallow, and deep failures can be due to different causes or complex hydro-mechanical processes.

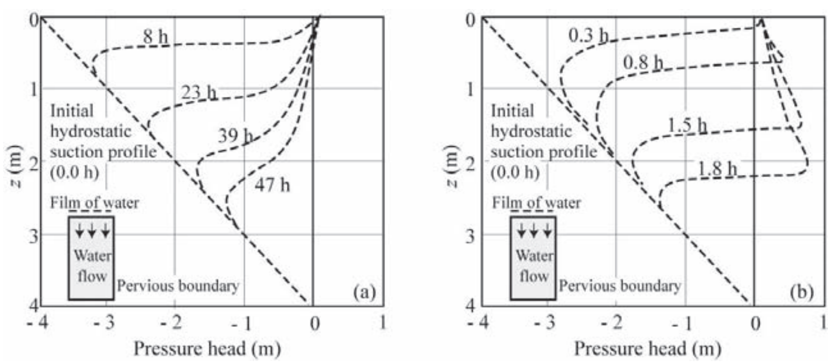

Fig. 7. Effect of vertical water infiltration in a column of fine grain soil (k_{sat} = 1.5 x 10^{-8} m/s) (a) and coarse grain soil (k_{sat} = 1.5 x 10^{-6} m/s) (b) (modified after Collins and Znidarcic, 2004).

Slope instability due to pore pressure increase. When a loose soil specimen of granular soil is subjected to an undrained loading, it reaches a peak on a line in a stress diagram (IL, between D and C on Fig. 9a) called Instability Line by Lade (1993), and then moves towards its critical state (C on Fig. 9a), on the critical state line (CSL) characterized by ϕ'_{cs}.

When a slope is subjected to pore pressure increase due to infiltration or rising water table, total stresses and shear stresses remain almost constant but effective stresses decrease. In a stress diagram such as the one in Fig. 9a, this corresponds to a stress path such as from I_{ls} towards Y_{ls}. If the stress path reaches the instability line at Y_{ls}, the soil has a tendency to move towards its critical state C. As the deviatoric stress at C is smaller than that due to gravity forces in the slope (q at I_{ls}), there will be static liquefaction of the soil and collapse of the slope. Major consequences of this phenomenon are that failure is triggered at an angle of strength mobilization smaller than ϕ'_{cs}, and that instability (at Y_{ls}) is followed by an increase in pore water pressure since p' decreases, with a significant part of the potential energy available at the onset of failure released into kinetic energy.

Chu et al. (2003) showed that the concept of instability (development of plastic deformations) applied to slopes can be extended to dense soils, with the instability line extending above the CSL (C towards Y_{ds} on Fig. 9a) and corresponding to a dilatant soil behavior. As shown by Wang and Sassa (2001), Damiano (2003) and more generally by Leroueil et al. (2009), flume tests and field evidences confirm this model for both loose and dense soils.

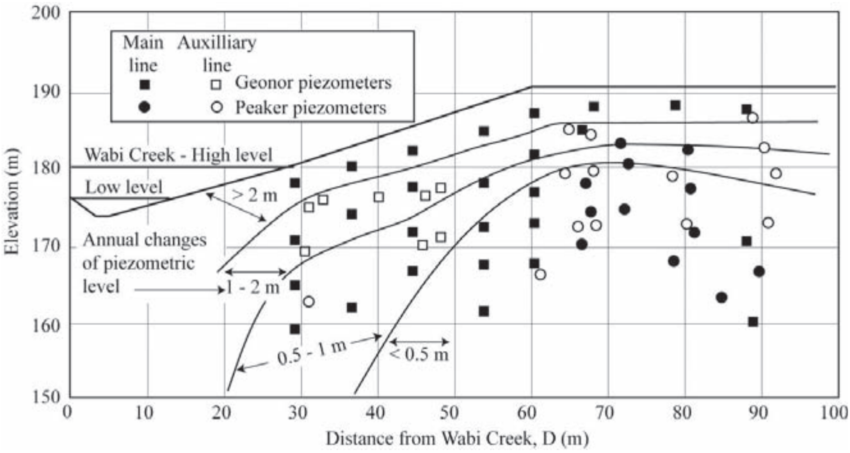

Fig. 8. Location of piezometers and annual changes of piezometric level at Wabi Creek (from Kenney and Lau, 1984).

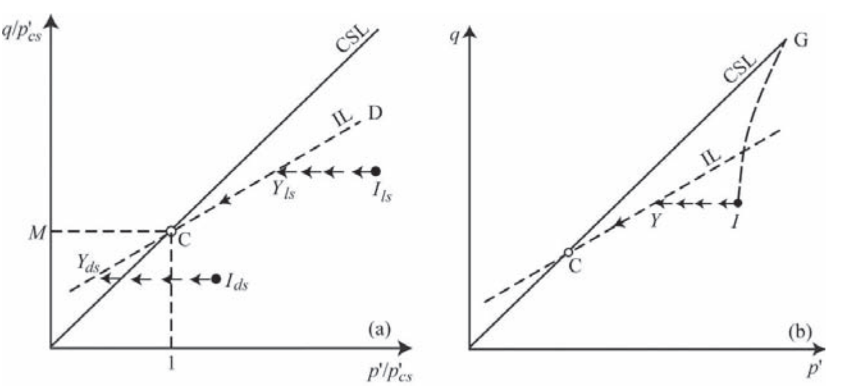

Fig. 9. (a) Stress path in sand followed at q = cst due to pore pressure increase (from Leroueil, 2004) and (b) Comparison between stress path in slope subjected to pore pressure increase and that implicitly assumed in stability analysis.

Progressive failure. If there is: (a) a brittle soil, i.e. a soil presenting a peak and a strain-softening behavior either in drained or undrained conditions; (b) non-uniformity in the distribution of shear stresses; (c) shear stresses that locally reach the peak strength of the soil; and (d) boundary conditions such that strains may develop, progressive failure will develop (see Urciuoli et al., 2007). As such conditions exist in many soils and slopes, progressive failure is quite common. Potts et al. (1997) studied the case of cut slopes in London clay considering $\phi'_p = 20°$ and $c'_p = 7$ kPa for the peak strength and $\phi'_r = 13°$ and $c'_r = 2$ kPa for residual conditions. These authors explained delayed failures in cut slopes and concluded that the average strength parameters at failure would be a friction angle of 18° and zero cohesion, which are parameters close to those back-calculated from first-time failures ($\phi' = 20°$ and $c' = 1$ kPa; Chandler, 1984). These average "mobilized" parameters are intermediate between peak and large deformation parameters which is typical of progressive failure.

Bernander (2000) indicated that a number of large landslides that occurred in Sweden were associated with a perturbation upslope and progressive failure, from upslope towards downslope. In eastern Canada on the other hand, local failure is generally initiated at the toe of the slope, generally by erosion; a failure surface then progresses horizontally over large distances into the soil mass and the soil above the failure surface adjusts to these new conditions, generally by forming horsts and grabens, typical of spreads. Limit equilibrium stability analysis methods performed either in drained or undrained analyses provide factors of safety in these spread landslides that are much higher than 1.0 and thus cannot be used. In fact, spreads are not well understood yet and are the object of researches by studying case histories in detail and performing numerical simulations (Locat et al., 2011; Locat, 2012).

It has also been shown that progressive failure may develop and explain failures in rock slopes (Eberhardt et al., 2004) and also in earth-dams, at Carsington, UK, (Potts et al., 1990) and at Aznalcollar, Spain, (Gens and Alonso, 2006). It is thus quite a common process, however generally ignored in practice.

Earthquake induced landslides.
The effects of earthquakes on the stability of slopes depend mostly on their magnitude, duration and frequency content, and on the main structural and mechanical characteristics of the rock/soil mass. Comprehensive reports on the effects of earthquakes have been extremely useful to develop our knowledge of the problem (e.g. Berz et al., 1980; D'Elia et al., 1985; Towhata et al., 2001; Ishihara, 2005; Lin et al., 2008; Wang et al., 2009; Locat, 2011). These studies have highlighted significant aspects, as the delay sometimes observed between the earthquake occurrence and the onset of slope failure (Seed et al., 1973; D'Elia et al., 1985; Seid-Karbasi and Byrne, 2007), the role of cyclic soil liquefaction in the triggering of catastrophic landslides, especially in saturated sands (e.g. Ishihara, 1993; Idriss and Boulanger, 2008)) and some complex post-cyclic lateral spreads in saturated clay (Fenelli et al., 1992; Olivares, 1997; Idriss and Boulanger, 2008). Also, performing two-dimensional dynamic response analyses, Ashford and Sitar (1994) showed that the peak acceleration computed at the crest of the slope is larger than the free-field value. Finally, field experience has been extensively used for defining conditions that can

trigger liquefaction (Youd et al., 2001) as well as the residual undrained shear strength of liquefied sands (Seed and Harder, 1990; Olson and Stark, 2002).

Studies have been performed about the effect on the stability of a slope of a low permeability soil layer in or above a liquefiable layer of sand. In case of liquefaction by shaking, the lower part of the liquefied sand layer densifies whereas the upper part, thus beneath the low permeability layer that acts as a barrier, expands. An extreme case of expansion is the formation of a water film immediately below the low permeability layer. As a result of this expansion, the shear strength decreases (to zero in the case of a water film) and there may be shear localization beneath the low permeability layer and slope failure. This has been observed experimentally in shaking tests (Kokusho, 1999) and in centrifuge tests (Kulasingam et al., 2004; Phillips and Coulter, 2005; Malvick et al., 2008), and numerically simulated (Seid-Karbasi and Byrne, 2007). Figure 10 shows a slope model of loose sand with a silt layer seen in lighter grey, in a centrifuge that was subjected to shaking (Malvick et al., 2008). Whereas the localization is clearly observed immediately beneath the silt layer about 120 s after shaking (Fig. 10c), it has not developed during shaking (Fig. 10b). This is explained by the fact that the redistribution of voids, and thus the change in strength, requires time. Because many natural soil deposits are stratified, it is thought that this process is the explanation for the failure of numerous submerged slopes, in particular for the failure of gentle slopes of a few percent. Details on the process and implications are given by the previously mentioned authors, in particular Kokusho (2003) and Malvick et al. (2008); it is emphasized in particular that conventional laboratory tests on undisturbed soil samples cannot reproduce this phenomenon and provide strength that cannot be representative of the strength of the soil beneath the low permeability layer.

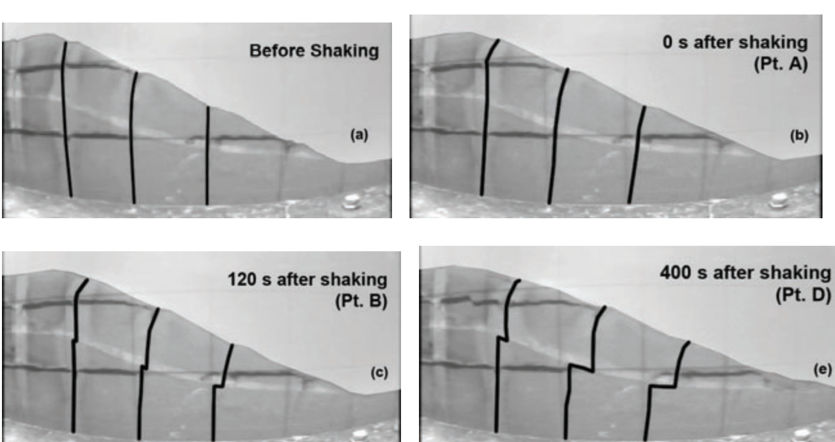

Fig. 10 Photographs of centrifuge model of Nevada sand with embedded silt arc (lighter layer) subjected to earthquake motion (modified from Malvick et al., 2008).

Numerical modeling

Numerical models for hydraulic or coupled hydro-mechanical problems have been developed for both saturated and unsaturated soil conditions; they constitute very powerful tools for studying a variety of problems and analyzing specific, sometimes complex case studies. Applications can be found in a variety of domains: infiltration in saturated and unsaturated conditions (e.g. Rahardjo et al., 2008); excavations in clayey deposits (e.g. Kovacevic et al., 2007); rapid drawdown (Pinyol et al., 2008 and 2011); slope subjected to earthquake (e.g. Byrne et al., 2006; Takahashi et al., 2008); flow-like landslides (e.g. McDougall and Hungr, 2004; McDougall et al., 2008). These developments have given a more detailed insight to a number of problems that were only coarsely understood. This can be illustrated by some examples.

Infiltration. The development of advanced theories and models about the behavior of unsaturated soils have favored the development of modern codes, working at slope and regional scales, for the analysis of the soil response to infiltration. SEEP (Geoslope, 2004) and CODEBRIGHT (Saaltink, 2005) are examples of codes working at slope scale that can relate the stability of slopes to precipitation via infiltration analysis. Codes conceived to perform analyses at regional scale start from a terrain model (e.g. TRIGRS by Baum et al., 2008). Even though losing the possibility to perform sophisticated infiltration analyses, they give the possibility to setting up reliable early warning systems in areas subjected to risk of rainfall-induced landslides and to produce in real time dynamic risk mapping by coupling infiltration analysis with stability analysis. Along this way, researchers of the Seconda Università di Napoli developed the so called I-MODE 3D Finite Volume Code (Olivares and Tommasi, 2008). It is used as a basic component of a modeling chain which includes the COSMO-LM code for weather forecasting. The geotechnical I-MODE 3D calculates the effects of precipitation, i.e. the increases in water content and decreases in suction or positive pore pressure, and performs stability analyses under the hypothesis of an infinite slope in unsaturated soils, using an extension of the Mohr-Coulomb failure criterion to unsaturated soils (Fredlund and Rahardjo, 1993). For the analysis of the hydraulic effects, the Water Retention Curve of the soil based on the Van Genuchten expression (1980) and the hydraulic conductivity based on the Brooks and Corey expression (1964) are used. Some applications are reported by Olivares and Tommasi (2008) and by Damiano and Olivares (2010).

Temporary cut in clay. Kovacevic et al. (2007) considered the process of pore pressure equilibration and progressive failure for minimizing the volume of an open-cut excavation in London clay for the below-ground construction of the London Heathrow Airport's Terminal 5. This excavation had a depth of 20 m and had to remain open for a period of up to 6 months. This was an uncommon problem that was complicated by the presence of horizontal tectonic shear surfaces at depths of 13 to 15 m, and could hardly have been examined without a sophisticated numerical model and some calibration. The authors used the numerical model ICFEP used by Potts et al. (1997) and similar soil parameters. The model and the input parameters were calibrated against the failure of 2 cuts that occurred a few kilometers away before being applied to the Heathrow Airport's Terminal 5 case. Finally, it was decided to accept a design that was giving a calculated deep-seated failure in 1.42 years.

Rapid drawdown. Water bodies may apply pressure on totally or partially submerged slopes and, if the level of water rapidly decreases (rapid drawdown), there are two effects: a reduction of the stabilizing pressure on the slope itself and a change in internal pore water pressures. As a consequence, several failures of slopes and of earth dams have occurred. In most cases, the problem is neither fully drained nor fully undrained, and rapid drawdown is accompanied and followed by progressive swelling of the slope soil and change in pore water pressure that are controlled by the coefficient of swelling/consolidation of the soil. Pinyol et al. (2008 and 2011) examined this problem, using the coupled hydro-mechanical model CODEBRIGHT. Pinyol et al. (2008), in particular, simulated the behavior of the Glen Shira Dam that was subjected to a drawdown of about 9 m in 4 days (Fig. 11). Figures 11b and 11c show the measured and calculated piezometric heads at the location of the piezometers. The agreement is quite good. Figure 11b shows in particular that in Piezometer 5, at an approximate elevation of 11 m, there is still a positive pore water pressure of about 10 kPa when the reservoir has been lowered from its initial elevation on day 9 to elevation 5.2 m at the end of drawdown, on day 13.

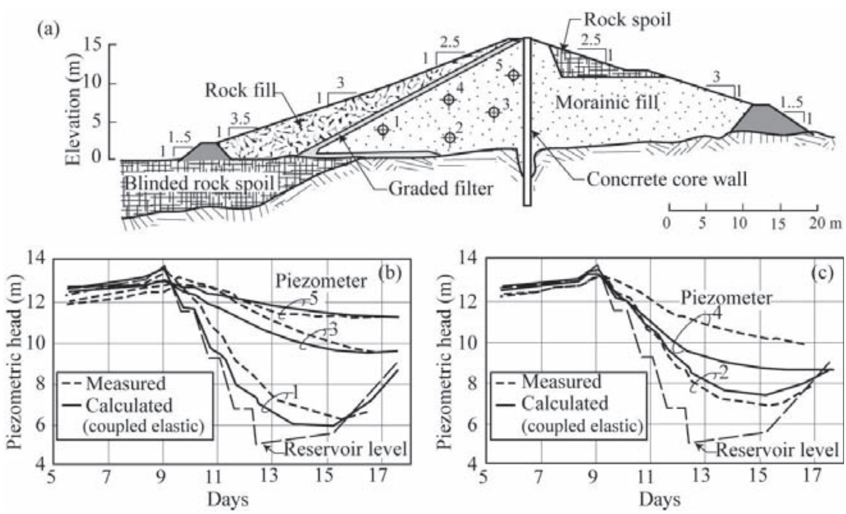

Fig. 11. Cross section of Shira dam subjected to rapid drawdown with the location of the piezometers 1 to 5 (a); comparison between measured and calculated piezometric heads (b and c) (modified after Pinyol et al., 2008).

Flow liquefaction. Several fully coupled hydro-mechanical models have also been developed for predicting the response of earth structures to earthquakes. It is the case of the model UBCSAND, run in FLAC, (Byrne et al., 2004) that is expressed in effective stresses and associates strength reduction to pore pressure increase.

Naesgaard et al. (2006) and Byrne et al. (2006), in particular emphasized the importance of water flow and pore water redistribution within the soil mass, and satisfactorily simulated both in space and time the failure of the Lower San Fernando dam that occurred in 1971 some 20 to 30 s after earthquake shaking (Fig. 12).

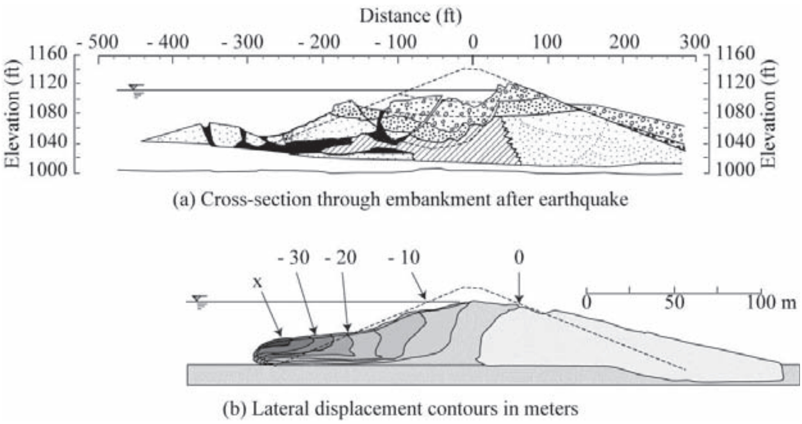

Fig. 12. (a) Cross-section of the Lower San Fernando dam after failure (after Seed et al., 1973) and (b) Simulated cross-section of the same dam after shaking (from Naesgaard et al., 2006).

As shown above by the presented examples, a large number of geotechnical problems are coupled, and the new generation of numerical models allows their study. It is also important to note that the combination of monitoring and numerical models provides important means to verify the hypotheses on the mechanisms involved and the representativeness of the model, and to calibrate the input parameters.

ASSESSMENT OF SLOPE STABILITY IN PRACTICE

The assessment of slope stability is difficult and requires an experienced geotechnical engineer. The approach suggested here is not new as such but reflects the way the authors and some of their colleagues follow or at least try to follow. It is summarized in the flow chart below (Fig. 13). The main stages are: preliminary office work; site visit; qualitative stability assessment; investigation; quantitative stability assessment; post-failure characterization; and management options. Fell et al. (2000) present a series of questions to be addressed in slope stability investigations and the reader is encouraged to refer to that work.

Preliminary office work

Slope stability assessment generally starts with a preliminary office work. At that stage, we are looking for:

- A general idea of the geology of the site as well as geological and

geomorphological features.
- The geometry of the slope (height, average inclination and profile) and possible changes in geometry due to human activity and/or erosion.
- The geomaterials involved.
- Surface waters (ponding, drainage, etc.).
- Preliminary regional and local hydrogeological models.
- Types of landslides that have occurred or may occur at the site and in the vicinity, and, if information is available, their characteristics (type and stage of movement; geometry; retrogression; runout distance).
- Hypotheses on possible causes of movements.
- Indicators of evidence of movement, evidence of previous landslides, evidence of erosion at the toe, presence of cracks, vegetation, presence of debris, etc.
- Local climatic and seismic conditions.

Documents that can be considered:
- If available, zonation maps.
- Topographic maps and LiDAR surveys.
- Historical data: witness reports; archives; newspapers; etc.
- Aerial photographs of the site and of its vicinity at different times for stereographic observations. Their comparison may show the evolution of the slope (erosion at the toe, surcharging at the top; changes in vegetation; etc.).
- Geological and geomorphological maps.
- Seismicity maps.
- Climatic conditions (rainfalls in particular).
- Previous investigations in the area (existing borings and soundings; laboratory and in situ tests; pore pressure observations in pits, tubes or piezometers; hydrogeological modeling; previous slope stability assessments).

The preliminary Office Work should be completed by a sub-report that should contain:
- The results of the preliminary analysis of the available information, in particular, the presupposed involved geomaterials, type of movement, predisposition factors, and possible triggering or aggravating factors.
- The additional information and parameters that may be required, and the type of investigation that should be performed to complete the information (boreholes, piezometers, inclinometers, etc.)
- The elements that should be specifically examined during the site visit. A site visit checklist could be used for that purpose.

Site visit

The preliminary office study being completed, a site visit is mandatory. There are a number of aspects to look for or to examine:
- Location of the site in its geomorphological and climatic environment.
- Geomorphological features of the site.
- Recent history of the site, including changes in surface drainage and geometry. Discussions with owners and neighbors are often helpful.

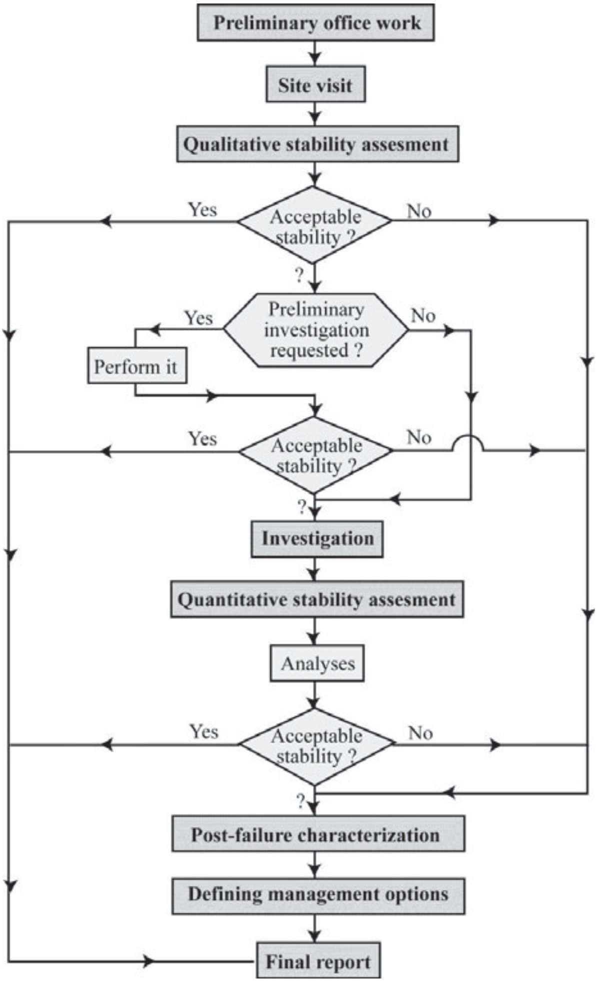

Fig. 13 Flow chart for slope stability assessment

- Evidences of geological processes such as landslides or erosion at the site and in the vicinity. The characteristics of the landslides in the vicinity of the slope (type, retrogression and runout distance of the debris) must be documented. The characteristics of the erosion, if any, (location, abruptness and height of the eroded zone) must also be documented.
- Geomaterials constituting the slope, which can be observed in eroded zones or along cuttings or scarps of previous landslides.
- Evidences of precarious stability and movements: open cracks, fissures and shears, possibly with vertical displacements; evidences of displacements;

bulging of the slope; heaving near the toe; evidences of deformation of man-made works (fissures and cracks in walls, road pavements, buildings; opening of joints in walls; tilting of walls or structures; etc.).
- Type of vegetation. If there are trees, describe the type, the approximate age (possibility to use dendrochronology), and the verticality.
- Resurgence or evidence of wet areas on the slope.
- Geomaterial at the toe of the slope, possibly in a river, which can indicate draining conditions in the slope.
- Access to the site for in situ investigation.
- Locations for possible soundings, boreholes and instruments such as piezometers, inclinometers, etc.
- Identification of potential problems that could occur during the study.

The site visit must be documented in a preliminary sub-report.

Qualitative stability assessment

Many slopes have remained stable for centuries or for thousands of years, and have already experienced extremely severe groundwater conditions. The probability of deep failure in the future is thus extremely low. This has led the government of the Province of Quebec to adopt an approach for the assessment of slope stability in sensitive clays in which a qualitative assessment of stability is part of the methodology (Lefebvre et al., 2008). This may lead to consider as stable slopes that possibly have a factor of safety smaller than the value generally required (e.g. F > 1.5), but it is thought that this approach based on performance is as reliable as the usual approach based on the calculation of a factor of safety (Lefebvre et al., 2008). It is thought that such qualitative assessment could be used in other geological environments. The conditions for considering a slope stable in the future are expressed hereunder in terms that are more general than those expressed by Lefebvre et al. (2008):

- The slope must be in a homogeneous geological and geomorphological context, thus with similar slopes in the vicinity.
- The slope should have had the same geometry for a long period of time (centuries), which means that it should not have been submitted to erosion and thus to a progressive decrease of its factor of safety. If the geometry has been recently modified by human activities, the changes must have a positive effect on stability; if the effect on stability is negative (e.g. excavation at the toe or loading at the top), even if minor, the slope cannot be a priori considered stable.
- The vegetation must not have changed for a long period of time. The presence of mature forest is a good indicator of stability; if the slope has been deforested, it generally has a negative effect on stability and the slope cannot a priori be considered stable.
- The slope geometry (height and slope angle) must be less critical than the neighboring slopes (and thus gives some stability reserve) or known from local experience to be generally stable.
- There should be absence of previous landslides in the neighboring and similar slopes, indicating that the area is not prone to landslides.

If such criteria are fulfilled (at least in the Quebec context), the slope is considered stable without any calculation of factor of safety. If there is uncertainty on the conclusion, a preliminary investigation may be required for verifying that the local soil conditions are not different from those assumed (e.g. by drilling boreholes or performing piezocone tests) or that there is no excessive pore pressures under the toe of the slope. Also, monitoring may help in confirming the stability of the considered slope or may be used in an observational approach.

This approach should be used with caution (or may not be used) for shallow landslides, for several reasons: (1) due to climate changes, slopes may be influenced differently from what has existed in the past. In other words, the principle put forward by Hutchinson (1995) that "past and present are keys for the future" may not be always true; (2) there are areas in the world, in particular in tropical regions, where weathering is active and may rapidly change the soil properties with time.

The qualitative stability assessment should also be summarized in a preliminary sub-report.

Investigation and instrumentation

The investigation must aim at defining the characteristics of the slope (in particular the potential predisposition factors), the characteristics of the geomaterials involved and the hydrogeological conditions. It may also help, with some instrumentation, at specifying the characteristics of the movement (e.g. existence of a pre-existing failure surface; rate of movement) and the variations of pore water conditions.

The in situ investigation may consist of:

- Boreholes, trenches and pits (for observation and recovering of soil samples) and soundings (piezocones and others);
- Geophysics: seismic P and S waves and resistivity for sub-aerial investigations; multibeam echo-sounding and seismic reflection for offshore investigations (Locat and Lee, 2002, 2009).
- Hydrogeology characterization: water levels in pits; piezometers; tensiometers and psychrometers. The installation of these instruments, their reliability and their monitoring have to be thought: number and location; possibility to have artesian pressures; except at shallow depths, simple tubes should not be used for pore water pressure measurements as the observations of water head can be misleading, and only isolated piezometers should be used; choice of the best instruments considering partial saturation, time lag, frequency of measurements and type (manual or remote) of readings; duration of the monitoring period. The hydraulic conductivity of the different soil units may also have to be estimated or measured.
- Weather station that may have to be installed on the site or may exist at small distance.
- Installation of instruments for monitoring.

In soils not containing gravels or coarser elements and in relatively soft soils, the piezocone is a very useful tool as it provides a detailed stratigraphy of the deposit as well as pore water pressure data after dissipation tests. In slopes in clay, the piezocone may also show evidences of destructuration, and thus indications of precarious stability (Demers et al., 1999; Leroueil, 2001).

The laboratory investigation aims at determining physical characteristics (unit weight; degree of saturation; grain size distribution; water retention curve; etc), mechanical properties (strength parameters in particular) and hydraulic properties (saturated hydraulic conductivity; hydraulic conductivity versus degree of saturation for unsaturated soils; etc.). Some remarks can be made: it is difficult to obtain undisturbed characteristics of soils that are heterogeneous or fissured; the size of specimens has to be established as a function of grain size and intensity of fissuring; most mechanical parameters are not "intrinsic" but depend on many factors (degree of disturbance of the specimens; size of the specimen; type of test; stress conditions; rate of testing; etc.), and this has to be thought before selecting testing conditions and the parameters for stability analyses. In some cases, only in situ tests (as direct shear tests or permeability tests in boreholes) can be performed to measure representative soil properties.

At this stage, instrumentation aims mostly at specifying the potential type of movement and its characteristics (e.g. inclinometers for specifying the stage of movement, pre-failure movement or reactivation, and, in this latter case, the location of the failure surface), the rate of movement and hydrogeological conditions.

If landslides have occurred in the vicinity in similar geological conditions, it may be interesting to perform back analyses in order to define strength parameters (without forgetting that they are associated with hypotheses on stratigraphy and pore pressures) and compare them with those deduced from laboratory tests. In the case of soils that cannot be easily sampled, back analyses may become the only way for assessing the mobilized shear strength parameters.

The investigation must be associated with an evaluation of its quality. In particular, the coherence of the data has to be checked: coherence of the different test results; coherence between soil stratigraphy and properties; coherence between stratigraphy and hydrogeological conditions. In relation with this latter point, it is important to mention that errors on pore pressure measurements are common. Also, when possible, it may be interesting and useful to try to correlate measured parameters such as weather conditions, pore pressures and rate of movement.

For the quantitative stability assessment discussed below, it is important to think that the worse conditions, in particular in terms of pore pressures, have to be considered.

A preliminary sub-report on the investigation performed and the results should be prepared at that stage. It should present the geological and geotechnical model of the slope with the geometry and the stratigraphy of the slope, and the hydraulic and mechanical characteristics of the different soil units.

Quantitative stability assessment

The quantitative stability assessment concerns first-time failures or reactivation, the post-failure stage being considered only if there is possibility of failure. It must be performed for the worse predictable conditions in terms of pore pressure conditions, changes in geometry due to erosion or human activity, earthquake activity, possibility of rapid drawdown, etc.

Traditionally, this assessment has been established by the determination of a factor of safety in 2-D limit equilibrium analyses. In addition to the geometry of the slope and the stratigraphy, these analyses require the unit weight and the strength parameters of the different soil units. They also require the hydrogeological regime or pore pressure distribution in the slope for the worse estimated conditions. Except for very simple cases where the pore pressure distribution can be easily estimated, it is recommended to use a hydrogeological model to define this distribution, remembering that in natural slopes pore pressures are not hydrostatic. Also, if there are variations of pore pressures at the boundaries, the variations at depth are not the same as at the boundaries (see Fig. 8). For detailed analysis of such cases, coupled hydro-mechanical models can be the only way to get reliable results.

For the selection of the strength parameters, when possible and for the case of brittle soils, the best way is from back-analyses of past landslides; otherwise, the engineer has to use his experience and judgment. In fact, if for ductile geomaterials, the strength parameters measured in the laboratory are generally representative of in situ behavior, it is not the case for brittle geomaterials where, mostly due to progressive failure and strain rate effects, the peak strength measured in the laboratory on good quality samples generally overestimates the strength mobilized in situ.

The required factor of safety for “stable conditions” depends on the country and/or on the author, but is most of the time equal to 1.5 or 1.4. This factor of safety covers to a large extent uncertainty on geometry, stratigraphy, strength parameters, pore pressure conditions, soil behavior and calculation method. However, if it is considered that the stability of a slope has to be improved, an increase in the existing factor of safety by 20 to 25% is generally considered sufficient. Fell et al. (2000) mention that for large landslides, an increase in F of 0.05 to 0.1 has been considered adequate.

It is worth noting that several countries are now using limit state methods of analysis based on partial factors applied on soil parameters and actions. This is in particular the case in Europe with the Eurocode 7 (EN 1997) mandatory since March 2010.

2-D Limit equilibrium methods of analysis present several shortcomings:

- They assume that the soil above the failure surface behaves as a rigid mass without any consideration for its stress-strain behavior and that the factor of safety is the same all along the failure surface. This is generally not the case.
- Most failure surfaces are three dimensional and several authors have examined the effect of the third dimension on the factor of safety (e.g. Skempton, 1985, Leshchinsky and Huang, 1992, and Morgenstern, 1992). According to Skempton (1985), it would approximately increase the factor of safety calculated in 2-D analyses by $(1 + KD/B)$, where D and B are the average depth and width of the sliding mass and K is an earth pressure coefficient. However, as 3-D analyses are more complex and less conservative than 2-D analyses, their use is not spread in the profession.
- As mentioned by Tavenas et al. (1980), when the analyses are performed in effective stresses, the definition of the factor of safety implicitly assumes a specific stress path that may be quite different from the stress path really followed. In a stress diagram such as the one shown in Figure 9b, the implicitly assumed stress path is approximately as IG whereas the stress path followed when failure is reached by pore pressure increase may be as IYC, thus quite different.

- There is a priori no consideration for the consequences of failure; in particular, the characteristics of the post-failure stage (velocity and runout distance) are not considered.

In spite of these limitations, limit equilibrium analyses have been very useful to the profession and are still widely used. It has to be remembered however that in case of soils with a strain softening behavior, the "average mobilized strength" is lower than the peak strength measured in the laboratory on good quality samples. In most geological environments, analysis of the stability of slopes by limit equilibrium method has thus to be seen as a semi-empirical approach, and, when possible, the back analysis of past failures in the same geological context can be used to specify strength parameters.

Factor of safety and probability of failure are often thought to be related: if F is close to 1.0, the probability of failure is considered very high; if it is about 1.5, the probability of failure is thought to be extremely small. However, as shown by several researchers (Christian et al., 1994; Duncan, 2000; Nadim et al., 2005), there may be no direct correspondence between factor of safety and probability. There have also been attempts for using concepts of probability and risk (Eq. 1) in the last decades; in particular, a specialty Conference on Landslide Risk Management was held at Vancouver in 2005. Picarelli et al. (2005) notice the difficulty in quantifying hazard and that a good understanding of the processes involved is an essential step to any quantification of hazard. There are several reasons that can explain the difficulties in quantifying hazard: the variability of soil units and soil characteristics; the uncertainty on the worse conditions to be considered; the uncertainty on the terrain model; and the uncertainty on the considered mechanical soil model. Nadim et al. (2005) indicate that probabilistic analyses can be seen as a complement to the conventional deterministic safety factor.

There are possibilities to consider the risk in deterministic approaches through the selected required factor of safety. Smaller values could be accepted for events with small consequences or small probability of occurrence. Along these lines: in 1984 in Hong Kong (GEO, 1984), the recommended F value was increasing with the potential loss (economic and life); in Sweden, Sällfors et al. (1996) indicate required factors of safety that decrease when the use of land changes from "New development" (high consequences) to "Undeveloped land" (low consequences), and when the quality of the investigation increases (lower uncertainty).

Factors such as historic information, geomorphological information and climatic-landslide activity relationships should also be considered for assessing the stability of slopes (Leroueil & Locat, 1998). Monitoring data, possibly in combination with numerical analyses, may also play an important role in the assessment of slope stability by specifying the slope conditions. Because of all these aspects that deserve consideration, the authors do not think that a single and fixed value of requested factor of safety should be considered for assessing the stability of slopes.

A way to overcome most of the limitations of limit equilibrium methods is to use finite element/finite difference methods, or possibly discrete element methods for some cases of rock slopes, which have been developed and used in the context of slopes (Potts et al., 1997; Griffiths and Lane, 1999; Eberhardt et al., 2004; Byrne et al., 2006). These methods present a number of advantages: they do not require assumptions on the shape and location of the failure surface; through the constitutive equations (that may be valid in saturated and unsaturated conditions), they provide information on deformation; they may allow the development of progressive failure; if there is hydro-mechanical coupling, they also allow pore water and pore pressure redistribution within the soil mass; they give detailed information on the processes involved and thus a better understanding of slope behavior. They also give the possibility to assess slopes in terms of their behavior (mostly acceptable deformations) rather than in terms of an obscure factor of safety. If their use has mostly been limited to research and the analysis of a few complex cases, it is certain that they will be more and more often used in practice in the years to come.

Special case: Cuts and rapid drawdown. These are problems for which changes of effective stresses, and thus in factor of safety, depend on the consolidation/swelling process of soil mass and thus on time. Except when the process is fully drained or fully undrained, the most relevant approaches for analyzing the evolution of stability consist in using numerical coupled hydro-mechanical models. The works done by Kovacevic et al. (2007) and Pinyol et al. (2008 and 2011 (for the analysis of the Canelles landslide)) and mentioned above are good examples of applications. Other approaches have been developed and are summarized by Cornforth (2005) and Duncan and Wright (2005).

Special case: Seismic slope stability analyses. Earthquakes induce cyclic accelerations, mostly horizontal, and thus shear stresses into the soil mass. They result in dynamic forces but also possible increase in pore pressure, destructuration of the soil, and consequently reduction in the shear strength of the soil. Consequences may be landslides or large movements, possibly with soil liquefaction or development of permanent deformations, or reactivation of landslides. It is then necessary to verify the slope response to these conditions. It is however not easy as the processes involved are very complex and the state of practice is still in evolution. The reader may refer to Blake et al. (2002) and Power et al. (2006) for guidelines and additional references. Essentially, it is important to differentiate between soils that cannot lose significant strength due to earthquake loading (ductile soils) and those that can lose strength and liquefy (brittle soils, in drained or undrained conditions).

In the first category, the most common approach is to perform a pseudo-static limit equilibrium analysis in which a destabilizing force equal to the soil weight multiplied by a seismic coefficient is applied horizontally; if the calculated factor of safety is lower than one, soil displacements arising from shaking can be determined on the basis of Newmark (1965)'s approach that considers that the mass of soil moves when the yield acceleration is exceeded (see Blake et al., 2002, and Power et al., 2006).

For soils that may show a strain softening behavior or liquefy, several steps are usually followed: the first one is to check if the earthquake may trigger soil liquefaction (Youd et al., 2001; Idriss and Boulanger, 2008); if so, the second one consists in verifying if the post-liquefaction strength (Seed and Harder, 1990; Olson and Stark, 2002) may prevent a flow-slide. Displacements can be determined on the base of Newmark (1965)'s approach (see Youd et al., 2002; Idriss and Boulanger, 2008).

Another but more complex approach would be to use a dynamic fully coupled hydro-mechanical numerical model incorporating constitutive equations expressed in effective stresses that associate strength reduction to pore pressure increase (e.g. DYNAFLOW (Prevost, 2002); UBCSAND (Byrne et al., 2004; Naesgaard et al., 2006).

This important stage of quantitative slope stability assessment should be completed with the preparation of a preliminary sub-report.

Post-failure characterization

The post-failure stage has to be examined separately from the failure stage as it corresponds to a completely different process. At the time of failure, some potential energy (E_P) becomes available, and what happens then depends on how this energy is redistributed. Part of the potential energy will dissipate through friction (E_F); the rest will be dissipated in breaking up, disaggregating and remolding the soil (E_D) and for generating movement (kinetic energy, E_K). Over a time interval during the post-failure stage:

$$\Delta E_P + \Delta E_F + \Delta E_D + E_K = 0 \qquad [2]$$

Leroueil (2001) discusses the different components of Equation 2 and their practical implications. The disaggregating energy in chalk and rocks in general has been examined by Leroueil (2001) and Locat et al. (2006) respectively; the remolding energy of sensitive clays has been examined by Leroueil et al. (1996) and Locat et al. (2008). Equation 2 also implies that in ductile materials, the available potential energy is dissipated through friction and, consequently, kinetic energy and post-failure rates of movement are small. On the other hand, if the material shows a loss of shear strength after failure, part of the available potential energy goes into movement (E_K) and high velocities as well as long runout distances could be reached.

As at the failure stage, it is essential to understand the mechanical process to predict the post-failure stage. For example, to have a flow-slide in sensitive clays, it is necessary to generate enough potential energy during the first-time failure to remold the clay, and to have clay with a liquidity index larger than 1.0 so that it can flow once remolded (Tavenas, 1984). Olivares and Damiano (2007) examined the post-failure mechanisms of initially unsaturated shallow deposits of pyroclastic soils on steep slopes from the Campania Region, Italy. These deposits are susceptible to landslides due to rainwater infiltration. From field and laboratory observations, and the understanding of unsaturated soil mechanics, Olivares and Damiano (2007) came to the following conclusions: (1) in steep slopes with a slope angle near or slightly larger than the friction angle ϕ' of the soil, this latter is practically saturated at the onset of failure and flow-slide can develop; (2) in very steep slopes with a slope angle significantly larger than ϕ', the soil at the time of slope failure is far from complete saturation and the possibility of having a flow-slide is less likely. These kinds of information can be used in a qualitative assessment of the possibility of flow-slide.

Post-failure assessment is important as it is often the most destructive stage of the landslide since the materials involved reach their maximum velocity, and their largest displacement. In many cases where the debris stops close to the toe of the slope, the runout distance can be defined on the basis of local observations and experience. However, if a flow-like landslide is expected, it is much more complex as the process depends on the geomaterial involved and its physical characteristics, and on the topography and other characteristics (materials that can be entrained, vegetation, etc.) of the travel path. Numerous numerical models have been developed for flow-like landslides (DAN3D, SHWCIN, RASH3D, TITAN2D, DFEM, FLO-2D, etc.) and a review is made by McDougall et al. (2008). The main difficulty with these models is the selection of input parameters that can provide representative bulk behavior (travel distance, velocities, thickness of debris). Most of the time, these parameters cannot be measured in the laboratory and result from trial-and-error adjustments of the simulation of previous events. McDougall et al. (2008) also discuss these aspects. To get more information about these special problems, the reader can refer to the Proceedings of the "International Workshop on Occurrence and Mechanisms of Flow-like Landslides in Natural Slopes and Earthfills, Sorrento, Italy, 2003" and of the "International Forum on Landslide Disaster Management, Hong Kong, 2007"; the first meeting considered experiences from a variety of places in the world, while the second one included a special session where some benchmark cases were analyzed by different researchers, using a number of numerical codes.

Post-failure may include the triggering of a tsunami, as in some cases of rock falls or rockslides along Norwegian fjords (Lacasse et al., 2008) or of coastal or submarine landslides.

If the study of post-failure is relevant, this stage should also be the object of a preliminary sub-report.

Defining management options

For defining management options, it is essential to well understand the mechanical processes that could lead to failure or large movements, and to know the characteristics of the potential landslide (size, velocity, travel distance).

"Risk assessment and Management" is the topic of an invited lecture to this GeoCongress, by Suzanne Lacasse, and only a few elements are provided here concerning management options. However, it is thought that, even without fully considering the risk, the engineer who concludes of the precarious stability of a slope must suggest possible options for mitigating the risk (Eq. 1). The general options are:

- Avoid the problem and choose a different lay-out or site.
- Decrease the hazard by reducing the driving forces or increasing the resisting forces or doing both (see Table 1). Mitigation methods and their design are not examined in this Paper. The reader is referred to Cornforth (2005) and/or LCPC (1998) for the selection and design of remedial and prevention methods. The mitigation methods are often associated with some monitoring in order to verify the hypotheses made and the efficiency of the adopted system.
- Decrease the risk by building in risk areas passive works, as check dams, barriers or retention basins, in order to protect the elements at risk (Versace et al. 2009).
- In some cases, mitigation measures cannot be applied due to economic,

morphological or technological constraints. The consequences of a potential event and thus the associated risk can then be reduced by monitoring rainfalls, or pore water pressures, or rates of displacement, or acoustic emission, etc. (what Picarelli et al. (2011) describe as geo-indicators) to define time close to failure at which planned measures can be taken for the protection of people and goods or at which people can be evacuated. This time can be defined on the basis of experience (e.g. antecedent rainfall and rainfall intensity exceeding some values), calculated factor of safety (e.g. pore pressures exceeding given values), or imminence of failure as indicated by the acceleration of displacements, or excessive acoustic emission. This can be seen as an observational approach or as an early warning system (Fell et al., 2000; Nadim and Intrieri, 2011; Picarelli et al., 2011). In such approaches, numerical models can be used for helping predicting and interpreting the measurements of the geo-indicators.

As for the other stages of the analysis, the management options should also be the object of a preliminary sub-report.

Final report

Preparing the final report on slope stability assessment is extremely important as it implies the responsibility of the geotechnical engineer involved. It thus has to be done in a very rigorous manner. The outline could coarsely follow the different stages indicated in Figure 13 and described above, and the preliminary sub-reports suggested for these different stages could be used to form a first draft. In order not to forget apparent details that could then be of extreme importance, it is important to write these preliminary reports as the project progresses.

CONCLUSION

Assessing the stability of slopes is a very difficult task that requires an experienced engineer and a rigorous approach. For that purpose, the geotechnical characterization of slopes that classifies the information on slopes is a very useful tool.

The paper evidences the tremendous progresses that have been made in the domain of slopes: there are new technologies, with possibilities to take automatically readings and send them via wireless links to an interpretation center; there are also interesting progresses in our understanding of some aspects of soil and slope behaviors; there has been development of very powerful coupled hydro-mechanical numerical models that can consider saturated and unsaturated soils, progressive failure, etc., and allow the analysis of complex problems. All this constitutes a real revolution in our way to monitor and analyze slopes.

A methodology, not new as such, is proposed for the assessment of slope stability. Its main characteristics are as follows:

- It includes a qualitative stability assessment based on past performance and on comparison with slopes in the vicinity that have similar geomorphology.
- It recognizes the shortcomings of limit equilibrium analyses but considers that they have been extremely useful to the profession and will remain in use in

practice.

- It is thought that the required factor of safety should not have a fixed value, but a value that integrates the concepts of risk, increasing with the consequences of a potential failure, decreasing when the uncertainty decreases and when the understanding of the processes involved progresses; also, the value could be reduced if associated with an adequate monitoring and an observational approach. The authors however recognize the difficulties for establishing guidelines for this approach.
- Because the processes involved are completely different from those at the stages of failure and reactivation, and because its consequences may be major, it is considered that the post-failure stage should be examined systematically and separately.

The coupled hydro-mechanical numerical models overcome most of the shortcomings of the limit equilibrium methods and give the possibility to assess slopes in terms of their behavior (mostly deformations) rather than in terms of factor of safety. There is no doubt that they will be more and more often used in practice in the future.

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Geotechnical Site Characterization in the Year 2012 and Beyond

Paul W. Mayne[1], M. ASCE, Ph.D., P.E.

[1]Professor, Civil & Environmental Engineering, Georgia Institute of Technology, 790 Atlantic Drive, Atlanta, GA 30332-0355; paul.mayne@gatech.edu

ABSTRACT: Due to the inherent complexities of natural soil materials, a thorough geotechnical site investigation program requires an integrated testing program of geophysics, rotary drilled borings with sampling, in-situ measurements, and various laboratory test series in order to provide ample information for design and analysis. This is only possible on large projects or situations where critical structures receive sufficient funding. For routine site exploration, realistic limitations in cost and time would dictate that the seismic piezocone test (SCPTù) and seismic flat dilatometer test (SDMTà) with dissipation phases be adopted in practice since as many as five separate measurements are obtained in an economical, continuous, and expedient manner from a single sounding. The collection of multiple readings is necessary to provide adequate information on geomaterials for rational engineering evaluations.

INTRODUCTION

In the past precedence of geotechnical site exploration programs on small- to medium size projects, it could have been considered adequate to drill a few soil borings on the property with standard penetration testing (SPT) and split-spoon sampling taken at regularly-spaced 1.5-m (5-ft) depth intervals, which were later supplemented by a few laboratory tests on recovered thin-walled tube specimens. Yet, the majority of information collected for analysis and design in this traditional approach rests mostly on N-values. A quick glance at a wide variety and diverse selection of geomaterials is shown in Figure 1. It is quite evident that these soils and rocks exhibit a vast and variable range in grain components, constituencies, and mineralogies with particle sizes varying from microns to millimeters to meters and larger. The corresponding array of geotechnical parameters applicable to each of these geomaterials can be expected to show tremendous ranges in strength, stiffness, stress state, and flow properties. Consequently, while valuable as an index, the SPT-N value is undisputedly a single number (an integer, at that), and as such, insufficient in and by itself to provide enough data for the full and proper evaluation of soil materials.

In the year 2012, the geotechnical engineer has many options towards preparing an advisory report to the client that offers solutions to the successful construction works

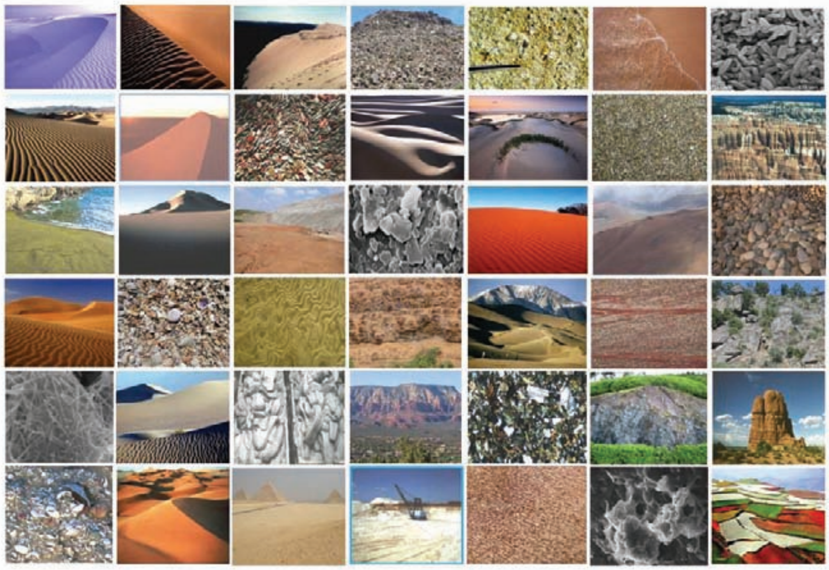


FIG. 1. Assorted geomaterials showing ranges of diversity and uniqueness

that involve foundation support, temporary & permanent excavations, short- & long-term stability, ground deformations, and seismic hazard concerns. For most civil engineering projects, a wide range of geotechnical solutions may be possible. For instance, the evaluation, comparison, and final choice of a suitable foundation system for an office building may include shallow footings, reinforced mat, driven pilings, bored piles, and geopiers, as well as over 40 different types of ground modification and site improvement. Certainly, the single N-value will not provide an adequate amount of information and input data for the engineering assessment of all of these possible solutions, even if the geoengineer has considerable experience, knowledge, and judgment in making her/his decisions.

Finding the solution(s) that offer the best value, least risk, most efficient, fastest construction time, and lowest cost should be a goal of the geotechnical engineer. As many methods are comparable in cost and performance, it will fall upon the designer to have as much understanding of the underlying ground conditions as possible, in order that the optimum solution is selected. Thus sets the stage for the necessary first step in geotechnics that always requires a site-specific subsurface exploration. In this mini-state-of-the-art (SOA) on geotechnical site exploration, a brief review is given on the various in-situ tests available for use (particularly the SCPTù and SDMTà), as well as, several upcoming new devices and procedures of merit.

CONSEQUENCES OF SUBSTANDARD SITE INVESTIGATION

A poorly-conducted and inadequate subsurface exploration program can have important and significant outcomes on the final constructed facilities, including possible overconservative solutions as well as unconservative designs. Some of the potential consequences may include:

- Excessively high construction costs and expenses due to unnecessary use of piled foundations or structural mats. In the case of truly-needed deep foundation systems, a substandard investigation may result in overdesigns with larger pile groups, diameters, and lengths than are warranted.
- Extra time and payments for implementation and conduct of unnecessary ground modification techniques.
- Unexpected poor performance of embankments, walls, excavations, and foundations, possibly including additional costs due to damage and/or retrofit and underpinning.
- Failure or instability during or after construction operations due to inadequate characterization of the geomaterials and/or missed anomalies, buried features, and/or weak layers and inclusions.
- Legal involvements, litigation, loss of professional reputation and/or license, and weakened credibility.

The upfront budgets for geotechnical site investigations should be sufficient to allow a reasonable amount of subsurface data to be procured and analyzed so that the design produces an efficient, safe, and economical solution.

TRADITIONAL SITE EXPLORATION

The tools of the trade and methods for subsurface investigation for geotechnical site characterization have primarily developed over the past century. A concise historical summary of the various field devices and test procedures is given by Broms & Flodin (1988). Moreover, as new technologies became available and accepted, the geotechnical profession placed a higher or lower reliance and dependency on each methodology, as depicted conceptually by Figure 2 (Lacasse 1988). Starting circa 1902 with the SPT, limited borings and auger cuttings with index testing provided the bulk of investigative data, coupled with a strong background in geology and engineering "judgment", thus serving towards design of a geotechnical solution. Over the next ten decades, the advent of a variety of in-situ tests, laboratory devices, geophysical methods, analytical modeling, numerical simulation, and probabilistic risk assessments have all emerged to play an important role in assisting the geoengineer towards an improved understanding on the material characteristics of the ground. Thus, while judgment is still warranted, more emphasis can now be placed on the utilization of direct-push probings with digital outputs, geophysical surveys with computer-enhanced imagery, trial searches for critical stability surfaces, numerical finite element simulations, and risk analysis with fuzzy logic.

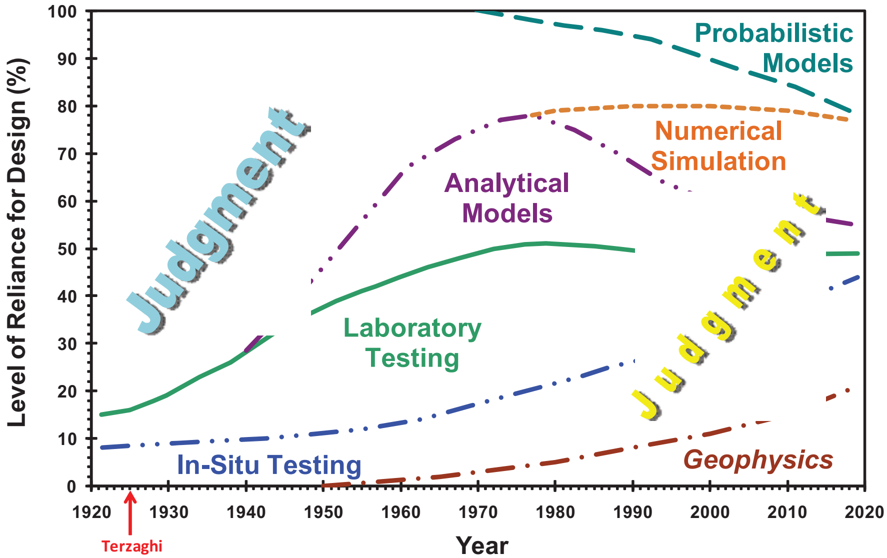


FIG. 2. Evolution of design-based processes for geotechnics (after Lacasse 1988)

Today, there are well over 150 different types of in-situ probes, field methods, and innovative gadgets available for purposes of geotechnical site investigation (Robertson 1986; Lunne et al. 1994). Figure 3 illustrates a number of the more well-known devices, many of these being rather narrowly focused on the quantification of a particular soil parameter or geomaterial property. The insertion type methods include an assortment of blades, probes, vanes, penetrometers, tubes, bars, plates, and/or cells that are either statically-pushed, dynamically-driven, drilled, torqued, twisted, inflated, vibrated, and/or sonically-advanced using hydraulics, pneumatics, rotation motion, electromechanics, or a combination thereof to collect data about the subsurface media. Some such devices include the cone penetration test (CPT), flat plate dilatometer test (DMT), borehole shear test (BST), and total stress cells (TSC). In addition, a number of nondestructive geophysical technologies, both invasive and noninvasive, have matured using either mechanical waves (compression, shear, Rayleigh, Love) and/or electromagnetic waves (resistivity, dielectric, electrical conductivity, permittivity) that can be deployed to ascertain very shallow, intermediate, and/or deep stratigraphic features, layering, and inclusions, as well as the small-strain elastic properties of the ground.

For a thorough investigation, a full suite of different field and laboratory tests must be conducted to ascertain the geostratigraphy, soil classification, site heterogeneity, and geotechnical engineering parameters. As an illustrative guide, Figure 4 shows one of the best available procedures for conduct of a detailed site exploration that include a series of soil borings to detail the stratigraphy, with in-situ strength testing by SPTs in

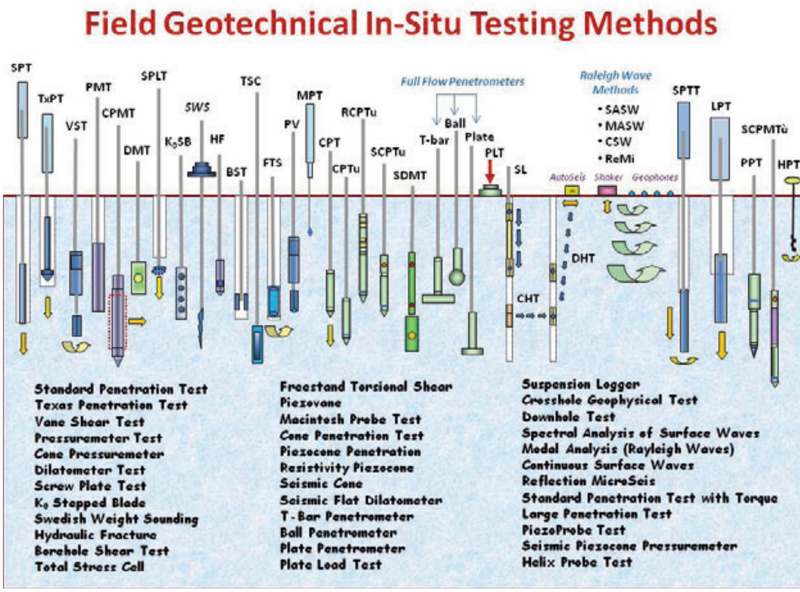


FIG. 3. Selection of in-situ devices and methods for ground investigations

sandy layers and field vane shear testing (VST) in clayey layers. The initial K_0 stress state and deformation modulus (E) can be evaluated using a series of pressuremeter tests (PMT), which also provide additional evaluations on the limit pressure (P_L) as well as the undrained shear strength (s_u) of clays and silts or effective friction angle (ϕ') in sands. The coefficient of permeability (k) of the ground can be assessed via field pumping tests, slug testing, or packer testing.

For small-strain measurements, crosshole tests (CHT) and/or downhole tests (DHT) provide means to map the profiles of compression wave (V_p) and shear wave (V_s) velocity with depth, although these may now be substituted with several noninvasive geophysical techniques that are readily available, including: spectral analysis of surface waves (SASW), refraction surveys (RfS), modal analysis of surface waves (MASW), continuous surface waves (CSW), and/or reflection microtremor (ReMi), the latter of which is a type of passive surface wave measurement.

In addition, with the collection of undisturbed thin-walled tube samples, a complementary laboratory program of index, grain size, hydrometer, triaxial (TX) shear, one-dimensional consolidation (CS), direct shear box (DSB) or direct simple shear (DSS), permeameter (PM), and resonant column (RC) or bender element (BE) testing can provide information about small specimens of the on-site materials and various soils strata. Undeniably, such an extensive site exploration program can only be afforded on large civil engineering projects such as interstate highway bridges and metropolitan water reservoirs or critical facilities including electrical power plants and

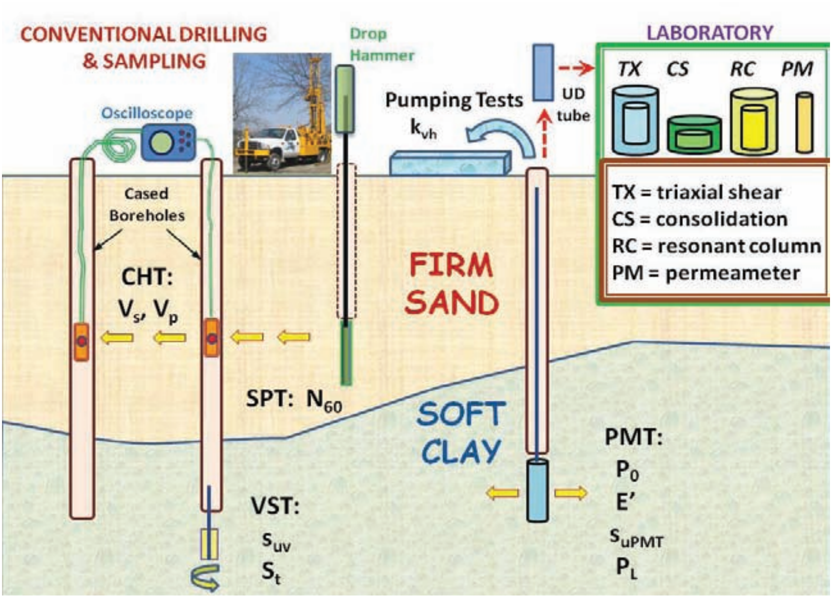


FIG 4. Current best practices for detailed geotechnical site exploration

nuclear facilities, due to high budgetary costs and lengthy durations for field deployment operations and laboratory test turnaround times. For routine projects of small- to medium-size concerns, a faster and less expensive alternative is needed, yet still able to provide adequate amounts and types of essential data for the analysis and design phase, as well as helping to avert legal issues which can arise because of reliance on too little or insufficient information.

INTERPRETATION OF IN-SITU TESTS

The results of in-situ tests are utilized to evaluate certain geotechnical parameters for input into analysis and design methodologies. The interpretations can be based on empirical or statistical relationships, analytical closed-form solutions, numerical simulations, and/or physical models, otherwise a combination of these approaches. The in-situ tests can be employed in a multi-stepped procedure, whereby the data are used to interpret soil engineering properties that are input into an engineering scheme or theoretical framework, or alternatively, used in a direct scaling to ascertain the necessary parameters. For instance, the use of CPT data can be applied to the evaluation of vertically-loaded pile foundations in a wide range of approaches, as depicted in Figure 5. Here, direct and/or indirect methods may be relied upon to calculate the side resistance, end-bearing, and total axial capacity in compression and tension, as well as the soil stiffness along the pile sides, beneath the pile tip, and top-

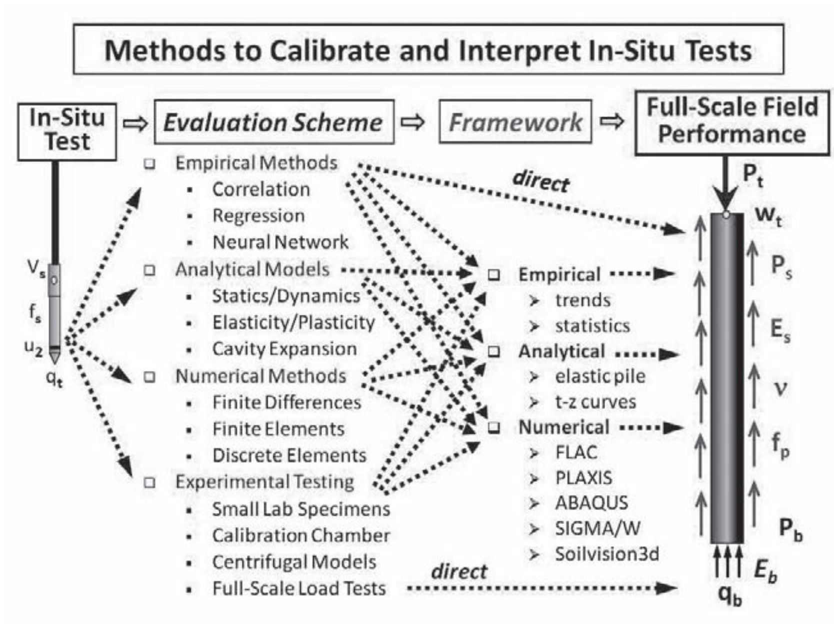


FIG. 5. Means of relating in-situ test results to full-scale performance

down load-displacement response and percentage of axial load transfer along the length of the pile.

From a practical standpoint, many in-situ tests have been calibrated against reference parametric values obtained from laboratory tests, primarily performed on specimens of natural clays cut from undisturbed tube samples or from reconstituted sand specimens. However, the issues of "sample disturbance" and "representative sample" must be raised for calibrations made in clay soils because of the wide assortment and quality of different tube samples (e.g., Shelby, piston, Laval, Sherbrook, Gus, JPN, ELE, etc.) as detailed by Tanaka (2000) and Lunne et al. (2006). This is furthermore complicated by the fact that certain parameters have multiple modes and thus non-unique benchmark values.

For clays, the notorious case is that concerning the evaluation of undrained shear strength (s_u = c_u), as this parameter exhibits a wide range (both theoretically and experimentally) depending upon which laboratory device is employed for the measurement (e.g., unconfined compression, triaxial shear, simple shear, fall cone, compression, extension, etc). Moreover, the undrained shear strength and stiffness are well-known to be influenced by strain rate effects, thus the faster the testing, the stiffer and stronger the clay appears (Randolph 2004; Peuchen & Mayne 2007). This too adds differences to the s_u values obtained from various modes of testing because of standardized testing rates (e.g., 1%/hour for DSS; 0.1°/sec for VST; 20 mm/s for CPT, etc.) which are not necessarily compatible. As a consequence, much confusion and uncertainty has arisen in the in-situ test evaluation of s_u profiles because of mixing of varied and inconsistent strength modes in their comparisons. As a solution to this dilemma, the author recommends calibrating the in-situ tests with the yield stress (σ_p'), or the effective vertical preconsolidation stress ($\sigma_p' \approx P_c' = \sigma_{vmax}'$), obtained from one-dimensional consolidation tests, because this parameter is uniquely defined (at least in concept). Then, the yield stress ratio (YSR = σ_p'/σ_{vo}'), or the more well-known overconsolidation ratio (OCR = P_c/σ_{vo}'), can be used to obtain the undrained shear strength via its relationship with normalized strength ratio (s_u/σ_{vo}'), as discussed elsewhere (e.g., Ladd & DeGroot 2003).

Reconstituted sand samples have been used in laboratory test programs to investigate the stress-strain-strength behavior of clean sands at different relative densities (D_R), effective confining stresses, drainage conditions (dry, saturated, undrained, drained, partially saturated), loading paths (e.g., triaxial compression, extension, plane strain, simple shear, direct shear), as well as other facets (e.g., Lade & Bopp, 2005). However, in the case of reconstituted sands, the dilemma and choice of which sample preparation method must be considered, since each of the various techniques (i.e., compaction, sedimentation, slurry, moist-tamping, vibration, static compression) can all result in significantly differing stress-strain-strength and stiffness behavior during shear and associated flow-permeability characteristics (Hoeg, et al. 2000; Vaid & Sivathayalan, 2000). Consequently, a number of incompatible and contradictory assessments have been made on the basis of mixed types of laboratory testing modes on sands, sample preparation techniques, stress paths, and other factors (e.g., large scale chamber boundary effects). These variables have considerable influence on calibrating in-situ test results, primarily in evaluating relative density, friction angle, and shear rigidity (Mayne et al. 2009).

GEOTECHNICAL EXPERIMENTATION SITES

Within the continental USA, six national geotechnical experimentation sites (NGES) have been established to provide a full range database that includes laboratory, geophysical, geotechnical, and full-scale prototype performance results for cross-referencing, comparative studies, and benchmark calibrations (Benoît and Lutenegger, 2000). Of particular value towards understanding soil behavior and the interpretation of test data, the NGES include test sites that are situated for research in sands (Texas A&M Site 1, TX; Treasure Island, CA), soft clay (Univ. Mass-Amherst, MA; Northwestern Univ., IL), sandy silts (Opelika, AL), and stiff clays (Univ. Houston, TX; Texas A&M Site 2; TX). For illustration, data from 5 different field tests obtained in the sandy soil layers at the Texas A&M site 1 are presented in side-by-side plots in Figure 6. Additional information and details concerning the characteristics and response of the sand can be found in Briaud (2007).

Recent symposia held in Singapore produced four full volumes on summary data from 66 international geotechnical experimentation sites (IGES) on the theme entitled: *Characterization and Engineering Properties of Natural Soils* (Tan et al. 2003; Phoon et al. 2007). In these proceedings, technical papers summarized the efforts of various prominent geotechnical research institutions, universities, and commercial testing firms in the detailed field and laboratory testing of a wide variety and array of differ-

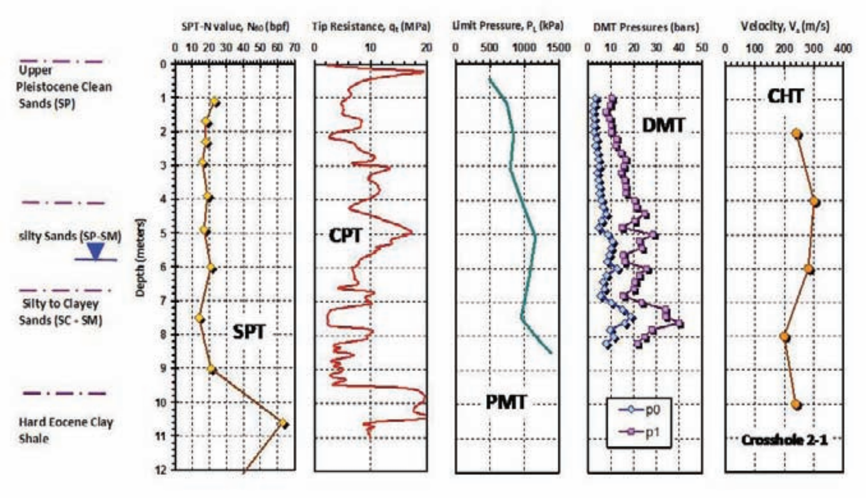


FIG. 6. Sand profile and results from SPT, CPT, PMT, DMT, and CHT at the Texas A&M national geotechnical experimentation site (data from Briaud 2007).

ing geomaterials, each within a particular geologic origin, setting, and location of a country or continent. In all cases, the IGES research programs have been underway for many years, in fact, often many decades, with most having not yet fully answered all of the behavioral subtleties within that particular soil formation. One excellent example is the Holmen sand site near Drammen, Norway, established and researched by the Norwegian Geotechnical Institute since 1956 (Lunne et al. 2003). At Holmen, extensive types of geotechnical lab sets, in-situ testing, pile foundation performance, building foundation settlements, and geophysical measurements have been collected in these sandy sediments, all of which can be cross-referenced.

Furthermore, note that a number of other well-documented sites are also available but were not included in these sets of proceedings yet would certainly qualify for IGES status, including the Canadian national test site at South Gloucester, Ontario. This site is underlain by the well-known Champlain sensitive marine clays and has served as the subject of geotechnical research for over 60 years (McRostie & Crawford 2001). At that site, the results of full-scale embankment performance and foundation settlements have been documented along with laboratory tests (e.g., index, triaxial, consolidation, time rate behavior, etc.) as well as soil borings, vane shear tests, pressuremeter, and piezocones. Of recent vintage, Yafrate & DeJong (2006) performed SCPTu, T-bar, and ball-penetration tests at the South Gloucester site. Also missing from the 2003 and 2007 IGES series is the infamous Boston Blue Clay, such as the site at Saugus, Massachusetts which has been used for studies involving embankment behavior, self-boring pressuremeter calibrations, piezoprobe tests, and series of laboratory tests (Whittle et al. 2001).

The geotechnical experimentation sites are of great value because many different types of measurements are taken in the same geomaterials in the same vicinity and location, hopefully validating issues related to test repeatability and minimizing issues of site or test variability. Furthermore, it is possible here to obtain a form of "ground truthing" in terms of interpretation, whereby the laboratory test data can be compared with field test results as well as the full-scale prototype structures. Geotechnical parameters acquired from analytical methods and/or numerical models can be calibrated properly with the recorded performance of full-scale geostructures, such as walls, pilings, footings, and excavations. Also, statistical or empirical correlations can be developed amongst different test methods. Having alternative methods of interpretation can in fact be helpful in geotechnical site characterization because there are not yet consensus procedures for assessing parameters for all types of geomaterials, thus multiple methods can be adopted in parallel towards their evaluation or range of values.

Although these research sites have been thoroughly studied using many series of field and laboratory tests, a number of unexpected facets in soil behavior have come to awareness following subsequent monitoring and construction. For instance, short- and long-term footing load tests on the IGES soft clays at Bothkennar in the UK indicated: (a) appreciable drainage, (b) lower excess porewater pressures less than expected for "undrained conditions", (c) deviations from traditional linear elastic behavior, and (d) significant long-term creep settlements comparable to those of primary consolidation (Lehane & Jardine 2003). In another full-scale instrumented case study, a large 40-m diameter circular fill over soft ground showed essentially drained behavior and significant long-term settlements due to secondary compression (Simonini et al. 2007). In yet another situation, the results of a symposium exercise involving 22 separate predictors using the same lab and field data gave a 6-fold range in bearing capacity evaluations for a large 2-m square footing pad load test on soft clay (Lehane 2003). Of final note, the unexpected large 14-m settlements of the reclaimed island of Kansai I in Osaka Bay may be attributed in part to inadequate characterization of the underlying clays (Puzrin et al. 2010). As a consequence, the findings and documented results from full-scale measurements at the IGES offer invaluable means for verifying our geotechnical engineering practice in terms of material testing, parameter assessment, constitutive modeling, and theoretical understanding on the complex nature of geomaterials.

NONINVASIVE GEOPHYSICAL MAPPINGS

In a traditional site investigation, borings or soundings are located on an established grid pattern, say 30-m on center, in order to systematically and hopefully capture any lateral variants in geostratigraphy and/or soil consistency across the site. Of course, in reality, this is merely a trial-and-error attempt since the gridded area may or may not coincide with Mother Nature's original coordinates. In fact, it would be plausible that a buried natural stream or area of old uncontrolled dumped fill could easily lie within the chosen grid points for the borings. If such buried anomalies and features were discovered during the construction operations, the contractor could demand a redesign of the geotechnical solution, otherwise alternatively claim "changed conditions". Of

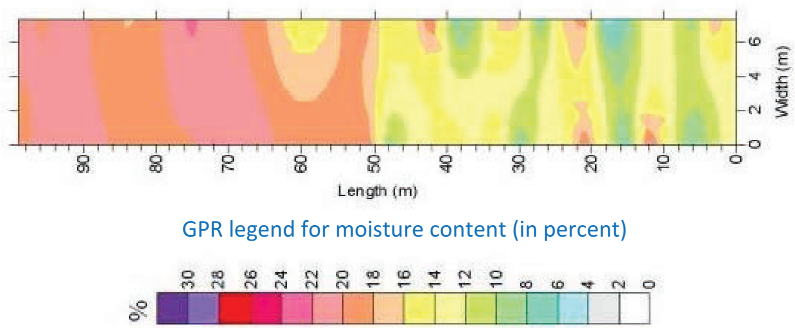


FIG. 7. Illustrative use of ground radar-mapped moisture contents for pavement studies in Waycross, GA (Larrahondo et al. 2008).

further concern, another unfortunate outcome may also include legal action by the owner, architect, structural engineer, and/or contractor against the geotechnical firm.

In 2012, a rational solution to these above situations is the utilization of surface geophysics via electromagnetic wave surveys. These well-established techniques include: ground penetrating radar (GPR), electrical resistivity surveys (ERS), and electromagnetic conductivity (EMC). Not only are these geophysical surveys quick and economical to perform, they offer a chance to rationally direct the site investigations towards the variants on the property, thus focusing on the mapping of relative differences in electrical ground properties (electromagnetic conductivity, resistivity, and/or dielectric) across the project area.

An example of the use of interpreted water contents from GPR measurements in the pavement subgrade for a GDOT pavement are presented in Figure 7 (Larrahondo et al. 2008). It can be seen that higher moisture contents ($16 < w_n < 24\%$) are notable across the western half of the site than for the eastern portion ($8\% < w_n < 16\%$). Similar mapping by EMC and SRS can be used for confirming site homogeneity and/or identifying anomalous zones and variability in the subsurface environment. The relative mapping of electrical conductivity or resistivity across a given site thus presents a rational opportunity to direct the next phase of site investigation using exploratory soundings and/or drilling and sampling methods.

HYBRID TECHNOLOGIES

The seismic piezocone test (SCPTù) and seismic flat dilatometer test (SDMTà) are hybrid in-situ exploratory methods that combine direct-push mechanical probings with downhole geophysics and therefore afford a modern means to site characterization for routine studies (Figure 8). These are not new methods, but were developed some three decades ago (Campanella et al. 1986; Hepton 1988). They offer continuous profiling

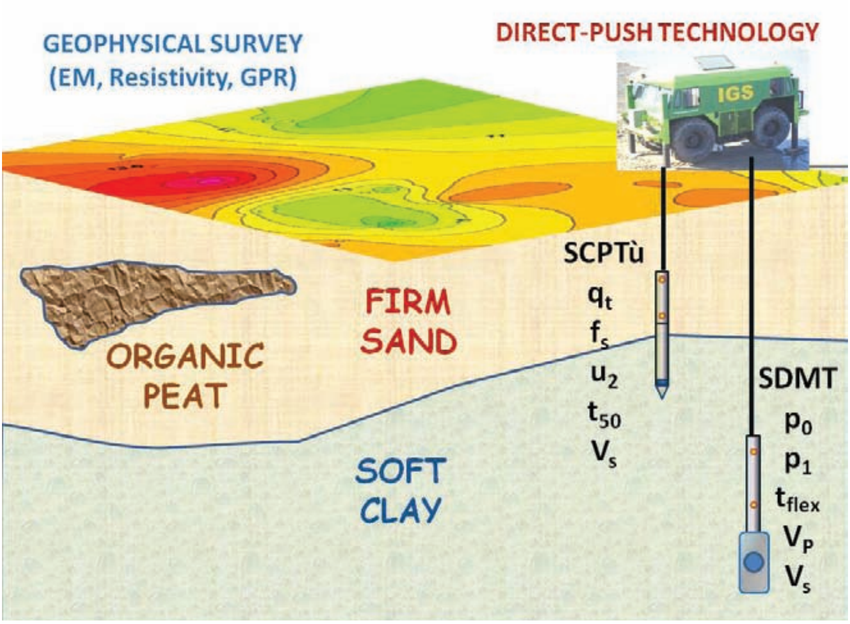


FIG. 8. Recommended routine site exploration: (a) quick areal mapping by electromagnetic geophysics; followed by: (b) direct-push soundings by either seismic piezocone or seismic dilatometer.

of strata and soil parameters with multiple readings taken at each depth in a quick and reliable manner. The SCPTù offers up to 5 readings with depth, including: cone tip resistance (q_t), sleeve friction (f_s), porewater pressure (u_2), time rate of dissipation (t_{50}), and shear wave velocity (V_s), as detailed by Mayne and Campanella (2005).

Two representative SCPTù soundings from Charleston, SC are presented in Figure 9 showing five separate measurements with depth. The soundings were performed for the recently completed Arthur Ravenel cable-stayed concrete segmental bridge over the Cooper River. The upper 20 m of soils consist of recent variable deposits of alluvial/marine origin that are underlain by the older Cooper Marl that is comprised of a stiff sandy calcareous clay (Camp et al. 2002). The marl is evident by the high penetration porewater pressures and rather high shear wave velocities.

In the SDMTà, the corresponding five readings can include: contact pressure (p_0), expansion pressure (p_1), deflation pressure (p_2), time rate of consolidation (t_{flex}), and either compression wave velocity (V_p), and/or shear wave velocity (V_s), or both. It is also possible to add additional readings such as blade thrust resistance (q_D) between successive push depths at 20-cm intervals (Marchetti et al. 2006), or resistivity, dielectric, and electrical conductivity. An illustrative example of five separate measurements taken during a SDMTà sounding performed in highly-stratified alluvial-

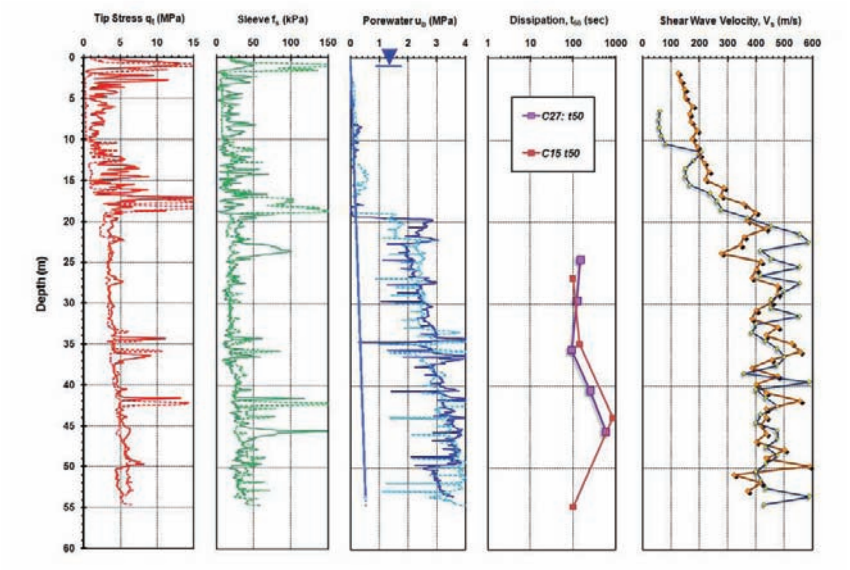


FIG. 9. Two seismic piezocone soundings at Charleston, SC

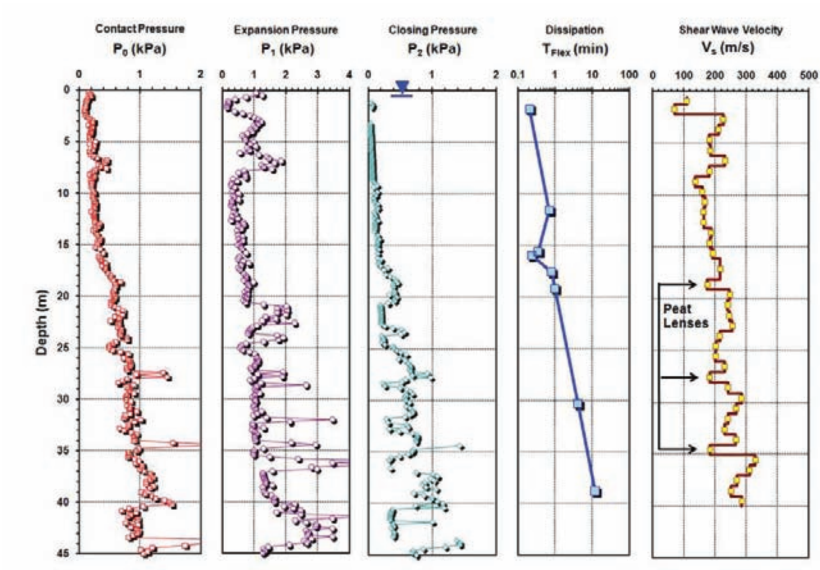


FIG 10. Seismic flat dilatometer sounding at Treporti site, Italy

marine sediments at the Treporti test embankment site near Venice, Italy is shown in Figure 10. Details on the extensive lab and field testing of these interwoven deposits are given by Simonini et al. (2007).

NEW DEVELOPMENTS

Advances in equipment, procedure, and interpretation of in-situ testing have occurred that can be used to the betterment of professional practice.

New Equipment

Over the past decade, several new in-situ devices have been introduced for site characterization, as well as a number of improvements in field testing equipment, as discussed subsequently.

While rotary drilling and augering methods are still widely used, sonic drilling and direct push methods have now become available for advancing boreholes and obtaining geomaterial samples (see Figure 11). Sonic drilling uses mechanic vibrations and resonance (50 to 150 Hz) to achieve penetration to depths of up to 300 m, thereby does not rely on air, water, or mud circulation. Spoils and excess cutting wastes are reduced to only 10 to 20 percent compared with traditional rotary drilling. Sonic drilling offers very fast penetration rates and the ability to collect continuous soil and/or rock samples with core diameters of between 100 mm 250 mm. In contrast, direct push sampling uses hydraulic or pneumatic systems to procure continuous soil samples by inserting and removing steel mandrels lined with plastic tubing.

For direct-push in-situ testing, large heavy-weight trucks and hydraulic track rigs continue to be employed to advance probes and penetrometers. Recent advances in

(a) (b) (c)

FIG. 11. Drilling equipment: (a) conventional rotary type truck rig by CME; (b) sonic rig by Boart Longyear; (c) rotosonic rig by Geoprobe Systems.

this arena include the development of light-weight rigs that have better accessibility and are easier to manuever and mobilize, particularly in areas of limited access. For capacity, either temporary anchors are installed or adjustable weights used in order to provide reaction (Figure 12). Essentially, the smallest versions of these rigs are also single-operator vehicles and therefore quite economical to deploy on small projects.

(a) (b) (c)

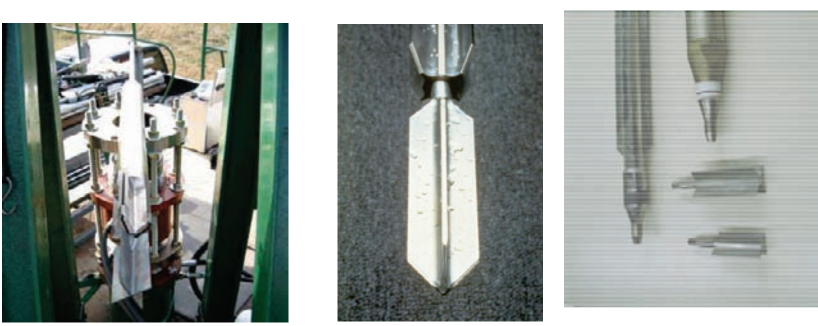
FIG. 12. Direct push vehicles: (a) 30-tonne track rig by ConeTec; (b) anchored rig by Pagani; (c) adjustable-weight Esme rig by IGS-Brisbane.

For cone penetration testing, the standard penetrometer sizes remain the 10-cm^2 and 15-cm^2 probes for production testing on routine geotechnical explorations (see Fig. 13). For special conditions involving soft soils and/or large coarse gravels, large 33-cm^2 (Fugro) and 40-cm^2 (ConeTec) penetrometers have been developed (Mayne 2010). Even larger cones have been developed for fast profiling seabeds by free-fall dropping off ships, thus requiring no pushing system whatsoever (Thompson et al. 2002; Stegmann et al. 2006). These free-fall penetrometers, or harpoon CPTs, have sizes of 60-cm^2 to 200 cm^2 and are reliant only on gravity impact to achieve penetration depths of up to 12 m (e.g, Aubeny and Shi 2006; Moser et al. 2007).

For increased resolution and profiling of shallow depths, smaller mini-cones have been built that have sizes of 1-cm^2 to 5-cm^2. These mini-CPTs are used for offshore seabed and pipeline studies (Peuchen et al. 2005), mapping of varved layers (DeJong et al. 2003), and pavement subgrades (Titi et al. 2000), as well as the obvious advantages associated with shortened dissipation times for porewater measurements (Kim 2004). Even smaller penetrometers have been devised for checking uniformity and density in centrifuge chamber deposits including 0.28-cm^2 (Wilson et al. 2004) and 0.38-cm^2 (O'Loughlin and Lehane 2010) size cones. Finally, a set of micro-cones (0.008-cm^2) have been developed using fiber bragg grating sensors with the results that diameters of 1-, 3-, 5-, and 7-mm have been built to look at soil-pile smearing, wick drains, and highly-stratified soil layering (Kim, et al. 2010). At this level, it may be possible to measure forces at the particle-penetrometer scale, thus perhaps useful in developing direct measurements for use in discrete element modeling (DEM).

(a) (b) (c)

FIG. 13. Cone penetrometer sizes: (a) standard 10- and 15-cm^2; (b) harpoon free-fall type (Moser et al. 2007); (c) micro-FBG type (Kim et al. 2010).

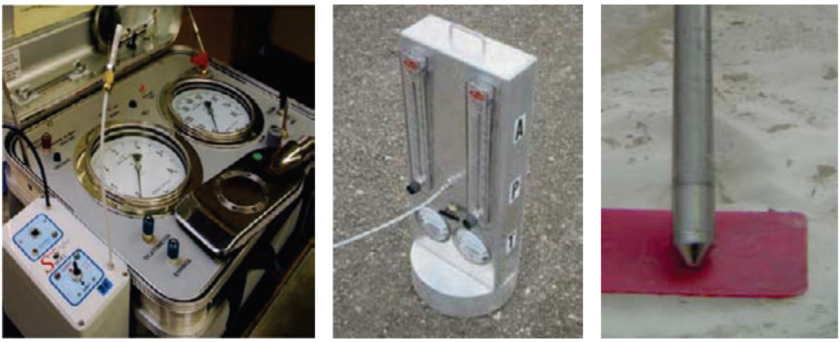


(a) (b) (c)

FIG. 14. Downhole electromechanical vanes: (a) Geotech AB onshore type; (b) McClelland offshore tapered vane; (c) Fugro offshore rectangular vane.

Electro-mechanical vanes now offer superior measurements and quality over the old long-standing mechanical vanes (Randolph et al. 2005). The vane shear test (VST) has been available for the in-situ determination of undrained shear strength (s_{uv}) and sensitivity (S_t) of clays for over 6 decades, but has long been plagued by issues of equipment maintenance problems because of rod coupling slippage, rusting, bent rods, soil-rod friction that all affect the measured torque. Moreover, the torquemeter resided at the surface and readings were either taken by hand or pen-plotted with ink onto paper. Modern electrovanes (Fig. 14) mitigate the aforementioned problems by deployment of the four-sided blade from a special housing that is located at downhole elevation and thus the measurements of torque and rotation are captured directly and digitally logged onto computer hard drives. The resulting data are quite similar in appearance to stress-strain-strength curves (Peuchen and Mayne 2007).

Some additional test devices of merit are shown in Figure 15 that include the commercial seismic flat dilatometer (Marchetti, et al. 2008) which is a hybrid pressure probe combined with downhole geophysics measurements, a gas or air permeameter



(a) (b) (c)

FIG. 15. Selected in-situ field test devices: (a) seismic flat dilatometer, (b) air permeameter, (c) scour probe.

(a) (b) (c)

FIG. 16. Modern electronic hand-held penetrometers for field use: (a) Spectrum Scout SC900, (b) Rimik CP40, (c) Eijkelkamp Penetrologger (Kees 2005).

for evaluating the hydraulic conductivity or specific permeability of granular surface soils, particularly base course materials (White et al. 2007), and an in-situ scour probe for providing a quick field evaluation of the erosion potential of soils (Caruso and Gabr 2011).

Of additional interest are the small portable electronic penetrometers for manual use by field personnel (Kees 2005). In lieu of having your site technician or field engineering being embarrassed by having to use the old heavy, burdensome and antiquated dynamic drop-weight penetrometer in a hand-bored hole, or the worse

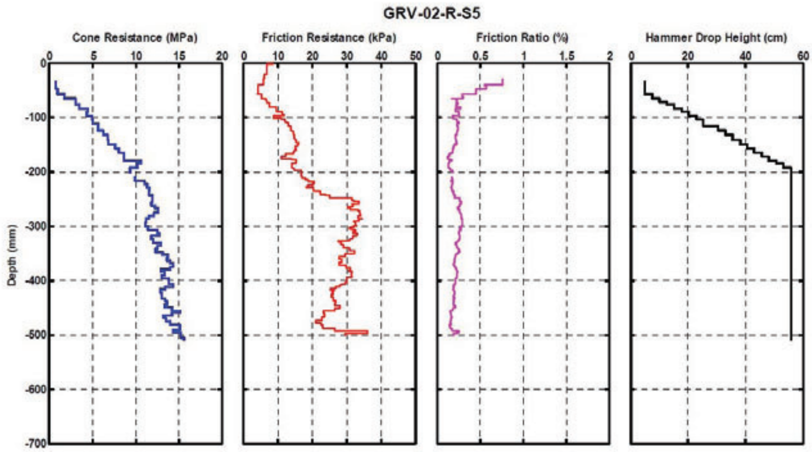


FIG. 17. Example multi-channel sounding results using the portable RapSochs system (Kianirad 2011)

alternative (e.g., a piece of No. 4 rebar), Figure 16 shows three available flashy models which offer means for quick and repeatable electronic profiling of tip resistance with depth. Note that the importance of showing a professional image and establishing a high respect for geotechnical engineering practice should not be underestimated.

Regarding light-weight and portable dynamic penetrometers, recent developments of a portable multi-channel probe termed "RapSochs" (rapid soil characterization system) have been made by Kianirad (2011), as presented in Figure 17. Here, equivalent readings of tip resistance, sleeve friction, and corresponding hammer drop height with depth are presented, using the calibrated algorithms developed by Kianirad et al. (2011). Future versions of the RapSochs device are to include measurements of moisture content and penetration porewater pressures.

New Procedures

Several new and improved testing procedures for soils have been made in the past decade, allowing for the evaluation of additional geotechnical parameters or producing better quality data.

Twitch Testing. An adjustable rate procedure termed "twitch testing" offers a means to evaluate viscosity strain rate effects during undrained loading of clays and silts, as well as the demarcation of drainage conditions (i.e., drained vs. partially drained vs. undrained), as described by Randolph (2004). Twitch testing procedures can be applied to vane shear, cone penetration, piezocone, t-bar, ball penetrometer, and piezoball results (Chung et al. 2006; Yafrate & DeJong 2007). It can be particularly useful in the evaluation of mine tailings (Oliveira et al. 2011). The piezocone and piezoball tests are especially valuable in twitch testing as both the tip resistances and porewater readings can be tracked together using a normalized and dimensionless velocity: $V = v \cdot d/c_v$ where v = test velocity, d = probe size, and c_v = coefficient of consolidation. A full range of twitch test rates using CPTu is presented by Kim et al. (2008) for clay-sand mixtures (Figure 18). For the soils tested, these results suggest that undrained conditions prevail for V > 10, while fully drained response occurs for V

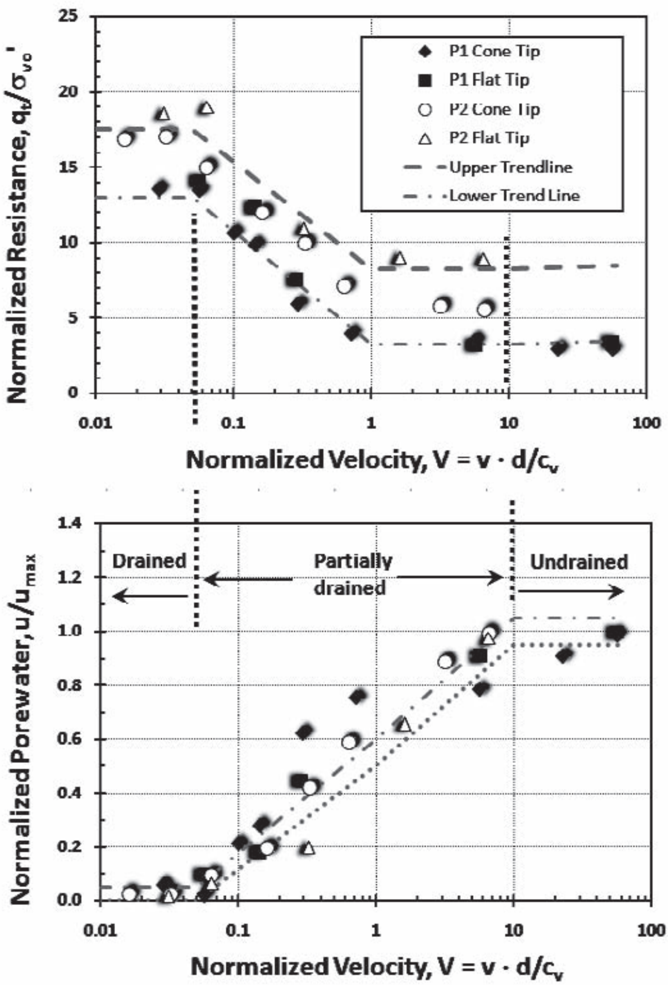


FIG. 18. Twitch testing results from two series of piezocones in clayey soils (data from Kim et al. 2008).

< 0.05, and intermediate V values correspond to partially drained and partly undrained cases. As laboratory testing forces an "undrained" response corresponding to conditions of constant volume, the twitch testing can be useful in confirming the validity of such cases and help to resolve difficulties in matching field and lab determinations of s_u. Twitch testing can also be used to quantify strain rate parameters during undrained shear (Peuchen & Mayne 2007).

Modulus Reduction Curves. For evaluating the in-situ dynamic properties of the ground, Stokoe et al. (2008) have developed a special pattern series of embedded geophones to measure wave arrivals, including arrival times for shear wave velocity and amplitudes for determining shear strain level: γ_s = PPV/V_s, where PPV = peak particle velocity. Using large portable shakers with "ominous" names (e.g., T-rex, Liquidator, Thumper), the hefty ground sources can be used to profile deep V_s profiles via low frequency waves. Moreover, the procedures can be used to obtain site-specific G/G_{max} reduction curves with logarithm of shear strain and in-situ damping responses on-site using variable frequencies and modes, supplemented with measurements taken at various locations and depths away from the shakers. Results appear comparable to those obtained on lab specimens using resonant column-torsional shear testing on clean to silty sands, as illustrated in Figure 19c.

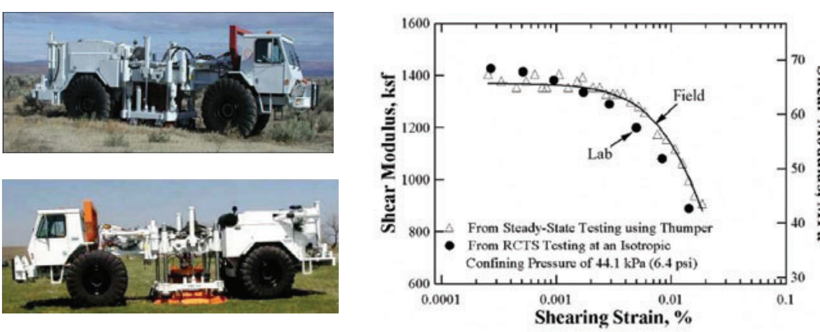


FIG. 19. In-situ dynamic measurements: (a) the Liquidator; (b) T-rex vibrator; (c) comparison of G/G_{max} reduction curves from field and lab testing on sands (Stokoe et al. 2008).

Downhole Shear Wave Testing. The original version of the SCPT used paired sets of left- and right-strikes to provide a crossover point and allow pseudo-interval downhole shear wave velocity profiling at 1-m intervals. As a single data point (arrival time) between consecutive test depths was paramount in the field measurement, a common procedure is to obtain two left strikes (for repeatability confirmation), as well as two right strikes. The result is that many SCPT today spend too much time collecting the DHT part of the SCPT. Two simple improvements to procure higher quality and faster results include: (a) autoseis source; (b) cross-correlation in the post-processing phase. Use of an autoseis (Figure 20a) is advantageous because the generated wave is repeatable (McGillivray and Mayne 2008). Cross-correlation utilizes the main wave arrival to match say 2000 data points (in lieu of the single sole data point in a first crossover). Plus the cross-correlation is automated in standard software (Excel, Matlab, Shearpro), whereas the cross-over point is usually evaluated by visual inspection.

Additional developments include the introduction of two improved field procedures: (1) frequent-interval DHT; (2) continuous SCPT (Mayne and McGillivray 2008). Frequent-interval downhole testing is the slowest approach, but offers the best detailing and delineation of V_s profiling with increased resolution at 20-cm intervals (Fig. 20b). Continuous SCPT is the fastest method available as wavelet data are generated and captured at approximately 1 to 5 second intervals. Such data require special considerations including fast sampling rates and noise filtering, time- and frequency-domain analysis, and post-processing methods.

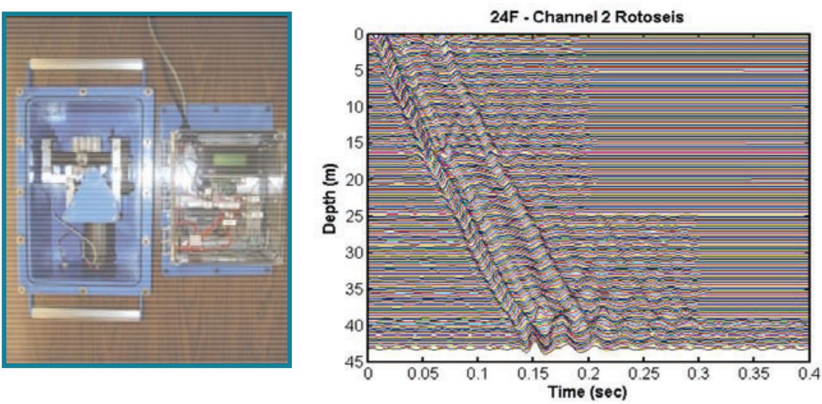


FIG. 20. Enhancements to downhole testing: (a) rotoautoseis source generator, (b) wavelet cascade from frequent-interval shear wave testing in Aiken, SC.

Interpretation Methods

The evaluation of geotechnical parameters from in-situ testing can be based on analytical, theoretical, numerical, and/or empirical-statistical trendline approaches. Table 1 gives a brief summary of selected references that address the interpretation of the primary set of in-situ field tests, yet is surely not comprehensive. Where appropriate, the test procedures per ASTM guidelines are also listed. Currently, no single framework or methodology has been established for interpreting the main tests (CPT, DMT, PMT, SPT, VST) together in a consistent manner. Instead, an assortment of different approaches are employed for each test; e.g. for clays: vane shear testing evaluated via limit equilibrium, cone penetrometer via strain path method, pressuremeter test via cavity expansion, etc. Moreover, the interpretations of in-situ tests are normally tackled assuming one of two extreme conditions: (a) undrained

Table 1. Select References for In-Situ Test Procedures and Interpretation

In-Situ Method	Title	Publisher	Author/Editor
General Overview on Field Tests	In-Situ Tests in Geomechanics (SPT, CPT, DMT, PMT, VST)	Taylor & Francis Group	Schnaid (2009)
	Geotechnical & Geophysical Site Characterization (ISC-3)	Taylor & Francis	Huang (2008)
	Site investigation and mapping in urban areas	14th ECSMGE: Millpress	Młynarek (2007)
	Geotechnical & Geophysical Site Characterization (ISC-2)	Millpress	Viana da Fonseca (2004)
	Geotechnical Site Investigation	Thomas Telford	Simons et al. (2002)
	Subsurface Investigations: Geotechnical Site Characterization	NHI Manual	Mayne et al. (2002)
	Evaluation of Soil & Rock Properties	FHWA Circular 5	Sabatini et al (2002)
	Geotechnical Site Characterization (ISC-1)	Balkema	Robertson (1998)
	Exploration of soft soil and determining design parameters	Port & Harbour Res. Inst. Japan	Leroueil & Jamiolkowski (1991)
	Manual on Estimating Soil Properties for Foundation Design	EPRI	Kulhawy & Mayne (1990)
	Developments in Field and Lab Testing of Soils	ISSMGE	Jamiolkowski et al. (1985)
Cone Penetration Test (CPT): ASTM D5778	Proceedings CPT '10	Omni Press	Robertson (2010)
	In-Situ Soil Testing	Lankelma	Brouwer (2007)
	Synthesis 368 on Cone Penetration Testing	TRB/NCHRP	Mayne (2007)
	CPT in Geotechnical Practice	Blackie Academic	Lunne et al. (1997)
	Proceedings CPT '95	Swedish Geotech-nical Society	Massarsch et al. (1995)
Dilatometer Test (DMT): ASTM D6635	The Flat Dilatometer Test in Soil Investigations	PCU, Indonesia	Marchetti et al. (2001)
	Flat Dilatometer Testing (International Symposium)	In-Situ Soil Testing	Failmezger and Anderson (2006)
Pressuremeter Test (PMT): ASTM D4719	Intl. Symposium: 50 Years of PMT	LCPC Press	Gambin et al. (2005)
	Cavity Expansion Methods in Geomechanics	Kluwer Academic	Yu (2000)
	Pressuremeters in Geotechnical Design	Blackie Academic	Clarke (1995)
	The Pressuremeter and Its New Avenues	Balkema	Ballivy (1995)
	The Pressuremeter	Swets & Zeitlinger	Briaud (1992)
	Pressuremeters	Thomas Telford	Houlsby (1990)

Table 1 (continued)

In-Situ Method	Title	Publisher	Author/Editor
Standard Penetration Test (SPT): ASTM D1586	Penetration Testing	Institution of Civil Engineers, UK	Stroud (1988)
Vane Shear Test (VST): ASTM D2573	Rate effects in VST	SUT	Peuchen (2007)
	On the evaluation of s_u and P_c' in clays	CGJ	Larsson, R. and Åhnberg, H. (2005)
	Vane Shear Strength Testing in Soils	ASTM	Richards (1988)
Full-Flow Penetrometers (T-bar; Ball)	Geotechnical Testing Journal	ASTM	DeJong et al. (2010)
	JGGE	ASCE	Yafrate et al. (2007)
	Proc. ISC-2, Porto	Millpress	Randolph (2004)
Geophysics: including Crosshole (CHT): ASTM D4428, Downhole (DHT): ASTM D7400; SASW, MASW; GPR; Refraction, Reflection; ReMi, and Resistivity	Synthesis on Geophysics for Transportation Projects	NCHRP/TRB	Sirles, P.C. (2006)
	Application of Geophysics to Highway Problems	FHWA	Wightman et al. (2003)
	Soils and Waves	Wiley & Sons	Santamarina et al. (2001)
	Dynamic Geotechnical Testing II	ASTM	Ebelhar, et al. (1994)

Notes:
ASTM = American Society for Testing & Materials; CGJ = Canadian Geotechnical Journal; EPRI = Electric Power Research Institute; FHWA = Federal Highway Administration; ISC = International Site Characterization; ISSMGE = Intl. Society Soil Mechanics & Geotechnical Engrg; NCHRP = National Cooperative Highway Research Program; NHI = National Highway Institute; SUT = Society for Underwater Technology; TRB = Transportation Research Board

behavior (i.e., loading at constant volume: $\Delta V/V_0 = 0$); or (b) drained response (i.e.,loading with no excess porewater pressure: $\Delta u = 0$). Consequently, many interpretation methods are established for "clays" (i.e., undrained) vs. "sand" (i.e., drained). However, partially-drained behavior is also plausible such that some Δu is generated at the same time that some ΔV changes occur. Partly undrained cases may also occur where some Δu is generated (but not fully developed).

What are needed are global interpretative schemes, able to address many types of soils (clays, silts, sands, mixtures) that exhibit possible responses for various drainage conditions under a range of strain rates of loading. For instance, the evaluation of the effective friction angle (ϕ') from piezocone measurements using an undrained/drained penetration theory has been developed for use in sands, silts, clays, and mixed soils by the Norwegian Institute of Technology (Senneset et al. 1989). The interpretation of DMT data has offered a generalized approach, primarily towards foundation settlements, that is found applicable to many soil types (Marchetti et al. 2001). An analytical-empirical CPTu soil classification system has been put forth by Schneider et al. (2008). Also, a generalized approach to permeability evaluation on-the-fly by piezocone using a moving volumetric dislocation model has been derived (Elsworth & Lee 2007; Lee et al. 2008).

Moreover, a universal approach should be able to assist in the evaluation of several types of in-situ test methods. An initial set of calibrations using a cavity expansion-critical state approach to evaluating PMT, CPTù, and DMT has been applied to data from six clay sites (Mayne 2007b, Mayne & Burns 2008). In that approach, only five input soil parameters are needed: effective friction angle (ϕ'), prestress (σ_p'-σ_{vo}'), rigidity index (I_R), plastic volumetric strain potential ($\Lambda = 1\text{-}C_s/C_c$), and void ratio ($e_o$) to produce full profiles of q_t, u_1, u_2, f_s, p_0, and p_1 with depth. With additional information (i.e., c_{vh}), the analytical model provides u_1 and u_2 dissipations with time.

For future research directions, it will be helpful to employ results from numerical simulations using finite elements, discrete elements, and/or finite difference solutions towards a unified approach to in-situ test evaluation for all test types under all drainage conditions, and can also be used to directly model the full prototype situation (e.g., driven offshore pile, jacked tunnel, supported excavation, etc.). This is the major challenge for the next generation of researchers in geotechnical site characterization.

CONCLUSIONS

The complexities of natural soil behavior are now evident from decades-long studies involving complementary suites of laboratory studies, in-situ testing, and full-scale structural performance results at international geotechnical experimentation test sites located in various geomaterials including sedimentary deposits of clays, silts, sands, mixed soils, and structured cemented geomaterials, as well as residua derived from in-place weathering of rocks. As such, site characterization is best-handled by deploying many types of in-situ geotechnical probings and geophysical surveys together in concert with laboratory testing programs on high-quality undisturbed samples. As this is only feasible on large projects or critical facilities with full funding, the profession would be best positioned to adopt the seismic piezocone and/or seismic dilatometer for routine subsurface explorations because up to 5 independent readings are collected in a single sounding, thus no compromise is made in acquiring the necessary and varied types of information about the ground conditions.

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Flexible Pavement Analysis and Design-A Half Century of Achievement

Carl L. Monismith[1], Dist. M. ASCE

[1]Robert Horonjeff Professor of Civil Engineering, Emeritus, Department of Civil and Environmental Engineering, University of California, Berkeley, Berkeley, California 94720-1712; monismith@ce.berkeley.edu

ABSTRACT: This paper provides one person's perspective on flexible pavement analysis and design methodology since about 1960. Some developments during the past 50-plus years briefly summarized include: mechanistic analyses; materials characterization; mechanistic-empirical (M-E) pavement design methodologies; accelerated pavement testing; non-destructive pavement evaluation and overlay design; pavement management; and, improved construction practices. Some observations on *education* and *training* in *pavement engineering* are also included.

INTRODUCTION

The purpose of this paper is to provide a perspective on flexible pavement analysis and design methodology since about 1960. This perspective is based on the author's view of developments in this area beginning in 1953, when he became involved with research in *flexible pavement design* and *asphalt technology*. These developments are the result of many engineers in the international community freely sharing their ideas and research through technical meetings, individual contacts made possible by such meetings, published technical papers, reports, and correspondence. Such perspectives are necessarily limited by the experiences and contacts of the preparer. The author's perspective has been influenced significantly by the International Conferences on Asphalt Pavement Design (starting at the University of Michigan in 1962) and the contacts made through these conferences, particularly with individuals from other parts of the United States, Europe, South Africa, and Australia. Moreover, this discussion has been significantly influenced by the engineers and researchers in California with whom he has had the privilege to work.

Some of the key developments during the past 50-plus years which are briefly summarized include: mechanistic analyses; materials characterization; mechanistic-empirical (M-E) pavement design methodologies; accelerated pavement testing; non-destructive pavement evaluation and overlay design; pavement management; and, improved construction practices.

A few observations on *education* and *training* have also been included since *pavement engineering*, like other Civil Engineering disciplines, require well educated engineers to insure that flexible pavements, an extremely important component of our infrastructure, are properly designed, constructed, maintained, and rehabilitated.

PRIOR TO 1960

A number of developments prior to 1960 contributed to those to be discussed subsequently. A few of these which are considered key will be briefly described in this section.

During World War II the U.S. Army Corps of Engineers (USACE), developed a pavement design procedure for airfield pavements, initially for military applications. This procedure made use of the California Bearing Ratio (CBR) procedure developed by O.J. Porter for the California Highway Department (*1*). The USACE, in addition to modifying the CBR test procedure to meet their needs, also modified the thickness design curve developed for highway loading to accommodate a range in aircraft wheel loads using elastic theory (Boussinesq) (*2*). Initially the aircraft operated on single wheel gears. In 1945, the B-29 was introduced with dual-wheel gears. This required additional considerations of the use of the Boussinesq solution and resulted in the introduction of the Equivalent Single Wheel Load (ESWL)[1] concept (*3*). Two of the key people in this development were W. Turnbull and R. Ahlvin (*4*). An important feature of the USACE studies was the use of accelerated pavement testing to validate and modify thicknesses arrived at for different aircraft load and gear configurations by the analytical procedure using the Boussinesq analysis (*2*).

Test roads have been used in the United States since at least 1921 with recorded reference to the Bates Test Road.[2] Two key developments in this area were the Western Association of State Highway Officials (WASHO) Road Test in Malad, Idaho in 1951 (*5*) and the American Association of State Highway Officials (AASHO) Road Test in the period 1958-60 in Ottawa, Illinois (*6*).

At the WASHO Road Test, A.C. Benkelman introduced the Benkelman Beam which permitted pavement deflections to be measured under slow moving wheel loads (*5*). This tool facilitated rapid measurement of pavement response, thus providing an early indication of future performance and a comparative measure against which to check calculated pavement response. It also provided an important tool for improved overlay pavement design. In this test road, the importance of thicker sections of asphalt concrete, now termed hot mix asphalt (HMA), (4 in. [100 mm] versus 2 in. [50 mm]) to improve pavement performance was also demonstrated.

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[1] Aircraft gear loads are expressed in terms of an equivalent single wheel load (EWSL) defined as the single wheel load which yields the same maximum deflection at a given depth as a multiple wheel load; the contact area of this ESWL is equal to the contact area of one of the wheels of the multiple wheel assembly. As will be seen subsequently, this definition is different than the equivalent single axle load (ESAL) used for highway pavement design.

[2] Discussed in: Older, C. "The Bates Experimental Road," and Goldbeck, A.T. "Highway Researches and What the Results Indicate", papers in *Proceedings of the American Road Builders' Association,* 1922.

The AASHO Road Test[3] (*6*) sparked a renewed interest in improved pavement design and provided the impetus for the development of many current analytically-based design procedures. Under the excellent leadership of W.N. Carey, Jr., the AASHO Road Test provided another important contribution to the engineering community *since well documented performance data were assembled and stored* permitting future researchers to have access to these data. Performance predictions by the new analytically-based procedures could be compared with actual field performance; reasonable comparisons confirmed the "engineering reasonableness" of the methodologies.

Following WW II, the Road Research Laboratory (RRL) (now the Transportation Research Laboratory [TRL]) of the United Kingdom installed test sections in a number of their major roadways to study the longer term performance of pavements under actual traffic loading in specific environments. One such experiment was the Alconbury Hill motorway reported by Croney et al. (e.g., Reference [*7*]). At the time, Sir William Glanville was the Laboratory Director and pavement research in the UK received considerable emphasis under his direction.

In the pavement analysis area, solutions were developed by Burmister in the 1940's for the response of two-and three-layer elastic systems to representative loading conditions (*8*). While these solutions were limited to conditions at layer interfaces and the results were generally presented in graphical form, they nevertheless introduced the engineering community to the important concept of treating the pavement as a layered system. Comprehensive use of these solutions would have to wait approximately 15 years for the advent of the electronic computer.

The work of F.N. Hveem and his staff in California had been measuring pavement deflections for a number of years prior to the WASHO Road Test using a GE travel gauge. Publication of his research in 1955 (9) provided a strong link between pavement deflections, truck loading, and fatigue failures in the asphalt-bound portion of pavement sections. This work had a significant impact on the development of procedures to predict fatigue cracking using analytically-based methodologies. During this period Hveem also introduced the concept of equivalent wheel loads (EWL) (*10*), the forerunner of equivalent single axle loads (ESALs), and the concept of layer equivalency with the use of the gravel equivalent factor (*11*).

1960 TO DATE

While the first International Conference on Asphalt Pavements (termed The International Conference on the Structural Design of Asphalt Pavements) was convened at the University of Michigan in August 1962. many of the elements for mechanistic empirical (analytically-based) pavement analysis and design were being worked on prior to the Conference. Examples include; Shell (*15, 16*); the Asphalt Institute (*14*); the RRL (*13*); and, University of Nottingham of the UK (17).

The purpose of the conference was to provide a technical venue for discussion of the results of the AASHO Road Test, as well as for worldwide developments in asphalt pavement analysis and design. J.E. Buchanan, President of the Asphalt

[3] Resulted from the 1956 Interstate Highway Act; its cost of $29 million would correspond to the cost of the Strategic Highway Research Program 30 years later, 1988-1993, which was $150 million.

Institute provided the primary impetus (and financial support for the Conference with assistance of F.N. Finn (who had been the Asphalt Institute's representative at the Test Road). The University of Michigan (UM) at Ann Arbor was selected as the conference site because of its long time association with asphalt pavements. W.S. Housel and W.K. Parr of the UM Civil Engineering Department, working with Asphalt Institute representatives and with key U.S. and international members of the asphalt paving community, developed a very successful conference.

Some of the key early U.S. and international participants (1962 and 1967) included: E.J. Yoder, W. Goetz, K. Wester, E. Nakkel, P. Rigden, and J. Kirk). The bound volumes of these Conferences, in addition to containing the technical papers, moderator reports, and discussions, include listings of the various committees which have contributed to the continued success of these conferences

Some of the key developments since 1960 are discussed in the following sections.

Mechanistic Analysis

The use of multi-layered analysis to represent pavement response, although developed by Burmister in the 1940s (*8*), did not receive widespread attention until the First International Conference on the Structural Design of Asphalt Pavements in 1962. While some agencies utilized solutions for two- and three-layered elastic solids in their design methodologies (e.g., the U.S. Navy [*12*]), use of these solutions was both limited and cumbersome.

At the 1962 Conference, however, important contributions were made by Whiffin and Lister (*13*), Skok and Finn (*14*), Peattie (*15*), and Dormon (*16*), and Pell (17). Both Whiffin and Lister and Skok and Finn illustrated how layered elastic analysis could be used to analyze pavement distress. Peattie and Dorman presented several concepts, based on such analyses, which would later become a part of the Shell pavement design methodology (and that of other organizations as well).Pell presented important data on the fatigue response of asphalt mixes.

A number of general solutions for determination of stresses and deformations in multi-layered elastic solids also were presented at the 1962 Conference. Additional related work was published in 1967 at the Second International Conference. These general solutions, coupled with rapidly advancing computer technology, fostered the development of the current generation of multi-layer elastic and viscoelastic computer programs. Table 1 contains a listing of *some* of the most commonly used programs. The ELSYM program, developed at the University of California, Berkeley by G. Ahlborn (*18*) and widely used, directly benefited from the 1962 and 1967 Conference papers.[4]

Computer solutions for layered systems in which the properties of each of the layers could be represented as linear viscoelastic materials were subsequently introduced; two available solutions, VESYS (*23*) and VEROAD (*24*), are listed in Table 1.

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[4] Although the work of the CHEVRON researchers never appeared in the published literature, it is important to recognize their significant contribution since they presented the first computer solution for a five-layer system (CHEV5L) in 1963 (*19*).

Table 1 Summary of Some Available Computer-Based Analytical Solutions for Asphalt Concrete Pavements.

Program	Theoretical Basis	Number of Layers (max)	Number of Loads (max)	Program Source	Remarks
BISAR (*20*)	MLE	5	10	Shell International	The program BISTRO was a forerunner of this program
ELSYM (*18*)	MLe	5	10	FHWA (UCB)	Widely used MLE analysis program
JULEA (*21*)	MLE	5	4+	USACE WES	Used in Program LEDFAA; also used in the new MEPDG of AASHTO
CIRCLY (*22*)	MLE	5+	100	MINCAD, Australia	Includes provisions for horizontal loads and frictionless as well as full-friction interfaces
VESYS (*23*)	MLE or MLVE	5	2	FHWA	Can be operated using elastic or viscoelastic materials response
VEROAD (*24*)	MLVE	15 (resulting in half-space)		Delft Technical University	Viscoelastic response in shear; elastic response for volume change
ILLIPAVE (*25*)	FE		1	University of Illinois	
FENLAP (*26*)	FE		1	University of Nottingham	Specifically developed to accommodate non-linear resilient materials properties
SAPSI-M (*27, 28*)	Layered, damped elastic medium	N layers resting on elastic half-space or rigid base	Multiple	Michigan State University/ University of California Berkeley	Complex response method of transient analysis—continuum solution in horizontal direction and finite element solution in vertical direction

MLE—multilayer elastic
MLVE—multilayer viscoelastic
FE—finite element

In the late 1960s, finite-element analyses to represent pavement response were developed by a number of researchers [e.g., Duncan, et al. (*29*)]. Increasingly, the finite-element method has been used to model pavement response, particularly to describe the nonlinear response characteristics of pavement materials. Examples of this approach include ILLIPAVE (*25*) and FENLAP (*26*).Solutions for the dynamic analysis of asphalt concrete pavements under moving, fluctuating loads have also been developed. The SAPSI-M program (*28*), Table 1, is one such example. In this program moving loads are modeled as a series of pulses with durations equal to the time required for a load to pass a specific location.

In terms of current analytically-based pavement design procedures, layered elastic analysis is the primary method for defining pavement response to load. Use of the finite element methodology has had limited application to date [e.g., ILLIPAVE (*25*)] possibly because of computational time constraints. Hence, it has been used primarily in special applications. However, improvements in both computer capabilities and in formulating finite element representations should allow it to become an integral part of routine pavement analysis of asphalt pavements in the future. (It should be noted that finite element analysis is an integral part of the design procedure for portland cement concrete (PCC) pavements in the Mechanistic-Empirical Pavement Design Guide [MEPDG] of AASHTO which is now available [30]).

MATERIALS CHARACTERIZATION

An important aspect of the development of analytically-based methodologies has been the evolution of procedures to define requisite material characteristics. A number of the analysis procedures, summarized in Table 1, are based on the assumption of linear response (either elastic or viscoelastic). The majority of materials used in pavement structures do not satisfy such an assumption. Accordingly, ad hoc simplifications of materials response have been used.

Materials characterization for analytically-based design methodologies requires definition of stress versus strain relationships, termed *stiffness* or *resilient modulus* (often referred to simply as ‘modulus’), for each pavement component. These moduli can be used to determine stresses, strains, and deflections within the pavement structure. As shown in Figure 1, results of such computations permit estimates of the various forms of distress which influence pavement performance. Methods currently used for these determinations include the following modes of load application: axial with or without confining pressure; shear: flexure; and, indirect tensile (diametral). Load forms include creep, repeated or dynamic loading (Reference [*31*] contains a more detailed tabulation of the loading forms and modes and forms associated with different pavement materials).

Determinations of distress criteria are also part of the characterization process. For asphalt concrete pavements (including mixes with recycled HMA, [RAP]), these include measures of the permanent deformation, fatigue, and fracture characteristics for the treated components and the permanent deformation response of soil and untreated granular materials. Fatigue and fracture characteristics are also measured for other treated materials (e.g., portland cement, lime, and lime fly-ash). As will be seen subsequently, these characteristics may be determined through laboratory testing for a specific project. Alternatively, estimated values of response representative of specific categories of materials may be selected based on previous research data. When defining response characteristics, service conditions must be properly considered. They include: stress state-associated with loading; environmental conditions-moisture and temperature; and construction conditions-e.g., water content and dry density for untreated materials and degree of compaction for asphalt-bound materials. To ensure that materials evaluation is accomplished at reasonable cost, these service conditions must be carefully selected.

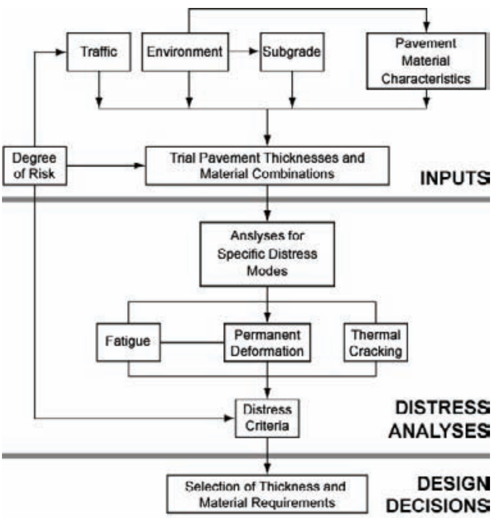

Figure 1 Simplified Analysis Design framework

Stiffness

Developments relative to pavement materials stiffness characteristics, which include *asphalt*-aggregate mixes, untreated fine-grained soils and granular materials, are summarized in this section. Reference (*32*) contains a summary of these characteristics for portland cement-treated and lime-treated materials.

Asphalt Mixes. The stiffness characteristics of asphalt-aggregate mixtures are dependent on the time of loading and temperature, i.e.:

$$S_{mix} = \frac{\sigma}{\varepsilon}(t,T) \tag{1}$$

where: S_{mix} = mixture stiffness; σ ,ε.= stress and strain, respectively; t = time of loading; and, T = temperature.

This approach was first presented by Van der Poel (*33*) in 1950, and expanded by Heukelom and Klomp (e.g., *34*). At temperatures above 25ºC, it is likely that the stress state has an influence on the stiffness characteristics of these materials, becoming more pronounced as the binder is less stiff. This effect may be reflected in an ad hoc manner when considering specific modes of distress.

Inherent in this approach is the assumption of the interchangeability of time and temperature. Such interchangeability is incorporated in a number of the design procedures to be discussed subsequently. It is useful to the design engineer since properties can be measured at more than one temperature for times of loading which are tractable and extended to other temperatures and times of loading which may be difficult to reproduce in the laboratory.

Mix stiffness can also be estimated from parameters such as the properties of the binders as they exist in the mix in the field and the volumetric proportions of the components. Two such examples are: 1) estimation procedures developed by the Shell investigators (*35, 36*) and incorporated in the Shell design procedure (*37*); and 2) procedure developed by Witczak (*38, 39*) and used both in the Asphalt Institute design procedure (*40*) and in the new AASHTO Guide methodology, under development (*30*).

The Shell method requires a measure of the stiffness of the binder and the volumetric properties of the aggregate, binder, and air in the compacted mix. Witczak's procedure incorporates aggregate grading characteristics as well (e.g., *39*)

Subgrade Soils. While there has been considerable research on the stiffness characteristics of fine-grained soils and granular materials by a number of investigators, impetus for these efforts was provided by the research of H.B. Seed and his associates (*41*). The term *resilient modulus* was introduced to describe relationships between applied stress and recoverable strain measured in a repeated load triaxial compression test.

For fine-grained soils, the stiffness characteristics are dependent on dry density, water content or suction, soil structure, and stress state. For a particular condition, the stiffness as defined by resilient modulus, for example, is dependent on the applied stress; that is:

$$M_r = F(\sigma_d) \tag{2}$$

where: M_r = resilient modulus, σ_d / ε_r ; σ_d .= repeated deviator stress; and, ε_r = recoverable strain measured after some prescribed number of applications of σ_d

The mechanical properties of soils and granular materials depend on the effective stress state (total stress - pore water pressure); accordingly Brown et al. (*42*) suggested a more general model, based on tests on saturated clays, of the form:

$$G_r = \frac{\sigma_d}{C}\left(\frac{\sigma_m}{\sigma_d}\right)^m \tag{3}$$

where: G_r= resilient shear modulus; σ'_m = mean normal effective stress; and C, m = constants for particular soil.

For partially saturated soils with degrees of saturation greater than 85 percent, soil suction can be used in place of the mean normal effective stress (*42*). To avoid unrealistically high values of G_r at low stresses, it was recommended (*42*) that the values of G_r be related to the magnitude of strain, as had been suggested by Seed and Idriss (*43*) for soil response to earthquake loads and used extensively in a variety of small strain geotechnical problems.

The influence of deviator stress on the stiffness response of the subgrade soil is illustrated, for example, in Reference (*40*); and the dependence of the subgrade stiffness on soil moisture suction as suggested by Equation (*3*) is illustrated in Reference (*44*).

Freeze-thaw action also influences the stiffness of fine-grained soils. When the soil is frozen, its stiffness increases; when thawing occurs, the stiffness is reduced substantially (*45*), even though its water content may remain constant. This was originally suggested by Sauer (*46*). Such variations should be incorporated into the design process where appropriate.

To ensure that fine-grained soils tested in the laboratory for pavement design purposes are properly conditioned, it requires an understanding of soil compaction (*47*), particularly the relationship among water content, dry density, soil structure, and method of compaction. At water contents dry of optimum for a particular compactive effort, clay particles are arranged in a random array termed a "flocculated" structure. At water contents wet of optimum (provided shearing deformations are induced during compaction), particles are oriented in a parallel fashion, often termed "dispersed" (*47*). As suggested by Lambe (*48*) and demonstrated by Mitchell (*49*), these dispersed and flocculated compacted soil structures can lead to significant differences in mechanical properties for specimens assumed to be at the same water content and dry density. Seed has demonstrated that selection of the soil structure after saturation is dependent on the type of soil (expansive or non-expansive) and, therefore, should the designer must select an appropriate compaction method to establish the soil structure for the water content selected for resilient modulus testing. Guidelines based on such considerations are available (*50*).

If equipment is not available to measure the stiffness modulus, a number of procedures to estimate this property from other tests have been developed. One of the well known approximate relations is that developed by the Shell Investigators (*51*) which was based on correlations between dynamic in-situ tests and the corresponding measured CBR values to estimate subgrade stiffness moduli:

$$E_{sub} = 10 \cdot CBR \text{ (in MPa)} \tag{4}$$

Note: Use of this relationship should be restricted to CBRs less than 20. In addition, it should be noted that stiffness values might range from 5 to 20 times the CBR.

Other relationships include, for example, those developed by the TRRL, also based on CBR (*52*); and by Dawson and Gomes Correia which covers the practical range for subgrades in the UK (*53*). Equation (4) has been used as a basis for estimation of soil modulus in a number of design procedures.

Untreated Granular Material. The stiffness characteristics of untreated granular materials are dependent on the applied stresses. This stress dependency has been expressed in several ways for pavement design and analysis purposes (*53–56*):

$$E_{gran} = K_1 \sigma_3{}^n \tag{5}$$

$$E_{gran} = K_1 \theta^{k_2} \tag{6}$$

$$E_{gran} = F(\sigma'_m, \sigma_d) \tag{7}$$

where: E_{gran} = stiffness modulus; σ_d, σ_3 deviator stress and confining pressure in a triaxial compression test respectively; θ = sum of principal stresses in triaxial compression (σ_d+3σ_3); σ'_m =.mean normal effective stress (σ_d+3σ_3)/3; and, *K, n,* K_1, K_2 = experimentally determined coefficients.

Equation (6) can be used in an ad hoc manner in layered elastic analyses (*39*) and in finite-element idealizations (*25*) both of which are used for pavement design and analysis purposes. This equation stresses the importance of effective stress and deviator stress on resilient response. Moreover, data suggest that the ratio of those stresses is most influential (*58*). References (*56* and *57*) includes ranges in value for various granular materials used in pavement structures.

Researchers who have contributed to the development of these stiffness relationships include, but are not limited to the following: G. Dehlen, R.G. Hicks, and R.D. Barksdale.

The degree of compaction has a significant influence on the stiffness characteristics of compacted granular materials. Another major factor affecting the stiffness of these materials is water content (degree of saturation) since this is directly related to effective stress. Although the method of compaction is important for fine-grained soils because of soil structure considerations, method of compaction has a comparatively lesser effect on soil structure (*60*). Accordingly, any method of compaction (e.g., vibratory) that produces the desired degree of compaction is suitable at the appropriate water content may be considered suitable for laboratory preparation of specimens for testing.

An alternative approach is to test the aggregate in a dry state (when pore pressures are zero) and select design values from estimates of the suction based on anticipated worst case drainage conditions.

As with fine-grained soils, estimates of the stiffness characteristics of granular materials may be utilized. For example, the Shell investigators suggest that since granular materials will only sustain very small tensile stresses, the ratio of the modulus of the granular layer to that of the subgrade is limited to a range of about 2 to 3 (*37*).

Permanent Deformation, Unbound Materials

Rutting in paving materials develops gradually with increasing numbers of load applications, usually appearing as longitudinal depressions in the wheel paths accompanied by small upheavals to the sides. It is caused by a combination of densification (decrease in volume and, hence, increase in density) and shear deformation and may occur in any or all pavement layers, including the subgrade. For well compacted HMA, available information suggests that shear deformation rather than densification is the primary rutting mechanism (*61, 62*).

From a pavement design standpoint, a number of approaches have evolved to consider rutting. The first considers limiting the vertical compressive strain at the subgrade surface to a value associated with a specific number of load repetitions, this strain being computed by means of layered elastic analysis. The logic of this approach, first suggested by the Shell researchers, e.g. G. Dormon (*16*), is based on the observation that, for materials used in the pavement, permanent (plastic) strains are proportional to elastic strains.[5] By controlling the elastic strain to some prescribed value, plastic strains will also be limited. Integration of the permanent strains over the depth of the pavement section provides an indication of the rut depth. By controlling the magnitude of the elastic strain at the subgrade surface, the magnitude of the rut is thereby controlled.

An equation of the following form has been used to relate the number of load applications to vertical compressive strain at the subgrade surface:

$$N = A\left(\frac{1}{\varepsilon_v}\right)b \tag{8}$$

where: N = number of load applications; ε_v .= elastic vertical strain at subgrade surface; A, b = empirically determined coefficients.

This approach has been quantified by the back-analysis of pavements with known performance, but is semi-empirical in nature since it applies to a particular range of structures with particular materials under particular environmental conditions. Values of the coefficients have been derived for different locations and circumstances. For example, the value for the exponent b is in the range 0.22 to 0.27.

Analyses suggest that beyond about 50×10^6 ESALs, Equation (9) may exhibit a flatter slope (*63*). For long life pavements, it is likely that this change in slope should be recognized to avoid overly conservative thick structural pavement sections for repetitions in excess of 50×10^6 ESALs if the designer is making use of this riterion.

A number of investigations have suggested that an alternative approach to control rutting in the unbound layer is to limit the vertical compressive stress at the surface of that layer, e.g., Thompson (*64*) and Maree (*65*).

Fatigue Cracking, Asphalt Concrete

Considerable research has been devoted to fatigue cracking in HMA. Results of this research have demonstrated that the fatigue response of HMA to repetitive loading can be defined by relationships of the following form (66–*70*):

$$N = A\left(\frac{1}{\varepsilon_t}\right)^b \textbf{ or } N = C\left(\frac{1}{\sigma_t}\right)^d \tag{9}$$

---

[5] Laboratory test data on soils, aggregates, and asphalt mixes support this assumption [e.g., Reference (*62*)].

where: N .= number of repetitions to failure; ε_t .= magnitude of the tensile strain repeatedly applied; σ_t = magnitude of the tensile stress repeatedly applied; and, *A, b, C, d* = experimentally determined coefficients. The stress or strain parameters are those which occur on the underside of the asphalt-bound layer and cracking which might result is referred to as *bottom-up cracking.* In thick asphalt concrete layers *top-down cracking* has been observed. While not well defined, research is underway to develop methodology to consider the potential for this type of distress.

Organizations contributing early to this area included the University of Nottingham, e.g., the work of Pell, (*71*) and the University of California, Berkeley (*72*). Others have included: Shell, both in the Netherlands (KSLA) (*73*) and France (*74, 75*); The Laboratoire Central Ponts et Chaussees (LCPC) (*76*); the TRL in the UK (*77*); CSIR, South Africa (*78*); RR, Belgium (*79*) and the Asphalt Institute, U.S (*80*).

A number of factors influence fatigue response as measured in the laboratory including the mode of loading i.e, controlled stress or load and controlled strain or deformation (*66, 67*).

A design relationship utilized today by a number of organizations is based on strain and uses an equation of the form:

$$N = K\left(\frac{1}{\varepsilon_t}\right)^a\left(\frac{1}{S_{mix}}\right)^b \tag{10}$$

This expression may involve a factor that recognizes the influence of asphalt content and degree of compaction proportional to the following expression:

$$\frac{V_{asp}}{V_{asp}+V_{air}} \tag{11}$$

where: V_{asp} is the volume of asphalt and V_{air} is the volume of air. Data developed by a number of researchers (e.g., *68, 69*) have permitted the quantification of Equation (11), for example, in the Asphalt Institute design procedure (*40*).

Equation (11) is used in the Shell (*37*) and Asphalt Institute (*40*) procedures with the coefficients set according to the amount of cracking considered tolerable, the type of mixture that might be used, and the thickness of the asphalt-bound layer. A few examples of fatigue design relationships following the form of Equation (10) are shown in Table 2.

An alternative approach to define fatigue response makes use of the concept of dissipated energy suggested by Chompton and Valayer (*82*) and van Dijk (*73*). The form of the relationship between the load repetitions and dissipated energy is:

$$WD = AN^Z \tag{12}$$

where: WD = total dissipated energy to fatigue failure; N = number of load repetitions to failure; and A, z = experimentally determined coefficients.

A number of researchers have utilized Equation (12) in lieu of Equation (10) as the damage determinant. To do this requires the use of viscoelastic rather than elastic analysis, and as of this date has not been widely practiced (*83*).

In the pavement structure, the asphalt mix is subjected to a range of strains caused by a range of both wheel loads and temperatures. To determine the response under these conditions requires a cumulative damage hypothesis. A reasonable hypothesis to use is the linear summation of cycle ratios (sometimes referred to as Miner's hypothesis) (*67*). This was originally suggested by Peattie in 1960 based on his wok on the fatigue of metals. The linear summation of cycle ratios hypothesis is stated as follows:

$$\sum_{i=1}^{n} \frac{n_i}{N_i} \leq 1 \tag{13}$$

where: n_i = the number of actual traffic load applications at strain level *i,* and N_i = the number of allowable traffic load applications to failure at strain level *i.*

This equation indicates that fatigue life prediction for the range of loads and temperatures (*66*) anticipated becomes a determination of the total number of applications at which the sum reaches unity.

While not discussed in this paper, other asphalt-treated (emulsion) mixes make use of the same relationships (*40*). Cement and lime treated materials make use of either stress or strain relationships represented by equation (e.g. Reference [*84*]).

Permanent Deformation (Rutting) in Asphalt Concrete Layer(s).

Two methodologies are briefly described in this section. The first approach, originally suggested by Heukelom and Klomp (*34*) and Barksdale (*86*) and used by MacLean (*62*) and Freeme (*87*), makes use of elastic analysis to compute stresses within the asphalt-bound layer together with constitutive relationships that relate the stresses so determined to permanent strain for specific numbers of stress repetitions (termed the layered-strain procedure). Integration or summation of these strains over the layer depth provides a measure of the rutting that could develop.

One version of this approach was developed by the Shell researchers and has been used in modified form to evaluate specific pavement sections (*37*). In this methodology, creep test results are incorporated into the following expression to estimate rutting:

$$\Delta h_1 = C_m \tag{20}$$

$$\Delta h_1 = C_m \sum_{i=1}^{n} \left[h_{1-i} \cdot \frac{(\sigma_{ave})_{1-i}}{(S_{mix})_{1-i}} \right] \tag{21}$$

where: Δh_1 permanent deformation in the asphalt-bound layer; h_{1-i} = thickness of sublayer of asphalt-bound layer with thickness; $(\sigma_{ave})_{1-i}$ = average vertical stress in layer; and $(S_{mix})_{1-i}$ = mix stiffness for layer for specific temperature and time of loading (obtained by summing the individual times of loading of the moving vehicles passing over that layer at the specific temperature).

Although this procedure is not sufficiently precise to predict the actual rutting profile due to repeated trafficking, it provides an indication of the relative performance of different mixes containing conventional asphalt cements. If it is planned to use mixes containing modified binders, use of creep test data for these mixes in Equation (15) will not provide correct estimates of mix performance since mixtures containing these binders behave differently under loading representative of traffic as compared with their behavior in creep. For example, the work of the Shell investigators suggest that use of creep test data may over predict rutting for mixes containing some modified binders (*88*). To consider the effects of stresses of different magnitudes on the development of rutting, which result from variations in traffic loads and environmental condition, a cumulative damage hypothesis is required, just as for fatigue. A "time-hardening" procedure suggested by Freeme (*87*) appears to provide a reasonable approach. The procedure used in the MEPD makes use of this approach (*30*).

The second approach was developed by Deacon and used to analyze the results of rutting in 34 of the 36 WesTrack test sections (*89*). In this methodology, the pavement is represented as a multilayer elastic system and the analysis consists of determining three parameters, τ, γ^e, and ε_v[6] on an hour-by-hour basis. Measured temperature distributions are used to define the moduli of the asphalt concrete which was subdivided into a number of layers from top to bottom with thicknesses 25 mm (1 in.), 50 mm (2 in.) for the first and second layers, and convenient thicknesses for the remainder of the asphalt concrete layer to simulate the effects of temperature gradients on mix stiffness.

In this model, rutting in the asphalt concrete is assumed to be controlled by shear deformations. Accordingly, the computed values for τ and γ^e at a depth of 50 mm (2 in.) beneath the edge of the tire were used for the rutting estimates. Densification of the asphalt concrete is excluded in these estimates since it has a comparatively small influence on surface rutting if the asphalt concrete layer is compacted to an air-void content not exceeding 8 percent.

In simple loading, permanent shear strain in the AC is assumed to accumulate according to the following expression:

$$\gamma^i = a \cdot \exp(b\,\tau)\gamma^e n^c \tag{22}$$

where: γ^i = permanent (inelastic) shear strain at 50 mm (2 in.) depth; τ = shear stress determined at this depth using elastic analysis; γ^e =corresponding elastic shear strain, n= number of axle load repetitions; a, b, c = regression coefficients.

[6] τ, γ^e =.elastic shear stress and strain at a depth of 50 mm (2 in.) below outside edge of tire
ε_v = elastic vertical compressive strain at the subgrade surface

Table 2 Design Fatigue Relationships

Design Method	Design Relationship	Equation	Remarks
NCHRP1-10B [Finn et al. (*85*)]	Greater than 45 percent wheel path cracking: $\log_{10} N(\geq 45\%) = 16.086 - 3.291 \log_{10}\left(\frac{\varepsilon_t}{10^{-6}}\right) - 0.854 \log_{10}\left(\frac{S_{mix}}{10^3}\right)$	(14)	ε_t = tensile strain in./in. × 10^{-6}
	For less than 10 percent wheel path cracking: $\log_{10} N(\leq 10\%) = \left(\frac{\log_{10} N(\geq 45\%)}{1.4}\right)$	(15)	S_{mix} = mix stiffness, psi
The Asphalt Institute (*40*)	$\left[\log_{10} N(\geq 45\%)\right] \cdot C$	(16)	Makes use of NCHRP 1-10B equation for 45 percent in more wheel path cracking modified by factor C to reflect the effect of asphalt content and air-void content
	$C = 10^M$	(17)	
	$M = 4.84\left(\frac{V_{asp}}{V_{asp} + V_{air}} - 0.69\right)$	(18)	
Shell International (*37, 91*)	$N = SF(K_{1r})\alpha\left(\frac{1}{\varepsilon_t}\right)\left(\frac{1}{S_{mix}}\right)$	(19)	
	$(K_{1r}) = f$ (mix volumetrics, pen. index of asphalt)		
	$\alpha = f$ (AC thickness)		
	$K_3 = f$ (AC thickness)		

The time-hardening principle is used to estimate the accumulation of inelastic strains in the asphalt concrete for in-situ conditions. The resulting equations are as follows:

$$\gamma_1^i = a_1\left[\Delta n_1\right]^c \tag{23}$$
$$a_j = a \cdot \exp(b\tau)\gamma_j^e \tag{24}$$
$$\gamma_j^i = a_j\left[\left(\gamma_{j-1}^i / a_j\right)^{(1/c)} + \Delta n_1\right]^c \tag{25}$$

where: $j = j^{th}$ hour of trafficking; γ_j^e .= elastic shear strain at the j^{th} hour; Δn = number of axle load repetitions applied during the j^{th} hour.

Rutting in the AC layer due to the shear deformation is determined from the following:

$$rd_{AC} = K\gamma_j^i \tag{26}$$

For a thick asphalt concrete layer, the value of K has been determined to be 10 when the rut depth (rd_{AC}) is expressed in inches (*61*).

To estimate the contribution to rutting from base and subgrade deformations, a modification to the Asphalt Institute subgrade strain criteria, i.e., a modification of Equation (8), can be utilized (*89*).

MECHANISTIC-EMPIRICAL PAVEMENT DESIGN

Currently, there are many mechanistic-empirical (analytically-based) design procedures which have been developed. Some, while not used, have served as the basis for other procedures. Several such procedures are briefly summarized in Reference (31) and a few are listed Table 3. Figure 1 illustrates a simplified framework which the procedures generally follow. All the procedures idealize the pavement structure as a multilayer elastic or viscoelastic system using programs like those described in Table 1.

While the procedures listed in Table 3 all received impetus from the 1962 Conference (as noted earlier), the U.S. Navy was using a pavement design procedure in the 1950's for airfield pavements which incorporated results of Burmister's solution for a two-layer elastic solid. A plate bearing test was used to measure the subgrade modulus and the thickness required was based on the requirement that the computed surface deflection not exceed 5 mm (0.2 in.) for the specific aircraft.

The procedures listed in Table 3 all consider the fatigue and rutting modes of distress in establishing pavement structures. Fatigue estimates are based on relationships like those shown in Table 4 and on subgrade strain or stress criteria. Some procedures utilize a layer strain procedure to estimate surface rutting contributed by the individual layers.

The linear sum of cycle ratios cumulative damage hypothesis is used in the majority of the methods to assess the effects of mixed traffic and environmental influences on fatigue cracking. Those procedures using a subgrade strain procedure incorporate a form of the linear sum of cycle ratios (based on compressive strain) for the same purpose. A few of the methods make use of the time-hardening procedure to estimate the cumulative effects of traffic and environment on rutting in the asphalt concrete (e.g., the Shell International method and the MEPDG). A study currently underway in Project NCHRP 9-30A is considering modifications to the rutting estimate procedure in the MEPDG. The Illinois DOT procedure includes the use of the fatigue endurance for full-depth asphalt pavements. This represents an excellent example of University (Illinois) researchers working with their DOT to arrive at a practical yet sound technical basis design methodology

Some of the engineers associated with the development of these methods are : P. Visser (Shell); M. Witczak, L. Santucci, J. Shook (Asphalt Inatitute); P.Leger, R.Sauterey, J. Bonnot (LCPC); S. Kuhn, N. Walker, H. Maree (South Africa); M. Witczak (MEPDG),.M. Thompson and S. Carpenter (IDOT).

Table 3 Examples of Analytically Based Design Procedures

Organization	Pavement Representation	Distress Modes	Environmental Effects	Pavement Materials	Design Format
Shell International Petroleum Co., Ltd., London, England (*37, 88, 90, 91*)	Multilayer elastic solid	fatigue in treated layers; rutting: · subgrade strain · estimate in asphalt bound layer	temperature	asphalt concrete, untreated aggregate, cement stabilized aggregate	design charts; the computer program BISAR is used for analysis
The Asphalt Institute, Lexington, KY (MS-1, MS-11, MS-23) (*40, 92, 93*)	Multilayer elastic solid	Fatigue in asphalt treated layers; Rutting: · subgrade strain	Temperature, freezing and thawing	Asphalt concrete, asphalt emulsion, treated bases, untreated aggregate	Design charts; computer program DAMA
Laboratoire Central de Ponts et Chaussées (LCPC) *(94, 95)*	Multilayer elastic solid	Fatigue in treated layers; rutting	Temperature	Asphalt concrete, asphalt-treated bases, cement stabilized aggregates, untreated aggregates	Catalogue of designs; computer program (ELIZE) for analysis
National Institute for Transportation and Road Research (NITRR) South Africa (*96, 97, 98*)	Multilayer elastic solid	Fatigue in treated layers; rutting: · subgrade strain · shear in granular layers	Temperature	Gap-graded asphalt mix, asphalt concrete, cement-stabilized aggregate, untreated aggregate	Catalogue of designs; computer program
Austroads (*99*)	Multilayer elastic solid	Fatigue in treated layers; rutting: · subgrade strain	Temperature, moisture	Asphalt concrete, untreated aggregates, cement stabilized aggregates	Design charts, computer program CIRCLY
AASHTO MEPDG (*30*)	Multilayer elastic	Fatigue in treated layers; rutting: · subgrade strain · asphalt concrete, time hardening Low temperature cracking	Temperature, moisture	Asphalt concrete, untreated aggregates, chemical stabilized materials	Computer program JULEA
IDOT *(100)*	Multilayer Elastic i	Fatigue in untreated layers; rutting: stress at subgrade surface	Temperature, moisture	Asphalt concrete, untreated aggregates, lime stabilized subgrade may be used depending on soil type	Design Charts based on M-E design methodology. AC fatigue endurance limit used to set thickness for AC layer

ACCELERATED PAVEMENT TESTING

As discussed earlier, engineers have utilized accelerated pavement tests as well as observations of performance of test roads and in-service pavements to calibrate, validate, and modify, if required, their procedures for the design of new and rehabilitated pavements. Examples included work developed by the USACE-WES and the TRL in the UK.

In the U.S., a number of test roads have been conducted under the aegis of the Transportation Research Board of the National Research Council, and the FHWA. Three that are of particular import are the WASHO Road Test (*5*), the AASHO Road Test (*6*), and WesTrack (*132*). The results of the AASHO Road Test have had a significant impact on pavement design, as already discussed earlier.

WesTrack, a federally-funded multi-million dollar hot-mix asphalt (HMA) located near Reno, Nevada, was completed in 2000. Its purpose was to further the development of performance-related specifications (PRS) technology and to provide field verification of the SHRP-developed Superpave asphalt mix design procedure. A series of asphalt mixes covering a range of aggregate gradations, asphalt contents, and degrees of compaction (35 sections in total) were evaluated (*101*).

Results from this test road provided important information on the effects of mix and construction variables on pavement performance. It has contributed to the concept of combining mix design and pavement design described in the previous sections and to the formulation of pay factors for use in performance-related specifications for hot mix asphalt construction (*89*).

Accelerated pavement testing (APT) has been an integral part of the development of the CBR procedure by the USACE. When this procedure was first developed, results of accelerated load tests at thirteen different locations were instrumental in establishing the initial thickness versus CBR relationships (*2*). The multiple wheel heavy gear load (MWHGL) tests of the 1960's played a significant role in establishing the methodology of the current procedure for airfield pavements (*3*).

The concept of subjecting in-service pavements to accelerated loading was successfully promulgated by the Council of Scientific and Industrial Research (CSIR) of South Africa beginning in the 1970's. They developed a series of Heavy Vehicle Simulator (HVS) units which could be used to test in-service pavements or specially developed test sections. The results of the tests using the HVS equipment together with laboratory test programs and pavement analyses have been successful in improving pavement technology including pavement construction practices. Early acceptance of analytically-based design in South Africa was assisted by the results of the HVS test program (*102*).

The success of the APT program in South Africa led to the development of an accelerated test unit in Australia termed the Accelerated Loading Facility (ALF) and a successful test program using the equipment to evaluate a range of paving materials (*103*).

The ALF technology was adopted by the FHWA and is currently being used at the FHWA Turner Fairbank Highway Research Center (TFHRC) to validate the binder specifications developed during the SHRP endeavor to control both permanent deformation and fatigue cracking in asphalt pavements (*104, 105*).

With the demonstrated success of APT in South Africa and Australia, a number of agencies both in the U.S. and abroad have developed their own capabilities. Currently, the states of Louisiana, Texas, California, Kansas, and Indiana have APT units. Reference (*103*) provides an excellent summary of the state of APT throughout the world as of 1996.

APT, to be effective, must be used in conjunction with both pavement analyses of the type described earlier and laboratory test programs (*106*). An example of this is described in Reference (*107*).

NON-DESTRUCTIVE PAVEMENT TESTING

A key element for pavement rehabilitation, particularly overlay design, is the non-destructive evaluation of the existing pavement. The Benkelman Beam, developed during the WASHO Road Test (*5*) by A.C. Benkelman has played a major role in the evolution of overlay design since it provided an inexpensive and reliable device to measure the surface deflection of a pavement under a standard load representative of actual traffic.

Mechanization of the Benkelman Beam concept to accelerate field deflection measurements was accomplished by the French with the introduction of the LaCroix Deflectograph (*108*) and by Hveem with the Traveling Deflectometer (*109*). As noted above, this type of equipment has played a major role in overlay design for asphalt pavements.

Notable among the methods (*110*) are those developed by the Asphalt Institute (Benkelman Beam measured deflections) (*111*), the TRRL procedure developed by N.W. Lister Benkelman Beam and TRRL Deflectograph deflections)[7] (*112*), and the State of California Procedure (Benkelman Beam and Traveling Deflectometer deflections) (*109*). The work of Lister is particularly noteworthy in this regard in that he introduced the concept of probability of achieving a given life in the overlay with thickness requirements based on probabilities of 50 and 90 percent of achieving the design life.

In addition to the devices noted above which measure pavement response to slowly moving loads, equipment has been introduced over the years that utilizes steady-state vibratory equipment and falling weight (impulse) loading equipment Some are listed in Table 4.

The vibratory equipment introduced by the Shell investigators (van der Poel and Nijboer) [e.g., Reference (*115*)] to measure dynamic pavement deflections was later extended to wave propagation measurements by Heukelomp and Klomp (*116*) and Jones and Thrower of the TRL (*117–119*). The work by Jones and Thrower is particularly important in that they used different vibratory equipment over a range in frequencies to measure waves of different types compression, shear, Rayleigh, and Love waves) and developed methodology to determine which type of wave was being measured. This, in turn, permitted estimates of both shear (G) and elastic (E) moduli of the various layers of a pavement system.

Currently, the falling weight deflectometer (FWD) equipment is used extensively for surface deflection measurements as a part of overlay pavement design. Various

[7] Also used for a number of years in South Africa by the CSIR (*113*) and in Australia (*114*).

analytical procedures to interpret the resulting measurements, referred to as back-calculation procedures, have been developed; examples include EVERCALC (*122*) and ELMOD (*123*). The FWD equipment, originally developed by the French, received impetus for general use by Danish and Shell investigators (e.g., Reference [*121*]). Table 5 lists some of the analytically-based (mechanistic-empirical) overlay design procedures developed over the years. It will be noted that the majority make use of the FWD.

Table 4. Examples of Deflection Measuring Devices and Wave Propagation Equipment

Load Application	Device	Remarks
Slowly moving wheel load (dual tires)	Benkelman Beam (deflection beam)	Developed at WASHO Road Test
	Traveling Deflectometer (California)	Developed by F.N. Hveem in California
	Deflectograph	Developed in France; used e.g., in South Africa, UK, and Australia
Vibratory load, steady state	Dynaflect	Developed in Texas
	Heavy Vibrator [U.S. Army Corps of Engineers, Waterways Experiment Station (WES)]	Developed at WES for airfield pavement evaluation (*120*)
	Heavy Vibrator	Developed by Shell
	Vibratory equipment for wave propagation; mass dependent on frequency	TRL, Shell , Texas
Falling weight (impulse load)	Falling weight deflectometer (FWD)	Developed in France and Denmark

PAVEMENT MANAGEMENT SYSTEMS

While engineers have managed their pavement systems in modern times, performing periodic maintenance and rehabilitation operations when deemed appropriate and tempered by available funds, it was not until the late 1960s that the concept of pavement management systems (PMS) was introduced. In the United States, this occurred under the aegis of the National Cooperative Highway Research Program (NCHRP) Project 1-10 with F.N. Finn, Finn and W.R. Hudson as the principal researchers (*125*). At about the same time, R. Haas initiated work in this area for the Canadian Provinces. Subsequently (1977) Hudson and Haas collaborated to provide the first textbook on pavement management systems (*126*).

The first state DOT in the United States to embrace the PMS concept was the State of Washington Highway Department. The State Materials Engineer, R. LeClerc, working with F.N. Finn and his associates, developed a system to manage the entire state highway network (*127, 128*). This included:

1. the introduction of condition surveys to be done on a systematic and continuous basis, and which included measures of surface distress, ride quality, and skid resistance,
2. development of pavement performance relationships,
3. establishment of levels of performance requiring maintenance and rehabilitation, and
4. life cycle cost analysis to permit effective use of existing funds.

The first truly network-level system was developed for the Arizona Highway Department in the 1970s by F.N. Finn and his associates working with G. Morris, the Arizona Research Engineer (*129*). The system was formulated to:

1. estimate costs to bring the network to and maintain it at some desired level of serviceability, or
2. in the face of budget constraints, estimate resulting serviceability's associated with the specific budget.

To accomplish this, an optimization model was utilized which based the formulation of the problem as a Markovian decision process and its conversion into a linear program, a remarkable advance for the time (*130*).

Since that time, pavement management systems have become a regular part of the activities of departments responsible for street and highway systems internationally. These systems provide the basis for maintaining entire networks at established levels of service commensurate with the designated functions of each of the segments of the system, e.g., interstate versus secondary routes.

In addition to the role of resource allocation, pavement management systems permit linking performance to the following databases: initial design: materials and pavement sections; construction, including as-built pavement sections, QC/QA data; traffic data; and environmental data.

Reference (*131*) provides an example of such a system in which the database with properties of mixes developed by the Superpave mix design have been linked to the pavement sections in the pavement management system so that their performance can be evaluated. This approach permits development of improved design and rehabilitation methodology and improved construction procedures. In effect, by this linking, each highway network becomes a long-term pavement performance project. For the MEPDG and newly developed new pavement design systems, this linkage will be extremely important for their validation.

IMPROVED CONSTRUCTION PRACTICES

As traffic and loadings continue to increase on our street and highway, airfield, and port and cargo transfer pavements, it is imperative that construction practices be improved to keep up with the demand. Results of mechanistic-empirical analyses allow engineers to establish such requirements. For example, References (*132* and *133*) contain data which illustrate the quantitative impacts of improved compaction and thickness control on fatigue cracking on HMA pavements By improving compaction (lower air-void contents) and reducing variability in air-void content (as measured by the standard deviation), longer pavement lives result. Similarly, the influence of thickness and its variability on affects fatigue performance; or a given target thickness, the lower the standard deviation in as constructed thickness, the better the performance. Moreover, the relative effects of variances in mix and thickness due to construction and that associated with testing can also be considered (*134*).

SOME THOUGHTS TO CONSIDER

From this brief discussion, it is apparent that considerable progress has been made in the methodology for the design and construction of long lasting pavements. This has resulted from the cooperation of many people at the international level, with those involved freely sharing their knowledge and experience to advance the field of Pavement Engineering. Sufficient evidence has been presented to conclude that technology is available to properly design and construct excellent performing asphalt pavements. However, *well educated people at all levels must be available.* This requires up-to-date education and training for engineers, technicians, and construction and maintenance personnel. As demonstrated herein, *Pavement Engineering is a "high-tech" profession!* In the U.S. at least, there is a growing concern that in the majority of Civil Engineering programs in Universities and Colleges, this premise is understood by many current faculty and that education in Pavement Engineering at both the undergraduate and graduate levels has been diminishing

A strongly recommended solution is to educate faculty as well as students. In the period 1956 to about 1965, the Asphalt Institute supported summer programs at a number of major Universities to educate Faculty in both asphalt pavement technology and pavement design and rehabilitation. Currently, an excellent example is the National Center for Asphalt Technology (NCAT) Instructor Training Course programs of this type, supported by Industry, can contribute to alleviating the faculty problem.

With the advent of web-based educational developments, *self-managed learning* programs like those developed by Professors J.P. Mahoney and S. Muench at the University of Washington provide up-to-date information for people new to the field as well as for more experienced personnel. An excellent example is the "Guide for Hot Mix Pavements"(*135*), which can be viewed at the following website:
http://hotmix.ce.washington.edu/wsdot_web/WSDOT_intro.htm

Programs like this can be used as prerequisites for other forms of education and training. Such activities include both short courses developed through Technology Transfer (T^2) Centers; certifications, e.g., for construction personnel, and testing technicians.

In conclusion, while there are still gaps in our knowledge, the pavement engineering community has made great strides since 1960 It is the responsibility of those who work in the pavements area to apply these developments with good engineering judgment.

Table 5. Examples of Analytically-Based Overlay Design Procedures

Procedure	Non-destructive Pavement Evaluation	Stiffness Modulus Determinations		Analysis Program	Distress Mechanisms		Considerations of Remaining Life, Existing Pavement	Overlay Thickness Determination	Remarks
		Back-calculations	Lab Testing		Fatigue	Rutting			
Shell (*121*)	FWD	Yes	No	BISAR	Yes	Yes	Yes	Overlay thickness selected to (a) limit fatigue and (b) limit rutting for anticipated traffic; thickness also selected assuming existing pavement is cracked	
Washington DOT EVERPAVE (*122*)	FWD	Yea	No	EVERCALC	Yes	Yes	No	Overlay thickness selected to (a) limit fatigue and (b) limit rutting for anticipated traffic; asphalt concrete assigned different stiffness values depending on condition	
Austroads *(99)*	FWD or Benkelman Beam	Yes	No	EFROMD2 (Based on CIRCLY)	Yes	Yes	Yes	Overlay thickness selected to (a) limit fatigue and (b) limit rutting for anticipated traffic; asphalt concrete assigned different stiffness values depending on condition	Two alternatives are available: • General mechanistic procedure (GMP)—extension of procedure for new pavements, • Ausroads simplified mechanistic overlay procedure (ASMOL)
University of Nottingham, UK (*124*)	FWD	Yes	Yes	PADAL	Yes	Yes	Yes	Overlay thickness selected to (a) limit fatigue and (b) limit rutting for anticipated traffic; asphalt concrete assigned different stiffness values depending on condition	

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APPENDIX
ENGINEERS MENTIONED IN THE TEXT

R. Ahlvin, Waterways Experiment Station, US Army Corps of Engineers
R. D. Barksdale, Georgia Institute of Technology
A. C. Benkelman, Highway Research Board
J. Bonitzer, L.C.P.C., France
J. Bonnot, L.C.P.C., France
S. F. Brown, University of Nottingham
J. E. Buchanan, The Asphalt Institute
D. M. Burmister, Columbia University
W. N. Carey, Jr., Highway Research Board
D. Croney, Road Research Laboratory, U.K.
G. Dehlen, CSIR, South Africa
G. M. Dormon, Shell U.K.
F. N. Finn, The Asphalt Institute
C. R. Freeme, CSIR, South Africa
Sir William Glanville, Road Research Laboratory, U.K.
W. Goetz, Purdue University
R. C. G. Haas, University of Waterloo, Canada
W. Heukelom, Royal Dutch Shell Laboratory, Netherlands
R. G. Hicks, Oregon State University
W. S. Housel, University of Michigan, Ann Arbor
W. R. Hudson, University of Texas, Austin
F. N. Hveem, California Division of Highways
J. Kirk, Danish Road Institute
A. J. G. Klomp, Royal Dutch Shell Laboratories, Netherlands
R. LeClerc, Washington State Highway Department
N. W. Lister, R.R.L., U.K.
H. Maree, CSIR, South Africa
N. W. McLeod, Imperial Oil Company, Canada
J. K. Mitchell, University of California, Berkeley
E. Nakkel, Federal Government, Germany

N. W. Nijboer, Royal Dutch Shell Laboratory, Netherlands
Ward K. Parr, University of Michigan, Ann Arbor
P. S. Pell, University of Nottingham
P. Rigden, CSIR, South Africa
L. Santucci, Chevron Research Corporation
R. Sauteray, L.C.P.C., France
H. B. Seed, University of California, Berkeley
J. Shook, The Asphalt Institute
M. R. Thompson, University of Illinois
W. Turnbull, Waterways Experiment Station, US Army Corps of Engineers
J. Verstraeten, R.R.L., Belgium
P. Visser, Royal Dutch Shell Laboratory, Netherlands
N. Walker, CSIR, South Africa
K. Wester, Dutch Study Center
M. Witczak, University of Maryland
E. J. Yoder, Purdue University

Shear Wave Velocity Profiling with Surface Wave Methods

Soheil Nazarian[1], F.ASCE

[1]Professor, Department of Civil Engineering and Director, Center for Transportation Infrastructure Systems, The University of Texas at El Paso, El Paso, TX 79968-0516, nazarian@utep.edu

ABSTRACT: Surface wave method was introduced as a tool to the geotechnical and infrastructure engineering fields in the early 1980's. Since then, the method has been continuously modified and improved. The adoption of the method has accelerated in the last ten years because of the interest of the engineering community, and due to the development of affordable and portable hardware and software. Despite numerous studies that demonstrate its effectiveness in a wide spectrum of applications, the method has not been fully embraced by the engineering community mainly due to lack of standardization. The common attributes of different approaches in implementing the surface method from the point of view of the engineering applications are discussed, followed by the practical and theoretical strengths and limitations of alternative approaches. Different approaches should provide satisfactory results as long as the inherent limitations of each approach are matched with the requirements of the engineering objectives of a given project.

INTRODUCTION

The main application of seismic methods has been shear wave velocity (VS) profiling for geotechnical earthquake engineering analyses. This application has become more widespread with the mandate for the seismic site classification based on the average shear wave velocity for the top 30 m of soil (a.k.a. VS30) in many codes such as the Uniform Building Code (UBC). The application of seismic wave measurements has been expanded significantly to other areas in the past two decades. In his Terzaghi lecture, Stokoe (2011) eloquently elaborates a number of applications that positively impact the evaluation and design with and on geomaterials. These applications range from estimating liquefaction potential (Andrus et al., 2004), to advanced three dimensional site characterization (Park et al., 2004), to construction quality control (Nazarian et al., 2002), and to anomaly detection (Gucunski et al., 1998).

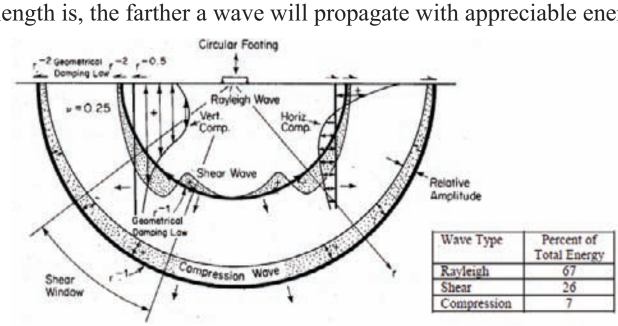
A large number of invasive and noninvasive techniques are available for estimating the VS profile at a given site (e.g., see Stokoe and Santamarina, 2000). For the sake of brevity these methods are not discussed here. The focus of this paper is on the state of the art in VS profiling with the surface wave method. The intended audience of this paper is the practicing engineers who are not experts in geophysical engineering.

Several different approaches have been introduced in the last three decades to implement the surface wave method. The most common approaches are the Spectral

Analysis of Surface Waves (SASW, Nazarian and Stokoe, 1985), Multichannel Analysis of Surface Waves (MASW, Park et al., 1999) and Refraction Microtremor (ReMi, Louie, 2001). Whether or when to use the surface wave method in a given engineering problem should be driven primarily by the technical requirements of the project, the sensitivity of the design process to the VS profile, and secondarily by the cost and time constraints. Depending on these factors, all, some or none of the approaches named above may be appropriate for a specific project. In the same context, the testing configuration and analysis approach have to be adjusted based on the range of depths of interest and the lateral uniformity of the site. To that end, the fundamentals of the surface wave method are introduced first, followed by different steps necessary to implement the approaches properly. The practical limitations and best practices for each step are then described.

FUNDAMENTAL PRINCIPLES OF SURFACE WAVES

The fundamental understanding of the propagation of surface waves goes back to more than a century ago (Lamb, 1904). A schematic of the propagation of waves in a geomedium is shown in Fig. 1. When steady-state (sinusoidal wave) energy is coupled to the surface of a medium, shear, compression, and Rayleigh-type surface waves are generated. In surface wave testing, the shear or compression wave energy is undesirable. When the energy is imparted in a vertical direction and the propagation of waves is monitored with receivers that are oriented vertically, the energy associated with the shear and compression waves is small after a reasonable source-receiver distance. Fig. 1 also implies that surface Rayleigh waves propagate along a cylindrical front with limited depth of penetration in the medium while the shear and compression waves propagate over a spherical front within the body of the medium. The effective depth that the surface wave penetrates and the horizontal distance that the wave propagates with appreciable energy are directly related to the wavelength of the wave generated. As a first approximation, the depth of penetration of the vertical component of surface wave is about the wavelength of the steady state energy imparted to the medium. As the wave propagates away from the source in the horizontal direction, its energy dissipates due to geometric and material damping (Rix et al., 2001). The longer the wavelength is, the farther a wave will propagate with appreciable energy.



Wave Type	Percent of Total Energy
Rayleigh	67
Shear	26
Compression	7

FIG. 1 – Wave Motion in a Half-Space from a Sinusoidal Surface Vibration (from Richart et al., 1970)

Other parameters can be estimated from surface wave velocity, V_R, by measuring or estimating the Poisson's ratio, ν, and total mass density, ρ (Richart et al., 1970). For example, shear modulus, G, can be estimated from

$$G = \rho\,[(1.13 - 0.16\upsilon)\,V_R]^2 \tag{1}$$

For an elastic half-space, V_R is a constant value and independent of wavelength, if the damping properties of the layer are ignored. When the geomedium contains layering the measured surface wave velocity varies with wavelength or frequency. The measured velocities in layered systems are usually not equal to V_R of any particular layer, but rather the apparent travel time of the surface wave divided by the horizontal distance it travels. This apparent velocity is called phase velocity, V_{ph}. A plot of variation in the phase velocity with frequency or wavelength is called a dispersion curve. The schematics of dispersion curves for a multi-layer geomedium shown in Fig. 2a are presented in Figs. 2b and 2c. These two figures contain the same information since the frequency, f, and wavelength, λ, are inversely proportional as per relationship $\lambda=V_{ph}/f$. Fig. 2b provides more detailed information about the deeper features of the site, while Fig. 2c provides more detail about the near surface features. A number of engineers prefer to inspect the dispersion curve based on wavelength as they perceive a fraction of wavelength as a surrogate for depth.

Surface waves propagate at several velocities at a given wavelength or frequency, with each velocity corresponding to a different mode of propagation. The heavy line in Fig. 2b or 2c is called the fundamental mode, and the others the higher modes. In most geotechnical applications, the fundamental mode exists for the entire frequency range of interest, while higher modes have cut-off frequencies, below which they do not propagate. The state of practice in surface wave testing usually considers only the fundamental mode of propagation, but higher modes may be considered to improve the interpretation (Calderón-Macías and Luke, 2010). The parameters that impact the shape of a dispersion curve are the shear wave velocity, mass density, and Poisson's ratio of each layer. Since the last two parameters impact the shape of the dispersion curve rather insignificantly (Nazarian and Stokoe, 1985), these values are typically assumed. The damping properties of each layer also impact the shape of the dispersion curves (Rix et al., 2001). However, their impact is either ignored or presumptive values are assigned to them (Stokoe and Santamarina, 2000).

An ideal dispersion curve can be divided into the following three critical zones as marked in Fig. 2b:

- An almost constant velocity region at shorter wavelengths (higher frequencies) that contains mainly the information about near surface VS (Zone 1)
- A transitional wavelength/frequency dependent region that contains combined information about the near surface and all of its underlying layers (Zone 2)
- An almost constant velocity region that contains information about the deepest strata of the site (Zone 3).

The dispersion curve is considered as the raw data for VS profiling. As such, one of the areas of intense research has been in recommending accurate and robust protocols to measure the dispersion curve. Most differences among SASW, MASW, ReMi etc. stem from this goal. Irrespective of the approach considered, there are common best practices that should be used to obtain the dispersion curve as discussed next.

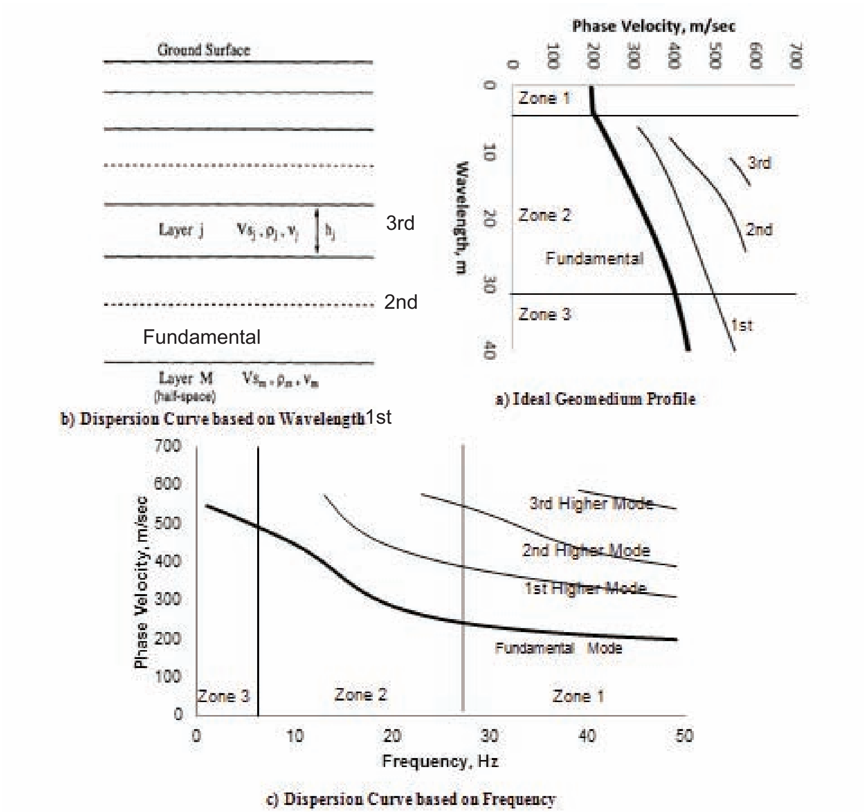


FIG. 2 – Example Dispersion Curves for a Typical Geotechnical Site (after Calderón-Macías and Luke, 2010)

FIELD TESTING PROCEDURES

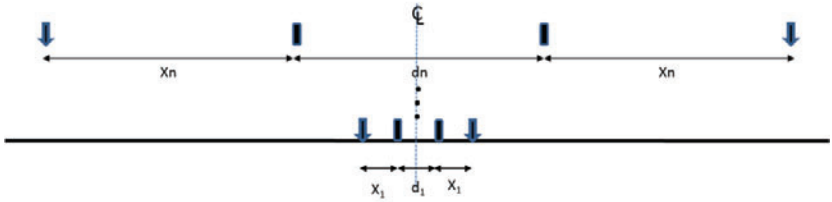
The main components of the field setup consist of a source, two or more receivers and a data acquisition device. These components should be selected carefully to ensure that the desired results are provided to the end user. The main requirement of the source is to couple surface wave energy over a range of frequencies so that ideally the three zones of the dispersion curve are defined. The source can be passive or active. The passive source often corresponds to the random vibration generated within the earth or environmental noise (wind, vehicular vibration etc.). Passive sources are especially attractive for deep soundings (Cox and Beekman, 2011). Since these sources usually do not contain high-frequency energy, field measurements may be supplemented with active sources to develop a complete dispersion curve.

The active sources include impulsive, steady state sweep using a vibratory device, or random vibration. The impulsive sources, especially sledge hammers, are the most common. A large-size sledge hammer typically couples adequate energy in the range of frequencies of 15 Hz to 100 Hz translating to a maximum effective depth of profiling of about 10 to 15 m (Li, 2008). For depths of investigations of up to 60 m, drop weights (either custom-made or improvised such as a bucket filled with concrete) have been used. Accelerated drop weights are also becoming popular. The vibratory sources are excellent for surface wave testing since they couple well-defined energy into the geomedium. Portable (up to 40 kg) shakers typically yield high-quality dispersion curves to a limited depth (less than 10 m). Customized vibroseises (Stokoe et al., 2004) can extend the depth of survey to 250 m and deeper. Active random vibration sources such as a bulldozer in-line with the sensor array have also been used in many investigations to extend the depth of investigation. Since most sources that extend the depth of investigation lack high-frequency energy, they should be used in conjunction with a high-frequency source.

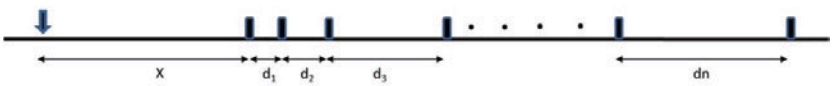
Velocity transducers (geophones) are by far the most common receivers. Important characteristics of a geophone for a proper field measurement are its natural frequency and its sensitivity. Geophones with natural frequencies close to 1 Hz are typically desirable for depths of survey greater than 30 m. The use of geophones with natural frequencies greater than 5 Hz is discouraged for routine geotechnical applications. The higher the sensitivity of a geophone is, the lower the magnitude of vibration that can be measured with certainty. The characteristics of all receivers used in the array should be as identical as possible, and the coupling of the receivers to the geomedium should be uniform and intimate. With the advancement in the software and hardware, most commercial data acquisition systems are appropriate for surface wave measurements as long as the acquisition parameters are selected correctly.

Typical source-receiver configurations used to acquire time records are shown in Fig. 3. The parameters of interest are the source-to-first-receiver spacing (a.k.a. source offset), X, and receiver spacings, d_1 through d_n. These parameters should be carefully selected to obtain the dispersion curve in the range of interest, to avoid the temporal and spatial aliasing in the analysis of the data, and to ensure the fidelity of the signal in the farther receivers (Zywicki and Rix, 2005). In the traditional SASW approach, two receivers are used at a time as shown in Fig. 3a (Nazarian and Stokoe, 1985). Several pairs of time records are collected by progressively doubling the distance between the receivers. The shortest spacing between the receivers is typically equal to the shallowest depth of investigation and the longest spacing is roughly equal to the maximum depth of interest for profiling. A source offset equal to the distance between the two receivers is suggested. The common receiver mid-point array, where the two receivers are moved symmetrically between measurements against a center point and two sets of data are collected by using the source on both sides of the array, is recommended. This is done as a measure to account for the dipping layers and to decrease the uncertainty in the final dispersion curve. The source is changed as the distance between the receivers increases to ensure that the energy in the appropriate range of frequencies is generated and detected for all receivers.

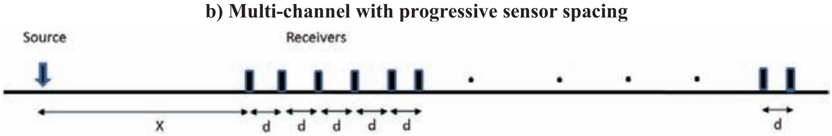
By today's standards, this process is tedious and time consuming because it was developed when even a two-channel data acquisition system was prohibitively expensive. With currently available data acquisition systems, the configuration can be modified so that many time records can be collected simultaneously based on the sensor configuration shown in Fig. 3b. Zywicki and Rix (2005) describe how to optimize the receiver spacings.



a) Traditional SASW



b) Multi-channel with progressive sensor spacing



c) Multi-channel with constant sensor spacing

FIG. 3 – Common Configurations for Surface Wave Testing

The configuration used for the MASW approach is shown in Fig. 3c. A series of usually equally-spaced receivers (typically 12 to 24) is placed on the surface and the seismic energy is recorded by all receivers simultaneously. The source offset, spacing between the receivers and the length of array (or the number of receivers) significantly impact the effective range of wavelengths for a given dispersion curve (Park et al., 1999; Lin and Chang, 2004). To ensure plane-wave propagation the source offset, X, should be at least as long as the maximum depth of investigation. The spacing between sensors, d_i, should be less than one half of the shortest wavelength measured. Finally, the distance between the first and last receiver should be at least equal to the maximum wavelength of interest.

The same configuration and criteria as the MASW approach are typically used for the ReMi approach but often with a passive source. Since the preferential direction of the source energy is usually unknown, the linear array should be oriented carefully or preferably a two-dimensional array should be considered (Tokimatsu et al., 1992; Cox and Beekman, 2011). In some studies, the passive source is augmented with cultural noise (e.g. jogging along the array) to introduce higher frequency surface wave energy. The effectiveness of this practice is ambiguous at this time.

The information provided in this section is only a general summary of best practices. A qualified and well-trained person may have to adjust test configurations during field testing to ensure that the collected data adequately meet the objectives of the project.

CONSTRUCTION OF DISPERSION CURVE

The end result of the field testing is a series of time records that has to be manipulated to construct a dispersion curve. Many algorithms are available for this purpose. In the traditional SASW approach, this step consists of analyzing each pair of time records collected individually to develop several individual dispersion curves and combining them to a representative dispersion curve (Nazarian and Stokoe, 1985). This process is well-documented in the literature for the last 25 years. Briefly, to obtain the dispersion curve for each receiver pair, the two time records are transformed into the frequency domain, and subjected to spectral analysis to obtain the so-called wrapped phase spectrum. The wrapped phase spectrum is unwrapped. Knowing the unwrapped phase, ϕ_u, at a given frequency, f, for a pair of receivers that are spaced a distance d_i, the phase velocity, V_{ph}, and wavelength, λ, can be estimated from:

$$V_{ph}(f) = 2 \pi f d_i / \phi_u ; \qquad \lambda = V_{ph}/f \tag{2}$$

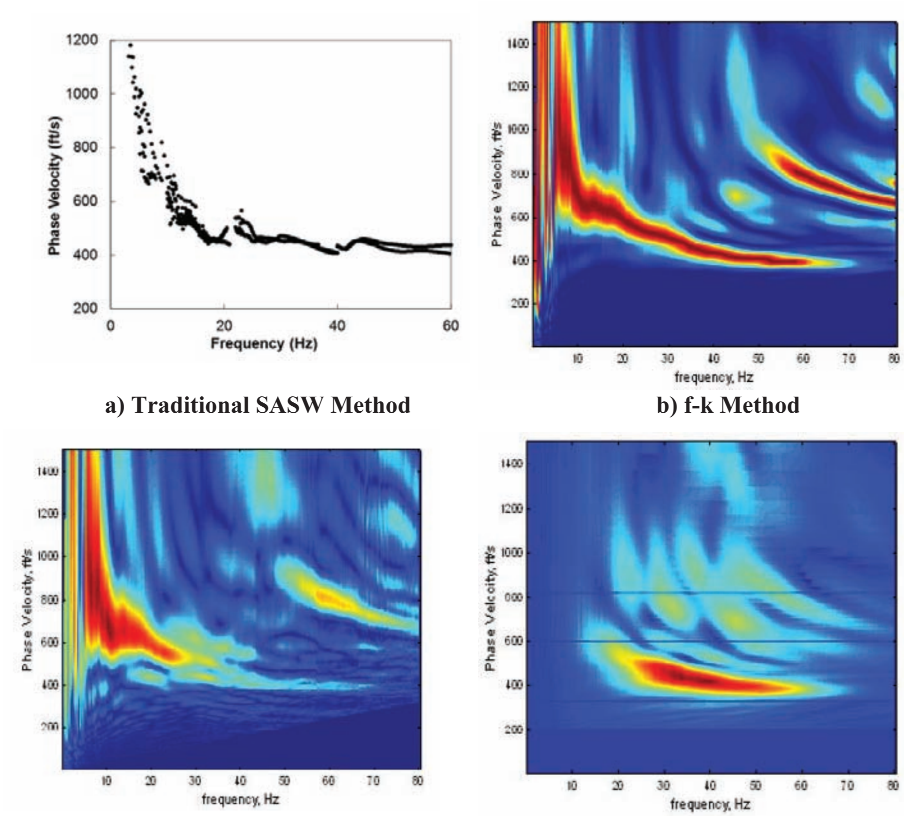
Coherence functions, an outcome of spectral analysis when data are collected repeatedly with the same configuration, are usually used as a quality control tool. A low coherence value (typically less than 0.9) is an indication of the lack of seismic energy or the contamination of energy with higher-mode surface waves or body waves in one or both receivers. Phase data in the regions with low coherence are typically removed from the construction of the dispersion curve. To ensure that the near-field energy contamination is minimized, the frequencies where the unwrapped phase is greater than 180^o (i.e. the wavelength is less than twice the distance between the receivers) should be used. To minimize the energy associated with the higher modes of propagation, phase velocities associated with phase greater than 720^o are sometimes not used. Nazarian and Desai (1993) proposed an algorithm to automate this process. The dispersion curve obtained from this algorithm is called the effective dispersion curve as it may be contaminated by higher modes (Gucunski and Woods, 1992).

The most uncertain and tedious portion of this activity is the phase unwrapping which has been studied by a number of researchers (e.g. Zywicki and Rix, 2005; Rosenblad and Bertel, 2008). This process relies also on the consistency between the dispersion curves from different receiver spacings. Aside from the limitations discussed above, the dispersion curves may not be consistent because of strong lateral heterogeneity of the site. Abdallah et al. (2005) proposed a process to minimize these problems by using a non-parametric fitting algorithm that utilizes a numerical fundamental mode dispersion curve as a constraint.

Examples of dispersion curves from different approaches using a number of receiver spacings are shown in Fig. 4. These dispersion curves are from a series of data collected at one of the National Geotechnical Experimentation sites for a project funded by the ASCE Geo-Institute (GI) for benchmarking of the surface wave method. For subsequent analysis, a number of representative points are sampled using statistical approaches (Nazarian and Desai, 1993; Joh, 1996; Rix et al., 2001) as shown in Figure 5. Each of the plots in Figures 4 and 5 is described in subsequent paragraphs.

The multi-channel data acquisition processes have permitted the use of more advanced signal processing algorithms. The common features of these methods are that (1) they can be readily automated, (2) the results are less sensitive to the environmental noise due to the redundancy in the measurements, and (3) the assumption that the surface energy is focused in the fundamental mode of the Rayleigh wave is not necessary. One can take advantage of the density of the measurement points with the multi-channel measurements to transform the time records not only temporally but spatially as well. Several multi-array approaches are in use at this time.

Zywicki (1999) adapted the frequency-wavenumber (f-k) process from the traditional geophysical field. Wavenumber is simply the rotational frequency (ω = $2\pi f$) divided by the phase velocity at a given frequency, $V_{ph}(f)$. The goal with the f-k



a) Traditional SASW Method **b) f-k Method**

c) FODI Method **d) p-f Method**

FIG. 4 – Construction of Dispersion Curve by Different Approaches (after Tran and Hiltunen, 2008)

process is to obtain a contour plot of the variation in amplitude of the energy P(f,k), associated with each frequency, f, and wavenumber, k, in the range of interest of the project. The fundamental equation associated with this process can be described as:

$$\mathbf{P}(f,k) = \mathbf{e}(k)\, \mathbf{W}^{T}(f)\, \mathbf{R}(f)\, \mathbf{W}(f)\, \mathbf{e}^{H}(k) \qquad (3)$$

where **e**(k) = phase shift vector associated with a trial k for a given receiver configuration, **W**(f) = a diagonal matrix that acts as a weighting function to adjust for the geometric spreading of the wave with distance from the source, **R**(f) = matrix of the cross spectral densities for each test frequency, and $\mathbf{e}^{H}(k)$ denotes the Hermitian transpose of vector **e**(k). As shown in Fig. 4b, the dispersion curve is estimated by determining the wavenumbers where the maximum amplitudes are observed at a given frequency. Higher surface wave modes may appear as secondary peaks in the contour plot. Several researchers have suggested marginal improvements to the method, especially by suggesting different parameters for the matrix **W**(f).

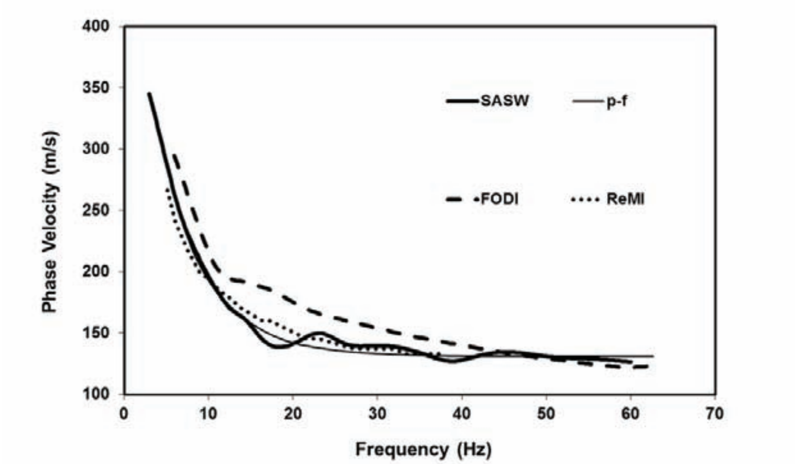


FIG. 5 – Dispersion Curves Obtained from Different Approaches (after Tran and Hiltunen, 2008)

Similar in concept, Park et al. (1999) proposed the following relationship for developing the contour plot of the amplitude vs. phase velocity and frequency:

$$P(V_{ph,f}) = \left(\frac{1}{N}\right)\sum_{i=1}^{N} e^{-j\delta_{i,T}}\, R_{i,norm}(\omega) \quad (4)$$

where $R_{i,norm}(\omega)$ is the normalized frequency-domain amplitude of the time record i at frequency f, and $\delta_{i,T} = 2\pi f\, x_i/V_{ph}$ with x_i being the distance between the source and the receiver i in the array. Parameter N corresponds to the number of receivers in the array. The normalization of amplitudes is usually done by dividing the amplitude of each record by the corresponding maximum amplitude in the frequency-domain. To obtain the dispersion curve, the phase velocities associated with the maximum $P(V_{ph},f)$ for each frequency are estimated as shown in Fig. 4c. Park (2011) termed this process as the Full-Offset Dispersion Imagining (FODI). To improve FODI further, Park (2011) proposed Selective-Offset Dispersion Imaging (SODI). The main difference between FODI and SODI is the number of receivers considered in Eq. 4. As in FODI, all receivers are used in the analysis, in SODI the number of receivers is reduced for different frequencies to only consider the offsets that will satisfy a wavelength criterion similar to the SASW approach to minimize the near- and far-field concerns. Lin et al. (2004) proposed an alternative means of implementing this approach that utilizes spatial and temporal Fourier transform of the data with the caveat that the receiver spacings should be uniform throughout the array.

Louie (2001) advocated the slowness-frequency (p-f) approach. This method builds on the well-established p-τ transform in reflection seismology. Slowness, p, is the inverse of the propagation velocity and τ is the projection of a linear best fit to variation in slowness with distance to a distance of zero from the source. The data in the p-τ domain are then subjected to an FFT algorithm to transfer them to the p-f domain. The summations of the normalized power spectra of the p-f data are used to develop a contour plot of the amplitude as a function of slowness and frequency (Fig.

4d). The determination of the dispersion curve from the contour plot is not as straightforward as the f-k or FODI approach. Zywicki (2007) has an insightful study of this process. Beekman (2008) proposed an algorithm for utilizing this approach in a more automated fashion. Uses of wavelet transform for constructing the dispersion curve (Kim and Park, 2002; Gucunski and Shokouhi, 2005) are also promising approaches that can be effective.

Figure 5 shows a comparison of the dispersion curves from different approaches presented in Figure 4. These dispersion curves are quite similar in their respective appropriate frequency ranges dictated by the array configurations and source characteristics. However, differences in the high and especially low-frequency ranges are apparent. Hebeler (2001), Li (2008) and Cox and Wood (2011) give practical examples of the benefits and uncertainties associated with different approaches. Even though simple in concept, to obtain high-quality dispersion curves the interaction of a number of sometimes conflicting parameters (discussed above) should be considered before field testing and should be adjusted in the field as necessary. As such, an experienced operator is needed for this task.

The SASW approach, even though tedious, can provide a dispersion curve that covers from near surface to depths of several hundred meters by changing the source characteristics. Since all the steps of the data reduction are transparent to the analyst, the quality and uncertainty of the measurement can be judged. The more convenient multi-channel methods that usually utilize one source with many receivers provide the dispersion curve in a preferential range of frequencies. For example, the MASW approach with a sledge hammer as a source is quite effective for shear wave profiling down to 10 m to 15 m, while the ReMi approach typically does not provide dispersion curves that contain information about shallow depths. Since all surface wave approaches are based on similar concepts, one can readily expand the range of dispersion curves by performing the same tests with different test configurations and sources (Nasseri-Moghaddam and Park, 2010), by combining two approaches (Liu et al., 2005, Park et al., 2005), or utilizing other complimentary geophysical tests (Rucker, 2007).

Any of these processes will increase the time necessary to conduct the field tests and analyze the data, but they will provide the information necessary for the engineering design or analysis with more certainty.

Estimation of Shear Wave Velocity Profile (Inversion Process)

The final step in the surface wave method includes estimating the variation in VS with depth from the dispersion curve. Since a closed-form solution is not available for this task, the so-called inversion process is usually considered. The inversion process consists of estimating an initial VS profile, developing a numerical dispersion curve from the assumed VS profile, comparing the experimental and theoretical dispersion curves, and adjusting the VS profile iteratively until the difference between the experimental and numerical dispersion curve is below an acceptable threshold. The inversion process consists of the following two components: (1) an algorithm to estimate the dispersion curve numerically (a.k.a. forward-model), and (2) an algorithm to adjust the assumed VS to minimize the differences between the experimental and numerical dispersion curves (a.k.a. optimization algorithm).

Forward Modeling

Ideally the numerical dispersion curve should be as representative of the field testing as possible. In that ideal model, the source and receiver configuration should mimic the field setup and the source signature should be similar to the field one. The output of the numerical model should be time records similar to the field measurements, so that the same algorithms used to construct the experimental dispersion curve can be applied to the numerical time records as well. The vertical and lateral heterogeneity of the geomaterial should also be considered in this ideal condition, since they impact the results. The computational engine for this purpose should be rigorous to handle the constitutive behavior of different geomaterials.

A carefully arranged all-purpose finite element/difference package or a custom algorithm developed for this purpose (e.g., Lai and Rix, 1998) may be used. These codes, which are computationally intense, are extremely important for understanding the role of different parameters that may impact the dispersive characteristics of the surface waves, optimizing the field measurement configuration, and understanding some of the complications with the surface wave method. However, these algorithms are not typically used in the inversion algorithms because of their complexity and time required to complete an inversion.

The first numerical formulation of the propagation of surface waves in multi-layered geomedia is attributed to Thomson (1950) and Haskell (1953). For a horizontally layered, vertically heterogeneous, isotropic, elastic medium over a half-space, they provided the differential equations for motion-stress vectors of surface waves that can be solved via a so-called transfer matrix. The major limitation of this approach is that the source-receiver configuration cannot be modeled. This pioneering work has been studied by numerous researchers to improve its computational efficiency, and to adopt it for the near surface applications as summarized in Pei et al. (2008). Nazarian and Stokoe (1985) adapted this formulation for the original SASW method with some adjustments to minimize its numerical instabilities associated with short wavelengths. Despite its simplified assumptions, the Haskell-Thomson type solutions are the most widely used algorithms in surface wave profiling due to their time-efficiency.

A more robust means of obtaining the dispersion curve is through the dynamic stiffness matrix advocated by Kausel and Roesset (1981) assuming plane waves propagating far from the source (a.k.a. 2-D analysis). This solution provides the fundamental mode of propagation. Kausel and Peek (1982) expanded the solution to a so-called 3-D analysis where both the surface and body waves are considered and the plane wave restriction is relaxed. In that solution the location of the source and receivers can be specified.

Gucunski and Woods (1992) utilized the above 3-D analysis to study how different modes of propagation practically contribute to the dispersion curve constructed with the traditional SASW approach. They indicated that the phase velocity determined by the SASW method is an effective phase velocity corresponding to the superposed mode resulting from the combination of fundamental and higher mode surface waves. Lai and Rix (1998) proposed a rigorous algorithm based on Green's functions to obtain the effective phase velocity dispersion curve that is equivalent in numerical efficiency to transfer matrix methods.

A number of other algorithms can be found in the literature that are essentially a variation of either the transfer matrix, 2-D stiffness matrix or 3-D stiffness matrix (see O'Neill and Matsuoka, 2005; Pei et al., 2008). Most methods discussed above can yield the fundamental and higher modes reasonably well since conceptually the higher mode dispersion curves may improve the inversion process. As indicated by many investigators (e.g. Gucunski and Woods, 1992; O'Neill and Matsuoka, 2005; Calderón-Macías and Luke, 2010) for a typical geomedium where the velocity contrast is not significant or where significantly high velocity inclusions are not present, all methods essentially yield the same dispersion curves. For more complex site conditions, the more rigorous algorithms may yield more representative results.

Optimization Algorithm

The optimization algorithm, which is the subject of interest and improvement by many applied mathematicians or computational scientists, is the process of iteratively comparing the experimental and numerical dispersion curves and minimizing the differences between them. N dispersion points are chosen from the experimental dispersion curve for comparison with corresponding numerical values. An M-layer profile is specified by the layers' shear wave velocities (with assumed constant Poisson's ratios and densities). In the simplest form, the optimization is carried out by an experienced analyst that manually changes the assumed VS profile and visually or with simple statistics judges when the two dispersion curves are close enough (Nazarian and Stokoe, 1985). Since this subjective and time-consuming process will allow the analyst to use his/her experience and any other a priori information about the site, the manual trial and error process may be preferred for very complex sites.

The state of the practice in surface wave testing is the use of an optimization or error minimization algorithm (Yuan and Nazarian, 1993; Joh, 1996; Ganji et al., 1998; Lai and Rix, 1998, Louie et al., 1999; Park et al., 1999). These algorithms automatically and rapidly provide a VS profile that statistically describes the measured dispersion curve well. Most minimization techniques are based on the generalized inversion technique as explained in many textbooks such as Santamarina and Fratta (2005). The generalized inversion technique assumes a linear relationship between perturbation in the VS of the layers and the resulting change in phase velocities. The partial derivative with respect to each parameter to be determined is carried out to estimate the change in that parameter for the next iteration. In most algorithms the partial derivatives are estimated numerically; however forward models that lend themselves to analytical differentiation are preferred (Lai and Rix, 1998). Also the percent change in layer shear wave velocities between iterations is typically constrained for stability (Lai and Rix, 1998; Yuan and Nazarian, 2003). In case of a sharp contrast between the properties of two adjacent layers, nonlinear minimization techniques such as those proposed by Ganji et al. (1998) are conceptually preferred, even though they are not as computationally efficient as the generalized inversion techniques (O'Neill and Matsuoka, 2005). Hadidi and Gucunski (2003) and Rydén and Park (2006) proposed the simulated annealing method for inversion. This algorithm utilizes the Monte Carlo method to ensure that the global minima are found. The use of the artificial neural networks (e.g. Williams and Gucunski, 1995 and Shirazi et al, 2009) or genetic algorithm (e.g., Yamanaka and Ishida, 1996) has also been proposed to accelerate the inversion process. These approaches are effective when the profile contains only a few uniform layers.

The VS profiles from the three approaches (i.e., SASW, MASW and ReMi) for the site described above are included in Fig. 6. The three VS profiles are in reasonably good agreement for the first 10 to 15 m, below which they diverge because perhaps in the uncertainties associated with the low frequency dispersion curves. For some rudimentary applications, such as VS30 determination, the three methods essentially yield the same outcome.

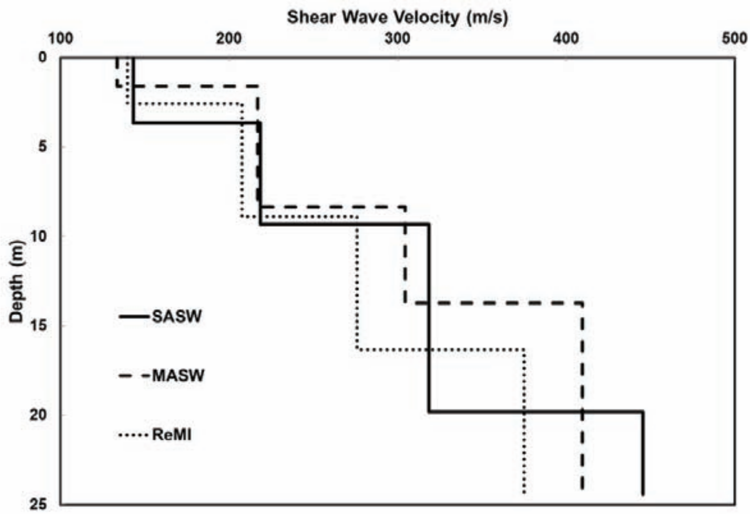


FIG. 6 – Comparison of Shear Wave Velocity Profiles from Different Approaches (after Tran and Hiltunen, 2011)

Practical Considerations

The state of the practice of surface wave testing is at least partially documented in Tran and Hiltunen (2011) as part of the Geo-Institute's benchmarking project. Up to ten participants analyzed several sets of data collected using different approaches. The VS profiles from each approach and among approaches were generally similar as long as the analysts adhered to the limitations of their approaches in terms of the range of frequencies where appropriate energy existed in their experimental dispersion curves and limited their inversion process to depths that were compatible with the ranges of frequencies contained in their experimental dispersion curves as described in Tran and Hiltunen (2011) and Cox and Wood (2011).

Any inversion process is conceptually a nonunique process where a number of VS profiles can yield a numerical dispersion curve that can describe a measured dispersion curve with a small tolerable mismatch. However, when the process (from field data collection, to construction of dispersion curve to inversion) is carried out properly such nonuniqueness may have small practical effect on the final engineering outcomes. Several desirable practical considerations to reduce the uncertainty and improve the accuracy of the reported VS are discussed below.

As shown in Fig. 2, an appropriate dispersion curve should contain three distinct zones. If the measured dispersion curve does not contain the short-wavelength (high-frequency) Zone 1, the VS of the near-surface layers cannot be estimated with certainty. This uncertainty will negatively impact the accuracy of the estimated shear wave velocities of the lower layers that may lead to an erroneous final interpretation. On the other hand, if the long wavelength (low-frequency) Zone 3 is not well defined, the shear wave velocities of the deep layers in the profile will be extremely nonunique. This phenomenon is critical when a soil profile is underlain by a stiff layer such as bedrock. If the depth and properties of the bedrock are critical information for the success of a project, the dispersion curve should contain Zone 3.

The number of layers assumed to describe the VS profile is also very important. It is perceived that a more detailed VS profile will be obtained if the number of layers in the profile is increased. The uncertainty in the reported shear wave velocities increases as the number of layers is increased since more unknown parameters become involved in the inversion process. It is a good practice to limit the number of layers to less than ten. The thickness of the first layer in the profile should be about two times the shortest wavelength reflected in the measured dispersion curve. The assumed thickness of each layer should not be less than 15 to 20% of its depth from the surface unless a priori information is available about a critical, relatively thin layer within the profile. The last layer (half-space) should not be placed deeper than one-half of the longest wavelength, and should be moved closer to surface if Zone 3 is not well-defined. A conscientious analyst will always take advantage of all additional geological and geotechnical site information to constrain the thickness and shear wave velocities of the layer.

The inversion process should be carried out with full consideration of the uncertainties in the measured dispersion curve. These uncertainties partially stem from the theoretical limitations of the method, and partially from the field setup and the algorithm used to construct the experimental dispersion curve. Marosi and Hiltunen (2004), O'Neill (2004), Lai et al. (2005) and Luke and Calderón-Macías 2007) contain excellent discussions on this topic. The consensus is that the dispersion points associated with lower frequencies are more uncertain and that the uncertainties in the measured dispersion curves should be propagated into the inversion process. The tolerable error when the inversion process is considered complete should be set based on these uncertainties to avoid the so-called "over-fitting" to ensure that the final VS profile is not erratic. Another pragmatic way of quantifying the uncertainties in the results is to conduct the inversion with a number of initial assumed models (e.g., using a Monte Carlo simulation) and statistically analyzing the uncertainty of layers' VS.

The utilization of the higher modes in the inversion process should conceptually be beneficial, especially for complex profiles, since the additional information may help in constraining the results (Supranata et al., 2007). Practically speaking, this task may be challenging. An example of a multi-mode dispersion curve for a normal geotechnical site where the VS gradually increases is shown in Fig.2. The dispersion curves from different modes are nicely separated. There is a consensus that for a typical geomedium the improvement in the final VS profile with multi-mode inversion may be small as compared to the fundamental mode inversion for these cases. As a comparison, computed dispersion curves for the fundamental and the three higher modes for a site containing 10 m of soil over bedrock are shown in Fig. 7. However, it may be difficult to practically delineate these modes with certainty as in some regions two adjacent modes are very close to one another and the energy transfer between modes is not theoretically limited to direct transfer between adjacent modes (Hebeler, 2001; Jin et al., 2009; Calderón-Macías and Luke; 2010). As such, it is possible for several intermediate modes to be indiscernible. Also body wave energy may appear in the dispersion curve and be misinterpreted as a higher surface mode. Some of these shortcomings can be overcome in the future as more work in this field is carried out.

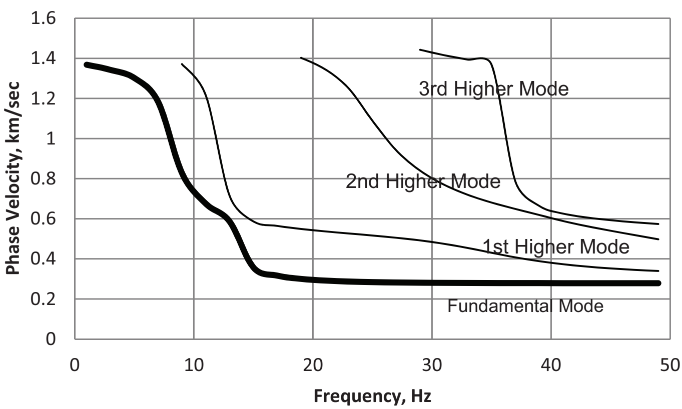


FIG. 7 – Dispersion Curves for a Site with Shallow Bedrock (after Calderón-Macías and Luke, 2010)

Finally, one of the most important factors in surface wave profiling is the self-consistency between the method used to obtain (construct) the measured dispersion curve, and the method used to compute the theoretical dispersion curve (Abdallah et al., 2005). A careful balance between the levels of sophistication of these two processes will improve the chances of obtaining reliable VS profiles.

CONCLUDING REMARKS

This paper contains a review of the state of the art associated with surface wave testing with emphasis on the practical considerations to obtain a representative shear wave velocity profile. Since its first applications about 30 years ago, surface wave testing has reached a level of maturity that can be used in many geotechnical and infrastructure engineering projects as a cost-effective and value-added tool. However, despite the hard work of a large number of researchers to expedite the field testing and automate the data reduction, a number of inter-related and sometimes conflicting parameters have to be considered and need be adjusted to obtain representative results. As such, the learning curve in using the surface wave method is steep, and these tests should be carried out by qualified personnel that are fully aware of the limitations of the methods. To that end, a certification program and national guidelines for surface wave testing may be beneficial.

As discussed above, and as has been shown by a number of major case studies, the different approaches to surface wave testing are equally valid as long as the data collection and analysis are compatible with the known theoretical and experimental limitations of each approach. The selection of the appropriate approach(es) should be driven by the requirements of the project. For some projects, it may be desirable to combine several approaches (e.g., MASW and ReMi) to obtain the VS profile over the range of depths required for the project. In others, it may be desirable to analyze the data collected by a given array with different approaches (e.g. constructing the dispersion curve from multi-channel data with the f-k and traditional SASW processes) to understand some of the inconsistencies in the results.

Depending on the uncertainty in the experimental dispersion curve, VS profiling can be mildly to significantly nonunique. Every effort should be made to quantify this nonuniqueness and all the available geological and geotechnical information from the site should be used in constraining the reported VS profile.

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I know that I might have offended a number of my very active colleagues such as Brady Cox, Sung-Ho Joh, John Louie, Choon Park and Brent Rosenblad by omitting to refer to all of their good work. I blame this on the page limit imposed on this paper.

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Offshore Geotechnics – the Challenges of Deepwater Soft Sediments

Mark F. Randolph[1]

[1]Professor of Civil Engineering, Centre for Offshore Foundation Systems, The University of Western Australia, 35 Stirling Highway, Crawley, WA 6009, Australia; mark.randolph@uwa.edu.au

ABSTRACT: In principle, design 'practice' is represented by gradually evolving national and international design guidelines. In reality, most designs rely on more sophisticated analysis techniques and principles, informed by recent research publications that may take several years to be reflected in industry guidelines. This may be considered as 'art'. The more rapidly a sector of industry is developing, the wider will be the gap between practice (as represented by design guidelines) and art. In the offshore industry, the most rapidly changing sector is the move to increasingly deep water, with new developments now in water depths exceeding 2 km. In those conditions, the seabed typically comprises relatively soft fine-grained material, with very slow sedimentation rates. The challenges to be discussed in this state-of-the-art paper include characterizing the shear strength and other engineering properties of the sediments, and design approaches for a range of deepwater infrastructure including different forms of anchoring systems, shallow mat foundations, pipelines, flowlines and riser systems. Particular focus in the paper is on secondary characteristics of the seabed sediments that reflect effects of loading (or strain) rate and disturbance (or remolding) on the response of the infrastructure in question.

INTRODUCTION

In the offshore arena, design 'practice' is captured formally in guidelines such as produced by the American Petroleum Institute (API), Det Norske Veritas (DNV) and the International Standards Organisation (ISO). The time scale over which new and revised design guidelines are published, which can exceed 10 years, results in an emphasis on rather traditional methods for foundation design with a lag before results of recent research can be implemented. New technology, such as different types of anchor, or new design approaches that arise from developments in more extreme conditions, such as the lateral buckling of high pressure, high temperature, flowlines, generally require design practices that are outside published guidelines.

Interpreting 'state-of-the-art' as representing analysis and design approaches that have resulted from recent research, but which mostly go beyond existing guidelines, this paper addresses some of the modern challenges in deepwater geotechnical engineering. The paper starts with comments on difficulties associated with assessing the strength profile and other characteristics of soft deepwater sediments, and the resulting trends towards more sophisticated site investigation and field model testing equipment. The consequences and quantification of varying degrees of rate effects, arising from either partial consolidation or viscous rheology, and remolding due to cumulative strains, are discussed. The paper then considers three groups of infrastructure: anchoring systems, shallow mat foundations for subsea systems, and pipelines; in each case showing how conventional geotechnical approaches need to be augmented to address particular features of the application.

The ideas and examples presented are mostly drawn from the author's research in conjunction with colleagues and postgraduate students at the Centre for Offshore Foundation Systems, The University of Western Australia, and through consultancy within the offshore industry. More extensive documentation of the topic can be found in related publications (Randolph et al. 2011; Randolph and Gourvenec 2011) and in the specialist symposium, ISFOG (Gourvenec and White 2010). Two significant areas that are not addressed here are those of geohazards and seismic design. Both of these are important aspects of offshore design and are generally approached through multi-disciplinary teams, starting with the regional geology and then gradually focusing towards the specific risks for the development in question. More details on geohazard assessment are provided in the complementary state-of-practice paper (Jeanjean 2012).

SEABED CONDITIONS AND SITE INVESTIGATION

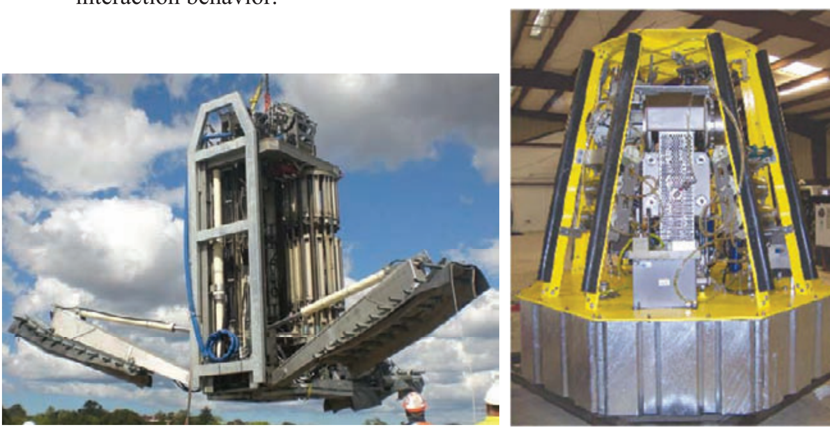
The seabed at deepwater sites typically comprises soft fine-grained sediments, with low sedimentation rates. The seabed topography may be affected by features such as previous mass transport events and channeling, and pock marks from gas expulsion or mud volcanoes, but if possible the development will be located away from these potentially hazardous features. A good example of how design strategies need to be adjusted where it is not possible to achieve this is provided by the suction anchor design for the Mad Dog development, where the geometry of one group of anchors was modified to cope with penetration through slump material on the edge of the Sigsbee Escarpment (Jeanjean et al. 2006).

Two different site investigation strategies are required for anchoring systems and shallow mat foundations, and for pipelines (including flowlines and steel catenary risers). The former require targeted investigation of the upper 20 to 30 m of the sediment profile, while the latter require extended investigations along the route of the pipeline, but to a depth of no more than 1 or 2 m.

The combination of deep water, generally soft sediments and focus on relatively shallow investigations has led to a number of trends in the last decade or so:

- a shift towards sea-bottom investigation systems and away from floating (drillships etc) systems;
- major advances in robotic techniques for drilling, sampling and in situ testing;
- improvements in gravity piston samplers, capable of retrieving samples of up to 20 or 25 m depth;
- development of lightweight sampling and penetrometer testing devices suitable for operation from remotely operated vehicles (ROVs);

- introduction of full-flow penetrometers to improve accuracy of strength measurement in very soft sediments, including assessment of remolded shear strength;
- introduction of seabed-based model testing equipment to assess pipeline-soil interaction behavior.



(a) Portable remotely operated drill (PROD) (b) Seabed penetrometer frame

FIG. 1. Examples of seabed-based site investigation equipment.

Seabed Equipment

Examples of modern seabed-based site investigation tools are shown in FIG. 1. On the left is the second generation portable remotely operated drill (PROD) developed by Benthic Geotech Pty Ltd, with capabilities of drilling, sampling and penetrometer testing down to a depth of 100 m. On the right is the small seabed frame developed by Gregg Drilling & Testing Inc. for cone and full-flow penetrometer testing to shallow depths in deep water. Another recent development is the Rovdrill-3, developed by Triton Group's Seafloor Geoservices, where power from a heavy duty work-class ROV (remotely operated vehicle) is used in conjunction with robotic drilling and penetrometer equipment, with a maximum investigation depth range of 200 m.

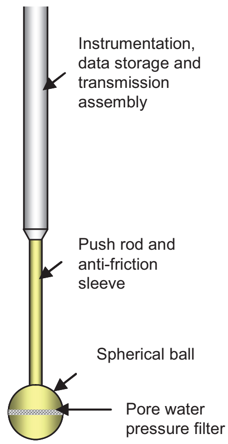
There are also a number of seabed frames and ROV-mounted equipment that target the upper 1 to 3 m of the seabed, specifically for pipeline investigations. Penetrometer equipment operated directly from an ROV was developed nearly a decade ago by the Danish Geotechnical Institute, with example results presented by Newson et al. (2004). A more recent development is Fugro's SmartSurf, which has a piston sampler of up to 2 m range, standard sized cone and T-bar penetrometers, and a miniature (12 mm diameter) T-bar with a 1 m stroke (Borel et al. 2010).

Large diameter gravity piston corers such as the STACOR (Borel et al. 2005) may be used to obtain relatively undisturbed samples up to 20 or 30 m long, although they tend to disturb the near surface material. Box-corers are used increasingly to cover bulk material from the upper 0.5 m of the seabed, allowing miniature penetrometer tests to be conducted through the samples (Low et al. 2008).

Full-flow Penetrometers

Full-flow cylindrical (T-bar) and spherical (ball) penetrometers have been used offshore increasingly over the last decade, with particular focus on the soft sediments encountered in deep water. A comparison of different cone, T-bar and ball penetrometers is shown in FIG. 2. The larger cone has a diameter of 44 mm, compared with the standard 36 mm diameter, while the standard T-bar is 40 mm by 250 mm long (so 10 times the projected area of the standard cone). Ball diameters of either 60 mm (as shown, with a 20 mm diameter shaft) or 78 mm (with a 25 mm diameter shaft) have been developed by different companies.

(a) Cones (standard and large) and T-bar



(b) Ball (Kelleher et al. 2005)

FIG. 2. Penetrometers of various shapes and sizes.

The principal advantages of full-flow penetrometers for assessing the shear strength of soft material are:

- increased resolution, for a given load cell, due to the larger projected area compared to the cone, and also the higher ratio of resistance from the soil to the ambient force on the instrument head arising from the depth of water;
- geometric simplicity, which has allowed more sophisticated and robust analysis in order to evaluate bearing factors and hence deduce the shear strength of the soil, and which renders these factors essentially independent of the pre-failure stiffness of the soil and the in situ effective stress ratio;
- the ability to confirm the accuracy of the load cell zero and to evaluate the sensitivity of the soil by conducting cyclic penetration and extraction tests.

Example data from a cyclic ball penetrometer test are shown in FIG. 3. The data show good ‘symmetry’ of remolded resistance during penetration and extraction, and a final resistance that is about 25 % of the initial penetration resistance. This is consistent with a soil sensitivity of about 5, noting that the bearing factor for fully remolded conditions will be somewhat larger than that for initial penetration (Zhou and Randolph 2009b).

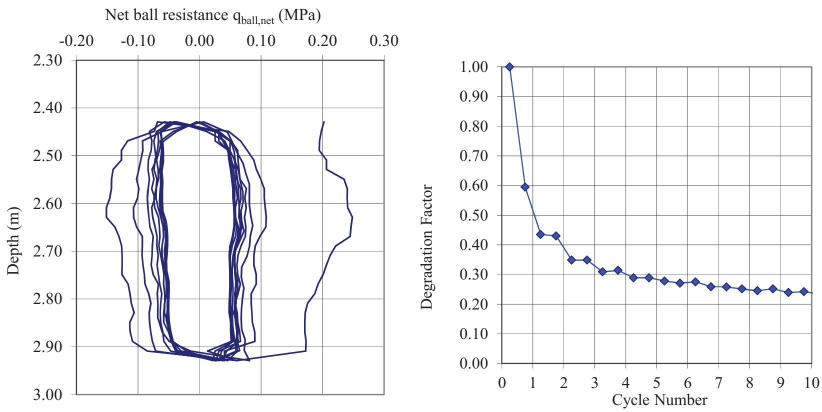


FIG. 3. Results of a cyclic ball penetrometer test.

Suggested guidelines for the conduct and interpretation of penetrometer tests have been published recently (DeJong et al. 2010, Lunne et al. 2011) – the first step in transforming ‘art’ into ‘practice’, which will be completed when the draft ISO standard on marine soil investigations is eventually published. Both guidelines encourage the performance of at least one cyclic test at every full-flow penetrometer test location to confirm the reference (zero) readings of the penetrometer load cell.

In early work on full-flow penetrometers, plasticity solutions for flow around a cylinder or sphere were seen as providing an appropriate bearing factor, N_k, from which to deduce shear strength, s_u, from the (average) resistance per unit area, q_u. This led to a suggested T-bar factor of 10.5, as appropriate for a moderately smooth interface condition (Stewart and Randolph 1994). However, the competing influences of high strain rate (characteristic shear strain rate of around 0.5 s^{-1}, so 4 orders of magnitude greater than a typical laboratory shearing rate) and partial softening as the soil is disturbed affect the value of N_k.

Numerical studies of the effects of strain rate and softening on the bearing factor for T-bar and ball penetrometers suggest $N_{T\text{-}bar}$ values in the range 10.5 to 13, with corresponding values of N_{ball} some 20 % greater (Zhou and Randolph 2009a). An alternative numerical approach (Klar and Pinkert 2010) has confirmed the range of T-bar values. Correlations based on the shear strengths (average of triaxial compression, simple shear and triaxial extension) measured on high quality samples from various onshore and offshore sites around the world have suggested an average value of 12 for $N_{T\text{-}bar}$ as shown in FIG. 4 (Low et al. 2010). There are, however, some indications that for many soils a slightly lower value of 10.5 to11 is appropriate, while in the high plasticity soils of the Gulf of Guinea, which appear to show greater strain rate dependency, the $N_{T\text{-}bar}$ value is greater.

The database referred to above was for moderately insensitive soils, with S_t

generally less than 8 as is typical for marine sediments. In more highly sensitive soils, such as some of the Canadian quick clays, lower bearing factors are observed, as indicated in FIG. 5 (DeJong et al. 2011). Comparison of ball and T-bar penetrometers have shown much more similar penetration resistance than theoretical solutions suggest, with bearing factors, N_{ball}, no more than 10 % greater than for the T-bar.

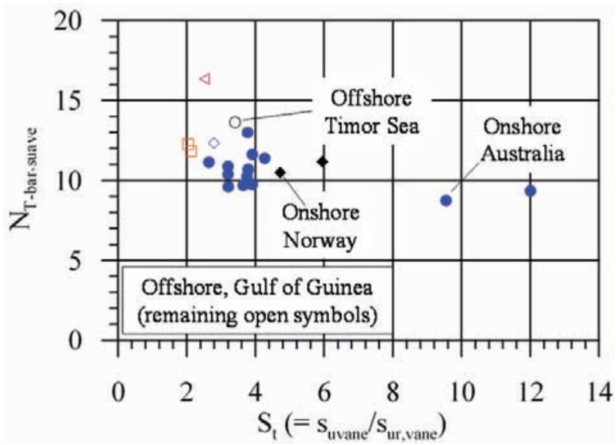


FIG. 4. Correlations of $N_{T\text{-bar}}$ based on average shear strength against S_t

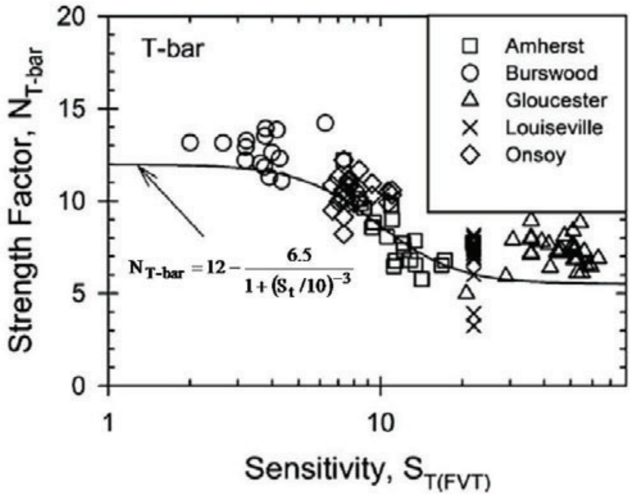


FIG. 5. Decrease of $N_{T\text{-bar}}$ values in soil of high sensitivity.

Penetrometer Testing in Intermediate Soils

The discussion so far has focused on fine-grained impermeable soils where a penetrometer test takes place under undrained conditions. However, in tropical regions where carbonate silts are encountered, and in onshore situations with siliceous silts, assessment of the degree of partial consolidation during a penetration test is a major challenge. There are two aspects of the problem: the first is to assess whether partial consolidation may have increased the bearing resistance; and the second, more challenging, is to choose an appropriate bearing factor from which to derive an undrained shear strength.

In offshore practice, pore pressure measurements during penetration testing are deemed far more reliable than friction sleeve measurements (Lunne and Andersen 2007). The pore pressure data is the key to assessing partial consolidation. In order to distinguish high penetration resistance arising from high overconsolidation ratio, from that arising from partial consolidation, it is useful to consider the normalized excess pore pressure, $\Delta u_2/\sigma'_{v0}$ (where σ'_{v0} is the effective overburden pressure) rather than the ratio, $B_q = \Delta u_2/q_{net}$. An alternative classification chart is shown in FIG. 6 (Schneider et al. 2008). A framework for 'transforming' penetrometer data affected by partial consolidation to the prediction of undrained bearing capacity of a much larger foundation has been suggested (Lee and Randolph 2011), considering the different normalized velocities, vD/c_v of penetrometer and foundation, where D is the relevant diameter and c_v the coefficient of consolidation.

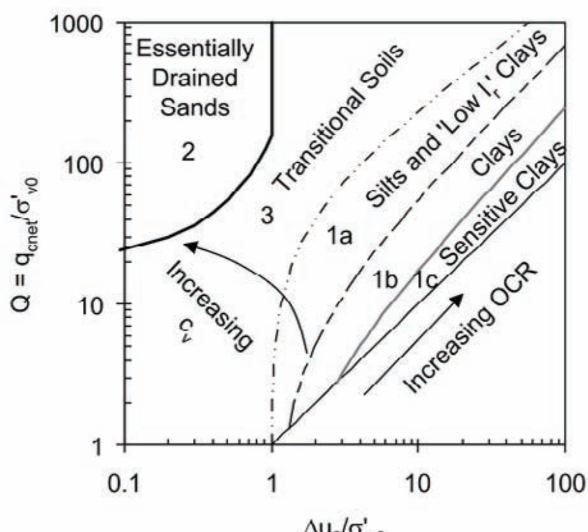


FIG. 6. Classification chart for piezocone data.

Design From Field Test Data

Improvements to, and more widespread use of, in situ testing have led to new design approaches based directly on field site investigation data. This is particularly true of sandy soil, where it is difficult to replicate field conditions within a laboratory. Modern approaches to pile design based on cone penetration testing are discussed extensively by Jeanjean (2012). Similar developments are occurring for (onshore) shallow foundations on sand, where the focus is more on settlement than capacity (Lehane et al. 2008).

In principle, it is logical to apply a similar approach for fine-grained soils, with the penetration resistance being used directly in design calculations, rather than going through the intermediate step of first estimating an appropriate undrained shear strength. In some applications, for example spudcan penetration resistance or lateral pile response, there is an obvious direct link, although even then some adjustment of the penetration resistance measured during site investigation might be required because of differences in the detailed strain path and rate of shearing.

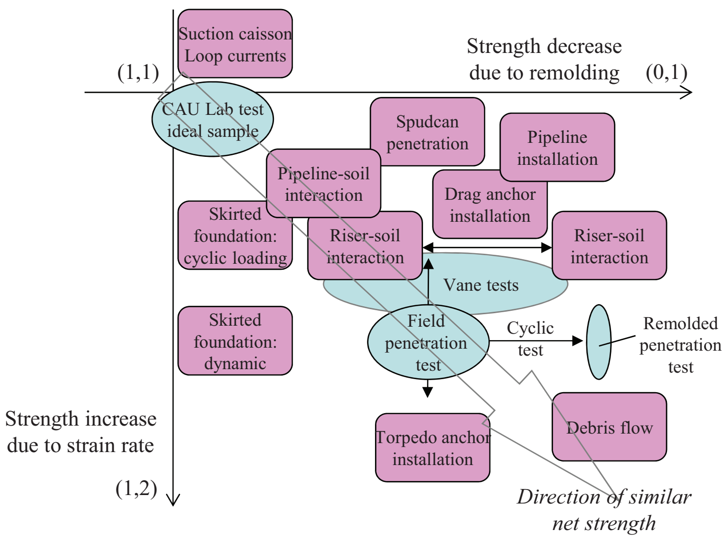


FIG. 7. Effects of strain rate and remolding for different design applications.

A schematic map of offshore design applications, together with different laboratory and field tests, is shown in FIG. 7 (Randolph et al. 2007). The map gives an indication of how the shear strength deduced from a particular test (or possibly the penetration resistance directly) may need to be adjusted to allow for different rates of shear strain or degree of remolding in a particular design application. Extremes of shear strain rate are provided by torpedo anchor installation (penetrating at around 25 diameters per second, so 50 times a penetrometer test) and a suction caisson designed to withstand loop currents in the Gulf of Mexico, where the governing load might be applied for several days. The difference in strain rates of 6 to 8 orders of magnitude might be expected to require adjustment of the relevant shear strength by a factor of 2 or more.

In terms of disturbance, the installation of skirted foundations will cause minimal localized remolding of the soil, the effects of which will be reduced by consolidation, while modeling of a debris flow or the soil response within the touchdown zone of a steel catenary riser would need to consider fully remolded conditions of the soil even before any allowance for additional water entrainment.

The two effects may be captured using a shear strength model with multiplicative adjustments for strain rate, using a logarithmic or power law function, and partial remolding, such as:

$$s_u = s_{uy}\left[1+\eta\left(\frac{\dot{\gamma}}{\dot{\gamma}_{ref}}\right)^n\right]\left[\delta_{rem} + (1-\delta_{rem})e^{-3\xi/\xi_{95}}\right] \tag{1}$$

where:

s_{uy} = yield stress at very low strain rates;
η = viscous parameter;
$\dot{\gamma}$ = shear strain rate;
$\dot{\gamma}_{ref}$ = reference shear strain rate, at which shear strength is $s_{u0} = s_{uy}(1 + \eta)$;
n = power law parameter;
δ_{rem} = remolded strength ratio, the inverse of sensitivity;
ξ = cumulative plastic shear strain; and
ξ_{95} = plastic shear strain to cause 95 % remolding.

The rate term is a normalized form of the Herschel-Bulkley model used widely to describe the rheology of non-Newtonian fluids (Herschel and Bulkley 1926) and applied in this form to fine-grained soils by Boukpeti et al. (2011).

Similar relationships to (1) have been used to assess suitable bearing factors for full-flow penetrometers, relative to shear strengths measured at low strain rates in the laboratory (Zhou & Randolph 2009a, Klar and Pinkert 2010). Other applications include spudcan penetration (Hossain and Randolph 2009) and the keying and failure of plate anchors. In both of these, for typical softening parameters relevant for offshore conditions, continuous motion leads to a reduction of 15 to 20 % in the bearing resistance, by comparison with results from plasticity solutions based on non-softening strengths. This is consistent with results of field performance of spudcan penetration in the Gulf of Mexico (Menzies and Roper 2008).

At low strain rates, or during sustained loading, consolidation may override effects of reduced viscosity as the water content of the soil is reduced. Penetrometer tests conducted at different penetration rates allow both viscous rate parameters and the consolidation coefficient, c_v, to be deduced (House et al. 2001, Chung et al. 2006). Modification of standard testing procedures in this way are attractive, essentially designing the field test in order to obtain the specific parameters required. Modern robotic tools for seabed site investigation now incorporate capabilities for varying the penetration rate to obtain additional data on viscous and consolidation parameters.

ANCHORING SYSTEMS

Although dynamic positioning of ships and mobile drilling units (MODUs) has become increasingly sophisticated and robust, often with three independent systems in case of failures, anchors are still required for extended position keeping. This includes temporary anchoring of MODUs for well drilling and permanent anchoring of floating production systems or as restraints to prevent pipeline movement. The choice for temporary anchors is dictated by considerations of how rapidly the anchor can be deployed, and also recovered and re-positioned, since the high day-rates for drilling units tend to swamp hardware costs. Low cost disposable anchors, such as the torpedo anchors discussed later, may be a viable alternative to recovering and re-positioning anchors. Conversely, permanent anchors require a robust and well-proven design basis, with less emphasis on the installation efficiency or ability to be recovered.

The main types of anchor in current use are:

1. Suction caissons, also referred to as suction piles or suction anchors, which are probably the most common choice for permanent moorings.
2. Drag anchors, either of conventional fluke design or the more recent low profile plate anchors designed to withstand a significant uplift component of loading (so-called vertically loaded anchors or VLAs).
3. Plate anchors installed with a mandrel, the most widespread being suction embedded plate anchors (SEPLAs), where a suction caisson is used as the mandrel (see FIG. 8a).
4. Gravity, or dynamically penetrated, anchors, which can range from very simple concrete-filled steel pipes fitted with a nose cone and flukes to the more sophisticated geometry of the Deep Penetrating Anchor (Lieng et al. 2010); a recent variant in this category, although it has some performance features similar to a drag anchor, is Delmar's proprietary OmniMax anchor (FIG. 8b).

Design approaches for suction caissons, both in regard to installation and in terms of long-term capacity are now well established (Andersen et al. 2005). The performance

(a) Inserting SEPLA in caisson

(b) Deploying OmniMax anchor

FIG. 8. Example anchoring systems.

of conventional drag anchors was originally explored empirically, with results of field testing by the US Navy summarized in NCEL (1987). Prediction of the kinematics and ultimate holding capacity of drag-embedded anchors of either type has become more sophisticated over the last decade (Murff et al. 2005). The general approach follows that of O'Neill et al. (2003): a yield function is defined in terms of normal, N, parallel (shear), S, and moment, M, loading, with hardening or softening linked to the change in embedment (and hence average shear strength), and kinematics determined according to an associated flow rule. The typical form of yield function is

$$f = \left(\frac{N}{N_{max}}\right)^q - 1 + \left[\left(\frac{M}{M_{max}}\right)^m + \left(\frac{S}{S_{max}}\right)^n\right]^{\frac{1}{p}} = 0 \tag{2}$$

Values of N_{max}, M_{max} and S_{max} may be obtained from numerical or analytical results and are expressed in terms of the area, A, of the anchor plate, the anchor dimension, B (in the plane of the mooring chain) and the local shear strength, s_{u0}, at the anchor centroid. Parametric solutions for drag anchors have been developed to explore the effect of geometry and shank resistance (Aubeny and Chi 2010). The form of the yield envelope may also be extended to investigate the effects of out of plane loading on the anchor capacity (Yang et al. 2010).

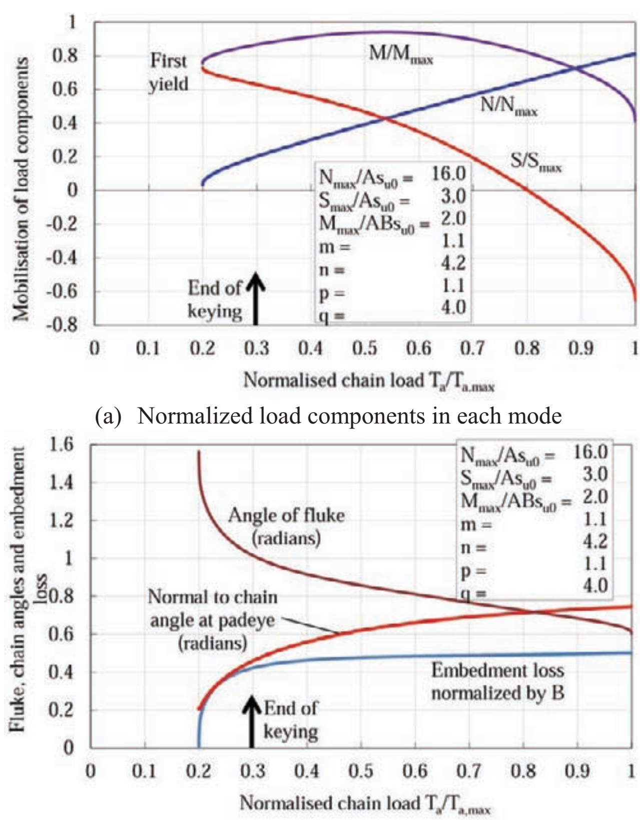
Mandrel-Installed Plate Anchors

The main challenge for drag anchors and VLAs is to evaluate how deep they will embed under the action of load applied through a mooring chain. Although the mooring chain is usually configured with a length of chain lying on the seabed, as the anchor embeds deeper, the chain profile within the soil becomes increasingly curved in an inverse catenary, so that the loading angle at the padeye increases. This eventually prevents further embedment of the anchor.

By contrast, mandrel-installed plate anchors such as the SEPLA are installed to a known depth, but with the main anchor plate (or fluke) in a vertical orientation. As the mooring chain (originally also vertical) is tensioned, it cuts through the soil with a gradually decreasing curvature; meanwhile the plate anchor rotates to become (approximately) normal to the chain – a process known as 'keying'. Since the mooring chain is initially vertical, it tends to lift the anchor, reducing embedment, in the early stages of keying, which will reduce the final anchor capacity because it moves into softer soil. Attention has therefore focused on the extent of embedment loss, which has been explored through a combination of reduced scale field tests (Wilde et al.2001), and extensive physical and numerical modeling (Gaudin et al. 2006, 2010, Song et al. 2008, Wang et al. 2011).

An interesting design feature of the SEPLA is the incorporation of a hinged flap at the top of the main fluke (see FIG. 8a). The flap has an eccentric hinge (on the shank side of the plane of the fluke) so that friction during vertical motion will act to rotate the flap to its limit of 20 º; it is also restrained from rotating towards the shank. However, experimental modeling shows that the flap does not rotate during the early part of keying, and indeed not until the anchor starts to undergo significant normal motion towards the shank and mooring chain (Gaudin et al. 2010). Recent numerical analysis (Tian et al. 2012) and analytical studies based on a yield-surface approach (Cassidy et al. 2012) have confirmed this behavior; rotation of the flap is prevented by rotation of the anchor as a whole, which results in pressure on the rear of the flap during the keying process.

The configuration of the SEPLA shank, and presence of the keying flap, results in the padeye being eccentric relative to the center of the combined fluke plate and flap. While this increases the moment during keying, helping the anchor rotate, the anchor ultimately rotates beyond the normal to the mooring chain. This leads to a significant reduction in the potential holding capacity of the anchor. This may be explored using software such as CASPA (Cassidy et al. 2012), where the yield function in (2) is used to determine the anchor kinematics during keying and loading.



(a) Normalized load components in each mode

(b) Fluke and chain angles, and normalized embedment loss

FIG. 9. Example response of SEPLA during keying and loading.

Example results are provided in FIG. 9, for a 4 m SEPLA, including a keying flap that represents 30 % of the anchor. Yield envelope parameters are as detailed. As the anchor is loaded, the software continually adjusts the angle of the mooring chain at the anchor padeye, according to the chain solution of Neubecker and Randolph (1995) assuming a fixed angle (in this case 40 º) at the seabed surface. Initial yield occurs for a padeye load (T_a) that is 20 % of the maximum (failure) value mobilized during the simulation. Initially the loading is dominated by shear and moment. As keying continues, the shear component decreases and the normal component increases monotonically, while the moment increases initially and then decreases.

In this example, the loss of embedment is about half the anchor dimension, B. The other key results are that: (a) at maximum mooring load ($T_a = T_{a,max}$) the normal force component is only 80 % of its potential maximum; (b) the anchor rotates beyond normal to the anchor chain (reached at about 80 % of the maximum mooring load), eventually reversing the sign of the shear force on the fluke. The maximum mooring load was only 85 % of the value attained for the same plate geometry but with the padeye located centrally relative to the overall fluke plate and keying flap.

Dynamically Penetrated Anchors

Over the last decade there has been a gradual increase in the use of dynamically penetrated anchors. These originated offshore Brazil with simple torpedo-shaped anchors, typically with four flukes towards the back (or upper end) of the anchor (Medeiros 2002, Araújo et al. 2004). Other developments include the DPA (deep penetrating anchor) in the North Sea (Lieng et al. 2010), and the OmniMax anchor (Zimmerman et al. 2009). These anchors, weighing 80 to 100 tonnes and 10 to 17 m long, are designed to be released from 50 to 100 m above the seabed, reaching velocities of between 20 and 30 m/s at the point where they enter the soil. Final tip penetrations are between 1.5 and 3 times the length of the anchor.

The actual penetration for a given soil profile and anchor properties may be estimated using conventional approaches for the bearing and frictional resistance on the anchor surfaces, but augmented significantly because of the high strain rates. In back-analysing a suite of centrifuge model tests, the deduced rate dependency was equivalent to between 20 and 30 % increase per logarithmic cycle (O'Loughlin et al. 2009). Such high rate dependency of the penetration resistance is a consequence of the extremely high strain rates; simple logarithmic and power law models fitted to data over 2 to 3 orders of magnitude rate increase tend to under predict the rate dependency when extrapolated over greater orders of magnitude change in strain rate (Lunne and Andersen 2007).

The torpedo anchor and DPA have the padeye at the back of the anchor, and are designed to be loaded at an angle from the vertical as the mooring chain (which is initially vertical above the penetrated anchor) cuts through the soil. Detailed investigation of the anchor holding capacity using numerical analysis shows that the optimum loading angle is such that the lateral (loading angle normal to axis) and axial capacities are mobilised equally (de Aguiar et al. 2009). For typical torpedo pile dimensions and strength profile, the two capacities are similar, so the optimum loading angle is about 45 º from the horizontal.

In deep water, the angle of the mooring line at the seabed is typically around 30 to 40 º. The loading angle at the anchor will be somewhat greater, because of the reverse catenary shape of the line through the soil. Axial capacity of the anchor is therefore critical and model tests have shown that this takes significant time to develop. Data from four series of tests are shown in FIG. 10 (Richardson et al. 2009). Two types of anchor, with no flukes or four flukes, were tested, and each type was installed either 'statically' from zero drop height above the seabed, or 'dynamically' with an impact velocity of 13 to 16 m/s. The consolidation time has been normalized by the anchor diameter and 'horizontal' coefficient of consolidation, c_h, obtained from piezocone dissipation tests. The anchors installed dynamically have extremely low axial capacity immediately after installation, and show T_{50} and T_{90} times of approximately 1 and 50. For a typical 1 m diameter anchor, and c_h values in the range 10 to 30 m^2/year, these would give actual consolidation times of 12 to 36 days for t_{50} and 1.5 to 2 years for t_{90}. These are significantly longer than the t_{90} times of 90 days or so reported for suction caissons (Jeanjean 2006).

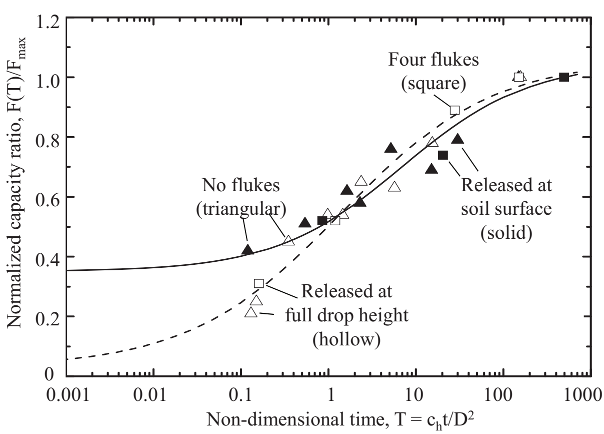


FIG. 10. Development of axial capacity with time for torpedo piles.

By contrast with torpedo anchors and the DPA, Delmar's OmniMax anchor has a fixed angle (but adjustable) shank positioned partway along the anchor shaft (FIG. 8b). The shank is able to rotate freely about the axis of the anchor, allowing for any misalignment during installation (Shelton 2007). The shape was optimized using model tests in transparent 'soil', such that the anchor tends to embed itself deeper as it is loaded, as shown schematically in FIG. 11. This aspect of their performance was demonstrated convincingly in 2008 during Hurricane Gustav in the Gulf of Mexico.

The 9 m long anchors, weighing 39 tonnes in air, achieved tip embedments of 16 to 18 m. After the extreme loading from Hurricane Gustav, which resulted in breaking of 6 of the 8 mooring lines, the anchors were recovered from depths ranging from 19 to 36 m. Back-calculated peak applied loads ranged from 3 to 5.5 MN, or efficiency factors of up to 14 in respect of the anchor weight in air (Zimmerman et al. 2009). The anchor design therefore combines the simplicity and low cost of dynamic self-weight installation, with robust performance and high efficiency from its tendency to dive.

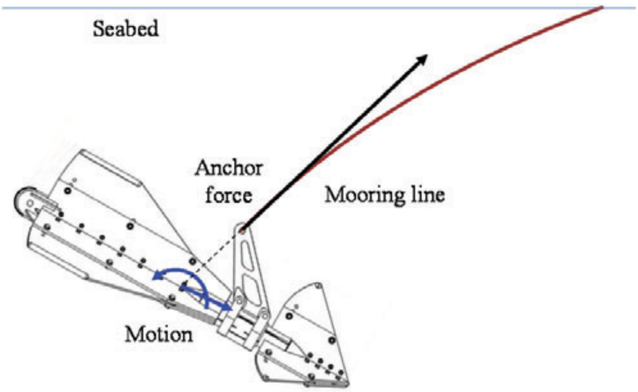


FIG. 11. Schematic of OmniMax performance and tendency to dive.

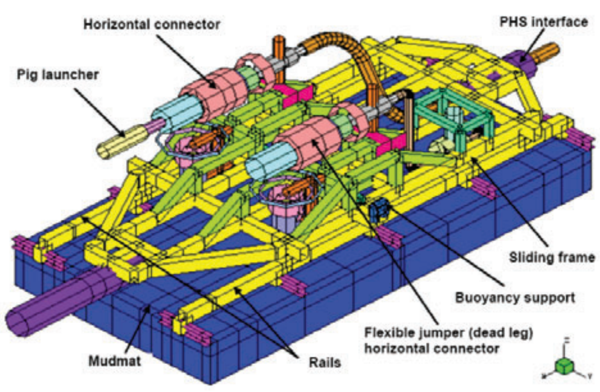


FIG. 12. Example mat foundation for pipeline end termination.

SHALLOW MAT FOUNDATIONS

Steel mat foundations are used extensively in deepwater projects in conjunction with pipelines, flowlines and risers, for example as end terminations (see example in FIG. 12, courtesy Subsea7), manifolds and riser bases. Typically, they are rectangular in plan area, with short skirts that penetrate the seabed by 0.5 to 1 m. Design approaches based on conventional bearing capacity theory, such as documented in API (2011), have led to a gradual increase in the required size of the foundation, up to the limit of approximately 10 m wide by 20 m long that can be handled by modern pipeline installation vessels.

Apart from the self-weight of the foundation and associated equipment, the various pipeline and jumper connections result in complex six degree of freedom loading, including a significant torsional component. Efficient foundation design requires more precise analysis than is provided in current design guidelines, and there is increasing focus on the use of failure envelopes in force and moment load space as a means to assess stability. Published work does not cover all aspects of the general design problem, such as general vertical V, horizontal H and moment M loading, strength heterogeneity (quantified by kB/s_{u0}, where k is the shear strength gradient, B the width or diameter of the foundation and s_{u0} the shear strength at skirt tip level), embedment and shape of the foundation, and the extent to which tensile stresses are permitted to develop beneath the foundation. A summary of various failure envelopes proposed for shallow foundations loaded under undrained conditions is provided in Table 1.

Table 1. Summary of existing failure envelopes for undrained limit states of shallow foundations under general loading

Reference	Load case	kB/s_{u0}	Surface S or embedment depth d/B	Strip S, Circular C or Rectangular R	Tension allowed	Closed form
Bransby & Randolph (1998)	VHM	0 to ∞	S	S	Y	Y*1
Bransby & Randolph (1999)	VHM	0 to 6	d/B = 0.17	S	Y	Y*2
Taiebat & Carter (2000)	VHM	0	S	C	Y	Y
Taiebat & Carter (2002)	VM	0	S	C	N	N
Gourvenec & Randolph (2003)	HM	0 - 10	S	S, C	Y	N
Finnie & Morgan (2004)	HT	0	S	S, C, R	-	Y
Yun & Bransby (2007)	HM	0 & 200	d/B = 0-1	S	Y	Y
Gourvenec (2007a)	VHM	0	S	R	N Y	Y N
Gourvenec (2007b)	VHM	0	S	S, C	Y	N
Gourvenec (2008)	VHM	0	d/B = 0-1	S	Y	N
Bransby & Yun (2009)	VHM	0 & 200	d/B = 0-1	S	Y	Y*3
Yun et al. (2009)	VHT	0	S	S, C, R	-	N
Gourvenec & Barnett (2011)	VHM	0 - 6	d/B = 0-1	S	Y	Y
Taiebat & Carter (2010)	VHM	0	S	C	N	N
Murff et al (2010)	HT	0	d/B = 0-0.05		-	Y

*1: Closed form expression intended for range of soil strength heterogeneity, although given exponents validated only for $kD/s_{u0} = 6$

*2: Expression from Bransby & Randolph (1999) adopted for low embedment ratios

*3: Based on expression in Yun & Bransby (2007)

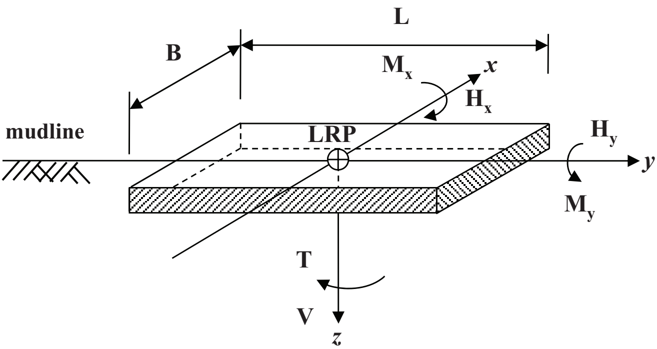


FIG. 13. General loading of a rectangular foundation of skirt depth, d.

The general loading of a rectangular foundation, following a right-hand rule (Butterfield et al. 1997), is shown in FIG. 13. From a design perspective it is convenient to define the load reference point for calculation of moments at the foundation center at seabed level, even though failure envelopes (particularly for combined H and M loading) are more consistent based on an adjusted moment at skirt tip level. In most cases, the soil strength profile may be idealized sufficiently as varying linearly with depth according to

$$s_u = s_{um} + kz \tag{3}$$

where s_{um} is the shear strength at the seabed (or mudline) and k is the gradient with depth z. The reference shear strength at skirt tip level is then $s_{u0} = s_{um} + kd$, and the maximum value of the strength heterogeneity, $\kappa = kB/s_{u0}$, is B/d.

During the later stages of design, particularly for critical cases, the foundation performance may be assessed by means of three-dimensional finite element analysis. However, initial sizing of mat foundations needs a simpler approach that allows the effect of different load combinations and mat geometry to be explored efficiently. Typically, the vertical load from the mat and equipment is well below 50 % of the 'uniaxial' vertical capacity, so that the main focus is on the 'live' loading H, M and T. The aim of the design process is to arrive at a foundation geometry that provides an adequate factor of safety on the live loading, or reserve strength ratio (inverse of material factor) for the soil. Ideally, the (factored) design load would be plotted on a design chart that shows relevant slices through the complex (six degree of freedom!) failure envelopes.

A set of possible steps to achieve this are summarized in Table 2. Uniaxial capacities for each load component are evaluated first, drawing on available published solutions for non-dimensional capacities V_{ult}/As_{u0}, $H_{x,ult}/As_{u0}$, $M_{x,ult}/ALs_{u0}$ etc (where A is the plan area, BL, of the foundation), expressed as functions of B/L, d/B and κ. Interaction effects between different loads are best evaluated by considering failure envelopes in a normalized form, for example V/V_{ult} versus $M_x/M_{x,ult}$ etc. Minor corrections (given the low ratio V/V_{ult}) may then be made to the ultimate values of H, M and T because of the partial mobilization of vertical capacity.

Table 2. Design steps for mat foundation

Step	Details
1	For given foundation geometry evaluate s_{u0} and non-dimensional quantities B/L, d/B and κ
2	Evaluate uniaxial capacities for vertical, horizontal, moment and torsional loading
3	Reduce ultimate horizontal, moment and torsional capacities to maximum values available, according to mobilised (design) vertical capacity, $v = V/V_{ult}$
4	For given angle of resultant horizontal load, evaluate corresponding ultimate horizontal capacity, and similarly for ultimate moment capacity
5	Evaluate reduced ultimate horizontal and moment capacities due to normalised torsional loading
6	Evaluate extent to which applied (design) loading falls within H-M failure envelope, and thus safety factors on self-weight V, live loading H, M, T or material strength s_{u0}

Table 3. Input data for example mat foundation design

Parameter	Value	Units	Parameter	Value	Units
Width, B	6	m	Vertical load, V	700	kN
Length, L	12	m	Horizontal load, H_x	100	kN
Skirt, d	0.5	m	Horizontal load, H_x	120	kN
Mudline strength, s_{um}	4	kPa	Moment, M_x	200	kNm
Strength gradient, k	1.5	kPa/m	Moment, M_y	-360	kNm
Skirt friction ratio, α	0		Torsion, T	540	kNm

The next stage is to consider the resultant horizontal and moment loads, H_{res} and M_{res} and the angles these make with the x axis. Ultimate values of H and M may then be assessed by considering failure envelopes of $H_x/H_{x,ult}$ versus $H_y/H_{y,ult}$ and similarly for moment. As a first approximately, these failure envelopes may be taken as circular, although finite element studies show that the actual envelopes deviate from a pure circle according to actual values of B/L, d/B and κ.

The effect of torsion may be treated in much the same way as for the vertical load, by adjusting the ultimate horizontal and moment capacities. Examples of failure envelopes for combined horizontal and torsional loading are available in the literature (see Table 1), but there is rather less in the public domain in respect of moment and torsion interaction.

The final step involves plotting the design loading relative to the assessed horizontal-moment failure envelope (in the relevant loading plane), for the given magnitude of torsional loading. An example of this is shown in FIG. 14, for the set of input parameters given in Table 3. The design load point falls well within the failure envelope, although the factor of safety with respect to the live loads is only about 1.45. The reserve strength ratio is also 1.45 (since failure dominated by sliding and torsion).

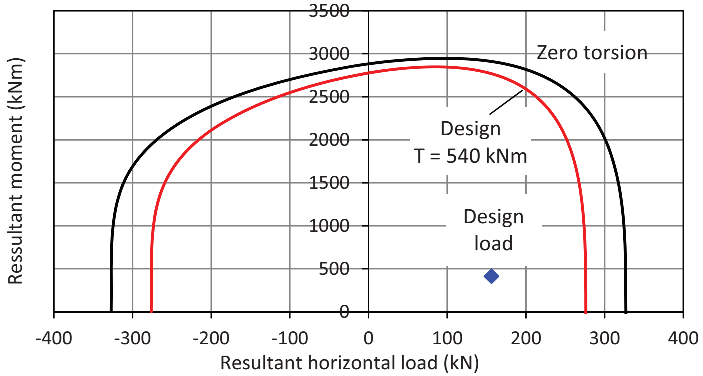


FIG. 14. Example mat foundation failure envelopes and design load in H-M space.

Pile-Enhanced Foundation Resistance

For typical design load combinations, failure tends to be dominated by sliding and torsion, since this occurs in the weakest material at the level of the skirt tips. It is therefore attractive to consider whether the sliding resistance could be improved by the addition of short piles at the corners of the foundation. BP has recently conducted a detailed study of such a 'hybrid' subsea foundation (HSF), with parallel physical and numerical modeling in addition to analytical solutions (Dimmock et al. 2012).

The piles may be relatively modest in size, for example with a penetration of the same order of magnitude as the foundation width, and length to diameter ratio of about 10. Even for an assumed pinned connection at the level of the foundation, the sliding and torsional resistance of such piles would be more than double that of the mat foundation alone; indeed for a given size of mat, the capacity to withstand the live loads on the foundation can be increased by a factor of more than 3.

The relatively high sliding capacity of the piles by comparison with the mat suggests that a rather simple (and conservative) design approach may be adopted, whereby the piles are assumed to carry the entire horizontal and torsional loads, while the mat is assumed to carry the full vertical load (self-weight of the mat and equipment). This effectively avoids the need for horizontal-moment interaction diagrams. Instead, design can be focused on ensuring adequate sliding and torsional resistance from the piles, and adequate moment capacity from the combined mat and pile resistance. This approach is discussed in detail by Dimmock et al. (2012).

PIPELINES AND RISERS

A wide range of different communication and transport lines are required within a typical deepwater field. These range from the primary export pipeline (often referred to as a trunkline), to smaller 'flowlines' connecting different wells within the field itself, MEG (mono-ethylene glycol – mixed with gas before export in order to avoid hydrate formation and help inhibit corrosion) supply lines and communication umbilicals. The main focus here is on flowlines and pipelines, with typical diameters from about 0.3 to 1.2 m, which are referred to using the generic term 'pipeline'.

The term 'riser' is used for steel pipes that carry hydrocarbon products between the seabed and a platform or floating facility. These may be in a vertical configuration, or in a catenary shape – hence the name steel catenary riser (SCR). Both types are suspended from a 'hang-off' point on the platform, but an SCR also requires a significant horizontal component of tension, provided by the length of pipeline on the seabed to which the SCR is connected and sometimes additional anchoring. One of the main design considerations for risers is a serviceability limit with respect to fatigue, which is generally assessed using specialist riser-dynamics software that models the complete system, including perturbation by waves and currents and interactions with the water and seabed. While this is outside the scope of the present paper, the configuration of an SCR is similar to that during laying of a pipeline.

In deep water, pipelines are generally laid directly on the seabed with no additional measures to provide stability, such as trenching, burial or gravity anchors. A recent review of the geotechnical challenges and design approaches for subsea pipelines has been provided by White and Cathie (2010). The main forces on the pipe arise from high internal temperature and pressure. Temperature increase due to the flow of hydrocarbon products causes longitudinal expansion, resisted by axial friction between pipeline and seabed and compressive force in the pipeline. High internal pressure, along with thermally induced compressive forces, may lead to lateral buckling of the pipe. Thermal cycles due to periodic shut down and start up of wells cause any buckles to reduce and grow again, but can also cause the pipeline to 'ratchet' axially – a process referred to as pipeline walking (Carr et al. 2003).

Design to mitigate and control lateral buckling and pipeline walking is the province of specialist pipeline engineers, but models for the lateral and axial pipe-soil interaction during these processes are a key input. The following discussion therefore considers three aspects of the geotechnical design of pipelines: pipeline embedment, lateral resistance to buckling, and axial friction.

Pipeline Embedment

The pipeline embedment is generally not a major consideration in itself, although it affects heat transfer rates from the pipeline to the surrounding water and soil and also exposure to geohazard events. However, the amount of embedment is critical in determining the lateral and, to a lesser extent, axial resistance offered by the seabed. Estimation of pipeline embedment requires consideration of the dynamic processes during laying of a pipeline on the seabed, during which it is suspended through the water in a catenary (see FIG. 15). This results in a local force concentration in the vicinity of the touchdown point, where the maximum force (per unit length), V_{max}, exceeds the submerged pipeline weight (per unit length), p. As successive segments of pipeline are welded on and fed out from the pipe-lay vessel, the contact force distribution will progress gradually along the pipeline.

The maximum (static) touchdown force relative to the submerged pipeline weight is a function of the pipeline bending rigidity, EI, the horizontal component of tension in the pipeline, T_0, and the seabed stiffness, k (ratio of vertical force to local penetration of the pipeline invert). The ratio V_{max}/p may be expressed directly in terms of a normalized stiffness parameter, K, defined as

$$K = \frac{\lambda^2 k}{T_0} \quad \text{where} \quad \lambda = \sqrt{\frac{EI}{T_0}} \tag{4}$$

as shown in FIG. 16 (Randolph and White 2008a). In water depths exceeding 500 m, and relatively soft seabed conditions, the normalized stiffness is unlikely to exceed 10 so that V_{max}/p is less than 2.

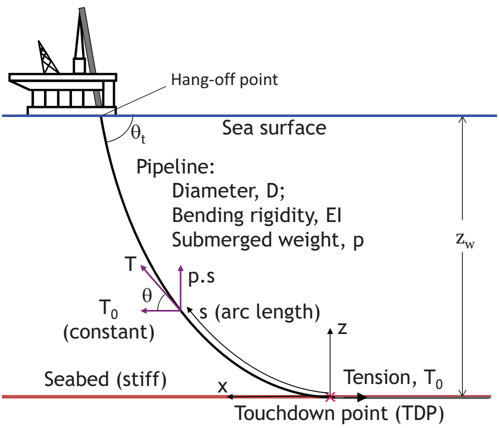


FIG. 15. Schematic of SCR or pipeline during lay process .

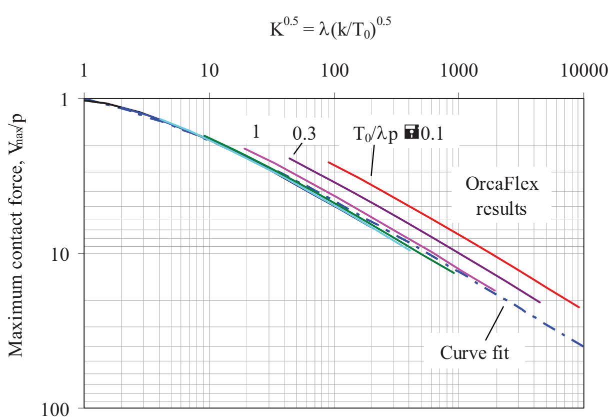


FIG. 16. Maximum pipeline contact force in touchdown zone.

Embedment under static conditions may therefore be evaluated by combining the maximum contact force with a penetration model relating the limiting resistance to the invert penetration, w, expressed as

$$\frac{V_{ult}}{s_{u,invert}D} = a\left(\frac{w}{D}\right)^b + f_b\frac{A'\gamma'}{s_{u,invert}D} \tag{5}$$

The right hand side includes two terms, the first of which relates to the geotechnical resistance, expressed in the power law form suggested by Aubeny et al. (2005), while the second accounts for buoyancy within the seabed. The (nominal) embedded cross-sectional area is A', while f_b is an adjustment factor that takes account of local heave of soil adjacent to the pipeline (Merifield et al. 2009). Detailed numerical analysis of static pipeline embedment has shown that typical values of the parameters a, b and f_b may be taken as 5.2, 0.2 and 1.5, where in addition the shear strength is adjusted for strain rate effects and partial softening (Chatterjee et al. 2011a).

The actual pipeline embedment for typical pipe-lay conditions has been found to exceed that estimated from static analysis alone. The ratio between the two may be expressed as a dynamic lay factor, f_{dyn}, the value of which can be as high as 8 (Lund 2000, Bruton et al. 2007). A simple approach, which appears to give reasonable accuracy for typical lay conditions, is to estimate the dynamic embedment directly from the static resistance given in (5), but replacing the in situ shear strength by the remolded shear strength (Westgate et al. 2010). A more detailed discussion of the various mechanisms involved in the dynamic embedment process is provided by Cheuk and White (2011).

Lateral Buckling Resistance

Following the pipe-lay process, a pipeline will generally be hydrotested (filled with water) and in due course become operational carrying hydrocarbon products. Under operational conditions, the submerged weight of the pipeline will be much less than the equivalent static vertical load, V_0, required to achieve its current embedment. The lateral resistance provided by the seabed can be expressed in terms of a yield function, the size of which is a function of the embedment, w/D. The maximum horizontal load, H_{max}, may be expressed in normalized form either as H_{max}/Ds_{u0}, where s_{u0} is the nominal shear strength at the level of the pipe invert, or H_{max}/V. These will both be an increasing function of the embedment, w/D, and respectively increasing or decreasing functions of the vertical load ratio, V/V_0.

Yield envelopes for 'ideal' geometries, ignoring the heave that occurs during penetration, have been presented by Randolph and White (2008b), based on an upper bound plasticity approach, and Merifield et al. (2008), derived from finite element analysis. A more sophisticated study using large deformation finite element (LDFE) analysis has recently been reported by Chatterjee et al. (2011b), where the full process of embedment (with associated heave) followed by lateral translation was simulated. That study led to a set of parabolic yield envelopes, as shown in FIG. 17, for pipeline embedment between 0.1 and 0.5 diameters. The yield envelopes are similar to, but rather more symmetric than, those from Merifield et al. (2008) where heave was neglected.

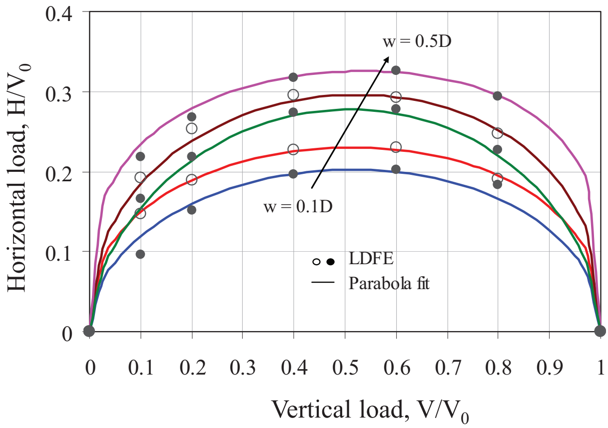


FIG. 17. Yield envelopes in V-H space comparing LDFE data with parabolic fits.

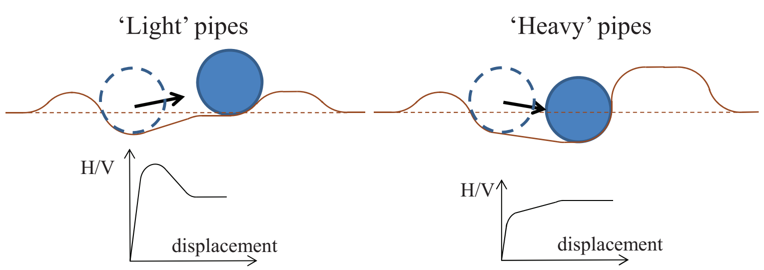


FIG. 18. Modes of failure under horizontal load for light and heavy pipes.

As the lateral buckle progresses, the horizontal resistance may either increase or decrease, depending on the weight of the pipeline relative to its embedment. Broadly, pipelines for which the normalized vertical load, V/Ds_{u0}, is less than about 2 will tend to reduce embedment as they move laterally, resulting in a softening response, while the reverse is true for heavier pipelines (Bruton et al. 2008). The differences in response are illustrated in FIG. 18. A berm of partially remolded soil is generated ahead of the advancing pipeline.

Eventually the horizontal resistance reaches a residual value, H_{res}, which is primarily a function of the current vertical load but also retains some memory of the original embedment condition of the pipeline, since that affects the size of the berm. An approximate expression for the residual horizontal resistance was proposed by White and Dingle (2011):

$$\frac{H_{res}}{V} = 0.3 + 2\left(\frac{w}{D}\right)_{initial}\sqrt{\frac{V}{V_0}} \tag{6}$$

Chatterjee et al. (2011b) proposed a more sophisticated approach from their finite element study, based on first estimating the residual embedment. Both approaches gave similarly close agreement with data from the SAFEBUCK JIP database, but the latter approach was found to have a much lower coefficient of variation.

Axial Resistance

Robust methods to assess the axial resistance of pipelines have proved remarkably elusive, with ratios of resistance, F, to submerged pipe weight, p, that can range from as low as 0.1 to over unity (White et al. 2011). There a number of factors that contribute to the difficulty in reducing this range:

- Average contact stresses beneath a pipeline are very low, typically less than 5 kPa, so that accurate data are difficult to obtain using standard laboratory equipment. Purpose designed ‘low-friction’ shear boxes, or simple tilt-table devices (Najjar et al. 2003) have been developed, yielding friction coefficients as high as 0.75 for low normal stresses.
- The curved geometry of the pipe-soil interface leads to a wedging effect, augmenting the friction ratio, F/p, compared with the friction from a planar test; the wedging factor varies with embedment to a maximum of 1.27 for w/D = 0.5 (White and Randolph 2007).
- The friction ratio is affected strongly by both the rate and extent of axial movement, due to generation and dissipation of excess pore pressures at the pipe-soil interface.

The effect of pore pressure generation and dissipation may be explored theoretically using finite element analysis. An example result is shown in FIG.. **19** (personal communication from PhD student, Ms Yue Yan), based on a coupled Modified Cam Clay soil model for initial normally consolidated conditions and a friction angle of 30 º. Depending on the rate of movement (normalized velocity vD/c_v), and length of time for which the movement is sustained (expressed as c_vt/D^2), the steady state friction shows a gradual transition from an ‘undrained’ value of 0.35 to a fully drained value of 0.65.

As noted by White et al. (2011), higher peak values of axial friction ratio may result following reconsolidation at the pile-soil interface, due to the effect of initial dilation on shearing. The resistance then drops to a residual value as axial motion continues. An interesting observation, however, was that even after very slow shearing that led to a drained residual resistance, further rapid shearing led to renewed generation of excess pore pressure at the interface, and correspondingly low friction ratios.

In order to capture these various phenomena, a theoretical framework is required that incorporates ideas from critical state soil mechanics. The effects of dilation and volumetric collapse need to be captured, but also a ‘damage’ component is required in order to explain observations from model testing. An initial attempt at such a framework is presented by Randolph et al. (2012).

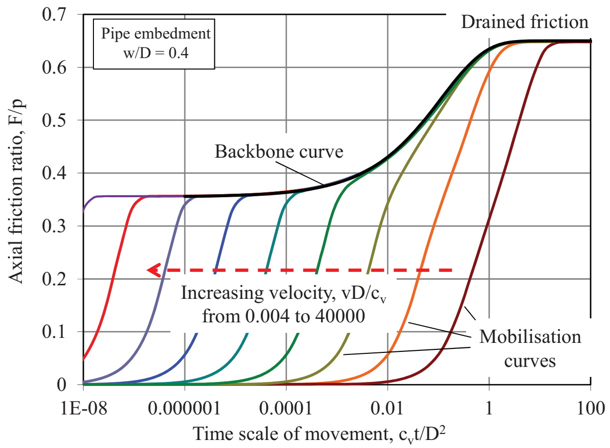


FIG.. 19. Variation of axial friction ratio from numerical analysis.

Art into Practice for Pipeline Design

Geotechnical design methods for pipelines is a rapidly developing area of knowledge, driven by deepwater developments over the last decade. The complexity of pipeline-seabed interactions, and the need to quantify the response over large cyclic displacements, has led to much greater reliance on experimental observations than for other aspects of offshore design. Initiatives such as Smartpipe, a deepwater model testing facility for pipelines developed by Fugro with support from BP (Hill and Jacob 2008), have allowed close integration of the observed response of a pipeline element in natural soil with data from immediately adjacent field penetrometer testing. Extensive model testing, either at 1 g or under centrifuge conditions, has also been conducted for many different projects using bulk samples of soil recovered from the seabed.

The experimental data have formed the basis for new design models and in parallel have spawned improved theoretical approaches that allow extrapolation outside the database. The SAFEBUCK joint industry project (Bruton et al. 2008, White et al. 2011) has proved an excellent role model in this respect for transforming art into practice.

The nature of pipeline geotechnical design, with sparsely distributed site investigation data and design calculations that rely on many different input parameters, has also led to more widespread adoption of stochastic approaches than in other geotechnical design areas. This is generally achieved using a Monte Carlo approach, with statistical input data interpreted from site investigation results leading eventually to fitted probability density functions for the output pipeline design parameters (White and Cathie 2010).

CONCLUSIONS

This paper has reviewed various aspects of offshore geotechnical design for deepwater projects, focusing on areas of recent development where design is informed by evolving evidence from experiments and numerical modeling, rather than established design codes. The principal challenges lie partly in characterizing seabed sediments that are too soft to sample and test in the conventional way, and partly in the design of infrastructure, such as anchoring systems and pipelines, that apply large deformations to the soil. Design calculations must account for damage, or softening, of the soil, and quantify the effects of strain or loading rates that may span regimes dominated either by partial consolidation or by viscous-enhanced response of the soil. Recent advances have been discussed, focusing on particular applications where the state-of-art has evolved significantly in recent years.

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Ground Improvement in the 21st Century: A Comprehensive Web-Based Information System

Vernon R. Schaefer[1], M. ASCE, P.E., James K. Mitchell[2], Dist. M. ASCE, P.E., Ryan R. Berg[3], F. ASCE, D.GE, P.E., George M. Filz[4], F. ASCE, P.E., and S. Caleb Douglas, M. ASCE, P.E.[5]

[1]Professor of Civil Engineering, Iowa State University, Ames, IA 50010-3232; vern@iastate.edu
[2]University Distinguished Professor Emeritus, Virginia Tech, Blacksburg, VA 24061-0105; jkm@vt.edu
[3]Ryan R. Berg & Associates, Inc., Woodbury, MN, 55125-3504; ryanberg@att.net
[4]Charles E. Via, Jr. Professor of Civil and Environmental Engineering, Virginia Tech, Blacksburg, VA 24061-0105; filz@vt.edu
[5]Ph.D. candidate, Civil Engineering, Iowa State University, Ames, IA 50010-3232; calebd@iastate.edu

ABSTRACT: There exist a large number of ground improvement methods that can be employed to overcome poor soil site conditions, some in use many decades, others recently developed. The growth in ground improvement methods, products, systems, and engineering tools has been tremendous, resulting in a very large body of knowledge. The selection of the most appropriate ground improvement technology is a complex undertaking that depends upon integration of available knowledge and a number problem-specific and site-specific factors. These factors are summarized and discussed in relation to the essential elements for success of a ground improvement project. A new comprehensive, web-based ground improvement information and guidance system, developed to summarize and organize this knowledge to facilitate informed decisions, is introduced and illustrated. The system can be used for engineering and construction practice that incorporates these essential elements.

INTRODUCTION

Ground improvement methods have developed markedly over the past five decades to the point where they are almost routinely used in geotechnical design and construction. The impetus for ground improvement has been both the increasing need to use marginal sites for new construction purposes and to mitigate risk of failure or potential poor performance. Every potential construction site presents the design engineer with several alternatives should unsuitable or marginal soil conditions be encountered. These alternatives include: (1) bypassing the poor soil through relocation of the project to a more suitable site or through the use of a deep foundation; (2)

removing and replacing the unsuitable soils; (3) designing the planned structure to accommodate the poor/marginal soils; or (4) modifying (improving) the existing soils, either in-place or by removal, treatment and replacement of the existing soils; or (5) completely abandoning the project (ASCE 1978; Mitchell 1981). Through a wide-variety of modern ground improvement and geoconstruction technologies, marginal sites and unsuitable in-situ soils can be improved to meet demanding project requirements, making the latter alternative an economically preferred solution in many cases.

Ground improvement is now recognized as a major sub-discipline of Geotechnical Engineering. The growth in ground improvement methods, products, systems, and engineering tools has been tremendous, with a very large body of knowledge and large number of technologies available. Progress in this development has been chronicled by means of many conferences, workshops, papers and reports - far too many for all to be cited herein. However, a few comprehensive references that describe the methods, their design and construction procedures, applications, advantages and limitations, and illustrate how the technologies have developed are noted. An early comprehensive State-of-the-Art (SOA) report on Soil Improvement was presented by Mitchell (1981) at the 10th ICSMFE in Stockholm. Recently, Chu *et al.* (2009) devoted a large part of their State of the Art report on Construction Processes prepared for the 17th ICSMGE in Alexandria to current developments in Ground Improvement. Within ASCE and the Geo-Institute three committee publications document the progress to 1997: *Soil Improvement History, Capabilities, and Outlook* (ASCE 1978), *Soil Improvement-A Ten Year Update* (ASCE 1987), and *Ground Improvement, Ground Treatment, Ground Reinforcement-Developments 1987-1997* (ASCE 1997). Numerous specialty sessions have been organized at Geo-Institute conferences, and many Geotechnical Special Publications are now available on different aspects of ground improvement.

The purpose of this paper is two-fold. First is to summarize the essential elements for success in any ground improvement project. Second is to introduce and illustrate a new comprehensive, web-based information and guidance system for use in engineering and construction of ground improvement works that incorporates these essential elements.

PART 1. ENGINEERING OF GROUND IMPROVEMENT PROJECTS

Ground improvement is the modification of site foundation soils or project earth structures to provide better performance under design and/or operational loading conditions (USACE 1999). Ground improvement objectives can be achieved using a large variety of geotechnical construction methods or technologies that alter and improve poor ground conditions where soil replacement is not feasible for environmental or technical reasons, or it is too costly (Elias *et al.* 2006). Several considerations that are essential in the selection, design, construction, validation, and monitoring of any successful ground improvement project are listed and discussed in the following sections.

Among the first questions to answer when considering ground improvement is: When and where is Ground Improvement an option? Ground improvement is an option when site soils are amenable to improvement in performance, sufficient expertise and equipment exists to accomplish the improvement, and, perhaps most importantly, the costs of improving the soils are warranted compared to other options available. Ground improvement has one or more of the following main functions (Munfakh 1997a; Elias *et al.* 2006), to:

- increase shear strength,
- increase bearing capacity,
- increase density,
- transfer embankment loads to more competent layers,
- control deformations (settlement, heave, distortions),
- accelerate consolidation,
- decrease imposed loads,
- provide lateral stability,
- form seepage cutoffs or fill voids and,
- increase resistance to liquefaction.

These functions can be accomplished by a wide variety of ground improvement and geoconstruction technologies. When improving soils at or near the ground surface, the improvement is generally easy to accomplish and relatively inexpensive (e.g. excavation and replacement). Ground improvement at depth becomes more difficult, requiring more investigation and analysis, and often specialized equipment and construction methods (Munfakh 1997a). Thus in making selections about the types of ground improvement to use at a given site, the engineer must have comprehensive and detailed knowledge of the methods available and the criteria to use in their selection.

Ground Improvement Methods

Many ground improvement and geoconstruction technologies are available to improve the properties of soils, and the methods can be categorized in a number of ways. Mitchell (1981) provided the following categories for his State-of-the-Art paper: compaction, with emphasis on in situ deep densification of cohesionless soils; consolidation by preloading and/or vertical drains and electro-osmosis; grouting; soil stabilization using admixtures and by ion exchange; thermal stabilization; and reinforcement of soil. More recently, Munfakh and Wyllie (2000) suggested eight main categories: Densification, Consolidation, Weight Reduction, Reinforcement, Chemical Treatment, Thermal Stabilization, Electrotreatment, and Biotechnical stabilization. The current International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) technical committee on ground improvement (TC 211, formerly TC17) lists five categories: improvement without admixtures in non-cohesive soils; improvement without admixtures in cohesive soils; improvement with admixtures or inclusions; improvement with grouting type admixtures; and earth reinforcement (Chu *et al.* 2009). Table 1 shows main categories, functions and methods of ground improvement methods adapted from several sources including Mitchell (1981), ASCE (1978, 1987, 1997), Munfakh and Wyllie (2000), Terashi and Juran (2000), Elias *et al.* (2006), and Chu *et al.* (2009).

Table 1. Ground Improvement Categories, Functions and Methods.

Category	Function	Methods
Densification	Increase density, bearing capacity, and frictional strength; increase liquefaction resistance of granular soils; decrease compressibility, increase strength of cohesive soils	Vibrocompaction
		Dynamic compaction
		Blasting compaction
		Compaction grouting
		Surface compaction (including rapid impact compaction)
Consolidation	Accelerate consolidation, reduce settlement, increase strength	Preloading without drains
		Preloading with vertical drains
		Vacuum consolidation
		Electro-osmosis
Load Reduction	Reduce load on foundation soils, reduce settlement, increase slope stability	Geofoam
		Foamed concrete
		Lightweight fills, tire chips, etc.
Reinforcement	Inclusion of reinforcing elements in soil to improve engineering characteristics; provide lateral stability	Mechanical stabilized earth
		Soil nailing/anchoring
		Micro piles
		Columns (aggregate piers, stone columns, geotextile encased columns, sand compaction piles, jet grouting)
		Fiber reinforcement
		Column supported embankments with load transfer platforms
		Geosynthetic reinforced embankment
Chemical Treatment	Increase density, increase compressive and tensile strength, fill voids, form seepage cutoffs	Permeation grouting with particulate or chemical grouts
		Bulk filling
		Jet grouting
		Compaction grouting
		Deep soil mixing-wet and dry
		Fracture grouting
		Lime columns
Thermal stabilization	Increase shear strength, provide cutoffs	Ground freezing
		Ground heating and vitrification
Biotechnical stabilization	Increase strength, reinforcement	Vegetation in slopes as reinforcing
		Microbial methods
Miscellaneous	Remediate contaminated soils	Electrokinetic methods, chemical methods

Selection Criteria

The selection of potentially suitable technologies, or the best suited technology for a specific project, can only be made after the evaluation of several problem-specific and site-specific factors. A list of the most important of these factors was originally developed by Mitchell (1981) and subsequently enhanced by Holtz (1989) and by Munfakh (1997b):

1. The operational criteria for the facility; e.g. stability requirements, allowable total or differential settlement, rate of settlement, seepage criteria, durability and maintenance requirements, etc. These criteria establish the level of improvement required in terms of soil properties such as strength, modulus, compressibility, and hydraulic conductivity.
2. The area, depth, and total volume of soil to be treated.
3. The soil type to be treated and its initial properties.
4. Depth to groundwater table.
5. Availability of materials, e.g., sand, gravel, water, admixtures, reinforcing elements.
6. Availability of specialized equipment and skilled labor force.
7. Construction and environmental factors; e.g., site accessibility and constraints, waste disposal, erosion, water pollution; and effects on adjacent facilities and structures.
8. Local experience and preferences; politics and tradition.
9. Time available.
10. Cost; generally construction cost, but life-cycle costs can be important.

Today the value of accelerating construction is often relayed into a cost to explicitly consider potential project savings by the use of a more expensive method to reduce time constraints. Many of the methods shown in Table 1 are best suited to certain types of soil. Figure 1 relates improvement methods to the range of soil grain sizes for which the method is most applicable.

Thus to decide among several methods, an engineer must be knowledgeable about the above factors for a wide variety of technologies. Fortunately such information is available, often in tabular form, in key references such as ASCE (1978, 1988, 1997), Mitchell (1981), Munfakh (1997a, 1997b), Elias *et al.* (2006), Holtz (1989), Holtz *et al.* (2001), and Chu *et al.* (2009). The incorporation of such information into an easily accessible on-line media is discussed in the second part of the paper.

Ground Improvement Design

The design of a ground improvement method for a particular problem is dependent upon the function of the improvement and the method(s) selected to carry out the function. The function will establish whether settlement, stability, density, geometry, and/or other parameters are the critical design parameters. Some technologies have well-established design procedures, some have a variety of published design procedures, some have proprietary design procedures, and for others design procedures are still being developed. In the second part of the paper, the design of particular ground improvement technologies is further addressed. For a particular technology, specific input and output items appropriate to the technology can be determined. These can be categorized in terms of Performance Criteria/Indicators,

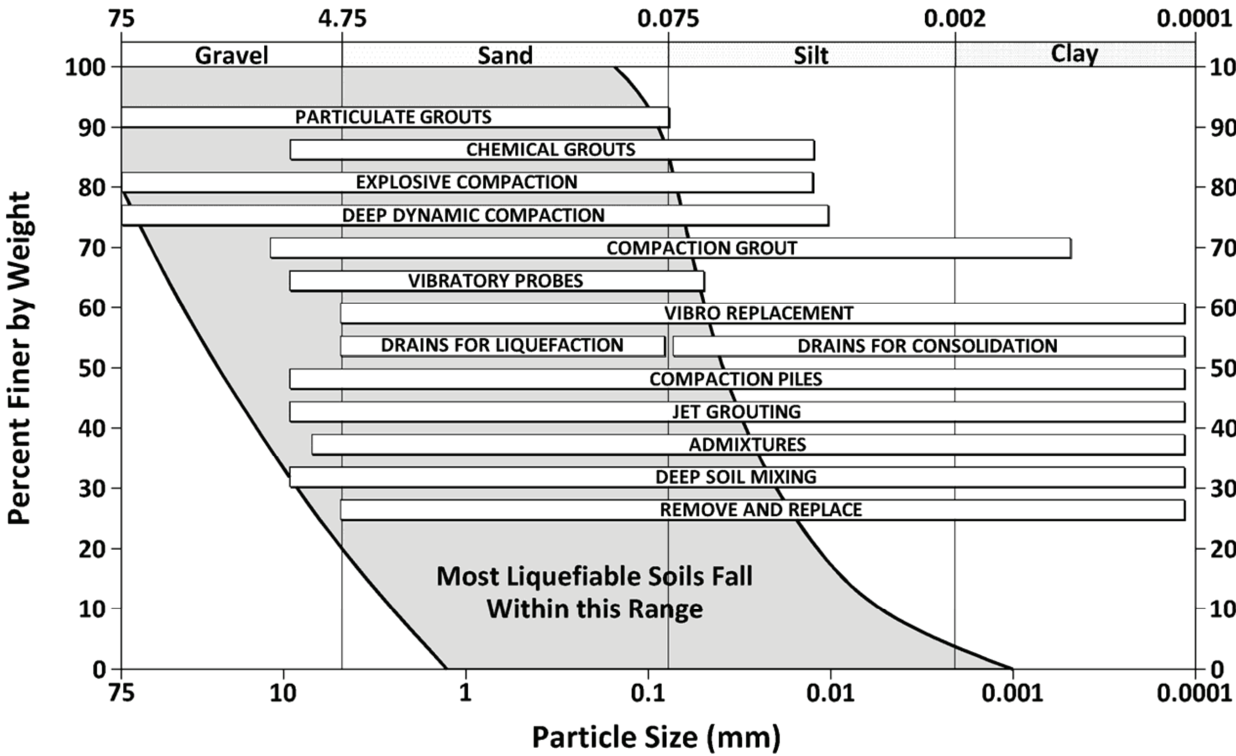


Figure 1. Available Ground Improvement Methods for Different Soil Types.

Subsurface Conditions, Loading Conditions, Material Characteristics, Geometry, and Construction Techniques. Examples of specific items in each category are listed in Table 2.

Table 2. Input and Output Items for Analysis and Design.

Categories of Input and Output Items for Analysis and Design Procedures	Some Example Items
Performance Criteria/Indicators	Minimum factor of safety values, load and resistance factor values, allowable settlements, allowable lateral deformations, reliability, drainage, time
Subsurface Conditions	Stratigraphy, ground water level, particle size distribution, plasticity, unit weight, relative density, water content, strength, compressibility, chemistry, organic content, variability
Loading Conditions	Traffic load, embankment pressure, structure loads, earthquake acceleration and duration, water pressures
Material Characteristics	Unit weight, water content, particle size distribution, internal friction angle, shear strength, inclusion dimensions, compressive strength, tensile strength, compressibility, modulus, stiffness, interface friction angle, permeability, equivalent opening size
Construction Techniques	Method of installation and/or densification, e.g., vibrocompaction
Geometry	Diameter, spacing, depth, thickness, length, area, slope

QC/QA Requirements

The satisfactory performance of improved ground is dependent upon verification that the ground improvement was constructed properly. Hence, suitable quality control (QC) and quality assurance (QA) procedures are absolutely imperative. As with design methods, the selection of appropriate QC/QA methods for a particular site is dependent upon the function of the ground improvement technology. Ground improvement effectiveness during and after construction is usually evaluated by standard methods and tests, including one or more of the following methods: inspection during construction; construction data records (power, energy input, pressures, quantities, spacings, rates, etc.); surface settlement and heave; sampling of admixture treated soil; penetration tests (SPT, CPT, BPT, DMT); shear wave velocity; undisturbed samples; pore pressure measurement; inclinometers; hydraulic conductivity (in-situ); and others. During construction, observations should be made and recorded at each improvement location. After construction, in-situ methods or performance monitoring can be used to verify that the required level of improvement is achieved; laboratory testing on undisturbed samples can also be used to verify some types of improvement.

Establishing suitable QC/QA procedures is arguably the critical limiting factor preventing more widespread application of some technologies. Providing clear, precise, and effective guidelines for QC/QA procedures will remove an important source of uncertainty that currently makes some designers hesitant to apply such technologies. Full scale test sections are an important means of assessing the expected performance of improved ground prior to full project application of a technology. In the second part of the paper, the QC/QA requirements for particular ground improvement technologies are further addressed.

Performance of Improved Ground

Several decades of experience in the use of the more well-known ground improvement methods in "conventional" applications such as bearing capacity improvement, slope stabilization, precompression and acceleration of consolidation, and construction of seepage barriers have shown that the required performance can be obtained if (1) the appropriate method is chosen for the problem and (2) the design and construction are done well. A common "trouble spot" with most methods is the difficulty in verifying that the desired level of improvement has been obtained, emphasizing again the need for a well-designed and implemented QC/QA program.

In recent years various ground improvement methods have been used at many sites to reduce the settlement and lateral spreading caused by earthquakes. Although some of these sites have been subjected to strong ground motions, few have been subjected to ground accelerations and durations of shaking as large as the design values. Nonetheless, the observed behavior confirms that ground improvement will help prevent liquefaction and ground failure from occurring, and reduce significantly the settlements and lateral displacements if liquefaction does occur (Mitchell *et al.*, 1995). Observations suggest that when sites are improved to the "no liquefaction" side of generally accepted liquefaction potential curves; e.g., Youd *et al.* (2001), the adverse effects of the earthquake shaking should be minor. Thorough analysis of sites affected by recent (2011) earthquakes in New Zealand and Japan is likely to further document quantitative understanding of improved ground performance in seismic areas.

Summary

The selection of appropriate ground improvement technologies for a particular problem is a complex undertaking that depends upon a number of the factors outlined above. The web-based guidance system described in the remainder of this paper provides an integrated and formalized basis for combining the above considerations in a way that can enable those new to the field of ground improvement, as well as experienced engineers, to obtain both guidance on the selection of methods for specific applications and projects, and detailed information about the many different ground stabilization and improvement technologies that are contained in the system. This system provides the user with critically important information with which to make sound engineering decisions appropriate for specific projects.

PART 2: A COMPREHENSIVE WEB-BASED INFORMATION AND GUIDANCE SYSTEM FOR GROUND IMPROVEMENT

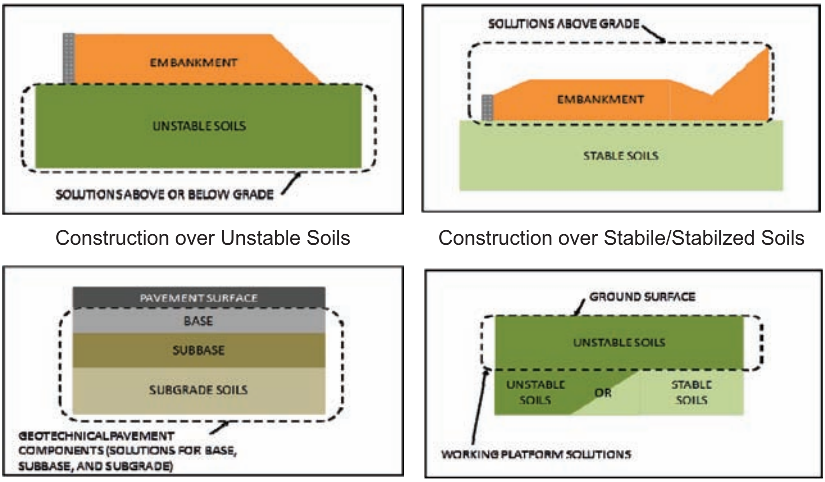
The second Strategic Highway Research Program (SHRP 2) was created by the U.S. Congress in 2006 to address challenges of moving people and goods efficiently and safely on the nation's highways. SHRP 2 has four main focus areas: Safety, Renewal, Reliability, and Capacity, with a least a dozen projects under each area. Geotechnical transportation issues are addressed under the SHRP 2 Renewal Focus Area, in which the goal is to develop a consistent, systematic approach to the conduct of highway renewal that is (1) rapid, (2) causes minimal disruption, and (3) produces long-lived facilities. The SHRP 2 R02 project is aimed at identifying geotechnical solutions for three elements: (1) construction of new embankments and roadways over unstable soils, (2) widening and expansion of existing roadways and embankments, and (3) stabilization of the working platform through a project titled: *Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform*. The R02 research team identified a large number of ground improvement and geoconstruction technologies and processes applicable to the three elements. The number of technologies was winnowed to 46 particularly applicable to the three elements, shown in Table 3. For each of these technologies the research team developed a comprehensive technical summary and assessed the current state of the practice of design, QC/QA, costs, and specifications. The resulting information was cataloged in a database and made accessible through a web-based system.

The main product of the R02 project is a web-based information, guidance, and selection system for geoconstruction and ground improvement solutions. The value of the system is that it collects, synthesizes, integrates, and organizes a vast amount of critically important information about geotechnical solutions in a system that makes the information readily accessible to the user. The target audience for the system is primarily public agency geotechnical engineering personnel at local, state, and federal levels. However, civil/structural, construction, pavement, and construction engineers in consulting, contracting, and academia will also find the system useful, as will transportation managers and decision makers. Although developed for the transportation industry, the technologies in the system can be applied equally well to non-transportation projects, and thus the system should have broad appeal to the overall geotechnical community.

The system was developed along the lines of the three elements; however, the final applications were divided into four areas, as shown in Figure 2. The system was developed with input from the research team members, the project Advisory Board, an Expert Contact Group, Federal Highway Administration (FHWA), and SHRP 2. Meetings were conducted throughout the project to bring together state agency transportation personnel, practitioners, contractors, and academics who work with the relevant geotechnical materials, systems, and technology areas. These meetings provided valuable brainstorming opportunities to identify technical and non-technical obstacles limiting widespread effective use of these technologies; to identify the available best opportunities for advancing the state of practice of existing and emerging technologies; and future directions of these technologies in transportation works. Comments from these meetings assisted in developing the objectives and strategies of the final system. The goal of the system is to provide a comprehensive tool that provides guidance for applying these geoconstruction solutions to transportation infrastructure.

Table 3. List of Technologies in the System

Aggregate Columns	Geotextile Encased Columns
Beneficial Reuse of Waste Materials	High-Energy Impact Rollers
Bio-Treatment for Subgrade Stabilization	Hydraulic Fill + Vacuum Consolidation + Geocomposite Drains
Blasting Densification	Injected Lightweight Foam Fill
Bulk-Infill Grouting	Intelligent Compaction
Chemical Grouting/Injection Systems	Jet Grouting
Chemical Stabilization of Subgrades and Bases	Lightweight Fill, EPS Geofoam, Low-Density Cementitious Fill
Column-Supported Embankments	Mechanical Stabilization of Subgrades and Bases
Combined Soil Stabilization with Vertical Columns (CSV)	Micro-Piles
Compaction Grouting	Mechanically Stabilized Earth Wall Systems
Continuous Flight Auger Piles	Onsite Use of Recycled Pavement Materials
Deep Dynamic Compaction	Partial Encapsulation
Deep Mixing Methods	Prefabricated Vertical Drains and Fill Preloading
Drilled/Grouted and Hollow Bar Soil Nailing	Rapid Impact Compaction
Electro-Osmosis	Reinforced Soil Slopes
Excavation and Replacement	Sand Compaction Piles
Fiber Reinforcement in Pavement Systems	Shoot-in Soil Nailing
Geocell Confinement in Pavement Systems	Screw-in Soil Nailing
Geosynthetic Reinforced Construction Platforms	Shored Mechanically Stabilized Earth Wall System
Geosynthetic Reinforced Embankments	Stone Columns
Geosynthetics Reinforcement in Pavement Systems	Vacuum Preloading with and without PVDs
Geosynthetics Separation in Pavement Systems	Vibrocompaction
Geosynthetics in Pavement Drainage	Vibro-Concrete Columns



Construction over Unstable Soils

Construction over Stabile/Stabilzed Soils

Geotechnical Pavement Components (Base, Subbase, and Subgrade)

Working Platforms

FIG. 2. Illustration of four application areas for the technologies.

Framework for the System

The development of the information system required planning on several levels. The framework for development required defining (1) overall system characteristics, (2) the user, (3) the knowledge, (4) the operating system, and (5) the approach to the system. The details of this development are summarized in Schaefer *et al.* (2011) and contained in the web-based system development report (Douglas *et al.* 2011).

The overall system developed is termed an *information and guidance system* because this system is meant to guide the user in selecting appropriate geoconstruction technologies for the project at hand. The knowledge base is contained in tables and the inference engine is shown graphically through flow charts. The flow charts and tables were programmed into a web-based system for ease of use. The system is intended to be used by both technical and nontechnical personnel, although to different levels.

The knowledge for identifying potentially applicable technologies to a set of geotechnical and loading conditions comes from the R02 team's work efforts, including the development of Comprehensive Technology Summaries (CTS), Design Procedure Assessments, and QC/QA Assessments for each of the technologies listed in Table 3. CTS development entailed development of an in-depth technology overview that included advantages, potential disadvantages, applicable soil types, depth/height limits, groundwater conditions, material properties, project specific constraints, equipment needs, and environmental considerations. Additionally, for each technology case histories, design procedures, QC/QA procedures and specifications were collected. The assessment efforts then qualitatively and quantitatively assessed the present design and QC/QA methods. The development of these CTS and assessment documents provided significant technical information related to each technology and the application of that technology with regard to geotechnical and loading conditions. Available FHWA manuals and guidance documents were identified in the CTS and assessment work efforts, and the information in those documents has been incorporated into the system.

The web-based system is programmed utilizing Adobe ColdFusion® software in conjunction with a Microsoft Access® database. This combination of software allowed the tables developed as part of the selection system to be ported to a database which could be dynamically queried via the web. The desired characteristics of the operating system were (Chouicha and Siller 1994):

1. Built-in mechanisms such as searching, control, and backtracking.
2. An internal database to hold the knowledge base.
3. Tools with windows, menus, frames, and drop boxes.
4. The ability to house the system on a server and allow the program to be run by multiple users via the World Wide Web.

Like most geotechnical analytical solutions, the results of the analysis must be measured against the opinion of an experienced geotechnical engineer practicing in the local area of the project. The system was developed with a "keep the system simple" philosophy, using two approaches. The first approach is that the system conservatively removes potentially inapplicable technologies during the process. The second approach, which will be a common theme throughout the selection procedure, is that the final selection of the appropriate technology will be the responsibility of the user. The system will lead the user to multiple technologies and provide all the means for technology explanation, design, and cost estimating. This system does not replace the project Geotechnical Engineer. The Geotechnical Engineer's "engineering judgment" is the final selection process, which takes into consideration the following: construction cost, maintenance cost, design and quality control issues, performance and safety (pavement smoothness; hazards caused by maintenance operations; potential failures), inconvenience (a tangible factor, especially for heavily traveled roadways or long detours); environmental aspects, and aesthetic aspects (appearance of completed work with respect to its surroundings) (Johnson 1975 and Holtz 1989).

The Web-based Information System

The homepage for the web-based information system is shown in Figure 3. The title of the web page is shown in the upper left. Along the left hand side of the page are buttons to the home page, project background, geotechnical design process, the catalog of technologies, the technology selection system, glossary, abbreviations, frequently asked questions, submit a comment, links, and an about this website, that are always available to the user. The part outlined in the bold box will change as other pages are selected. In subsequent screen shots only the material within the bold box will be shown. As shown within the bold box in Figure 3, there are four main parts to the system: Geotechnical Design Process, Catalog of Technologies, Technology Selection, and Glossary.

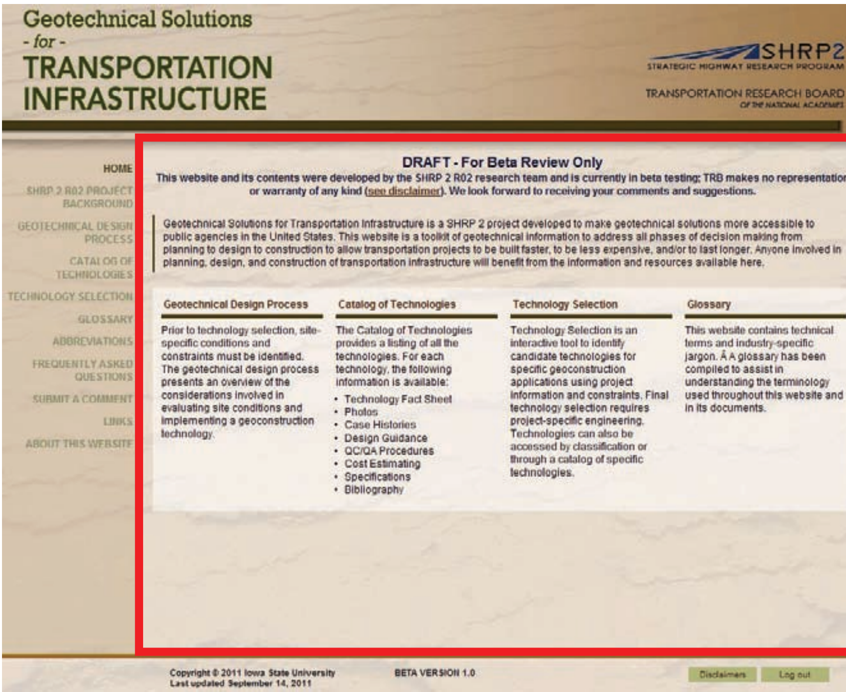


FIG. 3. Homepage for the SHRP 2 R02 project information and guidance system.

The Geotechnical Design Process page is included to alert the user to the basic background information needed to conduct geotechnical design such as project loading conditions and constraints, soil site conditions, and evaluation of alternatives. The page contains links to FHWA documents on review of geotechnical reports, evaluation of soil and rock properties, subsurface investigation and instrumentation. Additionally, links to several state departments of transportation geotechnical design manuals are provided. During the development of the system it was realized that a large number of technical terms and abbreviations were used and that in some cases different technologies used terms in different ways. Thus, an Abbreviations and Glossary is included with the system so that system users are able to find definitions of terms used in the various documents.

The technologies can be accessed in several ways. The Catalog of Technologies page provides a listing of the 46 ground improvement and geoconstruction technologies in the system that addresses the three element areas. Two traditional technologies—excavation and replacement, and traditional compaction—are included as they are often-used "base" technologies, to which ground improvement and geoconstruction methods are compared. The list of technologies in the catalog is shown in Table 3. The name of each technology is a hot-link button on the website that takes the user to a web page for that technology, which will be discussed in more detail subsequently. The Technology Selection page provides two further means of accessing technologies: through a classification system and through an interactive selection system. In the classification system, the technologies are grouped in the categories shown in Table 4. Thus an experienced engineer can access solutions according to particular categories of problems. The interactive selection system provides the user the opportunity to assess technologies based on several applications. An information and guidance procedure has been developed for each "application" area shown in Figure 2 and as defined in the R02 project work scope. In developing the system, the importance of properly identifying the potential applications was recognized. The Interactive Selection System is entered through the screenshot shown in Figure 4, wherein the first decision in the process is to select the potential application. In the selection system the list of applicable technologies is shown on the right-hand side of the page (see Figure 4), all of which are hot-linked to the respective technology pages. At the start of the selection all technologies will be shown on the right hand side, and as decisions are made, non-applicable technologies will be grayed out.

Table 4. Classification of Geotechnical Solutions by Application Categories

Earthwork Construction	Soft Ground Drainage & Consolidation
Densification of Cohesionless Soils	Construction of Vertical Support Elements
Embankments Over Soft Soils	Lateral Earth Support
Cutoff Walls	Liquefaction Mitigation
Increased Pavement Performance	Void Filling
Sustainability	

After clicking on one of the four application areas shown in Figure 4, the user will encounter a page requesting additional information to narrow the list of candidate technologies for the particular application. The number of possible queries for additional information is quite large and is dependent upon the application selected. The requested input and order of queries to the user were selected after considering the effect of the requested information on the determination of the potential technologies list. The potential queries (in no particular order) generated during development of the system are:

- What type of project is being constructed?
- What is the size of the project being constructed?
- Are there any project constraints to be considered in selecting a possible technology?
- What is the soil type that needs to be improved?
- To what depth do to the unstable soils extend?
- At what depth do the unstable soils start?
- Is there a "crust" or "rubble fill" at the ground surface?
- What is the depth to the water table?
- How does the water table fluctuate?

- What constraints exist? (i.e., utilities, material sources, existing adjacent structures, etc.)
- What is the desired outcome of the improvement? (i.e., decrease settlement, decrease construction time, increase bearing capacity, etc.)
- What technologies does the user already have experience with?

The questions used to narrow the technologies are dependent upon the application selected. Generally, three or four questions are used to develop a short-list; which can then be further defined with answering additional questions. To illustrate the use of the system, solutions for Construction Over Unstable Soils are presented herein in more detail.

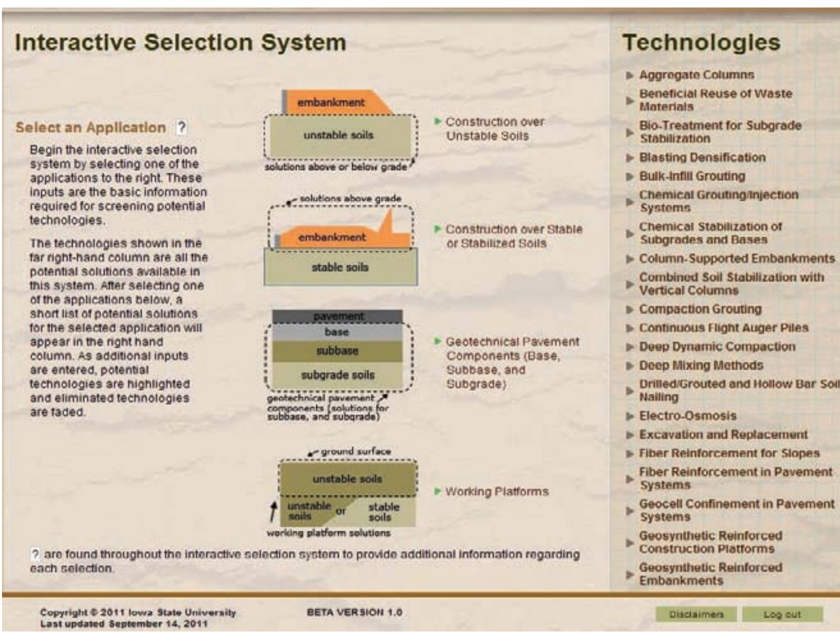


FIG. 4. Screenshot for the interactive selection system page.

Construction Over Unstable Soils

Selecting the Construction Over Unstable Soils application leads to a decision process for foundation soil improvement or reduced loading. This application is focused on ground improvement to support embankments of any height or transportation structures such as walls or box culverts over unstable soils. This system is focused on identifying geoconstruction solutions to these problems; however, users must also consider that structural solutions to such problems may be preferred alternatives.

From the list of potential queries, the two questions "What is the soil condition that needs to be improved?" and "To what depth do to the unstable soils extend?" were selected as the initial questions to reduce the number of potential technologies for this application. These two queries were found to be most useful in providing a preliminary short list of applicable technologies. A screenshot of the first page for the Construction Over Unstable Soils application is shown in Figure 5. The list of technologies shown on the right of this page has narrowed from the complete list shown on the previous Interactive Selection System page (Figure 4). The unstable soil conditions considered in the system are:

- Unsaturated and saturated, fine-grained soils
- Unsaturated, loose, granular soils
- Saturated, loose, granular soils
- Voids – sinkholes, abandoned mines, etc.
- Problem soils and sites – expansive, collapsing, dispersive, organic, existing fill, and landfills

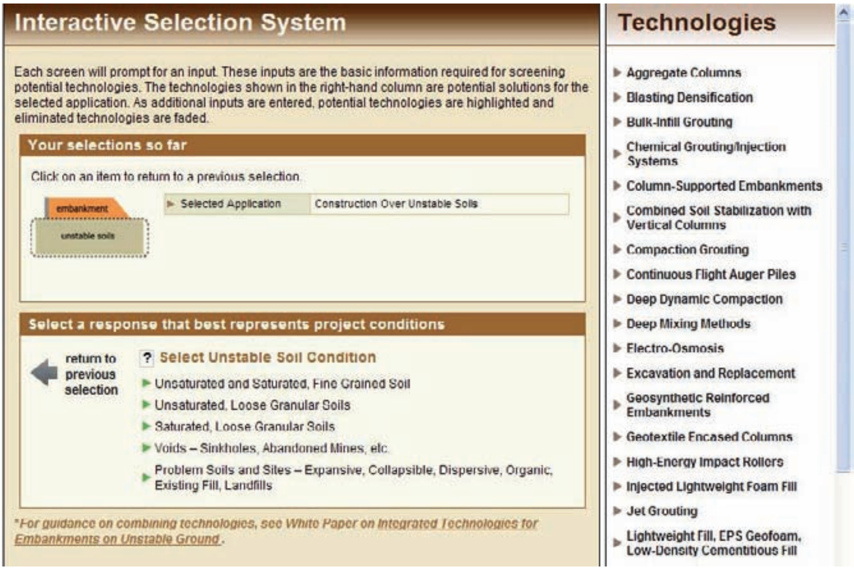


FIG. 5. Screenshot for the first Construction Over Unstable Soils page.

The screenshot after answering the soil type is shown in Figure 6. On the right-hand side of the screenshot it can be seen that several technologies are grayed out, indicating that they generally are not appropriate for the soil type selected (unsaturated and saturated, fine-grained soil).

The next question to be answered is the depth range for improvement. The depth ranges selected for inclusion in the system are

- 0 – 5 feet (ft) (0 – 1.5 meters (m))
- 5 – 10 ft (1.5 – 3 m)
- 10 – 20 ft (3 – 6 m)
- 20 – 50 ft (6 – 15 m)
- Greater than 50 ft (15 m)

After answering the unstable soil depth question additional technologies may be grayed out on the right-hand side. At this point the user can stop and assess the candidate list of technology solutions or enter additional project-specific information as shown in Figure 7. Since many of these technologies are used in combination with other ground improvement methods, guidance on combining technologies is contained in the linked Integrated Technologies for Embankments on Unstable Ground white paper (see Figure 5 or 6).

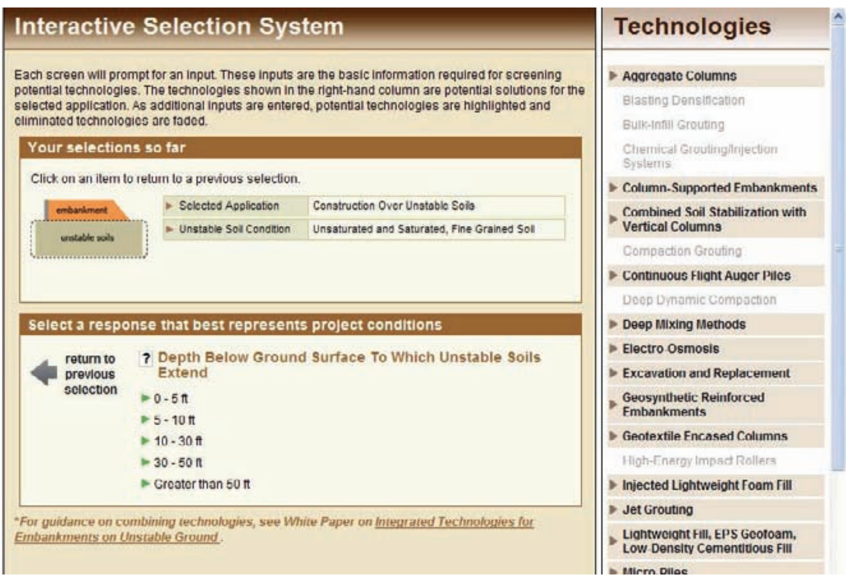


FIG. 6. Screenshot for the second Construction Over Unstable Soils page.

A final technology selection screenshot in Figure 8 shows the resulting candidate technologies on the right-hand side of the page, when the questions have been answered as shown. It can be seen that the list of technologies applicable to the selected conditions has been narrowed. At this point one can click on any of the highlighted technologies to obtain technology specific information. For example, clicking on Prefabricated Vertical Drains and Fill Preloading will bring up the screenshot shown in Figure 9. The documents listed can be accessed through hot-links on the website. Ratings are provided for each technology on the degree of technology establishment and a technology's potential application to SHRP 2 objectives.

As shown in Figure 9 a number of information documents about a given technology are accessible from the system. The list of documents available is shown in Table 5, which also indicates the format for the document. These documents are hot-linked and can be opened from this page or the box shown can be clicked and the selected documents can be printed or saved to a file for further use.

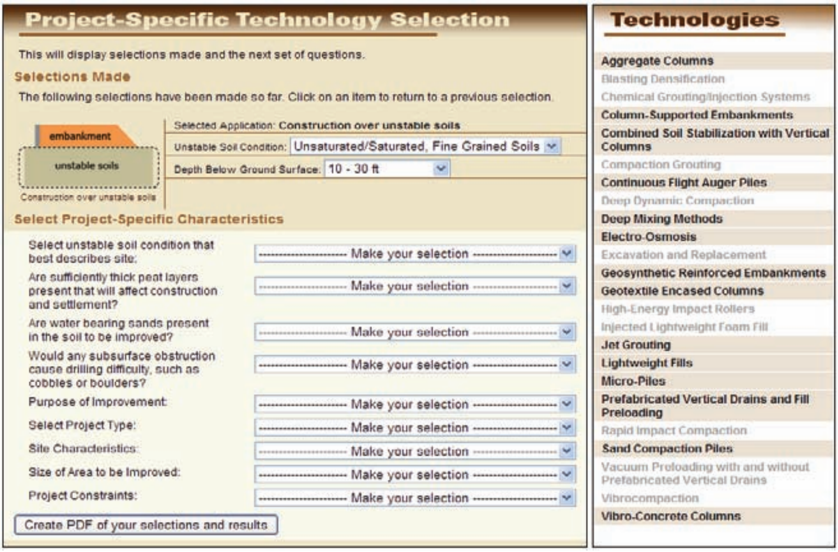


FIG. 7. Screenshot for the Project-Specific Technology Selection for Construction Over Unstable Soils.

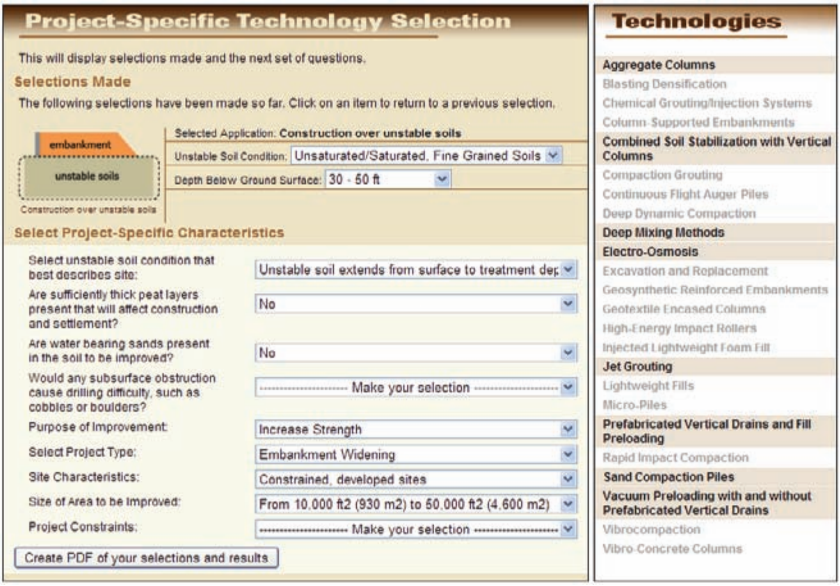


FIG. 8. Screenshot for the Project-Specific Technology Selection for Construction Over Unstable Soils.

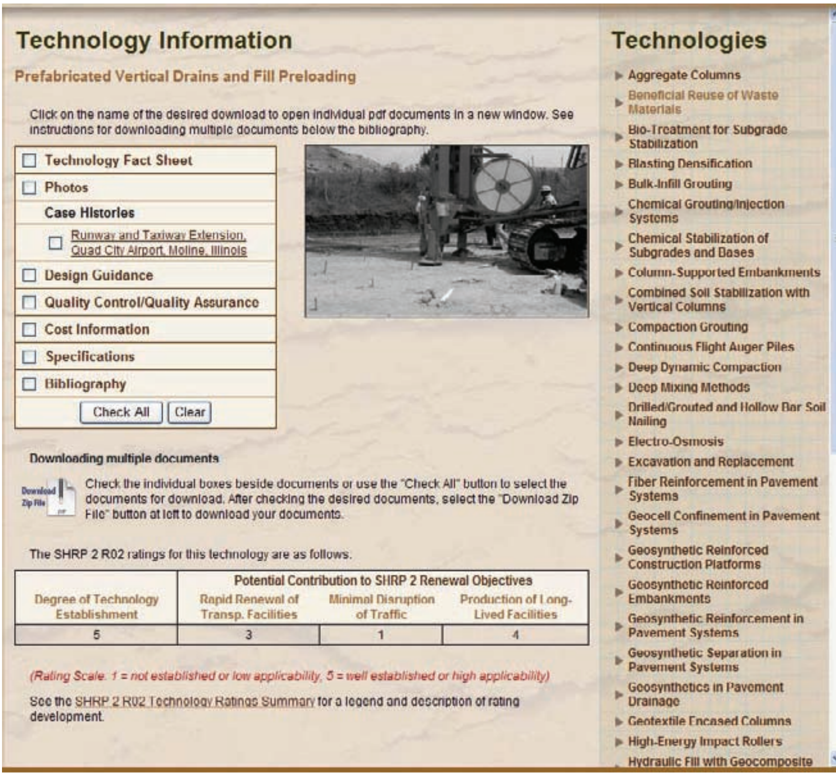


FIG. 9. Screenshot for the Prefabricated Vertical Drains and Fill Preloading Technology showing list of available documents.

TABLE 5 Documents Available Through the Information and Guidance System

Available for Review or Download	File Format
Technology Fact Sheets	Adobe pdf
Photos	Adobe pdf
Case Histories	Adobe pdf
Design Procedures	Adobe pdf
QC/QA Procedures	Adobe pdf
Cost Estimation	Adobe pdf and Microsoft Excel
Example and/or Guide Specifications	Adobe pdf and/or Microsoft Word
Bibliography	Adobe pdf

The information documents are generally provided in Adobe pdf format. The Technology Fact Sheets are two-page, summary information sheets that provide basic information on the technology including basic function, general description, geologic applicability, construction methods, SHRP 2 applications, complementary technologies, alternate technologies, potential disadvantages, example successful applications, and key references. The Photos show pictorially the equipment or methods used in the technology and can be valuable to get a perspective on the technology. The Case Histories provide a summary of project(s), in most cases conducted in the United States by a state department of transportation (DOT), that contains project location, owner, a project summary, performance, and contact information. The Design and QC/QA Procedures documents provide a summary of recommended procedures for the technology. The recommended design and QC/QA procedures come from an assessment of the current state of the practice of each technology. In cases where a well-established procedure (e.g., a FHWA manual) exists, that procedure is recommended. In cases of technologies with multiple procedures, but with no established procedure, the assessment led to a recommendation of procedure(s) to use. For a few technologies, design and/or QC/QA procedures were established based on additional research conducted during the project. For most technologies, there are two Cost Estimation documents available. The first provides an explanation of the cost item specific to the technology, generally emanating from the pay methods contained in specifications. Available regional and cost numbers, generally from DOT bid tabs or national data bases are compiled for each technology. The second document for Cost Estimation consists of an Excel spreadsheet developed to estimate costs for the use of the technology. The second document could not be prepared for some technologies due to insufficient information. The spreadsheet can be modified by the user to estimate specific project cost based on either a preliminary or final design. Guide specifications are provided for each technology in Adobe pdf and Microsoft Word (if available). The final document available for each technology is a bibliography compiled during the research project.

CONCLUSION

The selection of a suitable method of ground improvement and optimization of its design and construction of ground improvement to meet specific project needs requires extensive background knowledge of available ground treatment technologies and careful evaluation of several factors. These factors include understanding the functions of the method, utilization of several selection criteria, the use of appropriate design procedures, implementation of the right methods for quality control and quality assurance, and consideration of all relevant cost components and environmental factors.

A knowledge base has been compiled for 46 ground improvement and geoconstruction technologies, and a web-based information and guidance system has been developed to facilitate and organize this knowledge so that informed decisions can be made. The value of the system is that it collects, synthesizes, integrates, and organizes a vast amount of critically important information about ground improvement solutions in a system that makes the information readily accessible to the user.

The selection of ground improvement technologies and systems needs to be made with clear understanding of performance objectives relative to project needs and of agency governance and risk tolerance. The web-based system described herein provides the user with critically important information with which to make sound engineering decisions appropriate for specific projects.

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U.S. Levee and Flood Protection Engineering in the Wake of Hurricane Katrina

Raymond B. Seed[1], Adda Athanasopoulos-Zekkos[2], Diego Cobos-Roa[3], Juan M. Pestana[4], and Mike Inamine[5]

[1]Prof. of Civil Engin., University of California at Berkeley, CA. rmseed6@aol.com
[2]Asst. Prof. of Civil Engin., University of Michigan at Ann Arbor, MI. addazekk@umich.edu
[3]Graduate Student Researcher, Univ.of California at Berkeley, CA. dcobos@berkeley.edu
[4]Prof. of Civil Engin., University of California at Berkeley, CA. pestana@ce.berkeley.edu
[5]Dir. of Engineering, Sutter Butte Flood Control Agency, CA. m.inamine@sutterbutteflood.org

ABSTRACT: After more than half a century of increasing neglect, the nation's flood defenses are currently rated by the ASCE as having the very lowest levels of adequacy and reliability among all types of critical U.S. infrastructure, receiving a letter grade of straight "F" on the ASCE's Annual Infrastructure Report Card for the nation. Accordingly, the U.S. now finds itself at a crossroads in the wake of the recent catastrophic flooding of New Orleans by Hurricane Katrina, and the spate of other significant flood events of recent years. There is much to be learned from these recent events, and that learning opportunity has been seized upon at both the national and more local (State and regional) levels; and some promising changes have been initiated. This paper discusses key lessons learned from the Hurricane Katrina experience, and other recent flood disasters, and then presents a discussion of ongoing efforts to implement these lessons in effecting positive changes and improvements in U.S. flood protection practice.

INTRODUCTION

U.S. practice in the fields of levee engineering and flood protection has historically advanced in sporadic fits and starts, as decades of increasing neglect and lack of suitable funding and policy attention have been punctuated by infrequent major flood disasters, or groups of flood events, that drew national attention and policy and funding responses. One of the important lessons of the past century is that it is inadvisable to attempt to separate the intertwined issues of policy, standards, and funding and appropriations from the more narrowly confined "technical" issues that engineers tend more naturally to focus upon.

It is not feasible within the length constraints of this paper to attempt to discuss the nation's flood management history. Instead, three sets of events will be highlighted. The first of these were a series of catastrophic floods on the Mississippi and Ohio River systems during the 1920's and 1930's. In response, Congress passed the Flood Control Acts of 1928 and 1936 which essentially directed the U.S. Army Corps of Engineers (USACE) to establish levee engineering and flood protection methods and standards, and to apply these initially to the taming of the Mississippi River system and to California's Central Valley river systems, to be followed by further efforts in other areas of the nation. This represented a major leap forward, as prior to this there had been little or no oversight of flood control efforts (or levees), and most U.S. levees had little benefit prior to 1928 from engineering or science.

The second key event of the past century was the inception of the National Flood Insurance Program (NFIP) by Congress in 1968. This was in response to significant flood losses, often borne largely by the federal government, and was also done to provide a means for the general public to obtain more reasonably priced flood insurance. Unfortunately, a major policy gaffe occurred when the NFIP selected the 100-year recurrence level of water elevation as the standard around which the NFIP operates. This, in turn, strongly encourages communities to achieve and obtain certification only of a 100-year level of flood protection in order to exit from these onerous requirements and restrictions. That represents a level of protection lower than had routinely been targeted for important levee systems prior to 1968, and far too low a level for significant population zones and other important assets.

The third major event is the response to the catastrophic flooding of New Orleans by Hurricane Katrina, and the spate of other recent significant flood events in other parts of the nation. This, too, has triggered a Congressional response, and it has also triggered responses by the USACE and by state and local governments and agencies as well. These will be discussed at some length in the later sections of this paper, but it is worth noting at this early point that these responses have been very positive; including establishing the foundations for a National Levee Safety Program (NLSP), and a deliberative move in both policy and technical standards towards establishment of risk-based levels of protection significantly higher than the current de facto NFIP 100-year level of flood protection for non-rural populations and assets.

HURRICANE KATRINA

There had been a drumbeat of increasing flood losses across the nation in the wake of the instigation of the 1968 NFIP and the resulting wave of complacency associated with the public's perception that the 100-year level of certified protection might represent a reasonable standard of safety to aspire to for urban regions (NOAA, 2011). This complacency was shattered in 2005 by the catastrophic failures of the New Orleans regional flood protection system (FPS) and the consequent flooding of approximately 80% of the metropolitan area of a major American city. This was the most costly failure of an engineered system in U.S. history, resulting in approximately 1,600 deaths, and losses on the order of $100 to $150 billion (e.g.: GAO, 2006; RMS, 2006; U.S. Senate Committee on Homeland Security, 2006).

It is important to note that the New Orleans regional FPS system that failed so catastrophically during Hurricane Katrina in 2005 had been engineered and constructed in response to a previous flooding episode in 1965. In that year, Hurricane Betsy produced several levee breaches which resulted in flooding of approximately 20% of what was then the footprint of the New Orleans metropolitan region. The response was to upgrade the regional hurricane protection system, at a cost of several billions of dollars and over a design and construction period of slightly more than five decades (Rogers, 2008). Fifty years later, Hurricane Katrina produced far more numerous and catastrophic levee breaches in the "improved" flood protection system, flooding approximately 80% of the city and to significantly greater depths. This was not because Katrina produced a larger storm surge: the water levels produced by Hurricane Katrina were generally less than the "design levels" that the new regional flood protection system was intended to safely handle. Instead, this was one of the most costly engineering failures in history. It is therefore critically important to understand this disastrous paradox: how the investment of five decades of effort, and funds, led to the creation of a system that performed far worse than in 1965, as we now prepare to move forward to improve our nation's flood defenses.

(1) What Happened

Figure 1 shows a map of the three main protected basins of Metropolitan New Orleans. Blue stars indicate levee failures and breaches, and red stars indicate partially developed and/or incipient levee failures that did not progress to full breaches. Approximately 35 to 45 full and partial levee failures occurred during this event, depending on how one counts long sections of nearly continuous erosional failures on the east flanks of the region (either as continuous failures, or as series of discrete failures separated occasionally by an incompletely eroded segment.) Among these, there were seven main failures, or sets of failures, that together contributed a majority of the floodwaters that inundated most of the city. These are marked with numbers and arrows in Figure 1, and each will be briefly discussed.

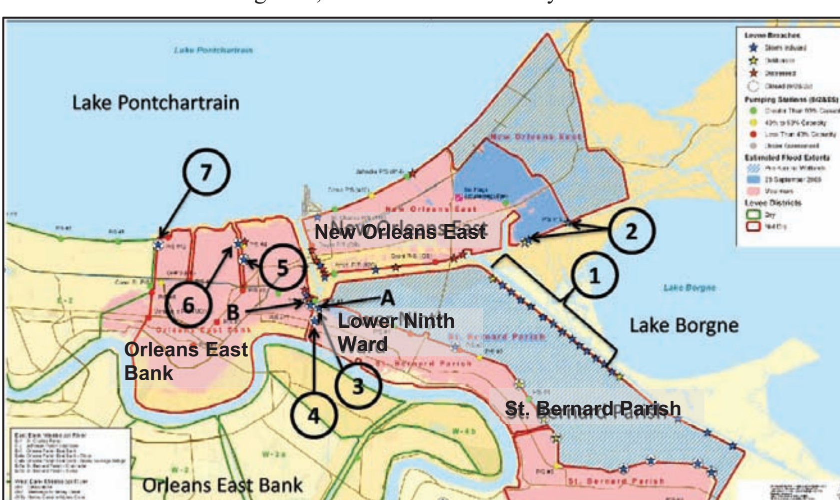

Fig.1: Map of the Three Main New Orleans Protected Basins

Fig. 2: Erosional Failure on the Lake Borgne Frontage (Seed et al., 2008a)

The eye of the storm tracked north just to the east of the city's easternmost basins, and the combination of storm surge and the counterclockwise swirl of the hurricane's winds threw a major storm surge and wind-driven waves against the east ends of the New Orleans East and Lower Ninth Ward/St. Bernard Parish protected basins. This produced catastrophic erosional failures along multiple miles of the levee frontages facing what had been "Lake" Borgne (which is actually a bay with direct connectivity to the open Gulf of Mexico.) These are indicated by arrows 1 and 2 in Figure 1. Figures 2 and 3 show eroded sections along these frontages. Figure 2 shows a section from the northern portion of the east-facing St. Bernard Parish levee that eroded fully, leaving only the imbedded dent in the soft foundation soils (as a

Fig. 3: Partially Eroded Levee Section on the Lake Borgne Frontage (Seed et al., 2008a)

result of consolidation and settlement) and the trail of eroded embankment detritus streamed back towards the swamps behind it to mark its former location. Figure 3 shows a section from farther to the south; where lesser erosion produced only partial damage to the frontal levee face and notching of the crest (or "crenellation").

There had been considerable debate between the three principal investigation teams as to the mechanism of failure along these two critical eastern flank frontages facing onto Lake Borgne; but that has now been formally resolved in federal court. The post-Katrina investigation conducted by the Interagency Performance Evaluation Team [IPET] initially concluded that these two levee frontages had been overtopped, and had eroded from the backside to the front as the overtopping waters accelerated down the backside faces of the earthen levee embankments producing high velocities and high erosive potentials at the lower back sides; as illustrated in Figure 4(a).

The two independent investigation teams; (the NSF-sponsored Independent Levee Evaluation Team [NSF/ILIT] and the State of Louisiana's independent investigation team [Team Louisiana]) both found instead that these levee frontages had breached, and over considerable lengths, long before any overtopping would have begun to occur (Van Heerden et al., 2006; Seed et al., 2006 & 2008a.) That produced a critically important difference in consequences, as the early and extensive failures meant that floodwaters raced through the breached systems while the storm surge was still rising; producing far more catastrophic damages than would have resulted if the failures had instead been produced by overtopping as the storm surge reached its peak and began to subside. This debate has now been resolved in federal court in favor of the two independent investigation teams (Duval, 2009).

The early erosional failures along these two frontages were principally the result of wind-driven storm waves attacking the frontal levee faces as illustrated in Figure 4(b); a mechanism that had not been formally considered during design. The

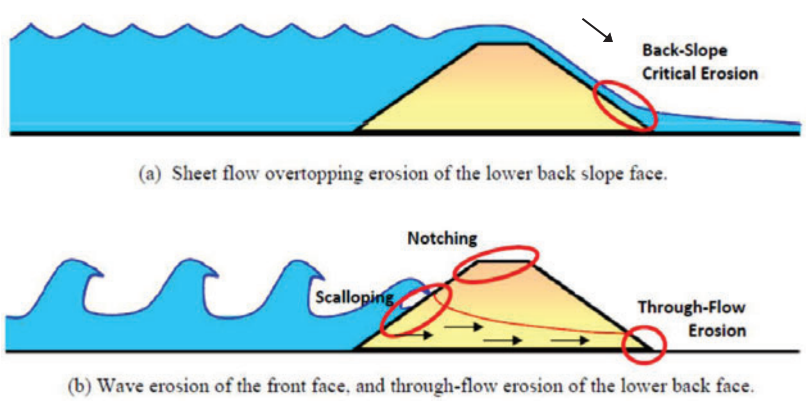

Fig. 4: Schematic Illustration of Several Potential Modes of Levee Erosion (Seed et al., 2008a)

levees themselves, along both frontages, were “sand core” levees constructed using materials that had been excavated during construction of the two adjacent navigational channels (the Mississippi River Gulf Outlet channel, or MRGO, and the Gulf Intracoastal Waterway channel, or GIWW). Both frontages had been constructed mainly of highly erodible cohesionless fine sands, with minimal compaction, and even with compacted clay veneers on some sections these levees were highly vulnerable to rapid erosion by wind-driven waves. The use of cohesionless levee embankment fill materials not conforming to normal USACE material specifications and standards for levees was occasioned in large part by the parsimony of Congress which, over the five decades of levee construction regularly appropriated lesser funds than requested by the USACE; forcing Corps designers to take small gambles in order to complete projects at lower cost (Seed et al., 2006). The failures along these two frontages represented two costly cases of such gambles lost.

The storm surge from the east flank was next driven through the East/West channel separating the New Orleans East basin from the Ninth Ward/St. Bernard Parish basin, and then into the North/South trending Inner Harbor Navigation Channel (IHNC). Multiple minor failures and partially developed failures occurred as a result of the rising storm surge within the IHNC, as shown in Figure 1, but only two of these failures managed to scour erosional holes below sea level and so only these two breaches contributed significantly to the flooding already underway.

These were the two large breaches at the west end of the Lower Ninth Ward, at the locations indicated by the numbers 3 and 4 in Figure 1. Figure 5 shows an aerial view of the massive south breach, more than 800 feet in length, and Figure 6 shows an aerial view of the much narrower north breach (delineated by the dashed lines.) Figure 7 shows a cross section of levee foundation conditions for the south

Fig. 5: Massive South Breach at the Lower Ninth Ward (Seed et al., 2008b)

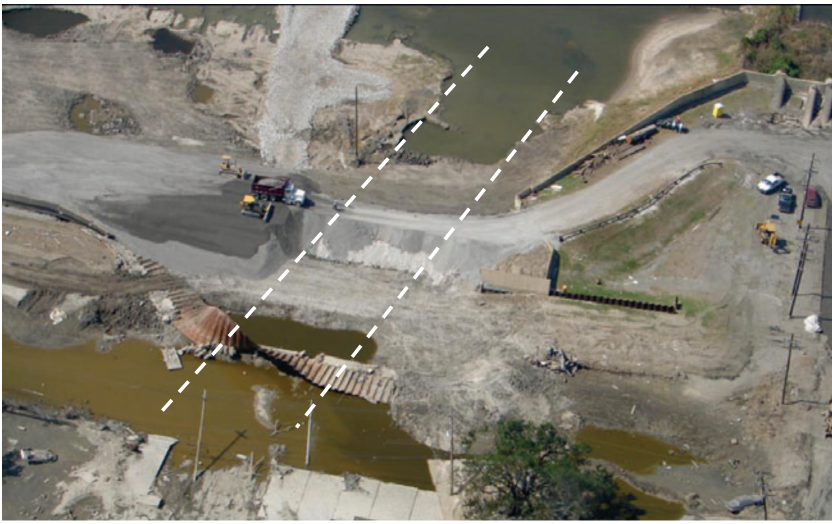

Fig. 6: North Breach at the Lower Ninth Ward (Seed et al., 2008b)

breach. Foundation conditions beneath both breach sections consisted of a surficial layer of relatively impervious soils, underlain by a layer of peaty marsh deposits. This was, in turn, underlain by a deep deposit of soft clays. Conditions at the much narrower North breach were essentially similar, except that the thickness of the landside blanket of relatively impervious soil overlying the peaty marsh deposits was thinner at the north breach.

The north breach occurred first, and the deep narrow breach was the classic result of underseepage through the laterally pervious deposits and then an uplift ("blowout") failure at the landside toe that subsequently retrogressed back beneath the levee as a classic piping failure. The massive south breach was also the result of underseepage, but with a slightly thicker landside surficial impervious "blanket" this section failed by overall lateral translational failure as the underseepage caused pore

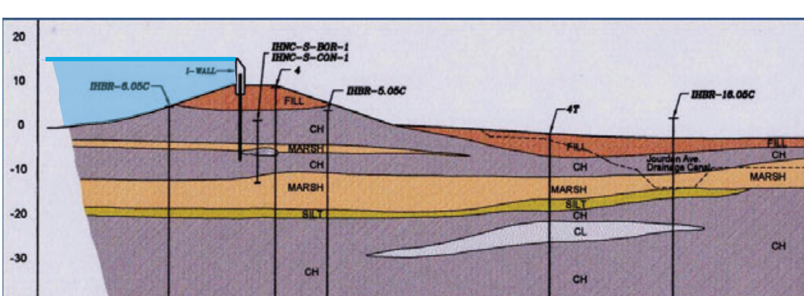

Fig. 7: Cross-Section at the South Breach at the Lower Ninth Ward
(Seed et al., 2008b)

pressure increases beneath the land side of the levee embankment, reducing shear strength, while the elevated water levels on the channel side pushed against the floodwall and its supporting sheetpiles to drive the composite levee section sideways.

There had been disagreement at these two breach sites between the NSF/ILIT and IPET investigation teams as to the causes of these two failures, but data daylighted since both teams filed their final reports in 2006 and 2008 now show that there existed several large holes in the waterside surficial “blanket”, and that these holes are exactly coincident with (1) the large south breach, (2) the narrower north breach, and (3) an incipient third breach nearly midway between the two full breaches that translated laterally more than 3 feet towards the land side but was stabilized there by the rising waters on the land side from the two already fully developed adjacent breaches (Bea, 2008).

The causes of both failures were the failure(s) of the sheetpiles to fully penetrate through the laterally pervious peaty marsh strata into the underlying clay in order to cut off underseepage. This was one of a number of locations where sheetpiles appeared to have been designed only to provide cantilever support for the concrete floodwalls topping the embankments, without also fully considering their role in underseepage cutoff. It had been the view of the local New Orleans District of the USACE that the peaty marsh deposits were relatively impervious, but these layered, peaty marsh deposits tend instead to have relatively high lateral hydraulic conductivity (Bea, 2008).

The remaining three major failures were the failures in the drainage canals within the main (metropolitan) protected basin, as indicated by numbers 5, 6 and 7 in Figure 1. The two eastern failures occurred on the London Avenue drainage canal. The southern breach (Location 5) was another classic underseepage and piping failure, as the sheetpiles supporting the concrete floodwall atop the earthen levee embankment were not extended deeply enough to suitably reduce underseepage. The northern breach (Location 6) was an underseepage-induced lateral translational failure, much like the large breach on the IHNC at Location 4 discussed previously. Here, too, the sheetpiles did not extend deeply enough to cut off flow through a sandy stratum beneath the levee. There was good agreement among the three principal post-Katrina investigation teams as to the causes of both of these failures.

The seventh major failure occurred on the Seventeenth Street drainage canal, at Location 7. Figure 8 shows cross-sections at this breach location both before and after the failure. This was the first of the drainage canal failures to occur within the main protected basin, and given the scope and cost of flooding that ensued, it is the single most costly geotechnical failure in U.S. history. The precise cause of this failure cannot be conclusively determined, as two potential failure modes appear approximately equally likely, and both would have produced the same post-failure condition as illustrated in Figure 8. One possible failure mode would have been underseepage-induced lateral translational stability failure, as with the failures discussed previously at Locations 4 and 6. The other possible failure mode would have been simple lateral translational instability, with the embankment and floodwall being pushed sideways by the elevated water levels in the canal and the composite

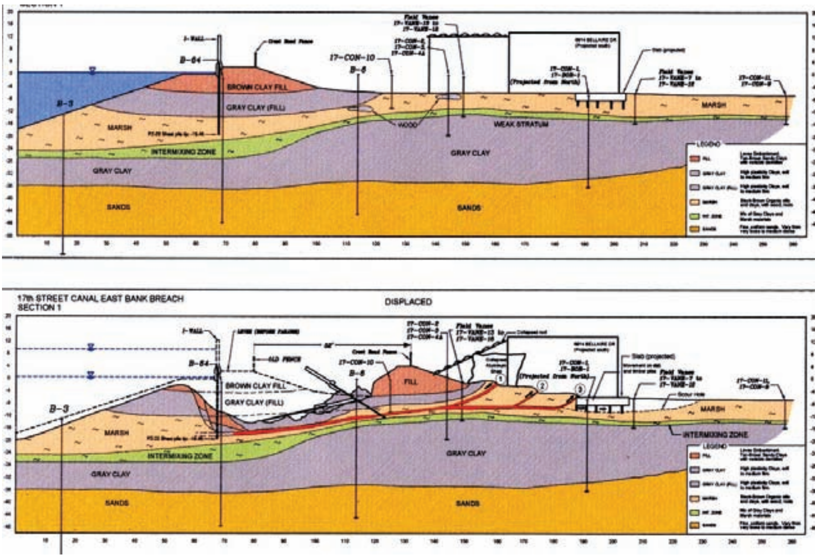

Fig. 8: Breach at the Seventeenth Street Drainage Canal (Seed et al., 2008c)

section translating laterally by sliding along a thin, weak and highly sensitive layer of organic silty clay within the layered marsh deposits underlying the embankment. Both mechanisms show calculated Factors of Safety of essentially unity at the water level of approximately +7 f.t above mean sea level at which the failure occurred (Bea, 2009). If the inception of the failure had been the result of underseepage-induced instability, then the highly sensitive layer would have quickly remoulded and lost strength, and it would have quickly captured the failure surface as the failure developed large displacements; so the final displaced geometry cannot serve to conclusively differentiate between these two mechanisms.

The thin, weak, and highly sensitive layer of silty organic clay that was the eventual sliding surface was less than 1 inch in thickness, and it was the result of a previous hurricane more than 800 years ago (based on carbon dating of pollens; Zobaa et al., 2009). This thin layer was missed by the pre-design site investigations, and also by the post-failure IPET investigation, because this thin stratum was overlain by a thicker layer of leaves, twigs and other vegetative detritus produced by that same hurricane event. It should be noted, however, that the pre-design site investigations "dropped" multiple attempted samples at this site (they were logged as "NR" or not recovered), all at the same elevation, and then failed to investigate further to see what was causing the repeated failure to successfully retrieve samples.

Having also initially missed this thin stratum, the IPET investigation concluded that the likely cause of failure was a deeper, rotational instability failure passing through the soft clays underlying the marsh deposits. This mechanism produces a Factor of Safety of approximately unity at a somewhat higher canal water level (about 3 to 4 feet higher) than that at which the failure occurred, but at a water level still below the intended design level. This was thus a badly flawed design, as it had three competing potential failure modes which would each have produced failure at less than the intended design water level.

In addition to these seven major failures, there were more than 30 lesser failures and partially developed or incipient failures. Most of these were associated either with "penetrations" where one or more facilities (usually either a roadway or a railroad line) passed across the alignment of the levee, or with "connections" where two adjacent levee frontage segments, constructed at different times in response to the tortuously incremental nature of the Congressional appropriations to fund construction over five decades, joined together. These failures did not develop fully by scour and erosion, usually because the landside protected areas were already filling so rapidly due to larger and more rapidly catastrophic failures, but in the absence of the other failures many of these would have had the potential to develop further and become serious in their own right. Thus the overall system appears to have been repeatedly and persistently flawed.

Notable among the lesser failures were the two failures that occurred on the east and west banks of the Inner Harbor Navigation Channel, at locations "A" and "B" in Figure 1. These are the east and west side crossings of the CSX Railroad line across the channel, and these were chronologically the first potentially serious failures to occur during this event. It should be observed that these were also the locations of two failures in the previous event, Hurricane Betsy of 1965, which was the reason for construction of the new regional FPS in the first place. Thus, the repeat failures at these same two locations 50 years later represent very daunting examples of failure to learn from the previous event.

(2) Principal Lessons

There were a great many lessons to be garnered from the disaster of Hurricane Katrina (e.g. Van Heerden, et al., 2006; Seed, et al, 2006 & 2008d, U.S. Senate Committee on Homeland Security, 2006; IPET, 2007; etc.), so we will focus here on only a subset of the most important of these. Lessons learned necessarily encompass a broad suite of technical issues as well as organizational and policy issues, and the focus here will be on lessons of particular import as the nation next attempts to upgrade its flood protection infrastructure.

One set of important lessons is the need for properly independent expert investigation of major failures. The failure of the IPET investigation to correctly identify the principal modes and mechanisms of failure at five of the seven main failures in New Orleans, and subsequent attempts to defend those findings in the face of mounting contradictory evidence, has resulted in debate and acrimony that has obscured the ability of the profession, and the nation, to absorb and implement the key lessons learned. As a result, a number of critical lessons were not implemented in the rapid re-construction and upgrading of the new (post-Katrina) New Orleans regional flood protection systems; a massive undertaking now nearing completion at a cost of $16.5 billion.

One of the key "technical" lessons learned was that proper geotechnical analyses were consistently well able to explain and diagnose all of the principal failures studied in detail. Indeed, properly performed site investigation, site and material characterization, and coupled seepage and stability analyses served not only to correctly identify the principal modes and mechanisms of failure; they were also able to closely identify the actually observed timings and water stages (levels) at which these failures occurred. That means that we have the engineering tools necessary to do this properly, and that the onus is therefore squarely upon us as engineers to consistently get it right.

A second key technical lesson was the importance of properly understanding the geology and geomorphology of project sites. Many of the failures, and a majority of the seven main failures, had their roots in poor understanding of the foundation units; leading to errors and misjudgments in later foundation characterization and engineering design analyses. Geology and geomorphology are increasingly being de-emphasized in much of modern U.S. geotechnical practice. This is a potentially dangerous trend.

A third lesson was that "connections" between project segments completed at different times, and "penetrations" where utilities or roads, etc. cross through levees or floodwalls, were locations of numerous partially developed failures. Connections and penetrations warrant special attention in critical systems where one weak link is one too many.

A fourth technical lesson was the importance of proper consideration of risk and reliability in the engineering of these complex and critical systems. Levels of reliability were too low; the low Factors of Safety (FS) required in key elements of the design process had historic roots in levees engineered for rural and/or agrarian regions, and they were inappropriately low for levees protecting major urban populations. Standards for acceptable seepage exit gradients (and uplift forces), and for Factors of Safety for levee and floodwall stability (required FS as low as 1.3) were simply too low; they left too little room for errors, oversights, misjudgments, undetected foundation geology nuances, etc. and each such oversight was unacceptably likely to result in failure.

A fifth critical technical lesson was the observation that most of the failures observed would likely have been prevented if suitably independent expert oversight had been implemented as a standard operating procedure within the New Orleans District of the USACE; as is increasingly common in other Districts. Unfortunately, the massive District had instead been allowed to become something of an independent fiefdom, and to successfully fend off both independent technical review panels as well as technical reviews from Division Headquarters in Vicksburg. This included outright rejection of reviews from Division Headquarters that might otherwise have prevented the main failures of the "downtown" drainage canals (Seed et al., 2006.)

One of the strongest common recommendations of the two independent investigation teams (NSF/ILIT and Team Louisiana) and of the two principal review panels for the IPET investigation (the ASCE External Review Panel and the NRC review panel) was the need to impose authoritative and suitably independent technical

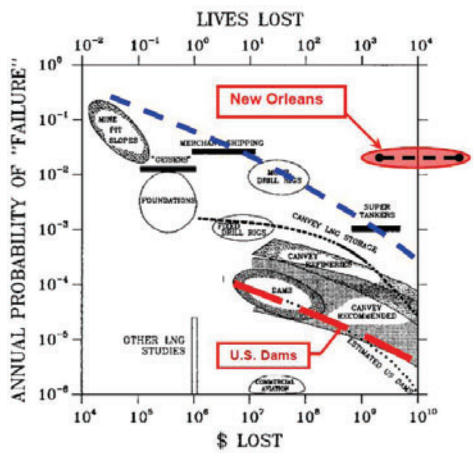

Fig. 9: Risk Plot for the New Orleans FPS

review during the post-Katrina upgrade and reconstruction of the New Orleans regional flood protection system. Unfortunately this was not done; ostensibly in the interest of avoiding delays in providing new protection in a timely manner. As a result, despite extensive re-organization of the District, major portions of the massive effort (six years, and \$16.5 billions of tax dollars) were performed without such review, and largely under the direction of many of the same individuals who had been responsible for the pre-Katrina levees.

There are also important lessons to be learned at the "policy" level. Many of these have a great deal to do with the inherent risks that can be associated with governmental neglect and governmental parsimony.

Current U.S. levee design standards are outdated, and they are especially outmoded when applied to urbanized regions with significant populations and property assets at risk. Figure 9 shows a risk plot based on the NSF/ILIT study's findings (Seed, et al., 2006.) The horizontal axis shows expected loss of life (top axis) or expected economic losses (bottom axis) in the event of failure, and the vertical axis shows annual probability of failure. Plotted on this figure are a number of human endeavors. The red dashed line in the lower right-hand corner corresponds approximately to targeted levels of risk associated with U.S. practice for large dams; structures that tend to be engineered very "safely" as they are viewed as potentially high risk systems. The blue dashed line higher on the figure approximately delineates the "normal" neighborhood, below which most engineered human endeavors occur.

The loss of life, and the economic losses, incurred due to the failure of the regional FPS during Hurricane Katrina are plotted in the upper right hand corner of the figure, based on the NSF/ILIT team's assessment that the level of reliability of the New Orleans regional FPS was such that the annual probability of failure was approximately represented by an average failure recurrence interval of one in 50 to 80 years (Seed et al., 2006). Flood protection is the only major engineered endeavor in the U.S., with broad ramifications with regard to public safety, which is consistently allowed to operate at risk levels above the dashed blue line. The contrast with dam safety practice, especially given the obviously massive population exposure for an urban system like that in New Orleans, is stark.

This is not to suggest that U.S. levee practice should be targeted to levels similar to those employed for large dams; as that would be economically unachievable in the foreseeable future. But it <u>is</u> suggested that <u>higher</u> levels of risk reduction should be targeted, and achieved, especially for “urban” levees protecting large numbers of persons and their homes and businesses. And it is noted that the Dutch, who saw much of their nation submerged by levee failures in a massive storm in the North Sea in 1953, do indeed target “dam-like” levels of risk prevention for levees protecting their largest urban areas.

Unfortunately, recent federal policy and Congressional actions over the past half century have instead tended to degrade national flood safety. And the threads of many of these actions, and inactions, are reflected directly in the New Orleans disaster. Federal parsimony, and budget cutting, have also done great harm to the engineering capability of the USACE, which takes a national lead in promulgating standards and procedures that are widely emulated, even on non-federal levee projects (Seed et al. 2006). Since the late 1970’s, inflation-adjusted budget cuts have increasingly required the USACE to swap out “conventional” engineering personnel assets to accommodate, without significant overall growth, a swelling volume of “environmental” work. Additional Congressionally mandated efforts to achieve improved cost efficiency have led to a progressive re-orientation of the USACE involving further significant transfer of “engineering” personnel assets to personnel assets targeted at “streamlined project management”. Geotechnical engineering has been particularly hard hit in all of this, as it is viewed as a high-overhead enterprise requiring field investigation and laboratory testing assets. The results of this have been two-fold: (1) geotechnical engineering design is now largely outsourced to private engineering firms, and (2) the formerly admirable “engineering” culture of the Corps has been quietly replaced with a “project management” culture. A project management culture is potentially more cost efficient, but is generally orthogonal to public safety as career advancement is premised mainly on achieving on-time and on-budget completion of projects. In such an institutional culture it takes a brave and very committed engineer to point out potential safety issues; as that can cause delays and cost increases.

Further, USACE engineers are currently underpaid, and the Corps is forced to outsource most of their engineering design work, with their own personnel relegated largely to review roles. This makes long-term retention of top engineers very challenging.

If the USACE is to continue to be the lead federal agency with regard to levees and flood protection, then it must be allowed to re-establish the necessary engineering capability and culture for this mission. To that end: (1) salaries should be raised, and the USACE should get back to targeting the hiring and long-term career retention of engineers from top level U.S. engineering graduate programs (as a majority of the geotechnical engineering efforts associated with flood works are properly associated with a graduate field of education), (2) Corps engineers should be allowed to retain and perform some fraction of the engineering design work in house (it is difficult to retain, and to properly train, top engineers if they are only permitted to review design work of others), and (3) a balance should be struck between an internal USACE culture of streamlined project management, and an engineering-first culture better able to advocate for public safety.

Finally, another significant contributing factor in the disastrous performance of the New Orleans regional FPS was the unacceptably long (50-year plus) planned period of design and construction of the FPS, which began in the wake of the 1965 Hurricane Betsy and was scheduled to have been finally completed in 2011. This allowed Congress to stretch out the incremental appropriations to fund the work; a situation that was further exacerbated by a tendency of Congress to appropriate somewhat lesser amounts than requested by the Corps. This meant that: (1) the New Orleans District was required to design and build the regional FPS in short segments, requiring many "connections" that eventually became locations of failures and partially developed failures, and (2) the District's engineers had to take small gambles to achieve cost-savings in design; many of these gambles were subsequently lost when Katrina arrived in New Orleans.

Moving Forward

The wakeup call delivered by Hurricane Katrina has been well received, and is spurring action and appropriate responses at both local and national levels. Further impetus has continued to be provided by additional flood events, including the great mid-western U.S. floods of 1993 and 2008, the catastrophic arrival of Hurricane Ike in 2008 in the coastal area of Galveston, Texas and the flooding of the Missouri River system in 2011.

There was also a partial victory, and a scare, in the passage of large flows down the Mississippi River system in 2011 that produced near record, and occasionally record, flood stages. The largely successful passing of these large flows, with no failures of urban levees (with only lesser damages due mainly to failures of lesser levees and to the opening of planned floodways to take pressure off main levees) was an important success. Particularly promising was the successful passage of the flow through the St. Louis region, where considerable effort has been expended over the past decade to upgrade the region's flood defenses. It should also be noted, however, that although record and near record flood stages were observed at many locations, these were not necessarily associated with record rainfall and flows. Continuing silting of many of the reservoirs behind dams created to reduce and control peak inflows into the Mississippi River system has significantly reduced storage capacity, and thus the ability to mitigate flows; a development which will only continue into the future (Rogers, 2011).

These events, and especially Hurricane Katrina, have brought into sharp focus the pattern of increasing flood risk and flood losses of the past half century. Having been made forcefully aware that the nation's flood defenses are in urgent need of improvement, the response has been admirable at many levels.

Congress, in the Water Resources Development Act (WRDA) of 2007, enacted legislation that established a National Committee on Levee Safety, an activity that is to eventually result in a National Levee Safety Program (NLSP), intended to largely parallel the National Dam Safety Program enacted by Congress in 1996. Recommendations by this committee provided to Congress in 2009 address many of the key lessons from Hurricane Katrina as enumerated in the previous section, and serve to lay the groundwork for a significant advance in national flood protection policy and standards. A key set of recommendations is the call for higher targeted levels of protection for "urban" regions than the de facto 100-year return level standard fomented by the current National Flood Insurance Program.

The USACE is also taking a lead in the move forward, and with Congressional support has performed the first attempted comprehensive survey of the nation's federal levees (USACE, 2011). It is estimated that approximately 15% of the nation's levees are federal levees, or joint federal/local partnership levees. These "federal" levees are often among the more important levees in any given region; but it is not uncommon for protection of even important urban areas to be comprised jointly of federal and non-federal levees. A significant fraction (but not all) of the remainder of the nation's levees have no significant federal involvement or oversight; many of these are poorly regulated or largely unregulated, and in many cases their condition and serviceability are largely unknown.

The USACE has also instigated a number of new programs and initiatives intended to lead to significant long-term advances in the levels of reliable flood protection afforded to the nation. One of the most important of these is a planned transition from simplified, formulaic levee design standards and criteria to formally reliability-based design standards; in which risk of failure and the associated hazard in terms of populations and properties at risk will both be factored into the design targets. The Corps has established a new national Risk Management Center in Denver, Colorado to perform the necessary background research and development work for this, and is next moving towards establishing as many as six regionally distributed Risk Centers, several each for dams and for levees, in USACE districts distributed across the nation.

Corollary to this effort is a similarly important effort to transition the USACE's basis for management of flood protection infrastructure inventory, and for prioritization of flood works and expenditures, to a formally risk-based process. This will represent an important sea change in USACE procedures for flood works, as the prioritization of projects and Congressional funding of them has historically been a largely politically interactive process. To date, most of the USACE's major project works have historically been funded by Congressional appropriations achieved through targeted "earmarks", and the USACE is an unusual federal institution in that it is permitted to regularly interact directly with Congress on issues associated with project funding. Transitioning to a risk-based management of flood protection infrastructure is very much the right thing to do, and it will be interesting to see how Congress responds to having thoughtful and well-developed risk based prioritization presented to them.

A third important USACE initiative is a planned transition from now-outmoded levee standards that had been historically focused essentially on prevention of property and economic losses, to a risk-based system with primary emphasis on the prevention of loss of life. This change is long overdue, as population growth has increasingly pushed urban development out into floodplains potentially at risk across much of the nation.

Individual states have also begun to take action. California, which has the largest overall levels of levee-related risk in the nation, has taken a strong lead in this regard; and this will be discussed as a case history in progress in the next section of this paper. As the re-engineering and reconstruction of an improved regional flood protection system for New Orleans nears completion, the State of Louisiana is taking a harder look at the rest of the State's flood defenses, and the intent is to address those next. Texas, stung by the damages produced by Hurricane Ike, is initiating a major effort to upgrade the inadequate flood defenses of the populous Dallas metropolitan region. And many other states are now beginning to assess and face up to longstanding flood protection issues.

What remains to be seen is how these and other efforts will be affected by the nation's ongoing budget crisis, and corollary efforts to reduce the long-term national debt. Given the need for local "matching" contributions from local (non-federal) partners, the weak national economy and the weak fiscal situations of many state governments will also be an issue. And for non-USACE levees, which currently appear to comprise approximately 85% of the nation's levees, local government capability, diligence and financing will of course be crucial factors.

One of the important lessons from Hurricane Katrina is the value of prevention of losses, and the cost of failing to do so. As discussed in the previous section, Congressional parsimony in the form of incremental appropriations stretched over five decades caused the New Orleans District of the USACE to seek ways to reduce costs of the region's evolving flood defenses in the face of Congressional parsimony with regard to appropriations for the work. The NSF/ILIT investigation team estimated that as much as $200 million in savings may have been achieved, but at the cost of increased risk of potential failures (Seed et al., 2006). Katrina's arrival produced numerous failures, and losses estimated at $100 to $150 billion; numbers that are a factor of 500 to 750 times larger than the "savings" that had preceded them. And at least 1,600 people died. In hindsight, it would have been far better to have spent 10% or even 20% more on system design and construction, and also to have implemented mandatory independent expert review throughout that process, and to have successfully prevented most or all of these tragic losses. The lesson that short-term parsimony must be juxtaposed against eventual likely losses is massively important for political leaders; and Hurricane Katrina is a very useful, albeit tragic, case in point that resonates with policy makers.

CALIFORNIA AS A CASE STUDY IN PROGRESS

The State of California has the distinction of having the largest share of inadvisable levee risk of any state in the nation. Accordingly, California was already beginning to address this prior to Hurricane Katrina; but the well-publicized catastrophe of Katrina has added impetus to those efforts. As a result, California has taken something of a lead over the past five years in attempting to address levee-related risk, and on a massive scale. The State's ongoing efforts to address levees and flood risk represent an excellent case study and provide some potentially significant lessons.

California's levee-related risks can be divided into four general areas: (1) the flood risk in California's great Central Valley, (2) levee-related risk to the State's main water supply as it passes through the Sacramento/San Joaquin Delta, (3) flood risk in the large and very densely urbanized metropolitan areas of Los Angeles and Orange Counties, and (4) everything else. The State is initially moving to address the first two of these, and it is here that insights and lessons may be learned that are applicable to much of the rest of the nation.

The Central Valley Flood Protection System Programs

California's state legislature passed a pair of bills (and bond measures) in 2006 to establish a program, FloodSAFE California, under the auspices of the California Department of Water Resources (DWR) and the re-organized Central Valley Flood Protection Board to address flood risk in the Central Valley. The perceived urgency in accomplishing this is such that the State set out ambitious timelines, and made available approximately $5 billion in State bond funds, so that the initial stage of the effort could proceed without the need to wait for federal support. With local (partial) matching funds, it is expected that more than $4 billion in levee improvement work will be achieved by about 2017. Eventual arrival of federal matching credits for elements of this work, and a planned Phase 2 of the State mandated effort, are expected to further extend the scope of these efforts.

The impetus for these extraordinary efforts was provided by two events, with Hurricane Katrina being the less important of the two. The primary impetus was the outcome of the Paterno lawuit; litigation involving a levee failure that occurred on the south side of the Yuba River in 1986, inundating the communities of Linda and Olivehurst with up to 10 feet of water. The levee that failed was a "Project" levee, meaning that it was a levee for which joint responsibility was vested with the USACE and the required "non-federal" (local) Project partner; in this case the State of California. The Central Valley was one of the first assignments of the USACE, beginning with the authorization of the Sacramento River Flood Control Project in 1917, followed by the landmark Water Resources Development Acts (WRDA) of 1928 and 1936; and in the Central Valley the non-federal partner for essentially all State/Federal Project levees is the State of California.

In the 1928 WRDA, Congress granted the USACE very strong protection from potential liability for failures and damages associated with levee failures. The State of California's legal system, in contrast, has a "deep pockets" system of liability in which any contributing party may be held liable for essentially all of the losses incurred, even if they are only marginally responsible, if the other parties cannot be made to pay.

The outcome of the Paterno lawuit was to hold the State of California responsible for the entirety of the losses incurred due to this levee failure. The levee segment that failed had originally been constructed by local agricultural interests. Subsequent maintenance and upgrades had been made mainly under USACE supervision, and involvement of State engineers on this particular levee segment had been slight at best. But the court held in 2003 that State involvement had, by definition, not been zero and with the Corps granted Congressional immunity and local interests unable to pay, the court held that the State would have to bear the full liability for damages. The State of California's liability rested in substantial part upon its formal acceptances of the levee dating back at least to 1951 as part of the overall Sacramento River Flood Control Project. Payment to plaintiffs was in excess of $450 million, the largest award in a flood litigation case up to that time in the United States (California Supreme Court, 2003).

By extension, that meant that the State could might be expected to be held potentially liable for essentially the full share of future flood damages from failures of joint State/Federal "Project" levees throughout the rest of the great Central Valley; a measure of potential liability representing hundreds of billions of dollars, and increasing each year as rapid population growth continues to push development from the over-developed adjacent San Francisco Bay Area increasingly into potentially flood prone areas in the Central Valley. The occurrence of the Katrina disaster only a year after the consummation of the Paterno lawsuit put a public face, and visceral images, on the issues of potentially catastrophic flood risk and further enhanced the ability of California's Legislature to act.

Scope of Ongoing Efforts:

The scope of the work is massive. Figure 10 shows a map of the levees, dams and floodways (bypasses) that comprise the Central Valley flood protection systems. There are approximately 2,120 miles of levees comprising the Central Valley's flood protection systems, along with multiple floodways or bypasses. Of these, 470 miles are "Urban" levees, and the rest are "Non-Urban" levees; where "Urban" levees are defined for purposes of this program as those levees whose failure would inundate more than 10,000 persons. Approximately 75% of both the Urban and Non-Urban levees are joint State/Federal "Project" levees, and the rest are non-Project levees with no formal federal involvement. The levees and bypasses operate interactively with a number of dams and reservoirs which can assist by helping to attenuate peak runoff flows from the Sierra Nevada Mountains to the east. The geology of the overall region is relatively diverse, giving rise to an excellent test case for current U.S. levee analysis and design methods.

Because the State is potentially solely liable, or nearly so, for failures of State/Federal "Project" levees, and because the State is initially (in Phase 1) expending its own funds without recourse to federal matching support, California was unusually free to draw heavily upon the important lessons from Hurricane Katrina.

One of the important lessons from Hurricane Katrina had been the risks associated with a long and drawn-out process extending over a period of many years (and even decades). The perceived urgency of risk reduction motivated the Legislature to proceed with the State's own funds, and to set forth an extremely ambitious time table. The first phase of the work is to consist of a first-ever comprehensive engineering investigation and evaluation of the Central Valley flood protection systems including both federal and non-federal components), currently due

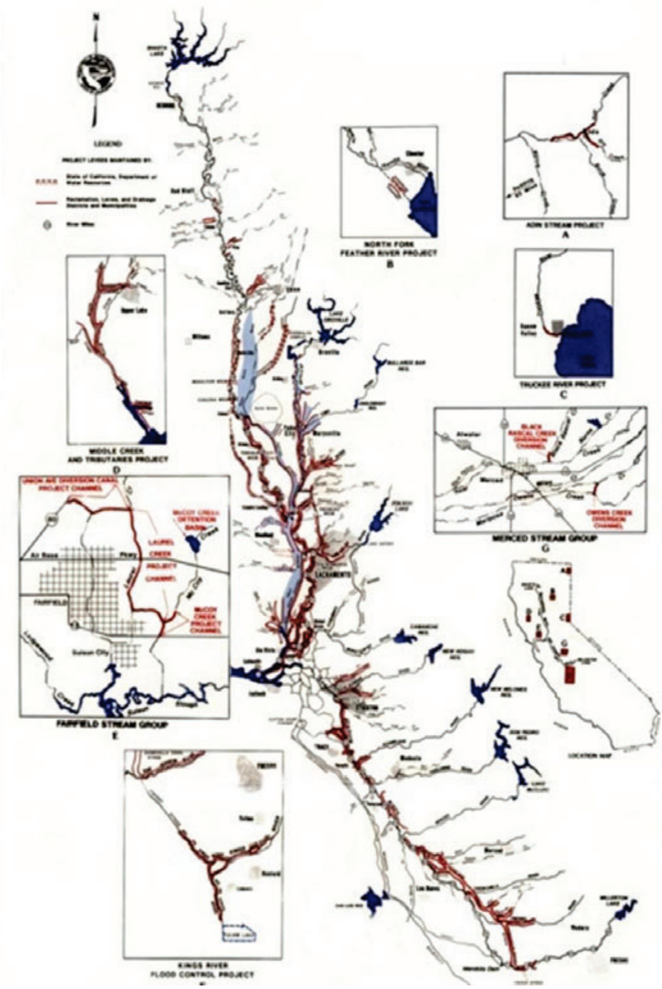

Fig. 10: The California Central Valley Flood Protection System
(Courtesy of the California Department of Water Resources)

to be completed by December of 2012, and mitigation works as then necessary to achieve robust 200-year level of protection by 2025, based on criteria developed by the California Department of Water Resources. If adequate progress to meet the 200-year level of protection is not made by 2015, severe development and other restrictions result, prompting communities to take immediate and decisive actions. Another major deliverable of the program is the development of a comprehensive Central Valley Flood Protection Plan (CVFPP); to be adopted by the Central

Valley Flood Protection Board (CVFPB) by July of 2012, and then to be revised every five years thereafter. Development of the CVFPP requires performing the first-ever formal risk assessment for the Central Valley's flood protection systems; an effort that has been carried forward in parallel with (and drawing heavily upon) the ongoing engineering evaluations of the Valley's flood protection systems that are now nearing completion.

A second important lesson was the value of independent expert input and review. To this end, a consortium of top-flight geotechnical firms with outstanding levee expertise was formed to perform a first-ever "independent" examination and analysis of the flood protection systems throughout the Central Valley; with engineers and firms recused from investigating and analyzing levee systems with which they had significant prior involvement. An independent panel of experts (the Independent Consulting Board; or ICB) was formed to oversee and review this effort, providing technical advice and review, and policy advice when required. Additional reviews were performed within the consortium (by experts from the different engineering firms), by the California Department of Water Resources (DWR), and by the USACE; a multi-tiered review process of unusual rigor. Indeed, at the urging of the ICB, maintaining high standards within the review process has been assigned a high priority.

The scope of the undertaking was massive, and it was necessary to establish consistently high standards for both the work, and for the multi-tiered review process, and to ensure that these would be maintained throughout the grueling effort. And so considerable effort was, and continues to be, devoted to these issues; from the review efforts of the USACE and the ICB right on through the interactive and mutually collaborative (and multi-tiered) review processes established by DWR and the contributing geotechnical firms.

Engineering Assessment and Design Criteria:

A key decision was to follow current USACE levee design criteria for the 470 miles of critical Urban levees as closely as possible, both because these are the most comprehensive and widely used criteria available, and because meeting these criteria is currently still required for projects eventually to be identified for eligibility for federal funding (or matching funding.)

Although current USACE levee design criteria formed the framework for the engineering evaluations process for the Urban Levees Evaluation Program (ULE), it was necessary to develop a large number of standards and protocols specific to this program as the USACE standards left a number of issues loosely defined. Interactively with a leadership team of senior technical experts from the consortia of engineering firms teamed up to execute the work, and engineers from DWR and USACE, and the ICB; a series of standards, protocols and procedures for detailed site characterization and geotechnical analyses were developed. In addition, new standards and procedures were developed for issues not yet addressed by USACE levee design criteria (e.g. seismic levee evaluations, longitudinal river erosion hazard assessment, etc.)

These standards and protocols are contained in a "Guidance Document"; a living document that continues to evolve as the project encounters new sets of challenges and conditions. This Guidance Document is a de facto supplement to the current USACE levee design guidelines, and although specifically developed for the California Central Valley levee systems, it is becoming a useful and more widely accessed reference (URS, 2011).

Space limitations will to allow a full treatment, so we will instead offer a few examples. One of the most important of these was the issue of "adding cohesion" in performing landside levee stability analyses. Current USACE levee criteria require the performance of seepage analyses for fully developed steady state flow conditions, and then the use of fully drained effective stress shear strengths in the coupled seepage/stability analyses that follow. Most levee embankments fare poorly when analyzed for fully developed seepage conditions and fully drained strengths (with little or no effective cohesion), and so it has become common across much of the nation to "add" some effective cohesion, especially to levee embankment materials, when performing these analyses. The engineers involved in the analysis efforts, and the reviewers and consultants, have all seen large amounts of cohesion added for these types of analyses (as much as 500 to 1,000 lbs/ft^2 or more). Blindly adding cohesion is equivalent to simply upwardly adjusting the calculated Factor of Safety; obviating the point of performing analyses in the first place; an issue with national ramifications.

So a pragmatic research program was undertaken to evaluate the actual amount of effective cohesion (c') that could be expected in the various cohesive soils comprising levee embankments and upper foundation soils in the Central Valley, and as a function of both compaction conditions, drying and dessication, and swell upon re-wetting. The Guidance Document presents the recommendations (guidelines) developed (in Table 5-4 of that document). These break the cohesive soils up into four general classes, based on USCS soil classification as well as Plasticity Index and Liquid Limit in a format that essentially represents a modified Plasticity Classification chart. Then, for each class of soils, and with differing values for embankment and foundation units, and varying as a function of Liquidity Index: (1) sets of "limiting" upper bound values of c' and φ' are set for cases wherein sufficient laboratory and/or field data is available, and (2) default values of c' and φ' are recommended for cases wherein lab and field data are sparse. For most CH and CL materials and conditions, upper bound values of c' are less than or equal to 100 to 200 lbs/ft^2, and upper bound values of φ' are less than 35°. For cases wherein data are sparse, lower (more conservative) default values of both c' and φ' are recommended in each case. The recommended "default" values are allowed only for initial stability assessments; for design and mitigation analyses, adequate data development is required, with allowed values being "capped" by the limiting upper bound values prescribed.

The importance of this is illustrated in the analyses shown in Figure 11. This figure shows a cross-section of a levee somewhere in the Central Valley consisting of compacted CL embankment material underlain by typical moderately overconsolidated upper foundation soils. Steady state seepage analyses have been performed for fully developed steady state conditions (using SEEPW), and then imported into SLOPEW to perform coupled slope stability analyses (Spencer's Method) for these seepage conditions. Analyses were performed for three cases. All material properties were the same in all three analyses, except for the value of c′ assigned to the levee embankment fill. Analyses were performed with c' = 0, 100 and 300 lbs/ft^2 in the embankment section. As shown, in Figure 11, varying c′ causes changes in the most critical failure surface, and produces significant changes in the Factor of Safety for landside slope stability; producing FS = 0.76, 1.45 and 2.19, respectively. Suitable values, and suitable bounds, of effective cohesion (c′) are critical in landside stability assessment.

A second good example are the guidelines developed for performance of coupled seepage and slope stability analyses. USACE criteria are clear with regard to targeted Factors of Safety, but leave analysis methods and details undefined. As a result, two general approaches are widely employed. One is to perform a full

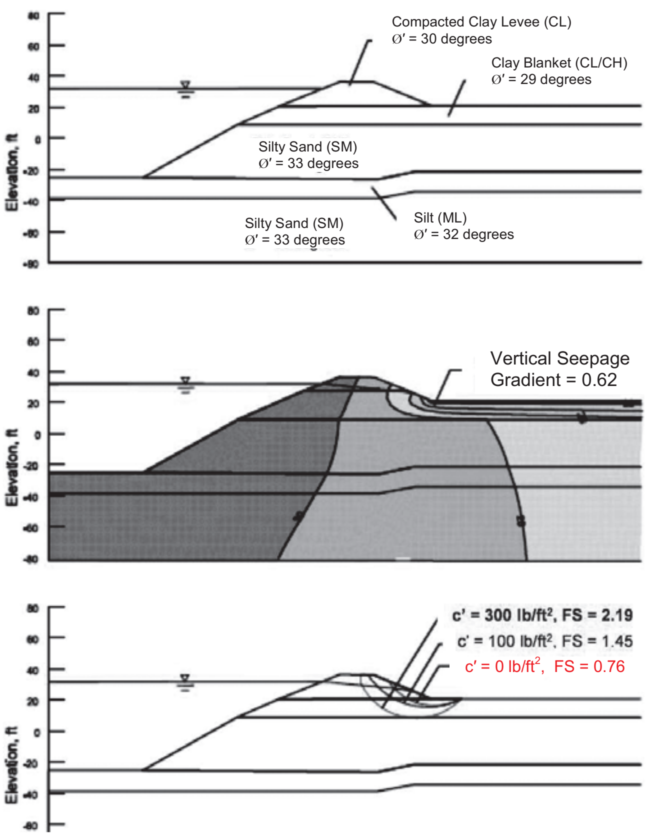

Fig. 11: Example of Effects of Varying Effective Cohesion of Levee Fill

seepage analysis (using finite element or finite difference methods), and then to import the calculated seepage forces and/or pore pressure fields into the subsequent slope stability analyses. The other approach is to specify a "reasonable" phreatic surface for assumed fully developed steady state conditions, and then enter this directly into the slope stability analyses; in which case the stability analysis software will make assumptions with regard to pore pressure fields based on the phreatic surface prescribed.

These two approaches generally produce different results for real-world situations wherein there are distinct differences between the hydraulic conductivities of embankment materials and the underlying upper foundation soils. In most cases, the assumption of a phreatic surface produces a somewhat higher calculated factor of safety for slope stability.

This is illustrated in Figure 12, again for an anonymous levee section somewhere in the Central Valley. The middle figure shows the results of steady state seepage analyses performed using SEEPW. Stability analyses are then performed (Spencer's Method) using (1) the SEEPW analysis results, and (2) just the phreatic surface from the SEEPW analyses; and the results are shown in the bottom figure. For the imported full seepage analysis results, the calculated Factor of Safety is FS =

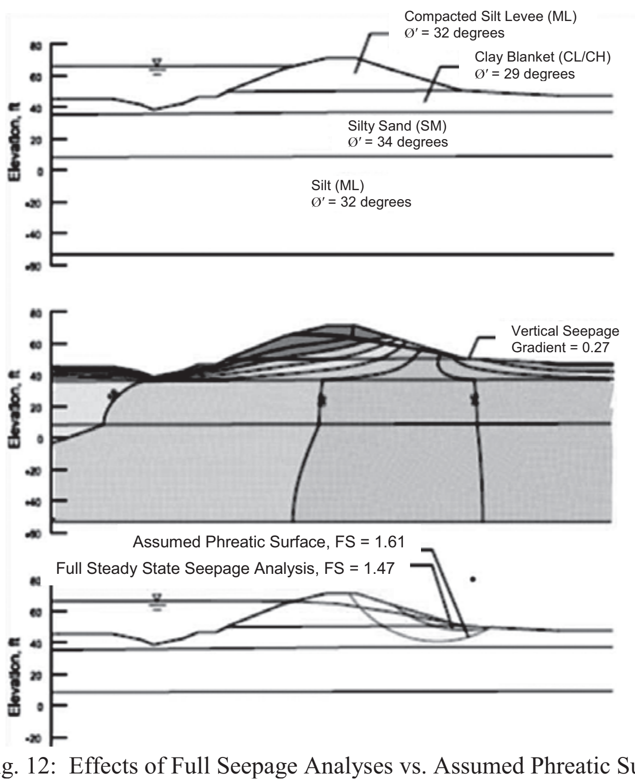

Fig. 12: Effects of Full Seepage Analyses vs. Assumed Phreatic Surface

1.47, versus FS = 1.61 with the assumed phreatic surface. The defined protocol for the Urban Levees Evaluation Program (ULE) is to perform full numerical seepage analyses for all cross-sections studied (using SEEPW), and then to import the calculated results into SLOPEW for the stability analyses.

Seepage analyses were found to consistently be a critical and challenging element of these overall levee analyses and levee assessments; both (1) with regard to seepage gradients and potential for erosion and piping failures, and (2) for landside stability assessments. As a result, the Guidance Document addresses the procedures and protocols to be used to assess coefficients of horizontal and vertical permeability, and also presents guidelines with regard to typical ranges of values for the soil types and conditions encountered over the project domain; a very large and geologically diverse region. Initially, full numerical seepage analyses were required to be performed in parallel with traditional (simplified) USACE blanket theory analyses, which greatly facilitated subsequent technical review. In most cases agreements between both sets of analyses were good, but for some cases resolution of potentially significant differences was required. In the absence of errors, or adverse effects of localized mesh details, etc. the numerical analyses are generally taken as the more accurate of the two when differences arise. In later analyses, after both the analysis teams and the reviewers gained experience, only the fully numerical analyses are now required. The Guidance Document also presents the program's criteria with regard to acceptable gradients for face-exiting seepage on the lower portion of the embankments' landside faces, as a function of material type and condition; a situation that is not addressed by the USACE's current design criteria.

An additional lesson from Hurricane Katrina was the importance of understanding regional and local geology. To that end, each Urban levee system studied was subjected to a sequential progression of study. The initial stage involved acquiring available geologic and geotechnical data and air photos, etc. This was followed by a formal geologic/geomorphic assessment and mapping, which (1) served as a basis for initial selection of drilling, sampling and other (e.g. CPT) field investigation procedures to be employed, and (2) provided significant insight with regard to preliminary definition of "reaches" (sub-elements of any regional system) and initial selection of cross-sections for analysis within each reach. This was then followed by two phases of geotechnical site investigation, each with field investigation and laboratory testing. Initial geotechnical analyses were performed at the end of the first phase, and the insights garnered served to inform the adjustment of reach and cross-section selection, as well as the second phase of site investigation and the final analyses that followed.

Another important lesson from Katrina was the value of avoiding institutional loss of knowledge during the course of an extensive and complex project. To that end, a single project leader (engineer) was assigned overall responsibility for each of the 14 Urban regions that are being studied and analyzed in detail. That individual is charged specifically with knowing his or her levees "as they would their own children"; a level of continuous familiarity that is regularly tested in the multi-tiered review process as well as with presentations and iterative discussions with the ICB.

Yet another important lesson from Katrina was the importance of "penetrations" and segmental connections. Segmental connections are receiving due attention as the main work effort progresses, and a special effort has been initiated to evaluate and assess penetrations. One of the principal challenges here is the difficulty of discovering undocumented penetrations, and then investigating their condition. Foot inspections, and geophysical methods, are being employed, but protocols and methods for investigation and assessment of penetrations are still evolving.

Life Safety:

The issue of life safety is widely misunderstood by both policy makers and the general public in the wake of Hurricane Katrina. In that event, approximately 80% of the city of New Orleans was flooded, but most of the flooding was relatively shallow in depth (less than 6 feet), and the warm August floodwaters from the Gulf of Mexico entered the city at temperatures of between 77° to 80°F. The evacuation prior to Katrina's arrival had been approximately 70% effective, and many of those who remained were located on what passes for "high" ground in New Orleans; ground several feet above mean sea level or more. Floodwaters were relatively shallow in many neighborhoods, and people were able to persist and to make their way through the relatively warm waters. As a result, although loss of life was tragic, the proportional rate of deaths in most of the flooded neighborhoods was relatively low.

California's flood risk in the Central valley is very different. Figure 13 shows a photo of Sacramento's low-lying "Pocket" neighborhood; a neighborhood at the south end of the State's capitol that would be inundated to depths of up to 20 feet and more if the levees were to fail with the adjacent Sacramento River flowing at the 200-year water surface elevation (WSL). As can be clearly seen in Figure 13, the adjacent neighborhood is well below river level at typical stage flood stages.

Figure 14 shows projected depths of inundation for communities in the Sacramento area for the 200-year WSL. The darkest blue color represents depths of flooding to depths of 10 to 26 feet. The Pocket, with a population of approximately 50,000, is indicated. North of Sacramento is a second and larger (and deeper) basin, the Natomas basin, which had historically been Lake Natomas before being leveed off for agricultural use. Natomas basin is currently being developed as a northward extension of the city of Sacramento and currently has about 40,000 residents, with a projected eventual population of more than 200,000 when development is completed. Development is currently halted

Fig. 13: The Sacramento "Pocket" Area

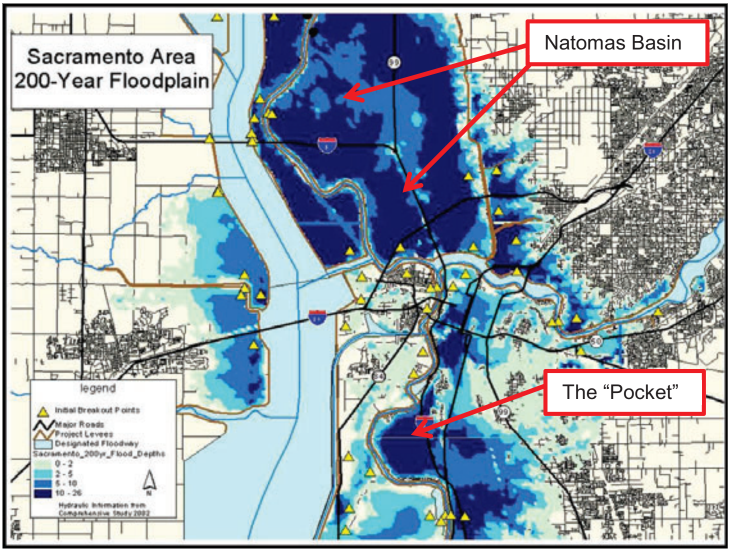

Fig. 14: The Sacramento Area 200-Year Floodplain
(Base figure courtesy of the Calif. Dept. of Water Resources)

due to recent FEMA de-certification of the Natomas levees, and urgent efforts are underway to upgrade these levees and re-achieve certification so that additional residential and light industrial development can resume.

Figure 15 is a photo taken near the south end of the Natomas basin. The engineer in the photo is holding a long staff, and the graphics department of DWR has added a “ceiling” to the photo to indicate the 200-year WSL at this location. The building in the background is a three-story structure; one and two-story structures throughout much of this area typically have rooftops below the 200-year WSL.

Fig.15: 200-Year WSL in South Natomas Basin
(Figure courtesy of the Calif. Dept. of Water Resources)

The percentile loss of life in flooded neighborhoods increases sharply after flooding depths exceed 8 feet (Jonkman et al., 2008). Many of the urban neighborhoods in California’s Central Valley are susceptible to flooding to greater depths than this, and the associated danger will be significantly further exacerbated by the water temperatures. Derived largely from snowmelt in the surrounding mountains, Central Valley floodwaters usually have temperatures on the order of 45° to 55°F. At these temperatures, few people can persist long before being overcome by hypothermia; at which point danger of drowning and loss of life increases rapidly.

The current Central Valley flood protection upgrade efforts will serve to improve levee safety, but they will not provide levees that can be guaranteed never to fail. And so more should be done with regard to the critical issue of life safety.

Additional steps would be more of a "policy" issue than a straightforward engineering issue, but engineers must involve themselves or nothing will be done. Several relatively inexpensive steps would make a significant difference with regard to life safety hazard exposure. A first step would be increased public awareness. In addition to notifications of property owners by mail, light blue lines could be added to lightposts and signs indicating the expected 200-year WSL. Residents could then look across their neighborhoods and visualize the flood levels, and understand.

A second set of important steps would be to provide safe locations, above the floodwater levels in flooded neighborhoods, where individuals could await rescue, and to provide for rapid rescue as well. Given the expected cold waters, people will need to be able to exit themselves from the floodwaters within about 30 minutes or less; otherwise the risk of loss of life will begin to increase rapidly.

Fortunately, this would be neither expensive nor difficult to achieve. Each neighborhood could be provided with some points of safety; buildings with accessible rooftops above the 200-year WSL where people could congregate and await rescue. That would require changes in building codes, and it would be especially effective in still-developing regions such as the Natomas basin. California has already begun to take the first steps here through voluntary building code provisions.

With regard to rescue, helicopters alone will not suffice to quickly rescue tens of thousands of people (or more) from flooded neighborhoods in the first few hours (the key window of time for cold, deep water.) Boats will be needed. Fortunately, these neighborhoods tend to have private boats (pleasure craft). Unfortunately, these boats are fastened to trailers, and they will be held down and submerged; ruining their electronics and rendering them useless. If these were, instead, unstrapped from their trailers and attached to 30 foot cords, then they would float up with the rising floodwaters. If they also had gas in their tanks, then they would be available for immediate service as neighborhood rescue craft. Even better, owners of such boats could become neighborhood boat marshals, and the locations of available rescue craft could be ascertained and mapped. Neighborhoods with few boats available could be given additional boats. The costs of arranging and overseeing such a program would be miniscule relative to the costs of the levee improvements underway.

These seem like simple, and very inexpensive ways to foment a major reduction in risk of loss of life; but such steps are tremendously difficult to implement because they are fiercely opposed by developers and real estate interests who have a vested interest in not wanting residents to understand the risks they face. And they are resisted by many residents as well, who justifiably fear reduction in their property values. And so risk of loss of life continues to be a significant issue in neighborhoods potentially subject to deep and cold flooding.

Lessons Learned:

The Central Valley levee programs have proven a fertile test-bed for current and evolving engineering approaches for assessment of levee adequacy and reliability. And so there have been many lessons learned.

One of the important lessons is that when the Urban levees of the Central Valley flood protection systems were subjected to thorough and independent evaluation; all 14 of the urban region's systems were found wanting. All 14 regions had FEMA-accreditated levees at the beginning of the process, but with evaluations now nearing completion it is evident that none of the 14 regions really had levees warranting full certification even at the 100-year return level required by the NFIP. One region, the Natomas basin, has already had its levees formally de-accredited by FEMA at the recommendation of the USACE, who are intimately involved in the overall process and thus well aware of the ongoing insights being garnered. It is daunting to realize that all of these levee systems had previously achieved formal certification under the FEMA and NFIP rules; a situation that raises doubt as to the robustness of FEMA's levee accreditation procedures, and with obvious potential ramifications at a fully national scale.

A second important lesson has been the large number of levee sections analyzed wherein there is a nearly simultaneous evolution of (1) low Factors of Safety with regard to seepage-induced internal erosion and potential for piping in the shallow foundation soils, (2) low Factors of Safety with regard to landside embankment slope stability due to underseepage and through seepage, and (3) hazardous conditions with regard to face exiting seepage and erosion potential on lower landside face of the levee embankment. This serves to further highlight the importance of properly performed seepage analyses, as noted previously; but there is more to it than that. These analyses show levees that are simultaneously achieving marginal stability with regard to toe seepage erosion, slope stability, and often erosion of the lower landside embankment face as well. This would suggest conditions not amenable to flood fighting (real time monitoring and emergency intervention during high water events as potentially serious conditions develop). And yet the ability to conduct effective flood fighting is the presumed basis for most U.S. levee design (excepting hurricane levees, where hurricane-induced wind fields would render the presence of personnel in the field too risky.)

Figure 16 shows an example of one such levee somewhere in the Central Valley. At the 100-year water surface elevation, the analyses show a vertical exiting gradient at the landside toe of 0.8, a Factor of Safety of 0.84 with respect to landside embankment stability failure of the lower landside face (and FS = 1.16 for a failure penetrating deeper and intersecting the levee crest), and face-exiting seepage that would be expected to cause erosional problems.

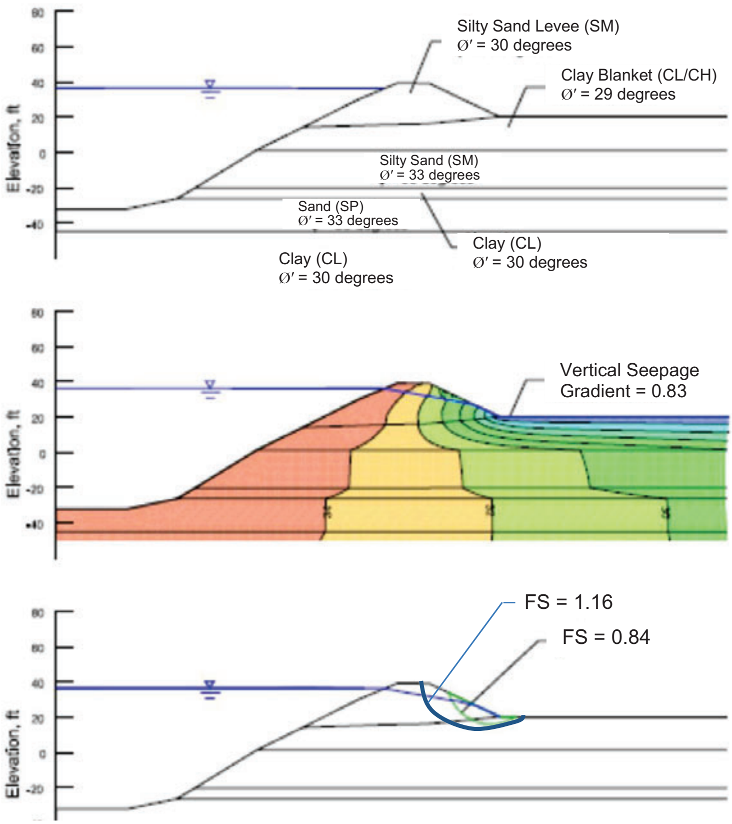

Fig. 16: Example Levee with Simultaneous Multiple Stability Problems

And there appear to be numerous similar examples of levee failures where essentially simultaneous achievement of marginal internal erosional stability (within both the foundation soils and the embankment soils), and marginal overall landside slope stability, have indeed led to catastrophic failures. Several of the most important failures during Hurricane Katrina would fall into this category, including the two failures at the west end of the Lower Ninth Ward, and the three failures on the drainage canals in the main (Metro) New Orleans protected basin. Interestingly, it would also apply to the "Paterno Case" levee failure on the Yuba River which was so pivotal in triggering California's current extensive efforts to upgrade flood protection in the Central Valley, as well as a second failure a few years later on the same levee system. In both cases, eyewitnesses descriptions are similar (and the second failure occurred with engineers and flood fighting equipment on site). In each case the ground at the landside toe began to show rising water and to soften; in one case a truck at the toe sank to its axles. At the same time face exiting seepage began to melt away the landside toe of the embankment. Then a sudden and catastrophic landside slope stability failure occurred. All of this happened quickly, with no possibility for constructive intervention, despite the presence of floodfighting personnel and equipment in one of the two cases.

There is a potentially important lesson here for the nation as a whole. If the ability to effectively flood fight is to be an assumed basis for design standards; then levee design standards should foment the design and construction of levees with adequate margins of safety so that flood fighting has a suitable chance to be effective. And there is a second lesson specific to California's Central Valley. The very challenging geology and difficult history of evolution of California's Central Valley levees, coupled with frequent high water events, has resulted in a situation wherein the Central Valley levees depend to an unusually high degree on frequent and often vigorous flood fighting. That is a very dangerous way to manage a levee system; because erosion (including internal erosion within levee embankments and within the underlying foundation soils) is a progressive mechanism and each successive high water event therefore poses an increased risk. The perception that this risk has now become unacceptable has a great deal to do with California's current efforts to upgrade these critical levees.

Another interesting lesson is the large degree of parallelism between California's ground breaking efforts, and the similarly ground breaking efforts of the USACE to develop new levee design standards and new tools for assessment of the nation's existing levees. Each set of efforts can benefit greatly from the other, and cross-communication and technology transfer between the two has generally been good. And this cooperation could be further improved, to the benefit of all.

An interesting example here is California's development of a Levee Assessment Tool (LAT) for initial assessment of the 1,650 miles of Non-Urban levees in the Central Valley, and the USACE's development of a Levee Screening Tool (LST), eventually likely to be applied to the even more massive problem of making a first assessment of the levees across the full nation for primarily policy and planning purposes. The scopes and scales differ, as the USACE may have to deal with up to 100,000 levee miles, but both "tools" are similar inasmuch as they provide for more rapid and economical initial assessments than would full application of standard geotechnical field investigations, laboratory testing, and analyses.

California's DWR has developed and used the Levee Assessment Tool (LAT), described in the Geotechnical Assessment Report (URS, 2011) for Non-Urban levees in the Central Valley. The LAT is intended to characterize the hazard associated with levee vulnerability. Likelihood of levee failure is characterized, while consequences of levee failure are not addressed by the LAT. Past performance data (i.e., what known occurrences of distress have resulted from previous high-water events) are entered into the LAT. Also, data about geotechnical and geologic conditions, levee geometry and landside topographic features, water surface elevation, and other physical factors that indicate relative levee robustness or vulnerability are entered into the LAT. The LAT uses a weighting and rating system to produce an initial weighted hazard indicator score (WHIS), and then compares this score to the past performance rating (PPR). These two aspects – factors that collectively indicate what vulnerability might be anticipated, and past performance that suggests what vulnerability has been demonstrated – are then considered in combination by a team of senior experts to assign an overall hazard classification for the levee. By comparing the analytical score against observed performance a more reliable assessment can be achieved. When analysis/evaluation and past performance do not reasonably corroborate each other, levee sections are identified for further study.

The USACE has developed and used the Levee Screening Tool (LST), described in USACE (2011). The LST is intended to characterize the risk associated with levee vulnerability. Likelihood of levee failure is characterized, and the consequences of levee failure are also estimated in the LST. The LST uses a simplified probabilistic framework to combine: (1) flood loading estimates based on hydrologic data, (2) engineering assessment ratings that correspond to ratings from routine levee inspections, and (3) loss of life and economic damage based on population at risk and value of infrastructure likely to be inundated. The results of the LST are presented as a life safety index and an economic risk index.

The LST is a more simplistic "geotechnical" evaluation, as is warranted by the number of levee miles to which it is likely to be applied, and it has the advantage of also considering likely consequences of flooding. The LAT benefits from not having to make a consequence assessment; as it is applied only to "Non-Urban" levees. It also permits a somewhat more detailed geotechnical assessment, as the number of levee miles is more limited, and so allows such things as the requirement that engineers walk and visually inspect the levees (and after the grass has been mown), and more detailed conversation and queries with the engineers responsible for maintenance and flood fighting (though this is not always optimally productive, as responses are not always fully open and unguarded).

Each of these tools addresses an important national need for rapid first-assessment methods as the nation attempts to address its longstanding levee risk. Hybridization of the two methods would pose some advantages, and a tool much like the LAT would be usefully applicable to selected levee reaches as a second pass tool after initial LST assessments. In California's case, the LAT will be used as a partial basis for prioritization and targeting of additional intrusive geotechnical site investigations and full analyses of selected sub-elements of the Non-Urban levees; with emphasis on levees protecting significant populations (though by "Non-Urban" definition less than 10,000 persons) and/or other especially significant assets not occurring in Urban regions.

Another set of lessons from California's current levee programs has to do with the institutional and policy hurdles that can deleteriously complicate and even obstruct efforts to improve levee safety. There have been three significant such hurdles over the past several years as California has worked to push forward its levee improvement programs.

The first of these was the USACE's re-invigoration of its "levee vegetation" policy, requiring removal of trees and mature, woody vegetation from levees and the immediately adjacent ground at the landside and waterside toes. This was not a new policy, but its enforcement had lapsed over the preceding several decades, to the extent that the USACE itself had designed and constructed deliberately vegetated levees in California as environmental remediation and as a means of reducing waterside erosion. So the sudden renewal of this vegetation policy had the same effect as the sudden introduction of a new policy. This renewal of the de-vegetation policy was initially a response to Hurricane Katrina. That is because, as vegetation is a "maintenance" issue, and operation and maintenance (O&M) were viewed as a potential route of attack for litigation in the wake of Katrina. This potential for legal vulnerability is due to the possibility that Congressional immunity for levee failures may not specifically cover the USACE for O&M (though Congress can now simply retroactively extend this immunity to cover it). There was a great deal of mature vegetation (large trees) "out of bounds" at the landside toes of levees throughout the New Orleans region, and this vegetation was quickly removed after the storm and flooding. This was then followed by an initiative to effect similar vegetation removal from levees across the rest of the nation; even though vegetation does not appear to have caused any of the New Orleans levee failures.

In the western U.S., this policy change has been resisted by environmental interests. In California's Central Valley, the trees on and near the levees are the last vestiges of vast riparian woods that once filled the valley, and their connection to the adjacent rivers and sloughs makes them precious habitat for a number of endangered species; setting up a natural conflict between environmental laws and USACE policy and standards.

This is a major issue for California's levee programs. California has estimated that it would cost approximately $7 billion to comply with the USACE's newly revived de-vegetation policy; a sum larger than the initial approximately $4 billion currently targeted for the far more urgent efforts to mitigate levee hazard associated with issues other than vegetation. And these high costs may be moot, as existing environmental laws, and the agencies that enforce them, may disallow the required vegetation removal.

California has studied 329 levee failures and more than 5,000 "incidents" (near failures, and incidents requiring intervention), and found that none of these could be specifically ascribed to vegetation. So although vegetation, especially at the landside toe region, <u>can</u> be potentially problematic with regard to inspection and flood fighting, and with regard to tree toppling and exacerbation of localized seepage hazards, it does not appear to be a priority relative to potential for levee failures due to the far more urgently dangerous issues of (1) overtopping, (2) underseepage and piping, (3) through seepage and erosion, (4) seepage-induced embankment stability failure, (5) river scour-induced undermining of levee toes, (6) rodent burrowing, etc.; all of which have contributed significantly to the history of past levee failures and incidents.

This puts the USACE in the awkward position of impeding efforts by a State to improve levees in urgent need of improvement, and at State expense. As a result, a solution is being sought. An initial attempt at a solution was the offer by the USACE to grant "variances" (partial exemptions) on a project by project basis. The initial application of this to selected projects resulted in significant reduction, but not elimination, of the numbers of trees to be removed. California, however, rejected this because there was no apparent rational engineering basis for determining which trees actually needed to be removed, rendering this an "arbitrary" process that would not serve as a sound basis for planning of the State's ongoing massive efforts. This "variance" solution for the newly re-invigorated vegetation policy has also been strongly rejected by multiple additional States; as it has also proven very difficult to meet the criteria necessary to achieve a variance. A second potential alternative recently offered by the USACE is the implementation of urgent levee improvements under a System-Wide Improvement Framework (SWIF). Under a SWIF, the levee maintainer agrees to make prioritized urgent investments (improvements), but also agrees to eventually make other required improvements (e.g. compliance with the vegetation policy) once the more urgent improvements have been completed. Given the costs of compliance with the current vegetation policy, and the likelihood that environmental laws and litigation might prevent eventual compliance with the USACE's current "Vegetation Policy" anyway, the State of California cannot agree to eventual full vegetation removal, and so the SWIF alternative also appears likely to be of limited utility at best for California's situation. Finally, for the past several years, there has been a parallel effort was to launch and conduct a collaborative research effort by the State of California and by the USACE to determine when, and to what extent, vegetation poses risks to levees, and also when vegetation is beneficial. In hindsight, it would have been a good idea to undertake that research effort before imposing the newly stringent national de-vegetation policy. This issue has not yet been resolved, and the USACE's "Vegetation Policy" continues to be a significant roadblock for California's urgent Central Valley levee efforts.

A second, and similar, problem arises with regard to the USACE's issuance of 408 permits; permits that are required for modification of levees with federal interests. When the first of California's applications to actually proceed with levee improvements were submitted, the response from USACE HQ in Washington was to deny these, because a fully comprehensive risk evaluation and prioritization of the Central Valley flood protection systems had not been completed. Under ordinary circumstances, that would have been a valid issue; as the USACE have to be good custodians of federal funds, and so they have a responsibility to ensure that levee funds are well spent and also effectively so. In this case, however, California was spending its own funds, and on levees already identified by the most thorough and comprehensive system-wide assessment ever performed to be among the most urgent levee sections in need of remediation within the Central Valley. So the USACE was again in the awkward position of obstructing urgently needed levee upgrades that were to be performed at no federal expense. To its credit, the USACE is working to develop a solution here as well. A likely solution would appear to again be the granting of "variances" or exemptions, but current indications are that this would entail a tedious and unpredictable process that would continue to slow California's efforts. Fortunately, California is also in the final stages of completing a Valley-wide risk assessment, for its own planning purposes, which may well serve to satisfy the USACE's newly evolving requirements with regard to risk-based prioritization; this might potentially solve everyone's problems here by addressing Federal, State and local common goals with regard to public safety and prioritization of expenditures.

A final problem has been recent changes in the USACE's methods for granting "credit" for matching contributions to joint federal/non-federal flood protection works. California was initially encouraged by the USACE to carefully adhere to USACE criteria in order to qualify for eventual federal cost-matching credits. Subsequent changes in USACE rules and procedures have, instead, made it increasingly difficult for the State to qualify projects for matching credits. This is not maliciously motivated; it is simply a natural outcome of the USACE's responsibility to be good curators of the nation's tax dollars. Once again, however, the USACE is in the awkward position of appearing to be non-supportive of efforts by a State to effect critical and urgent improvements in flood safety. That is important, as federal sources cannot be expected to fully fund all necessary upgrades of the nation's flood protection infrastructure; so State and local efforts (and funding) need to be encouraged. The initial solution offered would be to grant dispensations by the USACE's Chief at Washington HQ, but this has historically been a time-consuming and tedious (resource consuming) process, and an uncertain one as well; again a poor basis for planning and execution of the massive and urgent Central Valley levee programs underway, and one likely to result in unacceptable delays in urgently needed safety improvements. There is significant risk that this change in policy with regard to matching crediting may critically undermine California's major ongoing efforts.

These three issues do not appear to represent any malicious intent; in each case they are a natural result of well-intended policies that are now encountering novel and innovative California levee programs, and on a very large scale. They are, however, symptomatic of a USACE that is a massive and unwieldy bureaucratic organization, with a history of being less than optimally nimble in dealing with new situations. The USACE is also, correctly, focusing considerable efforts on what appear to be excellent changes in national levee risk assessment and levee design methods. This is a time of significant change, and also consequent challenge. A growing coalition of States is encountering these and similar issues with the USACE, and Congressional delegations have begun to become increasingly involved. It must be hoped that all will recognize the challenges inherent to a time of unprecedented change such as this, and that calm heads will prevail and will succeed in developing viable interim solutions to these types of issues until more comprehensive solutions can be implemented.

A fourth issue has caused enough concern among the public and Congress that the beginnings of a solution may be in sight. This problem is the inordinate duration, complexity and cost of Corps Feasibility Studies. These are important planning studies since they are necessary for federal authorization of funding for flood risk reduction projects. Risks associated with delays in project implementation had been another key lesson highlighted by Hurricane Katrina. These studies can take 10 years or more to complete, if they are completed at all, and they have long been a source of frustration for State and local flood management agencies, communities and the public. Often these studies become mired in do-loops of detailed studies and review iterations that do not produce better decisions or deliverables. In response, the Corps has recently proposed a new planning process designed to address deficiencies of the current pre-authorization process. This new proposal addresses five key elements: level of study detail, alternatives analysis, serial reviews, determination of federal interest, and resources for the planning studies themselves.

Throughout the new planning process, the level of detail necessary for technical studies will be managed with respect to risk to the study itself, thereby prioritizing and focusing resources. Rather than concentrating on a single best plan, the proposed process utilizes a multi-criteria evaluation approach that may expand the number of potential solutions. Serial reviews of projects by USACE District, Division, and Headquarters staff often result in endless cycles of comment/resolution; instead, the new process requires "vertical integration" of decision makers throughout the process and more frequent progress reviews that require early resolution of significant technical issues. Finally, the new process proposes to identify Federal interest early in the study, and to adequately resource (allocate funds and personnel for) the entire study duration (rather than incrementally, year by year). The resulting targeted timeline for this entire process is expected to be 18 months. If successful, this would be a quantum improvement over past processes.

To test this new process, the USACE has selected four pilot projects throughout the country. Of these, the Sutter Basin Project in California's Central Valley is by far the largest and most complex flood risk reduction project. The 285 square mile study-area east of the Sutter Buttes is bordered by the Feather River, Sutter Bypass, Cherokee Canal and Wadsworth Canal. Inundation depths of up to 20 feet threaten up to 80,000 residents and $14 billion of assets and infrastructure. In the catastrophic flood of 1955, a levee failure caused the deaths of 37 people and hundreds of millions of dollars in property damages. And in 1997, two additional costly and deadly failures occurred again on the Sutter Bypass and the Feather River directly adjacent to the Sutter Basin, effectively relieving pressures on other levees that also appeared to be on the brink of failure. In spite of the fact that this large and populous basin currently has less than a 10-year level of protection, the USACE planning study for eventual has been ongoing in fits and starts since 2000, and until recently it appeared doomed to the same fate of other studies: tangible improvement would eventually be initiated only following another catastrophe. To address this unacceptable risk, the State of California and the local agency (Sutter Butte Flood Control Agency) have already partnered on portions of an Early Implementation Project (EIP) as part of California's Central Valley levee programs; this EIP proposes to improve 44 miles of levee by 2015 at a cost of $300 million. California created the EIP program to construct urgent flood risk reduction projects in advance of the USACE, with the provision that the implementing agency would garner federal matching credit for an "Early Implementation Project". Unfortunately, this key strategy has been undercut by recent USACE policy changes as described above; an issue that must be resolved prior to construction of this ambitious project.

There has never been a time when the California State planning process, local support and assessments, State bond funding, an accelerated implementation program and an accelerated USACE planning process have coincided; thus there is intense pressure for the USACE to produce and deliver a cost-effective, timely and effective pre-authorization study. Given the urgency associated with the levels of obvious risk for the Sutter Basin, this project provides an excellent test of the USACE's new planning process.

Other Issues:

Finally, it should be noted that the ongoing Central Valley flood protection efforts represent only one of California's significant "levee" issues. A second issue is officially one of the most urgent and important overall problems facing the State, and that issue is the Sacramento/San Joaquin Delta (commonly referred to as the Sacramento Delta). In a typical water year between half and two-thirds of California's freshwater falls as snow or rain in the mountains surrounding the Central Valley, and then runs into the valley's rivers and eventually passes through the Sacramento Delta as it exits into San Francisco Bay. A labyrinthine system of meandering river channels and sloughs carry the freshwater through the Delta, and these are constrained by 1,150 miles of fragile levees that were never intended to become the main hub of the State's now extensive water systems. These rivers and sloughs are used to transport freshwater from the north and east across the Delta to its southern and western edges, where large amounts of water are withdrawn for use both in the metropolitan San Francisco Bay Area as well as in southern California's massively urbanized Los Angeles/San Diego region, and for southern California agricultural use as well. Twenty five million Californians get some or all of their water from the Delta, and these waters are also pivotally important in supporting the State's economy.

Figure 17 shows a map of the Sacramento Delta. The Delta was an estuarine region, largely at sea level, in the 1850's when it was initially leveed off to create dozens of islands and tracts for agricultural use. Because the uppermost soils are peaty marsh deposits, this was tremendously fertile land, and with a ready supply of water. Pumping was initiated to dewater the uppermost soils for farming, and that initiated a set of processes causing regional subsidence. The main mechanism is slow oxidation (essentially very slow "burning") of the organic peats due to exposure and sunlight, and additional mechanisms include wind-blown erosion, etc. Beginning in the 1860's, much the Delta has been subsiding at an annual rate of between 2 to 4 inches per year. Now, one and a half centuries later, much of the deep central and western Delta is deeply subsided to depths of as much as 15 to 20 feet below mean sea level.

The levees have also settled, largely as a result of ongoing consolidation and secondary compression in the underlying peats and soft clays that are common foundation materials. The initial levees of the 1860's were only a few feet tall, and were constructed by hand. In the early 1900's steam shovels operating from barges took over, and today many of the levees in the deep central and western Delta are up to 30 feet in height; built in successive and often largely unplanned stages upon the bases provided by the earlier (non-engineered) levee sections that preceded them. Some of the major levees along the main (navigable) river channels are Project levees with joint federal/local oversight, but a majority of the Delta's levees are non-Project levees with only local reclamation districts (often individual islands) responsible.

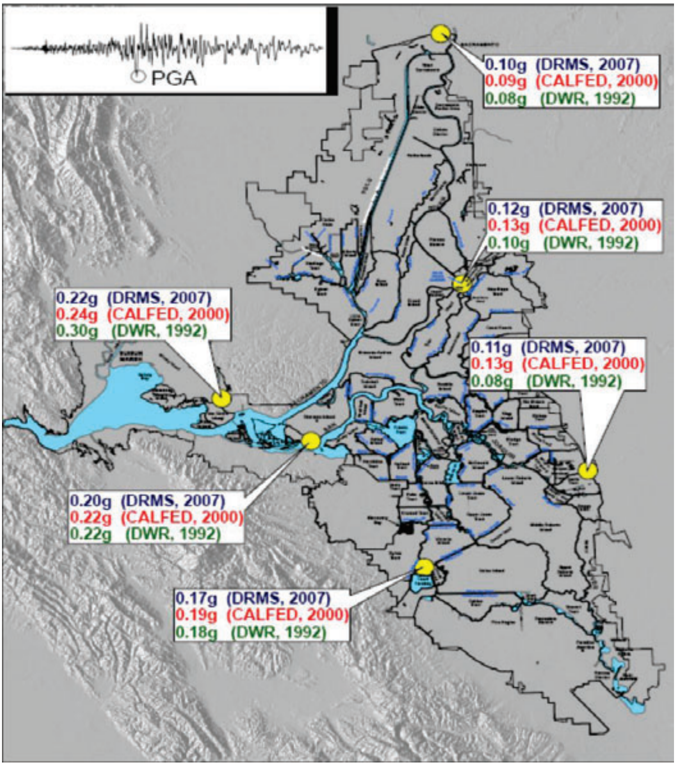

Fig. 17: The Sacramento Delta, with 200-year Return Level with Peak Ground Accelerations Indicated (Calif. Dept. of Water Resources, 2009)

Foundation conditions are variable and highly challenging, and the poor history of levee evolution massively compounds this. There have been 163 levee failures in the Delta since 1900, and this has become an increasingly expensive nuisance as it is generally necessary to repair the breach and then pump out the island to prevent intrusion of increased salt content from the seawater of San Francisco Bay which would disrupt the ability to transmit freshwater supplies across the Delta. These periodic levee failures are readily repairable, albeit at some cost, and so they do not present any imminent risk with regard to potential long-term disruption of the ability to transit freshwater across the Delta for use by people, agriculture and industry.

There are, however, two risks to the Delta's role as a principal hub of California's extensive water systems that are of sufficient gravity that the fragility of the Sacramento Delta as a water source was, in the Spring of 2010, the first "civil engineering" issue ever to make a U.S. President's list of the top ten strategic risks to the nation. They are also the reasons that the Delta is officially one of California's leading long-term concerns (as opposed to the ever-present short-term issues of State budgets and debt). The first of these is the dynamic tension between water needs for humans, and water needs for an eco-system that is of immense importance. Battles between water interests and eco-system interests have created a political logjam of more than 50 years duration. This logjam has only recently been broken open by the Governor's 2007-2010 Delta Vision process (Isenberg et al., 2008), which has led to a sweeping set of legislation and an important new window of opportunity to develop and implement comprehensive long-term solutions.

That is urgently important, because the second critical issue in the Delta is the constant risk of an earthquake that would produce devastating damages due to liquefaction and lateral spreading of many tens of miles of levees; a situation that would not be rapidly repairable and that would result in standing water rather than the flows currently constrained within narrow meandering streams and channels. That would produce salt intrusion from the adjacent San Francisco bay, and would salinate the Delta's waters producing a sustained interruption of water deliveries for multiple years until sufficient repairs could be made. Initial "direct" losses associated with such disruption of water deliveries are estimated at multiple tens of billions of dollars (DWR, 2009), but secondary and tertiary additional losses as the effects ripple through the State's economy and that of the nation are expected to be greater. Given the fragile state of the nation's economy, and the national debt, this poses a significant strategic risk with regard to potential disruption of the State's economy, and also to the economy of the nation.

Four sets of seismic fragility and/or risk studies have been made over the past three decades, culminating in the Delta Risk Management Strategy (DRMS) studies under the direction of the California DWR. Figure 17 shows 200-year return levels of PGA across the Delta from three studies. Current best estimates are that there is an annual likelihood of approximately 1% to 2% per year of catastrophic seismic disruption of the Delta's water delivery capabilities (California Department of Water Resources, 2009); an unacceptable level of such potentially catastrophic risk for a modern society.

Long-term solutions appear likely to involve construction of new water conveyance systems that will either be seismically robust, or at least rapidly repairable in the wake of a seismic event, that would bypass most or all of the fragile Delta. This could involve systems of canals and/or tunnels passing either to the east or west of the Delta, or tunnels passing directly beneath the Delta. Current cost estimates are on the order of \$8 to \$12 billion, but these continue to climb as NEPA/CEQA studies advance.

A key lesson in all of this is the massive importance of politics and policy. Engineering solutions can be readily developed once policy decisions are made, but there has been a half century and more of unremitting contention between the various stakeholders during which time: (1) the Delta's eco-systems have largely collapsed (and as a result water deliveries are currently largely under federal court management), (2) water deliveries have become increasingly unreliable (as a result of increasing eco-system fragility), and (3) seismic risk continues each year to pose a potentially catastrophic hazard to both. Even as solutions are developed, continued debate and litigation are expected to significantly delay eventual implementation of solutions to the current, and unacceptable, levels of ongoing hazard.

And California has numerous other levee issues as well. These include the flood protection systems for the massively urbanized Los Angeles/San Diego region, which appear likely to require re-evaluation as the USACE develops and promulgates new risk-based levee standards, and as these transition to an increased focus on life safety ahead of financial loss projections. California also has inadequate flood protection for communities from one end of the State to the other that are situated alongside numerous rivers that exit to the Pacific Ocean. And the vital water conveyance canals that carry water across large portions of the State are routinely contained over significant reaches by levee-like structures that have never been analyzed or designed with regard to potential seismic vulnerability.

Overall, it appears that California will need to spend approximately $10 billion or more to upgrade primarily Urban levees in the Central valley over the next 15 years, and additional expenses are likely to arise due to increased population growth into potentially flood-prone areas in the Central valley as the adjacent San Francisco Bay Area has become overbuilt. Additional expenditures of $10 billion or more will also be urgently needed to remedy unacceptable levels of risk of water supply disruption in the Sacramento Delta. And newly evolving national levee standards for urban regions are likely to require additional investments and upgrades for levees in densely populated southern California metropolitan regions, and for a number of other coastal communities as well. California appears to be heading towards expenditures of more than $30 billion over the next 20 to 30 years on levees and flood protection systems. Recent estimates are that the nation as a whole faces likely need for investments on the order of $50 to $100 billion for levees and flood protection works over the next 15 to 30 years (ASCE, 2009.) Based on California's situation, those national estimates appear to be on the low side.

SUMMARY

The wakeup call provided by Hurricane Katrina, and other recent U.S. flood disasters, is spurring multiple efforts at the federal level and at both the state and local levels to begin to address the chronic situation of the nation's current flood protection infrastructure. Good cooperation and collaboration between all of these levels will be vital; as all will be needed to address the massive shortcomings in U.S. flood defenses that have been allowed to develop over the past half century and more. Initial estimates suggest that expenditures of on the order of $50 to $100 billion will be required over the next 15 to 30 years to properly address the nation's flood protection infrastructure needs (ASCE, 2009), but experiences in California already suggest that these estimates may be too low and that considerably more may be required.

Similarly, it will be necessary to focus on policy, standards, organizational issues, and funding issues, as well as more traditional "engineering" analysis and design issues, if the nation is to resolve these massive problems. And policymakers must come to understand the importance of continuing operations and maintenance

(O&M), and periodic re-evaluations and upgrades as necessary, for flood protection infrastructure which tends to deteriorate over time.

The challenges are large, and they will only be further exacerbated by continuing population growth and increasingly risky development in potentially flood prone areas, by climate change, and by ongoing sea level rise. Fortunately, the nation appears to be rising to these challenges, and a number of promising initiatives are now underway; including efforts to create a National Levee Safety Program (NSLP) similar to the National Dam Safety Program initiated by Congress in 1996. It has been a long time in coming, and there are certainly challenges remaining to be resolved, but it now appears that the nation has an important opportunity to finally address flood risks that have been the nation's leading cause of catastrophic damages and loss over the past half century and more.

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Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls

Nicholas Sitar[1], Roozbeh Geraili Mikola[2], and Gabriel Candia[3]

[1]Edward G. Cahill and John R. Cahill Professor, Dept. of Civil and Environmental Engineering, UC Berkeley, Berkeley, CA 94720-1710, email: sitar@ce.berkeley.edu
[2]Ph.D. candidate, Dept. of Civil and Environmental Engineering, UC Berkeley, Berkeley, CA 94720-1710, e-mail: roozbehg@berkeley.edu
[3]Ph.D. candidate, Dept. of Civil and Environmental Engineering, UC Berkeley, Berkeley, CA 94720-1710, e-mail: gacandia@berkeley.edu

ABSTRACT

The evolution of the methodology for the evaluation of seismic earth pressures on retaining structures and basement walls is reviewed together with observations of field performance. The case history data and data from recent experimental work are used to show that the currently used methods are quite conservative and lead to excessively conservative designs in regions where design PGA exceeds 0.4g. Specifically, the experimental data from seismic centrifuge tests shows that the seismic earth pressure distribution for moderate size retaining structures, on the order of 6-7 m high, is triangular, increasing with depth. Moreover, there is no significant increase in seismic earth pressure between unbraced and braced structures with fixed base, while the loads on free standing cantilever structures are substantially lower owing to their ability to translate and rotate. The significance of the observed seismic earth pressure distributions is that the dynamic force can be applied at 1/3H, as is done for static loading, which substantially decreases the design level seismic moments on the structures.

INTRODUCTION

The problem of evaluating seismically induced lateral earth pressures on retaining structures has been first addressed in the 1920's in pioneering research carried out in Japan by Okabe (1926) and Mononobe and Matsuo (1929). Since then this problem has received periodic attention from the research community (e.g. Seed and Whitman, 1970; Nazarian and Hadjian, 1979; Prakash et al., 1969; Prakash, 1981; and Aitken, 1982); however, it had relatively little impact on design and engineering practice until relatively recently.

In the United States the Uniform Building Code (UBC) did not contain provisions for seismic design of retaining structures until 2003, although the California Building Code (CBC) contained provisions for certain types of building walls going back to 1980's (Lew et al., 2010b). Since then, however, the various provisions and recommendations have become more explicit and stringent, especially as it comes to the recommended method of analysis and the estimation of the design accelerations. The first comprehensive document to address this issue is the FEMA 450 document: "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" (NEHRP 2004) which has now been updated as FEMA 750 (NEHRP 2010). Both documents endorse the use of the M-O solution or the M-O solution as simplified by Seed and Whitman (1970) for "yielding walls" and the Wood (1973) solution for "non-yielding" walls. However, while Seed and Whitman (1970) use 0.85 PGA as the effective acceleration and it has been a general practice to use 0.67 PGA for design, the FEMA 750 document states:

> *"In the past, it was common practice for geotechnical engineers to reduce the instantaneous peak by a factor from 0.5 to 0.7 to represent an average seismic coefficient for determining the seismic earth pressure on a wall. The reduction factor was introduced in a manner similar to the method used in a simplified liquefaction analyses to convert a random acceleration record to an equivalent average series of cyclic loads. This approach can result in confusion on the magnitude of the seismic active earth pressure and, therefore, is not recommended. Any further reduction to represent average rather than instantaneous peak loads is a structural decision and must be an informed decision made by the structural designer."*

The immediate impact of this recommendation is on the applicability of the M-O method, since the earth pressure computed with the M-O method increases exponentially leading to very large forces at accelerations in excess of 0.6 g as discussed later. The practical consequence is that the M-O method cannot be directly applied in areas of high seismic demand and in practice requires a "work around", usually the addition of a small amount of cohesion. The Seed and Whitman (1970) approximation does not suffer from the same shortcoming as it is a straight line approximation to the M-O solution. On the other hand, this method recommends that the seismic earth pressure increment be applied at 0.6H, which, when combined with the recommended direct use of PGA, results in very large computed moments at the base of the retaining structure.

At this point the questions that have to be raised are: "What evidence do we have that indeed there is a problem requiring such step up in design recommendations?" and "Are the analysis and design methods appropriate for the proposed use?" The purpose of this paper is threefold: 1) address these questions using observations of seismic behavior of retaining structures and basement walls in past earthquakes; 2) review the theoretical and experimental basis behind the basic limit equilibrium design methods currently used in practice; and 3) present the results of recent experimental studies that introduce new data and suggest possible improvements to the currently used approaches.

OBSERVATIONS FROM EARTHQUAKES

A review of the performance of basement walls in past earthquakes by Lew et al. (2010a) shows that failures of basement or deep excavation walls in earthquakes are rare even if the structures were not explicitly designed for earthquake loading. Failures of retaining structures are most commonly confined to waterfront structures retaining saturated backfill with liquefaction being the critical factor in the failures. Failures of other types of retaining structures are relatively rare (e.g. Whitman, 1991; Al-Atik and Sitar, 2010) and usually involve a more complex set of conditions, typically, but not exclusively, involving sloping ground either above or below the retaining structure if not both. With these general observations in mind, the purpose herein is to illustrate the performance of retaining structures in different settings.

The photograph in Figure 1 shows failures of gravity retaining structures for a cut slope and a railroad embankment caused by the magnitude 7.9 (7.9 USGS, up to 8.2 in other sources) Great Kanto earthquake which devastated the greater Tokyo and Yokohama region of Japan in 1923. The retaining wall above the rail line has a slope above it while the base of the railway embankment is in a waterway. Thus, any number of possible failure modes can be postulated and the photograph is a good illustration of the shortcomings of attempting to evaluate the cause and mode of failure from photographic evidence alone. Nevertheless, it does not appear that seismically induced earth pressure would have been the principal cause of failure of both retaining structures, as bearing capacity and simple slope failure would also be good candidates.

FIG. 1. Failed retaining walls caused by the Great Kanto earthquake (courtesy of the Sykes Kanto Collection, NISEE, UC Berkeley).

A more recent failure shown in Figure 2 is a gravity retaining structure along a road cut in central Taiwan which partially failed in the 1999 Chi-Chi earthquake. This retaining wall was a classic trapezoid shaped gravity retaining structure with no apparent provision for seismic loading (i.e. broader section at the base). Most importantly, however, the steep slope above the wall failed and, while that was a seismically induced slope failure, the seismic earth pressure induced by ground motions alone was not the cause of failure. Failures of the kind shown in Figures 1 and 2 were also observed in the 1995 Hyogoken-Nambu earthquake in Kobe, Japan (Japanese Geotechnical Society, 1996) and they typically involved older structures supporting poorly compacted embankments and/or on sloping ground.

FIG. 2. Trapezoidal concrete gravity retaining wall which rotated outward due to failure of the slope above during the 1999 Chi-Chi earthquake in Taiwan (N. Sitar photo).

Overall, however, there is no evidence of a systemic problem with traditional static retaining wall design even under quite severe loading conditions (see e.g. Gazetas et al., 2004). No significant damage or failures of retaining structures occurred in the 1998 Wenchuan earthquake in China, or in the recent great subduction zone generated earthquakes in Chile (2010) and Japan (2011). Figure 3 is a photograph of a cobblestone gravity wall supporting an unfinished overpass in Mianzhu City, China, and Figure 4 which shows a series of highway underpass structures south of Concepcion, Chile, neither of which experienced any distress. This observation is consistent with the conclusion reached by Seed and Whitman (1970) who noted that gravity retaining structures designed for adequate factor of safety under static loading should perform well under seismic loading for PGA up to about 0.3 g.

Finally, the most challenging aspect of documenting and interpreting field performance is the fact that well documented case histories with actual design and performance data for modern retaining structures are very sparse. A rare, well documented case history of the performance of flood channel walls in the Los Angeles basin during the 1971 San Fernando earthquake is presented by Clough and Fragaszy (1977) who show that reinforced concrete cantilever structures, well designed and detailed for static loading, performed without any sign of distress at accelerations up to about 0.4 g, which is consistent with the previously mentioned conclusion by Seed and Whitman (1970).

FIG. 3. Cobblestone gravity retaining wall for an overpass, Mianzhu City, China, 2008 (N. Sitar photo).

FIG. 4. Reinforced concrete cantilever walls south of Concepcion, Chile, 2010 (N. Sitar photo).

PRIOR EXPERIMENTAL DATA

Prakash (1981) identified three questions that need to be answered when designing a retaining wall for seismic loads:

- What is the magnitude of total (seismic plus dynamic) earth pressure on the wall?
- Where is the point of application of the resultant?
- How much has the structure been displaced?

To answer these questions numerous experimental studies have been performed in the past and their results are very much a function of their experimental design. In order to be able to approach the problem in a general and cost effective manner, most of the physical model studies used either the shaking table or the geotechnical centrifuge to model this complex, dynamic, soil-structure interaction problem.

Shaking Table Experiments. The seminal experiments using a shaking table were performed by Mononobe and Matsuo (1929). Their original shaking table design consisted of a rigid base box mounted on rails and driven with an ingenious conical drum winch connected through a crankshaft to the base of the box (Figure 5). This arrangement allowed for simple application of sinusoidal excitation with linearly varying frequency, i.e. a frequency sweep. The ends of the box were trap doors, spring mounted at the base, with pressure gauges mounted at the top to measure the load as the "wall" tilted outward. As shown in the figure, the box dimensions were 9 ft long, 4 ft wide and 4 ft deep, with one door, door A, spanning the whole width of the box and the other door, door B, spanning only one half of the width of the box. Although, the box was quite substantial in size, the depth of the medium dense sand fill was only 4 ft and the sides of the box were rigid.

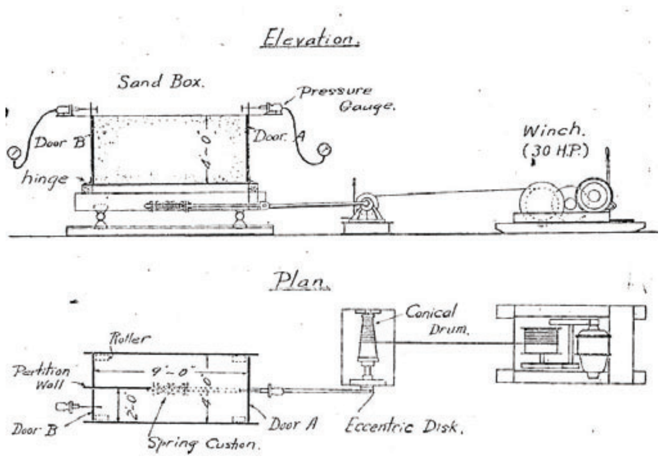

FIG. 5. Shaking table arrangement used by Mononobe and Matsuo (1929).

Since then numerous results of 1-g shaking table experiments have been reported in the literature, including Matsuo (1941), Ishii et al. (1960), Matsuo and Ohara (1960), Sherif et al. (1982), Bolton and Steedman (1982), Sherif and Fang (1984), and Ishibashi and Fang (1987). In general, the results of these studies were in agreement with the original results obtained by Mononobe and Okabe (1929) as far the total resultant thrust, but a consensus emerged that the point of application of the resultant thrust should be higher than one third the height of the wall above its base. However, as pointed out by a number of investigators (see e.g. Ortiz et al. 1983, Stadler, 1996), the 1-g shaking table experiments share a number of important limitations, namely: scaling of stress and stiffness using granular, cohesionless soil is problematic under 1-g conditions; the boundaries cannot be moved sufficiently far away from the structure as the size of the model increases; and, the most problematic of all is the boundary at the base of the model, since a model mounted directly on a shaking table is essentially founded on the outcrop generating the input motion, i.e. bedrock. Hence, a retaining structure founded on a compliant foundation cannot be readily modeled on a shaking table at 1-g due to scaling limitations.

Centrifuge Model Experiments. Centrifuge models have the advantage that they avoid the obvious limitations of the 1-g shaking table and scaling laws can be accurately followed. However, they also present challenges particularly when it comes to the ability to of conventional instrumentation to adequately respond at frequencies dictated by the scaled model response characteristics, as discussed later.

Dynamic centrifuge tests on model retaining walls with dry and saturated cohesionless backfills have been performed by Ortiz (1983), Bolton and Steedman (1985), Zeng (1990), Steedman and Zeng (1991), Stadler (1996), and Dewoolkar et al. (2001). Bolton and Steedman conducted dynamic centrifuge experiments on concrete (1982) and aluminum (1985) cantilever retaining walls with dry dense sand backfill and measured dynamic forces in agreement with the values predicted by the M-O method, but suggested that the point of application should be located at mid-height of the wall. More recently, Stadler (1996) performed a series of centrifuge model experiments on cantilever walls founded on a rigid foundation and observed that the total lateral earth pressure distribution was approximately triangular, while dynamic earth pressure distribution varied between triangular and rectangular. However, the magnitude of the forces was less than would be predicted using the M-O method and the experimentally measured moments were on the order of 20 to 75% less than would be predicted using the M-O method. Since Stadler's model walls were mounted directly on the base of the container, as was the case with most of the above mentioned centrifuge studies, they shared some of the limitations of placing the model on "bedrock" inherent in the 1-g shaking table experiments, albeit with much more rigorous model scaling.

A cantilever wall with dry medium-dense sand backfill founded on soil was examined in a series of dynamic centrifuge experiments by Ortiz et al. (1983). They also observed a broad agreement between the maximum measured forces and the M-O predictions; however, they found that the maximum dynamic force acted at about one third the height of the wall above its base. Similar conclusion regarding the point of application of the maximum dynamic force was reached by Al-Atik and Sitar (2010) who also modeled retaining structures on soil foundation, although they found that, in general, the loads predicted with the M-O method exceeded those measured in the centrifuge experiments.

Gravity walls with dry medium-dense sand backfill on soil foundation were studied by Nakamura (2006). He concluded that contrary to the M-O rigid wedge assumption, the part of the backfill that follows the displacement of the retaining wall deforms plastically while sliding down. Also, while the M-O theory assumes that no phase difference occurs between the motion of the retaining wall and backfill, Nakamura (2006) observed that the acceleration is transmitted instantaneously through the retaining wall and then transmitted into the backfill and, therefore, the dynamic earth pressures and inertia forces are not in phase. Most, importantly, while the wall as a whole tended to translate laterally, the dynamic increment in load did not significantly exceed the static load and the earth pressure remained roughly triangular with depth.

ANALYSIS AND DESIGN

Mononobe – Okabe (M-O) Method and Its Derivatives for Yielding Walls. The M-O method is based on the work of Okabe (1926) and Mononobe and Matsuo (1929) following the great Kanto Earthquake of 1923 in Japan. It was originally developed for gravity walls retaining cohesionless backfill materials and it is today the most common approach to determine seismically induced lateral earth pressures on a variety of structures.

This method uses a pseudo-static analysis based on the Coulomb wedge theory for active and passive earth pressure and includes additional vertical and horizontal seismic forces, as shown in Figure 6. The inherent assumptions in the method are as follows:

- The wall yields sufficiently to produce minimum active pressures
- When the minimum active pressure is attained, a soil wedge behind the wall is at the point of incipient failure and the maximum shear strength is mobilized along the potential sliding surface
- The soil behind the wall behaves as a rigid body

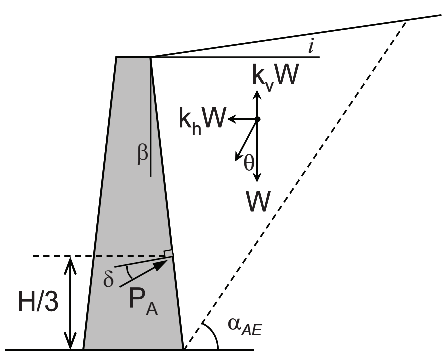

FIG. 6. Forces Considered in the Mononobe-Okabe Analysis.

Using force equilibrium the total active thrust P_{AE} per unit length of wall is determined by:

$$P_{AE} = 0.5\gamma H^2(1-k_v)K_{AE} \tag{1}$$

Where,

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta\cdot\cos^2\beta\cdot\cos(\delta+\beta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta)\cdot\cos(i-\beta)}}\right]^2} \tag{2}$$

and γ = unit weight of the soil, H = height of the wall, ϕ = angle of internal friction of the soil, δ = angle of wall friction, β = slope of the wall relative to the vertical, $\theta = \tan^{-1}(k_h/(1-k_v))$, k_h = horizontal acceleration (in g), and k_v = vertical acceleration (in g).

Equation (1) represents the total active thrust on the wall during seismic loading and the point of application of the resulting force is at 1/3H. As already mentioned, a serious limitation of equation (2) is that it increases exponentially and does not converge if $\theta < \phi - \beta$ (e.g. Kramer, 1996), which for typical values of angle of internal friction means accelerations in excess of 0.7 g. The approach adopted in such cases tends to be to use the simplified equation introduced by Seed and Whitman (1970) that separated the total force on the wall into a static and dynamic component. They then proposed a simplified expression for the dynamic increment of the active thrust as:

$$\Delta P_{AE} = \tfrac{1}{2}\gamma H^2 \Delta K_{AE} \quad \text{and} \quad \Delta K_{AE} \sim \tfrac{3}{4} k_h \tag{3}$$

where k_h is the horizontal ground acceleration as a fraction of the acceleration of gravity. This approximation is asymptotically tangent to the M-O solution at accelerations below about 0.4 g and it remains linear throughout. In addition, Seed and Whitman (1970) recommend that the resultant of the dynamic force increment be applied at 0.6H, hence introducing the concept of the "inverted triangle" to the distribution of the dynamic force increment.

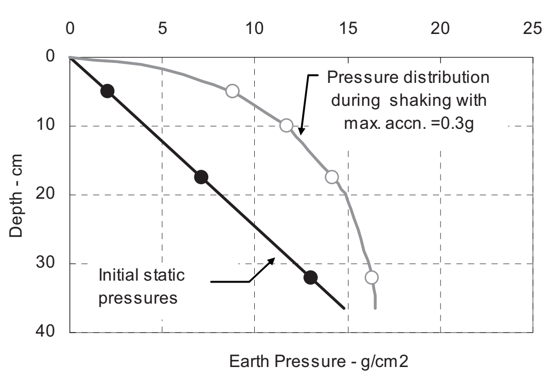

FIG. 7. Total (dynamic plus static) vs. static earth pressure (Seed and Whitman (1970) after Matuso, 1941).

The impetus for the recommendation by Seed and Whitman (1970) to place the resultant of the dynamic force increment at 0.6H is their interpretation of Matsuo's (1941) experimental results as reproduced in Figure 7. As stated earlier, Mononobe and Okabe considered that the total pressure computed would act at a height of H/3 above the base wall. However, a consensus has not been reached regarding the appropriate point of application of the dynamic force increment as can be seen from Table 1, which lists the recommendations of various researchers whose earth pressures agree more or less well with the M-O earth pressures.

Table 1: **Recommended Point of Application of The Seismic Force Increment**

Author	Point of Application
Mononobe-Okabe (1926-1929)	0.33H
Seed and Whitman (1970)	0.6H
Nandakumaran and Joshi (1973)	<0.65H
Krishna et al. (1974)	~0.5H
Sherif et al. (1982)	~0.42H
Prakash and Brasavanna (1969)	varies with acceleration
Ichihara and Matsuzawa (1973)	varies with acceleration
Ortiz et al. (1983)	varies, but higher than H/3
Woodward and Griffiths (1992)	varies with acceleration
Steedman and Zeng (1990)	varies, but higher than H/3
Mylonakis et al. (2007)	0.33H

Whereas the position of the point of application of the resultant has been the subject of a continuing discussion, many researchers (performing both analytical and experimental studies) have agreed that the earth pressures determined by the M-O method for "yielding walls" give adequate results (e.g. Matsuo (1941), Ishii et al. (1960), Prakash and Basavanna (1969), Seed and Whitman (1970), Ichihara and Matsuzawa (1973), Clough and Fragaszy (1977), Bolton and Steedman (1982), Sherif et al. (1982), Ortiz et al. (1983), Musante and Ortigosa (1984), Ishibashi and Fang (1987), Steedman and Zeng (1990), Woodward and Griffiths (1992), Anvar and Ghahramani (1995), Richards et al (1999)). Most recently, Mylonakis et al. (2007) proposed a modification of the M-O method which results in a single equation for both, active and passive condition. Their active solution is more conservative and gives a total force somewhat higher than M-O, whereas their solution for the passive case is less conservative than the M-O solution. They recommend 1/3H as the point of application of the total force resultant.

Given the above, it is important to reiterate that Seed and Whitman (1970) suggested that a "yielding" wall designed adequately for static forces will probably perform well under seismic loading. This suggestion has been supported by Clough and Fragaszy (1977) who proposed that a cantilever retaining structure with an adequate factor of safety for static forces will have enough reserve to resist dynamic loads up to an acceleration of at least 0.4g. Similarly, Whitman (1991) states that:

> "*Structures away from waterfronts have generally fared well during earthquakes. Examples of stability-type failures are rare…*"

The same sentiment is echoed by Huang (2000) who noted that failures of gravity walls are generally due to bearing capacity failures and not due to substantial increase in earth pressures. Finally, the commentary to ATC 32 states that most free-standing retaining walls not associated with other structures have performed well during past earthquakes even though no particular seismic design was implemented, which is consistent with the observations noted above and with the authors' own experience.

Not everyone agrees with this point of view. Richards and Elms (1979) argued that the M-O approach would lead to an unconservative estimate of the dynamic thrust because the inertia of the wall is not considered. They suggested a new design approach based on allowable displacements. This method based on Newmark's (1965) approach for determining seismically induced displacements in dams and embankments was later reviewed and recommended by Whitman and Liao (1984). Later, Steedman and Zeng (1996) and Zeng and Steedman (2000) present a displacement based method for the analysis of gravity walls using the same principles. Most recently, Bray et al. (2010) use the results of analytical and experimental studies to present a displacement based approach suitable for level and sloping ground conditions. Nevertheless, currently, the M-O method remains the most widely used method for "yielding" retaining walls.

Dynamic Earth Pressures on Non-Yielding/Rigid Walls. The currently accepted tenet is that seismically induced earth pressures on rigid walls, such as basement walls, are higher than those on yielding walls. This is of course true for the static case where earth pressures for yielding walls are performed with the active earth pressure coefficient K_A, whereas the at-rest earth pressures K_0 are used for unyielding walls. Theoretical results obtained by Wood (1973) are still considered as a standard for this case and the total dynamic thrust is approximated as $\Delta P_{AE} = \gamma H^2 A$ and the resultant acting at 0.6H above the base (FEMA 750). This results in a total thrust and corresponding moment approximately 2-3 times higher than that predicted by Seed and Whitman (1970) for yielding walls. The solution assumes a rigid wall being acted upon by elastic soil connected to a rigid foundation (Figure 8).

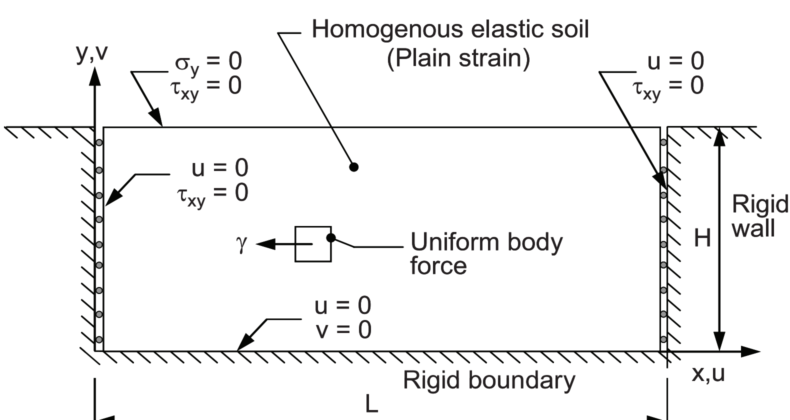

FIG. 8. Geometry and boundary conditions assumed by Wood (1973).

FEMA 750 interpretation of the above condition is as follows:

"*Most nonyielding walls will be located on rock or very stiff soil. Even in this condition, wall flexibility can be sufficient to develop active seismic earth pressures significantly reducing the loading on basement walls. Where a basement wall is located on rock or very stiff soil and where structural analyses determine that the wall flexibility is such that deformations will not develop seismic active earth pressures (i.e., deformations < 0.002H where H is the wall height), the wall should be designed as a nonyielding wall.*"

Note that the assumed geometry is more akin to a rigid box constraining elastic infill, as opposed to soil acting as an external load on a structure. In spite of these unique boundary conditions assumed by Wood (1973), this approach has been adopted by other researchers and similarly high pressures for unyielding walls were obtained by Matsuo and Ohara (1960), Scott (1973), Prakash (1981), Sherif et al. (1982), Veletsos and Younan (1997), Zhang et al. (1998), Ostadan and White (1998) and Ostadan (2005).

Ostadan (2005) (also Ostadan and White, 1998) proposed a simplified method which incorporated the main parameters affecting the seismic loading on basement walls and Ostadan's work is currently recommended by NEHRP (FEMA 750) provisions for seismic regulations of structures. His analyses show that the application of this method yields results that correspond in magnitude to the M-O method as a lower bound and the Wood solution as the upper bound which is as much as 2 to 2.5 times greater than the M-O solution.

However, if loads of this magnitude were to have been experienced by basement walls we would expect to have seen evidence of damage to deeply embedded structures/basements in recent earthquakes with very strong ground motions in heavily developed areas which has not been the case (Lew et al. 2010a, 2010b). Whitman (1991) addressed this apparent inconsistency by suggesting that because basements move relative to the foundation soil owing to soil structure interaction, dynamic earth pressures equal to those obtained by M-O are adequate except for a special condition of structures founded at a sharp interface between soil and rock. Similarly, Veletsos and Younan (1997) and Younan and Veletsos (2000) suggest that a substantial decrease from the typical "rigid" solutions is usually obtained for walls of realistic flexibilities. Most recently, Ahmadnia et al. (2011) performed a series of numerical analyses assuming typical basement wall configurations including cross struts and found that the M-O method, as simplified by Seed and Whitman (1970), gave adequately conservative results, with the point of application of the dynamic force at about 0.5 H.

Elastic Wave Propagation Methods. One alternative to the limit equilibrium method of analysis used in the M-O analysis is to consider elastic wave propagation. Based on Zeng's (1990) dynamic centrifuge experiments, Steedman and Zeng (1990) suggested that the dynamic amplification or attenuation of input motion through the soil and phase shift are important factors in the determination of the magnitude and the distribution of dynamic earth pressures. In their derivation they assumed the same Coulomb wedge as in the M-O method and they did not consider the reflected wave at the ground surface, as illustrated in Figure 9. The consequence of this assumption is that the computed earth pressure response is not amplified near resonance as shown in Figure 10. Their solution gives results which are equal or lower than the M-O method and suffers from the same limitations as the M-O method at high accelerations. It is important to note that the results in Figure 10 are for specific material properties in order to obtain the soil period, in this case $\varphi = 33°$, $\delta = 16°$, $V_s = 109$ m/s, $G_{max} = 20$ GPa, and H=10m. In addition, the computed earth pressure distribution at or below resonance i.e. the soil period matching the input motion period, is triangular, increasing with depth, and identical to that assumed in the M-O method. The resulting dynamic force increment is applied in the range 0.33 to 0.4H from the base of the wall for most typical conditions.

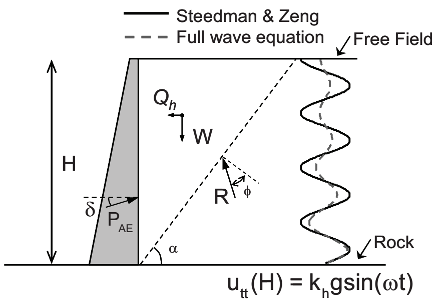

FIG. 9. Geometry of the problem as defined by Steedman and Zeng (1990).

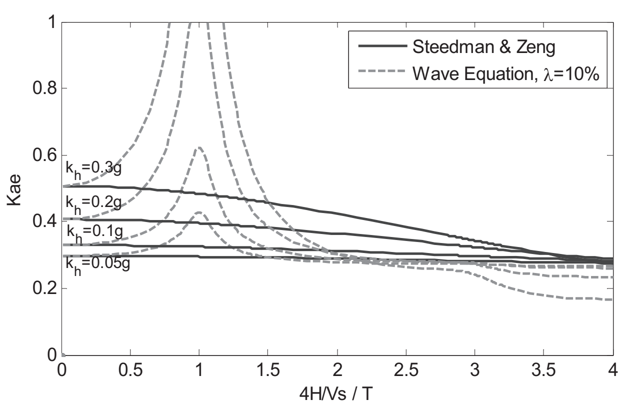

FIG. 10. Dynamic earth pressure increment as a function of the ratio of the natural period of the soil deposit and the input motion period.

Effect of Cohesion. All of the above mentioned analytical solutions assume ideal cohesionless backfill. Such conditions occur rarely and in such cases high water table and liquefaction susceptibility often drive the problem. In most typical cases, the natural deposits exhibit some degree of cohesion (see Figure 1) and, therefore, are unlikely to mobilize the kind of forces predicted assuming ideal cohesionless backfill.

The contribution of cohesion to the reduction of seismic earth pressure on retaining structures is explored by Anderson et al. (2008) who present a series of example charts to illustrate the reduction in computed seismic coefficient as a function of cohesion. They conclude that the "*reduction for typical design situations could be on the order of about 50 percent to 75 percent*" and ascribe the good observed seismic performance of retaining structures in part to the presence of cohesion in typical backfill. Similar conclusions are reached by Lew et al. (2010a,b) based on observations of seismic performance of different types of retaining structures in recent earthquakes.

DATA FROM RECENT EXPERIMENTS

The preceding discussion exposed some of the dilemmas facing researchers and practitioners in trying to determine the best approach to adopt in dealing with seismically induced earthpressures based on the results of previous experimental studies and field observations. To address some of these issues the authors are in the midst of an experimental program to evaluate seismically induced earth pressures on different types of retaining structures and to-date 3 dynamic centrifuge experiments with 3 different configurations of retaining structures and backfill were performed, as follows: 1) two identical U-shaped structures with cross bracing and medium dense sand backfill; 2) a cantilever U-shaped structure and a free standing cantilever wall with medium dense sand backfill ; and, 3) a cross-braced U-shaped structure and a free standing cantilever with compacted low plasticity silty clay (Yolo Loam) backfill.

Centrifuge Model Configurations. The centrifuge experiments were performed at the Center for Geotechnical Modeling at the University of California, Davis, using the flexible shear beam container. The centrifugal acceleration used in the experiments to-date has been 36g and all test results are presented in terms of prototype units unless otherwise stated. The experiments with the 6.5 m high, U-shaped cantilever walls with sand backfill followed the same model construction procedures and used the same model structures as used by Al-Atik and Sitar (2010) in order to allow a direct comparison of the results. The same aluminum model structures were used and lead was added to the structures to match the mass of the prototype reinforced concrete structures. The structures were fully embedded in dry sand backfill with relative density of 72% and were underlain by approximately 12.5m of sand or clayey silt in prototype scale.

The basement type (rigid structures) were modeled by modifying the U-shaped models by adding two levels of struts. The struts were instrumented with load cells in order to obtain direct measurements of the dynamic loads on the walls as shown in Figure 11. The free standing cantilever model was based on scaling of a 6.5 m prototype of California Department of Transportation design cantilever wall (Caltrans, 2010). This design incorporates a shear key in the footing in order to limit potential sliding of the footing during seismic loading. The silty clay (Yolo Loam) has PI = 11% and LL = 30%. It was hand compacted at a water content 2% above optimum to a relative compaction of 90% of Standard Proctor. Figure 12 is a diagram showing the configuration of the experiment with compacted clay silt which included one free standing cantilever wall and a cross-braced U-shaped structure. Scaled earthquake records developed by Al Atik and Sitar (2010) were used in order to maintain consistency for the purposes of comparison of the results. Multiple shaking events covering a wide range of predominant periods and peak ground accelerations were applied to each model in flight.

FIG. 11. Photograph of the cross-braced, U-shaped structure model with sand backfill. Note the load cell on each strut.

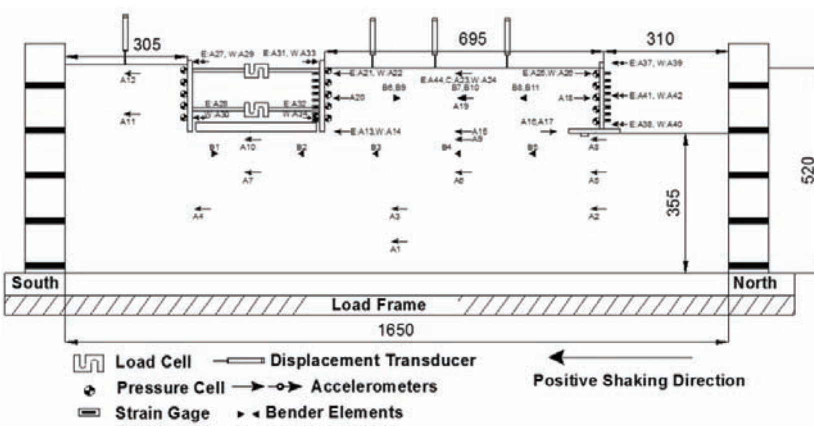

FIG. 12. Schematic of the layout of the 3rd experiment showing the positions of different transducers and the geometry of the models. All dimensions are in mm.

The models were instrumented to measure accelerations, displacements, bending moments and earth pressures. Soil settlement and the deformation and settlement of the structures were measured at different locations using a combination of spring loaded LVDTs and linear potentiometers. The lateral earth pressures were measured directly using flexible Tactilus™ pressure sensors. Lateral earth pressures on the cantiler structures were also calculated by double differentiating bending moments measured by the strain gages mounted on the model walls.

Dynamic Earth Pressures. As already discussed, the issue of the distribution and magnitude of the seismically induced earth pressures is very important since it bears directly on the computed forces and, most importantly, from a structural design perspective, it directly affects the magnitude of the computed shear and moment demand on the structure. Figures 13 and 14 show the measured distribution and magnitude of the dynamic earth pressure increment for structures retaining cohesionless and cohesive backfill, respectively, subjected to filtered and scaled Kobe-TAK 090 input motion (Al Atik and Sitar, 2009).

The corresponding dynamic earth pressure increment obtained from the M-O method and from the Seed and Whitman (1970) solution are plotted for comparison. All plots correspond to the time of maximum moment on the respective structures.

Overall, the data show that the seismic earth pressure increments increase with depth consistent with static earth pressure distribution and consistent with the M-O solution as the upper bound for the experimental results. The "inverted triangle" dynamic earth pressure increment, as computed using the Seed and Whitman (1970) solution, while producing a reasonable magnitude of the maximum dynamic earth pressure increment, does not reflect the observed distribution of the dynamic earth pressure increment. These results are quite consistent with the results previously obtained by Ortiz et al. (1983), Stadler (1996) and Al-Atik and Sitar (2010).

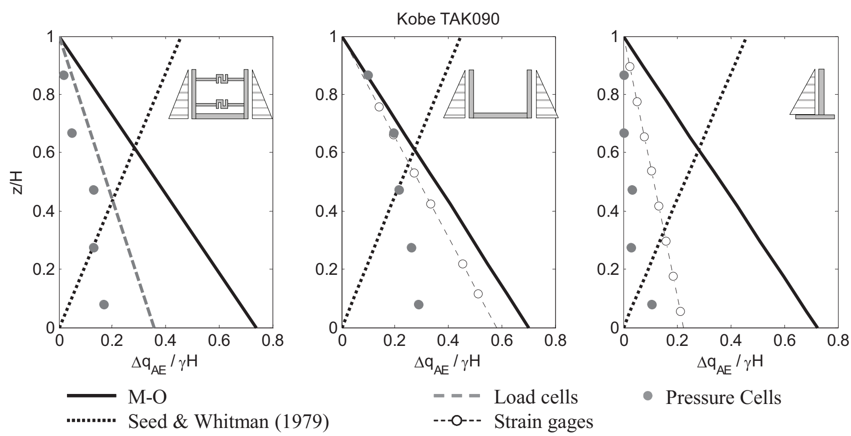

FIG. 13. Dynamic earth pressure increment from experiments with medium dense sand backfill in response to scaled Kobe-TAK090 input motion.

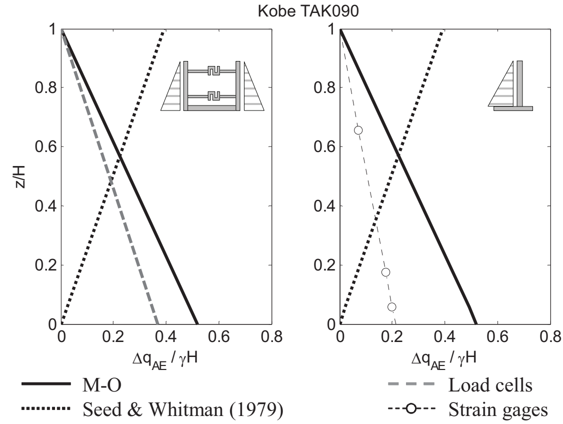

FIG. 14. Dynamic earth pressure increment from experiments with compacted silty clay backfill in response to scaled Kobe-TAK090 input motion.

Figures 15, 16 and 17 are plots of the experimental results in terms of the dynamic earth pressure coefficient, ΔK_{ae}, together with the curves obtained using the most common analytical solutions against free field PGA at the point of maximum measured moment on the respective structures. The plots show that the experimental data exhibit considerable scatter with increasing PGA and duration of the input ground motion. The scatter is particularly large for the models with the U-shaped cantilever structures and is most likely related to a combination of factors, including slight variations in relative density of the models and possible boundary effects that become more pronouced at higher accelerations.

Nevertheless, the overall trends in the data show that the Seed and Whitman (1970) approximation represents a reasonable upper bound for the value of the seismic earth pressure increment for both fixed base cantilever structures (U-shaped walls, Figure 15) and cross-braced, basement type, walls (Figure 16). In comparison, the M-O solution and the Mylonakis et al. (2007) solution are considerably higher than measured values at accelerations above about 0.4 g. The equivalent Wood (1973) seismic coefficient, computed using the prototype structure dimensions, clearly exceeds all other results by a considerable margin, as would be expected based on the assumptions used in deriving this solution which were discussed earlier.

The most significant difference between the analytically predicted seismic earth pressure increment and the observed data is for the free standing cantilever walls. The fact that small amount of rotation and translation can significantly decrease the forces acting in a retaining structure have been well recognized (e.g. Anderson et al., 2008, Bray at al., 2010) and the data presented in Figure 17 clearly shows this to be the case. In order to arrive at more moderate seismic earth pressures it is a common practice to include a small amount of cohesion (Anderson et al. 2008). The curve for cohesive backfill plotted in Figure 17 corresponds to the material properties of the compacted silty clay used in the experiment. As the plot show this is still a very conservative result. Overall, this data is consistent with the position presented by Seed and Whitman (1970) who suggested that well designed retaining structures should be well capable of withstanding ground motions with PGA on the order of 0.3g without the need for specific seismic design.

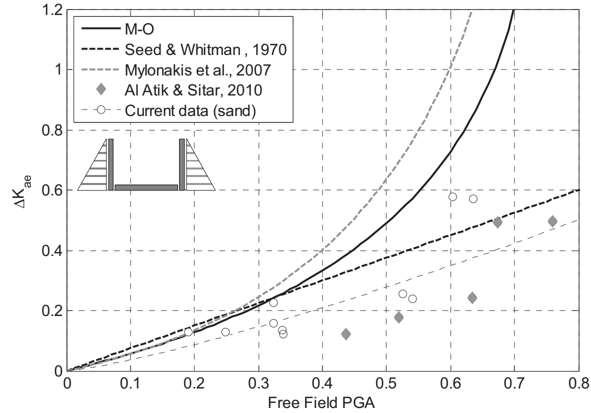

FIG. 15. Seismic earth pressure coefficient as a function of PGA for U-shaped cantilever walls with medium dense sand backfill.

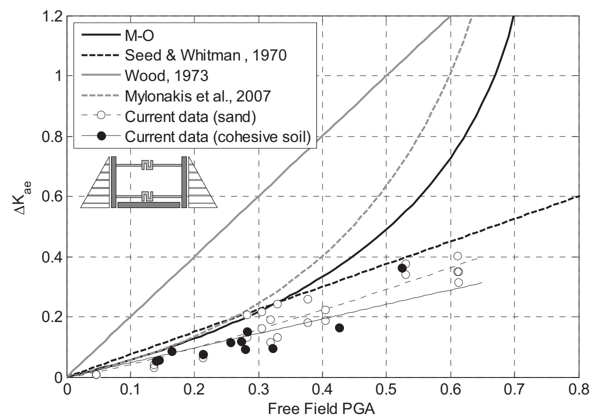

FIG. 16. Seismic earth pressure coefficient as a function of PGA for cross-braced walls.

Dynamic Moments. Dynamic moments are ultimately the quantities that dictate the structural design of the retaining structures. Clearly, the magnitude of the seismic earth pressure increment is very important in this regard. However, even more important is the point of application of the resultant, since the decision whether to apply the resultant at 0.6H versus 1/3H immediately changes the computed moment

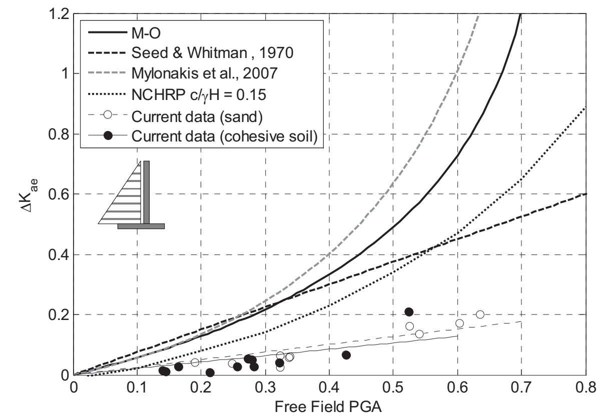

FIG 17. Seismic earth pressure coefficient as a function of PGA for free standing cantilever walls.

by a factor of about 2. The significance of this effect is illustrated in Figure 18 showing the dynamic moment increment plotted against PGA for the case of a free standing cantilever retaining wall. These results show that the M-O method gives amply conservative results over the full range of accelerations and that applying the seismic earth pressure increment at 0.6H, as recommended by Seed and Whitman (1970) and many others, leads to a significant, if not unnecessary, overdesign.

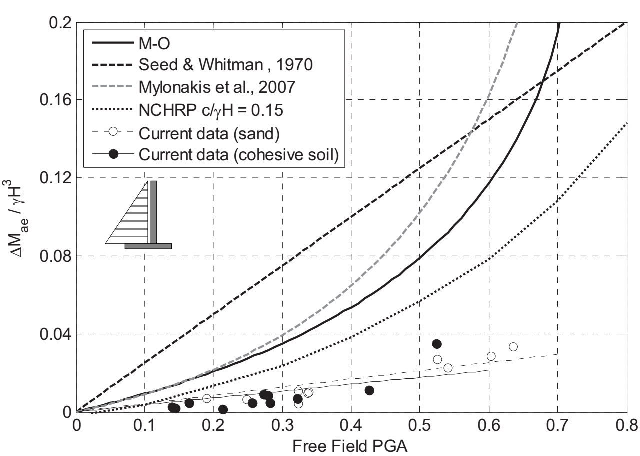

FIG. 18. Maximum dynamic moment increment as a function of PGA for free standing cantilever walls.

DISCUSSION AND CONCLUSIONS

The review of the previously developed analysis and design procedures and of the corresponding experimental data shows a significant difference in the observed and perceived distribution of the seismic earth pressure increment. The most likely difference between the different experimental results appears to be in the execution of the experiments. Specifically, 1-g shaking table experiments suffer from significant scaling and boundary condition limitations that cannot be easily overcome. The most significant limitation is the rigid base of the models built directly on the shaking table. In this respect, some of the past centrifuge experiments share the same limitation and it is the data from these experiments, starting with the work of Mononobe and Matsuo (1929) that suggest that seismic earth pressures increase toward the ground surface. In contrast, the seismic centrifuge experiments in which the structures are based on a soil foundation, show the opposite trend, with the seismic earth pressure increment increasing downward. These results point to the need to carefully evaluate the suitability of experimental facilities, especially 1-g shaking tables, for modeling of structures embedded in soil or based on soil foundation.

The most direct impact of the recognition that the point of application of the seismic earth pressure increment can be reasonably placed at 1/3H is the reduction in the computed design moments for the structure. Another important aspect of the results presented herein is the observation that stiff embedded structures do not seem to experience substantial increase in seismic earth pressure over that experienced by cantilever structures with fixed base. In this regard the Wood (1973) solution is not representative of conditions commonly encountered in practice and its continued use is not recommended.

The new experimental data suggest that the simplified M-O method proposed by Seed and Whitman (1970) provides an ample and reasonable upper bound for the expected magnitude of seismic earth pressure increment for moderate height retaining structures, 6-7 m high, in level ground, recognizing that the resultant force should be applied at 1/3H. The same applies to basement walls or cross-braced excavations. However, caution should be used in extrapolating these results to deeper embedded structures. There is no theoretical basis that would lead one to expect that seismic earth pressures continue to increase monotonically with depth. At some point the structures will begin to move with the soil and their response will become more consistent with that of tunnels which are well known to perform very well under seismic loading. An additional limitation of the experimental results presented herein is that they apply only to level ground. However, retaining structures are frequently placed on slopes with sloping backfill and sloping ground below. This type of setting requires a different approach, as slope stability, rather than earth pressure may be the governing mechanism of failure (Bray et al., 2010). At present this distinction frequently is not made.

All these issues deserve further careful evaluation, since the costs of an over-conservative design can be just as much of a problem as the cost of a future failure. In this respect, while there is a need for further experimental work, there is a much greater need for the development of a database of field observations from instrumented sites and structures. Only then we will be able to evaluate fully the actual performance under a variety of conditions. The development of such data is essential if we are to advance the state of the art and take the full advantage of advanced analytical tools that go hand in hand with modern performance based design.

ACKNOWLEGMENTS

The experimental program carried out in this research could not have been executed without the able assistance of Nathaniel Wagner and Jeff Zayas of the University of California at Berkeley. Dr. Dan Wilson and the staff of the Center for Geotechnical Modeling at UC Davis have been most accommodating and provided outstanding environment for a truly collaborative effort. The research funding was provide in part by a grant from the California Geotechnical Engineering Association (CalGeo), the State of California Department of Transportation (Caltrans) Contract No. 65N2170 and NSF-NEES-CR Grant No. CMMI-0936376: Seismic Earth Pressures on Retaining Structures. G. Candia was funded in part by a fellowship from the Chilean Commission for Scientific Research and Technology (CONICYT).

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Site Response in NEHRP Provisions and NGA Models

Emel Seyhan[1], S.M. ASCE and Jonathan P. Stewart[2], PE, F.ASCE

[1]Graduate Student, Civil and Environmental Engineering, University of California, Los Angeles, 90095; eseyhan@ucla.edu
[2]Professor and Vice Chair, Civil and Environmental Engineering, University of California, Los Angeles, 90095-1593; jstewart@seas.ucla.edu

ABSTRACT: Site factors are used to modify ground motions from a reference rock site condition to reflect the influence of geologic conditions at the site of interest. Site factors typically have a small-strain (linear) site amplification that captures impedance and resonance effects coupled with nonlinear components. Site factors in current NEHRP Provisions are empirically-derived at relatively small ground motion levels and feature simulation-based nonlinearity. We show that NEHRP factors have discrepancies with respect to the site terms in the Next Generation Attenuation (NGA) ground motion prediction equations, both in the linear site amplification (especially for Classes B, C, D, and E) and the degree of nonlinearity (Classes C and D). The misfits are towards larger linear site factors and stronger nonlinearity in the NEHRP factors. The differences in linear site factors result largely from their normalization to a reference average shear wave velocity in the upper 30 m of about 1050 m/s, whereas the reference velocity for current application is 760 m/s. We show that the levels of nonlinearity in the NEHRP factors are generally stronger than recent simulation-based models as well as empirically-based models.

INTRODUCTION

The site factors incorporated into ground motion prediction equations (GMPEs) differ from those in the building code, which are presented by BSSC (2009) (typically referred to as *NEHRP Provisions*) (e.g., Huang et al., 2010). These differences create practical difficulties because of caps imposed by regulatory agencies on the levels of ground motion from site specific analysis relative to those developed from prescriptive code procedures. In this paper, we document the differences in site factors and postulate reasons for the discrepancies. This work was performed to provide technical support for possible revisions to the NEHRP site factors, which are being considered for the next cycle of the NEHRP Provisions in 2014. Our conclusions on the factors causing the differences are preliminary because the research is ongoing.

BASIS OF SITE FACTORS IN NEHRP PROVISIONS

Review of NEHRP Site Factors

The NEHRP Provisions and Commentary (BSSC, 2009) provide the documentation from which seismic provisions in building codes are periodically updated. One important aspect of the NEHRP Provisions and Commentary is the specification of design-basis ground motions, which are derived for rock site conditions at 0.2 sec and 1.0 sec period from probabilistic seismic hazard analysis (PSHA) and then modified by site factors. The PSHA-based rock site ground motions used in building codes are mapped by the US Geological Survey (http://earthquake.usgs.gov/hazards/). In the 2008 version of the maps, the reference site condition is specified as V_{s30}=760 m/s, where V_{s30} is the average shear wave velocity computed as the ratio of 30 m to shear wave travel time through the upper 30 m of the site.

As shown in Table 1, NEHRP site factors are based on site categories derived from V_{s30} An exception to the V_{s30} criteria is made for soft clays (defined as having undrained shear strength <24 kPa, plasticity index >20, and water content > 0.40), for which category E is assigned if the thickness of soft clay exceeds 3 m regardless of $V_{s30.}$ The site factors are intended to modify ground motion relative to the reference condition used in development of the PSHA maps, which is at the boundary between categories B and C (V_{s30} = 760 m/s).

Table 1. Site categories in NEHRP Provisions (Martin 1994)

NEHRP Category	Description	Mean Shear Wave Velocity to 30 m
A	Hard rock	> 1500 m/s
B	Firm to hard rock	760 - 1500 m/s
C	Dense soil, soft rock	360 - 760 m/s
D	Stiff soil	180 - 360 m/s
E	Soft clays	<180 m/s
F	Special study soils, e.g., liquefiable soils, sensitive clays, organic soils, soft clays > 36 m thick	

Figure 1 presents the short- and long-period NEHRP site factors (BSSC 2009) F_a and F_v, which depend on both site class and intensity of motion on reference rock. The ground motion parameters for the reference site condition used with site factors are: (1) S_s - the pseudo spectral acceleration (PSA) at 0.2 sec (used with F_a) and (2) S_1 – pseudo spectral acceleration (PSA) at 1 sec (used with F_v).

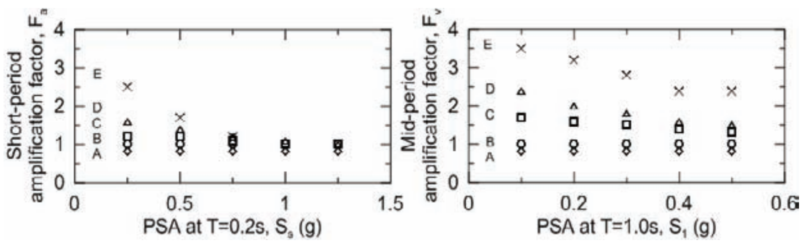

FIG. 1. Site factors F_a and F_v in NEHRP Provisions (BSSC 2009).

Some physical processes underlying the trends in the NEHRP site factors shown in Figure 1 are as follows:

1. Site factors decrease with increasing V_{s30}. This effect is related to the impedance contrast between the shallow soil sediments and the underlying stiffer sediments and rock. Slow velocities in shallow sediments will amplify weak- to moderate-amplitude input motions, especially near the fundamental frequency of the soil column.

2. Site factors decrease with increasing S_s or S_1 and the rate of decrease is fastest for soft soils. As ground motion amplitude increases, the shear strains in the soil increase, causing increased hysteretic damping in the soil. The increased damping dissipates energy and reduces ground motion levels. Because softer sediments develop larger strains than stiffer sediments, this effect is most pronounced for Category E and is less significant for stiffer sites.

3. Site factor F_a (short periods) attenuates more rapidly with increasing S_s or S_1 than does F_v. The damping effect described in (2) acts on each cycle of ground motion. High frequency ground motions will have larger fractions of wavelengths within the soil column than low frequency motions. Because the soil has more opportunity to influence high frequency motions, it produces greater nonlinearity.

Site factors can be developed using ground response simulations and empirical approaches. Existing NEHRP site factors were developed empirically for relatively low input rock ground motions (peak accelerations or S_l near 0.1 g) and have levels of nonlinearity derived from simulations. Additional details on the development of NEHRP factors utilizing empirical and simulation-based methods are given in the following sections.

Empirical Basis for Weak Motion NEHRP Site Factors

The empirical basis for the relatively weak motion NEHRP site factors was developed by Borcherdt (1994), Borcherdt and Glassmoyer (1994), and Joyner et al. (1994), who examined ground motions from the 1989 Loma Prieta earthquake recorded on a variety of site conditions varying from soft clay to rock in the San Francisco Bay Area. Site conditions at recording sites were generally characterized using bore-hole seismic-velocity measurements. A reference site approach was used in which Fourier spectral ratios were calculated for pairs of stations in which one is on soil and one is on reference rock. Figure 2 shows a map of the rock and soil sites considered by Borcherdt and Glassmoyer (1994) (BG). For a particular period T and rock-soil site pair, the site factor determined by this method is:

$$F(T) = \frac{FA_{Vs30}(T)}{FA_{ref}(T)} \tag{1}$$

where FA_{Vs30} (T) is the Fourier amplitude at period T from a recording on a site condition with velocity V_{s30} and $FA_{ref}(T)$ is a recording from a neighboring rock site that is taken as the reference (these sites generally have V_{s30} > 760 m/s). Fourier amplitude spectral ratios were computed at frequency intervals of 1/40.96 sec in the frequency domain. Period-specific spectral ratios calculated from Eqn. (1) were averaged across a short period band (0.1-0.5 sec) and mid period band (0.4-2.0 sec) to estimate F_a and F_v for each rock-soil pair. Resultant empirical estimates of F_a and F_v and corresponding linear regression curves are presented in BG (1994) and Borcherdt (1994b), and the Borcherdt (1994b) results are reproduced in Figure 3. The reference rock motions used by BG (1994) and Borcherdt (1994b) have bedrock peak ground accelerations that range from 0.075 to 0.11 g, with an average of about 0.1 g.

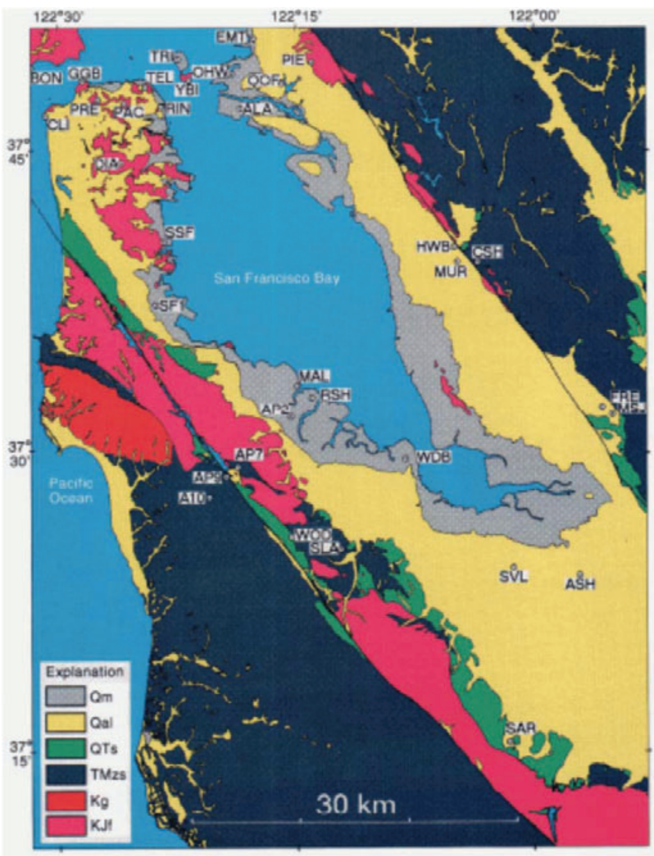

FIG. 2. Map of San Francisco Bay region, showing locations of 34 of 37 free-field stations that recorded 1989 Loma Prieta earthquake and generalized geologic units. KJf corresponds to Franciscan formation bedrock of Cretaceous and Jurassic age that was taken as reference rock. Borcherdt and Glassmoyer (1994).

Figure 3 shows the F_a and F_v factors presented by Borcherdt (1994b) for each station pair plotted as a function of V_{s30} along with linear regression results, 95% confidence intervals for the ordinate to the true population regression line, and the limits for two standard deviations above and below the estimate. The relatively narrow confidence intervals indicate that the scaling of the site terms with V_{s30} is statistically significant. It is apparent from the trends in Figure 3 that the scaling is more pronounced at mid periods than at short periods. This is thought to occur because most soil sites have fundamental vibration periods within the mid-period band, producing stronger site effects in that period range than at shorter periods.

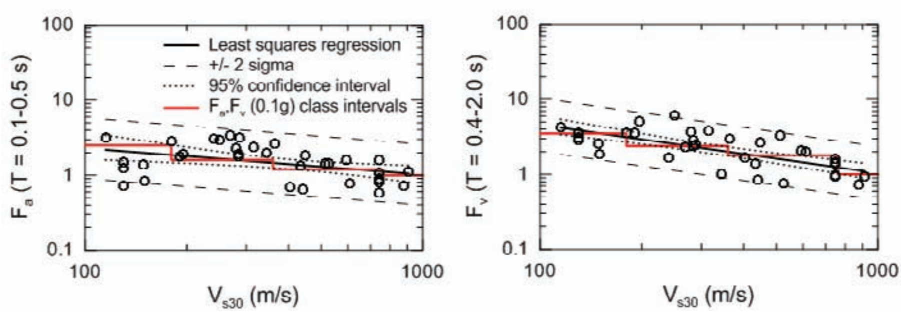

FIG. 3. Site factors F_a and F_v evaluated from reference site approach from recordings of 1989 Loma Prieta earthquake as function of V_{s30} (data from Borcherdt, 1994b). The reference motion amplitude for the data is PGA_r = 0.1g. Red stepped lines correspond to site factors in site class intervals.

The reference sites used by Borcherdt (1994b) correspond to a competent rock site condition, which in the San Francisco Bay Area corresponds specifically to Franciscan formation bedrock of Cretaceous and Jurassic age. The average values of V_{s30} among the reference sites is approximately 795 m/s, but the linear trend line through the data in Figure 3 reaches unity at V_{s30} = 1050 m/s.

In Figure 3 the red stepped lines correspond to F_a and F_v values in use since publication of the 1994 NEHRP Provisions (BSSC, 1995). As shown in Figure 3, the NEHRP F_a and F_v factors are consistent with the trend of the regression lines. The stepped site factors in Figure 3 are slightly different from those presented by Borcherdt (1994b), which match the lines at V_{s30} = 150, 270, 560 and 1050 m/s. The modifications in NEHRP factors relative to Borcherdt (1994b) are in (1) the velocity boundaries, the final values of which were selected in committee and (2) the amplification levels for particular categories (e.g., F_a for E) that were increased by committee consensus. As seen from Figure 3, the NEHRP factors match the regression lines at V_{s30} = 120, 290, 600 and 1050 m/s (for F_a) and at 160, 290, 450 and 1050 m/s (for F_v).

With regard to the V_{s30} = 1050 m/s reference condition provided by Borcherdt (1994b) and adopted for the 1994 NEHRP Provisions, it is useful to recall the national ground motion maps with which the NEHRP site factors were originally applied. As described by Algermissen and Perkins (1976), the GMPE used at that time was a model for rock conditions by Schnabel and Seed (1973), which was used directly for peak acceleration in the western US (non-subduction regions) and with some modification for other conditions (i.e., other regions and longer periods, as described by Algermissen and Perkins, 1976). The rock site conditions represented by the GMPE are poorly defined, although many of the motions used in GMPE development are from soil sites and were deconvolved to rock using wave propagation analysis (Schnabel et al., 1971). The rock conditions used in the deconvolution appear to have been hard (V_s = 2400 m/s), whereas the motions from rock sites were associated with much softer geologic conditions. Considering that the rock GMPE represents the average of these conditions, the 1994 national maps likely applied for firm rock conditions. Therefore, we postulate that general compatibility existed between those maps and the NEHRP site factors, which are referenced to firm rock (V_{s30} = 1050 m/s). By the time of the 1996 national maps (Frankel et al., 1996) as adopted by BSSC (1998), the reference condition used for the PSHA calculations was clearly defined as V_{s30} = 760 m/s (e.g., Frankel et al., 1996, p 5 & 17), but the incompatibility with the reference condition for site factors was either not recognized or not considered to be significant. This condition has remained to the present time.

Simulation-Based Nonlinearity in NEHRP Site Factors

Ground response simulations generally model the stratigraphy as one-dimensional and simulate the nonlinear soil behavior using equivalent-linear or nonlinear methods. Site factors can be evaluated from ground response analysis using the ratio of response spectra at the top of the soil column to that of the outcropping base motion. Some key issues in the utilization of ground response analysis to develop site factors are: (1) shear wave velocity profiles utilized for analysis should be representative of the application region, (2) selected modulus reduction and damping (MRD) curves should be appropriate for the predominant soil types, and (3) input motions should have appropriate amplitude and frequency content for the regional seismicity. Similar considerations apply for nonlinear ground response analysis.

Borcherdt (1994b) and Dobry et al. (2000) describe the process by which equivalent linear and nonlinear ground response simulations were used to supplement the linear site factors in Figure 3. Suites of profiles were analyzed by Seed et al. (1994) and Dobry et al. (1994) for categories C-E using velocity profiles from sites in California and Mexico City. The empirical amplification values shown in Figure 3 were found to be in good agreement with those derived independently by Seed et al. (1994), those computed parametrically by Dobry et al. (1994) at input ground motion levels near 0.1g, and response spectral ratios computed by Joyner et al. (1994). Hence, the modeling results were used to extrapolate the inferred amplification factors to higher input peak acceleration levels of 0.2, 0.3, and 0.4 g. Borcherdt (1994b) and Dobry et al. (2000) describe how the computed site factors were expressed in a linear form in log-log space as shown in Figure 4 and given by the following expressions:

$$F_a = \left(V_{ref} / V_{s30}\right)^{m_a} \tag{2}$$

$$F_v = \left(V_{ref} / V_{s30}\right)^{m_v} \tag{3}$$

where V_{ref} is the reference V_{s30} = 1050 m/s and m_a and m_v are fit coefficients that vary with input motion amplitude to capture trends in the simulations with the results shown in the legend of Figure 4 (Borcherdt, 1994b; Dobry et al., 2000). The black line in Figure 4 applies to PGA_r = 0.1g. For PGA_r > 0.1g the amplification levels decrease in accordance with the simulation results, with the amount of decrease being greatest at low V_{s30}. Note from Figure 4 that these expressions for site factors are referenced to a common V_{s30} = 1050 m/s. For the NEHRP site factors (Figure 1), the input motion ground motion amplitude was re-expressed as S_s and S_1 in lieu of PGA according to S_s=2.5PGA and S_1=PGA.

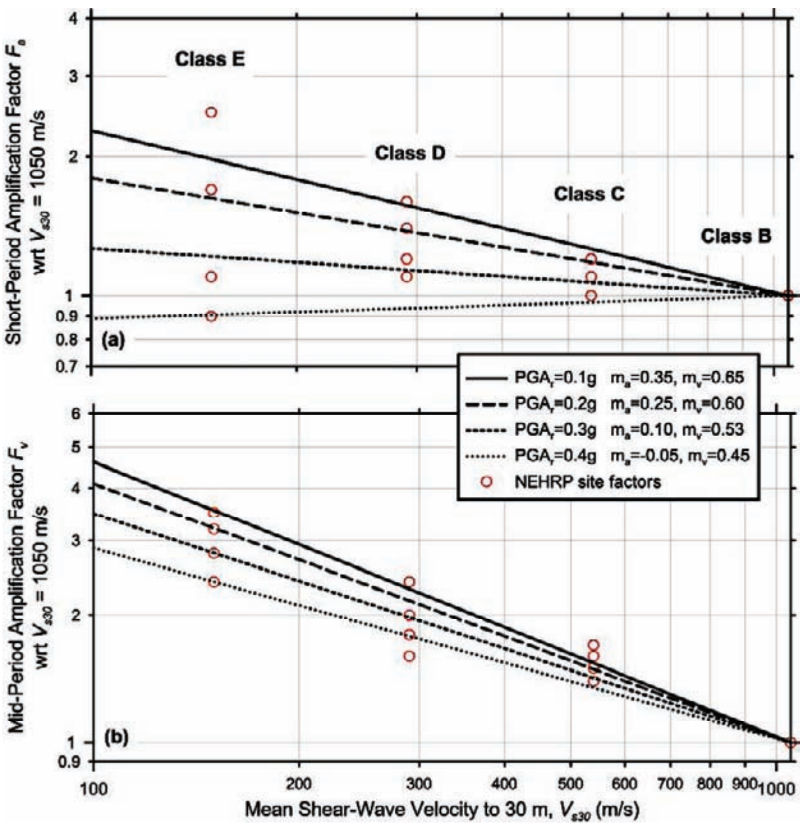

FIG. 4. Relationships between V_{s30} and (a) Short-period F_a and (b) mid-period F_v amplification factors used in the development of NEHRP factors. Figure adapted from Borcherdt (1994b) and Dobry et al. (2000). Parameters m_a and m_v are slopes of the amplification factors with V_{s30} in log-log space as given in Eqn. (2)-(3) with slopes originally from Borcherdt (1993, 1994a,b); PGA$_r$ corresponds to the input ground motion level on rock.

Figure 5 also shows the NEHRP site factors plotted at the V_{s30} values for which category-based site factors were originally developed by Borcherdt (1994b), as explained previously. The NEHPR factors have some discrepancies from the regression lines, especially for F_a in Category E and F_v in Categories C-D. As mentioned previously, those discrepancies arose from committee decisions.

SITE FACTORS IN NGA RELATIONS

The Next Generation Attenuation (NGA) project produced GMPEs for shallow crustal earthquakes in active tectonic regions (Power et al., 2008). GMPEs were developed by five teams consisting of Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), Chiou and Youngs (2008), and Idriss (2008). For ease of use, the abbreviations of AS, BA, CB, CY and Idriss are applied. The models are based on analyses of the PEER-NGA empirical strong ground motion database, which contains 3,551 recordings from 173 earthquakes (Chiou et al., 2008).

The NGA models are semi-empirical equations for peak ground acceleration (PGA), peak ground velocity (PGV) and 5% damped elastic pseudo-acceleration spectra (PSA) for periods up to 10 sec. These ground motion prediction equations (GMPEs) have a typical form of:

$$\ln Y = f_1(M) + f_2(R) + f_3(F) + f_4(HW) + f_5(S) + \varepsilon_T \tag{4}$$

where Y is the median geometric mean ground motion intensity measure (IM); f_i are functions of magnitude (M), source-to-site distance (R), style of faulting (F), hanging-wall effects (HW), and site conditions (S). Parameter ε_T is a random error term with a mean of zero and a total aleatory standard deviation given by

$$\sigma = \sqrt{\phi^2 + \tau^2} \tag{5}$$

where ϕ is the standard deviation of the intra-event residuals and τ is the standard deviation of the inter-event residuals.

The site factors in the NGA GMPEs express the effect of shallow site conditions on various ground motion IMs as a function of V_{s30}, and in the case of the AS, CB, and CY relations, a basin depth term as well. Different NGA developers used different methods to obtain site factors. AS and CB set coefficients describing the linear site response empirically and constrain the nonlinearity in site response based on simulations by Walling et al. (2008). BA and CY fit the coefficients for both the linear and nonlinear components of their site amplification model empirically.

When site amplification factors are developed empirically, the process can be described as a non-reference site approach. In contrast with the reference site approach utilized by BG, the non-reference site approach compares IMs from recordings (IM_{rec}) to median predictions from a GMPE for a reference site condition ($S_a^r(T)_{GMPE}$) as follows:

$$F(T) = \frac{\left(S_a^{rec}(T)\right)}{\left(S_a^{r}(T)_{GMPE}\right)} \tag{6}$$

Note that this approach does not require a reference site recording, hence a much larger set of ground motions can be used to develop site amplification levels, the median of which is taken as the site factor. In natural log units, $\ln F(T)$ can be viewed as the data residual relative to the rock GMPE:

$$\ln F(T) = \ln\left(S_a^{rec}(T)\right) - \ln\left(S_a^{r}(T)_{GMPE}\right) \tag{7}$$

The site factors are generally evaluated during the development of the GMPE in such a way as to minimize residuals.

As noted previously, the AS and CY GMPEs utilize site amplification models whose nonlinear component is set from the results of 1D ground response simulations. The simulation methodology and model building process are described in Walling et al. (2008). The ground response analyses used an equivalent-linear analysis method with random vibration theory as implemented in the program RASCALS (Silva and Lee, 1987). The velocity profiles were taken from a proprietary database maintained by Pacific Engineering and Analysis (PEA) for active tectonic regions. The MRD curves were taken from judgment-driven relations known as the Peninsular Range curves. For each soil profile, amplification factors were computed for input rock PGA values ranging from 0.001 to 1.5 g. For each case, the amplification with respect to $V_{S30}=1100$ m/ s was computed. Example site factors for V_{s30}=270 m/s and 560 m/s, obtained at T=0.2s from this process, are plotted against PGA for V_{s30}=1100 m/s (i.e. PGA1100) in Figure 5. Additional calculations were performed using MRD curves from EPRI (1993), with otherwise identical conditions. Models developed from those results are unpublished but were provided by Walling (personal communication, 2011).

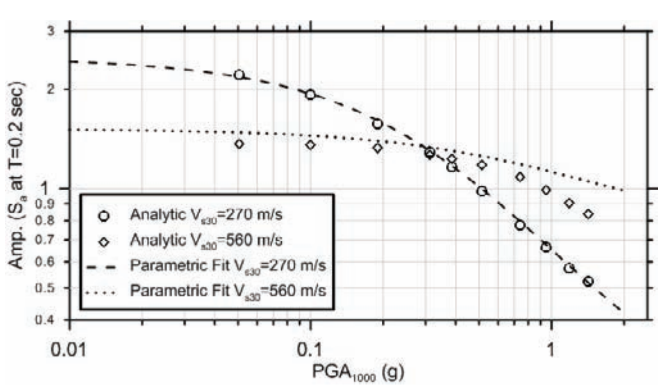

FIG. 5. Examples of the site factors computed by Walling et al. (2008) and parametric fits to the analysis results. Adapted from Walling et al. (2008).

DIFFERENCES BETWEEN NEHRP AND NGA SITE FACTORS

Site Factors Comparisons

In this section we compare the NEHRP site factors with NGA site factors derived from the four NGA GMPEs having site terms. Our objective is to identify discrepancies, with specific attention paid to evaluating differences in median amplification at low levels of rock ground motion as well as possible differences in the nonlinearity of site amplification.

The NGA relations use different functional forms for the site terms. The reference rock ground motion amplitude parameter used to drive nonlinearity in the models is taken as PGA for AS, BA and CB and as spectral acceleration at the period of interest for CY. Site terms $F_x(V_{s30}, A_x)$ are assumed to be log normally distributed and depend on A_x, the ground motion amplitude for a reference site condition having $V_{s30}=x$. Reference motion amplitude A_x is a median PGA for AS, BA, CB and an event-term adjusted median S_a at the period of interest for CY. The event term (η_i) is approximately the median residual for well recorded events, and is formally evaluated from random effects regression procedures (Abrahamson and Youngs, 1992). To summarize, input parameters for the site amplification models are:

- NEHRP: V_{s30}, S_s, S_1
- AS: V_{s30}, Median PGA_{1100} (PGA for V_{s30}=1100 m/s)
- BA: V_{s30}, Median PGA_{760} (PGA for V_{s30}=760 m/s)
- CB: V_{s30}, Median PGA_{1100} (PGA for V_{s30}=1100 m/s)
- CY: V_{s30}, Median + η_i $(S_a)_{1130}$ (S_a for V_{s30}=1130 m/s)

To facilitate comparisons between the NGA and NEHRP site factors, we compute site terms relative to the V_{s30}=760 m/s reference condition used in the national PSHA maps published by USGS. This condition is selected because the NEHRP factors are used to modify ground motions for site conditions that differ from the V_{s30}=760 m/s reference. NGA site factors are calculated relative to this reference condition as:

$$\ln\left(F_{760}(V_{s30},\ A_x)\right) = \ln\left(F_x(V_{s30},\ A_x)\right) - \ln\left(F_x(760,\ A_x)\right)$$

or (8)

$$F_{760}(V_{s30},\ A_x) = \frac{F_x(V_{s30},\ A_x)}{F_x(760,\ A_x)}$$

We define the reference site motion amplitude as A_x = median PGA for V_{s30} = 760 m/s, which is denoted PGA_r in the following text. Site factors are evaluated for PGA_r = 0.01-0.9g. The CY site term uses S_a at the period of interest instead of using the median PGA. For this model, reference motion amplitude is estimated from PGA_r as:

$$S_a\left(T = 0.2\sec, V_{ref} = 760 m/s\right) = 2.2 PGA_r \qquad (9a)$$

$$S_a\left(T = 1.0\,\text{sec}, V_{ref} = 760m/s\right) = 0.7PGA_r \tag{9b}$$

The factors of 2.2 and 0.7 in Eqn. 9a and 9b are based on differences in the median spectral ordinates (e.g., 0.2 sec S_a on rock vs PGA for Eqn. 9a) from the NGA GMPEs for rock site conditions and various ranges of M_w (5-8) and distance (0-50 km). Huang et al. (2010) use a procedure similar to that described above – instead of calculating the site term directly, they apply the NGA GMPEs for a range of magnitudes, distances, and other parameters to compute median S_a for selected V_{s30} values, which are normalized by medians for V_{s30} = 760 m/s. They take the ratio of median S_a at V_{s30} to median S_a at 760 m/s as a period-dependent site factor. Huang et al. (2010) average these values across three GMPEs (i.e. BA, CB and CY) and across period ranges to develop recommendations for F_a and F_v site factors.

We use the NGA site models at representative V_{s30} values for each NEHRP category. The representative velocities are evaluated from medians within the various categories B-E using the site database being compiled for the NGA-West2 project (http://peer.berkeley.edu/ngawest2/). That database contains 804 CA and international sites with measured V_{s30} values distributed as shown in Figure 6. The median V_{s30} values for each site category are indicated in Figure 6. More detailed histograms within the relatively well populated C and D categories are given in Figure 7. The representative category velocities given in Figure 7 are generally similar to those used by Borcherdt (1994b) to set the empirical site factors (i.e., 153 vs 150 m/s for E; 265 vs 290 m/s for D; 488 vs 540 m/s for C; 911 vs 1050 m/s for B).

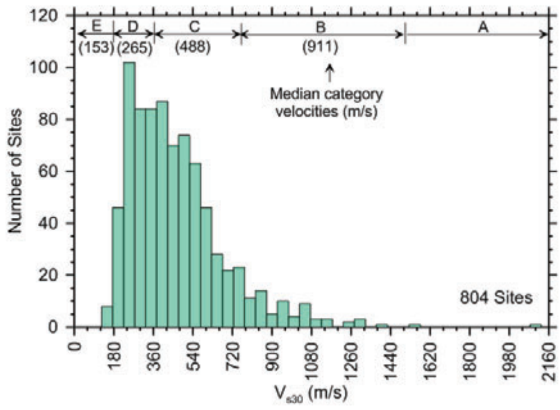

FIG. 6. Histogram of measured V_{s30} values for strong motion sites in NGA-West2 site database used to estimate representative category velocities.

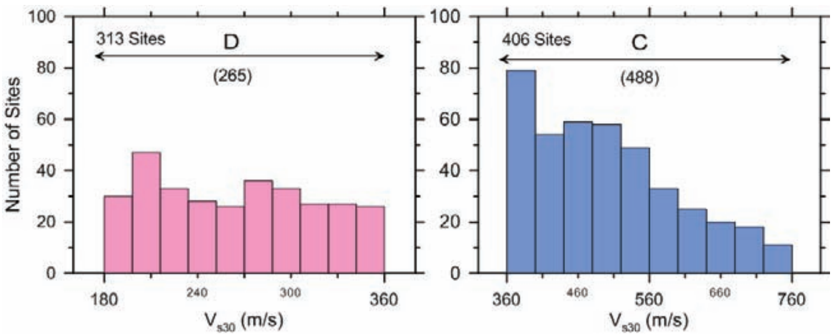

FIG. 7. Histogram of V_{s30} values within Categories C-D from NGA-West2 site database.

Figure 8 compares the discrete NEHRP site factors (black solid symbols) with NGA site amplification terms computed for median spectral accelerations across the period range for F_a (T = 0.1-0.5 sec) and F_v (T = 0.4-2.0 sec) relative to V_{s30} = 760 m/s. Adjustments to the NEHRP factors are also shown in Figure 8 (black open symbols), which are discussed further below. Also shown for comparison are site amplification factors from Huang et al. (2010) for Classes D and E (results for comparable V_{s30} values are not available for other site classes). Note the Huang et al. (2010) factors plotted in Figure 8 are the averaged from their values for specific spectral periods within the respective period ranges for F_a (0.1-0.5 sec) and F_v (0.4-2.0 sec). Because the reference rock amplitudes used by Huang et al. (2010) are 0.2 sec and 1.0 sec S_a, we convert to PGA_r using S_a/PGA_r ratios in Eqn. (9), which are compatible with the magnitude and distance range selected by Huang et al. (2010).

The spread of NGA site factors in Figure 8 reflects epistemic uncertainty, which is relatively large for Class E and modest elsewhere. We judged differences in NGA and NEHRP site factors to be significant when they clearly exceed the epistemic uncertainty for a given site class. In Classes C-D, NEHRP and NGA factors have different slopes for F_v, indicating different levels of nonlinearity. This issue is discussed further in the following section. In Classes C and D, NEHRP and NGA site factors are in reasonable agreement for F_a. In Classes B and E, NEHRP site factors are larger than NGA factors for F_a and F_v. NEHRP C and D factors for F_v are also larger than NGA factors for weak motions (i.e., PGA_r = 0.1g). The trends shown in Figure 8 are not changed appreciably if the V_{s30} values used to compute the NGA site factors are changed to the values selected by Borcherdt (1994b) of 150, 290, 540, and 1050 m/s. The Huang et al. (2010) site factors are generally similar to the NGA factors shown in Figure 8 for Classes D and E (and hence they also have similar discrepancies relative to NEHRP). The modest differences between our site factors and those of Huang et al. (2010) likely result from variability in the S_a/PGA_r ratios used to correct the abscissa, the use of different averaging procedures (i.e., different numbers of averaged spectral periods within F_a and F_v period bands) and other details. Huang et al. (2010) also report similar discrepancies between their site factors and NEHRP factors (e.g, their Figure 2).

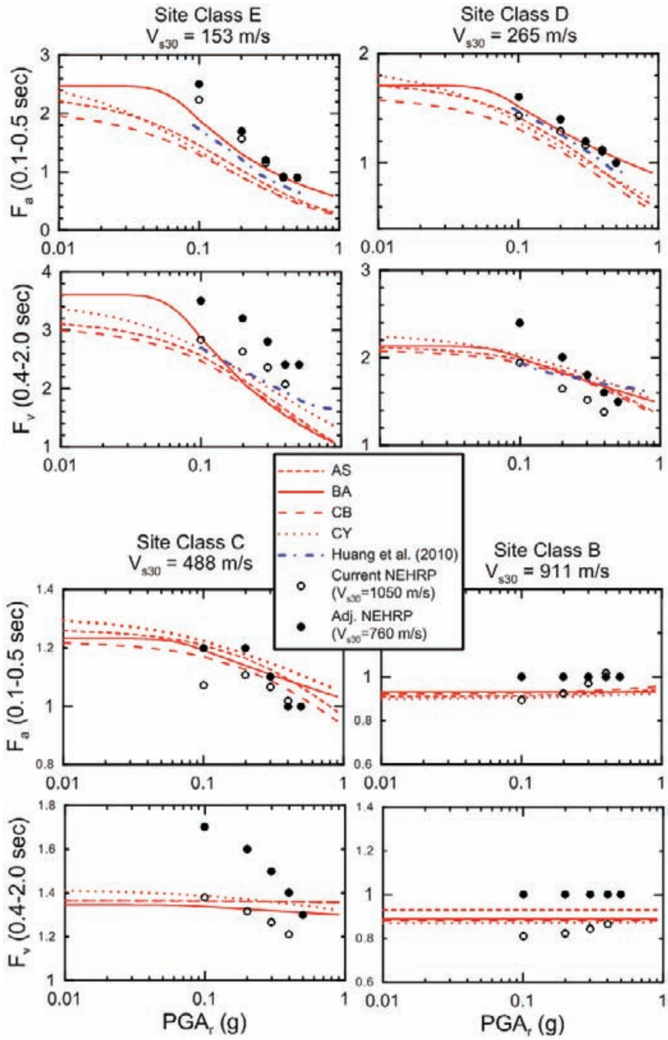

FIG. 8. Comparison of original and adjusted NEHRP site factors to site factors from NGA relationships averaged across corresponding period ranges (0.1-0.5 sec for F_a; 0.4-2.0 sec for F_v) and to those from Huang et al. (2010) (Classes D and E only).

As mentioned previously, adjusted NEHRP factors are also shown in Figure 8. The adjustment is computed to re-normalize the NEHRP factors from a reference velocity of 1050 m/s to 760 m/s as follows:

$$F_a^N = F_a\left(\frac{V_{ref}}{760}\right)^{-m_a} \tag{10a}$$

$$F_v^N = F_v\left(\frac{V_{ref}}{760}\right)^{-m_v} \tag{10a}$$

where superscript *'N'* indicates re-normalization, F_a and F_v are the original, published NEHRP factors, V_{ref} = 1050 m/s per Borcherdt (1994b) and Dobry et al. (2000), and m_a and m_v are taken from Borcherdt (1994b) and Dobry et al. (2000) (shown in Figure 4). No adjustments are made at PGA$_r$ = 0.5g due to a lack of published m_a and m_v values in Figure 4.

Shown with the open black symbols in Figure 8, the re-normalized NEHRP site factors are generally in better agreement with NGA site factors. The re-normalization essentially removes most of the misfit for Class D; significant misfits for other classes remain but are generally reduced. We wish to emphasize that the 'adjusted' NEHRP factors in Figure 8 are not being proposed for adoption in NEHRP, but are merely presented to demonstrate the reduction in site factors discrepancies that is possible through the use of a consistent reference rock condition (i.e. V_{s30}=760 m/s).

The variation of amplification factors with V_{s30} is also investigated to isolate the V_{s30} dependence of the amplification factors from the dependence on PGA$_r$. Figure 9 plots F_a and F_v from NEHRP and NGA (based on median spectral accelerations across the period range for T = 0.1-0.5 sec for F_a; T = 0.4-2.0 sec for F_v) versus V_{s30} for PGA_r = 0.01g, 0.1g, 0.3g, and 0.5g. The original and adjusted NEHRP factors are plotted at the category-averaged V_{s30} values of 153 m/s, 265 m/s, 488 m/s, and 911 m/s, correspond to categories E, D, C and B, respectively.

The results in Figure 9 indicate consistent slopes of the F_a and F_v vs V_{s30} relations for PGA_r = 0.01g and 0.1g. This indicates that the scaling of site factors with V_{s30} in the original BG (1994) and Borcherdt (1994b) relations is robust (i.e., similar V_{s30}-scaling is present in the NGA site terms). The offset between the NEHRP and NGA factors is largely due to the 1050 m/s reference condition in the NEHRP factors. For larger PGA$_r$ values, significant differences in site factors occur for V_{s30} < ~500 m/s, which encompasses conditions at most soil sites. Those differences arise principally from different levels of nonlinearity, which is addressed further in the following section.

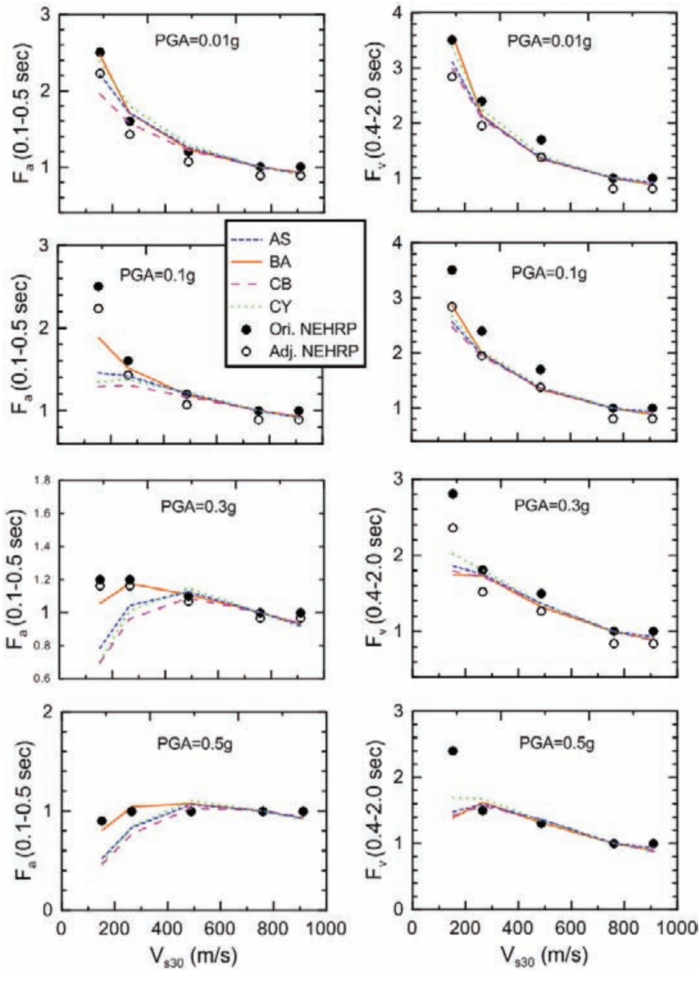

FIG. 9. Variation of site amplification with V_{s30}.

Evaluation of Nonlinearity in Simulation-Based Site Factors

Figure 10 compares the results of analytical studies presented by Dobry et al. (2000) (Fig. 4) with the site factors derived from more comprehensive equivalent-linear analyses by Walling et al. (2008), in which the "Peninsular Range" modulus reduction and damping (MRD) curves (i.e. PEN model) were used. Results are shown for short period band amplification factor, F_a (0.2 sec) and mid period band amplification factor, F_v (1.0 sec). The important conclusions to draw from this comparison relate to the relative slopes of the Walling et al. (2008) and Dobry et al. (2000) relations (not necessarily the vertical position of the curves). For instance, whereas the slopes for V_{s30} = 270 m/s are similar, the slopes for faster velocities are flatter in the more recent work.

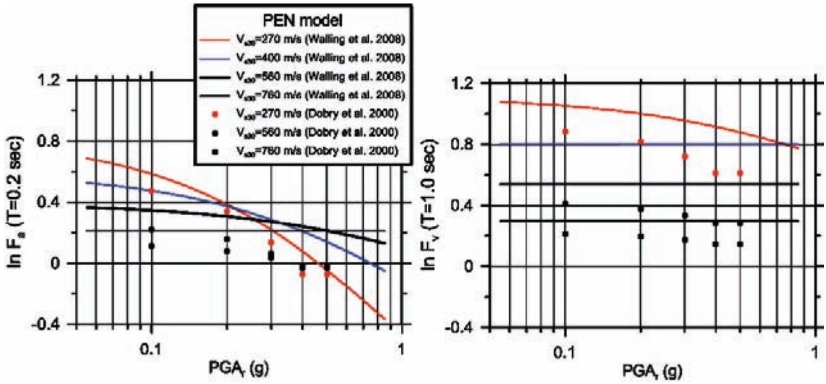

FIG. 10. Comparison of short-period F_a (0.2 sec) and mid-period F_v (1.0 sec) amplification factors between Dobry et al. (2000) and Walling et al. (2008) (PEN model). Results show flatter nonlinear relationship in the Walling et al. model for V_{s30} > 270 m/s.

Figure 11 illustrates the same type of comparison, but the results derived from the PEN model by Walling et al. (2008) are replaced with similar results provided by Walling (personal communication, 2011) that are derived from more nonlinear MRD curves from EPRI (1993). Using this soil model, the F_a slopes are steeper than those from Dobry et al. (2000). For F_v, the slopes are comparable at V_{s30} = 270 m/s; the Walling slopes are flatter for faster velocities.

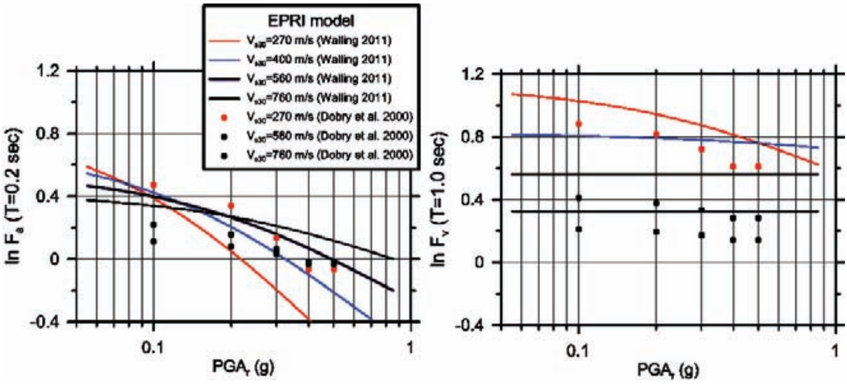

FIG. 11. Comparison of short-period F_a (0.2 sec) and mid-period F_v (1.0 sec) amplification factors between Dobry et al. (2000) and Walling (personal communication, 2011) (EPRI model).

The principal factor responsible for the varying levels of nonlinearity is different MRD models used in the ground response simulations. The Dobry et al. (2000) site factors are based on simulations by Seed et al. (1994) and Dobry et al. (1994), both of which used MRD curves from Vucetic and Dobry (1991) (i.e. VD91) for cohesive soils. For sands, Seed et al. (1994) used MRD curves from Seed et al. (1984) (i.e. S84) while Dobry et al. (1994) used the VD91 MRD curve for PI=0. Figure 12 compares the PEN curves from Walling et al. (2008) with the aforementioned curves that provide the basis for the Dobry et al. (2000) site factors. The PEN curves are more linear than VD91 MRD at PI=0 and the Seed et al. (1984) MR curves, although the VD91 PI=50 MRD curves are similar to PEN. Accordingly, the generally high nonlinearity in the MRD curves used in the studies behind the Dobry et al. (2000) amplification factors explains the relatively nonlinear site amplification.

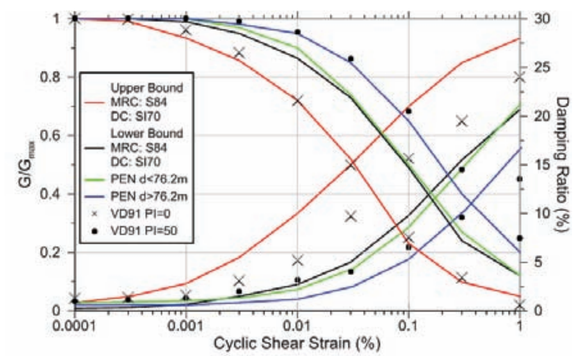

FIG. 12. Comparison of modulus reduction and damping curves from Dobry et al. (1994), Seed et al. (1984) and Walling et al. (2008) (PEN model). S84 = Seed et al. (1984), SI70 = Seed and Idriss (1970) and VS91 = Vucetic and Dobry (1991).

The varying levels of nonlinearity in amplification factors derived from the PEN and EPRI MRD curves reflects epistemic uncertainty, in the sense that we lack knowledge regarding which set of MRD curves are most "correct" for ground response calculations. Given that the simulation results from Walling et al. (2008) and Walling (personal communication, 2011) to some extent bracket the Dobry et al. (2000) curves (at least for F_a), we cannot conclude that the nonlinearity present in the NEHRP provisions is invalid on this basis.

However, nonlinearity from theoretical simulations can be checked against empirical data. Kwok and Stewart (2006) compared recorded ground motion recordings from various site conditions in California to predictions from rock GMPEs modified by theoretically-based site factors very similar to those of Walling et al. (2008). Residuals were calculated in a manner similar to Eqn. (7), but with the rock GMPE median modified with the theoretical site factor and event term η. An example result is shown in Figure 13, which shows no trend in residuals vs PGA_r, indicating that the nonlinearity in the theoretical site factors captures the data trends. This comparison provides support for the more linear recent amplification factors presented by Walling et al. (2008) and used in several of the NGA site terms.

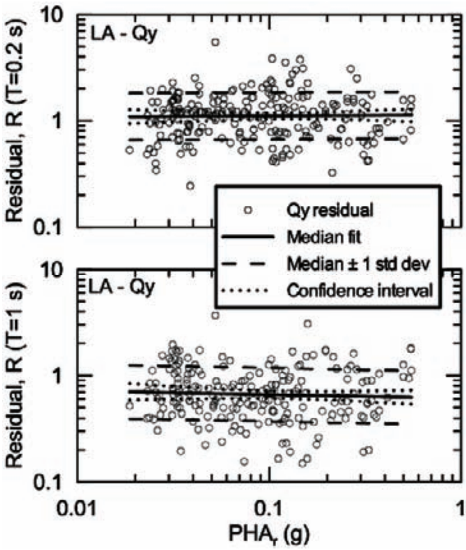

FIG. 13. Trend of residuals with PHA_r. From Kwok and Stewart (2006).

CONCLUSIONS

NGA and NEHRP site factor are consistent in certain respects (e.g., the scaling of linear site amplification with V_{s30}), but have discrepancies in linear site amplification (applicable for rock PGA≤0.1 g) for site Classes B to E and in the levels of nonlinearity for Classes C and D. The amount of these discrepancies ranges from up to 50% for Class E to amounts ranging from about 0 to 20% for Classes B-D. Previous work has identified similar discrepancies in NEHRP and NGA site factors (Huang et al., 2010), but the discrepancies were not clearly associated with differences in linear site amplification levels and nonlinearities. Such associations are useful to understand causes of misfits and to formulate possible future updates to NEHRP factors.

A major cause of the weak motion amplification misfit is that the NEHRP factors are normalized relative to a reference site condition of V_{ref}=1050 m/s, whereas their current application is relative to V_{s30} = 760 m/s. When re-normalized to V_{s30} = 760 m/s, the NEHRP factors are much closer to NGA factors (especially for Class D), although misfits remain for Classes B, C, and E.

We find that the nonlinearity in F_a and F_v from recent simulation-based work (Walling et al, 2008) is smaller than the nonlinearity in the NEHRP factors (Dobry et al., 2000). Those reduced levels of nonlinearity are consistent with trends from empirical ground motion data.

ACKNOWLEDGMENTS

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Tunneling in Difficult Conditions – The Squeezing Case

Fulvio Tonon[1], Ph.D., P.E.

[1]Assistant Professor, Dept. of Civil Engineering, Univ. of Texas at Austin, 1 University Station C1792, Austin, TX 78712-0280, USA, tonon@mail.utexas.edu

ABSTRACT: This paper deals with tunneling in squeezing conditions, and highlights different ways proposed to successfully drive a tunnel in such difficult conditions.

INTRODUCTION

Tunneling in difficult conditions includes squeezing conditions, swelling conditions, tunneling under the ground water table, and rock-burst conditions. The International Society for Rock Mechanics (ISRM) defines squeezing conditions as the "time dependent large deformation which occurs around the tunnel and is essentially associated with creep caused by exceeding a limiting shear stress. Deformation may terminate during construction or continue over a long time period", Barla (1995). Swelling refers to the time-dependent volume increase in the ground, and its induced deformations in tunnels (Einstein 1996); Einstein (1996) pointed out a possible interaction between squeezing and swelling. Rockbursting is the sudden ejection of rock from the walls of a tunnel caused by coalescing microfractures in the rock that end up isolating rock slabs under high stress conditions.

This paper concentrates on squeezing, with special regard to conventional tunneling, i.e. tunneling without the use of a Tunnel Boring Machine (TBM). In the latter case, the interaction between the TBM, the ground, and (for shielded TBMs) the grout is of utmost importance, and would take several papers to discuss. Indeed, the reader is referred to a comprehensive set of papers by Anagnostou (2006), Ramoni and Anagnostou (2006, 2009, 2010a, 2010b, 2011a, 2011b), and Ramoni *et al*. (2011), including the references therein.

TUNNELING IN SQUEEZING CONDITIONS

The prediction and modeling of the behavior of squeezing ground around tunnels has been the subject of many studies, and the reader is referred to the papers by Aydan et al. (1996), Hoek and Marinos (2000), Hoek (2001), Barla et al. (2004), Bonini et al. (2009), Debernardi and Barla (2009), Sterpi and Gioda (2009), Cantieni and Anagnostou (2009a and 2009b, 2011) as well as the papers by Ramoni and Anagnostou cited above for modern approaches and for further references. As indicated in Figure 1, the tunneling approaches may be divided into two large groups: those that proceed by sequential excavation (Figure 1a and b), and those that proceed by full face (Figure 1c). Several generations of NATM (New Austrian Tunneling Method) consultants have us believe that NATM necessarily uses sequential excavation. On the other hand, in many countries, such as the United States, sequential excavation is currently used to indicate soft ground tunneling without a tunnel boring machine (Romero, 2002). Many points of view on and definitions of the NATM have been proposed (Kovari, 1994) and reviewed by Karakuş and Fowell (2004). Brown (1990) and Romero (2002) suggest to differentiate NATM philosophy:

- The strength of the ground around a tunnel is deliberately mobilized to the maximum extent possible.
- Mobilization of ground strength is achieved by allowing controlled deformation of the ground.
- Initial primary support is installed having load-deformation characteristics appropriate to the ground conditions, and installation is timed with respect to ground deformations.
- Instrumentation is installed to monitor deformations in the initial support system, as well as to form the basis of varying the initial support design and the sequence of excavation.

From NATM construction method:

- The tunnel is sequentially excavated and supported, and the excavation sequences can be varied.
- The initial ground support is provided by shotcrete in combination with fiber or welded-wire fabric reinforcement, steel arches (usually lattice girders), and sometimes ground reinforcement (e.g., soil nails, spiling).
- The permanent support is usually (but not always) a cast in place lining.

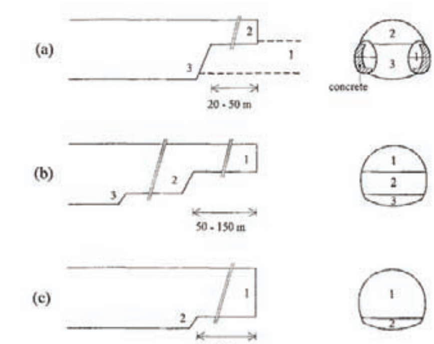


FIG. 1. General approaches to tunneling is squeezing conditions: a) side-drift method, b) top-heading and benching, c) full-face excavation. After Kovari and Staus (1996).

Tonon (2010) showed that the sequential excavation method dates back to the 1800s, and was devised by miners to excavated large cross-sections for the first railway lines at a time when there was no electricity, no compressed air, and power at the heading was mainly man-power, with animals used to muck out. Rabcewicz (the NATM inventor) actually maintained that "*tunnels should be driven full face whenever possible*". Indeed, in his abstract to the first 1964 paper on NATM, Rabcewicz refers to the NATM as: "*a new method consisting of a thin sprayed concrete lining, closed at the earliest possible moment by an invert to a complete ring – called "an auxiliary arch" - the deformation of which is measured as a function of time until equilibrium is obtained*". In the same paper, on page 454, Rabcewicz states that "*One of the most important advantages of steel supports is that they allow tunnels to be driven full face to very large cross sections. The resulting unrestricted working area enables powerful drilling and mucking equipment to be used, increasing the rate of advance and reducing costs. Nowadays, dividing the face into headings which are subsequently widened is used only under unfavourable geological conditions*." On page 457, Rabcewicz continues on this topic: "*There are still some difficulties to be overcome in normal methods of construction, as inverts are still usually built last of all, leaving the roof and sidewalls of the lining to deform at will. In the meantime, experience has taught us that it is by far more advantageous from all points of view, and frequently even imperative, to close a lining to a complete ring at a short distance behind the face as soon as possible. To comply with this requirement, tunnels should be driven full face whenever possible, although this cannot always be done, particularly in bad ground, where it often becomes necessary to resort to heading and benching. In the most difficult cases it may even be necessary to drive a pilot heading before opening it out to full section. An auxiliary arch executed in the upper heading (Belgian roof arch) though fairly effectively preventing roof loosening, represents an intermediate construction stage, which is still subject to lateral deformation. Such instability has to be removed as soon as possible by excavating the bench and closing the lining by an invert*."

In summary:

- NATM has nothing to do with sequential excavation.
- Rabcewicz realized that tunnels should be driven full face.
- Rabcewicz realized that full face allows for the use of large equipment i.e. deployment of large power at the face, which translates into fast tunnel advance and reduced costs.
- Rabcewicz never cared about nor mentioned the ground ahead of the tunnel face or ground support/reinforcement ahead of the tunnel face.
- Rabcewicz wanted but could not find a way to advance full face in difficult stress-strain conditions. His inability to proceed full face in all stress-strain conditions in 1964 was caused by a technological limitation in the normal methods of construction of those days.

The same way as Rabcewicz conceived of the NATM in the 1960's by observing tunnel behavior, in the 1970-80's Lunardi made the following basic observations in the tunnels that he designed and/or built:

1) Convergence (radial displacement of cavity wall) is only the last manifestation of ground deformation. The convergence is always preceded by and is the effect of the deformation of the advance core: preconvergence = radial displacement of ground at the future tunnel perimeter, and extrusion = horizontal displacement of the core.
2) Extrusion can be measured *in situ* and is related one-to-one with the preconvergence
3) In squeezing ground, everything else being the same, the deformation (convergence) of the cavity increases as the speed of tunnel advance decreases.
4) The collapse of the cavity is always preceded by the collapse of the face-core system.
5) In top-heading and benching, the tunnel face starts at the crown of the top heading and ends at the invert of the bench.
6) The arrival of the tunnel face reduces the confinement in the core and increases the major principal stress, giving rise to three basic face-core behaviors: A = stable; B = stable in the short term; C = unstable (FIG. 2).

In summary (FIG. 3):

- The ground behavior around the cavity and the convergence in the cavity at a given tunnel chainage X are controlled by the deformation and the behavior of the ground in the tunnel core when excavating the tunnel at chainage X (what Rabcewicz did not understand and could not do in 1960s).
- In difficult stress-strain conditions, counteracting convergence is not feasible. One needs to control preconvergence and extrusion, i.e. the deformations in the core ahead of the tunnel face (what Rabcewicz did not understand and could not do in 1960s).
- Sequential excavation extends the tunnel face even if the top heading is lined (same as Rabcewicz "*An auxiliary arch executed in the upper heading … represents an intermediate construction stage, which is still subject to lateral deformation*") and increases the volume of ground in the core that, by deforming, controls the behavior of the cavity (what Rabcewicz did not understand).
- If the extent of the face and of the core must be minimized, one has to proceed full face (same as Rabcewicz "*tunnels should be driven full face whenever possible*").

These results led Lunardi to the idea of engineering the core in order to use the core as a stabilization method for the cavity, the same way as rockbolts, shotcrete and steel sets are used to stabilize the cavity. One of the most striking proofs of the central role of the core is given by the re-excavation of tunnels that failed when the core was ignored: FIG. 4 offers two of many examples. The idea of engineering the core was implemented by developing new technologies, such as:

- Sub-horizontal jet-grouting (Campiolo tunnel, 1983).
- Pre-cut with full face excavation (Sibari-Cosenza railway line, 1985, evolution of the pre-decoupage used in the top heading in the Lille Metro, France).
- Fiberglass reinforcement of the core as a construction technology to be used systematically in full-face tunnel advance (1985, high speed railway line between

Florence and Rome), and not only as an ad-hoc means to overcome unpredicted tunneling problems.

The ADECO (Analysis of COntrolled DEformations) (Lunardi, 2008) is the culmination of these observations, experiments, and new technologies. The new technologies introduced with the ADECO can thus only be understood and properly used within the context of the ADECO approach. The ADECO workflow is illustrated in

FIG. 5. In the Diagnosis Phase, the unlined/unreinforced tunnel is modeled in its *in situ* state of stress with the aim of subdividing the entire alignment into the three face/core behavior categories: A, B, and C: these depend on the stress-strain behavior of the core (ground strength, deformability and permeability + *in situ* stress), not only on the ground class. The site investigation must be detailed and informative enough to carry out such quantitative analyses: this clearly defines what the investigation should produce.

In the Therapy phase, the ground is engineered to control the deformations found in the Diagnosis Phase. For tunnel category A, the ground remains in an elastic condition, and one needs to worry about rock block stability (face and cavity) and rock bursts; typically, rock bolts, shotcrete, steel sets and forepoling are used to this effect. In categories B and C yielding occurs in the ground; an arch effect must be artificially created *ahead* of the tunnel face (pre-confinement) when a large yielded zone forms in category B, and in all cases in category C. By looking at the Mohr plane (FIG. 6) two courses of action clearly arise:

- Protecting the core by reducing the size of the Mohr circle: this can be achieved either by providing confinement (increasing σ_3) or by reducing the maximum principal stress (reducing σ_1).
- Reinforcing the core, thereby pushing up and tilting upwards the failure envelope.

The rightmost column in FIG. 3 depicts the actual implementation of these two ideas as pre-confinement actions. The third line of action consists of controlling the convergence at the face by using the stiffness of the lining (preliminary or even final, if needed), which may also longitudinally confine the core. It is only in this context that the different technologies currently available and listed in FIG. 7 take their appropriate role. Notice that, at difference with the NATM, the ADECO embraces tunnels excavated with and without a tunnel boring machine.

Once the confinement and pre-confinement measures have been chosen, the cross-section is composed both in the transverse and longitudinal directions, and then analyzed. In all cases, full face advance is specified in all stress-strain conditions, thus fulfilling Rabcevicz's dream.

For each cross-section, displacement ranges are predicted in terms of convergence and extrusion. Besides plans and specs, construction guidelines are also produced during the design stage. The construction guidelines are used at the construction site to make prompt decisions based on the displacement readings. If the readings are in the middle of the predicted ranges, then the nominal cross-section in the plans and specs is adopted; if reading values fall to the lower end of the predicted displacement ranges, then the minimum quantities specified in the guidelines are adopted for the stabilization measures. Likewise, if reading values are on the upper end of the predicted displacement ranges, then the maximum quantities specified in the guidelines are adopted. Finally, if the readings are outside the predicted displacement ranges, the guidelines specify the new section to be adopted. In this way, ADECO clearly distinguishes between design and construction stages because no improvisation (design-as-you-go) is adopted during construction.

Monitoring plays a major role in the ADECO, but with two main differences with respect to the NATM:

- In categories B and C, not only convergence but also extrusion is measured because the cause of instability is the deformation of the core, and because stability of the core by pre-confinement actions is a necessary condition for the stability of the cavity.
- Monitoring is used to fine tune the design, not to improvise cavity stabilization measures, so that construction time and cost can be reliably predicted.

Tunnels are thus paid for how much they deform, which, unlike rock mass classifications carried out at the face, is an objective measure void of any interpretation. In addition, rock mass classifications are inapplicable to soils and complex rock mass conditions not included in classifications' databases. Experience in over 500 km of tunnels indicates that, when the ADECO has been adopted and tunnels were paid for how much they deformed, claims have decreased to a minimum.

In the Saint Martin La Porte access adit for the Lyon-Turin Base Tunnel, flexible solutions with control of the core deformations were successfully adopted (Barla *et al*., 2008). The support system initially implemented (FIG. 8a) consisted of yielding steel ribs with sliding joints (TH, Toussaint-Heintzmann type), rock anchors and a thin shotcrete layer in a horseshoe profile. Because of the lack of pre-confinement and sequential excavation, these sections of the tunnel underwent very large deformations with convergences up to 2 m and later needed to be re-profiled. The design concept finally chosen (FIG. 8b) to successfully drive the tunnel was based on allowing the support to yield while using full-face excavation with systematic face reinforcement by fiber-glass dowels.

The very well documented example of the Bolu tunnel (Brox and Hagedorn 1999, Dalgiç 2002) should also serve as an additional example in which ignoring the ground ahead of the tunnel face according to the NATM has led to the use of sequential excavation. Re-start of the excavation for the bench uncontrollably increased the displacements that far exceeded the design tolerances. Flexible linings/supports, overexcavation, longitudinal gaps in the shotcrete lining and yielding rock bolting according to the NATM (FIG. 10) forced the contractor to increase the excavation cross-section from 140 m^2 to 220 m^2, and to re-excavate the tunnel six times with dramatic effects on tunnel construction cost and schedule. The concept of monitoring the displacements to delay the installation of the final liner when convergences stop (or reach a small value, e.g., 2 mm/month) led to significant deformations (FIG. 11), unpredictable construction time, and, when an earthquake stroke, to failure of 400 m of already excavated and supported tunnel. As stated in the report to the re-insurers (Brox and Hagedorn 1999), the concept of allowing large (50 cm or more) convergence to occur in an attempt to reduce the "rock load" represents a high-risk approach to tunnel design because it leads to unpredictable stress-strain behavior where the rock mass is disturbed by large displacements and the yield zone around the tunnel may reach the surface (60-80 m in this specific case). Will re-insurers still be willing to issue insurance policies to tunnel projects designed according to this type of high-risk approach?

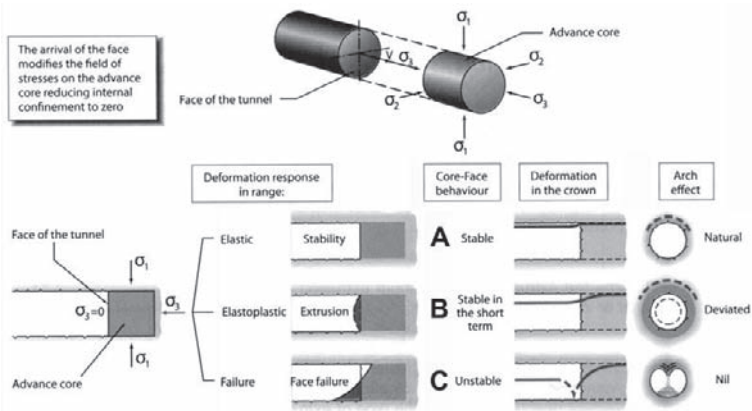


FIG. 2. Tunnel behavior categories based on face-core behavior. After Lunardi (2008).

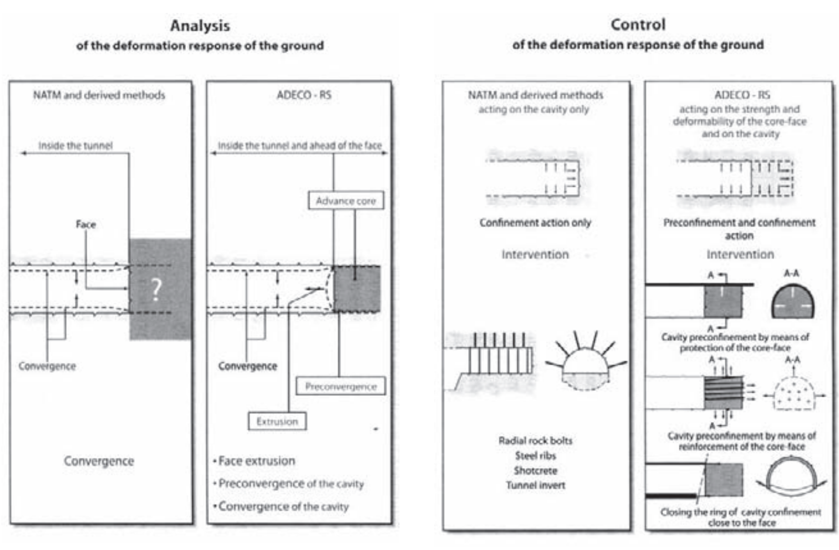


FIG. 3. NATM vs. ADECO. After Lunardi (2008).

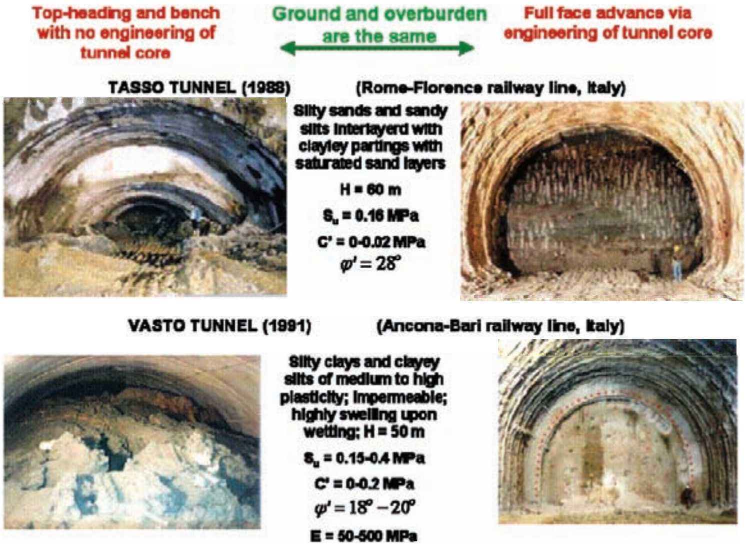


FIG. 4. Tunnels failed when the core was not used as a stabilization method (left-hand sides of FIG. 3 a and b); and re-excavated by using the core as a stabilization measure (right-hand sides of FIG. 3 a and b).

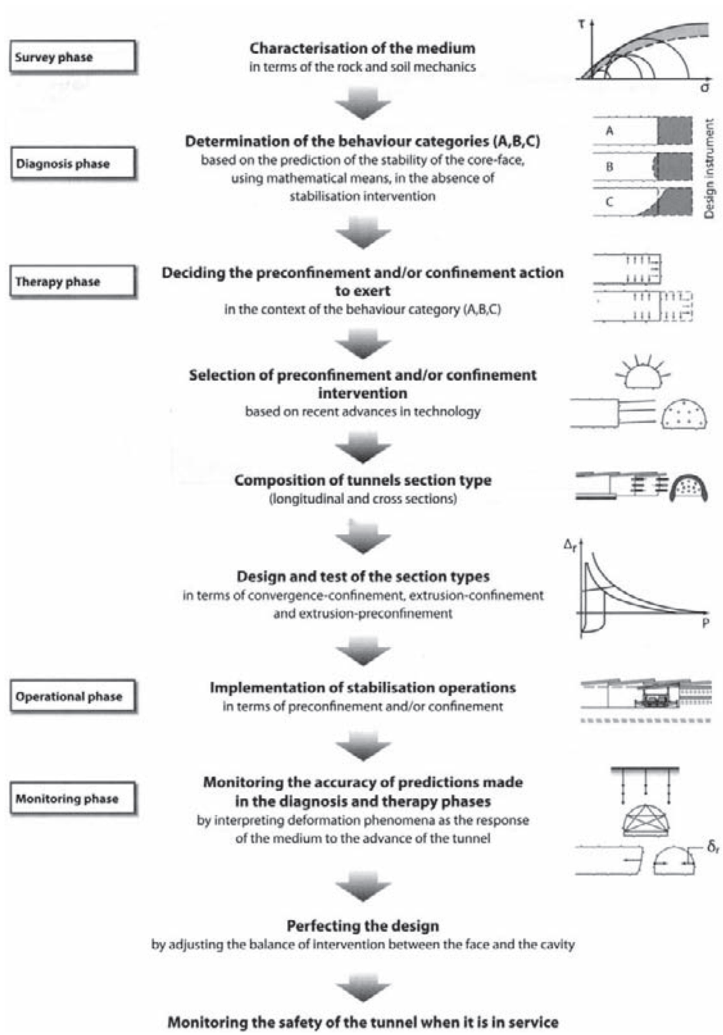


FIG. 5. ADECO workflow. After Lunardi (2008).

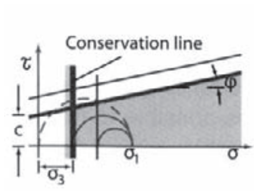


FIG. 6. Mohr-plane explanation of approaches to stabilize/stiffen the core. After Lunardi (2008).

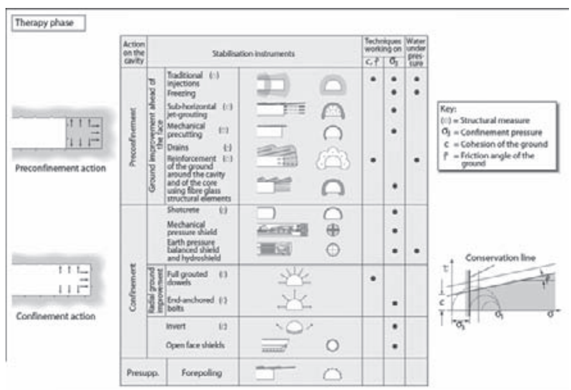


FIG. 7. Subdivision of stabilization tools based on their action as pre-confinement or confinement. After Lunardi (2008).

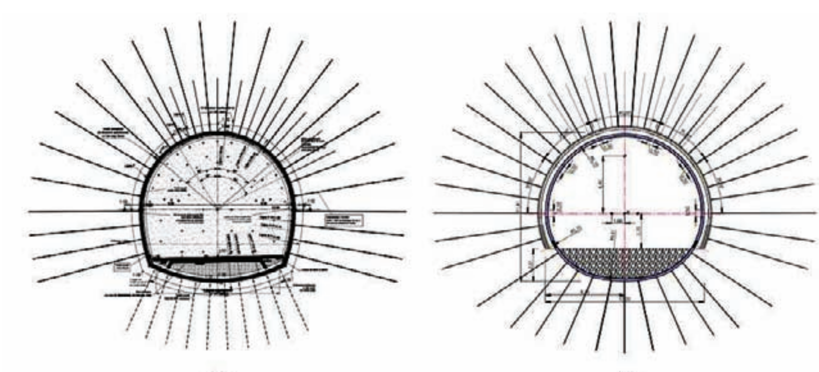


FIG. 8. Tunnel cross section showing the excavation-support systems adopted in the Saint Martin La Porte access adit between chainage 1267 and 1324 m (P7.3, a) and chainage 1325 and 1700 m (DSM, b). After Barla *et al*. (2008).

FIG. 9. Re-profiling of a highly deformed cross section is taking place in the Saint Martin access adit (Lyon-Turin Base Tunnel). After Barla *et al*. (2008).

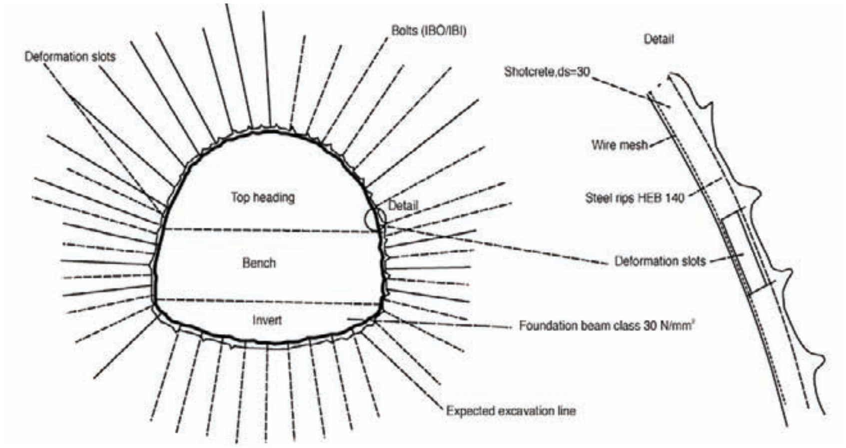


FIG. 10. Minimum support pattern for flexible approach in the Bolu tunnel original design by Geoconsult (1996). After Dalgiç (2002).

FIG. 11. Bolu tunnel. Heave in the Elmalık right tube at km 54 + 135 constructed per the original design by Geoconsult (1996). After Dalgiç (2002).

OUTSTANDING ISSUES IN THE UNDERSTANDING OF PRECONFINEMENT MEASURES

Preconfinement Limits Convergence

It is often maintained that preconfinement measures (FIG. 7) do not change the convergence curve of a tunnel (Pelizza and Peila 1993, Peila 1994, Peila *et al*. 1996, Oreste *et al*. 2004). This is equivalent to saying that the convergence measured at a large distance behind the tunnel face is not affected by the presence of preconfinement measures. As a consequence, when preconfinement is used, higher loads are predicted on primary lining/support and final lining than in the absence of preconfinement. It is thus believed that preconfinement leads to more expensive tunneling solutions.

However, Oreste's and Peila's (2000) 3D FEM elasto-plasticindicate that fiberglass elements inserted in the core reduce not only preconvergence, but also convergence in the cavity. The same conclusion is achieved from the analysis of FIG. 12b, where a lined tunnel is modeled with different densities of fiberglass reinforcement in the core: preconfinement by fiberglass reinforcement of the core effectively controls convergence and therefore core reinforcement does change the convergence curve.

Likewise, field evidence gathered in over 500 km of built tunnels indicates that substantial savings have been achieved in the final lining when pre-confinement was adopted (Lunardi 2008). This contradictory evidence is currently being investigated by the author and his graduate students within the International Tunneling Consortium (ITC) that he established at the University of Texas at Austin (www.caee.utexas.edu/prof/tonon/ITC.htm). The objective of the International Tunneling Consortium (ITC) is to establish a collaboration between academia and the industry to foster research and education in tunneling by listening to the industry needs. The mission of the ITC is twofold: 1) To carry out research on tunneling and underground construction as proposed by the members; 2) to educate the next generation of tunnel engineers.

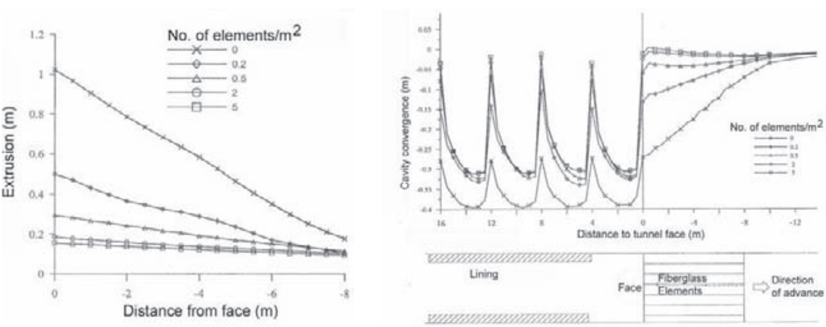
Comparison of FIG. 12a and FIG. 12b confirms that there is a 1-1 relationship between preconvergence and extrusion: as core reinforcement density increases, extrusion and preconvergence decrease together with maximum effect up to 1 element/m^2, beyond which small extrusion and preconvergence reductions are achieved for large increases in reinforcement density. Finally, FIG. 12b shows that convergence develops within only 1 m of the tunnel face, and this confirms the need to carefully design the preconfinement-confinement transitions in order not to waste the reduction in the preconvergence obtained with preconfinement ahead of the tunnel face as highlighted at the bottom of the rightmost column in FIG. 3b.

Ground Time-dependent Behavior

Until now, all analyses of tunnels with preconfinement have used elasto-plastic models (e.g., Pelizza and Peila 1993, Peila 1994, Peila et al. 1996, Wong et al. 2000, Oreste *et al*. 2004, Marcher and Jiřičný 2005, Serafeimidis *et al*. 2007, Serafeimidis and Anagnostou, 2007). An exception is the study by Bonini *et al*. (2009): the authors, after a review of characterization and modeling of time-dependent behavior in rock, described the mechanical behavior of the Italian scaly clays, a structurally complex Clay Shale (CS) formation of the Apennines (Italy). Then, they identified the key factors involved in the selection of the constitutive model for CS and selected and discussed two constitutive models. Finally, they analyzed the Raticosa tunnel and compared the results of the models with the monitoring data in terms of radial convergence of the tunnel and extrusion of the tunnel face.

Although the study by Bonini *et al*. (2009) elucidates the applicability of the constitutive models proposed, the interaction between time-dependent ground behavior resulting in squeezing and/or swelling conditions and preconfinement measures has never been investigated in detail and currently the key geomechanical parameters and engineering preconfinement measures governing this interaction are not understood. Under these conditions, the author thinks that the effect of preconfinement is even more beneficial. Consider a classic rate-dependent viscoplastic model (Perzyna 1971, Simo and Hughes 1998), in which the strain rate is higher the farther the stress point is from the yield surface. For example, FIG. 13 shows that, under sustained loading, the strain rate increases by nearly one order of magnitude even when the stress level (SL) increases from 50 to 86% of failure load. Because preconfinement allows stress points in the core and around the future cavity to remain closer to the yield surface, preconfinement should reduce strain rates in the ground and thus convergence and loads on primary support/reinforcement and final lining. In addition, full face advance allows for an immediate closure of the ring, and therefore even behind the face can stress points be kept as close as possible to the yield surface. Finally, fast rates of advance maintained constant for the entire tunnel excavation minimize displacements. A thorough investigation needed to confirm/disprove these arguments is currently being undertaken by the author and his graduate students within the International Tunneling Consortium (ITC) at the University of Texas at Austin. In order to help guide the preconfinement design in these difficult stress-strain conditions, the ITC research will also develop quantitative understanding of:

- The interaction between time-dependent ground behavior resulting in squeezing and/or swelling conditions and preconfinement measures, and;
- The key geomechanical parameters and effect of engineering preconfinement measures in time-dependent ground



(a) (b)

FIG. 12. Effect of number of fiberglass elements in core on: (a) extrusion; (b) Cavity convergence and preconvergence. Modified after Boldini *et al.* (2000).

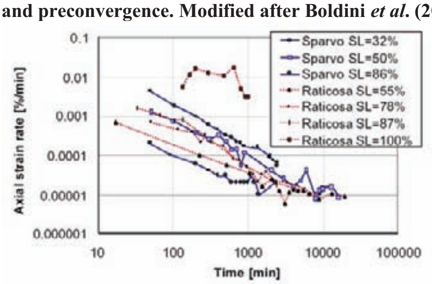


FIG. 13. Italian scaly clays: (a) axial strain rate versus time for creep tests. After Barla *et al.* (2004).

CONCLUSIONS

A wide variety of approaches exists to tunneling in squeezing conditions. These approaches were developed at different times, and it is important to understand the historical and technological conditions under which they were proposed and applied.

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Ingenuity in Geotechnical Design using Geosynthetics

Jorge G. Zornberg[1], Ph.D., P.E., M. ASCE

[1]Professor, The University of Texas at Austin, Austin, TX 78705; zornberg@mail.utexas.edu
President, International Geosynthetics Society (IGS)

ABSTRACT: Even though geosynthetics are now a well-established discipline within geotechnical engineering, ingenuity continues to be significant in projects involving their use. This is probably because of the ability to tailor the mechanical and hydraulic properties of geosynthetics in a controlled manner to address design needs in all areas of geotechnical engineering. This paper focuses on specific advances involving the use of geosynthetics in a wide range of geotechnical projects. Specifically, this paper addresses the creative use of geosynthetics in the design of earth dams, resistive barriers, unsaturated barriers, veneer slopes, coastal protection systems, foundations, bridge abutments, retaining walls, embankments, and pavements.

INTRODUCTION

Geosynthetics play an important role in geotechnical applications because of their versatility, cost-effectiveness, ease of installation, and good characterization of their mechanical and hydraulic properties. Probably because of these many attributes, the use of geosynthetics has often promoted ingenuity in multiple areas of geotechnical engineering. This paper discusses 10 (ten) cases of recent applications (or recent evaluations of pioneering applications) of geosynthetics in geotechnical projects. For each type of geotechnical project, the following aspects are discussed: (i) some difficulties in their design, (ii) a creative approach to address the difficulties using geosynthetics, and (iii) a recent project illustrating the creative use of geosynthetics.

CASE 1: INGENUITY IN EARTH DAM DESIGN

Some Difficulties in the Design of Earth Dams

Filters are both expensive and critical components of large earth dams. The objective of drains and their associated filters is to lower the phreatic surface within the dam to prevent water from emerging from the downstream slope, where flow could trigger erosion that can endanger the integrity of the structure.

The configuration of the filter zones depends on the type of embankment. In a homogenous dam, the filter is generally placed as a blanket of sand and fine gravel on the downstream foundation area, extending from the cutoff/core trench boundary to the edge of the downstream toe. Instead, in a zoned dam the filter is placed between the core and the downstream shell zone. A longitudinal chimney drain collects the intercepted seepage flow and, via one or more transverse drains, conveys the water to the toe drains outside the embankment. Satisfying the filter requirements may be particularly difficult in projects where the appropriate aggregate sizes cannot be obtained in sufficient quantities.

A Creative Approach using Geosynthetics: Geotextile Filters

Geotextiles can be used as filters in critical projects such as earth dams. They constitute a particularly attractive solution in projects where granular material is not readily available. While there has been significant resistance among dam designers towards the use of new filter materials such as geotextiles, the design base and experience in their use continues to grow. For example, a recent re-evaluation of filter criteria was conducted and confirmed the suitability of using geotextiles as filters in large earth dams (Giroud 2010).

The recent re-evaluation led to four criteria for geotextile filters: permeability criterion, retention criterion, porosity criterion, and thickness criterion. Filtration is governed by filter openings. The characteristics of filter openings are the size, shape, density (number per unit area) and distribution. The four criteria address three of these four characteristics: the size, density and distribution. The shape of filter openings is not addressed in the four criteria, but is likely to be a minor consideration (Giroud 2010). On the other hand, the shape of openings may be a relevant issue in the case of some woven geotextiles and some other types of man-made filters. Ultimately, the four proposed criteria for geotextile filters form a coherent set that allows safe design of geotextile filters.

The Recent Re-evaluation of a Pioneering Project: Valcros Dam, France

The pioneering project described herein, and reevaluated in light of a recently re-assessment of filter design criteria, is Valcros Dam. This is the first earth dam designed with geotextile filters. It was constructed in France in 1970 using a geotextile filter under the rip-rap protecting the upstream slope of the dam. In addition, a geotextile filter was used in the downstream drain of the dam.

Valcros Dam is a 17 m high homogeneous dam constructed with a silty sand having 30% by mass of particles smaller than 0.075 mm. Adequate sand filter could not be obtained for the downstream drain, leading to the use of a nonwoven geotextile as the filter. The construction of the downstream drain of the dam with a geotextile filter is shown in Fig. 1. The geotextile used in the downstream drain was a needle-punched nonwoven geotextile made of continuous polyester filaments, with a mass per unit area of 300 g/m^2. The performance of the drain has been satisfactory since its construction. This can be concluded from: (i) a constant trickle of clean water, (ii) a flow rate at the drain outlet that has been consistent with the hydraulic conductivity of the embankment soil, and (iii) no seepage of water ever observed through the downstream slope (Giroud 2010).

The good condition of the geotextile filter was confirmed using samples of geotextile removed from the actual filter after 6 and 22 years of completion of

Fig. 1. Construction of the downstream drain of Valcros Dam (Giroud 1992)

construction. In fact, the clogging was negligible (only 0.2% of the pore volume of the geotextile). The good performance of the geotextile filter can be explained by a recent reassessment provided by Giroud (2010). It was noted that the Valcros Dam filter was not designed using criteria derived directly from the classical Terzaghi's filter criteria. Instead, the geotextile filter was selected on the basis of limited experimental data available at that time (1970) involving the use of this geotextile under an experimental embankment constructed on saturated soft soil. The recent reevaluation of the use of a geotextile filter at Valcros Dam indicates that the geotextile indeed meets the current criteria for permeability, porosity, thickness and retention.

CASE 2: INGENUITY IN THE DESIGN OF RESISTIVE BARRIERS

Some Difficulties in the Design of Resistive Barriers

Conventional cover systems for waste containment involve resistive barriers, which may be particularly expensive when appropriate soils are not locally available. This includes the availability of topsoil, cover soil, drainage materials, and vegetation components. Additional costs include their annual operation and maintenance requirements, loss of revenue due to decreased landfill volume, and detrimental effects of post-construction settlements. In the case of steep landfill slopes, additional concerns involving the use of cover soils involve erosion as well as stability along interfaces with comparatively low interface shear strength.

A Creative Solution by using Geosynthetics: Exposed Geomembranes

Many of the cost- and performance-related concerns associated with the construction of conventional cover systems can be minimized or eliminated by constructing exposed geomembrane covers. These covers are particularly suitable for sites where the design life of the cover is relatively short, where future removal of the cover system may be required, where the landfill sideslopes are steep, where cover soil materials are prohibitively expensive, or where the landfill is expected to be expanded vertically in the future. In addition, the current trend towards the use of "leachate recirculation" or "bioreactor landfills" makes the use of exposed geomembrane covers a good choice during the period of accelerated settlement of the waste.

Key aspects in the design of exposed geomembrane covers are the assessment of the geomembrane stresses induced by wind uplift and the anchorage requirements against wind action. Wind uplift of the geomembrane is a function of the mechanical properties of the geomembrane, the landfill slope geometry, and the design wind velocity. Wind uplift design considerations involve assessment of the maximum wind velocity that an exposed geomembrane can withstand, of the required thickness of a protective layer that would prevent the geomembrane from being uplifted, of the tension and strain induced in the geomembrane by wind loads, and of the geometry of the uplifted geomembrane. Procedures for the analysis of geomembrane wind uplift have been recently developed (Giroud *et al.* 1995, Zornberg and Giroud 1997). A number of exposed geomembrane covers have been designed and constructed using these procedures (Gleason *et al*. 2001).

A Recent Project: The Tessman Road Landfill, TX

The Tessman Road Landfill, located near San Antonio (Texas), was designed and constructed with an exposed geomembrane cover. In order to accommodate the wind uplift, the geomembrane requires high tensile strength properties. The good mechanical properties of geomembrane required by the design made it feasible to mount an array of flexible solar laminate panels. This led to the first installation of a solar energy cover (Roberts *et al.* 2009). The solar energy cover was installed during only a two-month period in early 2009 and is now generating about 120 kW of renewable solar power (Fig. 2).

Fig. 2. Aerial view of the exposed geomembrane with arrays of solar panels at the Tessman Road Landfill (Roberts 2010)

The solar power is tied directly into the existing "landfill gas to energy" system. The Tessman Road Landfill Solar Energy Cover allows generation of renewable energy, creates a revenue stream, and reduces maintenance requirements. The material selected for the Tessman Road Landfill Solar Energy Cover is a green, 60-mil, fiber-reinforced, flexible polypropylene– based thermoplastic polyolefin product. The product offers high strength, flexibility, and a relatively low expansion-contraction coefficient.

The flexible solar panels are less than ¼-inch thick and with a surface of about 23 ft^2. A total of 30 solar panels are arranged in rectangular sub-arrays. A total of 35 sub-arrays fill about 0.6 acres, leaving room to expand the solar generation capacity over time. The 1,050 panels were adhered to the exposed geomembrane over a 5.6-acre project area, with flat areas (benches) separating the tiers. The panels are positioned parallel to final-grade contours with sideslopes angled about 15°. These panels were adhered to the geomembrane with an ethylene propylene copolymer designed for use on both the solar panels and the geomembrane surface. The Tessman Road Landfill Solar Energy Cover project is a good example of sustainable investment, with a high benefit-to-cost ratio, relatively low risk and increased energy efficiency.

CASE 3: INGENUITY IN UNSATURATED SOIL COVER DESIGN

Some Concerns in the Design of Unsaturated Soil Cover Systems

Resistive cover systems involve a liner (*e.g.* a compacted clay layer) constructed with a low saturated hydraulic conductivity soil (typically 10^{-9} m/s or less) to reduce basal percolation. While US regulations require resistive covers, they also allow the use of alternative cover systems if comparative analyses and/or field demonstrations can satisfactorily show their equivalence with prescriptive systems. Unsaturated soil covers are alternative systems that have already been implemented in several high-profile sites. Evapotranspiration, unsaturated hydraulic conductivity and water storage are parameters that significantly influence the performance of this system. The difficulty in adequately quantifying these important parameters has led to concerns regarding the long term performance of unsaturated soil covers. This has resulted in post-construction monitoring and in recommendations towards redundant measures such as additional capillary barrier systems.

A Creative Approach using Geosynthetics: Geotextile Capillary Barriers

The performance of evapotranspirative cover systems has been documented by field experimental studies (Anderson *et al.* 1993, Dwyer 1998), and procedures have been developed for quantitative evaluation of the variables governing their performance (Khire *et al.* 2000, Zornberg *et al.* 2003). However, recent studies have shown that the use of nonwoven geotextiles in a capillary barrier system provide superior performance than traditional coarse-grained soils (Zornberg *et al.* 2010).

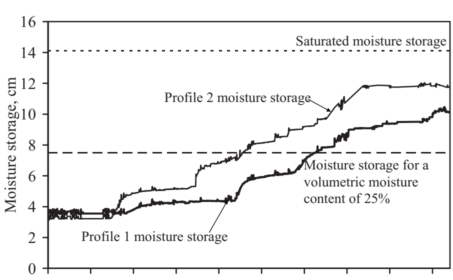

Fig. 3. Water storage in columns with geotextile (Profile 1) and granular (Profile 2) capillary barriers (McCartney *et al.* 2005)

The good performance of geotextiles as capillary barriers is shown in Fig. 3, which shows the water storage within a clay soil column as a function of time for columns involving geotextile and granular capillary barriers. This figure shows that the water storage increases as the infiltration front advances through the soil. Two values of water storage are shown as reference in the figure: the storage corresponding to a water content of 25% (the water content associated with free draining of the imposed impinging flow rate), and the water storage corresponding to saturated conditions. The water storage curves for Profile 1 (geosynthetic capillary barrier) and Profile 2 (granular capillary barrier) indicate that the clay stores water well in excess of the value expected from a freely-draining condition. Also, the results show that the geosynthetic capillary barrier outperformed the granular capillary barrier. In summary, geotextile capillary barriers provide higher water storage than granular soils. In addition, they also offer separation and filtration benefits that are necessary for a good long-term performance of capillary barriers involving granular soils.

A Recent Project: The Rocky Mountain Arsenal, CO

The Rocky Mountain Arsenal (RMA) is a Superfund site located near Denver (Colorado) that corresponds to one of the most highly contaminated hazardous waste sites in the US. One of the remediation components at the site involved the design and construction of alternative covers. The project includes over 400 acres of alternative covers. The climate in Denver is semiarid, with an average annual precipitation of 396 mm and an average pan evaporation of 1,394 mm. The wettest months of the year are also the months with the highest pan evaporation, which is appropriate for an evapotranspirative cover. The Record of Decision (ROD) for this hazardous waste site required a compliance demonstration to show equivalence of the alternative design with a prescriptive cover before construction of the final covers. The design and compliance of the covers at the RMA site are governed by a quantitative percolation criterion involving a threshold of 1.3 mm/year.

The compliance demonstration at the Rocky Mountain Arsenal involved a field demonstration, which was complemented with comparative numerical analyses (Kiel *et al.* 2002). Four evapotranspirative test covers were constructed on a rolling plain at the site in the summer of 1998. The instrumentation program involved monitoring of the basal percolation, precipitation, soil volumetric water content, and overland runoff in the four test covers. Basal percolation was collected in gravity lysimeters, which involved a geocomposite underlain by a geomembrane. Rain and snow were monitored using an all-season rain gauge. Surface water was collected in polyethylene geomembrane swales constructed around the cover perimeters. Water content reflectometer (WCR) probes were used to measure volumetric water content profiles.

While the test plots were well instrumented, the equivalent demonstration process initially focused almost exclusively on the lysimeter measurements. This was because the goal was that the water flux through site-specific soils under local weather conditions remains below the threshold of 1.3 mm/year. According to the lysimeter measurements, all test plots at RMA satisfied the quantitative percolation criterion over the period 1998-2003 of operation. However, subsequent evaluation of the water content records revealed that the presence of lysimeters had affected the flow of water due to the creation of a capillary barrier in the lysimeters. Even though this effect was not initially identified, the cover design was amended to include a capillary barrier.

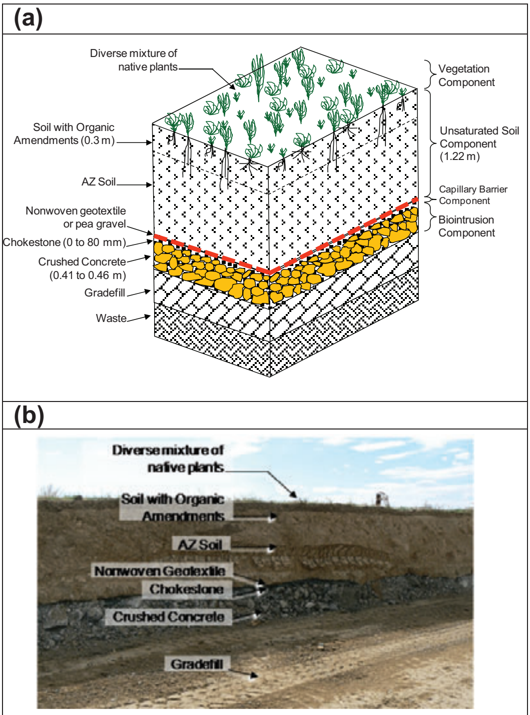

Fig. 4: Cross-section of the RMA Covers: (a) Schematic View; (b) Exposed cut in Shell Cover (Williams *et al.* 2010)

The final cover design for the first group of alternative covers constructed at RMA is shown in Fig. 4. As shown in the figure, the cover includes a geosynthetic capillary barrier (Williams *et al.* 2010). Specifically, the final design of the first cover constructed at the site includes a nonwoven geotextile over a chokestone layer (coarse gravel) to form a capillary break at the bottom interface of the barrier soil. The geotextile also helps minimizing the migration of soil particles into the chokestone layer. The chokestone is underlain by a biotic barrier consisting of crushed concrete from a demolition site. The performance of the final cover is currently being monitored. It may be concluded that geosynthetic capillary barrier may act as an essential component that contributes to the adequate performance of the system.

CASE 4: INGENUITY IN VENEER DESIGN

Difficulties in the Design of Veneer Slopes

The design of veneer slopes (*e.g.* steep cover systems for waste containment facilities) may pose significant challenges to designers. Considering the normal and shear forces acting in a control volume along the veneer slope (or infinite slope), and assuming a Mohr-Coulomb shear strength envelope, the classic expression for the factor of safety FS_u of an unreinforced veneer can be obtained as a function of the soil shear strength parameters. However, if the slope is comparatively steep or the veneer is comparatively thick, the designer is left with little options to enhance stability.

A Creative Solution by using Geosynthetics: Anchored Reinforcements

Geosynthetic reinforcement has been used as an alternative to stabilize veneer slopes. However, cases involving high, steep slopes lead to tensile requirements that are too high and for which reinforcement products do not exist in the market. A recent alternative involved the use of horizontal geosynthetic reinforcements, anchored in sound material underlying the soil veneer (Zornberg *et al.* 2001). In this case, the shear and normal forces acting on the control volume are defined not only as a function of the weight of the control volume, but also as a function of the tensile forces that develop within the reinforcements. In this case, the shear and normal forces needed for equilibrium of a control volume are defined by a formulation that depends on the tensile strength of the reinforcement and provides a convenient expression for stability evaluation of reinforced veneer slopes. Additional aspects that should be accounted for in the design of reinforced veneer slopes include the evaluation of the pullout resistance (*i.e.* embedment length into the underlying mass), assessment of the factor of safety for surfaces that get partially into the underlying mass, evaluation of reinforcement vertical spacing, and analysis of seismic stability.

A Recent Project: North Slopes at the OII Superfund Site

A cover reinforced using horizontally placed geogrids was constructed as part of the final closure of the Operating Industries, Inc. (OII) landfill. In 1986, the 60-hectare south parcel of the OII landfill was placed on the National Priorities List of Superfund sites. Beginning in 1996, the design of a final cover system consisting of an alternative evapotranspirative soil cover was initiated, with construction was carried out subsequently from 1997 to 2000. Stability criteria required a static factor of safety of 1.5, and acceptable seismically-induced permanent deformations less than 150 mm under the maximum credible earthquake.

One of the most challenging design and construction features of the project was related to the North Slope of the landfill. The north slope is located immediately adjacent to the heavily travelled Pomona freeway (over a distance of about 1400 m), rises up to 65 m above the freeway, and consisted of slope segments as steep as 1.5:1 (H:V) and up to 30 m high separated by narrow benches. The toe of the North Slope and the edge of refuse extends up to the freeway. The pre-existing cover on the North Slope consisted of varying thickness of non-engineered fill materials. The cover included several areas of sloughing instability, chronic cracking and high level of gas emissions. The slope was too steep to accommodate a layered final cover system incorporating geosynthetic components (*e.g.* geomembranes, GCLs).

After evaluating various alternatives, an evapotranspirative cover stabilized using geogrid reinforcements was selected as the appropriate cover for the North Slope (Fig. 5). Stability analyses showed that for most available evapotranspirative materials, compacted to practically achievable levels of relative compaction on a 1.5:1 slope (*e.g.* 95% of Standard Proctor), the minimum static and seismic stability criteria were not met. Veneer geogrid reinforcement with horizontally placed geogrids was then selected as the most appropriate and cost-effective method for stabilizing the North Slope cover. The veneer reinforcement consisted of polypropylene uniaxial geogrids, installed at 1.5-m vertical intervals for slopes steeper than 1.8:1, and at 3-m vertical intervals for slopes between 2:1 and 1.8:1.

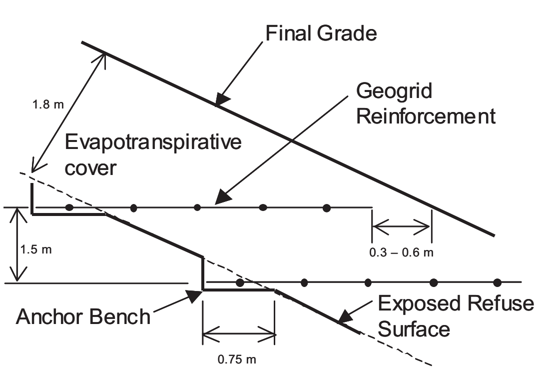

Fig. 5. Detail of the horizontal reinforcement anchored into solid waste (Zornberg *et al.* 2001a)

As shown in Fig. 5, the geogrid panels are embedded a minimum of 0.75 m into the exposed refuse slope face from which the pre-existing cover had been stripped. Construction of the North Slope was accomplished in 12 months. Approximately 500,000 m^3 of soil and 170,000 m^2 of geogrid were placed, with a total area exceeding 9.3 hectares. The covers have shown good performance since its construction, illustrating that geosynthetic reinforcement led to a successful approach where many other stabilization alternatives were not feasible.

CASE 5: INGENUITY IN COASTAL PROTECTION SYSTEM DESIGN

Some Concerns in the Design of Coastal Protection Systems

Coastal protection is often achieved through rock armor, or riprap, which involves large rocks placed at the foot of dunes or cliffs. This approach is generally used in areas prone to erosion to absorb the wave energy and hold beach material. Although effective, this solution is unsightly and may be extremely expensive. Also, riprap may not be effective in storm conditions, and reduces the recreational value of beaches.

A Creative Solution using Geosynthetics: Large Diameter Geotextile Tubes

Coastal protection can be effectively achieved through the use of geotextile tubes (Lawson 2008). While geotextile tubes have been used for hydraulic and marine structures since the 1960s, the use of relatively large-diameter geotextile tubes is comparatively new. They involve the use of strong woven geotextiles as the tube skin (with no impermeable inner liner). The major advantage of this system is that a large encapsulated mass, a tubular structure, could be designed directly to meet many hydraulic and marine stability requirements. Geotextile tubes ranging in diameter from 1.0 m to 6.0 m have been used in hydraulic and marine applications.

Geotextile tubes are laid out and filled hydraulically on site to their required geometry. Hydraulic fill is pumped into the geotextile tube through specially manufactured filling ports located at specific intervals along the top of the tube. During filling, the tube, being permeable, allows the excess water to flow through the geotextile skin while the retained fill attains a compacted, stable mass within the tube. For hydraulic and marine applications, the type of fill typically used is sand or a significant fraction of sand. The reasons for this are that this type of fill can be placed to a good density by hydraulic means, it has good internal shear strength and, once placed, it does not undergo further consolidation that would change the filled shape of the geotextile tube (Lawson 2008).

The geotextile skin performs three functions that are critical to the performance of the filled geotextile tube. First, it should resist (with adequate tensile strength and stiffness) the mechanical stresses applied during filling and throughout the life of the units, and must not continue to deform over time. Second, it must have the required hydraulic properties to retain the sand fill and prevent erosion under a variety of hydraulic conditions. Finally, it must have adequate durability to maintain working conditions over the design life of the units.

A Recent Project: Incheon Grand Bridge Project

Geotextile tubes were recently used for the construction of an artificial island at Incheon Grand Bridge Project, Korea (Lawson 2008, IFAI 2011). The project includes the construction of a freeway connecting the island that holds the new airport to mainland Korea (Fig. 6). This bridge is the longest in Korea and the fifth-longest cable-stayed bridge in the world. An artificial island was planned in order to construct the freeway viaduct and associated toll gate facilities. This artificial island is to be left in place once the freeway viaduct is completed, as the area will later be enveloped by a large land reclamation scheme to build a new high-technology city.

Fig. 6. Geotextile tubes for the construction of an artificial island at Incheon Grand Bridge Project, Korea (IFAI 2011)

The foundation conditions where the artificial island is located consist of very soft marine clays to an approximate depth of 20 m. Also, the tide range in this area is high, with a maximum difference in level of 9.3 m. This results in exposure of the soft clay foundation at low tide and inundation to around 5 m at high tide. To address these difficulties, it was decided to construct a containment dike for the artificial island using geotextile tubes. This approach was selected over the alternative of using sheet-pile walls, considering the low shear strength of the soft foundation and the height to which the artificial island would have to be raised.

The sand fill for the geotextile tubes was brought to the site by barge, mixed with water, and then pumped hydraulically into the geotextile tubes. The base of the wall has two tubes side by side, with a third tube placed on top. A fourth tube was subsequently placed to heighten the final system. The performance of the geotextile tube structure used in this project was studied by Shin *et al.* (2008). The results show that the filled tubes underwent very little deformation once filled, confirming the adequacy of the geotextile tube system.

CASE 6: INGENUITY IN FOUNDATION DESIGN

Some Difficulties in Foundations Design

Foundations on very soft soils are always problematic. However, when the undrained shear strength is below some 15 kN/m^2, even solutions such as stone columns are inadequate. This is because the horizontal support of the soft soil must equal the horizontal pressure in the column.

A Creative Approach using Geosynthetics: Geotextile-Encased Columns

Fig. 7. View of exposed Geotextile Encased Column (Alexiew *et al.* 2011)

High strength geotextiles have been used to construct Geotextile Encased Columns (GEC), which serve as foundation elements in very soft soils such as underconsolidated clays, peats, and sludge (Fig. 7). The columns are formed by using a special geotextile that cases granular material. The geotextile provides radial support while the casing is strained by ring tensile forces (Raithel *et al.* 2005, Alexiew *et al.* 2011). The first projects were successfully completed in Germany in the mid-1990s. Since their inception, over 30 successful projects have been completed in many countries including Germany, Sweden, Holland, Poland and Brazil.
Due to the presence of the geotextile casing, the soft soil can tolerate very low lateral support. This is because of the radial supporting effect of the geotextile casing. The horizontal support depends in turn on the vertical pressure over the soft soil, which can be relatively small. To withstand the high ring tensile stresses, the geotextile casings are manufactured seamlessly. The columns also act as vertical drains, but the main role of GECs is the load transfer to deep bearing layers. The GECs are arranged in a regular grid (Alexiew *et al.* 2011).

The vertical compressive stiffness of the GEC is lower than that of conventional deep foundation systems. Accordingly, the compacted vertical sand or gravel column settles under load due to radial outward deformations. The geosynthetic encasement, and to some extent the surrounding soft soil, provides a confining radial inward resistance, but some radial deformability is allowed. This deformability has been reported to provide better compatibility with the deformation of soft sols than more rigid systems. The use of geosynthetic reinforcements placed horizontally on top of the GECs (*e.g.* at the base of embankments founded using GECs) has been used to reduce differential settlements between the columns and the surrounding soil.

A Recent Project: Extension of Dockyards for the new Airbus, Germany

Geotextile Encased Tubes were used as part of the extension of the airplane dockyards in Hamburg-Finkenwerder for the production of the new Airbus A380. The area extension was conducted by enclosing a polder with a 2.4 km long dike, which was subsequently filled to provide an addition of 140 ha. The main problem facing this project was the construction in very soft soils (undrained shear strength ranging from 0.4 to 10 kPa), with thicknesses ranging from 8 to 14 m. The original design involved the construction of a 2.5 km long sheet pile wall, driven to a depth of 40 m. Ultimately, a dike was constructed over a foundation involving installing approximately 60,000 GECs with a diameter of 80 cm. They were sunk into the bearing layers to a depth ranging between 4 and 14 m below the base of the dike footing. This dike is the new main water protection for the airplane dockyard

This project was successfully implemented between 2001 and 2004. As part of the structural checks on the ground engineering concept, the stability and deformation predictions were verified by on-site measurements during construction. The comprehensive instrumentation included horizontal and vertical inclinometers, settlement indicators and measurement marks, as well as water pressure and pore water pressure transducers. Most of the measurement instrumentation was designed for continued monitoring after completion of the dike.

The dike surface was added to offset long-term settlement when much of the primary settlements were practically complete (after roughly one year). Additional predictions were conducted to estimate secondary settlements. An evaluation conducted in 2004 revealed significantly lower secondary settlements than initially predicted, confirming the soundness of the design involving GECs.

CASE 7: INGENUITY IN BRIDGE ABUTMENT DESIGN

Some Difficulties in the Design of Bridge Abutments

Conventional design of bridge abutments involve the use of a foundation approach to support the bridge (*e.g.* using a deep foundation) and a different type of foundation for the approaching roadway structure (*e.g.* foundations on grade). The use of two different foundation types for different components of the abutment has led to increased construction costs and times. In addition, vehicular traffic may not be smooth due to the development of a “bump at the bridge” caused by differential settlements between bridge foundations and approaching roadway structures.

A Creative Approach using Geosynthetics: GRS Integral Abutments

A comparatively recent approach involves the use of integral Geosynthetic Reinforced Soil (GRS) abutments, which support the bridge load by footings placed directly on a geosynthetic-reinforced wall, eliminating the use of traditional deep foundations altogether (Zornberg *et al.* 2001b, Keller and Devin 2003, Wu *et al.* 2006). Some additional advantages include their flexibility, and consequently added ability to withstand differential settlements and seismic loads as well as their ability to alleviate the bridge "bumps" commonly occurring at the two ends of a bridge supported by piles. In addition, this approach eliminates the need of excavations specialized drilling equipment needed for deep foundations, leading to comparatively rapid construction.

A Recent Project: Founders/Meadows Parkway Bridge, CO

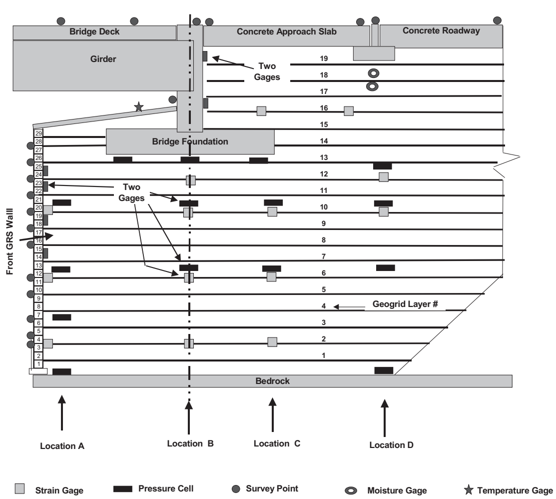

Fig. 8. Cross section of the Founders/Meadows Bridge abutment showing the geosynthetic reinforcements and instrumentation plan (Zornberg *et al.* 2001b)

A GRS abutment for bridge support, the Founders/Meadows Parkway Bridge, was constructed on I-25, approximately 20 miles south of downtown Denver, CO. This was the first major bridge in the US built on footings supported by a geosynthetic-reinforced system, eliminating the use of traditional deep foundations altogether (Fig. 8). Phased construction of the almost 9-m high, horseshoe-shaped abutments, located on each side of the highway, began in July 1998 and was completed twelve months later.

A comprehensive material testing, instrumentation, and monitoring programs were incorporated into the construction operations. Design procedures, material characterization programs, and monitoring results from the instrumentation program are discussed by Abu-Hejleh *et al.* (2002). Each span of the new bridge is 34.5 m long and 34.5 m wide, with 20 side-by-side pre-stressed box girders. The new bridge is 13 m longer and 25 m wider than the previous structure, accommodating six traffic lanes and sidewalks on both sides of the bridge. The bridge is supported by central pier columns along the middle of the structure, which in turn are supported by a spread footing founded on bedrock at the median of U.S. Interstate 25. Three types of uniaxial geogrid reinforcements were used in different sections of the wall. The long-term-design-strength of these reinforcements is 27 kN/m, 11 kN/m, and 6.8 kN/m, respectively.

Three sections of the GRS system were instrumented to provide information on the structure movements, soil stresses, geogrid strains, and moisture content during construction and after opening the structure to traffic. The instrumentation program included monitoring using survey targets, digital road profiler, pressure cells, strain gauges, moisture gauges, and temperature gauges. A view of the instrumentation plan for Phase II is also shown in Fig. 8. The figure shows the presence of the shallow footing resting on the reinforced soil mass.

Overall, the performance of the Founders/Meadows bridge structure, based on the monitored behavior, showed excellent short- and long-term performance. Specifically, the monitored movements were significantly smaller than those expected in design or allowed by performance requirements. Also, there were no signs of development of the "bump at the bridge" problem or any structural damage, and post-construction movements became negligible after an in-service period of 1 year.

CASE 8: INGENUITY IN THE DESIGN OF RETAINING WALLS

Some Concerns in the Design of High Retaining Walls

Flexibility of retaining walls is particularly relevant in the design of high (*e.g.* over 50 m) systems. This is important for the long-term response, to minimize differential settlements, and for adequate seismic response. In addition, the design of high structures using concrete retaining wall systems often requires deep foundations. Finally, and particularly in high walls, the time and cost requirements imposed by concrete retaining walls (*i.e.* formwork, placement of reinforcement bars, curing, removal of formwork) as well as technical limitations may be excessive.

A Creative Solution using Geosynthetics: Optimized Flexible Wall Systems

Geosynthetic-reinforced soil walls involve the use of continuous geosynthetic inclusions such as geogrids or geotextiles. The acceptance of geosynthetics in reinforced soil construction has been triggered by a number of factors, including aesthetics, reliability, simple construction techniques, good seismic performance, and the ability to tolerate large deformations without structural distress. That is, the very nature of geosynthetics in soil reinforcement applications has led to comparatively flexible systems. Yet, recent advances in the design of geosynthetic-reinforced walls have led to optimized systems that are particularly suitable for high walls. These systems include comparatively high tensile strength elements, comparatively low creep response, and comparatively flexible facing units.

A Recent Project: Sikkim Airport, India

An 80 m-high reinforced soil system has been recently constructed for the Sikkim Airport. The structure is a hybrid wall/slope system constructed in a very hilly road meandering along river Teesta, in the Himalayas region of India. This structure possibly constitutes the highest reinforced soil structure in the world built using geosynthetic reinforcements. Fig. 9 shows the front view and cross section of the recently constructed structure. The airport will provide connectivity to Gangtok, the capital of the state of Sikkim, which is nested in the Himalayas and remains often isolated during the rainy season. Site selection for an airport in this mountainous region required significant evaluation, as the airport's runway and apron requires flat land due to operational considerations. The new airport will be able to handle ATR-72 class of aircrafts. Its runway is 1,700 m long and 30 m wide. Its apron will be able to park two ATR-72 aircrafts.

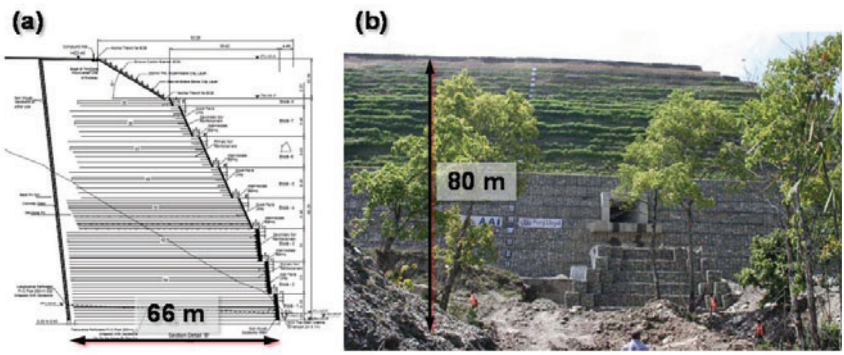

Fig. 9. High (80 m) geosynthetic-reinforced wall constructed for the Sikkim Airport: a) Cross-section, b) Front view (Zannoni 2011)

This innovative earth retention system involves the use of high strength geogrids as primary soil reinforcement with an ultimate tensile strength of 800 kN/m. The reinforcement vertical spacing ranges from 1.8 to 2.4 m. In addition, galvanized and PVC-coated wire mesh panels are used as secondary reinforcement (spaced every 0.6 m). A vegetated slope face is provided in a significant portion of the reinforced soil system by installing tailored units as fascia elements.

Seismic considerations played a significant role in the selection of the wall system. Indeed, the structure experienced a magnitude 6.8 earthquake during construction, with no signs of visible distress after the event. In addition, the selected system required significantly less stringent considerations regarding its foundation. Finally, environmental considerations such as the reduced carbon footprint of this alternative in relation to those involving concrete added to the decision of a geosynthetic-reinforced system. Locally available backfill material was used throughout the project. Sikkim is a green valley with rich flora and fauna. Accordingly, the selected reinforced soil structure, with local stone and green fascia, blends well with the surroundings causing minimum adverse effects on environment.

CASE 9: INGENUITY IN REINFORCED EMBANKMENT DESIGN

Concerns in the Design of Earth Embankments

If fine-grained soils constitute the available backfill material for an engineered embankment, the engineer is limited to the use of unreinforced systems and, consequently, comparatively flat slopes. This is because granular soils have been the preferred backfill material for reinforced soil construction due to their high strength and ability to prevent development of excess pore water pressures. Stringent specifications regarding selection of granular backfill are provided, for example, by the FHWA guidelines (Berg *et al.* 2009).

A Creative Solution using Geosynthetics: Reinforcements with In-Plane Drainage

A promising approach for the design of reinforced fine-grained soils is to promote lateral drainage in combination with soil reinforcement. This may be achieved by using geocomposites with in-plane drainage capabilities or thin layers of granular soil in combination with the geosynthetic reinforcements. This design approach may even lead to the elimination of external drainage requirements. The potential use of permeable inclusions to reinforce poorly draining soils has been documented (Tatsuoka *et al.* 1990, Zornberg and Mitchell 1994, Mitchell and Zornberg 1995).

The potential benefits of using marginal soils to construct steepened slopes are significant and include: (i) reduced cost of structures that would otherwise be constructed with expensive select backfill; (ii) improved performance of compacted clay structures that would otherwise be constructed without reinforcements; and (iii) use of materials, such as nearly saturated cohesive soils and mine wastes, that would otherwise require disposal. However, the significant benefits of using poorly draining soils as backfill material can be realized only if a proper design accounts for the adverse conditions. The adverse conditions and preliminary guidance are identified by Christopher *et al.* (1998) for the design of steep slopes using fine-grained soils

The Recent Re-evaluation of a Pioneering Project: Geotextile-Reinforced slope in Idaho National Forest

A geotextile-reinforced slope designed as part of the widening of US Highway 93 between Salmon, Idaho, and the Montana state line (Barrows and Lofgren 1993). The reinforced structure is a 1H:1V slope located in Idaho's Salmon National Forest along Highway 93. Esthetics was an important consideration in the selection of the retaining structures along scenic Highway 93 (Parfit 1992). The 172 m-long and up to 15.3 m-high geotextile-reinforced slope is vegetated, causing a minimum environmental impact to the Salmon National Forest.

The slope was designed using geotextile reinforcements that not only were required to have adequate tensile strength but were also expected to provide appropriate in-plane drainage capacity to allow dissipation of pore water pressures that could be generated in the fill. In this way, an additional drainage system was not necessary even though indigenous soils were used as backfill and groundwater seeping was expected from the excavation behind the fill. An extensive instrumentation program was implemented to evaluate its performance.

On-site soil coming from excavation of the road alignment was to be used as backfill material. Subsurface drilling revealed that the majority of subsurface material on this project is decomposed granite. Although the project specifications required the use of material with no more than 15% passing sieve no. 200, internal drainage was a design concern. This was because of the potential seepage from the fractured rock mass into the reinforced fill, especially during spring thaw, coupled with the potential crushing of decomposed granite particles that may reduce the hydraulic conductivity of the fill. Widening of the original road was achieved by turning the existing 2H:1V nonreinforced slope into a 1H:1V reinforced slope.

Fig. 10. Geosynthetic-reinforced slope in the Idaho National Forest, illustrating the use of reinforcement with in-plane drainage capabilities (Zornberg *et al*. 1997)

As shown in Fig. 10, the final design adopted two geosynthetic reinforced zones with a constant reinforcement spacing of 0.3 m (1 ft). At the highest cross-section of the structure, the reinforced slope has a total of 50 geotextile layers. A nonwoven geotextile was selected in the upper half of the slope, while a high strength composite geotextile was used in the lower half. The nonwoven geotextile, with an ultimate tensile strength over 20 kN/m, is a polypropylene continuous filament needle punched nonwoven. The composite geotextile, with an ultimate tensile strength over 100 kN/m, is a polypropylene continuous filament nonwoven geotextile reinforced by a biaxial network of high-modulus yarns.

The maximum geotextile strains observed during construction and up to eight weeks following the completion of slope construction are on the order of 0.2%. These are significantly low strain levels, mainly if we consider that extensometers report global strains, comparable with the soil strains obtained from inclinometer readings. The project was revisited in 2010, 17 years after its construction, in order to evaluate its post-construction behavior. The maximum strain in the geotextiles was measured to be only of 0.4%, that is, only 0.2% additional time-dependent strain. It is also possible that the post-construction reinforcement strains occurred due to settlement within the backfill material. The time-dependent strain behavior was found to be approximately log-linear.

Another means to evaluate the performance of the geotextile-reinforced embankment involved evaluation to determine the pavement condition index and the pavement condition rating. To provide a basis for comparison, two other pavement evaluations were conducted on earth structures of similar height in the same highway. Among the various retaining wall systems in the project, the pavement over the geosynthetic-reinforced slope was found to be the one with the highest pavement condition rating.

CASE 10: INGENUITY IN PAVEMENT DESIGN

Concerns in the Design of Pavements over Expansive Clays

The construction of pavements over expansive clay has often led to poor performance due to development of longitudinal cracks induced by moisture fluctuations. These environmental conditions are generally not fully evaluated as part of the design of pavements, which often focuses only on traffic loading conditions. Yet, volumetric changes associated with seasonal moisture variations have led to pavement heave during wet season and shrinkage during dry season.

The mechanisms leading to the development of the classical longitudinal cracks are expected to be due to tensile stresses induced by flexion of the pavement during settlements occurred in dry seasons. During the dry season, there is decrease in the moisture content of the soil in the vicinity of the pavement shoulders. This leads to settlements in the shoulder area, but not in the vicinity of the central line of the pavement, where the moisture content remains approximately constant throughout the dry season. On the other hand, during the wet season, the moisture content in the soil in the vicinity of the pavement shoulder increases.

A Creative Solution using Geosynthetics: Base Reinforced Pavements on Expansive Clays

Base reinforcement involves placing a geosynthetic at the bottom or within a base course to increase the structural or load-carrying capacity of a pavement system. Two traditional benefits are reported for reinforced pavements: (1) improvement of the pavement service life, and (2) equivalent pavement performance with a reduced structural section. A number of studies have been conducted to quantify the effectiveness of geogrids in pavements (Al-Qadi 1997, Perkins and Ismeik 1997, Zornberg and Gupta 2010). While field observations point to the good performance of geosynthetic-reinforced pavements, the actual properties governing the contribution of geosynthetics to the pavement reinforcement have not been clearly identified. A new application of basal reinforcement of pavements has been used in Texas with the purpose of mitigating the development of longitudinal cracks in pavements constructed over expansive clays.

A Recent Project: Low Volume Road over Expansive Clays in Milam County, TX

A project involving the use of geosynthetic reinforcements in a pavement over expansive clays is the reconstruction of FM 1915 located in Milam County, Texas. In 1996, an extensive network of longitudinal cracks was observed in over a 4 km stretch of the pavement section. Accordingly, the pavement was reconstructed with 0.25 m of lime treated subgrade and an asphalt seal coat on top. Due to the presence of expansive clays, a geogrid was placed at the interface between the base and subgrade. In order to evaluate the actual effect of the geogrid on the required base course thickness, two geogrid reinforced sections were constructed. The first section (Section 1) included a 0.20 m-thick base course, while the second section (Section 2) involved a 0.127 m-thick base course underlain by the same geogrid. In addition, a control (unreinforced) section was constructed with a 0.20 m-thick base course (Fig. 11).

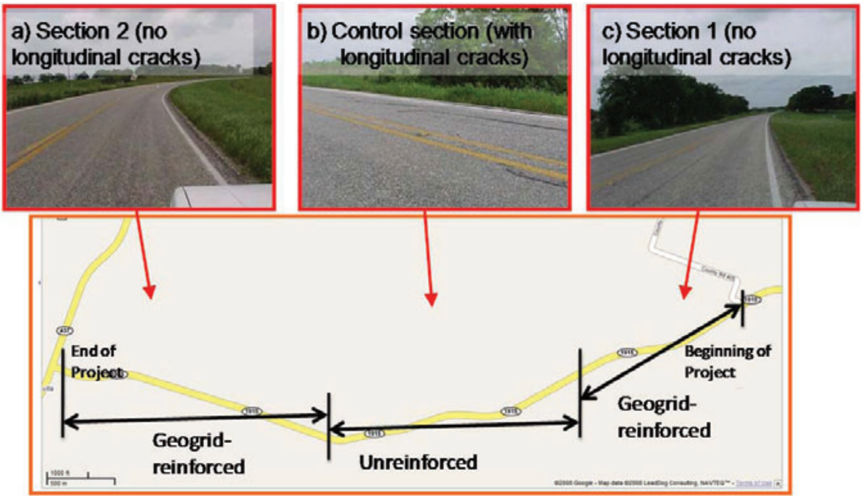

Fig. 11: Comparison of the performance of pavement sections over expansive clays: (a) Geogrid-reinforced Section 2; (b) unreinforced control section; (c) Geogrid-reinforced Section 1.

While falling weight deflectometer testing was conducted to quantify the pavement performance, the clearest evaluation was obtained based on condition surveys and visual inspection of the pavement. Specifically, the control section was found to develop significant longitudinal cracks only after a few months of use. On other hand, the two geogrid-reinforced sections were found to perform well, without any evidence of longitudinal cracking. Fig. 11 also illustrates the extent of the three experimental sections and details the performance of the three sections. An important lesson can be learned from this field experience: geosynthetic reinforcements have prevented the development of longitudinal cracks over expansive clays while unreinforced sections over similar clays have shown significant cracking.

CONCLUSIONS

Geosynthetics can now be considered a well-established technology within the portfolio of solutions available for geotechnical engineering projects. Yet, ingenuity continues to be significant in geotechnical projects that involve their use. This is probably because of the ability to tailor the mechanical and hydraulic properties in a controlled manner to satisfy the needs in all areas of geotechnical engineering. This paper discussed 10 (ten) recent applications or recent evaluations of old applications in geotechnical projects involving geosynthetics.

The discussion of each application identifies specific difficulties in geotechnical design, the creative use of geosynthetics to overcome the difficulties, and a specific case history illustrating the application. Specifically, this paper illustrates the merits of using geotextiles as filters in earth dams, the use of exposed geomembranes as a promising approach for resistive covers, the use of geotextiles as capillary barrier in unsaturated soil covers, the use of anchored geosynthetic reinforcements in stabilization of steep veneer slopes, the use of geotextile tubes for challenging coastal protection projects, the use of geotextile encased columns to stabilize very soft foundation soils, the use of integral geosynthetic-reinforced bridge abutments to minimize the "bump at the end of the bridge," the use of geogrids in the design of the highest reinforced soil wall involving geosynthetics, the use of reinforcements with in-plane drainage capabilities in the design of steep slopes, and the use of geosynthetic reinforcements to mitigate the detrimental effect of expansive clays on pavements.

Overall, geosynthetics play an important role in all geotechnical applications because of their versatility, cost-effectiveness, ease of installation, and good characterization of their mechanical and hydraulic properties. The creative use of geosynthetics in geotechnical practice is likely to expand as manufacturers develop new and improved materials and as engineers/designers develop analysis routines for new applications.

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State of the Practice

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State-of-the-Practice of Characterization and Remediation of Contaminated Sites

Jeffrey A. Adams[1], P.E., M.ASCE and Krishna R. Reddy[2], P.E., M.ASCE, D.GE

[1]Associate, ENGEO Incorporated, 2010 Crow Canyon Place, Suite 250, San Ramon, California 94583, USA, e-mail: jadams@engeo.com
[2]Professor, Department of Civil and Materials Engineering, University of Illinois at Chicago, 842 West Taylor Street, Chicago, Illinois 60607, USA, e-mail: kreddy@uic.edu

ABSTRACT: With increased manufacturing and industrial prowess following World War II, the United States entered into a period of unprecedented economic growth and advancement of the quality of life for its citizens. Nevertheless, as had occurred before in Western Europe, and is now occurring in the developing world, economic expansion and industrialization outpaced advances with respect to environmental stewardship. As a result, the environment was significantly impacted – often in the form of vivid, high-visibility events and disasters, but far more often with little notice or directly observable side effects on the subsurface environment. Regardless, in all of these cases, the cumulative effect was enormous and presented a very real threat to the gains made in quality of life in the preceding years. This state-of-the-practice paper presents a chronology of the effects of over 60 years of rapid industrialization, economic expansion, the detrimental effects to the environment, and the resulting reactions from private citizens, the corporate sector, and local, state, and federal government. An overview of the evolving regulatory framework as well as site characterization and remediation technologies used in practice is presented. Finally, the emerging holistic approaches and technologies to achieve green and sustainable remediation of contaminated sites are discussed. Many challenges and opportunities still exist for the development of efficient, reliable, simple and cost effective technologies to characterize and remediate numerous contaminated sites.

INTRODUCTION

Following victory in World War II, the United States entered into an unprecedented era of prosperity and industrial development. Industrial and manufacturing progress occurred over a wide range of the economy, from advances in aviation and aerospace to surface transportation, disposable and durable consumer goods, and other market sectors.

As industrial and manufacturing capacity expanded, the chemical industry also grew at an appreciable pace. Not only did the petroleum and petrochemical industries flourish, but numerous advances in synthetic chemical development and processing were achieved. Optimism grew that innovations in chemistry could improve the quality of millions of lives through the development of new products as well as emerging applications in agricultural, pharmaceuticals, and a wide range of other industries.

Nevertheless, as had occurred in Europe before and continues to this day throughout the developing world, industrialization and the resulting economic development and expansion outpaced management of wastes and by-products as well as overall environmental stewardship. In the 1950s and the 1960s, increased stresses to the environment began to manifest themselves in milestone events.

This state-of-the-practice paper describes the evolution of environmental laws and regulations and practical methods and technologies to assess, characterize and remediate contaminated sites, specifically soils and groundwater. While this paper is not meant to provide a comprehensive technical overview of remedial methods, it presents the lessons learned and advances made in addressing polluted sites chronologically from pre-Resource Conservation and Recovery Act (RCRA) and Comprehensive Environmental, Response, Compensation, and Liabilities Act (CERCLA) era to the present green and sustainable remediation era. Though the site remediation field has advanced significantly in the past few decades, many challenges and opportunities still exist to address holistic environmental impacts and mitigate them through modern and innovative technologies.

PRE-RCRA AND CERCLA ERA (1940s-1976)

Beginning of Environmental Issues

From the 1940s through the 1960s, very little if any collective energy was focused on environmental issues. The United States economy and population were both growing at an unprecedented pace, and individual, private sector, and public sector goals and initiatives were directed toward providing housing, and consumer and durable goods to growing families within an expanding middle class.

During this time, disposal practices of liquids and solids were quite rudimentary. Solids and liquids were often placed in uncontrolled dumps without any provisions for secondary containment, or in many cases, primary containment. Liquid wastes and solid wastes were also dumped into waterways without regard for chemical or thermal effects to the receiving waters. Despite some initial evolving legislation in the 1950s, air emissions from point or mobile sources were often unregulated or unchecked. As a result, the rapidly increasing pollutant loads to air, water, and soil were overwhelming the environment's ability to absorb these releases without manifested side effects. Additionally, numerous chemicals released to the environment could not be degraded though natural processes within a reasonable amount of time.

Air pollution was becoming increasingly prevalent; notable smog outbreaks in Donora, Pennsylvania (1948), London, UK (1952), New York (1953) and Los Angeles (1954) resulted in appreciable loss of life and significant disruptions to daily activities. In response, the Air Pollution Control Act was passed in 1955. This initial legislation acknowledged that air pollution was a growing hazard to public health; however, it deferred responsibility of combating air pollution to the individual states and did not contain power to sanction or hold air polluters responsible for their actions.

Water pollution was gaining notoriety with spectacular images and events. On multiple occasions (but most notably in 1969), the Cuyahoga River in Cleveland caught on fire. Downstream from the Cuyahoga River, its receiving water, Lake Erie, was declared biologically dead in the 1960s. Yet, Ohio was by far not the only source of impacted water bodies – they were found in every state, and the impacts were increasing.

In the Niagara Falls, New York area, development occurred over the previously abandoned Love Canal (Figure 1). The resulting development and infrastructure construction pierced the clay-lined canal. Over time, noxious odors were observed, and significant acute and chronic health problems were reported by the citizens. Eventually, follow-up testing and analysis determined the presence of widespread soil and groundwater contamination, and the United States Federal government paid for the relocation of hundreds from the Love Canal area.

(a) 1950s: Waste Disposal (b) 1978: Development (c) 2007: Post-Remediation

FIG.1. Love Canal Hazardous Waste Disposal Site

Several other notable environmental impacts entered the public consciousness. Among several large-scale oil platform and tanker disasters, in 1969, an offshore well accident resulted in crude oil washing ashore onto beaches along the Santa Barbara Channel in California. Additionally, nuclear fallout from above-ground nuclear weapons testing, first in the deserts of the western United States, and later in the Pacific Ocean, resulted in health impacts among those exposed.

These high-profile events as well as the everyday observations of "ordinary" citizens in their lives gave rise to a "grass-roots" environmental movement. Of the milestone occurrences associated with this movement, the first has been traditionally credited to the publication of Rachel Carson's *Silent Spring* in late 1962. This book lamented the observed death of song birds, ostensibly from the uncontrolled use of pesticides for vector abatement, most notably mosquitoes. Other evidence of DDT use and its deleterious impact to the environment began to emerge – declining bald eagle populations in the United States were attributed to bioaccumulation of DDT, resulting in adverse effects to their eggs. Public outrage increased, and eventually, DDT use was banned in the United States in 1972.

The 1969 Santa Barbara Channel oil spill also spawned the first observance of Earth Day in April 1970. The idea was well received by a wide range of audiences and interest groups, and millions took part in seminars, conferences, rallies, and demonstrations. Earth Day continues to this day and is celebrated in an ever-increasing number of countries by hundreds of millions of people.

Initiation of Environmental Regulations

The major environmental events as well as the evolving public interest in environmental protection began to coalesce in the 1960s and 1970s, and the federal government began to take notice. Beginning in the 1960s and into the 1970s, the federal government began to enact legislation designed to protect the environment. Some of these legislative acts and regulations include the following (Sharma and Reddy, 2004):

- Solid Waste Disposal Act (SWDA) (1965, 1970) – the first federal legislation attempting to regulate municipal solid waste. Provisions included: (1) An emphasis on the reduction of solid waste volumes; (2) An emphasis on the improvement of waste disposal practices; (3) Funding for individual states to better manage their solid wastes; and (4) Amendments in 1970 encouraged further waste reduction and waste recovery as well as the creation of a system of national disposal sites for hazardous wastes.
- Clean Air Act (CAA) (1970, 1977, and 1990) – represented the first comprehensive law that regulated air emissions from area, stationary, and mobile sources. Provisions of the law included: (1) The establishment of National Ambient Air Quality Standards (NAAQSs) for criteria pollutants; (2) Development of standards for other hazardous air pollutants, including asbestos, volatile compounds, metals, and radionuclides where NAAQSs have not been specified; (3) Establishment of Air Quality Regions within the United States for the purposes of regional monitoring toward attainment or non-attainment of quality goals; and (4) Later amendments established a comprehensive permitting system for various emissions sources toward the regulation of several common pollutants.
- Clean Water Act (CWA) (1977, 1981, and 1987) – established a structure for the regulation of pollutant discharge into U.S. waters. Provisions of the law included: (1) Identification of 129 priority pollutants as hazardous wastes; (2) Wastewater discharge treatment requirements mandating best available technologies; (3) Prohibition of discharge from point sources unless a National Pollutant Discharge Elimination System (NPDES) has been obtained; (4) Discharge of dredged material into U.S. waters is only allowed if a permit has been obtained; and (5) Discharges from Publicly Owned Treatment Works (POTWs) must meet pre-treatment standards.
- Safe Drinking Water Act (SDWA) (1974, 1977, and 1986) – passed to protect the quality of drinking water in the United States, whether obtained from above-ground or groundwater sources. Provisions of the law included: (1) Establishment of drinking water standards, including maximum contaminant levels, primary goals, and secondary goals that provide protection of health and aesthetic standards; (2) Protection of groundwater through the regulation of hazardous waste injections; and (3) Designation and protection of aquifers.
- Toxic Substances Control Act (TSCA) (1976) – enacted to regulate the use of hazardous chemicals. Provisions of the law included: (1) Requirement of industries to report or test chemicals that may pose an environmental or human health threat; (2) Prohibition of the manufacture and import of chemicals that pose an unreasonable risk; (3) Requirement of pre-manufacture notifications to

the United States Environmental Protection Agency (USEPA); (4) Prohibition of PCBs; and (5) The management and regulation of asbestos.

Lack of Guiding Framework

Despite these regulatory advances, several drawbacks and limitations still existed. First, with regard to solid waste disposal, a comprehensive framework was still not in place. The concept of "engineered landfill" still had not replaced the concept of a "dump". Further, although the regulatory framework had been developed to address the production, storage, and use of hazardous materials as well as regulations for controlled emissions and releases, a framework had not been developed for handling and remediating spills and other unauthorized releases of hazardous materials and petroleum products to environment.

RCRA/CERCLA ERA (1976-1986)

Initiation of Comprehensive Regulatory Framework

Many of the previously cited statutes and regulations lacked strong enforcement or sanctioning abilities. In other cases, these regulatory frameworks induced unintended and unfavorable behaviors and actions, such as unauthorized disposal, through perceived loopholes or exclusions. Additionally, since few regulations were in place for landfills, other disposal methods, such as deep groundwater injection, became increasingly common (Sharma and Reddy, 2004). To counteract these practices, two comprehensive major environmental laws were promulgated.

Resource Conservation and Recovery Act (RCRA)

The Resource Conservation and Recovery Act (RCRA) was passed in 1976 to manage and regulate both hazardous and nonhazardous wastes as well as underground storage tanks. RCRA also placed an emphasis on the recovery and reuse of materials through recycling (Sharma and Reddy, 2004). The major regulations include the following (USEPA, 2011):

- Subtitle C was developed to manage hazardous wastes, including a regulatory framework for the generation, transportation, storage and disposal of hazardous waste as well as technical standards for the design and operation of treatment, storage and disposal facilities (TSDFs).
- Subtitle D addresses non-hazardous solid wastes, including hazardous wastes from households and from conditionally exempt small quantity generators, general household waste, non-recycled household appliances, non-hazardous scrap and debris, such as metal scrap, wallboard and empty containers, and sludge from wastewater and water treatment plants.
- Subtitle I regulates underground storage tanks and includes provisions for notification, methods for release detection, design and construction standards, and reporting, recordkeeping, and financial responsibility. Corrective actions pertaining to releases from USTs are also regulated under Subtitle I. USTs containing hazardous wastes are regulated under Subtitle C.

Additional statutes were passed in 1984 in the Hazardous and Solid Waste Amendments. Much of the focus of these amendments was to protect groundwater, including the following (Sharma and Reddy, 2004): (1) Restrictions were placed on the disposal of liquids; (2) Requirements for the management and treatment of small amounts of hazardous wastes; (3) Additional regulations for underground storage tanks in urban areas; (4) Establishment of new standards for landfill facilities, including liner systems, leachate collection systems, groundwater monitoring, and leak detection; (5) Additional specific requirements for TSDFs; and (6) The USEPA was authorized to inspect and enforce these regulations as well as penalize violations.

Comprehensive Environmental Response, Compensation, and Liabilities Act (CERCLA) or Superfund

Numerous contaminated sites outside of RCRA jurisdiction (e.g., abandoned sites) posed a significant threat to human health or the environment. As a result, in 1980, the Comprehensive Environmental Response, Compensation, and Liabilities Act (CERCLA), or "Superfund", was passed to address cleanup of these hazardous sites. This extensive regulatory framework specifically addressed funding, liability, and prioritization of hazardous and/or abandoned waste sites.

Some key provisions of CERCLA include the following (Sharma and Reddy, 2004): (1) A \$1.6 billion fund was created from taxes levied on chemical and petroleum industries to finance the cleanup of hazardous waste sites and litigation brought against potentially responsible parties (PRPs); (2) A hazard ranking system (HRS) was developed to establish priority for contaminated sites to determine which sites could be placed on the National Priorities List (NPL); and (3) A framework was developed to outline site characterization and assessment of remedial alternatives.

Additional funds (\$8.5 billion) were appropriated in 1986 with the passage of the Superfund Amendments and Reauthorization Act (SARA). A \$500 million fund was also appropriated for the remediation of leaking underground storage tanks. Additionally, community right-to-know provisions were adopted. Controversially, SARA specified that cleanups were required to meet applicable or relevant and appropriate requirements (ARARs) and established provisions for cleanup-related legal and financial liability. Disclosure requirements related to annual releases of hazardous substances were also included.

Explicit liability provisions directed at current landowners and related innocent landowner provisions became a paramount concern for all entities associated with land transactions. As a result, standards were developed to assess the potential of contamination at properties. Three phases of environmental site assessments were developed. These include the following: (1) Phase I assessments are associated with a preliminary assessment to determine the potential for environmental impact at a site; (2) Phase II assessments include actual sampling of soil, groundwater, and soil vapor to determine the extent (if any) of environmental impact at a site; and (3) Phase III assessments include actual environmental remediation of impacts confirmed during previous phases of study.

As with CERCLA, SARA significantly underestimated the potential costs and timing associated with environmental cleanups. When CERCLA was first enacted, approximately 36,000 contaminated sites were identified; of these, 1,200 were placed on the NPL. At the end of Fiscal Year 2010, 1,627 sites remained on the NPL, and 475 sites had been closed (OSWER, 2011). However, these closures consumed a significant amount of resources; on average, \$40 million was expended per site (Gamper-Rabindran and Timmins, 2011) requiring an average of 11 years to achieve closure. Further, \$6 billion held in trust in 1996 had been exhausted by 2003.

Site Characterization Methods

Typically, soil impacts were characterized through the collection of soil samples from soil borings. Rotary soil borings, while effective, generate a relatively large volume of soil cuttings; in many cases these soils may be impacted and require special handling and disposal provisions. Monitoring wells installed using rotary borings also generate significant cuttings. Both of these characterization techniques are still widely used today; however, many improved techniques have been developed to improve production, ease construction, or limit the amount of waste materials. For instance, direct-push methods have become increasingly common. The small-diameter push probes greatly reduce the volume of investigation-derived waste (IDW), and in many cases, a continuous soil core can be recovered from the subsurface. Direct-push methods may also be used to facilitate groundwater collection, either by allowing "grab" groundwater samples or through the installation of pre-fabricated or "packed" wells.

Sampling and Analytical Methods

As important as the development of characterization techniques was the development of sampling and analytical methods for soil and water samples. The USEPA developed publication SW-846, *Test Methods for Evaluating Solid Waste, Physical/Chemical Methods*. This guide compiled analytical and sampling methods evaluated and approved for use in complying with RCRA regulations. SW-846 functions primarily as a guidance document that established acceptable sampling and analysis methods. SW-846 was first issued by USEPA in 1980. New editions have been issued to accommodate advances in analytical instrumentation and techniques.

Human and Ecological Risk Assessment

Human health and ecological risk assessments became important in feasibility evaluations, and the USEPA developed comprehensive methods to perform these assessments for Superfund sites (USEPA, 1997). Human health risk assessment consists of four basic steps: (1) hazard identification; (2) exposure assessment; (3) toxicity assessment; and (4) risk characterization. The most critical aspect of such assessment is developing a conceptual model, identifying receptors and exposure pathways, and determining exposure dosages under existing and potential remedial conditions. Figure 2 shows an example of such conceptual model. Knowing the toxicology data, risk is quantified, and risk less than 1×10^{-6} (one in one million) is generally considered acceptable. Unfortunately, this assessment is cumbersome and requires a large set of input data or necessity to make assumptions.

Remediation Technologies

With the evolving liability provisions assigned to landowners as well as the empowerment of USEPA to enforce cleanups and recover costs, significant attention was focused on cleanup technologies. At first, two "brute-force" remedial methods were prevalent – one each for contaminated soils and contaminated groundwater – soil excavation and groundwater extraction ("pump-and-treat"). Both methods are very straightforward. In the case of soil excavation, contaminated soils are removed from the subsurface until clean excavation bases and sidewalls have been established. Impacted soil is typically characterized through subsequent testing, allowing for appropriate transportation and disposal measures. Following excavation activities, clean fill materials are used to backfill the resulting excavation. In the cases of groundwater extraction, impacted groundwater is pumped to the surface, where it may be treated. The extracted groundwater may be disposed to a sewer facility, containerized and removed from the site, or re-injected into the subsurface. As the groundwater is extracted, dissolved contaminant mass is removed, which induces subsequent dissolution of non-aqueous phase liquid (NAPL) contaminant from free-phase sources or those adsorbed to the soil matrix. However, this process is rate-limiting and time-consuming, and as a result, pump-and-treat systems have commonly been operated for many years without achieving remedial goals.

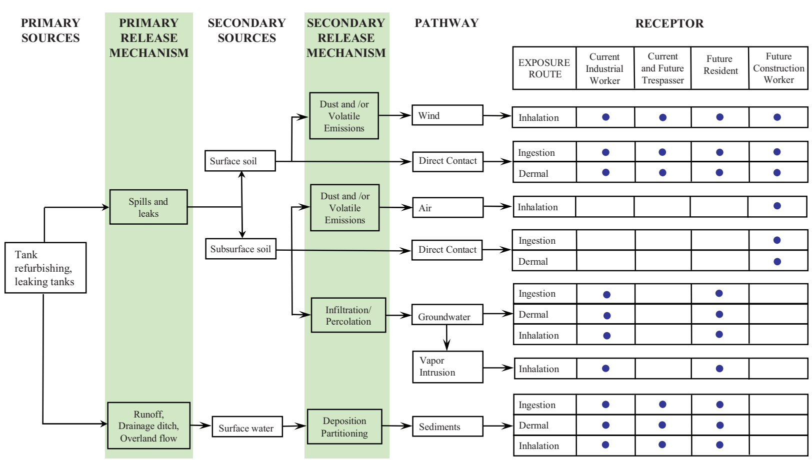

FIG.2. Human Health Conceptual Site Model

RBCA ERA (1986-1993)

Risk-Based Remediation

CERCLA and RCRA significantly changed the environmental regulatory landscape. For the first time, these landmark regulations induced compliance with intended waste disposal objectives. Additionally, responsible parties and landowners were compelled to remediate contaminated sites that posed a threat to human health and the environment. However, with such rapid change arose several complicating issues. The regulatory frameworks did not fully address indemnification to truly innocent parties. As such, perceptions about potential liability with respect to properties became a significant barrier to land transactions involving properties with confirmed or perceived contamination issues. Further, cleanup standards had not evolved with the passage of the legislation. Cleanup standards were motivated by an objective to restore contaminated soils and groundwater to a pristine condition. These cleanup objectives greatly affected the magnitude of cleanup effort required for site closure – with the same effect on related costs and time to closure.

Further, in many cases, these cleanup objectives could be considered unnecessarily conservative. In many cases, restored soil and water resources could not be functionally used due to other considerations, such as those associated with existing land uses. For instance, it may be considered unnecessary to remediate groundwater such that contaminants of concern (COCs) are reduced to drinking water standards in areas where groundwater is not considered potable due to naturally occurring conditions. Additionally, it may be considered inappropriate to mitigate contaminant concentrations within soils to non-detectable concentrations at ongoing industrial facilities. As a result, significant resources and time were often expended with little incremental benefit. While CERCLA and RCRA were significantly beneficial in protecting and remediating the environment, risk-based remediation approaches were determined to be necessary to more efficiently remedy these issues. To achieve this, various risk assessment methods have emerged to determine the remedial goals that can eliminate risk to public health and the environment.

Emergence of Tiered Risk Assessment Methods

The USEPA-recommended human health risk assessment method requires large input data and requires special expertise to perform such assessments. As a result, industry has initiated tiered risk assessment methods. Risk-based Corrective Action (RBCA) is a tiered assessment originally developed to help assess sites that contained leaking underground storage tanks containing petroleum. Although the standard is geared toward such sites, many state regulatory agencies use a slightly modified version for sites that do not contain underground storage tanks and are contaminated with a wide range of contaminants. This approach integrates risk and exposure assessment practices with site assessment activities and remedial measure selection. The RBCA process allows corrective action activities to be tailored for site-specific conditions and risks and assures that the chosen course of action will protect both human health and the environment.

Remediation Technologies

With an increased focus on risk-based cleanups, innovative soil and groundwater remediation techniques began to emerge. Soil vapor extraction (SVE) emerged as a popular, effective remedial alternative for vadose zone soil impacted with volatile organic compounds and petroleum products. In applying SVE, a vacuum is applied across an array of soil vapor extraction wells. The resulting pressure gradient and contaminant concentration gradients induce soil vapor migration toward the wells for extraction. Once extracted, the collected vapors are brought to the subsurface, where they may be treated in a number of ways, including filtration through granular activated carbon or combustion. SVE is an efficient remedial alternative when soils are permeable and homogeneous and contaminants are "volatile" (vapor pressure greater than 0.5 mm Hg and Henry's Law constant greater than 0.01) (USEPA, 1996). It is also attractive because it can be applied with readily available equipment and is relatively unobtrusive to sites (Sharma and Reddy, 2004).

While SVE has proven to be successful in applications to vadose zone contamination, it is not applicable to groundwater or saturated soil contamination. In-situ air sparging (IAS) emerged as an innovative technique for soil and groundwater remediation. Compressed air is transported through a manifold system which delivers air to an array of air injection wells below the lowest known point of contamination. The injected air rises towards the ground surface, and through variety of mass transfer, transport, and transformation mechanisms, the contamination present within the subsurface partitions into the vapor phase or is degraded. As the contaminant-laden air rises toward the subsurface, it encounters the vadose zone of soil, where it is often captured using SVE system. Further, if the conditions are favorable within the vadose zone, the native subsurface microbial population may degrade the contamination into harmless products (Adams et al., 2011). Similar to SVE, IAS is an efficient remedial alternative when soils are permeable and homogeneous and contaminants are volatile (Reddy and Adams, 2001). However, subsurface heterogeneity, such as layers and lenses of differing hydraulic conductivity, can adversely affect performance.

While air sparging has been successfully applied in permeable soils, another technology was under consideration for the remediation of fine-grained or heterogeneous soils. Electrokinetic remediation, or electrokinetics, consists of the application of an electric potential to soil to facilitate contaminant removal. Two electrode arrays (positively charged anodes and negatively charged cathodes) are placed at the ends of a discrete contaminated soil volume. Under electric potential, the contaminants are transported toward the electrodes under a variety of processes, where they are removed. Additionally, enhancement solutions may be applied to the subsurface at the electrodes as a flushing technique. When the contaminants reach the electrodes, they can be removed, or treated in place by electroplating or precipitation (Sharma and Reddy, 2004). Electrokinetics may be applied to a wide range of contaminants, including heavy metals, radionuclides, and organic compounds (Reddy et al., 1999). It can be applied to a wide range of soil and contaminant conditions, but complex subsurface conditions and resulting geochemical conditions can limit its effectiveness (Chinthamreddy and Reddy, 1999).

In some cases, aggressive in-situ remediation technologies may not be necessary for a given site. Monitored natural attenuation (MNA) approaches emerged as a viable option for contaminated sites. MNA has unfairly been labeled as a "leave it alone" technology. Although little effort is expended on enhancing remediation activity, it is crucial to properly characterize the subsurface to determine that conditions exist for MNA to occur (e.g., contaminant concentrations, groundwater and soil conditions conducive to necessary physical processes, the presence of microbial populations and requisite electron acceptors/donors and nutrients). Additionally, ongoing monitoring and modeling activities are necessary to assess progress and expected remediation timelines.

BROWNFIELDS ERA (1993-2010)

Liability and Resources: Impediments to Remediation

Risk-based approaches to site remediation encouraged more thoughtful, appropriate programs for impacted sites. However, in the absence of direct regulatory agency cleanup orders where site impact poses a significant danger to human health or the environment, financial incentives are almost always the motivation for site remediation. Further, environmental statutes for many years deterred investors from acquiring properties with either confirmed or suspected environmental impact. The deterrents were three-fold. First, entering into a purchase agreement in most cases exposed a buyer or owner to significant legal liability. Second, in the absence of a defined cleanup program with regulatory oversight, it was very difficult to predict costs associated with cleanups. Third, and almost as perilous to a prospective property purchaser, in many jurisdictions, low-risk contaminated sites were not assigned priority, and therefore, were very difficult to procure agency oversight to gain closure.

As a result, in many cases, impacted properties with significant re-use potential remained idle and contaminated for long periods of time. Many of these sites became known as Brownfields. A Brownfield is an abandoned, idled, or underutilized industrial or commercial site where expansion or redevelopment is complicated by actual or perceived environmental contamination (Reddy et al., 1999). The real or perceived contamination can range from minor surface debris to widespread soil and groundwater contamination. In many cases, these sites were located in decaying urban neighborhoods and contributed to overall neighborhood blight while exacerbating other social problems. Ironically, a percentage of these sites were located in areas undergoing extensive urban renewal, yet their potential as productive land remained unfulfilled.

Brownfields Programs

With time, many stakeholders and regulatory agencies associated with contaminated sites realized that CERCLA-induced liability was a significant deterrent to site remediation or redevelopment. In the early 1990s, the federal government took action to provide inducements to encourage Brownfield redevelopment. In 1993, the USEPA launched a Brownfields pilot program with a $200,000 grant used for a contaminated site in Cleveland, Ohio. Since then, millions of dollars in grants have been awarded to states, cities, counties, and tribes (Reddy et al., 1999).

In addition to inducements to pursue Brownfields redevelopment, the USEPA also took measures to clarify liability provisions as well as provide for indemnity for prospective purchasers. In 2002, the US Congress passed the Small Business Liability Relief and Brownfields Revitalization Act (SBLR&BRA) to accomplish the following: (1) The provision of certain relief for small businesses from liability imposed under the CERCLA; (2) Promotion of the cleanup and reuse of brownfields; (3) The provision of financial assistance for brownfields revitalization; and (4) The enhancement of state response programs. In 2005, the USEPA established the All Appropriate Inquiries (AAI) requirements, which became law on November 1, 2006.

The purpose of AAI was to establish liability protection under CERCLA for innocent landowners, contiguous property owners, or bona fide prospective purchasers. To establish this protection, prospective property owners must do the following (USEPA, 2009): (1) Conduct All Appropriate Inquiries in compliance with 40 Code of Federal Regulations (CFR) Part 312, prior to acquiring the property; (2) Comply with all Continuing Obligations after acquiring the property (CERCLA §§101(40)(C – G) and §§107(q)(A) (iii – viii)); and (3) Not be affiliated with any liable party through any familial relationship or any contractual, corporate or financial relationship (other than a relationship created by the instrument by which title to the property is conveyed or financed.

AAI has been a very important milestone in encouraging land acquisition and development. By establishing a framework, prospective land purchasers have a discrete set of actions they must perform to avoid open-ended liability and costs. In this manner, they can help eliminate the unknowns associated with a potential redevelopment project, which facilitates a return to productive use for many impacted properties.

Although financial and legal protections have been useful for larger projects or those that, in many cases, may have more acute environmental impacts, many more sites are impacted with low level contamination that, while not posing a significant risk to human health or the environment, still prevent site redevelopment. Further, in many cases where oversight could be made available, regulators and landowners often engaged in contentious relationships with respect to cleanup timelines, costs, and goals. In these cases, the lack of a positive relationship added unnecessary delays, expenditures, and problems for sites that may have been considered low-risk or straightforward with respect to remediation.

Triad – An Integrated Approach

As technological advances continued to evolve with respect to characterization and remediation methods as well as new approaches to regulatory oversight, new overall approaches to site remediation began to emerge. The USEPA compiled Triad, a comprehensive framework to site characterization and remediation. The goal of Triad is to manage uncertainty associated with all phases of a site remediation program. Triad advocates this with three approaches or categories. Through systemic planning, a conceptual site model (CSM) is developed, identifying key areas of uncertainty and allowing for the consideration of measures to mitigate this uncertainty. Key decisions regarding the remediation process with respect to operations and strategy are also made. Triad encourages dynamic work strategies that allow flexibility in characterization, remediation, and monitoring strategy as incoming ongoing information and data become available. Triad also advocates the use of real-time measurement through the use of such methods as rapid-turnaround analyses from fixed-base laboratories, mobile laboratories, and real-time in-situ and ex-situ technologies.

Voluntary Site Remediation Programs

With new resources and clarifications on liability, many states began to establish voluntary site cleanup or remediation programs. The goal was to create a framework in which regulatory agencies and property owners/purchasers could collaborate on a remediation program. Although the states' programs are typically administered on an individual basis, they feature common objectives and characteristics. Commonly, the owner/purchaser and the regulatory agency enter into a formal agreement. Often the agency is reimbursed for their oversight activities. The agency and owner/purchaser work together to establish a timeline and cleanup goals and to identify reasonable remedial system alternatives. Once the remediation has occurred, the regulatory agency issues a case closure through "No Further Action" status or similar finding.

As an example of a voluntary state program, California established a Brownfields program in late 1993. The Voluntary Cleanup Program (VCP) induces volunteer cleanup actions (the volunteer parties may or may not be responsible parties, or RPs) at eligible sites under the oversight of the California Department of Toxic Substances Control (DTSC) (DTSC, 2008). Project proponents enter into Voluntary Cleanup Agreements, which include reimbursement to DTSC for their oversight costs. Proponents develop a detailed scope of work, project schedule and services to be provided by DTSC. Importantly, project proponents do not admit legal liability for site remediation upon entering into a VCP agreement. Further, a 30-day "grace period" exists where either party (the Proponent or DTSC) may terminate the project with written notice (DTSC, 1995). Sites must be remediated to the same cleanup standards as those under DTSC jurisdiction by not within the VCP; however, the program allows for flexibility with respect to project timing and phasing (DTSC, 1995). Following remediation activities and the achievement of remedial action goals, DTSC may issue a "No Further Action" (NFA) letter or certification of completion, depending on the project circumstances. In either case, the issuance of this finding confirms that DTSC has determined that the site does not pose a significant risk to public health or the environment (DTSC, 1995).

The California plan is similar to programs that exist in other states (e.g., Illinois's Voluntary Site Remediation Program). Specifically, through the collaborative process, the project stakeholders can collectively assess and identify appropriate, efficient remedial alternatives. Many states require a cost-benefit analysis to study how proposed alternatives compare with respect to overall associated costs and remediation times. These programs have proven to be useful to all project stakeholders in facilitating site cleanups and restoring land to productive uses.

In many cases, alternative strategies other than full remediation may be considered. Using a risk-based exposure approach, site remediation may be coupled with site controls into an integrated strategy protective of future land use. As an example, institutional controls, including deed restrictions, covenants, or easements may be applied to a property that forbid specific activities (i.e., deep excavations or groundwater use) or allow for ongoing post-remedial monitoring activities. Engineering controls, including vapor barriers or subsurface active/passive ventilation systems, can also be used to mitigate potential exposure risks.

Expedited Site Characterization Technology

Several innovative site characterization methods have recently been developed and are being increasingly incorporated into site characterization. Although off-site, fixed-base laboratories continue to be popular and necessary for a range of analyses, mobile laboratories are also becoming increasingly popular. Confirmation sampling can be conducted in real-time as remediation activities are taking place.

In addition, Membrane Interface Probe (MIP) is a semi-quantitative, field-screening device that can detect volatile organic compounds (VOCs) in saturated and unsaturated soil and sediment (CLU-IN, 2011). Driven by direct-push technology, the probe captures a vapor sample, and a carrier gas transports the sample to the surface for analysis by a variety of field or laboratory analytical methods. Additional sensors may be added to the probe to facilitate soil logging and identify contaminant concentrations (CLU-IN, 2011). Essentially, it provides real-time, semi-quantitative data of subsurface conditions, reducing the need to collect soil and groundwater samples as well as the costs and lead times associated with sampling and analysis. It is especially efficient at locating source zones or "hot spots" associated with dense non-aqueous phase liquids (DNAPLs) and light non-aqueous phase liquids (LNAPLs).

X-ray Fluorescence (XRF) sensors may be employed with direct-push technology for real-time, in-situ monitoring of heavy metals. The XRF sensor system uses an x-ray source located in the probe to bombard the soil sample with x-rays. The bombardment excites various atoms and induces them to emit fluorescence that is correlated to specific metals.

Fiber Optic Chemical Sensors (FOCS) constitute another direct-push technology. FOCS use optical fiber to transmit light into the subsurface. During application, the light interacts with the COC within the subsurface, during which sensors observe a reaction or change (e.g., absorbance, reflectance, fluorescence, light polarization). One specific FOCS is laser-induced fluorescence (LIF). LIF is used for the detection of aromatic compounds or polycyclic aromatic hydrocarbons (PAHs). During application, the COCs fluoresce, providing information regarding location within the subsurface and relative degree of concentration.

Geophysical applications have also been increasingly employed for subsurface characterization. Although mostly used to identify subsurface objects or structures, they can be used to identify evidence or characteristics of potential subsurface contamination. Ground Penetrating Radar (GPR) uses high-frequency pulsed electromagnetic waves that are propagated downward into the subsurface. Energy is reflected back to the subsurface, where contrasts are used to identify objects or discontinuities, such as drums, tanks, landfill boundaries, or in some cases contamination. Magnetic technology is also employed, commonly using a magnetometer. Used to identify objects consisting of ferrous alloys, this technology is also used to identify drums, pipelines, tanks, and in some cases, mineralized iron ores. Obviously, this technology is not applicable to non-ferrous materials.

Much like Cone Penetration Testing (CPT) technology for geotechnical applications, although useful, these technologies do not eliminate the need for actual sampling, and it is subject to some limitations. These technologies provide screening level data that need to be supplemented with analytical soil or groundwater data to fully support human health risk assessments or remediation decisions (CLU-IN, 2011).

Yet another advance has been the rapid evolution and adoption of soil vapor sampling technology. The use of soil vapor sampling has increased dramatically in the past few years, due to both the introduction of more robust sampling technologies and procedures as well as increased favor of the use of soil vapor data in risk assessment. This has been used to replace modeling based on soil and groundwater data as well as passive indoor air vapor sampling.

Rapid Remediation Technologies

Not only has the focus been on rapid characterization methods, there has also been an increased focus on rapid remediation technologies. Excavation of impacted soil remains commonly used and is often cost-effective and efficient, especially when it can be combined with an on-site mobile laboratory. In spite of its drawbacks, it is still a rapid and useful technology when surface and subsurface conditions warrant its use.

Chemical oxidation technologies have also evolved as a preferred remedial alternative for in-situ or ex-situ remediation of soils and groundwater. With this technology, an oxidizing agent is introduced and mixed into the subsurface. The oxidizing agents most commonly used for treatment of hazardous contaminants in soil are ozone, hydrogen peroxide, hypochlorites, chlorine, chlorine dioxide, potassium permanganate, and Fenton's reagent (hydrogen peroxide and iron) (CLU-IN, 2011). The effectiveness of some of these oxidants can be enhanced through activation (Fenton's reagent, activated persulfate) and used in conjunction with other oxidants (perozone) (ITRC, 2005).

Another rapid technology is soil stabilization and solidification. With this method, additives or processes are applied to contaminated soil to chemically bind and immobilize contaminants, preventing mobility (Sharma and Reddy, 2004). Common binding agents include Portland cement, fly ash, cement kiln dust, and lime. The process may be applied in-situ or ex-situ. When performed ex-situ, the treated soil mass may be re-placed into the subsurface or off-hauled for disposal at an appropriate landfilling facility. In either in-situ or ex-situ, it is critical to assure that the reagent has been thoroughly mixed with the soil mass.

In addition, various thermal methods have been developed to accomplish contaminant remediation quickly. In-situ soil heating decontaminates soils through vaporization, steam distillation and stripping, and may be performed through power line frequency heating or radiofrequency heating. In-situ soil heating is applicable to both organic and semi-organic contamination; however, it may become cost-prohibitive when applied to deep-contaminated sites. Vitrification uses electric current to melt contaminated soil, destroying or removing organic materials and retaining radionuclides and heavy metals within the vitrified material. A number of ex-situ thermal methods are also effective in treating a variety of contaminants.

Soil flushing incorporates an aqueous solution used to flush and/or degrade contaminants from an impacted soil matrix. Common flushing agents include water, surfactants, co-solvents, acids/bases, and reactants/oxidants. Flushing technologies are best applied to high permeability soils.

Several other methods have been developed, including enhanced or augmented bioremediation, in which microbial populations are stimulated to degrade contaminants. Required nutrients, electron acceptors/donors, or bacterial populations are delivered to the subsurface. This method requires close monitoring to assess progress as well as the ongoing existence of suitable subsurface conditions.

In some cases, passive containment systems such as permeable reactive barriers (PRBs) and vertical barriers and caps have also been used as remedial strategy, especially if the contaminants are spreading rapidly, contaminants are non-uniformly distributed, or if the contaminated area is very large. PRBs incorporate a reactive media to adsorb, degrade or destroy contamination within groundwater as it passes through the barrier. Common reactants include zero-valent iron, zeolites, organobentonites, and hydroxyapatite. PRBs may be continuously installed perpendicular to a migrating plume, or they may consist of a "funnel-and-gate" design that diverts water flow through a treatment zone. PRBs must be monitored closely to ensure that suitable reactant mass is present as well as confirm that flow has not been lessened by clogging.

PRBs are also a technology where a mass flux/discharge analysis approach can be an effective analysis alternative. In contrast to the "point" approach utilized with numerous characterization and remediation technologies, the mass flux/discharge approach assesses the transport of contaminant mass across a monitoring interface over a period of time. It can be applied with pumping tests, in-well meters, or integrative approaches, such as the transect method. It can be especially useful in addressing plume stability and fate and transport assessment.

A single remediation technology often cannot cost-effectively address the technical challenges posed by contamination at a particular site. For instance, some technologies may be appropriate for removing source zone impacts, either in the form of free-phase product or very high concentrations, but may not be efficient or cost-effective at removing lower soil or groundwater concentrations. Based on the site-specific conditions, multiple technologies may be sequentially or concurrently used for remediation. This approach, commonly referred to as "treatment trains", can be very effective at accelerating cleanup timeframes and/or reducing the costs associated with remediation. This approach is also effective at addressing the presence of multiple disparate contaminants or multiple contaminated media.

Special Challenges

Despite the technological advances in characterization and remediation, several challenges remain. One example is the characterization and remediation of NAPLs, and in particular, DNAPLs. When introduced into the subsurface, DNAPLs migrate downward into the subsurface, and often are trapped in isolated areas near low permeable inclusions. They can be difficult to characterize and remediate, although some of previously mentioned innovative technologies have proven to be useful.

Further, contamination within fractured rock results in other unique challenges. When introduced, contamination will often migrate along bedding planes, joints, and fractures. Contamination can diffuse into and out of the solid matrix. Again, the aforementioned technologies, including geophysical methods, can be especially useful in such a setting.

GREEN AND SUSTAINABLE REMEDIATION ERA (2009-PRESENT)

Looking Beyond the Fence Line

During the Brownfields era, significant innovative technological advances were achieved, and the new collaborative regulatory environment resulted in productive land re-use and protection of the environment. However, many successful remedial programs were resulting in unwanted side effects and problems “beyond the fence”.

In many cases, contamination was not being destroyed but was instead transformed into a different media (i.e., from soil to liquid or from liquid to air). This resulted in unfavorable air emissions, contaminated extracted groundwater, or appreciable quantities of impacted soil. Additionally, secondary (but significant) effects were occurring, including significant energy and virgin material input, significant greenhouse gas emissions, and diversion of limited resources from other potential uses. These unintended side effects reduced the overall net environmental benefit when considering the overall effect of a site remediation program, or in rare instances, produced a negative overall environmental effect.

Emergence of GSR

Traditional risk-based site remedial approaches have not always been sustainable because they often do not account for broader environmental impacts or overall net environmental benefits. To address this, a focus on green and sustainable remediation (GSR) has begun to emerge. GSR is a comprehensive approach that protects human health and the environment while minimizing environmental side effects. Goals include: (1) Minimizing total energy use and promoting the use of renewable energy for operations and transportation; (2) Preserving natural resources; (3) Minimizing waste generation while maximizing materials recycling, and (4) Maximizing future reuse options for remediated land (USEPA, 2008; Ellis and Hadley, 2009; ITRC, 2011). In addition to the environment, GSR attempts to maximize social and economic benefits (often collectively known as the triple bottom line) associated with a remedial project.

GSR Framework

A GSR framework representing the confluence of environmental, social and economic factors in decision-making to increase sustainability for a project has not yet been developed or standardized, but it will likely address at least the following factors: (1) Reduced energy consumption but greater use of renewable energy sources when possible; (2) Minimized greenhouse gas (GHG) emissions; (3) The use of remedial technologies that do not require on-site or off-site waste disposal; (4) The use of remedial technologies that utilize recycled and/or reclaimed water sources; and (5) When appropriate, the use of remedial technologies or strategies that that do not restrict the potential future land use of a site.

GSR Assessment Tools

Qualitative and quantitative assessment tools are being developed to calculate sustainability metrics that consider all factors for GSR design and implementation. Qualitative assessment tools facilitate the screening of different remediation technologies based on potential impacts on the environment, society, and economics. Several public agencies have developed qualitative methods to preliminarily assess potential remedial technologies

Industry organizations and government agencies have recently started to develop quantitative GSR tools. For example, GSI Environmental Inc. developed the

Sustainable Remediation Tool (SRT) for the Air Force Center for Engineering and the Environment (AFCEE). The Microsoft Excel-based SRT estimates sustainability metrics for selected specific technologies, including carbon dioxide emissions, total energy consumed, change in resource services, technology cost, and safety/accident risk. SRT can be implemented as a preliminary Tier 1 analysis or a more detailed Tier 2 analysis based on user-defined, detailed, site-specific criteria.

SiteWise^{TM}, an Excel-based tool jointly developed by Battelle, US Navy and US Army Corps of Engineers, can be applied to select, design and implement GSR. It calculates the environmental footprint of remedial alternatives based on several quantifiable sustainability metrics, including: GHGs; energy usage; criteria air pollutants that include sulfur oxides (SOx), oxides of nitrogen (NOx), and particulate matter (PM); water usage; resource consumption; and accident risk.

EPA's Pollution Prevention Program developed a GHG Calculator tool to help quantify GHG reduction measures such as electricity conservation and water conservation.

Because there is no universally accepted way of calculating a carbon footprint, dozens of carbon calculators have become prevalent over the past few years, creating confusion and inaccurate information. It is recommended to use a life cycle assessment (LCA) to properly analyze carbon footprints and other impacts. LCA can provide a quantitative approach that provides an objective, scientific, and numerical basis for decision-making of GSR technologies. However, applications of LCA for contaminated sites are relatively scarce due mainly to their complex nature and a lack of training for environmental remediation professionals.

GSR Technologies

When analyzing potential technologies for a remediation program, the key principles and factors of GSR should be incorporated at all phases of site characterization, remediation, and monitoring. Technologies that encourage uncontrolled contaminant partitioning between media or those generate significant secondary wastes/effluents are not sustainable. Rather, technologies that destroy the contaminants, minimize energy input, and minimize air emissions and wastes, are preferred. In-situ systems are often attractive, as they typically minimize greenhouse gas emissions and limit disturbance to the ground surface and overlying soils.

As with traditional approaches, a single remediation technology often cannot cost-effectively address the technical challenges posed by contamination at a particular site, and multiple technologies may be sequentially or concurrently used for remediation. Further, technologies not typically considered sustainable may be combined with other technologies to develop multi-component remedial programs that are sustainable.

The duration of the remediation program can itself be a major governing factor in remediation system selection. The inclusion of power inputs derived from zero-emissions sources can also be beneficial.

GSR Challenges and Opportunities

Several challenges and opportunities exist in promoting GSR in practice, including a lack or absence of: (1) Education and training for stakeholders; (2) Guidance documents with clear and consistent definitions; (3) Standardized sustainability metrics and validated evaluation tools; (4) Well defined frameworks and processes to evaluate sustainability; (5) Well documented pilot studies/case studies involving sustainable remedies; (6) Incentives to adopt sustainable remedies; (7) Funding to support the research and development of sustainable approaches; and (8) Specific regulations requiring GSR.

CONCLUSION

This paper provides a chronology of environmental issues and evolution of regulations to manage wastes and contaminated sites. Technologies to characterize and remediate contaminated sites have also evolved. Risk-based, rapid technologies are still continuing to be innovated. The need for a holistic approach considering the net environmental impact based on activities involved in site characterization and remediation has led to emergence of new paradigm – green and sustainable remediation, which is rapidly being promoted by regulators, industry and other stakeholders. Challenges and opportunities still remain to develop efficient, simple, rapid and inexpensive techniques to characterize and remediate ever growing complicated contaminated site conditions.

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State of the Practice of MSE Wall Design for Highway Structures

Peter L. Anderson, P.E., M.ASCE[1], Robert A. Gladstone, P.E., M.ASCE[2], John E. Sankey, P.E., M.ASCE[3]

[1]The Reinforced Earth Company, 133 Park Street, North Reading, MA 01864; PH (978) 664-2830; FAX (978) 664-2831; Email: panderson@reinforcedearth.com

[2]Association for Metallically Stabilized Earth, P.O. Box 9142, McLean, VA 22102; PH (703) 749-3033; FAX (703) 749-3034; Email: bobgladstone@amsewalls.org

[3]The Reinforced Earth Company, 8614 Westwood Center Drive, Vienna, VA 22182; PH (703) 821-1175; FAX (703) 821-1815; Email: jsankey@reinforcedearth.com

ABSTRACT

The state of the practice of MSE wall design has become more complex as more and more systems, engineers and researchers have become involved in the practice. There are correct ways to design MSE walls, to apply traffic surcharge, to select design parameters and backfill, to assess service life, to address special design conditions such as bridge abutments, traffic barriers and earthquakes, and to select the wall design method itself. Yet the complexity persists, arising from policy changes and a multitude of design choices. This paper will discuss these and other areas of confusion and provide clarification regarding accepted, reliable methods of MSE design that have been proven in the field for more than forty years.

INTRODUCTION

At the invitation of the Federal Highway Administration (FHWA), Reinforced Earth® structures with their inextensible (steel) reinforcements were introduced in the United States in 1971. This new technology was quickly successful, both structurally and economically, giving rise to competing systems, a new industry and the generic name Mechanically Stabilized Earth (MSE). Design of MSE walls with inextensible reinforcements was, and still is, performed by assuming the MSE structure behaves as a rigid body, sizing it to resist external loads applied by the retained soil and by any surcharge, then verifying internal stability by checking reinforcement pullout and tensile rupture. This design method, derived from basic soil mechanics, is known as the Coherent Gravity Method (Anderson et al., 2010). In the 1970s, the Coherent Gravity Method was refined by several MSE-specific research studies to include a bilinear envelope of maximum reinforcement tension and a variable state of stress based on depth within the structure. From extensive usage, reinforcement pullout resistance parameters were developed for both ribbed reinforcing strips and welded wire mesh reinforcement and the behavior of steel-reinforced MSE structures became well understood and accepted.

The development of extensible (geosynthetic) reinforcements, in the late 1970s, necessitated use of the Tieback Wedge design method to account for differences in both internal stress distribution and deformation characteristics evident in MSE structures reinforced with extensible reinforcements. Confusion arose among engineers due to differences between the two design methods, giving rise to even more design methods, some of which were intended to work for both inextensible and extensible reinforcements. Meanwhile, the validity of the Coherent Gravity Method was being proven in tens of thousands of highway structures and it became the MSE structure design method either accepted or required by the majority of state departments of transportation (DOTs).

In 1975 the Federal Highway Administration promulgated its "Standard Specifications for Reinforced Earth Walls", later renamed for MSE walls, but this specification was abandoned in the early 1990s when the American Association of State Highway and Transportation Officials (AASHTO) created its specification of similar title. Although FHWA and AASHTO generally agree, they differ on some points, as seen by comparing training materials created by the FHWA's National Highway Institute (NHI, 2009) with design requirements in the AASHTO specifications.

In addition to differences between NHI and AASHTO, MSE designers have also had to deal with a transition from U.S. customary units to metric units and then back to U.S. units, as well as a mandated transition from Allowable Stress Design (ASD) to Load and Resistance Factor Design (LRFD). Today, MSE walls for highways are designed using the Coherent Gravity, the Simplified, and occasionally other methods, and according to numerous DOT specifications. There are still ASD and metric-unit designs ongoing for projects developed years ago, creating challenges for MSE wall designers and reviewers.

The practice of MSE wall design was once straightforward; today it can be confusing and complex. The following recommendations will help to minimize confusion and complexity:

- Continued and exclusive use of AASHTO specifications for MSE wall design,
- NHI courses should teach material consistent with AASHTO specifications,
- Use of the Coherent Gravity Method for inextensible reinforcements and the Simplified Method for extensible reinforcements.

This paper presents the state of the practice of MSE wall design for highway structures by discussing key aspects of MSE design and performance, the different reinforcement materials, and the reasons a different design method should be used for each reinforcement material.

DESIGN PLATFORM AND UNITS (LRFD vs. ASD)

The practice of MSE wall design is in an FHWA-mandated transition from Allowable Stress Design (ASD) to Load and Resistance Factor Design (LRFD). While some DOTs began implementing LRFD for MSE walls before the mandate, others still have not made that transition. Another transition is working through the backlog of projects designed in metric units. Taken together, these transitions require engineers to have daily familiarity with two design platforms (ASD, LRFD), two sets of units (metric, U.S.), multiple MSE design methods, and multiple state specifications.

DESIGN RESPONSIBILITY

Most DOTs are clear on who is responsible for each aspect of the design of MSE walls. The distribution of design responsibility is as follows:

External Stability. The owner (DOT), or the owner's geotechnical consultant, is responsible for the external stability of the proposed structure. The logic is simple: the owner is proposing to build the structure in the specified location and it is the owner's responsibility to investigate the feasibility of the proposed improvement, including the adequacy of the foundation soils to support the proposed structure. External stability analysis includes global stability of the structure, bearing capacity analysis of the foundation soils, and settlement analysis of the proposed structure.

An external stability analysis of an MSE structure is straightforward for a qualified geotechnical engineer and can follow the typical steps outlined below.

1. The foundation width for an MSE structure is taken equal to the soil reinforcement length, which is typically 0.7 times the height of the structure.
2. The height of the structure is taken from the top of leveling pad to the finished grade at the top of wall.
3. The reinforced soil mass may be modeled as a block, using a high cohesion value to force the failure surfaces being examined to be external to the structure. For example, design properties of $\gamma = 20$ kN/m^3, $\phi = 34°$ and $c = 70$ kN/m^2 may be used to model the reinforced soil mass in a global stability analysis.
4. The applied bearing pressure at the base of an MSE structure is approximately 135% of the overburden weight of soil and surcharge.
5. Factors of safety of 1.3 against global instability and 2.0 against bearing capacity failure are adequate for MSE walls (Anderson, 1991).

6. Settlement analysis is conducted by treating the MSE structure as a continuous strip footing of width equal to the strip length, with the applied bearing pressure as estimated in step 4.
7. Settlement at the wall face is approximately one-half of the value calculated in step 6.
8. MSE structures constructed with precast concrete facing panels (1.5 m x 1.5 m and 1.5 m x 3.0 m) and 20 mm thick bearing pads in the panel joints can tolerate large total settlements up to 300 mm, with up to 1% differential settlement (i.e., 300 mm in 30 m) without showing signs of distress in the wall facing.

Internal Stability. The MSE wall system supplier is responsible for internal stability design, including checking both pullout and rupture of the reinforcements. The supplier is also responsible for design of all wall system components, including the facing units, soil reinforcements, soil reinforcement connections to the facing units, bearing pads and joint-covering filter fabric. Wall suppliers also provide calculations that check sliding and overturning of the MSE gravity mass and determine the eccentricity of the structure and the applied bearing pressure at the base of the structure.

The state of the practice of MSE wall design is substantially about internal stability. Therefore, the following discussion reviews many aspects of internal stability design. It is useful to start at the beginning, by reviewing the basic mechanics of MSE structures.

BASIC MECHANICS OF MECHANICALLY STABILIZED EARTH

As explained by McKittrick (1978),

> "The basic mechanics of Reinforced Earth were well understood by Vidal and were explained in detail in his early publications. A simplification of these basic mechanics can be illustrated by Figure 1. As shown in Figure 1a, an axial load on a sample of granular material will result in lateral expansion in dense materials. Because of dilation, the lateral strain is more than one-half the axial strain. However, if inextensible horizontal reinforcing elements are placed within the soil mass, as shown in Figure 1b, these reinforcements will prevent lateral strain because of friction between the reinforcing elements and the soil, and the behavior will be as if a lateral restraining force or load had been imposed on the element. This equivalent lateral load on the soil element is equal to the earth pressure at rest ($K_o\sigma_v$). Each element of the soil mass is acted upon by a lateral stress equal $K_o\sigma_{v.}$ Therefore, as the vertical stresses increase, the horizontal restraining stresses or lateral forces also increase in direct proportion."

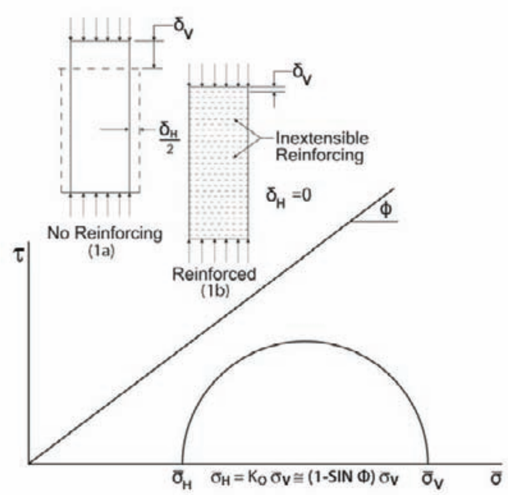

FIG. 1. Basic Mechanics of Reinforced Earth

Mechanically Stabilized Earth reinforced with inextensible (steel) reinforcements is, therefore, a composite material, combining the compressive and shear strengths of compacted granular fill with the tensile strength of horizontal, inextensible reinforcements.

A practical interpretation of McKittrick's explanation is that the larger the surcharge applied to an MSE structure, the stronger the composite MSE material becomes. Understanding this basic soil mechanics fact about MSE is crucial to the correct use of this composite construction material. With the addition of a facing system, MSE structures are well suited for use as retaining walls, bridge abutments and other, even more heavily loaded structures.

TRAFFIC SURCHARGE – *To Apply or Not To Apply?*

The design of MSE structures has moved from ASD, where many designers have developed an intuitive "feel" for the behavior of structures and the safety factors associated with their design, to LRFD, which uses non-intuitive factors derived through statistical analysis of past behavior. Statistical analysis can offer valuable insights. However, in the design of MSE structures, the resulting loss of intuition has led to a breakdown of logic regarding structure behavior when carrying a traffic surcharge. Therefore, it is worth repeating the fundamental concept of soil mechanics, stated above by McKittrick, that "… as the vertical stresses [on a sample of granular material] increase, the horizontal restraining stresses or lateral forces also increase in direct proportion." Rephrased in the context of surcharge loads on MSE structures, the larger the surcharge one applies to an MSE structure, the stronger the MSE structure becomes.

The AASHTO Specifications addressed this fundamental concept by a revision proposed in 2008 and issued in the 2009 Interim Specifications (AASHTO, 2009). AASHTO's explanation for this change is perfectly clear:

> "The application of live load surcharge with regard to pullout calculations for internal stability of MSE walls has been confusing, resulting in widely differing interpretations of the specifications regarding this issue. Based on a review of the historical development of this specification, the intent of the specification is to recommend that live load surcharge not be considered for pullout calculations. This applies to the calculation of reinforcement load (T_{max}) for evaluation of pullout stability, and the calculation of vertical stress for calculation of pullout resistance." (*2008 AASHTO Bridge Committee Agenda Item: 42; Subject: LRFD Bridge Design Specifications, Section 11, Article 11.10.6.2.1; Technical Committee T-15 Substructures and Retaining Walls*, unpublished.)

Therefore, two separate calculations of reinforcement tension (T_{max}) are required and engineers should be taught the following:

- When calculating T_{max} as part of performing reinforcement and connection rupture calculations, apply the surcharge to the MSE reinforced soil.
- When calculating T_{max} as part of performing pullout calculations, do not apply the surcharge to the MSE reinforced soil.

GRANULAR BACKFILL FOR MSE WALLS

The backfill used in MSE structures is critical to their overall performance. To meet performance requirements, MSE structure backfill is specified by AASHTO as granular material with a 100 mm maximum size and less than 15% fines (Table 1).

Table 1. Gradation Limits per AASHTO

U.S. Sieve Size	Percent Passing
100 mm	100
420 µm	0-60
75 µm	0-15

Additional requirements for plasticity index, internal friction angle, soundness and electrochemical properties are also given by AASHTO. Some DOTs vary the gradation limits or reduce the allowable fines content based on local material characteristics.

DESIGN PARAMETERS

Typically the design of an MSE structure is completed using assumed design parameters. The design is typically submitted, reviewed and approved before the contractor obtains approval of the select backfill that will be used to construct the MSE walls. The most common assumed design parameters are shown in Table 2.

Table 2. Assumed Design Parameters

Soil Type	Unit Weight	Friction Angle
Select Structure Backfill	20 kN/m^3	34°
Retained Fill	20 kN/m^3	30°
Foundation Soils	-----	30°

A 34° friction angle (ϕ) is typically assumed for the select backfill, as this is the maximum value permitted by the AASHTO specifications unless project-specific test data is provided. Use of this friction angle is a good choice for design of MSE walls because 34° is approximately the shear strength that will mobilize in the structure for most granular soils meeting the gradation requirements.

The actual material used to construct the structure will likely have somewhat different parameters. The unit weight may be higher or lower and the measured peak shear strength will likely be higher, often considerably higher than 34°. This is acceptable, since one of the purposes of using safety factors (or load and resistance factors) is to account for the uncertainties in the backfill properties. Use of peak shear strength in the design of structures should be avoided because peak shear strength is an intrinsic property that only develops if there is sufficient soil strain. Such strain is prevented by the soil reinforcements.

The mobilized shear strength that develops within a structure develops at strains less than those required to develop the peak shear strength. If too much deformation is allowed to occur, as may be the case if the design is based on peak shear strength, the shear strength of the soil will reduce to its residual value. The residual shear strength is likely much closer to 34° than it is to the peak shear strength. Therefore, use of 34° for design is a prudent choice, one that has been assumed in design and proven through four decades of MSE structure performance.

SERVICE LIFE

The service life of an MSE structure is defined as the period of time during which

- In terms of ASD, the tensile stress in the soil reinforcements is less than or equal to the allowable stress for the steel or,
- In terms of LRFD, the factored tensile resistance of the soil reinforcements is greater than or equal to the factored tensile load.

MSE retaining walls are routinely designed for a 75-year service life; those supporting bridges are typically designed for 100 years. The primary factor affecting the service life of an MSE structure is the long term durability of the reinforcements which, for inextensible (steel) reinforcement materials, is closely related to backfill electrochemical properties.

Research on buried galvanized steel, conducted by the National Bureau of Standards, Terre Armee Internationale (TAI), FHWA, and several state DOTs, confirms that the metal loss rates used in the design of MSE structures (Table 3) are conservative for steel soil reinforcements galvanized with 86 μm of zinc and buried in backfill meeting the electrochemical requirements shown in Table 4 (AMSE, 2006; Gladstone, et al., 2006). These loss rates and backfill electrochemical requirements have been codified in the AASHTO specifications (AASHTO, 2002; AASHTO, 2010).

Table 3. Metal Loss Rates

Material	Loss Rate
Zinc (first 2 years)	15 μm/yr
Zinc (subsequent years to depletion)	4 μm/yr
Carbon steel (after zinc depletion)	12 μm/yr

Table 4. Electrochemical Requirements

Property	Value
Resistivity	≥ 3000 ohm-cm (at saturation)
pH	5-10
Chlorides	< 100 ppm
Sulfates	< 200 ppm

The carbon steel loss rate in Table 3 is proportional to the loss of tensile strength for the sizes of strips and wires typically used as soil reinforcements in MSE structures. The minimum sizes of strips and wires that will assure validity of the metal loss model are discussed by Smith, et al. (1996). In general, according to Smith, small diameter wires (smaller than W10 for a 100 year service life and W7 for a 75 year service life) should not be used as the primary tensile members without additional data being developed on their long term tensile strength.

The loss rates in Table 3 determine the sacrificial thickness of steel that must be added to the load-carrying cross section to produce the design cross section. At the end of the service life, after 75 or 100 years of metal loss, the remaining steel will have a factor of safety of 1.8 against yield. In terms of LRFD, at the end of the service life the factored tensile resistance will be greater than or equal to the factored reinforcement tensile load.

DESIGN METHODS

Not surprisingly, MSE wall designers can become confused by the choices they must make. The two types of MSE wall reinforcements, inextensible (steel) and extensible (geosynthetic), behave differently. To obtain the required structure performance and service life, designers must understand reinforcement behavior and use the design method appropriate to each reinforcement type. The paragraphs below discuss design methods and reinforcement behaviors, match the design method to the reinforcement, and clearly show that the Coherent Gravity Method should be used for design of MSE walls with inextensible reinforcements and the Simplified Method should be used for MSE walls with extensible reinforcements. This is the state of the practice, with proper selection of the MSE design method being critical to successful design of MSE walls for highway structures.

Coherent Gravity Method. The Coherent Gravity Method was developed by postulating MSE structure behavior, observing actual structures, and interpreting observations in terms of the fundamentals of statics and soil mechanics (Anderson, et al., 2010). Three international symposia on soil reinforcement in 1978 and 1979, followed by publication of "Reinforced Earth Structures, Recommendations and Rules of the Art" (French Ministry of Transport, 1979), presented a substantial body of knowledge and defined the Coherent Gravity Method for the design of MSE structures. The method includes a bilinear envelope of maximum reinforcement tension, a state of stress varying with depth within the structure, high-pullout-resistance reinforcements (originally ribbed strips, with data becoming available later for welded wire mesh reinforcements), and minimal reinforcement movement and elongation. Design characteristics of the Coherent Gravity Method, as shown in Figure 2, are listed below.

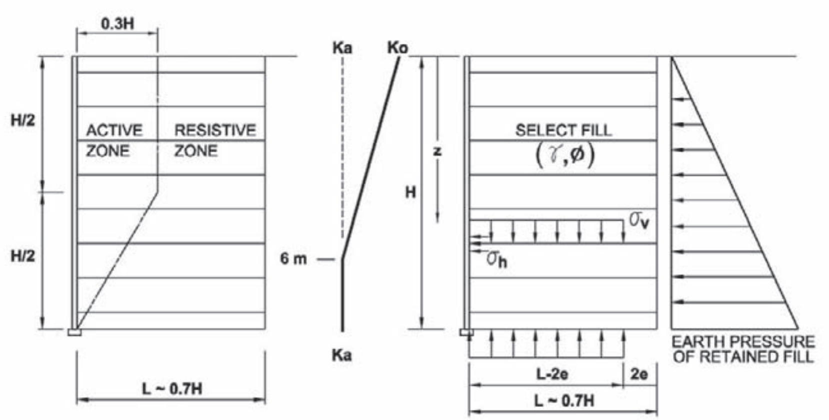

FIG. 2. Characteristics of the Coherent Gravity Method

- A rectangular cross section ("block") defined by the structure height, H, and the reinforcement length, L;
- Application of vertical and horizontal forces to the block, creating eccentric loading;
- A Meyerhof bearing pressure distribution at the base of the structure to determine foundation reactions and the repeated use of Meyerhof to determine the vertical earth pressure at each reinforcement level (Meyerhof, 1953);
- A state of stress decreasing from at rest (K_o) at the top of the structure to active (K_a) at a depth of 6 m and more;
- The resulting tensile forces in the reinforcements, determined from the horizontal earth pressure multiplied by the tributary area of the wall face restrained by the reinforcement at that level;
- The bilinear envelope of maximum reinforcement tension that separates the active from the resistive zone; and
- The inextensibility and high pullout resistance of the reinforcements which maintain the internal stability of the block.

The effects of externally applied loads on the reinforced soil mass, and the tendency of those loads to increase vertical and horizontal stresses within the structure, were confirmed by an extensive finite element study of 6 m and 10.5 m high walls (Anderson, et al., 1983). The Coherent Gravity Method was reported in its entirety, including worked example calculations, by Mitchell, et al., in NCHRP Report 290 (NCHRP, 1987).

Tieback Wedge Method. The Tieback Wedge Method was developed by Bell, et al. (1975) as an extension of the trial wedge method from traditional soil mechanics (Huntington, 1957), and has always been the appropriate design method for geosynthetic-reinforced MSE walls. In an MSE wall with geosynthetic reinforcements, the failure plane is assumed to develop along the Rankine rupture surface defined by a straight line oriented at an angle of $45+\phi/2$ from the horizontal and passing through the toe of the wall. Sufficient deformation is assumed to occur for an active earth pressure condition to exist from top to bottom of wall. The Rankine failure plane is not modified by inclusion of the extensible geosynthetic reinforcements. Therefore, reinforcement strain actually allows the failure plane to develop and the geosynthetic reinforcements, acting as tiebacks, restrain the active wedge from failing. This contrasts sharply with the Coherent Gravity Method, where the shape of the bilinear boundary between the active and resistive zones is based on the location of maximum reinforcement tension, the failure plane does not actually develop, the active wedge does not displace, and the inextensibility of the steel reinforcements prevents structure deformation.

Structure Stiffness Method. The Structure Stiffness Method was developed by Christopher, et al. (1990) based on instrumentation of full scale test walls and review of data reported in the literature from instrumented in-service walls. The Structure Stiffness Method is similar to the Tieback Wedge Method, however a bilinear envelope of maximum reinforcement tension is assumed for inextensible (steel) reinforcements and a Rankine failure plane angled at 45+ ϕ/2 from the horizontal is assumed for extensible (geosynthetic) reinforcements. The lateral earth pressure coefficient, K_r, is based on a complex formula that takes into account the global stiffness of the reinforcement, where the global stiffness is directly related to the area of tensile reinforcement times the reinforcement modulus of elasticity. Therefore, as the reinforcement density increases, both the global stiffness and the resulting coefficient of earth pressure, K_r increase. This method was not adopted by state DOTs and, therefore, does not appear in any AASHTO specifications. However, the method did lead to development of the earth pressure ratio K_r/K_a which is now used in the Simplified Method.

Simplified Method. The Simplified Method was developed from the Tieback Wedge Method to create a single design procedure applicable to MSE walls reinforced with either inextensible or extensible reinforcements. Instead of calculating the increase in internal vertical stress due to overturning at every inextensible reinforcement level, the Simplified Method approximates this stress increase by simply adding 0.2 γz to the soil overburden. However, no stress increase is used with extensible reinforcements. The Simplified Method uses the Coherent Gravity Method's bilinear envelope of maximum reinforcement tension for walls reinforced with inextensible reinforcements and the Rankine failure plane, inclined at 45+ϕ/2 from the horizontal, for extensible reinforcements. In general, the Simplified Method is the Tieback Wedge Method with K_r/K_a ratios adopted from development of the Structure Stiffness Method.

K_o-Stiffness Method. The K_o-Stiffness Method, developed by Allen, et al. (2001), is yet another method intended to be applicable to the design of MSE structures with either inextensible or extensible reinforcements. Similar to the structure stiffness method, the K_o-stiffness Method requires use of a complex equation to calculate the peak tension in each reinforcement layer. The components of this equation include a distribution factor, a local stiffness factor, a facing batter factor, a facing stiffness factor, and a global reinforcement stiffness factor, all as modifications to the at-rest earth pressure coefficient, K_o. Similar to the Structure Stiffness Method, this method has not been adopted by state DOTs and, therefore, does not appear in any AASHTO specifications.

COMPARING THE DESIGN METHODS

This is the state of the practice. Prior to development of the Simplified Method, the Coherent Gravity Method was used for design of MSE walls reinforced with inextensible (steel) reinforcements and the Tieback Wedge Method (now the Simplified Method) was used for design of MSE walls reinforced with extensible (geosynthetic) reinforcements. Both the Coherent Gravity Method and the Simplified Method are outlined in Section 11 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2010). To select the proper design method, one must understand reinforcement properties and behavior, so the following sections explain the behavior differences between inextensible and extensible reinforcements. This discussion clearly demonstrates why the Coherent Gravity Method should be used for design of MSE walls with inextensible reinforcements, and why the Simplified (Tieback Wedge) Method should be used for design of MSE walls with extensible reinforcements.

Differences in Reinforcement Behavior. Pullout tests on inextensible and extensible soil reinforcements begin the same, regardless of reinforcement type. The reinforcements are placed between layers of compacted soil in a pullout box and an overburden load is applied to the soil by an air bladder or mechanical means. The pullout force is applied to the leading end of the soil reinforcement and the pullout force and resulting displacement of the reinforcement are measured at frequent intervals. The pullout resistance of the reinforcement should be determined at a displacement of 20 mm (Christopher, et al., 1990).

The applied pullout force and resulting displacement of the reinforcement are measured simultaneously, but this is where the similarity between the reinforcement types ends. Because inextensible reinforcements experience virtually no elongation, displacement is measured at the leading end where the load is applied. For extensible reinforcements, however, displacement is measured at the trailing end, opposite from the end where the load is applied. This difference is necessary because significant elongation of the extensible reinforcement occurs, but by measuring displacement at the trailing end, deformation of the geosynthetic reinforcement is eliminated from the measurement. Recognizing this major difference in test protocol is fundamental to understanding why different design methods should be used for inextensible and extensible reinforcements.

Inextensible (Steel) Reinforcement. With inextensible reinforcements, the displacement at the leading end is nearly the same as the displacement at the free end because reinforcement strain is negligible. The friction developed between the reinforcement and the soil is determined for a leading edge displacement of 20 mm, and the transfer of load to the soil via friction is uniformly distributed over the full length of the reinforcement.

In an actual structure, the load is applied to the reinforcement by the soil within the active zone, which is trying to escape through the wall face. The magnitude of this earth pressure depends on the vertical stress and the coefficient of lateral earth pressure. Vertical stress is a function of the overburden pressure, which increases with depth in the structure, while the coefficient of lateral earth pressure varies from at rest (K_o) at the top of the structure to active (K_a) at a depth of 6 m and deeper. The horizontal earth pressure becomes tension in the reinforcements through the mechanism of friction.

The tension in the reinforcement is greatest at the line of maximum tension (Figure 3), and decreases gradually over the full reinforcement length until near the free end, where the tension decreases rapidly to zero. Significant tension is observed over the full length of the reinforcements.

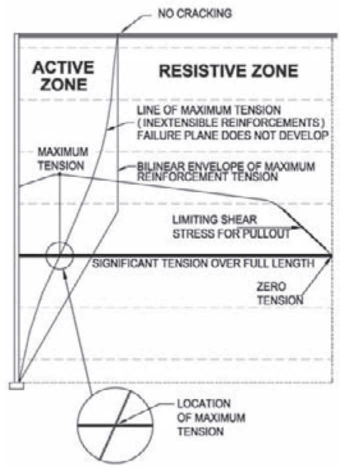

FIG. 3. Inextensible Reinforcement Tension

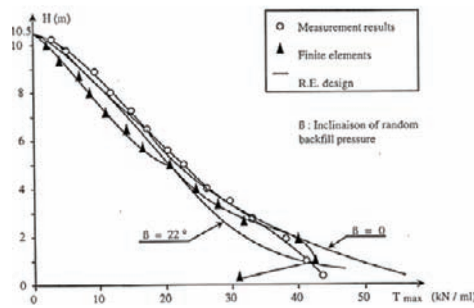

FIG. 4. Measured Loads in Reinforcement

Figure 4 is plotted from a full-scale test structure and the matching Finite Element Model (FEM) (Bastick, et al., 1993) and shows the maximum measured reinforcement tension, the maximum tension calculated by FEM, and the tension calculated by the Coherent Gravity method. Note that all are in close agreement, and that all three clearly show the overturning effect as curvature of the line near the bottom of structure. In the United States we use the $\beta = 0$ line to represent that the lateral earth pressure acts on the mass horizontally, not at an angle of inclination. This is the more conservative design line.

As was shown in Figure 3, the tension is distributed over nearly the entire reinforcement length. The compressive strength and shear strength of the soil combine to make the MSE structure behave as a rigid body. This rigid body behavior is also evident in Figure 4, in the magnitude of the maximum reinforcement tension, which increases toward the bottom of the wall. The increase is magnified by the overturning effect of the externally applied loads. This overturning effect, determined by the Meyerhof (1953) calculation, is considered by the Coherent Gravity Method but is not considered by the Simplified Method.

Extensible (Geosynthetic) Reinforcement. When performing pullout testing on extensible (geosynthetic) reinforcement, displacement must be measured at the trailing end of the reinforcement, not the end at which the load is applied. This is because extensible reinforcement undergoes significant strain under load, meaning when the leading edge has displaced 20 mm, the trailing end typically will not have displaced at all. Until trailing end displacement equals 20 mm, the length over which shear stresses have developed is unknown and load transfer from the soil to the reinforcement cannot be calculated over the full reinforcement length.

Terre Armee Internationale studied the difference in pullout resistance between inextensible and extensible reinforcements (Segrestin, et al., 1996). In this study, 40 mm wide ribbed steel strips and 100 mm wide polyester straps, in 6 m and 8 m lengths, were tested and compared in pullout. The steel strips are inextensible; the geosynthetic straps, though extensible, are among the least extensible geosynthetic soil reinforcements available. Figure 5 and Figure 6 show Segrestin's results for the 6 m long reinforcements; results for the 8 m long reinforcements were similar.

Figure 5 shows that, for a 40 mm leading edge displacement (δ), the trailing end of the inextensible (steel strip) reinforcement displaced 38 mm while the trailing end of the polyester strap had zero displacement. In addition, the displacement of the polyester strap at its mid-point was only 1.6 mm, indicating that virtually no load was induced on the back 3 m of this 6 m long reinforcement. Figure 6 shows the tensile load developed in the reinforcements during the pullout test. Note that when the leading edge of both reinforcements had displaced 40 mm ("δ_i = 40 mm"), the tensile load in the steel reinforcement was 38.4 kN, compared to the 22.5 kN load measured in the polyester reinforcement. The inextensible reinforcement carried 170 percent of the load with only 2.5 percent as much elongation as the extensible reinforcement.

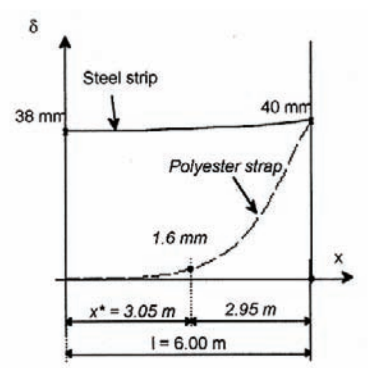

FIG. 5. Reinforcement Displacement

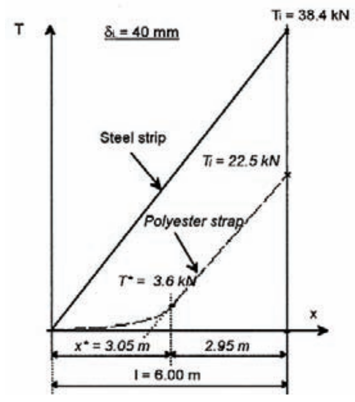

FIG. 6. Reinforcement Load

Taken together, Figures 5 and 6 show clearly that inextensible (steel) reinforcements work along their entire length. The friction which is mobilized along the inextensible reinforcement is uniform, but less than the limiting shear stress of the soil. The safety factor against pullout is due to the extra shear stress which can be mobilized along the full length of the reinforcement. Conversely, extensible (geosynthetic) reinforcements make use of only the minimum adherence length necessary to transfer the load to the soil. The friction which is mobilized along this length is equal to the limiting shear stress of the soil. The extra reinforcement length, which remains available but is not mobilized, provides the safety factor in pullout.

Due to the extensibility of geosynthetic soil reinforcements, the reinforcement will deflect at the failure plane as shown in Figure 7. Tension in the reinforcement will be greatest along the failure plane and will decrease rapidly behind the failure plane, based on the limiting shear stress of the soil (Figure 8, Carrubba, et al., 1999). As was seen in Figure 6 and confirmed in Figure 8, pullout tests indicate the tension in extensible reinforcements may reduce to zero a short distance beyond the failure plane, depending on soil-reinforcement friction and reinforcement extensibility.

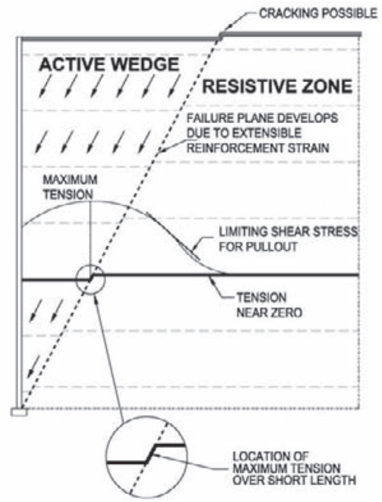

FIG. 7. Extensible Reinforcement

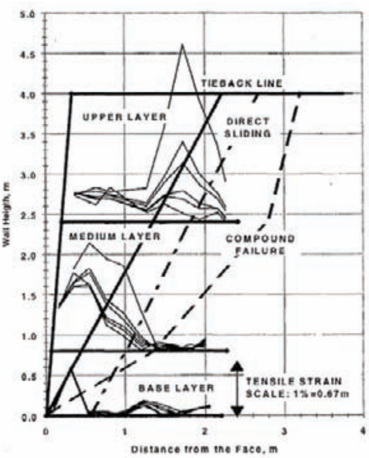

FIG. 8. Extensible Reinforcement Tension

This analysis shows that extensible reinforcements, including polyester straps, are not mobilized over their full length, and confirms what was found from the monitoring of actual structures reinforced with geosynthetic soil reinforcements (Simac, et al., 1990; Carrubba, et al., 1999) and from finite element studies (Ho, et al., 1993), especially at the bottom of structures. These observations mean that MSE structures reinforced with extensible (geosynthetic) reinforcements do not behave as rigid bodies (coherent gravity structures), while MSE structures with inextensible (steel) reinforcements do behave as coherent gravity structures.

IDENTIFY THE CORRECT DESIGN METHOD TO USE

In the current state of the practice of MSE wall design for highway structures, the AASHTO specifications include two design methods and two reinforcement types. Selecting the proper reinforcement system, then correctly applying the appropriate design method, is critical to achieving good structure performance and service life. The following statements summarize this thought process:

Coherent Gravity for Inextensible (Steel) Reinforcements. Inextensible (steel) MSE reinforcements are under tension over their full length, forming a coherent gravity mass. The measured reinforcement tensions clearly indicate an overturning effect consistent with Meyerhof (1953). The Coherent Gravity Method includes this overturning effect and predicts the measured tensions reasonably well. Therefore, the Coherent Gravity Method should be used for design of MSE walls reinforced with inextensible (steel) reinforcements.

Simplified (Tieback Wedge) for Extensible (Geosynthetic) Reinforcements. Extensible (geosynthetic) reinforcements are not under tension over their full length so an extensibly-reinforced MSE structure is not a coherent gravity mass. The Tieback Wedge Method is an accepted method for design of structures reinforced with extensible reinforcements and the Simplified Method is an MSE-specific version of the Tieback Wedge Method. Therefore, the Simplified Method should be used for design of MSE walls with extensible (geosynthetic) reinforcements.

SEISMIC DESIGN – *AASHTO's New No-Analysis Provision*

At the 2011 meeting of the AASHTO Subcommittee on Bridges and Structures, state and federal bridge engineers agreed that seismic analysis is not required for MSE walls which are ≤ 9.1 m high and subject to a design acceleration ≤ 0.4g. For taller walls, and for walls potentially subject to higher accelerations, seismic design in accordance with established (AASHTO) methods will still be required. This change in the state of the practice exempts most highway structures from requiring a seismic design.

TRAFFIC BARRIER DESIGN – *Confirmation of TAI Loads, NHI Recommends Higher Loads*

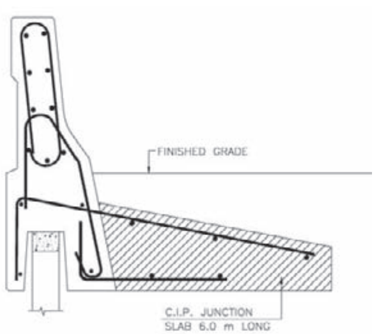

FIG. 9. Precast Concrete Traffic Barrier

Concrete safety barriers have been constructed on MSE walls in the United States since the early 1980s. Wall-mounted barriers were developed in France and crash tested in 1982 by Terre Armée Internationale (TAI, 1982). Hundreds of miles of both cast-in-place and precast barriers are in service and performing successfully throughout the United States and around the world. The typical cross section consists of the project-specific barrier shape and a nominally horizontal moment slab (Figure 9).

Safety barriers and their supporting MSE walls are designed using a pseudo-static design method developed nearly 30 years ago (Anderson, et al., 2008; TAI, 1982). The barrier and MSE wall are designed for an impact load of 45 kN, distributed over 1.5 m of wall for checking reinforcement tension and distributed over 6 m of wall for checking soil reinforcement pullout. For reinforcement pullout to occur, at least a 6 m length of both wall and barrier would need to move out as a unit. Figure 9 shows a typical precast concrete barrier and moment slab designed by the pseudo-static design method. This or similar barrier designs have been constructed atop thousands of MSE retaining walls since 1985. Their performance has been excellent.

Since 1994, the AASHTO Specifications for the design of MSE walls has included the pseudo-static design method discussed above.

The National Cooperative Highway Research Program (NCHRP) Project 22-20, *Design of Roadside Barrier Systems Placed on MSE Retaining Walls*, was begun in July 2004 with the goal of developing standardized procedures for MSE wall barrier design. The final report (same title, published as NCHRP Report 663) (NCHRP 2011), confirmed the validity of the TAI pseudo-static design load used since 1982 to size traffic barriers and design MSE walls subject to vehicular impact. Although the NCHRP research shows that the design method and the pseudo-static load specified by AASHTO are valid, NHI uses a significantly larger design load to check pullout of the top reinforcement layer. This unexplained difference is causing considerable confusion for DOTs, which are now uncertain whether to use the NCHRP crash test-validated load or the NHI-recommended load.

STRUCTURES SUPPORTING ABUTMENTS

The first Reinforced Earth bridge abutments were constructed in France in 1969 and in the United States in 1974. These MSE structures were "true" abutments where the bridge beams rested on a spread footing beam seat bearing directly on the reinforced backfill (Figure 10). Mixed abutments, with piles supporting the beam seat (Figure 11), were developed later. One of the first true abutments in France carried a remarkable 76 m span; spans up to 72 m have been constructed in the U.S.

After a long period of building only a few dozen MSE abutments per year, usage has increased rapidly since the late 1990s, as owners and engineers have become more familiar with and have developed greater confidence in this technology. Approximately 600 MSE abutments (300 bridges) are now constructed annually in the U.S., with 25% being true abutments supported directly on the reinforced soil (Anderson, et al., 2005). The main reasons for this growing usage are more rapid construction and lower cost of MSE abutments as compared to conventional concrete abutments.

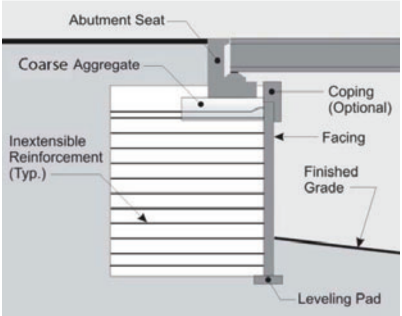

FIG. 10. MSE True Abutment

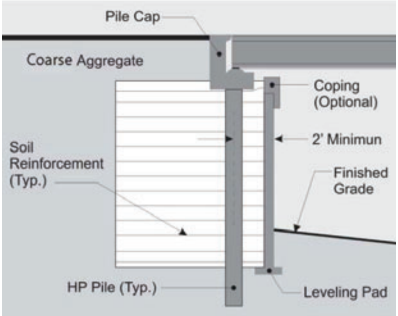

FIG. 11. MSE Mixed Abutment

MSE abutments – both true and mixed – should be designed using the Coherent Gravity method, discussed above and specified by AASHTO (2010), because the Coherent Gravity Method accounts for externally-applied loads and structure eccentricity.

BACK TO BACK WALLS

Back to back walls are commonly used as approach structures to vehicular and pedestrian bridges and for support of railways. Back to back walls consist of MSE structures with two wall faces separated by the width of the embankment. In some cases, the width of the embankment is narrow and the soil reinforcements from each face share the MSE backfill in the middle for pullout resistance. This is acceptable since the shear forces are in opposite directions.

Back to back walls with an aspect ratio (width of embankment divided by height) as small as 0.6 are safe and used in practice for many applications. The NHI, however, recommends that the minimum safe aspect ratio should be 1.1. This is not logical, as explained below:

The recommended aspect ratio for a standard MSE structure that retains a soil embankment is 0.7, but such an MSE structure may have an aspect ratio less than 0.7 if justified by calculation. Logically, therefore, if a structure with an aspect ratio of 0.7 can retain the soil load of an embankment, then a structure with the same aspect ratio certainly can stand alone, without an embankment to retain. The only load conceivably acting on a back to back structure is the wind load that may act on one face or the other. This load is far less than the load of a soil embankment. Clearly, the NHI-recommended minimum aspect ratio of 1.1 is incorrect.

CONCLUSIONS

The state of the practice of MSE wall design for highways has become more complex as more and more systems, engineers and researchers have become involved. There are correct ways to design MSE walls, to apply traffic surcharge, to select design parameters and backfill, to assess service life, to address special design conditions such as bridge abutments, traffic barriers and earthquakes, and to select the wall design method itself. Yet the complexity persists, arising from policy changes and from the multitude of conflicting design choices.

Designers are forced to be familiar with the Coherent Gravity, the Simplified (Tieback Wedge) and occasionally other methods, with the ASD and LRFD design platforms, with metric and U.S. customary units, with design guidelines issued by AASHTO and NHI, and with numerous, differing governmental specifications. Designers are understandably confused by the conflicting guidance and clarification is needed. This clarification can be achieved through the following steps:

- Continued and exclusive use of AASHTO specifications for MSE wall design
- NHI courses should teach material consistent with AASHTO
- Require the Coherent Gravity Method for design of MSE walls with inextensible reinforcements
- Require the Simplified Method for design of MSE walls with extensible reinforcements

Today's challenge is to restore basic principles to the design of MSE walls for highways to ensure that Mechanically Stabilized Earth structures continue to meet the structural and economic needs of transportation infrastructure for years to come.

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The Business of Geotechnical and Geoenvironmental Engineering

Rudolph Bonaparte, M. ASCE, Ph.D., P.E., D.GE, NAE[1]

[1]President and CEO, Geosyntec Consultants, Inc., 2002 Summit Boulevard, Suite 885, Atlanta, GA 30319; rbonaparte@geosyntec.com

ABSTRACT: This paper presents the author's perspectives on the state of the industry with respect to U.S. private-sector businesses providing geotechnical and geoenvironmental services to clients. The paper discusses: the history and evolution of the business sector; size and attributes of the sector; business models and practice characteristics of companies in the sector; and operational, management, and personnel priorities and challenges of these firms. The paper also addresses career considerations for personnel in this business sector, and the topics of clients and competition, risk management and contractual liability, mergers and acquisitions, and the author's thoughts on future directions for the business sector over the next 10 to 20 years. Sustained business success is being achieved by both large and small firms practicing geoengineering that can recruit and retain great people, provide superior work deliverables and solutions to their clients, perform their work efficiently and at a reasonable cost, and navigate both the competitive and risk management landscapes. A significant and exciting challenge for the senior management teams of these firms now and in the future is to achieve and integrate excellence in technical practice development, project management performance, personnel development and retention, risk and quality management, financial performance, and equity and ownership management to continually improve their firms and, in so doing, improve the profession.

1. INTRODUCTION

This state of practice (SOP) paper has been prepared at the invitation of the organizers of the GeoCongress 2012. The organizers requested that the author address "the business of geotechnics." This topic is indeed very broad and a paper on it could focus on any number of different subject areas, including, for example, any of the business sectors listed in Table 1, all of which routinely employ geotechnical and geoenvironmental engineers. It would be impossible in a single paper to address all of these different parts of our profession.

Table 1. Business Sectors in U.S. Economy Employing Geotechnical and Geoenvironmental Engineers

Sector	Example Participants
State and Municipal DOTs	NYSDOT, GDOT, CALTRANS
State Municipal Water Agencies	California Dept. of Water Resources
Federal Agencies	USGS, USACOE, USDOT, USBR
Port Authorities	Port Authority of NY/NJ, Port of Houston Authority
Power Utilities	TVA, PG&E, Duke, Constellation
Industry (Oil & Gas, Mining, Power)	ExxonMobil, Freeport-McMoRan, PG&E
EPCs (Engineer-Procure-Constructor)	Bechtel, Fluor, Shaw
Heavy Civil Contractors	Kiewit, Granite, Flatiron
“Geoengineering” Firms	Schnabel Engineering, Ninyo & Moore, Mueser Rutledge
“Multi-Service” E&C Firms	Haley & Aldrich, Geosyntec, Golder, Langan
“Full-Service” E&C Firms	CH2M Hill, URS, AECOM
Drillers/In-situ Testers/Geophysics	Gregg, Fugro, ConeTec,
Geotechnical Labs	TRI, GeoTesting Express, Excel Geotechnical, JTL
Geotechnical Lab Equipment Suppliers	Trautwein, Geoprobe, Geocomp, Hogentogler
Geotechnical Software Developers	Plaxis, gINT, RockWare, Geo-Slope, Itasca, Ensoft, Pile Dynamics
Geotechnical Instrumentation Suppliers	Geokon, RST, GRL, Slope Indicator, ITM
Public/Private University Faculty	Georgia Tech, Virginia Tech, U.C. Berkeley, Illinois
Public/Private Research Institutions	Sandia Laboratory, Battelle
Specialty Geotechnical and Geoenvironmental Contractors	Hayward Baker, Bauer, RECON, Moretrench, Menard, Nicholson, Schnabel Foundations
Foundation Material/System Suppliers	Geopier, Monotube, J.D. Fields,
Foundation Contractors	Malcolm Drilling, McKinney, Goettle, Morris Shea
Load/Non-Destructive Testing	GRL, Olson Engineering, LoadTest
Geosynthetics Manufacturers	Tensar Int., GSE, Huesker, CETCO
Geosynthetics Installers	Nilex, Plastic Fusion, American Wick Drain
Geoengineering Professional and Industry Organizations	ASCE Geo-Institute, ASFE, Deep Foundations Institute, Geosynthetic Materials Institute

Given the breadth of the industry, the selected focus of the paper is on private-sector U.S. businesses that are engaged in the provision of geotechnical and geoenvironmental engineering services: (i) as their principal service offering (i.e., “geoengineering firms”); (ii) as a significant capability within a large “full-service” engineering and consulting (E&C) firm; or (iii) as a business with a range of service offerings falling somewhere between the previous two categories (i.e., “multi-service” E&C firms). The rationale for choosing this focus is two-fold. First, private-sector engineering firms represent the largest employer for geoengineering professionals in the U.S., as reflected by the membership of the American Society of Civil Engineers (ASCE) Geo-Institute (Figure 1). Second, the author has for the past 25 years worked in a private-sector geoengineering firm that today numbers about 900 employees; hence it is the business sector he knows best.

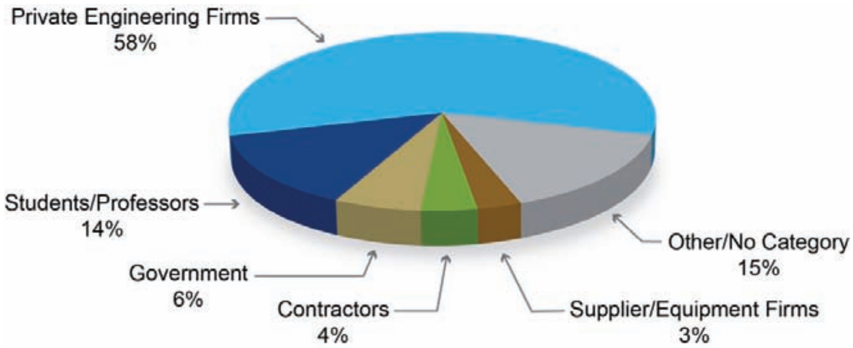

FIG. 1. Approximate Percentages of 10,750 ASCE Geo-Institute Members in Various Employment Categories. (Note: Data provided by the Geo-Institute in June 2011.)

The scope of the paper includes a discussion of: the history and evolution of the business sector; size and attributes of the sector; business models and practice characteristics of companies in the sector; and operational, management, and personnel priorities and challenges of these firms. The paper also addresses: career considerations for personnel in this business sector; the topics of clients and competition, risk management and contractual liability, and mergers and acquisitions; and the author's thoughts on future directions for the business sector over the next 10 to 20 years.

In the remainder of this paper, the term geoengineer will be used as a shorthand reference for "geotechnical and geoenvironmental engineer." It is interesting, and perhaps alarming, to note how over the past few years the term geoengineer has been usurped by the global warming/climate change community. For example, the U.S. National Research Council defines geoengineering as "deliberate, large-scale manipulations of the Earth's environment intended to offset some of the harmful consequences of GHG emissions" (NRC, 2011). A June 2011 Google© web search reveals that only 5 of the first 100 search results for "geoengineering" relate to geotechnical and geoenvironmental engineering. Notwithstanding this unfortunate usurpation, the author will continue to use the term geoengineer as defined at the beginning of the paragraph.

2. HISTORICAL PERSPECTIVE

Early Years: The private-sector practice of geoengineering emerged in the late 1930's, a time when the field of "soil mechanics and foundation engineering" was first gaining acceptance in the United States. As described by D'Appolonia (1983):

"The engineering of foundations and earthworks as practiced today by private practitioners dates back only to the late 1930s, when it was called "soil mechanics

and foundation engineering." The name was changed in 1972 to "geotechnical engineering" but – by no matter what name it may be known – the practice dates back to antiquity: a practice of empiricisms, generally quite parochial, with skills developed through the hard knocks of experience, painstakingly learned over long periods of time. The practice prior to 1940 had few unifying principles other than those agreed to by successful practitioners."

The situation in the late 1930's with respect to the acceptance of soil mechanics and foundation engineering as a rigorous discipline to be incorporated into the private practice of engineering was described vividly by Peck (1993):

"By no means, however, was soil mechanics universally accepted. Many prominent engineers, perhaps turned off by the mathematical formulations and unconvinced that the mechanical and hydraulic properties of such variable natural materials as soils could be reliably evaluated by tests, lost few opportunities to belittle the importance or practicality of the subject. On October 7, 1937, the American Society of Civil Engineers held a symposium on soil mechanics in Boston. Four papers were presented, one by Terzaghi on measurements of settlements of structures in Europe, to which Terzaghi had returned in 1928. Two of the other papers were by representatives of the Corps of Engineers: Spencer Buchanan's paper on "Levees in the Lower Mississippi Valley" and Ben Hough's paper on "Stability of Embankment Foundations". Spencer's paper was a thorough discussion of the side slopes, foundations, and control of seepage of the levee system for which the Corps of Engineers had long been responsible. It indicated the benefits of applying the principles of stability analysis and of soil mechanics in general. Characteristic of the skepticism regarding the subject was a discussion by Mr. A. Streiff, a Vice President of the Ambursen Engineering Corporation, designers and builders of buttress dams. A few quotations from his discussion characterized the feelings of the many engineers unimpressed by the developments. "The method of computing these slopes, described by Mr. Buchanan, does not seem to the writer to offer any better guaranty for their stability than that obtainable by older methods. Unfortunately, the great advances in soil mechanics are confined to the laboratory. The writer disagrees with Mr. Buchanan and feels that, for the present at least, the application of soil mechanics has caused no visible progress in: (1) the art of foundation design; and (2) the methods for computing soil stability. The first has always been quite adequate, and computation methods are as approximate as they ever were… ***Soil mechanics, at least to the present, has not visibly enriched the 'toolbox' of the practicing engineer."*** (Note: bold font added by author.)

Notwithstanding the early skepticism, the discipline of soil mechanics and foundation engineering received, over time, broader and broader acceptance, as reflected in: the establishment of teaching and research programs within the civil engineering departments of many universities; the acceptance and adoption of the new discipline by the U.S. Army Corps of Engineers, U.S. Bureau of Reclamation, and other federal, state, and municipal agencies; and the founding of private-sector firms practicing in the discipline. Table 2 provides a representative listing of some of the notable firms (and the date and location of their founding) that were early participants in the practice.

Table 2. Representative Private-Sector Firms that were Early Participants in the Practice of Soil Mechanics and Foundation Engineering (list stops at 1970)

Firm	Founding Metro Location	Year Founded
Mueser Rutledge	New York, NY	1910
Dames & Moore	Los Angeles, CA	1938
Tibbets-Abbett-McCarthy-Stratton	New York, NY	1942
Law Engineering	Atlanta, GA	1946
Soil Testing Services	Chicago, IL	1948
Woodward-Clyde	Oakland, CA	1950
Leroy Crandall	Los Angeles, CA	1954
Shannon & Wilson	Seattle, WA	1954
Greer & McClelland	Houston, TX	1955
D'Appolonia	Pittsburgh, PA	1956
Jacobs Associates	San Francisco, CA	1956
Haley & Aldrich	Boston, MA	1957
Harding Associates	San Francisco, CA	1959
Ardaman & Associates	Orlando, FL	1959
Golder Associates	Toronto, Canada	1960
Kleinfelder	San Francisco, CA	1961
Soil & Material Engrs	Detroit, MI	1964
Goldberg Zoino	Boston, MA	1964
GEI Consultants	Boston, MA	1970
Langan	New York, NY	1970
Earth Tech	Los Angeles, CA	1970

In the early years of private-sector practice, geoengineering firms primarily subcontracted to the owner's architect, engineer, or contractor who held the prime contract with the owner. The role of the geoengineer involved investigating site conditions, performing laboratory soil and rock testing, and developing design recommendations for foundation systems, excavation bracing and dewatering systems, and other geotechnical features of the project. The geoengineer's role evolved over time to also include construction "inspection" and material testing services.

Compared to today, it was uncommon for large engineering and construction firms designing buildings, industrial facilities, and public infrastructure to have their own in-house geoengineering staff. When geoengineering expertise was needed, the large engineers and constructors most often subcontracted to the firms in Table 2, and other like them.

1960's and 1970's: Public infrastructure programs, industrial expansion, and commercial and residential development in the decades following World War II provided opportunities for the growth and evolution of the geoengineering industry. Major initiatives included: (i) start of the U.S. interstate highway system, spurred by the Federal-Aid Highway Act (1956) and establishment of the U.S. Department of

Transportation; (ii) expansion of the U.S. water/wastewater storage, supply, and treatment infrastructure, spurred by the Clean Water Act (1972) and other federal legislation; (iii) development of 65 nuclear power plants in the U.S. between 1956 and the 1970's, spurred by the Atomic Energy Act (1954) and establishment of the Nuclear Regulatory Commission (NRC); (iv) development of mass transit systems in major urban centers around the country, such as the Washington Metro system and the Bay Area Rapid Transit system; and (v) industrial, commercial, and residential expansion around the nation, including major expansion of offshore oil and gas facilities in the Gulf of Mexico and construction of the Trans Alaska pipeline system. Further contributing to geoengineering opportunities was the continuance of older infrastructure development initiatives that remained active including western U.S. water storage, conveyance, and supply system expansion; flood control and navigation lock and dam system construction on major U.S. inland waterways, most notably the Mississippi River and its major tributaries (e.g., Missouri, Ohio, and Tennessee Rivers); continuing development of port and harbor facilities along the nation's coasts; and, building construction across the country, particularly in major urban centers. All of these activities provided opportunities for the growth and evolution of the private-sector practice of geoengineering. In parallel, geoengineering technology continued to advance at a rapid pace, as documented in the many classic papers published in the 1960's and 1970's in the ASCE Journal of Soil Mechanics and Foundation Engineering (becoming the Journal of Geotechnical Engineering in 1974).

By the 1970s, a number of geoengineering firms had grown to become substantial business concerns, several at a national scale. This is reflected in the list assembled by the industry news magazine Engineering News Record of the "Top 500 Design Firms" (i.e., largest E&C firms) in the U.S. for 1975. In that year, the following firms from Table 2 were amongst the 100 largest engineering firms in the country: Dames & Moore (#6), Woodward-Clyde (#16), Law Engineering (#50), McClelland Engineers (#58), D'Appolonia (#86), Fugro (#91), and Soil Testing Services (#98).

In 1969, leaders from ten U.S. geoengineering firms established the Associated Soil and Foundation Engineers (ASFE), which today exists as ASFE/The Geoprofessional Business Association. The concern of these firm leaders was the rising incidence of professional liability claims and litigation against their firms. The problem of claims liability had become so acute that insurance companies were refusing to provide professional liability (i.e., "errors and omissions") insurance to the firms to the point that the health and wellbeing of the firms were at risk. In response, the ASFE member firms established Terra Insurance Company as a captive of the company's policyholders to specifically serve the geoengineering industry. In the ensuing years, the ASFE member firms worked to reduce claims exposure by developing improved contracts, developing professional practice guidelines and training for its members, developing an organizational peer review system, and other measures.

In 1983, Dr. Elio D'Appolonia presented a memorial lecture sanctioned by ASFE, titled *Reflections on the Growth and Changes of the Private Practice of Geotechnical Engineering*. His paper describes the development and evolution of geoengineering business practices and challenges up through the 1960's and 1970's. In preparing his paper, he interviewed representatives of 25 ASFE member companies representing some of the larger geoengineering firms of the day. The paper contains a number of observations, cited below, which further help to illuminate the evolution of the geoengineering profession during the 1960's and 1970's.

- *"I believe the private practice of geotechnical engineering evolved from the early 1940's. The growth has been not only in numbers that have swelled the private practice from a handful of firms some four decades ago to more than 700 today."* The information presented by D'Appolonia indicates that much of this growth occurred during the 1960's and 1970's. He came up with his estimate of the number of geoengineering firms from review of ASCE and ASFE records and the telephone directories from major metropolitan areas. Interestingly, in 1983, ASFE, which is comprised mostly of geoengineering firms, had roughly 300 members. Today, ASFE has roughly the same number of members.
- *"During the 60's, the private practice of geotechnical engineering came into its own with the impetus of large-scale prototype testing. A few instances are: load transfer from piles and drilled shafts; the monitoring of slopes using inclinometers; the monitoring of excess pore water pressure of compressible soils loaded by fill; the settlement of industrial facilities placed on deep deposits of loose to medium dense granular soils…Data on the performance of completed facilities and large-scale tests were gathered, interpreted and reported in the literature for a significant number of cases contributing to the advance of the profession."*
- *"The development and growth of peripheral engineering services from our geotechnical base follows patterns that are consistent with the directions of construction activities since 1940….Particularly noteworthy is the increase in services related to earth science investigations and earthquake engineering in the 1960's during a time of environmental awakening by the nation as a whole."* Of the 25 firms directly interviewed for his paper, D'Appolonia found that all provided geotechnical engineering services, more than half provided services related to the earth sciences and earthquake engineering, and 40, 32, and 20 percent provided environmental engineering, surveying, and offshore services, respectively. All offered geotechnical laboratory testing services and 60 percent offered subsurface drilling and sampling services.
- *"Undoubtedly, our involvement in field operations and close ties with construction led to numerous claims and suits in the late 60's and early 70's…In the late 60's, the situation became so serious that engineering firms shied away from construction supervision and inspection. A litigious influence so permeated the building industry that we no longer spoke of construction inspection, or construction control. Rather, we called the critical activity "monitoring and observation".*
- *"You may recall that in the late 60's insurance companies would no longer insure geotechnical firms because of the high risk, particularly from third-party suits arising from accidents during construction. This action forced geotechnical firms to band together to form their own insurance company*

[Terra Insurance, in 1969]…*The incidence of claims has greatly diminished, particularly since the mid 70's."*

1980's to Present: By the late 1970's to early 1980's, a number of technology, practice, and market forces were either fully at play or beginning to emerge that greatly affected the direction of private sector geoengineering firms. These forces included:

Research and Development, New Technology, and New Publications: The amount of research and development (R&D) being conducted in the geoengineering discipline has been steadily increasing since the inception of the profession, with a seeming proliferation of activity over the last three decades. This has led to the ongoing introduction of new geoengineering theories, analysis and design methods, laboratory and field methods, and products and technologies across virtually all areas of the profession. The amount of geoengineering information being generated on an annual basis today is at least several times larger than the amount of information being generated in the late 1970's. This is illustrated by the fact that today, there are many more journals and magazines, conference proceedings, university reports, books, web-based resources, and other types of publications and information dedicated to the geoengineering discipline than there were in the late 1970's. Table 3 illustrates this point by presenting a representative list of English-language geoengineering journals, periodicals, and magazines available in 1975 and another list with additional geoengineering publications that have become available since the late 1970's. The practicing geoengineer today has many more tools and resources available to utilize in his/her practice than available 30 to 40 years ago. However, with the abundance of these tools and resources, the geoengineer today has the very real and significant challenge of keeping informed about recent developments and the even greater challenge of building experience and proficiency, and understanding the standard of care, across the practice range of profession.

Geotechnical Tool Box: The geoengineering R&D described above has resulted in the steady introduction of new and enhanced tools available to the geoengineering practitioner. Many of these tools have found wide use in the profession. These include, as examples, in-situ site investigation methods (e.g., cone penetrometer, pressuremeter), field instrumentation methods (e.g., slope inclinometers, vibrating wire piezometers), field testing techniques (e.g., pile driving analyzers, sealed double-ring infiltrometers), laboratory testing methods (e.g., automated triaxial shear strength tests, cyclic simple direct shear tests), and computer programs for a variety of geotechnical analyses, including the emergence of numerical tools such as the finite element and finite difference methods, and automated date acquisition, storage, analysis, and presentation tools.

Table 3. Growth in English-Language Geotechnical and Geoenvironmental Journals, Periodicals and Magazines from 1975 to 2011 (Note: list of publications is not exhaustive)

Publications Available in 1975	Additional Publications in 2011
Geotechnique (1948)	International Journal for Numerical and Analytical Methods in Geomechanics (1977)
Clays and Clay Minerals (1953)	ASTM Geotechnical Testing Journal (1978)
ASCE JSMFE (1956)	Soil Dynamics & Earthquake Engineering (1981)
Earth (1956)	International Journal of Integrated Waste Management (1981)
Canadian Geotechnical Journal (1963)	Geotechnical & Geological Engineering (1983)
Engineering Geology (1965)	Geotextiles and Geomembranes (1983)
Journal of Engineering, Geology & Hydrogeology (1967)	Geosynthetics (1983)
	Earthquake Spectra (1984)
	Geotechnical News (1984)
	Int. Journal of Engineering Geology and the Environment (1984)
	Computers and Geotechnics (1985)
	Deep Foundations (1993)
	Geosynthetics International (1994)
	Geo-Strata (2000)
	Natural Hazards Review (2000)
	Int. Journal of Geomechanics (2001)
	Landslides (2004)
	Geomechanics & Geoengineering (2006)
	Acta Geotechnica (2006)
	Int. Journal of Geoengineering Case Studies (2007)
	International Journal of Geotechnical Engineering (2007)

Geotechnical Specialization: In part as a response to the rapid growth in tools, technology, and information available to the geoengineering practitioner, and in part as a result of increasing project complexity and marketplace competition, firms developed and became recognized for specialized capabilities which helped to define their client and project niches. These specialties include, as examples, deep foundations and excavations in urban settings; bridge foundations and earth retaining structures; tunneling and underground construction; geotechnical earthquake engineering; geological hazard assessment and mitigation; offshore geotechnics and foundations for offshore structures; dams, reservoirs and canals; flood control structures, including levees; waterfront and marine structures; waste geotechnics and landfill engineering; contaminated site remediation and brownfields; and mine geotechnics and mining engineering.

Environmental Regulations: A number of federal environmental regulations were promulgated that we now know, with the benefit of hindsight, had a profound impact on the profession in the emergence of the practice of geoenvironmental engineering. The U.S. Environmental Protection Agency was established in 1970. A body of regulations followed that transformed the way society dealt with anthropogenic environmental impacts to water, air, and land. These regulations included the Clean Air Act (1970), Clean Water Act (1972), Resource Conservation and Recovery Act (1976), Toxic Substance Control Act (1978), Solid Waste Disposal Act (1984), and the Comprehensive Environmental Response, Compensation, and Liability Act

(Superfund, 1980). The Uranium Mill Tailings Radiation Control Act (1978), administered by the U.S. Department of Energy (DOE), also had a major impact. With the implementation of these regulations, many geotechnical engineering firms responded by building capabilities to help clients address the new regulatory requirements. Many firms grew rapidly in response to the emerging opportunities and added the word environmental to their branding. The recruiting emphasis in these firms shifted from a primary focus on geotechnical engineers and geologists, to also include hydrogeologists, environmental scientists, and others. Over time, the environmental and geoenvironmental practices of many firms outgrew their traditional geotechnical practices. In recognition of the growing body of geoenvironmental research and publications, in 1997, the ASCE journal for the geoengineering discipline was renamed the Journal of Geotechnical and Geoenvironmental Engineering.

Growth of Engineering and Consulting Industry: The E&C industry experienced substantial and sustained growth from the 1970s until the present, with that rate of growth slowing only since 2008 as a consequence of the deep U.S. recession that began in that year. The growth of the engineering industry can be seen in Figure 2 where the revenues generated by the ENR Top 500 Design Firms are reported for the time period 1965 to 2010. In the time period from 1975 to 2008, the total engineering revenues represented by the firms on the list rose from roughly \$3 billion to \$91 billion in non-inflation adjusted dollars. Considering inflation, the revenue increase would be from \$12 billion to \$91 billion in 2010 dollars (www.usinflationcalculator.com). This revenue increase reflects growth in the number, size, and complexity of projects and the parallel growth in the sizes and capabilities of the engineering firms on the ENR list.

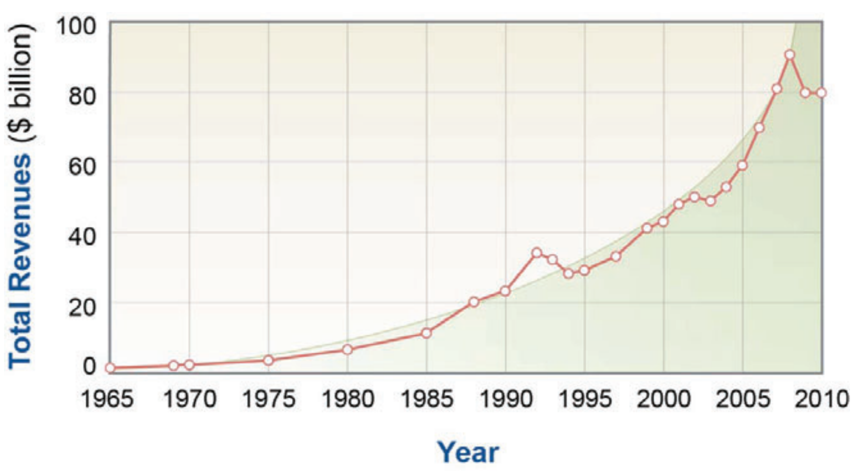

FIG. 2. Total Revenues of ENR Top 500 Design Firms. (Note: Dollar amounts correspond to the actual reported values in the given years. No adjustment has been made for inflation.)

In-Housing of Geotechnical Expertise: Many large engineering and construction firms that during earlier times had outsourced the geotechnical portions of their projects began to bring much of that work in house. Subsurface exploration, in-situ testing, laboratory testing, and construction-phase observation and material testing services continued to be outsourced by these firms, but data interpretation and analysis, and development of design recommendations and construction documents, were in many cases "in housed". As one specific example of this trend, in the early 1970s, the largest client for one of the largest geoengineering firms in the country was one of the country's largest water/wastewater design firms. Today, that water/wastewater design firm has its own in house geoengineering staff.

Federal/State/Municipal Procurement Requirements: Federal, state, and municipal regulations and policies have been established to protect and promote the well-being and interests of a variety of types of government-certified small businesses, small disadvantaged businesses, women-owned businesses, minority-owned businesses, HUBzone businesses, service-disabled veteran-owned businesses, and Alaskan native corporations. Procurement preferences supporting this goal have resulted in a significant number of private-sector practice geoengineering firms with these certifications. The geoengineering discipline has been fertile for this development for a number of reasons, including: (1) the ability to execute many geotechnical projects at a scale compatible with the sizes of many of these companies; (2) the value placed on these services by large engineers and contractors who must meet procurement goals on public-sector projects; and (3) the coincidence that the cost of geoengineering services as a percentage of all of the architectural, engineering, and related planning, design, and permitting costs for major projects, is typically of the same order of magnitude as the owner's subcontracting goals for certified businesses, (e.g., 10 to 15 percent).

Future Directions of Geoengineering Business Sector – View in 1988: In 1988, the ASFE Council of Fellows convened a meeting on "Future Directions in Geotechnical Engineering." The participants in the meeting developed 14 principal conclusions related to the future of geotechnical engineering practice and business (Aldrich, 2003). Ten of those conclusions (the ones most relevant to the subject of this paper) are presented in Table 4. It is interesting to compare this set of thoughts on the future direction of the profession with the author's view of geoengineering practice today, made 23 years after those conclusions were developed. The conclusions were prescient in the author's opinion and this is reflected in the discussion in the remainder of this paper. It is also interesting to compare the conclusions in Table 4 with the author's thoughts on future directions of the geoengineering business sector presented in Section 11 of this paper.

Table 4. Principal Conclusions from the 1988 Meeting of the ASFE Council of Fellows on the Future of Geotechnical Engineering (from Aldrich, 2003)

There will always be a market for individual consultants and small firms having widely-recognized expertise in specialized technical areas and emerging technologies.
Although the practice is mature, classical geotechnical engineering will continue to be practiced. Thus, there will be a need for small- to mid-sized firms, experienced in local areas to do drilling, materials testing, geotechnical engineering, and construction observation for new projects. The potential for growth is limited and competition among firms will be intense.
During the 1990s, the environmental geotechnology practice will continue to grow at a faster rate than classical geotechnical engineering. The scope of services will broaden and there will be more work outside of the United States.
Environmental practice is inseparable from the geotechnical practice. Most firms will need at least some capability in the environmental sciences to deal with sites which are contaminated. Health and safety programs and training will become mandatory.
The practices of many mid-sized to large firms will continue to grow and become broader in scope. Projects will become more complex and clients will be even more sophisticated, requiring a greater range of technical skills and more emphasis on project management. The practice will require greater consideration and understanding of social, economic, regulatory and political factors.
The practice will be motivated more by business principles. More firms will become business oriented professional service firms rather than professionally oriented businesses.
Turnkey work will increase in the private sector, with design-build firms and consortiums providing integrated services for both design and construction. Construction financing also may be offered. In the environmental area in particular, clients will want one consultant/construction combine to solve their problem. Turnkey work will provide more opportunity for innovation in design and construction.
Mergers and acquisitions of US firms by foreign firms will continue as international competition heats up. In turn, major US firms will buy into foreign firms to enter markets outside of the US. To compete, US firms must be innovative, technically and financially. US firms must stress quality.
There will be a healthy move away from litigation to alternate methods of dispute resolution. Greater emphasis will be placed on productivity and efficiency in our practice to remain competitive. Improved productivity will involve increased use of expert systems, sophisticated geographic information systems, and computer-aided engineering. Firms are more likely to patent their technology.
Greater emphasis will be placed on training programs for new employees, career long education, and professional development. There will be more women, minorities, and foreign born engineers in the practice.

Conclusion of Historical Perspective: All the foregoing developments, and others, have combined to shape the geoengineering business sector that exists in the U.S. today. It is fair to say that the *geoengineering practice* carried out by private sector firms has never been more complex, interesting, and diverse. Geoengineering practitioners are involved in virtually every type of infrastructure, industrial, and commercial development ongoing in the U.S. As projects become larger and more complex, and as performance tolerances become increasingly stringent, the opportunities increase for geoengineering practitioners to use the tools and technologies that have been developed and to stretch the state of practice. In parallel, private-sector *geoengineering business* is ever more complex and challenging. Geoengineering businesses operate in a dynamic marketplace that presents continual competitive, financial, personnel, and risk management challenges. The next sections of this paper present a closer look at the current state of the private-sector practice geoengineering business sector in the U.S.

3. SIZE OF BUSINESS SECTOR IN 2011

According to ASCE (www.asce.org), the society has 140,000 members, with just over 100,000 members in its eight practice institutes (Table 5). Note that society membership includes both U.S. and foreign members and that a member may elect to join more than one institute. Note further that not all civil engineers belong to ASCE. The U.S. Bureau of Labor Statistics (BLS) (www.bls.gov) 2010-2011 Occupational Outlook Handbook estimates that in 2008, there were 278,000 civil engineers, 54,000 environmental engineers, 8,000 marine engineers and naval architects, and 7,000 mining and geological engineers in the U.S. The American Council of Engineering Companies (ACEC) (www.acec.org), many of whose members are civil engineering firms, reports that it has 5,000 member firms representing more than 500,000 employees (with these employees including engineers, non-engineers, administrative staff, etc.). These data from ASCE, BLS, and ACEC provide indications of the size of the U.S. civil engineering industry.

Table 5. Membership Totals for ASCE Institutes (June 2011)

Institute	Membership
Architectural Engineering	8,330
Coasts, Oceans, Ports & Rivers	3,940
Construction	17,960
Engineering Mechanics	1,950
Environmental & Water Resources	23,380
Geo-Institute	10,750
Structural Engineering	21,010
Transportation & Development	15,480

The size of the geoengineering business sector can be roughly estimated by considering the number of U.S.-based members of the ASCE Geo-Institute (roughly 9,500) and increasing that number by the ratio of the number of civil engineers (lower estimate) or civil, environmental marine, mining, and geological engineers (upper estimate) reported by BLS to the membership of ASCE. Using this approach, it is roughly estimated that there are 20,000 to 25,000 geoengineering professionals in the U.S., participating in all of the business sectors identified in Table 1.

The number of private-sector geoengineering practitioners can be roughly estimated by considering the percentage of ASCE Geo-Institute members that identified private business as their employment category in the database reported in Figure 1 (58 percent) and multiplying that by the rough estimate of the total number of geoengineering professionals in the U.S. With this methodology, a rough estimate for the total number of geoengineers in U.S. private-sector practice is 12,000 to 15,000. This estimate may be low because it excludes any Geo-Institute members that did not identify a category in the database (i.e., about 15 percent of the respondents).

Another way to evaluate the size of the private-sector geoengineering business is to look at the results of an annual survey conducted by the Environmental Financial Consulting Group (EFCG), an investment banking firm located in New York City that focuses on the E&C industry (www.efcg.com). EFCG conducts an annual survey of firms including firms in the geoengineering, multi-service, and full service categories identified in Table 1. The 2010 survey involved 210 firms including many of the larger firms from the ENR Top 500 Design Firms list. The 2010 ENR list was published on April 25, 2011 (www.enr.com). The EFCG survey participants are asked to categorize their revenues into various market areas. The results for the 2010 survey are shown in Figure 3.

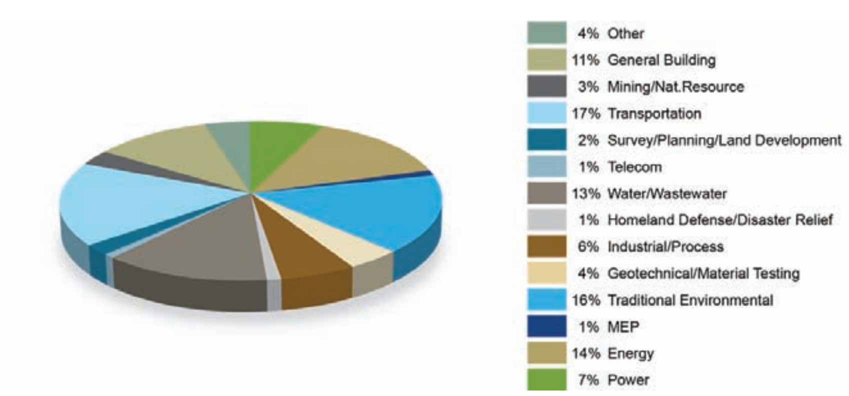

FIG. 3. Results of 2010 EFCG Survey of 210 E&C Firms on the Distribution of Revenues.

It can be seen that the survey participants allocated 4 percent of their revenues to "geotechnical/material testing." Whether this percentage truly reflects the size of the geoengineering market is dependent on how the survey participants allocated their revenues amongst the categories, which is not known. As just one example of this, it is not known if individual survey participants placed geoengineering revenues for their water/wastewater projects in that project category or in the geotechnical/material testing project category. As another example, it is likely that the traditional environmental sector also includes traditional geoenvironmental services, but the amount of these services is unknown. Nonetheless, applying the survey findings to the 2010 ENR Top 500 Design Firm total revenue of $79.8 billion for U.S.-based firms practicing both nationally and internationally results in a geoengineering private-sector market size of $3.2 billion. If one uses a reasonable estimate for the range of average gross revenue per employee in private-sector geoengineering firms of $150,000 to $200,000, and further uses a reasonable estimate for the average ratio of engineers to total staff of 0.75, a geoengineer headcount of 12,000 to 15,000 employees is calculated. This calculated total includes geoengineers for these U.S.-based companies who are working internationally. International project revenues make up more than 25 percent of the ENR total of $79.8 billion; the amount of these international project revenues generated by U.S.-based staff versus internationally-based staff is unknown. This total neglects staff associated with firms having 2010 gross revenues of less than $16.6 million (i.e., the size of the #500 firm on the ENR list). While there are many smaller firms and sole proprietorships in this category, their impact on total geoengineer industry headcount is believed to be relatively small (which is not to say that their impact on the geoengineering profession is small). These two factors tend to counterbalance each other, but to an unknown degree.

In summary, as a very rough estimate, there are today 12,000 to 15,000 geoengineers in private-sector practice in the U.S.

4. CHARACTERISTICS OF GEOENGINEERING FIRMS

Private-sector practice geoengineering firms can be differentiated based on a number of criteria, including size, geoengineering practice model, and ownership and legal structure.

Firm size: With respect to size, the author developed a classification system (Table 6) for use in this paper.

Table 6. Size Categories for Private Sector Firms with Geoengineering Practices

Category	Gross Revenue (million)	2010 ENR Rank Range
Very Large	> $250	1-50
Large	> $50 - $250	50-200
Medium	> $10 - $50	200-500
Small	≤ $10	NA
Sole Proprietor	NA	NA

The *very large* firm category includes the 50 largest firms from the 2010 ENR list, the largest 10 of which are given in Table 7. None of these 10 largest firms self-categorize (from categories pre-selected by ENR) as geotechnical engineer (GE), but instead self-categorize as engineer (E), engineer/architect (EA), engineer/contractor (EC), or engineer/architect/contractor (EAC). All provide a broad range of services for large infrastructure, industrial, commercial, and institutional projects. These firms may be considered "full service" and their focus is on large projects and programs. The organizational structures in these firms are often fundamentally different than in smaller, practitioner-led organizations. Organizationally, they are structured around market sectors (e.g., transportation, water/wastewater), client sectors (e.g., oil and gas, utilities, mining and minerals), and/or client types (e.g., private, municipal, and federal). The organizational structure is not built around practice disciplines, although "technical practice centers" may exist in the firm. These large firms typically have different professional staff responsible for firm management, business development, project management, and project execution. Notably, each has "in-housed" geoengineers and these in-house personnel perform much of the geotechnical evaluation, analysis, and design needed for the firms' projects. Table 7 also shows that for a number of these firms, in-house geoengineering capacity was built through acquisition.

Of the very large firms in the 2010 ENR list, only one Fugro - with revenues of $514 million and ENR rank #26 - self-categorizes as GE. In fact, Fugro is the only firm in the top 100 of the ENR list that self-categorizes as GE. Ninyo & Moore is the second GE firm on the 2010 ENR list at #187, with $53 million in revenues. By way of contrast, in the 1975 ENR top firms list previously discussed in this paper, Dames & Moore (#6), Woodward-Clyde (#16), Law Engineering (#50), McClelland Engineers (#58), D'Appolonia (#86), Fugro (#91), and Soil Testing Services (#99) were all in the top 100.

Table 7. Very Large Firms from the 2010 ENR Top 500 Design Firm List

ENR Rank (#)	Company Name	Self-Selected ENR Category	2010 Design Revenue ($ Million)	Geotechnical/ Geoenvironmental Acquisitions (Year)
#1	AECOM	EA	$5,920	Earth Tech (2008) TAMS (2002) (via Earth Tech)
#2	URS	EAC	$5,039	Woodward-Clyde (1997) Dames & Moore (2000)
#3	Jacobs	EAC	$4,748	Jordan, Jones & Goulding (2010)
#4	CH2M Hill	EA	$3,603	
#5	Fluor	EC	$3,128	
#6	AMEC	EC	$2,456	Agra Earth & Environmental (2000) Geomatrix Consultants (2008) Bromwell & Carrier (2011) MACTEC (2011) Law (2002) (via MACTEC) HLA (2000) (via MACTEC) Leroy Crandall (1982) (via Law)
#7	Tetra Tech	E	$2,210	Ardaman (2002) Gore Engineering (2007) Vector Colorado (2007) Bryan Stirrat (2009)
#8	Bechtel	EC	$2,170	
#9	KBR	EC	$2,010	
#10	Parsons Brinkerhoff	EA	$1,561	

Based on the comparison of 1975 and 2010 ENR firm rankings, the number of very large U.S. engineering firms who self-categorize as GE is much smaller today than it was 30 to 40 years ago. There are a number of possible reasons for this fact, perhaps the most significant being that the engineering marketplace as a whole has grown more rapidly than the geoengineering business sector and large projects are much more multidisciplinary than 30 to 40 years ago. As a consequence, the geoengineering component of many projects represents a decreased percentage of the total project scope. Additional factors include the rise of full-service firms in response to market forces and client demand, and the in-housing of geoengineering capability by large engineers and constructors has reduced the need by these firms for geoengineering subcontracting.

Table 8 presents representative *large firms* from the ENR list that either identify themselves as "GE" or who today self-categorize differently but started as geoengineering firms. Compared to the very large firms, these large firms typically still have traditional individual-practitioner led consulting practices. Senior personnel in these firms continue to operate as sellers/managers/doers (practitioner model). A number of these firms maintain in-house geotechnical testing capabilities, but only one maintains in-house field drilling and sampling capabilities. Firms in this size category have typically expanded beyond their geoengineering practice roots to provide services in selected other areas such as environmental engineering, water resource engineering, and structural engineering. In this regard, these firms may be considered "multi-service". These firms are also of a size that, to one degree or another, they incorporate organizational elements focused on specific market sectors, client sectors, and/or client types. While many of these firms characterize themselves as E in ENR ranking, their business may be better described as E&C, although this is not a defined ENR category.

Table 8. Representative Large Firms from the 2010 ENR Top 500 Design Firm List

ENR Rank (#)	Company Name	Self-Selected ENR Category	2010 Design Revenue ($ Million)
#58	Golder Associates	EC	$201
#74	Geosyntec Consultants	E	$163
#97	S&ME	E	$110
#114	Langan	E	$95
#119	GZA Geoenvironmental	EC	$92
#144	GEI Consultants	E	$75
#148	Haley & Aldrich	E	$74
#177	Shannon & Wilson	E	$57
#187	Ninyo & Moore	GE	$53
#190	Schnabel Engineering	GE	$51

Table 9 presents representative firms from the ENR list that are classified as *medium* based on Table 3. These firms share similarities with the large firms above, they are just smaller and typically do not have the breadth of practice capabilities or geographic coverage of the large firms. Note that in the medium firm classification, a higher percentage of firms categorize themselves as GE. Inspection of Tables 8 and 9 could be interpreted to indicate that (with one exception, Fugro), firms that self-categorize as GE are of a size (measured in terms of gross revenue) of about $50 million, or less. As the GE firms in Tables 8 and 9 grow larger in the coming years, it will be interesting to see if they continue to self-identify as GE, or if instead, they change their self-identification to E or some other category.

Table 9. Representative Medium Firms from the 2010 ENR Top 500 Design Firm List

ENR Rank (#)	Company Name	Self-Selected ENR Category	2010 Design Revenue ($ Million)
#219	Geoengineers	E	$44
#220	Raba Kistner	GE	$44
#292	Froehling & Robertson	GE	$34
#295	Paul C. Rizzo	E	$33
#326	Mueser Rutledge	GE	$31
#340	Engineering & Testing Services	GE	$29
#395	Soil & Material Engineers	E	$24
#404	Leighton Group	GE	$23
#408	Earth Systems	GE	$22
#435	Jacobs Associates	E	$21

There are many *small firms* in private-sector geoengineering practice that are not large enough to be on the ENR top firms list (revenues in 2010 less than $16.6 million). One estimate places the total number of geoengineering firms in the U.S. at about 1,500, with the great majority of these being very small firms (many with less than 10 employees) and sole proprietorships (Bachner, 2011). These firms typically have limited practice and geographic breadth. A few are highly specialized (subject matter experts) and are sought out for their expertise in a given area. Others may focus on a local presence where they principally serve local municipal, commercial and industrial clients. Many of these firms have a significant part of their business involved in the observation of construction activities and testing of construction materials. Representative firms in this latter category can be found on the websites for the California Council of Testing and Inspection Agencies (www.cctia.org), Texas Council of Engineering Laboratories (www.tcel.org), and Washington Area Council of Engineering Laboratories (www.wacel.org).

There are also a significant number of *sole-practitioner geoengineers*. In some cases, these sole practitioners are subject matter experts that are sought out by others for their special expertise. They consult to government, industry, and larger engineering firms. They may serve on expert panels for projects, or as experts in dispute resolution proceedings. For the individual whose expertise is mostly sought out by other engineering firms, being a sole practitioner can eliminate the potential hesitancy of these other engineering firms to retain the individual if he/she was instead employed with a different engineering firm for which there was an actual or perceived competitive conflict between the firms. Other sole practitioners may have a local, as opposed to expert practice, focus. Still others may choose a sole proprietorship because it gives them control over their time and commitments, which the author has observed to be attractive to some as they approach retirement, or for other reasons. It appears to the author that the number of geoengineers choosing to operate as sole practitioners has been steadily increasing over the past 10-20 years.

Geoengineering Practice Model: The geoengineering practice model for a company is influenced by how a number of factors are addressed by a firm as it grows and evolves. Factors that the author believes are important are: (i) technological approach to the market; (ii) business culture; and (iii) areas of technical competency and client focus. These three factors are interrelated, but are discussed individually below. The discussion is based on the author's personal observations and thus is most reflective of E&C businesses in the medium to large size category.

With respect to *technological approach*, Coxe et al. (1986) made the point for architectural firms that the status of the technology applied by a firm in the marketplace influences the types of projects that firm will perform and the project delivery processes it will use. The three technological approaches discussed by Coxe et al. are presented in Table 10. The technological approaches have been modified somewhat by the author to provide better relevancy to the geoengineering profession. Most private-sector geoengineering practices do not neatly fit into any single category. Instead, most exhibit attributes of several of the categories, with perhaps one of the categories being dominant in the firm. Notwithstanding this point, the author believes that Table 10 provides a useful framework for defining the technological style of a firm's practice.

Table 10. Technological Approaches of Geoengineering Practices [modified from Coxe et al. (1986) and Maister (1993)]

Brains – the technical knowledge and practice experience used on projects is at the forefront of the profession; the technology application requires creativity, innovation, and pioneering of new approaches. The project delivery process is driven by senior subject matter experts. Due to the very specialized nature of the work, delegation from senior to junior staff is more limited than in the other categories. The generation of new and innovative engineering tools and solutions is a hallmark of the brains practice.
Grey Hair – the technical knowledge and practice experience applied to projects are highly specialized, but include a lesser degree of innovation and creativity than for brains projects. The methods and approaches used to complete projects are well tested and similar to those that have been used on previous projects. Projects are more easily planned and scoped, and more of the work can be delegated, compared to brains projects. The project delivery process is driven by senior practitioners having deep experience and strong project management skills. Project teams are more structured and include a higher proportion of junior practitioners compared to brains projects. Strong service is a hallmark of the grey hair practice.
Procedures – the technical knowledge and practice experience applied to projects is well recognized and familiar, and the steps necessary to complete the projects are straightforward and well defined. The procedures for some parts of the work may be prescriptive (e.g., ASTM procedures), the project scope has been conducted many times previously, and much of the work can be delegated. The project delivery process is designed to achieve efficiency and cost effectiveness. Project managers are highly experienced in the practice and are evaluated by their ability to meet project delivery goals. Strong delivery is a hallmark of the procedures practice.

How does the technological approach drive the style of a firm's practice? As an example, a firm performing mostly "brains" projects will be highly dependent on the specialized expertise of its senior staff and may enjoy the advantage of having a limited number of qualified competitors. It may be able to command relatively high billing rates given the level of technical expertise embedded in its practice, but its opportunities to grow are limited and its staff leverage (i.e., ratio of junior staff to senior staff working on a project) will likely be low. It may also struggle to win work outside of its well-defined practice specialties. As a specialist firm, a brains-based practice will have the opportunity to work on very interesting technical assignments, but participation in large, multi-disciplinary infrastructure projects will only occur as a subcontractor to a full-service firm or as a consultant to the owner or regulator. In contrast, a firm performing mostly "procedures" work may be able to compete for a broader range of projects while achieving a higher level of staff leverage. It may also have more growth opportunity and staff recruiting may be easier because its personnel requirements are not as specialized. The business will be more scalable than a brains-based business. However, a procedures firm may face substantial competition from others because the technological barriers to market entry are not that high. Compared to the other categories, a procedures based firm may be the most scalable and replicable across regions. As the work requires less technical specialization, staff recruiting may be easier than in either brains or grey hair firms. Low cost may often be a primary selection factor for clients procuring procedures-based services, so operational and cost efficiency are more important management objectives in procedures firms compared to the other types. A grey hair practice fits between the other two categories. Many large and small firms have strong grey-hair attributes. Many of the largest E&C firms in Table 7 have large, sophisticated grey-hair teams focused on the execution of large, complex projects and programs.

Most firms publish a list of company values and other attributes that define the *business culture* of the firm. These typically include important principles such as honestly, integrity, work quality, employee safety, client service, lifelong professional development, equal opportunity, and others. Another attribute, not often stated, relates to whether the management and staff in the firm place more emphasis on the practice attributes of the company or on the company's business results. Coxe et al. used Webster's definition for practice ("the carrying on or exercise of a profession or occupation….as a way of life") and business ("a commercial or mercantile activity customarily engaged in a means of livelihood") to develop the following two end-point definitions for practicing professionals:

- *"Practice-centered professionals, who see their calling as "a way of life," typically have as their major goal the opportunity to serve others and produce examples of the discipline they represent. Their bottom line is qualitative: How do we feel about what we are doing? How did the job come out?"*

- *"Business-centered professionals, who practice their calling as "a means of livelihood," more likely have as their personal objective a quantitative bottom line, which is more focused on the tangible rewards of their efforts: How did we do?"*

Not only are these practice and business cultural end-points applicable to the individual, they are also applicable to firms. Where a company falls in the spectrum from a purely practice-based culture to a purely business-based culture is mostly dependent on the attitudes, focus, and demands of the firm's owners and senior management team. Figure 4, which the author has modified from PSMJ (2004), defines four types of professional service firms based on business culture. It is the author's observation that as firms grow, they tend to move from a more practice-centered culture to a more business-centered one, although the extent of this shift in business culture varies significantly between firms and there are exceptions to the observation. Similarly, and understandably, this culture shift also tends to occur if a firm's ownership structure (discussed subsequently) changes from partnership or employee ownership to outside private-investor or public-market ownership.

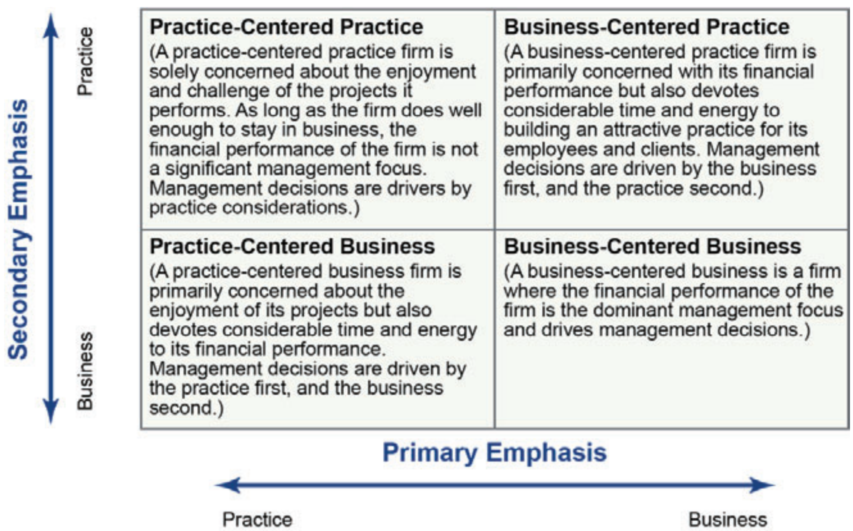

FIG. 4. Practice-Centered Verses Business-Centered Business Culture for Engineering Firms (modified from PSMJ, 2004).

The last factors related to the geoengineering practice model to be mentioned are *technical competency and client focus.* Representative areas for each are given in Table 11. It is emphasized that these lists are not exhaustive. Moreover, it should be clear that the nature of the specific services provided will be influenced by the technological approach and values of the firm providing the services. For example, virtually every geoengineering firm in the western U.S. will offer their clients geotechnical earthquake engineering services. However, the content of those services and the complexity of the projects to which they are applied may be significantly different for a brains practitioner in a practice-centered practice compared to a procedures practitioner in a business-centered business.

Table 11. Representative Areas of Technical Competency and Client Focus for Private-Sector Geoengineering Firms (lists are representative, not exhaustive)

Technical Competency	Client Focus
Site Investigations, In situ Testing, Geophysics, Laboratory Testing	Architects & Structural Engineers
Construction Observations and Material Testing	Residential Developers
Shallow and Deep Foundations for Buildings, Industry, and Infrastructure	Commercial Developers
Roads and Pavements, Pipelines, Transmission Towers	Municipalities, Cities, Counties
Excavation Bracing, Tiebacks/Soil Nails, Dewatering, Seepage Control	Water Resource Agencies
Instrumentation, Structure Integrity Monitoring	Ports and Harbor Authorities
Landslide Investigation/Repair, Natural Hazard Assessment/Mitigation	Transportation Agencies
Earth Retaining Structures, Abutments, Bridge Foundations	Federal Agencies (USACOE, USDOE)
Embankments, Dams, Slopes, Reservoirs	Regulatory Agencies (NRC, FERC, EPA)
Flood Control Structures, Levees, Pump Stations	Water/Wastewater Utilities
Tunneling, Underground Construction	Electric Power Utilities
Soil Improvement, Grouting	Oil & Gas (Offshore)
Offshore Geotechnics and Foundations	Oil & Gas (Onshore)
Waterfront Structures, Bulkheads, Quay Walls	Contractors (Heavy Civil)
Waste Geotechnics and Containment Systems	Contractors (Specialty)
Mine Geotechnics and Tailings Piles/Ponds	Manufacturers
Site Remediation, Brownfields	Industry, Superfund PRP Groups
Groundwater Flow, Contaminant Transport, Subsurface Barriers, Treatment Walls, and Drains	Landfill Owners and Operators
Seismic Site Response Analyses, SSI	Mine Owners
Liquefaction Assessment/Mitigation	R&D Funding Agencies
Arctic Engineering, Permafrost	International Monetary/Lending Agencies

Ownership and Legal Structure: Firms in the private-sector practice of geoengineering have a range of *ownership structures.* Ownership structure influences a firm's business culture, method of capitalization, decisions on debt and equity, financial rewards system, and ability to grow. Ownership structure is about who owns the firm, not how the firm is organized for legal and tax purposes (discussed below), although the two are interrelated. The types of ownership structures of engineering firms (along with the author's perspective on some of the advantages and disadvantages of each) are given in Table 12.

Table 12. Ownership Structures for Geoengineering Firms

Category	Advantages	Disadvantages
Employee Owners (Equity owned by employees)	Employee owners have control over company culture, style, and direction; employee owners control and allocate profits; company can use employee ownership as recruiting and reward tool; ownership helps build loyalty with employees; if firm avoids debt, firm retains high level of operational independence from bank restrictive covenants.	Company has limited ability to raise capital through employee shareholders, thereby limiting rate at which firm can grow and diversify without taking on debt; internal ownership transition (IOT) can be difficult; employee owners typically must value firm well below fair market value to achieve IOT without depleting firm equity.
Non-employee (outside) Private Investors (Equity owned by non-employee individuals or private companies; equity not publicly traded on a stock exchange)	Provides alternative to debt financing or public market to grow and diversify firm; sometimes part of exit strategy for retiring employee owners; investors can tailor investment amounts, terms, and conditions to firm's growth and development plans; investors are typically motivated to have firm management retain some level of ownership; equity investors often bring financial and management expertise.	Employee owners will often need to give up a majority of firm ownership to the private investors as a condition for investment; private investors may set goals for growth and profitability of the firm (and thus, the investors return on investment) that are aggressive and stress the capability of the firm or causes it to grow more quickly than management can handle; most private investors have a defined investment time window (e.g., 4 to 6 years) and then want to sell their investment.
Public Market Investors (Equity publicly traded on a stock exchange)	Public market typically values companies more highly than other categories and provides source of capital to aggressively grow and diversify the firm; high public valuation multiple on company earnings (P/E ratio) is advantageous in acquiring other companies; public market investors less intrusive of company daily operations than non-employee private investors.	Public markets have short-term focus on growth and earnings; meeting market expectations can dominate management focus; volatility of share price and continual need to grow earnings can be demoralizing to professional staff; extensive regulatory reporting requirement for public firms.

Employee-owned and private-investor geoengineering firms can have a number of different *legal structures.* These include sole proprietorships, limited liability companies (LLC), partnerships, professional corporations (PC), S corporations, and C corporations. Public market companies are almost always organized as C corporations. The treatment of profits and taxes, allowable number of owners, potential liability exposure of owners, and purchase and sale of company equity are generally different under the different types of organizational structures. A discussion on these details is beyond the scope of this paper, but they are important and must be carefully considered both in starting a company and during the life of a company as it grows and evolves.

The large majority of geoengineering firms in the size categories from individual practitioner through large-sized company are employee owned, with a relatively few firms in these size categories having non-employee private investment. Of the 210 firms that participated in the 2010 EFCG survey, 78 percent were employee owned, while 15 percent were publically owned, and 7 percent had outside private investors. Based on a review of the 2010 ENR Top Firms list, the percentage of public companies by size category does not become significant until the very large size category is reached. This fact is reflected in Table 7, where 8 of the 10 largest companies are public (only CH2M Hill and Bechtel are not public). In contrast, none of the firms in Tables 8 or 9 are public. While the overall number of public companies in the 2010 ENR Top Firms list is relatively small, the percentage of industry revenues flowing to them has been increasing in recent years and revenues to these firms represent a significant percentage of the total ENR Top 500 revenue. This can be seen by the fact that more than $35 billion (out of a total of $79.8 billion) in ENR-reported revenues came from the very large firms in the ENR list that are publically traded.

5. OPERATION AND MANAGEMENT OF GEOENGINEERING FIRMS

This section of the paper describes a number of important considerations in operating and managing a private-sector geoengineering business. Much of the discussion is based on the author's personal experiences and thus again most reflects E&C type-businesses in the medium to large size category.

Basic Business Model: An obvious necessity for the survival of any professional services business is to generate enough revenue from clients to cover the compensation of the firm's employees plus all overhead, expenses, and debt repayment obligations. This is of course true whether the business is practice-centered or business-centered, although the intensity of management focus on making a profit will vary as previously discussed.

Revenues: Revenues in E&C geoengineering firms are mostly generated by the hourly billings of the firm's personnel on contracted projects, plus reimbursable and subcontractor expenses incurred on the projects (e.g., subcontracted geotechnical drilling and laboratory services) and the mark-ups the firms put on these expenses. This revenue model is often referred to as "time and materials (T&M)." Sometimes clients want geoengineers to propose on their projects using "lump sum" pricing. Under the best circumstances, a project can be more profitable when performed on a lump sum instead of T&M basis, as lump sum pricing allows the firm to benefit from the efficiencies it can bring to the project. Often, however, the lump-sum pricing mechanism attempts to transfer significant project risk from the client to the geoengineering firm and the firm must pay careful attention to how it defines the scope of services, assembles the lump sum price, and negotiates the contract terms and conditions (including "pricing re-openers") if it is to be successful with this contracting method. In fact, due to the uncertainty (e.g., limited knowledge of subsurface geological conditions) associated with many geoengineering projects, lump sum pricing is often not appropriate. A third type of revenue model, falling between T&M and lump sum, is known as unit pricing and essentially involves providing the client with a lump sum price for each unit of work (e.g., a lump sum price to drill, sample, and log a borehole advanced to a prescribed depth). If the units to be priced are thoughtfully and carefully defined, unit pricing can be an attractive pricing mechanism for both the client and geoengineering firm.

Compensation, Overhead, and Expenses: The expenses incurred by geoengineering firms include labor, fringe benefits, building occupancy costs, marketing and proposal costs, professional development costs, and risk management (including insurance), IT/communications, human resources, finance/accounting, and miscellaneous costs (Table 13). Employee compensation represents the largest expense component, followed by fringe benefits and occupancy costs.

Profits: As a very general rule of thumb, if an E&C geoengineering firm can generate net revenues (gross revenues minus project reimbursable and subcontractor expenses) equal to twice its total labor wages, and if it reasonably manages its overhead, it will achieve an attractive level of pre-interest, pre-bonus, pre-tax profitability (EBIBT). At a net revenue/total labor multiple of two, EBIBT should be in the range of +/- 30 percent of total labor wages or +/- 15 percent of net revenues. This simple model is based on the typical overhead components for U.S. E&C firms given in Table 13. It is noted, however, that many firms have found it a challenge to achieve this net revenue/total labor multiple as evidenced by the fact that the historical median EBIBT of E&C firms in the EFCG survey has been in the range of 10 to 11 percent.

Table 13. Typical Profit/Loss Operations Model for E&C Geoengineering Firms (all values given as a fraction of the firm's total employee payroll amount)

Revenue (net)	2.0
Total Labor	1.0
Expenses	0.7
• Fringe Benefits	(0.23)
• Risk Management	(0.04)
• IT/Communications	(0.07)
• Finance/Accounting	(0.05)
• Human Resources	(0.03)
• Marketing/BD	(0.10)
• Building Occupancy	(0.11)
• Professional Development	(0.02)
• Interest/Depreciation/Amortization	(0.01)
• Bad Debt/Write-offs	(0.02)
• Miscellaneous	(0.02)
Pre-Tax, Pre-Bonus Profit	0.30

Income Statement and Profit Allocation: Pre-tax, pre-bonus profits may be allocated for a number of different pre-tax purposes. These include employee bonus programs, profit sharing plans, and pension plans. The amount of profits paid as bonuses to employees varies widely by company. EFCG has reported that the median bonus pool for the E&C firms in their annual survey has historically been in the range of 25 to 30 percent of EBIBT, but individual companies pay out from less than 10 percent to more than 50 percent of their EBIBT as bonuses. For firm's having an ESOP, a portion of the profits will typically be applied on a pre-tax basis to pay down debt held by the ESOP trust. For S corporations and partnerships, a major portion of the profits will be paid out pre-tax to the shareholders and partners, respectively, as these individuals have personal tax liabilities for their firm's profits as explained below. In the author's experience, the allocation of a healthy percentage of the company's profits to employee bonuses can motivate and incentive employees, while building goodwill and loyalty towards the firm.

The 2011 U.S. federal tax rate for C corporations varies depending on the amount of taxable income, but presently averages about 35 percent. State corporate tax rates range from zero to 12 percent, with the rates for most states falling in the range of 4 to 8 percent. S corporations, partnerships, and sole proprietorships, are not separate tax entities, like a C corporation, for purposes of federal tax obligations. Instead, they are defined as "pass-through entities". These entities are not subject to federal and state income taxes. Instead both the profits (after other pre-tax allocations) and the tax liability for those profits are allocated to the shareholders, partners, and members of the various entities. The taxable income liabilities for these individuals are addressed in their personal tax returns. At the federal level, the members of an LLC can elect to be taxed either as a corporation or partnership.

Profits retained in a firm after all pre-tax distributions, tax allocations, and dividends (for C corporations) are defined as retained earnings. Retained earnings build equity in a firm and can be used to fund growth, make capital expenditures, pay debt, repurchase stock, and for other purposes.

Balance Sheet: A balance sheet is a financial statement that summarizes a company's assets, liabilities, and shareholders' equity at a specific point in time. In short, it reports what a company owns and owes, as well as the investment amount of the shareholders. The components of the assets on the balance sheet of a typical E&C geoengineering firm include current assets (i.e., cash and short-term investments, accounts receivable [AR], and work-in-process [WIP]), property and equipment net of depreciation, other miscellaneous assets, and goodwill. Goodwill is an intangible asset (e.g., brand name, customer relationships, proprietary technology) that equals the price paid for an acquired asset (typically an acquired company) in excess of the book value of the asset. For most geoengineering firms, AR and WIP make up the most significant portion of the total assets. The liabilities side of the balance sheet is made up of current liabilities (the largest components of which are accounts payable, accrued expenses, and deferred income taxes) and long-term liabilities (including long-term debt obligations). Shareholders' equity is simply total assets minus total liabilities. Important balance sheet metrics include working capital (defined as current assets minus current liabilities), current ratio (defined as current assets divided by current liabilities), and debt-to-equity ratio (defined as total liabilities divided by shareholders' equity).

EFCG survey results indicate the following industry average financial metrics for E&C firms over the past decade: ratio of shareholders equity to gross revenues = 0.15

to 0.18; debt-to-equity ratio = 1.4 to 1.7; and ratio of cash and cash equivalents to gross revenues (exclusive of payroll coverage) = 0.04 to 0.06. EFCG found that all of these balance sheet metrics had become more conservative over the decade (i.e., firms were being managed to carry more equity and cash, and less debt). EFCG also found that these results reflect less financial leverage in the E&C industry compared to many other parts of the U.S. economy. Stated differently, compared to other industries, the E&C industry is more conservative with respect to taking on high levels of debt to fund operations, growth, product development, and acquisitions. As a result, the risk/reward calculus for civil and environmental firms is different than for those industries that take on more financial risk for the opportunity to try to earn more financial reward. This risk/reward calculus can be seen in the following comparisons. On the reward side, EFCG survey results show that the median annual EBIBT for civil and environmental E&C firms has for a number of years been in the range of 10 to 11 percent of net revenues and the median annual rate of stock appreciation has been about the same (upper quartile for both profits and shareholder return is in range of 15 to 20 percent). While these are healthy financial results and returns, they are somewhat modest compared to the financial goals for some other professions and industries. On the risk side, EFCG survey results show that it is rare in the E&C industry for a company to be unprofitable, and rarer still when one has suffered bankruptcy. EFCG reports that for its 2010 survey, which covers a portion of the worst economic time in the U.S. since the 1930's, only 4 percent of the 210 firms participating in the survey reported losing money. In contrast, the U.S. Federal Deposit Insurance Corporation (www.fdic.gov) reported 347 bank failures (from a population of roughly 7,800 U.S. banks) between the start of 2009 and mid-point of 2011. It is noted that the EFCG data are biased towards the larger and more financially successful firms in the E&C industry. Thus, the percentage of geoengineering firms, particularly small firms focused on residential and light commercial projects, suffering financial losses during the recession is likely larger than the percentage from the EFCG survey group. Nonetheless, the author's point regarding the risk/reward calculus for the types of firms represented in Tables 7,8, and 9 is believed valid.

Cash Flow: One other important aspect of the business management of E&C geoengineering firms is cash management. The profit and loss statements for the majority of E&C firms (including all C corporations with more than $5 million in revenues and all S corporations and partnerships with more than $10 million in revenues) are reported on an accrual basis. This simply means that revenues are recorded when work is done and expenses are recorded when they are obligated. This is in contrast to cash-based accounting where revenues are only recorded when payment is received and expenses are recorded only when paid. There are a number of important reasons why most firms, particularly larger ones, use accrual accounting. One disadvantage of accrual accounting, however, is that it does not reflect a firm's cash flow. For example, an E&C firm will recognize labor revenues on its books as soon as the labor effort is recorded in the firm's accounts. However, payment for that labor will typically not be received until 70 to 90 days after the charges were recorded (due to the time it takes to prepare client invoices and then get paid). If not careful, a growing E&C firm with the median profitability reported above can run short of cash.

A firm that is profitable on an accrual basis can have a cash flow shortage due to a variety of circumstances, including: (i) poor invoicing and collection practices; (ii) clients that hold payments in dispute or enter bankruptcy and do not make payment; (iii) unanticipated or excessive expenses requiring payment more quickly than income is received; (iv) cash consumption due to firm growth at a rate faster than the firm is generating free cash flow from operations; (v) cash consumption due to significant levels of capital expenditures; (vi) the firm incurs liability expenses not covered by insurance (e.g., deductibles); and (vii) the firm has stock repurchase obligations that consume cash in excess of free cash flow. The ability to predict cash needs and to plan for meeting those needs is a very important management responsibility. This planning must address both short- term cash requirements (e.g., expenses and bad debt) and long-term cash requirements (e.g., growth capital and cash for long-term stock repurchase obligations).

Management thinking varies regarding how much cash to carry on a firm's balance sheet to be able to handle the potential cash needs of a firm. The more financially conservative approach is to use time periods of strong profitability to accumulate enough cash for known needs, plus extra for contingencies (e.g., "rainy day fund" or for expansion opportunities). The more financially leveraged approach is to retain less cash in the company and to rely on a line of credit (short term) or loan (long term) to help address the cash needs of the firm. One measure used by the author's firm to evaluate its cash position is "payroll coverages", simply defined as the number of two-week payrolls that can be covered with cash on hand after setting aside enough funds for upcoming stock repurchase and tax obligations. A running cash balance of 3 to 4 payrolls beyond the set-aside funds is considered by the author to be conservative and healthy.

Employee Ownership and Transition: The distinction between a practice-entered firm and a business-centered firm is often, but not always, reflected in the firm's ownership structure. As the results of the 2010 EFCG survey demonstrate, the large majority of E&C firms are employee owned as S or C corporations, sole proprietorships, LLCs, or PCs. Ownership provides employees with the opportunity to have a voice in company practices, direction, and management, and to participate through their ownership stake in the financial rewards if the company is successful. Ownership can be a valuable motivator for employees and it can help align company and employee goals and incentives. Ownership opportunities can help in recruiting and retaining employees. As highly educated and trained professionals, many geoengineers see ownership as both an important career goal and as a component of their long-term personal financial plan.

Participation Levels for Employees in Ownership Programs: Employee ownership models in geoengineering companies vary considerably. At one extreme, a very few employees might own all or a very high percentage of the company's stock. At the other extreme, all of the company's employees are shareholders and no single employee owns a high percentage of stock. Both companies at these two extremes are "employee owned" but the two employee-ownership models are very different.

EFCG reports that the employee ownership participation level is less than 20 percent for most of the employee-owned firms in its 2010 survey. In the author's firm, the employee participation level is 35 to 40 percent and invitation to an employee to become a shareholder is based on a number of performance, leadership, length-of-service, and related criteria. For firms with ESOPs, all qualifying, full-time employees share in ownership of the firm as a consequence of the ESOP participation requirements. The level of ownership an employee accumulates in an ESOP will depend on the proportion of company stock held by the ESOP, the length of service for the employee, and the base compensation level of the employee (as the annual distribution of the ESOP ownership interest to the company's employees is proportional to their base compensation).

Ownership Transition: An extremely important management consideration for employee-owned firms is the process of ownership transition. The *more visible* form of ownership transition is an external sale (merger or acquisition) accomplished by selling the firm to another firm or private investor group as outlined in Table 12. However, the *more common* form of ownership transition is accomplished via internal transfer. As the name implies, this process involves the transition of the ownership of the company from one generation of employee shareholders to the next generation.

In a corporation, ownership transition involves the sale of stock by an existing employee shareholder to the treasury of the company, to another shareholder, or to an ESOP. With respect to the first two methods of transitioning shares, the method for valuing the company for internal ownership transactions (i.e, for establishing the price of company stock for internal "buy-sell" activities), the rules for buying and selling shares, and the requirements (including schedule) for payments are addressed in a company shareholder agreement, often called a "buy-sell" agreement. For non-ESOP firms, within the limitations of applicable law, the shareholders have flexibility to customize the shareholder agreement to fit their purpose. For firms with ESOPs, a trust is established to hold stock that the company purchases from existing shareholders. The purchased stock is allocated to employee accounts, as described above. There are very specific rules governing ESOP administration and these must be followed for the company to utilize the tax benefits and other features of the ESOP (Beyster Institute, 2005). For partnerships and LLCs, ownership transition for partners or members is governed by, respectively, the partnership agreement or operating agreement. The discussion on ownership transition that follows is directly applicable to private, employee-owned corporations, although many of the challenges and issues described are also relevant to partnerships and LLCs.

Valuation of Company (Share Value): As described above, the ownership of employee-owned corporations is governed by the shareholder agreement. In these firms, the shares cannot be sold to the public, so there is no "market value" as such. Instead, the shareholder agreement provides for either a formula (e.g., a multiple of EBIBT, book value, revenue, or some combination of these parameters) or appraisal approach to determine the value of shares on an annual or other periodic basis. The approach to valuation will reflect the company's culture and unique choices among a number of competing considerations, such as the payment of bonuses versus the enhancement of equity, the facilitation of bringing new shareholders into the firm while avoiding undue dilution of equity ownership, and maintaining the financial ability to repurchase firm shares while maintaining fair valuations. As a consequence, there is no one method for company valuations and, in fact, there is a significant range of methods used. This results in company valuations of shares that may have little correlation to the fair market value of the company (i.e., the value of the company if it were sold on the open market with willing and knowledgeable buyers and sellers). For an ESOP, the ESOP-held stock must be valued by an independent appraiser based on an assessment of the fair market value of the firm. Table 14, below, provides a summary of internal valuations for employee-owned firms from the 2009 EFCG survey. The valuations are presented as multiples of book value. The valuation methodology used by the 112 firms represented in Table 14 was roughly evenly split between a valuation formula and an appraisal. Based on this table, the median valuation is approximately 1.5 times book value.

Table 14. Internal Company Valuation for Employee-Owned Firms as a Multiple of Book Value (from EFCG 2009 Survey, 112 firms reporting)

Valuation	Firms (#)	Firms (%)
<1.0BV	5	4
1.0BV	20	18
≥1.0BV<1.3BV	18	16
≥1.3BV<1.5BV	13	12
≥1.5BV<2.0BV	18	16
≥2.0BV<3.0BV	26	23
≥3.0BV	12	11

The valuation method used by a company will have a significant influence on the ownership transition process. For example, the share valuation formula used by a company may set a low stock price compared to fair market value. This is not uncommon in employee-owned E&C firms. The rationale for a valuation methodology that results in a lower than fair market price is that it makes it easier for the company to pay for stock redemptions from selling shareholders and to help acquiring shareholders to purchase shares (because the company will often finance share purchases by its employees and/or provide bonuses to help employees be able to afford to purchase shares). However, with a low share price, when it comes time for a shareholder to sell, the shareholder may feel disadvantaged in being required (through the shareholder agreement) to sell his/her stock at a price believed to be well below fair market value. In this case, the selling shareholder may advocate to sell the entire firm to a buyer (i.e., another company or private investor) willing to pay a higher price. Another disadvantage of a low share price is that it makes it unattractive to use company stock to pay for acquisitions.

A share valuation formula or appraisal that results in a relatively high share price will be attractive to selling shareholders who hope to maximize their return. In this latter case, however, the company, buying shareholders, or ESOP may not be able to afford to purchase the stock. The company may need to take a bank loan, or find ways to generate cash (e.g., put off capital expenditures and/or reduce employee bonus programs) to pay the selling shareholders, or it may need to delay or stretch out payments to the selling shareholders. Under these circumstances, the selling shareholders may advocate to sell the firm to assure they get paid for their stock and the remaining shareholders may agree to a sale to obtain relief from the payment burden. All other things being equal, one fact is clear - the more profitable a firm's operations on a sustained basis, the higher the share valuation that is sustainable.

Rate of Stock Transition: Another important factor in transitioning shares is the rate of transition. It is much easier for a company to financially manage and support a long-term transition process wherein, for example, 5 percent of the shares are systematically transitioned every year versus an alternative scenario wherein selling shareholders want to sell a large percentage of the company's stock back to the company or to other shareholders within a few year time period. To help protect the firm against this latter scenario, shareholder agreements often contain repayment schedules that allow for the payment of the share purchase price over a time period that may be of several years duration, plus a right to suspend repayments if the company sinks into financial trouble as determined by one or more financial performance metrics. All other things again being equal, it is easier to effectuate internal ownership transition when the company's stock is widely distributed amongst many employees and there is no single person or small group that own a majority of the shares compared to the case where most of the stock is held by an individual or a very few employees. It is to the benefit of the substantial shareholders of the firm to work collaboratively with senior management to plan years in advance for a smooth process of stock sales and repurchases to avoid financial shocks to the company, to increase the likelihood that the selling shareholders can receive timely full payment for their shares, and to have buying shareholders feel they are making a safe and sound investment.

Demand for stock: To effectuate an internal ownership transition process, a company must maintain demand for its shares by employees. Internal demand for shares is easier to maintain in a profitable, growing, vibrant business where potential buyers of stock see a bright future for the company and a sound investment for them. These conditions help maintain the demand for stock. Conversely, in a static, declining, or less profitable business, demand for stock may wane and ownership transition becomes difficult. Hogan (2011) talks about "*buyer reluctance*" under these conditions. Should demand for stock and willingness to buy stock dry up, internal ownership transition becomes impossible and a firm will have no choice but to look externally for a buyer or source of equity investment.

In summary, if the ownership of an employee-owned firm is to be successfully transitioned from one generation of employees to the next, careful thought and planning must be given to: company performance and growth; share valuation method; creating sustained demand for share purchases; and, managing the share repurchase schedule, and managing the payment schedule for repurchased shares. An informative reference on ownership transition in employee-owned E&C firms is provided by Getz and Laurie (2002).

6. CAREERS IN GEOENGINEERING PRACTICE

Geoengineers typically begin their professional careers after obtaining a graduate degree in geotechnical or geological engineering or a related field. In private sector practice, career advancement will involve increasing levels of responsibility and authority as an individual gains experience and is promoted within a firm. While each firm has different names for the seniority levels in the firm, they typically fit into a framework that has four increasing levels of responsibility, performance expectation, and compensation. The four basic seniority levels are: (i) staff geoengineer; (ii) project geoengineer; (iii) senior geoengineer; and (iv) principal geoengineer. Table 15 presents a description of the engineering and operational responsibilities of geoengineers throughout this four-level career progression.

Table 15. Representative Engineering and Operational Responsibilities of Geoengineers during Career Life Cycle

	Staff Geoengineer	**Project Geoengineer**	**Senior Geoengineer**	**Principal Geoengineer**
Years Experience	**First 3-5 years after graduation**	**Next 3-5 years**	**Next 3-5 years**	**More than 12-15 years experience**
Engineering Responsibilities	Perform field investigation Provide construction QC/QA Perform calculation and analyses Prepare text, figures, tables for reports	Design/manage field programs Prepare/review calculation packages Prepare construction plans and specifications Manage 1 or 2 staff professionals Prepare small proposals Prepare complete reports	Design/manage field programs Develop scopes of work for major projects Review calculations and reports Review construction plans and specifications Manage a small team of project and staff personnel Interface with client/ prepare proposals	Direct practice areas Provide high level technical direction and consulting Peer review work products Lead major business development initiatives Manage a group of senior project, and staff personnel Manage company risks (contracts, safety, deliverables)
Operational Responsibilities	Safe work practices Meeting schedule (self) Working in teams Oral communications Technical writing	Budgeting and scheduling Quality control of work Management of junior staff Talking to clients Preparing complete, high quality work products	Project/program management Budgeting and scheduling Personnel and recruiting Proposal preparation and presentations Risk/quality management	Project and clients Business development Risk/quality management Personnel and recruiting Financial and legal matters Strategic planning
Time Management (typical)	90% project work 5% management and administration 5% professional development	75% project work 20% management and administration 5% professional development	65% project work 20% management and administration 10% new business development 5% professional development	0 to 50% project work 20 to 50% management and administration 20 to 50% new business development 0 to 5% professional development

Geoengineers in private-sector practice must be technically competent and experienced in the areas of their practice. The building of technical competency begins with their engineering education and then continues after the geoengineer starts his or her career at the staff level. A strong undergraduate education in civil engineering helps the aspiring student establish the engineering fundamentals needed to develop competency across one or more geoengineering practice areas. A strong graduate education in geotechnical, geoenvironmental, or geological engineering begins to directly build those competencies. The author believes that graduating geoengineers should look for employers where they can go through a career-long process of building those technical competencies, gaining experience, working on challenging projects, and building their reputation as accomplished and experienced geoprofessionals.

When these career steps, and the associated responsibilities, performance expectations, and compensation, are placed on a timeline, the trajectory of a geoengineer in private-sector practice can be defined. This trajectory is shown in Figure 5. The successful practicing geoengineer starts at the staff level and progressively advances through the project and senior levels to become a principal at which level the principal stays until he/she chooses to slow down as they approach retirement. The number of staff, project, and senior professionals in a firm for every principal (i.e., the "staff leverage") varies based on a number of factors, including the technological approach (Table 10) and business culture (Figure 4) described earlier in this paper. The larger the number of more junior personnel to more senior personnel, the more the leverage of each senior person. In the author's experience, an appropriate ratio of principals to senior, project, and staff personnel may range from about 1:5 to 1:15, depending on the nature of the practice. The author also notes that he has observed more than a few firms where there seems to be "position inflation" where a disproportionally large percentage of the firm's staff are principals.

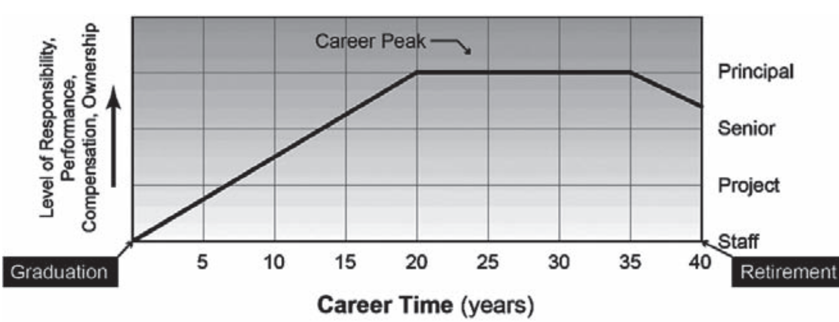

FIG. 5. Typical Career Trajectory for an Achieving Geoengineer in E&C Private-Sector Practice.

Many of the most successful practitioners in E&C geoengineering firms possess additional attributes beyond technical competency and project experience that help them to be successful in their careers. Table 16 provided a list of additional personal attributes that in the author's experience are often exhibited by young geoengineers who are on a track to become leaders in their organizations and in the profession.

Table 16. Common Positive Attributes of a Developing Geoengineering Practice Leader

No.	Positive Attributes
1.	Willing to work with passion for, and commitment to, projects, clients, and coworkers
2.	Bright and energetic, volunteers for assignments, good at multi-tasking; always willing to go the extra mile to get the job done
3.	Interested in a wide range of projects and work duties; eager to gain experience
4.	Creative thinker and lifelong learner; listens to supervisors, mentors, and coworkers
5.	Very good oral communicator and writer; able to organize and synthesize data, theories, and conclusions; works hard at becoming an excellent technical writer and oral communicator
6.	Demonstrates increasing levels of good judgment as career progresses
7.	Willing to take "non-engineering" duties seriously, such as health and safety, project management, risk management, and subordinate personnel mentoring and development
8.	Willing to jump in to solve a problem wherever found and fill a void if one exists
9.	Service and goal oriented, wants to help clients be successful in their enterprise
10.	Takes every opportunity to associate with and learn from academic, government, and industry leaders
11.	Active professionally, engaged with colleagues, contributor to, and cares about, profession
12.	Team player and valued colleague; helps others and shares successes

Figure 6 shows the same career trajectory as in Figure 5 (labeled as Geoengineer A) along with two other trajectories. For the Geoengineer B trajectory, the geoengineer exhibits more of the leadership attributes identified in Table 16 than does Geoengineer A. Geoengineer B also makes a greater investment of time and energy into his/her career compared to Geoengineer A. For Geoengineer C, the situation is the opposite. He/she exhibits fewer of the leadership attributes and makes a lesser time and energy career commitment compared to Geoengineer A. From Figure 6, it can be seen how factors such as leadership attributes and time/energy investment can influence the rate of career progression in a firm and the ultimate level of leadership achieved.

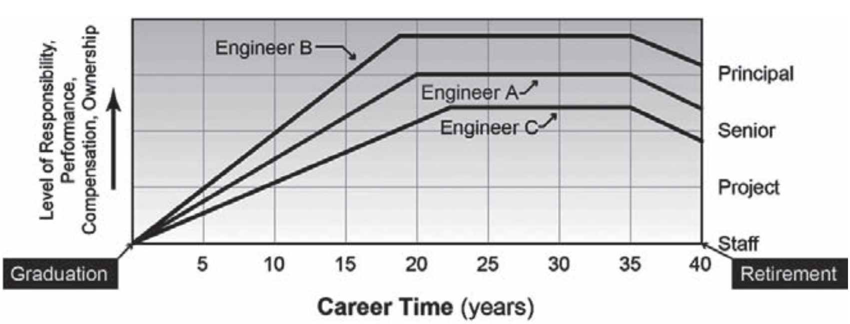

FIG. 6. Career Trajectories for Geoengineers Exhibiting Different Leadership Attributes and Time/Energy Commitment to Career.

One additional factor to consider with respect to career growth is whether the practicing geoengineer focuses his/her energies between practice and management. Figure 7 below shows the difference in focus and responsibilities for two hypothetical principals, with one focused on practice leadership and the other focused on management. E&C geoengineering firms need both types of individuals. The relative percentage of principals in either category, or along the spectrum of possibilities between the two, will depend in part on the technological approach (i.e., brains, grey hair, and procedures) discussed previously. Also, it is not uncommon to see individuals who provide both practice and management leadership within their firms.

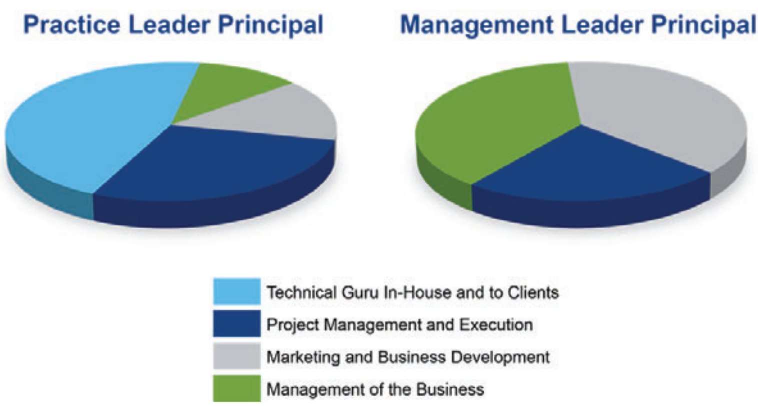

FIG. 7. Practice Leader versus Management Leader Career Track. (Note: Some firms have "technology" and "project delivery" leadership tracks compared to the "practice" and "management" leadership tracks in the figure.)

Taken together, Tables 15 and 16, and Figures 4, 5, and 6, provide a framework for considering career progression and focus in the private-sector practice of geoengineering.

7. GEOENGINEERING FIRM GROWTH

There is a conventional wisdom within the engineering business that firms either grow and evolve or they shrink and stagnate. The conventional wisdom holds that for a variety of reasons there is no such thing as "steady-state" for a professional services organization. With the exception of sole proprietorships, small partnerships, small, unique brains firms, and firms that are not concerned about their inter-generational continuity and well-being, the author believes this conventional wisdom to be true. Without some amount of growth it is not possible to build new capabilities and service lines to meet the evolving needs of clients, nor to provide growth and advancement opportunities for junior staff. If the staff leverage model described in the preceding subsection of this paper holds true, every time a geoengineer is promoted, one or more staff engineers must be recruited to maintain the leverage.

Similarly, and more impactful to growth, if a senior or principal level geoengineer is recruited into a firm, a number of staff engineers will need to also be recruited to maintain the personnel leverage needed to support the business model. For public firms and those with outside private-equity investors, the investors expect to see earnings growth (bottom line) which is mostly achieved through revenue growth (top line). As discussed in the previous section, ownership transition in employee-owned firms is easier to achieve in growing, vibrant firms compared to static firms.

With respect to growth, the chart in Figure 8 presents a useful framework for considering the opportunities and challenges for growth within a professional services organization.

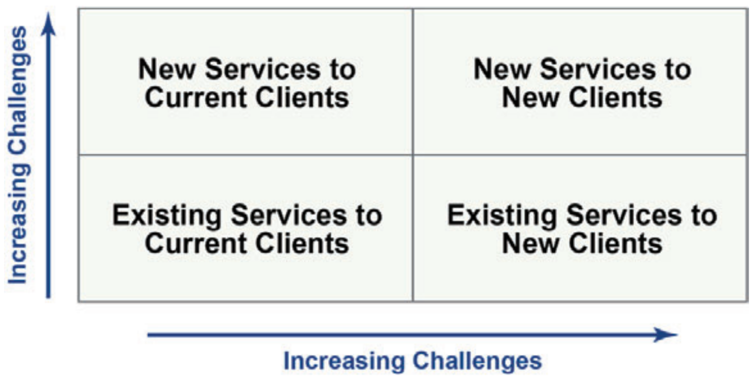

FIG. 8. Framework for considering business growth within a professional services organization.

From Figure 8, it can be deduced that the easiest way for a firm to grow is to sell more of the services it is well known for to the firm's current clients. While this is the easiest path to take, it is also one that has the potential to lead to stagnation of the firm, commoditization of the firm's business, and vulnerability for the firm associated with an insufficiently broad services offering and client base. Providing new services to current clients and providing existing services to new clients are well-established approaches for a firm to grow and evolve. They require more up-front effort and planning than a business model predicated on existing services to current clients; however, there is a logical progression to this model through either the existing client relationship or the recognized existing practice capabilities of the firm. The most challenging and difficult way for a firm to grow or evolve is through the provision of services that are being newly offered by a firm to prospective clients that have no pre-existing relationships with the firm.

In the author's experience, one of the most rewarding elements of the foregoing framework for firm evolution is in the building of new practice capabilities within an organization. The further the new practice capabilities are from the firm's existing core competencies, the greater the challenge. The more mature and established the market into which the firm intends to move, the greater the challenge. In the author's experience, building new practice capabilities in a firm to a high level of competency and proficiency takes substantial time and effort and can be accomplished in one of three ways:

- The first way is using an "*internal development*" model wherein new proficiencies are developed through the professional development and continuing education of existing staff. With this approach, new project opportunities may at first be modest due to the lack of corporate experience. More complex and challenging opportunities are only obtained progressively over time as project experience and a track record are developed. This process can take 5 to 10 years to bear enough fruit to be material to a business.

- The second way can be termed "*recruiting development*" and involves recruiting one or several leading, experienced practitioners in the new area to quickly create proficiency and experience within the firm. Recruiting the right professionals who are: (1) willing to be recruited; and, (2) have the right expertise, experience, personality, and values to fit into the new firm can be a huge challenge. When successful, this approach can lead to a material new business line within 2 to 5 years.

- The third way involves use of "*acquisition development*" wherein the firm acquires another firm that already has expertise and experience in the area that the acquiring firm wants to diversify into. While there are many challenges associated with achieving successful acquisitions, as discussed subsequently, they are increasingly common in the geoengineering profession. This approach can lead to a material new business line for a company essentially as soon as the acquisition closes.

Growth places a variety of stresses on a firm and these must be recognized and addressed to avoid potential negative impacts of the growth, including loss of company values and culture, loss of employee morale, work quality and/or delivery issues, and financial performance and cash flow issues. The starting point for managing growth is by recruiting the right people, both those just starting their careers and mid-career professionals that join the firm. Recruiting well is a tremendous challenge and an entire paper could be devoted to just that topic. The topic areas in this paper do, however, provide a conceptual framework for assessing the potential fit of a recruit with a firm: technological approach, business culture, values, areas of technical competency and client focus, education, leadership attributes, practice and/or management interests and focus, and willingness to make a substantial time and energy investment in his/her career. Once a person starts employment, energy and time need to be invested by the company in assimilating and integrating that individual so that he/she is able to become productive, efficient, and versed in the firm's work processes and products. The integration effort should be designed to build a collegial, team-oriented work atmosphere.

As a firm grows, it needs to create the systems for financial, quality and risk, human resources, IT, and health and safety management, and for professional and client development, that will allow the firm to continue to achieve its goals. Referring back to the discussion on cash flow and balance sheets, the process of growth consumes cash and leads to increasing working capital (AR and WIP) on the balance sheet. In the author's experience, as a rough rule of thumb, the revenue-based growth rate of a firm should be less than the firm's EBIBT in order to limit the cash flow impacts of that growth.

8. CLIENTS AND COMPETITION

Procuring Work: Geoengineering firms have a variety of ways of winning work with their clients and addressing marketplace competition from other firms for that work. A client may sole-source work to a firm on the basis of the firm's past performance for the client, special technical expertise, ability to meet schedule requirements, cost, or other factors. Sole-source awards are attractive for many reasons, including that they allow the firm to avoid some or all of the cost of competition (associated with preparing written proposals, making proposal presentations to clients, and discounting fees in an attempt to be competitive with other proposers) while having the opportunity to negotiate fees directly with the client. Most work in the geoengineering industry is not sole-sourced, but instead procured on a competitive basis. Typically, a client will seek technical and cost proposals from several geoengineering firms and make a selection based on a set of evaluation criteria. These criteria typically include technical approach to the scope of services, value added enhancements to the scope, qualifications and past performance, schedule, and cost. A good and fair competitive procurement in the author's view includes the following elements: (i) a limited number of pre-qualified proposers who are fully qualified and have good reputations for the quality of their work and services; (ii) proposal evaluation criteria that are weighted to qualifications, experience, technical approach, and schedule, with cost being only a secondary factor; and (iii) proposed contractual provisions that are fair and appropriate to both the client and the geoengineering firm (see subsequent section of paper). Procurements where many firms are allowed to propose, including firms that may be only marginally qualified, and where cost is the main selection criterion, are damaging to the geoengineering business sector as they have the potential to produce poorly performing projects, hurt qualified firms that could perform the project correctly, and result in disputes and claims between the client and geoengineering firm. In the author's view, the geoengineering profession has let too much of its work become captive to poor quality procurements. A number of professional organizations, perhaps most notably ASFE, have made this issue a major focus of their efforts.

With respect to "good and fair" procurements, many government contracting agencies use qualifications-based selection (QBS) to procure E&C firm design services. QBS involves owner ranking of proposing E&C firms based on competence, qualifications, and technical and management approach to the proposed scope of services, followed by negotiations between the owner and top-ranked firm on the final scope of services, schedule, and fee. The goal of QBS is to provide the project owner with the most competent and qualified services provider procured at a fair and reasonable price. The federal Brooks Act (PL 92-582, 1972) requires the use of QBS processes for federal government projects. Most, but not all, state governments have passed QBS "mini Brooks" statutes. In recent years, QBS has become less consistently applied to federal and state projects as some owner agencies have found ways to work around QBS requirements, or to bundle design and construction activities to obviate the QBS requirements. E&C firms today are often faced with procurements that involve the concepts of "best value", "performance-based contracting", "design-build", and others that depart from the principles of QBS. Particularly at the state, county, and local levels, some elected officials advocate for procurement reform and the concept of "low qualified bid". While this push is understandable in light of the severe revenue and budgetary challenges faced by many government bodies, the practical application of "low qualified bid" often results in selection of a marginally qualified firm working on the basis of an aggressively low bid, and producing inferior completed projects compared to projects performed under QBS. ACEC, ASFE, ASCE and others have worked diligently to protect and expand the use of QBS procurement procedures. The author believes that QBS is particularly appropriate in the geoengineering business sector due to the inherent uncertainties associated with the engineering characterization of subsurface geological conditions, coupled with the fact the geoengineering professional fees often represent a relatively small percentage of the total professional service fees for a project while the performance of the completed project is highly dependent on the quality of the geoengineering input provided.

Business Development Process: A distinction amongst geoengineering firms relates to how they carry out the business development function (i.e., how they get work). There are a few individuals within the profession with such strong technical reputations and/or client relationships that they can generate work simply by "waiting for the phone to ring". It is much more common, however, that firms must proactively seek out work through an organized business development effort. This effort will be influenced by the technological approach (Table 10) and business culture (Figure 4) of the firm, and the areas of technical focus (Table 11), types of clients (Table 11), and sizes of projects being pursued. The "sell/manage/do" approach to business development has senior engineering professionals in the firm taking the lead in developing new work and then managing the work and contributing to its execution. An extension of the sell/manage/do approach involves the firm's engineering professionals working as teams to systematically pursue development of multiple clients within a given client sector (Table 11). A third approach involves using business development professionals to lead the development of client opportunities, with the engineering professional providing secondary support to the effort. This latter approach is most common in large firms pursuing large, multi-disciplinary projects that require significant, multi-level, long-lead-time business development efforts. It is also common in businesses that mostly pursue procedures type projects that do not require significant specialized technical expertise.

Importance of Work Quality and Client Service: Needless to say, whatever the approach to business development, a geoengineering firm can only have sustained success if it executes that project well; to the complete satisfaction of the client; in a manner that is safe, meets the applicable project criteria and standard of care, and allows the firm's staff to professionally grow and develop their competencies; meets the project cost and schedule goals; and, results in a profit for the E&C firm and prompt payment of the firm's invoices. In doing all of these things well, the E&C

firm has laid the groundwork to build both long-term relationships with its clients and sustained business success. In the best case, continuing outstanding performance on subsequent projects will lead to higher and higher levels of confidence and trust on the part of the client and, likely, more opportunity for the geoengineering firm, sometimes under more favorable procurement processes. The geoengineering firm can often enhance this process further if members of the firm are able to build personal friendships with the client to complement their professional relationships. When the foregoing is accomplished, repeat business can amount to 80 percent, or more, of a firm's work. Project delivery excellence and client service are two hallmarks of the most successful E&C firms. A converse situation exists if a geoengineering firm struggles to successfully finish projects and completely satisfy its clients. In this latter case, the client is less likely to use the firm again. The firm is then faced with needing to find new clients, which takes time and is expensive. This latter E&C firm also doesn't have the benefit of being able to use its former clients as references. For this latter firm, the road to sustained health and profitability will be much more difficult than for the firm described at the start of the paragraph.

Importance of Good Clients: The importance of finding and keeping good clients cannot be over-emphasized. A firm cannot maintain a healthy workplace, recruit and retain high-quality staff, or maintain a profitable, sustainable business without good clients. In the author's experience, good clients share a number of attributes: they have ongoing, long-term needs for geoengineering services and they procure a substantial amount of interesting and challenging work; they value the geoengineering services they procure and treat their geoengineering consultants professionally and as informed members of the project team; they are ethical and are committed to doing their projects the right way; their procurement processes allow for sole-source awards, or at least QBS or "good and fair" competitive procurements, as described earlier; their contract terms and conditions are appropriate and reasonable and the scopes of services issued by the client are clear and well defined; they are willing to provide fair compensation for the services to be provided and they are open to compensation increases for increases in scope and schedule and for changed conditions; positive professional and personal relationships can be developed with them, communications are open and frequent, and expectations by all parties are aligned; and if issues arise, they are willing to engage with the geoengineering firm to proactively and constructively address and resolve the issues. Beyond these individual client attributes, it is helpful to geoengineering firms to have some market sector and geographic diversity in their client base. This fact can be seen in the 2008/2009 economic recession, when firm's narrowly focused in the residential and light commercial market sector struggled more than firms with more diverse clients in sectors such as transportation, water resources, flood protection, environmental remediation, oil and gas, power utilities, industrial development, and others. It is no easy task, but the senior management of geoengineering firms should approach their client development efforts with the notion of "continuous improvement" of their client base. While the goal is always to perform to the best of our ability for all clients and projects, when good clients are found, extraordinary efforts are called for to make those clients permanent.

9. PROFESSIONAL LIABILITY & RISK MANAGEMENT

Overview: The issue of professional liability for the geoengineering practitioner goes back to at least the late 1950's and 1960's and the issue still exists today. Remember the quotes from D'Appolonia (1983) cited earlier in this paper: "*In the late 60's, the situation became so serious that engineering firms shied away from construction supervision and inspection*", and "*You may recall that in the late 60's insurance companies would no longer insure geotechnical firms because of the high risk.*" The risk of professional liability claims remains today. The author suspects that not a year goes by when each firm in Tables 7, 8, and 9 does not need to address one or more actual or potential professional liability issues. Nonetheless, the professional liability climate for geoengineering firms today is better defined and more manageable than it was during the times described by D'Appolonia. This improvement was achieved through the initiatives of many organizations and individuals, including Terra Insurance, ACEC, and ASFE. In addition, senior managers of geoengineering firms, through training and experience, became more and more knowledgeable about professional liability risks, how to mitigate them, and, when a claim was threatened or made, how to proactively manage it. Many firms developed and embraced formal risk management programs. Alternative dispute resolution (ADR) involving direct negotiation, mediation, or arbitration became a more frequently used means of resolving disputes and is now frequently mandated in service agreements with clients and subcontractors. Risk/contract managers and legal departments have become an integral part of the senior corporate staff of many geoengineering firms. A strong general counsel can be of great value as a member of the senior management team in managing the firm's risk avoidance and claims management programs. Others playing a role in improving the situation included insurance brokers, insurance companies, and industry-focused consultants providing risk management training and advice (such as PSMJ Resources and ZweigWhite).

A recent paper by Brumund (2011) discusses a number of "geo-risks" potentially confronting geoengineering businesses. For the most part, the risk management topics covered by Brumund are different from those addressed in this section of the author's paper. The reader is referred to the Brumund paper for a discussion of: (i) risks associated with site characterization, modes of failure, and analytical tools; (ii) risks associated with large long-term projects; (iii) risks posed by rare, extreme impact events; and (iv) potential for personal professional liability of the geoengineer.

Risk Management Program: An effective risk management program must proactively address every aspect of project performance because each has the potential to give rise to claims and potential liability. The assessment of project risk must include the potential duties of, and impacts to, all relevant parties, including clients, contractors, construction workers, subconsultants, lenders, ultimate owners and users of the finished project, and third parties. Situations that may present risks to the geoengineering firm include: professional negligence (i.e., "errors and omissions"); pollution resulting from operations; unsafe work operations or worker physical or chemical exposures; risks associated with subsurface investigation activities (e.g., buried utilities); accidents involving employees, subcontractors, third parties, or others; third-party damage; contractual risk (e.g., indemnification obligations); subcontractor performance risks; risks associated with ill-defined scopes of work; risks associated with contractor implementing geoengineer's design; risks associated with corrupt practices; force majeure; payment risk; and client risks.

An effective risk management program requires the engagement and commitment of the firm's senior management team, general counsel, and all employees to address all forms of potential risks. The program components will typically be described in the firm's risk management, quality management, and health and safety plans. The challenge in most firms is not so much in preparing the plans themselves, but in having all employees embrace the plan contents and apply them consistently and methodically. At every level of the organization, employees have a role, whether it is a high level risk evaluation of a project opportunity, a senior engineer's review of the data, methods, and assumptions that will be used by project and staff engineers for an analysis, one staff engineer checking another staff engineer's detailed calculations, or a field crew holding a safety tailgate meeting at the start of each day of field work and reviewing the day's activities to identify potential safety or other issues. Effective risk management must produce a continuum of good practices, good decisions, and good responses to problems and challenges when they occur. This is no easy task. Table 17 provides a one-page summary of basic risk management concerns that project managers at the author's firm use as a day-to-day reminder of the many elements that go into an effective project risk management program. Each of the items in the table is supported through additional information contained in the firm's risk management plan, quality management plan, and health and safety plan, all accessible through a project delivery portal on the firm's intranet site.

Mitigating Project Risks: A firm's risk management program needs to address every stage of project performance from identification of project opportunities through procurement and contracting, execution, and work product delivery. As already noted, Table 17 provides a one-page document designed as a project risk mitigation guideline for geoengineering project managers. The discussion that follows addresses a number of points from the table.

Project Identification: At the project identification stage, the geoengineering firm must assess whether the scope of services to be executed is in its area of technical competency and that it has the expertise, experience, and resources to perform the work to the applicable standard of care. The firm must also assess whether it has the availability of staff to take on the project (i.e., is the firm too busy) and if any real or perceived conflicts of interest exist with respect to undertaking the assignment. During this initial phase, the geoengineering firm needs to conduct business due diligence on the prospective client, including: business strength and reputation; prior history using geoengineering firms; likely approach to procurement; and existing relationships of firm personnel with the prospective client. The firm must also consider subcontracting needs, certified business requirements, and any unusual risks or hazards posed by the project. The firm may make a go/no-go decision on the project opportunity at the project identification phase. An incomplete or badly executed project identification process may start the firm down the path of a high risk engagement. To aid their staff in the go/no-go evaluation process, firms should develop written project opportunity screening criteria and guidelines.

Table 17. Project Risk Mitigation for Geoengineering Project Managers

1. RELATIONSHIPS	**MAKE THEM SOUND** • Select clients who are ethical and financially responsible. • Create expectations that are realistic and achievable • Choose qualified, experienced, and insured subcontractors • Execute approved written contracts and subcontracts • Make sure all contracts comply with company policies
2. PROJECTS	**BE SURE THEY'RE ACCEPTABLE** • Confirm qualifications to perform the project • Verify compliance with state licensing requirements • Ensure qualified managers and staff will be available • Document scope and be sure it is adequate for the project • Include project contingencies in budgets and schedules
3. RISKS	**MAKE THE CLIENT AWARE** • Evaluate inherent project risks and discuss with client • Expressly state important cost and technical assumptions • Advise client that actual conditions may vary • Give prompt notice of any projected cost overruns or delays
4. PERFORMANCE	**COMPLY WITH APPLICABLE REQUIREMENTS** • Understand and comply with contract objectives • Know and comply with applicable laws and regulations • Meet or exceed applicable standards of care
5. COMMUNICATIONS	**FREQUENT AND IN WRITING** • Document milestone achievements • Discuss and document important decisions with client • Respond appropriately to all project communications • Communicate critical information to project team • Comply with retention policies for project documents
6. OPINIONS	**BE SURE THEY'RE SUPPORTABLE** • Acquire adequate data to support opinions • Qualify opinions to reflect limitations in data • Avoid use of red flag words • Be alert to third party reliance issues
7. QUALITY	**ACHIEVE QUALITY OBJECTIVES** • Regularly review Work Flow Guidance and QMP • Communicate quality requirements to project team • Conform services to quality management program • Follow peer review procedures
8. HEALTH & SAFETY	**COMPLY WITH PROGRAMS** • Make safety a top priority • Follow all applicable health and safety procedures • Seek help from health and safety staff
9. INCIDENT MANAGEMENT	**SEEK HELP IN DEALING WITH PROBLEMS** • Report any potential problems promptly to legal department • Follow incident reporting procedure • Develop coordinated response action plan

In considering whether to pursue a project, a firm must evaluate its ability to perform the work in accordance with the applicable standard of care. ASFE defines the standard of care as *".... the level of skill and competence ordinarily and contemporaneously demonstrated by professionals of the same discipline practicing in the same locale and faced with the same or similar facts and circumstances."* As discussed by Lucia (2012), the standard of care for most professional practice situations is not contained in any consensus documentation, but is instead typically defined by a jury or judge based on the competing testimony of expert witnesses in the context of specific legal disputes.

Project Procurement and Contracting: At the project procurement phase, the geoengineering firm must evaluate the process the client will use to procure services. Sole source, QBS and "good and fair" (as described earlier) procurements are preferred. Procurements with the negative attributes described in the previous section pose a potential for professional liability risks as the procurement reflects the client's attitudes about the value of the work, the skill and professionalism of firms proposing on the work, and the importance of low cost over quality. At the same time, the geoengineering firm must evaluate the contract terms and conditions (Ts&Cs) that will govern the project. Geoengineering firms should have their own standard contracts and attempt to have prospective clients agree to use them. Often, however, clients will want to use their own contracts. In the author's experience, it is a significant "red flag" if a prospective client will only provide the proposing firm with their contract Ts&Cs after submittal of proposals. The geoengineering firm should insist that the proposed contract be available for review early in the procurement process, so that the Ts&Cs can be evaluated, and, if necessary, negotiated to mutual acceptance, and so that the contract elements (such as payment terms, limits of liability, insurance limits, etc.) can be appropriately priced into the project. ASFE, ACEC, and Engineering Joint Contract Documents Committee (EJCDC) websites provide sample contract language that can be helpful in preparing or reviewing contracts. Use of specialized contracts staff and/or counsel experienced in engineering and construction contracting and knowledgeable of the firm's business is highly encouraged for review of any non-standard Ts&Cs. Particular attention should always be paid to Ts&Cs related to: scope and schedule provisions; financial terms and payment provisions; standard of care descriptions; third-party reliance provisions; provisions relating to risk allocation such as indemnification and limitation of liability; change order procedures; concealed conditions; delay and force majeure provisions; termination procedures; and choice of law, venue, and dispute resolution provisions.

Project Execution: Successful project execution involves conducting the project to satisfy the scope of services and contract provisions in a way that is professionally competent and meets applicable statutes, codes, regulations, and other requirements. The work must be done safely and in conformance with budget and schedule requirements. The author believes in the value of project management and quality management training and systems to help the professional staff of E&C firms achieve these project delivery goals. Tools such as written work flow guidance and intranet based project delivery portals can be used to integrate a firm's project, risk, quality, and health and safety processes.

It is essential during project delivery to keep the client informed of progress and any issues that arise. It is also important to fully comply with the contract provisions, especially with respect to progress reporting, budget and schedule forecasting, and prompt written notice of any changed condition that may lead to a change in the project scope, budget, and/or schedule. In the author's experience, E&C firms regularly fail to achieve prompt written notice when issues arise, instead communicating only orally, often only to a single client representative, and too frequently, not very promptly. Oral communication with the client is essential, but failure to meet contractual requirements for prompt written notice can result in liability and/or the loss of substantial rights, such as the right to an increase in compensation. It is also important for E&C firm professionals to develop the skills needed to be able to counsel their clients at every step of the project and influence their expectations appropriately with respect to the uncertainties and limitations associated with subsurface geological conditions. The failure to properly counsel a client is a major factor in leading to client disappointments. The ability to effectively communicate uncertainties to clients in a manner that clients appreciate is a very important consulting attribute.

Project Deliverables: Deliverables for geoengineering projects may include: site investigation reports; geotechnical baseline reports; database and GIS packages; calculation and analysis packages; study reports; design plans and specifications; permit application packages; construction bid document packages; construction observation, material testing, and/or quality assurance reports; and, forensic engineering reports. The preparation of high quality work products must be a passionate objective of geoengineering firm personnel. A high quality deliverable has three intrinsic components: (i) the results of a properly executed assignment that meets all of the scope, standard of care, contractual, and other requirements of the work, including all requirements for data evaluation, methodology evaluation, and peer review; (ii) a very good presentation in terms of organization, level of detail (appropriate to the nature of the work), and quality of text, figures, tables, and appendices; and (iii) a clear statement of the assumptions, limitations, and appropriate use of the work product.

During the author's career, he has seen many work products from many E&C firms that do not contain these three intrinsic components. The situation with respect to written work products (primarily reports) may actually be getting worse due to the increasingly fast pace of the industry and increasing budgetary constraints on many projects. All this leads to less effort devoted to preparing deliverables, an increasing tendency for engineers to draft reports electronically, and an increasing tendency for reviewers to provide their edits and comments on the draft electronic deliverables in "track change" mode. The latter two tendencies are particularly worrisome as they undermine the traditional process for young engineers to become good technical writers. The author has long believed that young engineers need to edit their draft reports through 3 to 4 versions before the reports are good enough to be given to their supervisors for review. It is this process of editing and correcting one's own work that leads to improvement. Traditionally, a supervisor would review the draft report using a red pen for mark-ups and the review comments would be discussed in a face-to-face meeting between the supervisor and more junior engineer. The junior engineer would then need to implement the changes and edits and, in so doing, take the time to understand why they were needed. This is a time-consuming, sometimes painstaking, but highly valuable teaching process. Today, however, it is too easy for the junior engineer to compose a report on a computer, send it (often before it has undergone sufficient levels of review by the junior engineer) by email to the senior engineer for review, receive edits and comments back in track change mode, briefly consider those edits and comments on his/her computer screen, and then click his/her mouse on the "accept all" icon. To the extent this is happening, the lack of self-review and face-to-face mentoring reflects a decline in professional practice.

Response to Potential Problems: Inevitably, even when every effort is made to identify, procure, deliver, and complete projects in accordance with the foregoing discussion, a problem arises, which leads to a disappointed, unhappy client, a payment dispute, or a claim for damages. The best way to address such situations is proactively with the involvement of senior management personnel not involved in the project. Whenever a project manager believes that an event has occurred that could give rise to a claim by the client, a contractor, a subcontractor, or a third-party against the E&C firm, regardless of the apparent validity of the claim, the project manager should immediately get the firm's senior management involved. To achieve this goal, the project manager needs to feel secure that the firm's response will be helpful and supportive, and not punitive. By bringing the matter to prompt attention of senior management, the project manager is taking the action that helps bring expert and objective assistance in thinking with respect to the best course of action at a time when the direction of the matter can most be affected. In addition, the action allows for the prompt notification of the firm's professional liability insurer.

Insurance: Professional liability insurance represents an important element of the risk management program for geoengineering firms. This insurance is customarily provided on a "claims made" rather than "claims occurrence" basis. This simply means that the covering policy is the one in effect at the time a claim is made, not the one in effect at the time of the underlying occurrence. This insurance is more widely available to E&C firms today than it was prior to the early-to-mid 1980s, due in large part to the improved professional liability claims practices and loss record of geoengineering businesses. The cost of such insurance has also moderated somewhat, particularly over the past decade. Today, the cost of professional liability coverage for medium to large geoengineering firms averages about 0.3 to 1.0 percent of the firm's gross revenues, with the policy providing per claim coverage equal to about 5 to 15 percent of the firm's gross revenues (typically higher percentages for smaller firms). The actual rates paid for the insurance will depend on the specific nature of the insured's business, its locations and claims history (i.e., historical loss ratio), the amount of per claim and aggregate coverage, the size of the policy deductible, and other factors. It is also noted that today, professional liability coverage for geoengineering firms frequently incorporates pollution liability coverage (i.e., coverage for clean-up costs for pollution arising out of the policyholder's covered operations). In the insurance industry, this combined insurance for E&C firms is called contractor operations and professional services (COPS) coverage.

10. MERGERS & ACQUISTIONS (M&A)

The author started his professional career at the geoengineering firm Woodward Clyde Consultants (WCC) in 1981. At that time, he implicitly assumed that the firm was a permanent fixture of the geoengineering landscape, as were other leading geoengineering firms of the day (Table 1). Approximately 25 years later, WCC became part of URS (in 1997) and many other firms having their roots in the geoengineering business sector have been acquired as illustrated by Table 18. This table was assembled from the files of the author and from those of Rogers (2011). EFCG survey data indicates that over the past decade, the percentage of E&C firms from their survey group involved in making acquisitions increased from about 40 percent to more than 80 percent for firms with revenues greater than $1 billion and from less than 20 percent to nearly 40 percent for firms with revenues less than $1 billion.

Table 18. Representative Listing of Acquired Geoengineering Firms

Seller	Acquirer	Year Acquired
ETCO	Woodward-Clyde	1972
Joseph S. Ward	Converse Consultants	1978
Leedshill	Woodward-Clyde	1982
Leroy Crandall	Law	1982
McClelland	Fugro	1987
S&ME	Westinghouse	1987
Ben C. Gerwick	COWI	1988
Chen & Associates	British Holding Company	1989
McBride, Ratcliff	Raytheon	1991
Kaldveer & Associates	Harza Engineering (CA)	1992
Chattahoochee Geotechnical	EMCON	1993
Wahler	Rust International	1993
Geospectra	Kleinfelder	1994
Wehren	EMCON	1994
Levine Fricke	Recon	1995
Lindvall Richter	Harza	1995
Roger Foott	GEI	1995
Earth Tech	Tyco	1996
Woodward-Clyde	URS	1997
Alton Geoscience	TRC	1999
Dames & Moore	URS	1999
EMCON	IT Group	1999
EQE Int.	ABS Group	1999
Mark Group	Harza	1999
Raamont	GZA	1999
Agra Earth & Environmental	AMEC	2000
Harding Lawson	MACTEC	2000
Lockwood-Singh	Exponent	2000
Lowney	TRC	2000
Harza Engineering	Montgomery Watson	2001
Ardaman	Tetra Tech	2002
Law Cos Group	MACTEC	2002
Subsurface Consultants	Fugro	2002
TAMS	Earth Tech	2002
Geologic Service Corp.	Kleinfelder	2005
Capozzoli and Associates	GeoEngineers	2007
Gore Engr.	Ardaman	2007
H.C. Nutting	Terracon	2007
Jaworski Geotechnical	Terracon	2007
Soil Testing Engineers	Ardaman/Tetra Tech	2007
STS Consultants	AECOM	2007
Vector Colorado	Tetra Tech	2007
William Lettis	Fugro	2007
Bryan Stirrat	Tetra Tech	2008
DCM Engineering	GeoEngineers	2008
Earth Tech	AECOM	2008

Fuller, Mossberger	Stantec	2008
Geomatrix Consultants	AMEC	2008
LFR	Arcadis	2008
Moore & Taber	Layne GeoConstruct	2008
Aquaterra	Terracon	2009
Gallet & Associates	Terracon	2009
Jacques Whitford	Stantec	2009
Load Test	Fugro	2009
ASC Geosciences	KCI Technologies	2010
Duane Miller & Associates	Golder Associates	2010
Geotechnical Engr. Group	Terracon	2010
Jordan, Jones, & Goulding	Jacobs Engineering	2010
Midwest Testing Laboratory	Terracon	2010
QORE	S&ME	2010
Treadwell & Rollo	Langan	2010
Bromwell Carrier (BCI)	AMEC	2011
MACTEC	AMEC	2011
Nordase & Associates	Terracon	2011
BCCM Engineering	S&ME	2011

As projects have grown larger and more complex, and as the industry has continued to grow and evolve (Figure 3), the pace of M&A activity involving geoengineering firms is increasing in parallel and must be recognized as a major force shaping the private-sector practice of geoengineering. Buyers and sellers have different motivations for entering M&A deals. For buyers, reasons include: (i) acquiring new capabilities and/or business locations; (ii) acquiring access to new client sectors and/or specific clients; (iii) gaining size and scale to achieve competitive advantage and/or pursue larger projects; (iv) developing turn-key service capabilities; (v) putting underutilized assets or investments to work; and (vi) increasing the firm's stock price if the accounting for the deal is immediately accretive to earnings and the firm's stock price is a function of earnings (especially true for public companies). For sellers, sale of the firm represents a way for shareholders, owners, and investors to monetize their investment in the firm. In the author's experience, tightly-held employee owned firms (i.e., firms where one person or a relatively small group of people own a significant percentage of the stock) frequently choose to be acquired as the owners near retirement because the ownership in the firm is not broad enough, and ownership transition planning has been insufficient, to support internal ownership transfer at a share valuation attractive to the owners. Firms of every size may choose to be acquired by a larger firm to gain access to larger, more diverse projects, to leverage their skills throughout a larger company, to provide more opportunity for their staff (particularly if their firm has struggled to grow and evolve), and/or to address financial problems or reduce financial risk, if they exist.

The author believes the pace of M&A seen in the industry today will continue for the foreseeable future. Of necessity, the boards of directors and senior management teams of private sector geoengineering practice firms need to think through their long-term positions on both acquiring and being acquired. The position of the company on these questions will influence every aspect of a firm's deliberations and strategic planning with respect to growth, financial performance, balance sheet management, investments in internal systems, and ownership structure, valuation, and transition.

The author also believes that an industry experiencing active M&A creates opportunities for firms choosing the path of organic growth (i.e., growth through recruiting and internal development of personnel). As large firms make acquisitions and grow even larger, their business emphasis shifts to larger projects and programs which they need to meet their revenue goals. This trend creates opportunities for other, typically smaller, firms to grow into those parts of the market being de-emphasized by the large acquiring companies. In addition, M&A activity often has an impact on the stability of some employees in acquired firms. When a firm is acquired, it must, over time, adapt to (if not completely adopt) the business and personnel culture and processes of the acquiring firm. Career advancement opportunities for personnel in the acquired firm may be better or worse after the acquisition. Commonly, there are personnel that end up happy after the acquisition and others who are not. Individuals in this latter category may never have considered leaving their firm prior to being acquired, but decide after the acquisition to leave because the acquiring firm is not as professionally and/or financially satisfying to them as their old firm.

11. FUTURE DIRECTIONS OF GEOENGINEERING BUSINESS SECTOR

The author believes that the next 10 to 20 years will see continuing excellent opportunities for geoengineers in private-sector practice to work on increasingly complex, interesting, and diverse projects. The author also believes that the next 10 to 20 years will see continuing opportunities for companies engaged in the practice. However, with the possible exception of firms serving the offshore oil, gas, mining, and energy industries (where the offshore setting creates substantial capital investment demands for ships, submersibles, drilling equipment, and geophysical equipment), the author does not see a comeback of very large geoengineering firms in the E&C industry, as would be reflected by an ENR Top Design firms list with very large firms reporting as GE (analogous to the list for 1975 described earlier in this paper). Geoengineers in private-sector practice will continue to be housed in full service and multi-service E&C firms, in small- to mid-size geoengineering firms, and in sole proprietorships. The list below presents additional thoughts of the author regarding the direction of geoengineering business practice in this timeframe.

- The geoengineering discipline will continue to evolve at a fast pace and geoengineering businesses will continue to find it a challenge to keep up with technical developments and incorporate those developments into practice.

- Geoengineering technology advancements will continue to occur across virtually all areas of the practice (Table 11). In addition, new technologies that have the potential to have a transformative impact on the practice may emerge. These include: geostructural sensors and sensing system technologies; geophysical investigation technologies; remote sensing technologies; information and cyberinfrastructure technologies; biogeoengineering technologies; and nanotechnologies. NRC (2006) describes a number of these technologies and the potential they hold for application in geoengineering practice. The development of innovative geoengineering materials (e.g., geosyntehetics) and systems (e.g., MSE walls and slopes) will continue at a rapid pace. The application of sustainability principles to geoengineering design and construction will increase.

- Notwithstanding the problems presently facing the U.S. and many other countries with respect to national debt and unemployment, the critical needs of infrastructure development and rehabilitation, drinking and agricultural water supply, oil, gas, and mining exploration, production, and delivery, electrical energy generation (including renewables) and transmission, flood management and control, natural hazard assessment and mitigation, and industrial capacity expansion will continue to generate large, complex projects having significant geoengineering components. This will create continuing opportunities for geoengineers to work on these projects. The impact of the overall poor economy on this portion of the geoengineering profession will likely continue to be relatively limited. In contrast, it may take 3 to 5 more years (from 2011) before geoengineering opportunities related to residential and commercial development rebound substantially from the anemic levels of the 2008/2009 recession.

- As the certainty of anthropogenically-driven global warming becomes more politically accepted, and the long-term impacts to climate, weather, sea level, natural systems, and social systems, become better defined, geoengineers will be integral to the development and implementation of mitigation measures that involve protecting and adapting virtually all forms of the civil infrastructure.

- The contracting practices of owners wherein responsibility for total project execution and financial risk is placed with a single firm, coupled with the increasing size and complexity of major urban, industrial, and infrastructure projects, will result in the continuing market dominance (in terms of revenues) of the very large EA, EC, and EAC firms, such as those listed in Table 6. The in-housing of geoengineering resources in these firms will continue.

- Other firms that desire to compete for the largest, most complex projects will pursue aggressive growth and acquisition strategies to reach their goals. This will be one factor that continues to drive M&A activity in the E&C business sector. These firms may also increasingly elect to grow or acquire in-house geoengineering resources.

- For firms that are not of a size or capacity to take on the largest and most complex projects, they will need to develop a subcontracting strategy attractive to the prime EA, EC, or EAC. Participation could be by offering services that are not in-housed by the prime, such as: drilling and sampling, in situ testing, laboratory testing, structural integrity testing, and foundation load testing; through certified business designations (i.e., small business); or, through specialized technical expertise not in-housed by the prime.

- As the largest firms continue to grow, more opportunities will be created for the next tiers of firms (in terms of revenues) to serve as the prime on large, complex projects for which the largest firms are not well suited or decide not to pursue. These types of projects will provide opportunities for firms of the size and type represented in Tables 8 and 9. These firms will need to execute strategies for achieving a competitive advantage for the work they pursue and they will need to be willing to accept or address increasingly complex contracting and risk transfer terms and conditions from owners.

- There will be increasing demand from owners, regulatory and review agencies, and the largest EA, EC, and EAC firms for high level, specialized technical expertise across the areas of geoengineering competency identified in Table 11. Firms that incorporate this type of expertise into their business model will have a significant positive differentiator when competing for projects where high level expertise is needed. Using the parlance from earlier in the paper, this could involve building a brains-focused firm, or a grey-hair firm with discrete, embedded brains practices.

- The market for procedures-focused firms, such as those focused on subsurface drilling, laboratory testing, construction materials testing and observations, and routine geoenvironmental and environmental services, will continue to be very competitive and driven by low cost. Government certified businesses will have a clear advantage for government procurements requiring that a portion of the work be done by such firms. Successful firms in this category will look to become more efficient to obtain competitive advantage. Others may try to develop specialized or unique services and/or tools they can offer clients to differentiate themselves from competitors and bring a value-added component to their procedures-based work.

- Ownership transition for employee-owned firms will continue to be a challenge for the reasons described earlier in this paper. However, improving management practices and increasing recognition of the important of long-range planning for ownership transition will make it easier to achieve that goal for those firms choosing the path of inter-generational transfer of company ownership.

- M&A activity will continue at an active pace within the E&C industry, regularly changing the corporate and competitive landscape, enhancing or disrupting careers, and resulting in the movement of people between companies. In 10 to 20 years, Table 18 will contain many more firms and the lists of firms in Tables 7, 8, and 9 will look much different.

- In 2011, the oldest of the baby-boomers turned 65. Over the next 10 years, many baby-boomer geoengineers will reach that age. Some will opt for retirement, but many others will continue to work due to good health, continuing professional interest, and/or financial necessity. While some will stay fully engaged, others will slow down to become full-time or part-time senior consultants in their companies. Others still will choose to become sole proprietors, working out of their homes. Geoengineering firms that can successfully incorporate these experienced resources into their businesses will be at an advantage.

- Firms will spend more and more time on the recruiting, development, and retention of staff. Competition for the most promising college recruits will be stiff. Firms will need to invest in their recruiting programs, including investing in, and building relationships with, the universities from which they recruit. Increasingly, they will need to work at making themselves attractive to recruits by providing exciting project, career advancement, professional development, and financial reward opportunities tailored to the performance of each individual employee.

- It is presently unclear to the author how the "Millennial generation" (born after about 1982) will influence and change the geoengineering industry. According to Schreiner (2011), as a set of generalizations, Millennials are the first generation that essentially grew up in the digital age. They work and communicate on line, with texting, instant messaging and social networking. They tend to think "outside the box" more than previous generations. They are comfortable working in teams and care about social issues, sustainability, and climate change. They also care about work-life balance and have been characterized by some as having a lower work ethic compared to earlier generations. They have also been characterized as being comfortable with a "job-hopping" approach to their careers. While these characterizations may be overly general, E&C firms will need to recognize and address them if their Millennial employees are to achieve their potential and maximize their contributions to their firms. At the same time, as with earlier generations, it is likely that as they grow and mature within their firms, and take on, and internalize, responsibility for the success of the firm and wellbeing of the firm's employees, the Millennials will accept and be up to the challenge.

- Issues related to commoditization of the profession and competitive pressures on professional service fees will not go away and may become worse. The role of professional associations, including as ASCE, ASFE, and ACEC will become increasingly important, in developing policies and positions, and driving them into the marketplace, will become increasingly important.

- For geoprofessionals, the establishment of the ASCE Geo-Institute (GI), and more recently, the start of the Academy of Geoprofessionals (AGP), have been important recent steps in "raising the bar" to address issues related to professionalism, commoditization, and pricing. It remains to be seen how far GI and AGP can go in addressing issues facing the profession. It also remains to be seen how ASCE Policy Statement 465 (Academic Prerequisites for Licensure and Professional Practice) will ultimately impact the profession.

- Related to "raising the bar", lifelong learning and continuing education will become more important for, and time consuming to, geoengineering practitioners. These efforts will go well beyond the current necessity of gaining professional development hours (PDHs) to maintain professional registration. The author believes that practicing geoengineers will increasingly avail themselves of the growing array of online graduate civil engineering programs and courses. Geoengineering companies will need to integrate this expanded level of professional development activity into their company operations.

As a final thought, the most successful geoengineering firms will continue to be the ones that can see where their opportunities lie in the marketplace, develop and implement a strategy to be successful with those opportunities given client spending levels and competition, achieve project delivery and client service excellence in their work, create a compelling work culture and environment that is attractive to both existing staff and recruits, and do all the other things, many of them discussed in this paper, that are needed to build a successful, sustainable company. The leaders of these firms will need to continue to embrace the inevitable continuum of change in societal needs for geoengineering services, marketplace drivers on those services, available technologies to use in the profession, competitive practices and pressures, and personnel, management, and financial priorities and challenges. Change is inevitable and E&C firm leaders will need to be forward looking and energetic in helping their firms achieve continuing success in this dynamic business environment.

12. CONCLUSIONS

This paper has presented the author's perspectives on the state of the industry with respect to private-sector businesses providing geotechnical and geoenvironmental services to clients. The paper discusses: the history and evolution of the business sector; size and attributes of the sector; business models and practice characteristics of companies in the sector; and operational, management, and personnel priorities and challenges of these firms. The paper also addresses: career considerations for personnel in this business sector; the topics of clients and competition, risk management and contractual liability, and mergers and acquisitions; and the author's thoughts on future directions for the business sector over the next 10 to 20 years. The *practice of geoengineering* carried out by private sector firms has never been more complex, interesting and diverse. Future opportunities for geoengineering practitioners are excellent. At the same time, the private sector *business of geoengineering* is increasingly complex, competitive, and challenging. Sustained success is being achieved by both large and small firms that can recruit and retain great people, deliver superior work products and solutions to their clients, perform their work efficiently and at a reasonable cost, and navigate both the competitive and risk management landscapes. Sustained success will also require that the firm be properly managed with respect to all of the topics described in this paper. The leadership of these firms will need to accept and embrace the inevitability of change and help their firms achieve continuing success in a dynamic business environment. In this environment, they will need to continually improve their firms and in so doing, improve the profession.

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Recent Advances in the Selection and Use of Drilled Foundations

Dan Brown[1], M.ASCE, Ph.D., P.E., DGE

[1]President, Dan Brown and Associates, PLLC, 300 Woodland Road, Sequatchie, TN 37374; Phone: 423-942-8681; dbrown@danbrownandassociates.com

ABSTRACT: The construction and design of drilled foundations in recent years has been most significantly affected by developments in drilling techniques related to materials, equipment, and generally improved capabilities in construction. In addition, advancements in technology for testing and QC/QA have resulted in improvements in performance, reliability and design. This paper describes some of the most significant developments affecting drilled foundations, including large diameter drilled shafts, continuous flight auger piles, drilled displacement piles, and small diameter micropiles.

INTRODUCTION

The drilled foundation options available in current practice include an incredible range of available technology, from very small diameter micropiles only a few inches in diameter to large drilled shafts that may be as large as 4m (13ft) in diameter. These foundations share a common feature in that the foundation is constructed by drilling a hole into the bearing formation and constructing the foundation into that hole by placing a cementitious material such as grout or concrete. This critical part of the structure is thus cast *in-situ* rather than prefabricated and installed into the ground as with a driven pile. (note: although helical anchors might also be considered "drilled" foundations, these will not be included as this paper already covers enough ground just dealing with cast-in-place drilled foundations!).

The differences between types of drilled foundations relate mostly to the method of installation and how the casting operation for the pile is completed, although this relates directly to the equipment used to construct the foundation. This paper will discuss the state of practice of drilled foundations in a way that is consistent with each particular drilled foundation type, as follows.

Micropiles are most often 30cm (12in) or less in diameter and often selected for use because of the advantages provided by the lightweight and maneuverable equipment available to install these piles. The distinguishing feature from a design perspective is that the pile itself is typically designed as a steel member such as a bar or tube which is bonded to the bearing stratum with a cement grout. Micropiles are most effectively used where the bearing materials allow effective utilization of the high strength of the steel. Granular soils or rock often provide suitable bearing formations and micropiles are often used in rock or in highly variable conditions where difficult drilling may be encountered.

Continuous Flight Auger Piles (CFA piles) are typically 30 to 100 cm (12 to 40 in) diameter and most often selected for use because of the advantages provided by the speed and cost-effectiveness of the installation method and equipment. Often called “augered cast-in-place (ACIP)” or “augercast” piles in U.S. practice, the distinguishing feature of the construction of these piles is the fact that the concrete (sometimes a sand-cement mix) is placed through the hollow center of the continuous flight auger drill string as the augers are withdrawn and then the reinforcement is placed into the wet fluid mix after the casting operation is complete. The pile is thus a reinforced concrete structural element and designed accordingly. CFA piles are usually most cost effective when used at lengths of 10 to 30 m (30 to 100 ft) and constructed entirely in soils, although occasionally these piles are used in weak rocks. Because of the speed with which the pile can be drilled and completed, it is not uncommon for a constructor to install several piles within a single hour of work.

Drilled Displacement Piles are constructed using a technique similar to CFA piles, but using tooling and more powerful equipment such that the drill tool is advanced while displacing the soil to form the hole rather than extracting the soil. These piles provide the obvious advantages that ground improvement is achieved during installation in some types of soils, and the handling and removal of spoils (which may include contaminants in some situations) is avoided. With the controls and monitoring equipment available on modern drill rigs used for these piles, there has also been progress in relating the torque and crowd pressures to the stratigraphy so that the performance of a specific pile can be related to installation measurements.

Drilled Shafts are most often 1 m (3ft) or more in diameter and constructed by excavating and stabilizing a hole into the bearing formation (often with drilling fluid and/or steel casing) followed by placement of reinforcement and then concrete. In this way, large diameter foundations are constructed which can transfer forces to deep, competent bearing strata and provide very large axial and lateral resistance. With the capabilities of modern equipment to install large drilled shafts and the improved testing capabilities for verification of structural integrity and geotechnical performance, the use of a single drilled shaft to support a single column is often used to maximize the foundation capacity in the smallest possible footprint.

This paper describes some of the most significant developments in the last 20 years affecting the selection and design of each of these types of drilled foundations. Selected examples from the author’s experience are included to illustrate the capabilities and use of modern drilled foundations, along with the factors influencing the selection of a specific drilled foundation type.

MICROPILES

The most significant advancement related to the use of micropiles for foundations and stabilization within the last 20 years has been the development and acceptance of practical design and construction guidelines, which in turn has lead to the adoption of micropiles in building codes and public works projects. The use of these foundations has grown dramatically during this period, and the popularity of this type of drilled foundation is largely the result of the versatility of the equipment (as shown in Figure 1) used to install them. Micropiles are used for applications including underpinning and seismic retrofitting and in locations and ground conditions where more conventional deep foundations would be difficult or impossible to construct.

a) Foothills Bridge, East Tennessee b) World Trade Center, New York City

FIG. 1. Micropile Drill Rigs in Restricted Access Locations.

Besides the ability to overcome difficult site access, micropiles can be drilled to provide good foundation support into materials which are impossible to penetrate with driven piling or which represent extremely difficult drilling conditions with larger diameter drilled foundations. Examples include piles through boulders, fills including rubble or other hard debris, and karstic formations in hard limestone. The micropiles may be advanced through porous layers with casing until a substantial thickness of sound material is penetrated, and then the casing partially withdrawn to form a permanently casing through the pervious strata and allow the pile to be grouted into the sound layer.

In a recent example, micropiles were installed to underpin a building for an industrial facility in Alabama. The single story building had been supported on steel H piles which had been driven to refusal on limestone layers within a zone of epikarst (weathered limestone) which had subsequently settled during a period of extreme drought that resulted in a drop in groundwater. Micropiles were used to penetrate into the underlying limestone, through karst solution features.

Micropile Design Details

Typical micropiles incorporate a tubular steel element or solid bar (sometimes multiple bars) which is grouted into rock or strong soil bearing stratum with a permanent steel casing extending through the overlying weak soils. Corrosion protection of the interior steel is provided by the grout and casing, and epoxy or galvanized coatings may be used in aggressive environments. In extreme cases, PVC or HDPE sheathing may be included to provide double corrosion protection as for an anchor. The micropile grout is typically a mixture of water and Portland cement that may be simply tremie-placed under gravity only, or may be pressure grouted during or after installation. Micropiles are often designed to support axial service loads of 1 MN (225k) or less per pile, but higher loads per pile can be achieved and there have been successful load tests of micropiles to loads in excess of 6 MN (1350k).

In a typical application, the structural loads dictate the size of the steel element and then the embedded length is determined to provide the geotechnical resistance necessary for the transfer of load from the steel through the grout to the soil or rock. The transfer of axial load is typically accomplished through side resistance in the portion of the pile below the casing (the bond zone), with no reliance upon side resistance in the permanently cased zone or in end bearing.

The unit side resistance in the bond zone is not only affected by the type of soil or rock, but may be strongly affected by the type of construction practice used for drilling and for grouting. As a result, the average nominal unit grout-to-ground bond strength is usually estimated empirically and verified through site-specific load testing, with the final micropile geotechnical design performed by the specialty contractor. By working in this way with either performance-based specifications or a design-build type of contract, the contractor has the ability and responsibility to select the most appropriate and cost-effective drilling and grouting techniques. The constructor thus typically has some design responsibility and incentive to improve geotechnical performance, and static load tests are routinely employed to provide verification of axial resistance.

A major factor in the broad acceptance and increased use of micropiles within the last 15 years has been the work of industry groups to develop and promote standards, share knowledge and expertise, and transform the technology into a more universally accepted foundation option. The practice in the U.S. has been shaped by the joint micropile committee of the Deep Foundations Institute and ADSC: The International Association of Foundation Drilling, whose members have worked with the Federal Highway Administration and the National Highway Institute to produce design and construction guidelines and training materials. An FHWA reference manual was published in 2000 (Armour, et al, 2000) and subsequently updated (Sabatini, et al, 2005), which provides a widely used reference for the design and construction of micropiles. Micropiles were only recently incorporated into the AASHTO Bridge Design Specifications in 2007 and the International Building Code (IBC) in 2006.

Example Applications of Micropile Foundations

The micropile design for the Foothills Parkway Bridge shown in the photo in Figure 1a, was part of a design-build project for the U.S. National Park Service. The bridge is constructed on a steep mountainside location in an environmentally sensitive area, with natural slopes approaching 1:1. Micropiles were used to support both a temporary work bridge and the piers for the permanent structure, and were drilled through residual overburden of decomposed rock to bear in the underlying metasandstone and metaconglomerate as illustrated in Figure 2. Anchors were incorporated into the foundation to resist passive earth pressures from the overburden soils which have marginal stability against downslope creep.

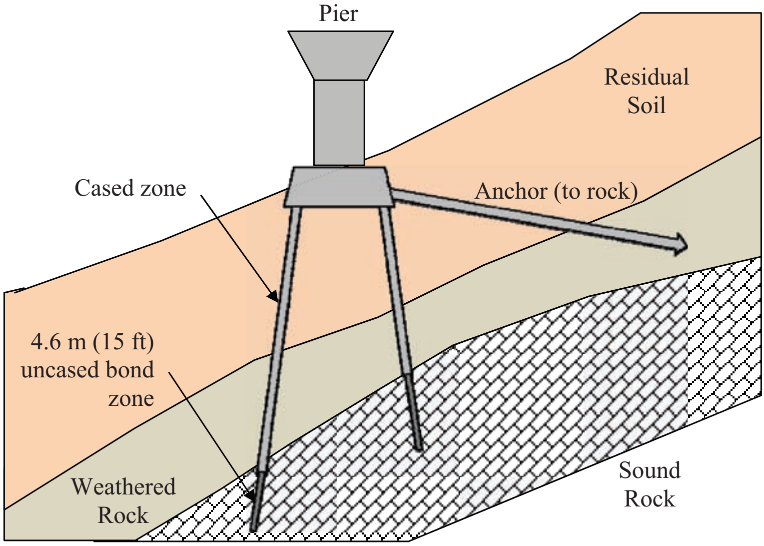


FIG. 2. Micropile Foundation Design for a Typical Pier, Foothills Parkway Bridge No. 2.

Analyses of the pile group foundation for bridge foundation loads from the pier were used to determine shear, moment, and axial demands on the individual piles and the design completed in accordance with the AASHTO 2007 LRFD guidelines. The shear and moment demand is resisted by the grout-filled permanent casing, which is 244 mm (9-5/8 inch) diameter, 12 mm (0.472 inch) wall thickness, 550 MPa (80 ksi) yield strength, and extends through the overburden soils and weathered rock zone. The casing was also installed so that no joint was located within 2.4 m (8 ft) of the top of the pile beneath the footing. The piles include a No. 18 center bar (57 mm, or 2-1/4 inch diameter) with 414 MPa (60 ksi) yield strength.

The maximum factored axial load demand of 1380 kN (310 kips) is resisted by the 203 mm (8 in.) diameter uncased portion of the pile which extends 4.6 m (15 ft) into the rock. This socket is designed for a nominal unit side resistance of 690 kPa (100 psi) and a resistance factor of 0.7. The axial resistance was confirmed by load tests.

The key factor in utilizing micropiles for the Foothills Bridge was the ability to position a small rig into place to install piles (shown in Figure 1a) with a minimum impact on the rugged and environmentally sensitive site. A tubular steel work trestle was installed atop the temporary micropiles, allowing construction of the permanent foundations and the remainder of the bridge from above ground. This type of solution requires a collaborative effort from both the designer and the constructor, as is facilitated by the design-build system for project delivery.

The photos in Figures 3 and 4 show another typical application where micropiles have been used for a foundation of a pedestrian bridge in Nashville, Tennessee. This foundation was installed as a part of a value-engineered alternate which was used because of the difficult access and unstable slope at this foundation location. The piles were drilled through boulder-filled debris using 24 cm (9-5/8 inch) O.D. permanent casing to facilitate drilling and casting through this zone, with a bond zone below the casing into an underlying limestone formation. The micropiles were constructed using a single threaded bar in each pile which extends into the reinforcement for the pile cap and utilizes plates threaded onto the bar to facilitate this connection. A load test was performed to a proof load of 4 MN (900 kips) with only 12 mm (½ inch) of elastic deformation observed during the test.

FIG. 3. Micropiles Cased Through Boulder Fill for the Cumberland River Pedestrian Bridge, Nashville, TN.

FIG. 4. Completed Micropile and Connection to Footing.

Micropiles in Slope Stabilization

The last 20 years has seen increased use of micropiles for problems of slope stability, whereby micropiles are utilized to transfer axial and shear forces across a sliding surface to provide restraining forces into an unstable soil mass. Brown and Loehr, (2007) document a rational but simple method to compute mobilized axial and shear forces across a failure surface and incorporate these into a limit equilibrium approach for estimating the contribution of micropiles to stability. This methodology is compared with measurements from the few available instrumented case histories in a research report sponsored by the aforementioned joint DFI/ADSC micropile committee (Loehr and Brown, 2008). One such example is the slide on U.S. 43 near Littleville, Alabama (Brown and Chancellor, 1997) which includes measurements of bending and axial forces in the micropiles. These piles were installed through a guide wall as shown in Figure 5 in an "A" configuration through fill and colluvium to restrain a soil mass sliding atop a weathered shale. The measurements documented the behavior of the piles to provide combined shear and axial tension or compression to restrain the failure, and this approach has now been employed on a number of slide repair projects across North America, e.g. Hasenkamp and Turner (2000).

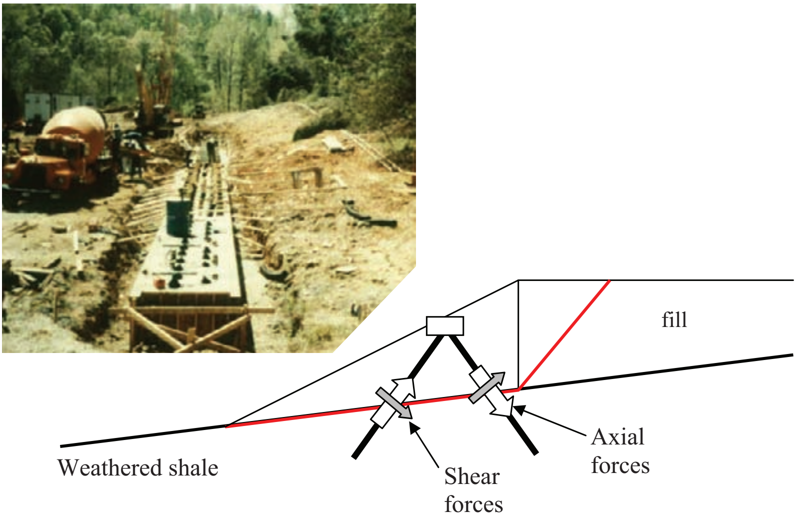


FIG. 5. Micropile Slide Repair at Littleville, Alabama.

Advancements in Micropile Drilling

Advancements in drilling technology and increases in load carrying capacity have been significant. The standardization of the flush joint threaded casing commonly used with micropiles has improved cost-effectiveness and the reliability of the flexural strength and structural performance of micropiles. Other advances and efficiencies are related to the combined use of the pile element as part of the drilling tool, for example with hollow bars or sacrificial drill pipe.

An example of innovation in drilling and grouting is illustrated in Figure 6 by the use of grout placement through a reverse circulation percussion drill tool, described by Atlaee et al (2010) for the construction of the micropiles for the Bronx-Whitestone Bridge in New York. The 35 cm (14 inch) diameter micropile foundations for the replacement of the Bronx approach structure for this bridge were constructed to bear in gneiss bedrock beneath overburden soils ranging from soft silts and peat to dense sand and glacial till including boulders. The reverse circulation percussion drilled flushed cuttings up through the drill rods and through the swivel atop the rods and into a discharge hose. Upon completion of drilling, grout was pumped down through the drill rods as the drill was extracted from the hole. The single #18 GR75 bar was installed into the grout-filled hole after the drill rods were removed. This innovative approach allowed the contractor to advance the drill rods to the bottom of the hole through the wide range of materials without the necessity to stop and replace tooling, and then accomplish the grouting without withdrawal of the drilling tools.

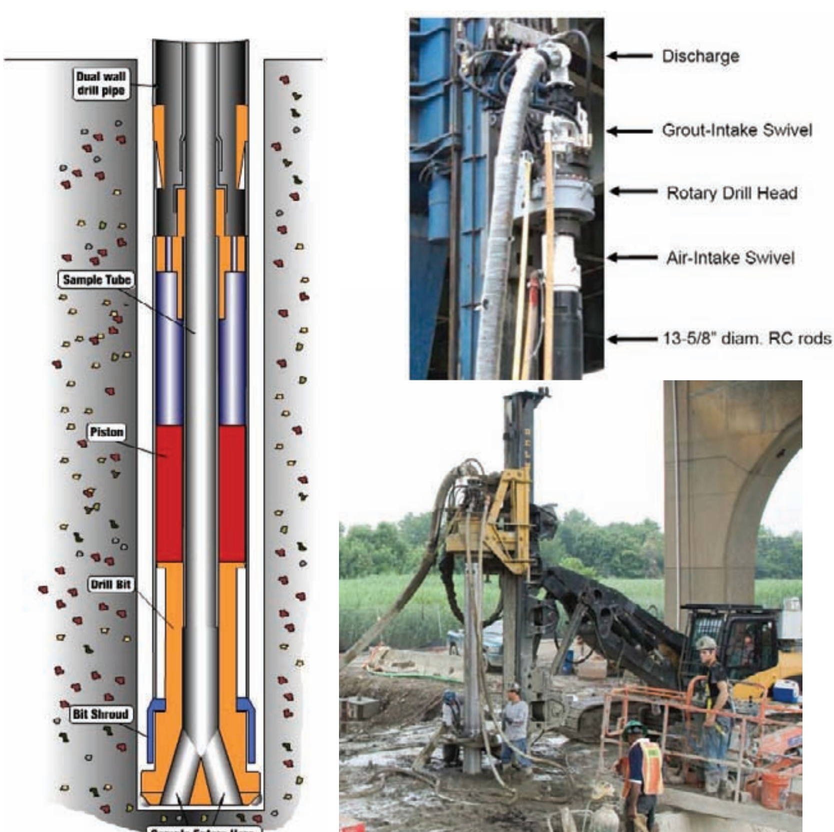


FIG. 6. Hammer Grout Piles at the Bronx Whitestone Bridge.

Another example of innovation in drilling and grouting is described by Szynakiewicz and Boehm (2008), whereby an office building in Houston, Texas was underpinned using micropile construction. The site of the project was composed of highly plastic clay soils, ground conditions that are generally not favorable to micropiles because of the low bond strength. However, the need for underpinning in a low headroom environment was most conducive to the small, versatile equipment used to install these piles. The underpinning was completed while the building remained occupied by constructing jet-grout columns within the deeper and more stable soils, and then installing micropiles into the jet-grout columns to provide load transfer from the building into the column. Low headroom drills were used for construction of both the columns and the micropiles. Load testing confirmed the load capacity of this hybrid pile, which transfers load from the micropile into the jet-grout column and then from the column into the clay soil.

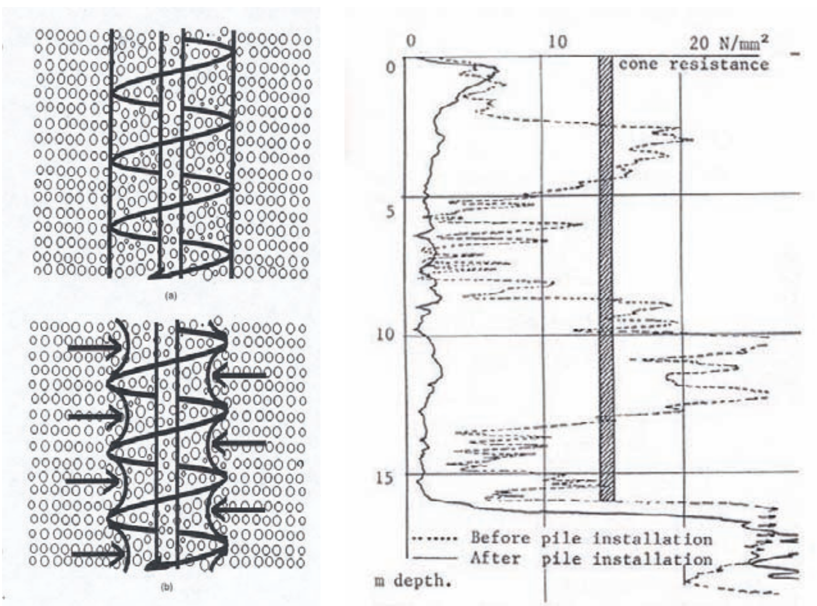
CONTINUOUS FLIGHT AUGER (CFA) PILES

The use of continuous flight auger piles or "ACIP" piles has been commonly performed in the U.S. since the 1970's, mostly using crane attached drills with a top-drive gearbox. Current practice includes the widespread use of this type of equipment as well as the adoption of many European practices. The current practices for design and construction of CFA and Drilled Displacement Piles in the U.S. is described in an FHWA reference manual by Brown, et al (2007).

The major advancements in the last 20 years have resulted primarily from two broad areas: 1) electronic controls for monitoring and guiding the drilling and casting process, and 2) more powerful drilling equipment with improved capabilities. The use of electronic monitoring equipment provides the drill operator with the feedback needed to ensure that each pile is constructed in a reliable and repeatable way, and the measurements provide the verification that the pile has been constructed properly. The use of more powerful fixed mast hydraulic-powered rigs provides greater torque and crowd or lifting capacity on the drill tooling, and promoted the use of larger diameter CFA piles.

Control of Drilling Process

The first key component is the control of the drilling process, in which the continuous flight auger drill must be advanced at the optimum rate. The drill is typically rotated at a constant rate and if the drill is allowed or forced to penetrate too quickly it can corkscrew into the soil and become "hung", i.e., the torque required to continue advancing exceeds the torque capacity of the rig. For this reason, the rate of advance must often be restrained to ensure that the soil is cut and loosened, but not so much that the augers are not kept charged full of soil to provide stability to the hole. The rate of advance of the drill must allow conveyance of enough material up the flights to allow for the volume of the drill itself and the bulking action of the soil as it is cut and remolded by the auger. If the soils have sufficient cohesion and/or arching action to stand vertically without the lateral support of the soil-filled auger, then the rate of penetration can be restrained to ensure easy drilling since conveyance of soils up the flights is not a significant issue. However, in loose sands below the groundwater or soft clays, excessive rotation of the drill without advancement will convey soils up the auger flights like a screw pump, the lateral stress around the pile is reduced, and the soil around the hole will side-load the auger resulting in loosening and ground subsidence around the pile. These effects have been described by Fleming (1995), and the effects on soil disturbance and pile behavior measured and described by Van Weel (1988) and Mondolini, et al (2002) (Figure 7). Observations of ground subsidence around CFA pile construction in sands and soft soils have been noted on numerous projects, for example as described by Esrig, et al (1994).



a) Side loading the auger due to excessive rotation (from Fleming, 1995)

b) Effect of soil loosening due to excessive rotation of CFA measured by CPT (from Van Weel, 1988)

FIG. 7. Effects of Excessive Rotation of CFA Auger.

Control of the Casting Process

The second key component in the construction of CFA piling is the control of the withdrawal of the auger during concrete or grout placement, and the need to synchronize this process with pumping so that:

a) positive pumping pressure is maintained at the point of discharge at the bottom of the augers,
b) a structural defect or neck in the pile does not result from pulling the auger string too fast, and
c) wasteful pumping of excess concrete or grout does not occur, particularly in soft soils where overconsumption would provide little or no benefit.

Automated Monitoring and Controls

Through much of the history of the use of CFA piles, the skill of an experienced drill rig operator has been recognized as a critical component, because the "feel" of the operator was always so important to both advance the drill effectively and withdraw the drill during concreting in the correct way. The use of modern electronic controls, shown in the photos of Figure 8, provides the operator with direct feedback measurements on the critical parameters and also the ability to document that the pile has been constructed in accordance with good practices. Many of today's rig operators, having grown up playing electronic games, are quite comfortable operating a joystick and using a graphical electronic display. For the constructor, the monitoring can also provide a measure of productivity, since some systems provide a minute-by-minute log of the activity of the drill rig. Equipment maintenance requirements represent another common function that may be included as a part of the on-board computer system.

a) Controls on a hydraulic system **b) Controls on a crane-mounted rig**

FIG. 8. Automated Monitoring Systems in Use with CFA Pile Construction

The most important control parameters include the rate of penetration and sometimes the applied torque and crowd (down force) on the tools, rotation rate, the concrete or grout pressures, and the volume of concrete or grout pumped as a function of the elevation of the auger tip and the theoretical volume required to that point. When these parameters are calibrated to site-specific load testing, the use of automated monitoring provides a high level of quality control and quality assurance. The monitored parameters are recorded and documented in a production log that provides a record of the successful completion of each pile.

Although not yet common in North American practice, there are available capabilities for the on-board computer to take over the casting process, automatically matching the rate of withdrawal of the auger to the rate of delivery of grout as measured through an in-line flowmeter while maintaining a specified delivery pressure in the pump line at the top of the auger string.

DRILLED DISPLACEMENT PILES

The use of more powerful fixed mast hydraulic-powered rigs provides greater torque and crowd on the drill, and promoted the use of drilled displacement piles. The drill tooling for these piles includes a feature that displaces rather than extracts the soil, as illustrated by a few of the different types of tools in use in the photos of Figure 9. These tools are characterized by a displacing body which is typically around 1.5 to 2 m (5 to 7ft) above the tip of the auger, with sometimes occasional reverse flights at various intervals above the displacing body. The short length of auger below the displacing body helps advance the tool by screwing into the soil below the displacing body and pulling it downward. Various types of cutting shoes on the bottom may be employed, depending on the type of soil to be penetrated. The photo on the left is from the construction of the Georgia Aquarium in Atlanta and shows a tool extracted from the soil upon completion of casting. The lack of spoils associated with the construction of this pile points to one of the advantages of this technique, i.e., spoil removal and the mess associated with CFA piles is avoided.

FIG. 9. Drilled Displacement Tools.

The torque and crowd required to construct a drilled displacement pile is substantial, and the modern fixed-mast hydraulic drill rigs are typically used for these piles. Because the pile fully displaces the soil, there are no issues related to over-rotation of the auger and potential loosening of the soil as described for CFA pile construction. The energy required to install the pile is related to the resistance of the soil to the displacement, and so the piles are often installed to a depth that is controlled by the capabilities of the drilling rig. The potential effect of lateral displacement or heaving on nearby structures may also be a consideration.

With the monitoring equipment capabilities described previously for CFA piles, it stands to reason that the measure of torque and crowd as a function of penetration might logically be related to the axial resistance of the completed pile in a manner similar to a CPT sounding. Variations in stratigraphy are readily detected, i.e., the penetration into a denser stratum is immediately evident by the measured torque and crowd required to maintain penetration. Although a broad methodology is not yet in widespread use, work is ongoing to develop site-specific correlations between "installation effort" (NeSmith and NeSmith, 2009) and load test results. Each individual drilled displacement pile is then installed to achieve a specific criterion based on the measurements of torque and crowd using the automated monitoring system. These advances offer improved efficiency as well as quality control and quality assurance.

Another advantage of the drilled displacement pile is the ground improvement that is naturally accomplished as the displacement tool densifies cohesionless soils and increases the in-situ stresses in the ground. In this way, the construction of multiple displacement piles in a group actually enhances the capacity of nearby piles, as demonstrated by Brown and Drew (2000). The ground improvement associated with the installation of displacement piles has been demonstrated by CPT soundings before and after installation, reported by Siegel, et al (2007) for a number of sites composed of sandy soils. Examples of these data are provided on Figure 10.

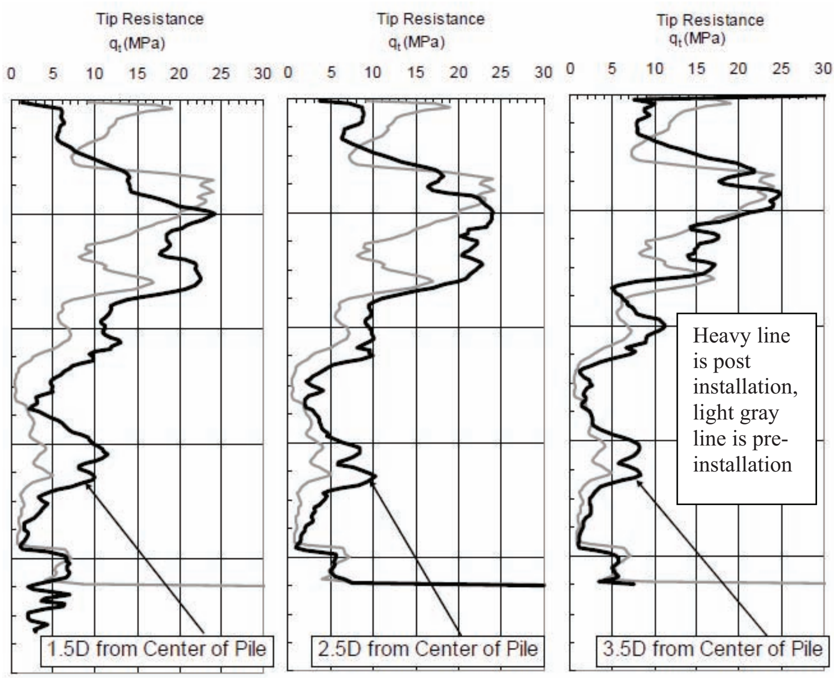


FIG. 10. Effect of Drilled Displacement Pile Installation on Cone Tip Resistance in a Sandy Soil (from Siegel et al, 2007).

As a result of the ground improvement benefits with drilled displacement pile equipment, these piles are popular for construction of pile raft foundations. The delineation between what is to be called a "pile" as opposed to a "rigid inclusion" or "column" in terms of ground improvement technology has become obscured and the terminology used may often reflect the design approach with respect to building code requirements. When used primarily as a ground improvement technique, the structure may in fact be designed to bear on spread footings that are not connected to the installed pile elements (or columns, since they are not really used as piles), and the columns may even be constructed of lower strength, unreinforced concrete.

An example of the use of drilled displacement pile construction techniques to achieve ground improvement is described by Siegel and NeSmith (2011) for a site composed of loose silty sand for a hospital in Kentucky. The technique was used to provide liquefaction mitigation and to increase subgrade stiffness so that the structure was founded on shallow footings bearing on the composite ground. The project included load tests on 3 m by 3m (10 ft by 10 ft) test foundations to applied bearing pressures of 335 kPa (7 ksf) to verify the performance. Photos of the footing construction and testing are shown on Figure 11, along with a plot of the results of the three load tests. The three tests were performed on nearly identical column layout in three different areas of the site. Instrumentation on the columns and on the subgrade between columns suggests that at the maximum bearing pressure the load was distributed about 40% to the columns and about 60% to the soil subgrade directly beneath the footing.

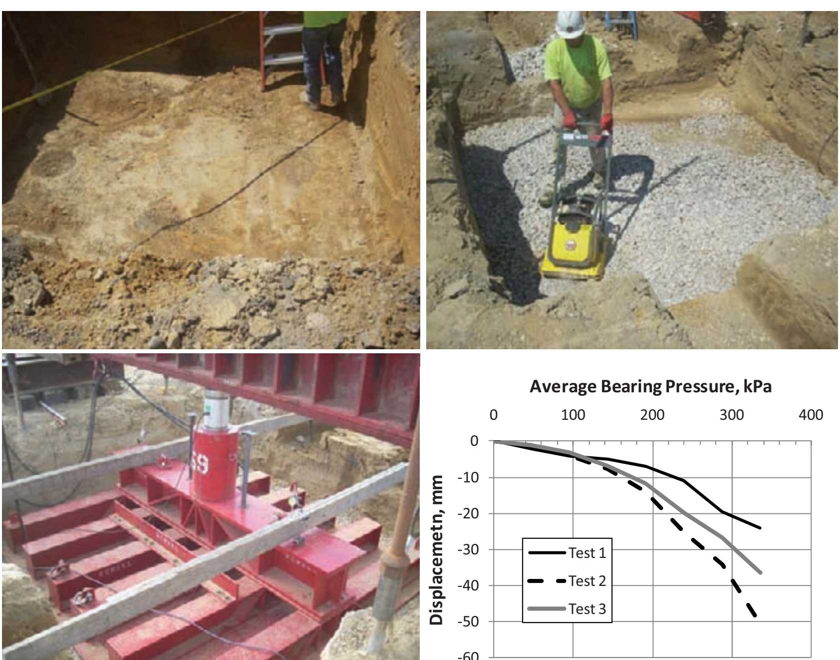


FIG. 11. Load Tests of Drilled Displacement Columns Supporting a Conventional Spread Foundation.

DRILLED SHAFTS

The major advancements in the use of drilled shaft foundations come from improvements in equipment and construction methods and improvements in testing and verification of performance. Drilled shafts can now be used with greater diameters and depths than ever before, a trend which opens opportunities for applications for foundation and geotechnical engineers to employ drilled shafts in new and creative ways. Methods of construction such as the use of base grouting and the use of polymer drilling fluids have been shown to provide improvements in performance of drilled shafts. Testing technology has also evolved to a point of routine use for verification of structural integrity and measurement of axial and lateral resistance to extremely large loads. Current practices for construction, design, and testing of drilled shaft foundations are provided by Brown, et al (2010).

Equipment for Larger and Deeper Drilled Shafts

The drilled shaft construction industry has evolved from a relatively small group of subcontractors to a much broader industry with a wide array of specialized equipment used for construction. Although it is still largely a craft performed by specialty subcontractors, a larger number of general contractors are self-performing this work and many subcontractors are concentrating on specialized types of drilled shaft and other specialty drilled construction techniques. The increased availability of specialized equipment which is focused on a particular construction technique contributes to this trend. On large or complex projects, drilled shafts have been employed with diameters of up to 4 m (13 ft) and depths of up to 80 m (260 ft).

One trend in recent years is a much increased use of oscillator or rotator equipment to install full length segmental casing. This type of equipment has been used to construct drilled shafts with diameter of up to 3.6 m (12 ft), and offers particular advantages in potentially caving ground conditions. The machines use hydraulic-powered jaws to clamp onto and twist the casing, and also pull or push the casing in the vertical direction. The oscillator machines twist the casing back and forth through a range of about 25° whereas the rotator provides the ability to twist the casing continuously through 360° and effectively use the casing as a full length coring tool. Photos of oscillator and rotator machines are provided in Figure 12.

One of the advantages of these machines is that drilled shafts can be installed in caving ground conditions with improved ability to stabilize the hole during excavation and concrete placement. The Benetia-Martinez bridge in the San Francisco Bay area is an example of a project with very challenging ground conditions composed of steeply bedded siltstone and shale with interbedded layers of soft and hard rock. After great difficulties with open hole drilling into this formation, the project was successfully completed using the rotator equipment shown in Figure 12b. The rock was removed from within the casing using a drop chisel to break the rock and a hammer-grab to extract it.

The installation of the casing by twisting it into place allows the casing to advance ahead of the excavation without the vibrations associated with the use of a vibratory hammer. Therefore the system allows installation of large diameter drilled shaft foundations in close proximity to existing structures with minimal risk of impact on the existing structure. Oscillator equipment was used to install 3.6 m (12ft) diameter drilled shafts for the Gilmerton Bridge in Chesapeake, Virginia in close proximity to two bascule bridges that remained operational during construction. The lack of vibrations adjacent to the drilled shaft construction was a primary feature in the selection of this method, and the use of large diameter drilled shafts minimized the size of the foundation footprint under each individual column for the new structure. Similar very large oscillator-installed drilled shafts were used on the Doyle Drive approach structures to the Golden Gate Bridge in San Francisco (Faust, 2011).

a) Drill-Mounted Oscillator **b) Rotator Attached to Crane**

FIG. 12. Oscillator and Rotator Machines

The presence of a fully cased hole also provides a reduced risk of soil caving during concrete placement, and therefore improved reliability for construction in applications such as bridge foundations where flexural demands require the use of large diameter drilled shafts. Where artesian groundwater conditions are present, the casing can be readily maintained at an elevation well above the ground surface to provide sufficient head within the shaft excavation to counterbalance the artesian condition.

Katzenbach, et al (2007) reviewed available load test information on drilled shafts constructed using the oscillator and rotator segmental casing method and report that the results are comparable and in some cases favorable to other installation techniques. One factor that favors the performance of drilled shafts constructed using this method is the fact that the teeth that are used on the cutting shoe at the bottom of the casing tend to produce a roughened surface at the concrete/soil interface as the casing is extracted. An opportunity to examine the surface of drilled shafts constructed using this construction method was provided recently at the Huey Long Bridge in New Orleans (Brown et al, 2010). The 2.8 m (9 ft) diameter drilled shafts were exposed within the sheet pile cofferdam after placement of the seal slab and during construction of the footing. These foundations were constructed prior to excavation of the cofferdam, with a corrugated metal pipe used as a temporary form above the top of the drilled shafts. The photo in Figure 13 shows the herringbone pattern left at the surface of the drilled shaft concrete due to the action of the teeth on the soil as the casing is extracted.

Another advantage is the control on verticality provided by the increased stiffness of the drilling system. While verticality is not often a critical factor for foundations, this aspect is important for applications in which drilled shafts are used near underground structures or to construct secant or tangent pile walls. Typical specifications for verticality of drilled shaft foundations using conventional construction techniques are 1.5% in soil and 2% in rock (Brown, et al, 2010). However, recent experiences in a test installation for the TransBay Terminal in San Francisco suggest that oscillator/rotator equipment is capable of maintaining verticality on the order of 0.35% to 0.5% for foundations as deep as 73 m (240 ft).

FIG. 13. Exposed Texture on the Drilled Shaft Surface, Huey P. Long Bridge.

Reverse-circulation drilling is another technique that has been increasingly used in recent years to construct drilled shafts to large diameters and depths. This drilling technique provides full face rotary cutting at the base of the excavation with the drilling fluid used to remove cuttings via air-lift pumping up through the center of the drill pipe. This closed system avoids the need to cycle in and out of the hole to remove cuttings from an auger and can also be very effective in excavating rock.

The photos on Figure 14 illustrate the equipment used with this technique. The system in Figure 14a is mounted onto a casing that was installed with a rotator, and is working in the space beneath an existing bridge on I-90 in Connecticut to install 2.8 m (9 ft) diameter drilled shaft foundations into the bedrock for the replacement bridge structure prior to demolition of the old one. The drill removes the cuttings by pumping the cuttings and fluid up through the center drill pipe, through the swivel at the top, and on to a spoil container via the discharge hose in the foreground. Drilling fluid is simultaneously pumped into the top of the excavation through a return line. An example of a full face rotary cutting tool is shown in Figure 14b. This tool was used on the Walter F. George Dam in Alabama to construct a cutoff wall into limestone. The bottom of the air-lift pipe is located slightly off-center so that this pipe moves around and suctions the cuttings across the face as the tool rotates.

a) Restricted Headroom Drilling **b) Full Face Drilling Tool**

FIG. 14. Reverse Circulation Drilling.

The Wolf Creek Dam project in Kentucky is an example of the advancement of drilled shaft equipment and technology to overcome challenges in a way that was not possible years ago. Seepage through the underlying limestone bedrock below has threatened the stability of the earth dam that retains Lake Cumberland, the largest reservoir east of the Mississippi. A previous cutoff wall had been constructed into the bedrock in the late 1970's using the best available technology at that time, and the seepage problem was not successfully resolved by that effort. Seepage has found new paths under and around the wall, leading to sinkholes and soft wet areas downstream as well as high measured pore water pressures in the embankment. The Wolf Creek Dam was in critical need of remediation to correct the problem.

The key component of the repair to the dam is the construction of a secant pile cutoff wall, and the construction of this wall utilizes reverse circulation drilling to construct drilled shafts to very great depths. After lowering the reservoir and completing an initial grouting program, the cutoff wall is constructed through the dam from a bench on the upstream face, as shown in the photo of Figure 15.

First, a 1.8 m (6 ft) wide concrete diaphragm wall is constructed through the embankment to the top of rock at a depth of around 25 to 30 m (80 to 100 ft). The secant pile wall is then constructed to depths of up to 84 m (275 ft) through the concrete diaphragm wall and the karstic limestone and into a sound limestone layer. The secant pile excavation is started using conventional drills with rock augers to open a hole to a depth of around 15 m (50 ft) into the diaphragm wall, and then completed using reverse circulation drilling as illustrated in the photos of Figure 16.

In order to maintain the alignment on such deep drilled shafts and ensure that the secant piles overlap to form a water-tight cutoff wall, a pilot hole is first installed using directional drilling techniques. The reverse circulation drill is equipped with a "stinger" to follow the pilot hole and maintain the alignment during drilling.

Construction of the cutoff wall is ongoing, with anticipated completion in 2013.

FIG. 15. Work Platform at Wolf Creek Dam.

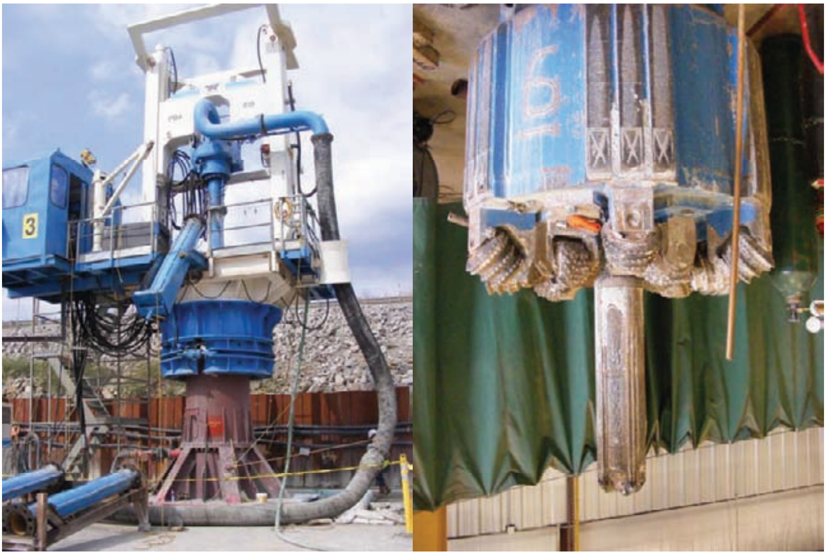


FIG. 15. Reverse Circulation Drilling for Secant Pile Cutoff Wall

Base Grouting

Base grouting to enhance the axial resistance of drilled shaft foundations is a technology that has been around for decades, but research in Tampa by Mullins et al (2000) has spawned a renewed interest in this technology in North American practice. Base grouting is a form of compaction grouting at the toe of a drilled shaft which compresses and preloads the soil below the toe, increases the state of stress in the ground, and can significantly increase the axial base resistance of drilled shafts which are founded in granular soils (Mullins, et al, 2006). There is also benefit from base grouting in that the grouting mitigates the effect of any loose granular material which might remain as a result of imperfect cleaning of the base of the drilled shaft excavation. The technique provides relatively little benefit in rock, cohesive soils, or strongly cemented materials. Another limitation to the improvements achieved with base grouting is that the available side resistance of the drilled shaft limits the magnitude of the pressure which can be applied.

The photos in Figure 16 illustrate some aspects of base grouting. Figure 16a) shows a typical base grouting apparatus attached to the base of the reinforcement cage. The photo in Figure 16b) shows a 1 m (3.5 ft) diameter and 8 m (25 ft) long drilled shaft at the Auburn University National Geotechnical Experimentation Site. This shaft was base grouted and subsequently exhumed to reveal the effects of base grouting in a very silty and medium dense soil. A relatively large volume of approximately 0.3 m^3 (10 ft^3) of grout was injected at the base of this shaft, representing a volume equal to about 30 cm (1 ft) length of shaft. The grout can be seen to have produced a bulge at the base and also to have migrated up along the side wall of the drilled shaft over the lower one to two diameters. The drilled shaft in the background of this photo was not base grouted.

a) Sleeve Port System for Grouting **b) Exhumed Base-Grouted Shaft**

FIG. 16. Base Grouting for Drilled Shaft Foundations

An example of the use of this technique on a major project is described by Dapp and Brown (2010) for the John James Audubon Bridge over the Mississippi River in Louisiana, which utilized drilled shafts founded in dense alluvial sand. Each of the two pylon foundations for the cable-stayed bridge included 21 drilled shafts which were 2.3 m (7.5 ft) diameter and approximately 60 m (200 ft) deep. The base grouting was accomplished via a sleeve-port system (tubes-a-manchette) that utilized the crosshole sonic logging tubes. The eight tubes were connected in pairs across the base of the drilled shaft to form four separate U-shaped circuits, as shown in the photo of Figure 16a).

The project included load tests using the Osterberg cell (O-cell) at the base of both grouted and ungrouted drilled shafts to provide a comparison of performance for full scale foundations. The data provided in Figure 17 illustrates the improvement in base resistance achieved by base grouting to a pressure of approximately 5 MPa (750 psi). The data shown are measured load at the base of the drilled shaft from base grouted shafts except the curve test T3. Shaft T3 was constructed in an identical way to the others but was not grouted and did not include the base grouting system.

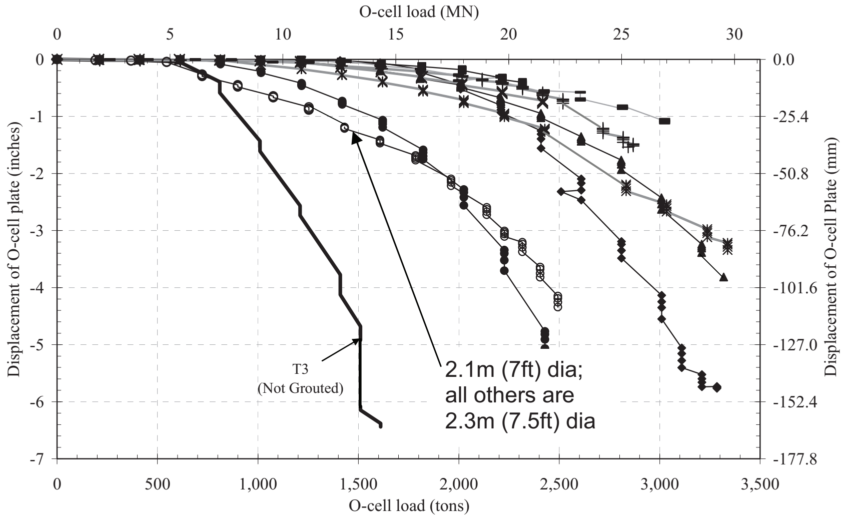
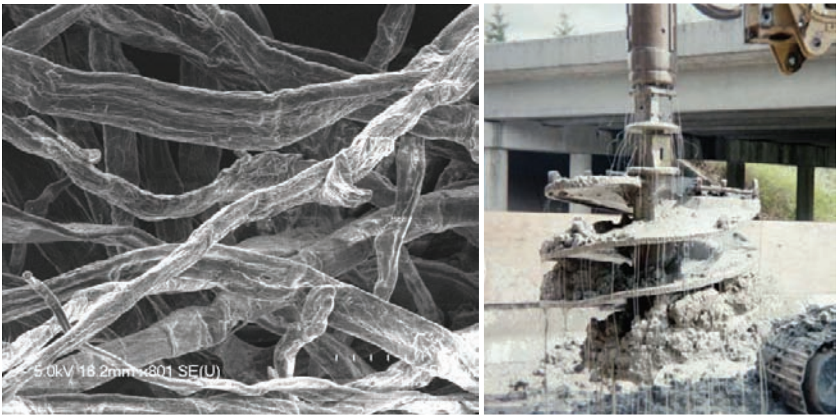


FIG. 17. Measured Base Resistance from the Base-Grouted Drilled Shafts at the John James Audubon Bridge, Louisiana.

Polymer Drilling Slurry

Although the versatility of drilled shaft construction emerged using mineral slurry (mostly bentonite) for drilling fluids, in recent years the use of polymer-based drilling fluids has become the prevalent practice where wet-hole techniques are used for construction. Polymers have several advantages over conventional bentonite for constructors, because it is more easily mixed, de-sanded, and disposed. The long-chain polymers (shown in Figure 18) mix easily with water and increase the viscosity of the fluid, and increased viscosity reduces the fluid loss into the surrounding soil and provides a stabilizing fluid pressure when a positive head is maintained within the drilled shaft excavation.

Unlike bentonite, polymers do not create a filter cake on the borehole wall, and therefore fluid loss tends to be a greater than with bentonite slurry. Also, the density of polymer slurry tends to be lower than bentonite fluids, and cannot be easily increased as with the addition of barite to bentonite fluids. In coarse or gravelly sand or areas with very high or artesian groundwater levels, these aspects may result in less effective performance with respect to borehole stability. If the fluid head in the shaft excavation is not actively maintained at a level higher than the groundwater level, the fluid level in the excavation will eventually fall and the supporting pressure may be lost. On the other hand, the lack of a filter cake mitigates one of the major design concerns associated with the use of bentonite slurry for drilled shaft construction, namely that excessive filter cake buildup will be detrimental to the bond at the soil/concrete interface and therefore reduce the available side resistance of the foundation.



a) Scanning Electron Micrograph, 800x **b) Polymer Slurry in Use**

FIG. 18. Polymer Drilling Fluids (photo at left courtesy of Likos, Loehr, and Akunuri, Univ. of Missouri).

Specifications for polymer slurry construction often have evolved from those used for bentonite, but the differences in performance of polymers require several modifications. Experiences with polymer slurry construction indicate that designers should be aware of several factors that affect practice.

The upper limits on viscosity used for bentonite are too restrictive for polymer; there is a need to limit the viscosity of bentonite to avoid excessive filter cake buildup, but polymers can utilize significantly higher viscosity in order to provide effective stabilization.

Unlike bentonite slurry, the density of polymer slurry will not be much higher than that of water, and so where groundwater levels are very near the top of the drilled shaft it is critical that a positive head of 2 m or more is maintained at all times. Where groundwater levels are near or above the ground surface, it will be necessary that the contractor extend the casing above grade to provide this head, plus additional distance to allow for fluctuations and working freeboard within the casing.

Because there is no bentonite filter cake, it is not necessary to limit the exposure time of the soil to slurry and/or require agitation of the sidewall. There is also substantial evidence that drilled shafts constructed using polymer slurry result in higher values of unit side resistance in sands and silts than similar foundations constructed using bentonite (Brown et al, 2002; Brown, 2002; Meyers, 1996).

Even where natural clays or shales are encountered in the soil, the polymers as shown in Figure 18b tend to stabilize these soils and prevent mixing of the clay with the drilling fluid. Many contractors like to employ a small amount of polymer slurry when drilling through clay because it reduces the tendency for clay to stick to the auger. There is also evidence that polymer drilling fluids may reduce wetting of some shales and thereby reduce the tendency for degradation of shale when the excavation is open. This behavior can result in improved side resistance for drilled shafts socketed into shale, especially for large drilled shafts where several days may be required to complete construction.

Axtell et al (2009) describe a case history for a bridge project in Kansas City where 3.2 m (10.5 ft) diameter rock sockets were constructed into a shale, and even though the polymer slurry filled hole was open for four days, load test measurements determined that the unit side resistance in the socket was a relatively favorable value 720 kPa (15 ksf). The polymer slurry was perceived to provide benefits with respect to preserving the integrity of the shale and was used for construction even though casing extended to the rock and slurry was not required to stabilize the hole. Slake durability tests of the shale with both river water and polymer slurry are summarized in Table 1. The higher slake durability index and durability rating of the rock specimens tested in polymer slurry indicates that the shale was subject to less significant degradation in the presence of polymer slurry compared to that observed when the rock was exposed to plain river water.

Table 1: Slake Durability Test Results.

Sample	**Natural Moisture Content**	**Slake Durability Index**		**Durability Rating Based on Shear Strength Loss**	
	(%)	**Type**	**$I_d(2)$ (%)**	**Type**	**DR_s**
River Water	8.3	II	72.2	Intermediate	61.9
Polymer Slurry	8.3	II	98.2	Hard, more durable	78.6

Where fine sands and silts are present, polymer slurry can present a challenge from the standpoint of cleaning the slurry. These sands and silts will not stay in suspension and will tend to settle out slowly after completion of excavation. The de-sanding units used with bentonite slurry construction do not work with polymer because the polymer molecules would be destroyed by the shearing process in the de-sander and polymers will also tend to clog the screens. De-sanding of polymer is normally accomplished by adding flocculants to help promote the settling of solids, a process that requires that the slurry be maintained in a calm environment so that the sands can settle out. Flocculation can occur either in the borehole (followed by pumping from the base of the hole to remove solids) or in a weir tank after removal and replacement of the fluid in the hole with clean slurry. In a very deep drilled shaft excavation filled with sand-contaminated polymer, solids can rain out of suspension for days and if left untreated could result in contamination of concrete during placement. Although flocculants can be used in the borehole to accelerate the process, the most reliable approach is to fully exchange the fluid by pumping slurry from the base of the shaft to a holding tank while adding clean slurry into the top of the excavation.

Verification Testing

Only within the last 20 years has integrity testing and load testing of drilled shaft foundations become commonplace, and the availability and use of these technologies has greatly improved the efficiency of designs and the reliability of the constructed foundations.

The vast majority of integrity testing performed in North America uses crosshole sonic logging (CSL) to verify the integrity of the concrete within the drilled shaft. CSL testing relies upon the measurement of compression waves between pairs of tubes that are typically attached to the reinforcement. The tubes are filled with water so that acoustic transponders and receivers can be used to perform measurements through the water, tube, and concrete between tube pairs at intervals of every few inches. By using multiple tubes, measurements can be performed at various angles and directions across the drilled shaft diameter and around the perimeter. There is some use of gamma-gamma testing (largely by Caltrans) to measure density of the concrete in the vicinity around embedded PVC tubes, although CSL testing is used as a backup if anomalies are detected. Recently, a promising new method called Thermal Integrity Profiling has been developed by Mullins (2010) based on thermal measurements; the heat of hydration is measured via downhole tubes and correlated with the presence of good concrete. This thermal technique offers the promise to verify integrity before the concrete has fully hardened.

CSL testing and other measures of integrity testing through downhole access tubes offer the potential to detect even relatively minor inclusions of soil, laitance, low strength concrete, or other deleterious material. Besides improving the reliability of the constructed foundation, the accountability provided by these test measurements provide quantifiable verification of an effective contractor's work plan and quality control for concrete placement. Effective construction methods are apparent because of the successful integrity test measurements; ineffective methods or poor controls are quickly detected. This accountability has had the effect of significantly improving the quality of construction on public works projects where CSL testing is routinely employed.

On the other hand, there is a distinct need for engineers and designers to recognize that perfection is not achievable in this challenging construction environment, and that drilled shaft designs should be relatively tolerant of minor imperfections. An example is illustrated in Figure 19 from an experimental drilled shaft at Lumber River, SC that was exhumed as a part of research (Brown et al, 2005). The source of an anomaly in the CSL measurement was exposed when the exhumed shaft was saw-cut at precisely the elevation revealed by the anomaly. The flaw was a small pocket of segregated concrete that was lodged against the CSL tube and was approximately the size of a tennis ball.

An imperfection detected as a result of integrity testing does not necessarily constitute a deficiency in the drilled shaft. The size of the flaw exposed in Figure 19 should not be of serious concern because the structural design of the drilled shaft must include adequate tolerance for such small imperfections. The magnitude of a potential flaw detected by CSL tests can usually be quantified by close examination of various signal paths across the cross section of the shaft and even by using tomography techniques if needed. If a potential imperfection is detected it may be at a location where the drilled shaft is not subject to the maximum flexural demands, and so a greater tolerance may exist. An engineering evaluation of the structural and geotechnical performance requirements at the elevation in question is required to determine if a deficiency exists in a drilled shaft with an imperfection.

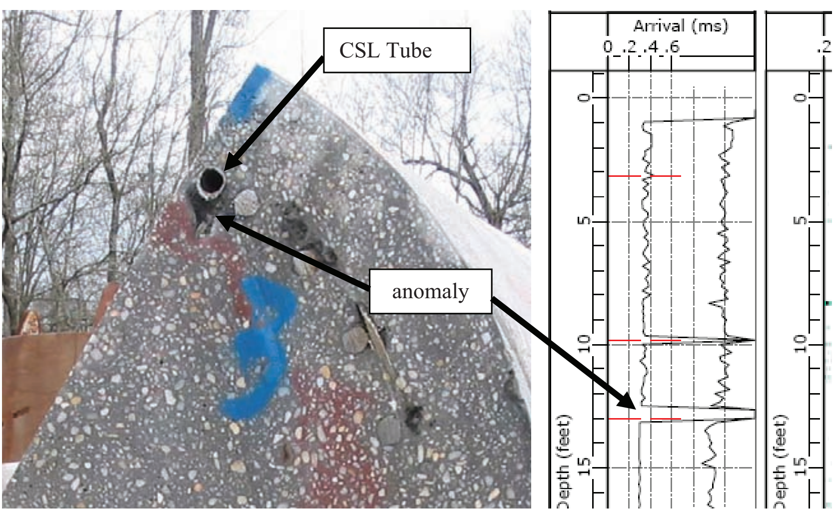


FIG. 19. Exposure of an Imperfection and CSL Anomaly.

Another consideration is that sometimes an anomaly in the signal occurs as a result of non-uniform concrete curing, tube debonding, or other artifacts of the test measurements. For these reasons, an anomaly in the integrity test measurement should typically be verified with coring or other means before expensive corrective action is warranted. An independent evaluation of the anomaly may be necessary to determine if a real imperfection exists and if any such imperfection is sufficient to constitute a deficiency.

The majority of load testing performed on drilled shafts in North America utilizes bi-directional embedded loading jacks, also known as the Osterberg Cell® (O-cell). The O-cell is embedded within the drilled shaft to engage the portion above the cell as a reaction against the portion of the shaft below the cell, with measured pressure in the cell calibrated to load and independent measurements of displacement of the two separate portions of the drilled shaft. This load testing technology has allowed the measurement of extremely large axial resistance because of the inherent simplicity in the test and the lack of need for a reaction system.

Conventional static top down load tests are still occasionally used with drilled shafts, and load tests of up to 50 MN (11,000 kips) have been performed with static reaction systems. Other advancements with drilled shaft load testing include the use of rapid load testing and high strain dynamic testing with signal matching, as would be performed on a driven pile. The rapid load testing method is most often employed using the Statnamic® device, which launches a reaction mass upward with about 20g of acceleration resulting in a downward thrust onto the drilled shaft. This test method offers a relatively economical means of verifying axial resistance from the top down without the need for a reaction system, and can often be performed on production foundations. The equipment available to perform rapid load testing is currently limited to a maximum applied force of around 45 MN (10,000 kips), and the maximum static resistance which can be mobilized is slightly lower due to inertial and rate-of-load effects.

The major implications of the advancements in testing for high capacity drilled shaft foundations are:

- Designers now have the means to obtain measurements that will provide the feedback necessary to improve design practices.
- Alternative forms of project delivery such as design-build can now include performance measurements for verification, and the availability of such testing can allow for performance-based specifications to be employed in the design-build process.

An example of the use of load testing for verification in design-build is the Honolulu Transit project currently under construction. The first phase of this project includes approximately 10 km (6 miles) of elevated guideway to be constructed in a tight space within existing right-of-way. A single drilled shaft foundation at each pier provides maximum support in the minimum footprint. Eight load tests using the O-cell method have been performed along the alignment in order to evaluate both the range of ground conditions encountered and the range of construction methods used.

Another example is provided by the New Mississippi River Bridge project in St. Louis, where load tests were used to verify a contractor-proposed "alternative technical concept" or ATC (Brown et al, 2011). This project utilized a conventional bid-build contract, but bidders were encouraged to submit confidential ATC's for review and possible approval during the pre-bid period. A bidder with an approved ATC could bid the project including the ATC in lieu of the base design for that portion of the work. The winning bidder submitted an alternative foundation design which included heavily loaded drilled shaft foundations and a plan for load testing to verify the axial resistance. The use load testing in the ATC design allowed the use of higher resistance factors in the LRFD design methodology and potential savings in foundation costs. The investment in load testing and increased performance risk to the contractor was considered as economically advantageous because of the potential savings.

Each of the two pylon foundations for this cable-stayed bridge is composed of a 2 x 3 group of drilled shafts, with permanent casing down to rock. Each shaft is supported entirely by the limestone bearing stratum through a 3.4 m (11 ft) diameter rock socket with depths into rock ranging from 5 m (16.5 ft) to 6.7 m (22 ft). The photos in Figure 20 show the excavation of the very hard limestone, which typically had compressive strength of around 140 MPa (20,000 psi), but included thin seams of weaker material. The load test successfully demonstrated a total axial resistance of 320 MN (72,000 kips), a value which exceeded the requirement for the ATC design.

FIG. 20. Load Test Shaft for the Mississippi River Bridge, St. Louis.

SUMMARY AND CONCLUSIONS

Today's engineers have a phenomenal variety of drilled foundation alternatives at their disposal. The tools that can be employed to solve deep foundation problems range from small diameter micropiles that can be installed in tight spaces using lightweight and portable rigs, to large diameter drilled shafts capable of supporting enormous loads. Reliability is enhanced by technology ranging from on-board rig monitoring and controls as well as post-construction integrity and load testing.

Innovative applications of micropiles have been described which exploit the capabilities of this technology, along with some new and innovative techniques for installing micropiles. The use of these drilled foundations is now becoming mainstream in North American practice, with published design guidelines by agencies such as FHWA and with micropiles now incorporated into building codes such as IBC and AASHTO.

The construction of CFA piles has matured so that these piles are recognized and more widely accepted, and the use and reliability of these economical piles is greatly improved by the use of onboard computer monitoring and control.

The advances in drilling equipment have lead to increased use of drilled displacement piles, a technology which offers advantages from the inherent ground improvement that is achieved. Displacement during drilling provides increased axial resistance and reliability in granular soils compared to CFA piles and the elimination of most excavated materials from the piling operations.

The capabilities of the equipment and methods for installing drilled shaft foundations has lead to larger and deeper drilled shafts so that effective solutions are provided for projects like the Wolf Creek Dam and the John James Audubon Bridge. Integrity and load testing provides reliability for improved design and construction as well as accountability for innovative project delivery methods such as design-build and the use of contractor-developed alternative technical concepts.

The capabilities of the equipment and drilling techniques present opportunities for engineers, as well as challenges. The opportunities are present because the work is more sophisticated than ever, and engineers who are in a position to utilize the available technology can provide value and efficiency to complex foundation engineering projects. The challenges are posed by the requirement to understand the complexities of new drilled foundation techniques and the impact of construction on the performance.

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Computer Monitoring in the Grouting Industry

Donald A. Bruce[1], Ph.D.

[1] President, Geosystems, L.P., P.O. Box 237, Venetia, PA 15367, U.S.A., Phone: 724-942-0570, Fax: 724-942-1911; dabruce@geosystemsbruce.com

ABSTRACT: Computer monitoring and specialized equipment in the grouting industry has undergone multiple levels of technological advancement over the past two decades. The implementation of these technologies over recent years has greatly enhanced the industry's ability to more efficiently collect and evaluate pertinent data. This has subsequently resulted in more reliable and sound decision making, an increase in efficiency, and ultimately, cost savings to the owner. Dreese et al. (2003) discussed advances in the grouting industry and defined Levels 1 to 3 Technology for real time monitoring and data collection. Level 3 Technology (Advanced Integrated Analytical Systems) represents the current level of advancement in real-time monitoring of grouting in the industry today.

The multitude of technical challenges faced during any grouting project, includes assessing the geologic formation and identifying characteristics most likely to impact the grouting program. This paper documents the progression of the industry in using state of the art technologies to not only collect data and make real-time assessments of the formations response to grout injection, but also the use of high resolution images to evaluate discontinuities with the formation prior to injection, the implementation of digital photogrammetry for assessing outcrops and/or structures in three-dimension, assessment of Monitoring While Drilling (MWD) information and its integration, and real-time monitoring of instrumentation data for evaluating the formations immediate response to grout injection.

INTRODUCTION

The application of new and existing technologies within the grouting industry has increased substantially over the past 15 years (Bruce, 2012). These technologies have led to balanced stable grouts, improved equipment for efficient grout mixing and quality control, and specialized drilling equipment to produce accurate grout holes at greater depths (Weaver and Bruce, 2007). The incorporation of new technology by the grouting industry has helped to solve various technical challenges involving grout curtain depth and location, grout mix characteristics and quality, and project schedule constraints. Interpreting the subsurface geologic conditions and response during production, however, often presents the biggest challenge for any grouting project. Fortunately, there are also technologies available to the industry that allow the engineer/contractor and owner (the project team) to make informed decisions to better effectively treat the formation and accomplish project goals in accordance with contract documents. Some of the more important technologies are discussed in this paper, and are summarized in Table 1.

Table 1. Summary of State of the Art Technologies

Method	Description
Real-Time Computer Monitoring	Real-time data collection and display of grouting and water testing operations. Enables operators to make sound engineering decisions, effectively measure project performance, and generate project records
Monitoring While Drilling (MWD)	Automated real-time collection, storage, and display of drilling parameters. Frequently recorded parameters include instantaneous advance speed, tool pressure, torque, injection pressure of the drilling fluid, rotation speed, penetration rate, and thrust (bit load). Specific energy is commonly determined based on recorded parameters.
High Resolution Borehole Imaging	High resolution images of the borehole sidewalls and discontinuities. Capable of measuring borehole deviation and producing stereonets and joint poles based on fracture picking
Digital Photogrammetry	Allows geologic mapping of areas with difficult accessibility. Capable of producing 3-dimensional models including digital terrain models (DTM) that can be directly incorporated into the project and subsequent geotechnical evaluations.
Automated Instrumentation	Real-time collection of piezometric and other data for monitoring of the subsurface geologic formation or piezometric surface response to grouting operations.

In the author's experiences, the use of MWD, high resolution borehole images, and automated instrumentation, integrated with computer monitoring, greatly enhance the project team's ability to interpret the geologic conditions. Each of these technologies is of limited value when used alone. The integration of these systems allows the user and owner to quickly visualize and understand large quantities of data and information in real-time to effectively manage a grouting program.

REAL-TIME COMPUTER MONITORING

The integration of automated computer monitoring of grouting operations has greatly enhanced the industry's ability to make better immediate decisions related to the formation's response to grout injection. Wilson and Dreese (2002) and Dreese et al. (2003), discussed the three levels of technology for the monitoring of grouting operations that were available in the industry as of 2003:

- *Level 1: Dipstick and Gage Technology*
- *Level 2: Real-Time Data collection & Display Systems*
- *Level 3: Advanced Integrated Analytical Systems (AIA Systems)*

This definition applied to rock grouting projects for dams, although the basic framework is equally applicable to other types of grouting, in soils as well as rock.

Level 1 Technology represents the general state of practice prior to around 1997 and does not utilize electronic pressure gauges and flow meters that are widely available in the market. Nor does Level 1 Technology incorporate the speed and power of the computer that makes most of the calculations required for monitoring fast and accurate. For this reason, only Level 2 and 3 Technologies are recommended as only they are truly able to produce real-time displays of grouting operations. Level 3 is considered the superior level of technology in the grouting industry today. Unlike the other levels of technology, the engineer or geologist operating the system has the tools and capabilities to provide onsite technical support and real-time assessment of the grouting results in addition to monitoring the operations. As discussed in detail in the referenced papers, Level 3 Technology is comprised of 4 major components that when combined, produce a unique and powerful monitoring system: (1) a real-time data display of information retrieved from field operations; (2) a central database to store all collected and calculated information; (3) linked customized CADD functions to automatically display up to the minute information stored in the database on demand; and (4) customized queries to quickly and accurately mine data from the database for daily report generation and up-to-date analytical capabilities. The CADD display allows the project team to visually observe the results as they are obtained, and assess the project status in relation to adjacent hole series and lines. The real-time monitoring and analytical capabilities allows the operator to make sound engineering decisions efficiently. Figure 1 shows a 3-dimensional closure plot of a single grout line through fourth order hole series at varying depth intervals. Analytical capabilities such as this allow the operator to assess grouting performance, resulting in better informed decision making.

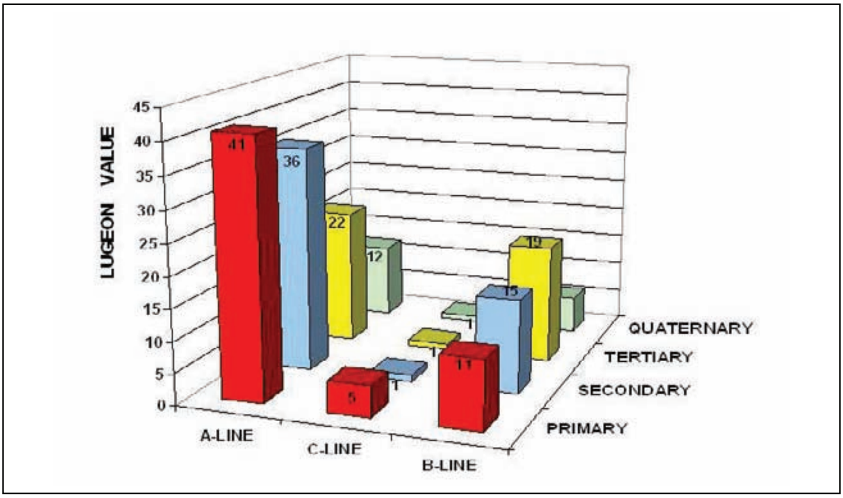


FIG. 1. 3-Dimensional Closure Plot.

The linked functionality of the grouting database to both CADD and customized queries is what makes true Level 3 Technology a powerful tool for fast and accurate analysis of grouting results. CADD plots and query analyses can be performed on demand and contain up-to-the-minute information. The rapid nature to which current grouting information can be presented and displayed for analysis by the project team is of substantial benefit over technologies that require a longer turn-around time of information in the form of drawings and queries, time on the orders of weeks, days or even hours. True Level 3 Technology gives the user the ability to present information on-demand in a matter of minutes. This is especially important for projects of a critical nature that require special attention to grouting progress and risk reduction. True Level 3 Technology is also important for projects with widely varied and unpredictable subsurface conditions, such as karst formations, that require careful, but relatively quick analysis and grouting method selection. The same level of care is warranted when conducting any grouting activity of high short-term risk potential, e.g., jet grouting under or adjacent to delicate, existing structures.

Technical challenges are faced frequently on any given grouting project, specifically due to the unknown nature of the subsurface. Technology in the grouting industry has fortunately advanced to new levels of sophistication to include additional methods of assessing the subsurface conditions to customize the program in order to effectively accommodate the geologic formations encountered. Two technologies that have proven to be valuable in analyzing subsurface conditions are Monitoring While Drilling (MWD) and High Resolution Borehole Imaging. While these technologies are not new, recent advances in the technology and their use in conjunction with the real-time monitoring and analytical tools has greatly enhanced and improved the ability to understand the subsurface conditions, resulting in a more efficient and productive program.

MONITORING WHILE DRILLING (MWD)

Interpreting the subsurface conditions and how it will impact the grouting program can often provide many challenges on a grouting project. Exploratory production holes only provide a small, unrefined picture of the subsurface conditions due to the limited amount of rock coring typically authorized, based on budget and schedule constraints. The limited number of exploratory core holes results in a relatively small number of intact subsurface specimens that can physically be evaluated. Although the majority of the holes drilled in a grouting program are considered production holes, the frequency and abundance of these production holes provides an opportunity to better define the subsurface conditions with the use of technologies that record drilling characteristics. Monitoring While Drilling (MWD) data allow each and every production hole to be treated as an 'exploration' hole (Bruce, 2003). That, combined with other advanced technologies such as borehole imaging, can provide the necessary subsurface information to optimize the grouting program. Each MWD production hole provides valuable information regarding the subsurface conditions. To compliment the subsurface investigation, Monitoring While Drilling (MWD) is recommended to collect and display real-time drilling parameters measured during advancement.

The subsurface investigation on a typical grouting project without MWD primarily consists of evaluating small cuttings and or water return produced from destructive drilling methods. In addition, some contracts specify that only one inspector (geologist or engineer) is required to inspect multiple drilling operations. Consequently, valuable information can be overlooked or missed. MWD provides continuous real-time information of the subsurface conditions for each and every drilling operation.

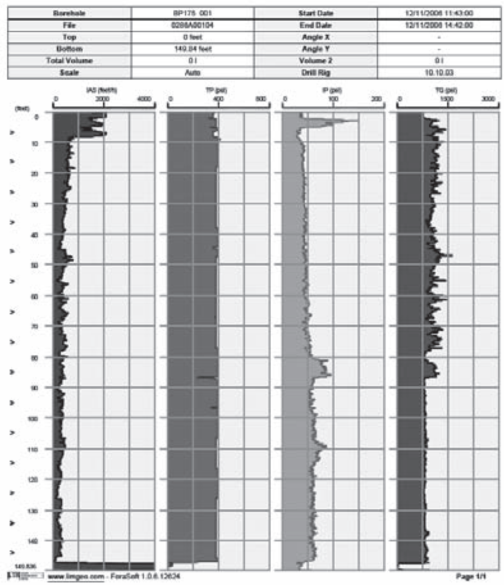


FIG. 2. Typical MWD Log.

Typical drilling parameters collected through MWD include instantaneous advance speed, tool pressure, torque, injection pressure of the drilling fluid, rotation speed, penetration rate, and thrust (hold back pressure). Figure 2 presents a typical MWD output log obtained from the field. There are other drilling parameters that exist and typically can be displayed. Specific energy is commonly determined based on recorded parameters. Plotted with depth, this parameter defines the energy required to advance through each lithological unit. The equation (Eq. 1) as defined by Weaver and Bruce (2007) is as follows:

$$e = \frac{F}{A} + \frac{2\pi N T}{A R} \qquad \text{Eq. 1}$$

where:
e = specific energy (kJ/m^3)
F = thrust (kN)
A = cross sectional area of hole (m^2)
N = rotational speed (revolutions/second)
T = torque (kN-m)
R = penetration rate (m/sec)

As discussed by Weaver and Bruce (2007), MWD information can benefit both the owner and the contractor. The data allow the owner to monitor the effectiveness of the program, and provides a basis upon which he can make a responsive change to the grouting program based on the results. The data also allow the contractor to optimize construction parameters and schedule work items.

As discussed by Bruce and Dreese (2010), meaningful electronic MWD data may be unobtainable for some drilling technologies. Drilling through critical zones should be observed by a geologist or engineer in these cases.

HIGH RESOLUTION BOREHOLE IMAGING

Borehole imaging has become a valuable tool for investigating subsurface conditions and identifying geologic attributes relevant to any grouting or foundation remediation project. High resolution borehole images provide many levels of detail and information that can be utilized for improving or optimizing any grouting program. In addition, composite cut-offs (the combination of concrete cut-offs and grout curtains) (Bruce et al., 2010) is being recognized as a superior approach to constructing seepage barriers through and below earthen embankments. The borehole images can be used to identify potential slurry loss zones that require additional attention through grouting to avoid major cut-off wall construction problems or dam safety issues. Imaging equipment exists in the industry that can perform the following:

- High resolution borehole imaging that provides a continuous oriented 360° image

- Fracture picking analysis: Software designed for feature analysis that determines strike, dip, and aperture thickness of identified fractures
- Produce tadpole plots and stereoplots
- Perform borehole deviation

Identifying the strike and dip of geologic discontinuities within the actual bedrock being treated will result in optimized grout hole orientation, the significance of which can lead to increased frequency of fracture and joint intersection during grout injection. Additionally, the aperture thickness identified during the fracture picking sequence provides a better understanding of injected grout travel distances as well as substantiating injected grout volumes.

As noted above, obtaining specimens via core drilling is generally limited to a few holes on production grouting projects due to budget and scheduling. Consequently, identifying the geologic attributes most likely to impact the grouting program is left to literature reviews, mapping of nearby outcrops, assessing the rock cuttings flushed during destructive rock drilling operations, and if used, MWD. The challenges inherent with interpreting the subsurface conditions are obviously intensified when limited samples can physically be observed. In addition, images of the in-situ condition provide a different interpretation than the core samples, for example solutioned zones with pipes often look like a "broken rock" zone in a core box.

Borehole images provide an accurate visual representation of the subsurface conditions that can be used to modify and improve the technical approach to the grouting program. For instance, a program that specifies high mobility grout (HMG) for rock treatment may require a sanded grout (medium mobility grout or MMG) or low mobility grout (LMG) based on the opening sizes observed in the images. If larger size cavities are observed in the images prior to treatment, such as the cavity shown in Figure 3, MMG or LMG may be identified as the appropriate initial grout type for a particular stage, prior to injecting HMG for final permeability reduction. Identifying these zones in advance can increase the contractor's efficiency with

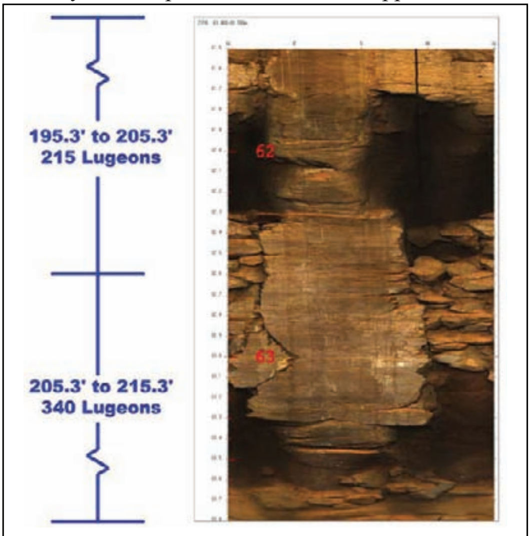


FIG. 3. Unwrapped Borehole Image (depth measured in meters).

regard to scheduling daily production work, and can reduce the amount of unnecessary grouting time and material by allowing a more appropriate grout mix to be used from the start. As discussed by Bruce and Dreese (2010), switching between HMG to MMG to LMG on a given hole is often specified in the contract, but is not easily achievable in the field as different equipment and delivery systems are required to inject MMG and LMG, as compared to HMG.

Borehole images can diagnose issues encountered during the production rock drilling program such as caving, 'rapid advancement' zones, rod drops, or water loss zones. While borehole imaging should not replace the need for core drilling, high resolution images can provide information that core specimens cannot. For instance, low recovery zones during core drilling can be attributed to a cavity, soil infilling or clay seam that washed out in the return water, or mechanical breaks due to the coring. The exact cause of the low recovery and location where the loss occurred can be difficult to ascertain. The borehole image allows the user to view these zones in situ.

Zones of high permeability are often difficult to explain without visual interpretation. High resolution images provide an understanding and meaning of pressure tests and magnitude of results as shown below in Figure 4.

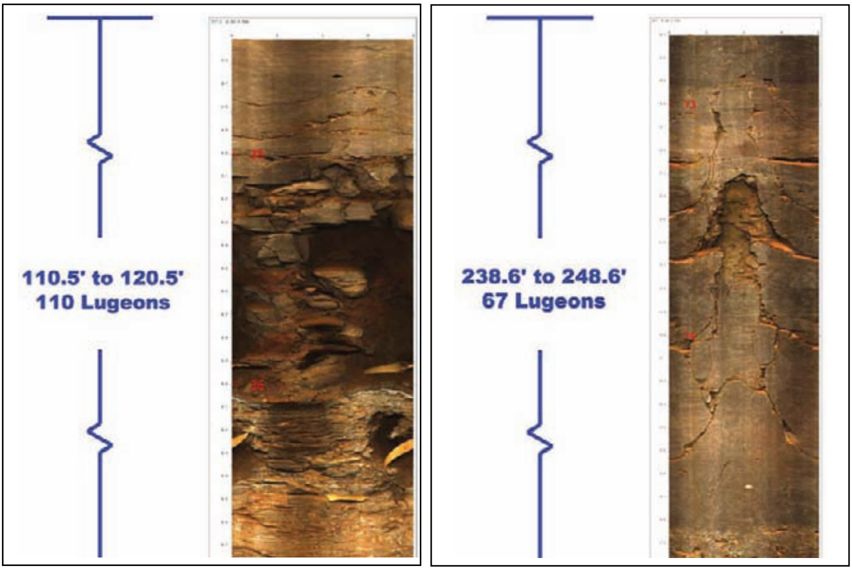
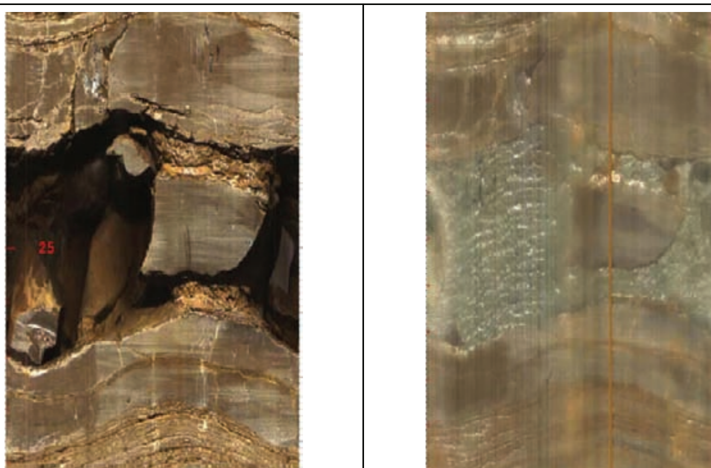


FIG. 4. Correlation Between Digital Borehole Image and Lugeon Value.

Borehole images can also be used to determine the effectiveness of the grouting program between each successive hole series (Primary to Secondary to Tertiary, etc) as well as between each grout stage along a hole during a downstage grouting program. Borehole images can identify grouted features and zones and can also identify un-grouted or incomplete grouted zones. Such information is valuable for evaluating closure plots and determining if or where additional treatment is required. Figure 5 below provides an example of a feature before and after treatment.



Pre-Treatment **Post-Treatment**

FIG. 5. Before and After Treatment.

Grouting projects often include a verification program to measure post-treatment permeability, and to determine if project goals were achieved. Typically, the verification holes are core drilled to visually assess the conditions of the rock specimens, but with the advancement in high resolution imaging technology, a large percentage of verification boreholes may be destructively drilled and imaged.

Images of the pre-treated boreholes should also be utilized as part of the daily computer monitoring operations throughout the grouting program. Inflatable packers are commonly lost or damaged due to inflation in cavities or highly fractured and broken zones that are often overlooked during production rock drilling. When available, borehole images should be reviewed for such issues. Over the duration of a large project, the potential cost savings could be quite high. Imaging systems are frequently overlooked due to the turbidity of the water generally encountered during the high production type grouting programs. High resolution acoustic televiewers are recommended as a viable alternative to optical televiewers, and generally can provide similar useful information produced by the optical televiewer.

In addition, such televiewers are often used to investigate the actual in-situ conditions of jet grouted or deep mixed columns or panels when core recoveries have been poor, but there is a likelihood that the coring process itself has caused the poor coring result.

DIGITAL PHOTOGRAMMETRY

Geologic mapping to determine orientation of bedding planes, structural features, and discontinuities in general is prudent to fully understand the bedrock characteristics and permeability relationships. This information serves to guide, plan and implement successful grouting programs. Such data provide the necessary information for determining hole orientation and spacing, and even the number of lines required to adequately treat the formation. Historically, such data have been obtained through classical geologic mapping using line surveys and a Brunton compass. Recently, very precise and accurate digital photogrammetric methods have become available to supplement historical methods. Digital photogrammetry allows for safe data collection of digital images taken of rock exposures with exceptional optical location and planimetric/depth accuracy. These methods offer improved data collection for relatively inaccessible rock outcrops such as steep dam abutments and highway rock exposures which are unsafe to access.

Digital photogrammetry allows geologic mapping of areas with difficult accessibility, and is capable of producing 3-dimensional models including digital terrain models (DTM) that can be directly incorporated into the project and subsequent geotechnical evaluations. Additionally, the number of discontinuity data sets collected through digital photogrammetry can be orders of magnitude higher than the number collected manually given the ability to collect images efficiently of the entire rock exposure without climbing or rappelling.

AUTOMATED INSTRUMENTATION

Monitoring the response of a structure, its foundation and any adjacent structures concurrent with grouting operations is an essential element of any grouting project. Instrumentation provides a quantitative measure of the formation's condition and the structure's performance. Automating the instrumentation provides real-time monitoring of the 'vital signs', and operating in conjunction with real-time computer monitoring, will provide the critical data necessary to make informed and educated decisions in a timely manner.

The subsurface foundation being treated during a grouting program is typically monitored with vibrating wire piezometers for pressure and weir monitors for seepage. Automated instruments can be installed in existing manual monitoring stations, such as existing casagrande piezometers, monitoring wells, and weir structures, or they can be installed at new defined locations. Important structures and foundations that may be adversely affected by the grouting operations can be monitored with crack gauges, strain gauges, tiltmeters, settlement sensors, inclinometers, load cells and earth pressure cells.

Automation of the various instruments installed provides a real-time response of the subsurface foundation and adjacent structures to the drilling and grouting operations performed throughout the duration of the grouting program (Figure 6). The information provided can be used to determine acceptable operating parameters during grouting operations such as appropriate grouting pressures to use due to existing pore pressures. The information can also be used to identify critical areas requiring special or immediate attention such as localized piezometric highs or lows, or zones of significant seepage, settlement or movement. During production, automated instrumentation data can be used by the project team to schedule work items, to analyze the performance of the grouting program, to minimize adverse effects of the grouting operations to the structure, foundation, and adjacent structures, and to help modify grouting methods and procedures. After the grouting program is complete, the automated instruments can also be used to analyze the post-construction effectiveness of the grouting program by recording any changes in piezometric levels, measured seepage, settlement, or movement of adjacent structures. In order to better determine the effectiveness of grouting program during production and post-construction, it is recommended that the automated instrumentation system be installed prior to the start of drilling and grouting operations to establish baseline conditions.

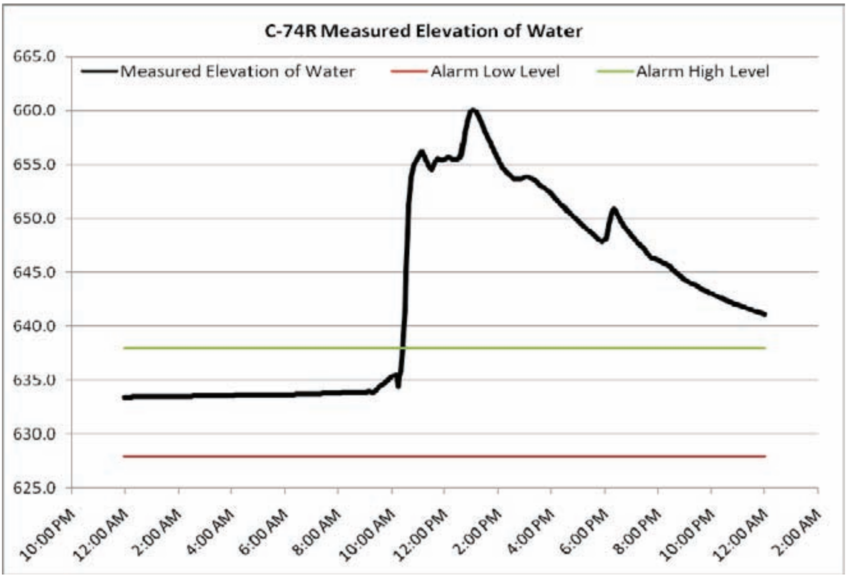


FIG. 6. Plot of Piezometer Response to Grouting Operations.

Communication between project team members is essential when viewing and analyzing instrumentation data, and when modifying project procedures based on observed responses of the subsurface foundation and adjacent structures. Open communication is essential to determine and disseminate the acceptable operating ranges and threshold values of each instrument. Standard Operating Procedures should be developed to address the monitoring and maintenance of the instruments as well as to provide an action plan for the proper notification of critical personnel in the event of an instrument exceeding its threshold value. Threshold values may need to be adjusted as grouting operations progress, program modifications are made, and subsurface conditions change.

CASE HISTORIES

Case histories that document the successful use of the state of the art technology discussed in this paper include three U.S. Army Corps of Engineers DSAC-1 Dams. DSAC, or Dam Safety Action Classification is a USACE initiated risk-informed approach and ranges from DSAC-1 which is the highest priority and highest risk to DSAC-5. The three DSAC-1 dams discussed herein are Clearwater Lake Dam, Missouri; Wolf Creek Dam, Kentucky; and Center Hill Dam, Tennessee.

Clearwater Lake Dam, Missouri

Clearwater Lake Dam, located in Piedmont, Missouri is a U.S. Army Corps of Engineers (USACE) owned dam. In 2003, a sinkhole was discovered on the upstream slope of the 4,200-foot-long, 150-foot-high earthen embankment dam. As an interim risk-reduction measure (IRRM), the USACE decided to install a grout curtain approximately 200 feet in length immediately downstream of the sinkhole to investigate and determine the cause and extent of the sinkhole. During this initial exploration, a large solution feature, approximately 25 feet wide by 170 feet tall was discovered in the foundation bedrock. Low mobility grout (LMG) was successfully utilized as the appropriate grout type to fill the feature, but it was determined that additional treatment would be necessary in the vicinity of the sinkhole. To accommodate this additional work, and to explore the foundation bedrock underlying the remaining embankment to identify potential locations of other solution features, two other projects were awarded; Phase 1 and Phase 1b Exploratory Drilling and Grouting. More extensive efforts were performed along the entire length of the embankment during the Phase 1 and 1b contracts. The two projects (essentially combined into one) consisted of a 2-line grout curtain with holes drilled on 10-foot centers from left to right abutment, with the intention of characterizing and pre-treating the foundation material in preparation for a proposed cutoff wall.

Level 3 Technology was utilized for real-time monitoring, display, and collection of all data. To complement this technology, high resolution borehole images were obtained to map bedrock discontinuities and other geologic features, identify opening sizes for determining most appropriate grout type, verifying complete treatment of openings, and for performing borehole deviation. MWD was also utilized at

Clearwater on various drilling rigs for obtaining real-time drilling characteristics of the underlying material for correlation of drilling characteristics with borehole images, pressure testing results, and injected grout volumes. The treatment of the solution feature and pregrouting of the karst bedrock has permitted the construction of the cutoff wall without a major slurry loss incident.

Wolf Creek Dam, Kentucky

Wolf Creek Dam is located near Jamestown, Kentucky within the USACE Nashville District. From 2007-2008, a grouting program was implemented both for interim risk reductions measures, and as an initial phase for the construction of a composite cutoff consisting of a grout curtain and a concrete barrier wall along the length of the embankment section of the dam. Additional grouting at the right abutment, the rock foundation below the concrete section of the dam, and along a downstream section adjacent to the concrete/embankment interface has also been performed, or is currently under construction.

In addition to the use of Level 3 Technology for monitoring grouting operations, other technologies utilized at the site included weekly upload of grouting as-built drawings to a website, and high resolution borehole imaging for the Phase I program. The Level 3 Technology of the grouting operations allowed for the rapid dissemination of grouting results to critical personnel at various levels of project oversight within the USACE and an independent Board of Consultants tasked with making technical decisions concerning grouting methods and determining whether additional holes were required and their appropriate locations. Borehole imaging was utilized to identify the condition of the rock foundation in critical areas, and to identify zones that required additional grouting. In addition to the image data, the camera probe also recorded borehole deviation data that was used as a quality control measure to ensure the consistent alignment of the grout curtain along the entire grout curtain length to a maximum depth of 345 feet from the top of work platform. The incorporation of these technologies resulted in the fast and accurate construction of 3,750 feet of a two-line curtain, which was completed within an accelerated project schedule and allowed for the timely start of the second phase of work, the construction of the barrier wall.

Automated instrumentation was added to key critical areas of the dam after Phase I for analysis of the performance of the subsequent work phases. The automated system was installed by the USACE to supplement the manual instrumentation system of piezometers, survey points, inclinometers, and extensometers that already existed at the site. The automated instruments were designed to record and display the response of the subsurface foundation and embankment to future grouting operations as well as the construction of the barrier wall. However, the real-time monitoring of the grouting operations and the automated instrumentation were not integrated. Consequently, operators were not always aware of issues for hours or days after completion of a stage, and forensic investigation to identify all activities at that specific time were not always possible.

Center Hill Dam, Tennessee

Center Hill Dam is also within the Nashville District and is located near Lancaster, Tennessee. From 2008-2010, Phase I of the remediation at Center Hill Dam was performed as both interim risk reduction measures and as part of an overall grout curtain and composite barrier wall cut-off, built to reduce overall seepage and instability at the site. Phase I work consisted of the grouting of the main dam embankment, left abutment groin, and left rim sections of the dam. Phase II of the project involves the construction of the barrier wall portion of the composite wall cut-off along the embankment section of the dam. Work for Phase II is scheduled to begin in late 2011.

State-of-the-art technologies utilized during Phase I operations included Level 3 computer monitoring, weekly update of grouting as-built drawings to a website for technical review, geophysical analysis of subsurface conditions using electrical resistivity, high resolution borehole imaging, down the hole camera technology for inspection of a large open-air cavity encountered during drilling operations in the left rim, and a real-time automated instrumentation system consisting of vibrating wire piezometers and weir monitors. Real-time monitoring results and on-demand grouting as-builts were used by USACE personnel and an independent Board of Consultants to make rapid, but informed technical decisions and program modifications, including the addition and deletion of holes based on grouting results.

The incorporation of the automated instrumentation system by the contractor that included alarm levels allowed for the real-time display and analysis of the piezometric response of the subsurface foundation to the drilling and grouting operations. The ability of the automated system to both record and display in real-time the changes in foundation pore pressures during operations allowed for the comfortable use of increased grouting pressures in specified zones to more effectively penetrate difficult rock formations with grout, and was integrated with the computer monitoring as a result of experiences at Wolf Creek Dam. Subsequently, automated instrumentation is required in the Center Hill Phase 2 Cut-off Wall contract.

CONCLUSIONS

The technological leaps in grouting in the few years prior to 2003 caused a renaissance in grouting in the United States, a rethinking of the usefulness and viability of grouting. The incorporation of new technologies over the last 10 years has redefined the role of grouting from use as a secondary measure, to that of a reliable and durable solution to a wide variety of projects, including interim risk reduction measures, permanent foundation remediation, and composite barrier construction.

The incorporation of computer monitoring, high resolution borehole imaging, MWD, and automated instrumentation provide many advantages when properly integrated. First is the ability to present relevant up to the minute information.

Second is the ability to "see" the actual subsurface conditions that exist at the site. Third is the ability to quickly and accurately reduce, filter out, and display pertinent information and grouting results. Finally, is the ability to view in real-time the response of the subsurface foundation and critical structures to current drilling and grouting operations. All of these program advantages result in the following project benefits:

- Fast and reliable information
- Rapid response to encountered foundation conditions and grouting results
- Informed decision making by project team
- Reduction in project schedule
- Reduction in project cost
- Increase in confidence of grouting results
- Greater success in meeting or exceeding project goals

The benefits that these integrated technologies provide also depends largely on the personnel responsible for its daily operation. Critical decisions made by the operators that may impact project performance and/or overall condition of the structure, require the assigned personnel to have experience with each of the tools as well as experience in dam safety issues and considerations.

The incorporation of these technologies into the field of grouting has allowed the solution of grouting to be considered as not only a viable alternative, but as a leading recommended, and even superior solution for a vast number of foundation stability and seepage remediation projects for critical infrastructure.

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Site Characterization for Cohesive Soil Deposits Using Combined In Situ and Laboratory Testing

Don J. DeGroot[1], M. ASCE and Charles C. Ladd[2], Dist. M., ASCE

[1]Professor, Department of Civil and Environmental Engineering, University of Massachusetts Amherst, Amherst, MA, USA, 01003; degroot@ecs.umass.edu
[2]Edmund K. Turner Professor Emeritus, Department of Civil and Environmental Engineering, Massachusetts Institute of Technology, Cambridge, MA, USA; ccladd@mit.edu

ABSTRACT: The paper presents recommendations for conducting geotechnical site characterizations to obtain design parameters for settlement and stability analyses. It focuses on relatively uniform, saturated cohesive soil deposits with near zero Standard Penetration Test blow counts and *soft ground* conditions, which means that construction will load the foundation soil beyond its preconsolidation stress. The site characterization program should select an appropriate combination of in situ tests for soil profiling (identify soil types and their relative state) and laboratory tests on undisturbed samples for strength-deformation-flow properties. Although the tools, procedures, and interpretation methods needed to conduct a reliable site characterization program are well developed, general practice often ignores this knowledge. Thus a prime objective of the paper is to provide recommendations for moving practice closer to the state of the art. Components of site characterization covered include site stratigraphy, drilling and undisturbed sampling, in situ testing, and laboratory consolidation and strength testing. Key recommendations include: fixed piston sampling using drilling mud and tubes with an appropriate geometry, piezocone testing for determination of site stratigraphy, radiography of sample tubes, debonding of samples from tubes, evaluation of sample quality, CRS testing to measure consolidation behavior, and anisotropic or K_0 consolidated strength tests to measure undrained shear strength behavior.

INTRODUCTION

Geotechnical site characterization is the determination of soil stratigraphy, in situ pore water pressure conditions, and soil properties for analyses and design of geotechnical engineering projects. It is best conducted using a combination of in situ testing and laboratory testing of high quality undisturbed samples. The tools and procedures that can be used to perform an effective and reliable site investigation are well developed and should be thoroughly embedded in practice. However, the authors encounter pervasive examples of poor site characterization procedures and believe that the state-of-practice has regressed over the past several decades. While limited budgets imposed by clients is a factor, of greater concern is the apparent ignorance of how to achieve more reliable information from a site investigation and the lack of appreciation of the extent to which data from poor quality testing and sampling can adversely affect the design, performance, and cost of geotechnical projects. It is thus a prime objective of this state-of-practice paper to identify common types of poor site characterization practice and to make recommendations for correcting these deficiencies. Significantly, most of these recommendations involve relatively little to no extra time and cost compared to current practice.

The focus of the paper is soft ground construction projects that involve relatively uniform deposits of saturated cohesive soils. Cohesive soils include clays, silts, and organic soils of low to high plasticity, although for convenience the paper will often refer to all cohesive soils as "clays". These soils have usually been deposited in an alluvial, lacustrine or marine environment and typically have Standard Penetration Test (SPT) blow counts that are weight-of-rod or hammer, except within surface drying crusts. They can have highly varied and complex stress histories, which is the dominant factor controlling their compressibility and strength behavior. A soft ground condition is defined as situations where the applied surface loads from construction produce stresses that exceed the preconsolidation stress of the underlying cohesive foundation soil. Projects with soft ground conditions require estimates of the amount and rate of settlement and an assessment of undrained foundation stability. As such, determination of the relevant soil properties for analyses and design is the major objective of a site characterization program after first defining the soil profile and groundwater conditions.

The paper begins with the objectives of a site characterization program and the general methodology for fulfilling these objectives. This is followed by a brief overview of the fundamentals of clay behavior, knowledge of which is essential for appreciating the types of tools and procedures that are essential for conducting an effective and reliable site characterization program. Common poor practice procedures and best practice recommendations are then presented covering: drilling and undisturbed soil sampling, in situ testing, soil classification, selection of laboratory test specimens, laboratory consolidation testing, and laboratory shear strength testing. The paper concludes with brief guidelines on selection of design parameters for settlement and stability analyses from the site characterization.

SITE CHARACTERIZATION: OBJECTIVES AND METHODOLOGY

The two key objectives of a site characterization program are to define the site stratigraphy and to estimate the relevant soil properties needed for design. The first objective encompasses determining the location and relative state of principal soil types (stiffness if cohesive and density if granular) and ground water conditions (location of water table and possible deviations from hydrostatic pore pressures). The second objective quantifies the engineering properties of the foundation soils that are needed for settlement and stability analyses, as listed in Table 1. All programs must include steps to properly classify the foundation soil types. In terms of properties, the in situ stress state (σ'_{v0}, u_0, and σ'_p) is the most important and required in all cases, while the need for other parameters depends on the specific design objectives.

Table 1. Clay Properties for Soft Ground Construction (from Ladd and DeGroot 2003)

A. SETTLEMENT ANALYSES		
Analysis	**Design Parameters**	**Remarks**
Initial due to undrained shear deformations (ρ_i)	- Young's modulus (E_u) - Initial shear stress ratio (f)	See Foott & Ladd (1981)
Final consolidation settlement (ρ_{cf})	- Initial overburden stress (σ'_{v0}) - Preconsolidation stress (σ'_p) - Final consolidation stress (σ'_{vf}) - Recompression Ratio (RR) - Virgin Compression Ratio [$CR = C_c/(1 + e_0)$]	- Check if hydrostatic u - Most important - Elastic stress distribution - RR ≈ 0.1 – 0.2 x CR - Very important
Rate of consolidation: vertical drainage ($\bar{U}_v$)	- Coef. of consolidation ($c_v = k_v/m_v\gamma_w$)	- Need NC value
Rate of consolidation: horiz. drainage ($\bar{U}_h$)	- Horiz. coef. of consol. ($c_h = c_v \bullet k_h/k_v$)	- Eff. c_h < in situ c_h from mandrel disturbance
Secondary comp. settlement (ρ_s)	- Rate of secondary compression ($C_\alpha = \Delta\varepsilon_v/\Delta\log t$)	- ρ_s only imp. for low t_p - $C_\alpha/CR = 0.045 \pm 0.015^\dagger$
B. UNDRAINED STABILITY ANALYSES		
During initial loading: assumes no drainage (UU Case)	- Initial in situ undrained shear strength (s_u)	- Isotropic vs. anisotropic s_u analyses - SH very desirable to evaluate s_u/σ'_{v0}
During subsequent (staged) loading: includes drainage (CU Case)	- Initial s_u for virgin clay - Increased s_u for NC clay ($S = s_u/\sigma'_{vc}$ at OCR = 1) - Results from rate of vertical and horizontal drainage	- Isotropic vs. anisotropic s_u - SH essential to determine when $\sigma'_{vc} > \sigma'_p$
Other Notation: NC = Normally Consolidated; OCR = Overconsolidation Ratio; SH = Stress History; t_p = time for primary consolidation; σ'_{vc} = vertical consolidation stress. †Note: ± is defined as a range unless followed by SD; then it defines ± one standard deviation.		

For settlement analyses the magnitude of the final consolidation settlement is always important and the key in situ parameters are stress history (SH = values of σ'_{v0}, σ'_p and $OCR = \sigma'_p/\sigma'_{v0}$) and the value of CR. Typical practice assumes that the total settlement at the end of consolidation equals the predicted one-dimensional final consolidation settlement (ρ_{cf}), i.e., initial settlements due to undrained shear deformations (ρ_i) are ignored. This is reasonable except for highly plastic (CH) and organic (OH) foundation soils with low factors of safety and slow rates of consolidation (large t_p). For projects involving preloading (with or without surcharging) and staged construction, predictions of the rate of consolidation are required for design. These involve estimates of c_v for vertical drainage and also c_h for horizontal drainage if vertical drains are installed to increase the rate of consolidation.

The values of c_v and c_h for soft ground conditions should be for normally consolidated (NC) clay. Settlements due to secondary compression become important only with rapid rates of primary consolidation, as occurs within zones having vertical drains. For such situations, designs often use surcharging to produce overconsolidated soil under the final stresses, which reduces the rate of secondary compression.

For undrained stability analyses, Table 1 describes two conditions: the UU Case, which assumes no drainage during (rapid) initial loading; and the CU Case, which accounts for increases in strength due to drainage that occurs during staged construction. Both cases require knowledge of the variation in s_u with depth for virgin soil. However, the CU Case also needs estimates of s_u for NC clay because the first stage of loading should produce $\sigma'_{vc} > \sigma'_p$ within a significant portion of the foundation. Most stability analyses use "isotropic" strengths, that is $s_u = s_u$(ave), while anisotropic analyses explicitly model the variation in s_u with inclination of the failure surface. Knowledge of the initial stress history is highly desirable for the UU Case, in order to check the reasonableness of the s_u/σ'_{v0} ratios selected for design, and is essential for the CU Case.

To fulfill the above objectives, the optimal site characterization program should use a combination of in situ tests and undisturbed sampling for laboratory testing. In situ tests, such as the piezocone (CPTU), are best suited for soil profiling since they can rapidly provide a (semi) continuous record of the in situ penetration data that are used for identifying the spatial distribution of soil types and information about their relative state. In situ tests, however, are not as well suited for direct determination of strength-deformation properties (such as those listed in Table 1) because of the effects of installation disturbance and poorly defined and uncontrollable boundary conditions. Therefore, derived soil properties from in situ tests depend on empirical correlations that are generally quite scattered and often unreliable without site specific verification. Conversely, most laboratory tests have well controlled boundary and drainage conditions that enable direct measurement of strength-deformation properties. However, the reliability of laboratory data is strongly dependent on proper test procedures and good quality undisturbed samples. Otherwise, variable, and often unknown, degrees of sample disturbance can cause poor quality laboratory data that produce erroneous spatial trends in soil properties.

The scope of a site investigation program should follow the advice of Peck (1994) for characterizing a relatively homogeneous clay layer versus a highly heterogeneous deposit. If prior knowledge indicates a reasonably well-defined profile having soft clay, then careful undisturbed sampling and laboratory testing is warranted if the design concept has a significant potential for settlement and stability problems. But with an ill-defined soil profile with heterogeneous layers, then the program should emphasize in situ testing to locate the weaker, more compressible soil layers with minimal collection of undisturbed samples for laboratory testing. The SPT has the advantage of giving relative stiffness values (e.g., gravel and sands vs. cohesive soils) from the blow counts and providing samples for visual classification that can be refined by lab testing, while the CPTU test offers detailed soil profiling but no collection of soils samples.

In summary, site characterization programs should combine the best aspects of in situ and laboratory testing, with the scope depending upon the extent of prior knowledge about the nature of the foundation soils and the size and complexity of the project. For small projects and for preliminary investigations for larger projects at sites of unknown subsurface conditions, the SPT is a prudent first choice as it is a robust tool and will provide relative stiffness values and soil samples for classification. For soft ground conditions involving relatively uniform saturated clay deposits, the focus should primarily be on the use of multiple CPTU soundings for soil profiling and assessment of spatial variability. This should be coupled with collection of good quality undisturbed samples from selected boreholes for laboratory strength-deformation testing. The laboratory data also can be used to develop site specific correlations for more reliable estimates of σ'_p and s_u from CPTU data.

FUNDAMENTALS OF CLAY BEHAVIOR

An understanding of clay behavior is necessary to formulate and conduct a site characterization program that will properly address all potential design requirements regarding adequate stability and allowable settlements. This includes selecting the optimal combination of in situ testing, soil sampling, and laboratory testing, along with the appropriate equipment and test methods. It is again essential during evaluation of the measured data sets leading to selection of key soil properties needed for analyses and final design (such as σ'_{v0}, σ'_p, CR, and s_u). This section summarizes information on clay behavior from Ladd and DeGroot (2003), with emphasis on four major aspects of behavior: stress history and its effects on consolidation and undrained strength; undrained strength anisotropy; rate effects; and sample disturbance. Leroueil and Hight (2003) present a detailed review of clay behavior.

Stress History

All important aspects of clay behavior are influenced by stress history (σ'_{v0}, σ'_p and OCR), as shown in Fig. 1 which illustrates the significant changes in the 1-D compressibility and flow properties when an intact structured clay is loaded beyond its preconsolidation stress. S-shaped virgin compression curves in ε_v-logσ'_v space (such as illustrated in Fig. 1) have continuous changes in CR with stress level, with the maximum value (CR_{max}) located just beyond σ'_p. As the loading changes from recompression (OC) to virgin compression (NC), c_v and C_α also undergo marked changes. For undisturbed clay, c_v(OC) is typically 5 to 10 times the value of c_v(NC), which is mostly due to a lower coefficient of volume change ($m_v = \Delta\varepsilon_v/\Delta\sigma'_v$) in the OC region. The rate of secondary compression increases as σ'_v approaches σ'_p and often reaches a peak just beyond σ'_p. This change in C_α is uniquely related to the slope of the compression curve as demonstrated by Mesri and Castro (1987), such that C_α/CR is essentially constant for both OC and NC loadings (Note: here "CR" equals $\Delta\varepsilon_v/\Delta\log\sigma'_v$ at all stress levels). For most cohesive soils, $C_\alpha/CR = 0.04 \pm 0.01$ and 0.05 ± 0.01 for inorganic and organic clays and silts, respectively (Table 16.1, Terzaghi et al. 1996). The vertical permeability decreases with increasing σ'_v to form an approximate linear relationship between e and logk_v. The permeability coefficient $C_k = \Delta e/\Delta\log k_v$ is empirically related to e_0 such that for most soft clays, $C_k \approx (0.45 \pm 0.1)e_0$ (Tavenas et al. 1983, Terzaghi et al. 1996).

The preconsolidation stress is more appropriately referred to as a vertical yield stress (σ'_{vy}) that separates small, mostly elastic strains from large plastic strains, although the familiar σ'_p notation is used in this paper. Jamiolkowski et al. (1985) divided the mechanisms causing the preconsolidation stress for horizontal deposits with geostatic stress conditions into four categories:

A. Mechanical due to changes in the total overburden stress and pore pressure conditions. Produces uniform stress history profile with constant $\sigma'_p - \sigma'_{v0}$.

B: Desiccation due to drying from evaporation and freezing. Often produces highly erratic stress history profile.

C: Drained creep (aging) due to long term secondary compression. Produces uniform stress history profile with constant σ'_p/σ'_{v0}.

D: Physico-chemical phenomena leading to cementation and other forms of interparticle bonding.

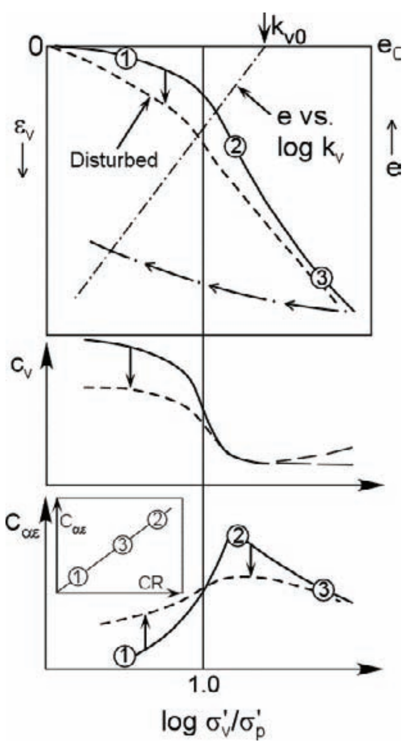


FIG. 1. Fundamentals of one-dimensional consolidation behavior: compression curve, k_v, c_v, and $C_{\alpha\varepsilon}$ vs. normalized vertical effective stress (from Ladd and DeGroot 2003).

Categories A, B and C are well understood and should be closely correlated to the geological history of the deposit. Although Category D mechanisms are poorly understood, they play a major role in some deposits, such as the sensitive, highly structured Champlain clay of eastern Canada. The authors hypothesize that various forms of cementation may be primarily responsible for the S-shaped virgin compression curves exhibited by many (perhaps most) natural soft clays. Cementation also can cause significant changes in σ'_p over short distances (i.e., even at different locations within a tube sample). In any case, very few natural clay deposits are truly normally consolidated, unless either recently loaded by fill or pumping if on land or by recent deposition if located under water. It is also common for two or more mechanisms to have occurred during the geologic history of a deposit. These factors highlight the importance of developing an understanding of the geologic history of a site so that stress history data from in situ and laboratory testing can be properly evaluated and understood.

Fig. 2 plots the undrained shear strength ratio (s_u/σ'_{vc}) versus OCR for two clays, each with three modes of shearing: triaxial compression (TC), triaxial extension (TE), and direct simple shear (DSS). The data show the dominant influence of stress history (i.e., OCR) on undrained shear strength that can be modeled by the SHANSEP equation

$$s_u/\sigma'_{vc} = S(OCR)^m \qquad (1)$$

The K_0 consolidated-undrained shear (CK_0U) tests were run using the SHANSEP reconsolidation technique for the plastic, insensitive AGS clay and the Recompression technique for the lean, highly sensitive James Bay clay (these techniques are described later in the paper). The strength increase with OCR for the AGS clay is mainly due to changes in the shear induced pore pressure from contractive (positive) to dilative (negative) with increasing OCR, as is typical of most non-structured soils. In contrast, the shape of the yield surface is mainly responsible for the strength increase for the highly structured and cemented James Bay clay. Also note the large decrease in the S values (solid symbols) caused by consolidating this clay beyond σ'_p which destroys the cementation bonds.

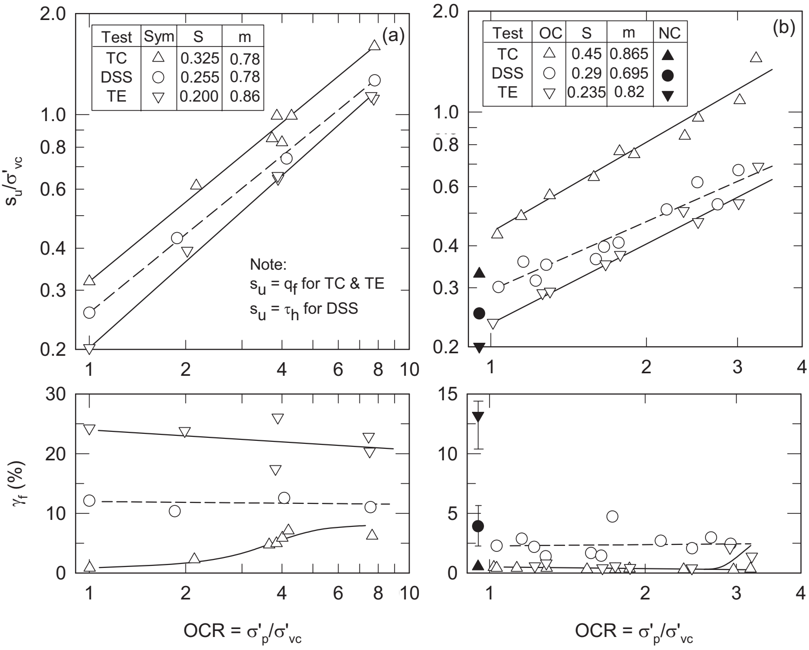


FIG. 2. OCR versus undrained strength ratio and shear strain at failure from CK_0U tests: (a) AGS plastic marine clay (PI = 43%, LI = 0.6) via SHANSEP (Koutsoftas and Ladd 1985); and (b) James Bay sensitive marine clay (PI = 13%, LI = 1.9) via Recompression (B-6 data from Lefebvre et al. 1983) [after Ladd 1991].

Undrained Strength Anisotropy

The undrained shear strength of clays depends on the orientation of the major principal stress at failure (σ'_{1f}), with s_u decreasing as σ'_{1f} rotates from vertical (δ = 0º, same direction as deposition) to horizontal (δ = 90º). Fig. 3 plots peak undrained strength ratios versus Plasticity Index (PI) from CK_0U TC (δ = 0º), TE (δ = 90º), and DSS ($\delta \sim 45°$) tests run on various normally consolidated clays and silts (but excluding varved deposits). The data show a constant ratio in TC; generally much lower DSS strengths that tend to decrease with lower PI; and even smaller ratios for shear in TE, especially at low PI. These data clearly demonstrate that most OCR = 1 soils exhibit significant s_u anisotropy that generally becomes more important in low PI clays, especially if also sensitive. Varved clays represent a special case wherein horizontal (DSS) shearing gives an unusually low peak τ_h/σ'_{vc} of only 0.15 to 0.18 for northeastern U.S. deposits (Ladd 1991). Overconsolidated clays also can exhibit pronounced anisotropic behavior, as illustrated by the data in Fig. 2. For relatively non-structured soils such as the AGS clay, the degree of anisotropy usually decreases with increasing OCR, i.e., the value of m is larger in extension than for compression. By contrast, OCR may have little effect on the anisotropy of sensitive, cemented soils such as the James Bay clay.

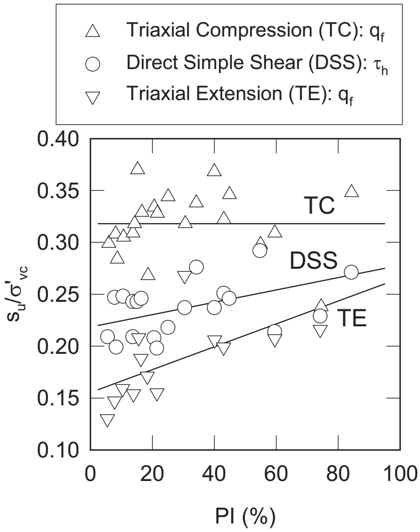
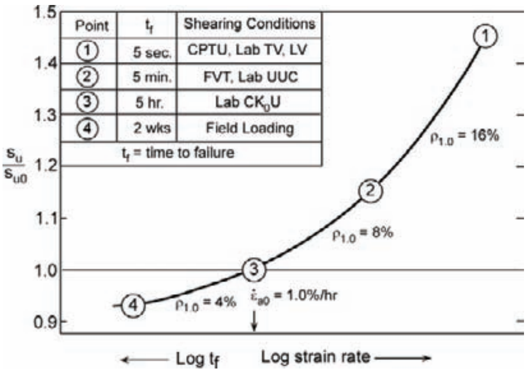


FIG. 3. Undrained strength anisotropy from CK_0U tests on NC clays and silts (from Ladd 1991).

Strain Rate

Clays are sensitive to the rate of strain (time to failure), with a significant increase in s_u at fast rates of undrained shearing. Fig. 4 was developed to illustrate how rate effects could influence the strength measured in different in situ and laboratory shear tests for a hypothetical low OCR clay. For the assumed values of $\rho_{1.0}$ = increase in s_u per change in log strain rate ($\Delta\varepsilon_a/\Delta t$) or time to failure ($t_f$), the sketch shows, that relative to a lab CK_0U test with an assumed reference strain rate = 1%/hr:

- extremely fast shearing ($t_f \approx 5$ sec), such as occurs in CPTU and lab strength index tests, increases s_u by almost 50%.
- fast shearing ($t_f \approx 5$ min), such as occurs in the field vane test (FVT) and lab unconsolidated-undrained triaxial compression (UUC) tests, increases s_u by about 15%.
- very slow shearing ($t_f \approx 2$ weeks), such as might occur in the field, reduces s_u by almost 10%.



These percentages (which assume adjustment to the same mode of shearing) are approximate and will vary with soil type and its stress history, and possibly temperature (e.g., Arctic soils with near freezing temperature can be extremely rate sensitive). But there is little question that fast shearing rates associated with in situ testing and laboratory strength index and UUC tests produce strengths that are too high for design, all else being equal.

FIG. 4. Illustration of effect of rate of shearing on low OCR clay (from Ladd and DeGroot 2003).

Sample Disturbance

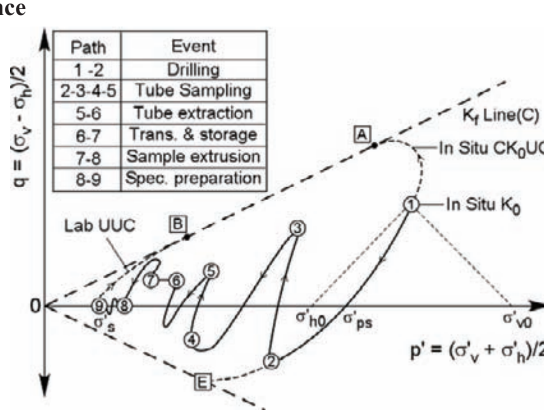


FIG. 5. Hypothetical stress path during tube sampling and specimen preparation of centerline element of low OCR clay (modified from Ladd and DeGroot 2003).

Sample disturbance is the most significant issue affecting the quality and reliability of laboratory test data for clays. It causes changes to the stress state and structure of an intact soil and as a result all key design parameters such as σ'_p, CR and s_u are adversely influenced. Each stage of the sampling process, from initiation of drilling to preparation of laboratory specimens, causes potential disturbance. Ladd and DeGroot (2003) developed Fig. 5 to illustrate the anticipated effective stress path for a low OCR clay as stresses change from the in situ state (Point 1) to that at the beginning of laboratory testing (Point 9) as a result of disturbance caused by drilling, sampling, storage and handling. The reduction in soil stress to the sample effective stress (σ'_s) is due to a combination of disturbance induced internal swelling and especially shear distortions. The magnitude of σ'_s differs markedly, and unpredictably, from the in situ stress state (σ'_{v0}, σ'_{h0}) and likewise from that of the "perfect" sample, which is defined as the effective stress (σ'_{ps}) resulting from undrained release of the in situ shear stress (i.e., reduction in σ_{v0} to σ_{h0} during borehole drilling). The reduced effective stress decreases s_u from laboratory index strength tests (e.g., TV, UUC) because shearing starts at σ'_s (Point 9 in Fig. 5) compared to the in situ behavior under field loading that starts at Point 1. But more importantly, poor sampling and handling will cause large shear distortions that "destructure" the soil and lead to even larger differences in undrained shear behavior relative to the in situ clay.

Fig. 1 further illustrates the adverse influence of sample disturbance on clay behavior by contrasting the one-dimensional consolidation response of a disturbed sample (dashed lines) relative to the intact (undisturbed) soil. Disturbance results in a more rounded compression curve with greater ε_v at all stress levels (giving higher RR and lower CR values). This tends to obscure and usually lower σ'_p, especially with more structured soils. During recompression, c_v(OC) is usually much lower and C_α(OC) much higher. The only parameters not significantly affected by sample disturbance are c_v well beyond σ'_p and the e–logk_v relationship, unless there is severe disturbance.

POOR PRACTICE: EXAMPLE CASE HISTORY

Fig. 6 presents data from an investigation conducted at a CL marine clay site that included SPT borings, Shelby tube sampling, field vane testing, and laboratory classification, strength index and consolidation testing on the tube samples. From extensive literature the main cohesive soil deposit in the region generally consists of low OCR Boston Blue Clay (BBC) with an overlying desiccated crust. Simple inspection of the field and lab test results indicate very poor quality data for this relatively uniform clay deposit with near hydrostatic pore pressure conditions. The consolidation tests produced $\sigma'_p < \sigma'_{v0}$ below El. – 20 m because of excess sample disturbance, which is also shown by the very low s_u(TV) values. The s_u(FVT) values are highly scattered and also very low below El – 20 m because of the poor equipment and test procedures used (with details provided in the section to follow on In Situ Testing). The large number of SPT tests performed below the crust with N values equal to zero only confirmed that a thick soft deposit was present and provided little meaningful design information. The SPT testing did collect samples for description and classification, which is very important, although samples for such testing were available from the subsequently collected "undisturbed" tube samples. For this project (thick uniform soft clay deposit) SPT testing was not a cost effective component of the site investigation program. Unfortunately this is a common occurrence in US practice since the SPT is by far the most popular site investigation tool and is often used simply by default regardless of the site conditions.

This case history serves as a preview to the remainder of the paper which presents best practice recommendations for characterizing sites involving soft ground construction on cohesive soils. The presentation nominally follows the chronological sequence of a site characterization program starting with field work, followed by laboratory testing, and concluding with selection of design parameters. As noted in the Introduction, the tools and procedures available for an effective and reliable site investigation are well developed, should be well known, and involve little additional time or cost compared to current practice. Some of these points are illustrated by reference to the poor quality data in Fig. 6.

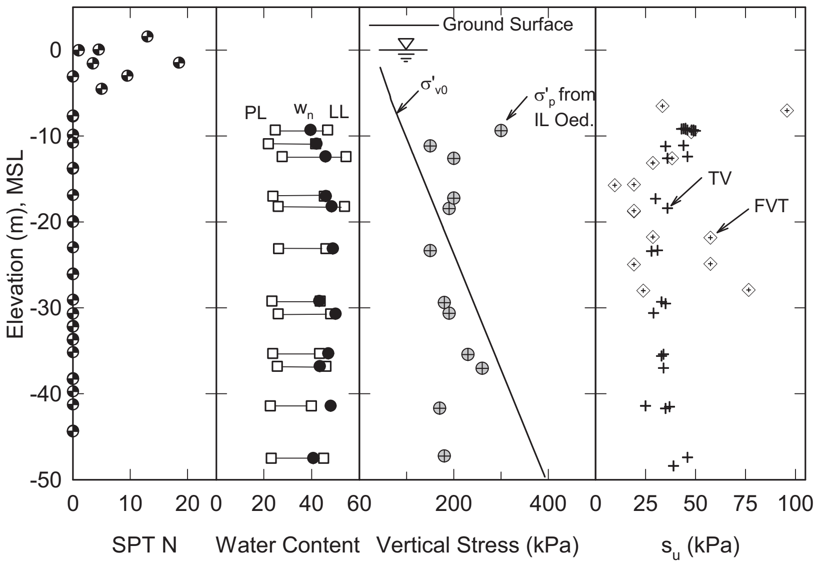


FIG. 6. Elevation vs. SPT blow counts, Atterberg Limits, stress history and measured s_u(FVT) and s_u(Torvane) for site investigation conducted on CL marine clay north of Boston, MA.

DRILLING AND SOIL SAMPLING

Drilling Procedures

Borehole drilling reduces the total vertical stress and subjects the clay at the bottom of the hole to undrained shear in triaxial extension as illustrated by Path 1-2 in Fig. 5. This path passes through a point for which the total vertical stress (σ_v) equals the in situ total horizontal stress (σ_{h0}) which represents the theoretical "perfect sampling effective stress" (σ'_{ps}) defined by Ladd & Lambe (1963), i.e., the effective stress for undrained removal of the in situ shear stress q_0. Because the weight of drilling mud (γ_m) is always less than the weight of overburden, the stress relief will generally cause the effective stress path to go well beyond the q = 0 axis (hence σ'_{ps}) to reach Pt. 2. The worst-case (but not atypical) scenario is failure of the clay in extension even before sampling takes place, i.e., beyond Pt. 2 to Pt. E. However, the magnitude of the stress relief during drilling can be managed by using a weighted drilling mud to keep Pt. 2 in Fig. 5 reasonably close to σ'_{ps} as illustrated in Fig. 7. The two sets of lines represent the conditions for NC clay ranging from low to high plasticity that result in q = 0 (upper lines) and q = s_u(E) [lower lines]. The insert gives the relevant clay properties used with Eq. 2 to calculate a failure state at the bottom of the borehole, i.e., $\sigma_{h0} - \sigma_v = 2s_u(E)$

$$\frac{\gamma_m}{\gamma_w} = 1 - \frac{z_w}{z} + (K_0 - 2s_u(E)/\sigma'_{v0})(\frac{\gamma_b}{\gamma_w} + \frac{z_w}{z}) \quad (2)$$

The weight of mud required to prevent failure before sampling increases significantly with boring depth. If the clay is overconsolidated, Eq. 2 can be used by increasing the values of K_0 and $s_u(E)/\sigma'_{v0}$ by OCR raised to the power 0.5 and 0.8, respectively.

Important aspects of good quality drilling for undisturbed sampling include:

- using a drilling mud with a weight ranging from about $1.2\gamma_w$ to $1.3\gamma_w$ (Fig. 7). Heavy weight additives such as barite ($G_s = 4.2$) can be used.
- maintaining drilling mud at top of borehole, especially when removing drill rods.
- use of larger diameter drill rods to reduce drill string wobble (e.g., NW vs. AW).
- upward or side discharge drill bits.
- decreasing the rate of drilling when approaching the targeted sample depth.

Line	Plasticity	γ_b/γ_w	K_0	$s_u(E)/\sigma'_{v0}$
L	Low	0.85	0.5	0.14
I	Inter.	0.65	0.6	0.18
H	High	0.45	0.7	0.22

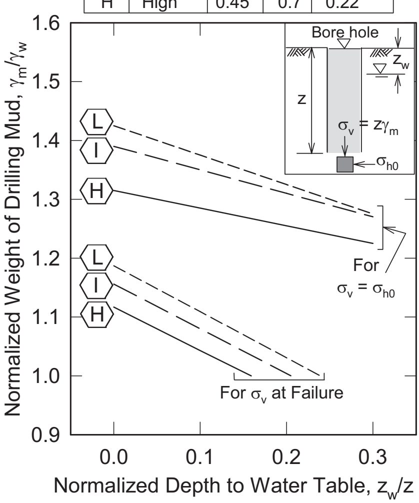


FIG. 7. Effect of drilling mud weight and depth to water table on borehole stability for OCR = 1 clays (modified from Ladd and DeGroot 2003).

The use of hollow stem augers has increased in popularity during the past two decades, especially in the US, for environmental drilling. Most environmental drilling projects require installation of groundwater monitoring wells and do not allow the use of a drilling mud. Hence, many drillers adopted hollow stem augers, which provide a cased borehole and a convenient means for placement of monitoring wells. Unfortunately, this trend has caused some drillers to abandon mud rotary drilling that is far better suited for geotechnical sampling.

Tube Sampling

Hvorslev (1949), Baligh et al. (1987), Clayton et al. (1998) and others have studied the influence of tube sampling techniques and sampler geometry on the quality of soft clay samples. Baligh et al. (1987) showed for tubes with an inside clearance ratio (ICR = $(D_i - D_e)/D_e$, where D_i and D_e are the inside diameters of the interior tube and its cutting edge) greater than zero, that the centerline soil experiences shear in compression ahead of the tube (Path 2 – 3 in Fig. 5), shear in extension as it enters the tube (Path 3 – 4), and compression again as it moves upward within the tube (Path 4 – 5). Theory predicts an axial strain of near one percent at the centerline for a standard 76 mm diameter US thin-walled Shelby tube (ASTM 1587: outside diameter D_0 = 76.2 mm, D_e = 72.1 mm, t = 1.65 mm, ICR = 1%, and Area Ratio AR = $(D_0^2 - D_i^2)/D_i^2)$ = 12%). For sampling of soft clays, sample tubes should have the following characteristics: 1) a minimum diameter (≥ 76 mm), 2) sharp cutting edge of about 5 to 10°, 3) AR < 10%, 4) ICR = 0, 5) made of noncorrosive materials (brass, stainless steel, or at a minimum galvanized or epoxy coated), and 6) be clean and free from dents or burrs. For the most commonly available US Shelby tube having a semi-blunt beveled cutting edge, removing the beveled edge and machining a 5° to 10° cutting angle will reduce the ICR to 0 and the AR to 9%.

Sample Extraction

The strength of the intact clay below the bottom of the tube and the suction created in the void upon tube removal creates additional sample disturbance (Path 5 – 6 in Fig. 5). Therefore a stationary (fixed) piston sampler, rather than a push sampler, should be used for three reasons: it prevents debris from entering the tube; it controls entry of soil during tube advancement; and the suction between the sample and the piston head during extraction helps to retain the sample. Thus piston samplers usually provide far better recovery and sample quality than push samplers. Stationary piston samplers use either actuating rods or hydraulic pressure (e.g., Osterberg) to control the piston head. Although samplers with actuating rods are more cumbersome to use than hydraulic samplers, they do allow for better control of the rate and amount of advance. Specially equipped rigs overcome the actuating rod handling problem, but are not common in the US, thus making hydraulic samplers more popular.

After advancing the tube, wait for about 15 minutes to allow the clay to partially bond to the tube via consolidation and strengthening of the remolded zone around the sample perimeter. Thereafter, slowly rotate the drill string two revolutions to shear the soil below the tube and then slowly withdraw the sampler.

Sample Sealing, Transportation and Storage

Path 6 – 7 in Fig. 5 depicts the decrease in effective stress that occurs due to an increase in water content within the central portion of the tube after sampling. The more disturbed clay around the perimeter consolidates, which causes swelling of the interior portion, and a stiffer, lower water content perimeter zone. Although some organizations extrude samples in the field mainly to reuse the tubes (and also to reduce swelling of interior clay, Hight 2003) the authors prefer to leave samples in the tubes because field extrusion and sample handling are more likely to create shear distortions. Important aspects of sample processing include:

- clean out about 1 to 2 cm from the bottom of the tube and seal the top and bottom with wax or a mechanical seal that uses an O-ring.
- transport tubes upright in boxes that damp shock and vibration as per ASTM D4220 and avoid excessive temperature changes, especially freezing conditions.
- store samples at constant temperature, which should ideally match the in situ temperature to reduce biologic activity.

IN SITU TESTING

This section discusses the piezocone and field vane which are the two most useful in situ tools for characterizing cohesive soils. The CPTU is excellent for soil profiling as it can rapidly provide detailed subsurface information and estimates of s_u and σ'_p via empirical correlations. The FVT generally provides more reliable estimates of s_u and σ'_p for relatively homogenous clay deposits.

CPTU Equipment and Test Procedures

ISO/FDIS 22476-1, ASTM D5778, and Lunne et al. (1997) present detailed guidelines on piezocone equipment and test procedures. The standard piezocone has a 10 cm^2 cross section, 60° tip, and a 150 cm^2 friction sleeve located behind the cone. Although the location of the filter for measurement of pore pressure varies, the recommended position is just behind the tip (called the u_2 position). Since the u_2 pressure acts on the shoulder behind the cone tip, the measured cone resistance (q_c) needs to be adjusted to obtain the corrected tip resistance

$$q_t = q_c + u_2(1\text{-}a) \qquad (3)$$

where a = net area ratio which is approximately equal to the area of the tip load cell (or shaft) divided by the projected area of the cone. Other equipment considerations include maintaining the cone and attached push rods of the same diameter for at least 40 cm behind the tip and equipping the cone with two perpendicular inclinometers to measure cone inclination. CPTUs equipped with geophones or accelerometers are used to measure downhole shear wave velocity (V_s) arrival times to compute the small strain shear modulus (G_{max}).

The standard CPTU test procedure consists of pushing the cone hydraulically at a nominal rate of 2 ± 0.5 cm/s using 1-m length push rods. Some cone trucks are equipped with a double clamping system such that rods can be added during advancement of the cone to enable a continuous push, which is ideal. For dissipation testing, i.e., monitoring changes in u_2 during a pause in penetration to estimate consolidation behavior, the cone should be held stationary via the push clamp to prevent self-weight settlement of the cone which will affect the dissipation data.

Four significant equipment and test procedure issues that commonly lead to poor quality data CPTU data for soft clays are: 1) low cone area ratio, 2) inaccurate friction sleeve data, 3) unreliable sensor measurements, and 4) poor saturation.

Cones should have as high an area ratio as practical, say about 0.8, because cones with lower area ratios (down to 0.5 or less) greatly decrease the accuracy of q_t because of the large pore pressure correction in Eq. 3. The area ratio should be determined using a pressure chamber for direct calibration of the influence of u_2 on q_c and not simply based on measured geometry. Sleeve friction values are notoriously inaccurate (e.g., see Lunne 2010) due to poor equipment design and the fact that the values are very low in soft clays (and even close to zero in soft, sensitive clays). As a result, f_s values are often unreliable although f_s is far less important than q_c and u_2 for interpretation of CPTU data in soft clays.

Quantitative interpretation of piezocone data in soft clays requires very accurate q_c and u_2 measurements, yet it is not uncommon to see very high capacity "universal" cones with insufficient data resolution being used in soft clays. Zero drift due to temperature changes is another sensor problem with some cones being quite sensitive to changes in temperature. It is thus important that zero readings are recorded (both before and after each sounding) at a temperature that is representative of in situ conditions. ISO/FDIS 22476-1 provides Application Classes that specify the equipment and procedures that shall be used for given site conditions (soil stiffness and uniformity of the deposit) and the corresponding minimum accuracy of the recorded values of q_c, u_2 and f_s that must be achieved.

Poor quality u_2 data is a pervasive problem mainly caused by: 1) poor initial saturation of the cone, and 2) desaturation due to cavitation in soil above the water table and in dilating sands below the water table. A key signature of poor saturation in soft clays is a sluggish pore pressure response upon pushing after a rod break. During the pause in a push the u_2 pore pressure will dissipate, once the push starts again the pore pressure should immediately (within several cms of penetration) jump back to the values that were recorded just before the rod break. Water is the ideal saturation fluid, but because of difficulties in maintaining saturation with water (especially with coarse filters), more viscous fluids such as silicone oil and glycerine are used. Different filter materials can be used although polypropylene plastic is the most common in practice. DeJong et al. (2007) studied the performance of various saturation fluids using coarse (15 to 45 μm) polyethylene filters and concluded that a low to moderate viscosity (100 – 1000 cS) silicone oil may be the preferable fluid for testing in saturated soils. For penetration above the water table and if dilating sands are anticipated then higher viscosity silicone oil, possibly in combination with finer filters, should be considered. ASTM D5778 provides guidelines on proper saturation procedures for various saturation fluids and filter materials including the use of high vacuum (< - 90 kPa) deairing for a minimum acceptable duration, and protecting the filters from desaturation during cone assembly and commencement of testing. Additionally it is recommended that filters be replaced after each sounding.

CPTU Data Interpretation

CPTU soundings provide a rapid and detailed approach for developing soil profiles as the data can be used to readily differentiate between soft cohesive and free draining deposits and for detecting the presence of interbedded granular-cohesive soils. Soil behavior type classification charts, such as those developed by Roberston (1990), are widely used and are especially useful for determining if the site consists of relatively uniform deposits versus complex heterogeneous foundation conditions. Multiple soundings across a site are also very useful for detecting spatial variations in soils units and their relative state.

CPTU data can be used to estimate σ'_p (Lunne et al. 2007)

$$\sigma'_p = k(q_t - \sigma_{v0}) = k(q_{net}) \tag{4}$$

where q_{net} is the net penetration resistance and k is typically within the range of 0.25 to 0.35. If a site has large variations in OCR and reliable site specific laboratory σ'_p

data are available, then a SHANSEP type of equation is preferred for developing site specific correlations

$$\sigma'_p = \sigma'_{v0}\left(\frac{q_{net}/\sigma'_{v0}}{S_{CPTU}}\right)^{1/m_{CPTU}} \qquad (5)$$

where values of S_{CPTU} and m_{CPTU} are determined from a regression to the data plotted as $\log(q_{net}/\sigma'_{v0})$ versus $\log(OCR)$.

The undrained shear strength can be estimated from CPTU data using empirical correlations that were developed using strength data from other testing methods via

$$s_u = q_{net}/N_{kt} \qquad (6)$$

where N_{kt} is the cone factor that varies with the selected reference undrained shear strength. For undrained stability analyses, it should equal s_u(ave) such as estimated from corrected field vane data (for homogeneous clays as discussed in the next section). Historically recommended N_{kt} values have spanned a large range [e.g., 10 to 20+ for s_u(ave)] which presumably reflect differences in the nature of the clay (lean and sensitive vs. highly plastic) and its OCR, the reliability of the reference strengths, and the accuracy of q_{net}. Continued research using high quality CPTU and reference s_u data is resulting in a narrowing in the range of recommended N_{kt} values (e.g., Low et al. 2010).

The universal CPTU correlations for σ'_p and s_u should always be used with caution because of the variations in k and N_{kt}. It is always preferable to develop site specific correlations, although this requires rather extensive and reliable reference σ'_p and s_u data. Other CPTU correlations such as for compressibility (CR) and undrained modulus (E_u) are far less reliable and are not recommended. Also note that some engineers simply use q_c rather than q_t in Eqs. 4 and 6 because of concerns about the reliability of u_2 data due to poor saturation (e.g., Duncan et al. 2009 closure to discussion by Ladd 2009). While proper saturation is a persistent concern it makes little sense to ignore the obvious effect of u_2 on q_c and more effort should be placed upon insisting that CPTU operators follow proper saturation procedures.

FVT Equipment and Test Procedures

The key aspects of good field vane equipment for testing soft to medium clays ($s_u \leq$ 50 kPa) include using a four bladed rectangular vane (diameter = 50 to 75 mm and height equal twice the diameter) with thin blades (2 mm), a gear drive system to rotate the vane and measure the torque, and the ability to account for rod friction (usually via a slip coupling). The most cost effective systems use rods that push the vane into the ground. When a borehole is needed for vane testing weighted drilling mud (Fig. 7) should be used to stabilize the borehole bottom and the vane should be inserted at least five times the borehole diameter. Other appropriate test procedures include performing the test about one minute after insertion and rotating the vane at a controlled rate of 6°/min. To measure the remolded undrained shear strength the vane should be rotated a minimum of 10 times after the peak torque is observed.

The highly scattered FVT data in Fig. 6 are from tests performed: in a hollow stem auger borehole without adequate drilling mud; and a device having thick tapered blades, rotated via a torque wrench. These procedures caused excessive insertion disturbance and a very fast rate of shearing that unfortunately commonly occur in practice.

FVT Data Interpretation

The FVT became the most reliable in situ test for estimating s_u(ave) after Bjerrum (1972) directly calibrated the measured s_u(FVT) values with undrained stability analyses of embankment failures. Bjerrum's empirical correction factor (μ) results from several effects, including disturbance during vane insertion, the complex mode of the vane failure, and, most importantly, the fast rate of shearing during the test (Terzaghi et al. 1996). These effects correlate well with plasticity index, as shown in Fig. 8, which includes Bjerrum's original data and other case histories. The majority of the data are for homogeneous clays, but note that the presence of sandy zones and shells ("FRT" in Fig. 8) can cause a large increase in s_u(FVT) and a corresponding reduction in μ. The correction factor does not include three-dimensional end effects, which typically increase the computed FS by 10 ± 5% compared to the plane strain (infinitely long) condition assumed for the embankment failures (Azzouz et al. 1983). Hence the μ factor should be reduced by about 10% for field situations approaching a plane strain mode of failure, or when end effects are being explicitly considered when selecting s_u for stability analysis (Ladd and DeGroot 2003).

FVT data can estimate the preconsolidation stress using the method proposed by Chandler (1988). Jamiolkowski et al. (1985) showed that the variation in $s_u(FVT)/\sigma'_{v0}$ with OCR is well approximated by the SHANSEP equation

$$\frac{s_u(FVT)}{\sigma'_{v0}} = S_{FVT}(OCR)^{m_{fvt}} \qquad (7)$$

where S_{FVT} is the NC undrained strength ratio at OCR = 1. Chandler (1988) used Eq. 7 with m_{fvt} = 0.95 together with Bjerrum's (1972) correlation between $s_u(FVT)/\sigma'_{v0}$ and plasticity index for OCR = 1 clays resulting in the following equation

$$\sigma'_p = \sigma'_{v0}\left(\frac{s_u(FVT)/\sigma'_{v0}}{S_{FVT}}\right)^{1.05} \qquad (8)$$

where the exponent 1.05 = $1/m_{fvt}$ and S_{FVT} is estimated as a function of plasticity index as represented by the solid line in Fig. 9. Also plotted in Fig. 9 are data from ten sites having homogeneous clays (no shells or sand) and PI ≈ 10 to 60%. The agreement with Chandler's relationship for S_{FVT} is good, especially for low PI clays. It is also good for m_{fvt} if the three cemented Canadian clays (for which $m_{fvt} > 1$) are excluded.

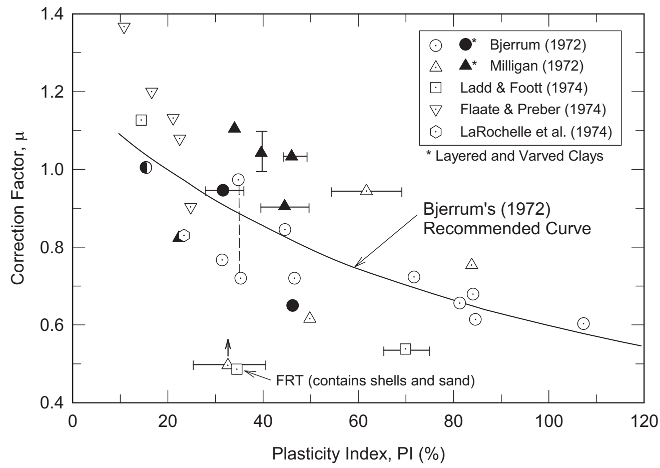


FIG. 8. Field vane correction factor vs. plasticity index derived from embankment failures (after Ladd et al. 1977).

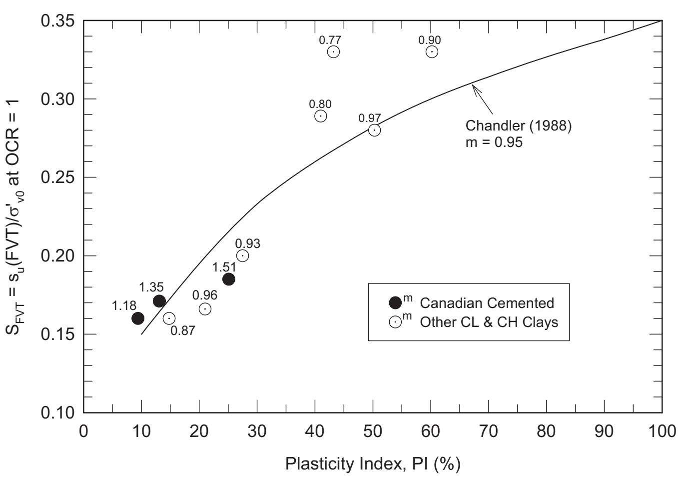


FIG. 9. Field vane undrained strength ratio at OCR = 1 vs. plasticity index for homogeneous clays (no shells or sand) [data points from Lacasse et al. 1978 and Jamiolkowski et al. 1985].

SOIL DESCRIPTION AND CLASSIFICATION

Description of each soil type encountered at a site should always include the Unified Soil Classification System (USCS) designation. Soils are typically first described during on site borings using SPT samples or the bottom end of tube samples and included as part of boring logs. The field and lab soil descriptions should follow ASTM D2488 Description and Identification of Soils (Visual-Manual Procedure) for engineering purposes that uses the USCS described in ASTM 2487 Classification of Soils for Engineering Purposes (USCS). Visual-manual selection of the correct USCS designation is an art that takes experience that used to be part of one's formal geotechnical education, but now seems to be largely ignored. The key visual-manual procedures for cohesive soils, as developed by Casagrande (1948) and described in ASTM D2488, include toughness, dry strength and dilatancy. Precise USCS classification of soils for engineering purposes requires use of D2487 that includes results from grain size distribution and Atterberg Limits tests. CPTU soil behavior type classification charts are very useful in developing preliminary soil profiles, but are imprecise compared to the USCS.

Cohesive soils for Atterberg Limits should be mixed at their natural water content to obtain the Liquidity Index (LI). Atterberg Limits on dried soil are appropriate only to distinguish between CL-CH and OL-OH designations (as per ASTM D2487) since drying can cause very significant reductions in plasticity. Atterberg Limits tests should also be performed on trimmings from all consolidation and consolidated-undrained shear strength tests. The geotechnical report should contain appropriate plots of soil profiles with USCS designation for each soil type, plus depth versus w_n-LL & PL and LI and especially the Plasticity Chart for all cohesive soils. These data provide preliminary estimates of the likely behavior of the foundation soils, and the location of cohesive soils. Casagrande's plasticity chart is very useful for predicting engineering properties via empirical correlations.

SELECTION OF LABORATORY TEST SPECIMENS

Specimens for consolidation and shear tests should avoid soil from within about 1 to 1.5 times the tube diameter at the top and bottom of the tubes due to the greater degree of disturbance that usually occurs at the ends. Specimen locations should ideally be selected using radiography in order to identify representative soil types and soils having minimal disturbance.

Radiography

Unfortunately, radiography of sample tubes is seldom performed in practice because: of the false perception of its high cost (~ $150 per tube in the U.S. Northeast); the lack of local facilities that are set-up to x-ray soil sample tubes; and the lack of appreciation that it provides a highly cost effective tool for planning and executing laboratory test programs.

ASTM D4452 describes the necessary equipment and procedures for x-raying soil samples. The resulting radiographs can identify many features including: 1) variations in soil type (i.e., granular vs. cohesive vs. peat); 2) soil macrofabric, especially the nature (thickness, inclination, distortion, etc.) of any bedding or layering; 3) the presence of inclusions such as stones, shells, sandy zones and root holes; 4) the presence of anomalies such as fissures and shear planes; and 5) the varying degree and nature of sample disturbance. Indications of sample disturbance that are visible in radiographs (see examples provided in ASTM D4452) include bending of horizontal soil layers near the tube perimeter, cracks due to stress relief, gross disturbance caused by the pervasive development of gas bubbles, and voids due to gross sampling disturbance (especially near the ends of a sample). It is significant that many features not noticed during physical inspection/trimming of soil samples would have been readily detected with radiographs.

Radiography provides a nondestructive means for examining samples and identifying locations of the best quality material of each representative soil obtained from the site. This allows the tubes to be cut, as explained below, at the best locations for each consolidation and strength test and greatly reduces the likelihood of running costly tests on poor quality or non-representative soil that produce misleading data.

Sample Extrusion and Specimen Preparation

The bond that develops at the soil-tube interface can cause very serious disturbance during sample extrusion. The bonding can be especially significant with prolonged storage times and with lower quality tube materials such as regular steel or galvanized steel. It is thus essential to break the bond at the soil-tube interface before extruding samples. After selecting the test specimen location, the tube should be cut adjacent to the desired location as shown in Fig. 10 using a horizontal band saw, or by hand (e.g., hack saw), or with a four point pipe cutter together with a set of blocks to minimize tube distortion during cutting. A thin wire is inserted into the soil/tube interface and rotated several times around the perimeter to destroy the bonding as further described by Ladd & DeGroot (2003). For example, portions of fixed piston tubes of Boston Blue Clay (BBC) for the Central Artery/Third Harbor Tunnel project in Boston were cut in short lengths of only several centimeters for a series of conventional oedometer tests on the deep (> 30 m), low OCR clay. Extrusion caused cracks to appear on the upper surface, and the resultant destructuring produced compression curves having OCRs less than one, whereas subsequent tests on debonded specimens gave OCR ≈ 1.1 to 1.2 (Fig. 4.6 in Ladd and DeGroot 2003).

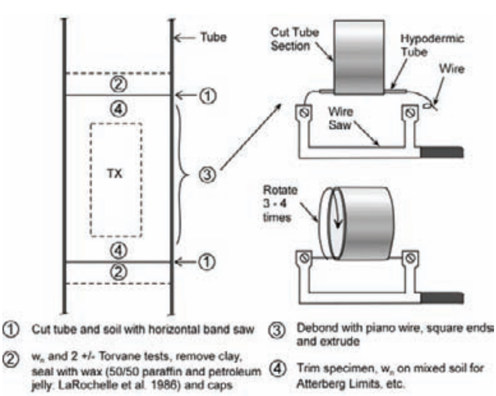


FIG. 10. MIT procedure for obtaining test specimen from tube sample (Germaine 2003).

Once soil has been extracted from the cut tube section, trimming should be conducted using sharp cutting tools, rings and wire saws in a humid room. The disturbed sample periphery should be removed during trimming. Laboratory sub-sampling using a miniature tube creates significant additional disturbance and should not be used.

CONSOLIDATION TESTING – PROCEDURES, INTERPRETATION AND PRESENTATION

The background section on Fundamentals of Clay Behavior reviewed clay consolidation behavior and this section describes laboratory methods and interpretation techniques for determining consolidation compression curves, preconsolidation stress, and flow characteristics.

Test Equipment and Procedures

The one-dimensional consolidation test is typically performed using an oedometer cell with application of incremental loads (IL) as this technique is widely available and relatively easy to perform. However, the constant rate of strain (CRS) test (Wissa et al. 1971) has significant advantages over the IL test as it produces continuous measurement of deformation, vertical load, and pore pressure for direct calculation of the stress-strain curve and coefficients of permeability and consolidation. Computer-controlled load frames allow for automation of IL and CRS testing (with the additional use of a flow pump for back pressure saturation during CRS testing) which generally results in improved data quality and test efficiency. General requirements for the IL test are covered by ASTM D2435 and for the CRS test by ASTM D4186. Computer controlled CK_0 stress path triaxial tests for specimens consolidated beyond σ'_p also give reliable data for determining the compression curve (i.e., σ'_p, CR). Furthermore, the automated K_0 consolidation measures K_0 for NC clay and provides sufficient data for estimating the in situ K_0 using the method of Mesri and Hayat (1993).

IL oedometer and CRS tests are usually conducted by first loading the specimen beyond the preconsolidation stress (σ'_p) to a maximum stress sufficient to define the virgin compression line (at least 2 to 3 points beyond σ'_p for IL tests), often followed by several unloading increments. In some cases an unload-reload cycle is used to better define the OC behavior (i.e., RR), although for most soft ground construction problems this is not an important design issue. The authors prefer using moist filter stones (as opposed to dry, e.g., Sandbækken et al. 1986) and adding water after application of the seating load. The specimen should be monitored to check for swelling and additional load applied, as necessary, to prevent swelling.

Traditional IL tests use a load increment ratio (LIR) of one and 24 hour load increments (t_c). For many soft clays, particularly those with S-shaped compression curves, doubling the load is too high to properly define the compression curve. Furthermore, 24 hour increments include secondary compression, which result in lower estimates of σ'_p by about 15 ± 5%. Better definition of the compression curve can be achieved using a reduced LIR (say 0.6 ± 0.1) for σ'_v increments bracketing σ'_p and a reduced t_c that approximates end of primary (EOP) consolidation for NC clay.

In the CRS test, the drainage is one-way and the base excess pore pressure (u_e) is measured with a pressure transducer. The measured ε_v-σ_v-u_e data are used with a linear CRS theory (e.g., Wissa et al. 1971) to compute continuous values of ε_v, e, σ'_v, k_v, and c_v. The strain rate for CRS tests needs to be selected such that the normalized base excess pore pressure (u_e/σ_v) is within acceptable limits. Too slow a rate results in $u_e \approx 0$ that prevents calculation of k_v and c_v and that may include secondary compression strains, while too fast a rate results in high excess pore pressures causing significant variations of void ratio and σ'_v. The selected strain rate should be such that $u_e/\sigma_v \approx 10 \pm 5\%$ in the NC range (e.g., Mesri and Feng 1992), although the resulting σ'_p will be about 10% greater than the EOP σ'_p due to strain rate effects (Mesri et al. 1994). ASTM D4186 suggests values for the strain rate as 10%/hr for MH soil, 1%/hr for CL soil, and 0.1%/hr for CH soil and that an appropriate adjustment should be made if needed during testing to keep u_e/σ_v within 3 to 15% in the NC range. At strain rates around 1%/hr, a CRS test with back pressure saturation takes about 3 to 4 days (without an unload-reload cycle), which is much faster than the traditional one day IL test. Test durations comparable to the CRS test are feasible for IL testing if load increments are applied soon after primary consolidation, which can be achieved using an automated computer controlled load frame (or frequent manual application), but the next paragraph describes a problem with having variable t_c increments.

Compression Curves

For consistent definition of the EOP compression curve generated from IL testing, data *for all increments* should be plotted at one constant consolidation time (t_c) using a value based on the maximum t_p estimated from increments in the NC region, which typically ranges from 10 to 100 min. For example, Fig. 11 compares IL (with LIR = 1) and CRS data (with u_e/σ_v < 10%) for tests conducted on two CL clays. The IL EOP data were determined using a constant t_c taken from t_p in the NC region as shown for selected increments of the BBC test in Fig. 12. These results highlight the difficultly with interpreting time curves for an increment near σ'_p. The 100 kPa (with greatly expanded ε_v scale) and 400 kPa time curves have distinct breaks that are easily interpreted using Casagrande's log time method to estimate t_p, whereas the 200 kPa increment almost coincides with σ'_p and contains a significant amount of secondary compression during the 24 hr loading period. Neither the log time method nor the root time method can be used to estimate the EOP for this increment.

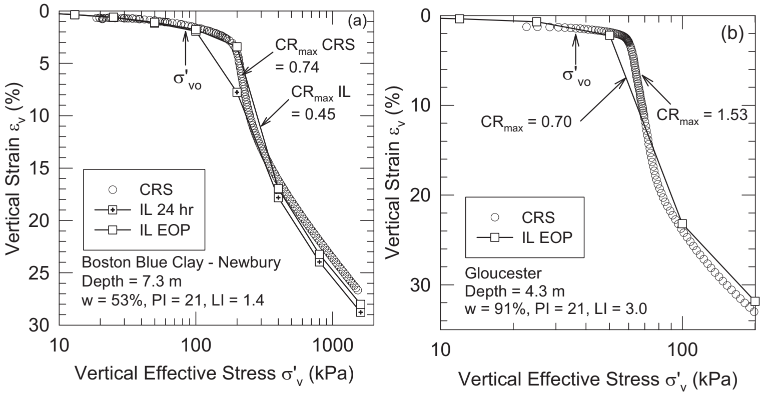


FIG. 11. Comparison of compression curves from CRS and IL tests on: (a) Boston Blue Clay, Newbury, MA; (b) Gloucester Clay, Ottawa.

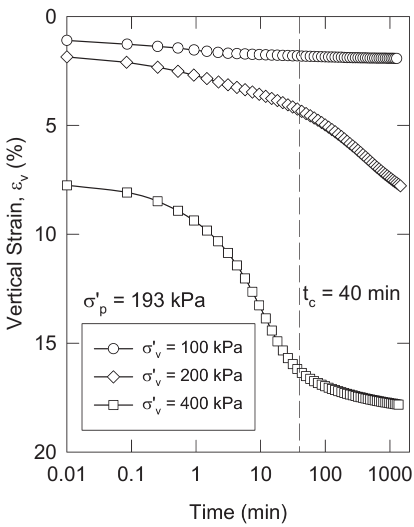


FIG. 12. Vertical strain – time curves for increments spanning σ'_p from the IL Test on BBC plotted in Fig. 11a.

In comparing the BBC compression curves, Fig. 11(a) shows that the IL 24 hr data gives a value of σ'_p that is too low whereas the EOP IL data, using t_c = 40 min. throughout the test (Fig. 12), produced a more realistic compression curve (although the good agreement was fortuitous in that the 200 kPa increment just happened to be close to σ'_p). The Fig. 11 IL data also demonstrate the importance of having at least 2 points on the virgin compression line to see if the compression curve is S-shaped and to help select CR_{max} and estimate σ'_p. However, for both soils, IL CR_{max} values are lower than that from the CRS tests.

Sample Quality

While no definitive method exists for determining sample quality, valuable information can be obtained from making use of both qualitative and quantitative methods. Qualitative (visual) assessment of sample quality is best made by examination of sample X-rays previously discussed. Ladd and DeGroot (2003) describe various quantitative approaches for assessing relative degrees of sample quality, including use of strength index test data, measurement of the sample effective stress (σ'_s), CR_{max} vs OCR, and the volumetric strain (ε_{vol}) upon reconsolidation to the in situ effective stress state (Note: this is equal to ε_{v0} at σ'_{v0} for 1-D consolidation). The volumetric strain method is the most well established procedure and should be reported for all consolidated test specimens (i.e., IL, CRS, and also the consolidation phase of CK_0U or CAU triaxial and DSS tests). Andresen and Kolstad (1979) proposed that ε_{vol} increases with increasing sample disturbance, which Terzaghi et al. (1996) adopted and coined the term Specimen Quality Designation (SQD) with sample quality ranging from A (best) to E (worst). They also suggest that reliable lab data requires samples with SQD of B (ε_{vol} < 2%) or better for clays with OCR < 3 – 5. More recently, Lunne et al. (2006) developed the normalized sample quality parameter $\Delta e/e_0$, which is computed as

$$\Delta e/e_0 = \varepsilon_{vol}(1 + e_0)/e_0 \quad (9)$$

where Δe = change in void ratio during reconsolidation to in situ stresses and e_0 = initial void ratio. By incorporating e_0 the method considers an equal amount of recompression strain as being more serious for clays with a lower e_0. The method also applies stricter criteria for higher OCR samples, as listed in Table 2, and the authors suggest that results from tests on samples with Quality Rating (QR) = 1 can be used as they are, results for QR = 2 samples must be carefully evaluated, and results for QR 3 & 4 samples are unreliable. While this quantitative method of assessing sample quality is not definitive, it provides valuable information that should be reported for all tests (although rarely done in practice). As an illustration, Fig. 13 applies the $\Delta e/e_0$ method to the IL results presented in Fig. 6. The results show that the deep tube samples had Quality Ratings well above 2, which explains why the lab tests gave values of σ'_p and s_u(TV) that are far too low, as discussed later. Note: Figs. 4.4 – 4.7 of Ladd and DeGroot (2003) illustrate application of strength index, ε_{vol} and CR_{max} data to assess sample quality.

Table 2 Evaluation of sample quality for low to medium OCR clays using Lunne et al. (2006)

OCR	$\Delta e/e_0$ at in situ stresses for Quality Ratings 1 to 4			
	1 (very good to excellent, VG/E)	2 (fair to good, F/G)	3 (poor, P)	4 (very poor, VP)
1 to 2	< 0.04	0.04 – 0.07	0.07 – 0.14	> 0.14
2 to 4	< 0.03	0.03 – 0.05	0.05 – 0.10	> 0.10

Preconsolidation Stress

All methods for estimating σ'_p require test data from samples of reasonable quality. The compression curves used for interpretation should either be from CRS tests having an appropriate strain rate (with perhaps a 10% reduction in σ'_p) or by plotted at a constant t_c from IL tests with appropriate LIRs and a maximum load sufficient to prove that CR is either constant or decreasing. Casagrande's method is the simplest and most widely used technique for estimating σ'_p, but it can be quite subjective and difficult to apply with rounded compression curves. The strain energy method of Becker et al. (1987) uses work per unit volume as the criterion for estimating σ'_p from a plot of strain energy versus σ'_v in linear scales. The method is easy to use and typically gives more reliable and consistent estimates of σ'_p compared to Casagrande, especially for stiffer clays with rounded compression curves. Note that the method uses the natural strain to compute the strain energy and that the *maximum slope* (not the average) of the strain energy vs. σ'_v plot in the NC range should be used to estimate σ'_p as illustrated in Fig. 6.4 of Ladd and DeGroot (2003).

Available CPTU and FVT data should also be used for estimates of σ'_p. The collective laboratory and in situ test results should be evaluated in the context of site geology to select final stress history profiles. The case history presented in Figs. 6 and 13 illustrate this approach. After the original site investigation (data in Fig. 6), a CPTU sounding was performed, leading to estimates of σ'_p using Eq. 4 with k = 0.3. The resulting CPTU σ'_p profile indicates an overconsolidated upper drying crust and a low OCR clay below El. -25 m. The selected σ'_p profile within the crust considered both the CPTU and lab data, whereas OCR = 1 was somewhat conservatively selected for the deeper clay after confirming σ'_{v0}.

Flow Characteristics

Computation of c_v for IL tests should be based on the average specimen drainage height for each increment (i.e., the H_d at t_{50} as per ASTM D2435) and not simply the initial H_d as suggested by Duncan (1993). The secondary compression that occurs in conventional 24 hr IL tests causes specimens to initially behave as overconsolidated soil with high c_v during the next load increment. Hence, derived values of c_v depend upon the graphical construction method selected to estimate c_v. For example, for one day tests with LIR = 1, $c_v(\sqrt{t}) \approx (2 \pm 0.5)$ times $c_v(\log t)$. The problem gets worse at lower LIRs and near σ'_p (see Fig. 12), such that c_v may become indeterminate. CRS testing not only avoids this problem, but also produces continuous values of k_v and c_v versus ε_v and σ'_v.

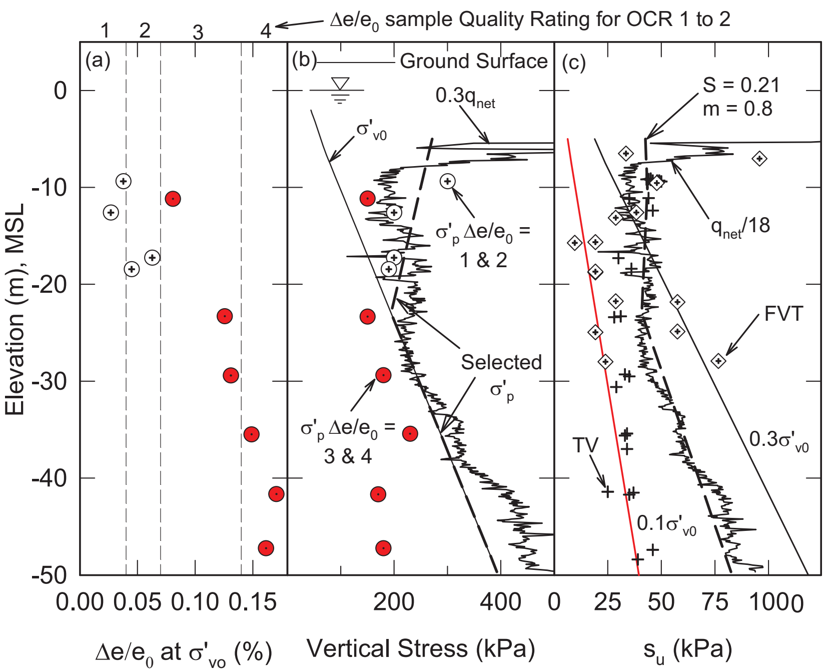


FIG. 13. Evaluation of sample quality, stress history, and undrained shear strength profiles for the BBC data presented in Fig. 6.

Secondary Compression

The rate of secondary compression (C_α) from IL testing should be computed using H_0 (like that for CR). One should not use the height at the end of primary (H_p) as recommended by Johnson (1970), because that gives C_α values that are progressively too high in the NC range. The C_α/CR ratios for all load increments should be approximately constant (independent of σ'_v and OCR) and fall within the range of 0.045 +/- 0.015 for CL, CH and OH soils as per Mesri and Castro (1987). [Note that this "CR" = $\Delta\varepsilon_v/\Delta\log\sigma'_v$ for both OC and NC soil].

Reporting Test Results

Compression curves should be plotted using strain, rather than void ratio, as this is better suited for practice in order to standardize scales and to obtain RR and CR values used to compute ρ_{cf}. Tabulated data for individual IL tests should include load and time increments, ε_v and e, strain energy, t_{50} and t_{90}, c_v, C_α and C_α/CR. Tabulated summary results for both CRS and IL tests should include σ'_{v0}, ε_{v0} and $\Delta e/e_0$ at σ'_{v0}, CR and estimates of σ'_p using both Casagrande and strain energy methods. For IL tests recommended plots include ε_v vs. log σ'_v (at a constant $t_c \approx$ the NC t_p) showing σ'_{v0} and σ'_p, and strain energy vs. σ'_v, plus at least representative $\sqrt{t}$ and logt curves for increments exceeding σ'_p. CRS test plots should include ε_v, u_e/σ_v, c_v versus log σ'_v, ε_v versus logk_v and strain energy.

STRENGTH TESTING – PROCEDURES, INTERPRETATION AND PRESENTATION

Laboratory testing for measuring undrained shear behavior of clays for soft ground construction stability problems should rely on a consolidated-undrained (CU) shear test program that accounts for anisotropy, strain rate effects and sample quality. This section first reviews the common, but poor practice, of excessive UU and CIUC testing, followed by recommendations for adopting a more rational testing program and interpretation of test results.

UUC and CIUC Triaxial Testing

Why are unconsolidated-undrained triaxial compression (UUC) tests (ASTM D2850) so widely used in practice when the measured s_u(UUC) will only equal the s_u(ave) appropriate for undrained stability analyses by a fortuitous cancellation of three errors? The test uses a fast strain rate of 60%/hr compared to field loadings, causes failure in triaxial compression that is too high because of effects of anisotropy, and is greatly affected by varying degrees of sample disturbance. That is, estimating s_u(ave) from UUC data inherently relies on compensating errors (i.e., the decrease in s_u from sample disturbance has to offset the increase in s_u from rate effects and only considering TC mode of shear) that cannot be controlled and seldom balance each other. UUC strengths from poor quality samples can be 25 to 50% too low, while those on high quality samples can yield s_u values that are 25 to 50% too high (Germaine and Ladd 1988). One should not be surprised that the data not only exhibit significant scatter but also give misleading trends in $s_u(UUC)/\sigma'_{v0}$ (especially with increasing depth). Therefore UUC tests are a waste of time and money compared to other less costly strength index tests such as the Torvane, lab vane and fall cone.

Isotropically consolidated-undrained triaxial compression (CIUC) tests (ASTM

D4767) are often used to estimate the initial strength of soft soils (UU Case, Table 1) and how it increases with consolidation (CU Case, Table 1). However, estimates of the initial s_u(ave) based on the measured strength from CIUC tests reconsolidated to the in situ σ'_{v0} for low OCR clays are almost always unsafe because the isotropic consolidation leads to a water content that is too low and shearing in triaxial compression ignores anisotropy. Similar problems occur when CIUC tests are used to measure q_f/σ'_c for CU stability problems because these values are much higher (unsafe) than the appropriate $s_u(\text{ave})/\sigma'_{vc}$. Thus CU strength testing should use anisotropic (CAU) or K_0 (CK_0U) consolidation and consider anisotropy as next described. Also note that Ladd (1991) does not recommend using the U.S. Army Corps of Engineers "QRS" methodology for calculation of design strengths during staged construction from CIUC test data as it also depends on compensating errors.

CU Test Equipment and Procedures

CU test equipment that is realistically available and suitable for practice include the triaxial compression (TC), triaxial extension (TE), and direct simple shear (DSS) devices. They can use K_0 or anisotropic consolidation, perform undrained shear at reasonable rates, and account for undrained strength anisotropy. Baldi et al. (1988), Germaine and Ladd (1988), and Lacasse and Berre (1988) give recommendations for triaxial equipment and test procedures while Bjerrum and Landva (1966), DeGroot et al. (1992) and ASTM D6528 do likewise for the DSS. Key components of good quality triaxial testing include:

- cells equipped with internal load cells to eliminate piston friction.
- proper alignment of the specimen and piston; this is especially important for conducting TE tests.
- back pressure saturation, ideally performed at the sampling effective stress.
- K_0 consolidation for geostatic stress conditions. Anisotropic consolidation (CAU) to an *estimated* K_0 value should be performed when $\sigma'_{vc} < \sigma'_p$ because 1-D laboratory reconsolidation produces K_0 values much less than in situ.
- undrained shear at a strain rate of about 0.5 to 1.0% per hour.

Computer automated stress path triaxial equipment make triaxial tests much simpler and more efficient to perform than manual methods. This is especially true for tests with K_0 consolidation on NC soil because manual consolidation requires the application of many small increments of vertical and radial stress in order to avoid excessive undrained shear deformations and to check that $K_c = \sigma'_{hc}/\sigma'_{vc}$ is close to K_0 (i.e., equal axial and volumetric strains), all of which is done automatically in a computer controlled system. For example, MIT uses an axial strain rate of about 0.1%/hr for NC consolidation with automated adjustments of the cell pressure (via a flow pump) to maintain a K_0 stress state (i.e., the computer control system targets keeping $\varepsilon_a = \varepsilon_{vol}$).

The direct simple shear (DSS) device simulates shearing along a horizontal plane after K_0 consolidation but with an indeterminate and non-uniform state of stress. However, the horizontal shear stress at the peak strength (τ_{hmax}) probably lies between $q_f = 0.5(\sigma_1 - \sigma_3)_f$ and $\tau_{ff} = q_f\cos\phi'$, the inclination of σ_{1f} probably equals $\delta \approx 45 \pm 15°$, and τ_{hmax} is thought to give reasonable estimates of s_u(ave) [except for varved clays].

Furthermore, the test requires less soil and less time and effort than CK_0U triaxial tests. DSS specimens should be trimmed into a circular wire-reinforced rubber membrane (or stacked rings) and loaded either incrementally or continuously to produce a K_0 compression curve that is the same as those measured in other 1-D consolidation tests. Undrained shear (actually constant volume) usually uses a shear strain rate of ≈ 5%/hour. Tests with $\sigma'_{vc} < \sigma'_p$ typically require the use of stones with embedded pins or with a rough waffle type surface to prevent slippage, which creates an unknown degree of disturbance and height of shearing. Another issue with $\sigma'_{vc} < \sigma'_p$ tests is the low horizontal stress developed during recompression such that the preshear K is lower than the in situ K_0 (Dyvik et al. 1985), especially with high OCR tests. Specimens should therefore first be loaded up to a higher stress level (NGI targets $0.8\sigma'_p$) and unloaded back to σ'_{vo} to develop additional horizontal stress, which can be tricky if σ'_p is uncertain.

Reconsolidation Techniques

The Recompression and SHANSEP strength testing techniques were independently developed to address the important soil behavior issues of anisotropy, strain rate, and sample disturbance. Both use CK_0U tests with shearing in different modes of failure at appropriate strain rates to account for anisotropy and strain rate effects. But the two methods have a major difference in how they deal with sample disturbance.

In the Recompression method, Bjerrum (1973) recognized the unreliable nature of the standard UU test and proposed using CU tests that are anisotropically reconsolidated to the in situ state of stress (σ'_{vo}, σ'_{ho}), as shown by point 3 in Fig. 14. This procedure assumes that the reduction in water content during reconsolidation to σ'_{vo} is sufficiently small to compensate any destructuring during sampling, so that the measured s_u data is representative of in situ clay for UU stability cases. Berre and Bjerrum (1973) recommended that the volumetric strain during recompression should be less than 1.5 to 4 percent. Note that Recompression tests can use a range of σ'_{vc} values (but $< \sigma'_p$), such as was done for the highly structured James Bay clay in Fig. 2(b), in order to compute s_u profiles via the SHANSEP equation.

The SHANSEP method (Ladd and Foott 1974, Ladd 1991) is based on the experimental observation that the undrained stress-strain-strength behavior of most "ordinary" clays, for a given mode of shear, is controlled by the stress history of the test specimen. The method assumes that these clays exhibit normalized behavior and uses mechanical overconsolidation to represent all preconsolidation mechanisms. The procedure explicitly requires knowledge of stress history profiles for the clay layer and was developed to obtain s_u profiles for both the UU and CU stability cases. Test specimens are K_0 consolidated to stress levels greater than σ'_p to measure the normally consolidated behavior (Points A and B in Fig. 14), which now can be readily performed using computer controlled systems that also produce compression curves for estimating σ'_p. At MIT, K_0 consolidation uses strain controlled loading (at 0.1%/hr as noted above) until reaching about 10% axial strain which is deemed to be a NC stress state. Specimens are also unloaded to varying OCRs (Points C and D) to measure OC behavior. Plots of log s_u/σ'_{vc} vs. log OCR are then used (Fig. 2(a)) to obtain the SHANSEP values of S and m to use with Eq. 1. The values of S will always have some inherent variability, in addition to a general trend of decreasing S with increasing σ'_{vc}/σ'_p (especially for structured clays having S-shaped compression curves). The DSS data summarized by DeGroot et al. (1992) for a variety of cohesive soils (i.e., ML to CH and OH) show that the coefficient of variation (COV[S]) was 6.5 ± 4% from DSS programs having 4 or more tests on 11 different soils.

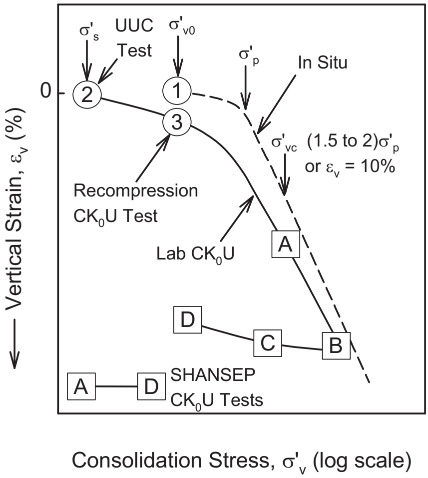


FIG. 14. Recompression and SHANSEP consolidation procedure for laboratory CK_0U testing (slightly modified from Ladd 1991).

For clay deposits that are or will become truly normally consolidated, the SHANSEP procedure should be used because reconsolidation to $\sigma'_{v0} = \sigma'_p$ for NC soil will result in high strains and low ε_v-logσ'_v slopes compared to the in situ soil (Fig. 14), and thus lead to unsafe design strengths. SHANSEP CK_0U tests for predicting the behavior of in situ OC clay should use $t_c \approx 1$ day so as to allow secondary compression to help "restore" some of the clay structure. But for in situ NC soil, the tests should use $t_c \approx t_p$ to minimize secondary compression (MIT uses $t_c \approx 2$ hr).

The SHANSEP reconsolidation technique of consolidating test specimens beyond the in situ σ'_p will destructure all OC clays to varying degrees. Furthermore, the mechanical overconsolidation used with SHANSEP will never exactly reproduce the undrained stress-strain behavior of natural overconsolidated deposits, especially for highly structured clays where mechanical consolidation is not the primary mechanism causing σ'_p. With highly structured clays, such as the Champlain clays, SHANSEP values of S are much too low, as shown by the solid points in Fig. 2(b). However, even though the SHANSEP technique gives imperfect results (except when the in situ OCR = 1 as noted above), it does offer several important practical advantages: 1) it forces the user to establish the initial stress history of the soil, which is needed to understand the deposit and is required for settlement analyses and stage construction; 2) the error in S should always be on the safe side and is probably < 5 to 10% for most cohesive soils of low to moderate sensitivity; 3) errors in m are not significant for low OCR deposits (thus can be estimated); and 4) with the advent of computer automation, SHANSEP tests yield continuous compression curves that provide values of σ'_p, plus those for CR, and K_0 vs. σ'_v (for TC/TE tests). In any case, the SHANSEP equation can always be used to calculate the range in s_u profiles for likely variations in stress history and SHANSEP values of S and m.

For problems requiring reliable predictions of deformations caused by undrained shear in OC deposits, the Recompression technique is much preferable (assuming good quality samples) because SHANSEP tests will always underestimate the in situ undrained stiffness. However, Recompression tests cannot, by definition, measure values of σ'_p and give only depth specific values of s_u. Thus a separate program of consolidation tests (to get stress history data) is needed to check the reasonableness of the measured s_u/σ'_{v0} ratios and for settlement and CU stability analyses.

Levels of Laboratory Strength Testing

Ladd (1991) proposed three levels of sophistication for obtaining strength data from laboratory testing for undrained stability analyses. Level A is the most sophisticated and involves performing CK_0U TC/TE and DSS tests to develop anisotropic strength parameters, $s_u(\alpha)$. Level B uses either DSS tests or a combination of CK_0U TC and TE tests to obtain isotropic strength values, s_u(ave). As noted above, DSS testing has the advantage of requiring less effort and soil and s_u(DSS) generally provides a good estimate of s_u(ave). However, Level B CK_0U TC/TE testing does have the advantage of providing s_u anisotropy information that is useful for CU Case stability analyses. But triaxial testing alone is not suitable for varved deposits because the lowest strength occurs for shear parallel to the varves, which requires DSS testing. For both Level A and B, the test program should focus on measuring S, with estimated values of m unless the deposit contains thick high OCR layers. Level C is the least sophisticated and uses the SHANSEP equation to compute s_u(ave) with S and m values derived from empirical correlations as summarized in Table 3. While Level C does not involve laboratory strength testing, stress history data are required. These can be obtained from laboratory consolidation testing or empirically from FVT or CPTU soundings (preferably with site specific correlations for the latter). Table 4 illustrates selection of appropriate levels of strength testing (A, B, or C) in order to use the SHANSEP equation in undrained stability analyses of increasing complexity.

Table 3 Level C Values of S and m for Estimating s_u(ave) via SHANSEP Equation (slightly modified from Section 5.3 of Ladd 1991)

Soil Description	S	m^a	Remarks
1. Sensitive cemented marine clays (PI < 30%, LI > 1.5)	0.20 Nominal SD = 0.015	1.00	Champlain clays of Canada
2. Homogeneous CL and CH sedimentary clays of low to moderate sensitivity (PI = 20 to 80%)	S = 0.20 + 0.05(PI/100) or Simply 0.22	$0.88(1 - C_s/C_c)$ ± 0.06 SD or simply 0.8	No shells or sand lenses-layers
3. Northeastern US varved clays	0.16	0.75	Assumes that DSS mode predominates
4. Sedimentary deposits of silts &organic soils (AL plot below A-line) and clays with shells	0.25 Nominal SD = 0.05	$0.88(1 - C_s/C_c)$ ± 0.06 SD or simply 0.8	Excludes peat

[a]$m = 0.88(1 - C_s/C_c)$ based on analysis of CK_0UDSS data on 13 soils with max. OCR = 5 - 10

Table 4 Levels of Sophistication for Evaluating Undrained Stability (from Ladd and DeGroot 2003)

Ex #	Stability Case	Method of Analysis	Strength Input	Strength Testing	Stress History	Typical Design FS*
1	UU	Circular Arc (Isotropic s_u)	s_u(ave) vs. z	FVT (no shells) or Estimated S and m (Level C)	Desirable Required	≥ 1.5
2	CU	Circular Arc (Isotropic s_u)	s_u(ave) vs. z for each zone	CK_0U TC & TE or CK_0UDSS (Level B)	Essential	1.3 – 1.5
3	CU	Non-circular Surface (Anisotropic s_u)	$s_u(\alpha)$ vs. z for each zone	CK_0U TC, TE & DSS (Level A)	Essential	1.25 – 1.35

*Design FS should be a function of the uncertainty in the "best estimate" FS, the consequences of failure, and the level of monitoring during construction. For major projects, a reliability analysis is recommended to help select a suitable FS (e.g., Christian et al. 1994).

SELECTION OF DESIGN PARAMETERS

The ability to select reliable soil parameters for settlement and stability analyses depends significantly on the scope and quality of the site characterization program. This section provides some general guidance on selection of design parameters, with greater detail available in Ladd (1991) and Ladd and DeGroot (2003). While often not part of the work scope of site characterization programs, knowledge of design requirements and the parameter selection process is essential for developing and executing an appropriate site characterization program and, most importantly, in convincing the client to adopt your recommendations.

Settlement Analyses

Estimating Final Consolidation Settlement

The magnitude of the final settlement caused by primary consolidation can be calculated using

$$\rho_{cf} = \Sigma[H_0 \cdot \varepsilon_{cf}] = \Sigma[H_0(RR\log\sigma'_p/\sigma'_{v0} + CR\log\sigma'_{vf}/\sigma'_p)] \quad (10)$$

where H_0 is the initial thickness of each layer (Note: σ'_{vf} replaces σ'_p if only recompression and σ'_{v0} replaces σ'_p if only virgin compression within a given layer). The initial (σ'_{v0} and σ'_p) and final (σ'_{vf}) stress histories, plus CR are most important when loading low-OCR clays. Selection of the initial stress history starts with a detailed evaluation of laboratory CRS and EOP IL consolidation tests to check correlations for estimating σ'_p from FVT and CPTU tests, and then using the in situ test data, along with geology of the site, to access spatial variations in σ'_p and development of stress history profiles for the settlement analyses. Consider reducing CRS σ'_p values by about 10% to better match EOP values for high quality samples. CR is the most important compressibility parameter. With structured clays, CR_{max} should be used for analysis, unless CR becomes significantly lower at the final σ'_{vf}. If CR_{max} is scattered, both due to natural variability and degrees of sample disturbance, assess changes in CR_{max} as function of ε_{vol} (or $\Delta e/e_0$), σ'_p (or OCR) and index properties (w_n, LL, LI, etc.) to help select appropriate design values.

Highly plastic (CH) and organic (OH) foundation soils with low factors of safety and slow rates of consolidation (large t_p) often have low values of undrained modulus that can lead to significant and continuing settlements (ρ_i) caused by undrained shear deformations and creep (Foott and Ladd 1981). For such conditions, the typical practice of assuming that the total settlement at the end of consolidation equals ρ_{cf} can be very unsafe and one should add estimates of ρ_i to ρ_{cf}.

The importance of secondary compression (or drained creep) during primary consolidation is controversial with two opposing theories (Ladd et al. 1977). Hypothesis A (Mesri et al. 1994) assumes that significant secondary compression occurs only after the end-of-primary (EOP) consolidation, so that the in situ σ'_p and ε_{cf} are independent of t_p (layer thickness/drainage distance). In contrast, Hypothesis B (Leroueil 1994) assumes that secondary compression occurs throughout primary consolidation, which basically means that σ'_p decreases and ε_{cf} increases with increasing t_p. There is little difference between the hypotheses for interpretation of standard laboratory IL consolidation tests. But very significant practical differences occur when predicting the final settlement at EOP in the field, especially with $\sigma'_{vf}/\sigma'_p < 2 - 3$ for thick clay layers having large values of t_p. While the validity of these opposing views are still controversial, settlement calculations using either hypothesis require essentially the same information from the site characterization program (i.e., σ'_p, CR, c_v, C_α, etc.).

Time Rate of Consolidation Settlement

Design of projects involving preloading (with or without surcharging) and staged construction require predictions of the rate of consolidation. These involve estimates of c_v for vertical drainage and c_h for horizontal drainage if vertical drains are installed to increase the rate of consolidation (where $c_h = r_k c_v$, and $r_k = k_h/k_v$). In both cases the selected values should focus on normally consolidated (NC) clay, even when using a program that can vary c_v and c_h as a function of σ'_{vc}. Selected c_v values should be compared with the useful empirical correlation in Navfac DM-7.1 (1982) between c_v and Liquid Limit. The value of r_k for marine clays is essentially one and for lacustrine varved clays may approach 5 to 10 (e.g., Tavenas et al. 1983, Mesri and Feng 1994, DeGroot and Lutenegger 2003). However, installation of wick drains (PVD) will cause soil disturbance that can reduce the effective c_h to values near c_v at close drain spacing (even with highly layered deposits), as documented by Saye (2003). Saye (2003) recommends that wick drains be spaced at no less than 1.5 m and that the size of the installation mandrel be as small as possible.

Estimating Secondary Compression

Post primary settlements due to secondary compression generally become important only with rapid rates of primary consolidation, as occurs within zones having vertical drains. For such situations, designs often use surcharging to produce overconsolidated soil under the final stresses, which reduces the rate of secondary compression. Selection of C_α(NC) is best done by first establishing C_α/CR for the deposit, and then applying this ratio to the design values of CR. Saye et al. (2001) provide correlations to estimate the reduction in C_α with OCR due to surcharging.

Stability Analyses

Interpretation of Strength Data

The selection of design strengths from CK_0U TC/TE and/or DSS measured peak strengths (q_f for TX and τ_{hmax} for DSS) may require adjustments depending on the particular stability problem. Potential corrections include how s_u is defined for the stability analysis, effect of the intermediate principal stress (σ_2), strain compatibility, and 3-D slope stability end effects.

When performing undrained stability analyses with a method of slices the authors recommend that s_u should be defined as the available shear strength on the failure plane at failure, i.e., $\tau_f = q_f\cos\phi'$ (the point of tangency of the Mohr Circle to the effective stress failure envelope) rather than selecting $s_u = q_f$ (peak point on Mohr Circle) as per Section 2.5 of Ladd (1991). The definition reduces s_u by about 10 to 15% and, if incorrect, at least errs on the safe side. For the σ_2 effect, undrained shear in plane strain (i.e., simulation of a long embankment) leads to higher strengths than measured in triaxial tests. The increase is 9 ± 6% in compression ($\delta = 0°$) and 22 ± 3% in extension ($\delta = 90°$), for an average increase of 15% (Ladd 1991). Strain compatibility considers the fact that the peak shear strength occurs at different shear strains (γ) for different modes of failure, followed by strain softening (especially in compression). Hence the average strength that can be mobilized along a potential failure surface is less than the average of the peak strengths. The reduction is roughly 10 to 15% in brittle, sensitive soils with a design $\gamma \approx 1 - 2\%$ (strain at maximum mobilized strength) and is about 5 to 10% in plastic, insensitive soils with a design $\gamma \approx 10 - 15\%$ (Ladd 1991). Finally, for typical embankment failures the actual factor of safety based on a 3-D analysis is about 10 ± 5% higher than the 2-D FS computed from conventional analyses that inherently assume an infinitely long plane strain failure (Azzouz et al. 1981).

Although each of these four factors can affect the outcome by a nominal 10 to 15%, they also generally tend to cancel out. This may partly explain why 2-D stability analyses using simplified interpretations of CK_0U strength data (i.e., a simple average of the peak q_f values) usually do not result in unexpected failures because the computed FS from the "simple" and "rigorous" analyses often are quite similar. However, one should be aware of these factors since the net result of the various corrections varies with soil type, failure geometry and the quality and type of strength data being evaluated.

Selection of s_u for UU Case

Example 1 in Table 4 is representative of a small project requiring a stability evaluation for the UU Case such as for a preload fill or water testing a storage tank. For this problem one needs to develop a profile of the initial in situ s_u(ave) versus depth for circular arc stability analyses using isotropic strengths (i.e., no variation of s_u with inclination of the failure surface) obtained via either laboratory or in situ testing. Laboratory consolidation tests are required to determine stress history (and also for settlement predictions) when s_u(ave) is calculated via the SHANSEP equation with Level C estimates of S and m selected from the empirical correlations given in Table 3. For clays without shells or sand lenses-layers, proper FVT with Bjerrum's correction should give fairly reliable s_u(ave) results. Alternatively, s_u(ave) from CPTU soundings could be used, but only if the selected N_{kt} has been verified from prior experience for the deposit. FVT and CPTU data also can help to define the stress history. Given this modest level of testing, Table 4 suggests a relatively high design factor of safety (FS).

Fig. 13 illustrates the above approach for estimating s_u(ave). With an average PI of about 20% for the clay (Fig. 6), Table 3 gives S = 0.21 and m = 0.8. Using these values and the selected stress history profile gives the computed s_u values plotted in Fig. 13(c). Note that the CPTU data with N_{kt} = 18 provide a reasonable match to the SHANSEP s_u profile. Another important feature of Fig. 13(c) is the addition of lines of constant s_u/σ'_{v0}. This is very simple to do (although not included with the original report as per Fig. 6) and the $0.1\sigma'_{v0}$ line shows that the lab TV (and half of the FVT) strengths below the crust are 50% less than a reasonable strength for NC clay.

Selection of s_u for CU Case

The analysis for a CU Case (e.g., stage construction as shown schematically in Fig. 15) must first establish a height for Stage 1 loading (which would be treated as a UU Case) in order to achieve the maximum benefit from consolidation prior to Stage 2 construction. Consolidation testing is essential in order to ensure that the σ'_{vc} from the Stage 1 loading significantly exceeds σ'_p within the weakest foundation soils. The strength test data for OC clay could come from either Recompression tests (assuming good quality samples) or SHANSEP tests (with t_c = 1 day), whereas SHANSEP tests (with $t_c = t_p$) should be used for NC clay. Consolidation analyses are needed to predict the rate of consolidation within the Zone 2 foundation soil to develop profiles of σ'_{vc} prior to Stage 2. The SHANSEP equation is then used to compute the vertical and lateral variations in s_u with design values of S and m for input for the Stage 2 stability analyses.

Example 2 in Table 4, as illustrated in Fig. 15 for a circular arc stability analysis, assumes the use of isotropic strengths derived from either DSS tests or a combination of TC and TE tests (except for varved deposits where DSS must be used). The design s_u(ave) profile usually can be taken as the peak τ_h from DSS tests or the average of the peak triaxial strengths [q_f(C) and q_f(E)] (with consideration of adjustments as previously noted). For the Stage 1 UU Case, using s_u(ave) is clearly valid since the critical failure surface will have modes of shearing ranging from compression to extension. But for the Stage 2 CU Case (Fig. 15), the failure mode within Zone 1 varies from DSS to extension and thus using s_u(ave) will overpredict the actual strength. Conversely, the failure mode within the strengthened Zone 2 varies from compression to DSS and using s_u(ave) will underpredict the actual strength. However, since Zone 2 is stronger than Zone 1, using s_u(ave) throughout usually leads to a FS that errs on the safe side. Example 2 is typical of projects of intermediate complexity. The suggested range for design FS can be lower than for Example 1 since the strength parameters are better defined. The lower value (1.3) would apply when field instrumentation will monitor the degree of consolidation and lateral deformations.

Example 3 in Table 4 represents a highly complex stability problem that justifies the use of non-circular failure surfaces and anisotropic values of s_u that vary with inclination of the failure surface, $s_u(\alpha)$. In addition to in situ testing to assess spatial variability and extensive consolidation tests, the CK_0U strength testing program now includes both triaxial and direct simple shear tests. Ladd (1991) and Ladd and DeGroot (2003) describe how to interpret triaxial and DSS data to develop design values of $s_u(\alpha)/\sigma'_{vc}$ versus failure plane inclination.

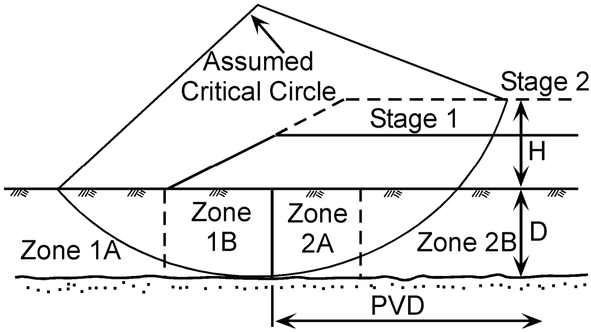


FIG. 15. Hypothetical cross-section for Example 2 in Table 4: CU case with circular arc analysis and isotropic s_u (from Ladd and DeGroot 2003).

SUMMARY AND CONCLUSIONS

The paper describes field and laboratory test programs for geotechnical site characterization which includes definition of the stratigraphy and selection of design parameters for settlement and stability analyses. It focuses on relatively uniform, saturated cohesive soil deposits for projects where construction will load the foundation soil beyond its preconsolidation stress. Although the tools, procedures, and interpretation methods needed to conduct a reliable site characterization program are well developed, general practice often ignores this knowledge, which often leads to either overly conservative designs (not economical) or problems with performance during and after construction (unhappy owners and even litigation). Thus the primary goal of the paper is to provide recommendations for improving the state-of-practice for assessing site stratigraphy, drilling and undisturbed sampling, in situ testing, and laboratory consolidation and strength testing. Table 5 lists common examples of poor practice and key recommendations for correcting poor practice. Significantly, most of these recommendations involve little to no extra time and cost compared to current practice.

Table 5 Site Characterization: Examples of poor practice and recommendations to correct common deficiencies

Poor Practice	Key Recommendations
Drilling and Sampling	
- Inadequate drilling mud weight - use of push samplers - poor tube geometry - field extrusion and reuse of tubes	- use proper drilling mud weight (Fig. 7) - use fixed piston sampler - use thin tubes with $D \geq 76$ mm, sharp cutting angle, and zero clearance - leave samples in tubes for shipping
In Situ Testing	
- poor FVT geometry with thick blades and use of torque wrench - poor saturation of CPTU (do not follow ASTM recommended procedure) - wide scatter in N_{kt} and k CPTU factors	- use FVT with proper geometry and gear drive system, FVT is most reliable in situ test for estimating s_u(ave) - CPTU is best tool for soil profiling - require saturation & interpret CPTU q_{net} data using site specific k and N_{kt} values
Soil Description & Classification	
- field logs and reports do not include USCS for all soils	- perform visual-manual tests for USCS - perform AL for all consol. & CU tests
Sample Disturbance and Selection of Lab Test Specimens	
- no evaluation of sample disturbance - use of test specimens from top or bottom of sample tube - sample extrusion without debonding	- x-ray tube samples and run s_u index tests - assess sample quality using ε_{vol} or $\Delta e/e_0$ from all consolidation tests (Table 2) - debond test specimens (Fig. 10)
Consolidation Testing and Parameters	
- IL test with LIR = 1, 24 hr increments, and σ'_{vmax} too low to define CR & σ'_p - use of variable t_c data from IL tests $\rightarrow$ erratic compression curves - use of c_v at σ'_{v0} or arbitrary σ'_v - selection of C_α without consideration of C_α/CR	- for IL tests, use reduced LIR near σ'_p, specify σ'_{vmax} to measure changes in CR beyond σ'_p, and use constant $t_c = t_p$ (NC) to better define σ'_p and CR_{max} - use more CRS consolidation testing - use both Casagrande and strain energy methods for estimating σ'_p
Stress History	
- reports do not define or consider stress history of the site - no σ'_p evaluation from lab & in situ data	- include profiles of σ'_{v0}, σ'_p & σ'_{vf} for relevant loadings - select design σ'_p using all available data
Strength Testing and Parameters	
- excessive reliance on UUC and CIUC tests - no tests to evaluate s_u anisotropy - no examination of most probable minimum s_u profiles via $s_u = S\sigma'_{vo}$	- replace UUC with TV, LV and FC - switch to CK_0U TC/TE and DSS tests - use good quality automated equipment for CK_0U TX and DSS testing - evaluate s_u/σ'_{v0} for all strength data

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NOTATION

The following symbols are used in this paper:

a	=	CPTU net area ratio
AL	=	Atterberg limits
b	=	$(\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3)$
C_α	=	rate of secondary compression = $\Delta\varepsilon_v/\Delta\log t$
CAU	=	anisotropically consolidated-undrained shear test
C_c	=	virgin compression index = $\Delta e/\Delta\log\sigma'_v$ for $\sigma'_v > \sigma'_p$
c_h	=	coefficient of consolidation for horizontal drainage
CIU	=	isotropically consolidated-undrained shear test
CIUC	=	isotropically consolidated-undrained TC test
C_k	=	hydraulic conductivity coefficient = $\Delta e/\Delta\log k_v$
CK_0U	=	K_0 consolidated-undrained shear test
COV	=	coefficient of variation
CPTU	=	piezocone penetration test
CR	=	virgin compression ratio = $\Delta\varepsilon/\Delta\log\sigma'_v$ for $\sigma'_v > \sigma'_p$
CRS	=	constant rate of strain
CU	=	consolidated-undrained
c_v	=	coefficient of consolidation for vertical drainage
d, D	=	diameter
DSS	=	direct simple shear
e	=	void ratio
e_0	=	in situ void ratio
EOP	=	end of primary consolidation
E_u	=	undrained Young's modulus
f_s	=	CPTU sleeve friction
FS	=	factor of safety
FV	=	field vane
FVT	=	field vane test
G_s	=	specific gravity of soil solids
h	=	height
H_0	=	initial layer thickness
ICR	=	inside clearance ratio
IL	=	incremental load
k	=	hydraulic conductivity
k	=	CPTU cone factor for preconsolidation stress
K_c	=	ratio of horizontal to vertical consolidation stress = $\sigma'_{hc}/\sigma'_{vc}$
k_h	=	horizontal hydraulic conductivity
K_0	=	coefficient of earth pressure at rest

k_v	=	vertical hydraulic conductivity
k_{v0}	=	vertical hydraulic conductivity at the in situ void ratio e_0
LI	=	liquidity index
LIR	=	load increment ratio = load increment/prior load
LL	=	liquid limit
m	=	strength rebound exponent
m_v	=	coefficient of volume change = $\Delta\varepsilon_v/\Delta\sigma'_v$
N	=	SPT blow count
NC	=	normally consolidated
N_{kt}	=	CPTU cone factor for undrained shear strength
OC	=	overconsolidated
OCR	=	overconsolidation ratio
p	=	$(\sigma_1 + \sigma_3)/2$
p_f	=	p at failure
p'	=	$(\sigma'_1 + \sigma'_3)/2$
p'_f	=	p' at failure
P_f	=	probability of failure
PI	=	plasticity index
PS	=	plane strain
PSC	=	plane strain compression
PSE	=	plane strain extension
q	=	$0.5(\sigma_v - \sigma_h)$ or $0.5(\sigma_1 - \sigma_3)$
q_c	=	CPTU tip resistance
q_f	=	q at failure
q_{net}	=	CPTU net tip resistance = $(q_t - \sigma_{v0})$
q_t	=	CPTU corrected tip resistance = $q_c + u_2(1\text{-}a)$
r_k	=	hydraulic conductivity ratio = k_h/k_v
RR	=	recompression ratio = $\Delta\varepsilon/\Delta\log\sigma'_v$ for $\sigma'_v < \sigma'_p$
S	=	undrained strength ratio at OCR = 1
SD	=	standard deviation
S_{FVT}	=	FVT undrained strength ratio at OCR = 1
SH	=	stress history
SPT	=	standard penetration test
SQD	=	Specimen Quality Designation
S_t	=	sensitivity = s_u(undisturbed)/s_u(remolded)
s_u	=	undrained shear strength
s_u(ave)	=	isotropic s_u for USA
$s_u(\alpha)$	=	anisotropic s_u for USA
t	=	thickness
t	=	time
t_{50}	=	time to 50% consolidation
t_{90}	=	time to 90% consolidation
t_c	=	time of consolidation
TC	=	triaxial compression
TE	=	triaxial extension
t_f	=	time to failure

t_p	=	time for primary consolidation
TX	=	triaxial test
u	=	pore (water) pressure
u_e	=	excess pore pressure
$\bar{U}_h$	=	average degree of consolidation for horizontal drainage
u_0	=	initial or in situ pore pressure
USA	=	undrained strength (stability) analysis
USC	=	Unified Soil Classification
UU	=	unconsolidated-undrained
UUC	=	UU triaxial compression test
$\bar{U}_v$	=	average degree of consolidation for vertical drainage
u_2	=	CPTU pore pressure for filter located at the cylindrical extension of the cone
w	=	water content
w_n	=	natural water content
z	=	depth
z_w	=	depth to water table
α	=	inclination of the failure surface from the horizontal
δ	=	angle between direction of σ_{1f} and vertical
Δu	=	change in pore pressure = $u - u_0$
ε	=	normal strain
ε_a	=	axial strain
ε_c	=	axial consolidation strain
ε_f	=	strain to failure
ε_v	=	vertical strain
ε_{v0}	=	vertical strain measured at σ'_{v0} in 1-D consolidation tests
ε_{vol}	=	volumetric strain
γ	=	shear strain
γ_b	=	buoyant unit weight
γ_f	=	shear strain at failure
γ_m	=	unit weight of drilling mud
γ_w	=	unit weight of water
μ	=	field vane correction factor
ρ_{cf}	=	final consolidation settlement
ρ_i	=	initial settlement due to undrained shear deformations
ρ_s	=	secondary compression settlement
σ_h, σ_v	=	horizontal, vertical total stress
σ'_h	=	horizontal effective stress
σ'_{hc}	=	horizontal consolidation stress
σ_{h0}	=	in situ total horizontal stress
σ'_{h0}	=	in situ horizontal effective stress
σ'_p	=	preconsolidation stress
σ'_{ps}	=	"perfect sample" effective stress
σ'_s	=	"sampling" or pretest effective stress
σ'_v	=	vertical effective stress

σ'_{vc}	=	vertical consolidation stress
σ'_{vf}	=	final vertical consolidation stress
σ'_{vy}	=	vertical yield stress
σ_{v0}	=	in situ vertical total stress
σ'_{v0}	=	in situ vertical effective stress
$\sigma_1,\sigma_2,\sigma_3$	=	major, intermediate, minor principal stresses
σ_{1f}	=	major principal stress at failure
τ	=	shear stress
τ_{ave}	=	$1/3(\tau_c + \tau_d + \tau_e)$
τ_c	=	τ for shear in compression
τ_d	=	τ for shear along horizontal failure surface
τ_e	=	τ for shear in extension
τ_f	=	τ on failure plane at failure
τ_h	=	horizontal τ or τ in DSS test
τ_{hc}	=	horizontal τ at consolidation
ϕ	=	friction angle in terms of total stresses
ϕ'	=	friction angle in terms of effective stresses

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The State of the Practice in Foundation Engineering on Expansive and Collapsible Soils

William N. Houston, Ph.D., P.E. M.ASCE[1] and John D. Nelson, Ph.D., P.E. F.ASCE[2]

[1] Professor Emeritus, School of Sustainable Engineering and the Built Environment, Arizona State University, Tempe, AZ 85287-5306 USA. E-mail: bill.houston@asu.edu
[2] Professor Emeritus, Colorado State University; CEO and Principal Engineer, Engineering Analytics, Inc. 1600 Specht Point Road, Suite 209, Fort Collins, CO 80525 USA. E-Mail: jnelson@enganalytics.com

ABSTRACT: Design of foundations for expansive and collapsible soils is an important challenge facing engineers. The state of the practice in this area has been changing over the past decades. The standard of care and definitions of the state of the practice and the state of the art are discussed, followed by an in-depth treatment of the state of the practice. The authors present what they consider to be the appropriate practice for field investigation techniques, laboratory testing, and foundation design on expansive and collapsible soils. They then discuss the general practice that is being followed and present recommendations for change. The experiences of the two authors represent different viewpoints in some areas. When that is the case both viewpoints are presented and the reader is left to consider each one on its own merits.

1.0 INTRODUCTION

The design of foundations on expansive and collapsible soils is one of the greatest challenges facing geotechnical engineers today. Although it has been recognized that such foundations pose unique problems, the history of rigorous approaches to this problem is short. Research findings relative to the behavior of these moisture sensitive soils have been available through publications for many decades, including the last two decades, in particular. Within the past 15 years, litigation against builders and design professionals has focused attention to this area, resulting in significant changes being made in the practice of foundation engineering.

Within the realm of the practice of foundation engineering for expansive and collapsible soils, there are three terms that warrant some attention. These are:

- State of the Practice,
- State of the Art,
- Standard of Care.

In the context of a paper to be prepared for publication and/or presentation at a conference, there appears to be very little controversy relative to the definitions of state of the practice and state of the art.

The **State of the Practice** refers to practices that are actually being carried out by practicing engineers. In other words, what is being done on a routine basis. If the audience for the paper is nationwide or even worldwide, then of course the state of the practice should probably be characterized in some average sense, given that practices vary significantly from state to state and even within a state. The state of the practice by the general geotechnical engineering community evolves with time and can be characterized only for a particular time or a modest period of time.

The **State of the Art** refers more or less to the best that we, as a professional group, can do. Employing the State of the Art may entail employment of the latest methods of analysis, research results, numerical models, laboratory testing equipment or methods, etc. In this case there is little need to "average" practices across the country or around the world, because the author seeks the "best". In a particular case it may be challenging for the author of a State of the Art paper to accomplish the preceding lofty characterization, but the concept is fairly easy to grasp.

Engineers often lament the fact that litigation and fear of litigation exert an inordinate influence on geotechnical practice. Although it might be possible to change that somewhat, change does not look easy or imminent. Therefore we must deal with the situation, as it is now. Accordingly, we choose to address herein some of the legal implications, at least superficially. The term standard of care has legal connotations (Smith, 2012; Mills, 2012; Durrant, 2012; Rossberg, 2012), as discussed in the following.

The **Standard of Care** represents that standard that an engineer should be following in order to provide the appropriate care to protect public health and safety. The Legal Dictionary (Law.Com, 2012) defines the standard of care as follows:

> *"The watchfulness, attention, caution and prudence that a reasonable person in the circumstances would exercise. If a person's actions do not meet this standard of care, then his/her acts fail to meet the duty of care which all people (supposedly) have toward others. Failure to meet the standard is negligence, and any damages resulting there from may be claimed in a lawsuit by the injured party. The problem is that the "standard" is often a subjective issue upon which reasonable people can differ."*

This definition is somewhat ambiguous in that one may choose to consider what is "prudent" in any particular case. Note that "duty of care" is embodied in the standard of care definition, which helps to explain why the terms are sometimes used interchangeably. This equivalence of terms does not seem problematic to the authors.

In the last book that Fu Hua Chen wrote (Chen, 1999), he defined the standard of care as, "Has the engineer in his/her work employed that degree of knowledge ordinarily possessed by members of that profession, and to perform (sic) faithfully and diligently any service undertaken as an engineer in the manner a reasonably careful engineer would do under the same or similar circumstance?" An important point that Chen brings into his discussion of standard of care is the state of knowledge regarding the pertinent area of engineering.

The ASCE Guidelines for Forensic Engineering Practice (Lewis, 2003) quote Weingardt (2000) as describing two generally accepted definitions of the standard of care.

> *"The first is that care which a reasonable person would exercise in a given situation with a certain set of facts, conditions, and circumstances. The second, more relevant definition calls for a comparison between the action of a professional relative to the skill and learning ordinarily possessed by other reputable engineers in the community. Under both definitions, the court also examines how prudently engineers handled the application of the engineering standards."*

In addition, these guidelines quote Peck and Hoch (1988) as asserting that the standard of care entails:

> *".... **The duty to stay informed** [emphasis added], which dictates that engineers must stay abreast of common subjects of discussion among professionals working in the same field ..."*

It is also clear that these guidelines seek to hold forensic engineers to a very high standard, requiring that their "…work must be based on practical experience and state-of-the-art awareness...".

Collectively considering the legal definition from the dictionary, Chen's definition, and the various quotes from the guidelines for forensic engineering practice, leads to the conclusion that just applying general field investigation techniques, laboratory tests, and design procedures that have been used in the past, albeit in a widespread fashion, does not necessarily meet the standard of care, when health and safety are an issue. It is clear that a prudent engineer must be aware of the current state of knowledge and either apply it or be prepared to defend the action of not applying it – when public health and safety are an issue. At the same time it must be acknowledged that the appearance of a conclusion, finding, or method in print does not necessarily make it accurate, valid, or worthy of application to practice. The state of knowledge tends to grow slowly over time and each new method requires a certain amount of confirmation and consensus within the research community, and perhaps, even recognition as being reasonable and practical by practitioners, before it should be widely applied to practice.

The preceding discussions of the standard of care rests heavily on the issue of public health and safety. Certainly, there are numerous geotechnical projects for which public health and safety is an issue including, but not limited to, foundations for nuclear power plants, dam and flood control structures, some bridge foundations, and often seismic loading and slope stability designs. However, cases for which foundation design in or on moisture sensitive soils would involve significant threat to public health and safety would be relatively scarce. In fact, if we eliminate general bearing capacity failure leading to toppling and failure to repair settlement damages in a timely fashion from the picture, author Houston would contend that cases of endangerment of public health and safety would be rare. Author Nelson, however, would contend that these cases would not be so rare.

In summary relative to definitions of terms, we see that the standard of care can potentially require a very stringent, high level of practice when projects are large and expensive and public health and safety are clearly at issue. On the other hand, for many small projects of a more routine nature, where health and safety are arguably not an issue, then the standard of care could shrink down to at least the state of practice nationwide, perhaps even lower in some cases. However, there appears to be a consensus that the standard of care should not be reduced to what is arguably the poorest practice in any state or community within a state – particularly if it can be reasonably argued that this local practice is unsuccessful with high frequency of failure. Certainly the authors endorse this position.

In the spirit of full disclosure, the authors will reveal a major part of their agenda. We seek to help bring about at least some modest upgrade relative to certain aspects of the state of the practice. It is our perception that the amount of meaningful testing and analyses done for a project today is commonly less than was done in the 1960's for a similar project, and we lament this fact.

In this paper the authors present what we consider should be the state of practice for field investigation techniques, laboratory testing, and foundation design on expansive and collapsible soils. We then discuss the state of practice that is being followed and present recommendations for change. The experiences of the two authors may represent different viewpoints in some areas. When that is the case both viewpoints are presented and the reader is left to consider each one on its own merits.

2.0 SITE INVESTIGATION

2.1 General Practice

The site investigation should be configured to gather sufficient information to define the entire soil profile to the depth and extent to which the geology of the area and the soil properties will affect the foundation behavior. The geology of the area and the particular geologic formation characteristics that can affect the foundation are important. Geologic maps of the area should be consulted and any geologic hazard maps should be studied. These types of materials are generally available from the U.S. Geological Survey as well as local or state geological agencies. For example, the Colorado Geological Survey and the Arizona Geological Survey are excellent sources for such materials.

The number and depth of exploratory borings must consider spatial variability of the soils in an area and the depth to which expansion or collapse will influence potential foundation movement. In the Front Range area of Colorado, where expansive soils are common, it is the general practice in the case of housing subdivisions to drill at least one hole for each lot, or in the case of multifamily buildings, to drill one or two holes for each building. For large commercial projects across the country it is common practice to space boreholes on a 30 m (100 ft) grid. This gives general coverage over the entire area.

For a single commercial building or large house, the general pattern of boreholes would be the same as for ordinary soil sites. The primary governing criterion is that the site be adequately covered to define the general soil profile. An example where drilling on each lot was not necessary is a site in Cannon Falls, Minnesota, where a housing subdivision that would include approximately 120 homes was proposed. The Colorado convention of drilling one hole on each lot would require about 120 exploratory boreholes. However, in this site the soil profile comprised about 5 to 10 meters of highly expansive shale overlying limestone which overlaid sound sandstone. Because of the predictability of the geology at this site, a total of about 15 exploratory holes to the depth of the limestone were sufficient to characterize the entire site.

The depth of exploration must extend to the depth to which the soil can influence the foundation. A school building had been constructed on a collapsible soil site in Rangely, Colorado. The collapsible soil extended to a depth of about 20 meters. Although the depth of collapsible soil had been determined prior to foundation design, drilled piers were extended to a depth of 13 meters. A few years after construction, wetting of the subsoils had progressed to a depth where hydrocollapse of the soil was causing the building to experience serious distress. The foundation was successfully remediated by compaction grouting the soil from the bottom of the piers down to the full depth of the collapsible soil. In this case, the depth of exploration was adequate to define the soil profile, but the design did not take that into account the relatively deep wetting which obviously occurred. The depth of wetting which should be adopted for design is an issue on which reasonable engineers may differ, including the authors. More discussion on depth of wetting and design philosophy will be presented subsequently.

At expansive soil sites, the exploration should extend to depths sufficient to define the zone in which heave can influence the foundation. This is especially important for deep foundations. Drilled pier foundations are widely used in the Colorado Front Range area. The drilled pier foundation relies on both dead load and skin friction in a stable anchorage zone to counteract uplift skin friction in the upper part of the pier (Nelson & Miller, 1992). Bearing capacity is usually not an issue for a pier foundation on these soils. The major concern for piers in expansive soils is the uplift force exerted by the swelling along the pier shaft within the active zone, z_a. The active zone, z_a, was defined in Nelson et al. (2001) as the zone of soil that is contributing to heave due to soil expansion at any particular time. The depth of potential heave, z_p, is the depth to which the overburden vertical stress equals or exceeds the swelling pressure of the soil (Nelson et al., 2001). When designing drilled piers, care must be taken in determining the depth of the active zone for design (i.e., the "design active zone"). A prudent designer should perform adequate analyses or investigation to establish within reasonable certainty the design active zone for the design life of the structure. Reasonable certainty can, for example, be established with some sort of risk-based analysis (Walsh et al., 2009; Walsh et al., 2011). If that cannot be done, it should be assumed that the design active zone is equal to the depth of potential heave (Chao et al., 2006; Overton et al., 2006; Chao, 2007).

The depth of exploration should extend sufficiently below the depth of expected heave to define adequately an anchorage zone for the pier. Soil samples should be taken to the entire depth of expected heave and anchorage and adequate testing should be done to that depth. The general practice of author Nelson is to drill a minimum of 12 meters initially and if highly expansive soil is encountered to that depth, additional deep borings are drilled.

In many cases that the authors have reviewed, the maximum depth of exploration has been only to depths of 6 to 8 meters below the ground surface. In the Front Range area of Colorado, it is common practice of some companies to test the soil at depths of 1.2, 2.7, and 4.2 meters with only a few tests being performed at a depth of 5.7 meters below the ground surface. If the foundation design includes a full basement with a structural floor and drilled piers with a design length of 7.6 meters, the piers would extend to a depth of about 10 meters below the ground surface. Thus, the samples tested at depths of 1.2 and 2.7 meters below the ground surface have no relevance for the pier design and the soil in only the upper 2 meters of the pier has been adequately tested.

Another commonly applied foundation design for expansive soils is referred to as overexcavation and replacement. In this method the expansive soil is excavated to a prescribed depth below the foundation and replaced with non-expansive or low expansive soil. The overexcavation and replacement method is generally used in conjunction with spread footings, but sometimes with drilled piers. In the Colorado Front Range area, depths of overexcavation are normally about 3 meters below the depth of the footing or grade beam. It is not uncommon for the excavated soil to be removed, remolded, moisture-conditioned, and replaced. If that is done, soil samples should be prepared to replicate the remolded and recompacted soil conditions and tested for both compression response when loaded at placement conditions and response to wetting under overburden plus structural loads.

Overexcavation and replacement is widely used for collapsible soils sites as well. The most common practice is to put more or less the same soil back in, but compacted to a higher density and wetter condition.

2.2. Soil Investigation Practices

The authors and their colleagues have reviewed many hundreds of geotechnical reports, perhaps as many as 600 to 700, and have interviewed geotechnical consultants in several states and have thus gained insight relative to geotechnical investigative practices across the USA. A brief summary of these practices is presented in the following sections, typically in tabular form, beginning with data derived from essentially all states and then focusing on a few regions known to have significant problems with moisture sensitive soils. The data presented is consistent with an estimated 95 percentile, meaning, for example, the depth of boreholes cited encompasses approximately 95% of the cases reviewed. For the sake of efficiency, Tables 1 – 6 were made to include data on soil testing and foundation design practices, even though these issues are discussed separately in later sections of this paper.

2.2.1 Soil Investigation Practices Across the USA Generally

Almost every state in the USA has some occurrences of either collapsible or expansive soils or both. The data in Table 1 below is weighted toward sites with soils that are not particularly moisture sensitive. The data provided were based on the experience of the authors and their colleagues and pertains to commercial buildings. Available data on nationwide residential practices were too sparse to include in the table, but the consensus is that the number of borings, depth of borings and testing, etc., is somewhat less for residential than for commercial.

2.2.2 Soil Investigation Practices in the Colorado Front Range Area

Over 300 geotechnical reports prepared for the Front Range area of Colorado have been reviewed and compiled into a database. These reports were prepared by a wide range of different geotechnical engineers during the period from 1978 to 2005. The reports provided recommendations for design of foundations for light structures on expansive soils. Pertinent information such as recommended foundation and floor type, maximum depth of exploration, and maximum depth of oedometer test were compiled into the database.

The maximum depth of exploratory boring is shown in Figure 1 for different years. In Figure 1, only one boring extended to a depth greater than 14 meters. The average depth of exploration is 8 meters. No obvious trend regarding change over time can be seen in Figure 1.

Table 1. General Soil Investigation Practices across the USA

Type of Structure	Borehole Spacing	Depth of Bore-holes	Depth of Testing Performed (exclusive of SPT and water content)	Types of Tests Typically Performed	Remediation Measures/Foundation Design
Commercial Buildings	30 m grid	3 to 3.5 m below final grade most common (up to 5.5 m in bldg area)	$\leq$ 4.6 m	SPT, moisture, gradation, CBR (occasionally in-situ density, response to wetting, compression or consolidation, shear tests)	Non-moisture sensitive: Most commonly shallow spread and strip footings, slab-on-grade, proof-rolling, over excavation and replacement of upper 1 to 4 m below footings and less below slabs. Moisture sensitive: Overexcavation and replacement (typical 1 to 4 m) and employ precautions and safeguards against deep wetting near buildings. Also, deep foundations, prewetting, surcharging, lime treatment.

The maximum depth of samples that was tested for expansion potential in the consolidation-swell test (response to wetting) is presented in Figure 2 for different years. Although there is no obvious trend in Figure 2, there are more samples tested from deeper depths after around 1994.

The maximum depth of sample tested for expansion potential is plotted against the depth of exploration in Figure 3. It is seen that about half of the samples tested for expansion potential were taken from depths of less than about half of the depth of exploration. This practice is potentially problematic in that the depth of potential heave in this area generally exceeds the maximum depth of exploration. Of course, it is only problematic if deep wetting is expected to occur.

The response to wetting tests are the most relevant to the expansive soil impact on foundation, but other tests that may be helpful in foundation design are typically performed on the samples retrieved. These additional tests include index tests, compaction tests, and occasionally shear tests and consolidation tests.

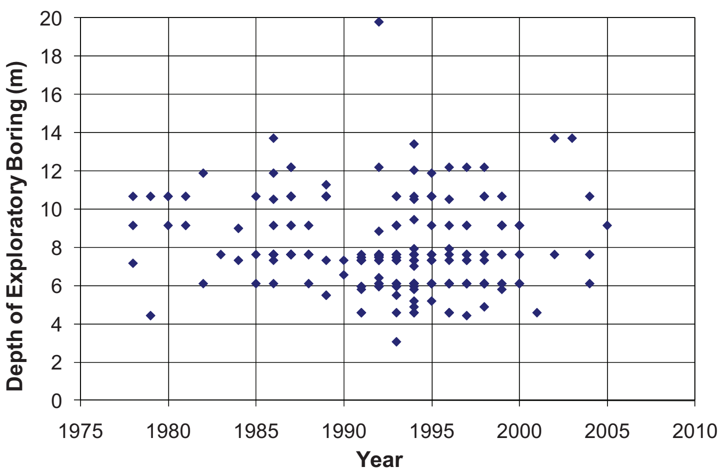

FIG. 1 Depth of exploration vs. time.

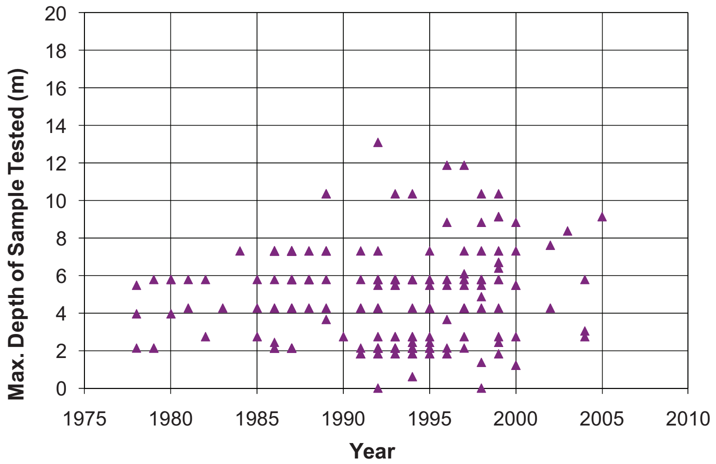

FIG. 2 Maximum depth of sample tested for expansion potential vs. time.

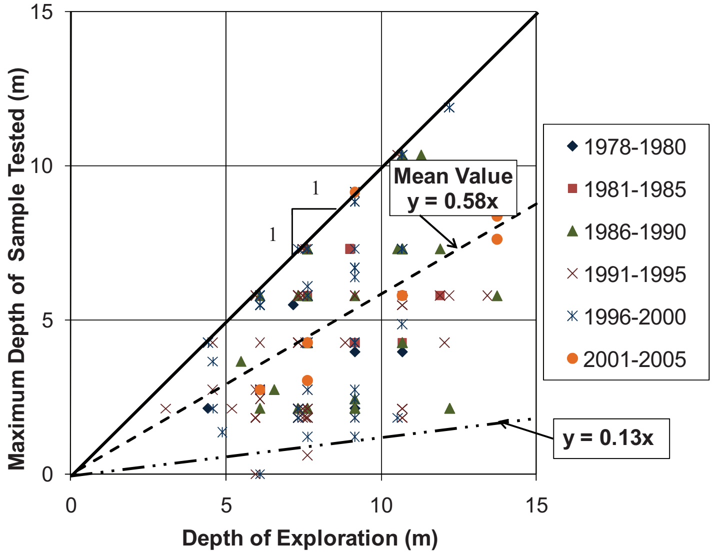

FIG. 3 Depth of exploration vs. maximum depth of sample tested for expansion potential.

2.2.3 Soil Investigation Practices in Utah

The data in Table 2 is derived primarily from southern Utah and was provided by David Black (Black, 2011). The data pertains to residential structures. The practices differ noticeably for sites expected to be on moisture sensitive soils (collapsible or expansive) compared to sites not expected to be moisture sensitive.

2.2.4 Soil Investigation Practices in Texas

The data in Table 3a relates almost entirely to expansive soils and was provided by Phil King (King, 2012). Phil King also provided the following input from the ASCE Texas Section (2007) as one of the sources for his input:

> *"As a minimum for unknown but believed to be uniform subsurface conditions, borings shall be placed at maximum 300 foot centers across a subdivision. Non-uniform subsurface conditions may require additional borings. One soil boring may be sufficient for a single lot investigated in isolation for a simple residence under 2500 square feet. However, more borings may be required on sites having fill, having large footprints, or noticeably varying geological conditions such as steep slopes or locations near known fault zones or geological transitions.*
>
> *Borings shall be a minimum of 20 feet in depth unless confirmed rock strata are encountered at a lesser depth. However, if the upper 10 ft of soils are found to be predominately cohesionless, then the boring depth may be reduced to 15 ft. Borings shall extend through any known fill or potentially compressible materials even if greater depths are required."*

The data in Table 3b also relates almost entirely to expansive soils and was provided by Marshall Addison (Addison, 2011). These data in both tables show differences between residential and commercial practices; for example, lime treatment and piers are extremely common for commercial and rare for residential. Note that there are regional differences across Texas.

2.2.5 Soil Investigation Practices in the Phoenix Valley, Arizona

The data in Table 4 was provided by Randy Marwig (Marwig, 2011). For the Phoenix Valley the differences between residential and commercial practices is modest. Post-tensioned slabs are very common and piers are rare.

2.2.6 Soil Investigation Practices in Southern California

The data in Table 5 is weighted toward sites characterized by collapsible soils and were provided by Sanjay Govil (Govil, 2011) and Iraj Noorany (Noorany, 2011; see also Noorany, 1997). Note that a distinction is made between smaller subdivisions requiring minimal cuts, fill, and regrading (relatively flat land) and larger projects requiring mass regrading with substantial cut and fill.

Table 2. Soil Investigation Practice in Utah - Residential

Borehole Spacing/Frequency	Depth of Boreholes	Depth of Testing (excluding SPT and moisture/density)	Types of Tests Typically Performed	Remedial Measures/ Foundation Design
Subdivisions: 1 per lot, occasionally 2	Non-moisture sensitive: ≤ 3.7 m Moisture sensitive: ≤ 6 m	Non-moisture sensitive: Upper 2 m Moisture sensitive: Upper 5 m	Index tests: moisture/density, solubility, occasionally consolidation and shear For moisture sensitive foundations: Response to wetting and swell pressure	Shallow foundations: Overexcavation and replacement- Collapsible soils, past practice approximately 0.6 to 1 m below footing; this is increasing now. (Note: primary problem with collapsible soils is failure to overexcavate deep enough). Expansive soils, 5 m of overexcavation is typical. Deep Foundations: Piers for expansive soils.

Table 3a. Soil Investigation Practices in Texas Provided by Phil King (King, 2012)

Borehole Spacing/ Frequency	**Depth of Boreholes**	**Depth of Testing (excluding SPT)**	**Types of Tests Typically Performed**	**Remedial Measures/ Foundation Design**
Residential Subdivisions: Varies from every lot to borings on 90 m spacings.	≤ 6 m	≤ 6 m	North Texas: Index and strength tests, pressure swell tests South Texas: Index and strength tests	North Central Texas (including Dallas): • Foundation design based on the condition with no remedial measures, typically stiffened slabs • Prewetting by water injection • “High-end residential: sometimes piers South Texas, including Houston and San Antonio: • Foundation design based on the condition with no remedial measures, typically stiffened slabs • Prewetting by water or chemical injection • “High-end residential: sometimes overexcavation and backfill with non-expansive material • “High-end residential: sometimes piers
Commercial 60 m spacing	On Grade ≤ 9 m Elevated ≤ 12 to 15 m	On Grade ≤ 9 m Elevated ≤ 12 to 15 m	Index and strength tests, and pressure swell tests	• Lime or chemical injection • Overexcavation and backfill with non-expansive material • Common to use piers to support walls and roof

Table 3b. Soil Investigation Practices in Texas Provided by Marshall Addison (Addison, 2011)

Borehole Spacing/ Frequency	**Depth of Boreholes**	**Depth of Testing (excluding SPT and moisture/density**	**Types of Tests Typically Performed**	**Remedial Measures/ Foundation Design**
Residential Subdivisions: Every other lot	≤ 6 m	≤ 4.6 m	Index tests compaction, response to wetting	North Central Texas (including Dallas): • Prewetting by water injection • Sometimes overexcavation and replacement South Texas, including Houston and San Antonio: • Overexcavation and backfill with non-expansive material • "High-end residential: sometimes piers • Low to medium cost residential: overexcavation and water injection; no piers
Commercial 60 m spacing	≤ 9 m	≤ 6 m	Index Tests, compaction, response to wetting	• Lime treatment very common • Almost always use piers

Table 4. Soil Investigation Practices in the Phoenix Valley, Arizona

Type of Structure	Borehole Spacing/Frequency	Depth of Boreholes	Depth of Testing (excluding SPT and moisture/density)	Types of Tests Typically Performed	Remedial Measures/ Foundation Design
Residential	1 Boring per 8 lots	≤ 4.6 m typically	Upper 2 m	Index tests, Expansion Index (EI), response to wetting, remolded swell test for fills and subgrades	• For EI ≥ 20: use post-tensioned slabs • For non-moisture sensitive soils: unreinforced slabs are used with lightly reinforced footings and stem walls
Commercial	30 to 60 m spacing	≤ 10 to 12 m	Upper 2 to 4 m	Index Tests, EI, response to wetting, remolded swell for fills and subgrades	• Residential and commercial about the same for expansive and non-moisture sensitive soils • For collapsible soils: Overexcavation and replacement of upper 1 to 1.5 m and avoid wetting

Table 5. Soil Investigation Practice in Southern California

Type of Structure	Borehole Spacing/Frequency	Depth of Boreholes	Depth of Testing (excluding SPT and moisture/density	Types of Tests Typically Performed	Remedial Measures/ Foundation Design
Residential	Smaller subdivisions requiring minimal cut, fills, and regrading: 1 boring per lot Larger projects requiring massive cuts, fills, and regrading: Boring and trenches configured to identify slope stability and moisture sensitive soil problems. Bucket augers common in slide areas	Smaller subdivisions with minimal regrading: 6 to 9 m typical, 15 m if liquefaction risk is anticipated Larger projects requiring massive regrading: 6 to 18 m generally, but almost always to "hard formational" material	Smaller subdivisions with minimal regrading: Most index testing in upper 3 m, but response to wetting tests are spread to full depths of borings Larger projects requiring massive regrading: EI tests tend to be shallow but other tests, including response to wetting and shear, are spread up and down the borings	Smaller subdivisions with minimal regrading; Index tests, EI tests, SPT tests, CA sampler, density, water content, consolidation, response to wetting, some shear tests, corrosion tests Larger projects requiring massive regrading: Index tests, Expansion Index, response to wetting, CA sampler penetration resistance, SPT for liquefaction studies, compaction tests, response to wetting on compacted samples, CPT rarely	Smaller subdivisions with minimal regrading Overexcavation and replacement denser, spread footings common, Post-tensioned slab very common, deepened footings, occasionally prewetting Larger projects requiring massive regrading: Overexcavation and replacement, commonly 6 to 9 m but occasionally much more. Post-tensioned slabs and shallow footings very common
Commercial	Warehouses, office buildings, and shopping centers: 30 m spacing	9 to 15 m, but always to hard formational deposits	Mostly in upper 3 to 5 m, but also throughout the depth of the borings	Same tests as for residential with additional shear tests	Slope stability, including seismic, is commonly a major issue. Construction "setbacks" from slope edge tend to be large. Post-tensioned slabs and shallow footings common. Use of piers is common. For non-moisture sensitive soils: non-post-tensioned, 125 mm slabs and lightly reinforced footings common.

2.3 Comments on General Practice

The general practice of site investigation with regard to the number and spacing of exploratory holes or test pits is usually adequate. The convention of drilling one hole on each lot in a housing development, which is a common practice in more than one state, is good for identifying spatial variability providing that the holes are drilled deep enough. However, depending on the depth of wetting adopted for design, it could be argued that the depth of borings and associated testing are often not deep enough to allow appropriate pier design. Even in those cases where relatively deep exploratory holes are completed, the majority of the samples that are tested are taken from depths that are too shallow to provide information on deeper moisture sensitive soils. The deeper soils can have a significant influence on free-field heave or soil collapse, provided the wetting reaches these deeper soils, of course. Whether or not wetting progresses to the tips of the piers, a lack of sampling and testing to this depth means that no data is obtained on skin friction. This in turn means that the probability is high that the piers will be under-designed, resulting in excessive heave or settlement, or over-designed resulting in excessive costs to the client. One of the projects investigated by author Nelson was a case where the samples that were tested represented only the soil that was excavated and no testing had been performed on the in-situ soil that remained. Thus, the foundation design was actually completed with no testing having been done on any of the soil that would influence the foundation behavior.

Author Nelson typically collects continuous core when doing exploratory boring. This is especially helpful in conducting final logging of the boring after inspection of the sample in the laboratory. It also provides the ability to observe and detect features in the soil profile that otherwise would not be seen.

3.0 LABORATORY TESTING

Laboratory testing of unsaturated soils for foundation engineering comprises the same general suite of tests as for ordinary saturated soils with one notable exception – the required testing for the response to wetting (or, in some cases, drying). This is, perhaps, the most important part of the testing program because it relates to analyzing potential heave or collapse settlement of the foundation.

The response to wetting test is an oedometer test in which the unsaturated sample is first subjected to a particular stress, called the inundation stress. Author Houston asserts that the usefulness of the test data obtained is maximized if this stress at inundation is the overburden plus the stress due to structural loads. After a period of time for compression of the soil to take place, the sample is inundated. After wetting, an expansive soil will swell and a collapsible soil will undergo compression, or collapse. After the swelling or collapse has been completed the sample can be further loaded. In the case of an expansive soil, it is compressed back to a thickness less than that at which the sample was inundated. The amount by which an expansive soil swelled during inundation is termed the “percent swell” and the stress required to compress the sample to its original height is termed the load-back swelling pressure, σ_{cs}. Empirical correlations can then be used to estimate the swell pressure. Alternatively, the sample could be prevented from swelling during inundation and the stress required to prevent swelling is measured. In the process of preventing swelling it is particularly important to account for apparatus compression. This result is the directly measured constant volume swelling pressure σ_{cv}. The percent swell and the swelling pressure are used in many methods to compute free-field heave (Nelson and Miller, 1992; Fredlund and Rahardjo, 1993; Nelson et al., 2006). In the case of collapsible soils, the strain upon inundation is termed the collapse strain for compacted specimens, but the collapse strain deemed most appropriate for most field conditions corresponds to the strain from the origin to the wetted curve (i.e., dry plus wetted strain) for undisturbed soils from the field (Houston and Houston, 1997). Further aspects of the response to wetting test are discussed in the paper by Chao et al. (2010) and Houston and Houston (1997).

In order to obtain an unbiased estimate of heave or collapse it is necessary to consider partial wetting (Houston, 1992; Overton et al., 2010). The strain derived from the conventional response to wetting test is the potential strain corresponding to full wetting. If the design engineer fails to evaluate (quantify) the probable partial wetting, then the engineer is left with the necessity of conservatively assuming full wetting, which is arguably over conservative in most cases – especially in the lower portion of the design wetted zone when wetting is from downward infiltration. The full wetting achieved in the laboratory test can ordinarily be achieved in the field by long term ponding, rise in groundwater table, or development of perched water zones on strata with relatively lower hydraulic conductivity. Even in the case of long term ponding, degrees of saturation in excess of 90% may be found only fairly near the bottom of the pond. As examples of the significance of partial wetting, Table 6 presents findings of degree of wetting from several cases of downward infiltration into moisture sensitive soil deposits. At many expansive soils sites where the soil has a high clay content and relatively high soil suction, degrees of saturation significantly higher than 90% can be found.

Soil suction measurements for application to practice usually are conducted using the filter paper method. In that test the sample is place in a sealed container along with a piece of calibrated filter paper. The soil and filter paper are allowed to come into equilibrium and the water content of the filter paper is measured. The calibration curve of the filter paper is used to determine the soil suction of the soil. Aspects of the filter paper method are discussed in Houston et al. (1994). The water content and suction of the soil can be used to calculate potential free-field heave (McKeen, 1992; Lytton, 1994).

There are advantages and disadvantages to the use of each method. The advantage of using the oedometer response to wetting test method is that almost all geotechnical engineering laboratories are equipped to perform this test. It is straightforward to perform, and it has less opportunity for error than the method based on soil suction measurement. A relative disadvantage of a rigorous soil suction method is that it usually requires lab capability of measuring and controlling suction.

Table 6. Examples of Partial Wetting for Various Sites

Site Location	Partial Wetting Conditions	Reference
New Mexico Site	**S_i ~ 15%; S_f ~ 35%.** Irrigated Lawn.	Walsh et al., 1993
Groundwater Recharge Sites in Arizona	**S_i ~ 15%; S_f ~ 60%** in upper 3 ft and essentially no change below that. Ponding for extended period.	Houston et al., 1999
China Loess	Reports of **settlement of about 10% of full collapse potential**. Therefore, partial wetting.	Houston, 1995
ASU Collapsible Soils Study Sites	**S_i ~15 to 20%; S_f ~ 50 to 70%**. Ponding for extended periods.	El-Ehwany and Houston, 1990
Denver, CO Site	**S_i ~ 68%; S_f ~ 96%** in the upper 25 feet for 4 years. Irrigated Lawn.	Chao et al. (2006)

The soil suction method in its most rigorous form is as follows. An undisturbed sample that is representative of a layer in the field to be modeled is transferred to an oedometer-type pressure plate device (Perez-Garcia et al., 2008) and wetted to a degree corresponding to the final (after wetting) moisture content in the prototype, while being subjected to the net normal stress projected for the corresponding point in the prototype. If only the heave (or collapse) strain for the point in the prototype is being sought, this strain can be measured directly in the oedometer-type device and it is not actually necessary to measure, estimate, or control the initial soil suction. However, it is necessary to estimate and control the final soil suction (after wetting) so that the lab specimen exhibits the appropriate strain. It could be argued that this method produces the most accurate value of the field strain that can be achieved because it entails an undisturbed sample, it is site-specific or layer-specific, and it more or less automatically incorporates the effects of partial wetting. Wetting until the specimen will absorb no more water is an end-extreme special case of the method.

If it is desired to extrapolate the lab results more widely to the field, for example to estimate the strain due to wetting at points above and below the field sample, but in the same layer, then it is necessary to obtain the slope of the log suction versus strain curve, sometimes symbolized as γ_h. This determination requires that both the initial and the final soil suction be evaluated and controlled in the laboratory. A shortcut to the more rigorous procedure outlined above is to simply estimate γ_h as being equal to the gross average of γ_h values for similar material, or to get an estimate of γ_h from correlations with index properties. Acceptance of the loss in accuracy, which can be substantial (Singhal, 2010), entailed in estimating γ_h does make it unnecessary to measure or control soil suction and makes it unnecessary even to test a sample in an oedometer-type device.

It is very tempting to compare and contrast the soil suction method with the more familiar response to wetting test method using the conventional oedometer device. However, the comparisons may make the two methodologies look more different than they are. Recall that the conventional response to wetting method leads first to a wetting-induced strain corresponding to full wetting, which is arguably overly-conservative for most cases. Once the commitment has been made to incorporate the effects of partial wetting the introduction of soil suction estimates into the procedure is more or less inevitable. To incorporate partial wetting into the final result it is necessary to use estimated initial and final suction values to arrive at a reduction factor to be applied to the potential strain associated with full wetting. It is possible to approximately quantify the effects of partial wetting via degree of saturation or water content (soil-specific relationships), if one wishes to accomplish this goal without uttering the word soil suction or thinking much about it. The authors do not endorse these substitutes unless site-specific/soil-specific correlations are developed and available.

In summary, the response to wetting test in the conventional oedometer described above yields the potential strain from full wetting and this strain must be reduced to account for partial wetting, if applicable, in order for unbiased results to be obtained. This reduction for partial wetting normally entails estimation of initial and final soil suction, in the prototype, but it does not require measurement/control of soil suction in the laboratory. On the other hand, the most rigorous soil suction method does entail measurement/control of the soil suction in an oedometer-type pressure plate device, but it incorporates partial wetting directly. If γ_h is estimated from correlations, then initial and final suction values must be estimated, but no laboratory measurements of soil suction or strain response are needed at all. This last method (estimating γ_h) is typically significantly less accurate than the response to wetting test with adjustment for partial wetting and the rigorous soil suction methods (Singhal, 2010).

3.1 General Practice

The general practice for laboratory testing utilizes primarily the response to wetting test for moisture sensitive soils. A fewer number of geotechnical engineers use the constant volume test and fewer yet use the soil suction measurements. There is not yet a standard value of inundation pressure that is used in performing the response to wetting test. Some engineers prefer to apply a stress approximately equal to the overburden pressure corresponding to the depth from which the particular sample was taken. Others use a standard stress for all tests. Obviously, if the resultant strain is to be representative of the expected strain in the prototype, the inundation stress should be overburden or overburden plus structural load, if a structure is present. Some classification schemes for expansive soils refer only to whether a soil has a low, moderate, high, or very high range of percent swell. In those cases, there are advantages to using the same inundation stress for all samples. A classification method has been proposed by Nelson et al. (2007a; 2011a) that utilizes both swelling pressure and percent swell to define the Expansion Potential, EP.

3.2 Comments on General Practice

The general practice of laboratory testing varies. Although the general procedures that are followed for oedometer testing are fairly consistent, there are variations with respect to the care taken during testing, and to some degree, test interpretation. The oedometer data must be corrected for compressibility of the apparatus. Also, if constant volume test data is being used, it must be corrected for sample disturbance, unless the sample has been subjected to overburden stress before wetting is commenced (Singhal et al., 2011). In some laboratories it is common only to load the sample to the inundation stress and then inundate it without reloading the sample in order to determine the load-back swelling pressure. The load-back swell pressure can be adjusted empirically (Singhal et al., 2011) to obtain an estimate of the constant volume swell pressure. This estimate is generally not as good as the result from a dedicated constant volume swell pressure test with adjustments for apparatus compressibility, but it may be satisfactory for many cases and it has the advantage that both a good value of strain at inundation stress and a fair value of constant volume swell pressure is obtained on one specimen. Thus, it would be good practice, in general, to determine the load-back swell pressure.

As an alternative to performance of the constant volume swell pressure test or the use of an adjusted load-back swell pressure, the following procedure can be used. Inundate the first specimen at overburden stress and measure the swell strain. Repeat the swell test on a companion specimen (as identical as possible) using an inundation pressure 3 to 5 times the overburden pressure. Plot these swell strains versus log of inundation pressure and determine where the straight line crosses zero strain, as a good estimate of the constant volume swell pressure (ASTM D4546; Singhal et al., 2011). The slope of this curve can also be used as the heave index, C_H (see Figure 5).

3.3 Recommendations for Changes

An important change that is recommended for oedometer testing is the need to assure that correction is made for equipment compressibility. Further, recent research (Singhal et al., 2011) has shown that the constant volume swell pressure increases significantly with confining stress at the time of inundation. Because of this dependence, the best practice for constant volume swell pressure determination is to load the specimen to overburden stress plus any structural load stress and then inundate, holding volume constant in consideration of equipment compressibility.

The best method for getting C_H is to measure swell on two or more specimens, one at overburden plus structural stress and the other at 3 to 5 times this value, as described in Section 3.2. This method also provides a good estimate of the constant volume swell pressure, requires less time than determination of the load-back swell pressure followed by correction, and finally requires less technician attention (or alternatively, less sophisticated computer control) than does the conventional constant volume swell pressure test.

The authors firmly agree that an insufficient number of response to wetting tests are being performed on moisture sensitive soils, in general, and that some of these tests should be on deeper soils, although the authors disagree on how much deeper these tests need to be, in most cases. Most of the geotechnical reports reviewed by the authors show substantial funds being expended for drilling and very little or nothing being spent on response to wetting and compression tests. Almost all of these reports would be improved if 25% of the boreholes were judiciously eliminated and replaced with an equal expenditure for response to wetting and compression tests and analyses of results – when moisture sensitive soils are involved. It should also be noted that good engineering requires that samples compare to expected density and moisture for fills be subjected to response to wetting tests. If the material at fill conditions is not expansive then it may be collapsible. Another relevant point here is that sometimes the replacement fill materials being used are too granular and too clean, yielding high permeability and a danger of creating the "bath tub" effect.

The authors also agree that an insufficient number of shear tests are being performed to adequately evaluate skin friction on piers and that these tests need to be deep enough to represent the lower portions of the piers. Simply designing piers with an extremely conservative skin friction value is not good engineering, in that it usually wastes the client's money.

Some guidance on the number of response to wetting tests that should be performed to characterize a collapsible soil site (for a given layer) is provided in Figure 4 . The data in Figure 4 were derived from a study of seven sites in Arizona and California, where the mean collapse strain, the standard deviation, and coefficient of variation, COV, were determined for each site (Houston et al., 1998). The COV was found to be the most stable parameter and averaged about 0.3. It was also found that the variation on a scale of 1 m or less was about the same as on a scale of 10 m or more, for a given soil type/layer, and that a normal distribution provided a satisfactory fit to the collapse strain values. Thus it was possible to prepare Figure 4, which can be used as follows.

- Assume a priori that COV = 0.3 for the new collapsible soil site
- Chose an error band which is acceptable and enter Figure 4 at COV = 0.3 and pick off the required number of samples (tests) to achieve the chosen error band. For example, assume that an error band of 20% is chosen. At COV = 0.3, n = 8. Suppose 8 tests are run and the mean collapse strain is 10%. The resulting statement that can then be made is that the true mean, resulting from a very large number of tests (millions of tests, e.g.) has a probability of 0.95 of falling between 8% and 12%.
- After the tests are done for the new site, a mean, standard deviation, and site-specific COV are available. By sliding horizontally along n = 8 to the site-specific COV, the error band can be fine-tuned, and the corresponding probability statement can be refined.

- The allowable error band can be chosen by posing the question: "How large would the error in measured average collapse strain have to be to change the foundation design?"

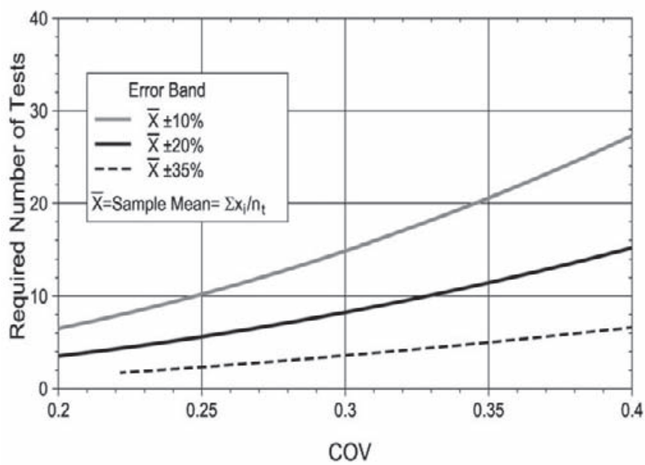

FIG. 4 Number of samples required for probability of 95% that true site (layer) mean lies within error band shown.

4.0 DESIGN METHODS

Foundation engineering for expansive and collapsible soils involves selection of an appropriate foundation type followed by rigorous analysis to design the foundation system to be used. Foundations on expansive soils generally consist of either stiffened mat foundations, deep pier and grade beam foundation, or overexcavation and replacement used with either spread footings or pier and grade beam foundations. Designs for collapsible soils sites are similar, but overexcavation and replacement are heavily favored options.

For stiffened mat or spread footing foundations, the analysis of expected movement involves calculation of heave at the level of the foundation. Free-field heave is defined as the heave (integration of vertical expansion strain over the zone where expansion occurs) due only to change in soil suction with no change in net normal stress. Therefore, it is the heave of the ground surface due to wetting with no structural load applied. For the free field case the net normal (vertical) stress is due only to overburden. When a structure is present the net normal stress is due to overburden plus structural load. The presence of a structure makes the behavior soil-structure interactive, because the magnitude of the expansive strain (upon wetting) is dependent on the magnitude of the net normal stress. The structural stress due to the footings or mats dissipates with depth and building load stresses likewise vary with depth along piers, as a function of the relative movement between pier and adjacent soil.

Heave calculation for foundation systems that incorporate the overexcavation and replacement method follows the same general procedure, but the soil profile includes the properties of the replacement soil to the depth of overexcavation. The design of deep piers can take one of two approaches. Chen (1988) presented a method for "rigid pier" design that assumes no heave of the pier. In that method, the force from the uplift skin friction in the active zone is equated to the required resisting skin friction and the length of a required anchorage zone is computed. That method typically results in longer piers than are necessary. Nelson and Miller (1992) presented a method to calculate pier heave for a given soil heave profile. This method is based on work by Poulos and Davis (1980), and is termed the "elastic pier" method. It facilitates calculation of the required pier length to limit pier heave to a tolerable amount, generally taken as 25 mm. That method has some limitations in that it assumes either a uniform soil or one in which heave varies linearly with depth. It is applicable over a limited range of pier slenderness ratios. Recently, Nelson et al. (2011c) have developed an improved finite element method of analysis for calculating pier heave. This computer code is termed APEX. It allows for calculating pier heave even for long slender piers, and it considers, as input, the soil heave profile as determined for computing free-field heave.

In Section 3.0, testing procedures for obtaining the expected strain due to wetting for a sample from a particular depth in the field have been described. If the strain at a point was derived from the soil suction based method, the strain value has already been reduced for partial wetting, as explained earlier. If the strain value was derived from conventional response to wetting tests with full wetting, then a reduction for partial wetting should be made as appropriate. In order to evaluate the total heave (or settlement) at the surface it is necessary to have these reduced strain values for the center of each layer that contributes significant strain. Then the total heave or settlement, ρ_t, is the summation of all the contributions:

$$\rho_t = \Sigma(\varepsilon_i \Delta z_i) \qquad (1)$$

Where ε_i and z_i are the estimated wetting induced strain and layer thickness, respectively.

A piece of information that is essential to the determination of the strain values is the pattern, depth, and degree of wetting of the subsoils. For sites with soils having high to very high expansion potential, the depth of potential heave, z_p, may be much deeper than the expected depth of wetting. For these sites it is especially important to devote some significant effort to assessing the probable depth and degree of wetting, although it is important for all sites with moisture sensitive soils.

Local experience with observed depths of wetting is very useful in making these projections, provided that irrigation practices in the future continue more or less unchanged, compared to past practices. Particularly when a change in irrigation practices is contemplated (either to wetter or to drier than that obtained by past practices), numerical models may be helpful. There are several challenges associated with modeling of flow through unsaturated soils, and potential numerical modeling problems associated with infiltration into expansive soil profiles have been discussed by Houston et al. (2011). If the inputs to the numerical model can be reliably assessed and constrained to match the inputs for the prototype over the last several decades, it may be possible to compare the numerical model "prediction" with what has actually been observed locally, thus calibrating to some extent the model. In general, the numerical modeling results could be a useful supplement to local experience on depth and extent of wetting. Computer software is available that will provide analyses that take into account the climatological and other environmental conditions. These include, for example, UNSAT-H; HYDRUS-2D; VADOSE/W; SVFlux; and MODFLOW SURFACT.

4.1 Current Design Practice

4.1.1 Determination of Heave Index

When a value of C_H, the heave index, is available, heave can be evaluated from Equation 2.

$$\rho = C_H \Delta z \log\left(\frac{\sigma_f}{\sigma_{cv}}\right) \tag{2}$$

Where:
- ρ = heave contribution for the soil layer
- C_H = heave index
- Δz = soil layer thickness
- σ_f = vertical net normal stress
- σ_{cv} = constant volume swelling pressure

Note that Equation 2 can be derived from Equation 1 above, with ε_i being replaced with $C_H \log(\sigma_f/\sigma_{cv})$. Typically, σ_f would be the vertical net normal stress due to overburden or overburden plus structural stress if a structure is present. Equation 2 is further illustrated in Figure 5 below.

If a soil profile has been divided into several layers and a lab test is available for the center of each layer and a strain value, reduced appropriately for partial wetting, is available for each of these center points, then Equation 1 can be used to sum up the heave, without direct evaluation of C_H or use of C_H. However, if heave is being evaluated for a soil layer for which no sample test result for strain is available, but a value of C_H, believed to be reliable, is available, then Equation 2 can be used.

The difference between the procedures used by various practitioners and researchers generally relate to the method by which C_H is determined. A method using the slope of the line connecting the percent swell for various values of inundation stress was described in Section 3 of this paper and also in Nelson et al. (2006) and Nelson et al. (2007b). This line is not exactly a stress-strain line because it is not derived from increasing (or decreasing) the stress on a single specimen and observing the resultant strain. Instead, the line represents the effect of applied confining stress on the amount of swell that will take place upon wetting. Different researchers and practitioners have applied correction factors or made variations on the way in which they define C_H. The authors agree that the method described above is rigorous and also the most efficient method for obtaining C_H.

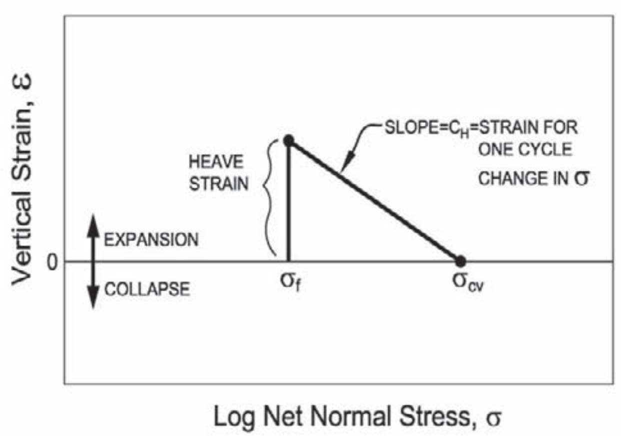

FIG. 5 Determination of C_H.

4.1.2. Depth and Degree of Wetting

The depth of wetting refers to the depth to which any increase in water content has occurred, regardless of how small. The degree of wetting refers to a particular point in the profile and relates to how much wetting has occurred at that point. For example, if the degree of saturation increased from 22% to 25% at a point the degree of wetting would be small, but if the increase were from 40% to 98% degree of saturation the degree of wetting would be very large.

Both the expected depth and degree of wetting in the soil profile in the future are very important parameters to be evaluated as a part of predicting the performance of a foundation. Unfortunately, this part of the prediction is the most difficult and also somewhat controversial. The authors have had rather strong disagreements on these issues in the past, relative to expansive soils in Colorado, and our respective positions are described in lengthy detail in Walsh et al. (2009), Walsh et al. (2011), and Nelson et al. (2011b). The reader is referred to the cited paper, discussion, and closure, from which the reader can judge the position that seems most reasonable. Depth and degree of wetting in collapsible soil profiles has been discussed extensively by author Houston and his colleagues (Houston and Houston, 1997; El-Ehwany and Houston, 1990).

The authors do agree that, in general, insufficient attention has been given to lateral sources of water, both for expansive and collapsible soils. These potential lateral sources may not be an important issue when the surrounding ground surface and underlying beds are horizontal. However if upper clay materials are cracked and weathered, the risk of future wetting from lateral sources should be considered if the ground surface or subsurface bedding is sloped. Likewise, in clay formations, if underlying seams of sand slope upward and outcrop where some future water source could feed the outcrop, the risk of lateral source wetting should be assessed. For collapsible soil profiles, the potential for lateral movement of perched water above lower permeability layers should be considered in assessment of probable depth of wetting. In addition to lateral movement of water, there are some hydrogeologic environments wherein rising groundwater table can become a significant source of subsurface wetting. The negative consequences of wetting of moisture sensitive soils from lateral inflow of water or rising groundwater table can be significant, highlighting the importance of consideration of geology, topography, and site grading as a part of the geotechnical investigation.

4.2 Recommendations

With regard to expansive and collapsible soils, the authors make the following recommendations.

4.2.1 Heave Calculation

For stiffened mat foundations, the value of heave to be provided to the structural designer for design purposes should be the maximum calculated free-field heave, corrected for applied load and partial wetting. The resulting heave of the slab may vary from a small amount at the center to the full amount at the edge of the slab or it may reach the full amount at the center and a very small amount at the edge. The structural slab design should consider what amount of final differential heave of the stiffened slab will be imparted to the structure.

Spread footings should not be used on highly expansive soil sites unless some type of soil modification is employed. This is commonly in the form of overexcavation and replacement. The overexcavated soil commonly is used as the replacement soil. The remolded soil usually retains some expansion potential after replacement. That expansion potential is sometimes significant. Therefore, it must be measured and taken into account in calculating the design heave.

There are two design considerations in the design of deep foundations with grade beams. One is the required length of pier for the tolerable design movement. The pier design generally requires consideration of soil-structure interaction issues such as relative movement between the pier and adjacent soil, load transfer curves or functions, and in some cases, compression or extension of the pier. These analyses can be conducted with various available methods including those presented in Coyle and Reese, 1966 (e.g., APILE), Poulos and Davis (1980), Nelson and Miller (1992) and a newly developed computer model called APEX developed by author Nelson and his colleagues (Nelson et al., 2011c).

The other consideration for pier and grade beam foundations is the void space that must be maintained beneath the grade beam. The required void space can be calculated using the heave prediction methods discussed above.

4.2.2 Collapse Potential Calculation

The estimation of total settlement of a foundation due to wetting induced collapse (hydrocollapse) follows more or less along the same lines as the computations for heave. For each identifiable layer deemed to have collapse potential, the potential strain due to full wetting is determined from response to wetting tests, using inundation pressures equal to overburden plus structural load. Figure 6 shows a schematic of the response to wetting for a natural cemented collapsible soil. Also shown in Figure 5 is the effect of disturbance. Depicted here is the case where the specimen is loaded to overburden stress and then wetted. Dry (in-situ moisture) loading corresponds to AC for an excellent undisturbed specimen. Dry loading from A to E corresponds to a significantly disturbed specimen. Thus, the more the disturbance the more the dry strain. After wetting, point F is reached. The wetted curve is a band because any soil layer exhibits some heterogeneity. Houston and Houston (1997) make the case that the best engineering approximation is to use the ordinate from the origin to the wetted curve, for natural soils, because the dry loading curve AB for the field prototype is nearly flat for essentially all unsaturated collapsible soil deposits. The research also showed that the final wetted curve ordinate was within a fairly narrow band, and not very dependent on the amount of disturbance.

The use of the change in ordinate from the origin to the wetted curve in Figure 6 does result in more strain than the more common practice of interpreting the collapse strain as the distance between the dry loading and wetted curves. However, once the effects of partial wetting are incorporated, the predicted strain comes back down, sometimes dramatically. A case in point is a commercial building in Albuquerque, New Mexico for which the actual settlement was only about 20% of the potential (fully wetted) settlement, even though the depth of wetting was about 14 m. This perhaps somewhat surprising result arose because the degree of wetting was small (initial S = 15%, final S = 35%).

Once the collapse strain, reduced for partial wetting, has been ascertained for each layer, the total settlement is computed using Equation 1, corresponding to integration of the strain profile over the depth of collapsible soil deposit. The differential settlement is normally taken as a fraction of the estimated total, depending on the foundation type and expected stiffness, and on the pattern of wetting anticipated.

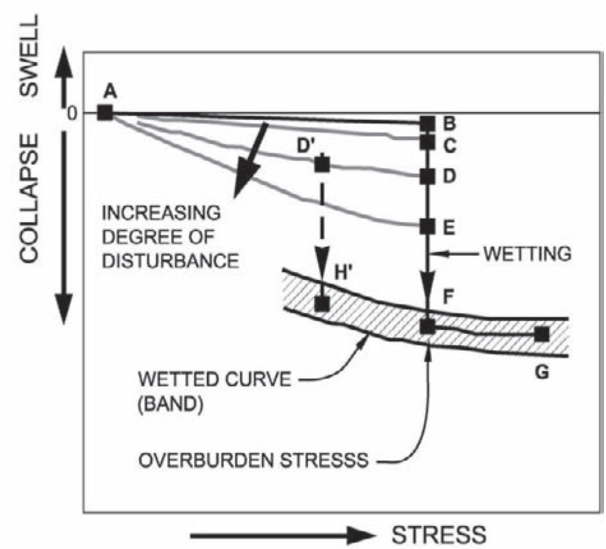

FIG. 6 Schematic of response to wetting for a collapsible soil, from Houston and Houston (1997).

5.0 SUMMARY AND CONCLUSIONS

The preceding discussions have highlighted areas in which the authors believe that the current state of practice is somewhat lacking. These are,

- Inadequate characterization of the site,
- Inadequate or absent laboratory testing, and
- Inappropriate foundation design for the site.

These deficiencies have been very costly as they have resulted in both underdesign (leading to failures of various degrees) and overdesign, because overly conservative assumptions were made in the absence of test results and analyses. Admittedly, the attention in the paper has been mostly focused on the underdesigned cases, but it should be emphasized that for every billion dollars wasted due to underdesign there are probably 5 billion dollars, or even more, wasted in overdesign. This is not to say that we should start making designs with higher probability of failure so we could save our client's money. Instead, what is being implied is that we should do more testing and analyses so that we more or less directly measure parameters and properties that we often now very conservatively estimate. This change together with more analyses and the study of possible alternative designs, would allow us to select designs that are usually much more economical, but with similar low probability of failure.

It is regrettable that geotechnical engineers generally feel helpless trying to improve the state of the practice because clients are typically unwilling to fund more in-depth studies and analyses and competition between consultants discourages anything other than slow change and improvement. It appears that, as a geotechnical community, our only hope of significantly improving the state of practice is to mount a nationwide organized campaign designed to demonstrate to clients (with case histories and life cycle cost analyses) that more testing and analyses saves them money at the bottom line. History shows some slow progress toward improvement due to litigation, but this is a painful path.

The deficiencies perceived by the authors are summarized in the sections below.

5.1 Site Characterization

Most notable in this area is failure to advance the exploratory holes to adequate depth in some cases. Furthermore, there is a tendency for engineers not to test samples from sufficiently deep depths. This is particularly true for skin friction tests for piers and also true when a reasonably good estimate of the depth of wetting exceeds the boring depth or the testing depth.

In addition to advancing the hole to sufficient depth and testing samples at those depths is the need for careful logging of the soil profile. Author Nelson asserts that an important element of careful logging is inspection of continuous core when practical. Granted that taking core during sampling increases the cost of investigation, that cost is minimal when compared to potential cost of damages. It is the responsibility of the engineer to educate the client as to the need to undertake that cost. Author Houston asserts that continuous coring is not practical in a great many cases.

Characterizing only the soil profile is not enough. The environmental conditions, climatological factors, irrigation practices and proximity to off-site water sources such as golf courses, irrigation ditches, streams, reservoirs, and others must be defined. As a part of the geotechnical report, owners should be admonished to provide good drainage, protect utility lines, quickly repair water lines, minimize irrigation input, and carry roof drainage as far away from the structure as possible, and at least 3 m if that can be achieved.

5.2 Laboratory Testing

The primary concern of the authors relative to laboratory testing is that not enough testing is being done, particularly response to wetting tests and compression tests where moisture sensitive soils are involved. When budgets are constrained it may be that some modest reduction in number of boreholes, particularly for fairly uniform or predictable profiles, and use of these funds for additional testing and analyses would be appropriate.

Laboratory testing includes the handling and storage of the samples and following of appropriate test procedures. Samples must be transferred with chain-of-custody documentation. They must be handled carefully and stored so as to prevent moisture change.

Very important is the need to make corrections for equipment compressibility. Correcting for equipment compressibility can result in a twofold increase, or more, in the measured swelling pressure.

The most efficient procedure for obtaining both the percent swell data and the swell pressure, needed for making heave predictions, is the performance of two swell tests (or more), as described in Section 3.2. Extrapolation of the straight line (on a semi log plot) to zero strain yields the swell pressure as a byproduct of the testing. The second most efficient (but somewhat less accurate) procedure for obtaining the swell pressure is to perform a swell test on a single test specimen, followed by load back to get a swell pressure, which is then corrected empirically (Section 3.2). The least efficient but most accurate means of obtaining the constant volume swelling pressure, σ_{cv}, is to perform a σ_{cv} test. It should be performed using an inundation pressure of overburden plus structural stress. The best method for obtaining C_H, when needed, is outlined in Section 3.3.

The most useful test for collapsible soils is the response to wetting test with Figure 4, or an alternative, suitable algorithm, used as a guide in selecting the number of test to be performed as a function of the accuracy required for decision making. Referring to Figure 6, the full wetting collapse strain should be taken as the origin to the wetted curve, and then reduced for partial wetting, as appropriate. The reduced strain values thus obtained for each layer are then used in Equation 1 (see Section 4.0) to obtain the total settlement. The differential settlement is then estimated as a fraction of the total settlement using an empirically-based factor.

5.3 Rigorous Design Methods

First and foremost in foundation design concerns is the need to define the design depth and degree of wetting accurately. When the depth of potential heave is large, it is not practical or economical to design for full wetting to the depth of potential heave. For many sites, an appropriate depth of design active zone can be defined using 1-D computer analyses (benchmarked to observed field data) that consider climate, irrigation, and soil profile. For low to moderate swelling sites the analyses may not be needed. It is prudent to utilize reliable local experience on the depth and degree of wetting observed in the past or to design for full wetting to the depth of potential heave if no data or analyses are available.

After an accurate depth of design active zone has been defined, rigorous analyses must be applied to the design of the foundations. In the authors' experience with litigation cases involving piers, it has been seen that, until recently, in almost no cases were design calculations performed, by either the geotechnical or structural engineer. In the cases where this criticism applies, a change in the state of the practice is needed.

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State of Practice:
Offshore Geotechnics Throughout the Life of an Oil and Gas Field.

Philippe Jeanjean[1], Ph. D., P.E., M. ASCE

[1]Technical Authority, Geotechnics, Global Upstream Projects, BP America Inc., 501 Westlake Park Boulevard, Houston, TX 77079, USA; e-mail: philippe.jeanjean@bp.com

ABSTRACT

The paper describes selected offshore geotechnical practices, from geohazards assessment in the early stages of a project, to the design of shallow and deep foundations, with a focus on deepwater applications and conventional siliceous sediments. The past and current recommendations of API and ISO are summarized and their differences are highlighted. The use of total and effective stress pile design methods is explored and recent developments in the design of piles in sands are discussed. Last, observations and lessons learned from the performance of offshore foundations during hurricanes are presented.

In summary, worldwide industry design practices are broadly aligned through the use of API and ISO standards. Nevertheless, unusual or difficult basin-specific soil conditions require modified design approaches and foundation solutions.

INTRODUCTION

Purpose and Outline

The paper is intended for an audience of onshore practitioners and focuses on offshore practices deemed of most interest and relevance to the onshore community. It does not aim to be an exhaustive presentation of all offshore geotechnics practices. Recent books dedicated to offshore geotechnical engineering by Randolph and Gourvenec (2011) and Dean (2010) can be consulted for in-depth reading.

A few basic facts about the offshore industry are first offered. Practices in deepwater geohazard evaluation in the early stages of a project are then presented. A discussion on foundation design, limited to driven piles and shallow foundations for conventional soils such as silica clays and sands, follows last. Difficult soils conditions such as calcareous or micaceous sediments are excluded. Space limitations also preclude discussions regarding the design of suction anchor and other anchoring systems, although they constitute an important part of deepwater foundation practice.

Last, geotechnical challenges and practices associated with the design of pipelines in very soft sediments at very low effective stresses are not addressed herein but can be found in Randolph (2012, this volume).

Offshore Oil and Gas Basins and Typical Sediment Types

The offshore oil and gas industry was born in 1947 in the Gulf of Mexico (GoM), with the installation of the first offshore platform in 6 m of water. The "Oily Rocks" development offshore Baku, Azerbaijan followed in the 1950's, and the industry expanded to the Arabian Gulf and the North Sea in the 1960's. Today the industry is present in many basins in all continents but Antarctica (Figure 1). In the Northern GoM, production from deepwater fields on the continental slope (i.e. in water depths greater than 305m or 1000 ft) started in the mid-1990s and has gained continued importance. It now represents 80% of total oil and 50% of total gas production. Other basins such as Brazil and Angola also see most of their production from deepwater areas. The geological settings of selected basins are summarized in Figure. 1 and, in broad terms, are (McClelland and Reifel, 1986):

- Deltaic: with thick sections of Holocene soft sediments, often over-pressured
- Relict: with Holocene and late Pleistocene sediments on the continental shelf exposed during the last glacial period
- Glaciated: with sediments on the shelf that were once covered by continental ice sheets, with possible boulders
- Carbonate: with sediments with high concentration of calcium carbonate.

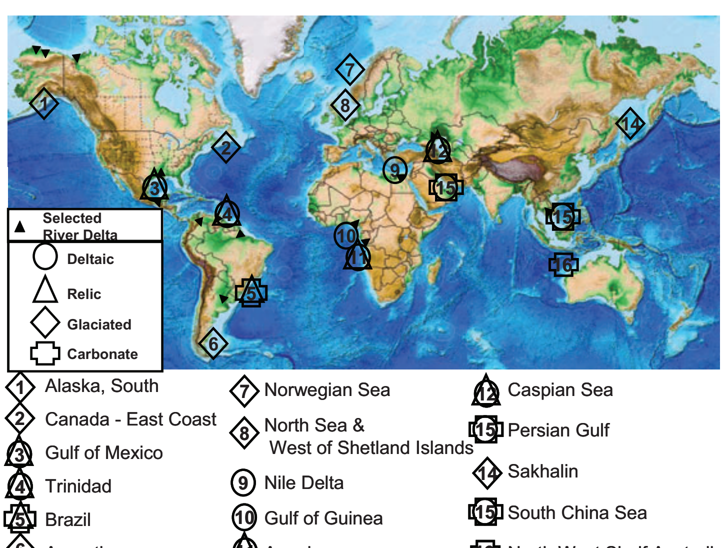
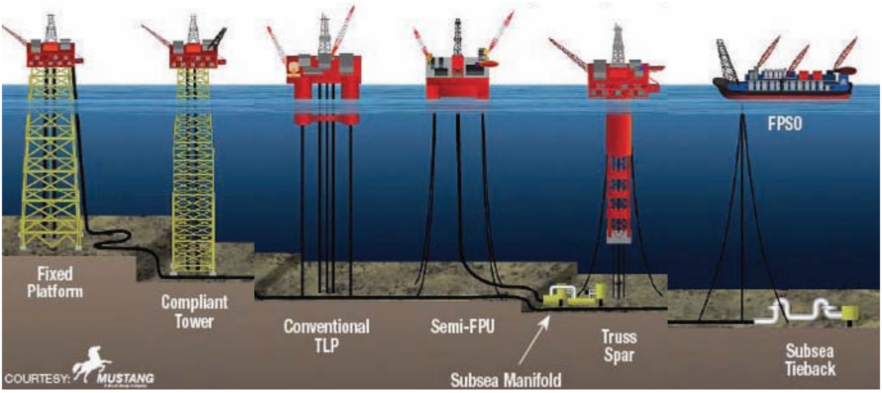


FIG. 1. Selected offshore hydrocarbon basins and typical sediment type in the seabed upper 150m (simplified and modified from McClelland and Reifel, 1986).

Type of Offshore Structures and Foundations

The main types of offshore platforms and the loads they impose on their foundation are summarized in Figure 2. The type of structure chosen for a field development will depend mainly on water depth, seabed conditions, regional preferences, and operators experience with a particular concept. The water depth record for a floating production system currently stands at an impressive 2600 m.



Platform Type	Fixed Platform	Compliant Tower	Tension Leg Platform (TLP)	Semi-Floating Production Unit (FPU), Spar, and FPSO[4]	Subsea Tieback
Proven water depth, m (ft)	Up to 412 m (1353ft)	305 to 531m (1000 - 1742 ft)	up to 1425 m (4674 ft)	Up to 2600 m (8530ft)	224 to 2747 m (738 - 9014 ft)
Water Depth Record holder	Bullwinkle, Gulf of Mexico	Petronius, Gulf of Mexico	Magnolia, Gulf of Mexico	Pioneer, Gulf of Mexico	Cheyenne, Gulf of Mexico
Typical foundation type	Driven piles[1]	Driven piles	Driven piles[3]	Suction piles, driven piles, or drag anchors	Shallow mat foundations, suction piles
Typical design loading conditions	Lateral in very shallow water, axial compression or tension in deeper water	Axial compression	Axial tension	Tension loads, oriented from 0 to 45 degrees from horizontal	See Note 2

Notes:

1- Drilled and grouted piles are used in calcareous soils.
2- Loads on subsea structures can range from simple compression loads to complex loading conditions along six degrees of freedom
3- Some TLP use shallow bucket foundation
4- FPSO: Floating Production, Storage, and Offloading platforms

FIG. 2. Typical platforms for offshore development (2011 Deepwater Solutions for Concept Selection - Mustang Engineering and *Offshore Magazine* - used with permission) and associated foundation loads.

Stages in a Typical Offshore Project

Offshore projects are typically divided into stages where specific activities take place. One such approach is summarized in Figure 3.

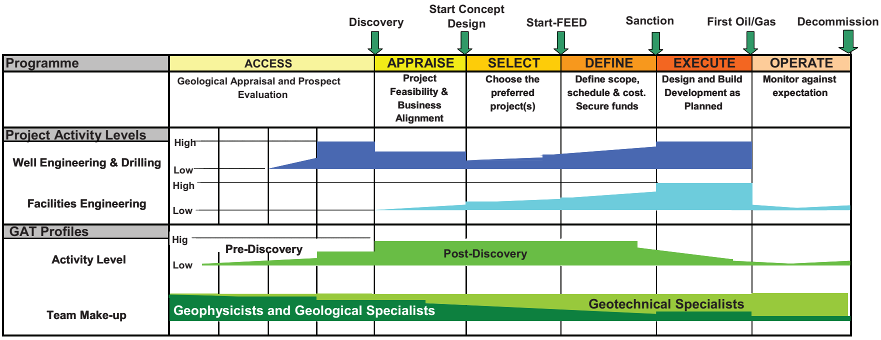


FIG. 3. Typical stages and level of geotechnical activities in offshore projects (from Evans, 2010) (FEED: Front-End Engineering Design; GAT: Geohazard Assessment Team).

The geotechnical engineer is involved during the entire life of the project and interacts closely with geophysicists and geologists in the early stages. Key activities and industry practice during each stage, except for the Access stage typically includes little geotechnical content, are now presented.

THE APPRAISE AND SELECT STAGES

In shallow waters, most developments have historically consisted of fixed platforms founded on driven piles. The physical footprint of such projects is very small, in contrast with deepwater projects where the footprint of the development can become quite large. A typical field layout for a deepwater FPSO project is shown on Figure 4. The lateral mooring system of the platform, along with subsea equipment and flowlines can extend over one hundred square kilometers.

At the same time, the deepwater continental slopes of many basins include widespread geohazards which pose challenges that have been documented in Jeanjean et al (2003a, 2005) for the Gulf of Mexico, Power and Clayton (2003) for West Africa, Hadley et al (2008) offshore Malaysia, as well as Kvalstad (2007) and Evans (2010).

Geohazards can be defined as local or regional soil conditions that may lead to unfavorable events causing loss of life, damage to the environment or loss of property. They typically include steep slopes, faults, gas hydrates, mud volcanoes, and earthquakes. An example of development in a geohazardous area is shown on Figure 5 with the Atlantis field located at the base of the Sigsbee escarpment in the Southern Green Canyon area of the GoM, where steep slopes with clear evidence of past failures are present, as discussed by Jeanjean et al (2003a).

By the end of the Appraise stage, the geotechnical engineer, working in multi-discipline teams with geophysicists and geologists, provides a preliminary understanding of the geohazards present within the area of interest and the risks that they may pose to the planned infrastructures. This understanding is refined and a final assessment is provided by the end of the Select stage, so that risks due to geohazards can be fully accounted for in selecting the preferred field development option.

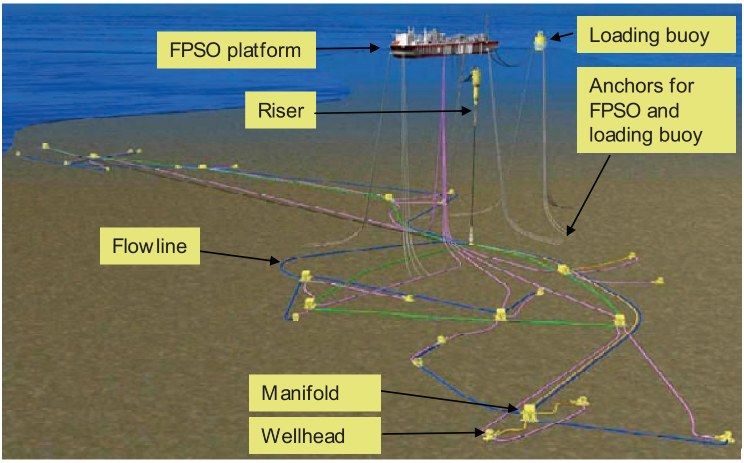


FIG. 4. Example of a deepwater field development extending over a large footprint. The water depth is nominally 1500 m.

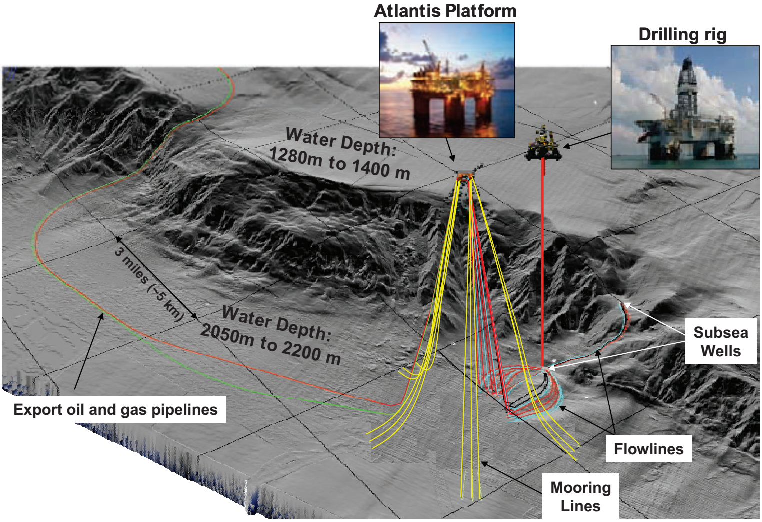


FIG. 5. Example of field development in a geohazardous area: the Atlantis development and the Mardi Gras Export Pipeline Routes in the Gulf of Mexico.

Codes and Standards and Geohazard Assessment

Although codes and standards clearly state the requirement for a rigorous geohazard assessment, they do not provide extensive guidance on how they should be performed, except for seismic hazards which have dedicated codes. For example, there is no advice on which method to use for a slope stability analysis or which safety factor is acceptable. Table 1 lists the key codes and standards from the American Petroleum Institute (API) and the International Standard Organization (ISO) that are used in the offshore industry and highlights how little content is devoted to geohazards.

Table 1. Offshore Codes and Standards Guidance on Geohazard Assessment

Standard	Design Application	Total Length (pages)	Geohazard-Related Text Length (pages)	Foundation-Related Text Length (pages)
API RP 2GEO	Shallow Foundations and Driven Piles for Fixed Structures	103	2 (2%)	101 (98%)
API RP 2A	Shallow Foundations and Driven Piles for Fixed Structures	225	1 (0.5%)	36 (16%)
ISO 19901-4	Shallow Foundations	34	2 (6%)	32 (94%)
API RP2T	Driven Piles for TLP	142	0.2 (0.2%)	6 (4.2%)
API RP2SK	Driven Piles and suction anchors for Spars, FPSO, and semi-sub	181	0.1 (0.05%)	38 (21%)
ISO 19901-7	Anchors for floating drilling systems	130	0.1 (0.07%)	10 (8%)
ISO 19904-1	Anchors for semi FPU and spars	194	0.1 (0.05%)	0.1 (0.05%)
ISO19905-1	Foundation for jack-up drilling units	288	2 (0.7%)	46 (16%)
Average for above offshore standards			1%	29%

As a result, it is not uncommon for deepwater projects to devote much more budget and geotechnical personnel time to understand geohazards than to design foundations. Jeanjean et al (2003a, 2003b), Kvalstad (2007), and Evans (2010) summarized industry practices and challenges in details and proposed rigorous frameworks for geohazard assessment. The following sequence of tasks is typical.

High resolution geophysical surveys

Geohazards are first imaged and mapped, at an appropriate level of resolution, through seismic reflection geophysical surveys. The various tools used are summarized in Figure 6 and range from low-resolution three-dimensional (3D) data to two-dimensional (2D) ultra-ultra high resolution (uUHR) data typically collected by Autonomous Underwater Vehicles (AUVs).

To illustrate how indispensable uUHR data are in geohazard assessment, the same seabed cross-section is shown on Figure 7, as imaged with conventional 3D data, UHR data, and uUHR data. The sequence of two buried debris flows in this example can only be mapped imaged and mapped (Figure 7d) with uUHR data, thereby dramatically improving the understanding of relevant geological processes.

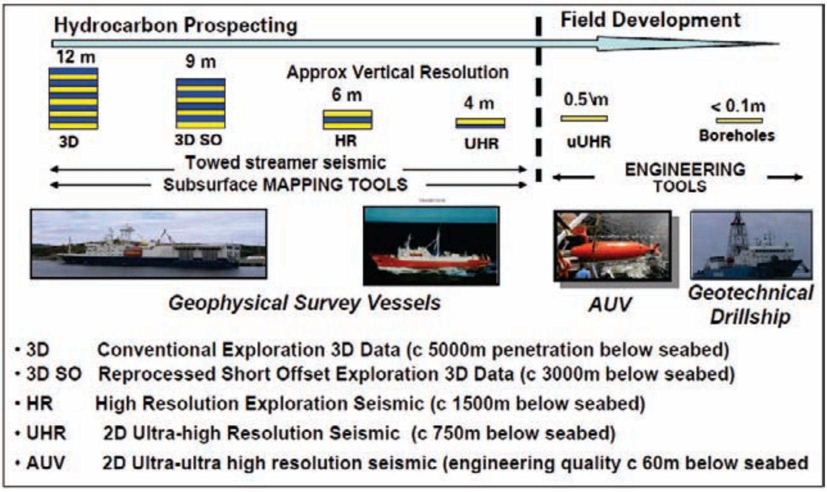


FIG. 6. Geophysical tools used in geohazard assessments (from Evans, 2010).

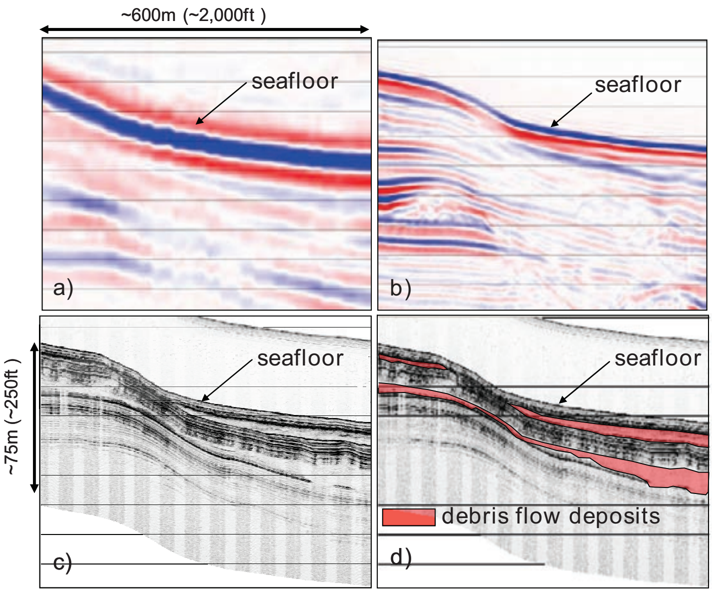


FIG. 7. Repeat seabed imaging with a) 3D data b) UHR data, c) uUHR data and d) interpreted cross section from uUHR data.

AUV data also include multibeam bathymetry. The same portion of the seafloor is shown on Figure 8, as imaged with 3D data and AUV uUHR data. The interpretation of the landslide processes at this site can only be performed with the highest resolution AUV data.

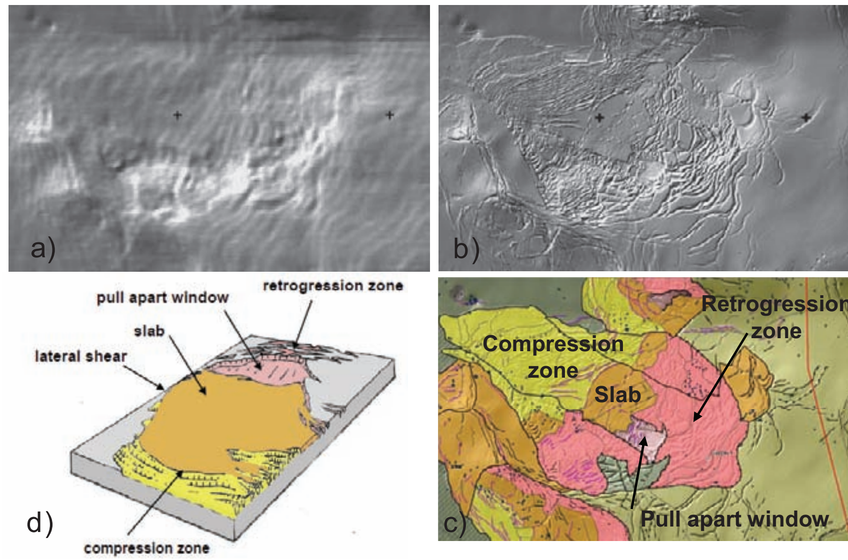


FIG. 8. Seafloor bathymetry of 3 km by 5 km area with a) 3D conventional data and b) AUV multibeam bathymetry data. c) mapped seafloor features. d) generalized morphology and processes.

Integration with geotechnical surveys

Once key geohazards have been mapped, a geotechnical site investigation is performed to "ground truth" or verify the interpretation of the geologist. Figure 9 a) and b) show an area that was suspected of having experienced past slope failures. A 100 mm diameter piston cores was collected, split and photographed. This single core is shown on Figure 9c) as a series of 13 sections, starting with the seafloor on the top left and ending with the deepest part of the core at the bottom right corner of the figure. The core revealed several sequences of debris flows, some of which below the resolution of the AUV data. The time of occurrence of each event (8510 years Before Present (BP), 14670 years BP, and 16120 years BP in this example) is typically measured using radioactive carbon C^{14} techniques and is a key input to estimate recurrence intervals of slope failures.

Understanding Trigger mechanisms

When faced with potentially unstable slopes, the project typically reflects on credible trigger mechanisms within engineering time frames in order to estimate the probability of occurrence of future failures. Jeanjean et al (2003b) found strong correlations between slopes failures and high sedimentation rates. Hance and Wright (2003) reported the trigger mechanism of 366 submarine landslides to also include earthquakes, gas hydrate dissociations, waves, erosion processes, and salt diapirism processes.

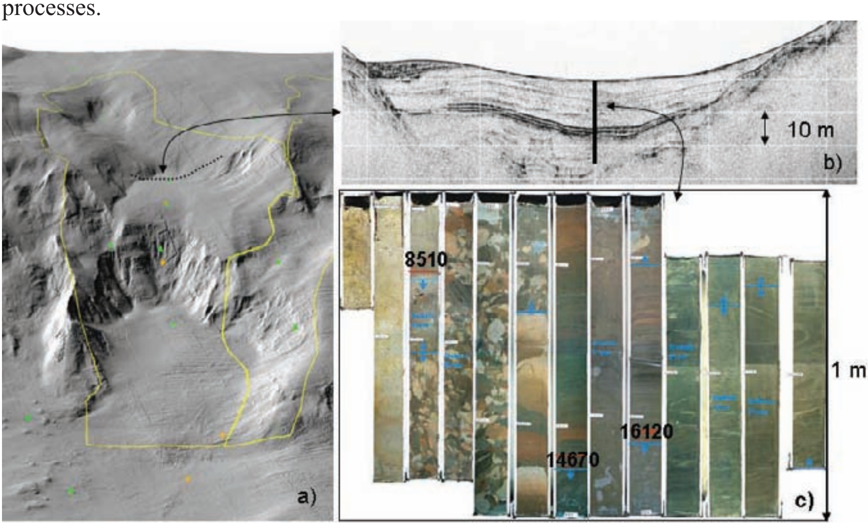


FIG. 9. Integration of a) seafloor rendering, b) AUV uUHR profile, and c) 100mm piston core in an area of past debris flows.

Communicating the Results of Geohazard Risk Assessment

Once the probability of occurrence and the consequences of an unfavorable geohazard event have been estimated by the project, the geohazard risk is typically reported in a semi-quantitative framework (Figure 10 a), although full quantitative risk assessments (Figure 10 b) are also used. This framework is also used for other project risks so that risks from all engineering disciplines can be compared in a consistent framework.

a)			Consequences				
			Negligible	Minor	Moderate	Major	Significant
			1	2	3	4	5
Cumulative Annual Frequency	>0.1	5	Medium	Medium	High	High	High
	>0.01	4	Low	Medium	Medium	High	High
	>0.001	3	Low	Low	Medium	Medium	High
	>0.0001	2	Low	Low	Low	Medium	Medium
	>0.00001	1	Low	Low	Low	Low	Medium

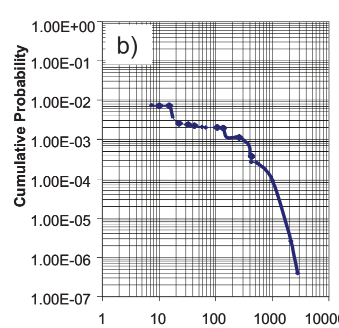


FIG. 10. Examples of risk reporting in a) semi-quantitative framework and b) full quantitative framework (modified from Evans, 2010).

THE DEFINE AND EXECUTE STAGES

During these stages, offshore foundations are designed, fabricated, and installed, mostly according to API or ISO standards, although local country regulations can add additional requirements. The discussion will therefore focus on these two sets of standards.

The early geotechnical practice was defined by the McClelland Engineers company, which developed the first practices for the design of driven piles in clays. Today, the recommendations of API remain influential worldwide, although they have been modified or discarded in certain basins faced with soil conditions significantly different than those of GoM or problematic soils such as calcareous sands. The API guidance for the design of driven piles has historically been included in RP 2A. In 2011, this guidance was updated and transferred to a new standard, RP 2GEO, which is dedicated to geotechnical and foundation issues.

The evolution and current status of industry practice for the design of deep and shallow foundations has been discussed in details by Pelletier et al (1993) and Jeanjean et al (2010) and is summarized in Figure 11.

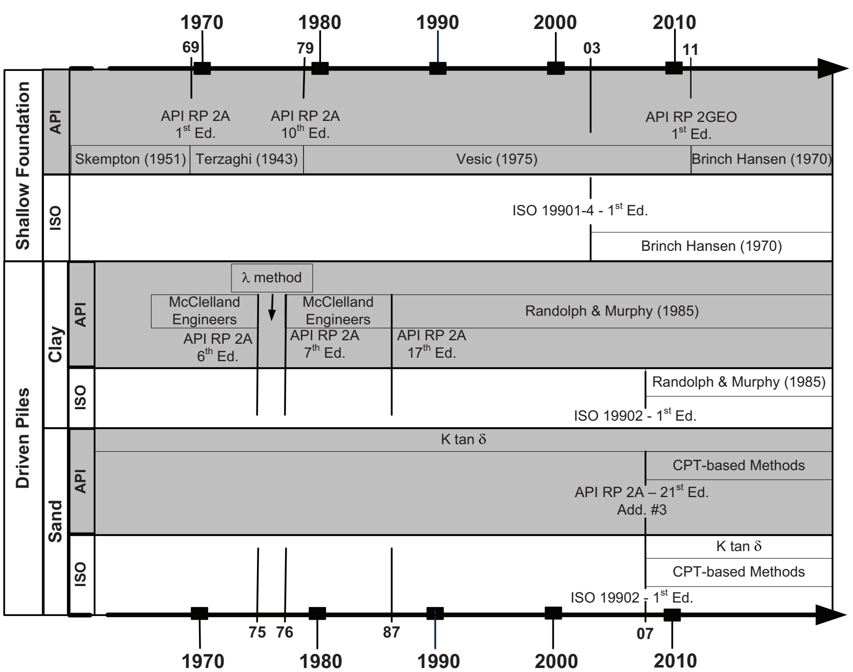


FIG. 11. Evolution and current status of recommended design methods in API and ISO codes for shallow and deep foundations.

Both lumped safety factors and limit state frameworks (known as LRFD or "Load and Resistance Factor Design" in North America) are used within the industry.

It should be noted that the API RP 2A standards exists both in the lumped safety factor format (known as the Working Stress Design), API RP2A WSD, and in an LRFD format (API RP2A LRFD). However, the API 2A LRFD document was withdrawn by API in 2008 and was rarely used for foundation design before that.

Design of Shallow Foundation

Settlement and stiffness considerations are usually secondary issues in the design of shallow foundations, although they can be of concern for the design of large gravity structures and permanent subsea foundations, and the discussion will focus on capacity.

Working Stress vs Limit State Design

API has historically used a lumped safety factor (i.e. working stress) approach with recommendations for safety factors against sliding (lateral) and bearing (vertical) failures. In API RP 2A 1st Ed. (1969) the safety factor on sliding failure was taken as 1.5 and was less than the safety factor against bearing failure, which was 2.0, because it was deemed that the consequences of a fixed platform (i.e. jacket) sliding during installation would be less than the consequences of a bearing failure under vertical load and overturning moment. These factors, which are independent of the type of load acting on the foundation, are still included in the latest API RP 2GEO standard.

Alternatively, the use of material factors in design guidelines was first introduced by Det Norske Veritas in DNV (1974) and this framework was adopted by the ISO 19901-4 (2003) standard on shallow foundations, which is therefore a load and *material* framework. When calculating the soil resistance (i.e. foundation capacity), the soil undrained shear strength and the tangent of the soil friction angle are divided by a material factor equal to 1.25. This factored resistance must then be greater than the external loads which are multiplied by a load factor ranging from 1.1 for gravity loads to 1.35 for environmental loads. The implications of these differences in framework will be explored later.

Calculation of Shallow Foundation Capacity

Prior to the first edition of RP 2A, shallow foundations in the Gulf of Mexico were designed primarily with the framework proposed by Skempton (1951). In API RP 2A 1st Ed. (1969) and the recommendations of Terzaghi (1943) were adopted. Later, starting with the RP 2A 10th Edition (1979), the framework proposed by Vesic (1975) was chosen over the Brinch Hansen (1970) recommendations and was still the recommended practice in API RP 2A 21st edition (2000).

During that time, the 1970s saw the rapid development of the North Sea basin. The early shallow foundation practice there, as used for the design of a large 90 m (300 ft) gravity-type storage tank in the Ekofisk field, was documented by Bjerrum (1973). The Brinch Hansen (1970) framework was adopted, with careful selection of the friction angle.

By the end of the 1970's, the UK Department of Energy, the Norwegian

Petroleum Directorate, Det Norske Veritas (DNV), and Lloyd's Register all had developed their own rules.

When ISO issued the first edition of its standard on foundation design, ISO 19901-4 (2003), the framework of Brinch Hansen was adopted. The API philosophy has since then been to align and merge its recommendations with those of ISO and to maximize the adoption of ISO standards (Wisch and Mangiavacchi, 2010). Therefore, when API issued the first edition of RP 2GEO, the bearing capacity framework recommended by Brinch Hansen was also adopted to maximize alignment of the two practices. The framework of lumped safety factors, rather than load and material factors, was nonetheless retained.

Remaining differences between ISO and API practices were summarized by Jeanjean et al (2010) and include, among others:

- *The removal of the effective cohesion, c' , term from the bearing capacity equations:* All geotechnical text books include a version of the general bearing capacity equation with both the soil effective cohesion, c', and the soil effective friction angle, ϕ'. In RP 2GEO, the equation for drained bearing capacity has deliberately omitted any component due to an effective cohesion, c', and the accompanying bearing capacity factor, N_c. The occasions when it might be appropriate to include a component for bearing capacity due to a presumed effective cohesion are believed to be rare.
- *Specification of triaxial friction angle when using RP 2GEO:* ISO 19901-4 (2003) recommends the use of plane strain ϕ' values, as per Brinch Hansen (1970). API decided to specifically recommend use of the triaxial values (typically taken as 10% lower than the plane strain values) primarily because this approach gave the closest agreement (for a given size foundation) between the allowable load envelopes that may be derived by ISO 19901-4 (2003) and RP 2GEO – i.e. use of triaxial friction factors helped to overcome some of the discrepancy that arises due to use of lumped safety factors in RP 2GEO, compared with material and load factors in ISO 19901-4 (2003). Additionally, performing plane strain tests is not standard industry practice.

Because offshore clay undrained shear strength profiles are often linearly increasing with depth, this design case is specifically addressed in codes and standards. The recommendations of Davis and Booker (1973) on how to account for linear increasing shear strength profiles in bearing capacity calculations are included in both ISO 19901-4 and API RP 2GEO. This typically results in lower capacities, when compared to past practices that used an "averaged" shear strength over various depths below the base of the foundation.

Comparison between API and ISO Practices

Differences in envelopes in V-H space (V: vertical load; H: Horizontal load) according to API and ISO practices are illustrated with two simple examples.

Envelopes are first developed for a surface mudmat (no embedment) with effective dimensions of 10 m by 10 m and founded on soil with constant s_u = 10 kPa (0.21ksf). The material factor was taken as 1.25 in the ISO 19901-4 (2003) calculations. The results are shown in Figure 12 below.

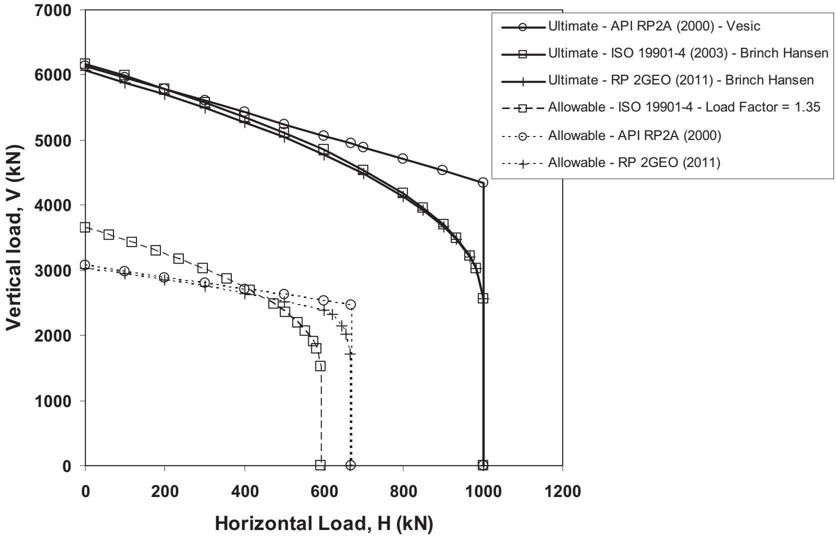


FIG. 12. Ultimate and allowable V-H envelopes for 10 m square mudmat founded on soil with s_u = 10 kPa (0.21ksf), according to API and ISO practices.

Comparable envelopes of ultimate capacity are generated using ISO 19901-4 and RP 2GEO. However, differences in factoring between WSD and load/material factor design approaches leads to some differences in allowable load. In comparing envelopes from ISO and API RP 2GEO for foundations on undrained soil, it is typically seen that:

- Foundations subject to predominantly vertical load will have higher allowable load when assessed using ISO 19901-4. This is because the combined effect of load and material factor (1.35 x 1.25= 1.68) is less onerous than the safety factor of 2.0 recommended by API for bearing.
- Foundations subject to predominantly lateral loads will have higher allowable load when assessed using RP 2GEO. This is because the combined effect of load and material factor (1.35 x 1.25= 1.68) is more onerous than the safety factor of 1.5 recommended by API for sliding.

Next, envelopes are developed for the same foundation now installed on seabed soils that will act as drained under the design loads. Figure 13 shows the results for a soil with a triaxial friction angle of 32 degrees. In comparing envelopes from ISO and API for foundations on drained soil, it is typically seen that:

- Because RP 2GEO uses the smaller triaxial friction angle and ISO 19901-4 uses the 10% larger plane strain friction angle, the ultimate load envelopes are significantly smaller when using RP 2GEO compared to ISO 19901-4.
- However, due to differences in the way safety factors in RP 2GEO and

load/material factors in ISO 19901-4 are applied, envelopes of allowable loads are similar from both codes.

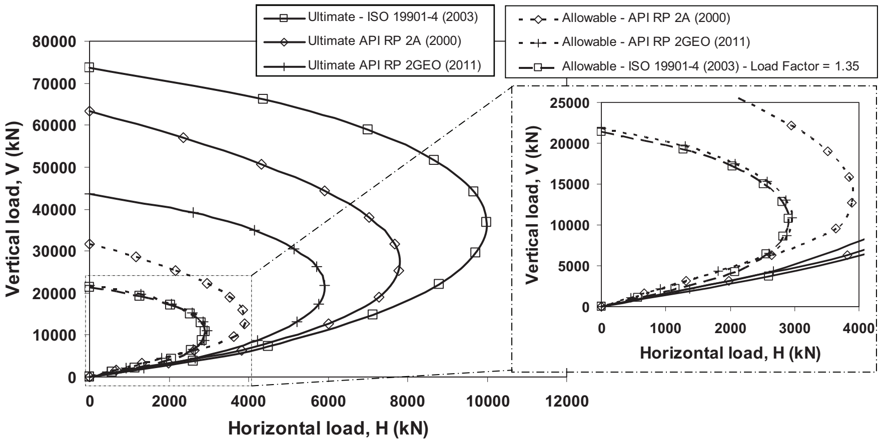


FIG. 13. Ultimate and allowable V-H envelope for a 10 m square mudmat founded on soil with triaxial $\phi' = 32^{o}$, according to API and ISO practices.

While not entirely consistent, the industry practice is overall well aligned. Calculations performed with API RP 2GEO and ISO 19901-4 will result in broadly comparable foundation size to resist a given set of unfactored (allowable) load. The greatest challenge to fully align worldwide practices resides in how to reconcile the lumped safety factor, working stress design (WSD), API framework with the partial load factor and material factor limit state ISO framework. The author believes that the most promising path forward is for both frameworks to adopt a load and resistance approach.

Introduction of State-Of-the-Art techniques into codes and standards

State-of-the-Art techniques for the design of offshore shallow foundations are described by Randolph (2012, this volume). They include so-called "yield surfaces methods" for the analysis of complex loading conditions. Gourvenec and Deeks (2011), in a report to API and ISO, compared these yield surface methods with the conventional design methods described above and with bespoke finite element analyses and illustrated the advantages of yield surface methods.

These state-of-the-art methods, as described by Gourvenec and Deeks (2011), are being introduced into industry practice by appearing in the informative Annex A of API RP2GEO (2011). Because Annex A is informative (and not normative), its use is considered to be good advice but is not mandatory in practice.

Driven Pile Foundation Design

Lumped safety factors vs limit state

In the API WSD framework, piles are designed against axial failure with a safety factor of 1.5 for extreme conditions and 2.0 for operating conditions (roughly equivalent to a 1-year load event).

Contrary to the ISO framework for shallow foundation, ISO uses a load and *resistance* factor framework for deep foundation. The pile ultimate capacity is calculated using unfactored characteristic values of soil properties. The factored capacity is obtained by dividing the ultimate capacity by a resistance factor of 1.25 for extreme conditions and 1.5 for operating conditions (ISO 19902). This factored resistance must then be greater than the external loads which are multiplied by a load factor ranging from 1.1 for gravity loads to 1.35 for environmental loads.

Therefore, although the design methodologies are similar (see Figure 11), the piles dimensions will be different, for a given design case, depending on whether API or ISO standards are used.

Piles Design in clays

Planning an Offshore Site Investigation

Faced with a specific design case, the geotechnical engineer must first determine the appropriate scope of work for the site investigation. Codes and standards do not prescribe a set number of borings or cores or in-situ tests to be acquired. Industry practice is to determine the appropriate level of geotechnical site investigation based on:

- the outcome of uUHR geophysical surveys and the resulting understanding and complexity of the surficial geology over the area of interest,
- the local installation experience for the chosen foundation type(s),
- and any local regulation requirements.

There is no "one-size fits all" answer for the number, location, and termination depths of the borings and cores to be acquired. Newlin (2003) reported that for the Na Kika platform, the soil borings are between 3 and 6 km away from the platform anchors, demonstrating that if the shallow geological setting is uniform and well imaged through uUHR surveys, the borings need not be close to the anchors. By contrast, Jeanjean et al (2006) reported that, for the Mad Dog spar, borings had to be located within 10 m of the anchors, due to extremely variable seabed conditions.

The first offshore in-situ test to be routinely used for clays was the remote vane (Doyle et al, 1971). It still remains popular in the Gulf of Mexico with some operators but has been increasingly replaced by the Cone Penetration Test (CPT). CPTs are typically performed in seabed mode as a continuous push up from the seafloor to depth of up to 40 m below seafloor and in downhole mode (i.e. inside a rotary boring) in a series of 3 m contiguous pushes.

Laboratory Testing - the SHANSEP framework

In its informative annex, API RP 2GEO recommends that the undrained shear strength be measured with UU triaxial compression tests, miniature vane tests, in-situ vane tests or CPT tests and also advocates the use of the SHANSEP framework (see Ladd and DeGroot, 2003, and DeGroot et al, 2010), where the shear strength of clays that exhibit normalized behaviors can be approximated by:

$$\frac{s_{uDSS}}{\sigma_v'} = S \cdot OCR^m$$

with s_{uDSS} being the shear strength measured in a DSS test, σ_v' the vertical effective stress, S the normally consolidated value of s_{uDSS}/σ_v', OCR the overconsolidation ratio, and m the strength increase component.

With the advance of deepwater activities, sample disturbance has generally increased in part because the soil samples, most of which contain some amount of biogenic gas, experience a large decrease in total stresses that can cause the gas to come out of solution, expand, and form cracks. Samples targeted for advanced laboratory tests such as the Direct Simple Shear (DSS) tests or consolidated triaxial tests are not extruded in the field but are kept in stainless steel thin-wall sampling tubes. Back in the laboratory they undergo x-ray radiography and the least disturbed portions of the samples is identified and selected for testing.

Design shear strength profiles are typically obtained for GoM clays with the use of DSS tests run and interpreted in the SHANSEP framework. The samples are consolidated to two to three times the in-situ vertical effective stress and unloaded to a chosen over consolidation ratio (OCR), which minimizes the effects of sample disturbance. Figure 14 and Table 2 show the probability density obtained from 346 DSS tests.

Table 2. Distribution of DSS Normalized Shear Strength Ratio for GoM Clays

Stress Range (kPa)	0 to 280	0 to 1100	0 to 4800
Nominal Equivalent Burial Depth (m)	0 to 50	0 to 150	0 to 650
Design Application	Shallow foundations and suction anchor	Driven Piles	Stability of large slopes; borehole stability during well drilling
Mean of Normal pdf	0.26	0.24	0.24
Standard Deviation of Normal pdf	0.027	0.029	0.030

The largest influencing parameter is the normal effective stress and the normalized strength decreases with an increase in normal stress. Little correlation has been found between normalized strength ratio and plasticity index, which ranges from 30 to 80 for the results in Figure 14.

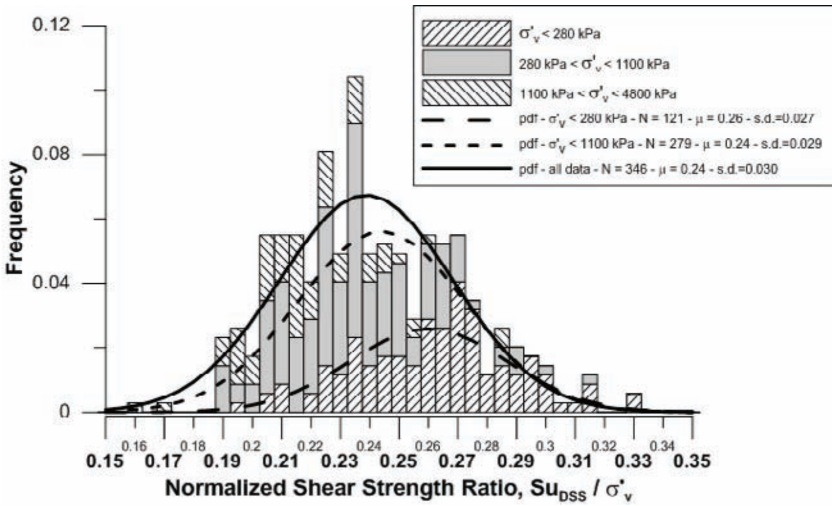


FIG. 14. Probability Density Function of Normalized Shear Strength Ratio measured in 346 DSS Tests on Normally Consolidated Gulf of Mexico Clays.

The SHANSEP framework is also used to provide strength for overconsolidated clays. Figure 15 shows data from 475 DSS tests run at OCR of up to 25. As noted by previous studies (i.e. Wroth, 1984) the best fit lines tend to show lower "m" factors as OCR increases.

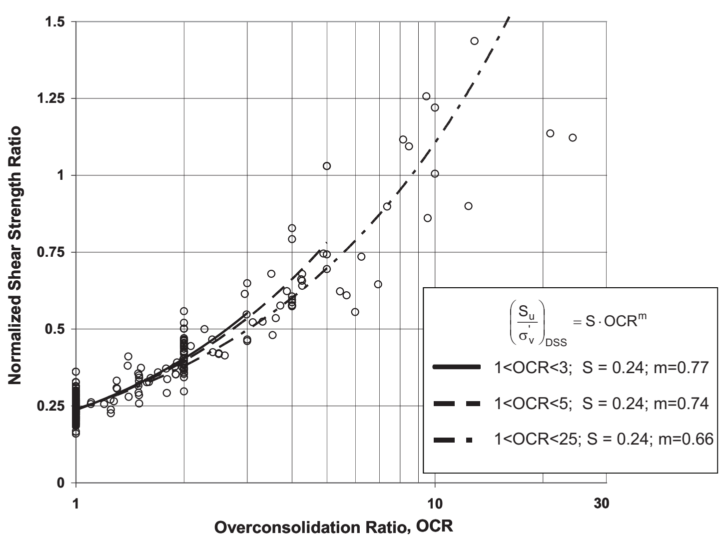


FIG. 15. Normalized strength ratio vs OCR for Gulf of Mexico clays, as measured in 475 DSS tests, and interpretation in the SHANSEP framework.

Limitations of the SHANSEP framework

While the Gulf of Mexico non-sensitive and non-structured clays do exhibit normalized behavior, clays in some other offshore areas do not. Le et al (2008) and Evans (2010) reported that the SHANSEP framework is not used offshore Egypt and is not recommended for West of Africa clays (Gulf of Guinea and Angola) which are highly structured, highly compressible, and sensitive. The "Sensitivity Framework" of Cotecchia and Chandler (2000) was adopted to provide design strength values and Le et al (2008) reported that high quality triaxial testing under confining stresses representative of the in situ stress level appears as the only way to provide geotechnical parameters that are representative of the in situ clay strength.

The API Total Stress Method

It has been long recognized that friction for piles in clays is controlled by the effective stress along the pile wall (Burland, 1973, Meyerhof, 1976). However, McClelland (1974) noted that total stress approaches had been favored in the Gulf of Mexico. The evolution of the API guidance, to which most offshore piles in clays are designed, can be found in Pelletier et al (1993). The current method is still a total stress approach and is a special case of the method proposed by Randolph and Murphy (1985), who recommended that the unit friction along a pile in clay, f_α, be calculated as follows:

$$f_\alpha = \left[\left(\frac{S_u}{\sigma'_v}\right)_{nc}\right]^{0.5} \times S_u^{0.5} \times \sigma_v'^{\,0.5} \text{ for} \left(\frac{S_u}{\sigma'_v}\right) \leq 1 \tag{1}$$

$$\text{and } f_\alpha = \left[\left(\frac{S_u}{\sigma'_v}\right)_{nc}\right]^{0.5} \times S_u^{0.75} \times \sigma_v'^{\,0.25} \text{ for} \left(\frac{S_u}{\sigma'_v}\right) \geq 1$$

with $\left(\frac{S_u}{\sigma'_v}\right)_{nc}$ being the normalized strength ratio for a normally consolidated soil.

API first implemented the above equation in the 17th edition of RP2A (1987) (see Figure 11), assuming that, for Gulf of Mexico clays, $(S_u/\sigma'_v)_{nc} = 0.25$. This choice of normalized strength is validated by the data presented on Figure 14 and allows Eqn. (1) to be reduced to:

$$f_\alpha = \alpha_{API} \times S_u \tag{2}$$

$$\text{with } \alpha_{API} = 0.5 \times \left(\frac{S_u}{\sigma'_v}\right)^{-0.5} \text{ for} \left(\frac{S_u}{\sigma'_v}\right) \leq 1 \text{ and } \alpha_{API} = 0.5 \times \left(\frac{S_u}{\sigma'_v}\right)^{-0.25} \text{ for} \left(\frac{S_u}{\sigma'_v}\right) \geq 1$$

The clays offshore West Africa exhibit quite different normalized properties. They are highly structured, with low effective unit weights, and as a result their DSS normalized strength ratio for normally consolidated conditions is higher than those of GoM clays (because of lower effective stresses) and typically ranges between 0.3 and

0.4. They are also sensitive, with sensitivity routinely as high as 10. They clearly do not fit the underlying $(s_u/\sigma'_v)_{nc}$ = 0.25 assumption in the API method and it is not clear if and how the API method should be modified for these clays. Industry practice is to nevertheless use the API α method as is because the high $(s_u/\sigma'_v)_{nc}$ ratio will be treated as if the soil were overconsolidated and low alpha values will be calculated, thereby providing a perceived conservative solution.

Effective Stress Methods

Criticism of the API total stress design method typically includes (e.g. Burland, 1993, Patrizi and Burland, 2001) lack of fundamental rigor, poor accuracy, and a high dependency of s_u on the test method (s_u is not unique for given soil).

In trying to address the above criticisms, it should be noted that the Randolph and Murphy method does have a rigorous framework and is consistent with the principle of a purely frictional bond between pile and soil (Randolph and Murphy, 1985). Next, the accuracy of the API method will be compared, for typical GoM conditions, to that of simple effective stress methods such as the β method, which is *not* used in design in GoM.

The β Method Applied to Gulf of Mexico Clays

The β method, in its simplest form (Burland, 1973, Patrizi and Burland, 2001), calculates the limiting pile shaft friction, τ_{sf}, as:

$$\tau_{sf} = \beta \times \sigma'_v \qquad \text{with} \qquad \beta = K_0 \times \tan\phi' \tag{3}$$

where K_0 is the coefficient of earth pressure at rest and ϕ' is the critical state angle of shearing resistance.

Meyherhof (1976) proposed to calculate K_0 as

$$K_0 = (1 - \sin\phi') \times OCR^{0.5} \tag{4}$$

Figure 16 shows the results of 121 K_0-consolidated triaxial compression tests (so-called Consolidated-Anisotropically-Undrainded-Compression (CAUC) tests) and compares the measured K_0 against values predicted by Eqn. (4), for OCR values between 1 and 5. The critical state friction angle is taken as the angle of friction at the maximum obliquity (i.e. when the ratio of the normalized shear stress over the normalized mean effective stress is maximized).

Linear regression analysis indicates that the predicted mean values are close to the measured values, albeit with a large scatter. Therefore, for normally to lightly consolidated GoM clays, combining Eqn. (3) and (4) gives:

$$\beta = K_0 \times \tan\phi' = (1 - \sin\phi') \times OCR^{0.5} \times \tan\phi' \tag{5}$$

As suggested by Burland (1993), the β values calculated by Eqn. (6) are plotted in Figure 17 against the normalized shear strength ratio, as measured in DSS tests.

The correlation is strong and β can be given by:

$$\beta = a + b \times \left(\frac{S_u}{\sigma'_v} \right)_{DSS} \tag{6}$$

with a = 0.14; b = 0.53 for $\left(\frac{S_u}{\sigma'_v} \right)_{DSS} < 0.5$ and a = 0.10; b = 0.68 for $\left(\frac{S_u}{\sigma'_v} \right)_{DSS} < 1.1$

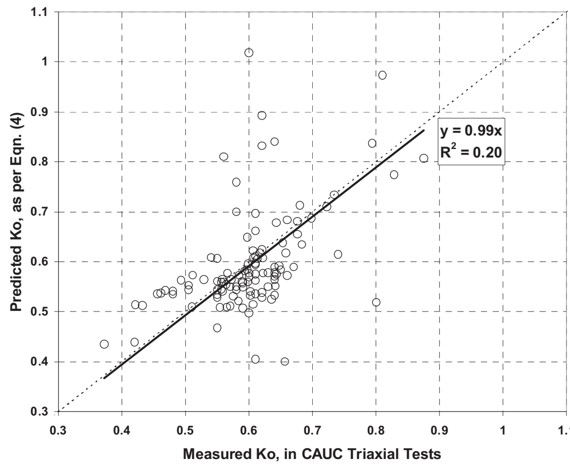


FIG. 16. Predicted K_0 values as per Eqn (4) vs measured values of K_0 in 121 K_0-consolidated triaxial compression (CAUC) tests.

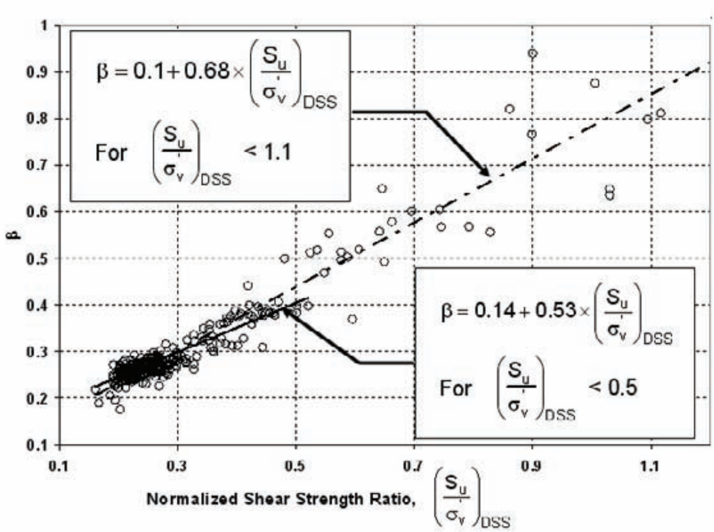


FIG. 17. β factor, as per Eqn. (6) vs DSS normalized shear strength ratio.

These values are higher than, but compare favorably with, those proposed by Patrizi and Burland (2001) ($\beta = 0.1 + 0.4\ Su / \sigma_v'$).

Pile Capacity with α and β methods for Gulf of Mexico Clays

The unit friction, f_β , along a pile can be estimated according to the β method by combining Eqn. (6) and the SHANSEP framework as follows:

$$f_\beta = \beta \cdot \sigma_v' = \left(a + b \cdot \left(\frac{S_u}{\sigma_v'} \right)_{DSS} \right) \cdot \sigma_v' = \left(a + b \cdot \left(S \cdot OCR^m \right) \right) \cdot \sigma_v' \tag{7}$$

The same unit friction according to the API α method, f_α , is, for $\left(\frac{S_u}{\sigma_v'} \right) \leq 1$:

$$f_\alpha = \alpha_{API} \cdot S_u = 0.5 \cdot \left(\frac{S_u}{\sigma_v'} \right)^{-0.5} \cdot \frac{S_u}{\sigma_v'} \cdot \sigma_v' = 0.5 \cdot \left(S \cdot OCR^m \right)^{-0.5} \cdot S \cdot OCR^m \cdot \sigma_v'$$

or $$f_\alpha = 0.5 \cdot \left(S \cdot OCR^m \right)^{0.5} \cdot \sigma_v' \tag{8}$$

The ratio of the two calculated unit frictions is therefore

$$\frac{f_\beta}{f_\alpha} = \frac{a + b \cdot \left(S \cdot OCR^m \right)}{0.5 \cdot \left(S \cdot OCR^m \right)^{0.5}} \tag{9}$$

and is plotted on Figure 18 with a = 0.14, b = 0.53, S = 0.24, and m = 0.77, which are best fit parameters for typical GoM driven pile design applications.

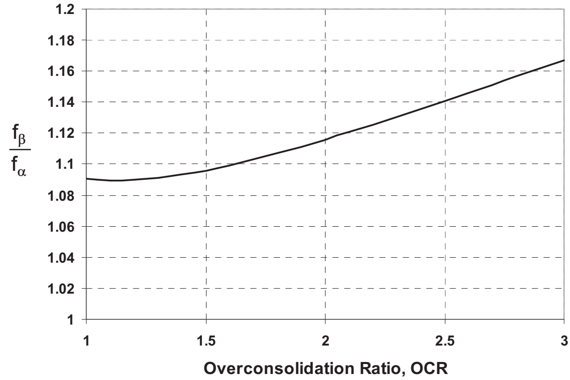


FIG. 18. Ratio of unit friction according to the β method, f_β, over unit friction calculated with the API α method, f_α .

The two methods give unit friction values within twelve percent of each other for OCR less than 2, which is a typical design condition, and within 18% for OCR less than 3. Therefore, for typical GoM clays, the API α method used by the industry gives results consistent with those from simple effective stress methods such as the β method, if the shear strength test method is the DSS test.

The above discussion is intended as a validation of industry practice in GoM and should not be construed as a criticism of the β method.

The Imperial College Pile (ICP) method

In the ICP method (Jardine et al, 2005a), the local shear stress along the pile at failure, τ_f, is calculated as:

$$\tau_f = 0.8 \cdot \sigma'_{rc} \cdot \tan(\delta_f) \tag{10}$$

with σ'_{rc} being the local radial stress after equalization (see Jardine et al, 2005a for details), and δ_f an angle between the peak and ultimate interface angles of friction, which are to be measured in interface ring shear tests.

In addition, the unit end-bearing is taken as proportional to the cone tip resistance, with the proportionality factor depending on plugging and drainage conditions.

The industry is sometimes faced with clay properties clearly outside those in the API pile load tests database. An extreme case of such conditions is found West of the Shetland Islands. Figure 19 compares the range of shear strength and normalized shear strength ratio encountered at the site described by Aldridge et al (2010) with those of the API load test database and the Chow (1996) load test database, which was used in the development of the ICP method.

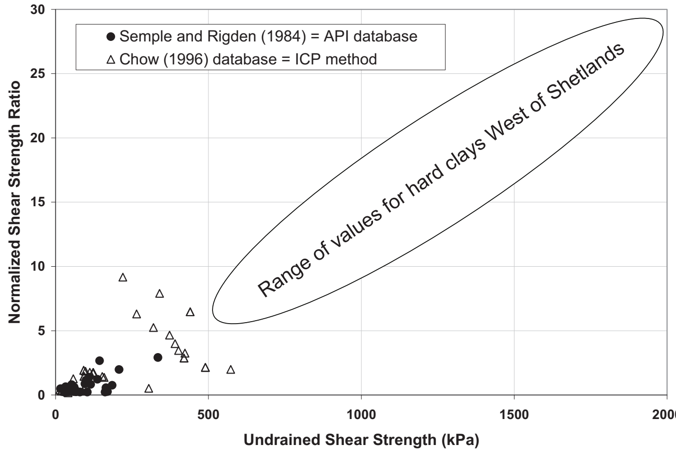


FIG. 19. Normalized shear strength ratio for selected clays offshore West of Shetlands Islands.

Aldridge et al (2010) reported that the above ICP method for clays was used at the site with specialized ring shear tests performed in the laboratory. Overy et al (2007) also reported the successful use of the ICP method for hard clays at nine North Sea platform sites.

Piles design in sands

The historical development of the API RP 2A design method is well documented by Pelletier et al (1993) and Jeanjean et al (2010). The reader will find extensive details of the studies that led to the API recommendations and a summary of historical and current practice in the Gulf of Mexico and in the North Sea. Important points are summarized herein.

The API and ISO "Main Text" method

In the main text (as opposed to the appendices) of API RP 2GEO and in the normative section of ISO 19902, the shaft friction, f, in sands is calculated (and has been since the 1st Edition of RP 2A in 1969) by the following equation, which is known as the "Main Text" method:

$$f = K \times \sigma_v^{'} \tan \delta \tag{11}$$

where K is the coefficient of lateral earth pressure, $\sigma_v^{'}$ is the vertical effective stress, and δ is the angle of soil friction *on the pile wall.*

Design practices have been and are still mostly aligned around the world, except for the choice of K. API RP 2GEO and ISO 19902 both recommend a K value of 0.8 for both compression and tension and this value has been used in the Gulf of Mexico over the past 25 years. However, K values of 0.7 in compression and 0.5 in tension (as per the recommendations of RP 2A 1st Ed., 1969) are still often applied for North Sea projects and are the values recommended by DNV (1992) and Lloyd's Register (1989) and (2008). Hobbs (1993) also reported that the API limiting values, both for friction and end bearing, are sometimes increased on the basis of high-quality Cone Penetration Tests (CPT) records. The general consensus based on recent pile load tests (Euripides and Ras Tanajib II, see discussion below) is that API RP 2A is very conservative regarding the friction in dense to very dense sands. Therefore K values of 0.8 for both compression and tension are often also considered for North Sea projects for which the high density is confirmed by CPTs.

Studies and Load Tests from the past 20 years

Numerous studies were undertaken in the last 20 years to address the capacity of piles in sand and have been summarized by Jeanjean et al (2010). Work that directly impacted the current API and ISO recommendations and industry practice includes:

- Studies by the Offshore Technology Research Center (OTRC) and Fugro McClelland (see Hossain and Briaud, 1993, and Fugro-McClelland, 1994).

- The EURIPIDES pile load tests (1995) (see Zuidberg and Vergobbi, 1996, Fugro Engineers, 2004, and Kolk et al, 2005).
- Studies and load testing by Imperial College and development of the ICP method (see Jardine and Chow, 1996, Jardine et al, 2005a, 2005b).
- The Ras Tanajib I and II pile load tests (1996-1999) (see Helfrich et al, 1985, Al-Shafei et al, 1994, and Fugro Engineers, 2004).
- API-sponsored Fugro Engineers Pile Study (see Fugro Engineers, 2004) and development of the Fugro-05 design method.
- Studies at the Norwegian Geotechnical Institute (NGI) from 1995 to 2005 and development of the NGI-05 design method (see NGI, 2001, and Clausen et al, 2005).
- Studies at the University of Western Australia (UWA) and development of the UWA design method (see Lehane et al, 2005).

The above studies consistently demonstrated that the API RP 2A (2000) main text method exhibits biases with soil density and embedment ratio (see Figures 20 to 23). It underpredicts the capacity of relatively short piles ($L/D < 40$), particularly in tension, and piles in dense to very dense sands and overpredicts the capacity for relatively long piles ($L/D > 40$) and pile in loose sands, with L being the embedded pile length and D the pile diameter.

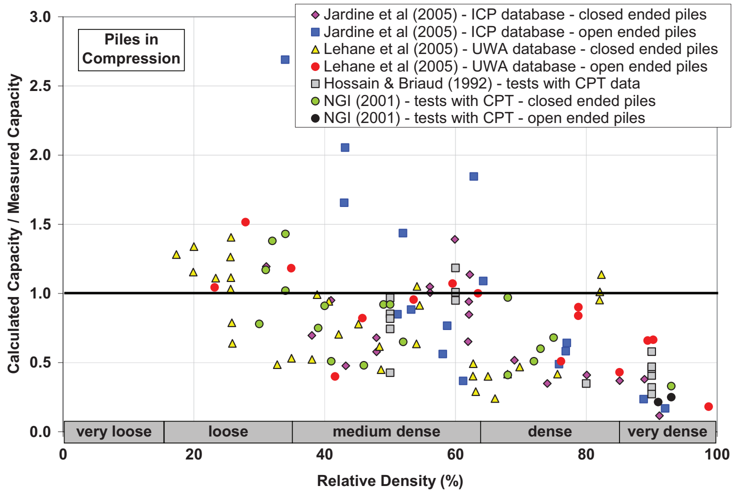


FIG. 20. Ratio of Calculated-to-Measured pile *compression* capacity, according to the API main text method, as a function of sand relative density.

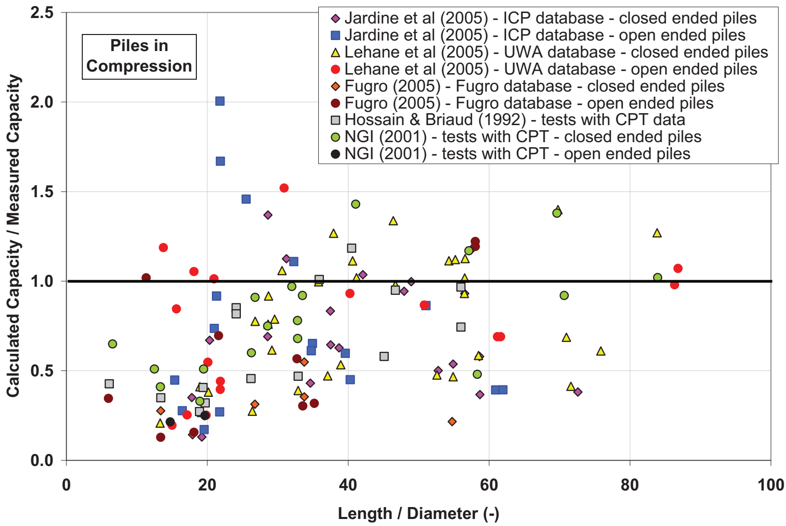


FIG. 21. Ratio of Calculated-to-Measured pile *compression* capacity, according to the API main text method, as a function of pile Length-to-Diameter ratio.

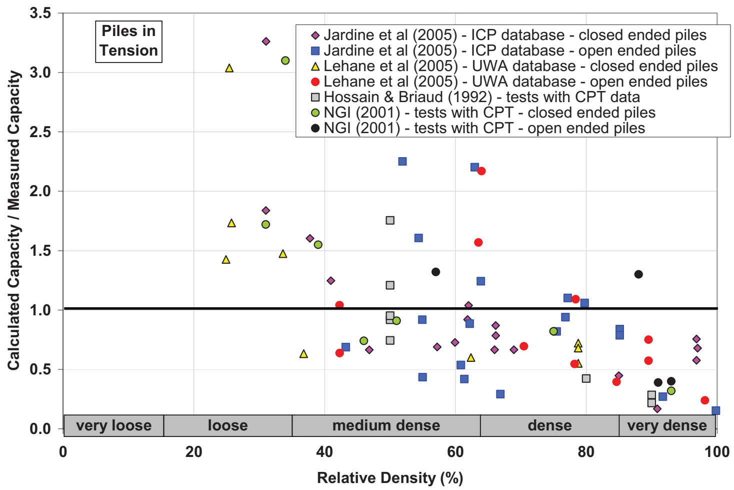


FIG. 22. Ratio of Calculated-to-Measured pile *tension* capacity, according to the API main text method, as a function of sand relative density.

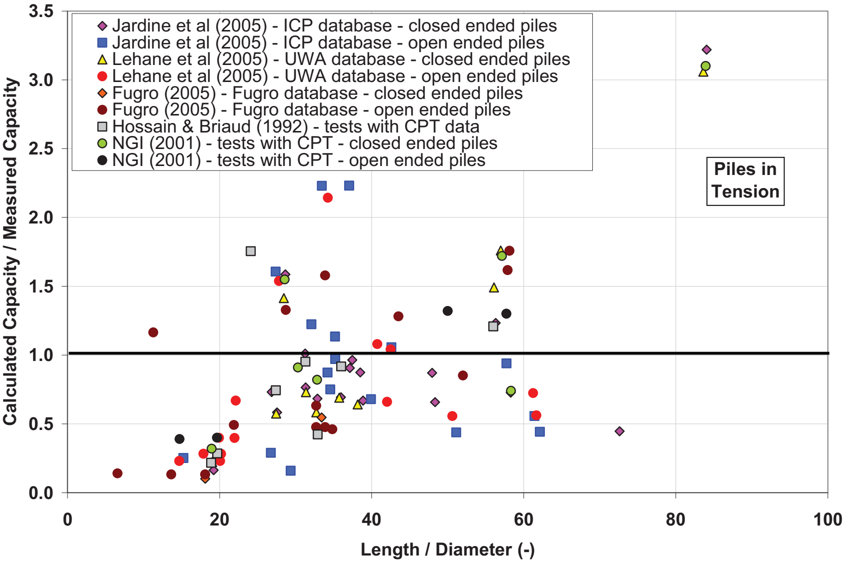


FIG. 23. Ratio of Calculated-to-Measured pile *tension* capacity, according to the API main text method, as a function of pile Length-to-Diameter ratio.

Recent Updates in API and ISO recommendations

In view of the above studies and test results, important changes were subsequently made to both API and ISO codes, thereby ensuring a consistent and improved practice worldwide (see Jeanjean et al, 2010, for further details).

Modification to the "main text" method

The documented conservatism in the main text method for piles in dense to very dense sands is consistent with the API and ISO positions that the API method is a design method and not a prediction method and provides an estimate for prudent design based on the (best) available experience. The track record of the method has been good, although there is probably only a fraction of the API-designed foundations that have been subjected to their design loads. Therefore the method has remained unchanged for dense to very dense sands. Nevertheless the "K·tan δ" term was replaced by its product "β = K·tan δ" and only recommended values for β are presented in RP 2GEO to avoid the documented mistakes (OTRC, 2009) by some practitioners in the choice of K and confusion between the angle of sand internal friction and the angle of friction at the pile-soil interface.

However the documented unconservatism, as summarized in Figures 20 to 23, was of concern and, as a result, the API main text method is no longer recommended for very loose and loose sands, loose sand-silts, and medium dense silts, and dense silts. Four CPT-based methods are recommended instead in RP 2GEO and ISO 19902:

- the Simplified-ICP method, which is a simplified version of the ICP method in Jardine et al (2005a) where the coefficients have been rounded off and the favorable dilatancy term has been ignored,
- the Fugro-05 method (as per Fugro Engineers, 2004)
- the NGI-05 method, (as per Clausen et al, 2005)
- and the Offshore-UWA methods (an adaptation of the method in Lehane et al, 2005 for large diameter offshore piles where the favorable effects of the pile driving partially plugged and the increase in radial stress during loading have been ignored).

Further discussion of the methods can be found in Jardine and Chow (2007) and Schneider et al (2008), and Jeanjean et al (2010).

Designers should be aware that areas of potential unconservatism still remain in the RP 2GEO and ISO 19902 main text method. In comparison to all CPT-based methods, the method may not be conservative in tension for thick deposits of medium dense sands (Figure 22). Fortunately, this is a rare design situation.

CPT-based design methods

Both RP 2GEO and ISO 19902 clearly state in the main text that:

> *"In comparison to the Main Text method (...) the CPT-based methods are considered fundamentally better, have shown statistically closer predictions of pile load test results and, although not required, are in principle the preferred methods."*

Designers around the world are therefore clearly encouraged to maximize the use of CPT-based methods, for *all* silica sand relative densities. These methods have all been shown to reduce scatter, skewing and bias when compared to the API method.

As discussed in Jeanjean et al (2010), the reason why these methods appear in the annexes of API and ISO codes and standards is that industry currently cannot agree on a single preferred method.

Important aspects of these CPT methods include:

For compression loads:

- All CPT methods predict higher friction close to the pile tip than the main text method limiting values.
- The end-bearing values are fairly consistent between methods.
- The pile capacity is higher with the CPT methods than with the API main text methods for short pile penetrations (L/D < 40) but the bias becomes less clear as the pile penetration increases.

For tension loads:

- CPT methods generally predict higher friction than the main text method for short pile penetrations (L/D < 40).
- CPT methods generally predict lower capacity than the main text

method for long pile penetration (L/D > 40).

The use of the above CPT methods will therefore not automatically result in capacities higher than those calculated with the API main text method. In order to take full advantage of CPT methods and to avoid cone refusal in very dense sand layers, industry practice increasingly includes the use of 5cm^2 cone systems with tip refusal pressure of 100 MPa (as opposed to the industry standard 10 cm^2 cone with 50 MPa maximum tip pressure). The need to have closely spaced CPT records near the expected pile tip penetration is also more scrutinized in order to properly identify potential thin clay layers which would be detrimental to end bearing and high friction values close to the pile tip.

Industry Practice

Based on published literature to date, the Simplified-ICP method appears to be the preferred CPT-based method although some designers use an arithmetic average of the above four CPT methods to calculate pile capacity, while others are cautiously using the lowest bound estimate.

Overy (2007) documented the use of the ICP method for nine platforms installed in the North Sea in loose to very dense sands. The CPT-based methods are also increasingly used for platform re-assessment in the North Sea, Gulf of Mexico and offshore Trinidad. Achmus and Muller (2010) compared the methods for large-diameter offshore piles for renewable energy applications and concluded that the ICP and UWA methods appear the most applicable for such applications.

Extrapolating outside the database

The length, diameter, and ultimate capacity of offshore piles can often be much larger than those in load tests databases (Bond et al, 1997, Jardine, 2009). Figure 24 shows the characteristics of the piles in the Semple and Rigden (1984) and the Chow (1996) tests databases and compares them with the *calculated* ultimate capacities of large piles for compliant towers and TLPs. Most of the piles in the databases are less than 1 m in diameter, less than 40 m in length, and have a measured ultimate capacity less than 8 MN. By contrast, large offshore piles can be up to 2.5 m in diameter, 160 m in length, with *calculated* ultimate capacity exceeding 120MN.

For TLPs, industry practice, as defined in API RP 2T (2010), for the design of such large piles is to use the API α method but to cautiously increase the minimum recommended API RP 2A safety factor of 1.5 to 2.25. In addition, the piles have typically experienced only partial setup when production starts because it may take up to 1 year for the pile to experience full set-up. As a consequence, safety factors under fully set-up conditions typically exceed 2.7.

For compliant towers, Will et al (2006) reported that the Benguela-Belize pile foundations had a safety factor of 1.75 for extreme conditions and 2.35 for operating conditions.

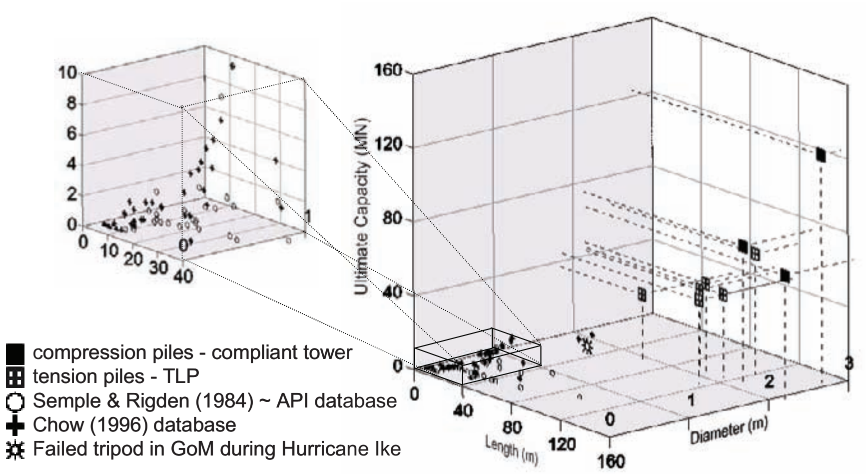


FIG. 24. Length, diameter, and *calculated* ultimate capacity of large offshore piles for compliant towers and TLP compared with those of piles in databases of load tests in clays.

THE OPERATE STAGE

During operations, the platform may experience environmental loads in excess of design events. Learnings from foundation field performance during hurricanes are now discussed.

Numerous investigators (e.g. Aggarwal et al., 1996, Horsnell and Toolan, 1996, Bea et al., 1999, and Energo, 2006, 2007) have well documented that driven pile foundations of fixed offshore structures have performed much better than expected during events such as Hurricanes Andrew (1992), Roxanne (1995), Lili (2002), Ivan (2004), Katrina (2005), Rita (2005), and Ike (2008). Although hundreds of platforms were destroyed, documented cases of foundation failure are rare. This fact has been traditionally explained by conservative biases in pile capacity. Jardine (2009) also pointed to unaccounted for pile capacity growth with time to account for the low failure incidence rate. Additionally, Najjar and Gilbert (2009) highlighted that the existence of lower-bound capacities can significantly increase the calculated reliability of the foundation. To understand whether conservatism truly exists in geotechnical industry practice, OTRC (2009) compared the predicted and observed performance of thirteen fixed structures where the foundation was loaded to near or beyond its capacity.

Gilbert et al (2010) summarized the findings and noted that field performance is consistent with expectations based on design. The survival of the platforms could be explained by structural arguments, without invoking any increase in pile capacity from API practice. In summary, Gilbert et al (2010) concluded that although conservatism may exist in design methods (particularly in sands), it cannot be demonstrated by the better-than-expected platform survival performance during hurricanes.

In addition, the only foundation failure documented by Gilbert et al (2010) consists of an axial pile pull-out in an all-clay profile on a tripod structure during Hurricane Ike (see Figure 24). A very significant finding of the study was that the calculated capacity of the pile matches very well the hindcast load at the pile head, using the API recommendations and performing proper load-displacement analyses (so-called t-z analyses) accounting for the softening of the axial load transfer curves. In addition, considerations such as beneficial rate effects and detrimental cyclic loading effects did not seem to have a noticeable net effect on pile capacity.

The author believes that the above study constitutes a strong validation of industry practices for pile design in GoM clays.

CONCLUSIONS

Mitchell (2009) stated that:

> *"(...) the greatest advances in geotechnics have come from (...) the need to undertake projects in uncharted environments such as the Arctic, the oceans, and space where there is little or no experience but the risks are high (...)"*

The paper showed how deepwater projects in geohazardous areas embody this pioneering spirit and described the typical steps in geohazard evaluation, starting with high-resolution geophysical surveys and concluding with quantitative risk assessment analyses.

The latest contributions of the offshore industry to driven pile design, through the sponsoring of key load tests and studies, and the resulting development of CPT-based design methods have been described. For clays typical of Gulf of Mexico conditions, the adequacy of the API total stress method has been validated through analyses of laboratory soil behavior and review of foundation field performance.

Last, the paper emphasized that the industry has made great improvements in aligning its shallow foundation and pile design practices through the consistent use of API and ISO standards.

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State of Practice: Excavations in Soft Soils

Demetrious C. Koutsoftas,[1] M.ASCE, P.E., G.E.

[1]Principal DKGC, Inc., 60 Joost Avenue, San Francisco, CA 94131, deme@dkgeotech.com.

ABSTRACT: This paper examines the state of practice of the design and construction of excavations in soft soils. The review is based primarily on the author's experience, published case histories, and discussions with colleagues involved in the design and construction of excavations in soft soils. It is not a survey of current practices, because such a survey would not be practicable, or even useful, because of many variations in local practices. The paper examines major milestones in the development of the design and construction of excavations in soft soils, and reviews recent developments and trends in design and construction practices. Traditionally, the design and construction of excavations is handled contractually as a design-build contract, with the responsibility for the design and construction of the shoring system delegated to the contractor. However, new developments in analytical design tools and construction methods, as well as wider recognition of the potential legal consequences of the impacts of excavation-induced deformations on existing structures, and the need to address these issues in advance of construction, is leading to new approaches in the design and construction of excavations in general, and for soft soils in particular. Recent innovations in construction methods, such as the introduction of deep-soil mixing and jet grouting, are providing greater flexibility in the design and construction of excavations for deformation control. This paper reviews these developments and summarizes data from case histories to illustrate practical application of these new methodologies and construction techniques for the design and construction of excavations in soft soils.

INTRODUCTION

The design of excavations is fundamentally a soil-structure interaction problem. However, up until recently, the complexity of the problem and limitations in analytical capabilities necessitated simplifying assumptions to be made that would allow designing shoring systems based on structural considerations, with geotechnical input limited to the earth and water pressures for which the shoring system would be designed. Goldberg et al. (1976) provide a comprehensive summary of traditional design and construction practices for lateral support of excavations, with many design examples and practical considerations. Although the basic design concepts and practical considerations have not changed significantly in the last 25 years, there have been major advances in analytical capabilities and construction methods that make possible new approaches to the design and construction of excavations in soft soils. The new approaches include design of excavations for deformation control, rather than just designing the shoring system to safely support the surrounding ground retained by the shoring system. They also allow innovative contractual approaches that lead to more equitable allocation of risks, and better control of the design and construction process by the owner.

Excavations in soft soils present the greatest challenge because of the large lateral loads that need to be supported by the shoring system, and because of the potential for large lateral deformations and their impacts on existing adjacent structures. Because of the potential for large deformations, excavation in soft soils in urban areas generate significant public involvement; and in particular, greater participation by stakeholders, who often demand that the impacts of excavations on their structures be addressed in advance of construction. The new realities brought about by public participation are forcing project owners and their designers to reconsider the traditional approaches and contractual practices. As a matter of practical necessity, the project owners and their designers have to address ground deformations caused by the proposed excavations and their impact on the adjacent structures before contract award, and it is no longer practicable or effective to delegate that responsibility to the contractor. Preconstruction and post-construction surveys of adjacent structures are becoming necessary. These requirements lead to more comprehensive designs, better instrumentation and monitoring programs, and more comprehensive documentation of construction processes and the impacts of construction on adjacent structures. This paper reviews the new trends that necessitate a holistic approach to the design and construction of excavations.

The paper begins with a historical review of the major developments in the design and construction of excavations to provide historical perspective. This is followed by a review of some of the basic principles that govern the behavior of excavations in soft soils, followed by a discussion of the current state of practice; and concludes with a review of current trends, and projections for future developments.

HISTORICAL PERSPECTIVE ON DESIGN AND CONSTRUCTION DEVELOPMENTS

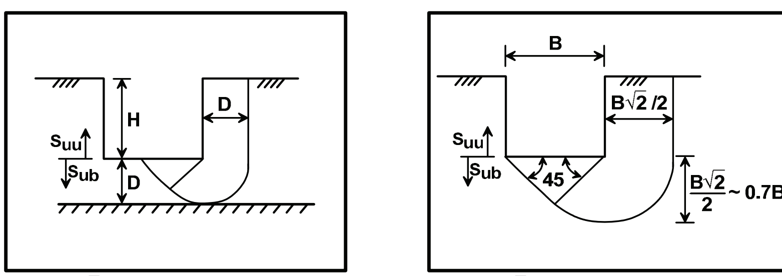
This section reviews important milestones in the development of design and construction practices for excavations in soft soils.

Design Developments

Terzaghi (1943) presented a method for analysis of the stability of the base of excavations in clay. Although the significance of this method did not become fully apparent to the profession in general for many years, it is now recognized that base stability is the most important factor in the design of excavations in soft soils. Strut loads and excavation-induced deformations are critically dependent on base stability.

Up until 1948, estimation of strut loads and the design of shoring systems were based on lateral pressures estimated on the basis of Rankin's theory of earth pressures. The publication by Terzaghi and Peck (1948) of the apparent earth pressure diagrams for estimating strut loads, and the recognition that strut loads were generally higher than what could be estimated based on Rankin's theory, is one of the most important milestones in the design of excavations. The Terzaghi and Peck (T&P) apparent pressure diagrams for soft clays were modified later by Terzaghi, Peck, and Mesri (1996), based on recommendations made by Henkel (1972), and remain as the basis for estimating strut loads for the design of braced excavations.

Bjerrum and Eide (1956,) presented improvements to Terzaghi's (1943) base stability analysis, and include bearing-capacity factors, Nc, for analysis of base stability of excavations in soft clays. The improvements include direct account in the Nc values of the effects of excavation size and geometry in terms of the length-to-width (L/B), and depth-to-width (H/B) ratios. Figure 1 summarizes the two methods.



(a) $D < (\sqrt{2}/2)\,B$ *(b) $D > (\sqrt{2}/2)\,B$*

FACTOR OF SAFETY AGAINST BASAL HEAVE		
CASE	*(a)*	*(b)*
Terzaghi 1943	$FS = \frac{1}{H} \times \frac{N_c S_{ub}}{\gamma - S_{uu}/D}$	$FS = \frac{1}{H} \times \frac{N_c S_{ub}}{\gamma - 2S_{uu}/\sqrt{2}\,B}$
Bjerrum and Eide 1956	$FS = \frac{1}{H} \times \frac{N_c S_{ub}}{\gamma}$	

FIG. 1. Methods of Analysis of Basal Stability in Soft Clays

Although the primary objective of the shoring system design is to control deformations of the retained earth, it was not possible to obtain accurate and complete measurements of excavation-induced deformations until the introduction of the slope inclinometer (slope indicator) by Shannon and Wilson (1958). Inclinometer measurements became widespread very quickly and provided information that allowed better understanding of the behavior of excavations. Inclinometer measurements showed that the major portion of lateral deformations was in the soil below the base of the excavations, which led to the realization that traditional structural analyses for estimating the deformations of the shoring system could not account for these deformations, and therefore were of limited value, because they were not representative of actual movements.

Peck (1969) used inclinometer measurements to compare the volume of soil involved in the lateral deformations with the volume of soil represented by the profiles of surface settlements, and documented the close relationship between the two. Peck (1969) also summarized the results of settlement measurements for many case histories that show how the settlements, δ_v, normalized with respect to the depth of the excavation (δ_v/H), vary with distance from the excavation, with soil type, base stability, and workmanship. The largest deformations were associated with excavations in soft clays, particularly those with high-stability numbers N ($N = \gamma H/Su$; γ is the total unit weight of overburden soils, H is the depth of excavation, and Su is the undrained shear strength of the soil), and with significant depth of clay below the base of the excavation. Although Peck (1969) did not provide charts for estimating lateral deformations, his observation that the volume of soil represented by the lateral deformations was approximately equal to the volume of soil represented by the surface settlements could be used to make approximate estimates of lateral deformations, as well.

In the subsequent 20 years, after the publication of Peck's (1969) normalized settlement profiles, there was intensive research in developing numerical models for estimating excavation-induced deformations, and in developing a better understanding of the behavior of excavations in soft soils. The report by Goldberg et al. (1976) summarizes some of the research efforts, including preliminary charts for estimating lateral deformations, and the recognition of the importance of system stiffness on excavation-induced deformations. Although the literature includes numerous papers and research reports presenting the results of numerical analysis of excavations, it was not until the publication by Clough et al. (1989) of their method for estimating excavation-induced deformations that a simple and practical methodology became available for estimating excavation-induced deformations. The most important contribution of the paper is the establishment of a numerical relationship between normalized maximum wall deformations (δ_h/H), the system stiffness, and the factor of safety against basal heave. The system stiffness is defined as $S = EI/\gamma_w h^4$ (E is the Youngs modulus of the wall, I is the moment of inertia per unit length of the wall; h is the average strut spacing, and γ_w is the unit weight of the water), and it is similar to the definition of system stiffness used by Jaworski (1973) in his assessment of the effects of system stiffness on stability. Figure 2 shows the chart developed by Clough et al. for estimating lateral deformations of excavations.

Clough and O'Rourke (1990) present a comprehensive summary of methods of estimating excavation-induced deformations. They review the Clough et al. (1989) method in detail, and also present updates of envelopes of the normalized settlement profiles presented initially by Peck (1969). Figure 3 summarizes settlement data that were the basis for the Clough and O'Rourke (1990) envelope of settlements for excavations in soft to medium clays. It must be recognized that the envelopes recommended by Clough and O'Rourke do not provide information for estimating actual settlement profiles for specific cases; they simply indicate that in the majority of cases, the settlements would be within the area bounded by the envelope. Actual settlement profiles can be very different from the envelope, and therefore these envelopes should not be used to estimate profiles of settlements for analysis of the impacts of excavation-induced settlements on adjacent structures.

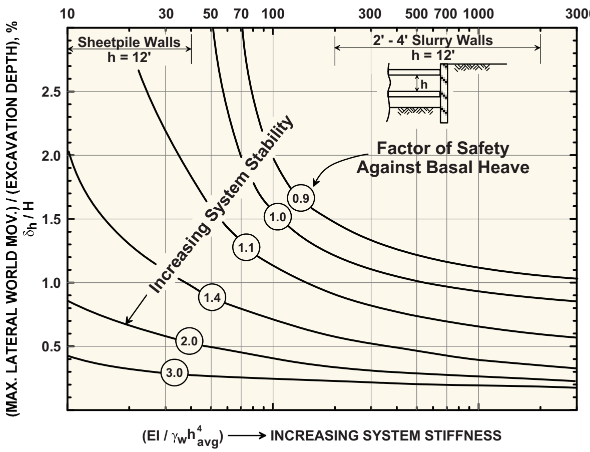


FIG. 2. Chart for Estimating Lateral Movement (and Settlements) for Support Systems in Clays (Clough et al., 1989)

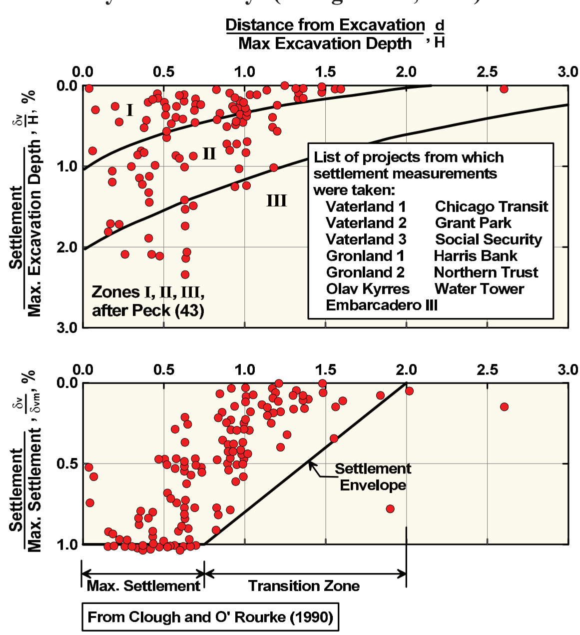


FIG. 3. Summary of Measured Settlements Adjacent to Excavations and Normalized Envelope Proposed by Clough and O'Rourke (1990) for Soft to Medium Clays

In the last 20 years, a very large number of technical papers and reports have been published presenting the results of theoretical studies and empirical measurements aimed at refining the settlement profiles and envelopes proposed by Clough and O'Rourke (1990). The results of these studies showed that the settlements immediately behind the shoring wall may be significantly smaller than the envelope values represented by the Clough and O'Rourke (1990) envelope. Hsieh and Ou (1998) and Kung et al. (2007) summarized some of these developments.

Efforts to develop numerical models for analysis of excavations have been ongoing since the early 1970s (Jaworski 1973, Clough and Tsui 1974, Clough and Mana 1976). Commercially available computer programs that are sufficiently robust for practical applications have become available relatively recently; and although they represent considerable improvements over earlier versions, still require considerable expertise, and their use for practical applications has been rather limited. Use of these programs is, however, becoming more widespread, as experience is gained from their gradual, but expanding, use for practical applications. The greatest weakness of currently available programs is in the definition of the constitutive model for representing the stress-strain behavior of soft soils. The development of the constitutive model requires considerable manipulation of the results of conventional laboratory tests; and in general, there is much less emphasis in correctly performing the necessary laboratory tests than in the analysis.

Construction Developments

Up until the early 1960s, construction methods for braced excavations were limited to soldier piles and lagging, and steel sheet piles for constructing the shoring walls; and cross lot struts, or berms and rakers for lateral support. Dewatering was an essential element for most excavation projects, because it was often impractical to advance the shoring wall deep enough to create a positive cut-off against underseepage. However, the inevitable settlements resulting from compression of soft soils due to groundwater lowering have been a source of problems for many projects.

The introduction of diaphragm (slurry) walls in Europe in the early 1950s (Tamaro 1975), in combination with development of more powerful construction equipment, were major developments in excavation support methods. The improvements in wall stiffness and strengths and the ability to extend diaphragm walls to great depths were major leaps forward in improving base stability, reducing underseepage, and the need for dewatering, and for better control of excavation-induced deformations. Diaphragm walls were introduced in the United States in 1962, and became popular very quickly because of the greater flexibility in the design of the shoring system. A large number of case histories have been published in the last 50 years, including proceedings of workshops (D'Apollonia et al. 1974), Federal Highway Administration reports (1979), and a book on slurry wall construction (Xanthakos 1979). Tamaro (1975) provides a comprehensive summary of early applications, opportunities, and challenges with the method. The Institution of Civil Engineers (UK) organized a specialty conference on "Diaphragm Walls and Anchorages" in 1974, which brought together experiences from around the world, and was an important step in expanding the use of slurry walls for excavation support.

While diaphragm wall applications enjoyed great success in Europe and the United States, in Japan, research focused on the development of in situ mixed systems, including the cement deep-soil mixing (CDSM) method and jet-grouting (Welsh 1992). The CDSM method was introduced in the United States in the mid-1980s (Jasperse and Ruyan 1987), and started to compete effectively with diaphragm wall applications for shoring support. The first major application of CDSM in the United States was for a 45-foot-deep excavation for a large underground storage basin at a treatment plant facility in the Bay Area (Koutsoftas 1999). Since then, there has been a steady increase in CDSM applications for excavation support as reported, among others, by Taki and Yang (1991), Yang (1997), Yang et al. (2001), and Yang (2003). Some of the most recent advances include the Cutter Soil Mixing (CSM) method (Kvinland et al. 2010), and the Trench Remixing Deep (TRD) method (Garbin et al. 2010). Steel soldier piles inserted in the in situ soil-cement mix, while it is still soft, provide the necessary strength and stiffness for the shoring wall.

The construction industry has seen a tremendous increase in the application of tiebacks, soil and rock anchors, soil nails, and other similar techniques for lateral support of excavations. The advantages of these methods include speed of construction, economy, and the elimination of internal supports, which leave the excavation free for the contractor's use. However, tiebacks and other similar anchorage techniques have limited applicability in soft soils because of the inability to develop economically in the soft soils the high load-capacities that are needed to support the large lateral loads.

Jet grouting (Welsh 1992) is an innovation introduced in the United States in the early 1990s, and has great potential as a ground improvement technique for excavation support. A number of case histories, along with the advantages and limitations of the method, are reviewed in the next section.

BASIC DESIGN CONSIDERATIONS

The state of practice reflects, to a large degree, the state of our understanding, or lack thereof, of the behavior and the factors that affect/control the performance of excavations. Therefore, as a prelude to the discussion of the state of practice, it is essential to review the current state of understanding of the behavior of excavations in soft soils.

There are a number of important factors that affect the behavior of excavations in soft soils that need to be clearly understood in order to produce a safe and economical design. They include: (1) basal stability and its effects on strut loads and excavation-induced deformations; (2) stability of the base against hydrostatic uplift; (3) excavation-induced deformations and their potential for adversely impacting adjacent structures; (4) sources of ground deformations from construction activities other than the actual excavation and bracing processes; (5) control of groundwater; and (6) recent construction innovations in excavation support and ground improvement.

Base Stability

The performance of excavations in soft soils is controlled primarily by the stability of the base of the excavation. The strut loads, bending moments in the wall, and the excavation-induced deformations are affected by the stability of the base. The factor of safety of the base (Bjerrum and Eide 1956) is a good index of the expected performance of the excavation. The stability number, $Nb = \gamma H / Su$, is often used in lieu of the factor of safety as a criterion for base stability; where γ is the average total unit weight of the overburden soils, H is the depth of the excavation, and Su is the undrained shear strength of the soft soils below the base of the excavation (Peck 1969). If surcharge loads are present, the surcharge pressure is added to the overburden stresses to calculate the factor of safety or the stability number. The factor of safety is estimated following either the Bjerrum and Eide (1956) or the Terzaghi (1943) method. The two methods and applicable equations are illustrated on Figure 1. It should be noted that for the Terzaghi method, the value of Nc is taken as 5.14, while for the Bjerrum and Eide method, the value of Nc varies depending on the geometry of the excavation. Both methods have certain limitations. The Terzaghi method tends to overestimate the contribution of shear resistance along the vertical portion of the assumed failure surface, but the Nc value does not account for the effects of the excavation geometry and depth on stability; although it could be argued that the likely overestimation of the effects of side friction are offset by the generally lower Nc values. On the other hand, the Bjerrum and Eide method does not properly account for the effects of limited thickness of soft clay below the base of the excavation. Both methods neglect the effects of the shoring system on base stability. However, it has been shown conclusively that the stiffness of the shoring system, and particularly the stiffness of the embedded portion of the shoring wall, are important factors in improving base stability. Jaworski (1973) studied the effects of system stiffness on the initiation of yielding, and showed that increasing the system stiffness could delay the initiation of yielding; and, as a consequence, leads to improved stability and smaller deformations. O'Rourke (1992) presented theoretical analyses to show that the work expended by the bending of the embedded portion of the shoring wall can contribute to improved base stability, which is reflected in increased Nc values. Ukritchon et al. (2003) present a comprehensive review of factors affecting base stability, including: wall adhesion, the depth of soft clay below the base, and the bending strength of the shoring wall. Their study led to the following conclusions: (1) wall adhesion can increase the stability coefficients, Nc, for narrow excavations (i.e., H/B much greater than 1); (2) for wide excavations (i.e., H/B < 1.0), as the thickness of the soft clay below the excavation decreases, the Nc values increase, even beyond what is reflected in the Terzaghi (1943) method; and (3) the values of Nc increase as the strength of the shoring wall increases, and the effects are more pronounced in cases where the depth of embedment of the shoring wall (D) is significant (i.e., D/H > 1.0).

The undrained shear strength of soft soils is critical to the evaluation of base stability. However, the undrained shear strength does not have a unique value, but it is affected by many factors, including the in situ vertical effective stress, σ'_{vo}, the overconsolidation ratio (OCR), the mode of shear (i.e., anisotropy), and time. Fortunately, most soft clays follow normalized behavior, and the effects of overburden stress and OCR can be readily accounted for by using the SHANSEP

(Ladd and Foott 1974) method. Most soft soils exhibit anisotropic characteristics with respect to their undrained shear strength (Bjerrum 1972, Bjerrum et al. 1972, Clough and Hansen 1981, and Koutsoftas and Ladd 1985). Figure 4 shows the variation in the mode of failure along typical failure surfaces in an excavation. Different parts of the failure surface involve shear in compression, extension, and direct simple shear. Clough and Hansen (1981) review the effects of strength anisotropy on base stability and deformations. They show that for typical anisotropic strength ratios ($K_s = Su_h/Su_v$), the factor of safety can be 15% to 20% lower than the values estimated for isotropic soil, assuming that the isotropic strength is as estimated from triaxial compression tests. Bjerrum et al. (1972) review factors affecting the undrained strength of soft soils as it relates to base stability, and provide recommendations for correction factors to be used with the results of field vane shear tests before being used for base stability analysis.

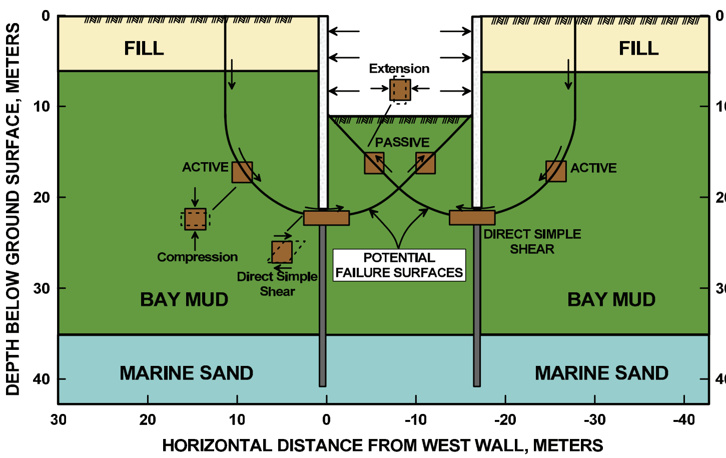


FIG. 4. Anisotropic Strength Characteristics of Soft Clays and Their Effects on Basal Stability

Another important factor in the selection of undrained shear strength for base stability evaluation is the fact that failure develops at significantly different shear strain levels under different modes of shear. In compression, failure can take place at strain levels of 1% or less, while in extension failure takes place at shear strain levels between 10% and 20%; and in direct simple shear, failure may develop at strain levels between 5% and 15%. It is therefore important to consider compatibility of strains along the failure surface. Koutsoftas and Ladd (1985) provide recommendations for selection of undrained shear strengths for stability accounting for the effects of anisotropy and compatibility of deformations. Recommendations for selection of undrained shear strengths for base stability are presented in a later section of this paper.

Base Stability Against Hydrostatic Uplift

Soft clays are often underlain by permeable strata under high hydrostatic pressures. Deep excavations, even in relatively thick clay deposits, can lead to a situation where at the end of excavation, a clay plug of only limited thickness remains below the base. Examples of this condition are shown on Figures 5 and 6 from two projects in San Francisco.

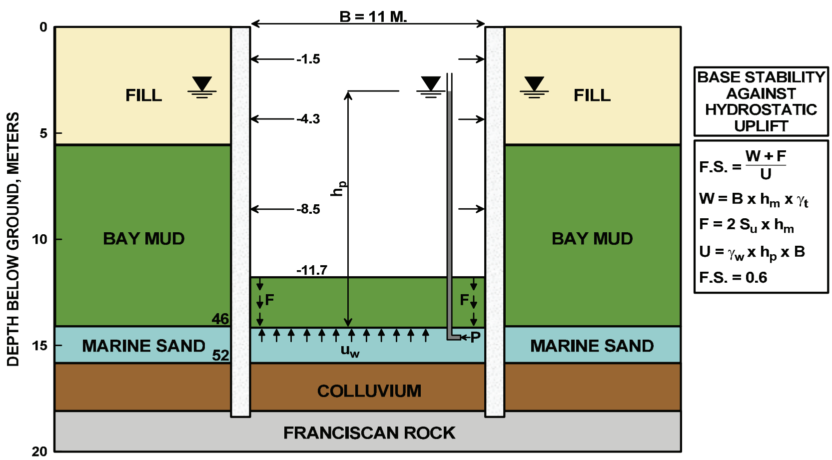


FIG.5. Excavation Stability against Hydrostatic Uplift: Islais Creek Contract D – Army Street Segment

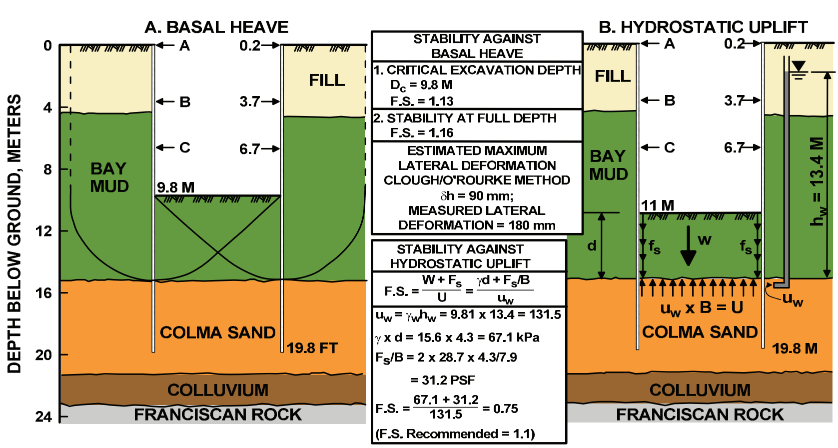


FIG.6. Base Stability against Hydrostatic Uplift – Islais Creek Contract C, San Francisco

Under these conditions, there is a serious risk of instability of the base of the excavation caused by the unbalanced hydrostatic pressures in the permeable layers. In the first case, relief wells (without pumping) were installed to relieve the unbalanced water pressures. The wells flowed continually during the excavation process, which confirmed the concerns about the unbalanced pressures. In the second case, dewatering had been specified, but the contractor concluded that the sand layer was not permeable enough to warrant dewatering. The excavation experienced very large lateral deformations (in excess of 180 mm) and settlements, which were in part due to the unbalanced hydrostatic pressures. The analysis of base stability against hydrostatic uplift is rather simple, but is often not considered, which can lead to serious consequences. Figure 7 shows the results of the analysis for a recent project in San Francisco, illustrating the variation in factor of safety as a function of excavation depth, H, and the thickness of the clay plug (D) below the base. It is evident from these results that under certain conditions, instability can result even in cases that may involve relatively thick deposits of clay below the base of the excavation.

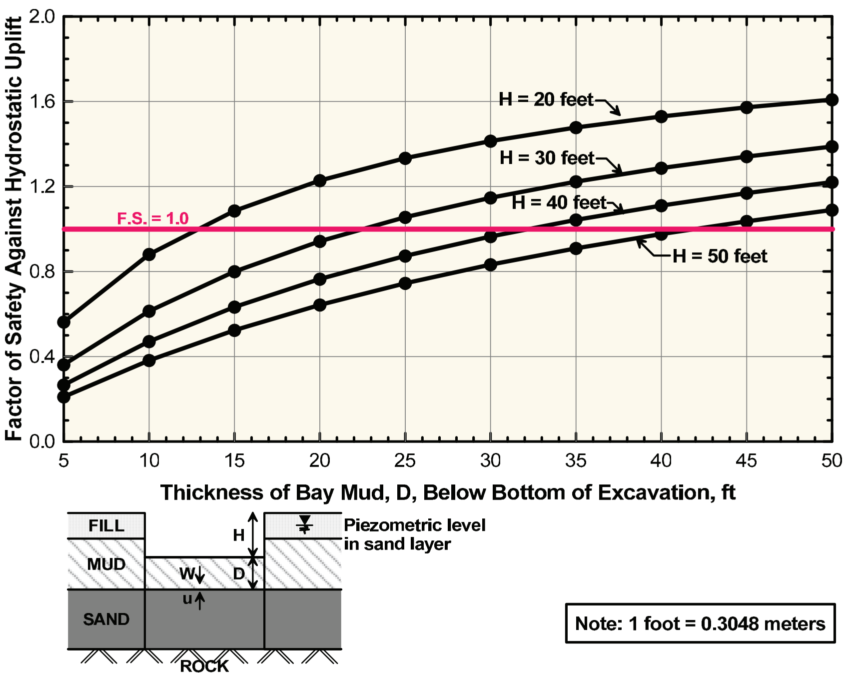


FIG. 7. Factor of Safety against Hydrostatic Uplift Excavations in Bay Mud Underlain by a Thick Permeable Stratum

The effects of unbalanced hydrostatic pressures on ground deformations are evident from the results of numerical analyses performed for a recent project in San Francisco, and shown on Figures 8a and 8b. The soil profile consists of 6 mof fill over 20.5-m-thick Bay Mud, which overlies dense Colma Sand. Groundwater was encountered at a depth of 3 mbelow the surface. As shown on Figure 8a, there was a significant increase in lateral deformations and settlements as the excavation was advanced from 8.8 mto 12.2 m. At this stage, the factor of safety against hydrostatic uplift was 1.1, and the factor of safety against basal heave was 1.15. When the excavation depth was increased to 15.2 m, the factor of safety against hydrostatic uplift dropped below 1.0 (see Figure 6), while the factor of safety against basal heave was still 1.1, not including the contribution of the very stiff shoring system that penetrated through the Bay Mud into the dense Colma Sand. At this stage, the numerical analysis failed to converge because of the large deformations that were developing. The solution was to extend the cut-off wall (just the soil cement and not the soldier piles) through the sand layer to cut off the seepage into the excavation, with the results shown on Figure 8b. The numerical analysis was completed, although lateral deformations were still rather large, because of the low factor of safety against basal heave, but the excavation remained stable.

Excavation-Induced Deformations

Excavation-induced deformations refer to the deformations caused by the physical removal of soil between the shoring walls and the installation of bracing. The vast majority of monitoring data presented in the technical literature refer to these deformations. Instrumentation is normally installed after the shoring walls are constructed, and monitoring begins shortly before commencement of excavation. Currently, available methods for estimating excavation deformations, such as the Clough and O'Rourke (1990) method, as well as numerical modeling, simulate the excavation-induced deformations. Often, monitoring is terminated soon after the excavation is completed, although the process of removal of struts and other operations such as backfilling and continued dewatering can cause additional deformations.

Most of the cases reported in the literature include lateral deformations of the shoring wall, and in some instances at short distances behind the wall. It is rare to have measurements of lateral deformations at significant distances from the wall. An example of lateral deformations measured at various distances behind the shoring wall are shown on Figure 9, from measurements made for the Muni Metro Turnback (MMT) project (Koutsoftas et al. 2000) in San Francisco. They show significant lateral deformations at distances, as far as 1.5H, behind the excavation. These deformations are important in evaluating impacts on existing adjacent structures, because of the differential extension that existing buildings would be subjected to (see O'Rourke et al. 1977, Boscarding and Cording 1989, Clough and O'Rourke 1990, and Cording et al. 2010). The response of adjacent buildings to such differential extension depends on the structural system of the building, particularly whether the foundations are tied together, or they behave as individual footings.

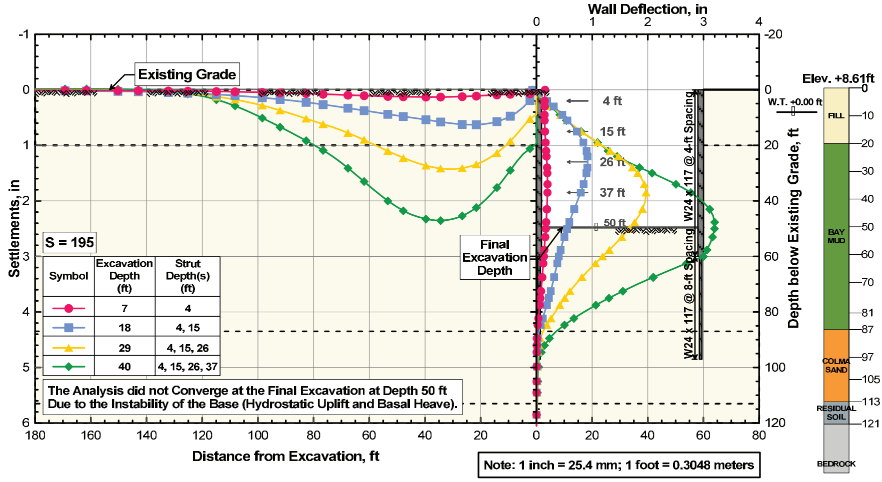


FIG.8a. Profiles of Lateral Deformations and Surface Settlements: Failure of Numerical Analysis to Converge at Final Excavation Step Because of Instability Due to Hydrostatic Uplift

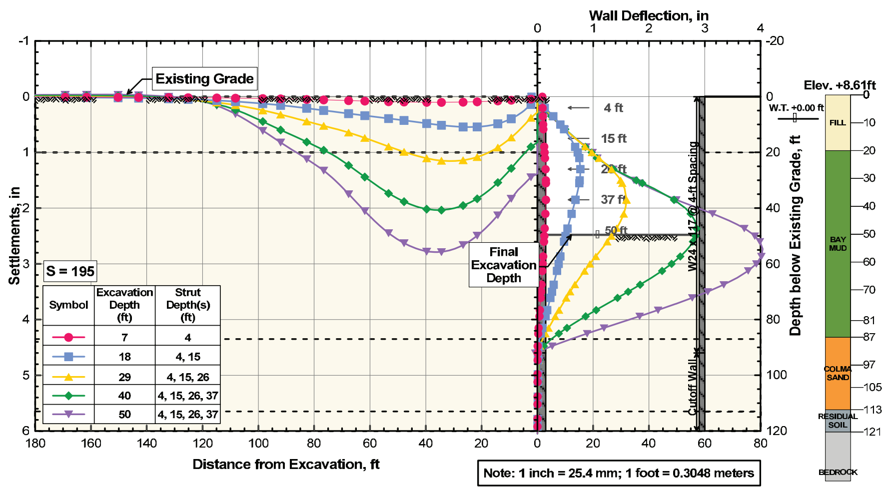


FIG.8b. Profiles of Lateral Deformations and Surface Settlements: Completed Numerical Analysis after Mitigation for Instability against Hydrostatic Uplift

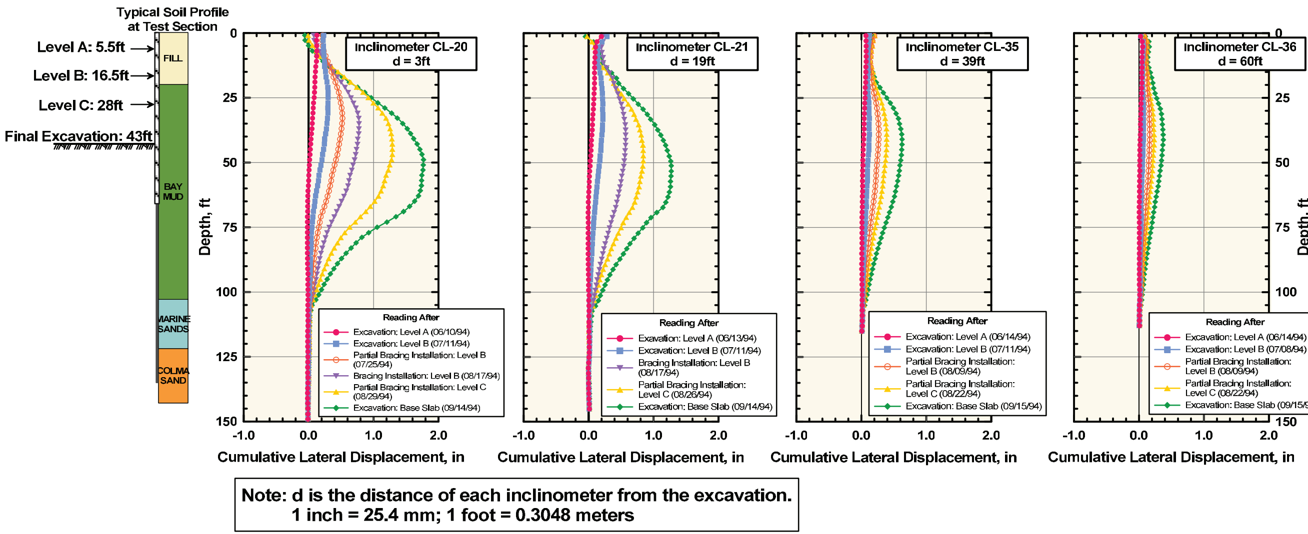


FIG.9. Lateral Deformations Measured at Various Distances from a Deep Excavation in Bay Mud: MUNI Metro Turnback Project, San Francisco

The effects of the removal of struts on lateral deformations are evident in the profiles shown on Figure 10 (Koutsoftas 1999) for an 11-m-deep excavation in Bay Mud. The removal of the struts caused an increase in lateral deformations above the base slab. It is therefore essential, when important structures are present adjacent to an excavation, to include monitoring of lateral deformations within the zone of influence of the excavation, and that the monitoring continue until the project is completed, which is always much longer than the end of excavation.

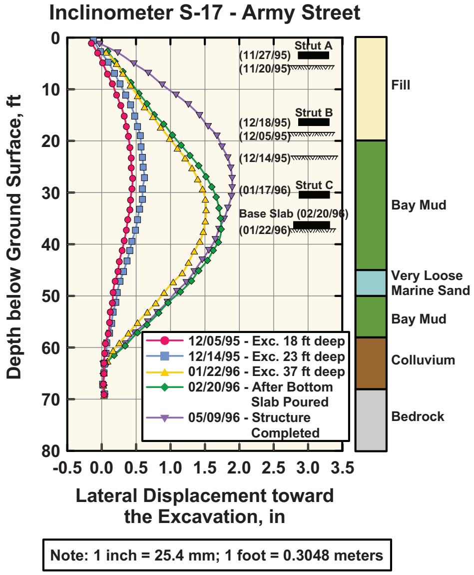
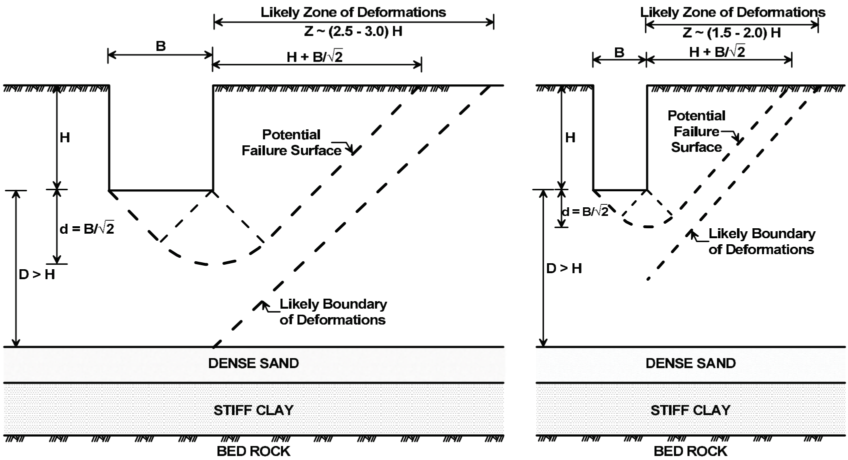


FIG.10. Typical Lateral Deformation Profiles Illustrating the Effects of Strut Removal on Deformations: Army Street Segment of Islais Creek Site, San Francisco

Although in theory, measurements of settlements should be easier than measurements of lateral deformations, actually, accurate measurement of excavation-induced settlements is much more difficult because the measured settlements often include settlements that are not caused by the excavation process, such as: settlements of loose fills (that often overlie soft clays) due to vibrations, settlements caused by dewatering, pile driving, and other activities. There are conflicting data regarding the zone of influence of settlements, as well as the profile of settlement, as discussed earlier. There is an apparent inconsistency between the shape of the settlement profiles from actual measurements and the envelope proposed by Clough and O'Rourke (1990). It is important to recognize that the envelope proposed by Clough and O'Rourke (1990) reflects the maximum expected settlements, and not the actual profile of settlements. Some recent studies (Hsieh and Ou 1998, Kung et al. 2007, and Wang et al. 2010) suggest that the shape of the settlement profiles behind excavations may be considerably different from the empirical profiles presented by Peck (1969), and by the envelope proposed by Clough and O'Rourke (1990). It appears that fundamentally, a difficulty arises when attempting to relate the zone of influence of excavation-induced deformations to the depth of the excavation, without considering other factors that may affect the zone of influence of settlements. The width of the excavation, as well as the depth of the clay below the base of the excavation, are important factors, affecting the zone of influence of settlements behind excavations. For very narrow excavations (H/B much greater than 1), the zone of influence of excavation-induced settlements is likely to be limited to a zone less than 2.0H, because, as shown on Figure 11, the zone of deformations behind the excavation is affected by the depth of the potential failure zone, which is theoretically equal to $B/\sqrt{2}$. If B is much smaller than H, it is highly unlikely that significant settlements can develop beyond a distance equal to $H+B/\sqrt{2}$. For excavation widths, B, equal to H/2 or less, the zone of influence of settlements would be less than 2H, and in some instances significantly less than 2H. The same effect, of limited zone of influence of settlements, develops when the thickness of soft clay below the excavation is small relative to the width of the excavation; i.e., D/B significantly less then 1.0. On the other hand, when very thick deposits of clay are present below wide excavations (B/H > 1.0), the zone of influence of excavation-induced deformations can extend to a distance that can be significantly greater than 2H, as illustrated on Figure 11. Figure 12 presents suggested normalized profiles of settlements, based on the author's experience, that attempt to account for the thickness of soft clay below the base of the excavation, and also to recognize the accumulating evidence that settlements immediately behind the wall are significantly smaller than the maximum settlements that can develop behind an excavation. It is important to realize that the shape of the settlement profile is very important in evaluating potential impacts on adjacent structures, and that settlement profiles, if assumed to have the shape of the envelopes shown on Figure 3, can lead to unsatisfactory results.

Other Sources of Ground Deformations Related to Excavations

Excavation-induced deformations are just one component of the deformations that can develop during excavation in soft soils. There are a number of other construction processes associated with excavation that can cause additional deformations, including the following: (1) construction of diaphragm (slurry) walls; (2) construction of secant-pile walls; (3) removal of old foundations or other obstructions prior to installation of the shoring walls; (4) jet-grouting that might be performed either for ground improvement or to supplement the bracing system; (5) installation of foundation piles to support the permanent structures; (6) removal of existing piles; (7) dewatering; (8) construction vibrations; and (9) removal of struts, backfilling, and other activities. The sources of additional deformations are reviewed below.



(a) WIDE EXCAVATION; DEEP SOFT CLAYS (b) NARROW EXCAVATION

FIG. 11. Effects of Excavation Geometry and Depth of Soft Soils on Zone of Influence of Excavation-Induced Deformations

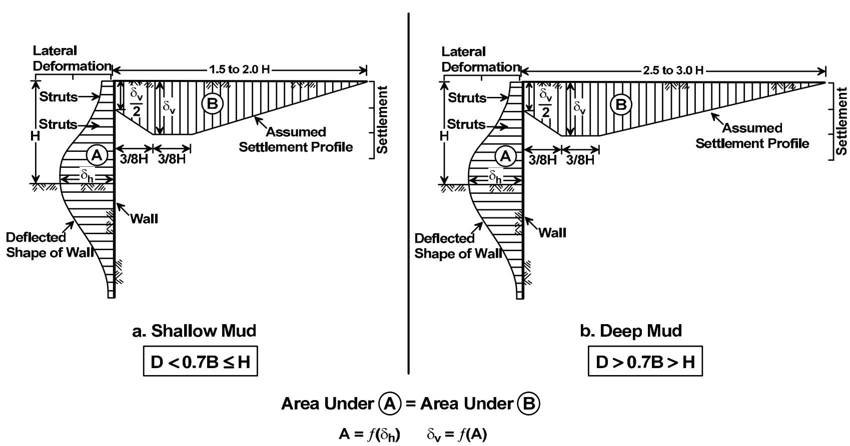


FIG.12. Proposed Method for Estimating Excavation-Induced Settlements from Predicted Lateral Wall Deformations

1. Construction of Diaphragm (Slurry) Walls.

Clough and O'Rourke (1990) summarized settlements caused by the construction of slurry walls from a number of case histories. The data show maximum settlements developing adjacent to the trench, and decreasing with distance from the trench, with a zone of influence ranging between 1.0H and 1.5H (where H is the depth of the trench). The maximum settlements can be up to 0.14% of the depth of the trench.

In soft soils, the excavation of the trench and subsequent backfilling with concrete causes significant stress changes in the soil around the trench. Figure 13 illustrates stress changes in terms of stress paths, for an element of soil behind the trench. During excavation the soil experiences increased shear stresses due to reduction of the lateral stresses, causing axial compressive strains and lateral extension and associated settlements and lateral deformations towards the trench. During this phase, small negative excess pore pressures develop. During subsequent filling of the trench with concrete, the horizontal stresses increase, causing shear in extension, and development of significant positive excess pore water pressures. During this phase, there can be significant horizontal deformations away from the trench and possibly heave. This behavior is illustrated below from the results of two case histories.

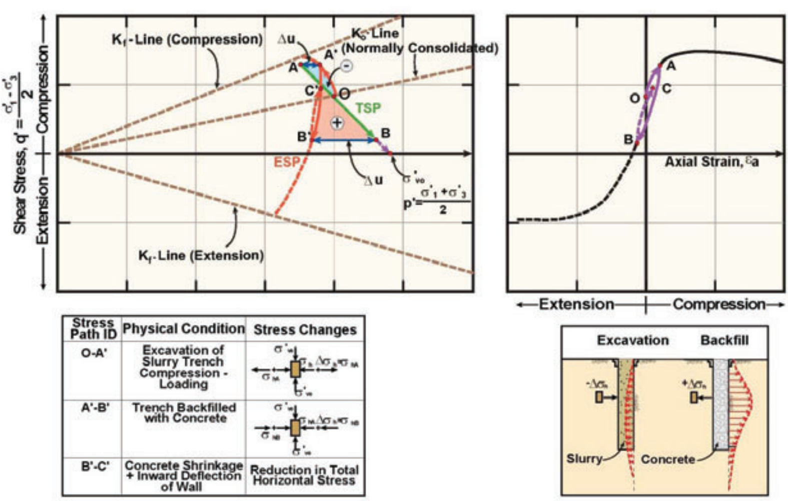


FIG.13. Stress Changes and Deformations during Diaphragm Wall Construction

Figure 14 shows lateral deformations and settlements measured during a test program that involved construction of a deep slurry wall in soft soils in Singapore (Poh and Wong 1998). During excavation, lateral deformations of up to 40 mm towards the trench were recorded in the soft marine clay, and settlements up to 32 mm were measured adjacent to the trench. The zone of influence of the trench excavation, as reflected by the settlement profile, was approximately 14 m, although the trench was 40 m deep. During concreting, there was a reversal of the lateral deformations, causing deformations away from the trench, and there was a small amount (about 5 mm) of surface heave (reduction in settlement). These observations are consistent with the behavior anticipated from the stress paths on Figure 13.

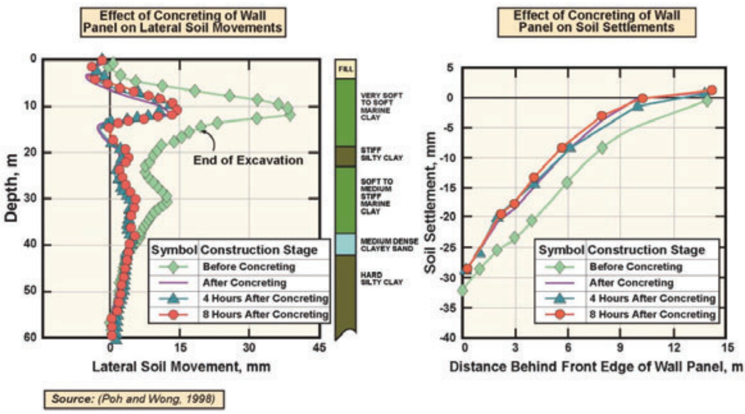


FIG.14. Deformations Caused by Construction of Slurry Walls in Soft Clays: Singapore Project

Similar results were observed during construction of the Soldier Pile and Tremie Concrete (SPTC) walls for the MMT project in San Francisco (Koutsoftas et al. 2000). Figure 15 shows a cross-section of the subsurface conditions, the excavation and bracing, the SPTC walls, and inclinometers installed prior to the construction of the SPTC walls. Figure 16 shows profiles of settlements measured during trench construction along two lines of surface settlement markers. The data indicate a maximum settlement of about 25 mm, which is approximately 0.1% of the depth of the trench. The zone of influence was 21.3 m wide, approximately equal to the depth of the trench. These measurements are consistent with the Clough and O'Rourke (1990) data.

Figure 17 shows excess pore water pressures measured with two piezometers that were installed behind the trench. They show rapid and significant increase in pore water pressures during concreting, as expected from Figure 13. The excess pore water pressures had dissipated before the excavation began, because of lateral deformations and the associated reduction in lateral stresses caused by pile driving, which is discussed later in this section.

Figure 18 shows the lateral deformations measured with an inclinometer located 6.1 m behind the trench. The effects of construction of several SPTC panels are shown. During construction of each of the panels in the vicinity of the inclinometer, there were distinct increments in lateral deformations away from the trench. Maximum lateral deformations at the end of construction of the SPTC wall approached 30 mm. Lateral deformations were taking place not only transverse to the wall, but also parallel to the wall, although the deformations parallel to the wall were much smaller, because the deformations reversed direction as panels were constructed on either side of the inclinometer. Koutsoftas et al. (2000) provide more detailed discussion of these data, and show a zone of influence of lateral deformations caused by slurry wall construction close to 30 m wide.

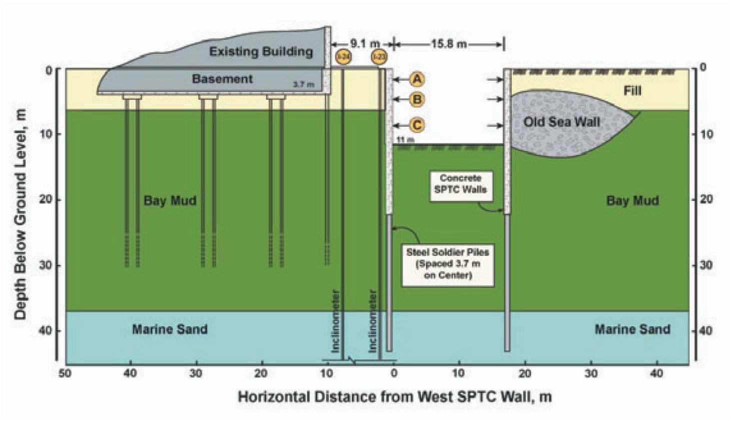


FIG.15. Excavation for MUNI Metro Turnback Project: SPTC Walls andBracing System

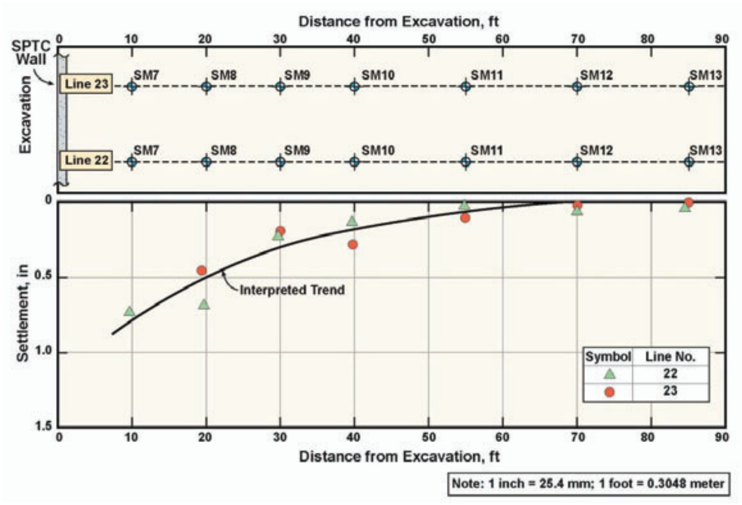


FIG.16. Settlements Measured at the End of SPTC Wall Construction – MMT Project, San Francisco

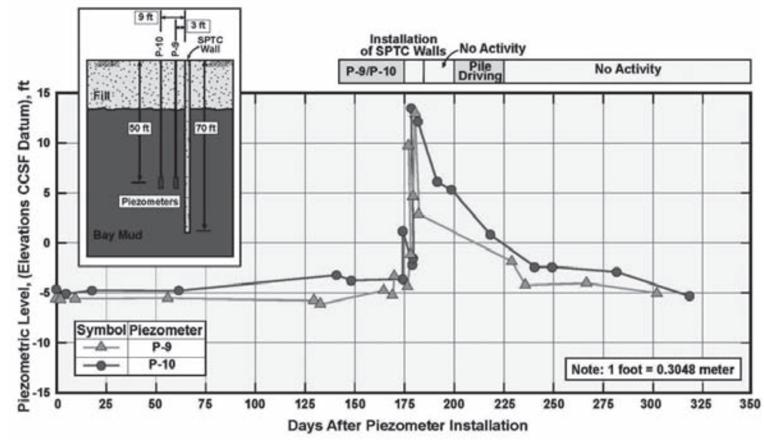


FIG.17. Excess Pore Water Pressures Caused by Slurry Wall Construction

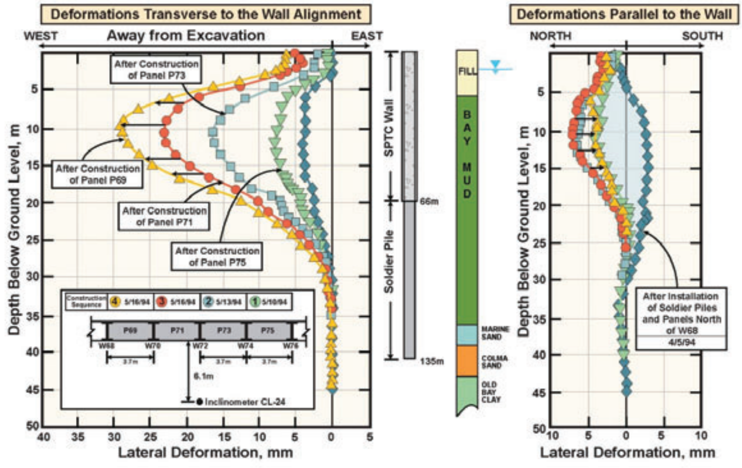


FIG.18. Lateral Deformations Caused by Slurry Wall Construction – MMT Project, San Francisco

The lateral deformations during excavation of the trench at the MMT project were much smaller than the values reported for the Singapore case history. One important difference in these two cases is the depth of groundwater. In the Singapore case, the water table was close to the surface, and therefore there was minimal positive differential head (stabilizing effect) from the slurry in the trench. For the MMT project, the groundwater table was 3 m to 3.7 m deep, which resulted in significant positive head, providing lateral support to the walls of the trench.

2. Deformations Caused by Secant Pile Wall Construction.

Construction of secant pile walls can cause significant deformations due to sloughing, and loss of ground, caused by squeezing of clays during excavation of the holes for the construction of the soldier piles. Squeezing of the hole in soft soils can also cause overexcavation and exacerbate the deformations caused by loss of ground.

The effects can be particularly severe if drilling fluid is not used to stabilize the excavated holes. Examples of significant deformations caused by construction of secant pile walls are provided by Lucas and Baker (1978), Finno (1992), Finno et al. (2002), Calvello and Finno (2003), and Finno (2010). During construction of the soldier piles for the SPTC walls for the MUNI Metro Turnback project, settlements up to 3 inches were observed on the sidewalk adjacent to several soldier piles. The cause of the settlements was sloughing of the loose fill around the uncased holes that were being drilled for the soldier piles. The problem was exacerbated during concurrent excavation for adjacent soldier piles. The excessive deformations were mitigated by using casing, introducing slurry in the hole at the early excavation stages, and avoiding concurrent excavation of adjacent soldier piles.

In a recent project in San Francisco, a prototype test was performed involving the construction of a partial secant wall that included construction of 5 secant piles, each 7 feet in diameter, up to 230 feet deep. The use of casing in combination with slurry, together with a very rigorous control of the excavation process (maintaining the casing ahead of the excavation at all times to eliminate squeeze of the deep Old Bay Clay layer), was successful in virtually eliminating settlements and lateral deformations during construction of the secant piles. This prototype test demonstrated that in soft soils, the use of casing in combination with slurry and proper control of the excavation process can control settlements caused by construction of soldier piles to very small values.

3. Ground Deformations Caused by Jet Grouting.

Jet grouting is a very versatile technique for ground improvement, with the potential for numerous applications in soft ground construction, including deep excavations. Jet grouting has been used on several projects to improve excavation support, as described by Whittle and Davies (2006), Tam and Li (2011), Adams and Robison (1996), and Wong and Poh (2000), as well as for another project in San Francisco discussed later in this paper. The most common application is to construct a jet-grouted strut at the base of the excavation to restrain the lateral deformations of the shoring walls, and thus improve base stability and reduce lateral deformations that typically develop deep below the base of the excavation. However, jet-grouting can, under certain circumstances, cause significant ground deformations that could have detrimental impacts on existing adjacent structures. Measurements of deformations caused by jet grouting are reviewed below from three case histories.

Poh and Wong (2001) reported the results of a comprehensive field testing program in Singapore that involved jet grouting to treat a block of soft marine clay adjacent to a concrete diaphragm wall. Figure 19 shows the layout of the test, including the diaphragm wall, the jet-grouted block, and the instrumentation that was installed to monitor lateral deformations, pore water pressures, and lateral stresses. Figure 20 shows the stratigraphy of the site, and the lateral deformations measured with four inclinometers during jet grouting. Inclinometers were installed behind the wall, and in the free field. The data showed that the entire wall deflected away from the jet-grouted block, and deflections extended well below the zone of jet grouting. The top of the wall moved approximately 8 mm, with the maximum deflection of 10 mm occurring just above the zone of jet grouting. Deformations in the free field were much larger, although they decreased with distance from the zone of treatment.

Figure 21 plots the maximum lateral deformations measured in the free field and behind the diaphragm wall as a function of distance from the treatment zone and the wall. The measurements indicate a zone of influence of up to 25 m wide behind the wall, and perhaps up to 30 m in the free field.

Figure 22 plots maximum excess pore water pressures measured in the marine clay layer as a function of the measured incremental lateral stresses. The data suggest that the excess pore water pressures are the result of the increased lateral stresses caused by displacement of the soil during jet grouting.

In another case, deep sewers were constructed in deep, soft Bay Mud in the Islais Creek area in San Francisco as part of the Islais Creek Transport/Storage project (Dames & Moore 1997). The specifications required a very stiff shoring system extending deep below the base of the excavation, as shown on Figure 23. The contractor proposed an alternative system that included a pre-constructed jet-grouted strut 2 m thick at the base of the excavation, in lieu of the specified deep shoring walls. The advantages included much shorter shoring walls, and reduction of the number of horizontal supports from 3 to 2 levels of struts. The jet-grouted strut was anchored, to prevent uplift and to reduce bending moments in the grouted strut, using the steel H-piles that were designed to support the permanent structure. During jet grouting, lateral deformations were measured with inclinometers, which were installed behind the shoring walls. Detailed results of the monitoring program are presented in Dames & Moore (1997). Figure 24 shows typical lateral deformations measured at two locations along the alignment of the sewer. The data show that the entire wall was displaced laterally, although the bottom of the wall deflected more than the top, causing tilting of the wall. Lateral deformations up to 150 mm were measured. The maximum deformations developed at the level of the jet-grouted strut, but significant deformations developed also in the Bay Mud below the base of the excavation to depths of 6 m to 12 m below the jet-grouted slab. The maximum shear strain represented by the deformations of the Bay Mud is approximately 1.1%, which is well below the failure strain in direct simple shear. The behavior observed at the Islais Creek project is consistent with the behavior observed at the Singapore site, although the magnitude of the deformations is significantly larger because the shoring wall was much shorter.

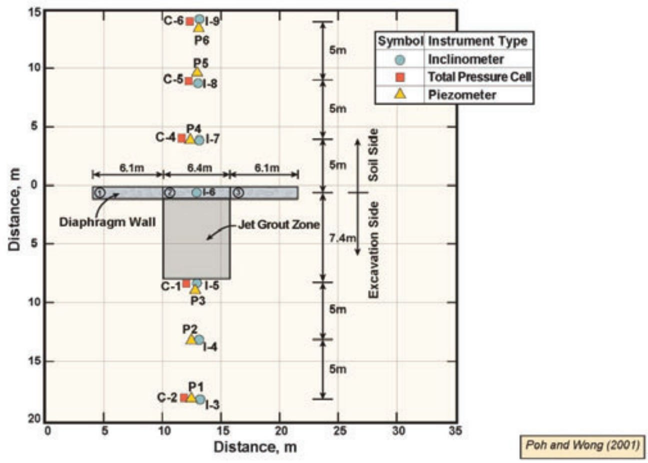


FIG.19. Plan of Singapore Jet-Grouting Field Test

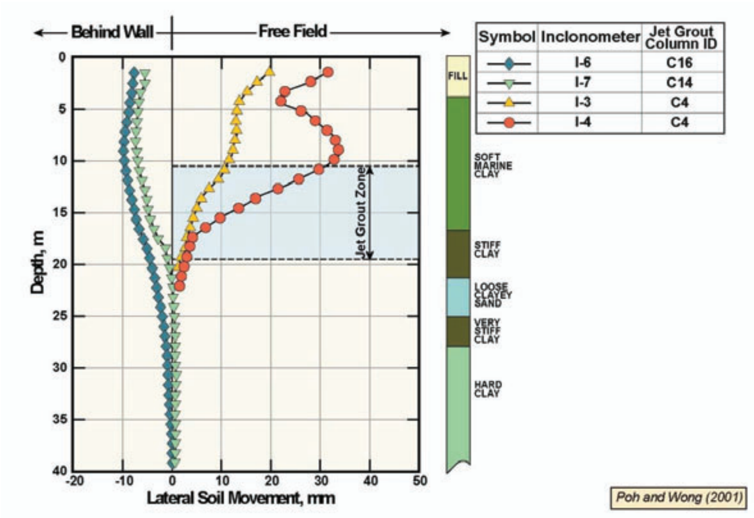


FIG.20. Lateral Deformations Profiles Recorded during Jet Grouting: Singapore Field Test

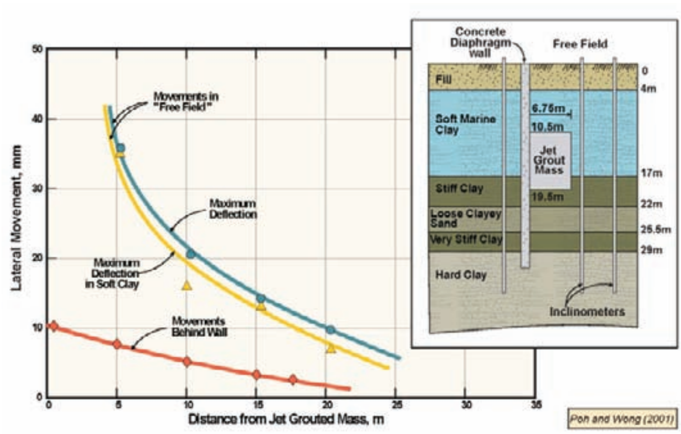


FIG.21. Lateral Deformations Caused by Jet Grouting: Singapore Field Test

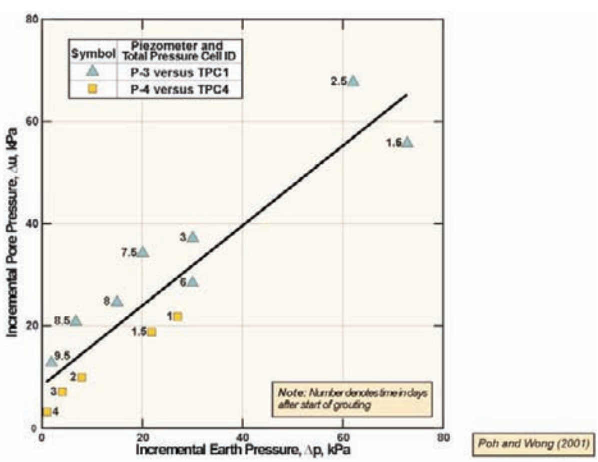


FIG.22. Excess Pore Water Pressures and Lateral Stresses Caused by Jet Grouting: Singapore Field Test

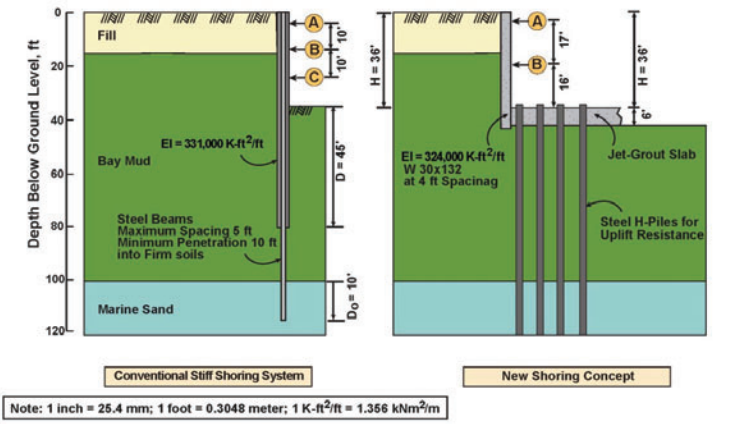


FIG.23. Islais Creek Contract D Excavations: Use of Soil Cement Walls and Jet Grouting Strut for Shoring

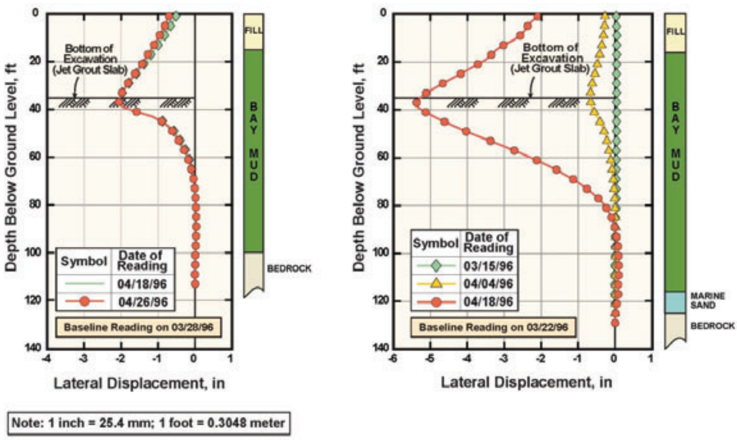


FIG.24. Lateral Deformations Caused by Jet Grouting: Islais Creek Contract D

It is inevitable that significant heave must have developed as a result of the lateral deformations shown on Figure 24. Unfortunately, the settlement measurements, which according to the contract were the responsibility of the contractor, had never been submitted to the design team; because, according to the contractor, a baseline survey had not been performed prior to jet grouting.

In another case, which was also part of the Islais Creek Transport/Storage project, a segment of the sewer involved a deep, but rather narrow, excavation in deep Bay Mud adjacent to a railroad embankment. Diaphragm walls, 0.9 m thick, were specified extending through the Bay Mud and were socketed into bedrock. A 4-m-thick jet-grouted slab was specified to improve stability and to restrain lateral deformations of the walls. Figure 25 shows the cross-section of the 14-meter-deep excavation and lateral deformations measured with inclinometers that were attached to the steel soldier piles (Adams and Robison 1996). The data show that jet grouting had caused the diaphragm walls to move outward, with a maximum lateral deformation of 25 mm developing at the level of the jet-grouted strut, and approximately 20 mm at the top of the wall. The wall deflected all the way to the bottom of the Bay Mud. The benefits of the jet-grouted strut are evident by the rather small lateral deformations that were measured during excavation.

The results from these three case histories show clearly the potential for large excess pore water pressures, lateral deformations, and heave to develop during jet grouting. Proper planning, in combination with real-time monitoring, is necessary to allow sequencing of the jet-grouting operations in a manner that would minimize the cumulative effects of jet grouting.

4. Deformations Caused by Pile Driving and Pile Extraction

When the permanent structures are supported on pile foundations, it is customary to drive the piles before excavation begins, because in most cases that involve deep excavations, it is not practicable to drive the piles after completion of the excavation. In such cases, the piles are driven using rather long followers. When the followers are extracted from the ground, after driving of each pile, a hole is left in the ground equal to the diameter of the follower. Figure 26 shows schematically the holes that are left after extraction of the follower, and the potential for settlements due to the collapse of the holes. Simple calculations can be made to estimate the loss of ground that can result from collapse of the holes, if they are left open. Three case histories are reviewed below to illustrate this condition.

The first case involved installation of over 1,000 prestressed concrete piles for uplift support of a deep concrete storage basin, which required a 13.5-m-deep excavation, 72 m × 81 m in plan.

The soil profile consisted of a layer of loose fill 3 m deep, over soft Bay Mud to a depth of 10 m, which was underlain by very stiff clays to great depth. The piles were to be installed in pre-drilled holes (to minimize heave), to a depth of 8 to 10 m below the base of the excavation, as shown on Figure 26. Simple calculations demonstrated that unless the holes that would be left by the extraction of the followers were filled, unacceptably large settlements and lateral deformations would develop. To prevent the anticipated ground deformations, the specifications required backfilling the holes with grout or pea gravel. The contractor elected to fill the holes by pouring grout from the surface into the holes, after extracting the followers. Careful monitoring of the volume of grout that was poured to fill the holes, and comparison with the theoretical volume of the voids suggested that the process was partially successful, and approximately 75% of the voids had been filled with grout.

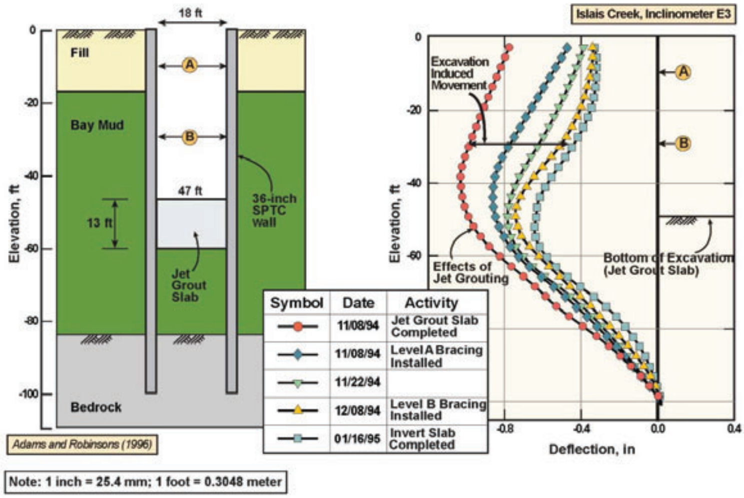


FIG.25. Lateral Deformations Caused by Jet Grouting and by Excavation: Islais Creek Contract E

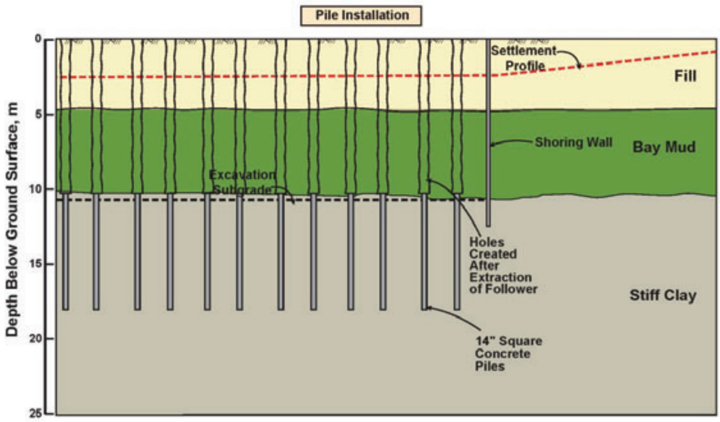


FIG.26. Schematic of Pile Installation in Deep Excavations using Followers and Likely Settlement Profile

Lateral deformations were monitored during pile installation with inclinometers placed behind the shoring wall; the results from two inclinometers are summarized on Figure 27. The measurements show that at depths greater than 6 m (in the stiffer soils) the lateral deformations varied essentially linearly with depth, with maximum values in the range of 25 mm to 50 mm. However, in the softer soils, in the upper 6 m, the lateral deformations were much larger, ranging between 100 mm and 150 mm. These deformations are much larger that the excavation-induced deformations (Koutsoftas 1999), which were typically less than 50 mm. This case history illustrates that in soft soils, large deformations could develop during pile driving with long followers, even if mitigation measures are implemented.

The second case involved driving four rows of 450-mm, square, prestressed-concrete piles within the footprint of an excavation 16 m wide, and 11 m to 13 m deep, for the MMT project (see section on Figure 15). The piles were spaced 2 m to 3 m apart. Because of the depth of the excavation and close spacing of the struts, it was necessary to drive the piles before excavation began, using long followers. Because of concerns with pile heave, the specifications required driving the piles in predrilled holes. However, of equal concern were potential deformations that could be caused by collapse of the holes left after extraction of the followers, similar to what is shown on Figure 27. The specifications required special procedures to be implemented to minimize deformations due to pile driving. The specified sequence of driving is illustrated on Figure 28. Lateral deformations measured during pile driving are summarized on Figure 29.

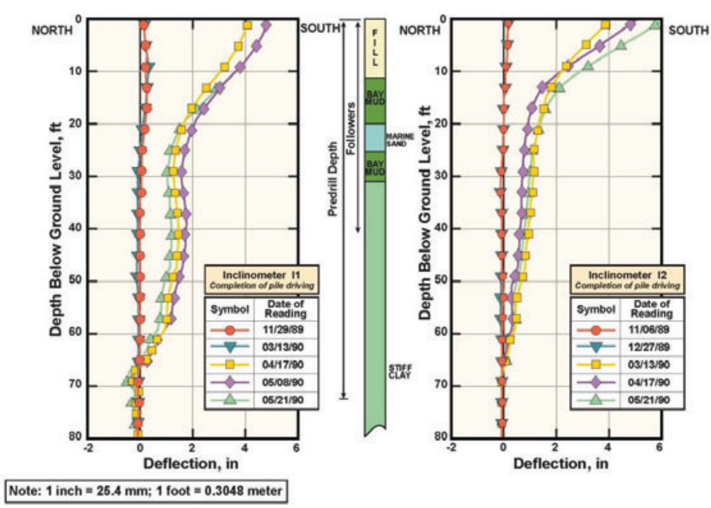


FIG.27. Effects of Predrilling and Use of Long Followers on Lateral Deformations during Pile Driving

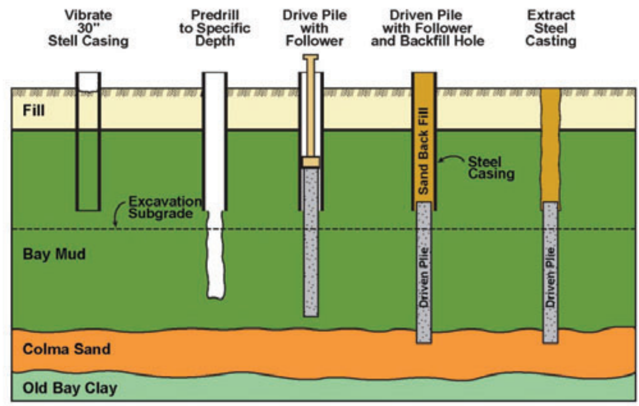


FIG.28. Pile Installation Procedure for Control of Heave and Settelements During Pile Driving with Deep Followers: MMT Project – San Francisco

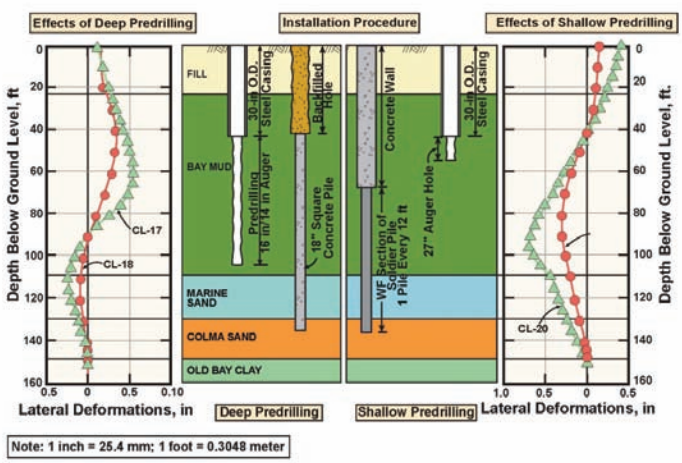


FIG.29. Typical Lateral DeformationProfiles: Pile Driving using Predrilling and Followers: MMT Excavation – San Francisco

The inclinometers shown on Figure 29 were located 1 m and 3 m behind the shoring wall, respectively. The data show outward deformations below the depth of predrilling, and inward deformations above the depth of predrilling. The lateral deformations were relatively small, typically less than 15 mm. The mitigation measures that were employed to minimize deformations caused by pile driving were effective, but increased the cost of pile installation, although the installation of the casings, the backfill, and the extraction of the casings were very fast, and had no impact on the schedule. Koutsoftas et al. (2000) discuss this case in greater detail and show cumulative lateral deformations towards the excavation of up to 20 mm, and a zone of influence approximately 20 m wide. This case demonstrates that even with extensive mitigation measures, deformations caused by pile driving are unavoidable, but it is important to take the necessary steps to minimize them.

The third case involved an 11-meter-deep excavation for a box sewer supported on prestressed concrete piles for uplift. The piles were spaced 2 m to 3 m apart and penetrated up to 8 m below the base of the excavation. Concrete piles were installed in predrilled holes using long followers, after the steel sheet piles had been installed, but before the excavation started. The holes left by the extraction of the followers were not backfilled, but allowed to collapse. Figure 30 shows a schematic of the installation process and the likely profile of settlements that might result from collapse of the holes that were left after excavation of the followers. Figure 31 shows building settlements measured around the excavation after pile driving was completed, but before commencing the excavation process. Settlements ranged between 33 mm and 130 mm. Figure 32 shows settlements measured at various distances from the excavation, and compares them with theoretical estimates made using the assumed profile shown on Figure 30. The results of the simplified analyses are consistent with the observed settlements.

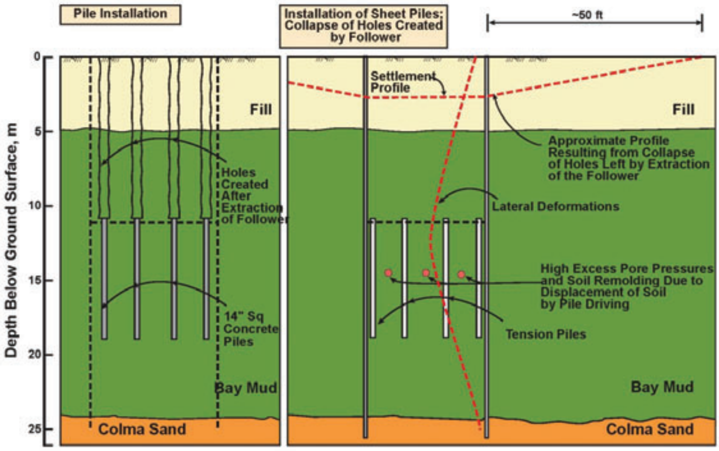


FIG.30. Schematic of Procedure Used to Drive Tension Piles for a Deep Box Sewer and Effects on Lateral Deformations and Settlements: Islais Creek Contract C – San Francisco

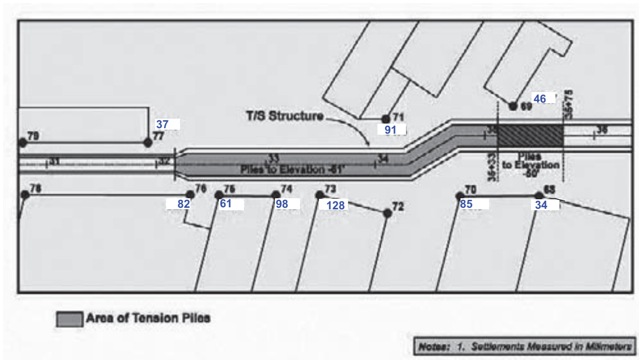


FIG.31. Building Settlements Caused by Pile Driving Using Predrilling and Followers: Islais Creek Contract C – San Francisco

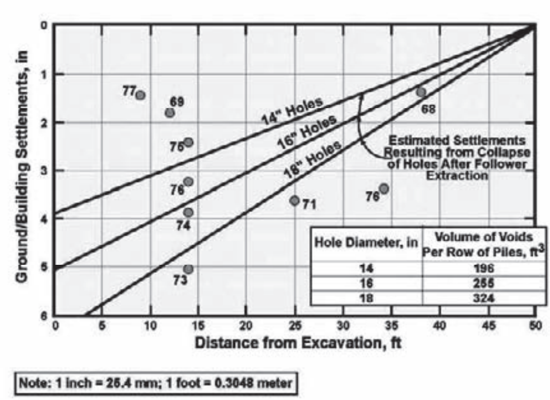


FIG. 32. Evaluation of Settlements Caused by Pile Driving Using Long Followers: Islais Creek Contract C – San Francisco

The deformations that can develop as a result of extraction of pre-existing piles are similar to the effects of the collapse of the holes left from extraction of the followers. The author is aware of a number of unpublished cases that involve large lateral deformations and settlements caused by extraction of wood piles, which are commonly found at many sites in San Francisco where old buildings are often demolished to construct new buildings. As a result of the past unfavorable experiences with extraction of piles, in recent projects, the trend has been to specify that the piles not be extracted, but the excavation be carried around the piles, and that the piles be cut after each excavation stage. Where the piles extend below the base of the excavation, they are left in place. This is a good practice and should become standard.

Deformations Caused by Vibrations

Settlements and possibly lateral deformations can develop from densification of loose sands caused by construction vibrations. Activities that can produce levels of vibration sufficiently large to cause densification include: (1) driving of steel sheet piles for shoring; (2) pile driving; and (3) use of jackhammers to break up obstructions such as concrete and boulders found in fills, and occasionally to break up concrete that protrudes beyond the vertical alignment of diaphragm walls, in cases where large overbreaks might occur during excavation and backfilling of the slurry trenches.

Clough and Chameau (1980) and Clough et al. (1989) summarize the results of studies involving settlements caused by sheet-pile driving for several projects in San Francisco. Lacey and Gould (1985) describe case histories involving settlements caused by vibrations due to pile driving.

Deformations Cause by Drilled Pier Construction

During construction of drilled piers through soft soils, if the drilled holes are not cased, significant settlements can develop; partially due to sloughing, and partially due to squeezing of the soil around the excavated hole. The effect is very similar to what was described earlier for the effects of the construction of secant pile walls. Finno (1992) summarizes several cases involving ground deformations caused by construction of drilled piers in the Chicago area. Lateral deformations measured with inclinometers installed behind shoring walls show deformations caused by caisson construction, as large as 30% to 40% of the total deformations of the shoring system that were measured at the end of excavation. In one case (Lucas and Baker 1978) construction of a single row of caissons resulted in lateral deformations of 25 mm to 50 mm.

Figure 33 shows the layout of drilled piers for a port terminal structure at a site in Hong Kong. The drilled piers varied in diameter from 1 m to 3 m and were up to 30 m deep. The ground consisted of deep reclamation fill over a relatively thin deposit of soft clay. The drilled piers were constructed in cased holes extending to the bottom of the soft soils. During construction, settlements were monitored along the perimeter of the site, and profiles of settlements measured at various times during construction are also shown on Figure 33. Settlements up to 35 mm were measured along the two lines of the surveys, which were up to 10 m away from the closest caissons.

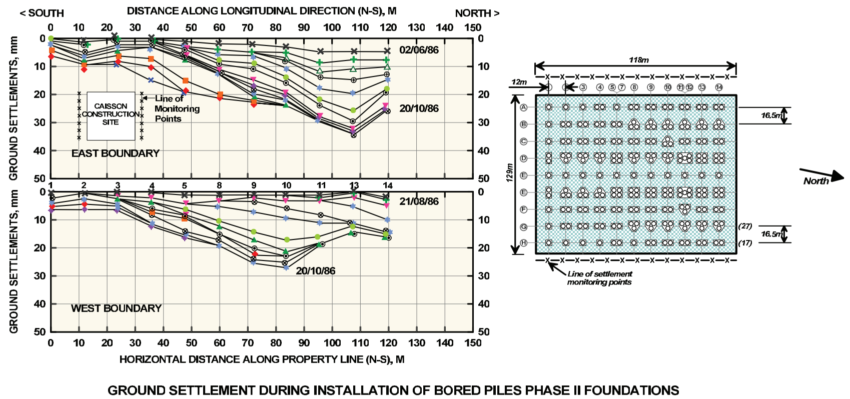


FIG.33. Profiles of Settlements Caused by Caisson Construction for a Port Terminal Structure in Hong Kong

In a recent project in San Francisco, a prototype test was performed to evaluate the feasibility of installing large-diameter secant piles to a depth of 70 m. As part of the test, five piles were installed, each 2.13 m in diameter. Kluzniak et al. (2010) describe subsurface conditions at the test site. A comprehensive monitoring program was implemented, including measurements of settlements and lateral deformations. Steel casings were advanced ahead of the excavation, and carefully controlled procedures were implemented during excavation, backfill, and extraction of the casings intended to minimize ground deformations caused by the installation process. The results of the prototype test showed imperceptible lateral deformations, and settlements were typically less than 5 mm, probably the result of near-surface sloughing or compression of the 6-m-deep fill. The results of this test show that appropriate procedures can be specified to control settlements and lateral deformations that can be caused by caisson construction to very small (5 mm or less) values.

The deformations reported by Finno (1992), as well as the case history from Hong Kong, show the potential for significant settlements and lateral deformations can develop during caisson construction. However, as the case history in San Francisco demonstrated, it is possible to mitigate the deformations caused by caisson construction by specifying construction methods that can keep the deformations to very small values.

Deformations Caused by Dewatering

In most cases involving excavations in soft clays, the general rule is to extend the shoring walls to penetrate deep enough to tie into an impermeable layer, thus creating a bathtub effect that limits the requirements for dewatering. Water-bearing strata, if present within the excavation, are predrained using wells or wellpoint systems. It is not customary, nor desirable, to dewater water-bearing strata behind or below the excavation because of the potential for consolidation settlements of the soft soils due to reduction in pore water pressures. The typical case where dewatering becomes essential is to relieve hydrostatic uplift pressures below the base of the excavation to improve the stability of the base, as illustrated in Plates 5, 6, and 7. If the shoring walls can be extended to penetrate through the water-bearing strata, and tie them into an impermeable layer below it, then dewatering is not necessary, and relief wells can be installed to relieve the water pressures in the sand strata. However, in cases where it is either not possible or practical to extend the shoring walls to penetrate through the water-bearing strata (below the base), dewatering is inevitable. Such was the case illustrated in Plate 6, where the sand layer was too dense and the steel sheets met refusal before penetrating through the sand layer.

Consolidation settlements can be estimated using one-dimensional consolidation theory. Powers (1985) includes several examples involving dewatering of sand strata either from above or below a compressible clay layer. Because of variations in compressibility characteristics, particularly the coefficients of consolidation, accurate estimates over the time duration of the dewatering are difficult. A difficulty that is not always recognized in such cases is related to the estimation of the radius of influence of the dewatering, and the inability to accurately estimate the drawdowns beyond the limits of the excavation. For these reasons, dewatering is often considered as the last resort, in cases where the effects of dewatering can affect existing adjacent structures, and in such cases, mitigation measures to protect the adjacent structures may be necessary. More detailed discussion of the effects of dewatering is provided by Powers (1985 and 1992) and Gould et al. (1992).

Deformations Caused by Removal of Old Foundations

Although not widely recognized, or reported in the literature, removal of old foundations can often cause unexpected large deformations. The task of demolition of old foundations is often delegated to the piling contractor, or a demolition expert, and is often treated in the construction documents as incidental work. Because of this practice, little attention is paid to the potential deformations that can be caused by the removal of old foundations. The author has witnessed large deformations caused by the voids left during removal of old foundations, and by the significant vibrations caused by hoe-ram equipment that often is used to break old concrete foundations. During one project in the Bay Area, the demolition of an old foundation wall immediately adjacent to an existing old brick building resulted in large settlements that caused large cracks in the brick walls of the adjacent building. The cracks were so severe that the construction team and the owner felt it necessary to implement structural remedial measures before resuming construction of the new building. Finno (2011) also cited a number of cases from his experience in the Chicago area.

In San Francisco and other major cities, new construction often requires demolition of existing structures, including basement structures. In many cases, the existing basement walls can be left in place and incorporated into the shoring system, although in other cases, the basement walls have to be removed to make room for the construction of the new shoring wall. In a recent case in the Bay Area, after demolition, large vertical cracks appeared in the area behind the removed old basement walls.

CONSTRUCTION INNOVATIONS

The introduction of in situ deep-soil mixing and jet grouting provide new opportunities for innovative design approaches to minimize and control ground deformations within acceptable limits. These two methods are briefly reviewed below.

The Cement Deep Soil Mixing (CDSM) Method

This general category includes methods that involve mixing cement slurry, or cement-bentonite slurry with in situ soils to form a series of overlapping panels that are reinforced with steel soldier piles to construct shoring walls for excavation support. Currently available equipment includes: (1) overlapping multi-auger equipment (Taki and Yang 1991, and Japanese Coastal Development Institute of Technology (JCDIT) 2002); (2) the CSM method described by Kvinsland et al. (2010); and (3) the TRD method described by Garbin (2010).

The mixing of soil in place with cement (cement-bentonite) slurry has distinct advantages over other shoring construction methods: (1) it eliminates the need for excavation of a slurry trench and the associated deformations that occur during construction of diaphragm walls; (2) eliminates the need for excavating deep holes, as is the case for secant pile walls; and (3) avoids the potential for settlements caused by vibrations as is the case with driven-steel sheetpile walls. The in situ mixing methods produce minimal vibrations, very low noise, and involve very low or practically no soil displacement or loss of ground during mixing. In a recent prototype test for a major project in San Francisco (Kluzniak et al. 2010), shoring walls 0.9 m thick and up to 43 m deep were constructed using the CDSM method. A comprehensive instrumentation program, involving inclinometers and surface settlement markers, showed lateral deformations and settlements limited to less than 5 mm as close as 1 m from the CDSM panels. The author's experience, which involves a large number of CDSM projects, is consistent with the results of this test.

Other applications of CDSM for excavation support are described by McGinn and O'Rourke (2003) and O'Rourke and O'Donnel (1997), involving construction of a soil-cement buttress to stabilize a deep excavation in Boston that experienced large deformations.

Jet Grouting

Jet grouting (Welsh 1992) is a method that involves a combination of mixing cement slurry with soil in place, as well as displacement/replacement of soil with cement slurry. In cases that involve soft soils, displacement/replacement of the soft soils is the dominant characteristic of the method. As discussed earlier, jet grouting has the potential to cause significant lateral deformations, undrained heave, and possibly subsequent consolidation settlements. However, when properly applied, the method offers great potential for improving base stability and controlling excavation-induced deformations. Experience has shown that by proper attention to detail, by using appropriate tools (triple tube, rather than single- or double-tube method) and by properly sequencing the jet grouting operations, the displacement of soil can be minimized, although it may be impossible to eliminate soil displacement and its adverse effects. Cases reviewed in this paper show that the most promising application of this method is for construction of pre-installed jet-grouted struts between the shoring walls, at depth, particularly below the base of the excavation.

CURRENT PRACTICE

This section is based largely on the author's personal experience, and review of the literature on the subject matter, which is quite extensive. In addition to numerous articles in professional journals, the proceedings of four specialty conferences on Earth Retention (1970 conference on lateral stresses and design of the earth retaining structures; Lambe and Hasan 1990; Finno et al. 2010; and O'Rourke and Hobelman 1992), as well as the FHWA Reports prepared by Goldberg et al. (1974), provide a wealth of information from which the state of practice can be assessed. A handbook on earth retention systems (Macnab, 2002) includes many examples of practical applications of different shoring systems, reflecting some of the practices in the U.S.A. The available literature, which is consistent with the author's experience, suggests the following design process in the majority of cases.

1. The geotechnical engineer for the project prepares a report, which provides information on subsurface conditions, design parameters, and often includes recommendations for lateral pressure diagrams to be used for design of the shoring.
2. The owner's designer (typically a licensed structural engineer) prepares specifications for the design and construction of the shoring system. The specifications typically include requirements for the stability of the excavation; for control of groundwater; disposal of excavated spoils and groundwater; requirements for instrumentation and monitoring (if applicable); protection of the planned structures; the qualifications of the shoring contractor and his designer; and requirements for submittals and the review process and approval of the submittals for the excavation support system (ESS). Typically, the specifications delegate the responsibility for the design and construction of the shoring and completion of the excavation to the contractor. In essence, design and construction of the excavation and shoring is a design-build contract.
3. After the contract is awarded, the contractor's designer(s) prepares a series of submittals for the design of the shoring, dewatering, and instrumentation (if instrumentation and monitoring is also delegated to the contractor), for review by the owner's design engineers. Even though the owner's designers review the submittals and provide comments, it is rare that the owner will compel the contractor to make changes to the design, unless issues of safety are involved. Often the review and approval process may become protracted and contentious, as the owner's designers try artfully (or not so artfully) to steer the contractor's designer to address all the issues that are of concern to them, without becoming contractually entangled in the design of the shoring.
4. Even when the geotechnical report includes recommendations for earth pressure diagrams for design of the shoring, the contractor's designer often develops his own diagrams based on design parameters included in the report. A common pitfall for geotechnical engineers is that the design parameters may not be entirely

consistent with the recommended pressure diagrams, which can be a source of disagreement and disputes. Although the contractor's designer is required to evaluate the design of the shoring system for each excavation stage, it is rare that the geotechnical report will provide that level of detailed recommendations, because the actual sequence of excavation is not known to the geotechnical engineer, and that also can be a source of misinterpretation and disputes.

5. Sometimes the specifications (the author is aware of a number of such projects in the Bay Area) require the contractor to estimate excavation-induced deformations. The shoring designers typically perform a structural analysis using commercially available computer programs for structural analysis that basically treat the shoring system as a beam with simple supports at the strut levels, loaded with the active-passive pressure diagrams developed by the shoring designer. An iterative approach is followed to estimate the required embedment of the shoring wall, and to also estimate deformations. This method does not have the ability to account for the deformations that develop deep below the base of the excavation, and often below the shoring wall. Therefore, such analyses are not consistent with the actual behavior of the system, and are thus inappropriate. Although most geotechnical engineers recognize the limitations of the method, it has become so entrenched that it is difficult to replace it, as long as the design remains in the province of the contractor, and as long as the shoring (structural) engineer bears sole responsibility for the design of the shoring.
6. It is rare for geotechnical reports to include estimates of ground deformations caused by excavations, although the author is aware of a number of exceptions when major projects are involved. In the Bay Area, the author is aware of at least seven projects where the geotechnical report included estimates of ground deformations, and guidelines for minimum specification requirements regarding the system stiffness (EI of the wall, spacing of struts, and embedment of the shoring wall). The Clough and O'Rourke (1990) method is simple and relatively easy to apply, and geotechnical engineers have an opportunity to improve the practice of the design of shoring systems by including estimates of deformations and by providing information that would help the contractor's designer to adopt the recommendations in the report, or at least make an attempt to understand and apply the Clough and O'Rourke (1990) method. In this regard, it is important to note that the Clough and O'Rourke (1990) method provides estimates of excavation-induced deformations only. Other sources of deformations also need to be considered.
7. Unfortunately, in current practice, the plans and specifications rarely include specific requirements for the evaluation of base stability, and stability against hydrostatic uplift. This may be a tricky point, because it is presumed that the geotechnical engineer has checked the stability of the excavation to verify that the proposed construction is technically feasible and economical. Instability of the base of the excavation can add substantial costs for ground improvement, or for shoring walls that may need to be stiffer and deeper than anticipated by the contractor at the bid stage, which could be a source of claims and disputes. As pointed out by Marr and Hawkes (2010), failure to recognize instability of the base can lead to serious consequences.

8. Alternative approaches to the "conventional practice" of delegating the design of the shoring to the contractor have been used with considerable success on some major projects, as early as 1988. Koutsoftas (1999) and Koutsoftas et al. (2000) describe three major excavations in soft soils where the owner either took responsibility for design of the shoring (MMT project), or included minimum requirements for the system stiffness and embedment of the shoring wall. Marr and Hawkes (2010) and Bryson and Zapata-Molina (2010) present recommendations for the design of excavation support systems for deformation control, which are endorsed by the author.
9. For major projects, particularly in urban areas where the impacts on adjacent structures may be the driving factor in the design, an instrumentation and monitoring program is specified, and is often included in the contract documents, and thus becomes the responsibility of the contractor. However, in major projects, instrumentation and monitoring is often focused on evaluating the impacts on adjacent structures. Public involvement during the review period of environmental impact reports is forcing owners to pay more attention to the design and performance of excavations. The stakeholders (owners of adjacent structures and utilities) often demand an assessment of the impacts of construction on their structures during the design stage. It is therefore not practical or advisable to defer this aspect of excavation planning and design until the contract is awarded, because the contractor would not have the necessary time to attend to the demands of the process of satisfying third-party requirements. Pre-construction and post-construction surveys are becoming a necessity for protection of both parties (owner and stakeholders), and for allocation of liability in the event of poor performance. In such cases, the instrumentation must be in place well in advance of construction to establish baselines; and under these conditions, the project owner's design engineer is the logical choice for installing and monitoring the instrumentation. However, it must be recognized that without detailed and accurate records regarding the sequence of construction activities to establish the cause-and-effect relationship, the results of the instrumentation and monitoring program can be of limited effectiveness. It is therefore essential that the design engineer, who is often responsible for interpreting the results of the instrumentation, be given access and be charged with the responsibility for the collection of independent construction records that will allow detection of the causes of the deformations, especially when trigger levels are reached, so that effective remedial measures can be implemented. It is indeed regrettable that competing interests, concerns for liability, and cost considerations cloud the decision-making process for assignment of the responsibility for the instrumentation and monitoring program. Dunnicliff (2011) discusses some of the issues that pertain to the selection of the entity responsible for instrumentation. The geotechnical engineer and the project owner carry a great deal of liability, and place great faith that the contractor will execute the project in a responsible manner to protect their interests. It is therefore in the best interests of the project, the owner, and the geotechnical engineer of record to assign the instrumentation and monitoring responsibility to the geotechnical engineer under direct contract with the owner.

10. For excavations in soft clays, the in situ, undrained shear strength of the clay is the most critical parameter affecting the design. The general practice is to use Unconsolidated Undrained (UU) triaxial tests to estimate undrained shear strength. However, UU tests are unreliable because of the effects of sample disturbance and test procedures on the measured strengths (see Germaine and Ladd 1988, and Ladd and Lambe 1963). Figure 34 shows undrained shear strengths determined from UU tests performed on high-quality piston samples of Bay Mud, and compares them to undrained shear strengths determined from field vane shear tests at the MMT site in San Francisco (Koutsoftas et al. 2000). The scatter in the UU data (which is considerable) does not reflect in situ variability of the soil, because the field vane shear strengths show very little scatter. The scatter in the UU data is the result of disturbance during sampling and specimen preparation, but also of test procedures. The continued use of UU tests is unfortunate, but this practice is unlikely to change because of competitive pressures, and because UU tests are easy to perform, although difficult to perform well. As Finno (2010) laments, laboratory testing is becoming a lost art.

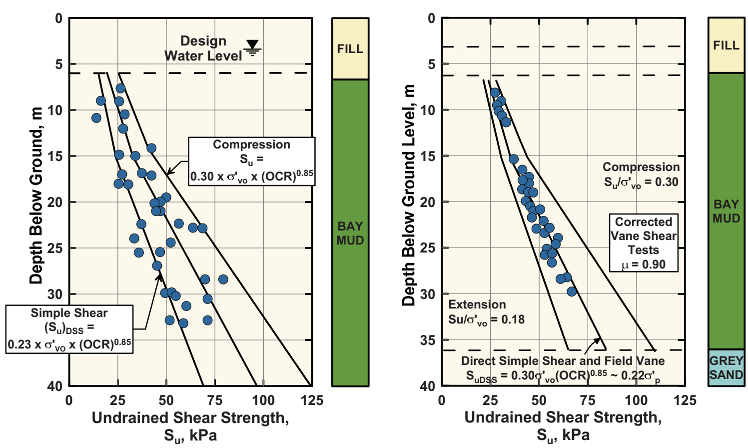


FIG.34. Comparison of Undrained Shear Strengths of Bay Mud Determined from UU Laboratory Tests and In Situ Vane Shear Tests

Unfortunately, the author cannot dispute this claim, although in many areas around the world, including the Bay Area, extensive experience has been accumulated with the application of undrained shear strengths measured with the field vane apparatus that provides geotechnical engineers with the option to use in situ vane shear testing, which can produce, under expert guidance, high-quality undrained shear strength data. Furthermore, it has been documented in the literature that many soft clays around the world follow normalized behavior, and therefore the SHANSEP (Ladd and Foott 1974) method can be used to estimate in situ undrained shear strengths in-lieu of the current practice that focuses on the results of UU tests to estimate undrained shear strengths. The basic information required for successful application of SHANSEP is the stress history of the deposit (i.e., the variation of preconsolidation pressures, σ'_p, with depth), and the normalized undrained shear strength ratio, S, for the normally consolidated soil. Ladd et al. (1977) summarize data for many clays that can be used to estimate the value of S, if site-specific data are not available. Also Mesri (1975 and (1989) has shown that as an approximation, the in situ undrained shear strength can be estimated as, Su = 0.22 × σ'_p. This is consistent with data summarized by Koutsoftas and Ladd (1985) and Koutsoftas et al. (2000). This simple equation also incorporates the effects of anisotropy and strain compatibility. However, high-quality undisturbed samples and testing are necessary to determine the in situ stress history. The question remains as to the selection of the appropriate average undrained shear strength along the potential failure surfaces shown on Figure 4 for base stability analysis. Based on the author's experience, the average strength can be approximated by the value of the strength at a depth equal to one-half of the maximum depth of the failure surface below the base of the excavation, or approximately 35% of the width of the excavation, B.

11. Earthquake loading is not normally considered in the design of temporary shoring systems, even in parts of the world where severe earthquake hazards are anticipated. The author knows of only two cases where the temporary shoring system had been designed for earthquake forces. The first was the MUNI Metro Turnback project in San Francisco. The second is the Transbay Transit Center excavation that is currently under construction, also in San Francisco. The primary concern is with safety. Typically, the strut loads are checked for a relatively small earthquake with 50- to 200-year return periods. The low return periods are justified in view of the relatively short period of time that the excavation may remain open. The shoring designer needs particularly to check the connections to make certain that the supports for the walers, and connections of the struts to the walers, have sufficient flexibility to accommodate the anticipated racking deformations. Also, for wide excavations where pin piles are necessary, to provide intermediate support to long struts, the details of the pin pile-strut connections need to be checked to verify that the anticipated deformations can be accommodated without loss of support. In the future, in areas where the risk of earthquake hazards is high, building departments are likely to begin to require that the design of the shoring system should consider earthquake loading. Such was the case for a recent major underground project in San Francisco.

SUMMARY AND FUTURE TRENDS

For the vast majority of excavation projects, the responsibility for the design of the shoring is delegated to the contractor as a design-build contract. This practice is unlikely to change because it is driven by the owner's and the designer's justified fears of liability, because the design of the shoring is not straightforward (considered by many as a black box), and because performance is affected by many factors that are solely under the control of the contractor. The process is further complicated by the fact that building departments require that the design be prepared and stamped by a licensed structural engineer, and therefore the geotechnical engineer has no control over the design process, but is always in the firing line when poor performance occurs.

It has long been recognized that the current practices are problematic, because they do not promote teamwork, but foster an environment of gamesmanship and mistrust. Fortunately, during the past 20 years there has been great progress in our understanding of the behavior of excavations, as well as in the development of new analytical tools that make alternatives to the traditional approach for design and construction of excavations possible, as discussed below.

Starting as early as 1988, a small number of projects began to deviate from the traditional practice, as discussed earlier, and the success realized from those projects makes a strong case for alternative design and contracting practices. These alternative procedures became available to owners and geotechnical engineers after publication of the Clough and O'Rourke (1990) method for estimating excavation-induced deformations. The method is simple, robust, reliable, and most geotechnical engineers with an advanced degree should be able to apply this method successfully. In addition, experience with the application of numerical analyses is accumulating at an accelerated rate, and it is quite likely that numerical analysis techniques will become more user-friendly, and see greater applications for design. Geotechnical engineers and project owners have the tools and the opportunity to confidently develop minimum requirements for the design of the shoring system that can control the excavation-induced deformations within desirable limits, without having to assume additional responsibilities and liability beyond what they are exposed to with the current process. Actually, their liability is likely to be reduced!

The late professor Ralph B. Peck played an important role in the transition from delegating the design of the shoring system to the contractor towards a system in which the owner and designer become active players in the design of the shoring system. He was instrumental during the design (1988) of the MUNI Metro Turnback project in San Francisco, in convincing the owner that it was in his best interests to authorize the designer of record to design the shoring system. In his opening address at the 1990 conference on Earth Retaining Structures, he stated the following:

> "It is now possible, largely as a result of the work of Wayne Clough and his associates, to select the bracing system and excavation sequence at a given site to ensure that the settlement or lateral movement will not exceed predetermined limits. The economy of various possible arrangements to achieve this end can be investigated readily. For many problems a satisfactory solution can be reached by use of summary diagrams relating movement, wall stiffness, support capacity, soil stiffness, and factor of safety against failure. More detailed studies can be made with an interactive computer program."

Although some progress has been made towards achieving the advancements envisioned by Peck (1990), the profession in general has made little progress in implementing the approach advocated by Peck on a broader scale. More significant progress can be made only when the geotechnical engineers, the project design engineers, and owners understand and recognize that the approach advocated by Peck (1990) and strongly endorsed by this paper not only does not increase the owner's and designer's liability, but can actually reduce liability for all parties, improve the quality of the excavation construction, and level the playing field among bidders by eliminating bids that are likely to ignore the risks associated with deep excavations in soft soils.

In urban centers, the impacts of excavation-induced deformations on adjacent structures are becoming a matter of concern for the owners of the adjacent structures, who often demand that the impacts on their property be addressed at an early stage in the design process. This trend is likely to become more entrenched, and it is inevitable that designers will be called upon to estimate not only the anticipated deformations, but also to evaluate the impacts of the deformations on adjacent structures. The geotechnical engineer of record has the opportunity to take the lead in estimating excavation-induced deformations, and to be an important player in evaluating impacts on adjacent structures, although the evaluation of structural response may best be carried out by a structural engineer. However, research in the response of structures to deformations has traditionally been led by geotechnical engineers. O'Rourke et al. (1977), Clough and O'Rourke (1990), and Cording et al. (2010), among others, describe many of the advances in our understanding of the response of structures to excavation-induced deformations.

For major projects in urban areas, the most desirable option is to assign the responsibility for evaluating excavation-induced deformations to the geotechnical engineer, and to develop specification requirements for the shoring that would limit the deformations within acceptable limits. The extent of the specification requirements can vary depending on the confidence and expertise of the designer, but at a minimum, it would be advantageous to include the following:

- Specification of the EI of the shoring wall per unit length of the wall. Although this requirement in itself should be sufficient, the process can be enhanced considerably by including design charts in the geotechnical report such as the one shown on Figure 35, which illustrates the benefits of increasing wall stiffness in reducing wall deformations.
- Specification of the minimum depth of the shoring wall, which can be based either on analytical considerations, or can be based on experience, or practical installation considerations.
- Specification of the maximum allowable strut spacing, and/or a minimum number of levels of horizontal supports.

The above criteria can be included in the specifications as minimum requirements, which does not relieve the contractor from the responsibility of performing his own analyses, and deciding whether a more robust design is required to meet the project design objectives.

Other design and construction criteria that would be beneficial if included in the specifications are the following: (1) requirements for preloading the struts, and the amount of preloading; (2) maximum depth of excavation of each lift before the supports are installed; and (3) in long excavations, requirements for limiting the length of excavation ahead of the struts.

If the above items are specified for the design and construction of the shoring, a reliable independent estimate of excavation-induced deformations can be made by the geotechnical engineer, and it can be reasonably expected that a competent shoring contractor (and designer) would construct the excavation to meet the owner's expectations.

Instrumentation and monitoring are becoming an integral part of the design and construction process. As discussed in this paper, excavation-induced deformations is but one of the sources of deformations associated with excavations in soft soils. Proper

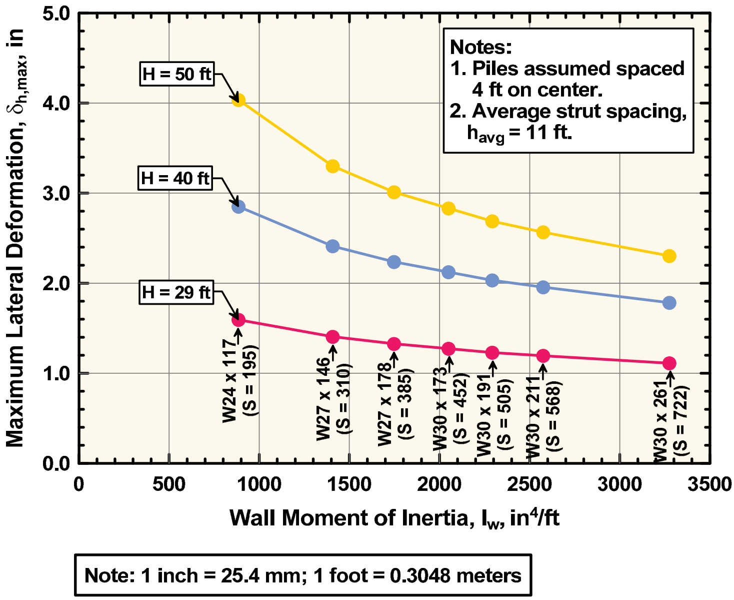


FIG.35. Effects of Soldier Pile Stiffness on Maximum Lateral Deformations: Excavation in Deep Bay Mud

planning, installation, timely monitoring and interpretation of the data, combined with comprehensive documentation of relevant construction activities, is the only defense of the owner and his designers from sources of deformations unrelated to the project, and which might not have been anticipated during the design phase. New advances in automation are in the development stage that allow remote monitoring of the instrumentation, but also remote and timely monitoring of the construction activities (Hashash and Finno 2008, Hashash 20011). Implementation of these innovations is in its infancy, but it is anticipated that automation in monitoring will become accepted as the method of choice for major projects, in a rather short period of time.

Preconstruction and post-construction surveys are becoming essential for managing the issues that might arise from potential impacts (real or alleged) of excavations on adjacent structures. The preconstruction activities cannot be delayed until the construction contract is awarded, because coordination with the owners of adjacent structures is time-consuming. Furthermore, in order to protect the project from impacts that may not be related to the excavation work, but may be ongoing, it is important that the monitoring program begin well in advance of construction to establish reliable baseline data that can be used to differentiate between excavation-induced deformations, and pre-existing conditions that may be continuing concurrent with the excavation work. In a recent case, the project owner began monitoring the settlements of an adjacent building more than a year in advance of construction of his project to establish the rate of ongoing settlement, as one of the baselines for evaluating the impacts of his excavation on the adjacent structure.

In conclusion, it is the author's opinion that in the not-too-distant future, there will be significant changes in design and construction practices that are likely to improve the performance of excavations, and provide project owners and their designers with the necessary tools to manage in a comprehensive manner the design and construction of excavations, improve the quality of the project, and reduce liability for all the parties involved in design and construction of excavations.

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Risk Assessment and Mitigation in Geo-Practice

Suzanne Lacasse[1], Farrokh Nadim[2] and Kaare Høeg[3]

[1]Managing Director, Norwegian Geotechnical Institute (NGI), suzanne.lacasse@ngi.no
[2]Director, International Centre for Geohazards (ICG), NGI, farrokh.nadim@ngi.no
[3] Expert Adviser, Norwegian Geotechnical Institute (NGI), kaare.hoeg@ngi.no
Address: NGI, P.O.Box 3930 Ullevaal Stadion, NO-0806 Oslo, Norway.

ABSTRACT: The notions of risk assessment, risk management, acceptable risk and risk mitigation are addressed, and the State-of-Practice of geo-risk assessment and risk mitigation is exemplified with case studies. The examples illustrate the assessment of hazard, vulnerability and risk, and the treatment (mitigation) of risk under different design situations, including underwater slopes subjected to earthquake hazard, the stability of quick clays, and the potential breach of embankment dams.

INTRODUCTION

Society is exposed to both natural and human-induced risks, and while the risk can never be eliminated, the engineer's goal is to reduce the risk to levels that are acceptable or tolerable. Coordinated, international, multi-disciplinary efforts are required to develop effective societal response to geo-risks. The needs in practice are accentuated by recent events with disastrous impact:

- Tsunamis (e.g. Indian Ocean 2004 and Tohoku, Japan 2011) cause enormous personal and societal tragedies. The tsunami in Japan showed the vulnerability of a strong and prosperous society, and how paralysing one entire nation has repercussions around the world. Germany even decided to close all its nuclear power plants. The effects of the cascading events (earthquake-tsunami-nuclear contamination) will reverberate for years, with irreparable political, economical and psychological damage. Since 2004, at least eight tsunamis have caused fatalities.
- Examples of other cascading hazards include: flood events in France (1999, 2002) causing severe damage and triggering release of hazardous material.
- In 2010 and 2011, extreme floods ruined many parts of the world, including regions in Australia, Pakistan, China, Sri Lanka, India, Indonesia, Venezuela and Columbia.
- The Baia Mare tailings dam failure in Romania (2000) released cyanide contaminated fluid, killing tons of fish and poisoning the drinking water of 2 million people in Hungary. The Aznalcóllar tailings dam failure in Spain (1998) released 68 million m^3 of contaminated material in the environment.
- Recent earthquakes in El Salvador (2001), India (2001), Iran (2003), Pakistan (2005), China (2008) and Haiti (2010) caused numerous fatalities and made many

more homeless. Besides Haiti, earthquakes ravaged Chile, China, Sumatra and Iran in 2010. In 2011, earthquakes damaged Japan and Christchurch in New Zealand.

- The losses in 2010 were more than double those of 2009, for about the same number of events (Tables 1 and 2). The frequency and severity of climate-driven hazards (sea level rise, storms, floods) are increasing. In the past decade, natural disasters caused 1 million fatalities and affected 2.5 billion people across the globe. In 2010, 373 natural disasters were registered, 300.000 persons died and 200 million others were affected. Total costs were 650 billion NOK (Munich RE 2011).

Many lives could have been saved if more had been known about the risks associated with the hazards and if risk mitigation measures had been implemented. Developing countries are more severely affected than developed countries, especially in terms of lives lost (UNISDR 2009).

Table 1. Natural catastrophes in 2010 (Munich Re 2011).

Events and losses	2010	2009	Average 2000-2009	Average 1980-2009
Number of events	950	900	785	615
Overall losses (US$m)	130,000	60,000	110,000	95,000
Insured losses (US$m)	37,000	22,000	35,000	23,000
Number of fatalities	295,000	11,000	77,000	66,000

Table 2. Natural disasters between 1991 and 2000 (IFRC 2001).

Countries	No. of disasters	No. of lives lost
Low and medium developed countries	1838	649,400
Highly developed countries	719	16,200

Most of the increase in losses is due to the increase in the exposed population. Mitigation and prevention of the risk have not attracted widespread and effective public support in the past. However, it is now accepted that a proactive approach to risk management is required to reduce the loss of lives and material damage associated with natural hazards. A milestone in recognition of the need for natural disaster risk reduction was the approval by 164 United Nations (UN) countries of the "Hyogo Framework for Action 2005-2015: Building the Resilience of Nations and Communities to Disasters" (ISDR 2005). This document defines international working modes, responsibilities and priority actions for the coming 10 years. The European statistics from the past 100 years (Table 3) indicate that the frequency of landslides in Europe (about 20 major events per year) is the highest among natural hazards, compared to floods, earthquakes and cyclones. However the number of fatalities and the quantity of material damage is far greater under earthquakes.

Since the 80's, hazard and risk assessment of the geo-component of a system has gained increased attention. The offshore, hydropower and mining industry were the pioneers in applying the tools of statistics, probability and risk assessment. Whitman (1996) offered examples of how probabilistic analysis can best be used in geotechnical engineering, and what types of projects the approach is appropriate for, and concluded that probabilistic methods are tools that can effectively supplement traditional methods for geotechnical engineering projects, provide better insight into the uncertainties and their effects and an improved basis for interaction between engineers and decision-makers. The most effective applications are those involving relative probabilities of failure or illuminating the effects of uncertainties in the parameters. Gradually, environmental concerns and natural hazards soon implemented hazard and vulnerability assessment. In the 21st century, the awareness of the need for mitigation of natural hazards has greatly increased. Nowadays, the notion of hazard and risk is a natural question in the design of most foundation aspects. Griffiths *et al.* (2012) presents, in a companion paper, the state-of-the-art on risk assessment in geotechnical engineering with particular emphasis on stability analysis of highly variable soils.

Table 3. Socio-economic impact of natural hazards in Europe (1900-2000).

Hazards	Loss of life	Natural damage
45 floods	10,000	105 B€
1700 landslides	16,000	200 B€
32 earthquakes	239,000	325 B€

TERMINOLOGY

The terminology used in this paper is consistent with the recommendations of ISSMGE Glossary of Risk Assessment Terms (listed on TC304 web page: http://jyching.twbbs.org/issmge/home_2010.htm):

Danger (Threat): Phenomenon that could lead to damage, described by geometry, mechanical and other characteristics. Its description involves no forecasting.

Hazard: Probability that a danger (threat) occurs within a given period of time.

Exposure: The circumstances of being exposed to a threat.

Risk: Measure of the probability and severity of an adverse effect to life, health, property or environment. Risk is defined as Hazard × Potential worth of loss.

Vulnerability: The degree of loss to a given element or set of elements within the area affected by a hazard, expressed on a scale of 0 (no loss) to 1 (total loss).

In UNISDR terminology on Disaster Risk Reduction (2009), "disaster" is defined as "a serious disruption of the functioning of a community or a society causing widespread human, material, economic or environmental losses which exceed the ability of the affected community or society to cope using its own resources". The term "natural disaster" is slowly disappearing from the disaster risk management vocabulary. Without the presence of humans, one is only dealing with natural processes; these become disasters only when they impact a community or a society.

RISK ASSESSMENT AND MANAGEMENT

Risk management refers to coordinated activities to assess, direct and control the risk posed by hazards to society. Its purpose is to reduce the risk. The management process is a systematic application of management policies, procedures and practices. A risk management framework comprises the following main tasks: (a) Danger or hazard identification; (b) Cause analysis of the dangers or hazards; (c) Consequence analysis, including vulnerability analysis; (d) Risk assessment combining hazard, consequence and uncertainty assessments; (e) Risk evaluation or is the risk acceptable or tolerable?; and (f) Risk treatment or what should be done?

Risk management integrates the recognition and assessment of risk with the development of appropriate treatment strategies (e.g. Fig. 1). Understanding the risk posed by natural events and human-induced activities requires an understanding of its constituent components, namely characteristics of the danger or threat, its temporal frequency, exposure and vulnerability of the elements at risk, and the value of the elements and assets at risk. The assessment systemizes the knowledge and uncertainties, i.e. the possible hazards and threats, their causes and consequences. This knowledge provides the basis for evaluating the significance of risk and for comparing options. Risk assessment is specifically valuable in detecting deficiencies in complex technical systems and in improving the safety performance, e.g. of storage facilities. Risk communication means the exchange of risk-related knowledge and information among stakeholders. Despite the maturity of many of the methods, broad consensus has not been established on fundamental concepts and principles of risk management. The field suffers from a lack of clarity on key scientific pillars, e.g. what risk means and how risk is best described, making it difficult to communicate across disciplines and between risk analysts and stakeholders.

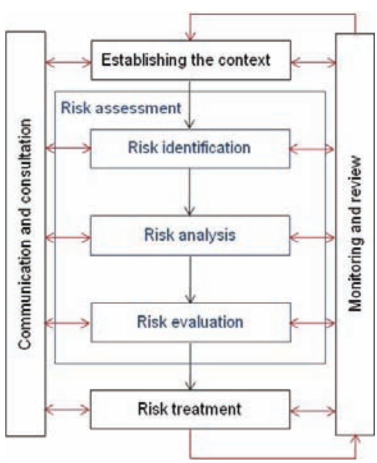


FIG. 1. Risk management process (ISO, 2009).

The ISO 31000 (2009) risk management process (Fig. 1) is an integrated process, with risk assessment, and risk treatment (or mitigation) in continuous communication and consultation, and under continuous monitoring and review. Due to the aleatory (inherent) and epistemic (lack of knowledge) uncertainties in hazard, vulnerability and exposure, risk management is effectively decision-making under uncertainty. Today's risk assessment addresses the uncertainties and uses tools to evaluate losses with probabilistic metrics, expected annual loss and probable maximum loss. Future-oriented quantitative risk assessment should include uncertainty assessments, consider technical feasibility, costs and benefits of risk-reduction measures and use this knowledge for the selection of the most appropriate risk treatment strategies.

Fell *et al.* (2005) made a comprehensive overview of the state-of-the-art in landslide risk assessment. Düzgün and Lacasse (2005) listed a large number of proposed risk formulations. The first step in any risk reduction process is a quantitative risk assessment. For example for landslides, one would (1) define scenarios for triggering the landslide and evaluate their probability of occurrence; (2) compute the run-out distance, volume and extent of the landslide for each scenario; (3) estimate the losses for all elements at risk for each scenario; and (4) estimate the risk.

Two emerging issues

General frameworks for risk and multi-risk assessment and risk treatment are needed. A joint analysis and quantification of all the human-induced and natural risks that can affect an area does not exist. Risk assessment is still fragmented, where geo-risks are considered independently of each other and separately from industrial or human-induced risks. A multi-risk approachis required for sound environment management and land use planning, as well as for a competent emergency preparedness and response. Multi-risk evaluation is a new field across a number of expertise areas, and with incomplete theory. The issues that need to be addressed fall into three classes: (a) *Interaction and amplification of risks, including cascading effects* (i.e. the recognition that multi-risk is more than a simple aggregation of single risks); (b) *Dynamic vulnerability to multi-risks* (i.e. how does the vulnerability change with time when one or more events hit., and how does the vulnerability change when different threats occur almost simultaneously); and (c) *Application of multi-risk assessment* (i.e. implementation in practice to mitigate risk more effectively).

In many contexts experts acting alone cannot choose the "appropriate" treatment. A multi-disciplinary approach is heavily underlined by all UN organizations working in this field. A deciding factor on whether an extreme event turns into a disaster is the social vulnerability of the population at risk, i.e. the capacity to prepare for, respond to and recover from the extreme event. Policy-makers and affected parties are recognizing that traditional expert-based decision-making processes are insufficient in controversial risk contexts. The approach tends to de-emphasise the affected interests in favour of "objective" analyses, and the decisions lack popular acceptance. The approaches can slight the local and anecdotal knowledge of those most familiar with the situation and can in the worst case produce irrelevant or unworkable outcomes. Conflicting values, interests and expert evidence characterise many risk decision processes. The decisions become more complex with long time horizons and uncertainties on climate and demographic and other global changes. Risk communication and stakeholder involvement have been widely acknowledged for supporting decisions on controversial environmental risk events. The value of a participatory approach is multiple. The decision-makers are those who need to manage the threats. The stakeholders are the most knowledgeable on the decision-making process, and on the concerns (or lack thereof) for hazards. When stakeholders are contributors to the development of participatory documents, the value of the document increases because the stakeholders have ownership of the written work, making it more likely that guidelines become more effective.

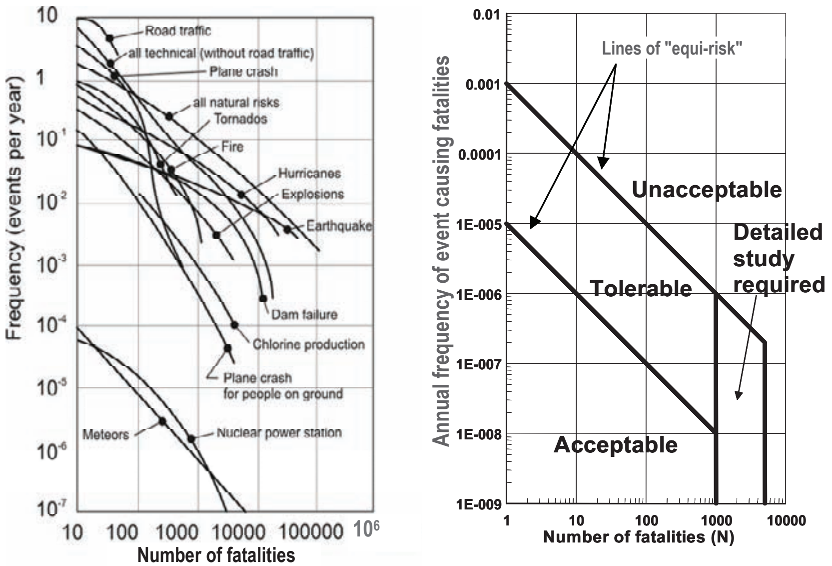
Natural and human-induced risks

Many factors complicate the risk picture. Urbanisation and changes in demography are increasing the exposure of vulnerable population. Climate change is altering the geographic distribution, frequency and intensity of hydro-meteorological hazards and threatens to undermine the resilience of poorer countries and their citizens to absorb loss and recover from disaster impacts. The earthquake-tsunami-nuclear contamination chain of events in Japan is a telling example of cascading hazards and multi-risk: the best solution for earthquake-resistant design (low/soft buildings) may be a less preferable solution for tsunamis (high/rigid buildings). The sea walls at Fukushima gave a false sense of security. The population would have been better prepared if told to run to evacuation routes as soon as the shaking started. The 2008 China earthquake is a second recent example of cascading hazards where an earthquake caused large rock falls, landslides and landslide dams ("quake dams") that could lead to disastrous dam breach. Storage facilities for hazardous materials, e.g. underground spaces and tailings dams for nuclear waste, contaminated materials, industrial waste and waste water) are often in the chain of cascading events. To reduce the hazard and risk, improved design, improved models, reduced uncertainty and improved risk management are required, in addition to improved risk assessment and management.

Acceptable or tolerable risk?

A difficult task in risk management is establishing risk acceptance criteria. As guidance to what risk level a society is apparently willing to accept, one can use 'F-N curves'. The F-N curves relate the annual probability (F) of causing N or more fatalities to the number of fatalities, N. The term "N" can be replaced by other quantitative measure of consequences, such as costs. The curves are used to express societal risk and to describe the safety levels of particular facilities. Figure 2a presents a family of F-N-curves. Man-made risks tend to be represented by a steeper curve than natural hazards in the F-N diagram (Proske 2004). Figure 2b presents an interim risk criterion recommendation for landslides on natural hillsides in Hong Kong (GEO 1998). On the log-log F-N diagram (Fig. 2), lines with slope equal to 1 are curves of equi-risk (the risk is the same for all points along the line). A slope greater than 1 reflects risk aversion, where society is less tolerant when a large number of lives are lost in a single event, than if the same number of lives is lost in several separate events. An example is the public concern at the loss of large numbers of lives in airline crashes compared to the much larger number of lives lost in separate road traffic accidents.

Who should define acceptable and tolerable risk level: the potentially affected population, government, or the design engineer? Acceptable risk refers to the level of risk requiring no further reduction. It is the level of risk society desires to achieve. Tolerable risk presents the risk level reached by compromise in order to gain certain benefits. A construction with a tolerable risk level requires no action/expenditure for risk reduction, but it would be desirable to control and reduce the risk if the economic and/or technological means for doing so are available.

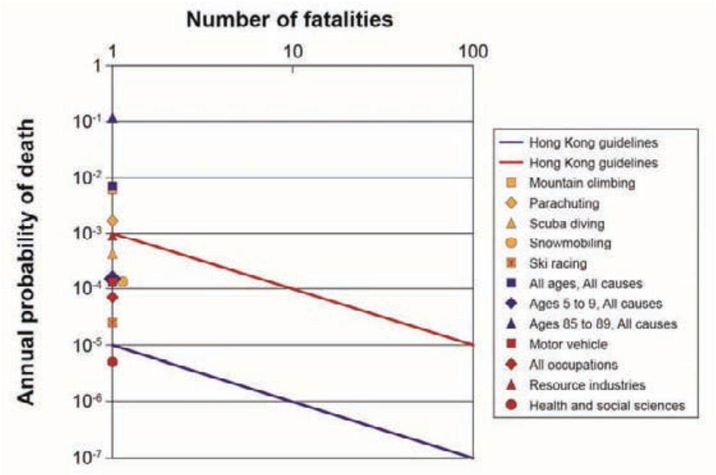


(a) US Nuclear Regulatory Comm. (1998). (b) Hong Kong criteria (GEO 1998).

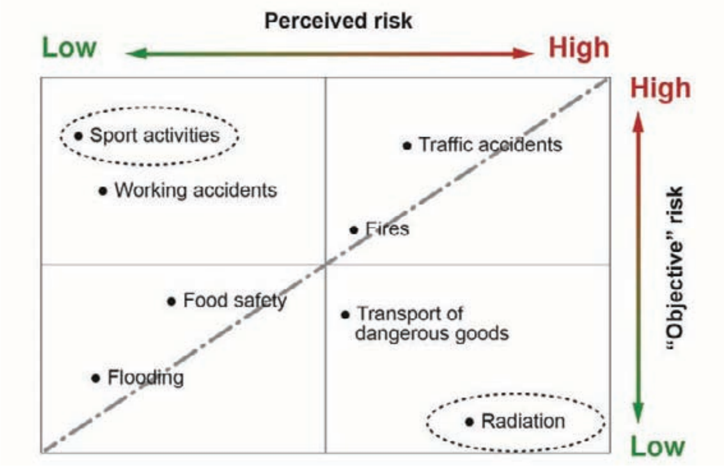
FIG. 2. Examples of F-N curves

Risk acceptability depends on factors such as voluntary vs. involuntary exposure, controllability vs. uncontrollability, familiarity vs. unfamiliarity, short vs. long-term effects, existence of alternatives, type and nature of consequences, gained benefits, media coverage, information availability, personal involvement, memory, and level of trust in regulatory bodies. Voluntary risk tends to be much higher than involuntary risk. If under personal control (e.g. driving a car), the risk is more acceptable than the risk controlled by other parties. For landslides, choosing to live close to a natural slope is a voluntary risk, while having a slope engineered by the authorities close to one's dwelling is an involuntary risk. Societies that experience geohazards frequently may have a different risk acceptance level than those experiencing them rarely.

Total risk is defined by the sum of specific risks. It is difficult to evaluate the sum, since the units for expressing each specific risk differ. Individual risk has the unit of loss of life/year, while property loss has the unit of loss of property/year (e.g. USD/yr). Risk acceptance and tolerability have different perspectives: the individual's point of view (individual risk) and the society's point of view (societal risk). Figure 3a presents an example of accepted individual risks. The value of 10^{-4}/year is associated with the risk of a child 5 to 9 years old dying from all causes. Risk perception is a complex issue. Figure 3b illustrates how perceived and "objective" risk can differ. Whereas the risk associated with flooding, food safety, fire and traffic accidents are perceived in reasonable agreement with the "objective" risk, the situation is very different with issues such as nuclear energy and sport activities.



(a) Accepted individual risks (Thomas and Hrudey, 1997; Hutchinson, J., Personal communication. Queens' University, Kingstom, Canada. May 2011).



(b) Perceived vs. "objective" risk (Max Geldens Stichting, 2002).

FIG. 3. Acceptable, tolerable and perceived risk.

GEO compared societal risks as described in a number of national codes and standards Figure 4 presents the comparison. Although there are differences, the recommended risk level centers around 10^{-4}/year for ten fatalities. It is also possible to present the confidence in a risk estimate rather than the above F-N curves. Figure 5 illustrates this with a log-log graph of risk in terms of expected annual fatality as a function of time, with a lower bound, average and upper bound risk estimate. The example is from the study done by NGI on the risk of future tsunamis on the west coast of Thailand, in the aftermath of the tsunami disaster on 26 December 2004.

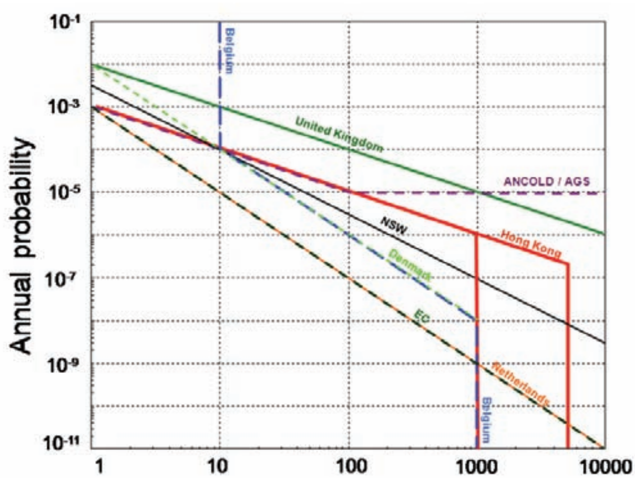


FIG. 4. Acceptable Societal Risk criteria in different countries (Ho, K. Personal communication. Gov. of Hong Kong SAR, CEDD, GEO, Nov. 2009)

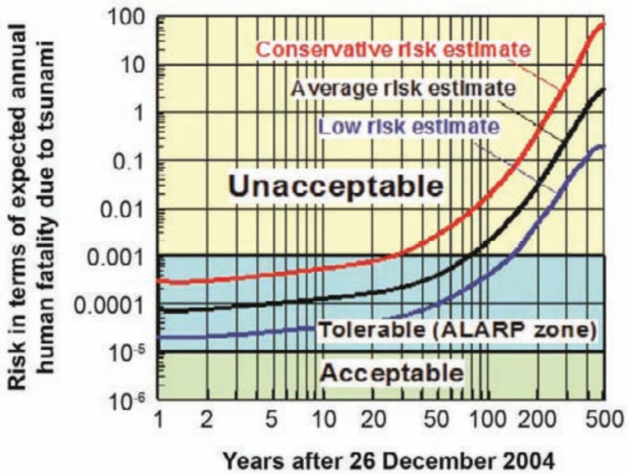


FIG. 5. Confidence in risk estimate for future tsunamis on west coast of Thailand (Nadim and Glade, 2006).

Risk mitigation

The strategies for the mitigation of risks associated with geohazards can broadly be classified in six categories: 1) active application of land use plans, 2) enforcement of building codes and good construction practice, 3) use of early warning systems, 4) community preparedness and public awareness campaigns, 5) measures to pool and transfer the risks and 6) physical measures and engineering works. The first five categories refer to non-structural measures, which aim to reduce the consequences, while the sixth includes active interventions such as construction of physical protection barriers, which aim to reduce the frequency and severity of the threat.

A mitigation strategy involves: 1) identification of possible disaster triggering scenarios, and the associated hazard level, 2) analysis of possible consequences for the different scenarios, 3) assessment of possible measures to reduce and/or eliminate the potential consequences of the danger, 4) recommendation of specific remedial measures and, if relevant, reconstruction and rehabilitation plans, and 5) transfer of knowledge and communication with authorities and society.

In many situations, an effective risk mitigation measure can be an early warning system that gives sufficient time to move the elements at risk out of harm's way. Early warning systems are more than just the implementation of technological solutions. The human factors, social elements, communication and decision-making authorities, the form, content and perception of warnings issued, the population response, emergency plans and their implementation and the plans for reconstruction/recovery are also essential parts of the system. An early warning system without consideration of the social aspects could create a new type of emergency (e.g. evacuating a village because sensors indicate an imminent landslide, but without giving the village population any place to go, shelter or means to live). Failures in communication and in response can be more dramatic than failures in the technology and maintenance. Challenges in designing an early warning system include the reliable and effective specification of threshold values and the avoidance of false alarms. The children's story about the little shepherd boy who cried "wolf" is the classic example of how false alarms can destroy credibility in a system. Initially an implicit trust exists between the affected population and the "experts", and an acknowledgment of the system exists. Lost trust can only be won back with difficulty.

CASE STUDY 1: PROBABILISTIC ANALYSIS OF SEISMIC STABILITY OF UNDERWATER SLOPE

The stability of submarine slopes under earthquake loading is a challenging issue in offshore geohazards studies. This is especially the case where seafloor installations such as platforms and pipelines are founded on a slope or within the potential run-out distance of a failed slope.

Three scenarios of earthquake-induced slope instability should be assessed for submarine clay slopes (Biscontin *et al.,* 2004):

1. Failure occurs during the earthquake: in this scenario, the excess pore pressures generated by the cyclic stresses degrade the shear strength so much that the slope is not able to carry the static shear stresses.

2. Post-earthquake failure due to increase in pore pressure at critical locations caused by seepage from deeper layers.
3. Post-earthquake failure due to creep.

Soils with strong strain-softening and high sensitivity are most susceptible to failure during earthquake shaking: excess pore pressure migration from deeper layers into critical areas, leading to instability, could occur over a time span of years or even decades in deep marine clay deposits. Post-earthquake creep-type failure is believed to be the most common mechanism for clay slopes. Special cyclic direct simple shear tests (DSS) run on marine clay specimens in laboratory suggest that the earthquake-induced shear strains correlate well with the reduction of the pre-cyclic (static) undrained shear strength (Nadim 2011). Figure 6 presents the results of such tests on marine clay from the Ormen Lange field in the North Sea (Nadim *et al* 2005). The shear strain-shear strength ratio type of diagram in Figure 6 is in fact the original method of cyclic strength presentation introduced by Harry Seed in the early 60's. In the diagram, γ_{cy} is the cyclic shear strain, N the number of cycles and γ_{tot} the total (cyclic + average) shear strain.

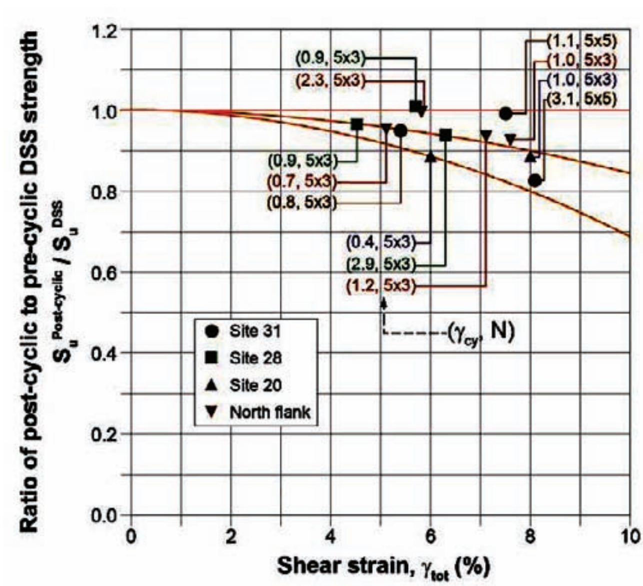


FIG. 6. Effect of permanent strains during cyclic loading on post-cyclic undrained shear strength, special cyclic DSS-tests on clay (Nadim *et al.*, 2005).

Nadim (2011) calculated the annual probability of earthquake-induced slope failure using a procedure developed through a number of joint-industry research projects and offshore geohazards studies in the North Sea, the Caspian Sea, the Black Sea, offshore Indonesia, and the Gulf of Mexico. The approach accounts for the uncertainties in all steps of the assessment and utilizes the available information to come up with a rational estimate. The analysis has eleven steps:

1. Identify the critical slopes and establish the geometry and mechanical soil properties for the critical slopes in a probabilistic format.
2. Use Monte Carlo simulation, FORM or other technique to compute the cumulative distribution function (CDF) of the static safety factor for the slope, F_{FS}.
3. Update the CDF for static safety factor using the fact that the slope is standing today. This implies that the current factor of safety, although unknown, is greater than unity. The annual probability of failure becomes the question of the likelihood that the current factor of safety will fall below unity during a reference time of one year. Its probability distribution can be computed (from FORM analysis or Monte Carlo simulation), but is truncated to reflect that the slope is stable today. This is basically a Bayesian updating procedure where the a-priori information is that FS ≥ 1. The updated (or posterior) distribution of the factor of safety is:

$$P[FS < z \mid FS \geq 1] = \frac{F_{FS}(z) - F_{FS}(1)}{1 - F_{FS}(1)} \tag{1}$$

   Seismic slope failure occurs if the safety factor falls below unity as a result of the earthquake loading effects.
4. Do a probabilistic seismic hazard assessment and identify representative acceleration time histories for return periods of interest.
5. Establish the reduction in the post-earthquake undrained shear strength as a function of the maximum earthquake-induced shear strain from laboratory tests or literature survey (e.g. Fig. 6).
6. Perform dynamic response analyses for various combinations of dynamic soil properties and representative earthquake ground motions using the Monte Carlo simulation technique. The analyses should be done for at least two return periods, as discussed in Step 9.
7. Using Steps 5 and 6, establish the distribution function for the undrained shear strength reduction factor, reflecting the effects of earthquake loading.
8. From Steps 3 and 7, establish the CDF for post-earthquake static safety factor. The conditional probability of failure (given that the earthquake with the specified return period occurred) is the value of this CDF at FS equal to 1.
9. The annual failure probability is the sum (integral) of all conditional failure probabilities for a given return period, divided by that return period. The analyses should be done for at least two return periods, ideally above and below the return period that contributes most to the annual failure probability. Iteration might be necessary as this is not known beforehand.
10. With the result of Step 9, establish a model with load and resistance that matches the computed failure probabilities at the return periods of interest. The most usual load parameter is the input annual peak ground acceleration (PGA), with typically exponential or Pareto distribution. If PGA is used as the representative load parameter, the slope resistance needs to be specified as an acceleration parameter. A log-normal distribution for resistance is commonly assumed.

11. Estimate the probability that the resistance of the slope is less than the applied load (e.g. the annual PGA), which is the annual probability of earthquake-induced slope failure.

The example below applies to a slightly overconsolidated clay slope in a moderately seismic area. The computed and the updated CDFs for the static safety factor under undrained loading prior to the earthquake (Steps 2 and 3) using the FORM approximation are shown on Figure 7. Figure 8 presents the results of the probabilistic seismic hazard study (Step 4). A linear relationship in the log-log plot of PGA vs. Annual Exceedance Probability implies a Pareto distribution for the annual PGA. The earthquake events with return periods between 1,000 and 10,000 years contribute most to the annual failure probability. The dynamic response analyses were therefore done for earthquake events with return periods of 3000 and 10000 years. Each of these events was represented by 4 sets of properly scaled acceleration time histories.

Figure 9 shows the results of the dynamic response simulations in terms of the maximum earthquake-induced shear strain and shear strength reduction factor for this slope (Step 5). Figure 10 shows the histograms of the shear strength reduction factors obtained from the simulations and a fitted distribution function to the data. The reduction factors are listed in Table 4.

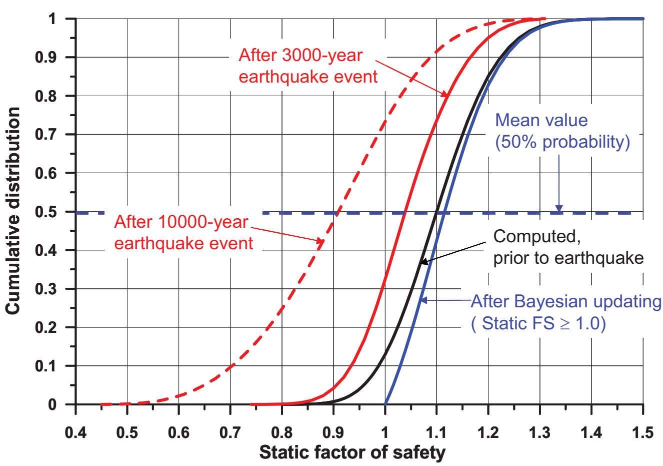


FIG. 7. Results of probabilistic analyses of static undrained stability, prior (black) and after earthquake (red).

Using the shear strength reduction factors on Figure 9, the conditional CDF of the post-earthquake safety factor (Step 8) was obtained. The post-earthquake static safety factor is the product of the shear strength reduction factor and the (corrected) pre-earthquake static safety factor. The CDFs for the post-earthquake undrained safety factor following the 3,000 and 10,000-year earthquake events are shown with the red curves on Figure 6.

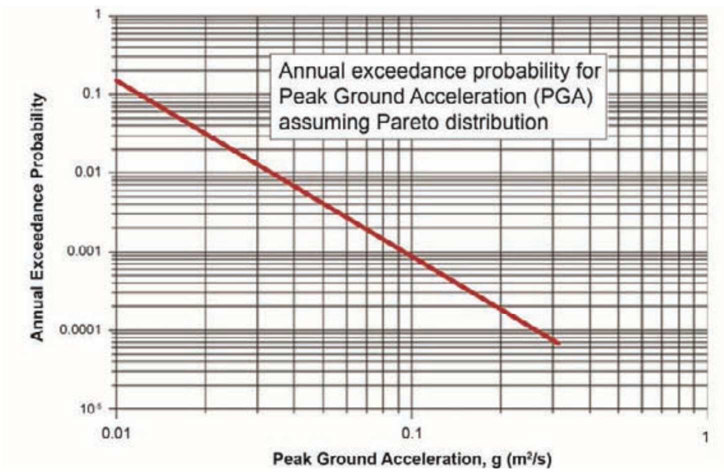


FIG. 8. Calibrated (for return periods 1,000 to 10,000 yrs) Pareto distribution with $\mu = 0.0077g$ and $\sigma = 0.0106g$ for annual PGA A_{max}.

To estimate the annual probability of slope failure (Step 9), a simplified model similar to that suggested by Cornell (1996) was developed. The limit state function for the seismic resistance of the slope was defined as:

$$G = \text{Seismic resistance} - \text{Earthquake load} = A_{resist} - \varepsilon \cdot A_{max} \tag{2}$$

where A_{max} is the annual peak ground acceleration representing the earthquake load, A_{resist} is the resistance of the slope to earthquake loading in terms of the peak ground acceleration causing slope failure, and ε describes the variability of the peak ground acceleration at a given return period.

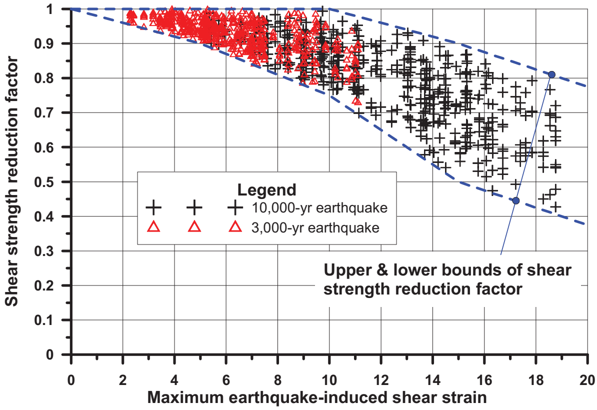


FIG. 9. Dynamic response simulations, return periods of 3,000 and 10,000 yrs.

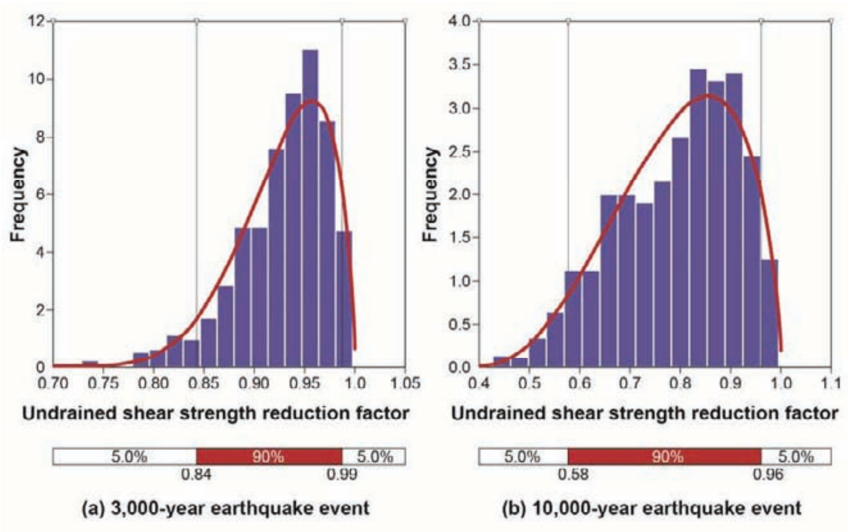


FIG. 10. Histograms and best-fit distributions of earthquake-induced undrained shear strength reduction factors.

Table 4. Beta distribution of undrained shear strength reduction factors.

Return period	Lower bound	Upper bound	Mean	Standard deviation
3,000 yr	0.72	1.0	0.94	0.04
10,000 yr	0.42	1.0	0.81	0.12

The resistance parameter A_{resist} was given a lognormal distribution, and the parameters were calibrated to match the conditional failure probabilities for the 3,000-year and the 10,000 year earthquake events. A_{resist} was found to have mean value of $\mu = 0.22g$ and standard deviation of $\sigma = 0.11g$.

The limit state function in Eq. 2 with the parameters in Table 5 and the FORM approximation were used to estimate the annual failure probability and reliability index (defined as $\beta_{annual} = \Phi^{-1}[1 - P_{f,annual}]$):

Annual probability of failure: $P_{f,annual} = 3.7 \cdot 10^{-4}$
Annual reliability index: $\beta_{annual} = 3.4$

Table 5. Random variables for evaluation of annual failure probability.

Parameter	Assumed distribution	Mean value	Standard deviation
A_{max}	Pareto	0.008g	0.01g
ε	Normal	1.0	0.12
A_{resist}	Lognormal	0.22g	0.11g

Experience with practice

Christian and Baecher (2011) suggested that the calculated failure probability in geotechnical engineering is generally much greater than the observed probability of failure, and that this is one of the unresolved problems in geotechnical risk and reliability. The hazard evaluation in the example shows how one can address this issue by using a minimum of additional information (slope is standing at time of analysis, therefore FS>1) to obtain a reasonable estimate of the probability of slope instability under earthquake loading and to provide a rational basis for stakeholders to make decisions on appropriate design and reliability of the design.

CASE STUDY 2: HAZARD AND RISK ASSESSMENT AND RISK MITIGATION FOR QUICK CLAYS

As part of work for The Norwegian Water Resources and Energy Directorate (NVE), NGI (Gregersen, 2001) developed a practical simplified method to map the risk associated with quick clay slides. Potential slide areas are classified according to "engineering scores" based on an evaluation of the geology, local conditions and persons or properties exposed. Hazard classes are described as low, medium and high, consequence classes as not severe, severe and highly severe. The risk, based on engineering experience, is divided in three risk classes (Lacasse *et al.* 2004).

The hazard level depends on topography, geology, geotechnical conditions and changes due to human activities, climatic effects, erosion, etc. The weights given to each hazard factor in Table 6 (and in Table 7) describe each factor's importance relative to the hazard (or loss). The hazard classes are: (1) *Low:* Favourable topography and soil conditions; extensive site investigations; no erosion; no earlier sliding; no planned changes, or changes will improve stability; (2) *Medium:* Less favourable topography and soil conditions; limited site investigations; active erosion; important earlier sliding in area; planned changes give little or no improvement of stability; (3) *High:* Unfavourable topography and soil characteristics; limited site investigations; active erosion; extensive earlier sliding in area; planned changes will reduce stability. A zone with score 0 to 17 (up to 33% of maximum score) is mapped as "low hazard", with low probability of failure. A zone with weighted score 18 to 25 (up to 50% of maximum score) is mapped as "medium hazard", with a higher, though not critical, probability of failure. A zone with weighted score 26 to 51 is mapped as "high hazard", with a relatively high probability of failure. The consequence level is evaluated with Table 7. The consequence classes are: (1) *Not severe:* No or small danger for loss of human life, costly damage or consequences; (2) *Severe:* Danger for loss of life or property or important economical or social loss; (3) *Highly severe:* High exposure of human life loss or large economical or social loss. The zones with weighted score between 0 and 6 (13% of maximum score) are mapped as "not severe". The zones with weighted score between 7 and 22 (up to 50% of maximum score) are mapped as "severe". The zones with weighted score between 23 and 45 are mapped as "highly severe"; they would hold a large number of persons, either as residents or as persons on the premises temporarily. The risk score to classify the mapped zones into a risk class is obtained from the relationship $R_S = H_{WS} \times C_{WS}$, where R_S is the risk score, H_{WS} is the weighted hazard score and C_{WS} is the weighted consequence score.

Table 6. Evaluation of hazard for slides in quick clay in Norway.

Factor on hazard	Weight	Score for hazard			
		3	2	1	0
TOPOGRAPHY					
Earlier Sliding	1	Frequent	Some	Few	None
Height of slope, H [i)]	2	>30 m	20-30 m	15-20 m	<15 m
GEOTECHNICAL CHARACTERISTICS					
Overconsolidation ratio (OCR)	2	1.0-1.2	1.2-1.5	1.5-2.0	>2.0
Pore pressures [ii)]					
- In excess (kPa)	3	> + 30	10-30	0-10	Hydrostatic
- Under pressure (kPa)	-3	> - 50	-(20-50)	-(20-0)	Hydrostatic
Thickness, quick clay layer [iii)]	2	>H/2	H/2-H/4	<H/4	Thin layer
Sensitivity, St	1	>100	30-100	20-30	<20
NEW CONDITIONS					
Erosion [iv)]	3	Active	Some	Little	None
Human activity					
- Worsening effect	3	Important	Some	Little	None
- Improving effect	-3	Important	Some	Little	None
MAXIMUM SCORE	--	51	34	16	0
% of maximum score		100%	67%	33%	0%

i) For study, inclination was identical for all slopes (1:3). In general Slope inclination should be added in the list of hazards.
ii) Relative to hydrostatic pore pressure.
iii) In general, the extent and location of the quick clay are also important.
iv) Erosion at the bottom of a slope reduces stability.

Table 7. Evaluation of consequence for slides in quick clay in Norway

Loss	Weight	Score for consequence			
		3	2	1	0
HUMAN LIFE AND HEALTH					
Number of dwellings[i])	4	> 5 (close)	> 5 (wide)	≤ 5 (wide)	0
Industry building	3	> 50	10-50	< 10	0
INFRASTRUCTURE					
Roads (traffic density)	2	High	Medium	Low	None
Railways (importance)	2	Main	Required	Level	None
Power lines	1	Main	Regional	Network	Local
PROPERTY					
Buildings, value[ii])	1	High	Significant	Limited	0
Flooding impact[iii])	2	Critical	Medium	Small	None
MAXIMUM SCORE	--	45	30	15	None
% of maximum score		100%	67%	33%	0%

i) Permanent residents in sliding area (close means closely spaced. Wide means widely spaced).
ii) Normally no one on premises, but building(s) have historical or cultural value
iii) Slides may cause water blockage or even dam overflow, flooding may cause new slides; need time for evacuation; losses depend on interaction of several factors.

Risk mitigation

To make decisions on the need for remedial action, Table 8 gives recommendations for quick clay areas in Norway. The volume of the sliding mass is probably the most important factor for the extent of the run-out zone. If several millions of cubic metre is involved, the run-out cannot be evaluated by simple dynamic or topographic models. This is especially important if large rivers are blocked and huge amounts of water are dammed with the possibility of subsequent generation of catastrophic flood waves downstream.

Table 8. Activity matrix as a function of risk class.

Activity	**Risk class**			
	12 (low risk)	**3** (low to medium risk)	**4** (medium to high risk)	**5** (high risk)
Soil investigations	None	Consider additional *in situ* tests and pore pressure measurements	Require additional *in situ* tests and pore pressure measurements	Require additional *in situ* tests, pore pressure measurements and lab.'y tests
Stability analyses	None	None	Consider doing	Require
Remediation[1)]	None	None	Consider doing	Require

1) e.g. erosion protection, stabilizing berm, unloading, soil stabilization, moving of residents

As an example, the city of Drammen, along the Drammensfjord and the Drammen River, is built on a deposit of soft clay. Stability analyses were done in an area close to the centre of the city, and indicated that some areas did not have satisfactory safety against a slope failure. Based on the results of the stability analyses and the factors of safety (FS) obtained, the area under study was divided into three zones (Gregersen 2005): Zone I, FS satisfactory; Zone II, FS shall not be reduced; Zone III, FS too low, area must be stabilised. In Zone III, a stabilizing fill (berm) was immediately placed in the river to support the river bank (Fig. 12), and the factor of safety checked again.

In Zone II, no immediate geo-action was taken, but a ban was placed on any new structural and foundation work without first ensuring increased stability. Figure 12 illustrates possible measures: (1) if an excavation is planned, it will have to be stabilised with anchored sheetpiling or with soil stabilisation, e.g. with lime-cement piles; (2) new construction or new foundations cannot be built without first checking their effect on the stability down slope; for example, adding a floor to a dwelling may cause failure because of the added driving forces due to the additional loading, and new piling up slope will cause a driving force (increased pore-pressures and possible vibrations) on the soil down slope.

Experience with practice

Simple concepts allow quantitative estimates of vulnerability and risk. The example shows how one can provide assistance to making important decisions with simple tools and engineering experience and judgement. The estimates are relative, but will help prevent catastrophic events.

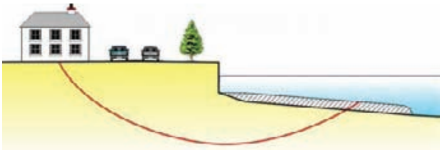


FIG. 11. Mitigation with stabilizing fill (berm) in Zone III in Drammen.

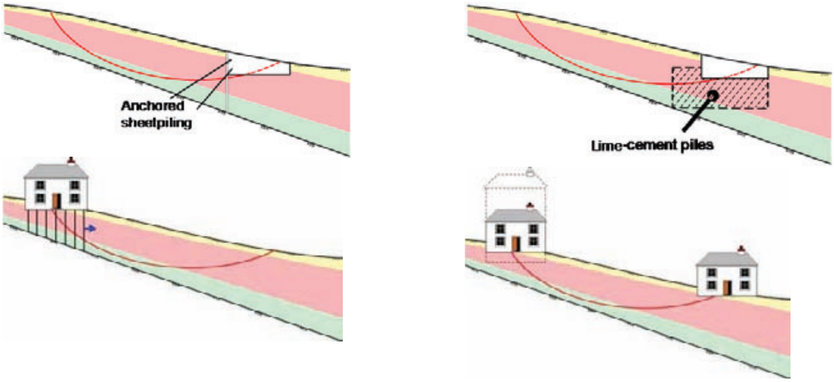


FIG. 12. Preventive measures in Zone II in Drammen (Gregersen 2008; Karlsrud 2008).

CASE STUDY 3: VULNERABILITY TO LANDSLIDE

As an example of vulnerability to landslides and consequences, a vulnerability study was done in an urban area in Norway, along the banks of a river where it meets a fjord. The quaternary geology indicates that the soils are normally consolidated. Typical soil profiles display a 1.5 m to 3.5 m thick river sand layer at the surface, underlain by marine deposits, consisting mainly of silty clay and clayey silt with some fine sand layers with thickness 10 to 23 m. The clay deposits are exposed to river erosion. The sensitivity of the clay was measured as close to or over 100, indicating "quick clay" behaviour. A plan view of the hazard and exposed areas is shown in Figure 13.

Vulnerability has two perspectives. The social vulnerability, or the "capacity of a society to cope with hazardous events" and the physical vulnerability, or the "degree of expected loss in a system from a specific threat", quantified between 0 (no loss) and 1 (total loss). Vulnerable categories include people, structures, lifelines, infrastructure, vehicles, and the environment. The vulnerability model used addresses the physical vulnerability quantitatively. The vulnerability is evaluated with the VIS formulation developed at NGI, where $V = I \cdot S$, with V as the vulnerability, I the landslide intensity and S the susceptibility of the vulnerable elements. Lacasse and Nadim (2011) presented the model in more detail. The intensity (I) expresses the potential damage caused by a landslide as a function of its kinematics (volume, depth and displacement), and its kinetics (velocity, momentum):

$$I = k_S \cdot [r_D \cdot I_D + r_G \cdot I_G] \tag{3}$$

where k_s = Spatial impact ratio; r_D, r_G = Dynamic and geometric relevance factors; and I_D, I_G = Dynamic and intensity components

The spatial impact ratio expresses how much an element at risk is affected spatially, and is defined in the range [0 to 1]. The relevance factors, specific to landslide type and vulnerable category reflect the available knowledge (or belief) on the relevance of the dynamic and geometrical characteristics of the landslide in causing loss (Table 9). The sum (r_D + r_G) shall be equal to unity. The dynamic intensity (I_D) expresses the destructive potential of a landslide's kinetic energy and momentum, and is defined in the range [0 to 1]. The geometric intensity (I_G) accounts for the dimensional properties of the sliding masses (e.g. depth, volume, displacement, area), and is defined in the range [0 to 1].

Table 9. Examples of relevance factors.

Category	**Landslide**	r_D	r_G
Structures	Rapid	0.90	0.10
Structures	Slow	0.15	0.85
Persons	Rapid	0.75	0.25
Persons	Slow	1.00	0.00

The susceptibility in the VIS model expresses the propensity of the vulnerable element to undergo loss, and depends on the physical characteristics (resistance and geometry) of the element. The susceptibility is independent of the landslide intensity. The susceptibility model is expressed as:

$$S = 1 - \prod_{i=1}^{n_s}(1-\xi_i) \tag{4}$$

where the susceptibility factors (ξ_i) are category-specific and reflect the user's belief or knowledge on the susceptibility. The factors range between [0 to 1] and should be mutually independent (Kaynia *et al.* 2008).

Table 10 illustrates the approach for determining the susceptibility of structures and persons, and gives examples of the influence of factors such as population density, income and age. The susceptibility of structures (S_{STR}) depends on the structure type and the state of maintenance. The susceptibility of persons (S_{PSN}) depends on the population density, the income and the age. For the study area shown in Figure 13, the landslide intensity was calculated as 0.20, and the susceptibility was obtained as 0.54, giving a vulnerability of 0.11. The spatial input ratio k_s (ratio of area occupied by structure to the selected reference area) was 0.22. The vulnerability factor is low, due to the relatively low spatial impact ratio. The vulnerability would have been significantly higher for the same landslide if a smaller reference area, for example only half of the urban area, had been considered, because the spatial impact ratio would have been much higher (and close to 1). The extent of the reference areas should be carefully considered because of this limitation in the model.

Table 10. Susceptibility of structures and persons.

Structures. Model: $S_{STR} = 1-(1-\xi_{STY})(1-\xi_{SMN})$

Type	Resistance	ξ_{STY}	Maintenance	ξ_{SMN}
Lightest simple structures	None	1.00	Very poor	0.50
Light structures	Very low	0.90	Poor	0.40
Rock masonry, concrete and timber	Low	0.70	Medium	0.25
Brick masonry, concrete structures	Medium	0.50	Good	0.10
Reinforced concrete structures	High	0.30	Very good	0.00
Reinforced structures	Very high	0.10		

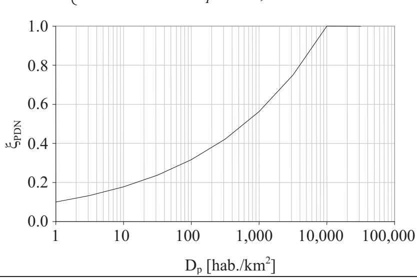
Persons (outdoors and in vehicles)

Susceptibility as function of age, ξ_{AGE}

Age (yrs)	ξ_{AGE}	Age (yrs)	ξ_{AGE}	Age (yrs)	ξ_{AGE}
0-5	1.00	20-50	0.00	65-70	0.70
5-10	0.90	50-55	0.10	70-75	0.90
10-15	0.70	55-60	0.30	> 75	0.95
15-20	0.30	60-65	0.50		

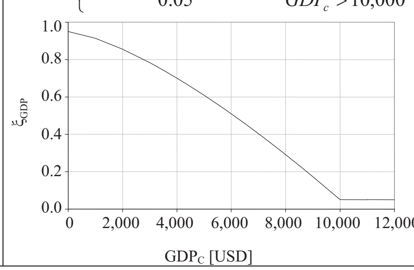
Susceptibility as f (population density), ξ_{PDN}

$$\xi_{PDN} = \begin{cases} 0.1 \cdot D_P^{\;0.25} & D_P \leq 10{,}000 \\ 1.00 & D_P > 10{,}000 \end{cases}$$



Susceptibility as f (income), ξ_{GDP}

$$\xi_{GDP} = \begin{cases} 0.95 - 0.90\left(\dfrac{GDP_c}{10{,}000}\right)^{1.4} & GDP_c \leq 10{,}000 \\ 0.05 & GDP_c > 10{,}000 \end{cases}$$



Experience with practice

The quantification of vulnerability is only, and at best, approximate. The relative values obtained depend on the model used, and can be subjective. However, it is useful that some estimate is done in order to rank vulnerable categories and the possible consequences, and to prioritise the mitigation measures that should be implemented.

$V = I \cdot S = 0.20 \times 0.54 = 0.11$

$C = 200$ m/s
$A_t = 2300$ m^2
$A_i = 512$ m^2

Intensity, I
$k_s = 0.22$
$r_D = 1.00$
$r_G = 0.00$
$I_D = 0.89$
$I = 0.22\ (1.0 \times 0.89) = 0.20$

Susceptibility, S
40 structures
(5 light, 22 masonry, 13 reinforced concrete; 50% in poor state, 50% in good state)
$S = 0.54$

FIG. 13. Vulnerability calculation in study area for Case Study 3.

RISK ASSESSMENT FOR DAMS

In many countries, a systematic risk analysis and assessment are required for dams with potentially high failure consequences. This applies to the construction of new dams as well as to the safety evaluation of existing dams. Three books, among others, present the state-of-the art on risk analysis of dams: Vick (2002), Hartford and Baecher (2004) and Fenton and Griffiths (2008). Scott (2011) presented an extensive review of the risk assessment to dam safety, based on the wide experience (370 high/significant dams and dikes) of the US Bureau of Reclamation.

Høeg (1996) described using the probabilistic risk approach for different purposes: e.g. (a) at the dam design stage to achieve a balanced design and to place the main design efforts where the uncertainties and the consequences seem the greatest; (b) as a basis for decision-making when selecting among different remedial actions and upgrading for old dams within time and economical constraints; (c) to relate dam engineering risk levels to acceptable or tolerable risk levels.

The event tree approach (Ang and Tang 1984; Vick 2002; Hartford and Baecher 2004) is very useful for the hazard and risk assessment of large and complex facilities such as dams. The analysis involves breaking down a complex system into its fundamental components, and determining the potential "failure" modes and the physical processes that could cause "failure". The approach provides insight into how a series of events leading to non-performance of the dam might unfold. The probability of each event, given the occurrence of an initiating event, is quantified. As the number of events increases, they fan out like the branches of a tree. Each mutually exclusive path represents a specific sequence of events, resulting in a particular outcome. The probability for each branch of events is the multiplication of probabilities on the branch. The result is a set of frequency-outcome pairs (the outcome could be, e.g. "failure" or "no failure"). The total probability is the sum of all events contributing to an outcome. A presumption of event tree analysis is that engineering judgement and experience is needed in addition to the results of statistical and deterministic analyses. To achieve consistency in the evaluation of the probabilities (from one expert to the other and from one situation to another), a convention should be used to anchor the probabilities. One scale often used can be found in Table 11.

Table 11. Example of verbal descriptors of uncertainty.

Descriptor of uncertainty	Event probability	Definition
Virtually impossible	0.001	Due to known physical conditions/ processes that can be described and specified with almost complete confidence.
Very unlikely	0.01	The possibility cannot be ruled out on the basis of physical or other reasons.
Unlikely	0.1	Event is unlikely, but it could happen.
Completely uncertain	0.5	There is no reason to believe that one outcome is more or less likely than the other (for three possible outcomes, use 0.33).
Likely	0.9	Event is likely, but it may not happen.
Very likely	0.99	Event is very likely, and one would be surprised if it did not happen.
Virtually certain	0.999	Due to known physical conditions or processes that can be described and specified with almost complete confidence.

One can debate whether there is in practice too much emphasis on engineering judgment in the event tree approach. However, all deterministic approaches, including statistics of data, also involve engineering experience and judgment. In probabilistic analysis, the selection of relevant and appropriate data sets also requires subjective judgment. Each dam-foundation system is unique, and it is difficult to apply statistics to the available data on dam failures and observed incidents.

CASE STUDY 4: TAILINGS DAM IN ROMANIA

As an example, the estimated annual probability of non-performance of a new tailings management facility at Roşia Montană (www.gabrielresources.com/prj-rosia.htm) in Romania is presented. The analyses established whether the dam provides acceptable safety against release of tailings and toxic water, and whether additional hazard reducing measures are needed. The project lies within the existing Roşia Montană mining district north-east of the town of Abrud in the Apuseni Mountains of Transylvania. The project aims at mitigating the consequences of the historic and future mining operation with the interception and containment of contaminated water currently entering the system, treatment of the contaminated waters and isolation and recovery of the waste rock piles within the project boundary. The operation of the project will generate tailings for approximately 17 years, producing tailings from the processing of a total of approximately 215 Mt of ore. The proposed mining and processing operation requires the construction and operation of a Tailings Management Facility (TMF) in the valley. The TMF (Fig. 14) includes a Starter Dam as a first stage of the Completed Dam, a Secondary Containment Dam (SCD), a tailings delivery system, a reclaim water system and a waste rock stockpile. The TMF is designed as a depository for the treated tailings residue. The Corna Valley TMF site is to provide the required design storage capacity for the life of the mine, plus an additional contingency capacity.

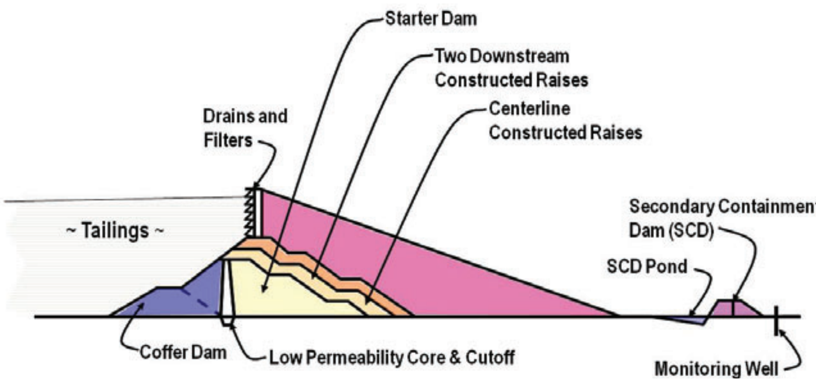


FIG. 14. Cross-section of tailings dam (Corser, P. (2009). Personal communication. MWH Americas, Inc.).

To establish whether the dam provides acceptable safety against "uncontrolled" release of tailings and water during its life, an event tree approach was used to do the hazard analyses.

The event tree hazard analyses considered the dam at different stages of its life and estimated the probability of non-performance. A non-satisfactory performance of the dam was defined as an uncontrolled release of tailings and water from the dam over a period of time. The analyses looked at critical scenarios, including all potential modes of non-performance for the dam under extreme triggers such as a rare, unusually strong earthquake and extreme rainfall in a 24-hour period.

<u>Design</u>

Requirements that influenced the estimated probabilities in the hazard analyses include:

– Operational freeboard of one meter above storage elevation for maximum reclaim pond including 2 PMP (probable maximum precipitation) volumes. The freeboard means an extra storage capacity of 2 PMP (equivalent to 5.5 million m^3 volume), corresponding to a capacity of two 1/10,000-year rainfall occurring within the same 24 hours;
– Gentle slope for the rockfill Starter Dam (≈1V:2H upstream and ≈1V:2H downstream);
– Gentle downstream slopes for the Completed Dam (1V:3H);
– Good quality rockfill for the Starter Dam construction and for the downstream shoulder of the final Completed Dam;
– "Well drained" tailings beach at the upstream face of the dam, where equipment can be moved for repairs, in case of movement or partial breach;
– Secondary Containment Dam (SCD) with about 50,000 m^3 containment capacity;

- Diversion channels along the sides of the valley to divert excess rainfall runoff away from the TMF pond to minimise the risk of overtopping;
- Emergency spillway to control any excess water released;
- Comprehensive geotechnical monitoring system planned for safety surveillance;
- Careful construction control by owner and contractor/engineer.

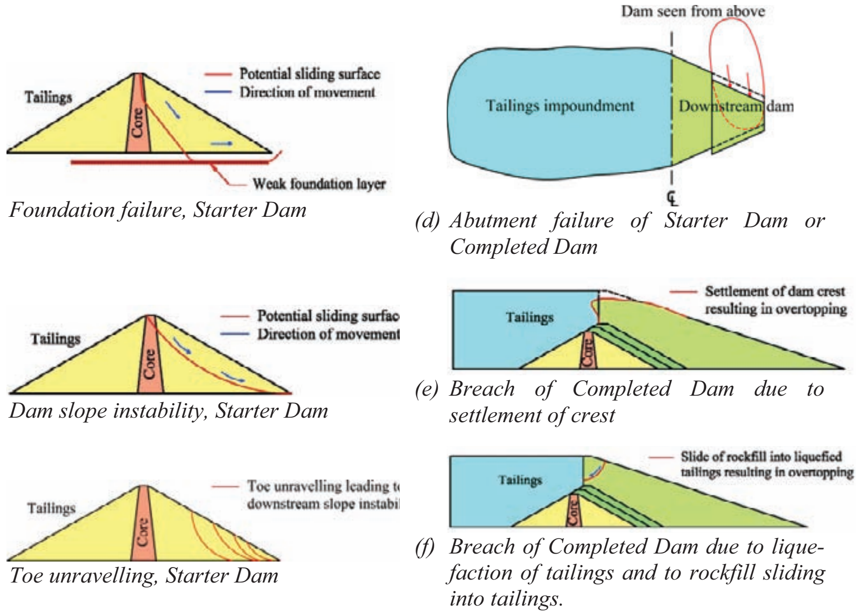
Event tree analysis

To establish whether the dam provides acceptable safety against "uncontrolled" release of tailings and water during its life, an event tree analysis was done. The event tree team reviewed all site investigation and design documents and visited the site. An event tree workshop was organised to develop the event trees and reach a consensus when quantifying the hazards (e.g. Vick, 2002). The analysis involved breaking down the complex system into its fundamental components, and determining the potential "failure" mechanisms leading to non-performance of the dam and the physical processes that could cause such mechanisms. The "non-performance modes" considered included:

- Foundation failure due to e.g. excess pore pressures or weak layer in foundation leading to cracking, instability and breach of the dam.
- Dam slope instability downstream or upstream, due to e.g. high construction pore pressures in core of Starter Dam, excessive pore pressures caused by static or earthquake loads, or instability due to inertia forces.
- Unravelling of downstream toe and slope due to e.g. overtopping or excessive leakage through or under the dam. This can be caused by a slide into the depository, dam crest settlement due to deformations of the Starter Dam, piping, internal erosion and sinkhole formation, or excessive deformations (slumping) of the top vertical part of the Completed Dam during earthquake shaking.
- Dam abutment failure followed by breach due to e.g. slide close to and/or under part of the dam.
- Liquefaction of the tailings.

The analyses looked into construction deficiencies, e.g. filter material segregation leading to uncontrolled internal erosion, inadequate drainage, very weak construction layers or zones in the embankment, inadequate types of material(s) in the embankment fill, or insufficient quality control and unforeseen construction schedule changes which often happen in mining operations. These conditions were integrated in the event trees as separate events during the course of the construction of the Starter Dam and Completed Dam. Figure 15 presents some of the configurations and examples of the non-performance modes analysed. Overtopping without breach of the dam, and under-capacity/damage of the SCD were also considered as events in the event trees.

At the event tree workshop, a discussion was first held to screen the most critical, and yet plausible, times in the life of the TMF to analyze, e.g. during construction of the Starter Dam, during the downstream construction stages, during the centreline construction of the final dam, and/or in the early years after completion of the final dam. A matrix of dam configuration and time was prepared, and the following modes were seen as most critical and susceptible to lead to the highest probabilities of non-performance. As part of the mode screening, the following considerations were subjected to a consensus decision: most critical times during life of the dam, extreme and critical precipitation (rainfall, flood and snowmelt), likelihood of failure of the waste stockpile adjacent to the depository, critical situations after construction of the dams, and geo-environmental considerations.



(a) Foundation failure, Starter Dam

(d) Abutment failure of Starter Dam or Completed Dam

(b) Dam slope instability, Starter Dam

(e) Breach of Completed Dam due to settlement of crest

(c) Toe unravelling, Starter Dam

(f) Breach of Completed Dam due to liquefaction of tailings and to rockfill sliding into tailings.

FIG. 15. Sketches showing examples of non-performance modes.

Table 12 presents the short version of the analyses prioritised. They are grouped by triggering event and by the dam "Configuration" analysed, the Starter Dam, the Completed Dam or an intermediate construction stage. Event trees were developed for each trigger, with each non-performance mechanism looked at separately. In some cases, two non-performance mechanisms were considered successively.

The total probability of non-performance is the sum of all contributing probabilities to the non-performance for each of the dam configurations. The estimated probabilities were presented as a function of the release of tailings and water associated with the non-performance of the Dam. The highest estimated probabilities of non-performance were associated with earthquake shaking of the main dam and the static liquefaction of the tailings at time 9 to 12 years. The scenarios would result in some material damage and some contamination, but only in the vicinity downstream of the dam. For the Starter Dam, no plausible expected scenario led to a significant release of tailings and water because of the limited quantity of water available and the reserve capacity provided (2 PMP's). Essentially all material released could be contained by the Secondary Containment Dam.

Table 12. Overview of event tree analyses.

Configuration	Time (t)	Trigger	Non-performance mode
Starter Dam	1.5 yrs	Earthquake shaking	Foundation failure
			Dam slope instability
			Abutment failure
			Toe unravelling
Completed Dam	16 yrs		Foundation failure
			Downstream instab-t‘y and liqu.'tion
			Abutment failure
Starter Dam	1.5 yrs	Precipitation, flood, snowmelt	Foundation failure
			Dam slope instability
			Abutment failure
			Internal erosion and toe unravelling
			Operational delays
Starter Dam + 2 raises	4 yrs	Precipitation, flood, snowmelt	Operational delays
			Natural terrain slide in valley
Intermediate	9-12 yrs	---	Failure of waste stockpile
			Liquefaction of tailings
Completed Dam	16 yrs		Failure of waste stockpile
Starter dam	1.5 yrs		Internal erosion

The analyses showed the following:

- No sequence of plausible accidental events results in a probability of non-performance of the dam greater than once in a million years (or 10^{-6}/yr). For a lifetime of say 20 years, the probability of non-performance is 2 x 10^{-5}.
- The estimated probabilities of non-performance are lower than what is considered acceptable as design criteria for dams and other containment structures around the world and lower than probabilities of non-performance for most other engineered structures. Figure 16 presents such a comparison.

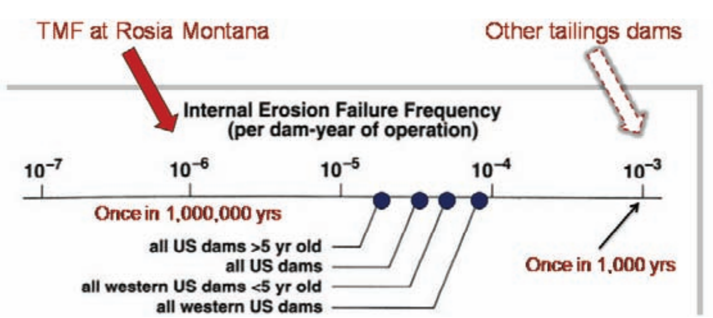


FIG. 16. Annual probability of failure for different types of dams.

The factors that contribute to the low estimated probability of non-performance include the use of good quality rockfill for the downstream shoulder of the dam, gentle downstream slopes for both the Starter and the Completed Dam, dam capacity to store extreme precipitation and/or snowmelt events, spillway to release excess water in a controlled manner, the planned safety monitoring to warn of any early signs of unexpected performance, and the proposed preparedness to remediate any indication of unexpected behaviour.

The physical impact in terms of damage to the environment was also studied, in the extremely unlikely event of a breach in the dam should occur. The analysis suggested that the tailings would flow a couple of 100 meters and that the tailings volume would be very limited (Fig. 17). Studies of groundwater and river water flow and contamination were also conducted. The levels of contamination were found to be above surface water discharge standards for a limited period of time and in the immediate neighbourhood of the tailings dam only under the worst case conditions (low flow in the downstream river). However, implemented monitoring, early warning installations and emergency procedures would help contain damage to a minimum.

The weather and flow conditions for this to occur combined with the probability of dam breach occurring at the same time resulted in the probability of occurrence of 1 in 10 million years (Whitehead, P. 2009. Private communication. Aquatic Environments Research Centre, University of Reading).

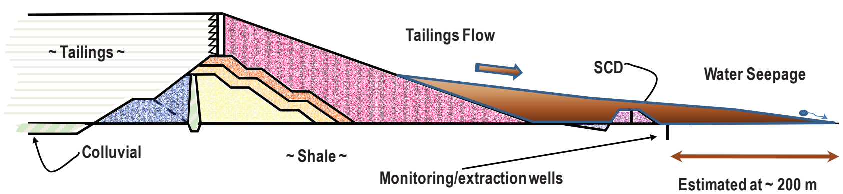


FIG. 17. Physical impact of TMF dam breach at Roşia Montană (Corser, P. (2009). Personal communication. MWH Americas, Inc.).

Experience with practice

The concepts of probabilistic risk analyses for dams have been around for a long time (e.g. Whitman 1984; Serafim 1984; Vick and Stewart 1996). The example illustrate that the event tree analysis is a systematic application of engineering judgment. Its application does not require the prior existence of extensive statistics or the application of complex mathematics. The process provides meaningful and systematic estimates and outcomes on the basis of subjective probabilities.

With increasing frequency, society demands that some form of risk analysis be carried out for activities involving risks imposed on the public. At the same time, society accepts or tolerates risks in terms of human life loss, damage to the environment and financial losses in a trade-off between extra safety and enhanced quality of life.

The role of the dam engineering profession is to explain the uncertainties involved in the construction and operation of dams and to present the likelihood of incidents and failure in informative and meaningful terms. The conventional use of a factor of safety just does not do that, and concepts from probability theory and reliability analyses should be applied.

The key to making the risk analysis of dams effective begins with a detailed overview of all potential failure modes. If shortcuts are taken, the results could be unreliable and misleading. Once the potential failure modes are understood, the screening process will identify the critical modes. A variety of tools are available for making the quantitative risk estimates. The event tree approach is useful and illustrative. It is recognized that risk estimates and risk assessment guidelines are only approximate, but they are useful for choosing among alternatives, comparing risk levels, and making decisions.

CASE STUDY 5: ROCKFILL DAM WITH MORAINE CORE IN NORWAY

For the Viddalsvatn dam in Norway completed in 1971 (Høeg, 1996), the probability of failure due to internal erosion was evaluated. Viddalsvatn dam, owned by Oslo Energi, is a rockfill dam with moraine core located in mid-Norway. It has height of 80 m crest length of 425 m and reservoir volume of 200 x 106 m^3. During the first years, Viddalsvatn dam experienced several incidents of intermittent increased seepage, caused by internal erosion. Sinkholes developed on the crest of the dam following these. Automatic seepage monitoring was carried out continuously. Grouting of the core stopped further leakage and internal erosion.

The event tree analysis workshop assembled a group of specialists on the different aspects of the dam design. Potential failure sequences were broken down into individual events in either logical or temporal progression to form event trees. The process of internal erosion was considered in four stages: initiation, continuation, progression and breach development. Field observations and subjective field experience were considered. Hydrologic (100-yr and 1000-yr flood), earthquake and normal loading conditions were considered. Breach was defined as the uncontrolled release of the reservoir. Lacasse and Nadim (1998) presented examples of the event trees.

The probabilities were obtained by assigning verbal descriptors (Table 11). Observations and statistics were used. The estimates also relied heavily on engineering judgment. The probabiliity of progressive erosion was set as high because of the possibility of damage due to previous incidents that had been observed. For leakage initiating erosion but not leading to progressive erosion, the probability (p) of toe and downstream slope unravelling and of failure is believed to be virtually impossible (p=0.001) because of the rockfill discharge capacity and of the dam's observed performance during three earlier such incidents. However, should internal erosion be progressive, toe unravelling was considered as more likely (p became 0.1 and p=0.5, depending on the success of leakage control by reservoir drawdown). Each outcome in the event tree ended up as dam breach or no dam breach. Some component events were treated statistically, e.g. the 100-yr and 1000-yr flood were based on historical data, and the earthquake frequency and response spectrum were based on the Norwegian database for earthquakes. The event trees for each of the loading cases resulted in the following annual probabilities of failure:

Loading	Annual probability of failure
Flood	1.2×10^{-6}/yr
Earthquake	1.1×10^{-5}/yr

Normal (internal erosion) 5.5×10^{-4}/yr

The total annual probability of failure for all modes was 5.6×10^{-4}.

Experience with practice

The probabilistic estimates represent a relative order of magnitude for the different scenarios. They should not be interpreted to be an accurate probability. In practice, the results of the analysis proved even more useful when done on several dams and compared, as was done in Johansen *et al.* (1997).

CASE STUDY 6: EARLY WARNING SYSTEM FOR THE USOI LANDSLIDE DAM IN TAJIKISTAN

Lake Sarez is located in the Pamir Mountain Range in eastern Tajikistan. The lake was created in 1911 when an earthquake triggered a massive rock slide (volume: ~2 km^3) that blocked the Murgab river valley. A landslide dam, Usoi Dam, was formed by the rockslide which retains the lake. The dam at altitude 3200 meters has a height of over 550 meters, and is by far the largest dam, natural or man-made, in the world. Lake Sarez is about 60 km long, with maximum depth of about 550 m and volume of 17 km^3. The lake has never overtopped the dam, but the current freeboard between the lake surface and the lowest point of the dam crest is only about 50 m. The lake level is currently increasing about 30 cm per year. If this natural dam were to fail, a worst-case scenario would be a catastrophic outburst flood endangering thousands of people in the Bartang, Panj, and Amu Darya valleys downstream.

There is a large active landslide on the right bank (Fig. 18), with observed movement rate of 15 mm/year. If this unstable slope should fail and slide into the lake, it would generate a surface wave large enough to overtop the dam and cause a severe flooding downstream. Experts who studied the hazards agree that the most probable scenario at Lake Sarez is failure of the right bank slope and overtopping of the dam.

In 2000, an international "Lake Sarez Risk Mitigation Project" was launched under the auspices of the World Bank to deal with the risk elements posed by Usoi dam and Lake Sarez. The objective of the project was to find long-term measures to minimize the hazard and to install an early warning system to alert the most vulnerable communities downstream. The early warning system for Lake Sarez has been in operation since 2005, with 9 remote monitoring units linked to a central data acquisition system at a local control centre near the dam and the monitoring listed in Table 13. Data is transmitted via satellite to the main control centre in Dushanbe, Tajikistan's capital. Alerts and warning messages are sent from Dushanbe to 22 communities connected to the system. The local control centre is manned 24 hours per day, every day. The warning system has three alarm levels. Each level is based on monitored data and/or visual observations. Threshold values for triggering alarms include both maximum measured values and rate of change with time (Table 14). Table 15 describes the alarm states and emergency warning plans. The main problem has been insufficient power in some of the remote villages. The system was turned over to the Ministry of Defence who now has responsibility for its operation. The plan is to keep the early warning system in operation until 2020 which is the target date for completion of the mitigation works.

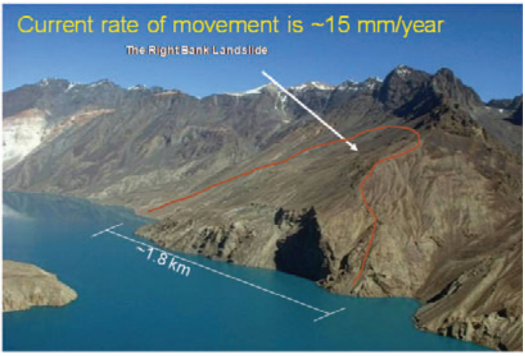


FIG. 18. Active landslide on the right side of Lake Sarez.

Table 13. Early warning system measurements at Lake Sarez (Stucky 2007).

Measurement	Methodology
Lake elevation	Pressure transducer in the lake
Detection of large surface wave	Pressure transducer in the lake
Seismic event	Strong motion accelerometers
Surface displacements	GPS
Flow in Murgab river downstream	Radar type level sensor
Turbidity in the outflow water	Turbidity meter
Flood conditions down stream	Level switches
Meteorological data	Complete weather station

Table 14. Threshold values for Level 1 and Level 3 alarm states (Stucky 2007).

Level	Source	Threshold value
1	Seismic acceleration	$a > 0.05$ g
	Lake level elevation	$H > 3270$ m above sea level
	Rate of change of lake level	$dH/dt > 25$ cm/day
	River flow downstream	$Q > 300\ m^3/s$ or $Q < 10\ m^3/s$
	Manual alarm input	Unusual visual observation
3	Height of detected wave on lake	Wave height > 50 m
	Flood sensor	$Q > 400\ m^3/s$
	River flow down stream	$Q > 400\ m^3/s$ or $Q < 5\ m^3/s$
	Rate of change of river flow	$dQ/dt > 15\ m^3/s/h$
	Manual alarm	Major event observed

Figure 19 illustrates the effect of mitigation on the risk level. The first estimate gave a risk level [P_f; number of fatalities] of [10^{-4}/yr; 5000 fatalities]. This was reduced to [10^{-4}/yr; 200 fatalities] with the installation of the early warning system, and to [10^{-7}/yr; 200 fatalities] by in addition the lowering of the lake reservoir. A permanent lowering of the lake reservoir by about 120 m using a diversion tunnel around the landslide turned out to be the most cost-effective mitigation measure. The possibility of at the same time producing electrical power is being evaluated, but the transmission of power to potential users also presents a big (unsurmountable – because of the mountains, Fig. 19) challenge.

Experience with practice

Risk assessment has become a necessary tool for managing the risks associated with storing large amounts of water and/or tailings upstream of populated areas and/or environments that need to be preserved. The analysis is of a complex nature. Its success and reliability need the interaction of a wide spectrum of expertise. To gain the trust of the stakeholders, the assessments need to include many components that are wider than what the geotechnical engineer usually deals with.

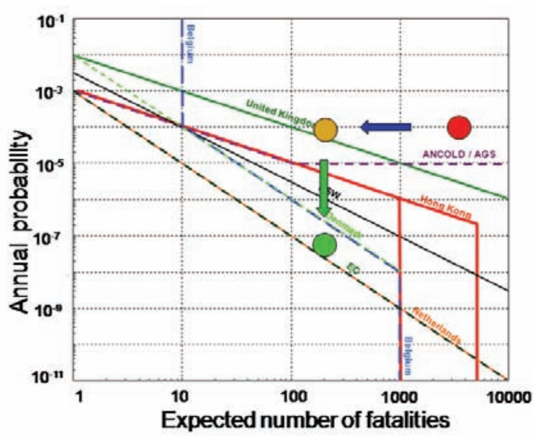


FIG. 19. Effect of mitigation measures on risk (criteria from Figure 6)

Table 15. Alarm states and emergency warning plan.

Level 0 –Normal state		Level 1 – Abnormal state but not critical	
Definition	All systems operating properly No abnormal conditions detected	*Definition*	Abnormal situation due to a external or internal problem
Origin of warning	Early Warning System Local operating personnel	*Origin of warning*	Early warning system Local operating personnel
Destination of warning	Local control centre & Dushanbe	*Destination of warning*	Local control centre and Dushanbe
Action	Daily operation and maintenance	*Action*	Inspection, checking, repair and observation
Level 3 –Escape Signal		Level 4 –Back to normal signal	
Definition	Abnormal condition detected based on several sources	*Definition*	Normal conditions confirmed after a Level 3 alarm
Origin of warning	Early Warning System Local control centre or Dushanbe	*Origin of warning*	Dushanbe
Destination of warning	Local control centre and all villages downstream	*Destination of warning*	Local control centre and all villages
Action	People in villages evacuate to predefined safe areas	*Action*	Back to Level 0

SUMMARY AND CONCLUSIONS

The paper presented six examples of hazard and risk assessment. The most effective applications of risk approaches are those involving relative probabilities of failure or illuminating the effects of uncertainties in the parameters. The continued challenge is to recognize problems where probabilistic thinking can contribute effectively to the engineering solution, while at the same time not trying to force these new approaches into problems best engineering with traditional approaches.

The tools of statistics, probability and risk can be intermixed, to obtain the most realistic and representative estimate of hazard and risk. It is possible to do reliability and risk analyses with simple tools, recognizing that the numbers obtained are relative and not absolute. It is also important to recognize that the hazard and risk numbers change with time, and as events occur or incidents are observed at a facility.

The assessment models for hazard and risk be transparent and easy to perceive and use, without reducing the reliability, suitability and value of the models required for the assessment. At the same time, for the purpose of communication with the stakeholders, the profession needs to focus on reducing the complexity of their explanations.

There should be an increased focus on hazard- and risk-informed decision-making. Integrating deterministic and probabilistic analyses in a complementary manner (e.g. Whitman 1996; Lacasse *et al.* 1989) will enable the user (with or without scientific background) to concentrate on the analysis results rather than the more complex underlying information. The complementary deterministic-probabilistic approach brings together the best of our profession and the required engineering judgment from the geo-practicians and the risk analysis proponents.

The civil engineer's task is about assessing hazard, vulnerability and risk in the context of safety, and to treat (reduce) the risk with appropriate mitigation measures. The civil engineers know, perhaps much more than the public affected by the risk, that the analysis of safety in facilities or on natural slopes is a very complex task and subject to important uncertainties. In an analysis, the engineer "runs the risk" of not seeing "the forest for the trees", or not seeing "safety for the steps in the risk assessment". The engineer also "runs the risk" of using the numbers, to arrive at, even subconsciously, the answers one wants to achieve. Unbiased estimates, when data are lacking, are difficult to achieve, even for the most experienced professional.

Focus needs to remain on "safety". Faced with natural and man-made hazards, society's only resource is to learn to live and cope with them. One can live with a threat provided the risk associated with it is acceptable or promises are made to reduce the risk to a tolerable level. After the recent natural catastrophes around the world, risk mitigation is gaining increasing interest and opens new challenges and new opportunities for our profession. Emerging issues are accounting for multi-risk, climate and global changes and risk communication, especially bridging the gap between the technological and social aspect of hazard, vulnerability and risk.

The profession needs an enhanced capacity to address hazards and assist in the making of informed decisions on actions to reduce their impacts. Focus needs to shift from response-recovery towards prevention-mitigation, building resilience and reducing risks, learning from experience and avoiding past mistakes.

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The Practice of Forensic Engineering

Patrick C. Lucia[1], Ph.D., P.E.

[1]Principal, Geosyntec Consultants, 1111 Broadway, 6th Floor, Oakland, California 94607

ABSTRACT

The practice of forensic engineering is one of the most interesting and challenging for geotechnical engineers. Geotechnical engineers are naturally drawn to failures as opportunities to increase our judgment by learning how applications of our knowledge may have failed to achieve the intended result. The challenge for many engineers is that these failure studies typically must be done within the context of a legal system where the primary objective is to assign or allocate responsibility for the failure. The adversarial process can be intimidating, especially for engineers not accustomed to the process. Engineers typically work together collaboratively in an atmosphere where they challenge each other to produce the best engineered result. In litigation, collaboration is displaced by criticism of even the most scientifically grounded professional opinions. Forensic engineering must therefore meld the best science of failure analysis to the art of conflict resolution, where the financial consequences of the resolution can be a matter of grave consequence to the responsible party.

INTRODUCTION

This paper is not intended to specifically address technical approaches to evaluating causes of failure, although several case histories will be discussed. Technical approaches for evaluating failures vary greatly depending upon the problem and are discussed in great detail in other publications. The intent of this paper is to discuss the framework under which a technical evaluation of a failure takes place in the legal system and how under some circumstances the requirements of the legal system can affect the work and the opinions of the experts.

The discussion in this paper is intended to be about the practice of forensic engineering as practiced by individuals who serve as expert witnesses. Admittedly, expert witnesses comprise a small percentage of the geotechnical community, but their practice has an impact on almost every company that engages in geotechnical engineering. Most engineers live in fear that someday they will be entangled in this process. In today's litigious society a failure typically results in the intersection of the legal and the engineering professions, two professions in which the members are highly trained and intelligent. An evaluation of a failure that warrants the time and expense of a forensic evaluation is generally done for the purpose of allocating responsibility. In fact, the geotechnical expert's client is typically a lawyer who is retained by the defendant geotechnical engineering company. Engineers too often think of the evaluation of a failure only in terms of the technical issues involved. However, that is only part of the practice of forensic engineering; the evaluation of the failure takes place within the context and rules of the legal system and it is that involvement of the legal system that, in my experience, frustrates most engineers. A definition of forensic engineering edited to apply to geotechnical forensic engineering is as follows:

> **Forensic geotechnical engineering** is the application of geotechnical engineering to answer questions of interest to the legal system. The word *forensic* comes from the Latin adjective *forensis*, meaning "of or before the forum." In Roman times, a criminal charge meant presenting the case before a group of public individuals in the forum. Both the person accused of the crime and the accuser would give speeches based on their side of the story. The individual with the best argument and delivery would determine the outcome of the case. This origin is the source of the two modern usages of the word *forensic,* as a form of legal evidence and as a category of public presentation.

As defined above the word forensic is both as a form of legal evidence and a category of public presentation. By the time both parties to the dispute get to court, both believe that their evidence will determine the outcome of the litigation. While the evidence must meet the standards of the court, the ability of the expert to present often complicated technical facts to either a judge or lay person in a jury can be the key to the outcome of the conflict. Unfortunately, the facts themselves are only a part – a very important part, of course– of the resolution of the conflict. A poor presentation of the facts by both sides in the conflict can result in a judge or jury having to decide the meaning and implication of complex technical issues based on their own life experiences and education.

The evaluation of a failure within the legal system has to incorporate the laws of science, the practice of engineering, and the rules of evidence within the court system. Typically an expert will have an opinion on the cause or causes of the failure, which will point liability towards one or more of the participants in the project. The basis of that expert's opinion can vary depending on the state or the court system in which the case is being tried. If the finding of the court is that there was negligence on the part of the engineer, that finding must be based on a determination that the engineer failed in their duty to meet the standard of care practiced by other engineers in their profession at the same time.

Allocation of responsibility leading ultimately to a financial sharing of the costs of the failure is that portion of the forensic study that most engineers find exceedingly difficult. The forensic evaluation leads to an opinion on the cause of the failure and then to responsibility for that cause. The technical evaluation then turns toward an accusation of responsibility to either an individual or company. Engineers are typically ill equipped to deal with this part of the forensic process. Lawyers are on the other hand quite skilled at conflict and the resolution of conflict through mediation, arbitration, or trial. Solving the conflict ultimately becomes the point of the forensic evaluation; this happens not only within the laws of science and engineering, but also within the rules of discovery and evidence within the court. A number of the issues that the forensic engineer must deal with in this practice are discussed below with a few case histories to illustrate the issues. The discussion is from the perspective of a geotechnical engineer who frequently intersects with the legal profession. The opinions on how the legal system works relative to geotechnical engineering are just that: opinions based on observations over 25 years of forensic investigations and expert testimony.

HOW DO WE DEFINE FAULT – STANDARD OF CARE

In the allocation of responsibility regarding the engineer's performance the issue is always directed at the engineer's compliance with the standard of care. An engineer is not held to a strict liability standard where the performance of their work is guaranteed. The courts recognize that the practice and standards of engineering can vary over time. ASFE states that the standard of care is "… that level of skill and competence ordinarily and contemporaneously demonstrated by professionals of the same discipline practicing in the same locale and faced with the same or similar facts and circumstances." More recently courts have ruled that the geographic location may be a factor but not a determining one.

The implications of the standard of care defense for the defendant geotechnical engineer is that in the forensic evaluation the plaintiff's expert is looking for factors that contributed to the failure that were not representative of the standards met by other engineers. These can include many things such as the number of borings drilled, the frequency of sampling, the number and types of tests conducted, the interpretation of the data, the assumptions made in the analyses, the types of analyses performed, the recommendations made based on the results of the analyses, and the performance of the engineer during construction observation services.

In the practice of geotechnical engineering the engineer cannot specify the materials for the project. The engineer is given a site for which he or she may only observe a small percentage of the material and can test an even smaller percentage to evaluate its properties. The engineer must then interpolate between the limited data, apply reasonable assumptions to an analysis, provide the client with design and construction recommendations, and then provide construction observation services that will result in a project that performs to the client's expectations. At each step in the process substantial judgment is required.

Given so many opportunities to use an individual's judgment, how does anyone evaluate compliance with the standard of care? Two engineers with different levels of experience in a particular geologic formation may conclude that quite a different number of borings and tests are required. The extremes are easy; if no borings were drilled, no tests conducted, and recommendations were based entirely on prior experience, most would conclude that in most circumstances that would be outside the standard of care. Textbooks, handbooks, and a variety of publications give guidance on conducting studies that, in most cases, are conservative and defer to the judgment and experience of the engineer. While the extreme cases are easy, and quite rare, most of the time the evaluation of the compliance with the standard of care by engineers falls into a gray area that results in a conflict with opinions by plaintiffs and defense experts.

The definition of standard of care as proposed by ASFE and incorporated in other definitions seems simple enough. Merely polling other engineers is likely to produce results with a distribution of responses that correspond to the complexity of the problem. Ultimately the question is answered, as far as the court is concerned, by the experts in the case. The plaintiff's expert will present an argument that the defendant geotechnical engineer failed to meet the standard of care either based on their experience or by reciting technical documents that, in their opinion, establish the failure on the part of the engineer. The defense expert will, of course, counter those arguments with their own experience and documents supporting the work of the defendant geotechnical engineer.

The difficulty with this system is that, in my experience, many plaintiffs' experts are highly educated, intelligent engineers with excellent academic credentials but with limited experience practicing the profession for which they now offer opinions on how others are conducting their work. Most practicing geotechnical engineers are reluctant to criticize their peers and rarely take on the role of the plaintiff's expert. The standard of care is never written down in a manner that has gained universal acceptance and therefore is always subject to debate.

In establishing whether negligence has occurred the testimony of the expert is subject to the rules of the court. As discussed below, under one set of rules of admissibility of evidence great weight is given to the education and training of an engineer to testify as an expert. Under another set of rules, while great weight is given to the education and training of the expert, a substantial weight is given to published peer-reviewed literature on the technical matter. Geotechnical engineers often believe that their technical strength is the local knowledge they have gained over many years of practice, which often is not recorded in publications. As in Roman times, the arguments are presented before the forum or in the present day equivalent, a judge or jury, and the side with the best facts and presentation will prevail.

THE RULES OF EXPERT EVIDENCE VERSUS THE PRACTICE OF ENGINEERING

The forensic evaluation is typically intended to come to an opinion as to the factors that led to the failure and were ultimately responsible for the failure. Geotechnical engineering, probably more than any other civil discipline relies on both the science of engineering and the empirical knowledge gained by an engineer in many years of practice. That combination of science and empiricism is considered the definition of the Standard of Care. The plaintiff's expert must reach the conclusion and offer the opinion that the defendant geotechnical engineer breached the standard of care at one or more points in the process leading to design. Because such arguments are so subjective, the defendant geotechnical engineer often says, "How can he say that?" There are in fact rules in the submission of evidence and the development of opinions. Experts are generally given great latitude in developing and presenting opinions, but there are rules nonetheless.

There are two main rules for the admissibility of expert testimony evidence in the United States. In all federal courts and in some state courts Daubert (Dejnožka, 2004), decided by the U.S. Supreme Court in 1993, is the basis for admissibility of evidence. In other states Frye (Dejnožka, 2004) is considered the basis by which admissibility of evidence is judged, and a few states have their own rules. These two rules have different criteria as to the admissibility of expert testimony in trial. The status of Frye or Daubert in state court varies. As shown in Table 1, 30 states have adopted Daubert or have rules of evidence consistent with Daubert, 14 states have rejected Daubert, and seven states have neither rejected Daubert nor accepted it. Daubert is the standard for expert testimony in all federal courts.

Consistent with Frye and as contained in the California Evidence Code 720, to qualify to give expert testimony:

> "(a) A person is qualified to testify as an expert if he [or she] has special knowledge, skill, experience, training, or education sufficient to qualify him as an expert on the subject to which his testimony relates."

The opposing side can object that the expert lacks the qualifications to testify as an expert but the court is given broad discretion in this judgment and commonly the testimony is admitted and the judge or jury must decide what weight to give the testimony.

Generally under Frye, judges are not considered "gatekeepers," meaning that the testimony is typically allowed and the jury or judge can weigh the testimony relative to testimony of other experts. The opposing side can challenge the testimony when that testimony is based on assumptions not supported by the record, is considered speculation, or based on factors or methods not accepted by other experts. Under Frye in California expert testimony must establish general acceptance and that correct scientific procedures were used. General acceptance implies that a consensus of the relevant qualified scientific community accepts the technique. This interpretation is consistent with the standard of care definition. However, the discretion of the court in allowing opinions in testimony is broad and can often lead to the situation when the defendant geotechnical engineer asks, "How can he say that?," the answer simply is that it is allowed by the law based on the expert's qualifications. Under Frye it is really up to the judge or jury to decide the credibility or believability of the testimony.

In 1993 the Supreme Court ruled that the trial judge must "ensure that any and all scientific testimony or evidence admitted is not only relevant, but reliable." This ruling established the trial judge as the gatekeeper of expert testimony in federal court and in those states that have adopted Daubert. The rules of admissibility of expert testimony are based on four criteria:

- Whenever a scientific theory or technique is used in the development of an opinion, has the theory or opinion been tested?
- Has the scientific theory or technique been subjected to peer review and publication?
- Are there standards to control the application of the scientific theory or technique and is there a known or potential error rate?
- Has the scientific theory or technique gained acceptance within the relevant scientific community?

Importantly Daubert requires that testimony be based on scientific knowledge. That knowledge must be more than a subjective belief or unsupported speculation and must apply to a body of known facts. The methodology used in arriving at the opinion must evaluate a testable hypothesis, been subject to peer review, and have a known or potential rate of error. Daubert does not require expert testimony to be accurate to 100% certainty since it is recognized that uncertainties exist in science and engineering.

Table 1. The Status of Daubert in State Courts 2006

States Adopting Daubert		
Alaska	Maine	Oklahoma
Arkansas	Massachusetts	Oregon
Connecticut	Michigan	Rhode Island
Delaware	Mississippi	South Dakota
Georgia	Montana	Tennessee
Idaho	Nebraska	Texas
Indiana	New Hampshire	Utah
Iowa	New Jersey	Vermont
Kentucky	New Mexico	West Virginia
Louisiana	Ohio	Wyoming

States Rejecting Daubert		
Arizona	Kansas	Pennsylvania
California	Maryland	South Carolina
Colorado	Nevada	Washington
District of Columbia	New York	Wisconsin
Florida	North Dakota	

States Neither Accepting Nor Rejecting Daubert		
Alabama	Minnesota	Virginia
Hawaii	Missouri	
Illinois	North Carolina	

The differences between Frye and Daubert can be seen in a dispute between plaintiffs and defense experts. Assume a slope failure occurs during construction adjacent to existing homes. A lawsuit is filed against the geotechnical engineer of record for the slope failure. The slope movements do not cause any visible or measurable damage to the homes. The plaintiff's expert claims that the slope movements have resulted in a reduction of lateral confinement for the soil under the existing homes. The loss of lateral confinement has, in their opinion, resulted in lateral expansion of the soil and the opening of cracks that cannot be seen at the surface. It is hypothesized that over the passage of time the infiltration of water from irrigation and rainfall will result in migration of surface soils into those cracks and eventually lead to differential settlement at the surface and subsequent damage to the homes. The plaintiff's expert acknowledges that the cracks cannot be detected as the process of investigating the cracks will obscure them.

The above example happens frequently in litigation. An expert will take an event such as a slope failure adjacent to homes and state that loss of lateral support leads to horizontal movement in the slope. That statement leads to a theory that the consequence of that event will have a future impact of substantial magnitude on the property. The current damage that will eventually lead to future damage is not testable, and therefore results in the proverbial dueling experts. Under Frye this testimony can be allowed based on the credentials of the expert. The credibility of the testimony will be determined by a judge or jury weighing the testimony against the testimony of other experts. Under Daubert this testimony can be challenged by the opposing side and a hearing can be held on the admissibility of the expert's testimony; the judge will have to make a determination as to its admissibility based on the criteria by Daubert. The defense will argue that the testimony amounts to nothing more than a subjective belief or unsupported speculation. There is a lack of peer reviewed literature supporting this hypothetical scenario. Additionally, the fact that this is not a testable hypothesis will increase the likelihood that the testimony will not be admitted.

CONSIDERATIONS FOR THE FORENSIC INVESTIGATION

The forensic investigation starts with the fact that a failure has occurred likely as a result of a geologic feature, soil property, groundwater condition, loading condition, constructed feature, negligence on the part of the engineer, construction defect, or some other condition that differs from those assumed during design. All parties begin the process knowing that something is different than assumed by the engineer. The question is whether that thing could have been foreseen, was knowable, was the result of an error in calculations, an omission, a negligent act on the part of the engineer, or a defect in construction caused by the contractor. The burden of proof rests with the plaintiff to demonstrate that the geotechnical engineer breeched the standard of care. In some cases the plaintiff's expert may simply review the existing data and conclude that the engineer was negligent in the number of borings, tests conducted, analyses performed, or observations made during construction.

In other cases there may be extensive investigations that uncover a previously unknown condition. The plaintiff's expert, in reviewing the existing or new information, will arrive at a completely different interpretation of data. The plaintiff's expert will conclude that the result of their investigation and their interpretations conclusively show the geotechnical engineer of record breached their duty to meet the standard of care. In addition to the conclusion on standard of care he or she will present a cost associated with mitigating the damages associated with the failure.

The time for investigation is limited to the discovery period defined by the court. The plaintiff presents its case and then the defense has the opportunity to conduct whatever investigation is felt necessary to rebut the allegations. Typically there is but one opportunity to conduct the defense's investigation. A consideration the defense will evaluate is "did the plaintiff prove their case?" Often in a standard of care case additional investigation does not necessarily help the defense's case regarding the negligence aspect. If in the course of their investigation the plaintiff uncovers some unknown condition. The defense will argue that the defendant engineer conducted the investigation, testing, analyses, and observations in accordance with the standard of care. The newly uncovered condition was not knowable or foreseeable. The plaintiff's investigation was done with the benefit of hindsight, unavailable to the geotechnical engineer at the time of their investigation.

Additional investigation is useful for the defense to develop its own cost of repair, regardless of how strong they may believe their case to be in arguing that the geotechnical engineer is not liable for the damage. In the court's deliberations the judge or jury may determine that an allocation of responsibility is appropriate and the defense would prefer that the allocation consider costs they believe to be more realistic. The defense expert must consider that the plaintiff may say that they accept the expert's cost estimate and ask that they conduct whatever repairs are believed necessary and guarantee the work. This is typically a negotiating strategy and is done prior to court as part of settlement discussions. In my experience, I have never seen a defendant geotechnical engineer agree to resolve a problem that they do not believe they have any responsibility for, for a plaintiff who has sued them, and then guarantee the work.

SUMMARY OF THE LITIGATION PROCESS

After the investigations have been completed and both sides in the conflict have an idea of the costs involved and an understanding of the strengths and weaknesses of their arguments, the process of financially resolving the matter can truly begin. In mediation I have seen mediators, when meeting privately with the engineers and their attorneys, ask the attorneys to tell their engineer client the cost of going to trial, a cost that the attorney would accept as a lump sum price. It is common that these costs can range from hundreds of thousands of dollars to over a million dollars. Generally attorney's fees are not recoverable unless the original contract provides for these fees.

Engineers often have difficulty agreeing to settle a case when, in their view, the facts do not indicate that they were negligent. Ultimately at this point the decision becomes based on business facts. The costs yet to be incurred and the risks of losing must be considered. The cost, both financially and emotionally, of the involvement of very senior personnel in the firm must also be considered. It is at this point that the ability of attorneys to deal with conflict far outweighs that of the engineer. More than 90% of all litigation cases I have been involved in have settled prior to trial.

The costs associated with settling the case can be mitigated by the quality of the forensic investigation. Having the best facts along with the best presentation will convince the other side of the weakness of their case and move settlement toward a reasonable amount. These arguments are illustrated in the case histories that follow.

CASE HISTORIES

The case histories described below are intended to demonstrate outcomes in court when juries have to decide technical matters based on their own life experiences. In one case a simple technical fact is well presented before a jury, contrasted with the second case where several complex technical issues were presented to a jury. Based on polling of the jury after they reached their verdict it was clear that all the experts had failed to present convincing arguments. The last case history presents a failure and the outcome was more in line with what all engineers would like to see as a result of a forensic investigation.

Good Facts and a Good Presentation

The first case history involves a very familiar scenario in geotechnical engineering. The project was the installation of utilities over a length of several miles with trench depths reaching 25 feet. Placing and compacting backfill in trenches at depths to 25 feet can be difficult and present challenges for all involved. The geotechnical engineer did not test the compaction at depths below 10 feet due to concerns about safety. All the compaction tests met the project specifications as the specifications were understood by all at the time. Shortly after completion of the backfilling of the trenches, settlements of up to four inches of the backfilled trenches had occurred at numerous locations.

After the settlements had been observed the geotechnical engineer evaluated the characteristics of the backfill through a series of borings. Samples were taken through the full depth of the backfill, density and water content measurements were made, and the compressibility of the backfill was measured. The degree of compaction of the samples was measured by combining the individual samples collected in the borings and then developing a compaction curve using ASTM 1557. The measured dry densities of the soil samples were typically from about 65 pounds per cubic foot (pcf) to about 80 pcf, with water content from about 35% to about 50%. The engineer concluded that, based on the lowest measured maximum dry density of 87 pcf, samples at dry densities less than 78 pcf would fail to meet the 90% relative compaction criteria. Additionally, based on compression test data, the engineer estimated that at the insitu densities and water content, settlements on the order of 1.5 inches to 3.5 inches were to be expected. The engineer concluded that the entire depth of backfill did not meet specifications and should be removed and recompacted and the settlement was due to construction defect.

The owner of the project demanded the contractor remedy the defect. The contractor declared bankruptcy at which point the owner called the bond the contractor put up for the project. At this point I was requested by the contractor's bonding company to review the data and the conclusions reached by the geotechnical engineer. The low dry density and high water content data certainly seemed quite inappropriate for a backfill of this use and likely to produce the results observed at the site. Despite what may appear to be obvious, it is very important to review the available documents and to the extent possible to understand the history of the project prior to reaching opinions regarding causation.

The original geotechnical report for the site recommended that the compaction requirements for the site be in accordance with CalTrans standard CTM 216, with the caveat that dry density be used. This is an important caveat since CTM 216 is not a dry density specification, as stated in the general scope for CTM 216:

> "Relative compaction in this method is defined as the ratio of the in-place wet density of a soil or aggregate to the test maximum wet density of the same soil or aggregate when compacted by a specific test method."

When the final specifications for the project were produced, the compaction requirement for trenches included the use of CTM 216 without any caveats as recommended by the geotechnical engineer on the use of dry density as opposed to a wet density; additionally there was a requirement that 90 percent compaction be achieved at depths below 12 inches. The engineer's recommendation in their report to use a dry density specification was not carried forth in the final specifications for the project, despite the geotechnical engineer's review of the final plans and specifications.

In subsequent meetings with all the parties it was pointed out that the investigation by the geotechnical engineer on the characteristics of the backfill was requiring the contractor to meet a standard that was not part of the construction documents. Despite what was intended on the part of the engineers regarding the use of a dry density compaction requirement that was not what the contractor was required to do by the contract. The engineers believed that they had significant experience in prior projects with the CTM 216 specification and that the fault had to be on the part of the contractor regardless of whether a wet or dry density specification was used. A decision was made by the owner of the project to remove all the backfill and recompact the same fill in place with the CalTrans CTM 216 specification as a dry density specification.

All the parties were allowed to sample the original backfill during the removal process. The wet density and water content data collected as the soil was excavated agreed fairly well with the data previously collected in borings by the geotechnical engineer. Despite the depth of the trenches and length of the project, there were a limited number of tests taken during construction. All the samples taken during construction met the specification both for wet densities and for dry densities.

Samples taken after construction generally had slightly lower wet densities and higher water content than samples tested during construction. However, attempting to determine density and water content data existing at the time of compaction at a later date can be imprecise. The data does show that soils at the recorded densities and water content will not perform as expected for a project of this type. The soils have very low dry densities and high water content and are susceptible to significant settlement. In my opinion, the data did indicate that based on the CTM 216 wet density specification as included in the project specifications, it was highly probable that the contractor placed the soils in accordance with the contract specifications.

The obligation of the contractor is to build the project according to the plans and specifications. The specifications, as included in the contract documents, required that the soils be placed in accordance with CTM 216, which is a wet density specification. The placement conditions of the backfill at the time of construction cannot be determined to an absolute certainty; however, in my opinion, it is highly probable that the compacted soils would have met the specifications. It is also true that the soils as they existed at the time of sampling following construction did meet the specifications. Based on the results of investigations the contractor's bonding company filed a lawsuit against the owner of the project and the geotechnical engineering to recover the bond money that had been paid. The complaint alleged that the settlement problems were the result of negligence and not a construction defect. The project had been built according to the specifications.

When a failure occurs it is universally true that the attorneys for the parties go back to the contract documents to understand what was agreed to by the parties at the initiation of the project. They are not looking for what people intended to do or what people thought should be done, but what was actually agreed to be done. At times this can be a matter of dispute; however, in this project it was clear that the parties contractually agreed to place the soils in accordance with CTM 216. That compaction specification was inappropriate for these soils and would allow for placement of backfill that would meet the specifications but would have a performance that would not meet the expectations of the project.

Several attempts were made to mediate the litigation that were ultimately unsuccessful. Lacking a resolution, the parties were committed to going to court for a jury trial. The contractor's bonding company's position was that the design was defective as a result of negligence on the part of the engineer. The negligence on the part of the engineer was the failure during their review of the specifications to note that the specifications did not meet their recommendations. The specification was inappropriate for this site, not consistent with the standard of care, and was always likely to lead to failure. The defense had several legal arguments as to the ability of the plaintiff to bring suit, but also argued that this specification was appropriate, used substantially in the practice, and the cause of the failure was defective construction.

The technical issues in the case were narrow and apparently easily understood by the jury. The inappropriateness of the specification was demonstrated to the jury by a container of soil that given its weight, would not meet the specification. However, for a total density specification, merely adding water to the soil to increase its total weight eventually would allow the soil to meet the specification. This simple demonstration helped the jury to understand the technical issues being debated by the experts. The jury returned a verdict in favor of the contractor and its bonding company for nearly the full amount of the damages against the owner of the project and the engineer.

Convincing a jury or judge in a case involving technical matters is a matter of having the best facts supporting the case and being able to present them in a way a lay person can understand, for either complex or simple technical issues.

Good Facts and a Bad Presentation

The second case history is far more complex technically. The site was a residential development with about 1400 home sites, and involved a massive grading project, with lots constructed on fill over gently sloping hillsides. The project had, in addition to the housing lots, a significant amount of open space surrounding the homes. Shallow failures occurred in the open space causing concern on the part of the Homeowners Association (HOA). A lawsuit was initiated by the HOA against the developer and subsequently all the parties involved in the design and construction of the project were parties to the lawsuit. A series of investigations were conducted by several different geotechnical consulting firms representing the different defendants. During the original investigation for the development, a number of existing landslides and colluvium were found in areas that were to receive fill for construction of lots for homes. Records by the geotechnical engineer of record of removal of the colluvium and landslide deposits during construction were sparse or non-existent. Additional investigation through boring and tests pits indicated that perhaps as many as 30 areas where homes were built could be fill constructed over soils that had been intended to be removed. The homes had been built several years prior to the litigation and some had existed up to about five years. No homes had been damaged by earth movement although, as indicated, landslides had occurred in adjacent open space.

The issue at dispute was the extent to which the stability of the existing fills supporting homes would need to be mitigated through construction of buttresses and installation of drainage to achieve appropriate factors of safety. The issues debated by the experts included:

- The areas underlain by the colluvium and/or landslides;
- The properties of the colluvium;
- The residuals strengths of the colluvium and landslides;
- The assumptions on groundwater levels;
- The effects of earthquake loading on soils at their residual strength; and
- Appropriate factors of safety to be used.

A cross section typical of the conditions at the site is shown below in Figure 1. While there was significant scatter in the soil data, there was agreement on the strength properties for the fill and colluvium. There was disagreement on the residual strength properties to be used, the location of the groundwater table, what would be appropriate factors of safety, and the meaning of calculated factors of safety for landslides at their residual strength.

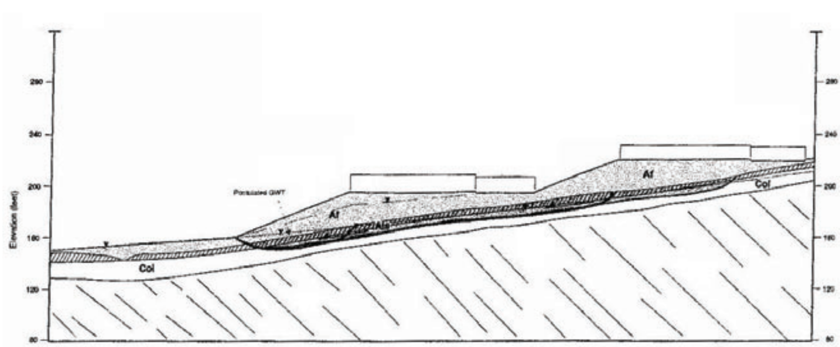

FIG. 1. Typical Cross Section.

The disagreement on the residual strength was primarily due to the curved nature of the residual strength envelopes. The calculated vertical stresses on nearly all the residual surfaces ranged from 500 pounds per square foot (psf) to about 2500 psf. Depending on the depth of the slide plane, a different linear approximation of the strength envelope was utilized, incorporating a different value of cohesion (c) and friction angle (phi) to get the correct shear strength for the appropriate range of stresses. Others choose to use a singular value of c and phi to approximate the shear strength at all depths.

There was significant disagreement on the appropriate factor of safety to be used. Commonly a factor of safety of at least 1.5 is used in residential construction. I argued that the use of a factor of safety of 1.5 or even higher is appropriate at a stage of investigation where data is being collected on the site for design purposes. However, when a failure has occurred or through substantial additional investigation the uncertainty on the data is greatly reduced, allowing for a lower factor of safety. This is particularly relevant when residual strengths are being used. The uncertainty associated with the strength of the soil when residual strengths are used is quite small. The uncertainty due to a lack of long term data on the phreatic surface led to conservative assumptions on location of groundwater. The conservative assumption that groundwater was at or near the ground surface combined with the equally conservative assumption on strength through the use of residual strengths warranted the use of a factor of safety lower than 1.5. It was my opinion that the only true variables in the analyses were the location of the groundwater surface and the shear strength of the soil. Give the conservative assumptions on both variables a factor of safety of 1.25 was appropriate.

Additionally, there was significant debate on the use of seismic coefficients and the meaning of factors of safety against earthquake loads. There are well established methods to establish the factor of safety against seismically induced deformations. These methods involve total stress conditions and the use of undrained strength properties. However, it was my opinion that these methods were not necessarily applicable to calculation of deformations of slopes where the soils are at their residual strengths. The residual strength of soils is calculated or measured as an effective stress condition. The use of an effective stress parameter in a total stress analyses would be inconsistent with basic soil mechanics.

There was no literature available on this topic to provide insight on this matter. In a State like California where Daubert is not accepted the testimony of all experts on this matter is allowed and the jury must decide this issue, weighing each expert's opinion. In other states where Daubert has been accepted and an opinion lacked peer reviewed technical literature the judge could be asked to not allow that opinion in the trial, essentially having the judge serve as the "gatekeeper" as intended by Daubert.

In developing recommendations as to which sites needed mitigation substantial cost differences occur when analyses are done with factors of safety of 1.25 versus a factor of safety of 1.5. The differences in assumptions by the experts resulted in an order of magnitude difference in the cost estimates to mitigate the problems at the site. My estimate was that approximately $2.5M dollars were required to provide the homes with an adequate factor of safety while the opposing expert estimated the cost at about $25M based on the assumptions in their analyses.

The trial was heard by a jury and lasted about five months. Testimony by experts lasted for several weeks. As an example of the uncertainty of jury decisions, the geotechnical engineer, who was a minor party to the case, settled for the full amount of their insurance policy while the jury was deliberating. When the jury returned their verdict they did not find that the geotechnical engineer was guilty of negligence. The jury came back with a verdict that the developer and contractor owed the HOA about $6.5M.

The jury members who agreed to be interviewed by the lawyers after the trial as to how they arrived at their verdict indicated that they never understood what the experts were talking about. At the end they did believe there was a problem and they thought the HOA should have money available to fix any issues that developed so they figured out the dollar amount on their own. This case illustrates the perils of allowing a jury to decide complex technical issues. It is not clear from the jury's perspective who had the best facts; it is clear that both sides had a poor method of presenting their facts and the story of their case. The issues were very complex and difficult to present in a simplified form. This case history also demonstrates the perils of letting technical issues go to jury trial for decisions.

Forensic Engineering as It Should Be

The third case history involves slope stability problems that occurred during the early phases of the Panama Canal widening project. The Panama Canal in the Gaillard Cut area is well known by geotechnical engineers as historically unstable. The widening of the Gaillard Cut area was a major part of the expansion of the Panama Canal designed to increase the flow of traffic and increase revenue. At the time of the beginning of the project the United States still had authority over the Canal.

During this early phase of the Canal widening a major landslide occurred at an area known as Hodges Hill. Historically this area was well known to be unstable and had been studied by Casagrande. Figure 2 below from Casagrande shows the general vicinity of Hodges Hill in 1912, 1915 and 1947. Over a period of about 35 years the slope progressively moved to become stable at its residual strength. The original designers were unaware of the concept of residual strength and believed that the soils could sustain a much steeper slope. The slopes are comprised of formations ranging from basalts to clay shale with low residual strengths. A cross section of the lithology is shown in Figure 3, where the various soil and rock types can be seen. The harder basalts overlay the weaker shale making back calculation of the residual strength a complex undertaking.

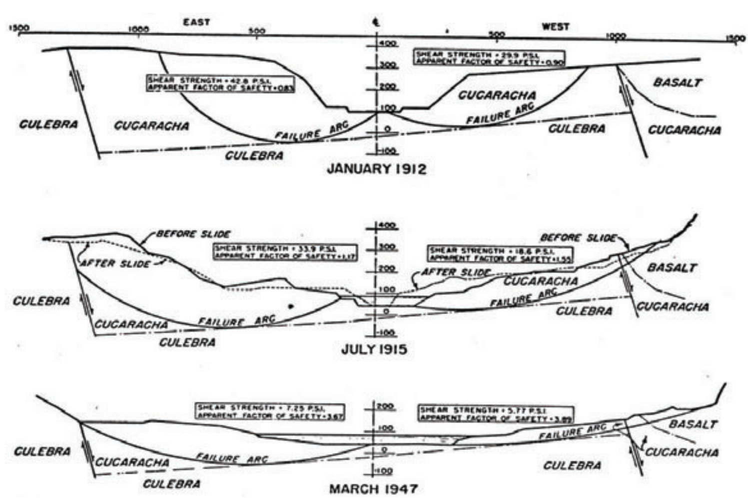

FIG. 2. Historical Landslide Stability.

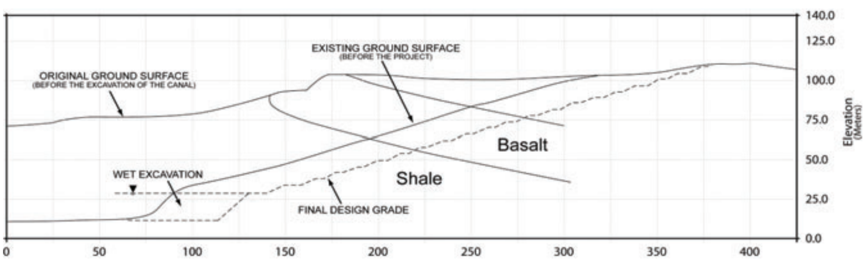

FIG. 3. Geologic Cross Section.

The geotechnical engineers at the Panama Canal Commission and the designers of the project are a very experienced and knowledgeable group with a good understanding of the properties and behavior of soils at their residual strength. They were very aware of the history of the slopes and the fact that the existing factor of safety was very near to unity, and that over steepening of the slopes during construction would result in a high probability of failure. The intent of the design was to cut the slope back to an angle equal to the original slope angle before removing the toe in what was called the "wet excavation" that would create the additional width for the canal. With this sequence of excavation the overall factor of safety should remain equal to or greater than the original factor of safety.

The construction specifications were clear as to how the excavation was to be sequenced, as taken from the specifications and shown below:

> "General …The initial part of the excavation at any location shall be performed by complete progressive benching commencing from the top of the cut and working down to the lowest bench. Any other method of constructing the slopes shall be submitted for approval of the Contracting Officer. The contractor shall never leave slopes at angles exceeding design grades during excavation work."

The landslides occurred in late July of 1997 and in January 1998.The slides occurred from about an elevation of 85m down to the Canal elevation of about 26m. This area of Hodges Hill has been subject to previous movements in 1910 and 1912. My review consisted of a site visit, extensive meetings with the staff of the Panama Canal Commission reviewing the design and subsequent construction of the cut slope, and the analyses of the failures. The specific issues to be addressed in my evaluation were as follows:

- Was the original design performed correctly?
- Was the slope constructed in accordance with the plans and specifications?
- What was the cause of the 1997 and 1998 landslides?
- What are the impacts of the slope failure on the wet excavation project?

In 1968 cracks were discovered on Hodges Hill at approximately the same location as the 1997 and 1998 landslides. When the movements causing the cracks observed in 1968 actually took place is unknown. Lutton's conclusion regarding the 1968 cracks is as follows (Lutton, 1975):

> "Activity in April 1968 is seen in retrospect to have fallen short of failure. There seems to be no strong evidence or reason to suppose a through going sliding surface had developed. It was suggested in a letter to members of a board of consultants on the problem (Memo, E and C Director, July 16, 1968) that a logical shear zone pattern was not in evidence from data collected on subsurface motion. The depths and areas in which motions were recorded suggested shearing activity on several planes with separate blocks of material shifting independently. No reactivation followed despite the fact that at least two heavy rains fell that wet season."

No subsequent significant movements were noted in the Hodges Hill slope even during the record rainfall year of 1986.

The project was required to facilitate the subsequent wet excavation for the widening of the Canal. The basic criteria for the design of this and other cut slopes is that the factor of safety at all stages of construction and at completion of excavation be greater than or equal to the factor of safety that existed prior to excavation. This criterion is imposed due to difficulty of evaluating the actual factor of safety in the Cucaracha Shale and the underlying Culebra Formation.

The Hodges Hill slope was considered stable based on criteria used at the Canal (factor of safety greater than 1.0) and based on its historical performance and the results of an analytical evaluation. The analytic evaluation used the existing slope geometry, assumptions on piezometric surfaces and strengths, and assumed modes of failure.

For the purpose of design, piezometric data was available from piezometers installed in 1968 and from piezometers installed in 1995. The highest piezometric levels from the period of 1968 to 1972 were compared to data collected from 1995 to 1997. The data from the 1968 to 1972 period was more conservative (higher piezometric evaluation) than the 1995 to 1997 data for the purposes of evaluating the stability of the slope and the design of the excavation of the slope. Using residual strength values for the Cucaracha and Culebra Formations, the calculated factor of safety of the slope was less than one, indicating the slope was unstable. Since this conclusion contradicted the observed performance of the slope, the higher, fully softened strength values were appropriately used for design.

The analyses for design of the cut slope assumed that the Culebra and Cucaracha Formations had similar strength properties. Evaluation of both fully softened and residual strength envelopes derived from previous analyses and laboratory testing for the Culebra and Cucaracha Formations indicate that the Culebra Formation has similar strength to the Cucaracha Formation. For the purpose of design, the assumption that both strengths are the same is appropriate. The analyses considered several modes of failure including failures at the bottom of the wet excavation cut and failure surfaces within the slope. The various modes of failure considered were sufficient to evaluate the most critical failure surfaces.

Based on the fully softened shear strength parameters used in the design, the slope was considered stable in its pre-construction configuration. If the slope angle was maintained or flattened during and after construction, the factor of safety of the slope would not be decreased from its pre-construction stable state. During construction this critical condition was to be maintained.

The specifications clearly require an unloading of the slope from the top to the bottom. With the final slope equal to or flatter than the existing slope the basic design criteria would not have been violated.

The design was performed in an appropriately conservative manner. The design evaluated the previous evidence of movement in the slope and through careful analyses concluded that the slope was not at its residual state allowing the use of the fully softened strength. This conclusion was consistent with that of Lutton and with the prior performance of the slope. The specifications provided a means for excavating the slope in a way that the basic design criteria would be met at all stages of construction and at completion. It was my opinion that the design was performed correctly and was consistent with the standard of care normally exercised by engineers.

Based on review of cross sections provided by the contractor as a basis of progress payments, the geometry of the excavation can be evaluated at any time during construction. The cross sections clearly indicate that the excavation did not proceed in accordance with the required specifications. The cross section, shown in Figure 6, at approximately the center of the 1997 and 1998 landslides clearly shows that, by July 1997, prior to the landslide, substantially more excavation had occurred at the toe than at the top of the slope. This was in violation of the specifications and in violation of the design criteria.

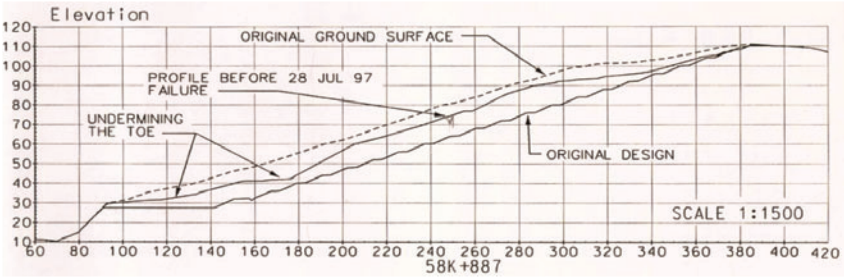

FIG. 4. Cross Section of Excavation Prior to Failure.

The 1997 landslide at the Hodges Hill excavation is a result of the failure of the contractor to follow the plans and specifications. The overexcavation of the toe of the slope resulted in a steeper slope than existed prior to construction. This violated the basic design criteria of maintaining the factor of safety equal to or greater than the factor of safety that existed prior to construction. The excavation sequence during and prior to July 1997 resulted in a reduction in the factor of safety to less than one and the subsequent movement of the slope. Back analyses of the failed slope indicated a significant reduction in the factor of safety and the resulting reduction in the friction angle due to the movement of the slope.

The January 1998 movement was almost entirely within the 1997 movements and was merely a continuation of the earlier movements with the strength reduced from fully softened to a residual value. After the 1997 movements the slope was in an unstable condition and small, seemingly insignificant changes in the geometry of the slope could lead to additional movements.

The original design required that following the completion of the project and the subsequent wet excavation project the factor of safety of the slope would not be reduced below its value prior to the dry excavation. The slope movements of 1997 and 1998 resulted in a reduction of the strength of the soils in the slope. At the slope configuration existing following the 1997 and 1998 movements the factor of safety of the slope has been reduced due to the reduced soil strength. Excavation of the toe of the slope, as proposed for the wet excavation project, would likely result in a significant failure of the slope and potentially a major impact on the Canal. It was my recommendation that the Panama Canal Commission continue with its plan, which included the following:

- Installation of instruments to evaluate the depth of movements,
- Continued reevaluation of the reduced friction angle of the slope using the 1997 piezometric data,
- Design of reconstructed slope angles that result in a slope with the same degree of stability as originally considered in the design, and
- Allowing the contractor to continue the current excavation in accordance with the plans and specifications.

I advised the Commission that continuing excavation on the slope may result in additional movements of the impacted area. At that time all the evidence indicated that the movements were entirely within the slope and not impacting the Canal. It was my opinion that any additional movements would be insignificant, and similar to the movements observed in January 1998 of a few feet. Localized regrading may be required to flatten slopes to mitigate movements prior to development of a mitigation plan for the entire slope.

While the conclusion was that the landslide was a construction defect, it was pointed out to the Commission that the specifications for the sequencing of the excavation were clear the plans were less specific. The plans showed the initial and final configurations of the slope but made no mention of the required sequence of construction. For the purposes of a construction claim against the contractor there was enough data to pursue such a claim. However, one of the issues was to address future construction issues to facilitate a more efficient project. I recommended that future plans be more specific on the proposed sequence of construction.

Secondly, the contract had a single unit price for excavation. The excavation at the top of the slope was in basalt while the excavation at the toe of the slope was in shale. The contractor initially started the excavation at the top of the slope but moved to the toe, likely as a result of the slower rate of excavation in the basalt. The cash flow for the contractor on the project could be improved by the faster rate of excavation of the shale. I recommended that the Commission consider a provision in the bid documents for different unit rates for excavation at the top of the slope as opposed to the toe of the slope to provide an incentive to the contractor.

Thirdly, the Commission had a division of labor based on the responsibility of different departments within their organization. The construction management group was separate from the design group. A lack of communication between the designers and the construction managers on the importance of the excavation sequence contributed to the failure. I recommended that a member of the design team be involved in the construction of the project and future projects.

While a claim against the contractor was contemplated, to the best of my knowledge, none was ever filed. While this case history could be cited as forensic engineering in a perfect world, it remains a unique case history in my experience. This project was at the beginning of a project of major scope in the world and could be considered a "lessons learned" situation for the remainder of the project, where potential financial savings could be achieved.

CONCLUSIONS

The practice of forensic geotechnical engineering is the application of geotechnical engineering to answer questions of interest to the legal profession. In today's litigious society the failure of a geotechnical engineering project brings together the engineering and legal professions to resolve the conflict of responsibility for the failure. The geotechnical engineer must apply science and engineering within the rules of the legal system in order that their work can be effective in helping to bring about a resolution to the conflict. The rules of the legal system governing the admissibility of opinions can vary from state to state and between state and federal court.

When an expert presents their opinion on the cause or responsibility for a failure, that opinion must be well founded and presented in a way that the judge or jury understands the technical matters at dispute in the litigation. Judges and jurors must reach a decision in the matter. When left with a poor presentation or poorly understood conflicts on complex technical matters they must use their own life experiences to decide responsibility for the failure. While jurors and judges do their best to sort out the issues the results can often be confusing. Settlement of the dispute prior to proceeding to trial is almost always the preferable outcome.

A thoughtful, high quality forensic investigation consistent with good science and engineering combined with an ability to clearly present the matters being disputed will always aid in the settlement of the dispute.

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Foundation Design for Tall Buildings

Harry G. Poulos[1], Dist. MASCE

[1]Coffey Geotechnics, 8/12 Mars Road, Lane Cove West, NSW Australia 2066; Phone +61 2 9911 1000; Fax +61 2 9911 1001; harry_poulos@coffey.com

ABSTRACT: This paper sets out the principles of a limit state design approach to design a pile or piled raft foundation system for tall buildings, and involves three sets of analyses:

1. An overall stability analysis in which the resistances of the foundation components are reduced by the appropriate geotechnical reduction factor and the ultimate limit state (ULS) load combinations are applied.
2. A serviceability analysis, in which the best-estimate (unfactored) values foundation resistances and stiffnesses are employed and the serviceability limit state (SLS) loads are applied.
3. An analysis to obtain foundation loads, moments and shears for structural design of the foundation system.

The importance of appropriate parameter selection and load testing is emphasized. The approach is illustrated via its application to a high-rise building in Korea.

INTRODUCTION

The last two decades have seen a remarkable increase in construction of tall buildings in excess of 150m in height, and an almost exponential rate of growth. A significant number of these buildings have been constructed in the Middle East and Asia, and many more are either planned or already under construction. “Super-tall” buildings in excess of 300m in height are presenting new challenges to engineers, particularly in relation to structural and geotechnical design. Many of the traditional design methods cannot be applied with any confidence since they require extrapolation well beyond the realms of prior experience, and accordingly, structural and geotechnical designers are being forced to utilize more sophisticated methods of analysis and design. In particular, geotechnical engineers involved in the design of foundations for super-tall buildings are increasingly leaving behind empirical methods and are employing state-of-the art methods.

There are a number of characteristics of tall buildings that can have a significant influence on foundation design, including the following:

1. The building weight increases non-linearly with increasing height, and thus the vertical load to be supported by the foundation, can be substantial.

2. High-rise buildings are often surrounded by low-rise podium structures which are subjected to much smaller loadings. Thus, differential settlements between the high- and low-rise portions need to be controlled.
3. The lateral forces imposed by wind loading, and the consequent moments on the foundation system, can be very high. These moments can impose increased vertical loads on the foundation, especially on the outer piles within the foundation system.
4. The wind-induced lateral loads and moments are cyclic in nature. Thus, consideration needs to be given to the influence of cyclic vertical and lateral loading on the foundation system, as cyclic loading has the potential to degrade foundation capacity and cause increased settlements.
5. Seismic action will induce additional lateral forces in the structure and also induce lateral motions in the ground supporting the structure. Thus, additional lateral forces and moments can be induced in the foundation system via two mechanisms:
    a. Inertial forces and moments developed by the lateral excitation of the structure;
    b. Kinematic forces and moments induced in the foundation piles by the action of ground movements acting against the piles.
6. The wind-induced and seismically-induced loads are dynamic in nature, and as such, their potential to give rise to resonance within the structure needs to be assessed. The fundamental period of vibration of a very tall structure can be very high, and since conventional dynamic loading sources such as wind and earthquakes have a much lower predominant period, they will generally not excite the structure via the fundamental mode of vibration. However, some of the higher modes of vibration will have significantly lower natural periods and may well be excited by wind or seismic action.

This paper will review some of the challenges that face designers of foundations for very tall buildings, primarily from a geotechnical viewpoint. The process of foundation design and verification will be described for a proposed tall tower in Korea.

TYPICAL HIGH-RISE FOUNDATION SETTLEMENTS

Before discussing details of the foundation process, it may be useful to review the settlement performance of some high-rise buildings in order to gain some appreciation of the settlements that might be expected from two foundation types founded on various deposits. Table 1 summarizes details of the foundation settlements of some tall structures founded on raft or piled raft foundations, based on documented case histories in Hemsley (2000), Katzenbach et al (1998), and from the author's own experiences. The average foundation width in these cases ranges from about 40m to 100m. The results are presented in terms of the settlement per unit applied pressure, and it can be seen that this value decreases as the stiffness of the founding material increases. Typically, these foundations have settled between 25 and 300mm/MPa. Some of the buildings supported by piled rafts in stiff Frankfurt clay have settled more than 100mm, and despite this apparently excessive settlement, the performance of the structures appears to be quite satisfactory. It may therefore be concluded that the tolerable settlement for tall structures can be well in excess of the conventional design values of 50-65mm. A more critical issue for such structures may be overall tilt, and differential settlement between the high-rise and low-rise portions of a project.

Table 1. Examples of Settlement of Tall Structure Foundations

Foundation Type	Founding Condition	Location	No. of Cases	Settlement per Unit Pressure mm/MPa
Raft	Stiff clay	Houston;	2	227-308
	Limestone	Amman; Riyadh	2	25-44
Piled Raft	Stiff clay	Frankfurt	5	218-258
	Dense sand	Berlin; Niigata	2	83-130
	Weak Rock	Dubai	5	32-66
	Limestone	Frankfurt	1	38

FOUNDATION DESIGN ISSUES

The following issues will generally need to be addressed in the design of foundations for high-rise buildings:

1. Ultimate capacity of the foundation under vertical, lateral and moment loading combinations.
2. The influence of the cyclic nature of wind, earthquakes and wave loadings (if appropriate) on foundation capacity and movements.
3. Overall settlements.
4. Differential settlements, both within the high-rise footprint, and between high-rise and low-rise areas.
5. Possible effects of externally-imposed ground movements on the foundation system, for example, movements arising from excavations for pile caps or adjacent facilities.
6. Earthquake effects, including the response of the structure-foundation system to earthquake excitation, and the possibility of liquefaction in the soil surrounding and/or supporting the foundation.
7. Dynamic response of the structure-foundation system to wind-induced (and, if appropriate, wave) forces.
8. Structural design of the foundation system; including the load-sharing among the various component of the system (for example, the piles and the supporting raft), and the distribution of loads within the piles. For this, and most other components

of design, it is essential that there be close cooperation and interaction between the geotechnical designers and the structural designers.

A general process by which these issues can be considered is set out below.

FOUNDATION DESIGN PROCESS

The process of foundation design is well-established, and generally involves the following aspects:

1. A desk study and a study of the geology and hydrogeology of the area in which the site is located.
2. Site investigation to assess site stratigraphy and variability.
3. In-situ testing to assess appropriate engineering properties of the key strata.
4. Laboratory testing to supplement the in-situ testing and to obtain more detailed information on the behaviour of the key strata than may be possible with in-situ testing.
5. The formulation of a geotechnical model for the site, incorporating the key strata and their engineering properties. In some cases where ground conditions are variable, a series of models may be necessary to allow proper consideration of the variability.
6. Preliminary assessment of foundation requirements, based upon a combination of experience and relatively simple methods of analysis and design. In this assessment, considerable simplification of both the geotechnical profile(s) and the structural loadings is necessary.
7. Refinement of the design, based on more accurate representations of the structural layout, the applied loadings, and the ground conditions. From this stage and beyond, close interaction with the structural designer is an important component of successful foundation design.
8. Detailed design, in conjunction with the structural designer. As the foundation system is modified, so too are the loads that are computed by the structural designer, and it is generally necessary to iterate towards a compatible set of loads and foundation deformations.
9. In-situ foundation testing at or before this stage is highly desirable, if not essential, in order to demonstrate that the actual foundation behavior is consistent with the design assumptions. This usually takes the form of testing of prototype or near-prototype piles. If the behavior deviates from that expected, then the foundation design may need to be revised to cater for the observed foundation behavior. Such a revision may be either positive (a reduction in foundation requirements) or negative (an increase in foundation requirements). In making this decision, the foundation engineer must be aware that the foundation testing involves only individual elements of the foundation system, and that the piles and the raft within the system interact.
10. Monitoring of the performance of the building during and after construction. At the very least, settlements at a number of locations around the foundation

should be monitored, and ideally, some of the piles and sections of the raft should also be monitored to measure the sharing of load among the foundation elements. Such monitoring is becoming more accepted as standard practice for high-rise buildings, but not always for more conventional structures. As with any application of the observational method, if the measured behavior departs significantly from the design expectations, then a contingency plan should be implemented to address such departures. It should be pointed out that departures may involve not only settlements and differential settlements that are greater than expected, but also those that are significantly smaller than expected.

DESIGN CRITERIA

Limit State Design Approach – Ultimate State

There is an increasing trend for limit state design principles to be adopted in foundation design, for example, in the Eurocode 7 requirements and those of the Australian Piling Code (1995). In terms of limit state design using a load and resistance factor design approach (LRFD), the design criteria for the ultimate limit state are as follows:

$$R_s^* \geq S^* \tag{1}$$
$$R_g^* \geq S^* \tag{2}$$

where R_s^* = design structural strength = ϕ_s. R_{us}, R_g^* = design geotechnical strength = ϕ_g. R_{ug}, R_{us} = ultimate structural strength, R_{ug}= ultimate strength (geotechnical capacity), ϕ_s = structural reduction factor, ϕ_g = reduction factor for geotechnical strength, and S^* = design action effect (factored load combinations).

The above criteria are applied to the entire foundation system, while the structural strength criterion (equation 1) is also applied to each individual pile. It is not considered to be good practice to apply the geotechnical criterion (equation 2) to each individual pile within the group, as this can lead to considerable over-design. R_s^* and R_g^* can be obtained from the estimated ultimate structural and geotechnical capacities, multiplied by appropriate reduction factors. Values of the structural and geotechnical reduction factors are often specified in national codes or standards. The selection of suitable values of ϕ_g requires considerable judgment and should take into account a number of factors that may influence the foundation performance. As an example, the Australian Piling Code AS2159-1995 specifies values of ϕ_g between 0.4 and 0.9, the lower values being associated with greater levels of uncertainty and the higher values being relevant when a significant amount of load testing is carried out.

Load Combinations

The required load combinations for which the structure and foundation system have to be designed will usually be dictated by an appropriate structural loading code. In some cases, a large number of combinations may need to be considered. These may include several ultimate limits state combinations, and serviceability combinations incorporating long-term and short-term loadings.

Design for Cyclic Loading

In addition to the normal design criteria, as expressed by equations 1 and 2, it is suggested that an additional criterion be imposed for the whole foundation of a tall building to cope with the effects of repetitive loading from wind and/or wave action, as follows:

$$\eta\, R_{gs}^* \geq S_c^* \tag{3}$$

where R_{gs}^* = design geotechnical shaft capacity, S_c^* = maximum amplitude of wind loading, and η = a reduction factor.

This criterion attempts to avoid the full mobilization of shaft friction along the piles, thus reducing the risk that cyclic loading will lead to a degradation of shaft capacity. In most cases, it is suggested that η can be taken as 0.5, while S_c^* can be obtained from computer analyses which give the cyclic component of load on each pile, for various wind loading cases.

Soil-Structure Interaction Issues- Analyses for Structural Foundation Design

When considering soil-structure interaction to obtain foundation actions for structural design (for example, the bending moments in the raft of a piled raft foundation system), the worst response may not occur when the pile and raft capacities are factored downwards (for example, at a pile location where there is not a column, load acting, the negative moment may be larger if the pile capacity is factored up). As a consequence, additional calculations may need to be carried out for geotechnical reduction factors both less than 1 and greater than 1. As an alternative to this duplication of analyses, it would seem reasonable to adopt a reduction factor of unity for the pile and raft resistances, and the factor up the computed moments and shears (for example, by a factor of 1.5) to allow for the geotechnical uncertainties. The structural design of the raft and the piles will also incorporate appropriate reduction factors.

Serviceability Limit State

The design criteria for the serviceability limit state are as follows:

$$\rho_{max} \leq \rho_{all} \tag{4}$$

$$\theta_{max} \leq \theta_{all} \tag{5}$$

where ρ_{max} = maximum computed settlement of foundation, ρ_{all} = allowable foundation settlement, θ_{max} = maximum computed local angular distortion and θ_{all} = allowable angular distortion.

Values of ρ_{all} and θ_{all} depend on the nature of the structure and the supporting soil. Table 1 sets out some suggested criteria from work reported by Zhang and Ng

(2006). This table also includes values of intolerable settlements and angular distortions. The figures quoted in Table 2 are for deep foundations, but the authors also consider separately allowable settlements and angular distortions for shallow foundations, different types of structure, different soil types, and different building usage. Criteria specifically for very tall buildings do not appear to have been set, but it should be noted that it may be unrealistic to impose very stringent criteria on very tall buildings on clay deposits, as they may not be achievable. In addition, experience with tall buildings in Frankfurt (see Table 1) suggests that total settlements well in excess of 100mm can be tolerated without any apparent impairment of function. It should also be noted that the allowable angular distortion, and the overall allowable building tilt, reduce with increasing building height, both from a functional and a visual viewpoint.

Table 2. Suggested Serviceability Criteria for Structures (Zhang and Ng, 2006)

Quantity	Value	Comments
Limiting Tolerable Settlement mm	106	Based on 52 cases of deep foundations.
Observed Intolerable Settlement mm	349	Based on 52 cases of deep foundations.
Limiting Tolerable Angular Distortion rad	1/500	Based on 57 cases of deep foundations.
Limiting Tolerable Angular Distortion rad	1/250 (H<24m) to 1/1000 (H>100m)	From 2002 Chinese Code H = building height
Observed Intolerable Angular Distortion rad	1/125	Based on 57 cases of deep foundations.

Dynamic Loading

Issues related to dynamic wind loading are generally dealt with by the structural engineer, with geotechnical input being limited to an assessment of the stiffness and damping characteristics of the foundation system. However, the following general principles of design can be applied to dynamic loadings:

- The natural frequency of the foundation system should be greater than that of the structure it supports, to avoid resonance phenomena. The natural frequency depends primarily on the stiffness of the foundation system and its mass, although damping characteristics may also have some influence.
- The amplitude of dynamic motions of the structure-foundation system should be within tolerable limits. The amplitude will depend on the stiffness and damping characteristics of both the foundation and the structure.

It is of interest to have some idea of the acceptable levels of dynamic motion, which can be expressed in terms of dynamic amplitude of motion, or velocity or acceleration. Table 3 reproduces guidelines for human perception levels of dynamic motion, expressed in terms of acceleration (Mendis et al, 2007). These are for vibration in the low frequency range of 0-1 Hz encountered in tall buildings, and incorporate such factors as the occupant's expectancy and experience, their activity, body posture and orientation, visual and acoustic cues. They apply to both the translational and rotational motions to which the occupant is subjected. The acceleration levels are a function of the frequency of vibration, and decrease as the frequency increases. For example, allowable vibration levels at a frequency of 1 Hz are typically only 40-50% of those acceptable at a frequency of 0.1 Hz. It is understood that, for a 10 year return period event, with a duration of 10 minutes, American practice typically allows accelerations of between 0.22 and 0.25m^2/s for office buildings, reducing to 0.10 to 0.15 m^2/s for residential buildings.

Table 3. Human Perception Levels of Dynamic Motion (Mendis et al, 2007)

Level of Motion	Acceleration m^2/s	Effect
1	<0.05	Humans cannot perceive motion
2	0.05 - 0.1	Sensitive people can perceive motion. Objects may move slightly
3	0.1 – 0.25	Most people perceive motion. Level of motion may affect desk work. Long exposure may produce motion sickness.
4	0.25 – 0.4	Desk work difficult or impossible. Ambulation still possible.
5	0.4 – 0.5	People strongly perceive motion, and have difficulty in walking. Standing people may lose balance.
6	0.5 – 0.6	Most people cannot tolerate motion and are unable to walk naturally.
7	0.6 – 0.7	People cannot walk or tolerate motion.
8	> 0.85	Objects begin to fall and people may be injured.

Design for Ground Movements

Foundation design has traditionally focused on loads applied by the structure, but significant loads can also be applied to the foundation system because of ground movements. There are many sources of such movements, and the following are some sources that may be relevant to tall buildings:

1. Settlement of the ground due to site filling, reclamation or dewatering. Such effects can persist for many years and may arise from activities that occurred decades ago and perhaps on sites adjacent to the present site of interest. Such vertical ground movements give rise to negative skin friction on the piles within the settling layers.
2. Heave of the ground due to excavation of the site for basement construction. Ground heave can induce tensile forces in piles located within the heaving ground. Excavation can also give rise to lateral ground movements, which can induce additional bending moments and shears in existing piles.

3. Lateral and vertical movements arising from the installation of piles near already-installed piles. These movements may induce additional axial and lateral forces and bending moment in the existing piles.
4. Dynamic ground motions arising from seismic activity. Such kinematic motions can induce additional moments and shears in the piles, in addition to the inertial forces applied by the structure to the foundation system.

Such ground movements do not reduce the geotechnical ultimate capacity of the piles, but have a two-fold influence:

The foundations are subjected to additional movements which must be considered in relation to the serviceability requirements. Because the action of ground movements on piles is a soil-structure interaction problem, the most straight-forward approach to designing the piles for the additional forces and moments is to compute the best-estimate values, and then apply a factor on these computed values to obtain the design values, as suggested previously for on soil-structure interaction.

DESIGN METHODS AND TOOLS

Once the necessary geological and geotechnical information has been obtained, the design process generally involves three key stages:

1. Preliminary analysis, assessment and design;
2. The main design process
3. Detailed analyses to check for complexities that may not be captured by the main design process.

The methods and tools that are employed in each of these stages need to be appropriate to the stage of design. Some typical design methods, and their mode of use, are set out below.

Preliminary analysis and design

In this stage, use can make use of spreadsheets, MATHCAD sheets or simple hand or computer methods which are based on reliable but simplified methods. It can often be convenient to simplify the proposed foundation system into an equivalent pier and then examine the overall stability and settlement of this pier. For the ultimate limit state, the bearing capacity under vertical loading can be estimated from the classical approach in which the lesser of the following two values is adopted:

1. The sum of the ultimate capacities of the piles plus the net area of the raft (if in contact with the soil);
2. The capacity of the equivalent pier containing the piles and the soil between them, plus the capacity of the portions of the raft outside the equivalent pier.

For assessment of the <u>average</u> foundation settlement under working or serviceability loads, the elastic solutions for the settlement and proportion of base load of a vertically loaded pier (Poulos, 1994) can be used, provided that the geotechnical profile can be simplified to a soil layer overlying a stiffer layer. Figures 1 and 2 reproduce these solutions, from which simplified load-settlement curves for an equivalent pier containing different numbers of piles can be estimated, using the procedure described by Poulos and Davis, 1980).

An alternative approach can be adopted, using the "PDR" approach described by Poulos (2002). In this approach, the simplified equations developed by Randolph (1994) can be used to obtain an approximate estimate of the relationship between average settlement and the number of piles, and between the ultimate load capacity and the number of piles. From these relationships, a first estimate can be made of the number of piles, of a particular length and diameter, to satisfy the design requirements.

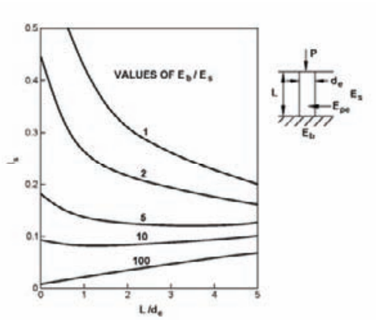

FIG. 1. Settlement of equivalent pier in soil layer (Poulos, 1994).

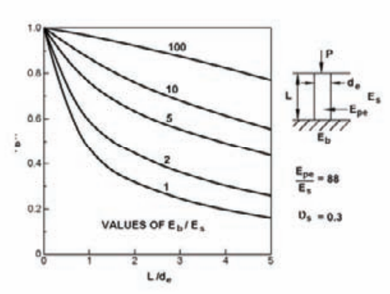

FIG 2. Proportion of base load for equivalent pier (Poulos, 1994).

Main design evaluation and sensitivity study

For this stage, it may be appropriate to use computer methods for pile and pile-raft analysis such as, DEFPIG (Poulos, 1990), PIGLET (Randolph, 2004), GROUP8 (Ensoft, 2010), REPUTE (Geocentrix, 2006), GARP (Small and Poulos, 2007) and NAPRA (Mandolini et al, 2005). All such programs have some limitations; for example, some assume a rigid cap or raft, some do not allow for contact between the cap/raft and the soil, and some can only consider vertical loading. However, all these programs are capable of allowing for non-linear pile-soil behavior, albeit in an approximate manner, and accordingly, the following procedure may be employed:

1. For the stability of the foundation system in the ultimate limit state, the ultimate limit state loading combinations are applied and the pile resistances are reduced by a geotechnical reduction factor. The foundation system satisfies the criterion in equation (1) if it does not collapse under any of the imposed load combinations.
2. For the average settlement of the foundation system, the serviceability limit state loadings (or the working loads) are applied to the foundation system. In this analysis, the pile resistances are *not* factored, but are instead best estimates, while the geotechnical stiffness characteristics employed in the analysis are also best estimates. For long-term loadings, long-term geotechnical and structural parameters are used, whereas, for wind and earthquake loadings, short-term stiffness and strength parameters, and short-term structural stiffness characteristics, are used.

From these analyses, the number and arrangement of piles in the foundation system can be adjusted in order to seek an optimal computed performance.

Detailed design and the final design check

For the final stages of design, once the basic pile configuration has been decided, it may be desirable to use a finite element and finite difference analyses, preferably three-dimensional, such as PLAXIS 3D and FLAC3D to verify that the foundation performance is consistent with that computed from the main design stage, and also to examine the influence of any factors such as the effects of the lateral resistance of the raft and or surrounding walls on the lateral response of the foundation system. Caution should be exercised in using two-dimensional analyses as they can often be misleading and can give settlements, differential settlements and pile loads which are inaccurate, for example, as noted by Prakoso & Kulhawy (2001).

In this stage, every effort should be made to ensure the following requirements are satisfied:

1. The geotechnical model used is appropriate for the ground conditions;
2. The constitutive behavior of the soil layers is consistent with the behavior of the foundation soils;
3. The geotechnical parameters have been assessed appropriately;
4. Account is taken of the stiffness of the superstructure when computing the foundation performance.

In many cases, the key outputs from the geotechnical analyses are the equivalent spring stiffnesses of each pile within the foundation system, as well as those of the various portions of the raft. These stiffnesses are for both vertical and lateral responses, and if necessary, torsional responses, of the piles. These characteristics are provided to the structural designer who can then incorporate them into the complete structure-foundation model. In this way, the most realistic estimates may be made of the settlement and differential settlements, and proper account can be taken of the interactions between the structure and the foundation. Clearly, such a process requires close cooperation and understanding between the structural and foundation designers.

Geotechnical parameter assessment

A key element in undertaking each of the three stages of design is to try and employ geotechnical parameters that are consistent with the method being used. In a preliminary analysis, when there is a paucity of geotechnical data, it may be appropriate to employ parameters based on SPT values However, such parameters would be quite inappropriate in a three dimensional finite element analysis carried out for the detailed design stage (although no doubt this does happen on occasions). Reliable quantitative data on major high-rise projects can generally be derived from in-situ testing, especially cone, pressuremeter and dilatometer tests. While high-level laboratory testing remains feasible, it is often overlooked because of cost, timing and availability problems.

In deriving soil stiffness values to be used for settlement predictions, seismic testing, either via seismic cones or geophysical methods, is becoming increasingly important. Such testing enables the small-strain shear modulus, G_0, to be derived from the measured shear wave velocity. This small-strain value may then be used either with a suitable constitutive model which considers the strain-dependency of stiffness, or alternatively to estimate the operative stiffness for the stress or strain levels appropriate to the foundation system. Mayne et al (2009) describe a simple approach which has been used successfully with elastic theory to predict non-linear load-settlement characteristics of single piles. Such an approach may be able to be extended to consider pile groups.

Poulos et al (2001) have developed an approximate approach in which values of the secant Young's modulus (relative to the small-strain value) can be estimated as a function of the factor of safety against failure An example of such relationships are reproduced in Figure 3 for a clay soil, and for axially loaded piles, laterally loaded piles and shallow foundations. It will be noted that, for a given factor of safety, different values of the secant modulus apply to the different foundation situations, because of the differences in the strain levels induced in the soil.

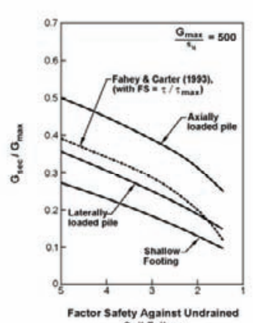

FIG. 3. Secant modulus ratio for various foundation types on clay: G_o/s_u = 500 (Poulos et al, 2001).

When modeling a foundation system using a soil model that does not incorporate the stress- or strain–dependency of soil stiffness, it is still possible to make approximate allowance for the increase in stiffness with increasing depth below the foundation by using a modulus that increases with depth. From approximate calculations using the Boussinesq theory to compute the distribution of vertical stress with depth below a loaded foundation, it is possible to derive a relationship between the ratio of the modulus to the small strain value, as a function of relative depth and relative stress level. Such a relationship is shown in Figure 4 for a circular foundation, and may be used as a convenient, albeit approximate, means of developing a more realistic ground model for foundation design purposes. When applied to pile groups, the diameter can be taken as the equivalent diameter of the pile group, and the depth is taken from the level of the pile tips.

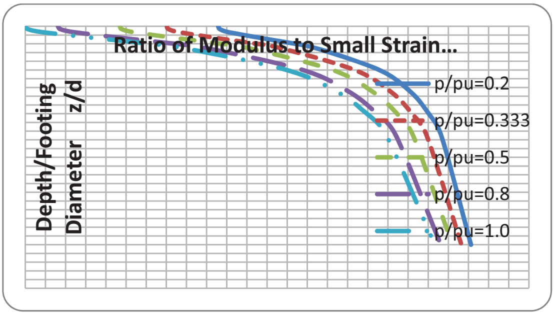

FIG. 4. Ratio of modulus to small-strain modulus for circular foundation (p=applied pressure, p_u = ultimate pressure).

The Role of Pile Testing

Pile testing is an essential component of tall building foundation design. The results of such tests serve several purposes, including:

1. Verification of the design assumptions regarding pile shaft and base capacity;
2. Verification of design assumptions regarding pile head stiffness;
3. Verification of the construction technique and the integrity of the as-constructed shaft and base.

With the increase in required pile capacities as buildings have become taller, there has been increasing use made of the Osterberg cell test technique (Osterberg, 1989). This test is attractive because it is self-reacting, and with suitable placement of the cells, can load the pile base and pile shaft to failure, unlike most other types of test. Of particular interest is the ability to identify "soft toes" developed during construction and flaws in pile shaft construction if the shaft is suitably instrumented.

APPLICATION TO THE INCHEON TOWER, KOREA

Introduction

A 151 storey super high-rise building project has been under design since 2008, located in reclaimed land constructed on soft marine clay in Songdo, Korea. This building is illustrated in Figure 5 and is described in some detail by Badelow et al (2009); thus, only a brief summary is presented here.

FIG. 5. Incheon 151 Tower (artist's impression).

Ground Conditions and Geotechnical Model

The Incheon area has extensive sand/mud flats and near shore intertidal areas. The site lies entirely within an area of reclamation, which is likely to comprise approximately 8m of loose sand and sandy silt, constructed over approximately 20m of soft to firm marine silty clay, referred to as the Upper Marine Deposits (UMD). These deposits are underlain by approximately 2m of medium dense to dense silty sand, referred to as the Lower Marine Deposits (LMD), which overlie residual soil and a profile of weathered rock.

The lithological rock units present under the site comprise granite, granodiorite, gneiss (interpreted as possible roof pendant metamorphic rocks) and aplite. The rock materials within about 50 metres from the surface have been affected by weathering which has reduced their strength to a very weak rock or a soil-like material. This depth increases where the bedrock is intersected by closely spaced joints, and sheared and crushed zones that are often related to the existence of the roof pendant sedimentary / metamorphic rocks. The geological structures at the site are complex and comprise geological boundaries, sheared and crushed seams - possibly related to faulting movements, and jointing.

From the available borehole data for the site, inferred contours were developed for the surface of the "soft rock" founding stratum within the tower foundation footprint and it was found that there was a potential variation in level of the top of the soft rock (the pile founding stratum) of up to 40m across the foundation.

The footprint of the tower was divided into eight zones across the site, these being considered to be representative of the variation of ground conditions and geotechnical models were developed for each zone. Appropriate geotechnical parameters were selected for the various strata based on the available field and laboratory test data, together with experience of similar soils on adjacent sites. One of the critical design issues for the tower foundation was the performance of the soft UMD under lateral and vertical loading, hence careful consideration was given to the selection of parameters for this stratum. Typical parameters adopted for foundation design are presented in Table 4.

Foundation Layout

The foundation comprises a 5.5 m thick concrete mat and piles supporting columns and core walls. The numbers and layout of piles and the pile size were obtained from a series of trial analyses through collaboration between the geotechnical engineer and the structural designer. The pile depth was determined by considering the performance and capacity of piles of various diameters and length. The pile depths required to control settlement of the tower foundation were greater than those required to provide the geotechnical capacity required. The pile design parameters for the weathered/soft rock layer are shown in Table 5 and were estimated on the basis of the pile test results in the adjacent site and the ground investigation data such as pressuremeter tests and rock core strength tests.

The final design employed 172 piles of 2.5m diameter, founded in the "soft rock" stratum, with lengths below the base of the raft varying from about 36m to 66 m, depending on the depth to the desired founding level. The base of the raft was about 14.6m below ground surface level. The pile layout was selected from the various options considered, and is presented in Figure 6.

Table 4. Summary of Geotechnical Parameters

Strata	Typical Thickness m	E_v MPa	E_h MPa	f_s kPa	f_b MPa
UMD (4 layers)	25.2	7 – 15	5-11	29-48	-
LMD	2.5	30	21	50	-
Weathered Soil	2.0	60	42	75	-
Weathered Rock	13.5	200	140	500	-
Soft Rock (above EL-50m)	10.0	300	210	750	12
Soft Rock (below EL-50m)	36.5	1700	1190	750	12

E_v = Vertical Modulus f_s = Ultimate shaft friction
E_h = Horizontal Modulus f_b = Ultimate end bearing

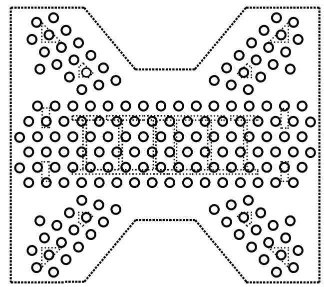

FIG. 6. Foundation Layout.

Table 5. Ultimate Resistances for Pile Analysis

Material	Ultimate Friction f_s(kPa)	Ultimate End Bearing f_b(MPa)
Weathered Rock	500	5
Soft Rock	750	12

Loadings

The overall loadings used for the foundation design were developed by the structural designer and are summarized in Table 6.

Table 6. Design Load Components

Load Component	Value
Dead Load	5921.4 MN
Live Load	639 MN
Horizontal wind (x-direction)	149 MN
Horizontal wind (y-direction)	115 MN
Earthquake (x-direction)	110 MN
Earthquake (y-direction)	110 MN
Moment (x-direction)	21600 MNm
Moment (y-direction)	12710 MNm

Preliminary Assessment

For the preliminary assessment of the foundation performance, a simplified geotechnical model was adopted, with constant layer thicknesses being assumed beneath the building footprint. A constant pile length of 50m was adopted for the calculations.

For the preliminary assessment, only the effects of vertical loading were considered, with emphasis being placed on the load capacity and the settlement under the dead plus live loading. The ultimate axial capacity of a single pile was computed to be 244 MN. Thus, for the single pile failure mode, the computed group capacity under vertical loading was 172*244 = 41968 MN, while the net raft capacity was 5930 MN, giving a total of 47898 MN. For the block failure mode, the total capacity of the block containing the piles, the soil between the piles, and the portions of raft outside the piles was computed to be in excess of 60000 MN, and thus the single pile failure was found to be critical. The overall factor of safety under purely vertical load was therefore 47898/6560.4 = 7.3 which was considered to be more than adequate. In terms of the limit state criterion in equation 2, a reduction factor of 0.65 was used to factor down the foundation capacity, together with load factors on dead load and live load of 1.25 and 1.5 respectively. It was found that the criterion was easily satisfied.
The average settlement was computed using the equivalent pier approach and the curves in Figure 1. Under the dead plus live loading, the average settlement was computed to be about 75 mm, which again was considered to be acceptable for preliminary design purposes.

Detailed Assessment

Analysis of overall stability. For the detailed design phase, the ultimate limit state (ULS) combinations of load were input into a series of non-linear pile group analyses using a computer program CLAP (Combined Load Analysis of Piles) developed by Coffey (2007). The pile axial and lateral capacities were reduced by geotechnical reduction factors of 0.65 for axial load, and 0.40 for lateral load). The smaller factors for lateral load reflected the greater degree of uncertainty for lateral response. For the detailed analysis with CLAP, it was possible to take account of differing soil profiles and hence the eight different profiles identified during the ground interpretation process were employed.

In all cases analyzed, the foundation system was found to be stable, i.e. the computed foundation movements were finite, and generally the maximum computed settlement under the ULS loadings was less than 100mm. Thus, the overall stability condition was deemed to be satisfied.

Cyclic stability. From the CLAP analyses, the components of cyclic wind loading were obtained and used to check the cyclic stability criterion in equation 3. Figure 7 plots the ratio η for each pile in the group. The largest cyclic load component in any pile was 29.2 MN, and the ratio η of this cyclic load to the factored-down pile shaft resistance was found to be 0.43, which was less than the maximum allowable value of 0.5. Thus the cyclic load criterion was satisfied and little or no cyclic degradation of pile capacity should be expected.

Predicted performance under vertical loading. For the settlement analysis of the foundation system, the computer program GARP (Small and Poulos, 2007) was used as the main analysis tool. The GARP analysis used as the main design tool, and without taking any account of the stiffness of the superstructure, the analysis gave a maximum settlement of 67mm and a maximum differential settlement of 34mm. The maximum angular rotation (not taking into account the stiffness of the superstructure) was found to be 1/780. Both the settlement and angular rotation values were considered to be acceptable.

It can be noted that the very rough average preliminary settlement from the chart in Figure 1 of 75mm was of a similar order to that obtained from the GARP analysis.

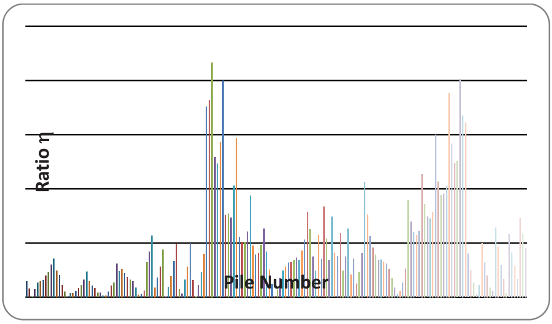

FIG. 7. Results of cyclic loading analysis – Load Case 0.75(DL+LL+WL).

Detailed assessment of final foundation design. To provide a check on the GARP analyses, and to examine the effects on foundation performance of the basement wall surrounding the piled raft, analyses were also carried out using the commercially-available program PLAXIS 3-D Foundation. For the purposes of this paper, and subsequent to the execution of the foundation design, the PLAXIS analyses were also used to examine the effects of including the presence of the raft, and two separate cases were analysed:

1. The piles being connected to the raft which is in contact with the underlying soil but not with the surrounding soil above the raft base level (Case 1). This is the usual case considered for a piled raft, where only contact below the raft is taken into account.
2. The piles being connected to the raft, which is in contact with both the underlying soil and the soil surrounding the basement walls of the foundation system (Case 2). This is the actual case that is to be constructed. In this case, account was taken of vertical walls that are 14.6m deep and 1.2m thick.

Plate elements had to be fixed to the bottom of the solid elements of the raft and the pile heads fixed to the plate as this is required in PLAXIS if the pile heads are to rotate with the raft. The sides of the excavation were supported by retaining walls that were modelled in the mesh. The finite element mesh for the problem (Case 2) is shown in Figure 8, and it may be seen that the soil is divided into layers representing the materials of Table 2. Because of the limitations of PLAXIS 3D, the soil profile was assumed to be horizontally layered below the foundation footprint with the same profile as that employed in the preliminary assessment. The soil layers were treated as Mohr-Coulomb materials to allow for non-linear effects,

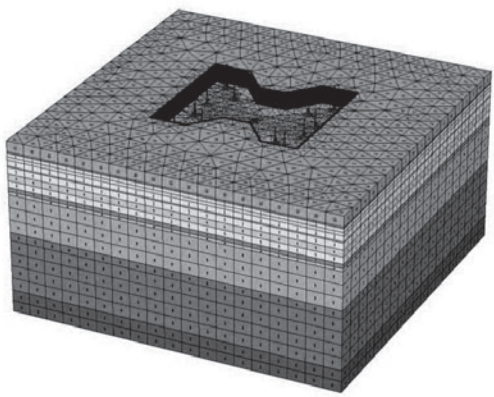

FIG. 8. Finite element mesh showing material layers.

Vertical loading. In the first analysis, vertical loading only was applied to the foundation for both Case 1 and Case 2. As may be expected, when contact between the basement walls and the soil is considered, the deflection of the raft is reduced. This may be seen from the load-deflection plot of Figure 9 where the percentage of load applied to the raft versus the vertical deflection is plotted. The reduction in vertical displacement caused by taking the embedment of the raft into consideration is about 8 mm in this case.

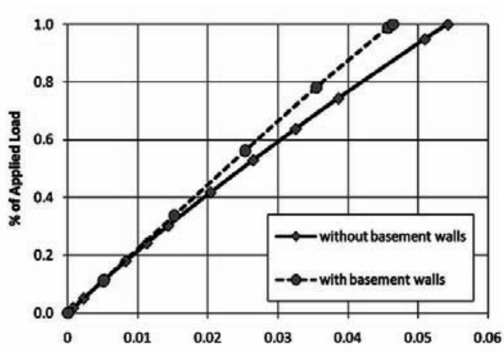

FIG. 9. Load-deflection behaviour at raft centre (vertical loading).

These values of maximum settlement and maximum differential settlement were somewhat less than those from the GARP analysis (56mm max. settlement and 40mm differential, versus 67mm and 34mm from GARP). This difference may reflect the inherent conservatism in the use of interaction factors within the GARP analysis. Nevertheless, the agreement between the two analyses was considered to be adequate and the comparison indicates that a program such as GARP can be a very useful design tool, particularly when a large number of different cases (pile number and configurations) is to be analysed prior to deciding upon the final layout and number of piles.

Horizontal loading. In order to examine the effects of including the soil above the raft in the analysis, a PLAXIS 3-D analysis was undertaken for lateral loading only for both Case 1 and Case 2 as well as the case where the raft was assumed not to be in contact with the ground. The latter case was modelled in PLAXIS by placing a thin soft layer of soil underneath the raft. The results of the analysis showed that the predicted lateral deformation at working load was less than the conventional case of no raft contact, when the contact or embedment of the foundation was taken into account as shown in Figure 10. This figure shows the lateral deflection at the central point of the raft versus the percentage of lateral load applied to the foundation.

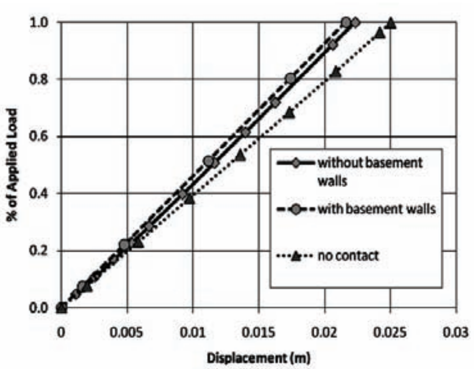

FIG. 10. Load-deflection behaviour of central point of the raft (Horizontal loading).

Deformed meshes in the case of horizontal loading are presented in Figures 11 (Case 1) and 12 (Case 2). Because of the bending of the piles under lateral loading, it is of interest therefore to compare the moments induced into one of the piles in the leading row for each of the cases. Figure 13 shows bending moment distributions for a pile on the leading edge of the raft. It may be seen that, when the raft is in contact with the soil at the sides of the basement above raft level, and/or the raft is in contact with the ground, the bending moments that were calculated via the finite element analysis are lower than from the conventional type of analysis (where the raft is assumed to make no contact). However, in this case because of the large number of piles, the effect of the walls on the reduction of pile moment is small.

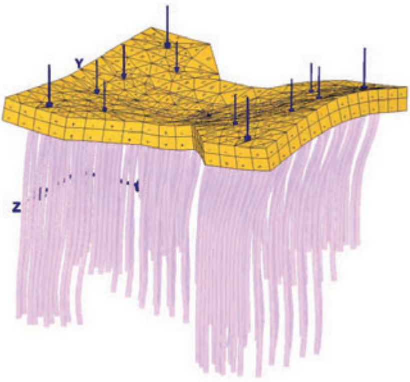

FIG. 11. Deformed raft and piles under lateral loading (Case 1).

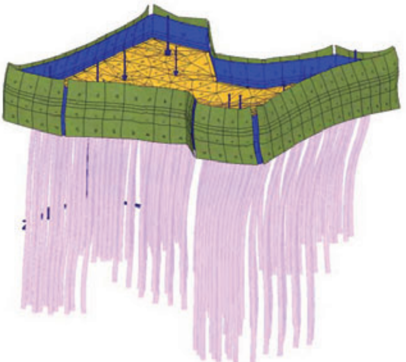

FIG. 12. Deformed raft and piles under lateral loading(Case 2).

The program CLAP, which is a modified version of the computer program DEFPIG (Poulos, 1990), was used as the main design tool for considering the lateral response of the foundation. It is interesting to note that CLAP gave a maximum lateral displacement of 22mm and a maximum pile bending moment of 15.7MNm. These values are comparable to those obtained from PLAXIS 3D and indicate that, for the design of piled rafts with a large number of piles, it is probably adequate to ignore the presence of the cap when computing the lateral response of the foundation and the distribution of bending moment within the piles.

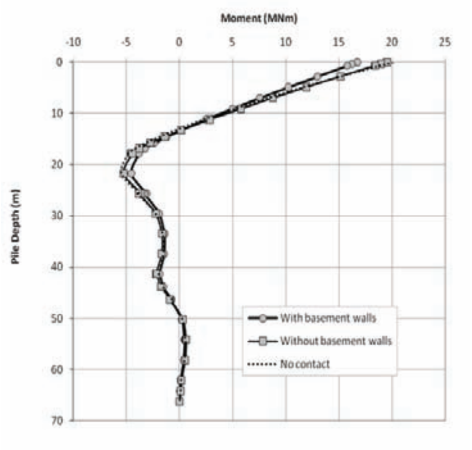

FIG. 13. Pile moments for horizontal loading, with and without contact.

Pile Load Testing

A total of five pile load tests were planned, four on vertically loaded piles via the Osterberg cell procedure, and one on a laterally loaded pile jacked against one of the vertically loaded test piles. For the vertical pile tests, two levels of O-cells were installed in each pile, one at the pile tip and another at between the weathered rock layer and the soft rock layer. The cell movement and pile head movement were measured by LVWDTs in each of four locations, and the pile strains were recorded by the strain gauges attached to the vertical steel bars. The vertical pile tests were undertaken in early 2010 after which the project was put on hold. One of the tests was found to have construction-related defects and was excluded from consideration. The average and range of results of the remaining three tests are shown in Table 7 for the two main supporting layers. It can be seen that the performance of the test piles exceeded design expectations, and that there could be scope for re-evaluating the foundation pile configuration.

Table 7. Assessed Average Performance of Three Test Piles

Location	Parameter	Ultimate Design Value	Average Mobilized & Range
Soft Rock	End Bearing (MPa)	12.0	24.3 (18.9-37.6)
	Shaft Friction (kPa)	750	1534 (1326-1994)
Weathered Rock	Shaft Friction (kPa)	500	708 (356-1054)

CONCLUSIONS

This paper has set out an approach for the design of pile foundation systems for high-rise buildings, using a limit state design approach. This approach involves three sets of analyses: an overall stability analysis using factored-down soil and pile resistances, an ultimate limit state analysis using unfactored soil and pile resistances to obtain design structural actions, and a serviceability analysis. In addition, a check can be carried out to assess the ratio of cyclic load amplitude to factored-down pile shaft resistance. It is suggested that if this ratio for a pile is less than about 0.5, there should be a low risk of cyclic degradation of shaft resistance occurring.

The application of the approach has been illustrated via its use for the 600m tall Incheon tower. It has been demonstrated that it is possible to obtain reasonably consistent outputs from the various stages of design, ranging from preliminary analysis using hand calculation tools, through the main design process using appropriate design software, to the detailed analysis and final checking phase using high-level numerical analysis. Via the latter analyses, the effect of considering the embedment of the raft was found to have a relatively modest effect on foundation settlements and pile load lateral response and bending moments.

A finding of practical importance is that for tall buildings supported by piled raft foundations with a large number of piles, a conventional pile group or piled raft analysis may often be adequate, albeit conservative, for estimating the vertical and lateral behaviour of the foundation, and the distributions of pile load and bending moment within the piles in the foundation system.

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Geotechnical Engineering Education: The State of the Practice in 2011

Andrea L. Welker[1], M. ASCE, P.E.

[1]Associate Professor, Civil and Environmental Engineering Department, Villanova University, Villanova, PA 19085; andrea.welker@villanova.edu

ABSTRACT: The current state of geotechnical engineering education in the United States is examined in this paper. K-12 outreach efforts, undergraduate and graduate education, continuing education opportunities, faculty demographics, the influence of the American Society of Civil Engineers (ASCE), and the challenges and changes facing geotechnical engineering education are examined. Several sources of data were used to develop this paper including results from a survey to members of the United States Universities Council on Engineering Education and Research (USUCGER), results from an informal survey provided to practitioners, university websites, and published literature. The K-12 outreach efforts appear to have been successful as enrollments in civil engineering have demonstrated strong growth over the past decade. Nearly all (93%) accredited civil engineering programs require soil mechanics and most (83%) require soil mechanics laboratory. Geotechnical engineers comprise 11% of the civil engineering faculty and about three quarters of all programs have two or fewer geotechnical engineering faculty members. Geotechnical engineering faculty are supportive of ASCE's Policy Statement 465 and The Body of Knowledge. The key challenges facing geotechnical engineering education are falling credit-hour requirements for the attainment of a bachelor's degree, time and resources needed to support laboratories, effectively incorporating complex topics into classes, and balancing the importance of hands-on practical experience with ever-increasing research demands.

INTRODUCTION

There is increasing emphasis on teaching and learning at all levels within engineering. There are sustained efforts to reach out to students in K-12, several programs focused on "teaching the teachers," journals and conferences focused on education, and new standards requiring continuing education for licensed professional engineers. Lifelong learning is no longer a goal, it is the norm.

Sessions at conferences and publications on education by geotechnical engineers are one indication of the importance of this topic. Geotechnical engineers, to their credit, periodically examine the current state of engineering education; most recently in 2005 (Culligan, et al. 2005 and Mullen, et al. 2005). At the annual Geo-Institute conference in 2000 there was a session devoted to education, and each conference since 2004 has also had an education session. Seventy-one records were found in the American Society of Civil Engineers (ASCE) database when performing a combined search for engineering education and geotechnical engineering. Geotechnical engineers are also well-represented at the annual American Society for Engineering Education (ASEE) conference.

Geotechnical engineering faculty members created their own advocacy organization, the United States Universities Council on Geotechnical Education and Research (USUCGER), in 1986. There are 385 accredited engineering programs in the United States; of these programs, 224 grant accredited civil engineering degrees (ABET 2011). At the time of writing, USUCGER had 122 member universities: 117 grant accredited civil engineering degrees and five grant some other accredited engineering degree. Thus USUCGER represents more than half of the accredited civil engineering programs. When USUCGER was originally founded, the “E” stood for “Engineering,” but in 2003 “Engineering” was replaced by “Education” to highlight the importance of education to its members. The mission of USUCGER is “to provide advocacy for the continued development and expansion of high quality geomechanical, geotechnical, and geo-environmental engineering research and education which will enhance the welfare of humankind and meet the needs of the nation.” More information on USUCGER, including its history and member organizations, can be found online at http://www.usucger.org/. Periodically, USUCGER holds workshops; the most recent was in 2008 in Sacramento, CA. Nearly one-half of the time at this workshop was devoted to educational issues.

Various sources were used to gather the data needed to write this paper and are described below. The information found in these sources was then reinforced by published literature on engineering education.

- *University websites*: university websites were used to obtain information about geotechnical engineering programs such as the number of geotechnical engineering faculty members, their gender, whether soil mechanics was required, other required and elective geotechnical courses, and graduate course offerings.
- *Informal interviews with 14 practicing geotechnical engineers*: interviews were conducted for another paper (Kunberger, et al. 2011), but some of the information gathered is relevant to this paper. A summary of the 14 practitioners’ demographics is provided below:
  - the number of years with employer varied greatly (2 to 20) as did the rank of the interviewee (field engineer to principal);
  - ten were men (71%) and four were women (29%);
  - one was from the public sector and 13 from private-sector consulting firms; and

   - twelve were Professional Engineers in at least one state and two were Engineers in Training.
- *American Society for Engineering Education data*: geotechnical engineering is widely recognized as a fundamental sub-specialty within civil engineering. Nationwide data specific to geotechnical engineering is difficult to obtain, so data for civil engineering will be used to elucidate trends.
- *Survey of the USUCGER membership*: a ten-question survey was administered electronically to the USUCGER membership. The survey was open for 15 days and had 70 respondents. It was used to obtain information that is not readily available on the Internet. For example, respondents were asked if they supported the ASCE Policy Statement 465 and what they perceive as the biggest challenges to geotechnical engineering education.

K-12 OUTREACH

Geotechnical engineers actively reach out to younger students to ensure that there is an adequate pipeline of undergraduate students. There are a plethora of outreach programs, both formal and informal, across the country, thus a comprehensive listing is not practical. Opportunities to interact with younger students abound, e.g., both the Girl Scouts and Boy Scouts offer badges for completing engineering projects and many middle schools have technology or engineering clubs.

There are many resources for those participating in an outreach program, examples of which are presented in this paper. The Museum of Science in Boston has created a comprehensive engineering curriculum for elementary school students entitled "Engineering is Elementary." There are approximately 20 units in the series, one of which is devoted to geotechnical engineering. This unit includes a storybook entitled *Suman Crosses the Karnali River* and supplementary curricular materials. For a broad array of outreach ideas and practices, the sessions offered at the annual ASEE conference by the K-12 and Precollege Division provide papers and workshops. Many of the techniques described in these sessions can be applied to geotechnical engineering. Examples of specific geotechnical engineering outreach efforts can be found in recent publications, such as Iskander, et al. (2010); Kunberger and Csavina (2010); Elton, et al. (2006); and Fiegel, et al. (2000).

UNDERGRADUATE

A positive outcome of K-12 outreach efforts is that civil engineering has experienced robust growth over the past ten years and that we continue to attract women to the profession. The growth in the number of civil engineering degrees awarded since 2000 (34%) is greater than the growth experienced by engineering degrees overall (23%) (Figure 1). Most of this growth has occurred since 2004, about four years after the total number of engineering degrees awarded started to increase. These rates of growth in degrees granted are anticipated to continue as enrollments have also shown robust growth and the economic downtown has increased interest in the value of STEM degrees. In 2010, 20% of civil engineering degrees were awarded to women, as compared to 18% overall (Gibbons 2011). Civil engineers directly improve the lives of others, which plays a significant role in our ability to attract women to our profession. This role in society must continue to be emphasized if civil engineering is to continue attracting women and other underrepresented groups.

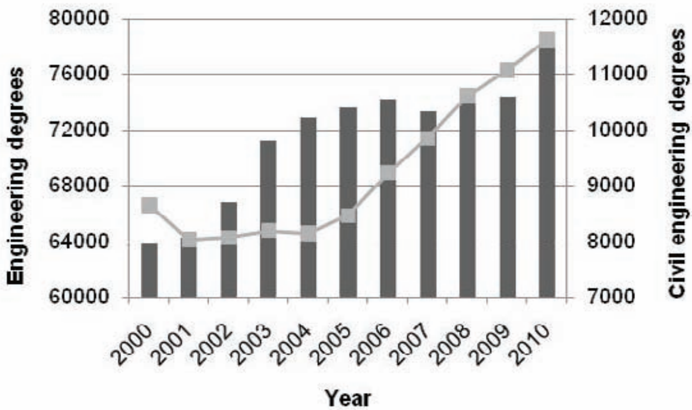

Figure 1. Number of bachelor's degrees awarded yearly; all engineering degrees (bars) on the primary y-axis and civil engineering degrees (line) on the secondary y-axis (Gibbons 2011)

Geotechnical engineering is a fundamental component of a civil engineering education; thus, Soil Mechanics, or its equivalent (Geotechnical Engineering I, Introduction to Geotechnical Engineering, etc), is a required course for 93% of accredited civil engineering programs. Over the past decade, many engineering programs have sought to increase the flexibility of their programs; this change is reflected in the fact that only 37% of accredited programs require a second course in geotechnical engineering, while 75% offer a geotechnical engineering elective after soil mechanics. Foundation Design is the most frequently offered second geotechnical engineering course whether it is required or an elective. Some programs allow qualified undergraduates to enroll in graduate classes, which greatly increases the number of courses a student can choose from for their electives.

Geotechnical engineering professors are interested in what text book is used for Soil Mechanics because it is a required course at nearly all universities. The respondents to the USUCGER survey reported using eight different texts for undergraduate soils mechanics (Table 1). Three professors reported they do not use any text book and two reported using more than one text book. *Principles of Geotechnical Engineering* by Das (various editions) is the most widely-used textbook, while *Geotechnical Engineering: Principles and Practices* by Coduto or Coduto, Yeung, and Kitch and *Introduction to Geotechnical Engineering* by Holtz and Kovacs or Holtz, Kovacs, and Sheahan, are the second-most widely used. This distribution is similar to that reported by Grigg, et al. (2005), although the gap between the most popular text and the second most popular texts has narrowed.

Table 1. Textbook used for undergraduate soil mechanics (or equivalent) as reported in the USUCGER survey

Textbook	Number	Notes
B. Das, *Principles of Geotechnical Engineering*	23	Most were satisfied, but not enthusiastic
D. Coduto or D. Coduto, et al., *Geotechnical Engineering: Principles and Practices*	15	Several noted that the 2nd edition is greatly improved from the 1st
R. Holtz and W. Kovacs or R. Holtz, et al., *Introduction to Geotechnical Engineering*	15	Most reported being very happy with both editions of the text
M. Budhu, *Soil Mechanics and Foundations*	5	Covers critical state soil mechanics
None	3	
R. Craig, *Craig's Soil Mechanics*	2	Both respondents reported being very satisfied with this text
D. McCarthy, *Essentials of Soil Mechanics and Foundations*	2	
I. Dunn, et al., *Fundamentals of Geotechnical Analysis*	1	Out of print
H-Y. Fang and J. Daniels, *Introductory Geotechnical Engineering: An Environmental Perspective*	1	

Bonwell and Eison (1991) are often credited with coining the term “active learning.” Since the publication of their study for the Association of the Study of Higher Education (ASHE), the use of active learning techniques has become widespread. There is no universally accepted definition for active learning, rather, it is a broad term that includes all forms of instruction that place the onus of learning onto the learner. The Civil Engineering Division of ASEE (which has five standing subcommittees: Committee on Education Policy, Committee on Professional Practice, Committee on Effective Teaching, Committee on Instructional Technology, and the ASCE Liaison Committee) publishes many papers on innovations in the classroom each year. Some recent examples of publications that address the use of active learning techniques for geotechnical engineering courses are:

- Problem based learning: Akili (2010)
- Case histories: a workshop on the use of case histories in geotechnical engineering was recently held (September 2011) at the XV European Conference on Soil Mechanics & Geotechnical Engineering and the use of

case histories was described by Hagerty (2010) and Godoy and Covassi (2010)
- Discussion: Kunberger and O'Neill (2010)
- Technology: Hanson, et al. (2010).

The professors responding to the USUCGER survey believe that laboratory experiences, which are an example of active learning, are a key component of undergraduate education (Table 2). Nearly all of the respondents to the survey (63/64) believe that a laboratory is critically important or important to teaching soil mechanics; however, the website analysis revealed that only 83% of accredited civil engineering programs require soil mechanics laboratory. This disconnect between the importance that professors place on laboratory work and the curriculum is a challenge facing geotechnical engineering education.

Table 2. Faculty responses to the question: *How important are laboratory experiences to geotechnical engineering classes?* This table includes undergraduate classes

Class	Critically Important	Important	Minimally Important	Not Important
Soil Mechanics or Equivalent	60	3	1	0
Foundation Design	0	5	11	13
Geology	1	1	0	0

Interest in involving undergraduates in research has increased dramatically since the Boyer Commission (1998) released its seminal report. Involvement in research is used to retain undergraduates and entice students to continue their education in engineering by attending graduate school. Nearly 96% of responding professors involve undergraduates in research at least sometimes (Figure 2). These students are often paired with graduate students to assist with more mundane or repetitive tasks. Professors are more likely to use undergraduates if there are existing support programs, which serve to reduce the costs, in terms of both time and money, of hiring the student. This indicates that if universities value this type of activity, adequate support systems must be in place.

The education of civil engineers must be responsive to the needs of the workplace. The Bureau of Labor Statistics (2011) predicts the number of civil engineering jobs will increase by 24% through 2018. It is anticipated that geotechnical engineering will grow at the same rate because geotechnical engineers interact with and provide support for the other civil engineering disciplines. Of notable importance, however, is that the growth in degrees awarded is currently exceeding the projected growth of jobs.

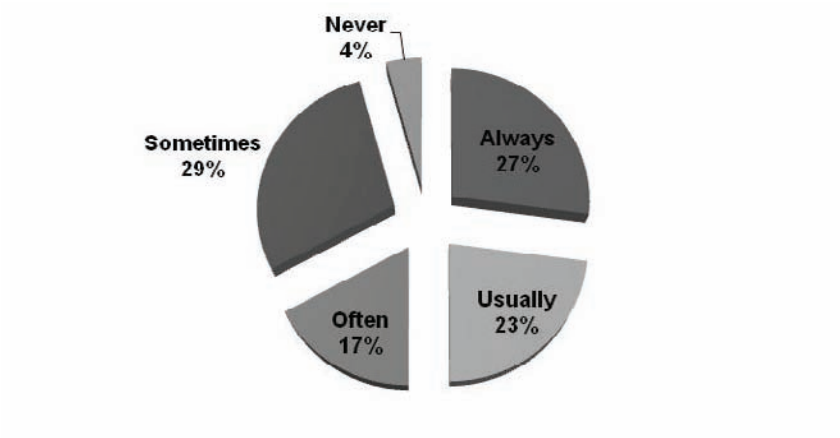

Figure 2. Faculty responses to the question: ***Do you involve undergraduates in your research?***

In recognition of the need for universities to provide employable graduates, ABET, Inc. accreditation criteria requires that programs obtain input from their constituents (ABET 2010). Many programs have created advisory committees comprised of alumni and/or practitioners to provide this input. Despite this format for communication between academia and industry, *CE News* reported that only 36% of practitioners believe that entry-level civil engineers are well prepared for the workforce, compared to 70% of academics (Fauerbach 2010). The informal interviews conducted with practicing engineers by the author, however, did not corroborate this data, as these engineers were satisfied with their more recent hires. As reported in Kunberger, et al. (2011), when asked of their expectations of new hires, some common themes emerged:

- A master's degree or the willingness to obtain a master's degree in a timely manner
- A solid understanding of the fundamentals and the ability to use those fundamentals to obtain practical solutions
- Solid communication skills: written, oral, and graphical
- In a word, a professional. One practitioner summed this up by saying "We don't want technicians, but people who are smart technically, can be active professionally, can build relationships, and advance in our organization."
- Work experience through co-operative education, internships, or summer employment.

GRADUATE

Interest in graduate education is expected to increase in response to market demands and falling undergraduate credit requirements. Of the 224 programs that grant an accredited bachelor's degree in civil engineering, 62% offer graduate-level geotechnical courses that satisfy the requirements for a master's or doctoral degree. Most of these programs offer between five and ten geotechnical courses. It is postulated that many programs rely on adjunct faculty to teach some of their graduate level courses, as the depth, breadth, and number of course offerings exceed the number of full-time geotechnical faculty. Graduate level courses are reflective of faculty expertise, and, as expected, there is no consistent curriculum offered at every university.

The number of students receiving master's degrees in civil engineering has varied over the past 10 years (Figure 3). A substantial decline was noted from 2006 to 2007, but since the number of master's degrees awarded has been steadily increasing, which is partly attributable to the weak job market since 2009. About 30% of civil engineering master's degrees in 2010 were awarded to women. This percentage is greater than the total number of master's degrees awarded for all engineering disciplines, approximately 23% (Gibbons 2011).

The number of doctoral degrees awarded in civil engineering has declined slightly over the past five years (Figure 4). This trend is similar to the trend demonstrated for engineering overall. In 2010, approximately 26% of doctoral degrees in civil engineering were awarded to women, a percentage that is higher than the national average for all engineering disciplines (23%) (Gibbons 2011).

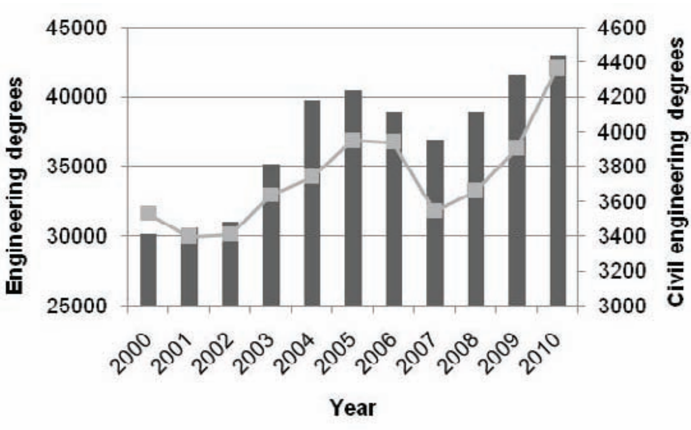

Figure 3. Number of master's degrees awarded yearly; all engineering degrees (bars) on the primary y-axis and civil engineering degrees (line) on the secondary y-axis (Gibbons 2011)

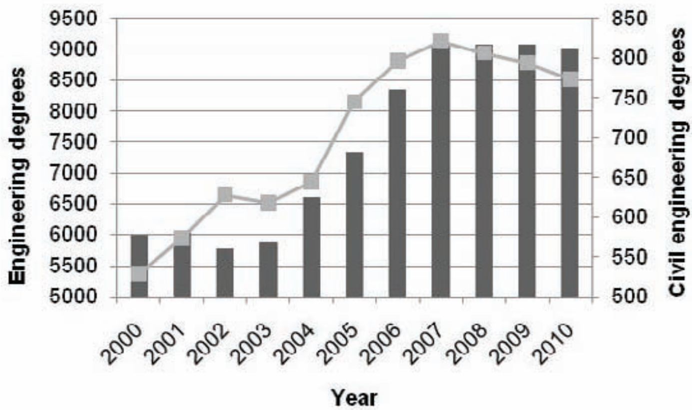

Figure 4. Number of doctoral degrees awarded yearly; all engineering degrees (bars) on the primary y-axis and civil engineering degrees (line) on the secondary y-axis (Gibbons 2011).

The USUCGER survey revealed that nearly 81% of professors are able or are usually able to find students qualified for graduate study. Although most professors are satisfied with the quality of their students, they identified several successes and challenges in recruiting graduate students for geotechnical study. Several respondents stated the importance of engaging students in undergraduate research as a means to recruit talent to geotechnical engineering – even if these students went to another institution for graduate study (Figure 2). The major challenge is finding domestic students that are able to manage the non-prescriptive and open-ended nature of graduate study. The number of foreign national students increases with the level of the degree and, in 2009, about 44% of master's-level graduates in engineering were foreign nationals (Gibbons 2011).

Most geotechnical engineering graduate-level courses are taught in a traditional setting. Only 18.5% of survey respondents stated that they teach distance education courses and nearly all (17.1%) were at the graduate-level. Most comments regarding distance learning were negative; a loss of personal interaction with the students and an increase in time spent delivering and grading exams were described as the major disadvantages to distance education. Likely, this resistance to distance education is also related to the importance geotechnical engineering professors place on laboratory work; professors stated that laboratories are critically important to important to 66% of the graduate courses they teach.

CONTINUING EDUCATION

Geotechnical engineers continue to learn long after their formal education has ended, whether in formal or informal venues. In recent years, formal continuing education opportunities have increased dramatically; including, webinars, seminars, professional organization meetings, workshops, and conferences. ASCE and the Geo-Institute are especially active in creating products to help professional engineers fulfill their continuing education requirements. In 2010 the Geo-Institute formed the Continuing Education Committee to spearhead these efforts.

Much of the growth in continuing education opportunities can be attributed to more stringent continuing education requirements for professional engineers. At the time of writing, 40 of 50 states, plus the District of Columbia require professional development hours (PDHs) for a professional engineer to maintain licensure. Most states require between 12 and 15 PDHs per year. Some states require a certain number of PDHs be completed each year, while others provide a two year time frame. New York State requires the most PDHs with 36 hours biannually while Florida requires only 8 PDHs biannually. The regulations on what constitutes a PDH vary widely from state to state. Some examples of a qualified PDH are: completion of graduate-level, continuing-education, short, and webinar courses; attendance at professional meetings and conferences; teaching qualified courses or presenting at professional meetings; authoring a published paper, article, or book; or obtaining a patent.

Although professors participate in many continuing education products designed for practitioners, there are several courses that are designed specifically to "teach the teachers." Two courses that focus on teaching are *ExCEEd (Excellence in Civil Engineering Education)* and *How to Engineer Engineering Education*. *ExCEEd*, which is sponsored by ASCE, is a one-week intensive workshop for civil engineering professors (Estes, et al. 2008). This workshop, which started in 1998, is typically held in two different locations in the United States each July. *ExCEEd* is unique to civil engineering and demonstrates the support that ASCE has for ensuring that future civil engineers are well educated, by improving the quality of their instruction. *How to Engineer Engineering Educatio*n is a three-day workshop held annually at Bucknell University for engineering and science faculty.

There are also two long-running short courses specifically designed for geotechnical engineering professors: the *ADSC Faculty Workshop*, which is held every eight years, and the *Professors' Driven Pile Institute*, which is held biannually (Caliendo, et al. 2009). These courses demonstrate a commitment on behalf of educators and industry to provide students a current, practical education. Furthermore, these types of courses are increasingly important as more professors enter academia immediately after obtaining their doctorate.

FACULTY

University websites were analyzed to obtain information on the number and gender of geotechnical engineering faculty at universities with accredited civil engineering programs. This analysis revealed that about 11% (397/3,652) of civil engineering faculty are geotechnical engineers. About 17% of all civil engineering programs have no geotechnical engineering faculty, and 75% have two or fewer (Figure 5). The mode is one geotechnical engineering faculty member per institution. Geotechnical faculty members were identified by the courses they taught or by examining the publications and research interests cited on their websites. Approximately 11% (42/397) of geotechnical faculty members are women. A majority of civil engineering departments (184) have no women faculty members, 131 have one, four have two, and one department had three. These results are similar to those reported by Laefer and McHale (2010) for 50 of the most research-active universities in the US.

Professional registration is important to geotechnical engineering educators. The results of the USUCGER survey revealed that 81% are registered professional engineers and 13% plan on obtaining their license soon. Only 6% do not have their license and do not intend to obtain one. One respondent noted that their institution requires all engineering faculty to become professionally licensed.

More in-depth information on funding trends, personal backgrounds, educational training, professional ranking, and productivity for geotechnical engineering faculty at 50 of the most research-active universities in the US can be found in Laefer, et al. (2011) and Laefer and McHale (2010).

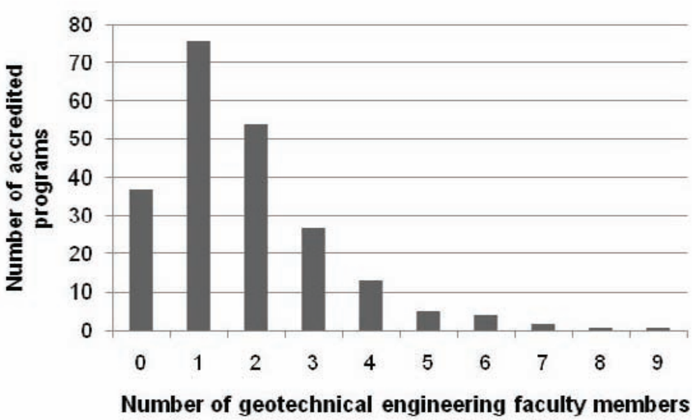

Figure 5. Distribution of geotechnical engineering faculty members at accredited civil engineering programs

THE INFLUENCE OF ASCE

ASCE yields an enormous influence on the education of civil engineers. ASCE is instrumental in providing continuing education opportunities (including *ExCEEd*), providing webinars, publishing journals and books, and hosting conferences and seminars.

The most important way ASCE has influenced education over the past decade was the adoption of Policy Statement (PS) 465. PS 465 describes the preparation that will be required for future civil engineers to attain licensure. This policy recommends that an engineer obtain an additional 30 coordinated credits beyond the bachelor's degree, along with progressive engineering experience. According to ASCE's website, the Board of Direction created the Committee on Academic Prerequisites for Licensure and Professional Practice in 1998 to "develop, organize, and execute a detailed plan for full realization of Policy Statement 465." In 2007, PS 465 was adopted by the Board of Direction (ASCE 2011). Some key components of PS 465 are the concept of a Body of Knowledge (BOK) and the additional educational requirement of a master's degree or 30 coordinated credits. Much has been written about PS 465 (e.g., Russell, et al. 2009 and 2008; and Russell 2003) and the topic often leads to heated debate. Once this policy was adopted, the BOK had to be developed.

According to the BOK Committee of the Committee on Academic Prerequisites for Professional Practice (BOK CAP^3 2008), *The Body of Knowledge*, 2nd edition, describes:

- the knowledge, skills, and attitudes needed to enter into professional practice;
- how to fulfill the BOK; and
- guidance for faculty, students, interns, and practitioners.

The adoption of PS 465 and the creation of the BOK necessitated changes to ABET's program criteria for accreditation. Program criteria are written by the lead professional organization for each degree accredited by ABET; these statements place additional requirements on each degree program beyond the general ABET criteria (specifically, Criterion 3 a-k). Although the current ASCE program criteria is more reflective of the 1st edition of the BOK released in 2004, it provides the link between what is required of undergraduate civil engineering programs and PS 465 (ABET 2010).

As both PS 465 and the BOK are written for all civil engineers, and ABET Criterion 3 a-k are written for all engineers, the key question for geotechnical engineers is 'how can geotechnical engineering classes help to fulfill accreditation requirements?' From a curricular standpoint, there are several ways. The program criteria require graduates to apply knowledge in four technical areas. This requirement relates to the outcome in the BOK (BOK CAP^3 2008) for breadth in civil engineering areas in which geotechnical engineering is identified as one of seven traditional technical areas. Geotechnical engineering also plays a major role in fulfilling the requirement for experimentation. In addition, the high licensure rate reported by those responding to the USUCGER survey will help departments fulfill the faculty requirements of the program criteria.

Nearly 86% of the respondents to the USUCGER survey support PS 465 with no or with few reservations (Figure 6). Several respondents commented that the additional credits are necessary to replace the credits that have been removed from most undergraduate programs over the past 30 years. In addition, several commented that a master's degree is already the *de facto* entry degree for geotechnical engineering, which limits the effect that this policy will have on those pursuing a career in geotechnical engineering. Some professors voiced concern about what will be allowed to comprise the "30 coordinated credits."

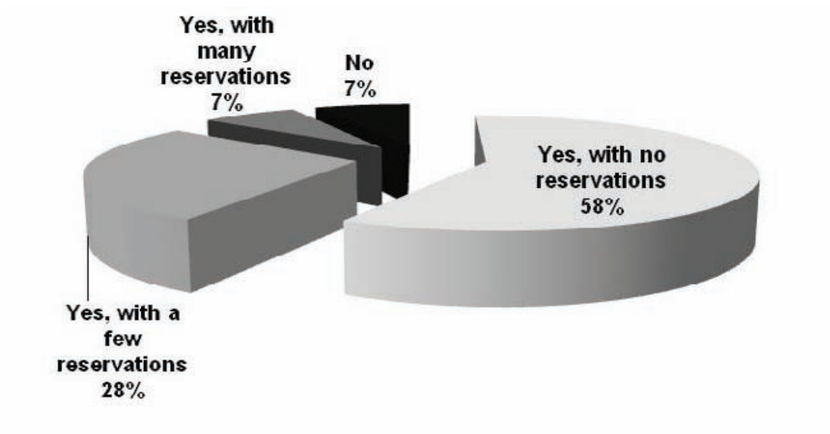

Figure 6. Faculty responses to the question: *Do you support ASCE Policy Statement 465, which recommends that an engineer obtain an additional 30 coordinated credits beyond the BS, along with progressive engineering experience?*

USUCGER members generally support the BOK, with nearly 80% answering either yes, they believe it is a useful representation of what every engineer should know or yes, but there are some flaws (Figure 7). Although the document has support amongst geotechnical engineering professors, nearly 16% did not know what the BOK was. Several respondents commented there is too much emphasis on professional (soft) skills and not enough emphasis on technical skills at the undergraduate level. Furthermore, several commented that employers have a large responsibility in teaching young engineers professional skills.

A positive outcome of the adoption of the ABET program criteria, the ASCE discipline-specific criteria, and the BOK is that it has created a culture of assessment. Engineers should applaud efforts to create a systematic, fact-based approach to determining whether we have achieved our goals as educators. As geotechnical engineers, we should be finding ways to make our classes indispensable to the effort (Dewoolkar, et al. 2009).

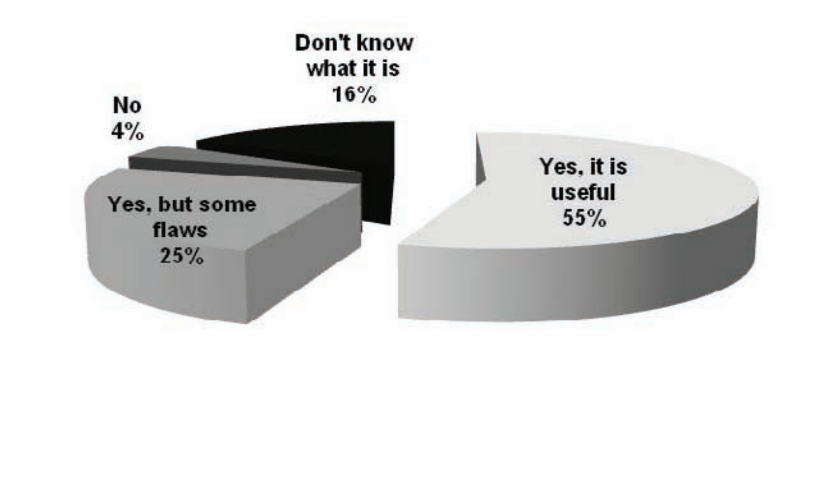

Figure 7. Faculty responses to the question: *Do you support ASCE's Body of Knowledge?*

CHALLENGES AND CHANGES

The challenges and changes identified by the faculty responding to the USUCGER survey are summarized below:

- Falling credit-hour requirements for the attainment of a bachelor's degree
- De-emphasizing technical content and placing greater emphasis on professional skills
- Increasing importance of a master's degree
- Decreasing resources to support laboratories and hands-on instruction
- Attracting quality students and improving the image of geotechnical engineering
- Incorporating topics like unsaturated soil mechanics, Load and Resistance Factor Design (LRFD), critical state soil mechanics, fundamental physical/mathematical concepts, risk assessment, etc.
- Losing practice-oriented faculty and the need for productive partnerships between academics and practitioners

The national trend of falling credit requirements for the attainment of a bachelor's degree was cited by many as a challenge facing geotechnical engineering education. On average, in 1920, 151 credit hours were required to obtain a BSCE (or equivalent), by 2005, this number had dropped to 130 credit hours and has stayed at 130 for the past five years (Russell and Stouffer 2005 and Fridley 2011). This downward trend, coupled with greater emphasis being placed upon teaching professional skills at the undergraduate level, has placed tremendous pressure on civil engineering departments to cut credits and technical content. The result of these trends is that most students have only one required undergraduate course in soil mechanics; geology and foundation design are often electives. In light of these facts, it is no surprise that a master's degree is required by employers. Recently, ABET removed its prohibition on dual-level accreditation (previously a program could not have both an accredited bachelor's and master's program). Given the significance of a master's degree to entering the geotechnical engineering profession, one would expect to see an increase in students seeking advanced degrees and a push towards accreditation of these degrees.

Many professors responding to the USUCGER survey lamented limited resources to support laboratories and other hands-on experiences. These concerns were especially prominent for those facing increasing enrollments.

Professors report that attracting the best students to geotechnical engineering remains a fundamental challenge. Some tied this concern to frustrations about balancing the need to teach fundamental topics with the desire to incorporate more advanced (and perhaps more interesting) topics into their courses. A concern voiced by one respondent was the lack of a foundation design textbook that incorporates LRFD principles: "we will never get LRFD into design practice if we don't teach it and, of course, we need a text."

A challenge facing faculty themselves is the tension between the hands-on practical nature of geotechnical engineering and the need/desire to do research on more arcane topics. Several provided solutions to this challenge such as developing meaningful partnerships between academics and practitioners, elevating the status of applied research, and encouraging young professors to get field experience.

CONCLUSIONS

Lifelong learning has become the norm for geotechnical engineers; thus this paper explored the state of geotechnical engineering education from K-12 outreach efforts through continuing education opportunities. Special emphasis was placed on college-level education and geotechnical engineering faculty members in accredited engineering programs.

Geotechnical engineering education is facing challenges, but also has enormous opportunities to explore. For example, the isolation of geotechnical engineering faculty members presents both a challenge and an opportunity. Eleven percent of civil engineering faculty members are geotechnical engineers and many departments have two or fewer geotechnical engineers on faculty. To maintain graduate programs, many graduate classes appear to be taught by adjuncts, who are, presumably, practitioners. This existing connection between academics and practitioners should be exploited to bridge the gap between the two (the divide between the two was a common challenge cited by respondents to the USUCGER survey) and to allow many of the geotechnical engineering educators working in isolation to form productive partnerships. Another challenge that also presents opportunities are the ABET and

ASCE program-specific accreditation criteria. While the criteria, along with falling credit counts, has reduced the number of required geotechnical engineering classes in most curriculums, these classes are well-suited to being indispensible parts of the assessment process.

Market forces, falling undergraduate credit requirements, and PS 465 have increased the importance of a master's degree to those wishing to practice geotechnical engineering. More than 60% of programs that offer an accredited civil engineering bachelor's degree also offer graduate degrees. Most of these programs are taught face-to-face because of the hands-on nature of geotechnical engineering, as well as the desire to maintain a personal relationship with students.

Every discipline of study will always be faced with challenges in attracting the best and brightest. Geotechnical engineering professors routinely engage undergraduates in research as a means to encourage students to pursue advanced degrees and career opportunities. Most of the students entering graduate school are qualified, although the ability to attract well-qualified domestic students continues.

ACKNOWLEDGEMENTS

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Geo-Seismic Design in Eastern US: State of Practice

Sissy Nikolaou[1], M. ASCE, PhD, PE

[1]Senior Associate, Director of Geo-Seismic Department, Mueser Rutledge Consulting Engineers, 14 Penn Plaza, New York NY 10122; snikolaou@mrce.com

ABSTRACT: Earthquakes in the Eastern United States (US) could affect not only millions of people, but potentially impact the worldwide economy. This region is home to densely populated urban centers, including the nation's capital, Washington, DC, and the financial capital of the world, New York City. Because most of the buildings and infrastructure on the East Coast have not generally been designed to accommodate dynamic lateral loading, the anticipated moderate earthquake hazard places the region at high seismic risk. The practice of geotechnical earthquake engineering has a short history in the area, with seismic codes in effect for less than two decades.

The author will discuss engineering approaches presently followed to solve common geo-seismic problems such as seismic hazard assessment, site response analysis, and liquefaction potential evaluation. Most of these approaches are based on knowledge and developments from the Western US and other seismically active areas of the world. Certain concepts and requirements from these areas may not be applicable in the Eastern US due to its unique local geologic and seismological characteristics. In this case, the geotechnical engineer should pay special attention to identifying parameters that may not be as significant in other regions, such as: (i) the predominantly harder bedrock with shear wave velocities of more than 5,000 feet per second; (ii) the nature and properties of overburden soils; (ii) significant impedance contrast between overburden soils and bedrock; (iv) high frequency content of input rock motions; (v) selection of liquefaction assessment approach; (iv) treatment of uncertainty due to lack of significant historic and recorded strong ground motion data.

Ideas on how the unique geo-seismic design aspects of the Eastern US could be treated in site characterization, analysis approaches, and code development will be presented. Design concepts based on performance- or risk-based philosophy instead of the conventional factor of safety approach will be discussed, particularly for critical applications that must be designed for extreme events that inherently hold larger uncertainties. Finally, the author will present recent findings from the Mineral, Virginia Earthquake of August 2011 and discuss the impacts that this event may have in future practice.

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