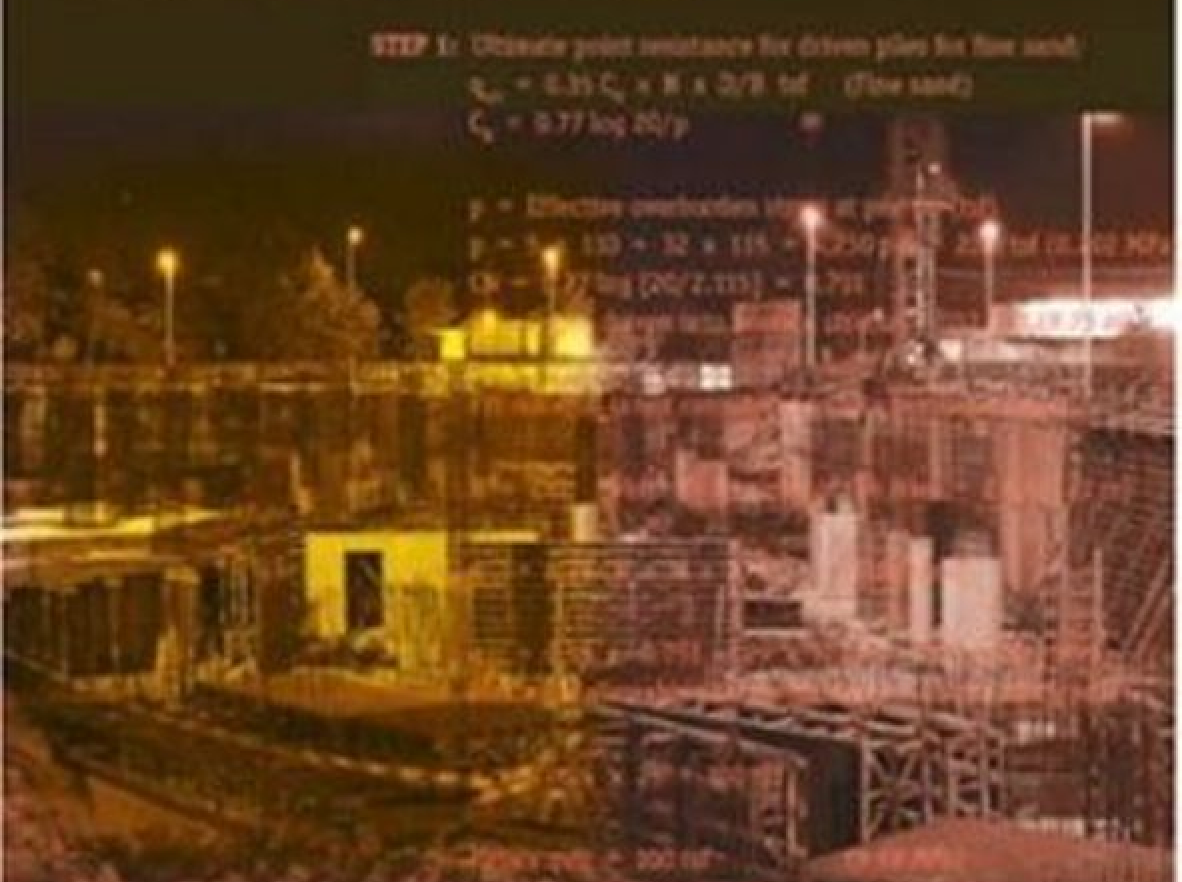
Geotechnical Engineering Calculations and Rules of Thumb



Ruwan Rajapakse

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1

Site Investigation and Soil Conditions

1.1 Introduction

Soils are of interest to many professionals. Soil chemists are interested in the chemical properties of soil. Geologists are interested in the origin and history of soil strata formation. Geotechnical engineers are interested in the strength characteristics of soil.

Soil strength is dependant on the cohesion and friction between soil particles.

1.1.1 Cohesion

Cohesion is developed due to adhesion of clay particles generated by electromagnetic forces. Cohesion is developed in clays and plastic silts. Friction is developed in sands and nonplastic silts. See Fig. 1.1.

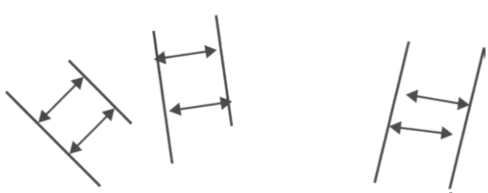

Figure 1.1 Electrochemical bonding of clay particles that gives rise to cohesion

1.1.2 Friction

Sandy particles when brushed against each other will generate friction. Friction is a physical process, whereas cohesion is a chemical process.

Soil strength generated due to friction is represented by friction angle φ. See Fig. 1.2.

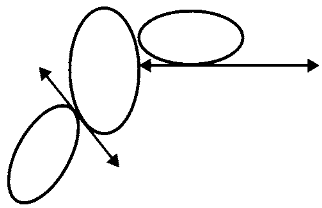

Figure 1.2 Friction in sand particles

Cohesion and friction are the most important parameters that determine the strength of soils.

Measurement of Friction

The friction angle of soil is usually obtained from correlations available with the standard penetration test value known as SPT (N). The standard penetration test is conducted by dropping a 140 lb hammer from a distance of 30 in. onto a split spoon sample. The number of blows required to penetrate 1 ft is known as the SPT value. The number of blows will be higher in hard soils and lower in soft clays and loose sand.

Measurement of Cohesion

The cohesion of soil is measured by obtaining a Shelby tube sample and conducting a laboratory unconfined compression test.

1.2 Origin of a Project

Civil engineering projects are originated when a company or a person requires a new facility. A company wishing to construct a new facility is known as the owner. The owner of the new proposed facility would consult an architectural firm to develop an architectural design. The architectural firm would lay out room locations, conference halls, restrooms, heating and cooling units, and all other necessary elements of a building as specified by the owner.

Subsequent to the architectural design, structural engineers and geotechnical engineers would enter the project team. Geotechnical engineers would develop the foundation elements while structural engineers would design the structure of the building. The loading on columns will usually be provided by structural engineers. See Fig. 1.3.

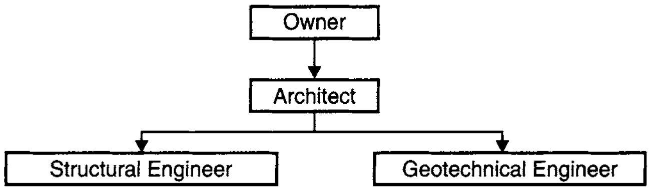

Figure 1.3 Relationship between owner and other professionals

1.3 Geotechnical Investigation Procedure

After receiving information regarding the project, the geotechnical engineer should gather the necessary information for the foundation design. Usually, the geotechnical engineer would start with a literature survey.

After conducting a literature survey, he or she would make a field visit followed by the subsurface investigation program. See Fig. 1.4.

1.4 Literature Survey

The geotechnical engineer's first step is to conduct a literature survey. There are many sources available to obtain information regarding topography, subsurface soil conditions, geologic formations, and groundwater conditions. Sources for the literature survey are local libraries, the Internet, local universities, and national agencies. National agencies usually conduct geological surveys, hydro-geological surveys, and topographic surveys. These surveys usually provide very important information to the geotechnical engineer. Topographic surveys will be useful in identifying depressed regions, streams, marshlands, man-made fill areas, organic soils, and roads. Depressed regions may indicate weak bedrock or settling soil conditions. Construction in marsh areas will be costly. Roads, streams, utilities, and man-made

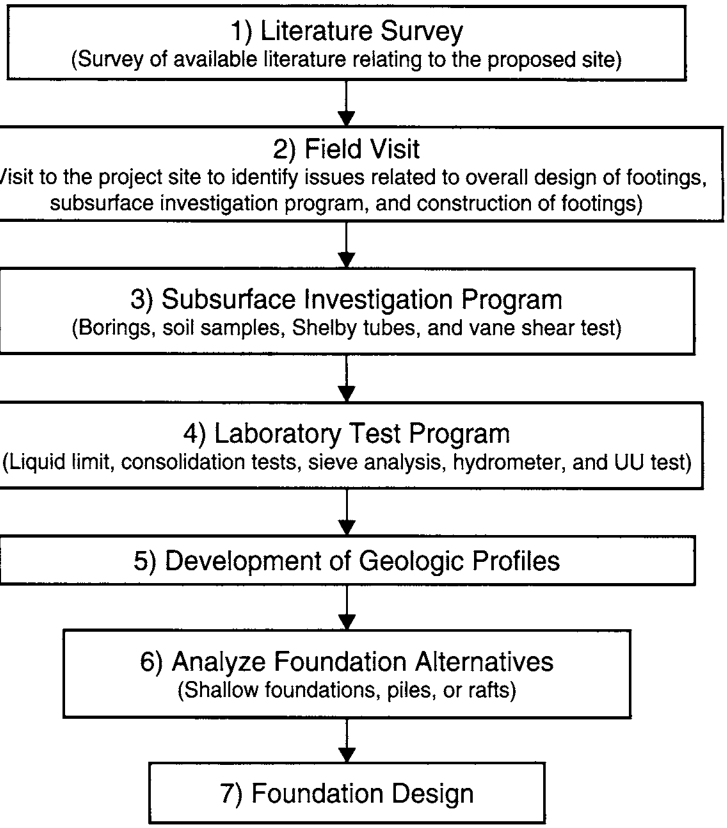

Figure 1.4 Site investigation program

fills will interfere with the subsurface investigation program or the construction. See Fig. 1.5.

1.4.1 Adjacent Property Owners

If there are buildings in the vicinity of the proposed building, the geotechnical engineer may be able to obtain site investigation studies conducted in the past.

1.4.2 Aerial Surveys

Aerial surveys are done by various organizations for city planning, utility design and construction, traffic management, and disaster

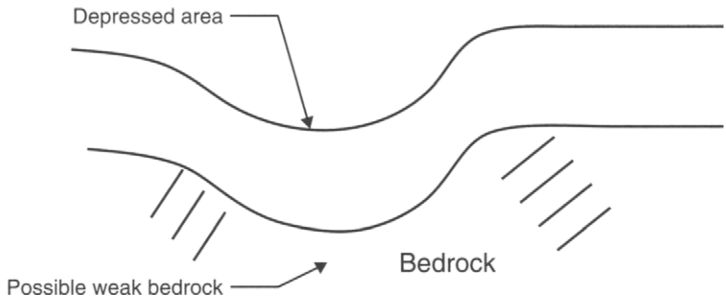

Figure 1.5 Depressed area

management studies. Geotechnical engineers should contact the relevant authorities to investigate whether they have any aerial maps in the vicinity of the proposed project site.

Aerial maps can be a very good source of preliminary information for the geotechnical engineer. Aerial surveys are expensive to conduct and only large-scale projects may have the budget for aerial photography. See Fig. 1.6.

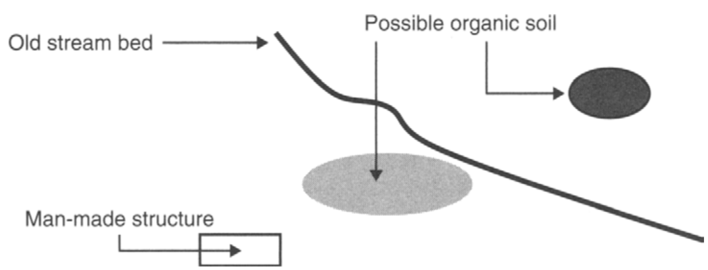

Figure 1.6 Important items in an aerial photograph

Dark patches may indicate organic soil conditions, a different type of soil, contaminated soil, low drainage areas, fill areas, or any other oddity that needs the attention of the geotechnical engineer. Darker than usual lines may indicate old streambeds or drainage paths or fill areas for utilities. Such abnormalities can be easily identified from an aerial survey map. See Fig. 1.7.

Figure 1.7 Aerial photograph

1.5 Field Visit

After conducting a literature survey, the geotechnical engineer should make a field visit. Field visits would provide information regarding surface topography, unsuitable areas, slopes, hillocks, nearby streams, soft ground, fill areas, potentially contaminated locations, existing utilities, and possible obstructions for site investigation activities. The geotechnical engineer should bring a hand augur such as the one in Fig. 1.9 to the site so that he or she may observe the soil a few feet below the surface.

Nearby streams could provide excellent information regarding the depth to groundwater. See Fig. 1.8.

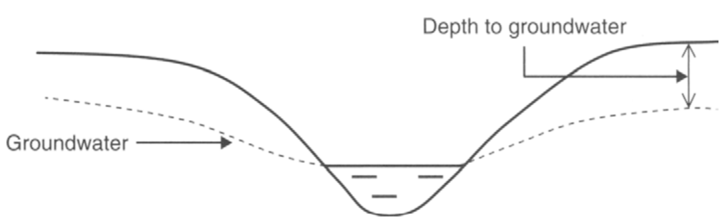

Figure 1.8 Groundwater level near a stream

1.5.1 Hand Auguring

Hand augurs can be used to obtain soil samples to a depth of approximately 6 ft depending upon the soil conditions. Downward pressure (P) and a torque (T) are applied to the hand auger. Due to the torque and the downward pressure, the hand auger penetrates into the ground. The process stops when human strength is not sufficient to generate enough torque or pressure. See Fig. 1.9.

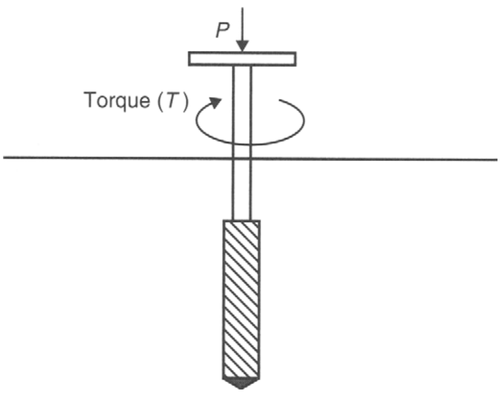

Figure 1.9 Hand auger

1.5.2 Sloping Ground

Steep slopes in a site escalate the cost of construction because compacted fill is required. Such areas need to be noted for further investigation. See Fig. 1.10.

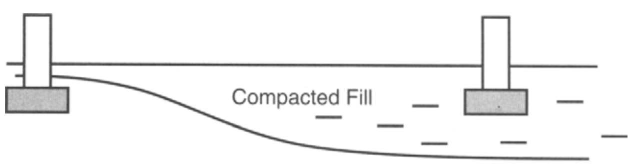

Figure 1.10 Sloping ground

1.5.3 Nearby Structures

Nearby structures could pose many problems for proposed projects. It is always good to identify these issues at the very beginning of a project.

The distance to nearby buildings, schools, hospitals, and apartment complexes should be noted. Pile driving may not be feasible if there is a hospital or school close to the proposed site. In such situations, jacking of piles can be used to avoid noise.

If the proposed building has a basement, underpinning of nearby buildings may be necessary. See Fig. 1.11.

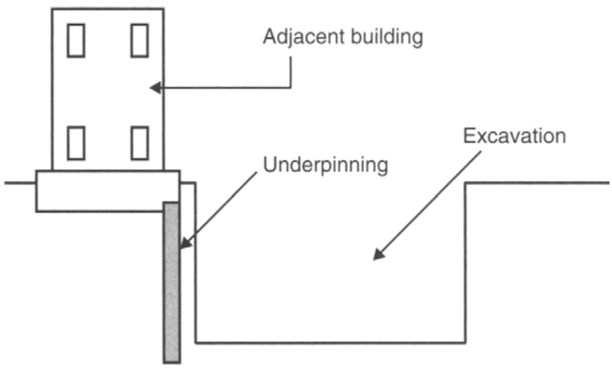

Figure 1.11 Underpinning of nearby building for an excavation

Other methods, such as secant pile walls or heavy bracing, can also be used to stabilize the excavation.

The foundation of a new building can have an effect on nearby existing buildings. If compressible soil is present in a site, the new building may induce consolidation. Consolidation of a clay layer may generate negative skin friction in piles of nearby structures. See Fig. 1.12.

1.5.4 Contaminated Soils

Soil contamination is a very common problem in many urban sites. Contaminated soil will increase the cost of a project or in some cases could even kill a project entirely. Identifying contaminated soil areas at an early stage of a project is desirable.

1.5.5 Underground Utilities

It is a common occurrence for drilling crews to accidentally puncture underground power cables or gas lines. Early identification of existing

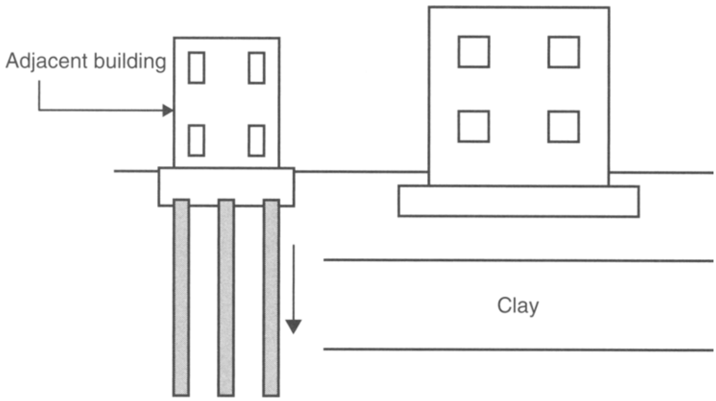

Figure 1.12 Negative skin friction in piles due to new construction

utilities is important. Electrical poles, electrical manhole locations, gas lines, and water lines should be noted so that drilling can be done without breaking any utilities.

Existing utilities may have to be relocated or left undisturbed during construction.

Existing manholes may indicate drainpipe locations. See Fig. 1.13.

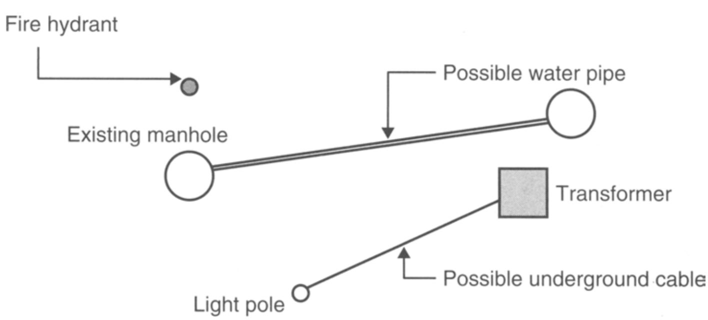

Figure 1.13 Observation of utilities

1.5.6 Overhead Power Lines

Drill rigs need to keep a safe distance from overhead power lines during the site investigation phase. Overhead power line locations need to be noted during the field visit.

1.5.7 Man-Made Fill Areas

Most urban sites are affected by human activities. Man-made fills may contain soils, bricks, and various types of debris. It is possible to compact some man-made fills so that they could be used for foundations. This is not feasible when the fill material contains compressible soils, tires, or rubber. Fill areas need to be further investigated during the subsurface investigation phase of the project.

1.5.8 Field Visit Checklist

The geotechnical engineer needs to pay attention to the following issues during the field visit.

1. Overall design of foundations.
2. Obstructions for the boring program (overhead power lines, marsh areas, slopes, and poor access may create obstacles for drill rigs).
3. Issues relating to construction of foundations (high groundwater table, access, and existing utilities).
4. Identification of possible man-made fill areas.
5. Nearby structures (hospitals, schools, courthouses, etc.).

Table 1.1 gives a geotechnical engineer's checklist for the field visit.

1.6 Subsurface Investigation Phase

The soil strength characteristics of the subsurface are obtained through a drilling program. In a nutshell, the geotechnical engineer needs the following information for foundation design work.

- Soil strata identification (sand, clay, silt, etc.).
- Depth and thickness of soil strata.
- Cohesion and friction angle (two parameters responsible for soil strength).
- Depth to groundwater.

Table 1.1 Field visit checklist

Item	Impact on site investigation	Impact on construction	Cost impact
Sloping ground	May create difficulties for drill rigs	Impact on maneuverability of construction equipment	Cost impact due to cut and fill activities
Small hills	Same as above	Same as above	Same as above
Nearby streams	Groundwater monitoring wells may be necessary	High groundwater may impact deep excavations (pumping)	Impact on cost due to pumping activities
Overhead power lines	Drill rigs have to stay away from overhead power lines	Construction equipment have to keep a safe distance	Possible impact on cost
Underground utilities (existing)	Drilling near utilities should be done with caution	Impact on construction work	Relocation of utilities will impact the cost
Areas with soft soils	More attention should be paid to these areas during subsurface investigation phase	Possible impact	Possible impact on cost
Contaminated soil	Extent of contamination need to be identified	Severe impact on construction activities	Severe impact on cost
Man made fill areas	Broken concrete or wood may pose problems during the boring program	Unknown fill material generally not suitable for construction work.	Possible impact on cost
Nearby structures		Due to nearby hospitals and schools some construction methods may not be feasible such as pile driving Excavations for the proposed building could cause problems to existing structures Shallow foundations of new structures may induce negative skin friction in piles in nearby buildings	Possible impact on cost

1.6.1 Soil Strata Identification

Subsurface soil strata information is obtained using drilling. The most common drilling techniques are

- Augering.
- Mud rotary drilling.

Augering

In the case of augering, the ground is penetrated using augers attached to a rig. The rig applies torque and downward pressure to the augers. Machine augers use the same principle as in hand augers for penetration into the ground. See Figs. 1.14 and 1.15.

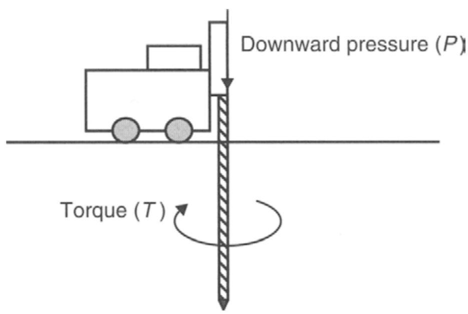

Figure 1.14 Augering

Mud Rotary Drilling

In the case of mud rotary drilling, a drill bit known as a roller bit is used for the penetration. Water is used to keep the roller bit cool so that it will not overheat and stop functioning. Usually drillers mix bentonite slurry (also known as drilling mud) to the water to thicken the water. The main purpose of the bentonite slurry is to keep the sidewalls from collapsing. See Fig. 1.16.

Drilling mud goes through the rod and the roller bit and comes out from the bottom. It removes the cuttings from the working area. The mud is captured in a basin and recirculated. See Fig. 1.17.

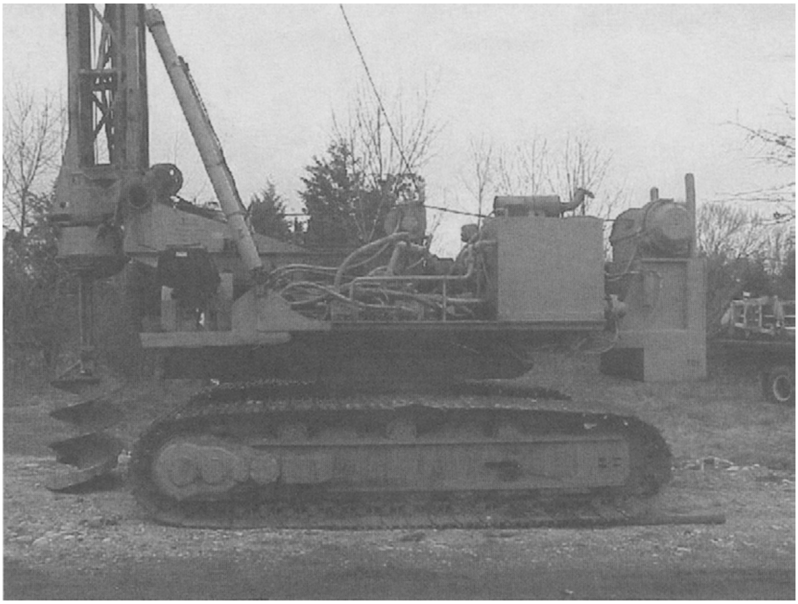

Figure 1.15 Auger drill rig (*Source*: http://www.precisiondrill.com)

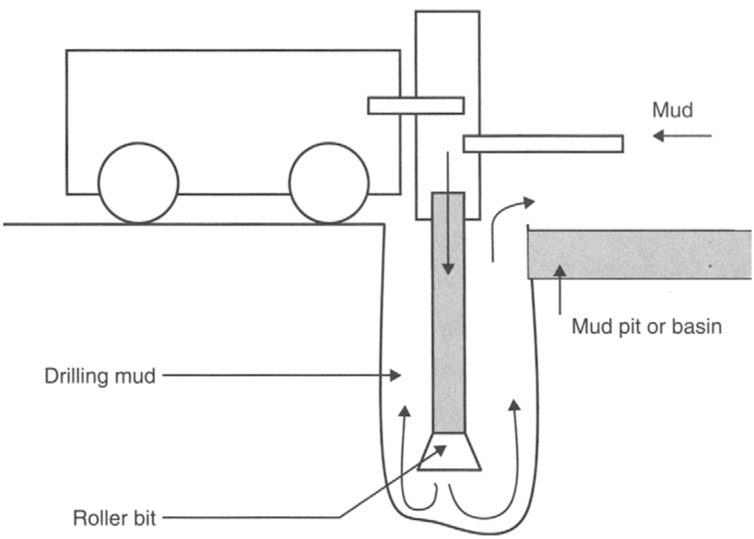

Figure 1.16 Mud rotary drilling

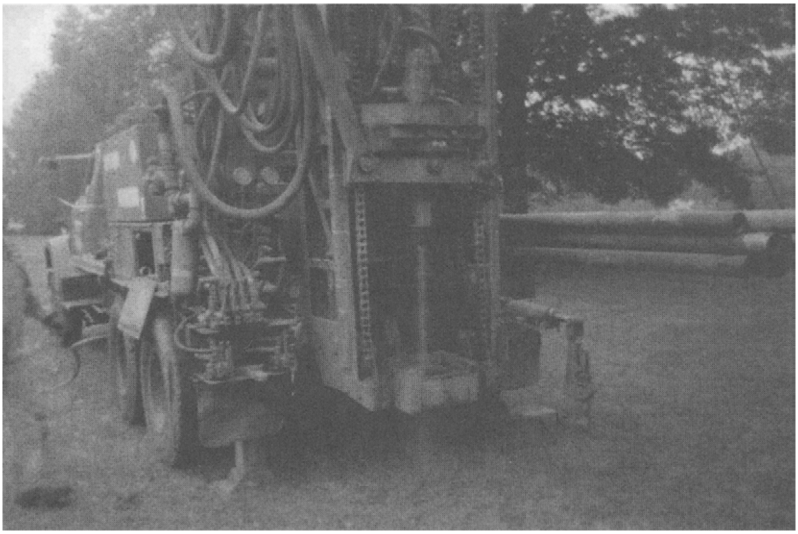

Figure 1.17 Mud rotary drill rig

Boring Program

The number of borings that need to be constructed may sometimes be regulated by local codes. For example, New York City building code requires one boring per 2,500 sq ft. It is important to conduct borings as close as possible to column locations and strip footing locations. In some cases this may not be feasible.

Typically borings are constructed 10 ft below the bottom level of the foundation.

Test Pits

In some situations, test pits would be more advantageous than borings. Test pits can provide information down to 15 ft below the surface. Unlike borings, soil can be visually observed from the sides of the test pit.

Soil Sampling

Split spoon samples are obtained during boring construction. Split spoon samples typically have a 2 in. diameter and have a length of 2 ft. Soil samples obtained from split spoons are adequate to conduct sieve analysis, soil identification, and Atterberg limit tests. Consolidation tests, triaxial tests, and unconfined compressive strength tests need a large quantity of soil. In such situations, Shelby tubes are used. Shelby tubes have a larger diameter than split spoon samples. See Figs. 1.18 through 1.20.

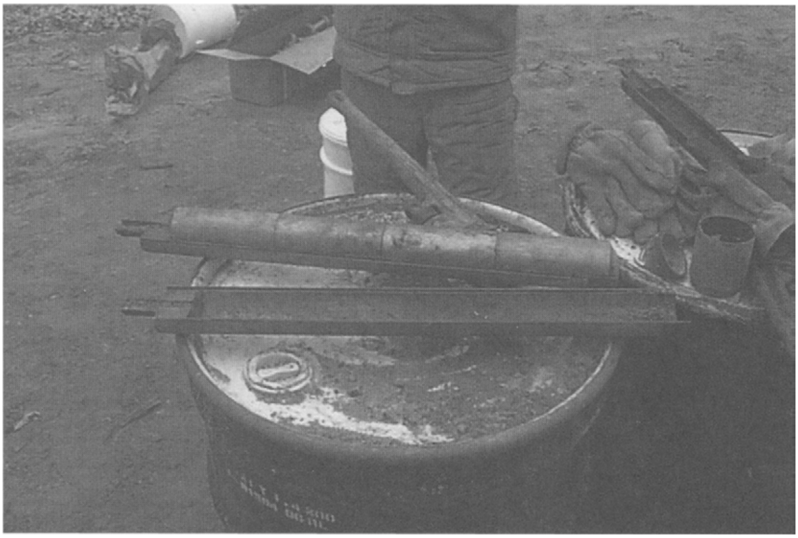

Figure 1.18 Split spoon sampling

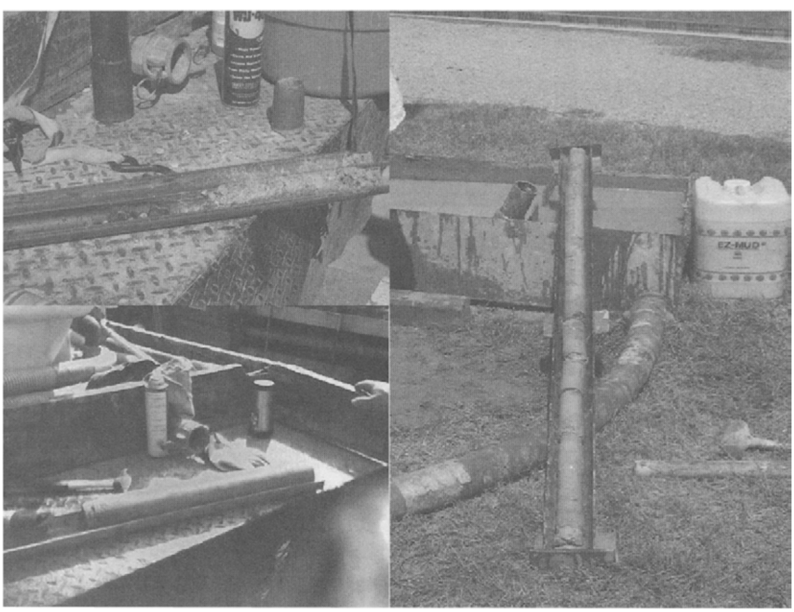

Figure 1.19 Split spoon sampling procedure

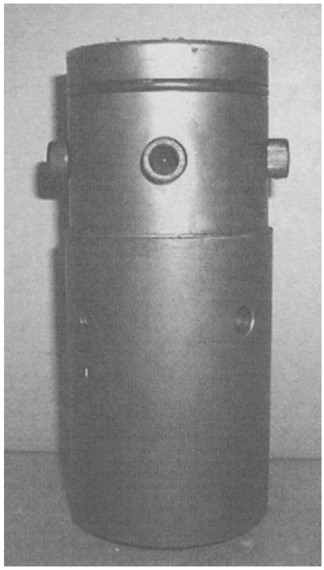

Figure 1.20 Shelby tube (*Source*: Diedrich Drilling)

Hand Digging Prior to Drilling

Damage to utilities should be avoided during the boring program. Most utilities are rarely deeper than 6 ft. Hand digging the first 6 ft prior to drilling boreholes is found to be an effective way to avoid damaging utilities. During excavation activities, the backhoe operator is advised to be aware of utilities. The operator should check for fill materials, since in many instances utilities are backfilled with select fill material. It is advisable to be cautious since there could be situations where utilities are buried with the same surrounding soil. In such cases it is a good idea to have a second person present exclusively to watch the backhoe operation.

1.7 Geotechnical Field Tests

1.7.1 SPT(*N*) Value

During the construction of borings, the SPT (*N*) values of soils are obtained. The SPT (*N*) value provides information regarding the soil strength. The SPT (N) value in sandy soils indicates the friction angle in sandy soils and in clay soils indicates the stiffness of the clay stratum.

1.7.2 Pocket Penetrometer

Pocket penetrometers can be used to obtain the stiffness of clay samples. The pocket penetrometer is pressed into the soil sample and the reading is recorded. The reading would indicate the cohesion of the clay sample. See Figs. 1.21 and 1.22.

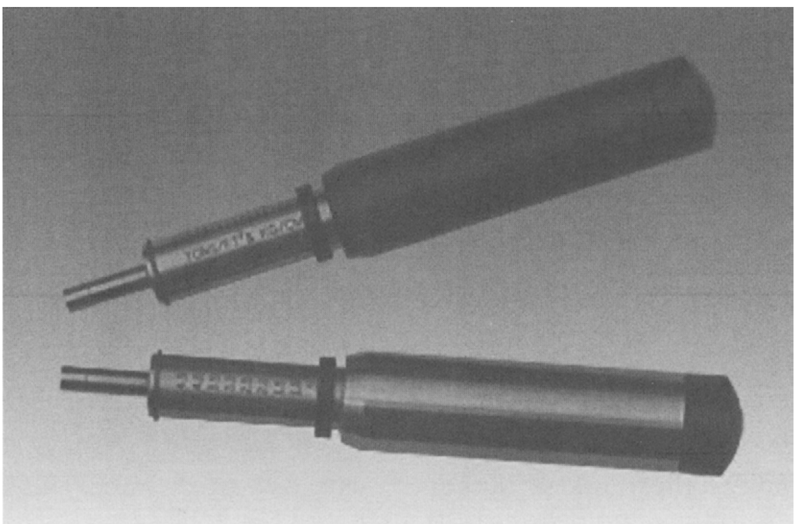

Figure 1.21 Pocket penetrometers

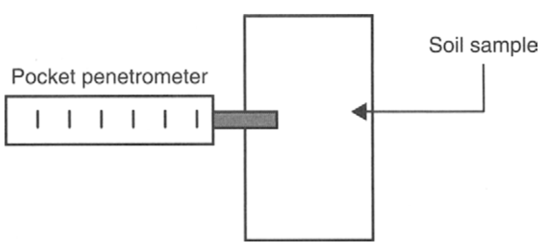

Figure 1.22 Pocket penetrometer

1.7.3 Vane Shear Test

Vane shear tests are conducted to obtain the cohesion(c) value of a clay layer. An apparatus consisting of vanes are inserted into the clay layer and rotated. The torques of the vane is measured during the process. Soils with high cohesion values register high torques. See Fig. 1.23.

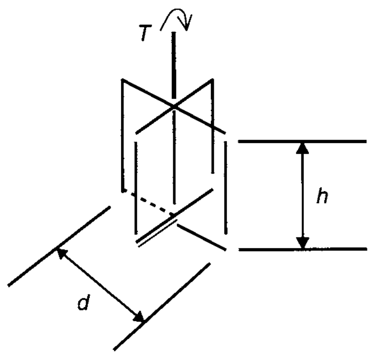

Figure 1.23 Vane shear apparatus

Vane shear test procedure:

- A drill hole is made with a regular drill rig.
- The vane shear apparatus is inserted into the clay
- The vane is rotated and the torque is measured.
- The torque gradually increases and reaches a maximum. The maximum torque achieved is recorded.
- At failure, the torque reduces and reaches a constant value. This value refers to the remolded shear strength. See Fig. 1.24.

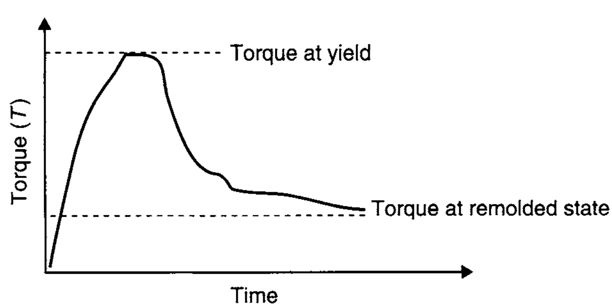

Figure 1.24 Torque vs. time curve

The cohesion of clay is given by

$$T = c \times \pi \times (d^2h/2 + d^3/6)$$

where

T = torque (measured)
c = cohesion of the clay layer
d = width of vanes
h = height of vanes

The cohesion of the clay layer is obtained by using the maximum torque. The remolded cohesion was obtained by using the torque at failure.

1.8 Correlation Between Friction Angle (φ) and SPT (N) Value

The friction angle (φ) is a very important parameter in geotechnical engineering. Soil strength in sandy soils solely depends on friction.

Correlations have been developed between the SPT (N) value and the friction angle. See Table 1.2.

1.8.1 Hatakanda and Uchida Equation

After conducting numerous tests, Hatakanda and Uchida (1996) derived the following equation to compute the friction angle using the SPT (N) values.

$$\varphi = 3.5 \times (N)^{1/2} + 22.3$$

where

φ = friction angle
N = SPT value

This equation ignores particle size. Most tests are done on medium to coarse sands. For a given N value, fine sands will have a lower friction

Table 1.2 Friction angle, SPT (N) values and relative density

Soil type	SPT (N_{70} value)	Consistency	Friction angle (φ)	Relative density (D_r)
Fine sand				
	1–2	Very loose	26–28	0–0.15
	3–6	Loose	28–30	0.15–0.35
	7–15	Medium	30–33	0.35–0.65
	16–30	Dense	33–38	0.65–0.85
	<30	Very dense	<38	<0.85
Medium sand				
	2–3	Very loose	27–30	0–0.15
	4–7	Loose	30–32	0.15–0.35
	8–20	Medium	32–36	0.35–0.65
	21–40	Dense	36–42	0.65–0.85
	<40	Very dense	<42	<0.85
Coarse sand				
	3–5	Very loose	28–30	0–0.15
	6–9	Loose	30–33	0.15–0.35
	10–25	Medium	33–40	0.35–0.65
	26–45	Dense	40–50	0.65–0.85
	<45	Very dense	<50	<0.85

Source: Bowles (2004).

angle while coarse sands will have a larger friction angle. Hence the following modified equations are proposed.

$$\text{fine sand } \varphi = 3.5 \times (N)^{1/2} + 20$$
$$\text{medium sand } \varphi = 3.5 \times (N)^{1/2} + 21$$
$$\text{coarse sand } \varphi = 3.5 \times (N)^{1/2} + 22$$

Design Example 1.1

Find the friction angle of fine sand with an SPT (N) value of 15.

Solution

Use the modified Hatakanda and Uchida equation for fine sand.

$$\begin{aligned}\varphi &= 3.5 \times (N)^{1/2} + 20\\ &= 3.5 \times (15)^{1/2} + 20\\ &= 33.5\end{aligned}$$

1.8.2 SPT (*N*) Value vs. Total Density

Correlations between the SPT (*N*) value and total density have been developed. These values are given in Table 1.3.

Table 1.3 SPT (*N*) value and soil consistency

Soil type	SPT (N_{70} value)	Consistency	Total density
Fine sand			
	1–2	Very loose	70–90 pcf (11–14 kN/m^3)
	3–6	Loose	90–110 pcf (14–17 kN/m^3)
	7–15	Medium	110–130 pcf (17–20 kN/m^3)
	16–30	Dense	130–140 pcf (20–22 kN/m^3)
	<30	Very dense	<140 pcf (<22 kN/m^3)
Medium sand			
	2–3	Very loose	70–90 pcf (11–14 kN/m^3)
	4–7	Loose	90–110 pcf (14–17 kN/m^3)
	8–20	Medium	110–130 pcf (17–20 kN/m^3)
	21–40	Dense	130–140 pcf (20–22 kN/m^3)
	<40	Very dense	<140 pcf (<22 kN/m^3)
Coarse sand			
	3–6	Very loose	70–90 pcf (11–14 kN/m^3)
	5–9	Loose	90–110 pcf (14–17 kN/m^3)
	10–25	Medium	110–130 pcf (17–20 kN/m^3)
	26–45	Dense	130–140 pcf (20–22 kN/m^3)
	<45	Very dense	<140 pcf (<22 kN/m^3)

1.9 SPT (*N*) Value Computation Based on Drill Rig Efficiency

Most geotechnical correlations were done based on SPT (*N*) values obtained during the 1950s. However, drill rigs today are much more efficient than the drill rigs of the 1950s. If the hammer efficiency is high, then to drive the spoon 1 ft, the hammer would require a lesser number of blows compared to a less efficient hammer.

For an example let us assume one uses an SPT hammer from the 1950s and it required 20 blows to penetrate 12 in. If a modern hammer is used, 12 in. penetration may be achieved with a lesser number of blows. Therefore, a high efficiency hammer requires a smaller number of blows. Conversely, a low efficiency hammer requires a larger number of blows.

It is possible to convert blow count from one hammer to another. The following equation gives the conversion from a 50% efficient hammer to a 70% efficient hammer.

$$N_{70}/N_{50} = 50/70$$

Design Example 1.2

An SPT blow count of 20 was obtained using a 40% efficient hammer. What blow count is expected if a hammer with 60% efficiency is used?

Solution

$$N_{60}/N_{40} = 40/60$$
$$N_{60}/20 = 40/60$$
$$N_{60} = 13.3$$

Design Example 1.3

An SPT blow count of 15 was obtained using a 55% efficient hammer. A second hammer was used and a blow count of 25 was obtained. What was the efficiency of the second hammer?

Solution

Let us say the efficiency of the second hammer is N_x.

$$N_x/N_{55} = 55/X$$
$$N_x = 25 \text{ and } N_{55} = 15$$

Hence

$$55/X = N_x/N_{55} = 25/15 = 1.67$$
$$X = 55/1.67 = 32.9$$

The efficiency of the second hammer is 32.9%.

1.10 SPT-CPT Correlations

In the United States, the Standard Penetration Test (SPT) is used extensively. On the other hand, the Cone Penetration Test (CPT) is popular in Europe. A standard cone has a base area of 10 sq cm and an apex angle of 60°.

European countries have developed many geotechnical engineering correlations using CPT data. Fig. 1.25 shows a standard CPT device.

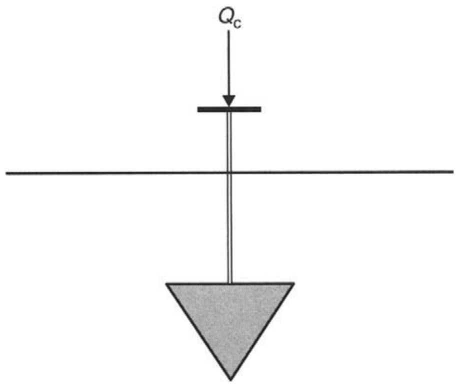

Figure 1.25 Standard CPT device

The correlation between SPT and CPT shown in Table 1.4 can be used to convert CPT values to SPT numbers or vice versa.

$$Q_C = \text{CPT value measured in bars}$$

Table 1.4 SPT-CPT correlations for clays and sands

Soil type	Mean grain size (D_{50}) (measured in mm)	Q_c/N
Clay	0.001	1
Silty clay	0.005	1.7
Clayey silt	0.01	2.1
Sandy silt	0.05	3.0
Silty sand	0.10	4.0
Sand	0.5	5.7
	1.0	7.0

Source: Robertson et al. (1983).

where

1 bar = 100 kPa
N = SPT value
D_{50} = size of the sieve that would pass 50% of the soil

Design Example 1.4

SPT tests were done on a sandy silt with a D_{50} value of 0.05 mm. The average SPT (N) value for this soil is 12. Find the CPT value.

Solution

From Table 1.4, for sandy silt with a D_{50} value of 0.05 mm

$$Q_c/N = 3.0$$
$$N = 12$$

Hence

$$Q_c = 3 \times 12 = 36\,\text{bars} = 3{,}600\,\text{kPa}$$

1.11 Groundwater

A geotechnical engineer needs to investigate groundwater conditions thoroughly. Groundwater conditions in a site are important for a number of reasons. The groundwater level of a site is obtained by constructing groundwater wells.

1.11.1 Dewatering

Dewatering is done when excavation is needed for building foundations, earth retaining structures, and basements.

1.11.2 Landfill Construction

Hydrogeology plays a major role during the design of landfills. Groundwater flow direction, flow rate, and aquifer properties need to be thoroughly studied during the design phase of a landfill.

1.11.3 Seismic Analysis

Groundwater location plays a significant role in liquefaction of sandy soils during earthquakes.

1.11.4 Monitoring Wells

Monitoring wells are installed to obtain the groundwater elevation. Monitoring wells are typically constructed using PVC pipes. See Fig. 1.26.

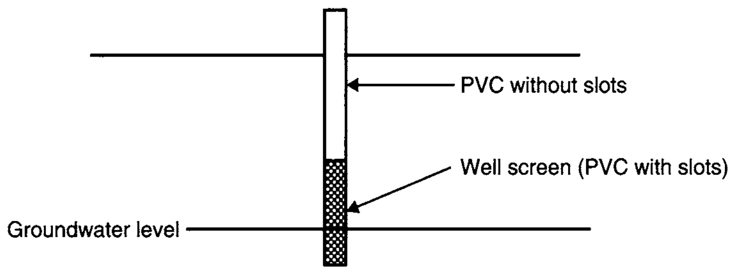

Figure 1.26 Groundwater monitoring well

A slotted section of the PVC is known as the well screen and allows water to flow into the well. If there is no pressure, the water level in the well indicates the groundwater level.

1.11.5 Aquifers with Artesian Pressure

Groundwater in some aquifers can be under pressure. The monitoring wells will register a higher water level than the groundwater level. In some cases, water would spill out from the well due to artesian pressure. See Fig. 1.27. Since the aquifer is under pressure, the water level in the well is higher than the actual water level in the aquifer.

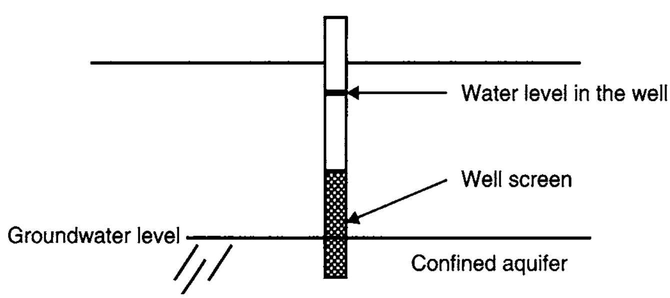

Figure 1.27 Monitoring well in a confined aquifer

In Fig. 1.28., an impermeable clay layer is shown lying above the permeable sand layer. The dotted line shows the groundwater level if the clay layer were absent. Due to the impermeable clay layer, the groundwater cannot reach the level shown by the dotted line. Hence, the groundwater level is confined to point A in the monitoring well. When a well is installed, the water level will rise to point B, higher than the initial water level due to artesian pressure. See Fig. 1.29.

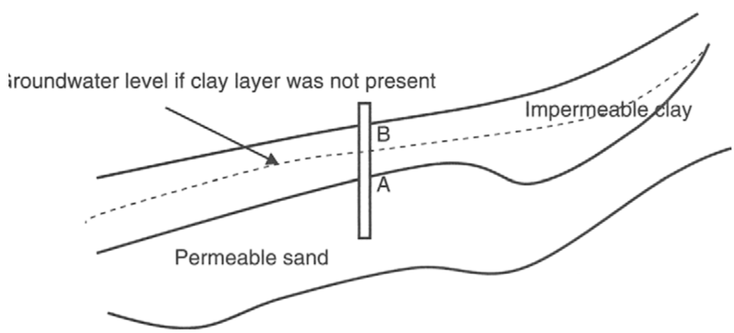

Figure 1.28 Artesian conditions

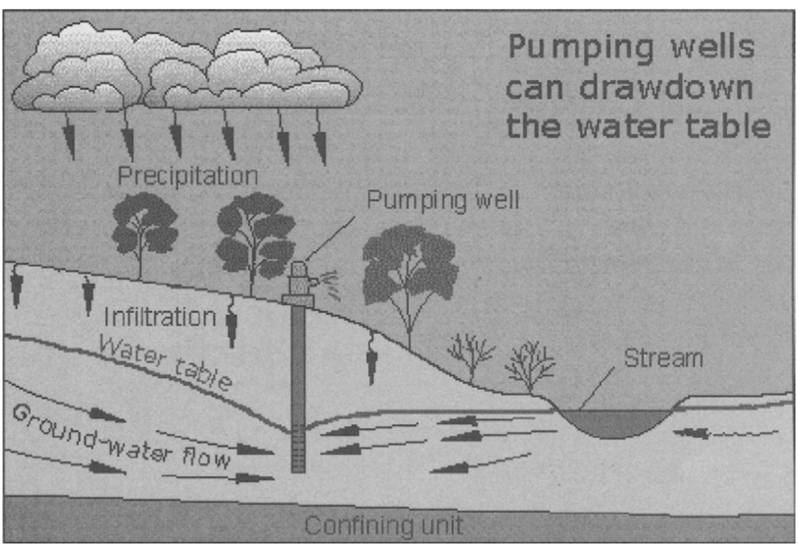

Figure 1.29 Pumping well (*Source*: http://ga.water.usgs.gov/edu/graphics/wcgwstoragewell.gif)

1.12 Laboratory Testing

After completion of the boring program, laboratory tests are conducted on the soil samples. The laboratory test program is dependant on the project requirements. Some of the laboratory tests done on soil samples are given below.

1. Sieve analysis.
2. Hydrometer.
3. Water content.
4. Atterberg limit tests (liquid limit and plastic limit).
5. Permeability test.
6. UU tests (undrained unconfined tests).
7. Density of soil.
8. Consolidation test.
9. Tri-axial tests.
10. Direct shear test.

1.12.1 Sieve Analysis

Sieve analysis is conducted to classify soil into sands, silts, and clays. Sieves are used to separate soil particles and group them based on their size. This test is used for the purpose of classification of soil.

Standard sieve sizes are shown in Table 1.5. In general, any particle greater than a no. 4 sieve is considered to be gravel. Sands are defined as falling in the range of no. 4 to no. 200 sieves. Silts and clays are particles smaller than no. 200 sieve.

A hypothetical sieve analysis test based on selected set of sieves is given in Fig. 1.30 as an example.

Sieve no. 4, size = 4.75 mm, percentage of soil retained = 0%
Sieve no. 16, size = 1.18 mm, percentage of soil retained = 20%
Sieve no. 50, size = 0.30 mm, percentage of soil retained = 25%

Table 1.5 U.S. and British sieve number and mesh size

Sieve no.	Mesh size (mm)
U.S. sieve number and mesh size	
No. 4	4.75
No. 6	3.35
No. 8	2.36
No. 10	2.00
No. 12	1.68
No. 16	1.18
No. 20	0.85
No. 30	0.60
No. 40	0.425
No. 50	0.30
No. 60	0.25
No. 80	0.18
No. 100	0.15
No. 200	0.075
No. 270	0.053
British sieve number and mesh size	
8	2.057
16	1.003
30	0.500
36	0.422
52	0.295
60	0.251
85	0.178
100	0.152
200	0.076
300	0.053

Sieve no. 80, size = 0.18 mm, percentage of soil retained = 20%
Sieve no. 200, size = 0.075 mm, percentage of soil retained = 30%

If we know the percentage of soil passed through a given sieve, we can find the percentage of soil retained in that sieve.

- Sieve no. 4 (4.75 mm): all soil went past sieve no. 4.

$$\text{percent retained} = 0\%$$

$$\text{percent passed} = 100\%$$

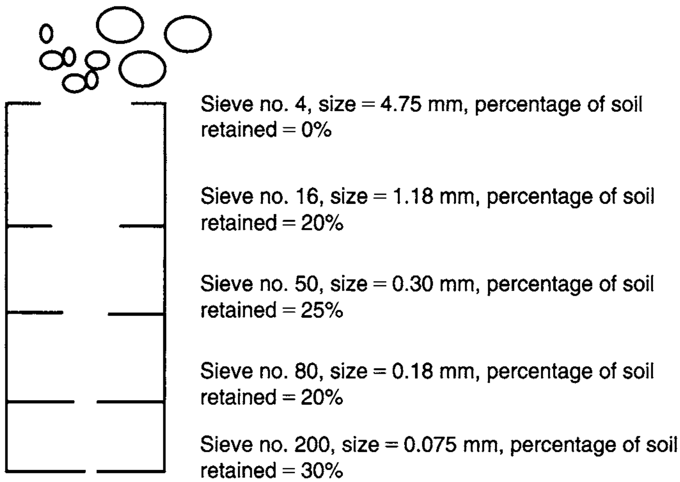

Figure 1.30 Sieve analysis

- Sieve no. 16 (1.18 mm): 20% of soil was retained in sieve no. 16.

$$\text{percent retained at sieve no. } 16 = 20\%$$
$$\text{percent passed} = 100 - 20 = 80\%$$

- Sieve no. 50 (0.30 mm): 25% of soil was retained in sieve no. 50.

$$\text{total retained so far} = 20 + 25 = 45\%$$
$$\text{percent passed} = 100 - 45 = 55\%$$

- Sieve no. 80 (0.18 mm): 20% retained in sieve no. 80.

$$\text{total retained so far} = 20 + 25 + 20 = 65\%$$
$$\text{percent passed} = 100 - 65 = 35\%$$

- Sieve no. 200 (0.075 mm): 30% retained in sieve no. 200.

$$\text{total retained so far} = 20 + 25 + 20 + 30 = 95\%$$
$$\text{percent passed through sieve no. } 200 = 100 - 95 = 5\%$$

Now it is possible to draw a graph indicating percent passing at each sieve, as shown in Fig. 1.31.

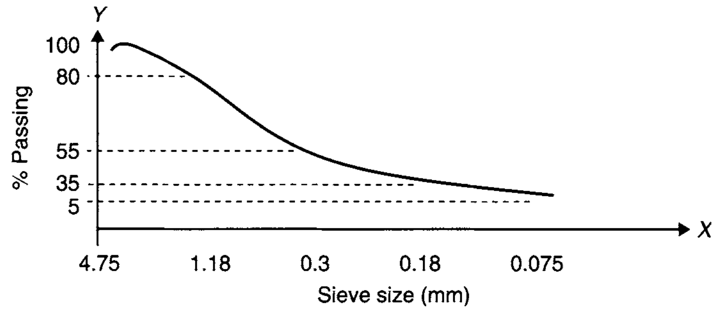

Figure 1.31 Percent passing vs. particle size

D_{60}

The variable D_{60} is defined as the size of the sieve that allow 60% of the soil to pass. This value is used for soil classification purposes and frequently appears in geotechnical engineering correlations.

As can be seen in the 0.3 mm sieve in Fig. 1.31, 55% of the soil would pass though this sieve. If we make the sieve bigger, more soil would pass. On the other hand, if we make the sieve smaller, then less than 55% of the soil would pass.

To find the D_{60} value, draw a horizontal line at the 60% passing point. Where the horizontal line meets the curve, drop it down to obtain the D_{60} value. See Fig. 1.32.

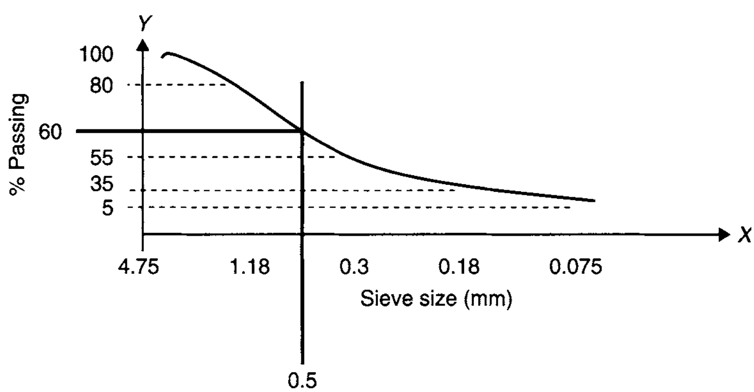

Figure 1.32 Finding D_{60}

In this case D_{60} is closer to 0.5 mm.

Now let us use Fig. 1.31 to find D_{30}. As before, draw a line at the 30% passing line. In this case D_{30} happened to be approximately 0.1 mm.

Fig. 1.33 shows a sieve set. Table 1.6 shows size ranges for soils and gravels, and Table 1.7 gives a range of specific gravity values for soil and gravel components.

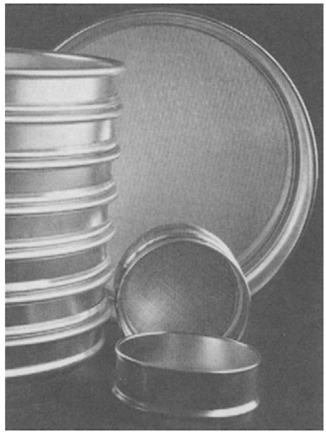

Figure 1.33 Sieve set (*Source*: Precisioneforming LLC., http://www.precisioneforming.com)

Table 1.6 Size ranges for soils and gravels

Soil	Size (in.)	Size (mm)	Comments
Boulders	>6	>150	
Cobbles	3–6	75–150	
Gravel	0.187–3	4.76–75	Greater than #4 sieve size
Sand	0.003–0.187	0.074–4.76	Sieve #200 to sieve #4
Silt	0.00024–0.003	0.006–0.074	Smaller than sieve #200
Clay	0.00004–0.00008	0.001–0.002	Smaller than sieve #200
Colloids	<0.00004	<0.001	

Table 1.7 Specific gravity (Gs)

Soil	Specific gravity
Gravel	2.65–2.68
Sand	2.65–2.68
Silt (inorganic)	2.62–2.68
Organic clay	2.58–2.65
Inorganic clay	2.68–2.75

1.12.2 Hydrometer

Hydrometer tests are conducted to classify particles smaller than a no. 200 sieve (0.075 mm).

When soil particles are mixed with water, larger particles settle quickly. On the other hand, smaller particles tend to float or settle at a low velocity. See Fig. 1.34. Since D_2 is greater than D_1, the velocity of the sand particles (V_2) will be greater than the velocity of the silt particles (V_1). Similarly V_3 will be greater than V_2.

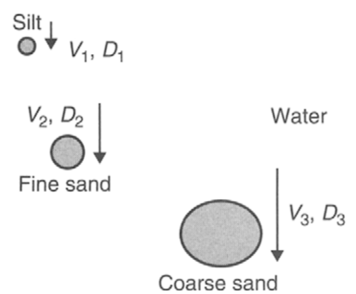

Figure 1.34 Settling soil particles

The velocity of settling particles is given by Stokes law.

$$V = \frac{980 \times (G - G_W) \times D^2}{30\eta}$$

where

G = specific gravity of soil
G_W = specific gravity of water
D = particle diameter (mm)
V = velocity of particles (mm/sec)
η = absolute viscosity of water (in Poise)

The variables V and D are unknown quantities in the above equation.

If the settling velocity V can be found by experiment, then the particle diameter D can be computed.

Hydrometer Test Procedure

The reader is referred to the ASTM D422 standard for a full explanation of the hydrometer test procedure. An overview of the test is provided here. See Fig. 1.35.

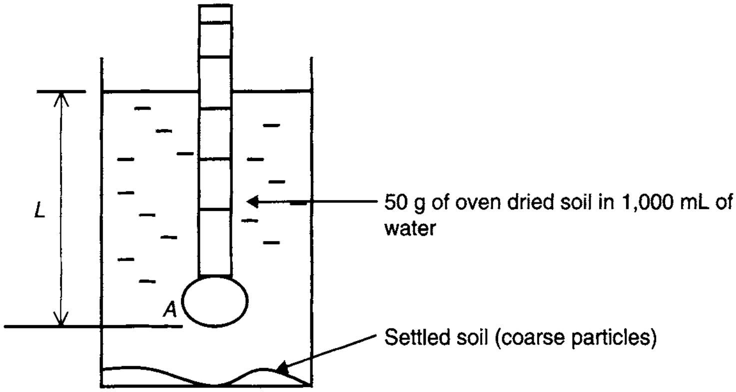

Figure 1.35 Hydrometer

The hydrometer reading is dependant upon the density of the liquid. When soil is mixed with water, very fine particles will be suspended in water while the heavy particles will settle to the bottom. If more soil is suspended in water, then the hydrometer reading L will be smaller. The reading L is an indication of amount of fine particles suspended in water.

Hydrometer test procedure

- Mix 50 g of oven dried soil and 1,000 mL of distilled water.
- Mix soil and water thoroughly.
- Insert the hydrometer and obtain the hydrometer readings L.
- The hydrometer reading provides grams of soil in suspension per liter of solution.
- Obtain readings for different time periods, typically 2, 5, 15, 30, 60, 250, and 1,440 min.
- Compute D for every reading.
- Plot a graph between grams of soil per liter of water vs. D.
- Soil particles will increase the density of water/soil mixture. Hence, initially L would be a lower value. As time passes, larger diameter particles settle. The density of the soil/water mixture would go down and the hydrometer would sink (L value would increase).

- At time T_1, the length measured is L_1. Use the Stokes equation to find D_1.

$$V_1 = \frac{L_1}{T_1} = \frac{980 \times (G_1 - G_W)}{30\eta} \times D_1^2$$

- Since L_1 and T_1 are known, D_1 can be computed.
- L_1 is the hydrometer reading and T_1 is the time passed.
- G_1 is the specific gravity of the soil and has to be measured separately.
- According to the Stokes equation, all particles larger than D_1 have settled below the hydrometer level.
- Obtain the hydrometer reading (grams of soil in suspension). The D value computed from the Stokes equation gives the largest particles that could possibly be in suspension. In other words, all the particles in the solution are smaller than the D value obtained using the Stokes equation.
- Hence the hydrometer reading is similar to the percent weight passing reading given by a sieve.
- Use the weights passing reading to obtain the percent passing reading by dividing the weight per each size by the total sample.
- Use the table below to fill in the information required for the analysis.

Sieve size	**Percent passing**
Sieve analysis	
No. 4	________
No. 10	________
No. 40	________
No. 200	________
Hydrometer analysis	
0.074 mm	________
0.005 mm	________
0.001 mm	________

Combine the hydrometer readings with sieve analysis readings and prepare a single graph. See Fig. 1.36.

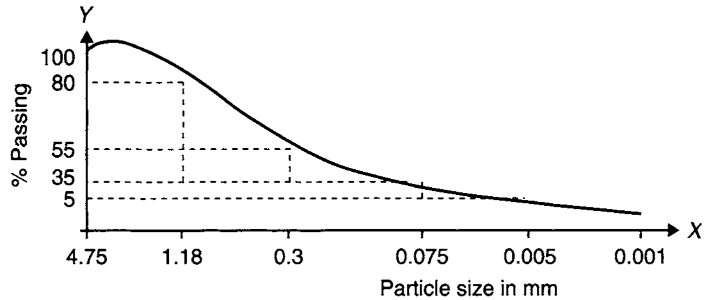

Figure 1.36 Hydrometer readings incorporated to sieve analysis curve

1.12.3 Liquid Limit and Plastic Limit (Atterberg Limit)

Liquid Limit

The liquid limit (LL) is the level of water content at which the soil starts to behave as a liquid.

The liquid limit is measured by placing a clay sample in a standard cup and making a separation (groove) using a spatula. The cup is dropped until the separation vanishes. The water content of the soil is obtained from this sample. The test is performed again by increasing the water content. A soil with a lower water content would yield more blows, and a soil with a higher water content would yield fewer blows.

A graph can be constructed comparing the number of blows and the water content. See Fig. 1.37. The liquid limit of a clay is defined as the water content that corresponds to 25 blows.

What is the significance of the liquid limit? Liquid limits for two soils are shown in Fig. 1.38. Soil 1 would reach a liquid-like state at water content of LL1. Soil 2 would attain this state at water content LL2.

In the graph shown in Fig. 1.38, LL1 is higher than LL2. In other words, soil 2 loses its shear strength and becomes liquid-like at a lower water content than soil 1.

Plastic Limit

The plastic limit is measured by rolling a clay sample to a 3 mm diameter cylindrical shape. During continuous rolling at this size, the clay

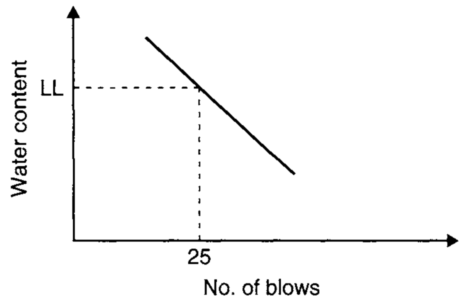

Figure 1.37 Graph for liquid limit test

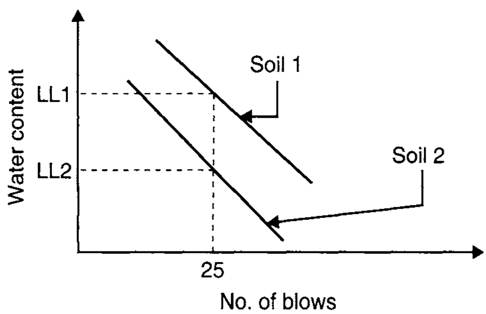

Figure 1.38 Liquid limits for two soils

sample tends to lose moisture and cracks start to appear. The water content at the point where cracks start to appear is defined as the plastic limit. See Fig. 1.39.

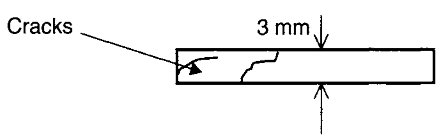

Figure 1.39 Plastic limit test

Practical Considerations of Liquid Limit and Plastic Limit

The water content where a soil converts to a liquid-like state is known as liquid limit. Consider the two slopes as shown in Fig. 1.40. Assuming all other factors to be equal, which slope would fail first, slope 1 with a liquid limit of 30%, or slope 2 with a liquid limit of 40%?

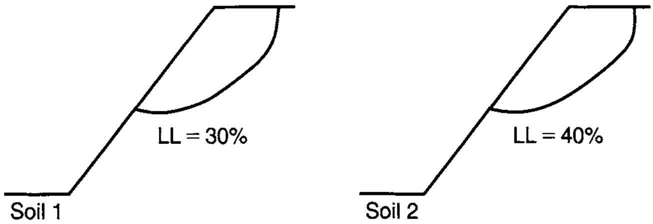

Figure 1.40 Slope stability in different soils

During a rain event, soil 1 would reach the liquid limit prior to soil 2. Hence soil 1 would fail before soil 2.

During earthquakes, water tends to rise. If soils with a low liquid limit were present, then those soils would lose strength and fail.

The plastic limit indicates the limit of plasticity. When the water content goes below the plastic limit of a soil, then cracks start to appear in that soil. Soils lose cohesion below the plastic limit. See Figs. 1.41 and 1.42.

1.12.4 Permeability Test

Transport of water through soil media depends on the pressure head, velocity head, and the potential head due to elevation. In most cases, the most important parameter is the potential head due to elevation. See Fig. 1.43.

Water travels from point A to point B due to high potential head. The velocity of the traveling water is given by the Darcy equation.

$$v = k \times i$$

where

v = velocity
k = coefficient of permeability (cm/sec or in./sec)
i = hydraulic gradient = h/L
L = length of soil
$Q = A \times v$ = volume of water flow
A = area
v = velocity

Figure 1.41 Slope failure (*Source*: http://www.dot.ca.gov)

Design Example 1.5

Find the volume of water flowing in the pipe shown in Fig. 1.44. The soil permeability is 10^{-5} cm/sec. The area of the pipe is $5\,cm^2$. The length of the soil plug is 50 cm.

Solution

Apply the Darcy equation

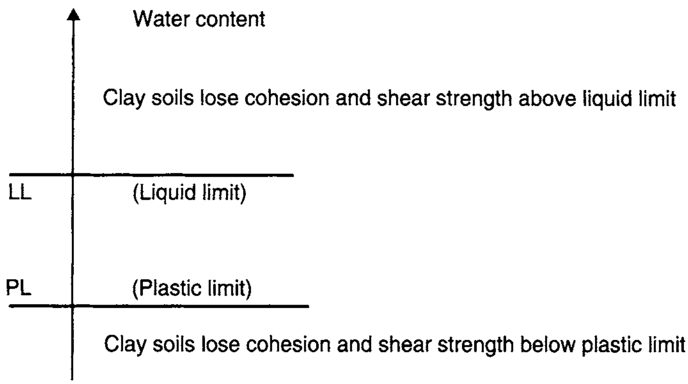

Figure 1.42 Liquid limit and plastic limit

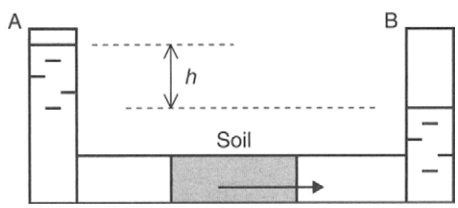

Figure 1.43 Water flowing through soil

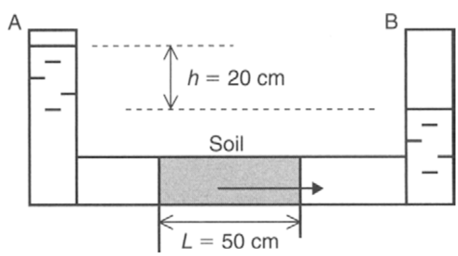

Figure 1.44 Water flow due to 20 cm gravity head

$$v = k \times i$$

$$v = k \times (h/L)$$

$$v = 10^{-5} \times 20/50 = 4 \times 10^{-6}\ \text{cm/sec}$$

$$\text{volume of water flow} = A \times v = 5 \times 4 \times 10^{-6}\ \text{cm}^3/\text{sec}$$

$$= 2 \times 10^{-5}\ \text{cm}^3/\text{sec}$$

Seepage Rate

Water movement in soil occurs through the voids within the soil fabric. The more voids there are in a soil, the more water can flow through. This seepage path can be seen in Fig. 1.45.

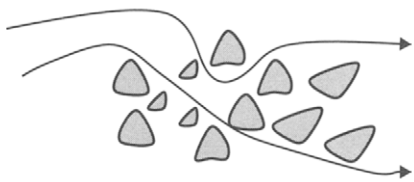

Figure 1.45 Seepage path

The velocity of water seepage through a soil mass is dependant upon the void ratio or porosity of the soil mass. This seepage velocity can be seen in Fig. 1.46. The volume of water traveling through the soil is

$$Q = v \times A$$

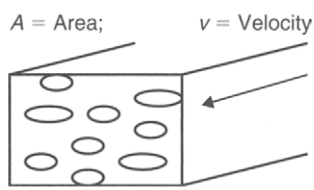

Figure 1.46 Seepage velocity

where

$A =$ area
$v =$ velocity

The porosity (n) of a soil is defined as

$$n = V_v / V$$

where

$V_v =$ volume of voids $= L \times A_v$
$A_v =$ area of voids

V = total volume = $L \times A$
L = length
A = total cross-sectional area

Hence

$$n = (L \times A_v)/(L \times A) = A_v/A$$
$$A_v = n \times A$$
$$Q = v \times A$$

The velocity of water traveling through voids (v_s) is known as the seepage velocity.

$$Q = v_s \times A_v$$
$$Q = v \times A = v_s \times A_v = v_s \times (n \times A)$$
$$v \times A = v_s \times (n \times A)$$
$$v_s = v/n$$

1.12.5 Unconfined Undrained Compressive Strength Tests (UU Tests)

The unconfined compressive strength test is designed to measure the shear strength of clay soils. This is the easiest and most common test done to measure the shear strength.

Since the test is done with the sample in an unconfined state and the load is applied quickly so that there is no possibility of draining, the test is known as the unconfined undrained test (UU test).

The soil sample is placed in a compression machine and compressed to failure. Stresses are recorded during the test and plotted. See Fig. 1.47 for the UU apparatus and stress-strain curve. Also see Fig. 1.48 for Mohr's circle for the UU test.

$$q = \text{stress at failure}$$
$$c = \text{cohesion} = q/2$$

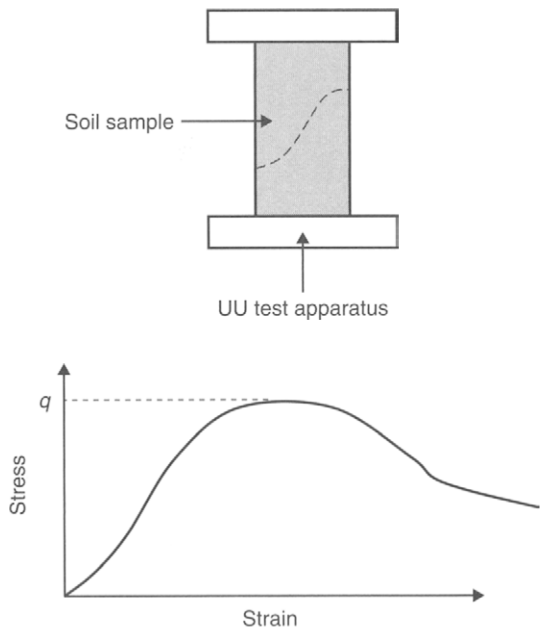

Figure 1.47 UU apparatus and stress-strain curve

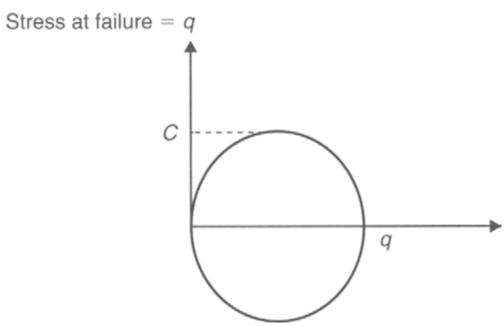

Figure 1.48 Mohr's circle for the UU test

1.12.6 Tensile Failure

When a material is subjected to a tensile stress, it will undergo tensile failure. See Fig. 1.49 for the tensile strength test.

Figure 1.49 shows material failure under tension. Tensile failure of soil is not as common as shear failure, and tensile tests are rarely conducted. On the other hand, tensile failure is common in tunnels. Rocks in tunnel roofs are subjected to tensile forces and proper supports need to be provided. See Fig. 1.50.

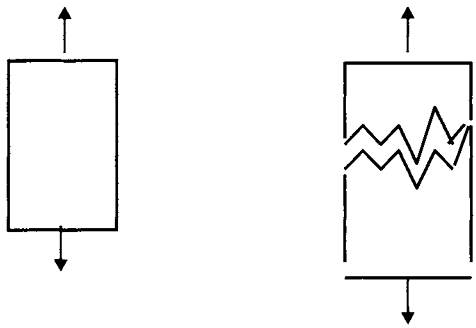

Figure 1.49 Tensile strength test

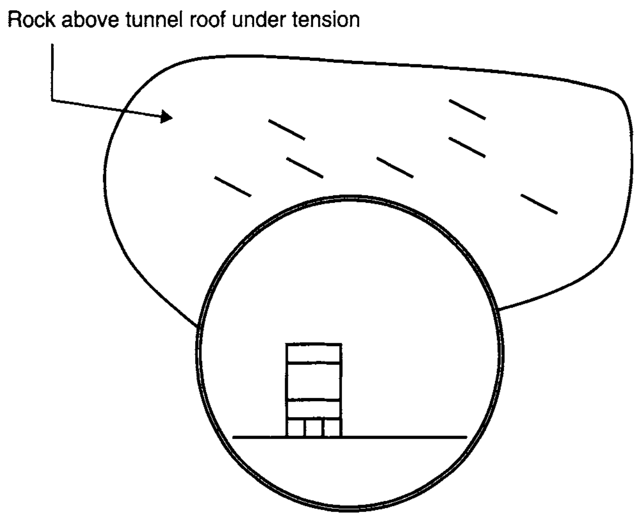

Figure 1.50 Rock under tension

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2

Geotechnical Engineering Theoretical Concepts

This chapter will discuss geotechnical engineering concepts that are needed for design.

2.1 Vertical Effective Stress

Almost all problems in geotechnical engineering require the computation of effective stress.

Prior to venturing into the concept of effective stress, we will consider a solid block sitting on a table. The density of the block is given as γ, and the area at the bottom of the block is A. See Fig. 2.1.

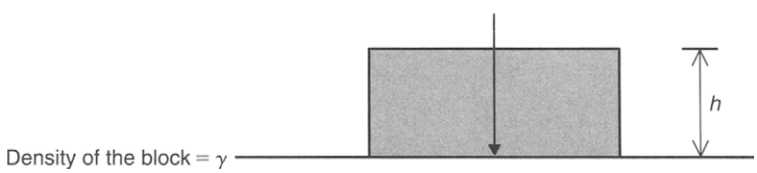


Figure 2.1 Block on top of a plane

$$\text{volume } (V) = A \times h$$

$$\text{weight of the block} = \text{density} \times \text{volume} = (\gamma \times A \times h)$$

$$\text{vertical stress at the bottom} = \text{weight/area} = (\gamma \times A \times h)/A = \gamma \times h$$

What happens when water is present? See Fig. 2.2.

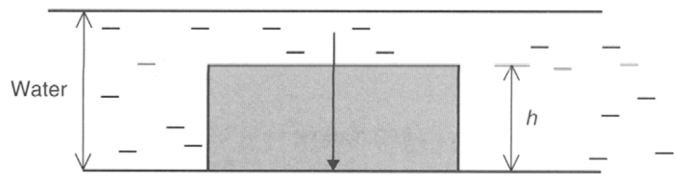


Figure 2.2 Block under buoyant forces

Due to buoyancy, the effective stress is reduced when water is introduced. We experience this fact daily when we step into a pool. Inside the pool, we feel less weight due to buoyancy.

$$\text{new vertical stress at the bottom} = (\gamma - \gamma_W) \times h$$

where

γ_W = density of water

What happens when water only partially fills the space? What is the pressure at the base, if the water is filled to a height of y ft as shown in Fig. 2.3? The new vertical stress at the bottom is given as

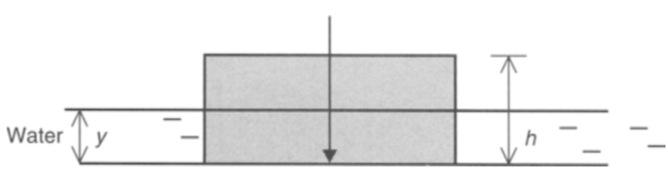


Figure 2.3 Partially submerged block

Soils also have a lesser effective stress under the water table. Consider the following example.

Design Example 2.1

Find the effective stress at point A (no water present). See Fig. 2.4.

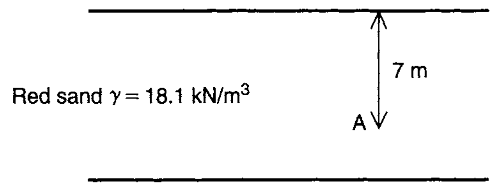


Figure 2.4 Effective stress

Solution

$$\text{effective stress at point A} = 18.1 \times 7 = 126.7\,\text{kN/m}^3$$

Since there is no groundwater, the total density of soil is all that is necessary to compute the effective stress.

Design Example 2.2

Find the effective stress at point A. For this example, the groundwater is 2 m below the surface. See Fig. 2.5.

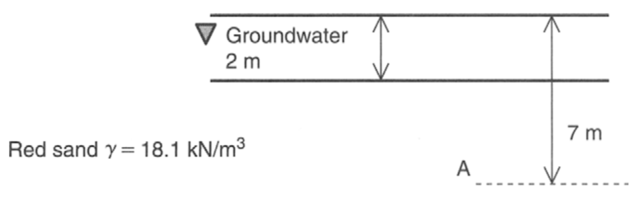


Figure 2.5 Effective stress when groundwater present

Solution

$$\text{effective stress at point A} = 18.1 \times 2 + (18.1 - \gamma_W) \times 5\,\text{kN/m}^3$$

$$\gamma_W = 9.81\,\text{kN/m}^3$$

$$\text{effective stress at point A} = 18.1 \times 2 + (18.1 - 9.81) \times 5\,\text{kN/m}^3$$

$$= 77.7\,\text{kN/m}^3$$

$$\gamma_W = 9.81\,\text{kN/m}^3$$

Note that there is no buoyancy acting on the first 2 ft of the soil. Hence the density on the first 2 ft of soil is not reduced. There is buoyancy acting on the soil below the groundwater level.

Hence the density of the soil below the groundwater level is reduced by 9.81 kN/m^3 to account for the buoyancy.

2.2 Lateral Earth Pressure

Once the vertical effective stress is found, it is a simple matter to calculate the lateral earth pressure.

Consider the water pressure at point A. See Fig. 2.6.

$$\text{vertical pressure at point A} = \gamma_W \times h$$

$$\text{horizontal pressure at point A} = \gamma_W \times h$$

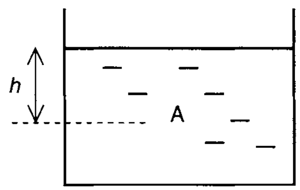


Figure 2.6 Pressure in water

In the case of water, the vertical pressure and horizontal pressure at a point are the same. This is not the case with soil. In soil, the horizontal pressure (or horizontal stress) is different from the vertical stress. See Fig. 2.7.

$$\text{horizontal and vertical pressure at point A in water} = \gamma_W \times h$$

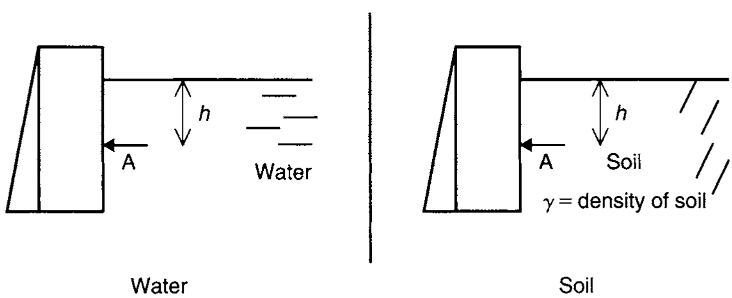


Figure 2.7 Lateral earth pressure in water and soil

As mentioned earlier, pressure in water is the same in all directions. In the case of soil, it has been found experimentally that the horizontal pressure is given by the following equation.

horizontal pressure at point A in soil = $K_0 \times$ (vertical effective stress)

where

K_0 = lateral earth pressure coefficient at rest

To further define horizontal pressure in soil,

$$\text{horizontal pressure at point A in soil} = K_0 \times \gamma \times h$$

where

γ = density of soil
h = height of soil

When the soil can move, K_a and K_p values should be used instead of K_0. See Fig. 2.8.

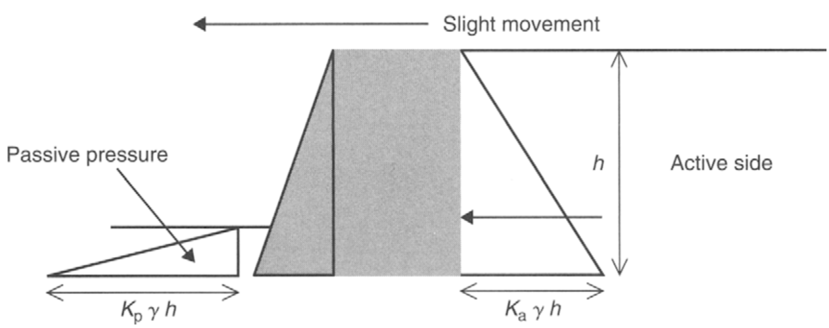


Figure 2.8 Active and passive earth pressure

The retaining wall will move slightly to the left due to the earth pressure. Due to this slight movement, the pressure on one side will be relieved and the other side will be amplified. K_a is known as the active earth pressure coefficient and K_p is known as the passive earth pressure coefficient. The passive earth pressure coefficient is larger than the active earth pressure coefficient.

$$K_a = \tan^2(45 - \varphi/2)$$

$$K_p = \tan^2(45 + \varphi/2)$$

$$K_a < K_0 < K_p$$

2.3 Stress Increase Due to Footings

When a footing is placed on soil, the stress of the soil layer will increase. The stress of the footing would be reduced due to the distribution.

Assume a 3×3 m square footing as shown in Fig. 2.9. From the figure, the column load is 180 kN. The stress on the soil just below the footing is

$$180/9 = 20\,\text{kPa}$$

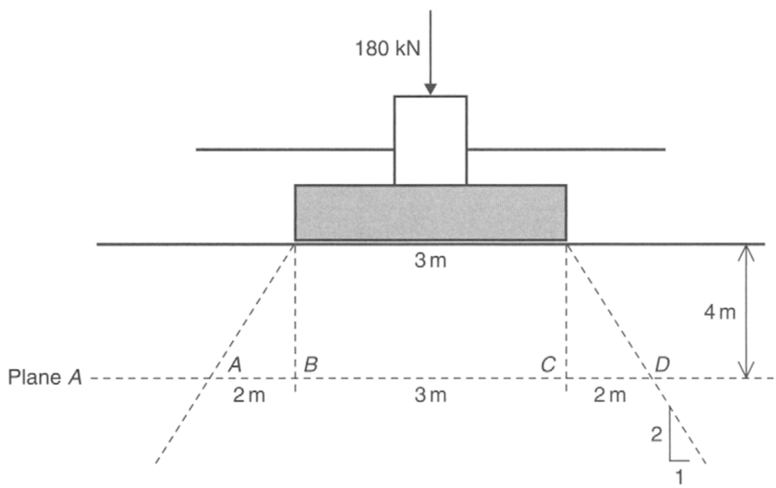


Figure 2.9 Stress distribution in a column footing

The length of one side at plane A with a 2:1 stress distribution is given as

$$AB + BC + CD = 3 + 2 + 2 = 7\,\text{m}$$

The stress at plane A can be calculated as

$$180/(7 \times 7) = 3.67\,\text{kN/m}^2$$

Note that lengths AB and CD are equal to 2 m since the stress distribution is assumed to be 2:1. The depth to plane A from the bottom of the footing is 4 m. Hence lengths AB and CD are 2 m each.

For an illustration of loading on strip footings, see Fig. 2.10.

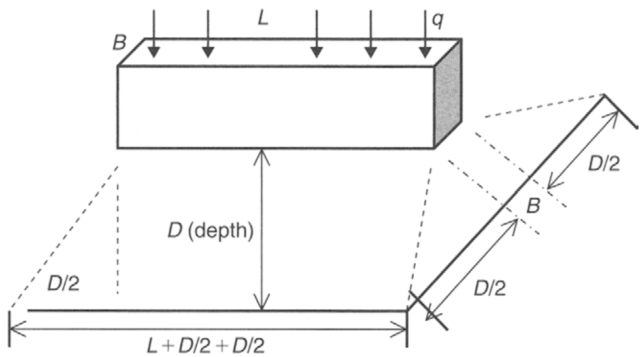


Figure 2.10 Pressure distribution under a wall footing assuming a 2:1 distribution

The total load at footing level is given by

$$q \times (L \times B)$$

where

$q =$ pressure at footing level
$L =$ length of the wall footing
$B =$ width of the wall footing

The pressure at D m below the bottom of footing is given by

$$\frac{q \times (L \times B)}{(L + \frac{D}{2} + \frac{D}{2}) \times (B + \frac{D}{2} + \frac{D}{2})}$$
$$= \frac{q \times (L \times B)}{(L + D) \times (B + D)}$$

where

$D =$ depth to the layer of interest measured from the bottom of the footing

Design Example 2.3

Find the stress increase at a point 2 m below the bottom of footing. The strip foundation has a load of 200 kN per 1 m length of footing and has a width of 1.5 m. Assume 2:1 stress distribution. See Fig. 2.11.

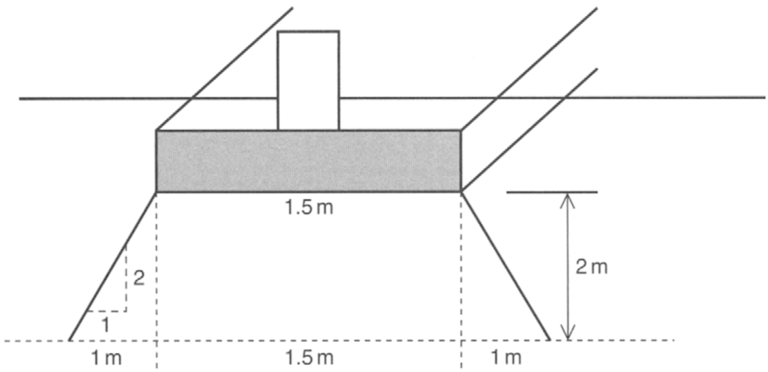


Figure 2.11 Strip footing loading

Solution

The loading at the bottom of footing per 1 m length of the footing is 200 kN. The stress increase at 2 m below the bottom of footing is

$$200/(3.5 \times 1) = 57.1 \text{ kN per 1 m length of footing}$$

Note that in this case, the load distribution at the ends of the strip footing has been neglected.

2.4 Overconsolidation Ratio (OCR)

The overconsolidation ratio is defined as the ratio of past maximum stress and present existing stress. The existing stress in a soil can be computed based on the effective stress method. In the past, the soil most probably would have been subjected to a much higher stress.

$$\text{overconsolidation ratio} = p'_c/p'_0$$

where

p'_c = maximum stress that soil was ever subjected to in the past
p'_0 = present stress

Soils have been subjected to larger stresses in the past due to glaciers, volcanic eruptions, groundwater movement, and the appearance and disappearance of oceans and lakes.

2.4.1 Overconsolidation Due to Glaciers

Ice ages come and go approximately every 20,000 years. During an ice age, a large percentage of land will be covered with glaciers. Glaciers generate huge stresses in the underlying soil. See Fig. 2.12.

$$p'_c = \text{maximum effective stress encountered by clay}$$

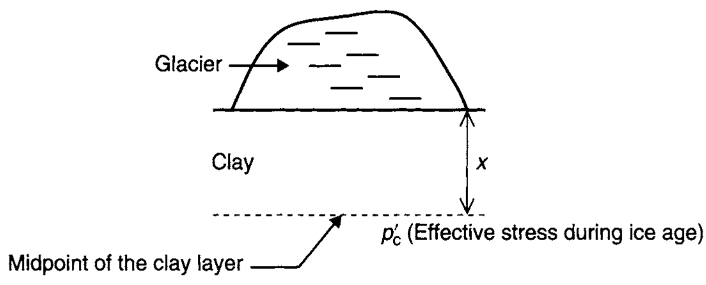


Figure 2.12 High stress levels in soil during ice ages due to glaciers

When the load is removed, the clay layer will rebound. See Fig. 2.13. The effective stress after the load is removed is p'_0. The overconsolidation ratio is again

$$p'_c / p'_0$$

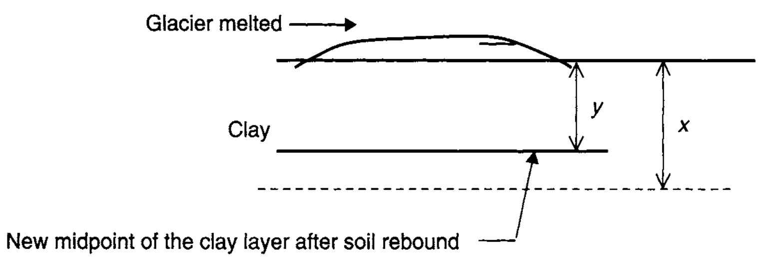


Figure 2.13 Rebound of the clay layer due to melting of the glacier

When the glacier is melted, the stress on the soil is relieved. Hence $p'_c > p'_0$. See Fig. 2.14.

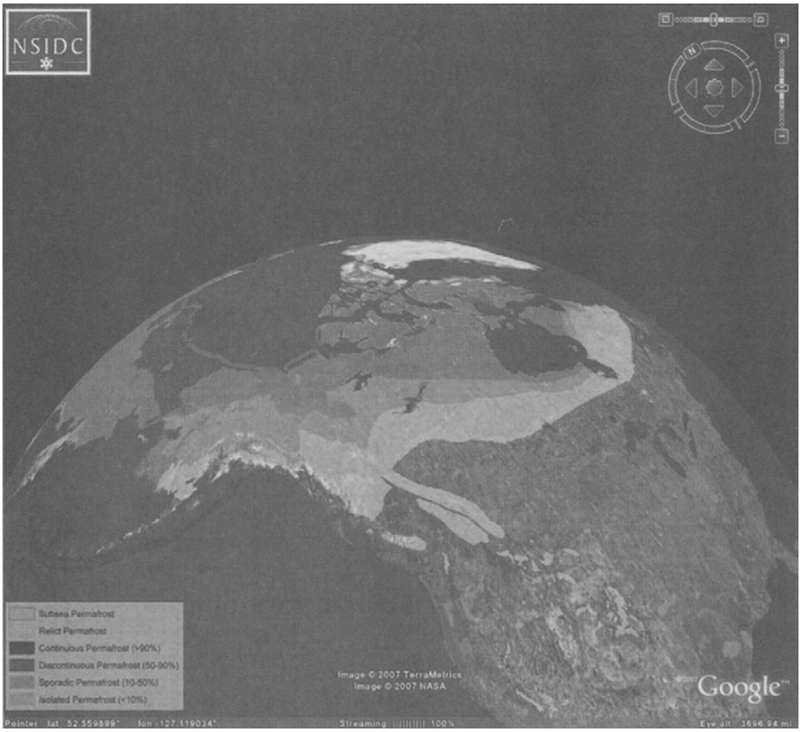


Figure 2.14 NASA image of present day distribution of glaciers in North America

Design Example 2.4

Find the overconsolidation ratio of the clay layer at the midpoint. The maximum stress that the soil was subjected to in the past is 150 kN/m^2. See Fig. 2.15.

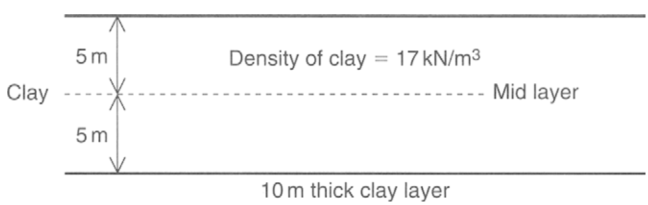


Figure 2.15 Overconsolidation ratio

Solution

STEP 1: Find the present stress at the midpoint of the clay layer (p'_0)

$$p'_0 = 17 \times 5 = 85\,\text{kN/m}^2$$

STEP 2: Find the overconsolidation ratio (OCR):

$$\text{OCR} = \text{past maximum stress/present stress}$$

The past maximum stress is given as 150 kN/m^2.

$$\text{OCR} = 150/85 = 1.76$$

Design Example 2.5

Find the overconsolidation ratio of the clay layer at the midpoint. The maximum stress that the soil was subjected to in the past is 150 kN/m^2. The groundwater is at 3 m below the surface. See Fig. 2.16.

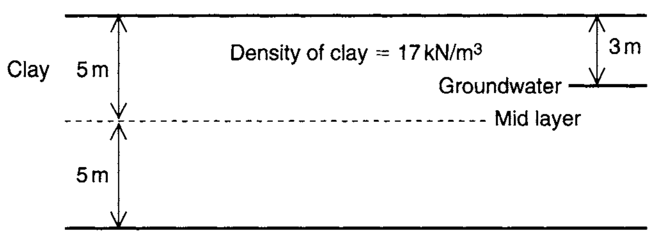


Figure 2.16 Soil profile for clay layer with groundwater at 3 m below the surface

Solution

STEP 1: Find the present stress at the midpoint of the clay layer (p'_0)

$$p'_0 = 17 \times 3 + (17 - \gamma_w) \times 2$$

$$\gamma_w = \text{density of water} = 9.81\,\text{kN/m}^3$$

$$p'_0 = 65.4$$

STEP 2: Find the overconsolidation ratio (OCR)

$$OCR = \text{past maximum stress/present stress}$$

The past maximum stress is given as $150\,kN/m^2$.

$$OCR = 150/65.4 = 2.3$$

2.4.2 Overconsolidation Due to Groundwater Lowering

When groundwater is lowered, the effective stress at a point will rise. This is explored in the following example.

Design Example 2.6

Find the effective stress at point A. The density of the soil is found to be $17\,kN/m^3$ and the groundwater is at 2 m below the surface. See Fig. 2.17.

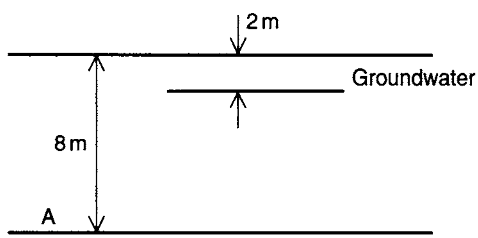


Figure 2.17 Soil profile with groundwater at 2 m below the surface

Solution

$$\text{effective stress at point A} = 17 \times 2 + (17 - \gamma_W) \times 6$$

$$\gamma_W = 9.81\,kN/m^3$$

$$\text{effective stress at point A} = 77.1\,kN/m^2.$$

What would happen to the effective stress if the groundwater were lowered to 4 m below the surface? See Fig. 2.18.

$$\text{new effective stress at point A} = 17 \times 4 + (17 - \gamma_W) \times 4 = 96.8\,kN/m^2$$

When the groundwater was lowered, the effective stress increased.

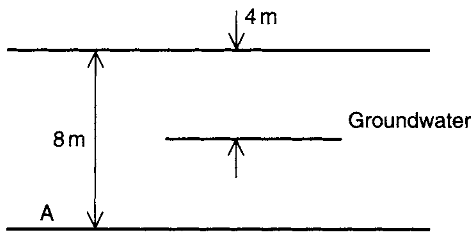


Figure 2.18 Soil profile with groundwater at 4 m below the surface

Design Example 2.7

A 10 m thick sand layer is underlain by an 8 m thick clay layer. The groundwater is found to be at 3 m below the surface at the present time. Old well log data shows that the groundwater was as low as 6 m below the surface in the past. What is the overconsolidation ratio (OCR) at the midpoint of the clay layer? The density of sand is 18 kN/m^3 and the density of clay is 17 kN/m^3. See Fig. 2.19.

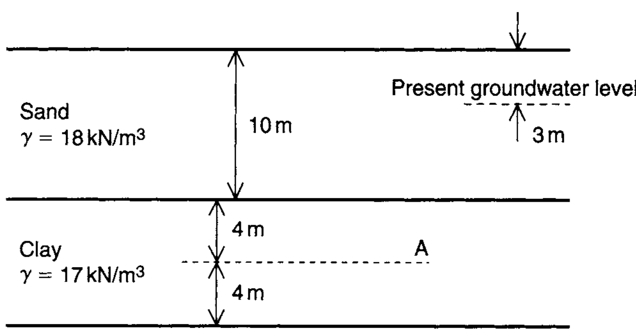


Figure 2.19 Soil profile and present groundwater level

Solution

STEP 1: First, consider the present situation.

Find the effective stress at the midpoint of the clay layer (point A)

$$3 \times 18 + 7 \times (18 - \gamma_W) + 4 \times (17 - \gamma_W)$$

$$\gamma_W = 9.81\,\text{kN/m}^3$$

$$\text{effective stress at point A} = 3 \times 18 + 7 \times (18 - 9.81) + 4 \times (17 - 9.81)$$

$$= 140.1\,\text{kN/m}^2$$

STEP 2: Find the effective stress at the midpoint of the clay layer (point A) in the past.

In the past, groundwater level was 6 m below the surface. See Fig. 2.20.

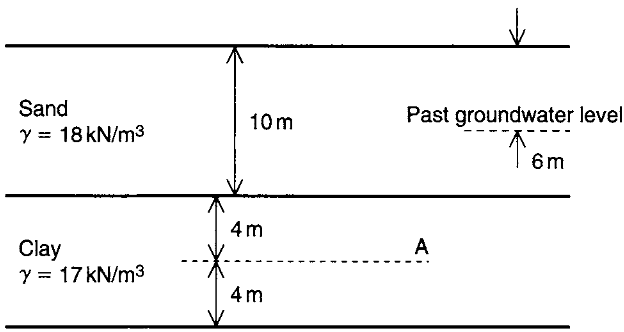


Figure 2.20 Soil profile and past groundwater level

Find the effective stress at the midpoint of the clay layer (point A).

$$6 \times 18 + 4 \times (18 - \gamma_W) + 4 \times (17 - \gamma_W)$$

$$\gamma_W = 9.81\,\text{kN/m}^3$$

$$\text{effective stress at point A} = 6 \times 18 + 4 \times (18 - 9.81) + 4 \times (17 - 9.81)$$

$$= 169.5\,\text{kN/m}^2$$

STEP 3: Find the overconsolidation ratio (OCR) due to past groundwater lowering.

$$\text{OCR} = \text{past maximum stress/present stress}$$

$$\text{OCR} = 169.5/140.1 = 1.21$$

2.5 Soil Compaction

Shallow foundations can be rested on controlled fill, also known as engineered fill or structural fill. Typically, such fill materials are carefully selected and compacted to 95% of the modified Proctor density.

The modified Proctor test is conducted by placing soil in a standard mold and compacting it with a standard ram.

2.5.1 Modified Proctor Test Procedure

STEP 1: Soil that needs to be compacted is placed in a standard mold and compacted. See Fig. 2.21.

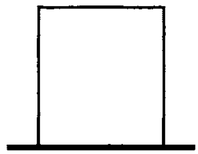


Figure 2.21 Standard mold

STEP 2: Compaction of soil is done by dropping a standard ram 25 times for each layer of soil from a standard distance. Typically soil is placed in five layers and compacted.

STEP 3: After compaction of all five layers, the weight of the soil is obtained. The soil contains solids and water. The solid is basically soil particles.

$$M = M_S + M_W$$

where

M = total mass of soil including water
M_S = mass of solid portion of soil
M_W = mass of water

STEP 4: Find the moisture content of the soil.
The moisture content is defined as

$$M_W/M_S$$

A small sample of soil is taken and placed in the oven and measured.

STEP 5: Find the dry density of soil.

The dry density of soil is given by

$$M_S/V$$

where

M_S = dry weight of soil
V = total volume of soil

STEP 6: Repeat the test a few times with soils of different moisture content, and plot a graph comparing dry density and moisture content. See Fig. 2.22.

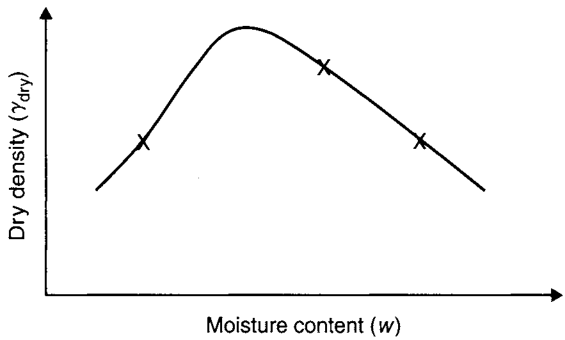


Figure 2.22 Dry density and moisture content

STEP 7: Obtain the maximum dry density (γ_{dry}) and the optimum moisture content.

See Fig. 2.23. For a given soil, there is an optimum moisture content that would provide the maximum dry density.

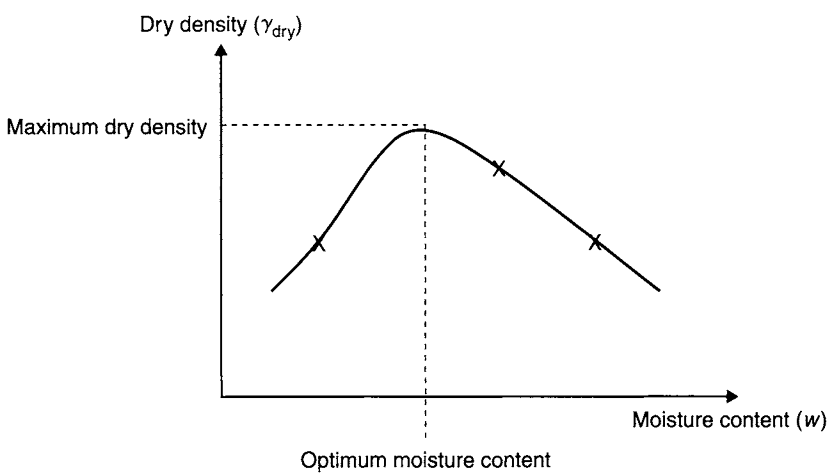


Figure 2.23 Optimum moisture content

It is not easy to attain the optimum moisture content in the field. Usually, soil that is too wet is not compacted. If the soil is too dry, water is added to increase the moisture content.

2.5.2 Controlled Fill Applications

Controlled fill can be used to build shallow foundations, roadways, parking lots, building slabs, and equipment pads. See Figs. 2.24 and 2.25.

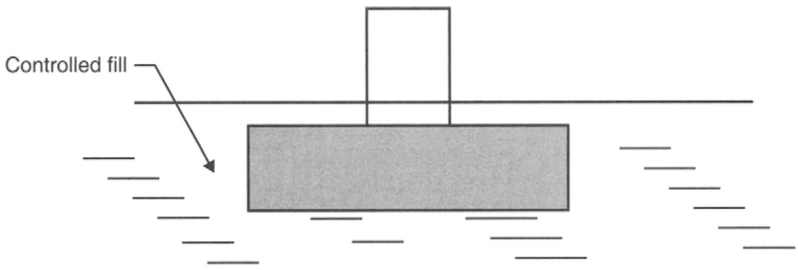


Figure 2.24 Shallow foundation placed on top of controlled fill

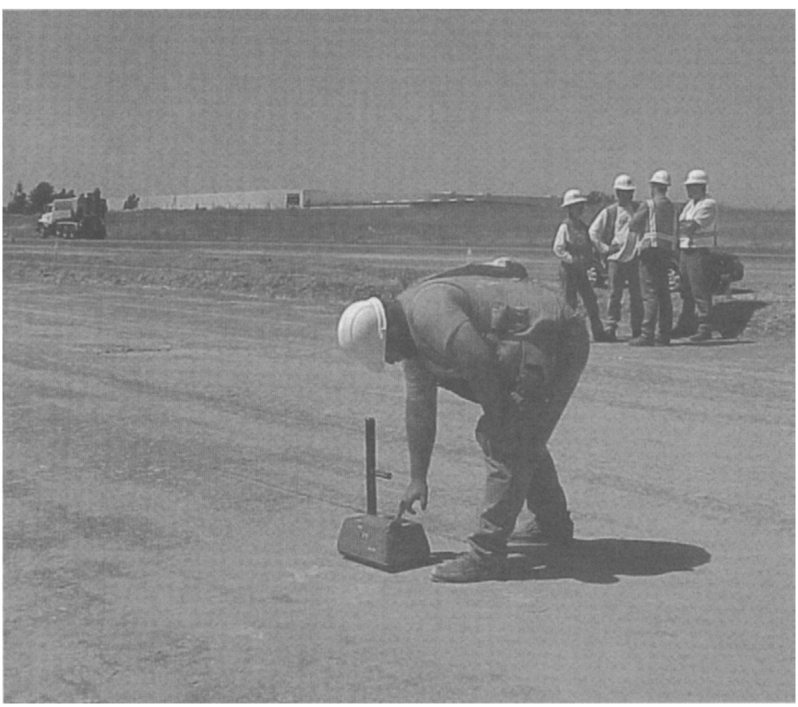


Figure 2.25 Checking the compaction status with a nuclear gauge *Source*: (http://www.dot.ca.gov.)

2.6 Borrow Pit Computations

Fill material for civil engineering work is obtained from borrow pits. The question is, how much soil should be removed from the borrow pit for a given project?

Usually, the final product is controlled fill or compacted soil. The total density, optimum moisture content, and dry density of the compacted soil will be available. This information can be used to obtain the mass of solids required from the borrow pit. If the soil in the borrow pit is too dry, water can always be added in the site. If the water content is too high, then soil can be dried prior to use. This could take some time in the field since it takes a few sunny days to get rid of water.

Water can be added or removed from soil. What cannot be changed is the mass of solids.

2.6.1 Procedure

- Find the mass of solids required for the controlled fill.
- Excavate and transport the same mass of solids from the borrow pit.

Design Example 2.8

A road construction project needs compacted soil to construct a road that will be 10 ft wide and 500 ft long. The road needs a 2 ft layer of soil.

The modified Proctor density was found to be 112.1 pcf at an optimum moisture content at 10.5%. See Fig. 2.26.

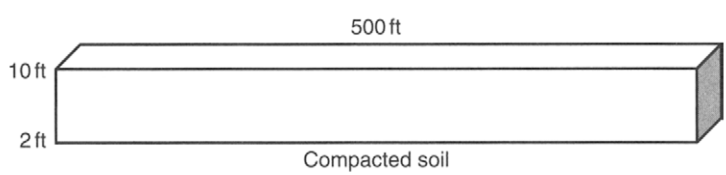


Figure 2.26 Compaction of soil

The soil in the borrow pit has the following properties:

total density of the borrow pit soil = 105 pcf

moisture content of borrow pit soil = 8.5%

Find the total volume of soil that needs to be hauled from the borrow pit.

Solution

STEP 1: Find the mass of solids (M_S) required for the controlled fill.

$$\text{volume of compacted soil required} = 500 \times 10 \times 2 = 10{,}000\,\text{cu ft}$$
$$\text{modified Proctor dry density} = 112.1\,\text{pcf}$$
$$\text{moisture content required} = 10.5\%$$

STEP 2: Draw the phase diagram for the controlled fill.

M_a = mass of air (usually taken to be zero)
V_a = volume of air
M_W = mass of water
V_W = volume of water
M_S = mass of solids
V_S = volume of solids
M = total mass of soil = $M_S + M_W$
V = total volume of soil = $V_S + V_W + V_a$

The soil after compaction has a dry density of 112.1 pcf and a moisture content of 10.5%.

$$\text{dry density} = M_S/V = 112.1\,\text{pcf}$$
$$\text{moisture content} = M_W/M_S = 10.5\% = 0.105$$

The road needed a 2 ft layer of soil at a width of 10 ft and length of 500 ft. Hence the total volume of soil is given by

$$2 \times 10 \times 500 = 10{,}000\,\text{cu ft}$$
$$V = \text{total volume} = 10{,}000\,\text{cu ft}$$

Since

$$M_S/V = \text{dry density}$$
$$M_S/10{,}000 = 112.1$$
$$M_S = 1{,}121{,}000\,\text{lb}$$

The mass of solids M_S of 1,121,000 lb should be hauled in from the borrow pit.

STEP 3: Find the mass of water in the controlled fill.

$$\text{moisture content in the controlled fill} = M_W/M_S = 10.5\% = 0.105$$

$$M_W = 0.105 \times 1{,}121{,}000\,\text{lb} = 117{,}705\,\text{lb}$$

STEP 4: Find the total volume of soil that needs to be hauled from the borrow pit.

The contractor needs to obtain 1,121,000 lb of solids from the borrow pit. The contractor can add water to the soil in the field if needed. The mass of solids needed is

$$M_S = 1{,}121{,}000\,\text{lb}$$

The density and moisture content of borrow pit soil are known.

$$\text{density of borrow pit soil} = M/V = 105\,\text{pcf}$$

$$\text{moisture content of borrow pit soil} = M_W/M_S = 8.5\% = 0.085$$

Since $M_S = 1{,}121{,}000\,\text{lb}$

$$M_W = 0.085 \times 1{,}121{,}000\,\text{lb} = 95{,}285\,\text{lb}$$

Therefore, the solid mass of 1,121,000 lb of soil in the borrow pit contains 95,285 lb of water.

$$\text{total weight of borrow pit soil} = 1{,}121{,}000 + 95{,}285 = 1{,}216{,}285\,\text{lb}$$

The total density of borrow pit soil is known to be 105 pcf.

$$\text{total density of borrow pit soil} = M/V = (M_W + M_S)/V = 105\,\text{pcf}$$

Insert known values for M_S and M_W

$$M/V = (M_W + M_S)/V$$

$$(95{,}285 + 1{,}121{,}000)/V = 105\,\text{pcf}$$

Hence

$$V = 11{,}583.7\,\text{cu ft}$$

The contractor needs to extract 11,583.7 cu ft of soil from the borrow pit. The borrow pit soil comes with 95,285 lb of water.

The compacted soil should have 117,705 lb of water. (This was calculated above in Step 3.) Hence water needs to be added to the borrow pit soil. The amount of water needed to be added to the borrow pit soil is calculated as

$$117{,}705 - 95{,}285 = 22{,}420\,\text{lb}$$

The weight of water is usually converted to gallons. One gallon is equal to 8.34 lb.

$$1\,\text{gal} = 8.34\,\text{lb}$$

So the amount of water that needs to be added to the soil is

$$\frac{22{,}420}{8.34} = 2{,}688\,\text{gal}$$

2.6.2 Summary of Steps for Borrow Pit Problems

STEP 1: Obtain all the requirements for compacted soil.

STEP 2: Find M_S or the mass of solids in the compacted soil. This is the mass of solids that needs to be obtained from the borrow pit.

STEP 3: Find the information about the borrow pit. Usually the moisture content in the borrow pit and total density of the borrow pit can be easily obtained.

STEP 4: The contractor needs to obtain M_S of soil from the borrow pit.

STEP 5: Find total volume of soil that needs to be removed in order to obtain M_S mass of solids.

STEP 6: Find M_W of the borrow pit (mass of water that comes along with soil).

STEP 7: Find M_W (mass of water in compacted soil).

STEP 8: The difference between the mass of the water in the borrow pit and the mass of the water in the compacted soil is the amount of water that needs to be added.

3

Shallow Foundation Fundamentals

3.1 Introduction

Shallow foundations are the very first choice of foundation engineers. They are, first and foremost, cheap and easy to construct compared to other alternatives, such as pile foundations and mat foundations. Shallow foundations need to be designed for bearing capacity and excessive settlement. There are a number of methods available to compute the bearing capacity of shallow foundations. The Terzaghi bearing capacity equation, which was developed by Karl Terzaghi, is still widely used.

3.2 Buildings

Shallow foundations are widely used to support buildings. In some cases, doubt is cast upon the ability of a shallow foundation to carry necessary loads due to loose soil underneath. In such situations, engineers use piles to support buildings. Shallow foundations are much more cost-effective than piles. Building footings are subjected to wind, earthquake forces, and bending moments. See Fig. 3.1.

3.2.1 Buildings with Basements

Footings in buildings with basements need to be designed for lateral soil pressure as well as vertical soil pressure. In most cases, the building frame has to support forces due to soil. See Fig. 3.2.

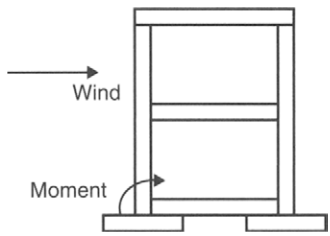


Figure 3.1 Building footings are subject to both vertical and lateral forces

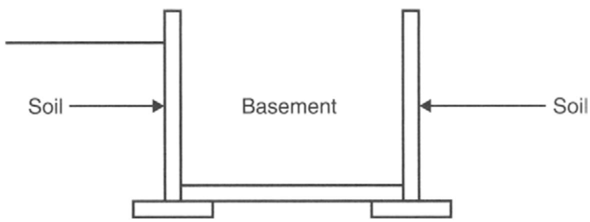


Figure 3.2 Building with basement

3.3 Bridges

Bridges consist of abutments and piers. Due to large loads, many engineers prefer to use piles for bridge abutments and piers. Nevertheless, if the site conditions are suitable, shallow foundations can be used for bridges. See Fig. 3.3.

Shallow footings in bridge abutments can be subjected to large lateral loads due to approach fill, in addition to vertical loads.

Scour is another problem that needs to be overcome in bridges. Scour is the process of erosion due to water. Shallow foundations could fail if scouring is not noticed and corrected. See Fig. 3.4.

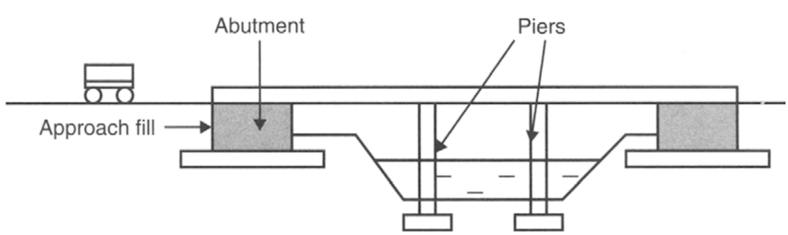


Figure 3.3 Lateral forces in a bridge abutment due to approach fill

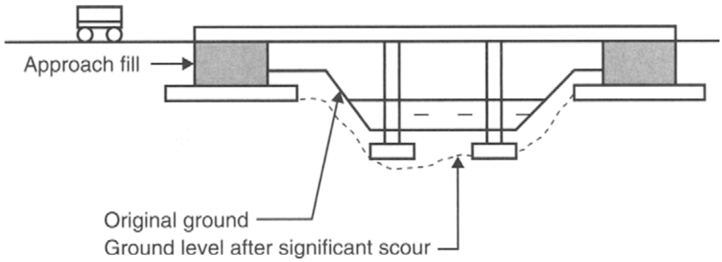


Figure 3.4 Effect of scour (Erosion of soil due to water)

Shallow foundations are vulnerable to the effect of scouring, and for this reason many bridge engineers prefer piles for bridge abutments and piers. See Fig. 3.5.

Scouring is not a problem for bridges over highways and roadways. See Figs. 3.6 and 3.7.

Usually due to scour, many bridge engineers prefer piles over shallow foundations. See Fig. 3.8.

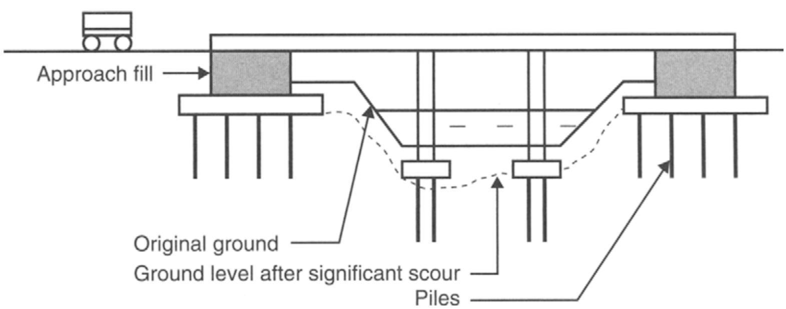


Figure 3.5 Use of piles to counter scour

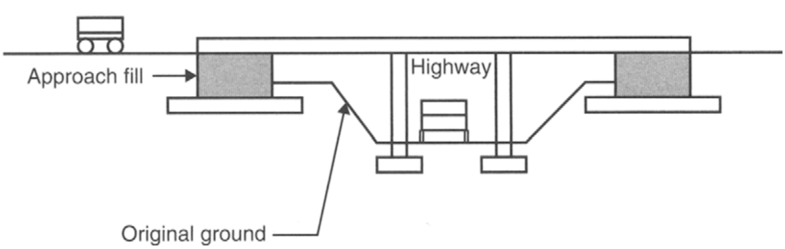


Figure 3.6 Bridge over a highway

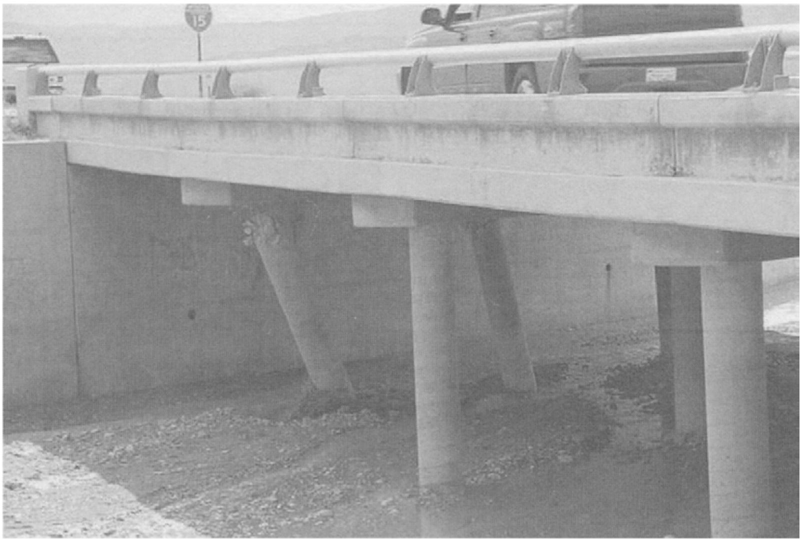


Figure 3.7 Pier failure due to scour. (*Source*: http://www.dot.ca.gov/)

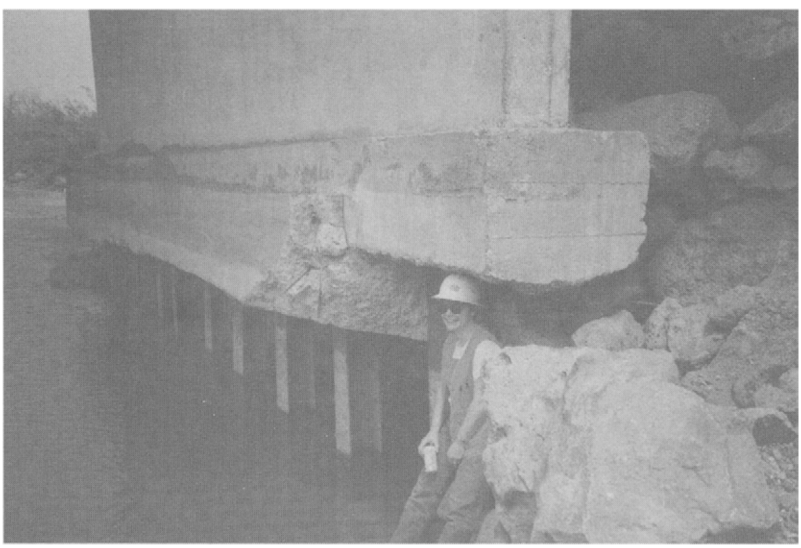


Figure 3.8 Thanks to piles, bridge foundation did not collapse. (*Source*: http://www.dot.ca.gov/)

3.4 Frost Depth

Shallow foundations need to be placed below frost level. During wintertime, the soil at the surface is frozen. During the summer, the frozen soil melts again. This freezing and thawing of soil generates a change in volume. If the footing is placed below the frost depth, the freezing and melting of the soil will generate upward forces and eventually cause cracking in concrete. See Figs. 3.9 and 3.10.

During the summer, the water in the soil melts and the footing settles. For this reason, all footings should be placed below the frost depth in that region. See Fig. 3.11.

The frost depth is dependant upon the region. The frost depth in Siberia and some parts of Canada could be as much as 7 ft, while in places like New York it is not more than 4 ft. Since there is no frost in tropical countries, frost depth is not an issue.

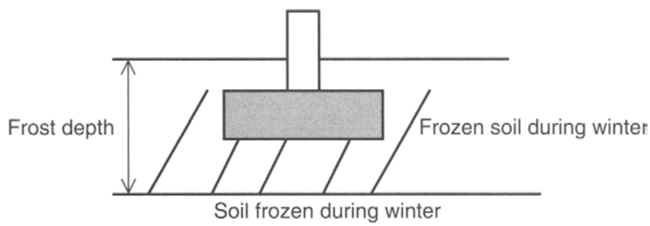


Figure 3.9 Footing on frozen soil

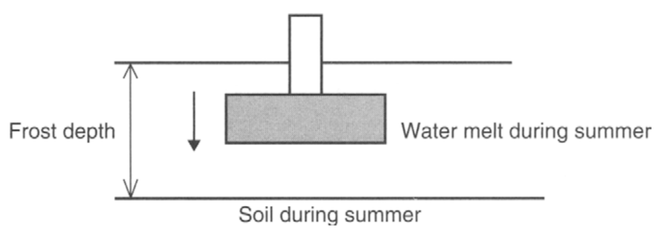


Figure 3.10 Freezing and thawing of soil

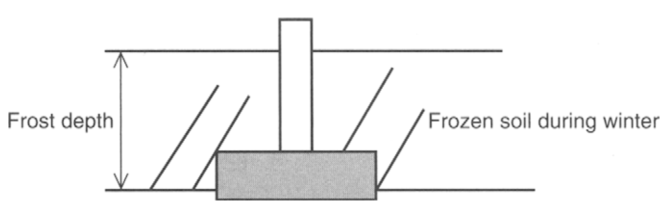


Figure 3.11 Footing placed below the frost depth

4

Bearing Capacity: Rules of Thumb

4.1 Introduction

Many engineers use the bearing capacity equations developed by Terzaghi, Hansen, Meyerhoff, and Vesic to find the bearing capacity of foundations. On the other hand, there are rule of thumb methods that have been used for many years by engineers. In this chapter some of the rule of thumb methods to compute bearing capacity will be presented.

4.2 Bearing Capacity in Medium to Coarse Sands

As mentioned in Chapter 1, the strength of sandy soils is dependent upon the friction angle. The bearing capacity in coarse to medium sands can be obtained using the average SPT (N) value. This can be known in standard units or SI units. For standard or fps (foot pound second) units, (where ksf refers to 1,000 pounds per square foot):

- Bearing capacity of coarse to medium sands (allowable) = 0.2 N_{average} (ksf) (not to exceed 12 ksf).

And for SI units:

- Bearing capacity of coarse to medium sands (allowable) = 9.6 N_{average} (kPa) (not to exceed 575 kPa).

The above equation can be used by the following procedure.

STEP 1: Find the average SPT (N) value below the bottom of footing to a depth equal to width of the footing.
STEP 2: If the soil within this range is medium to coarse sand, the above rule of thumb can be used.

If the average SPT (N) value is less than 10, the soil should be compacted.

Design Example 4.1

A shallow foundation is placed on coarse to medium sand. The SPT (N) values below the footing are as shown in Fig. 4.1. The footing is 2 m × 2 m. The bottom of the footing is 1.5 m below the ground surface.
Find the allowable load bearing capacity of the footing.

Solution

STEP 1: Find the average SPT (N) value below the footing to a depth that is equal to the width of the footing.

$$\text{average SPT}(N) = (10 + 12 + 15 + 9 + 11 + 10)/6$$
$$\text{average SPT}(N) = 11$$

STEP 2: Calculate the allowable bearing capacity.

$$\begin{aligned} 9.6 \times \text{SPT}(N)\,\text{kPa} &= 9.6 \times 11\,\text{kPa} \\ &= 105\,\text{kPa} \end{aligned}$$

$$\begin{aligned} \text{total allowable load on the footing} &= \text{allowable bearing capacity} \times \\ &\quad\ \text{area of the footing} \\ &= 105 \times (2 \times 2)\,\text{kN} \\ &= 420\,\text{kN}\ (94\,\text{kip}) \end{aligned}$$

where

kip = 1,000 lb

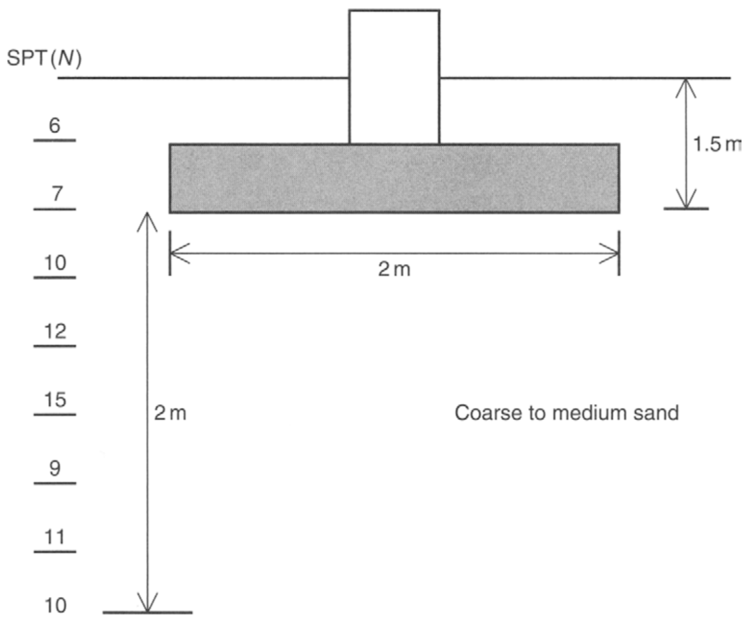


Figure 4.1 Load bearing capacity of footings

4.3 Bearing Capacity in Fine Sands

Same equation can be used for fine sands, but restricted to a lower maximum value. For fps units:

$$\text{bearing capacity (allowable)} = 0.2\,N_{\text{average}}(\text{ksf}) \quad \text{(not to exceed 8 ksf)}$$

And for SI units:

$$\text{bearing capacity (allowable)} = 9.6\,N_{\text{average}}(\text{kPa}) \quad \text{(not to exceed 380 kPa)}$$

5

Bearing Capacity Computation

Terzaghi theorized that the soil underneath the footing would generate a triangle of pressure as shown in Fig. 5.1. Terzaghi considered the angle between the bottom of the footing and the pressure triangle to be as shown in Fig. 5.1. The bearing capacity of the soil is developed in a footing due to three properties of soil.

1. Cohesion.
2. Friction.
3. Density of soil.

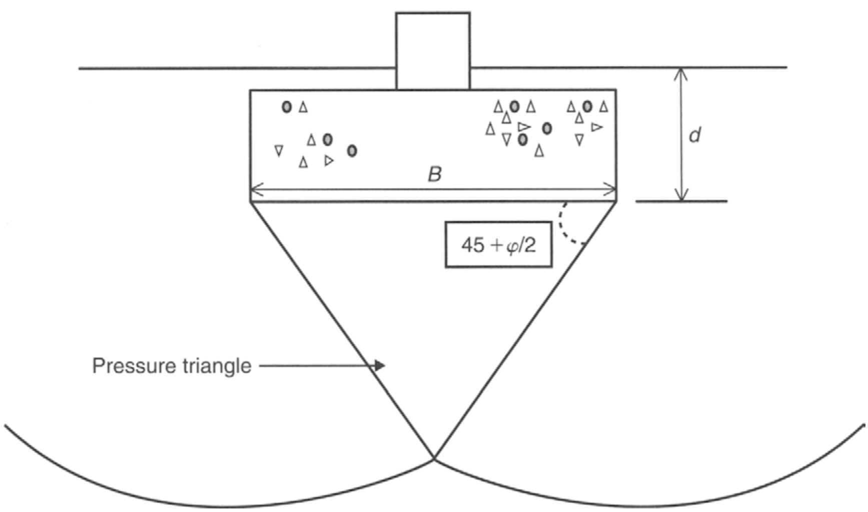


Figure 5.1 Bearing capacity computation

5.1 Terms Used in the Terzaghi Bearing Capacity Equation

The terms used in the Terzaghi bearing capacity equation are the ultimate bearing capacity, the cohesion term, the surcharge term, and the density term.

The ultimate bearing capacity (q_{ult}) of a foundation is the load beyond which a foundation would fail and no longer be useful.

Cohesion (c) is a chemical process. Clay particles tend to adhere to each other due to electrical charges present in clay particles.

Unlike cohesion, friction is a physical process. The higher the friction between particles, the higher the capacity of the soil to carry footing loads. Usually, sands and silts inherit friction. For all practical purposes, clays are considered to be friction-free. The friction of a soil is represented by the friction angle φ, where B is the width of the footing, and the angle of the soil pressure triangle is given by

$$45 + \varphi/2$$

The general Terzaghi bearing capacity equation is given below.

$$q_{\text{ult}} = \underbrace{(c \times N_c \times s_c)}_{\text{Cohesion term}} + \underbrace{(q \times N_q)}_{\text{Surcharge term}} + \underbrace{(0.5 \times B \times N_\gamma \times \gamma \times s_\gamma)}_{\text{Density term}}$$

5.2 Description of Terms in the Terzaghi Bearing Capacity Equation

The term q_{ult} represents the ultimate bearing capacity of the foundation (in tons per square foot, tsf, or pounds per square foot, psf).

5.2.1 Cohesion Term

The cohesion term

$$c \times N_c \times s_c$$

represents the strength of the soil due to cohesion, where

c = cohesion of the soil
N_c = Terzaghi bearing capacity factor (obtained from Table 5.1)
s_c = shape factor (obtained from Table 5.2)

Table 5.1 Terzaghi bearing capacity factors

φ Friction angle	N_c	N_q	N_γ
0	5.7	1.0	0.0
5	7.3	1.6	0.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5.0
25	25.1	12.7	9.7
30	37.2	22.5	19.7
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5
50	347.5	415.1	415.1

Table 5.2 Shape factors for Terzaghi bearing capacity equation

	s_c	s_γ
Square footings	1.3	0.8
Strip footings (wall)	1.0	1.0
Round footings	1.3	0.6

5.2.2 Surcharge Term

The surcharge term

$$q \times N_q$$

represents the bearing capacity strength developed due to surcharge. This term deserves to be explained further.

The surcharge load is the pressure exerted due to soil above the bottom of the footing. See Fig. 5.2. The effective stress is given as

$$q = \gamma \times d$$

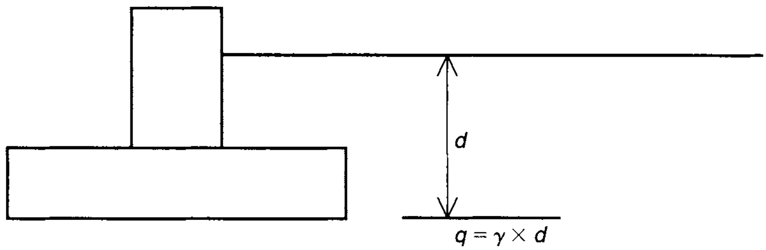


Figure 5.2 Effective stress at bottom of footing

where

q = the effective stress at bottom of footing
d = distance from ground surface to the bottom of the footing
γ = effective density of soil

Considering again the surcharge term

$$q \times N_q$$

where

N_q = Terzaghi bearing capacity factor obtained from Table 5.1

The bearing capacity factor, N_q, depends upon the friction angle of the soil, φ.

For example, if one places the bottom of the footing deeper, the d term in the equation would increase. Hence the bearing capacity of the footing would also increase. Placing the footing deeper is a good method to increase the bearing capacity of a footing. This may not be a good idea when softer soils are present at deeper elevations.

5.2.3 Density Term

The density term

$$0.5 \times B \times N_\gamma \times \gamma \times s_\gamma$$

represents the strength due to the density of soil, where

B = width of the shorter dimension of the footing
N_γ = Terzaghi bearing capacity factor obtained from Table 5.1, dependent on the friction angle of soil, φ
γ = effective density of soil
s_γ = shape factor obtained from Table 5.2.

If the soil has a higher density, the bearing capacity of the soil would be higher.

5.3 Discussion of the Terzaghi Bearing Capacity Equation

It is important to see how each term in the Terzaghi bearing capacity equation impacts the bearing capacity of a footing. See Fig. 5.3.

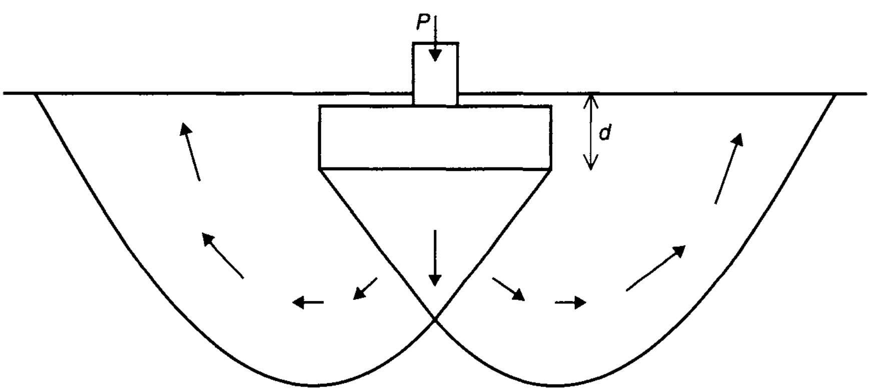


Figure 5.3 Movement of soil below a shallow footing at failure

When the footing load (P) is increased, the pressure triangle that is below the footing would be pressed down. The soil on the sides would get pushed upward. At failure, soil from the sides of the footing would be heaved. If there is a larger surcharge, or the dimension d increases, the soil will be locked in and the bearing capacity would increase. It is possible to increase the bearing capacity of a footing by increasing d.

The bearing capacity increase due to surcharge (q) is given by the second term (qN_q) as discussed earlier.

An increase of bearing capacity when a layer of thickness X is added is shown in Fig. 5.4.

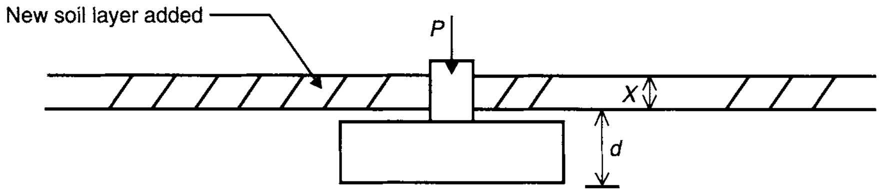


Figure 5.4 New soil layer added to increase the bearing capacity

In Fig. 5.4, an additional soil layer of thickness X is added. The new ultimate bearing capacity will be given by the following equation

$$q_{\text{ult}} = (cN_c \times s_c) + (q \times N_q) + (0.5 \times B \times N_\gamma \times \gamma \times s_\gamma)$$

The first and third terms will not undergo any change due to the addition of the new soil layer. The new surcharge q in the second term will be

$$q = \gamma (d + X)$$

A foundation engineer can increase the bearing capacity of a footing by adding more surcharge as shown above.

5.3.1 Effect of Density

The effect due to the density of soil is represented by the third term

$$0.5 \times B \times N_\gamma \times \gamma \times s_\gamma$$

The higher the density of soil, the higher will be the ultimate bearing capacity. When γ increases, the ultimate bearing capacity also increases.

5.3.2 Effect of Friction Angle φ

One may wonder, where is the effect of the friction angle in the Terzaghi bearing capacity equation? All of the bearing capacity coefficients (N_c, N_q, and N_γ) are obtained using the friction angle.

The higher the friction angle, the higher will be the bearing capacity factors. (See Table 5.1.) Hence, the effect of the friction angle is indirectly incorporated into the Terzaghi bearing capacity coefficients.

5.4 Bearing Capacity in Sandy Soil

Cohesion in sandy soils and nonplastic silts is considered to be zero. The following example shows the computation of bearing capacity in sandy soils.

Design Example 5.1

This example concerns a column footing in a homogeneous sand layer. Find the ultimate bearing capacity of a (1.5 m × 1.5 m) square footing placed in a sand layer. The density of the soil is found to be 18.1 kN/m3 and the friction angle is 30°. See Fig. 5.5.

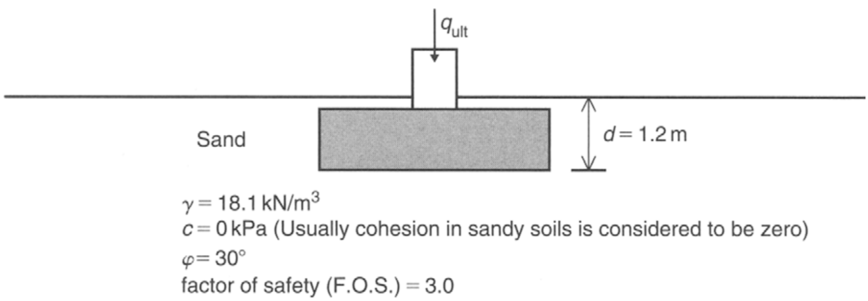


Figure 5.5 Column footing in a homogeneous sand layer

Solution

$$q_{\text{ult}} = (cN_c \times s_c) + (q \times N_q) + (0.5 \times B \times N_\gamma \times \gamma \times s_\gamma)$$

STEP 1: Find the Terzaghi bearing capacity factors from Table 5.1, using $\varphi = 30°$.

$N_c = 37.2$

$N_q = 22.5$

$N_\gamma = 19.7$

STEP 2: Find the shape factors using Table 5.2.
For a square footing $s_c = 1.3$ and $s_\gamma = 0.8$

STEP 3: Find the surcharge (q).

$$q = \gamma \times d = 18.1 \times 1.2 = 21.7 \text{ kPa}$$

STEP 4: Apply the Terzaghi bearing capacity equation.

$$q_{\text{ult}} = (cN_c \times s_c) + (q \times N_q) + (0.5 \times B \times N_\gamma \times \gamma \times s_\gamma)$$
$$q_{\text{ult}} = (0 \times 37.2 \times 1.3) + (21.7 \times 22.5) + (0.5 \times 1.5 \times 19.7 \times 18.1 \times 0.8)$$
$$q_{\text{ult}} = 702.2 \text{ kPa}$$

where

allowable bearing capacity ($q_{\text{allowable}}$) $= q_{\text{ult}}/\text{F.O.S.} = q_{\text{ult}}/3.0 =$ 234.1 kPa
F.O.S. = factor of safety

The total load ($Q_{\text{allowable}}$) that could safely be placed on the footing is calculated as

$$Q_{\text{allowable}} = q_{\text{allowable}} \times \text{area of the footing}$$
$$= q_{\text{allowable}} \times (1.5 \times 1.5) = 526 \text{ kN}$$

In this example, we computed the bearing capacity of a column footing or a square footing placed in sandy soil. In the next example, the computation of the bearing capacity of a wall footing placed in sandy soil is shown.

Design Example 5.2

This example explores a wall footing in a homogeneous sand layer. Find the ultimate bearing capacity of a 1.5 m wide wall (strip) footing placed in sandy soil. The density of the soil is found to be 18.1 kN/m^3 and the friction angle is 20°. See Fig. 5.6.

$\gamma = 18.1 \text{ kN/m}^3$

$c = 0 \text{ kPa}$

$\varphi = 20°$

factor of safety (F.O.S.) = 3.0

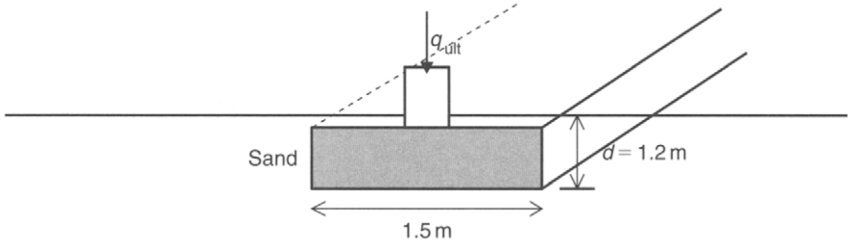


Figure 5.6 Strip footing

Solution

$$q_{ult} = (cN_c \times s_c) + (q \times N_q) + (0.5 \times B \times N_\gamma \times \gamma \times s_\gamma)$$

STEP 1: Find the bearing capacity factors from Table 5.1.

$N_c = 17.7$

$N_q = 7.4$

$N_\gamma = 5.0$

STEP 2: Find the shape factors using Table 5.2.
For a wall footing, $s_c = 1.0$ and $s_\gamma = 1.0$.
STEP 3: Find the surcharge (q).

$$q = \gamma \times d = 18.1 \times 1.2 = 21.7 \text{ kPa}$$

STEP 4: Apply the Terzaghi bearing capacity equation.

$$q_{ult} = cN_c \times s_c + q \times N_q + 0.5 \times B \times N_\gamma \times \gamma \times s_\gamma$$

$$q_{ult} = 0 \times 17.7 \times 1.0 + 21.7 \times 7.4 + 0.5 \times 1.5 \times 5.0 \times 18.1 \times 1.0$$

$$q_{ult} = 228.5 \text{ kPa}$$

$$\text{allowable bearing capacity}(q_{allowable}) = q_{ult}/\text{F.O.S.} = q_{ult}/3.0$$
$$= 76.2 \text{ kPa}$$

The total load that could safely be placed on the footing is calculated as

$$Q_{allowable} = q_{allowable} \times \text{area of the footing} = q_{allowable} \times (1.5 \times 1.0)$$
$$= 114.2 \text{ kN/linear meter of footing}$$

Note that in the case of wall footings, the area of the footing is taken as the width × 1 m, or the capacity of the footing is given in terms of kN per 1 m length of the wall footing.

5.5 Bearing Capacity in Clay

In clay soils and plastic silts, the cohesion term plays a major role in bearing capacity. The friction angle is considered to be zero for clays and plastic silts.

Design Example 5.3

This example explores a column footing in a homogeneous clay layer. Find the ultimate bearing capacity of a 1.2 m × 1.2 m square footing placed in a clay layer. The density of the soil is found to be 17.7 kN/m^3 and the cohesion was found to be 20 kPa. See Fig. 5.7.

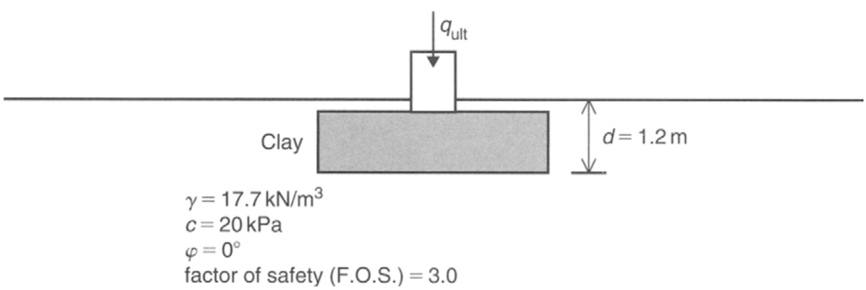


Figure 5.7 Column footing in a homogeneous clay layer

Solution

$$q_{\text{ult}} = (cN_c \times s_c) + (q \times N_q) + (0.5 \times B \times N_\gamma \times \gamma \times s_\gamma)$$

STEP 1: Find the Terzaghi bearing capacity factors from Table 5.1.
For clay soils, the friction angle (φ) is considered to be zero.

$N_c = 5.7$

$N_q = 1.0$

$N_\gamma = 0.0$

STEP 2: Find the shape factors using Table 5.2.
For a square footing $s_c = 1.3$ and $s_\gamma = 0.8$.
STEP 3: Find the surcharge (q).

$$q = \gamma \times d = 17.7 \times 1.2 = 21.2 \text{ kPa}$$

STEP 4: Apply the Terzaghi bearing capacity equation.

$$q_{\text{ult}} = (cN_c \times s_c) + (q \times N_q) + (0.5 \times B \times N_\gamma \times \gamma \times s_\gamma)$$
$$q_{\text{ult}} = 20 \times 5.7 \times 1.3 + 21.2 \times 1.0 + 0$$
$$q_{\text{ult}} = 169.4 \text{ kPa}$$
$$\text{allowable bearing capacity}(q_{\text{allowable}}) = q_{\text{ult}}/\text{F.O.S.} = q_{\text{ult}}/3.0 = 56.5 \text{ kPa}$$

The total load ($Q_{\text{allowable}}$) that could safely be placed on the footing is found as

$$Q_{\text{allowable}} = q_{\text{allowable}} \times \text{area of the footing}$$
$$Q_{\text{allowable}} = q_{\text{allowable}} \times (1.5 \times 1.5) = 127.1 \text{ kN}$$

Design Example 5.4

A 7 ft tall dike is to be constructed on top of a soft clay layer using sandy material. The dike has to be constructed immediately.

Will the dike undergo bearing failure? An unconfined compressive strength test of the clay sample yielded 700 psf. See Fig. 5.8.

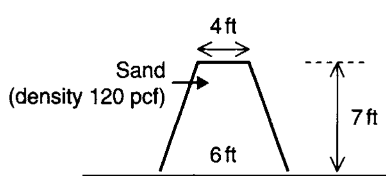


Figure 5.8 Dike built on soft clay

STEP 1: Compute the pressure exerted on the clay soil stratum due to the dike.

$$\text{volume of the dike per linear foot of length} = (4+6)/2 \times 7 \text{ cu ft}$$
$$= 35 \text{ cu ft}$$
$$\text{weight of the dike per linear foot of length} = 35 \times 120 = 4{,}200 \text{ lb}$$
$$\gamma \text{ (soil density)} = 120 \text{ pcf}$$
$$\text{pressure exerted on the clay stratum} = 4{,}200/6 = 700 \text{ psf}$$

STEP 2: Assume a factor of safety of 2.5.
Hence, the required bearing capacity of the soft clay layer is

$$2.5 \times 700 \text{ psf} = 1{,}750 \text{ psf}$$

STEP 3: Determine the bearing capacity of clay by using the Terzaghi bearing capacity equation.

$$\text{bearing capacity} = (c \times N_c) + (q \times N_q) + (0.5 \times \gamma \times N_\gamma \times B)$$

For clay soils, $N_q = 1$, and $N_\gamma = 0$. Hence, the bearing capacity reduces to

$$c \times N_c + q \times N_q$$

where

q = overburden soil on the bearing stratum

Since there is no overburden soil on the bearing stratum, $q = 0$. Hence, the bearing capacity of the clay stratum reduces to

$$c \times N_c$$

$N_c = 5.7$ from Table 5.1 for friction angle zero. The bearing capacity of the clay stratum can be calculated as

$$5.7 \times c$$

where

c = cohesion parameter

STEP 4: Find the cohesion parameter of the clay layer.

The cohesion of the clay layer can be obtained by conducting an unconfined compressive strength test.

The load due to the dike is applied immediately, and there is not enough time for the clay stratum to consolidate. The load is applied rapidly during an unconfined compressive strength test. For the above scenario, an unconfined compressive strength test is appropriate.

STEP 5: The unconfined compressive strength test yielded a value of 700 psf. See Fig. 5.9.

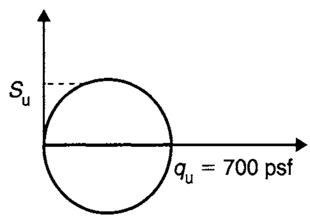


Figure 5.9 Mohr's circle

Hence the unconfined compressive strength is given as

$$q_u = 700\,\text{psf}$$

and the shear strength is

$$S_u = c(\text{cohesion})$$

The shear strength of clay soil is therefore

$$S_u = q_u/2 = 700/2 = 350\,\text{psf}$$

See Fig. 5.9.

Since $\varphi = 0$ for undrained clay soils, shear strength is solely dependent on cohesion. (However, note that drained clays have a φ value greater than zero.) Hence

$$S_u = c + \sigma \tan \varphi = c + 0 = 350\,\text{psf}$$

The bearing capacity of the clay layer is therefore

$$5.7c = 5.7 \times 350 = 1{,}995\,\text{psf (from Step 3)}$$

The bearing capacity that is required with a factor of safety of 2.5 is therefore 1,750 psf (from Step 2). Hence the soft clay layer will be safe against bearing capacity failure.

5.6 Bearing Capacity in Layered Soil

In reality, most sites contain more than one layer of soil. In such situations, average values of friction angle and cohesion need to be obtained. This procedure can better be explained with examples. See Fig. 5.10.

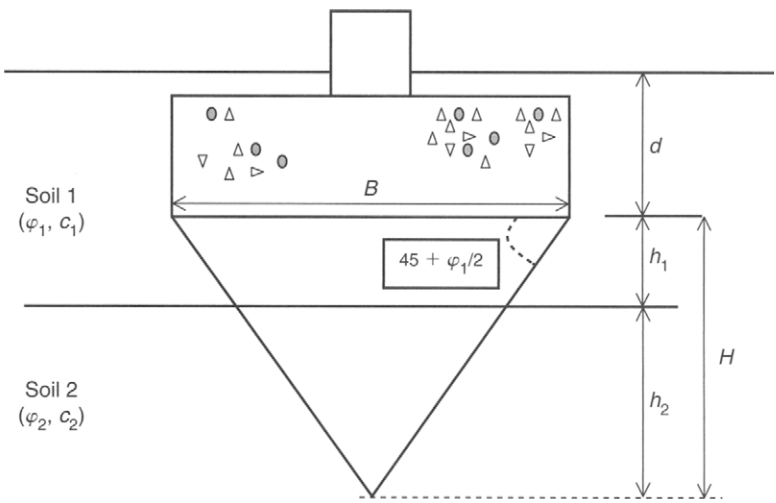


Figure 5.10 Bearing capacity in layered soil

The average friction angle and cohesion within the pressure triangle needs to be obtained. The pressure triangle is drawn as shown in Fig. 5.10 using the friction angle of the top layer (φ_1).

The average cohesion within the pressure triangle is given by

$$c_{ave} = (c_1 \times h_1 + c_2 \times h_2)/H$$

The variables c_1 and c_2 are the cohesion values of first and second layers, respectively. h_1 and h_2 are the thickness of the soil layers measured as shown in Fig. 5.10. H can be found using trigonometry.

$$H = B/2 \times \tan(45 + \varphi_1/2)$$

Note that H is equal to $(h_1 + h_2)$.

Notice that the pressure triangle is found using φ_1, the friction angle of the top layer. A similar procedure could be used to find the average friction angle within the pressure triangle.

$$\varphi_{\text{ave}} = (\varphi_1 \times h_1 + \varphi_2 \times h_2)/H$$

The average cohesion (c_{ave}) and the average friction angle (φ_{ave}) are used in the Terzaghi bearing capacity equation.

Design Example 5.5

This example explores a column footing in soil layered with clay and sand. Find the ultimate bearing capacity of a 2 m × 2 m square footing placed in layered soils as shown in Fig. 5.11. The density of the soil is found to be 17.7 kN/m^3 for all soils, and the cohesion of clay was found to be 28 kPa. The friction angle of sand is 20°.

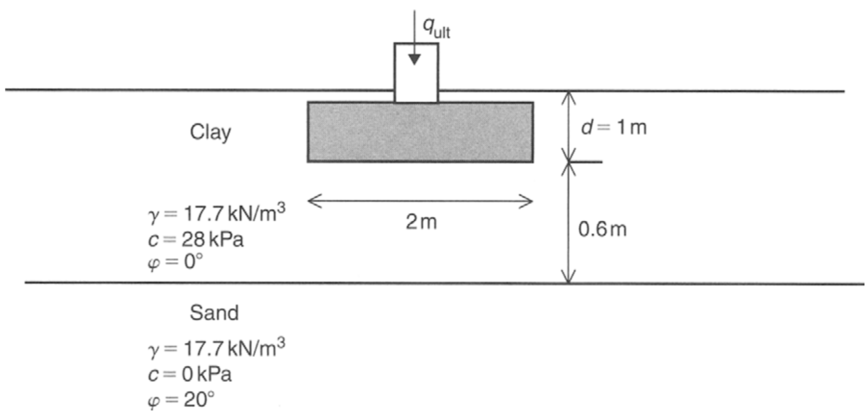


Figure 5.11 Shallow foundation on layered soil

Solution

Refer to the Terzaghi bearing capacity equation

$$q_{\text{ult}} = (cN_c \times s_c) + (q \times N_q) + (0.5 \times B \times N_\gamma \times \gamma \times s_\gamma)$$

Find the average friction angle and cohesion. To find the average friction angle and cohesion, the depth of the pressure triangle needs to be computed.

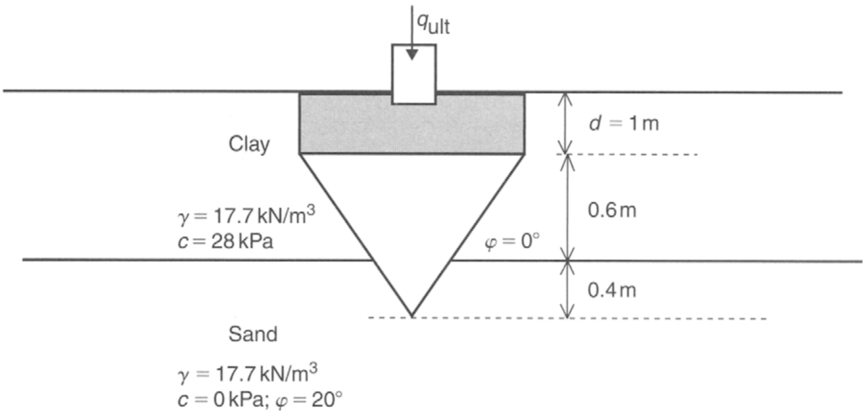


Figure 5.12 Effective depths for the bearing capacity equation

STEP 1: Find the depth of the pressure triangle (H). See Fig. 5.12.

$$H = B/2 \times \tan(45 + \varphi_1/2)$$

where

B = width of the footing = 2 m
φ_1 = friction angle of the top layer = 0°

$$H = 2/2 \times \tan(45 + 0) = 1\,\text{m}$$

The depth h_1 is given to be 0.6 m and h_2 can be found.

$$h_2 = H - h_1 = 1 - 0.6 = 0.4\,\text{m}$$

STEP 2: Find the average friction angle, φ_{ave}.

$$\varphi_{ave} = (\varphi_1 \times h_1 + \varphi_2 \times h_2)/H$$

$$\varphi_{ave} = (0 \times h_1 + 20 \times h_2)/H$$

Since $H = 1.0$ m, $h_1 = 0.6$ m, and $h_2 = 0.4$ m, we can calculate φ_{ave}

$$\varphi_{ave} = (0 + 20° \times 0.4)/1 = 8°$$

STEP 3: Find c_{ave}.

$$c_{\text{ave}} = (c_1 \times h_1 + c_2 \times h_2)/H$$

Since $H = 1.0\,\text{m}, h_1 = 0.6\,\text{m}, h_2 = 0.4\,\text{m}, c_1 = 20\,\text{kPa}$, and $c_2 = 0$, we can calculate c_{ave} as

$$c_{\text{ave}} = (28 \times 0.6 + 0)/1 = 16.8\,\text{kPa}$$

We will use φ_{ave} and c_{ave} in the Terzaghi bearing capacity equation to find the bearing capacity.

STEP 4: Using 8° for φ_{ave}, the Terzaghi bearing capacity factors can be found using Table 5.1.

$N_c = 8.7$
$N_q = 2.3$
$N_\gamma = 0.9$ (using interpolation)

From Table 5.2 for column (square) footings, we find

$s_c = 1.3$
$s_\gamma = 0.8$

STEP 5: Use the Terzaghi bearing capacity equation to obtain q_{ult}. Assume a factor of safety of 3.0 to find $q_{\text{allowable}}$.

$$q_{\text{ult}} = (cN_c \times s_c) + (q \times N_q) + (0.5 \times B \times N_\gamma \times \gamma \times s_\gamma)$$

$$q_{\text{ult}} = 16.8 \times 8.7 \times 1.3 + (17.7 \times 1) \times 2.3 + 0.5 \times 2 \times 0.9 \times 17.7 \times 0.8$$

$$q_{\text{ult}} = 191.1 + 40.7 + 12.7 = 244.5\,\text{kN/m}^2$$

$$q_{\text{allowable}} = 244.5/\text{F.O.S.} = 81.5\,\text{kN/m}^2$$

$$\text{F.O.S.} = 3.0$$

STEP 6: Find the total allowable load on the footing, Q.

$$Q = \text{allowable bearing capacity} \times \text{area of the footing}$$

$$Q = q_{\text{allowable}} \times 4 = 326\,\text{kN}$$

Design Example 5.6

This example explores a column footing on layered soil with two sand layers. Find the ultimate bearing capacity of a 2 m × 2 m square footing placed in layered soils as shown. The density of the soil is found to be 17.7 kN/m^3 for all soils. The friction angles of the sand layers are 15° and 20°, respectively. See Fig. 5.13.

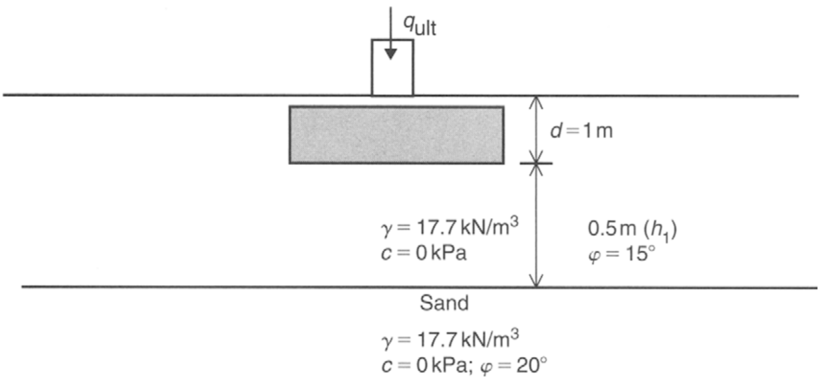


Figure 5.13 Shallow footing in two sand layers

Solution

Refer to the Terzaghi bearing capacity equation

$$q_{ult} = cN_c \times s_c + q \times N_q + 0.5 \times B \times N_\gamma \times \gamma \times s_\gamma$$

STEP 1: Find the depth of the pressure triangle, H. See Fig. 5.14.

$$H = B/2 \times \tan(45 + \varphi_1/2)$$

where

B = width of the footing = 2 m
φ_1 = friction angle of the top layer = 15°

$$H = 2/2 \times \tan(45 + 15/2) = 1.3\,\text{m}$$

h_1 is given to be 0.5 m and h_2 can be found. $h_2 = H - h_1 = 1.3 - 0.5 = 0.8$ m

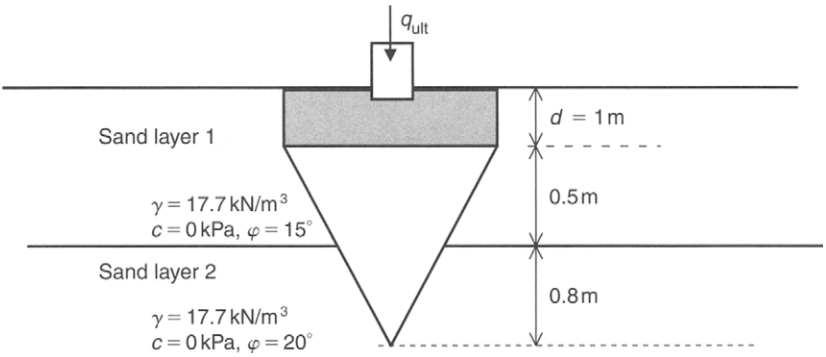


Figure 5.14 Effective depths of two sand layers

STEP 2: Find the average friction angle, φ_{ave}

$$\varphi_{ave} = (\varphi_1 \times h_1 + \varphi_2 \times h_2)/H$$
$$\varphi_{ave} = (15 \times h_1 + 20 \times h_2)/H$$

Since $H = 1.3\,\text{m}$, $h_1 = 0.5\,\text{m}$, and $h_2 = 0.8\,\text{m}$

$$\varphi_{ave} = (15° \times 0.5 + 20° \times 0.8)/1.3 = 18°$$

STEP 3: Find c_{ave}.

$$c_{ave} = 0 \text{ since } c_1 \text{ and } c_2 \text{ are zero}$$

Use φ_{ave} and c_{ave} in the Terzaghi bearing capacity equation to find the bearing capacity.

STEP 4: Using φ_{ave} equal to 18°, the Terzaghi bearing capacity factors can be found.

Using Table 5.1, through interpolation, the following bearing capacity factors are obtained.

$N_c = 15.8$
$N_q = 6.2$
$N_\gamma = 4.0$

From Table 5.2 for column (square) footings

$s_c = 1.3$
$s_\gamma = 0.8$

STEP 5: Use the Terzaghi bearing capacity equation to obtain q_{ult}.

$$q_{\text{ult}} = cN_c \times s_c + q \times N_q + 0.5 \times B \times N_\gamma \times \gamma \times s_\gamma$$

$$q_{\text{ult}} = 0 + (17.7 \times 1) \times 6.2 + 0.5 \times 2 \times 4.0 \times 17.7 \times 0.8$$

$$q_{\text{ult}} = 0 + 109.7 + 24.6 = 134.4\,\text{kN/m}^2$$

Assume a factor of safety of 3.0.

$$q_{\text{allowable}} = 144.4/\text{F.O.S.} = 44.8\ \text{kN/m}^2$$

STEP 6: Find the total allowable load on the footing, Q

$$Q = \text{allowable bearing capacity} \times \text{area of the footing}$$

$$Q = q_{\text{allowable}} \times 4 = 179.2\ \text{kN}$$

What if the pressure triangle is located in one soil layer as shown in Fig. 5.15?

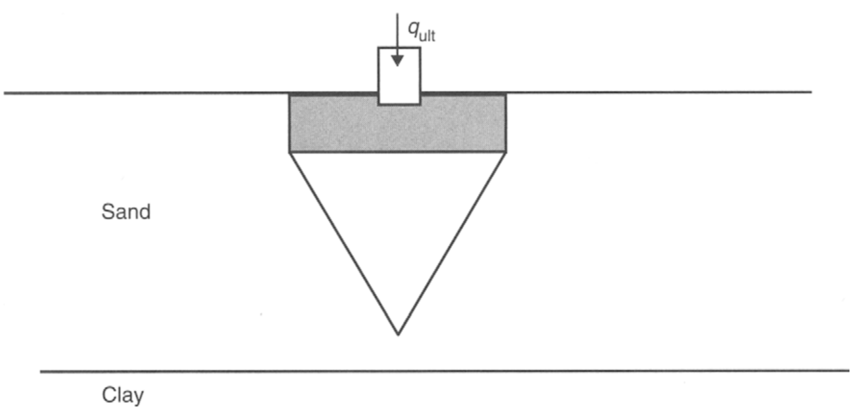


Figure 5.15 Shallow foundation in sand and clay layers

In Fig. 5.15, the effect of clay on the bearing capacity can be reasonably ignored. The designer should still look into the settlement aspects of the footing separately.

In the previous example, a column footing placed on a soil strata consisting two different sand layers was considered. In the next example, a wall footing placed in two different clay layers will be considered.

Design Example 5.7

This example covers a wall footing in soil with two different layers of clay. Find the ultimate bearing capacity of a 2 m wide square wall footing placed in layered soils as shown in Fig. 5.16. The density of the soil is found to be 17.7 kN/m^3 for all soils. The cohesion of the top clay layer was found to be 28 kPa, and the cohesion of the bottom clay layer was found to be 20 kPa.

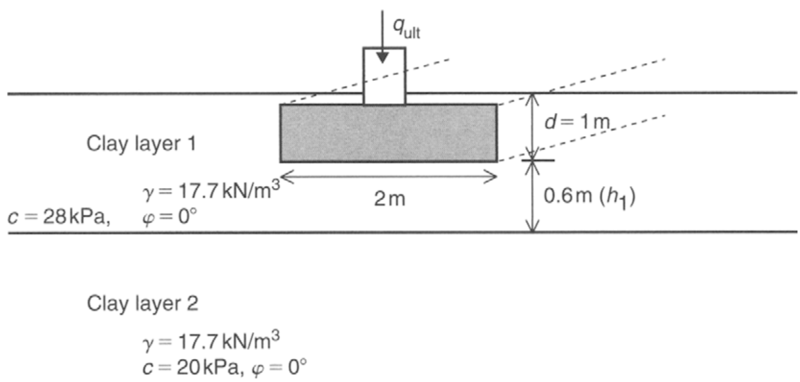


Figure 5.16 Shallow foundation in two clay layers

Solution

Refer to the Terzaghi bearing capacity equation

$$q_{ult} = cN_c \times s_c + q \times N_q + 0.5 \times B \times N_\gamma \times \gamma \times s_\gamma$$

STEP 1: Find the depth of the pressure triangle, H. See Fig. 5.17.

$$H = B/2 \times \tan(45 + \varphi_1/2)$$

where

B = width of the footing = 2 m
φ_1 = friction angle of the top layer = 0°

$$H = 2/2 \times \tan(45 + 0) = 1\text{ m}$$

h_1 is given to be 0.6 m and h_2 can be found

$$h_2 = H - h_1 = 1 - 0.6 = 0.4\text{ m}$$

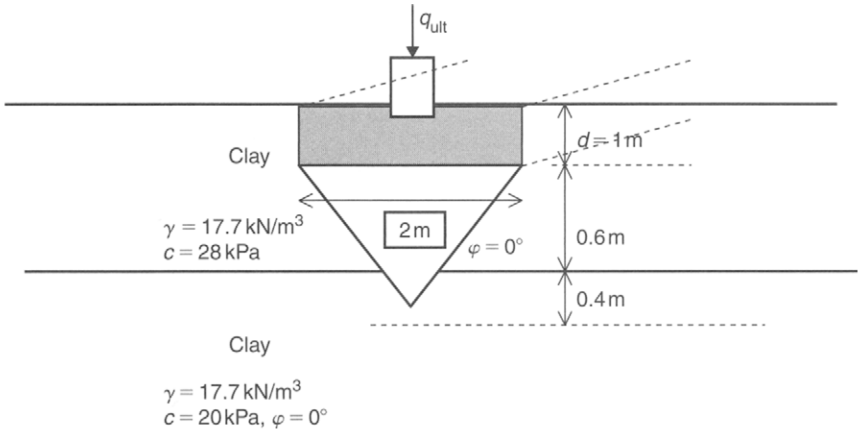


Figure 5.17 Pressure triangle

STEP 2: Find the average friction angle, φ_{ave}

The value of φ_{ave} is zero since both layers are clay.

STEP 3: Find the average cohesion, c_{ave}

$$c_{ave} = (c_1 \times h_1 + c_2 \times h_2)/H$$

Since $H = 1.0\,m$, $h_1 = 0.6\,m$, $h_2 = 0.4\,m$, $c_1 = 28\,kPa$, and $c_2 = 20\,kPa$

$$c_{ave} = (28 \times 0.6 + 20 \times 0.4)/1.0 = 24.8\,kPa$$

Use φ_{ave} and c_{ave} in the Terzaghi bearing capacity equation to find the bearing capacity.

STEP 4: Using φ_{ave} equal to 0°, the Terzaghi bearing capacity factors can be found.

Using Table 5.1,

$N_c = 5.7$

$N_q = 1.0$

$N_\gamma = 0.0$

From Table 5.2 for wall (strip) footings

$s_c = 1.0$

$s_\gamma = 1.0$

STEP 5: Use the Terzaghi bearing capacity equation to obtain the q_{ult}

$$q_{ult} = cN_c \times s_c + q \times N_q + 0.5 \times B \times N_\gamma \times \gamma \times s_\gamma$$
$$q_{ult} = 24.8 \times 5.7 \times 1.0 + (17.7 \times 1) \times 1.0 + 0$$
$$q_{ult} = 141.4 + 17.7 = 159.1\,\text{kN/m}^2$$

Assume a factor of safety of 3.0.

$$q_{allowable} = 159.1/\text{F.O.S.} = 53.0\,\text{kN/m}^2$$

In a wall footing allowable load is given per linear meter of the footing.

$$q_{allowable} = 53.0 \times \text{(width of the footing)}$$
$$= 53.0 \times 2 = 106\,\text{kN per linear meter of the footing}$$

Design Example 5.8

This example considers a strip footing design in sand using SPT (N) values, for the no groundwater case. Find the allowable bearing capacity of the strip foundation shown in the Fig. 5.18. Sand was encountered below the footing and SPT (N) values are as shown.

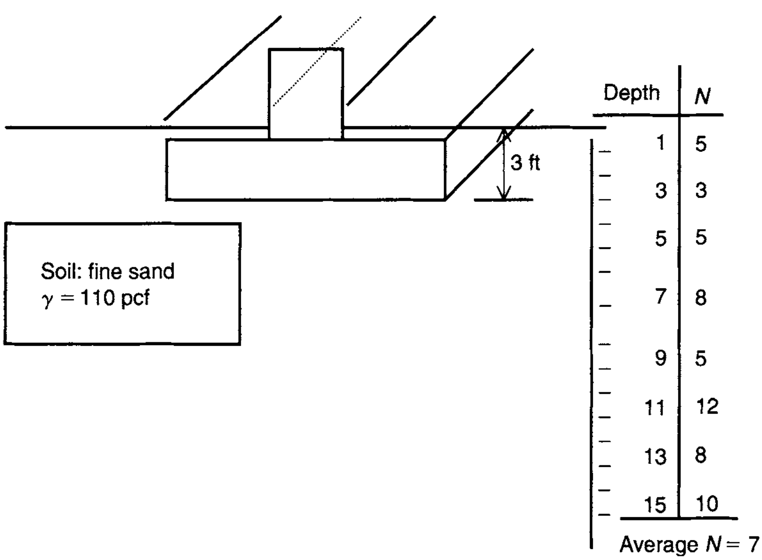


Figure 5.18 Strip footing in sand

Assume a width of the footing to be 4 ft for initial calculations.

Solution

STEP 1: Find the SPT (N) values. These are obtained from Fig. 5.18.
STEP 2: Find the average friction angle, φ_{ave}, and total density.
Use Tables 1.2 and 1.3 to obtain φ_{ave} and total density using the average N value.
STEP 3: In this case $\varphi = 30$ (for fine sand with an average N value of 7 from Table 1.2).
STEP 4: Refer to the Terzaghi bearing capacity equation

$$q_{\text{ult}} = cN_c \times s_c + q \times N_q + 0.5 \times B \times N_\gamma \times \gamma \times s_\gamma$$

where

c = cohesion (zero for sands)

N_c, N_q, and N_γ are the Terzaghi bearing capacity factors

B = width of footing

For $\varphi = 30°$, obtain $N_c = 37.2$, $N_q = 22.5$, $N_\gamma = 19.7$.

$$\gamma = \text{total soil density} = 110\,\text{pcf}$$

Note that above groundwater level, γ' is equal to total soil density, γ. Below groundwater level

$$\gamma' = \gamma - \gamma_{\text{W}}$$

where

γ_{W} = density of water

Next, determine the effective stress, q

$$q = \text{effective stress at bottom of footing} = 110 \times 3 = 330\ \text{lb/ft}^2$$

From Table 5.2 for wall (strip) footings

$s_c = 1$

$s_\gamma = 1$

Substitute all the values into the Terzaghi bearing capacity equation

$$q_{\text{ult}} = cN_c \times s_c + q \times N_q + 0.5 \times B \times N_\gamma \times \gamma \times s_\gamma$$
$$q_{\text{ult}} = 0 + 330 \times 22.5 + 0.5 \times 110 \times 5 \times 19.7 \times 1.0 = 12{,}842 \text{ psf}$$

The allowable bearing capacity, which will be referred to as q_a from this point on, can be determined, using a factor of safety of 3.0, as

$$q_a = q_{\text{ult}}/3.0$$
$$q_{\text{ult}}/3.0 = 12{,}842/3.0 = 4{,}280 \text{ psf}$$

5.7 Bearing Capacity when Groundwater Present

Groundwater reduces the density due to buoyancy. When groundwater is present, the density of the soil needs to be modified. When a footing is loaded, a triangle of stress is developed below the footing. If groundwater is present below the stress triangle, it does not affect the bearing capacity. See Fig. 5.19.

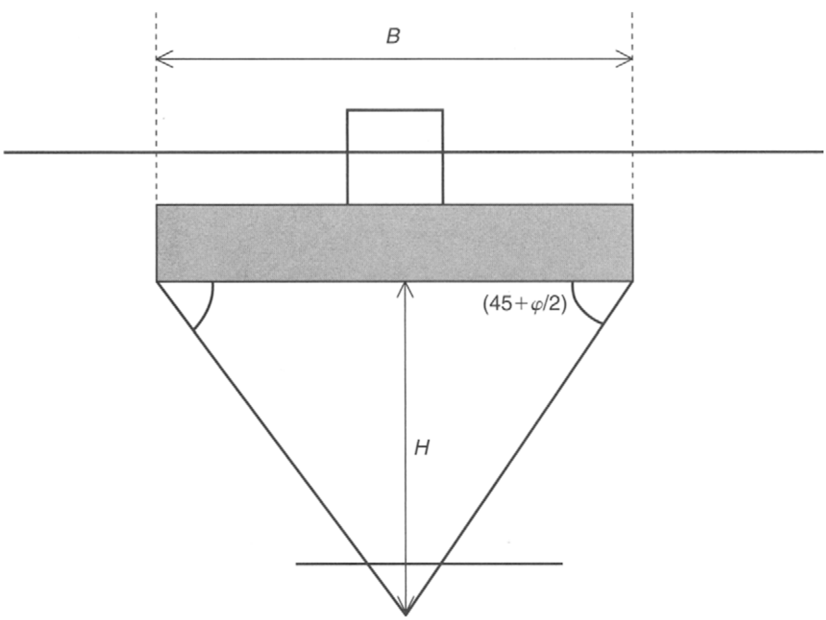


Figure 5.19 Pressure triangle below a footing

It is assumed that the angle between the stress triangle and the footing is

$$45 + \varphi/2$$

Hence from trigonometry we can obtain

$$H = B/2 \times \tan(45 + \varphi/2)$$

where

H = depth of the stress triangle
B = width of the footing

The equivalent density of soil γ_e depends on whether groundwater is present. If groundwater is within the stress triangle, the equivalent density of the soil is used. See Fig. 5.20. The equivalent density of soil γ_e is given by

$$\gamma_e = (2H - d_w)/H^2 \times d_w \times \gamma_{total} + \gamma' \times (H - d_w)^2/H^2$$

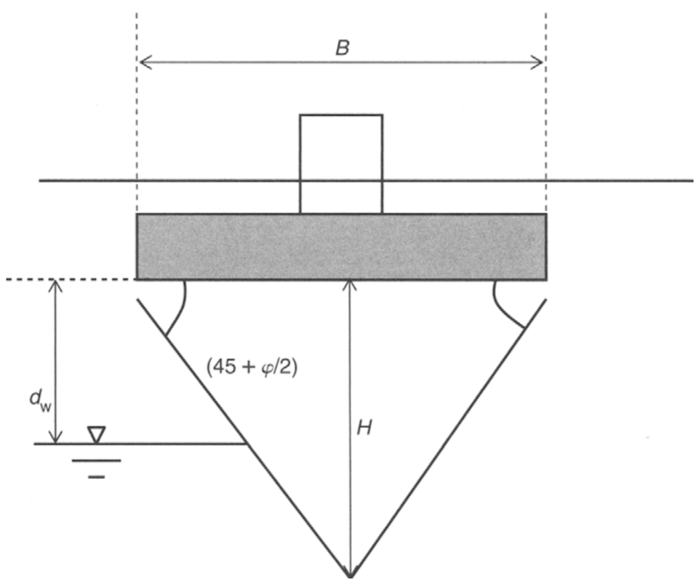


Figure 5.20 Groundwater within the pressure triangle

where

$H = 0.5\,B\tan(45 + \Phi/2)$
d_W = depth to groundwater measured from the bottom of footing
γ' = buoyant density of soil = $\gamma - \gamma_W$
γ_W = density of water

5.8 Groundwater Below the Stress Triangle

If the groundwater level is below the stress triangle ($d_W > H$), the effect due to groundwater is ignored. See Fig. 5.21.

When groundwater is below the stress triangle, γ (total soil density) is used in the bearing capacity equation.

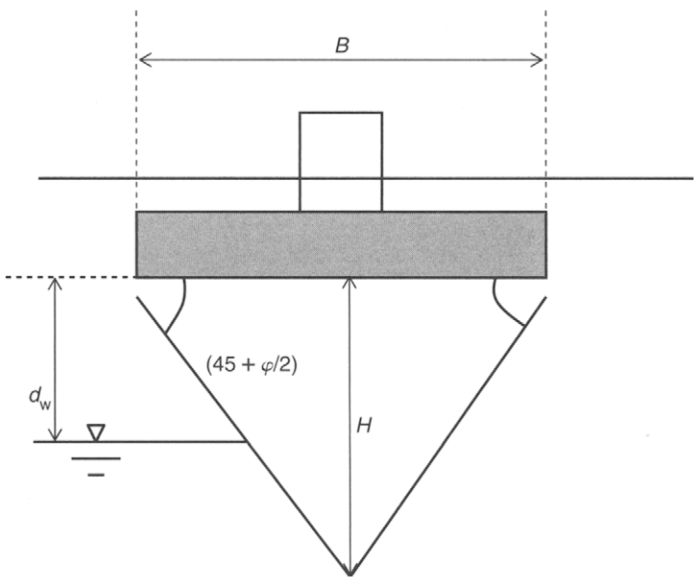


Figure 5.21 Groundwater is below the stress triangle

5.9 Groundwater Above the Bottom of Footing Level

If groundwater is above the bottom of footing, then γ' is used in the bearing capacity equation. See Fig. 5.22.

$$\gamma' = \gamma - \gamma_W$$

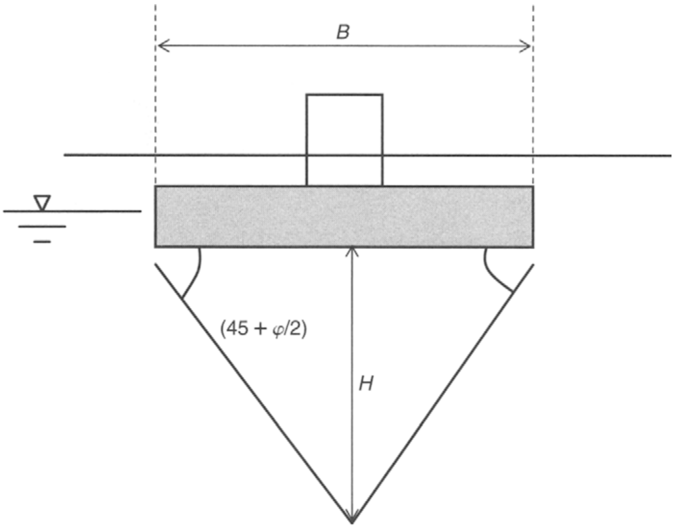


Figure 5.22 Groundwater above bottom of footing

where

γ' = buoyant density of soil
γ = density of soil
γ_W = density of water

5.10 Groundwater at Bottom of Footing Level

When groundwater is at the bottom of the footing level, then d_W becomes zero. In this case, the first term of the equation becomes zero, and the second term becomes γ'.

$$\gamma_e = (2H - d_W)/H^2 \times d_W \times \gamma_{total} + \gamma' \times (H - d_W)^2/H^2$$

$$d_W = 5$$

Hence when the groundwater is at the bottom of the footing level

$$\gamma_e = \gamma'$$

Design Example 5.9

This example considers a shallow foundation design in the case where groundwater is present. The loading given by the structural engineer is

15,000 lb per linear foot of wall. The average SPT (N) value is found to be 7, which relates to a friction angle of 30° for fine sands.

The groundwater is 4 ft below the ground surface. The bottom of the footing is placed 3 ft below the ground level surface to provide protection against frost. Compute the bearing capacity of a 4 ft wide strip footing. See Fig. 5.23.

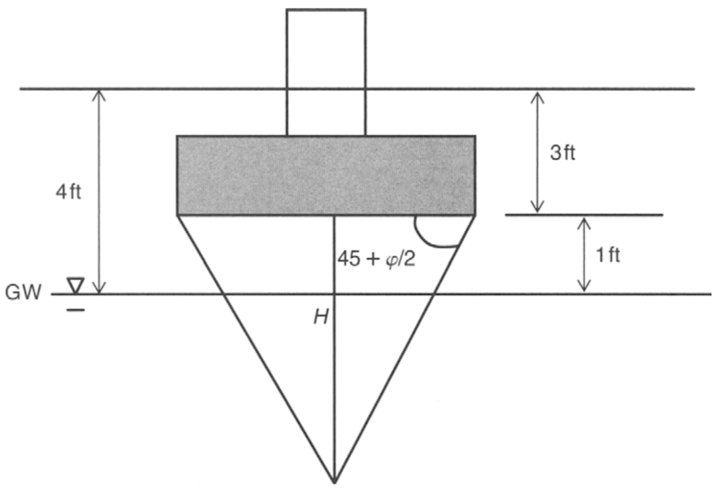


Figure 5.23 Groundwater within the pressure triangle

Solution

STEP 1: Determine the equivalent density of the soil, γ_e.

The width of the footing B is equal to 4 ft. The groundwater in this case is below the bottom of the footing.

$$H = 0.5 \times B \times \tan(45 + \Phi/2)$$

$$= 0.5 \times 4 \times \tan(45 + 30/2)$$

$$= 0.5 \times 4 \times \tan(60)$$

$$= 2 \times 1.73$$

$$H = 3.46$$

$$\gamma_e = (2H - d_w)/H^2 \times d_w \times \gamma_{total} + \gamma' \times (H - d_w)^2/H^2$$

where

d_W = depth to groundwater measured from the bottom of footing
$d_W = 1$ ft

$$\gamma' = 110 - 62.4 = 47.6 \text{ pcf}$$

$$\gamma_e = (2 \times 3.46 - 1)/(3.46)^2 \times 1 \times 110 + 47.6 \times (3.46 - 1)^2/(3.46)^2$$

$$\gamma_e = 54.4 + 24.1 = 78.5 \text{ pcf}$$

STEP 2: Find the terms of the Terzaghi bearing capacity equation.
Refer to the Terzaghi bearing capacity equation

$$q_{\text{ult}} = cN_c s_c + q' N_q + 0.5\gamma' B N_\gamma s_\gamma$$

where

c = cohesion (zero for sands)
$\varphi = 30$ for $N = 7$, for fine sands (Table 1.2)

N_c, N_q, and N_γ are the Terzaghi bearing capacity factors
For $\varphi = 30$, use Table 5.1 to obtain $N_c = 37.2$, $N_q = 22.5$, and $N_\gamma = 19.7$. From Table 5.2, $s_\gamma = 1.0$ for strip footings.

STEP 3: Calculate the terms of the Terzaghi bearing capacity equation.
The first term ($cN_c s_c$) in the Terzaghi bearing capacity equation does not change due to groundwater elevation. Note that for sands, c (cohesion) is zero.

The variable q is defined as the effective stress at the bottom of the footing.

$$q = (110) \times 3$$

$$= 330 \text{ psf}$$

$$qN_q = 330 \times 22.5$$

$$= 7{,}425 \text{ psf}$$

STEP 4: Apply the Terzaghi bearing capacity equation.

$$q_{\text{ult}} = cN_c \times s_c + q \times N_q + 0.5 \times \gamma_e \times B \times N_\gamma \times s_\gamma$$

$$q_{\text{ult}} = 0 + 7{,}425 + 0.5 \times 78.5 \times 4 \times 19.7 \times 1$$
$$= 10{,}518 \text{ psf}$$

allowable bearing capacity = ultimate bearing capacity/factor of safety

$$= 10{,}518/3.0$$
$$= 3{,}506 \text{ psf}$$

Design Example 5.10

This example concerns a strip and column footing design in sand, considering the effects of groundwater. The loading given by the structural engineer happens to be 15,000 lb per linear foot of the wall. The average SPT (N) value is found to be 7, which relates to a friction angle of 30° for fine sands.

The groundwater is 2 ft below the ground surface. The bottom of the footing is placed 3 ft below the ground surface to provide protection against frost. Compute the bearing capacity of a 4 ft wide strip footing. See Fig. 5.24.

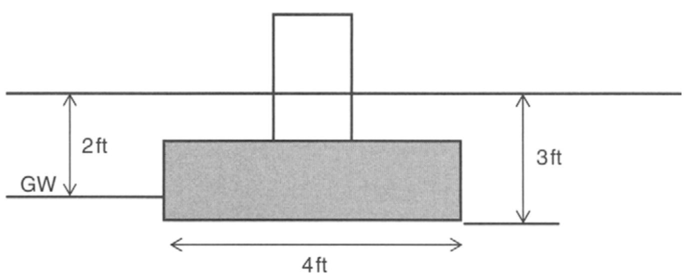


Figure 5.24 Shallow foundation (groundwater effect considered)

Solution

STEP 1: Find γ_e.

The groundwater in this case is above the bottom of footing. Hence

$$\gamma_e = \gamma'$$

STEP 2: Find the terms of the Terzaghi bearing capacity equation.

Refer to the Terzaghi bearing capacity equation

$$q_{\text{ult}} = (cN_c \times s_c) + (q' \times N_q) + (0.5 \times \gamma' \times B \times N_\gamma \times s_\gamma)$$

where

c = cohesion (zero for sands)
$\varphi = 30°$ for $N = 7$, for fine sands, from Table 1.2
N_c, N_q, and N_γ are the Terzaghi bearing capacity factors

For $\varphi = 30°$, obtain $N_c = 37.2$, $N_q = 22.5$, and $N_\gamma = 19.7$.

γ' = buoyant soil density

γ' is taken to be equal to $\gamma' = \gamma - \gamma_W$ where γ_W is the density of water.
STEP 3: Find c_{ave}.

The first term (cN_cs_c) in the Terzaghi bearing capacity equation does not change due to groundwater elevation. Note that for sands c (cohesion) is zero.

The second term ($q'N_q$) will change depending on the elevation of the groundwater. The variable q' is defined as the effective stress at the bottom of the footing.

$$q' = (110) \times 2 + (110 - 62.4) \times 1 = 267.6\ \text{psf}$$
$$q'N_q = 267.6 \times 22.5 = 6{,}021\ \text{psf}$$

The term (110×2) gives the effective stress due to soil above the groundwater level. The total density, γ, is used to compute the effective stress above the groundwater level.

The next term $[(110 - 62.4) \times 1]$ gives the effective stress due to soil below the groundwater table. The buoyant density, γ', is used to compute the effective stress below the groundwater level.

STEP 4: Find the allowable bearing capacity.

γ_e is given by

$$\gamma_e = (2H - d_W)/H^2 \times d_W \times \gamma_{\text{total}} + \gamma' \times (H - d_W)^2/H^2$$

When the groundwater is above the bottom of the footing

$$\gamma_e = \gamma'$$
$$\gamma_e = (110 - 62.4) = 47.6\ \text{psf}$$

Apply the Terzaghi bearing capacity equation.

$$q_{\text{ult}} = (cN_c \times s_c) + (q' \times N_q) + (0.5 \times \gamma' \times B \times N_\gamma \times s_\gamma)$$

$$q_{\text{ult}} = 0 + 6{,}021 + 0.5 \times 47.6 \times 4 \times 19.7 \times 1$$

$$= 7{,}896 \text{ psf}$$

allowable bearing capacity = ultimate bearing capacity/factor of safety

$$= 7{,}896/3.0 = 2{,}632 \text{ psf}$$

5.11 Shallow Foundations in Bridge Abutments

Shallow foundations in bridge abutments undergo heavy lateral loads, in addition to vertical loads, due to the approach fill. See Fig. 5.25.

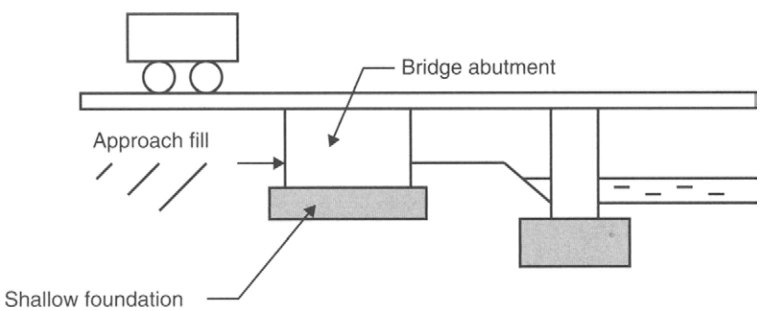


Figure 5.25 Shallow foundation for a bridge abutment

Design Example 5.11

Find the safety factor against lateral forces acting on the foundation shown in Fig. 5.26. The soil and concrete interphase friction angle, δ, is found to be 20°, while the friction angle of soil, φ, is 30°. The density of the soil is 18.0 kN/m^3. The weight of the abutment is 2,000 kN per 1 m length of the abutment.

Solution

STEP 1: Find the lateral earth pressure coefficient (K_a).
See Fig. 5.27.

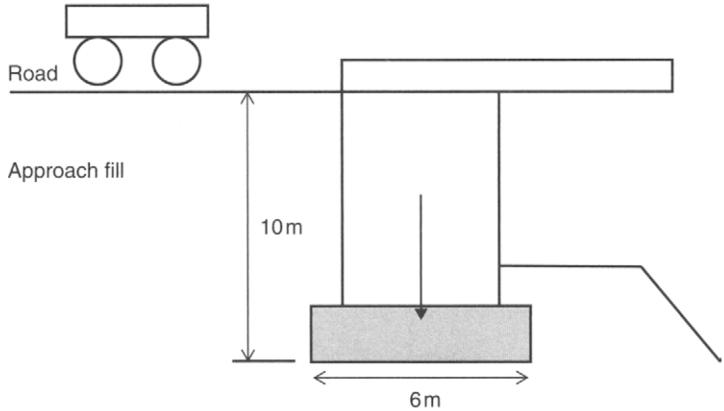


Figure 5.26 Bridge abutment

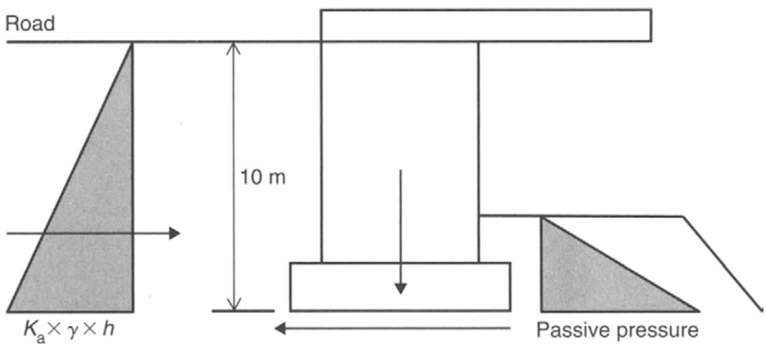


Figure 5.27 Bridge abutment stress diagram

$$K_a = \tan^2(45 - \varphi/2)$$

$$K_a = \tan^2(45 - 30/2) = 0.333$$

STEP 2: Find the stress at the bottom of the footing and the total lateral force.

$$\text{stress at the bottom of the footing} = K_a \times \gamma \times h$$

$$= 0.33 \times 18 \times 10 = 59.4 \text{ kN/m}^2$$

Passive pressure is ignored.

$$\text{total lateral force} = \text{area of the stress triangle}$$
$$= 59.4 \times 10/2 = 297\ \text{kN}$$

STEP 3: Find the safety factor.

$$\text{lateral resistance against sliding} = \text{weight of the abutment} \times \tan(\delta)$$

where

δ = friction angle between concrete and soil
$\delta = 20°$ and the weight of the abutment is given to be 2,000 kN per 1 m length

$$\text{lateral resistance against sliding} = 2{,}000 \times \tan(20°)$$
$$= 728\ \text{kN}$$
$$\text{factor of safety} = \text{lateral resistance/lateral forces}$$
$$= 728/297 = 2.45$$

6

Elastic Settlement of Shallow Foundations

6.1 Introduction

All soils undergo elastic settlement. Elastic settlement is immediate and much higher in sandy soils than in clay soils. Due to the high Young's modulus values in clay, elastic settlement is low in clay soils.

Elastic settlement, S_e, can be found in Table 6.1. S_e is also given by the following equation from AASHTO, the American Association of State Highway and Transportation Officials

$$S_e = \left[q_0/(1 - \nu^2) \times (A)^{1/2}\right] / E_s \times \beta_z$$

where

S_e = elastic settlement
q_0 = footing pressure (should be in same units as E_s)
ν = Poisson's ratio (see Table 6.2 for Poisson's ratio)
A = area of the footing
E_s = elastic modulus or Young's modulus of soil (see Table 6.1 to obtain E_s)
β_z = factor to account for footing shape and rigidity

AASHTO provides another method to obtain the elastic modulus.

Table 6.1 Elastic Modulus vs. SPT (N) Values

Soil type	E_s (ksf)	E_s (kN/m^2)
Silts and sandy silts	8 N	383 N
Fine to medium sand and slightly silty sand	14 N	670 N
Coarse sand and sands with little gravel	20 N	958 N
Sandy gravel and gravel	24 N	1,150 N

Closer examination of Table 6.2 shows that the elastic modulus of clay soils is significantly higher than the elastic modulus of sandy soils.

$$\text{elastic modulus} = \text{stress/strain}$$

$$\text{strain} = \text{stress/elastic modulus}$$

Design Example 6.1

Find the immediate elastic settlement of the foundation shown in Fig. 6.1. The dimensions of the foundation are 1.5 m × 1.5 m. The foundation rests on medium dense coarse to medium sand. The column load is 1,000 kN. The average SPT (N) value below the footing is 12.

Solution

STEP 1: The elastic settlement is given by

$$S_e = \left[q_0/(1 - \nu^2) \times (A)^{1/2}\right] / E_s \times \beta_z$$

S_e = elastic settlement
q_0 = footing pressure = 1,000/(1.5 × 1.5) = 444 kN/m^2
ν = Poisson's ratio is between 0.2 and 0.35 (from Table 6.2), assume $\nu = 0.25$
A = area of the footing = 1.5 × 1.5 = 2.25 m^2
E_s = elastic modulus or the Young's modulus of soil
E_s = 670 N kPa (from Table 6.1, using the "fine to medium sand and slightly silty sand" category)
N = SPT (N) value
E_s = 670 × 12 = 8,040 kPa
β_z = factor to account for footing shape and rigidity

Table 6.2 Elastic Modulus and Poisson's Ratio for Soils

Soil type	Elastic modulus (ksf)	Elastic modulus (kN/m^2)	Poisson's ratio (ν)
Clay soils			
Soft clay	50–300	2,390–14,360	
Medium stiff clay	300–1,000	14,360–47,880	0.4–0.5 (for all clays in the undrained condition)
Stiff clay	1,000–2,000	47,880–95,760	
Very stiff clay	>2,000	>95,760	
Fine sand			
Loose fine sand	160–240	7,660–11,490	
Medium dense fine sand	240–400	11,490–19,150	0.25 for fine sand
Dense fine sand	400–600	19,150–28,720	
sand			
Loose coarse to medium sand	200–600	9,570–28,720	
Medium dense coarse to medium sand	600–1,000	28,720–47,880	0.2–0.35 for coarse to medium sand
Dense coarse to medium sand	1,000–1,600	47,880–76,600	
Gravel			
Loose gravel	600–1,600	28,720–76,600	0.2–0.35
Medium dense gravel	1,600–2,000	76,600–95,760	0.2–0.35
Dense gravel	2,000–4,000	95,760–191,500	0.3–0.4
Silt			
Silt	40–400	1,915–19,150	0.3–0.35

Source: AASHTO (1998).

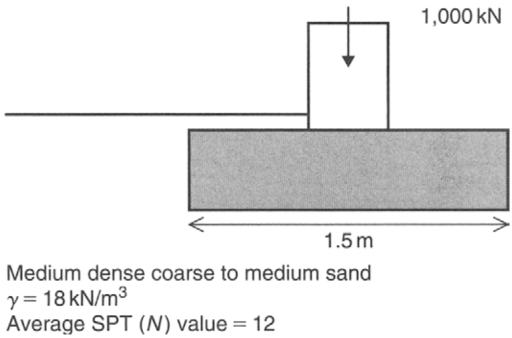


Figure 6.1 Elastic settlement of footing

Assume $\beta_z = 1.0$.

$$S_e = \left[444/(1 - 0.25^2) \times (2.25)^{1/2}\right]$$
$$= 8{,}040 \times 1.0$$
$$S_e = 0.088\,m\,(3.47\,in.)$$

Reference

AASHTO. 1998. American Association of State Highway and Transportation Officials.

7

Foundation Reinforcement Design

7.1 Concrete Design (Refresher)

7.1.1 Load Factors

Load factors need to be applied as a safety measure against uncertainties that could occur during computation of loads. The different loads of a building are computed by adding the weight of all the slabs, beams, columns, and roof elements.

$$\text{load factor for dead loads} = 1.4$$
$$\text{load factor for live loads} = 1.7$$
$$\text{load factor for wind loads} = 1.7$$

7.1.2 Strength Reduction Factors (φ)

The following are strength reduction factors, φ.

concrete shear $\varphi = 0.85$ concrete compression $\varphi = 0.70$
concrete tension $\varphi = 0.90$ concrete beam flexure $\varphi = 0.90$
concrete bearing $\varphi = 0.70$

Many parameters affect the strength of concrete. Water content, humidity, aggregate size and type, cement type, and method of preparation are all important factors. Strength reduction factors are used to account for variations that could occur during concrete preparation. Different φ values are used for shear, tension, compression, and beam flexure.

Stress (σ) due to beam flexure is obtained using the following equation.

$$M/I = \sigma/y$$

where

M = bending moment
I = moment of inertia
y = distance to corner fibers from the neutral axis

7.1.3 How Do We Find the Shear Strength?

If the compressive strength of a concrete is known, the following equation can be used to find the shear strength, v_c, of the concrete.

$$v_c = 4 \times (f'_c)^{1/2}$$

where

f'_c = compressive strength of concrete
v_c = shear strength

The shear strength value needs to be reduced by the strength reduction factor, φ.

$$v_c = 4 \times \varphi \times (f'_c)^{1/2}$$

where

$\varphi = 0.85$ for concrete shear

7.2 Design for Beam Flexure

Figure 7.1 shows a beam subjected to bending. The beam is simply supported from two ends and loaded at the middle. The upper concrete fibers are under compression and the lower steel is under tension.

The compression of concrete fibers is represented by a stress block as shown in Fig. 7.2. This is an approximate representation. The actual stress distribution is a parabola, and not a block shape.

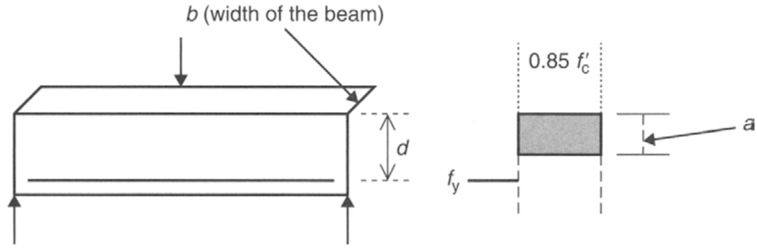


Figure 7.1 Beam subjected to bending

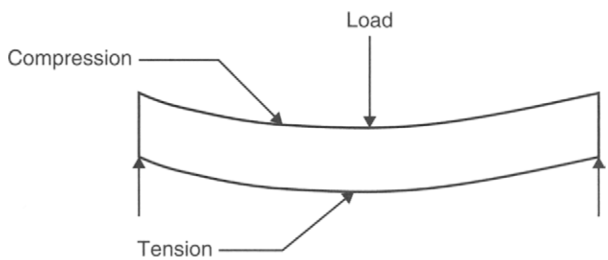


Figure 7.2 Top fibers are subjected to compression while bottom fibers are subjected to tension

The depth to reinforcements is d and the depth of the stress block is a ft. See Fig. 7.3.

The a value depends on the concrete strength and the steel strength.

By balancing forces, we get

$$f_y \times A_s = 0.85 f_c' \times a \times b$$

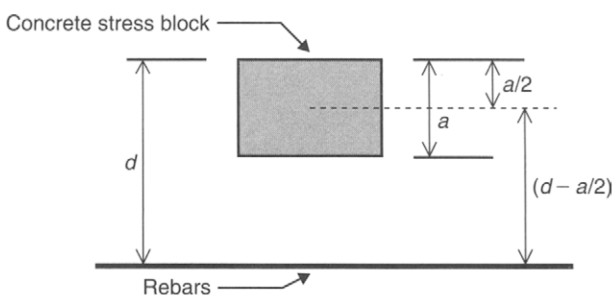


Figure 7.3 Concrete stress block and rebars

where

f_y = yield stress of steel
A_s = steel area
b = width of the beam
a = depth of stress block

The area of the stress block is therefore

$$A = 0.85 f'_c \times a \times b$$

Taking the moments about the steel gives us

$$M = (0.85 f'_c \times a \times b) \times (d - a/2)$$
$$M/0.85 f'_c = a \times b \times (d - a/2)$$

7.3 Foundation Reinforcement Design

7.3.1 Design for Punching Shear

Figure 7.4 shows a shallow foundation with bottom rebars.

Punching failure occurs $d/2$ distance from the edge of the column. See Fig. 7.5.

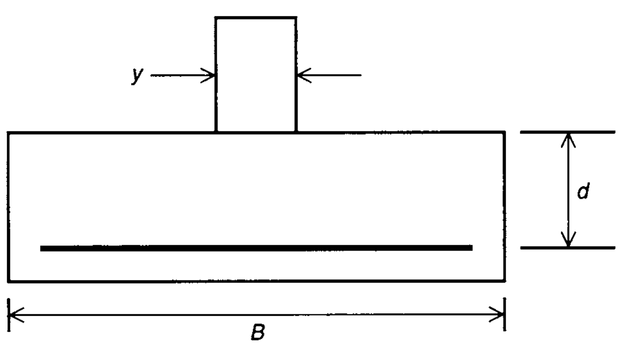


Figure 7.4 Shallow foundation with bottom rebars

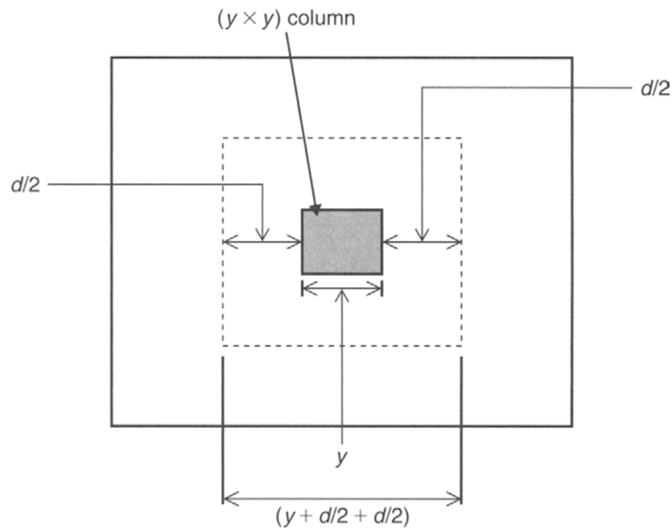


Figure 7.5 Punching shear zone in a footing

7.3.2 Punching Shear Zone

Due to the load of the column, the footing could be punched through. Normally, the punching shear zone is taken to be at a distance of $d/2$ from the edge of the column. The value d is the depth of the footing measured to the top of rebars from the top of the footing. Hence the dimensions of the punching shear zone are $(d + y) \times (d + y)$. For punching failure to occur, this zone needs to fail.

The dimensions of the punching shear zone are given by

$$4 \times (y + d/2 + d/2) \times d \times v_{\mathrm{C}} = 4 \times (y + d) \times d \times v_{\mathrm{C}}$$

The perimeter of the zone is represented by

$$4 \times (y + d/2 + d/2)$$

Here d represents the depth or thickness of the footing.

The perimeter is multiplied by the depth, d. This would give the resisting area for shear. See Fig. 7.6. v_{C} is the shear strength per unit area of concrete. The force that needs to be resisted is equal to

$$[B^2 - (y + d)^2] \times q_{\text{allowable of soil}}$$

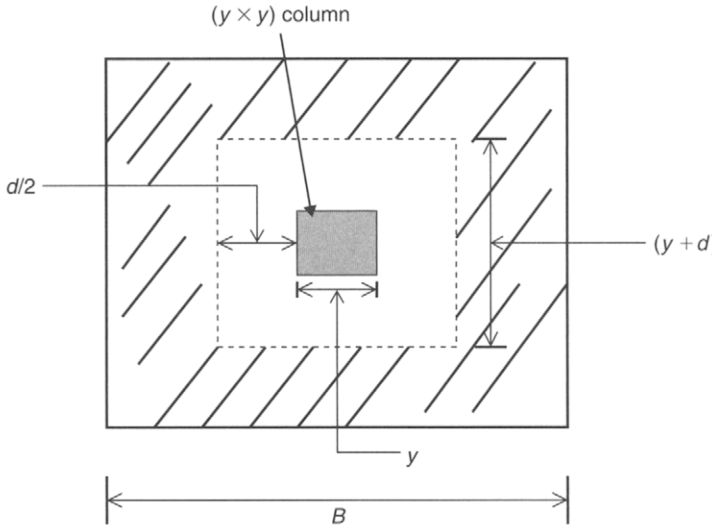


Figure 7.6 Stressed area of the footing

The area of the hatched area outside the punching zone is

$$B^2 - (y + d)^2$$

$$\left[B^2 - (y + d)^2\right] \times q_{\text{allowable of soil}} = 4 \times (y + d) \times d \times v_c$$

It is possible to find the depth of the footing required to avoid punching shear failure. The load from the column is given by

$$B^2 \times q_{\text{allowable of soil}}$$

where

v_c = shear strength of the concrete

v_c is computed using the following equation. The shear strength of concrete, v_c is given by

$$v_c = 4\varphi \times (f_c')^{1/2}$$

where

f_c' = compressive strength of concrete

The value of f_c' normally varies from 3,000 psi to 6,000 psi. For 3,000 psi concrete

$$v_c = 4 \times 0.85 \times (3{,}000)^{1/2} = 186.2 \text{ psi}$$

Hence d can be computed.

7.3.3 Design Reinforcements for Bending Moment

It is clear that a bending moment would be developed in the footing due to the load from the column and the soil pressure. See Fig. 7.7.

Assume a 1 ft wide strip of the footing (the shaded strip in Fig. 7.7). The forces acting on this strip are shown in Fig. 7.8. The soil pressure acts from below. The footing is fixed to the column.

The footing/column boundary is considered to be cantilevered as shown in Fig. 7.8. Now the problem is boiled down to a beam flexure problem.

$$L = (B - y)/2 \text{ (for a cantilevered beam)}$$

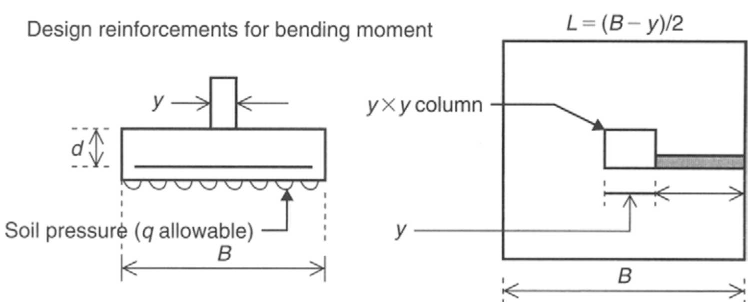


Figure 7.7 Soil pressure and rebars

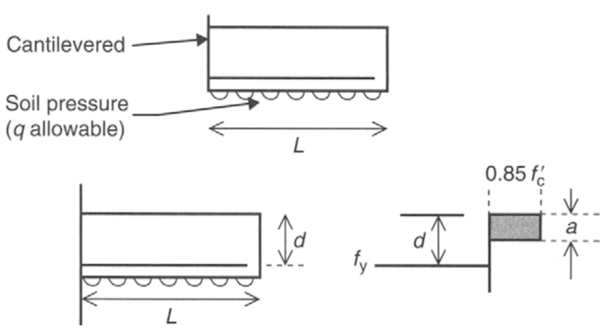


Figure 7.8 Cantilevered strip

The bending moment, M, can be determined by

$$(M) = \frac{(q_{\text{allowable of soil}}) \times L^2}{2}$$

$$\text{bending moment} = \text{total load} \times \text{distance to center of gravity}$$
$$\text{total load} = q_{\text{allowable of soil}} \times L$$
$$\text{distance to center of gravity} = L/2$$

Since we are considering a 1 ft wide strip, the width of the beam considered is 1 ft.

$$\text{force in the steel} = A_s \times f_y$$

where

A_s = steel area
f_y = yield stress of steel

This force in the steel should be equal to the force in concrete. Hence

$$0.85 f_c' \times b \times a = A_s \times f_y$$

where

f_c' = concrete compression strength
f_y = yield stress of steel
A_s = steel area
b = width of the beam
a = depth of the stress block

Note that a value of above 0.85 represents the factor of safety against concrete workmanship. No factor of safety is assumed for steel.

$$a = (A_s \times f_y)/(0.85 f_c' \times b) \tag{7.1}$$
$$\text{moment arm} = (d - a/2)$$

Table 7.1 Reinforcing Bars

Number of bars	Nominal diameter (in.)	Nominal diameter (mm)	Nominal weight (lb/ft)
3	0.375 (3/8)	9.5	0.376
4	0.500 (4/8)	12.7	0.668
5	0.625 (5/8)	15.9	1.043
6	0.750 (6/8)	19.1	1.502
7	0.875 (7/8)	22.2	2.044
8	1.000	25.4	2.670
9	1.128	28.7	3.400
10	1.270	32.3	4.303
11	1.410	35.8	5.313
14	1.693	43	7.650

Where the moment arm is the distance between the steel and the center of the stress block.

Hence,

$$\text{resisting steel moment} = A_S \times f_y \times (d - a/2)$$

This resisting moment should be equal to the moment induced due to loading.

$$\text{moment induced due to soil pressure} = \frac{(q_{\text{allowable of soil}}) \times L^2}{2}$$

Hence

$$(q_{\text{allowable of soil}}) \times L^2 = A_S \times f_y \times (d - a/2)/2 \tag{7.2}$$

In the above equation, $q_{\text{allowable}}$, L, and f_y are known parameters. The value of a is obtained from Eq. 7.1, in terms of A_S. f_c', f_y, and b are known. The only unknown in Eq. 7.1 is A_S.

Hence, in Eq. 7.2, two parameters are unknown. They are the required steel area A_S and the required depth of the footing d. To solve Eq. 7.2, guess a reasonable footing depth d, and estimate A_S.

8

Grillage Design

8.1 Introduction

Typical load bearing concrete foundations are designed with steel reinforcements. It is not possible to design reinforcements for very high column loads. In such cases, grillages are used. See Figs. 8.1 and 8.2.

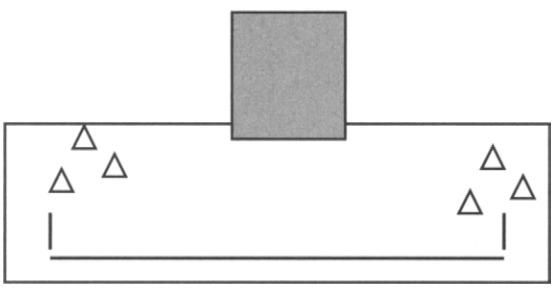


Figure 8.1 Regular footing with steel reinforcements

8.1.1 What Is a Grillage?

A grillage consists of two layers of I-beams as shown in Fig. 8.2.

The load from the column is transferred to the base plate. The base plate transfers the load to the concrete. The concrete transfers the load to the top layer of I-beams and then to the bottom layer of I-beams.

The bottom layer of I-beams transfers the load to the concrete below and then to the rock underneath.

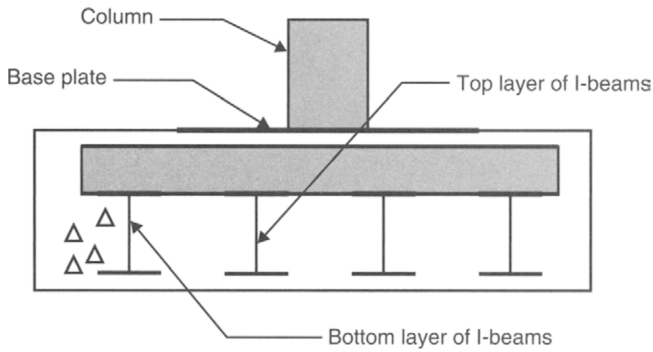


Figure 8.2 Grillage footing

Design Example 8.1

This example explores grillage design. A concrete encased steel column carrying 2,000 tons needs to be supported. The allowable bearing capacity of the rock is 20 tsf (tons per square foot). The steel column is supported on a 24 in. × 24 in. base plate. It is decided to have a grillage consisting of two layers of I-beams. The engineer decides to have three I-beams in the top layer and five I-beams in the bottom layer.

a. Design the top layer of I-beams.
b. Design the bottom layer of I-beams.

Solution

STEP 1: Find the size of the footing required.

$$\text{size of the footing required} = 2{,}000\,\text{ton}/20\,\text{tsf} = 100\,\text{sq ft}$$

Use a footing of 10 ft × 10 ft. See Fig. 8.3.

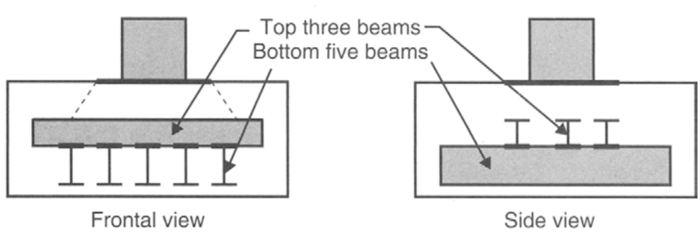


Figure 8.3 Grillage (front and side view)

The following assumptions apply.

- Assume that the beams are 10 ft long. In reality, the beams are less than 10 ft, since the footing has dimensions of 10 ft × 10 ft.
- Assume that the base plate is 24 in. × 24 in. and the load is transferred to the top layer of beams as shown in Fig. 8.3.
- Assume that the load is transferred to a section of 30 in. (See Fig. 8.4).

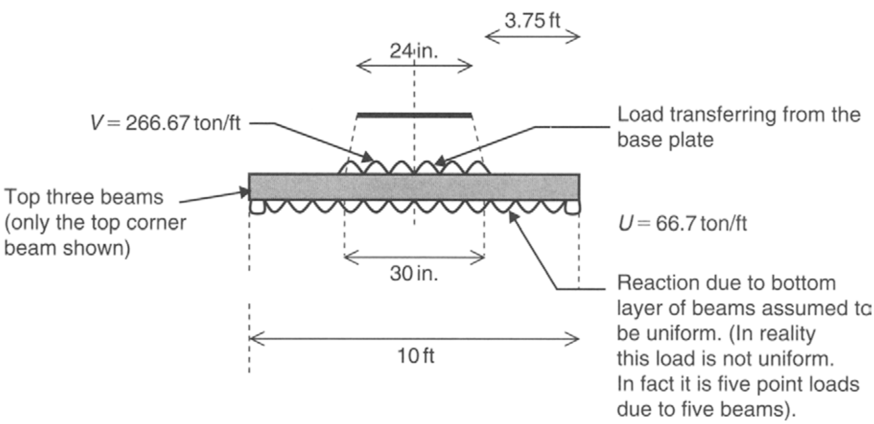


Figure 8.4 Loading on a grillage

STEP 2: The loads acting on the top three beams are shown in Fig. 8.4.

$$\text{total load from the base plate} = 2{,}000\,\text{tons} = 4{,}000\,\text{kips}$$

where

1 ton = 2,000 lb
1 kip = 1,000 lb
1 ton = 2 kips

Since there are three I-beams in the top layer, one I-beam would take a load of 666.67 (2,000/3) tons. This load is distributed in a length of 30 in. Hence the distributed load on the beam is

$$666.67/2.5 = 266.67\,\text{ton/ft}$$

The load from the top I-beams is distributed to the bottom layer of I-beams. The bottom layer of I-beams generates an upward reaction on the top I-beams. This reaction is assumed to be uniform.

In reality, this upward reaction (U) is basically five concentrated reactions acting on the top layer of I-beams. As mentioned above, there are five beams in the bottom layer and each exerts a reaction.

A uniformly distributed load due to the reactions from the bottom layer equals

$$666.67/10 = 66.7\ \text{ton/ft}$$

The total load that needs to be transferred from one top beam is 666.67 tons and it is distributed over a length of 10 ft. See Fig. 8.5.

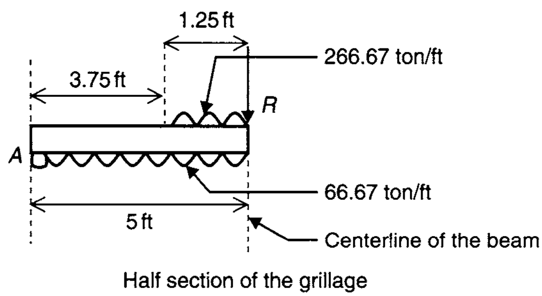


Figure 8.5 Half section of the grillage

Now the problem is to find the maximum bending moment occurring in the beam. Once the maximum bending moment is found, an I-beam section can be designed.

The maximum bending moment occurs at the center of the beam.

STEP 3: Find the maximum bending moment in the beam.

The reaction at the centerpoint of the beam is taken to be R.

Assume the bending moment at the center to be M. For this type of loading the maximum bending moment occurs at the center. (Take moments about the centerpoint.)

$$M = (66.67 \times 5 \times 2.5) - 266.67 \times 1.25 \times 1.25/2 = 625\ \text{ton ft}$$

The above value of 66.67×5 represents the total load, and 2.5 represents the distance to the center of gravity. Similarly, 266.67×1.25

represents the total load, and 1.25/2 represents the distance to the center of gravity from the center of the beam.

The beam should be able to carry this bending moment. Select an I-section that can carry a bending moment of 625 ton ft.

$$M = 625\,\text{ton ft} = 2 \times 625\,\text{kip ft} = 1{,}250\,\text{kip ft}$$

$$M/Z = \sigma$$

M = bending moment
Z = section modulus
σ = stress in the outermost fiber of the beam

Use an S-section (i.e., an American Institute of Steel Construction "S" shape beam) with allowable steel stress of 36,000 psi.

$$\sigma = 36\,\text{ksi}$$

$$Z = M/\sigma$$

$$Z = \frac{(1{,}250 \times 12)\,\text{kip in.}}{36\,\text{ksi}}$$

$$Z = 417\,\text{in}^3$$

Use W (i.e., an American Institute of Steel Construction "W" shape beam) 36 × 135 section with section modulus 439 in^3.

STEP 4: Design the bottom layer of beams.

Three top beams rest on each bottom layer beam. Assume the top beams are 12 in. apart (see Fig. 8.6).

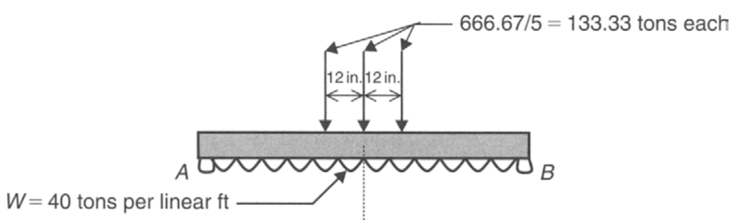


Figure 8.6 Forces on bottom layer of beams

Each top layer beam carries a load of 666.67 tons. Each of the top layer beams rests on five bottom layer beams. Hence 666.67 tons is distributed over five bottom layer beams. Each bottom layer beam gets a load of 666.67/5 tons (or 133.33 tons) from each top beam.

There are three top layer beams. Hence each bottom layer beam carries a load of

$$3 \times 133.33 = 400 \text{ tons}$$

All bottom layer beams sit on concrete. This load needs to be transferred to the concrete. The reaction from concrete is considered to be uniformly distributed.

$$W = \text{concrete reaction} = 400/10 = 40 \text{ tons per linear ft}$$

STEP 5: Find the maximum bending moment.

The maximum bending moment occurring in the bottom layer beams can be computed. See Fig. 8.7.

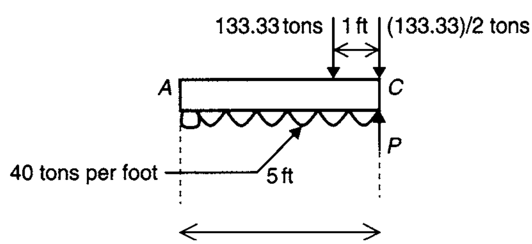


Figure 8.7 Half section of the grillage bottom layer

Cut the beam at the center. Then the concentrated load at the center needs to be halved since one half goes to the other section.

Take moments about point C.

$$M = 40 \times 5 \times 2.5 - 133.33 \times 1 = 366.7 \text{ ton ft}$$

Hence the maximum bending moment is 366.7 ton ft.

$$M = 366.7 \text{ ton ft} = 2 \times 366.7 = 733.4 \text{ kip ft}$$

$$M/Z = \sigma$$

M = bending moment
Z = section modulus
σ = stress in the outermost fiber of the beam

Use a steel section with allowable steel stress of 36,000 psi.

$$\sigma = 36\,\text{ksi}$$

$$Z = M/\sigma$$

$$Z = \frac{(733.4 \times 12)\,\text{kip in}}{36\,\text{ksi}}$$

$$Z = 244.4\,\text{in}^3$$

Use S 24 × 121 section with a section modulus of 258 in^3.

9

Footings Subjected to Bending Moment

9.1 Introduction

Shallow footings are subjected to bending moments and horizontal forces in addition to vertical loads. Foundation engineers need to address all the forces and moments in the design. Horizontal forces and bending moments are generated mostly due to wind loads. See Fig. 9.1.

Due to wind loads, the bending moment M will act on corner footings. See Fig. 9.2.

Wind load can be simplified to a resultant horizontal force and a bending moment at the footing level. See Fig. 9.3.

The wind load acting on the building is equated to a horizontal force, H, and a bending moment, M. The lateral resistance, R, of the bottom of footing should be greater than the horizontal force, H.

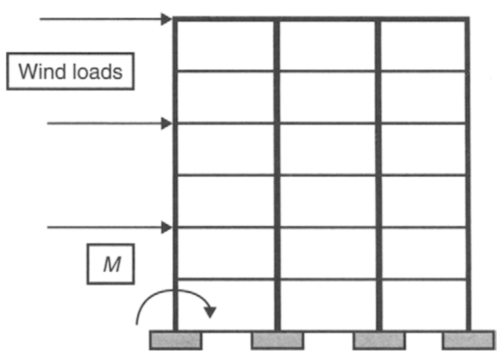


Figure 9.1 Loads on footings

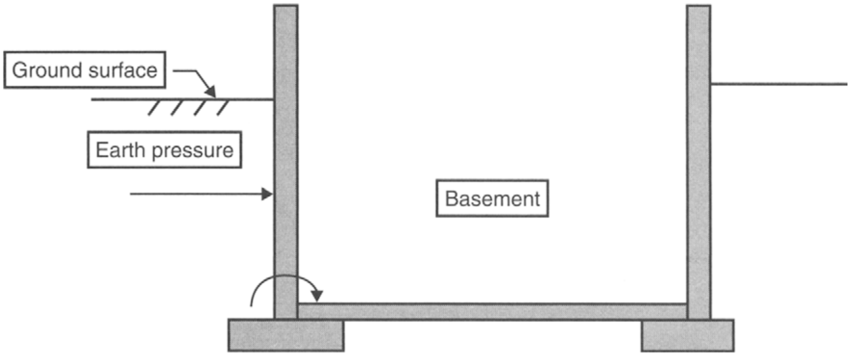


Figure 9.2 Building with a basement

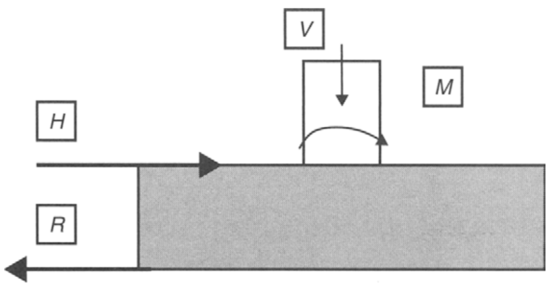


Figure 9.3 Forces and moments acting on a footing

R can be calculated by finding the friction between the bottom of the footing and the soil. The friction, *R*, at the bottom of footing is given by

$$R = V \tan \delta$$

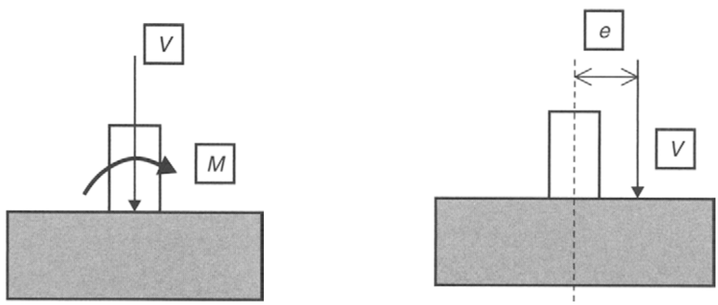
where

δ = friction angle between bottom of footing and soil
V = vertical force acting on the footing

For stability, $R > H$. Note that the soil friction angle, φ, can be used as an approximation for δ.

9.2 Representation of Bending Moment with an Eccentric Load

The bending moment acting on a footing can be represented by an eccentric vertical load. A vertical load that is not acting at the center of the footing will generate a bending moment. See Fig. 9.4.



Footing with bending moment

Bending moment is equated to an eccentric vertical load

Figure 9.4 Equivalent footing

Essentially, a footing subjected to a bending moment can be represented by a footing with an eccentric load. e is selected in a manner so that

$$M = V \times e$$

where

e = eccentric vertical load
M = bending moment due to wind load
V = vertical load

Due to bending moment, one side would undergo larger compression than the other side. See Fig. 9.5.

The left side of Fig. 9.5 consists of only a vertical load. The right side has a bending moment that has been represented by an eccentric vertical load.

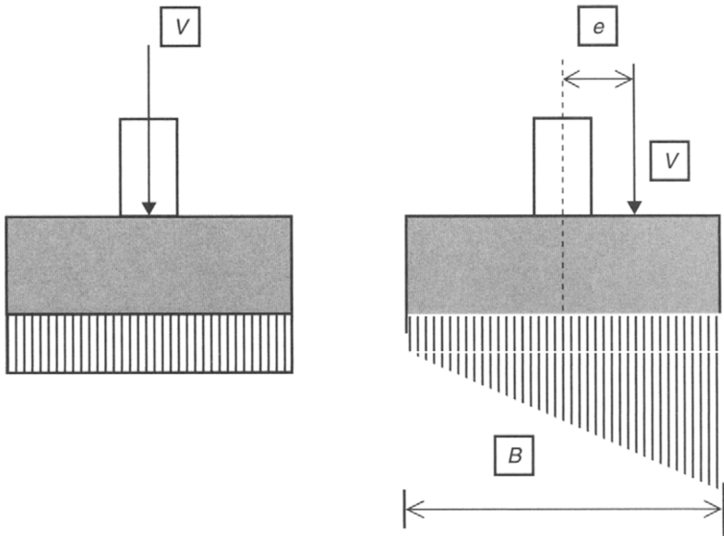


Figure 9.5 Stresses underneath footings

When the bending moment is increased, the stress at the bottom of footing becomes uneven. See Fig. 9.6.

If the bending moment were to increase more, the pressure under the footing on one side would become zero. The foundation engineer should make sure that this would not happen.

The soil pressure underneath the footing should be equal to the applied vertical force, V. The force due to the soil is equal to the area of the pressure triangle.

$$\text{area of the pressure triangle} = q_{max} \times B/2$$

where

q_{max} = maximum stress at the edge of the footing
B = width of the footing

For stability

$$\text{vertical load} = \text{area of the pressure triangle}$$

$$V = q_{max} \times B/2$$

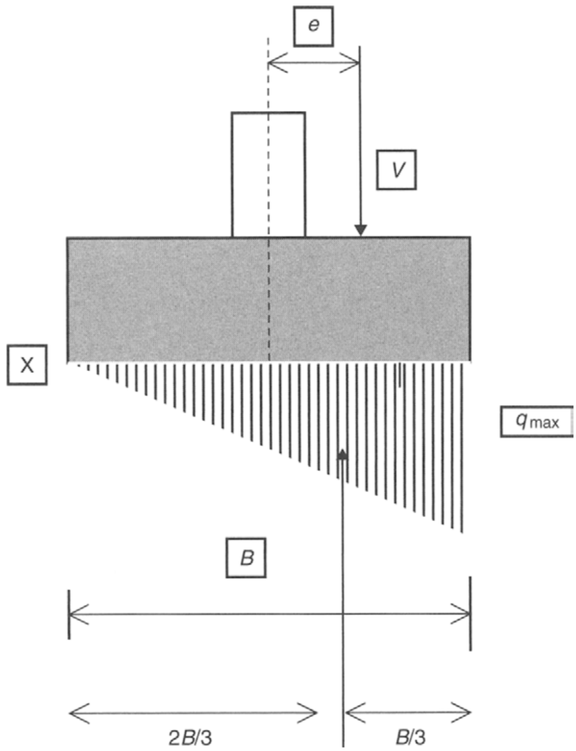


Figure 9.6 Stress distribution under a footing

Take moments around point X.

$$\text{overturning moment} = V \times (B/2 + e)$$
$$\text{overturning moment} = (q_{\max} \times B/2) \times (B/2 + e)$$

Since

$$V = q_{\max} \times B/2$$

The overturning moment is obtained by taking moments around point X.

resisting moment = (area of the pressure triangle) × (distance to center of gravity of the stress triangle from point X)

resisting moment = $(q_{\max} \times B/2) \times 2B/3$

For stability

$$\text{overturning moment} = \text{resisting moment}$$

$$(q_{max} \times B/2) \times (B/2 + e) = (q_{max} \times B/2) \times 2B/3$$

$$B/2 + e = 2B/3$$

$$e = B/6$$

If e is greater than $B/6$, then negative forces would be generated underneath the footing. In other words, the footing would be lifted. Hence it is the responsibility of the geotechnical engineers to make sure that eccentricity, e, does not exceed $B/6$ from the center of the footing.

10

Geogrids

Geogrids can be used to increase the bearing capacity of foundations in weak soils. See Fig. 10.1. Geogrids distribute the load so that the factor of safety against bearing failure is increased. See Fig. 10.2.

Due to the geogrid, the foundation load is better distributed. Hence the pressure on the weak soil layer is reduced. Pressure reduction occurs since the load is distributed into a larger area.

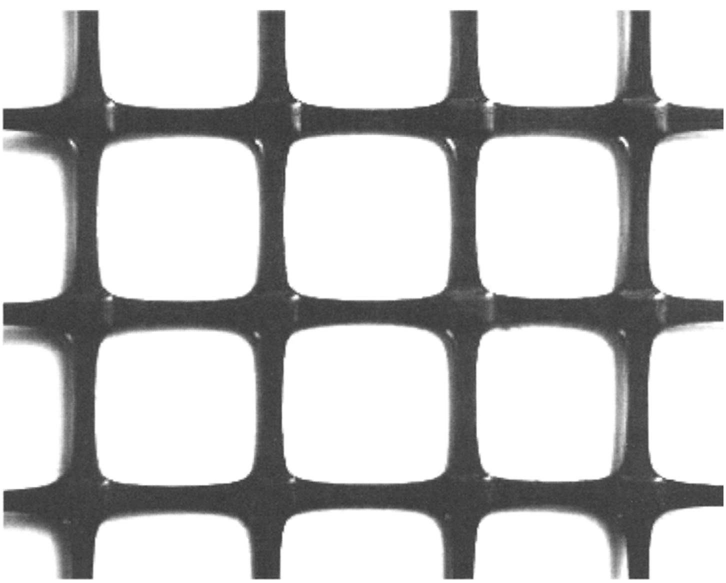


Figure 10.1 Geogrid

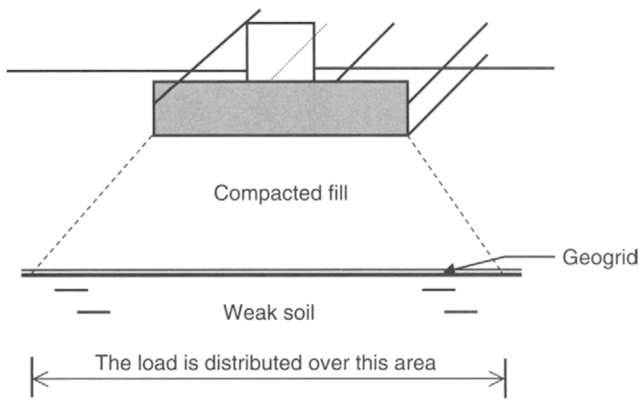


Figure 10.2 Load distribution in a geogrid

For this reason, settlement can be reduced and bearing strength can be improved.

Geogrids basically distribute the load in a much more even manner. It is not advisable to use geogrids for major settlement problems.

10.1 Failure Mechanisms

The foundation can fail within the compacted zone. See Fig. 10.3. This could happen if the soil is not properly compacted.

The foundation can fail due to failure of the soil under the geogrid. Geogrids need to be large enough to distribute the load into a larger area. See Fig. 10.4.

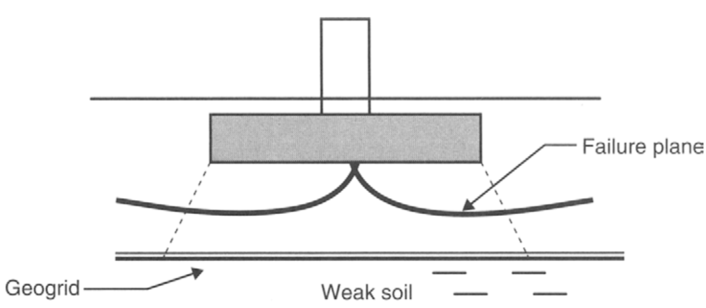


Figure 10.3 Failure in the compacted zone

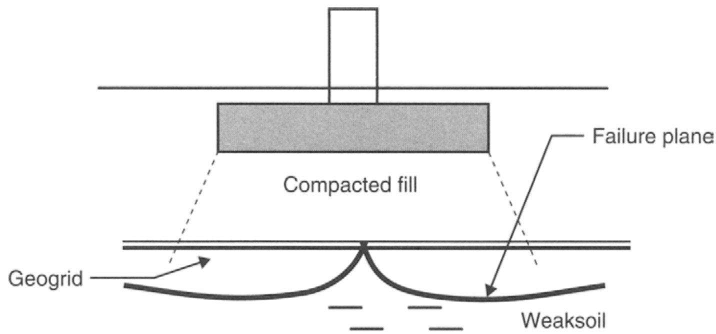


Figure 10.4 Failure within the weak soil layer

Reference

Xanthakos, P. 1994. *Ground control and improvement.* New York: John Wiley and Sons.

11

Tie Beams and Grade Beams

11.1 Tie Beams

The purpose of tie beams is to connect column footings together. Tie beams may or may not carry any vertical loads, such as walls and so forth. See Fig. 11.1.

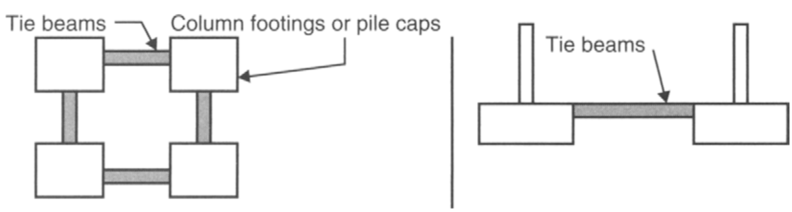


Figure 11.1 Tie beams

11.2 Grade Beams

Unlike tie beams, grade beams can carry walls and other loads. See Fig. 11.2.

Grade beams are larger in size than tie beams, since they carry loads.

After construction of pile caps or column footings, the next step is to construct tie beams and grade beams. Qualified personnel must supervise the placement of rebars prior to concreting. See Fig. 11.3.

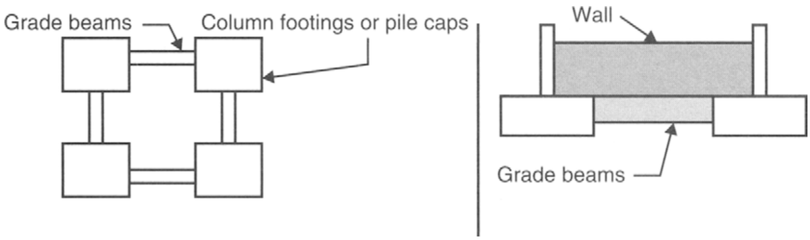


Figure 11.2 Grade beams

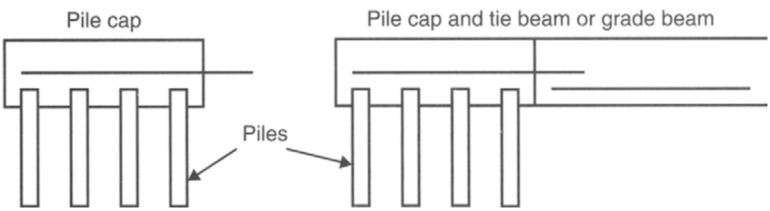


Figure 11.3 Pile cap and tie beams

Note that the contractor should provide rebars jutting out prior to concreting of pile caps. That way, when the grade beams are constructed, continuous rebars could be provided.

11.3 Construction Joints

During the process of setting up, concrete contracts. If construction joints were not provided, cracks would be generated due to this contraction process. See Fig. 11.4.

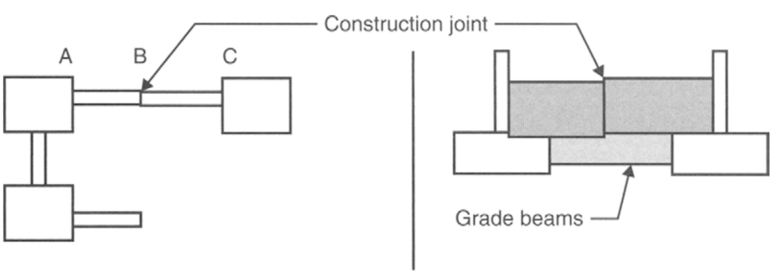


Figure 11.4 Construction joints

Construction Procedure

- Concrete a section (as an example, concrete from point A to point B in Fig. 11.4).
- Wait a reasonable time period for the concrete to contract.
- Concrete the next section.
- This way any contraction in the concrete is eliminated.
- Typically, engineers recommend construction joints every 60 to 100 ft of beams.

12

Drainage for Shallow Foundations

12.1 Introduction

Geotechnical engineers need to investigate the elevation of groundwater, since groundwater can create problems during the construction stage. The stability of the bottom may be affected by the presence of groundwater. In such situations, a dewatering method needs to be planned. Maintaining sidewall stability is another problem that needs to be addressed. See Figs. 12.1 and 12.2.

Continuous pumping can be used to maintain a dry bottom. Stable side slopes can be maintained by providing shoring supports.

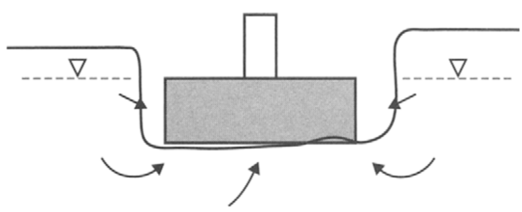


Figure 12.1 Groundwater migration during shallow foundation construction

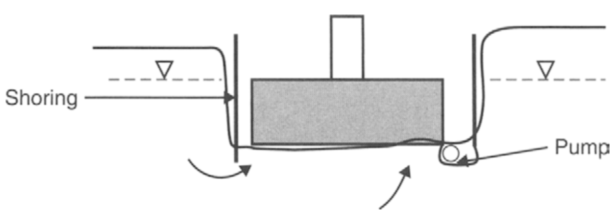


Figure 12.2 Pumping and shoring the sides

12.1.1 Well Points

In some cases, it is not possible to pump out enough water to maintain a dry bottom for concrete shallow foundations. In such situations, well points are constructed to lower the groundwater table. See Fig. 12.3.

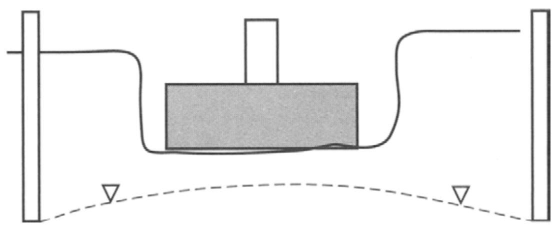


Figure 12.3 Use of well points to lower the groundwater table

12.1.2 Small Scale Dewatering for Column Footings

In small scale dewatering, water is pumped from the excavation. The groundwater level is lowered by constructing a small hole or a trench inside the excavation (as shown in Fig. 12.4) and placing a pump (or several pumps) inside the excavation. For most column footing construction work, this method will be sufficient.

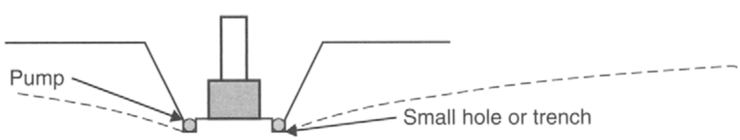


Figure 12.4 Dewatering of a column footing

12.1.3 Medium Scale Dewatering for Basements or Deep Excavations

In medium scale dewatering, water is pumped from trenches or wells. For medium scale dewatering projects, trenches or well points can be used. See Fig. 12.5

It may be necessary to add more pumps to keep the excavation dry. A combination of submersible pumps and vacuum pumps can be used.

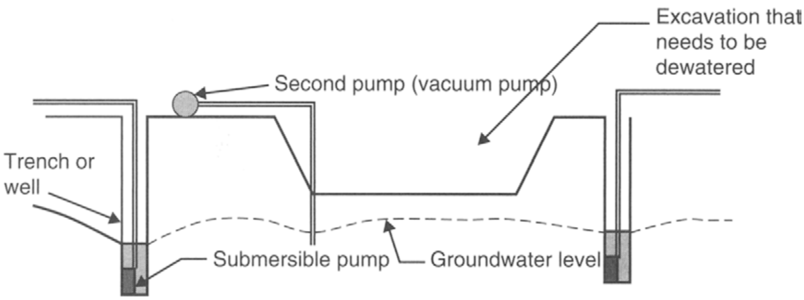


Figure 12.5 Groundwater lowering using well points

12.1.4 Large Scale Dewatering for Basements or Deep Excavations

Two alternatives are available for large scale dewatering for basements or deep excavations.

Alternative 1: Well points or trenches are constructed. A main artery pipe is connected to each pump as shown in Fig. 12.6. A strong high capacity pump sucks water out of all the wells, as shown.

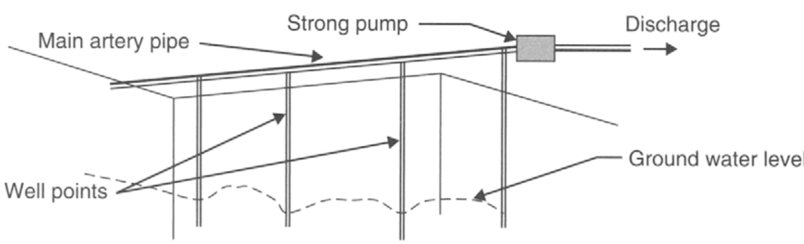


Figure 12.6 Well points in series with one large pump

Alternative 2: A similar dewatering system can be designed using submersible pumps. In this case, instead of one pump, each well would get a submersible pump as shown in Fig. 12.7.

Alternative 2 is more effective than alternative 1. The main disadvantage of alternative 2 is high maintenance. Since there is more than one pump, more maintenance work will occur compared to alternative 1.

On the other hand, alternative 2 can be modified to include fewer well points, since the pumping effort can be increased significantly.

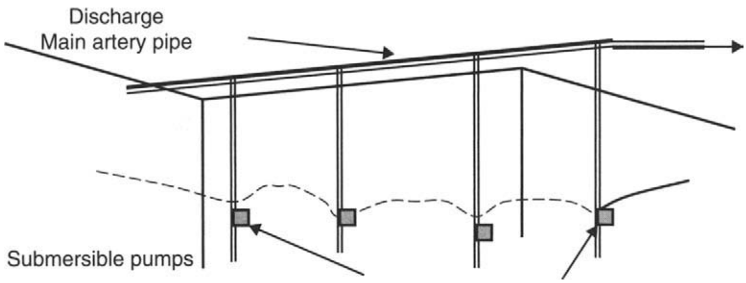


Figure 12.7 Well points in series with submersible pumps

The following notes should be taken into account when planning dewatering.

- For most construction work, groundwater should be lowered at least 2 ft below the excavation.
- A water quality assessment study needs to be conducted. If the groundwater is contaminated, discharge permits need to be obtained, depending on local regulations. In most cases, local and national environmental protection agencies need to be contacted and proper procedures should be followed. Discharging contaminated water into rivers, lakes, or storm water systems could be a major offense.
- If water needs to be treated prior to discharge, dewatering may become extremely expensive. In that case, other methods, such as cutoff walls or ground freezing, may be cheaper.

12.1.5 Design of Dewatering Systems

Initial Study

Study the surrounding area and locate nearby rivers, lakes, and other water bodies. Groundwater normally flows toward surface water bodies, such as rivers and lakes. If the site is adjacent to a major water body (such as a river or lake) the groundwater elevation will be the same as the water level in the river.

Construct Borings and Piezometers

It is important to gather data on existing soil conditions. To do this, create soil profiles based on borings. Assess the permeability of the existing soil stratums. (Sandy soils will transmit more water than clayey soils.)

The graph in Fig. 12.8 shows approximate permeability values of sands and gravel with respect to D_{10} values. A D_{10} value is the size of sieve that will allow 10% of the soil to pass through.

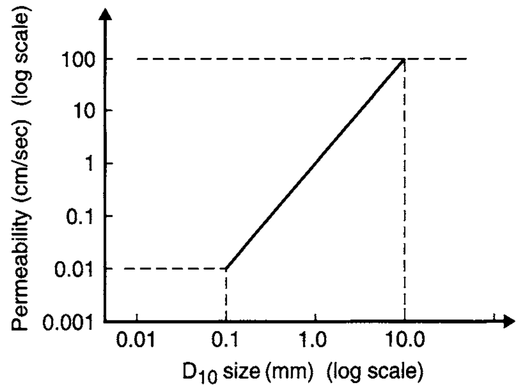


Figure 12.8 Permeability sandy soils

STEP 1: Obtain the D_{10} value by conducting a sieve analysis.
STEP 2: Use the graph in Fig. 12.8 to find the permeability.

Note that this graph should not be used for silts and clays.

The experimental values in Table 12.1 were used to draw the graph in Fig. 12.8 (Xanthakos, 1994). Again, the D_{10} value is the size of the sieve that only 10% of the soil would pass through. For example, if 10% of the soil would pass through a sieve that has an opening of 1.5 mm, then the D_{10} value of that soil is 1.5 mm. The D_{10} value is obtained by drawing a sieve analysis curve and then locating the 10% passing point in the curve. The corresponding sieve size is the D_{10} value.

Table 12.1 D_{10} Value vs. Permeability

Soil type	D_{10}(mm)	Permeability (cm/sec)
Sand	0.06	0.0036
Sand	0.01	0.01
Gravel	0.30	0.09
Gravel	1.5	2.3
Gravel	9.0	81

Source: Xanthakos (1994).

The graph in Fig 12.8 should be used with reservations. If highly variable soil conditions are found, the graph should not be used to obtain the permeability. In such situations, a well pumping test needs to be conducted to obtain the permeability of the soil.

- **Seasonal Variations** Groundwater elevation readings should be taken at regular intervals. In some sites, groundwater elevation could be very sensitive to seasonal changes. The groundwater elevation during the summer could be drastically different from the level in winter.
- **Tidal Effect** The groundwater elevation changes with respect to tidal flow. In some sites, groundwater elevation may show a very high sensitivity to high and low tides.
- **Artesian Conditions** Check for artesian conditions. The groundwater could be under pressure and well pumping or any other dewatering scheme could be a costly procedure.
- **Groundwater Contamination** If contaminated groundwater is found, then groundwater cutoff methods should be studied.

12.2 Ground Freezing

Ground freezing can be used as an alternative to pumping. In some situations, pumping of groundwater to keep excavations dry may be very costly. When the groundwater is contaminated, discharging of groundwater is a major problem. If contaminated groundwater is pumped out, that water needs to be treated prior to disposal. In such situations, ground freezing can be an economical way to control groundwater. See Fig. 12.9.

12.2.1 Ground Freezing Technique

Soil pores are full of moisture. The moisture in soil pores is frozen to produce an impermeable barrier. See Fig. 12.10. Ground freezing is achieved by circulating calcium chloride brine. Calcium chloride brine is sent through one pipe and retrieved from two pipes. See Fig. 12.11.

The brine solution is sent through pipe A and removed from pipes B and C. During circulation, the brine solution extracts heat from the

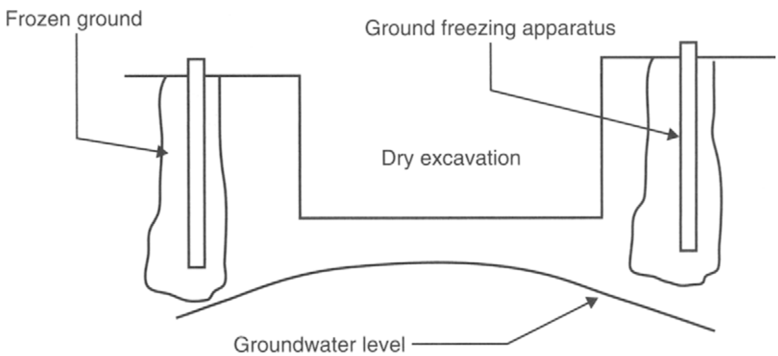


Figure 12.9 Ground freezing to keep excavations dry

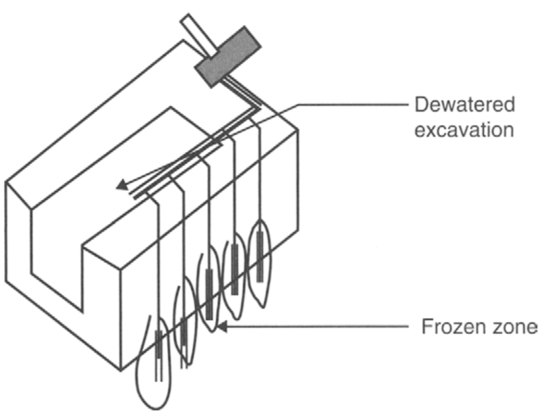


Figure 12.10 Ground freezing methodology

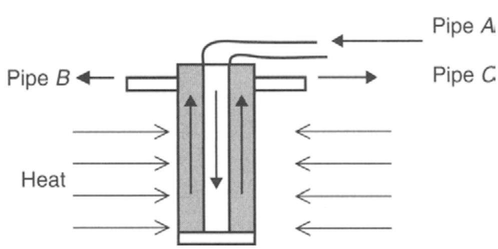


Figure 12.11 Brine flow for ground freezing

surrounding soil. When heat is lost from the moisture in soil, ground freezing occurs.

Brine solution is removed from pipes *B* and *C*, is pumped to the plant and energized, and sent again to the well.

A decade or so ago, ground freezing was considered to be an exotic method that no one knew exactly how to do. Recently many projects have been successfully completed using the ground freezing technique. When construction work needs to be conducted in a contaminated groundwater media, the ground freezing technique becomes economical.

12.2.2 Ground Freezing—Practical Aspects

Ground freezing is a complex operation. There are instances where the method failed to cut off the groundwater from the foundation during construction. The following sections deal with important aspects of ground freezing, the type of soil, refrigeration technology, groundwater flow and ground heave, ground freezing in the presence of moving water, tunneling, and underpinning.

Sands

Ground freezing is easier in granular soils. Groundwater in granular soils occurs in the pores of the soil. The chemical bonds between soil grains and water are negligible. Hence water would freeze quickly after the temperature is brought below the freezing point (32° F or 0° C).

Clays

Ground freezing is unpredictable and difficult in clay soils. Water molecules are chemically bonded to clay particles. For this reason, water is not free to freeze. Substantially low temperatures are required to freeze clay soils.

Refrigeration Plant Size

Refrigeration plants are rated by tons (T_R).

$$1\ T_R = 12{,}000\ \text{BTU/hr}$$

Typical refrigeration plants for ground freezing range from 100 to 200 T_R.

Liquid Nitrogen (LN_2) vs. Brine

Liquid nitrogen is another chemical used for ground freezing. Extremely low temperatures can be achieved with liquid nitrogen. The cost of liquid nitrogen plants can be higher than brine plants.

As mentioned earlier, clay soils may not be easy to freeze with chlorides. In such situations, liquid nitrogen can be used. Ground freezing with liquid nitrogen is much faster compared to brine.

Groundwater Flow Velocity

Groundwater flow velocity changes from site to site. High groundwater flow velocities can create problems for the freezing process. It is well known that more energy is needed to freeze moving water than still water.

Prior to any ground freezing project, groundwater flow velocities should be assessed. If the flow velocity is high, the temperature may have to be brought down well below the freezing point to achieve success.

Ground Heave

The ground can heave due to freezing. When water changes to ice, the volume increases by approximately 10%. When performing ground freezing, ground heave could cause problems for nearby buildings. In sandy soils, water freezes quickly and ground heave would be noticeable. In clay soils, water freezes gradually. Hence, ground heave may not be apparent at the beginning. See Fig. 12.12.

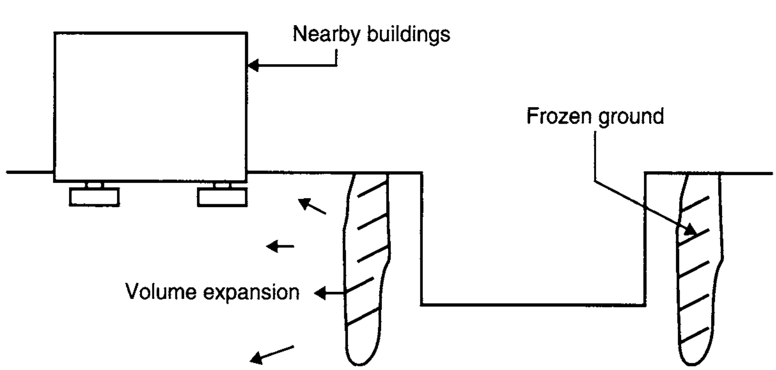


Figure 12.12 Volume expansion due to ground freezing could create problems for nearby buildings

Utilities

Ground freezing could freeze nearby utilities. If utilities such as gas lines or water lines are present, then the proper authorities need to be consulted prior to recommending the ground freezing method to control groundwater.

Groundwater Control Near Streams and Rivers

Groundwater control near streams and rivers could be a challenging affair. In such situations, ground freezing may be more effective than well pumping.

Groundwater Control in Tunneling

The ground freezing method can also be utilized in tunnels. Fractured ground can be encountered unexpectedly during tunneling. See Fig. 12.13.

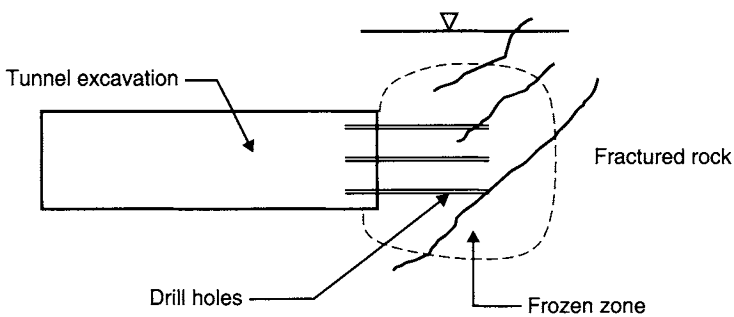


Figure 12.13 Ground freezing in tunneling

Holes are drilled ahead of the tunnel excavation and brine solution is injected. This can create a frozen zone in front of the tunnel excavation to freeze the groundwater.

Contaminant Isolation

Ground freezing can be used for long periods of time to isolate contaminants. In such situations, durable piping and machinery need to be used. Contaminant isolation efforts could range from 10 to 30 years. During this time period, maintenance of the system is required. See Fig. 12.14.

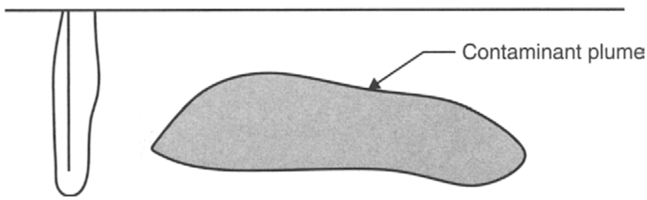


Figure 12.14 Ground freezing for contaminant isolation

Directional Ground Freezing

Due to the advancement of directional drilling techniques, freezing tubes can be installed at an angle. See Fig. 12.15.

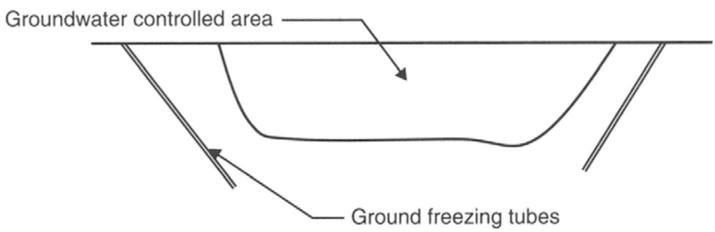


Figure 12.15 Ground freezing tubes installed at an angle

Ground Freezing for Underpinning

When excavations are planned near other buildings, underpinning of these buildings has to be conducted. Ground freezing can be used instead of underpinning in such situations. See Fig. 12.16.

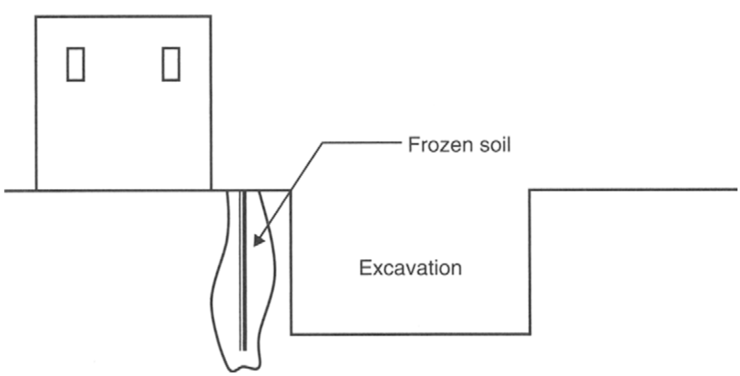


Figure 12.16 Stabilization of soil near existing buildings

12.3 Drain Pipes and Filter Design

Groundwater can create many problems for geotechnical engineers. Controlling groundwater is a challenge in many situations. Drainage in shallow foundations, basements, and retaining walls is done using drain pipes. See Figs. 12.17 and 12.18.

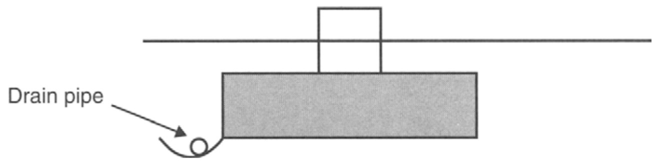


Figure 12.17 Drain pipe in a shallow foundation

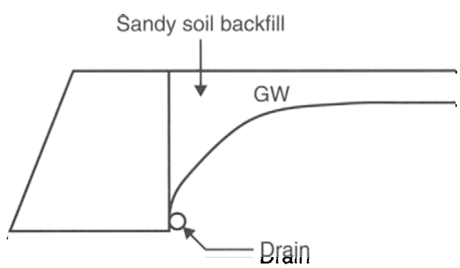


Figure 12.18 Drainage in retaining walls

Drain pipes tend to get clogged due to the migration of silt and clay particles. It is important to filter fine particles to avoid clogging.

Gravel filters have been one of the oldest techniques to filter fine particles. Gravel filters were very common prior to the arrival of geotextiles. See Fig. 12.19.

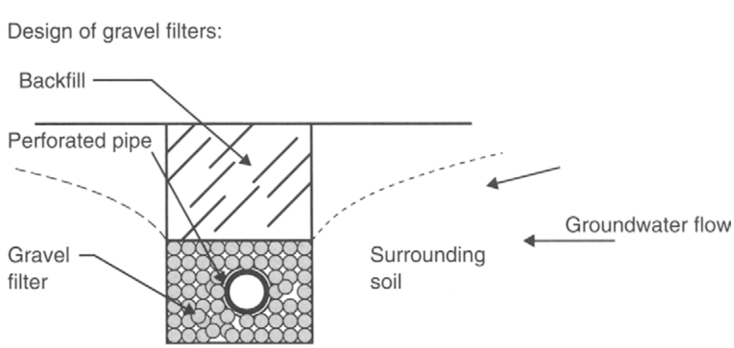


Figure 12.19 Gravel filter

12.3.1 Design of Gravel Filters

Purpose of Gravel Filters

Gravel filters stop any soil particles from entering the pipe. At the same time, the gravel allows ample water flow. If fine gravel is used, water flow can be reduced. On the other hand, large size gravel can allow soil particles to pass through and eventually clog the drain pipe.

Empirically it has been found that if the gravel size is selected in accordance with two rules given below, very little soil would transport through the gravel filter and at the same time let water flow smoothly.

RULE 1: To block soil from entering the pipe, the size of the gravel should be

$$D_{15}(\text{gravel}) < 5 \times D_{85}\,(\text{soil})$$

In this case, the D_{15} size means that 15% of particles of a given soil or gravel would pass through the D_{15} size of that particular soil or gravel. The D_{85} size means that 85% of particles of a given soil or gravel would pass through the D_{85} size of that particular soil or gravel.

In order to obtain the D_{15} size for a gravel, conduct a sieve analysis test and draw the sieve analysis curve. Draw a line across the 15% passing point. Find the D_{15} size using the sieve analysis curve.

For the filter to stop soil from washing away, the D_{15} value of gravel should be smaller than ($5 \times D_{85}$) of soil. However, the gravel size should not be too small. If the gravel size is too small, no water would pass through the gravel, which is not desirable.

Rule 2 is proposed to address that issue.

RULE 2: To let water flow, the size of the gravel should be

$$D_{50}\,(\text{gravel}) > 25 \times D_{50}\,(\text{soil})$$

It has been found that the D_{50} value of gravel should be larger than ($25 \times D_{50}$ of soil) in order for the water to drain properly.

When selecting a suitable gravel to be used as a filter, these two rules should be adhered to.

Design Example 12.1

Sieve analysis tests show that the surrounding soil has the following D_{85} and D_{50} values.

$D_{85} = 1.5$ mm
$D_{50} = 0.4$ mm

Find an appropriate gravel size for the filter.

Solution

First, apply Rule 1.

$$D_{15}(\text{gravel}) < 5 \times D_{85}\ (\text{soil})$$
$$D_{15}(\text{gravel}) < 5 \times 1.5$$
$$D_{15}(\text{gravel}) < 7.5$$

Hence the D_{15} size of the gravel should be less than 7.5 mm.

Next, follow Rule 2.

$$D_{50}(\text{gravel}) > 25 \times D_{50}$$
$$D_{50}(\text{gravel}) > 25 \times 0.4$$
$$D_{50}(\text{gravel}) > 20\ \text{mm}$$

Hence, select a gravel with a D_{15} value less than 7.5 mm and D_{50} value greater than 20 mm.

12.4 Geotextile Filter Design

Geotextile filters are becoming increasingly popular in drainage applications. Ease of use, economy, and the durability of geotextile filters have made them the number one choice of many engineers.

12.4.1 Geotextile Wrapped Granular Drains (Sandy Surrounding Soils)

Gravel is wrapped with a geotextile to improve the performance. See Fig. 12.20. The geotextile filters the water, and the gravel acts as the drain. There are two types geotextiles (woven and nonwoven) available in the market. For sandy soils, both woven and nonwoven geotextiles can be used. The geotextile is wrapped around stones and backfilled with original soil.

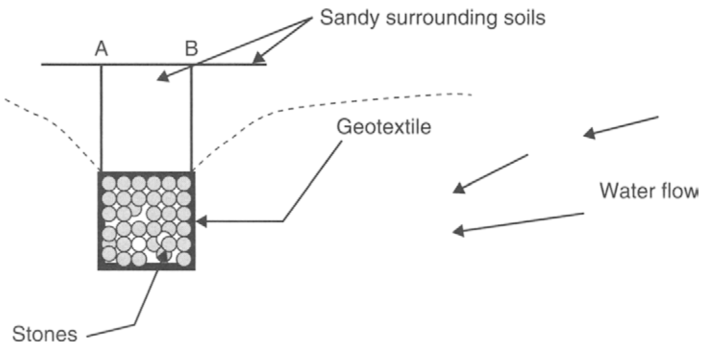


Figure 12.20 Geotextile wrapped drain filter

The stones act only as a medium to transport water. The task of stopping soil from entering the drain is done by the geotextile.

Equations have been developed for the two types of flow: one way flow and two way flow (also called alternating flow).

One Way Flow

In the case of one way flow, the flow of water is always in one direction across the geotextile. In Fig. 12.20, water enters the drain from the drain sides. Use the following equation for geotextiles in sandy soil (Zitscher, 1975).

$$H_{50}\,(\text{geotextile}) < 1.7 \text{ to } 2.7 \times D_{50}\,(\text{soil}) \quad \text{(for one way flow for sandy soils)}$$

H_{50} indicates that 50% of the holes in the geotextile are smaller than the H_{50} size. D_{50} (soil) indicates that 50% of soil particles are smaller than the D_{50} size.

Two Way Flow (Alternating Flow)

In some instances, water goes through the geotextile in both directions. In a heavy rain, water enters the drain from the top (between points A and B in Fig. 12.20), flows into the drain, and then flows out of the drain to the surrounding soil. If flow through the geotextile is possible in both directions, a different set of equations needs to be used (Zitscher, 1975).

$$H_{50}(\text{geotextile}) < (0.5 \text{ to } 1.0) \times D_{50}\,(\text{soil}) \quad \text{(for two way flow for sandy soils)}$$

Design Example 12.2

Design a geotextile wrapped granular filter drain for a sandy soil with $D_{50} = 0.2$ mm. It is determined that the flow across the geotextile is always in one direction.

Solution

STEP 1:

$$H_{50}(\text{geotextile}) < 1.7 \text{ to } 2.7 \times D_{50}\,(\text{soil}) \quad \text{(for one way flow for sandy soils)}$$

The above equation is valid for one way flow in sandy soil. Above H_{50} indicates that 50% of the holes in the geotextile are smaller than H_{50} size. D_{50} (soil) indicates that 50% of soil particles are smaller than D_{50} size. Since D_{50} is found to be 0.2 mm, H_{50} can be computed.

$$H_{50}(\text{geotextile}) < (1.7 \text{ to } 2.7) \times 0.2\,\text{mm}$$

H_{50} should be between 0.34 mm and 0.54 mm. Hence, select a geotextile with a H_{50} size of 0.4 mm.

Alternating Flow in Sandy Soils

In some situations, flow can be in both directions. In such situations, the procedure in the following example should be adopted.

Design Example 12.3

Design a geotextile wrapped granular filter drain for a sandy soil with $D_{50} = 0.2$ mm. It is determined that the flow across the geotextile could alternate.

Solution

STEP 1: For two way flow, find the H_{50} from the two way flow equation.

$$H_{50}(\text{geotextile}) < (0.5 \text{ to } 1.0) \times D_{50}\,(\text{soil}) \quad \text{(for two way flow for sandy soils)}$$

Since D_{50} was found to be 0.2 mm

$$H_{50}(\text{geotextile}) < (0.5 \text{ to } 1.0) \times 0.2\,\text{mm}$$

H_{50} should be between 0.1 mm and 0.2 mm. Select a geotextile with an H_{50} size equal to 0.15 mm.

12.4.2 Geotextile Wrapped Granular Drains (Clayey Surrounding Soils)

The theory behind using geotextile wrapped granular drains is that for cohesive soils, the direction of flow is not significant since the flow is not rapid as in sandy soils. Most engineers prefer to use non-woven geotextiles for cohesive soils. For these soils, use the equation (Zitscher, 1975)

$$H_{50}(\text{geotextile}) < (25 \text{ to } 37) \times D_{50}\,(\text{soil}) \quad (\text{one way and two way flow for clayey soils})$$

Design Example 12.4

Design a geotextile wrapped granular filter drain for a cohesive soil with $D_{50} = 0.01$ mm.

Solution

$$H_{50}(\text{geotextile}) < (25 \text{ to } 37) \times D_{50}\,(\text{soil}) \quad (\text{one way and two way flow for clayey soils})$$

The above equation is valid for cohesive surrounding soils for any type of flow.

$$H_{50}(\text{geotextile}) < (25 \text{ to } 37) \times 0.01 \text{ mm}$$

Since D_{50} is equal to 0.01 mm, H_{50} should be between 0.25 mm and 0.37 mm. Select a geotextile with H_{50} size equal to 0.30 mm.

12.4.3 Geotextile Wrapped Pipe Drains

On some occasions, flow through granular media alone is not enough. See Fig. 12.21. In such situations, a perforated pipe is used at the center, as shown in Fig. 12.21. The size of the pipe depends on the flow rate required.

The design of the geotextile in this case is very similar to the previous cases if the surrounding soil is sand and only one way flow is expected.

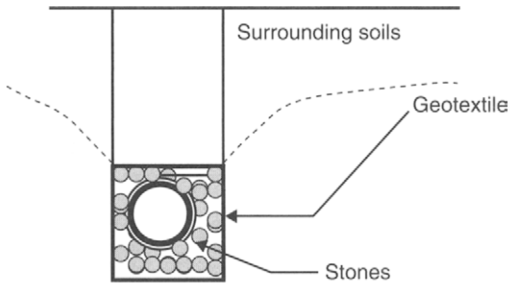


Figure 12.21 Geotextile wrapped pipe drain

12.5 Summary

Sandy soils (one way flow)

$$H_{50}(\text{geotextile}) < 1.7 \text{ to } 2.7 \times D_{50}\,(\text{soil})$$

Sandy soils (two way flow)

$$H_{50}(\text{geotextile}) < 0.5 \text{ to } 1.0 \times D_{50}\,(\text{soil})$$

Clayey soils (one way and two way flow)

$$H_{50}(\text{geotextile}) < 25 \text{ to } 37 \times D_{50}\,(\text{soil})$$

References

Rankilor, P. R. 1981. *Membranes in ground engineering*. New York: John Wiley & Sons.
Xanthakos, P. 1994. *Ground control and improvement*. New York: John Wiley & Sons.
Zitscher, F. F. 1975. Recommendations for the use of plastics in soil and hydraulic engineering. *Die Bautechnik*, 52(12):397–402.

13

Selection of Foundation Type

There are a number of foundation types available for geotechnical engineers: shallow foundations, mat foundations, pile foundations, and caisson foundations.

13.1 Shallow Foundations

Shallow foundations are the cheapest and most common type of foundation. See Fig. 13.1.

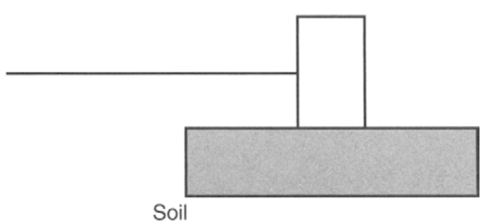


Figure 13.1 Shallow foundation

Shallow foundations are ideal for situations when the soil immediately below the footing is strong enough to carry the building loads. In some situations, the soil immediately below the footing could be weak or compressible. In such situations, other foundation types need to be considered.

13.2 Mat Foundations

Mat foundations are also known as raft foundations. Mat foundations, as the name implies, spread like a mat. The building load is distributed over a large area. See Figs. 13.2 and 13.3.

Figure 13.2 Mat foundations

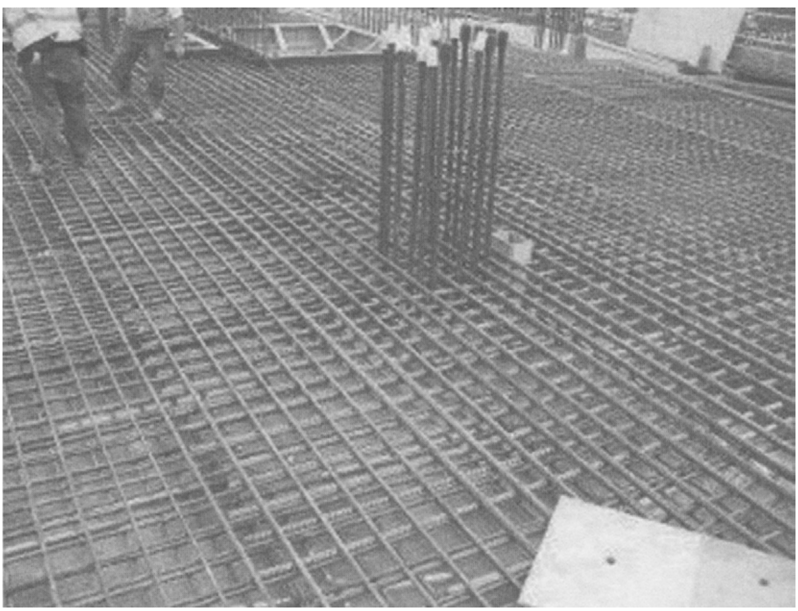


Figure 13.3 Rebars for a mat foundation
Source: http://www.integer-software.co.uk

13.3 Pile Foundations

Piles are used when the load-bearing soil is at a greater depth. In such situations, the load has to be transferred to the load-bearing soil stratum. See Fig. 13.4.

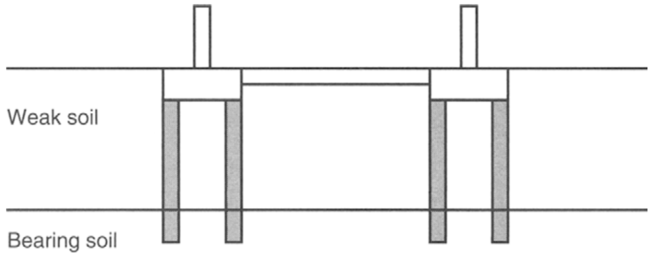


Figure 13.4 Pile foundation

13.4 Caissons

Caissons are nothing but larger piles. Instead of a pile group, a few large caissons can be utilized. In some situations, caissons could be the best alternative. See Fig. 13.5.

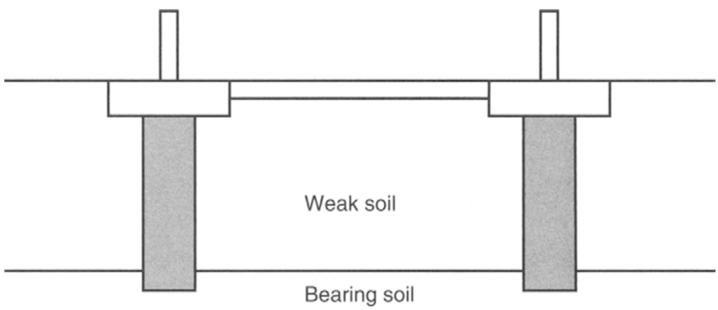


Figure 13.5 Caissons

13.5 Foundation Selection Criteria

Normally every attempt is made to construct shallow foundations. This is the cheapest and fastest foundation type. The designer should look into bearing capacity and settlement when considering shallow foundations.

The geotechnical engineer needs to compute the bearing capacity of the soil immediately below the footing. If the bearing capacity is adequate, settlement needs to be computed. Settlement can be immediate or long term. Both immediate and long term settlements should be computed.

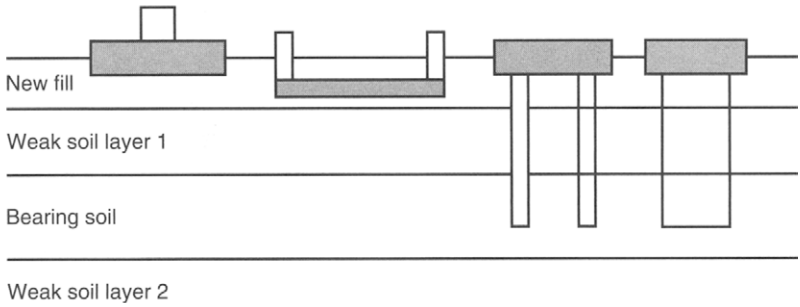


Figure 13.6 Different foundation types

Figure 13.6 shows a shallow foundation, mat foundation, pile group, and a caisson. The geotechnical engineer needs to investigate the feasibility of designing a shallow foundation, due to its cheapness and ease of construction. In the above situation, it is clear that the weak soil layer just below the new fill may not be enough to support the shallow foundation. Settlement in weak soil due to loading of the footing also needs to be computed.

If shallow foundations are not feasible, then other options need to be investigated. Mat foundations can be designed to carry large loads in the presence of weak soils. Unfortunately, cost is a major issue with mat foundations.

Piles could be installed as shown in Fig. 13.7, ending in the bearing stratum. In this situation, it is important to be careful of the second weak layer of soil below the bearing stratum. Piles could fail due to punching into the weak stratum.

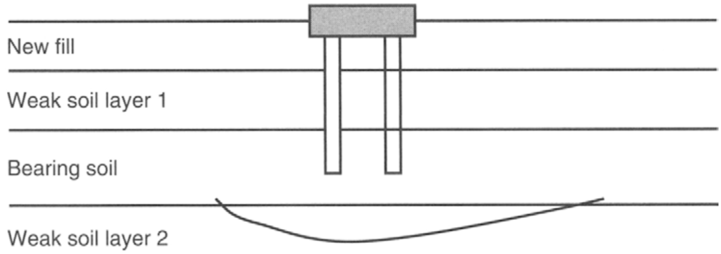


Figure 13.7 Punching failure (soil punching into the weak soil below due to pile load)

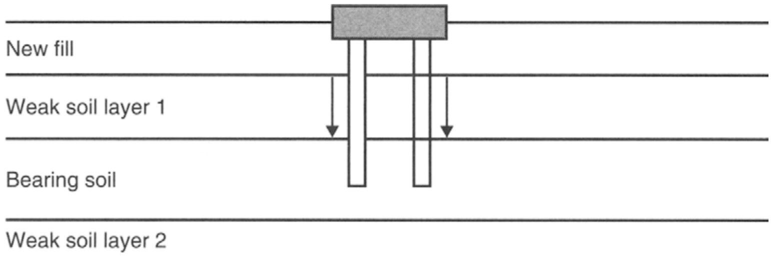


Figure 13.8 Negative skin friction

The engineer needs to consider negative skin friction due to the new fill layer. Negative skin friction would reduce the capacity of piles, as shown in Fig. 13.8. Due to the new load of the added fill material, weak soil layer 1 would consolidate and settle. The settling soil would drag piles down with it. This is known as negative skin friction or down drag.

14

Consolidation

14.1 Introduction

What happens when a foundation is placed on top of a clay layer? It is obvious that the clay layer will compress and, as a result, the foundation will settle. Why does the foundation settle due to an applied load?

Clay is a mixture of soil particles, water, and air. When a clay soil is loaded, the water particles inside the soil mass are pressurized and tend to squeeze out. See Fig. 14.1.

The settlement due to water squeezing out is known as primary consolidation.

The squeezing out of water is caused by the dissipation of excess pore pressure. When all the water in the soil is squeezed out, primary consolidation has been achieved.

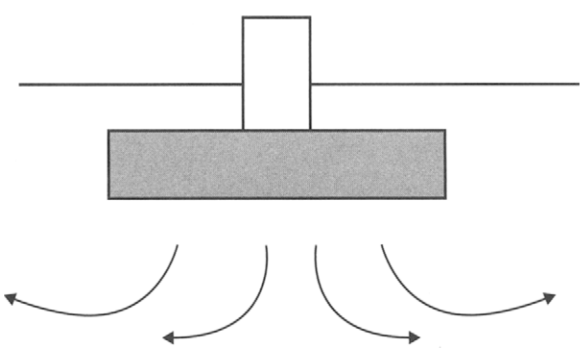


Figure 14.1 Water squeezing out from the soil

In reality, 100% primary consolidation usually can never be achieved. For all practical purposes, 90% primary consolidation is taken as the end of the primary consolidation process. See Fig. 14.2 for the dramatic result of clay consolidation as shown by the Leaning Tower of Pisa.

Figure 14.2 Tower of Pisa (clay consolidation)

14.1.1 Secondary Compression

After primary consolidation comes the phenomenon of secondary compression.

Once all the water is squeezed out, it might be expected that settlement would stop. Interestingly, that is not the case. Settlement continues even after all the excess pore water is squeezed out. This phenomenon, secondary compression, is due to the fact that soil particles start to rearrange their orientation. See Fig. 14.3.

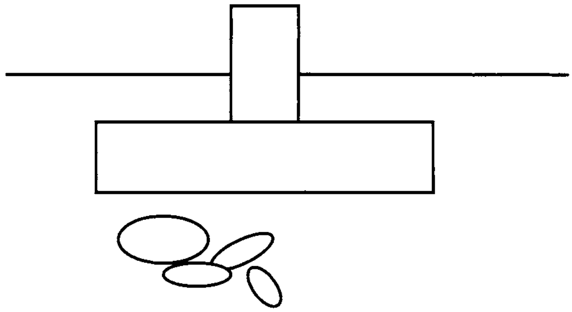


Figure 14.3 Soil particles below a footing

What does this mean?

These soil particles settle to rearrange into a more stable configuration. See Fig. 14.4.

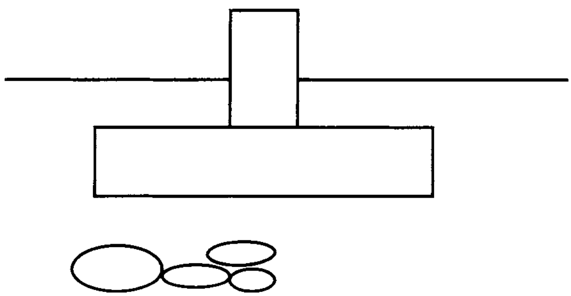


Figure 14.4 Stable rearrangement

Rearrangement of soil particles thus creates further settlement of the foundation. In reality, secondary compression starts as soon as the foundation is constructed. But for simplicity, the usual practice is to compute the secondary compression after the primary consolidation is completed.

14.1.2 Summary of Concepts Learned

- Primary consolidation occurs due to dissipation of excess pore pressure.
- In reality, the primary consolidation never ends. For all practical purposes, 90% consolidation is taken as the end of the process.

- Secondary compression occurs due to rearrangement of soil particles, which is due to the load. Secondary compression has nothing to do with the pore water pressure.

14.2 Excess Pore Pressure Distribution

Figure 14.5 shows a group of piezometers installed at increasing depth in the soil close to a shallow foundation. Piezometers measure fluid pressure.

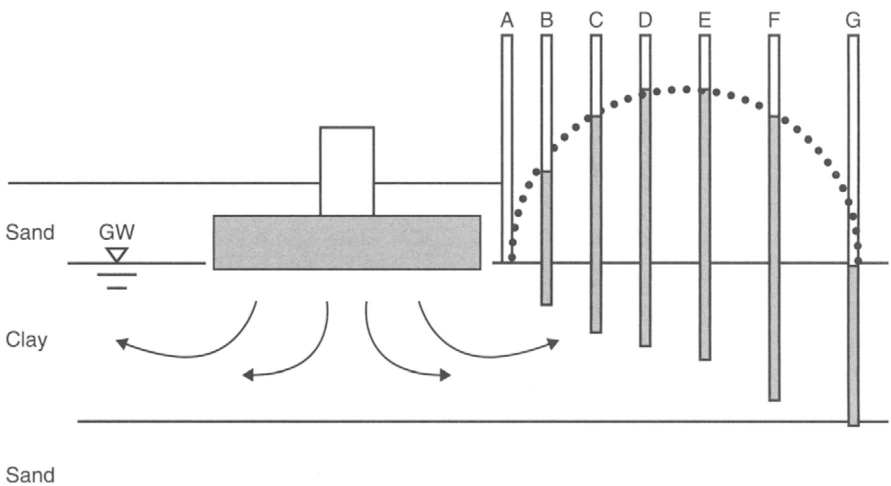


Figure 14.5 Pore pressure distribution near a loaded footing

Piezometer A is located at the same level as the bottom of the footing in the sand layer and Piezometer G is the deepest measuring device. What information does each piezometer report?

Piezometer A: Any excess pore pressure generated due to the footing load would be dissipated immediately, since the sand is highly permeable.

Piezometer B: The bottom of piezometer B is at a lower depth than piezometer A. Excess pore pressure will not dissipate immediately, since water particles have to travel upward toward the sand layer. This would take some time, depending upon the permeability of the clay layer.

Piezometers C, D, and E: Piezometers C, D, and E are lower than piezometer B. Hence, more time is required for the dissipation of pore pressure.
Piezometer F: The water level of piezometer F has dropped compared to piezometer E. This is due to the fact that water could dissipate to the sand layer below the clay layer.
Piezometer G: Excess pore pressure in piezometer G will dissipate immediately, as in the case of piezometer A, due to the proximity of the bottom sand layer.

Dissipation of excess pore pressure usually takes months, or in some cases years, to complete. Clays with low permeability would take a long time to complete primary consolidation. As mentioned earlier, 100% primary consolidation would usually never be achieved.

14.3 Normally Consolidated Clays and Overconsolidated Clays

Settlement due to consolidation occurs in clayey soils. Clay soils usually originate in lake beds or ocean floors. Sedimentation of clay particles occurs in calm waters. See Fig. 14.6.

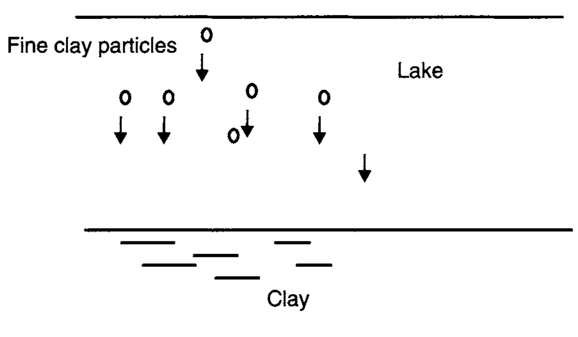


Figure 14.6 Formation of a clay layer due to sedimentation

Years down the road, the lake dries up and the clay layer is exposed. See Fig. 14.7.

The newly formed clay layer was never subjected to any loads other than the load due to the body of water. Such clays are known as

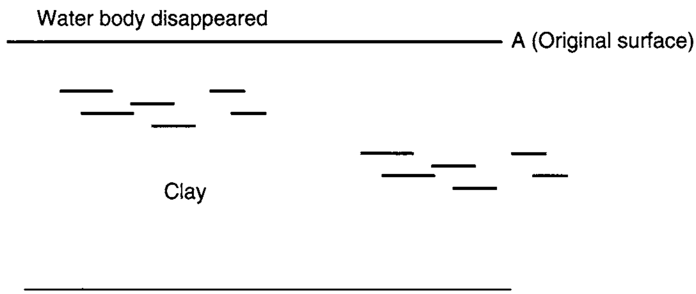


Figure 14.7 Normally consolidated clay

normally consolidated clays. The earth is a dynamic planet. Changes occur on the earth's surface every day. Hurricanes, tsunamis, landslides, the coming and going of glaciers, and volcanoes are some of the events that occur on the earth's surface. During the last ice age, a large area of northern hemisphere was under a few hundred feet of ice. When glaciers are formed during an ice age, the clays in that region are subjected to a tremendous load. Due to the ice load, the clay undergoes settlement. See Fig. 14.8.

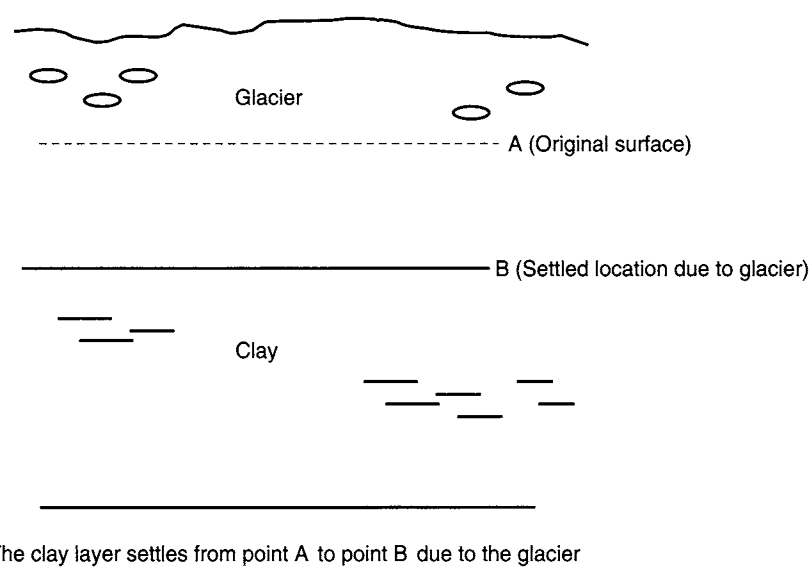


Figure 14.8 Overconsolidation process

Glaciers normally melt away and disappear at the end of ice ages. Today, glaciers can be seen only at the North Pole and the South Pole, or in high mountain ranges. After the melting of the glaciers, the load on the clay is released. Hence, the clay layer rebounds. Clays that had been subjected to high pressures in the past are known as preconsolidated clays or overconsolidated clays. See Fig. 14.9.

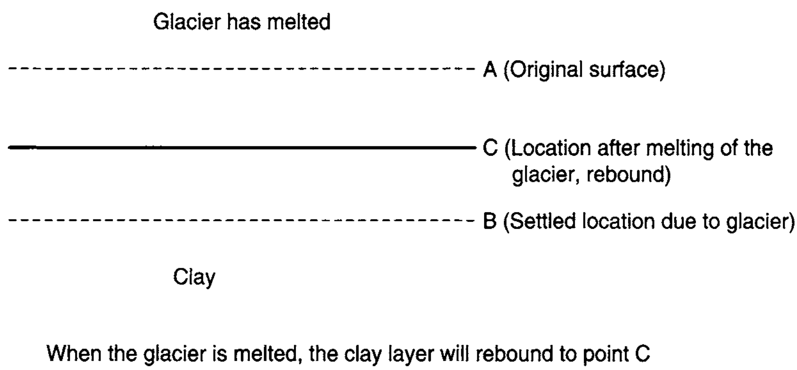


Figure 14.9 Overconsolidated clay

Now assume that a footing has been placed on the clay layer. The clay layer with the newly placed footing is shown in Fig. 14.10.

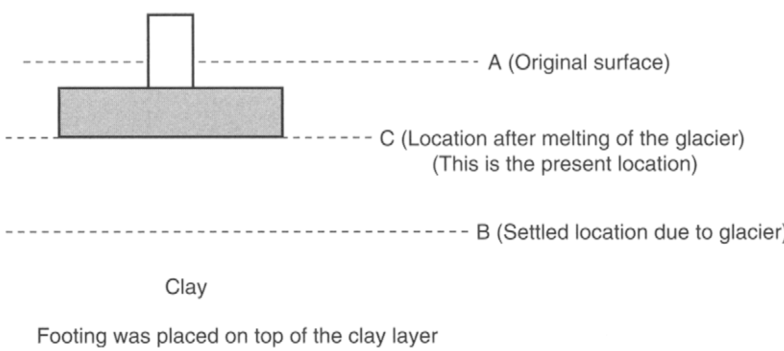


Figure 14.10 Footing load on overconsolidated clay

The clay layer then undergos settlement due to the footing load. The new location of the top surface of the clay layer is shown in Fig. 14.11.

Figure 14.12 shows the change in void ratio, e, due to the settlement of the footing.

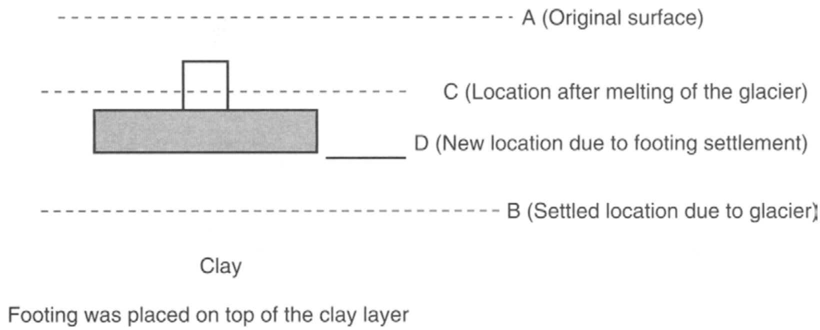


Figure 14.11 Footing settlement

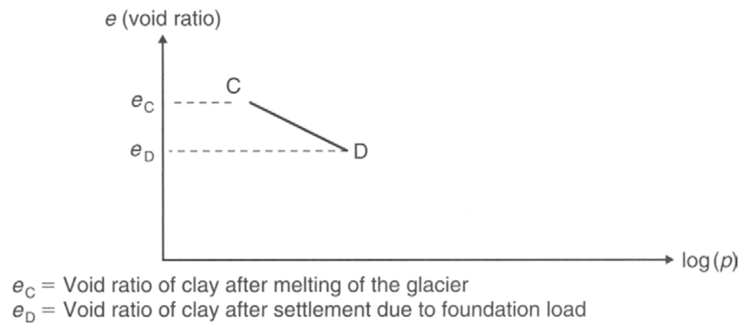


Figure 14.12 Change of void ratio

Now assume that the footing load is increased. When the footing load is increased, the clay layer settles further. If the load is large enough, the clay layer settles beyond point B. Point B is the surface of the clay layer during the period in which the glaciers were present. Figure 14.13 shows the new location of the settled footing at point D. See Fig. 14.14 for a graph showing the further change in void ratio, e.

When the load is increased, the void ratio of clay decreases due to settlement.

In the above example, in the time when the glacier was present, the void ratio of the clay layer was e_B. When the glacier melted

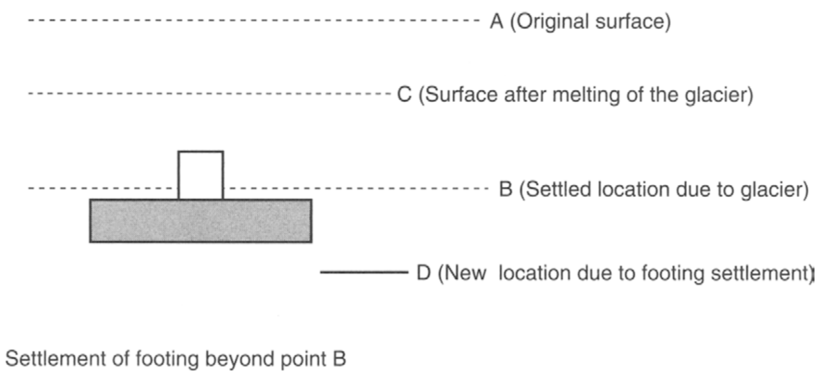


Figure 14.13 Settlement of footing beyond previous level

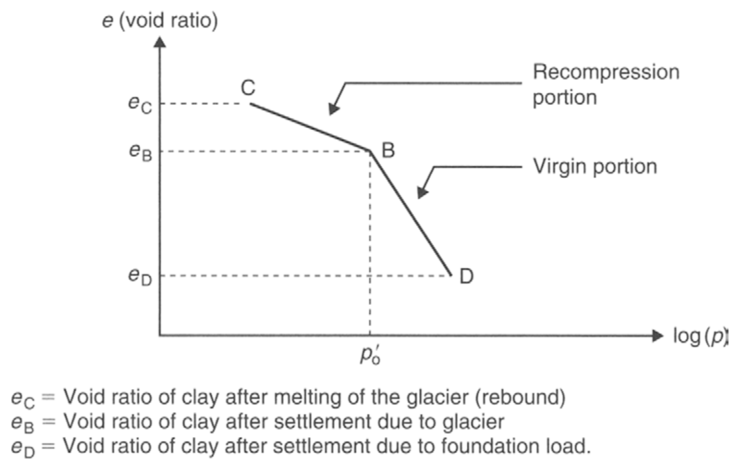


Figure 14.14 Void ratio after consolidation due to footing load

away, the clay layer rebounded. Hence, the voids inside the clay layer increased. After the melting of the glacier, the void ratio increased from e_B to e_C.

This was the state of the clay layer until a footing was placed there. When the footing load was increased, the void ratio decreased. As soon as the void ratio goes below e_B (void ratio when the glacier was present) the curve bends.

The top portion of the e vs $\log(p)$ curve shown in Fig. 14.14 is known as the recompression curve, and the bottom portion is known as the virgin curve.

C to B: Recompression curve
B to D: Virgin curve

The gradient of the virgin curve is steeper than the recompression curve.

The gradient of the recompression curve is usually denoted by C_r and the gradient of the virgin curve is denoted by C_c. The settlement is high for a given pressure within the virgin portion of the graph.

The pressure at point B is the largest pressure that the clay layer was subjected to prior to placing the foundation. The pressure at point B is known as the preconsolidation pressure and is denoted by p'_c. The stress prior to placement of the footing is usually denoted by p'_0.

14.4 Total Primary Consolidation

When a clay layer is loaded, as we learned in the previous section, it starts to consolidate and settle. See Fig. 14.15.

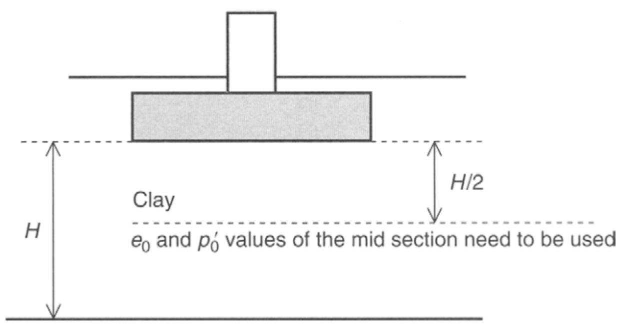


Figure 14.15 Shallow foundation on clay soil (properties of midsection)

Settlement due to total primary consolidation in normally consolidated clay is given by the following equation. Please note that normally consolidated clays have never been subjected to a higher stress than they have now.

$$\Delta H = H \times C_c/(1 + e_0) \times \log\left(\frac{(p'_0 + \Delta p)}{p'_0}\right)$$

where

ΔH = total primary consolidation settlement
H = thickness of the clay layer
C_c = compression index of the clay layer
e_0 = void ratio of the clay layer at the midpoint of the clay layer prior to loading
p'_0 = effective stress at the midpoint of the clay layer prior to loading
Δp = increase of stress at the midpoint of the clay layer due to the footing. See Fig. 14.16.
e_0 = present void ratio
e_f = final void ratio after consolidation due to the footing load
p'_0 = present vertical effective stress at the midpoint of the clay layer
p'_f = final vertical effective stress at the midpoint of the clay layer after placement of the footing

$$\Delta p = p'_f - p'_0$$

C_c = gradient of the curve (compression index)

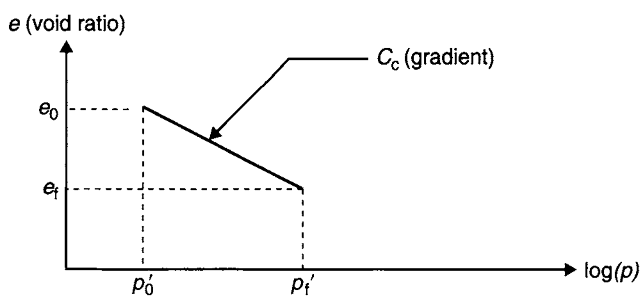


Figure 14.16 Change of void ratio for normally consolidated clay

Design Example 14.1

This example explores consolidation settlement in normally consolidated clay, for the no groundwater case. Find the settlement due to consolidation of a 3 m × 3 m column foundation with a load of 200 kN. The foundation is placed 1 m below the top surface, and the clay layer is

9 m thick. There is a sand layer underneath the clay layer. The density of the clay layer is 18 kN/m^3, the compression index, C_c, of the clay layer is 0.32, and the initial void ratio, e_0, of the clay is 0.80. Assume that the pressure is distributed at a 2:1 ratio and the clay is normally consolidated. See Fig. 14.17.

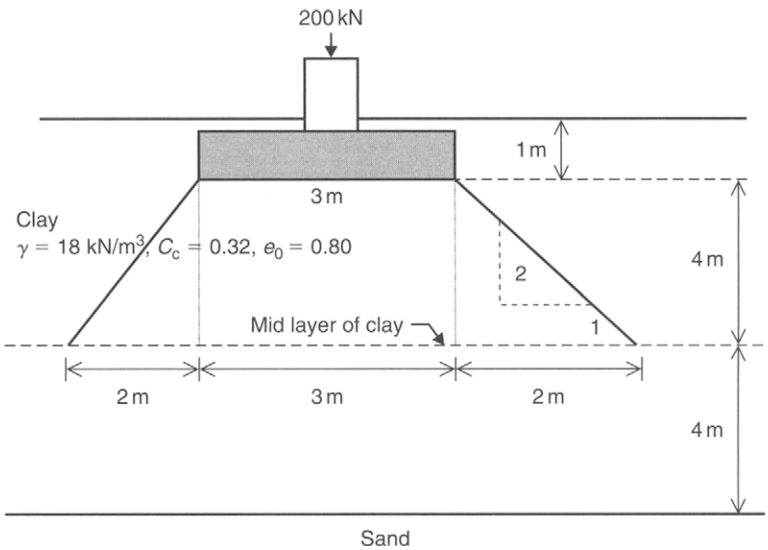


Figure 14.17 Footing on normally consolidated clay layer

Solution

STEP 1: Write down the consolidation settlement equation

$$\Delta H = H \times C_c/(1 + e_0) \times \log\left(\frac{(p_0' + \Delta p)}{p_0'}\right)$$

ΔH = total primary consolidation settlement
H = thickness of the compressible clay layer
C_c = compression index of the clay layer
e_0 = void ratio of the clay layer at the midpoint of the clay layer prior to loading
p_0' = effective stress at the midpoint of the clay layer prior to loading

STEP 2: The clay layer is 9 m thick and the footing is placed 1 m below the surface. The top 1 m is not subjected to consolidation. A clay layer with a thickness of 8 m below the footing is subjected to consolidation due to footing load. Find the effective stress at the mid layer of the compressible clay stratum, p'_0.

$$p'_0 = \gamma_{\text{clay}} \times 5$$
$$p'_0 = 18 \times 5 = 90\,\text{kN/m}^2$$

STEP 3: Find the increase of stress, Δp, at the midpoint of the clay layer due to the footing. The total load of 100 kN is distributed at a larger area at the midsection of the clay layer.

$$\text{area of the midsection of the clay layer} = (2 + 3 + 2) \times (2 + 3 + 2) = 49\,\text{m}^2$$

$$\Delta p = 200/49 = 4.1\,\text{kN/m}^2$$

STEP 4: Apply values in the consolidation equation.

$$\Delta H = H \times C_c/(1 + e_0) \times \log\left(\frac{(p'_0 + \Delta p)}{p'_0}\right)$$

The following parameters are given:

$C_c = 0.32$
$e_0 = 0.8$

$$\Delta H = 8 \times 0.32/(1 + 0.8)\log(90 + 4.1)/90$$
$$\Delta H = 0.0275\,\text{m (1 in.)}$$

Note that H, the thickness of the clay layer, should be calculated starting from the bottom of the footing. Although the total thickness of the clay layer is 9 m, the first 1 m of the clay layer is not compressed.

In the next example, groundwater is considered.

Design Example 14.2

This example covers consolidation settlement in normally consolidated clay, in the case where groundwater is present. Find the settlement due to consolidation of a 3 m × 3 m column foundation with a load of 180 kN. The sand layer is 4 m thick and the clay layer below is 10 m thick. The density of the sand layer is $17.5\,kN/m^3$, and the density of the clay layer is $18\,kN/m^3$. The groundwater level is 1.5 m below the surface. The compression index, C_c, of the clay layer is 0.3, and the initial void ratio, e_0, of the clay is 0.8. Assume that the pressure is distributed at a 2:1 ratio and the clay is normally consolidated. See Fig. 14.18.

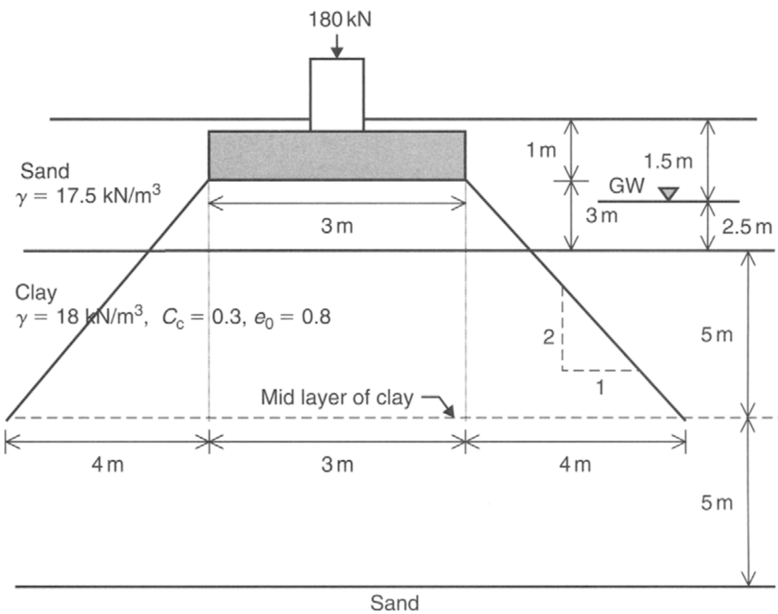


Figure 14.18 Shallow foundation on clay soil (parameters)

Solution

STEP 1: Write down the consolidation settlement equation

$$\Delta H = H \times C_c/(1 + e_0) \times \log\left(\frac{(p'_0 + \Delta p)}{p'_0}\right)$$

where

ΔH = total primary consolidation settlement
H = thickness of the clay layer

C_c = compression index of the clay layer
e_0 = void ratio of the clay layer at the midpoint of the clay layer prior to loading
p'_0 = effective stress at the midpoint of the clay layer prior to loading

STEP 2: Find the initial effective stress at the mid layer of the clay stratum, p'_0.

$$p'_0 = \gamma_{\text{sand}} \times 1.5 + (\gamma_{\text{sand}} - \gamma_{\text{water}}) \times 2.5 + (\gamma_{\text{clay}} - \gamma_{\text{water}}) \times 5$$
$$p'_0 = 17.5 \times 1.5 + (17.5 - 9.807) \times 2.5 + (18 - 9.087) \times 5$$
$$p'_0 = 86.5\,\text{kN/m}^2$$

STEP 3: Find the increase of stress, Δp, at the midpoint of the clay layer due to the footing. The total load of 180 kN is distributed at a larger area at the midsection of the clay layer.

$$\text{area of the midsection of the clay layer} = (4 + 3 + 4) \times (4 + 3 + 4) = 121\,\text{m}^2$$

$$\Delta p = 180/121 = 1.5\,\text{kN/m}^2$$

STEP 4: Apply the values in the consolidation equation.

$$\Delta H = H \times C_c/(1 + e_0) \times \log\left(\frac{(p'_0 + \Delta p)}{p'_0}\right)$$
$$\Delta H = 10 \times 0.3/(1 + 0.8)\log(86.5 + 1.5)/86.5$$
$$\Delta H = 0.0124\,\text{m}\ (0.5\,\text{in.})$$

14.5 Consolidation in Overconsolidated Clay

The e vs $\log p$ curve for preconsolidated clay is shown in Fig. 14.19. The symbols used in Fig. 14.19 are defined as follows.

e_0 = initial void ratio
e_c = void ratio at preconsolidation pressure
e_f = final void ratio at the end of the consolidation process.
p'_0 = initial pressure prior to the footing load

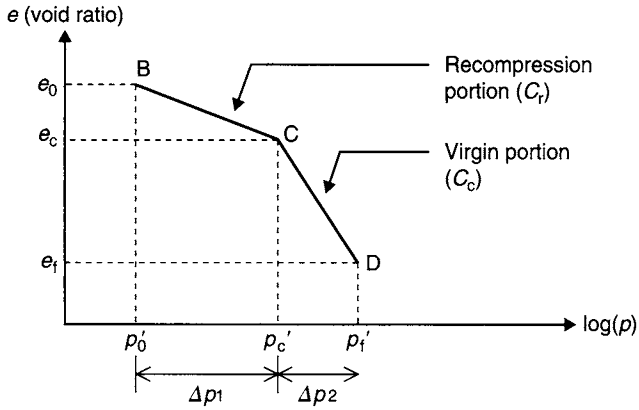


Figure 14.19 *e* vs log*p* curve for overconsolidated clay

p_c' = overconsolidation pressure (also known as preconsolidation pressure)
p_f' = pressure after the footing is placed
C_c = compression index
C_r = recompression index
$\Delta H = \Delta H_1 + \Delta H_2$
ΔH_1 = settlement from B to C (C_r)
ΔH_2 = settlement from C to D (C_c)

$$\Delta H = H \times C_r/(1 + e_0) \times \log\left(\frac{(p_0' + \Delta p_1)}{p_0'}\right)$$
$$+ H \times C_c/(1 + e_0) \times \log\left(\frac{(p_c' + \Delta p_2)}{p_c'}\right)$$

In this equation, point B indicates the present void ratio and existing vertical effective stress, and point C indicates the maximum stress that the clay had been subjected to in the past. The void ratio and the past maximum stress have to be obtained by conducting a laboratory consolidation test. The geotechnical engineer should obtain Shelby tube samples and send them to the laboratory to conduct a consolidation test. Point D indicates the expected stress after the footing is constructed.

Unlike in the normally consolidated soils, in preconsolidated soils, the settlement has to be computed in two parts.

The consolidation settlement from B to C is computed as

$$\Delta H_1 = H \times C_r/(1 + e_0) \times \log\left(\frac{(p_0' + \Delta p_1)}{p_0'}\right)$$
$$\Delta p_1 = p_c' - p_0'$$

and the consolidation settlement from C to D is computed as

$$\Delta H_2 = H \times C_c/(1 + e_0) \times \log\left(\frac{(p'_c + \Delta p_2)}{p'_c}\right)$$

$$\Delta p_2 = p'_f - p'_c$$

Hence the total settlement is given by

$$\Delta H = \Delta H_1 + \Delta H_2$$

Design Example 14.3

This example concerns consolidation settlement in overconsolidated clay, in the case where groundwater is not considered. Find the settlement due to consolidation in a 3 m × 3 m column foundation with a load of 270 kN. The foundation is placed 1 m below the ground surface, and the clay layer is 9 m thick. There is a sand layer underneath the clay layer. The density of the clay layer is 18 kN/m^3, the compression index, C_c, of the clay layer is 0.32, the recompression index, C_r, is 0.035, the preconsolidation pressure (p'_c) is 92 kN/m^2, and the initial void ratio, e_0, of clay is 0.80. Assume that the pressure is distributed at a 2:1 ratio and the clay is normally consolidated. See Fig. 14.20.

Solution

STEP 1: Write down the consolidation settlement equation.

$$\Delta H = H \times C_r/(1 + e_0) \times \log\left(\frac{(p'_0 + \Delta p_1)}{p'_0}\right)$$

$$+ H \times C_c/(1 + e_0) \times \log\left(\frac{(p'_c + \Delta p_2)}{p'_c}\right)$$

Recompression portion (C_r) Virgin compression (C_c)

where

ΔH = total primary consolidation settlement
H = thickness of the compressible clay layer
C_c = compression index of the clay layer (virgin portion)
C_r = recompression index of the clay layer (recompression portion)

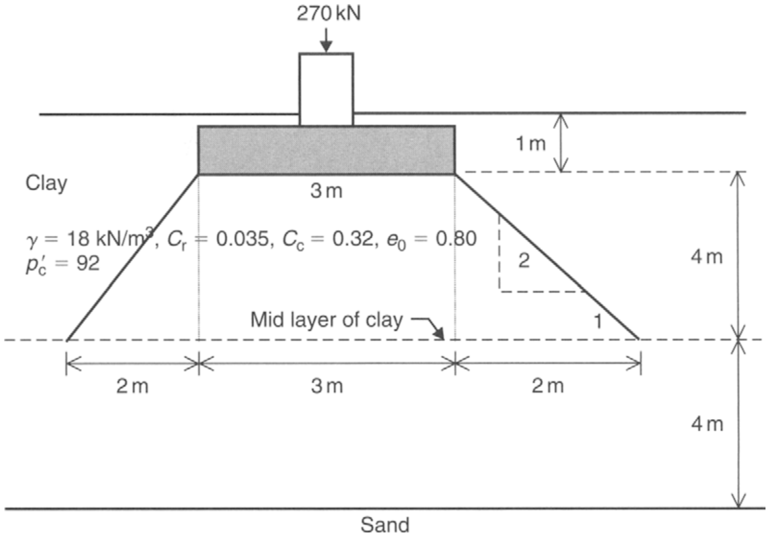


Figure 14.20 Settlement in overconsolidated clay

ΔH_1 = settlement in recompression portion
ΔH_2 = settlement in virgin portion
e_0 = void ratio of the clay layer at the midpoint of the clay layer prior to loading
p'_0 = effective stress at the midpoint of the clay layer prior to loading
p'_c = preconsolidation pressure
p'_f = pressure after the footing is placed

STEP 2: The clay layer is 9 m thick and the footing is placed 1 m below the surface. The top 1 m is not subjected to consolidation. A clay layer with a thickness of 8 m below the footing is subjected to consolidation due to footing load.

Find the effective stress at the mid layer of the compressible clay stratum, p'_0

$$p'_0 = \gamma_{\text{clay}} \times 5$$
$$= 18 \times 5 = 90\,\text{kN/m}^2$$

STEP 3: Find the increase of stress, Δp, at the midpoint of the clay layer due to the footing. The total load of 270 kN is distributed at a larger area at the midsection of the clay layer.

area of the midsection of the clay layer $= (2+3+2) \times (2+3+2) = 49\,\text{m}^2$

$$\Delta p = 270/49 = 5.5\,\text{kN/m}^2$$

$$\text{existing stress, } p'_0 = 90\,\text{kN/m}^2$$

$$\text{preconsolidation stress is given as} = 92\,\text{kN/m}^2$$

$$\text{final pressure after foundation load, } p'_f = p'_0 + \Delta p = 90 + 5.5 = 95.5\,\text{kN/m}^2$$

STEP 4: Apply the values in the consolidation equation.

$$\Delta H = \Delta H_1 + \Delta H_2$$

$$\Delta H = H \times C_r/(1+e_0) \times \log\left(\frac{(p'_0 + \Delta p_1)}{p'_0}\right) + H \times C_c/(1+e_0) \times \log\left(\frac{(p'_c + \Delta p_2)}{p'_c}\right)$$

where

p'_0 = effective stress at the midpoint of the clay layer prior to loading $= 90\,\text{kN/m}^2$
Δp_1 = stress increase from initial stress, p'_0 to preconsolidation pressure p'_c
Δp_2 = stress increase from preconsolidation pressure, p'_c, to final pressure, p'_f

Since p'_0 is 90 and p'_c was given as 92, Δp_1 is $2\,\text{kN/m}^2$. Since p'_c is 92 and the final pressure, p'_f was determined as 95.5, Δp_2 is $3.5\,\text{kN/m}^2$. Therefore ΔH can be calculated as

$$\Delta H = 8 \times 0.035/(1+0.8)\log(90+2)/90 + 8 \times 0.32/(1+0.8) \log(92+3.5)/92$$

For the line diagram showing ΔH, see Fig. 14.21.

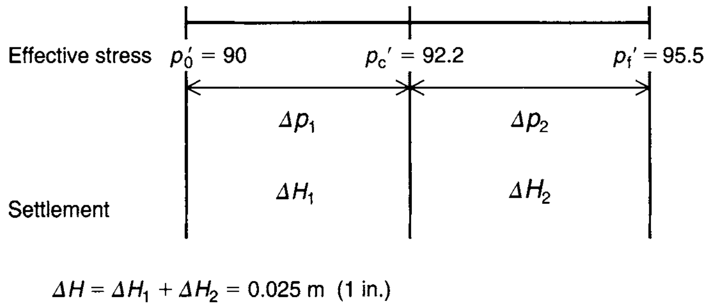


Figure 14.21 Line diagram

14.6 Computation of Time for Consolidation

In the previous chapter, we studied how to compute the total settlement due to primary consolidation. In this chapter, we discuss how to compute the time taken for the consolidation process.

The time taken for primary consolidation is given by the following equation.

$$t = \frac{H^2 \times T_V}{c_V}$$

where

t = time taken for the consolidation process
H = thickness of the drainage layer (discussed in the explanation below)
T_V = time coefficient
c_V = consolidation coefficient

T_V can be obtained from Table. 14.1 if $U\%$, the percent consolidation, is known.

14.6.1 Drainage Layer (*H*)

The thickness of the drainage layer is defined as the longest path a water molecule has to take for drainage. See Fig. 14.22.

In the case of single drainage, drainage can occur only from one side. In Fig. 14.22, water cannot drain through the rock. Water can drain

Table 14.1 $U\%$ and T_V

$U\%$ (percent consolidation)	T_V
0	0.00
10	0.048
20	0.090
30	0.115
40	0.207
50	0.281
60	0.371
70	0.488
80	0.652
90	0.933
100	1.0 (approximately)

from the sand layer on top. In this case, the thickness of the drainage layer $= H$. See Fig. 14.22.

The water molecule in Fig. 14.23 has the opportunity to drain either from the top or from the bottom. The longest drainage path for a water molecule is only $H/2$.

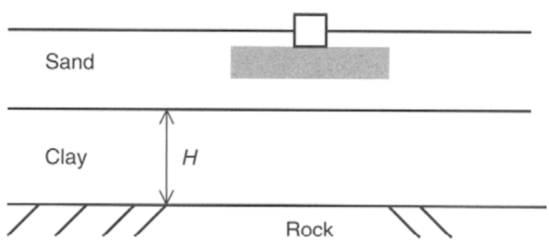


Figure 14.22 Thickness of the drainage layer (single drainage)

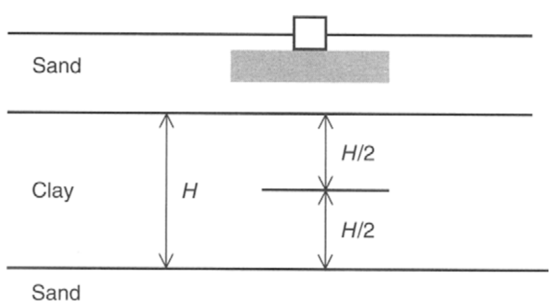


Figure 14.23 Thickness of the drainage layer (double drainage)

Design Example 14.4

Find the approximate time taken for 100% consolidation in the clay layer shown in Fig 14.24.

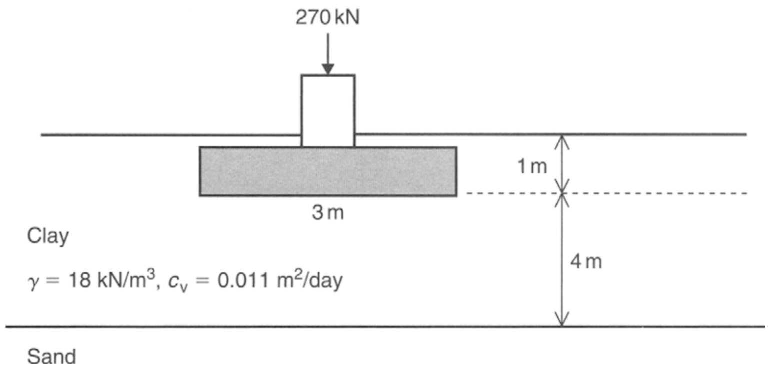


Figure 14.24 Consolidation in clay (double drainage)

Solution

The time taken for consolidation is given by

$$t = \frac{H^2 \times T_V}{c_V}$$

where

t = time taken for the consolidation process
H = thickness of the drainage layer
T_V = time coefficient

From Table 14.1, the approximate T_V value for 100% consolidation is 1.0. The value of c_V is given as 0.011 m^2/day.

The clay layer above can drain from the top and also from the bottom. From the top it can drain to the surface, and from the bottom it can drain to the sand layer below. Hence, H should be half the thickness of the clay layer.

$$H = 4/2 \text{ m}$$

$$H = 2 \text{ m}$$

$$t = (2^2 \times 1.0)/0.01 = 363 \text{ days}$$

We could reasonably assume that all the settlement due to clay consolidation has occurred after 363 days.

Design Example 14.5

Find the settlement after 1 year in the clay layer shown for the 3 m × 3 m column footing as shown in Fig. 14.25. The groundwater level is 1.5 m below the surface. The following parameters are given for the clay layer: $\gamma = 18\,\text{kN/m}^3$, $C_r = 0.032$, $C_c = 0.38$, $e_0 = 0.85$, $p'_c = 50\,\text{kPa}$, $c_v = 0.011\,\text{m}^2/\text{day}$.

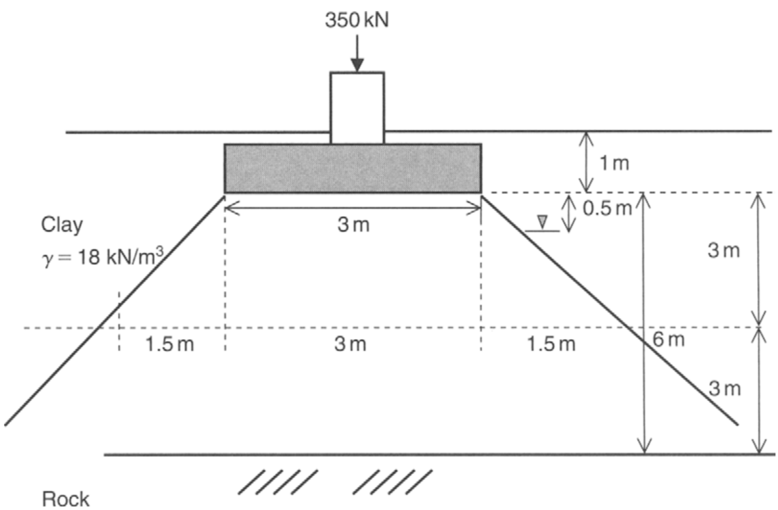


Figure 14.25 Consolidation in clay (single drainage)

Solution

ΔH = total primary consolidation settlement
H = thickness of the drainage layer
C_c = compression index of the clay layer in the virgin zone
C_r = recompression index of the clay layer in the recompression zone
e_0 = void ratio of the clay layer at midpoint of the clay layer prior to loading
p'_0 = effective stress at the midpoint of the clay layer prior to loading

p'_c = preconsolidation pressure
p'_f = pressure after the footing is placed
c_V = consolidation coefficient

STEP 1: The first step is to find the percent consolidation (U%) after one year

$$t = \frac{H^2 \times T_V}{c_V}$$

t = 1 year = 365 days
H = thickness of the drainage layer

In this case drainage is only possible from the top. Drainage is not possible from the bottom, due to bedrock. Therefore, this is a single drainage situation. Hence $H = 6$ m.

It should be mentioned here that in reality, the longest drainage path is 7 m, since the footing is placed 1 m below the surface.

$$T_V = \text{time coefficient}$$

Find T_V

$$T_V = c_V \times t/H^2 = 0.011 \times 365/6^2 = 0.111$$

STEP 2: Use Table 14.1 to determine T_V.

From Table 14.1, for $U\% = 20\%$, $T_V = 0.009$, and for $U\% = 30\%$, $T_V = 0.115$. Through interpolation, find U% for $T_V = 0.111$.

$$(U - 20)/(0.111 - 0.009) = (30 - 20)/(0.115 - 0.009)$$

For $T_V = 0.111$, U = 29.6. Therefore, when t (time) = 1 year = T_V = 0.111 and U% = 29.6.

STEP 3: Find the total settlement due to consolidation.

$$\Delta H = H \times C_r/(1 + e_0) \times \log\left(\frac{(p'_0 + \Delta p_1)}{p'_0}\right) + H \times C_c/(1 + e_0) \times \log\left(\frac{(p'_c + \Delta p_2)}{p'_c}\right)$$

The clay layer is 7 m thick, and the footing is placed 1 m below the surface. The top 1 m is not subjected to consolidation. The compressible portion of the clay layer is 6 m thick. Find the effective stress at the mid layer of the compressible clay stratum, p'_0.

$$p'_0 = 1.5 \times \gamma_{\text{clay}} + 2.5 \times (\gamma_{\text{clay}} - \gamma_{\text{w}})$$
$$= 1.5 \times 18 + 2.5 \times (18 - 9.807) = 47.5\,\text{kN/m}^2$$

STEP 4: Find the increase of stress, Δp, at the midpoint of the clay layer due to the footing. The total load of 350 kN is distributed at a larger area at the midsection of the clay layer.

$$\text{area of the midsection of the clay layer} = (3 + 1.5 + 1.5) \times (3 + 1.5 + 1.5) = 36\,\text{m}^2$$
$$\Delta p = 350/36 = 9.72\,\text{kN/m}^2$$

STEP 5: Find the final pressure after application of the footing load.

$$p'_{\text{f}} = \text{initial pressure + pressure increase due to footing load at midpoint}$$
$$p'_{\text{f}} = p'_0 + \Delta p = 47.5 + 9.72 = 57.22\,\text{kN/m}^2$$

STEP 6: Apply values in the consolidation equation.

$$\Delta H = H \times C_{\text{r}}/(1 + e_0) \times \log\left(\frac{(p'_0 + \Delta p_1)}{p'_0}\right) + H \times C_{\text{c}}/(1 + e_0) \times \log\left(\frac{(p'_{\text{c}} + \Delta p_2)}{p'_{\text{c}}}\right)$$

where

Δp_1 = stress increase from initial stress, p'_0, to preconsolidation pressure p'_{c}

Since p'_0 is 47.5 and p'_{c} was given as 50 kN/m^2, Δp_1 is $(50 - 47.5)$ kN/m^2.

$$\Delta p_1 = 2.5\,\text{kN/m}^2$$

Next, we find Δp_2, the stress increase from preconsolidation pressure p'_c to final pressure p'_f. Since p'_c is 50 kN/m^2 and p'_f was found to be 57.2, Δp_2 is (57.22 – 50) kN/m^2.

$$\Delta p_2 = 7.22\text{kN/m}^2$$

$$\Delta H = 6 \times 0.032/(1 + 0.85)\log\left(\frac{(p'_0 + \Delta p_1)}{p'_0}\right) + 6 \times 0.38/(1 + 0.85)$$

$$\log\left(\frac{(p'_c + \Delta p_2)}{p'_c}\right)$$

$$= 0.1037\log(47.5 + 2.5)/47.5 + 1.23\log(50 + 7.22)/50$$

$$= 0.0023 + 0.072$$

$$= 0.0743\,\text{m}\ (2.92\,\text{in.})$$

STEP 7: Find the settlement after 1 year.

The percent consolidation (U%) after 1 year is 29.6% (see Step 2).

settlement after 1 year = total settlement $\times$ percent consolidation

settlement after 1 year = 0.0743 $\times$ 0.296 = 0.022 m (0.87 in.)

15

Earth Retaining Structures

15.1 Introduction

Earth retaining structures are an essential part of civil engineering. Retaining walls are everywhere, yet few people notice them. For instance, take a drive for 20 minutes and count how many retaining walls you see.

There are many types of retaining walls. See Fig. 15.1.

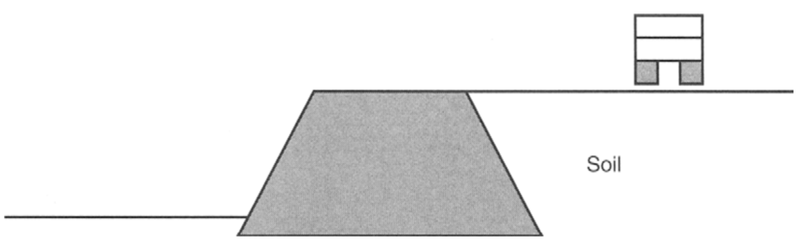


Figure 15.1 Gravity walls

In the case of gravity walls, the pure weight of the retaining wall holds the soil. Gravity walls are made of rock, concrete, and masonry. Presently, concrete is the most common material used to construct gravity walls. See Figs. 15.2 and 15.3.

Gabions are baskets filled with rocks. These rock baskets can be used to construct retaining walls. Gabion walls are designed as gravity walls. Easy drainage through gabions is a major advantage. Gabion walls are, in most cases, cheaper than concrete gravity walls.

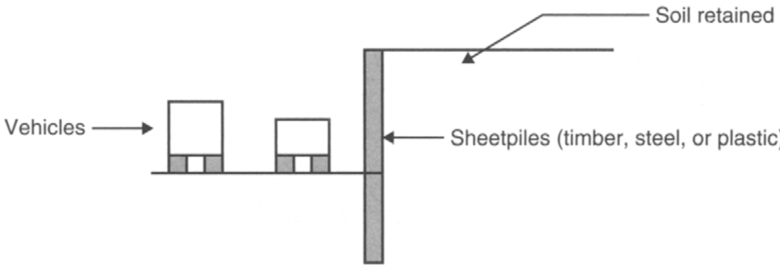


Figure 15.2 Sheetpile walls

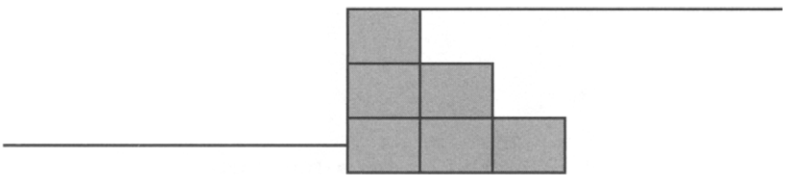


Figure 15.3 Gabion walls

15.2 Water Pressure Distribution

Before we discuss the horizontal force due to soil, consider the horizontal force due to water.

Water pressure is the same in all directions, since it is a liquid. Vertical stress at a point inside water is the same as the horizontal stress at that location. See Fig. 15.4.

$$P_h = P_v = \gamma_w \times h$$

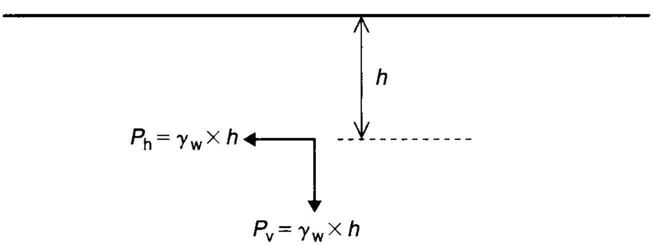


Figure 15.4 Pressure in water

where

P_h = horizontal pressure
P_v = vertical pressure
γ_w = density of water, usually taken as 62.4 pcf (pounds per cubic foot) or 9.81 kN/m^3
h = depth to the point of interest
P_h = horizontal pressure = $\gamma_w \times h$
P_v = vertical pressure = $\gamma_w \times h$

In the case of water, the horizontal pressure is equal to the vertical pressure. See Fig. 15.5.

$$\gamma_w = \text{density of water} = 9.81\,\text{kN/m}^3 (62.4\,\text{pcf})$$

$$\text{total force, } F = \text{area of the triangle}$$

$$F = \gamma_w h \times h/2 = \gamma_w \times h^2/2$$

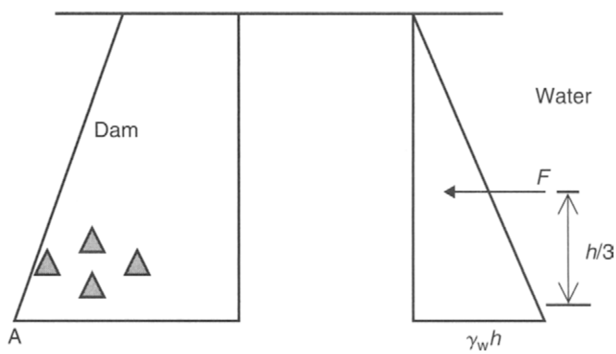


Figure 15.5 Water pressure on a dam

The moment around point A can be computed.

$$\text{total moment, } M = \text{force} \times \text{distance to the force}$$

$$M = \gamma_w \times h^2/2 \times h/3 = \gamma_w \times h^3/6$$

The resultant pressure of the triangle acts at a distance $h/3$ from the bottom.

Design Example 15.1

Find the horizontal force and the overturning moment of the dam shown in Fig. 15.6.

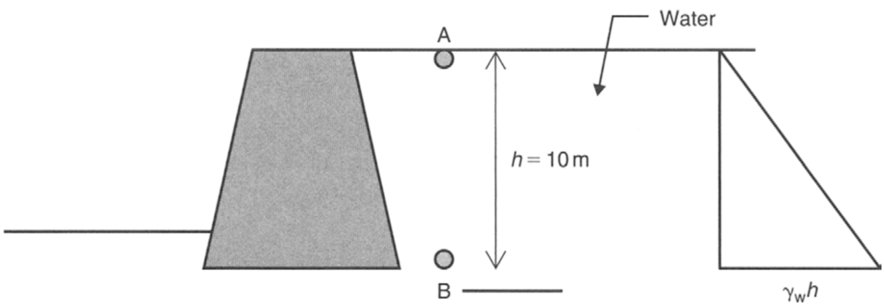


Figure 15.6 Dam subjected to water pressure

Solution

$$\gamma_w \times h = 10 \times 9.81 = 90.81\,\text{kN/m}^2$$

The total pressure acting on the dam due to water is obtained by computing the area of the pressure triangle.

$$\begin{aligned}\text{total force acting on the dam} &= \text{area of the pressure triangle}\\ &= 1/2 \times 10 \times 90.81\\ &= 454.1\,\text{kN}\end{aligned}$$

In the case of water, the pressure in all directions is the same.

15.2.1 Computation of Horizontal Pressure in Soil

The following equations are used to compute the horizontal pressure in soils. In the case where there is no groundwater,

$$\begin{aligned}\text{vertical pressure in soil} &= \text{density of soil} \times \text{depth} = \gamma \times h\\ \text{horizontal pressure in soil} &= \text{lateral earth pressure coefficient}\\ &\quad \times \text{density of soil} \times \text{depth}\\ &= K \times \gamma \times h\end{aligned}$$

where

K = lateral earth pressure coefficient

There are three lateral earth pressure coefficients.

- Active earth pressure coefficient, K_a.
- Passive earth pressure coefficient, K_p.
- Lateral earth pressure coefficient at rest, K_0.

15.3 Active Earth Pressure Coefficient, K_a

The active earth pressure coefficient is used when the retaining wall has the freedom to move. See Fig. 15.7.

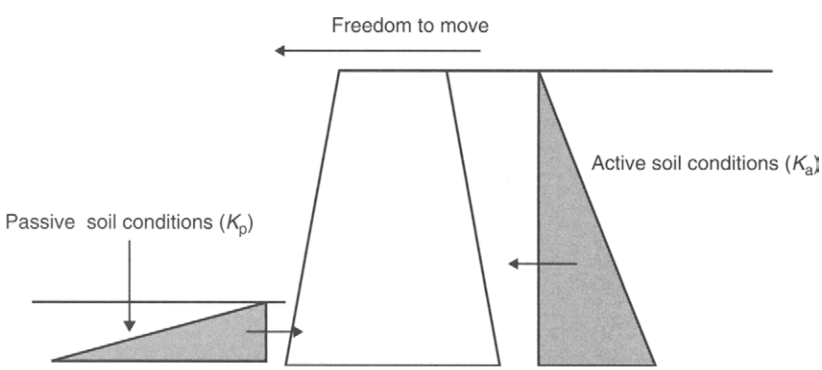


Figure 15.7 Freedom of movement of the retaining wall

The retaining wall in Fig. 15.7 is free to move to the left. When a slight movement in the retaining wall occurs, the pressure on the right-hand side will be reduced. On the other hand, the pressure on the left-hand side will be increased.

Hence we can see that K_a is smaller than K_p.

The following equation is used to compute K_a and K_p.

$$K_a = \tan^2(45 - \varphi/2)$$

$$K_p = \tan^2(45 + \varphi/2)$$

Many designers do not consider the passive soil conditions in front of the retaining walls on the conservative side. Some codes require that erosion protection be provided when passive earth pressure in front of a retaining wall is considered for the design.

15.4 Earth Pressure Coefficient at Rest, K_0

The following equation is used to compute K_0.

$$K_0 = 1 - \sin\varphi$$

The earth pressure coefficient at rest is used for cantilever retaining walls. See Fig. 15.8.

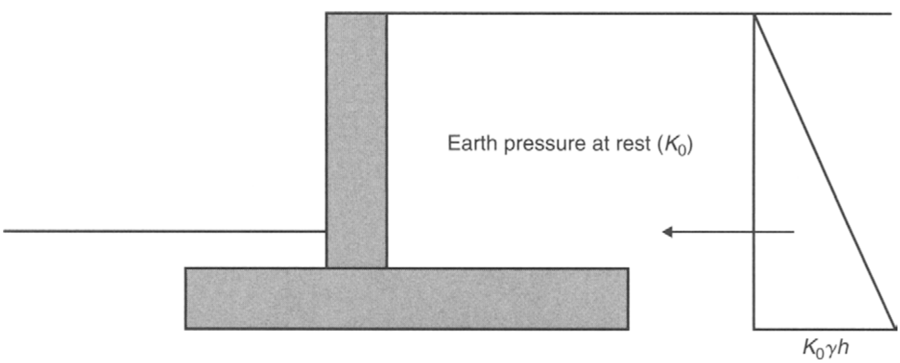


Figure 15.8 Cantilever retaining walls

In a cantilever retaining wall, it is difficult to say whether the wall can move enough to create active conditions behind the wall. Hence, the earth pressure coefficient at rest, K_0, is used for cantilever retaining walls. K_0 is larger than K_a.

$$K_a < K_0 < K_p$$

15.5 Gravity Retaining Walls: Sand Backfill

In gravity walls, the soil pressure is restrained by the weight of the retaining wall. In Fig. 15.9, a simple gravity retaining wall is shown. Modern gravity retaining walls are made of concrete.

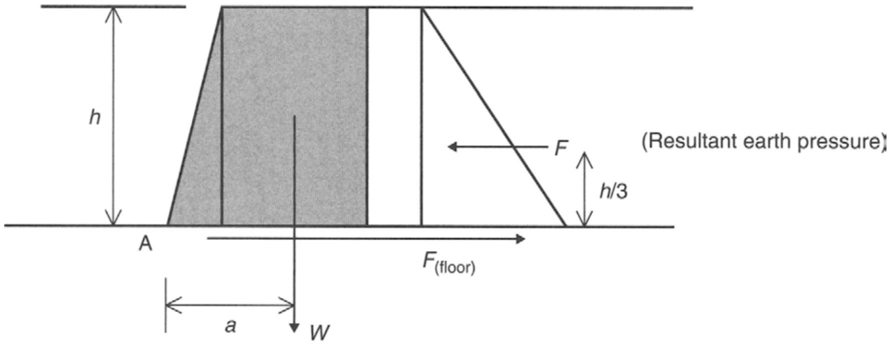


Figure 15.9 Gravity retaining wall

The retaining wall can fail in three different ways. See Fig. 15.10.

1. The retaining wall can slide. This is called a sliding failure.
2. The retaining wall can overturn around the toe (point A).
3. Bearing failure of the foundation.

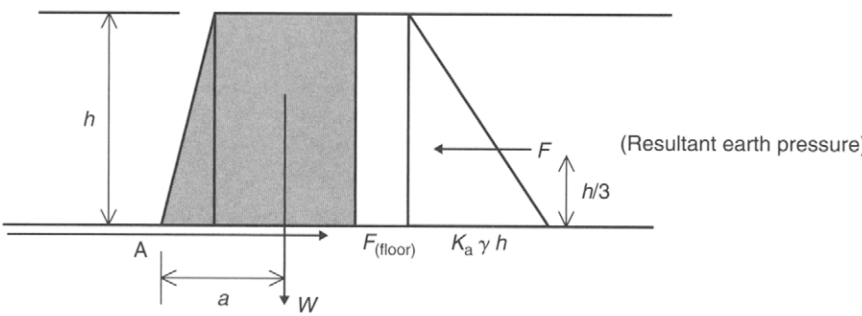


Figure 15.10 Gravity retaining wall with soil pressure

15.5.1 Resistance Against Sliding Failure

For stability against sliding failure,

$$F_{(\text{floor})} > F$$

where

F = resultant earth pressure
$F_{(\text{floor})}$ = friction between concrete and soil at the bottom face

$$F_{(\text{floor})} = \tan\delta \times W$$

where

W = weight of the retaining wall
δ = friction angle between concrete and soil

The factor of safety against sliding failure is

$$\text{F.O.S.} = F_{(\text{floor})}/F$$

15.5.2 Resistance Against Overturning

The retaining wall can overturn around the toe (point A).

$$\text{overturning moment} = F \times h/3$$

$$\text{resisting moment} = W \times a$$

$$\begin{aligned}\text{factor of safety against overturning} &= \text{resisting moment/overturning moment} \\ &= 3\,Wa/Fh\end{aligned}$$

What is the method to find the resultant earth pressure?
The earth pressure at any given point is given by $K_a \times \gamma \times h$.

$$K_a = \text{active earth pressure coefficient} = \tan^2(45 - \varphi/2)$$

where

γ = density of soil
h = height of the soil

$$\begin{aligned}\text{resultant earth pressure, } F &= \text{area of the pressure triangle} \\ &= K_a \times \gamma \times h \times (h/2) \\ &= K_a \times \gamma \times h^2/2\end{aligned}$$

The resultant earth pressure acts at the center of gravity of the pressure triangle, $h/3$ above the bottom.

total moment, M, around point A = force, F × distance to the force

$$M = (K_a \times \gamma \times h^2/2) \times h/3 = K_a \times \gamma \times h^3/6$$

Design Example 15.2

This example explores a gravity retaining wall with sand backfill, in the no groundwater case. Find the factor of safety for the retaining wall shown in Fig. 15.11. The height of the retaining wall, H, is 10 m, the weight of the retaining wall is 2,300 kN for a 1 m length of the wall, and the weight acts at a distance of 5 m from the toe (point X). The friction angle of the soil backfill is 30°. The soil backfill mainly consists of sandy soils. The density of the soil is 17.3 kN/m³. The resultant earth pressure force acts at a distance $H/3$ from the bottom of the wall. The friction angle between soil and earth at the bottom of the retaining wall was found to be 20°.

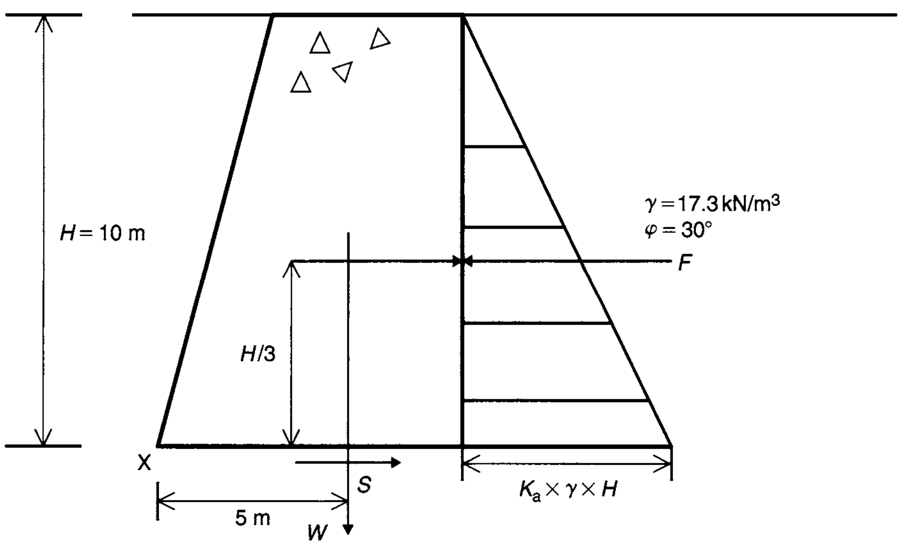


Figure 15.11 Gravity retaining wall with sand backfill

Solution

STEP 1: Find the resultant earth pressure.

$$K_a = \tan^2(45 - \varphi/2) = \tan^2(45 - 30/2)$$

$$K_a = \tan^2(30°) = 0.33$$

$$\text{earth pressure at the bottom of the base} = K_a \times \gamma \times H$$
$$= 0.33 \times 17.3 \times 10$$
$$= 57.1\,\text{kN/m}^2$$

$$\text{resultant earth pressure, } F = \text{area of the pressure triangle}$$
$$= 57.1 \times 10/2$$
$$= 285.5\,\text{kN per meter length of the wall}$$

STEP 2: Find the resistance against sliding at the base, S.

$$S = \text{weight of the wall} \times \tan(\delta)$$

where

δ = friction angle between concrete and soil at the bottom of the retaining wall

$$S = W \times \tan(\delta) = 2{,}300 \times \tan(20°)$$
$$= 837\,\text{kN per 1 m length of the wall}$$

The weight of the retaining wall is given to be 2,300 kN per 1 m length.

$$\text{factor of safety against sliding} = 837/285.5 = 2.93$$

STEP 3: Find the resistance against overturning.

Overturning will occur around point X of the retaining wall.

Resistance to overturning is provided by the weight of the retaining wall.

$$\text{overturning moment} = F \times H/3 = 285.5 \times 10/3$$

$$= 951.7\,\text{kN} \times \text{m (per 1 m length of the wall)}$$

$$\text{resisting moment} = W \times a = 2{,}300 \times 5$$
$$= 11{,}500\,\text{kN} \times \text{m (per 1 m length of the wall)}$$

$$\text{factor of safety against overturning} = 11{,}500/951.7 = 12$$

15.6 Retaining Wall Design When Groundwater Is Present

Groundwater exerts additional pressure on retaining walls when present. This concept needs to be understood when drawing pressure triangles.

- Total density should be used above groundwater level to compute the lateral earth pressure. Below the groundwater level, buoyant density of soil should be used.

$$\text{buoyant density} = (\gamma - \gamma_W)$$

Pressure due to water should be computed separately.

Calculation of pressure acting on a gravity retaining wall when groundwater is present needs to be calculated in two parts.

- First, draw the pressure diagram with effective stress of the soil.
- Second, draw the pressure diagram for the groundwater.
- Both pressure diagrams are considered when computing the sliding force and the overturning moment. See Fig. 15.12.

$$\text{earth pressure above groundwater} = K_a \times \gamma \times h_1$$

$$\text{earth pressure below groundwater} = K_a \times (\gamma - \gamma_W) \times h_2$$

$$\text{pressure due to water alone} = \gamma_W \times h_2$$

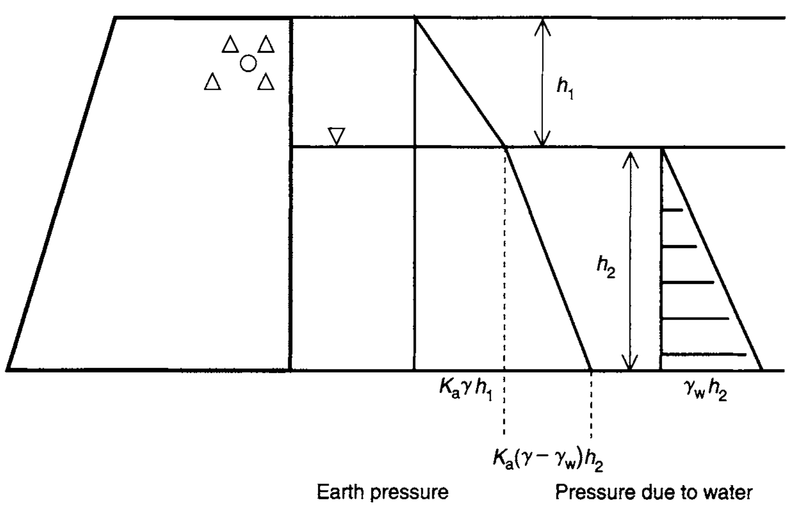


Figure 15.12 Earth pressure and pressure due to water

where

γ = total density of soil
γ_W = density of water
$(\gamma - \gamma_W)$ = buoyant density of soil (or soil density below the groundwater level)

Figure 15.13 shows all the factors used to compute the factor of safety of a retaining wall when groundwater is present.

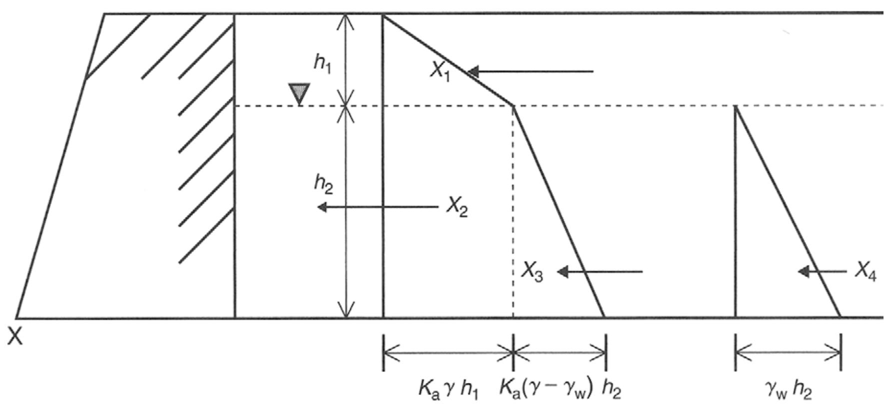


Figure 15.13 Forces acting on the retaining wall

total force = areas of all the triangles and rectangles

$$\text{area of the triangle } X_1 = K_a \times \gamma \times h_1^2/2$$
$$\text{area of the rectangle } X_2 = K_a \times \gamma \times h_1 \times h_2$$
$$\text{area of the triangle } X_3 = K_a \times (\gamma - \gamma_W) \times h_2^2/2$$
$$\text{area of the triangle } X_4 = \gamma_W \times h_2^2/2$$
$$\text{total force} = X_1 + X_2 + X_3 + X_4$$

$$= K_a \times \gamma \times h_1^2/2 + K_a \times \gamma \times h_1 \times h_2 + K_a \times (\gamma - \gamma_W) \times h_2^2/2 + \gamma_W \times h_2^2/2$$

total moment, M around point X = moment of each area

$$\text{moment of triangle } X_1 = K_a \times \gamma \times h_1^2/2 \times (h_2 + h_1/3)$$
$$\text{moment of rectangle } X_2 = K_a \times \gamma \times h_1 \times h_2 \times (h_2/2)$$
$$\text{moment of triangle } X_3 = K_a \times (\gamma - \gamma_W) \times h_2^2/2 \times (h_2/3)$$
$$\text{moment of triangle } X_4 = \gamma_W \times h_2^2/2 \times h_2/3$$

$$\text{total moment} = K_a \times \gamma \times h_1^2/2(h_2 + h_1/3) + K_a \times \gamma \times h_1 \times h_2 \times (h_2/2) + K_a \times (\gamma - \gamma_w) \times h_2^2/2 \times (h_2/3) + \gamma_w \times h_2^2/2 \times h_2/3$$

Design Example 15.3

This example explores a gravity retaining wall with sand backfill, in the case where groundwater is present. Find the factor of safety for the retaining wall shown in Fig. 15.14. The height of the retaining wall, H, is 10 m, the weight of the retaining wall is 2,300 kN for a 1 m length of the wall, and the weight acts at a distance of 5 m from the toe (point X). The friction angle of the soil backfill is 30°. The soil backfill mainly consists of sandy soils. The density of the soil is 17.3 kN/m^3. The resultant earth pressure force acts at a distance $H/3$ from the bottom of the wall. The friction angle between soil and earth at the bottom of the retaining wall was found to be 20°. The groundwater level is found to be at 4 m below the surface.

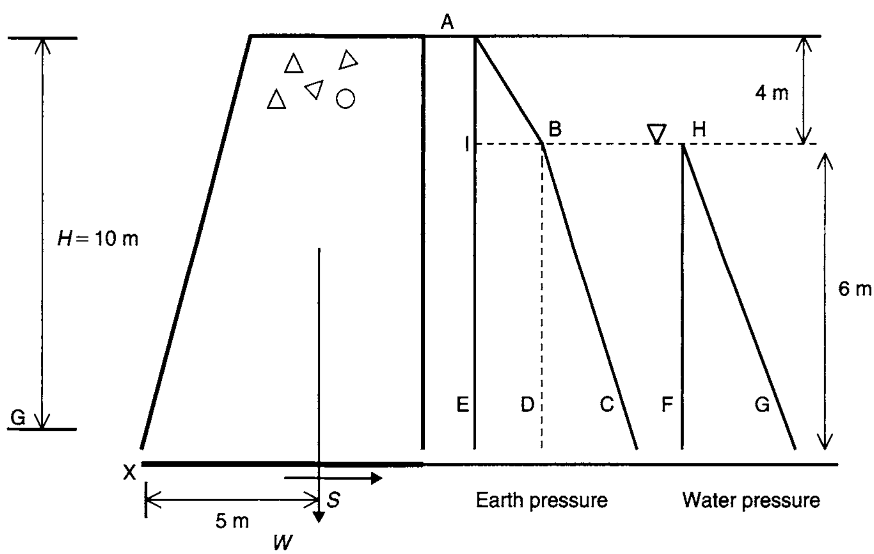


Figure 15.14 Overturning moment

Solution

STEP 1: Find the coefficient of earth pressure.

$$K_a = \tan^2(45 - \varphi/2) = \tan^2(45 - 30/2)$$

$$K_a = \tan^2(30°) = 0.33$$

STEP 2: Find the earth pressure at point B.

$$\text{earth pressure at point B (IB)} = K_a \times \gamma \times h = 0.33 \times 17.3 \times 4$$
$$= 22.8\,\text{kN/m}^2$$
$$\text{IB} = \text{ED}$$

STEP 3: Find the earth pressure at point C.

$$\text{Earth pressure at point C} = \text{ED} + \text{DC}$$

The distance ED was found to be 22.8 in the previous step.

$$\text{DC} = K_a \times (\gamma - \gamma_w) \times h = 0.33 \times (17.3 - 9.81) \times 6 = 14.8\,\text{kN/m}^2$$

$$\text{Earth pressure at point C} = 22.8 + 14.8 = 37.6\,\text{kN/m}^2$$

STEP 4: Find the pressure due to water at point G.
See Fig. 15.15.

$$\text{Water pressure at point G} = \gamma_w \times 6 = 9.81 \times 6 = 58.9\,\text{kN/m}^2$$

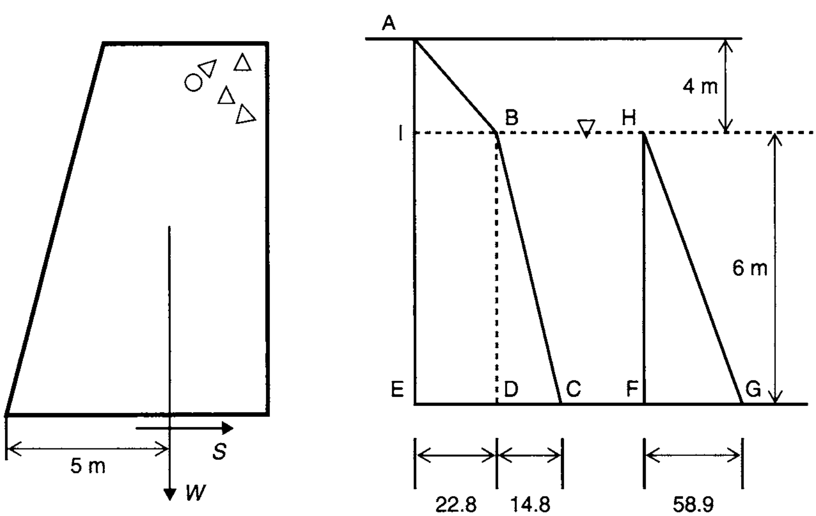


Figure 15.15 Forces and moments acting on earth retaining wall

STEP 5: Find the horizontal force due to the soil and water.
The horizontal force due to soil and water can be computed by adding the areas of all of the triangles.

$$\text{area of triangle ABI} = 22.8 \times 4 \times 1/2 = 45.6\,\text{kN}$$

$$\text{area of rectangle IBED} = 22.8 \times 6 = 136.8\,\text{kN}$$

$$\text{area of triangle BCD} = 14.8 \times 6 \times 1/2 = 44.4\,\text{kN}$$

$$\text{area of triangle HGF} = 58.9 \times 6 \times 1/2 = 176.7\,\text{kN}$$

$$\text{total area} = 403.5\,\text{kN}$$

STEP 6: Find the resisting force against sliding.

$$S = \text{weight of the retaining wall} \times \tan(\delta)$$

where

δ = friction angle between concrete and soil at the bottom of the retaining wall

$$S = W \times \tan(\delta) = 2{,}300 \times \tan(20°)$$

$$= 837\,\text{kN per 1 m length of the wall}$$

The weight of the retaining wall is given to be 2,300 kN per 1 m length.

$$\text{factor of safety against sliding} = 837/403.5 = 2.07$$

Usually, a factor of safety of 2.5 is desired. Hence it is advisable to increase the size of the retaining wall to increase the weight.

STEP 7: Find the resistance against overturning, *O*.

See Fig. 15.16. The retaining wall can overturn around point X. To find the overturning moment, we need to find the position of the forces in each unit.

For triangle ABI, F_1 is the force acting at the center of gravity of the triangle. The center of gravity of the triangle is 1/3 of the height of the triangle.

$$\text{distance to } F_1 \text{ from the bottom} = L_1 = 6 + 4/3\,\text{m} = 7.33\,\text{m}$$

For rectangle IBED, the force F_2 is acting at the center of the rectangle.

$$\text{distance to } F_2 \text{ from the bottom} = L_2 = 6/2\,\text{m} = 3\,\text{m}$$

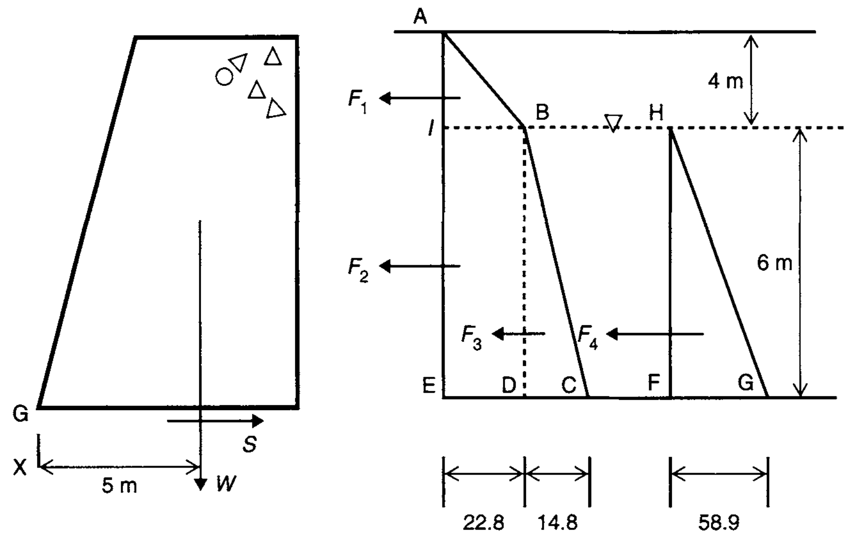


Figure 15.16 Resistance against overturning

For triangle BCD, F_3 is the force acting at the center of gravity of the triangle. The center of gravity of the triangle is 1/3 of the height of the triangle.

$$\text{distance to } F_3 \text{ from the bottom} = L_3 = 6/3\,\text{m} = 2\,\text{m}$$

As in the previous triangles, for triangle HGF, F_4 is the force acting at the center of gravity of the triangle.

$$\text{distance to } F_4 \text{ from the bottom} = L_4 = 6/3\,\text{m} = 2\,\text{m}$$

$$\text{overturning moment} = (F_1 \times L_1) + (F_2 \times L_2) + (F_3 \times L_3) + (F_4 \times L_4)$$

$$\text{overturning moment} = 45.6 \times 7.33 + 136.8 \times 3 + 44.4 \times 2 + 176.7 \times 2$$

$$\text{overturning moment} = 1{,}186.8\,\text{kN m per 1 m length of the wall.}$$

STEP 8: Find the resistance against overturning, O.

The resistance to overturning is provided by the weight of the retaining wall.

$$\text{resisting moment} = W \times a = 2{,}300 \times 5$$

$$= 11{,}500\,\text{kN m per 1 m length of the wall}$$

$$\text{factor of safety against overturning} = 11{,}500/1{,}186.8 = 9.7$$

A factor of safety of 2.5 to 3.0 is desired against overturning. Hence the factor of safety against overturning is sufficient.

15.7 Retaining Wall Design in Nonhomogeneous Sands

The soil behind an earth retaining structure may not be homogeneous. When there are different types of soil adjacent to a retaining wall, the earth pressure diagram will look different. See Fig. 15.17.

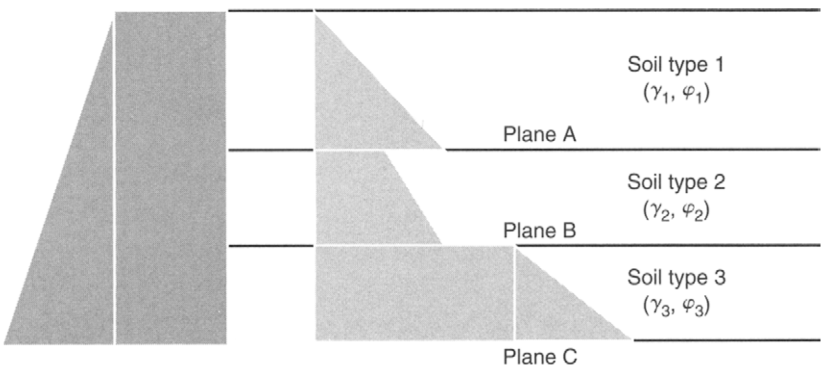


Figure 15.17 Lateral earth pressure in mixed soil

The lateral earth pressure at any location in soil is equal to the vertical effective stress multiplied by the lateral earth pressure coefficient.

lateral earth pressure = vertical effective stress at that location × lateral earth pressure coefficient

$$P_h = P_v \times K_a$$

where

P_h = horizontal stress
P_v = vertical effective stress (or simply effective stress)

When different types of sandy soils are encountered, K_a (the lateral earth pressure coefficient) will be subject to change. It is possible that different sandy soils are used for backfilling. In some cases, soil from the neighborhood is used for economic reasons. See Fig. 15.18.

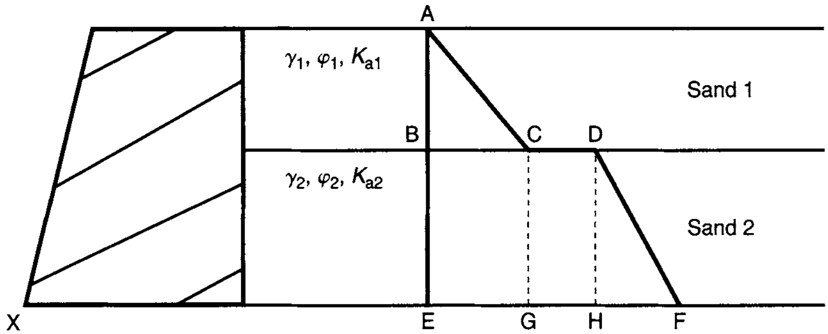


Figure 15.18 Lateral earth pressure diagram

$$\text{horizontal stress in sand layer } 1 = K_{a1} \times \gamma_1 \times h$$

$$\text{horizontal stress in sand layer } 2 = K_{a2} \times \gamma_2 \times h$$

Since K_{a1} and K_{a2} are different, there is a break at the start of the second layer.

The above concepts can better be explained using an example.

Design Example 15.4

This example concerns a gravity retaining wall in nonhomogeneous sandy soil. Find the factor of safety for the retaining wall shown in Fig. 15.19. The height of the retaining wall, H, is 10 m, the weight of the retaining wall is 2,300 kN per 1 m length of the wall, and the weight acts at a distance of 5 m from the toe (point X). The sandy backfill consists of two sand layers. The friction angle of the top sand layer is 30° and the density of the soil is 17.3 kN/m^3. The friction angle of the bottom sand layer is 20° with a density of 16.5 kN/m^3. The friction angle between the soil and the earth at the bottom of the retaining wall was found to be 20°. See Fig. 15.19.

Solution

STEP 1: Find the lateral earth pressure coefficients of sand layers.

$$K_{a1} = \tan^2(45 - \varphi/2)$$

$$\text{sand layer 1: } K_{a1} = \tan^2(45 - 30/2) = 0.33$$

$$\text{sand layer 2: } K_{a2} = \tan^2(45 - 20/2) = 0.49$$

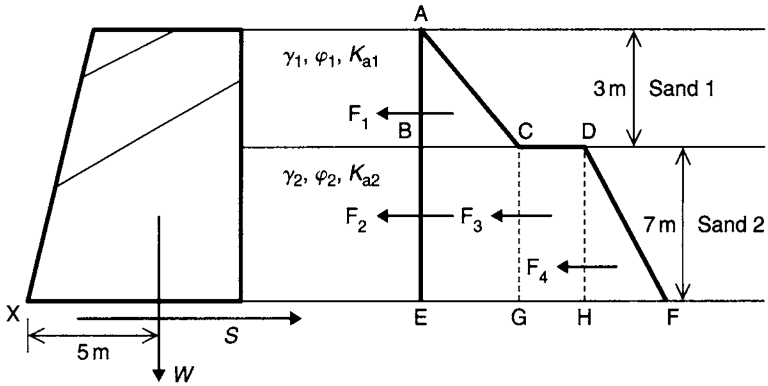


Figure 15.19 Forces due to mixed soil conditions

STEP 2: Find the horizontal force acting on the retaining wall.

$$\text{length BC} = K_{a1} \times \gamma_1 \times h = 0.33 \times 17.3 \times 3 = 17.1\,\text{kN/m}^2$$

$$\text{length BD} = K_{a2} \times \gamma_1 \times h = 0.49 \times 17.3 \times 3 = 25.4\,\text{kN/m}^2$$

The K_a value in sand layer 1 is 0.33 and the K_a value in sand layer 2 is 0.49. The horizontal stress is found by multiplying the vertical effective stress with the lateral earth pressure coefficient, K_a. In sand layer 1, the lateral earth pressure coefficient is K_{a1}, and in sand layer 2, the lateral earth pressure coefficient is K_{a2}.

$$\text{length BD} = \text{length BC} + \text{length CD}$$

$$25.4 = 17.1 + \text{length CD}$$

$$\text{length CD} = 8.3\,\text{kN}$$

The effective stress at both cases at point B is ($\gamma_1 \times h$) or (17.3×3). At point B, the density of soil layer 2 has no impact.

Find length EF.

First find the vertical effective stress at point E.

$$\text{vertical effective stress at point E} = (\gamma_1 \times 3) + (\gamma_2 \times 7) = 17.3 \times 3 + 16.5 \times 7 = 167.4\,\text{kN/m}^2$$

$$\text{horizontal effective stress at point E} = \text{EF} = K_{a2} \times \text{vertical effective stress at point E}$$

horizontal effective stress at point E = $0.49 \times 167.4 = 82\,\text{kN/m}^2$

$$EF = EH + HF$$
$$EH = BD = 25.4$$
$$82 = 25.4 + HF$$
$$HF = 56.6$$

STEP 3: Find the total horizontal force exerted on the retaining wall.

total horizontal force exerted on the retaining wall = area of all triangles and rectangles

area of triangle ABC $= 1/2 \times 3 \times 17.1 = 25.7\,\text{kN/m}^2$

area of rectangle BCEG $= 17.1 \times 7 = 119.7\,\text{kN/m}^2$

area of rectangle CDGH $= 8.3 \times 7 = 58.1\,\text{kN/m}^2$

area of triangle DHF $= 1/2 \times 7 \times 56.6 = 198.1\,\text{kN/m}^2$

total horizontal force $= 25.7 + 119.7 + 58.1 + 198.1 = 401.6\,\text{kN/m}^2$

STEP 4: Find the resisting force against sliding.

$$S = \text{weight of the retaining wall} \times \tan(\delta)$$

where

δ = friction angle between concrete and soil at the bottom of the retaining wall

$$S = W \times \tan(\delta) = 2{,}300 \times \tan(20°)$$
$$= 837\,\text{kN per 1 m length of the wall}$$

The weight of the retaining wall is given to be 2,300 kN per 1 m length.

$$\text{factor of safety against sliding} = 837/401.6 = 2.08$$

Typically, a factor of safety of 2.5 is desired. Hence, it will be necessary to increase the weight of the retaining wall.

STEP 5: Find the resistance against overturning.

The retaining wall can overturn around point X. To find the overturning moment, we need to find the position of the forces in each unit.

For triangle ABC, F_1 is the force acting at the center of gravity of the triangle. The center of gravity of the triangle is acting at 1/3 of the height of the triangle.

$$\text{distance to } F_1 \text{ from the bottom} = L_1 = 7 + 3/3\,\text{m} = 8\,\text{m}$$

For rectangle BCEG, F_2 is the force acting at the center of the rectangle.

$$\text{distance to } F_2 \text{ from the bottom} = L_2 = 7/2\,\text{m} = 3.5\,\text{m}$$

For rectangle CDHG, F_3 is the force acting at the center of gravity of the rectangle.

$$\text{distance to } F_3 \text{ from the bottom} = L_3 = 7/2\,\text{m} = 3.5\,\text{m}$$

For triangle DHF, F_4 is the force acting at the center of gravity of the triangle. The center of gravity of the triangle is acting at 1/3 of the height of the triangle.

$$\text{distance to } F_4 \text{ from the bottom} = L_4 = 7/3\,\text{m} = 2.33\,\text{m}$$

$$\text{overturning moment} = (F_1 \times L_1) + (F_2 \times L_2) + (F_3 \times L_3) + (F_4 \times L_4)$$

$$\begin{aligned}\text{overturning moment} &= 25.7 \times 8 + 119.7 \times 3.5 + 58.1 \times 3.5 \\ &\quad + 198.1 \times 2.33 \\ &= 1{,}289.5\,\text{kN m per 1 m length of the wall}\end{aligned}$$

STEP 6: Find the resistance against overturning.

The resistance to overturning is provided by the weight of the retaining wall.

$$\begin{aligned}\text{resisting moment} &= W \times a = 2{,}300 \times 5 \\ &= 11{,}500\,\text{kN m per 1 m length of the wall}\end{aligned}$$

$$\text{factor of safety against overturning} = 11{,}500/1{,}289.5 = 8.9$$

A factor of safety of 2.5 to 3.0 is desired against overturning. Hence the factor of safety against overturning is sufficient.

15.7.1 General Equation for Gravity Retaining Walls

The retaining walls considered so far had a vertical surface, and the backfill was horizontally placed. This may not be the case in some situations. Consider a case where the backfill is at an angle β to the horizontal. The vertical surface of the retaining wall is at an angle θ to the horizontal. The friction angle between the soil and the concrete is δ.

In such situations, the K_a value is obtained from the equation shown below.

$$K_a = \frac{\sin^2(\theta + \varphi)}{(\sin^2(\theta) \times \sin(\theta - \delta) \times [1 + (p/q)^{1/2}]^2)}$$

$$p = \sin(\varphi + \delta) \times \sin(\varphi - \beta)$$

$$q = \sin(\theta - \delta) \times \sin(\theta + \beta)$$

See Fig. 15.20.

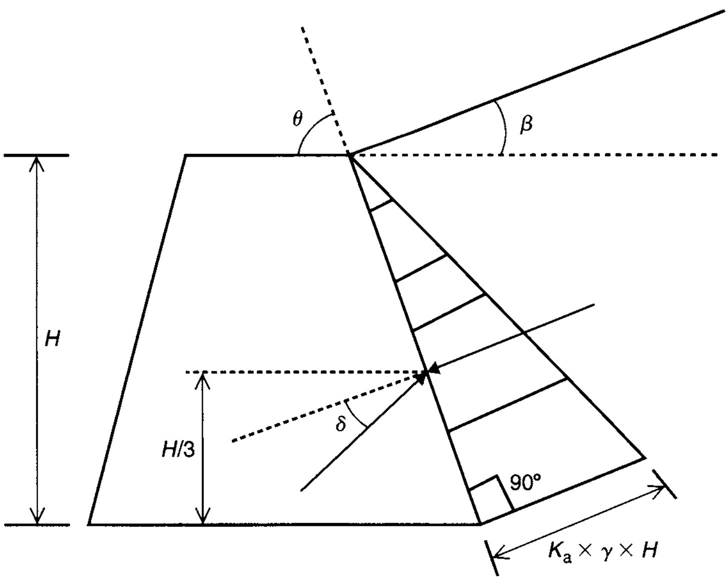


Figure 15.20 Retaining wall with inclined backfill

When $\theta = 90°$, $\beta = 0$, and $\delta \neq 0$:

$$\sin(90° + \varphi) = \cos(\varphi)$$

$$\sin\theta = 1.0 \text{ for } \theta = 90°$$

$$\sin(90^\circ - \delta) = \cos(\delta)$$

$$\sin(90 + \beta) = \cos\beta$$

$$\cos\beta = 1.0 \text{ for } \beta = 0^\circ$$

$$K_a = \frac{\cos^2(\varphi)}{\cos(\delta) \times [1 + (p/q)^{1/2}]^2}$$

$$p = \sin(\varphi + \delta) \times \sin(\varphi)$$

$$q = \cos(\delta)$$

See Fig. 15.21.

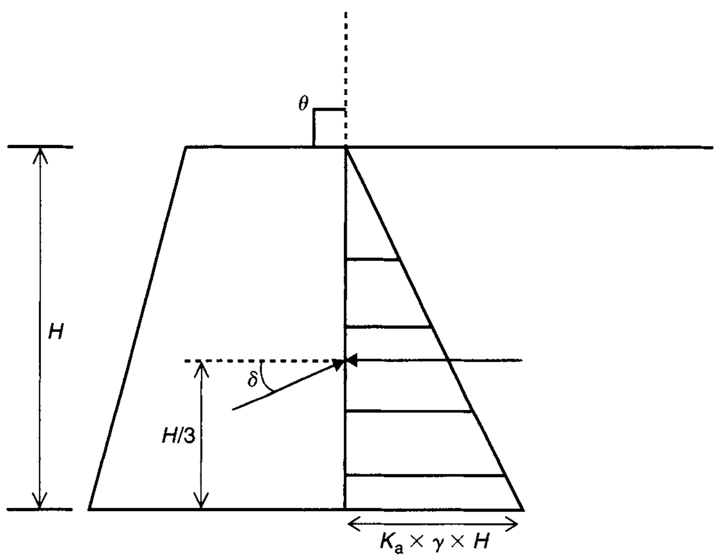


Figure 15.21 Retaining wall with nonzero concrete–soil friction angle

When $\theta = 90^\circ, \beta = 0$, and $\delta = 0$

$$\sin(\theta + \varphi) = \cos(\varphi) \text{ for } \theta = 90^\circ$$

$$\sin 90^\circ = 1.0$$

$$\sin(\theta - \delta) = \cos(\delta) \text{ for } \theta = 90^\circ$$

$$\sin(\theta + \beta) = \cos\beta \text{ for } \theta = 90^\circ$$

$$\cos\delta = 1.0 \text{ for } \delta = 0^\circ$$

$$K_a = \frac{\cos^2(\varphi)}{[1 + (p/q)^{1/2}]^2}$$

$$p = \sin^2(\varphi)$$

$$q = 1.0$$

Hence

$$K_a = \frac{\cos^2(\varphi)}{[1 + \sin\varphi]^2}$$

This equation can be simplified to

$$K_a = \tan^2(45 - \varphi/2)$$

See Fig. 15.22.

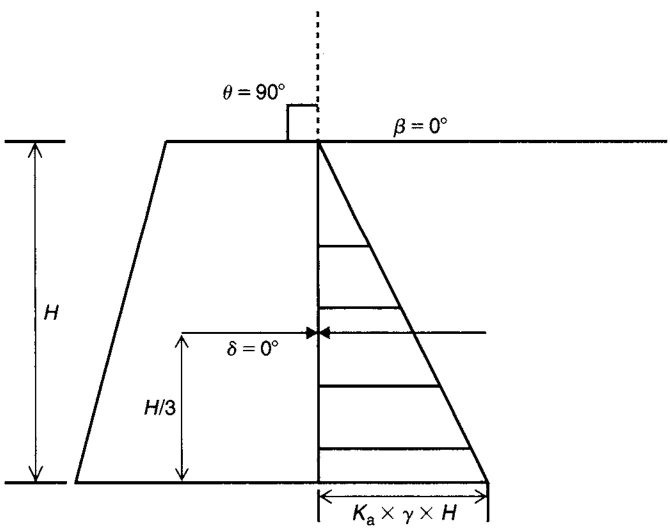


Figure 15.22 Gravity retaining wall and forces

15.7.2 Lateral Earth Pressure Coefficient for Clayey Soils (Active Condition)

The earth pressure in clay soils is different from the earth pressure in sandy soils. In the case of clay soils, lateral earth pressure is given by the following equation.

$$\text{lateral earth pressure} = \gamma \times h - 2c$$

In this case, c is cohesion and γ is the density of the clay. When clay backfill is used, a portion of the clay layer will crack. The thickness of the cracked zone is given by $2c/\gamma$. See Fig. 15.23.

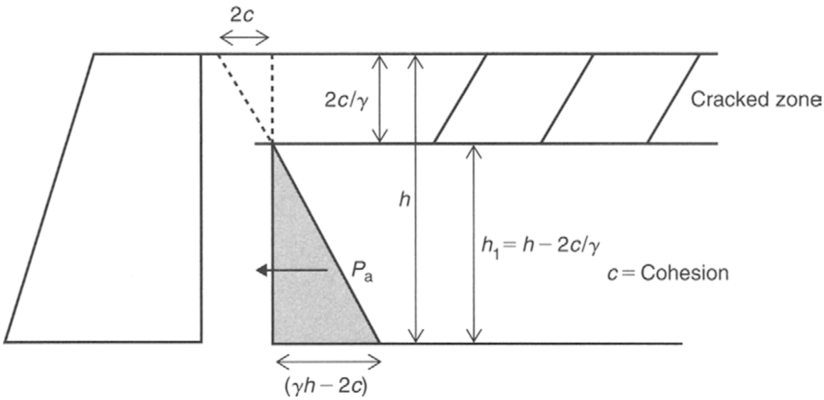


Figure 15.23 Gravity retaining wall in clay soil

No active pressure is generated within the cracked zone.

$$\text{force } (P) = \text{area of the shaded portion of the triangle}$$

$$P_a = (\gamma h - 2c) \times h_1/2$$

$$P_a = (\gamma h - 2c) - (h - 2c/\gamma)/2$$

Usually, the cracked zone will get filled with water. Hence, the pressure due to water also needs to be accounted for. See Fig. 15.24.

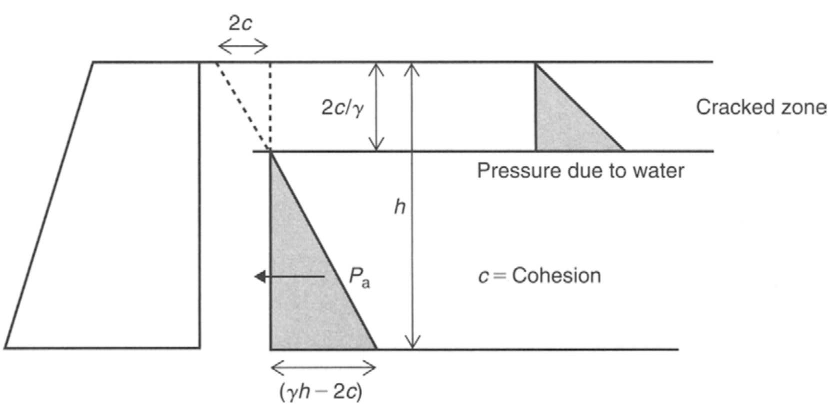


Figure 15.24 Forces due to clay soil

Design Example 15.5

This example explores a gravity retaining wall with a clay backfill. Find the factor of safety of the retaining wall shown in Fig. 15.25 against sliding failure and overturning. The cohesion of the clay backfill is 20 kPa. The density of soil is $\gamma = 17.1\,\text{kN/m}^3$. Assume that the cracked zone is not filled with water. The weight of the retaining wall is 2,000 kN per meter run acting 3 m from the toe of the retaining wall, X, as shown. The height of the retaining wall is 8 m. The friction angle between the base of the retaining wall and the soil is 25°. Assume that the tension cracks are not filled with water.

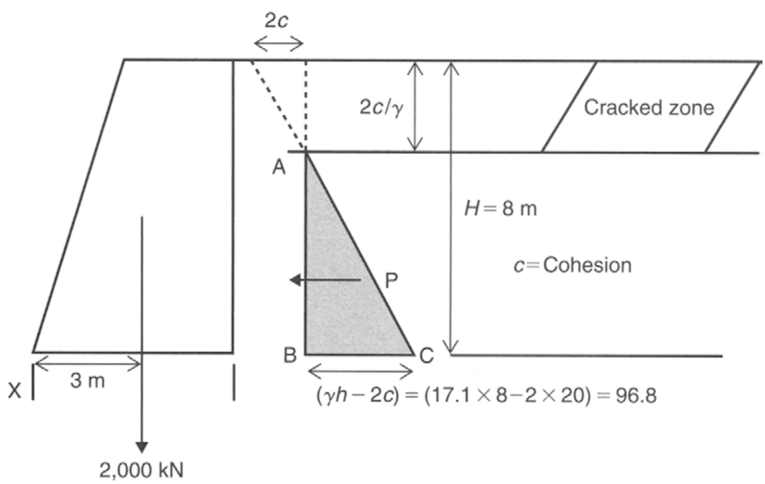


Figure 15.25 Force diagram

Solution

STEP 1: Find the thickness of the cracked zone.

$$\text{thickness of the cracked zone} = 2c/\gamma = 2 \times 20/17.1 = 2.34\,\text{m}$$

STEP 2: Find the pressure at the bottom.

$$\text{pressure at B} = (\gamma h - 2c) = (17.1 \times 8 - 2 \times 20) = 96.8$$

STEP 3: Find the lateral forces acting on the wall.

$$\text{lateral forces acting on the wall} = \text{area of triangle ABC}$$

Note that in this case it is assumed that the cracked zone is not filled with water.

$$\text{area of triangle ABC} = 1/2 \times 96.8 \times 8 = 387.2\,\text{kN}$$

STEP 4: Find the resistance to sliding.

$$\text{resistance to sliding} = \text{friction angle} \times \text{weight of the retaining wall}$$

$$\text{resistance to sliding} = \tan 25^\circ \times 2{,}000\,\text{kN} = 0.466 \times 2{,}000 = 932\,\text{kN}$$

$$\text{factor of safety against sliding} = 932/387.2 = 2.4$$

Usually, a factor of safety of 2.5 is desired. Hence, it will be necessary to increase the weight of the retaining wall.

STEP 5: Find the overturning moment.

The resultant of the triangle ABC acts at a distance 1/3 from the base.

$$\text{distance to the resulting force} = (8 - 2.34)/3 = 1.89\,\text{m}$$

The overturning moment is obtained by obtaining moments around point X.

$$\begin{aligned}\text{overturning moment} = \text{force} \times \text{moment arm} &= 387.2 \times 1.89\\ &= 731.8\,\text{kN m}\end{aligned}$$

Resistance to overturning is provided by the weight of the retaining wall.

$$\text{resistance to overturning} = 2{,}000 \times 3 = 6{,}000\,\text{kN m}$$

$$\text{factor of safety against overturning} = 6{,}000/731.8 = 8.2$$

The resistance against overturning is sufficient.

Design Example 15.6

This example explores a gravity retaining wall with a clay backfill and a cracked zone filled with water. Find the factor of safety of the retaining wall shown in Fig. 15.26 against sliding failure and overturning. The cohesion of the clay backfill is 20 kPa. The density of the soil is $\gamma = 17.1\,\text{kN/m}^3$. Assume the cracked zone is filled with water.

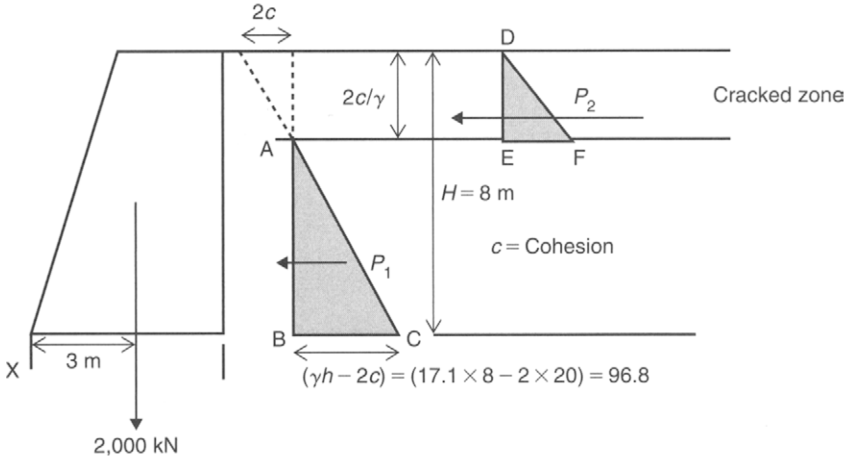


Figure 15.26 Water pressure due to cracked zone

The weight of the retaining wall is 2,000 kN per meter run, acting at a distance 3 m from the toe of the retaining wall, *X*, as shown. The height of the retaining wall is 8 m. The friction angle between the base of the retaining wall and the soil is 25°. Assume that the tension cracks are filled with water.

Solution

STEP 1: Find the thickness of the cracked zone.

$$\text{thickness of the cracked zone} = 2c/\gamma = 2 \times 20/17.1 = 2.34\,\text{m}$$

STEP 2: Find the pressure at the bottom.

$$\text{pressure at B} = (\gamma h - 2c) = (17.1 \times 8 - 2 \times 20) = 96.8$$

$$\text{pressure at E} = \gamma_w h = 9.81 \times 2.34 = 22.9\,\text{kN}$$

$$\gamma_w = \text{density of water} = 9.81\,\text{kN/m}^3$$

STEP 3: Find the lateral forces acting on the wall.

$$\text{lateral forces acting on the wall} = \text{area of triangle ABC} + \text{area of triangle DEF}$$

Note that in this case it is assumed that the cracked zone is filled with water.

$$\text{area of triangle ABC} = 1/2 \times 96.8 \times 8 = 387.2\,\text{kN}$$
$$\text{area of triangle DEF} = 1/2 \times 22.9 \times 2.34 = 26.8\,\text{kN}$$
$$\text{total sliding force} = 414\,\text{kN}$$

STEP 4: Find the resistance to sliding.

$$\text{resistance to sliding} = \text{friction angle} \times \text{weight of the retaining wall}$$
$$\text{resistance to sliding} = \tan 25^\circ \times 2{,}000\,\text{kN} = 0.466 \times 2{,}000 = 932\,\text{kN}$$

$$\text{factor of safety against sliding} = 932/414 = 2.25$$

Usually, a factor of safety of 2.5 is desired. Hence, it will be necessary to increase the weight of the retaining wall.

STEP 5: Find the overturning moment.

The resultant of the triangle ABC and DEF acts at a distance 1/3 from the base.

$$\text{distance to the resulting force ABC from the bottom} = (8 - 2.34)/3 = 1.89\,\text{m}$$
$$\text{distance to the resulting force DEF from the bottom} = (8 - 2.34) + 2.34/3 = 6.44\,\text{m}$$

The overturning moment is obtained by obtaining moments around point X.

$$\text{overturning moment of ABC} = \text{force} \times \text{moment arm} = 387.2 \times 1.89 = 731.8\,\text{kN m}$$
$$\text{overturning moment of DEF} = \text{force} \times \text{moment arm} = 26.8 \times 6.44 = 172.6\,\text{kN m}$$
$$\text{total overturning moment} = 904.4\,\text{kN m}$$

The resistance to overturning is provided by the weight of the retaining wall.

$$\text{resistance to overturning} = 2{,}000 \times 3 = 6{,}000\,\text{kN m}$$
$$\text{factor of safety against overturning} = 6{,}000/904.4 = 6.63$$

The resistance against overturning is sufficient.

15.7.3 Lateral Earth Pressure Coefficient for Clayey Soils (Passive Condition)

Passive conditions occur in front of a retaining wall as shown in Fig. 15.27. Due to pressure from the backfill, the retaining wall will try to move toward direction Z. During the process, the soil in front of the retaining wall will be subjected to the passive condition. As mentioned earlier, if the soil is subjected to a force that loosens the soil, then active conditions prevail. On the other hand, if the force applied subjects the soil to a denser condition, then passive conditions prevail.

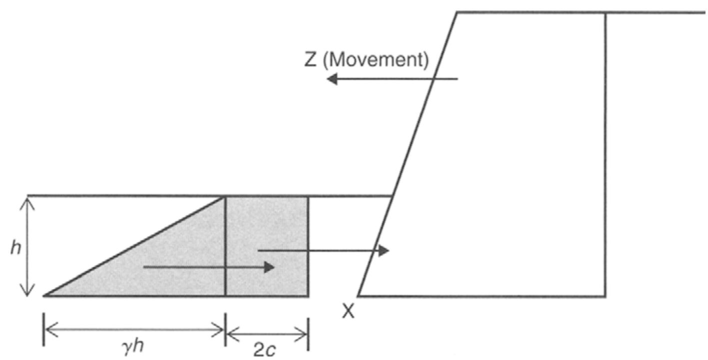


Figure 15.27 Passive earth pressure in clay soil

Here c is cohesion and γ is the density of the clay.

$$\text{total force} = \text{area of the triangle} + \text{area of the rectangle}$$
$$= \gamma \times h^2/2 + 2 \times c \times h$$

The force due to the triangle portion acts at a distance $h/3$ above the bottom. The force due to the rectangular portion acts at a distance $h/2$ from the bottom.

$$\text{moment around point X} = \text{moment of the triangle} + \text{moment of the rectangle}$$

$$\text{moment of the triangle} = (\gamma \times h^2/2) \times h/3 = \gamma \times h^3/6$$

$$\text{moment of the rectangle} = 2c \times h^2/2$$

$$\text{total moment around } X = \gamma \times h^3/6 + 2c \times h^2/2$$

Design Example 15.7

This example explores a gravity retaining wall with a clay backfill. In this case, the cracked zone is filled with water, and passive pressure in front of the retaining wall is considered. Find the factor of safety of the retaining wall shown in Fig. 15.28 against sliding failure and overturning. The cohesion of the clay backfill is 20 kPa. The density of the soil, γ, is 17.1 kN/m^3. The weight of the retaining wall is 2,000 kN per meter, acting 3 m from the toe of the retaining wall, X, as shown in Fig. 15.28. The height of the retaining wall is 8 m. The friction angle between the base of the retaining wall and the soil is 25°. Assume that the tension cracks are filled with water. Assume that a clay with a cohesion of 20 kPa is placed in front of the retaining wall, and the passive pressure is not negligible. The depth of the soil in front of the retaining wall is 1.5 m.

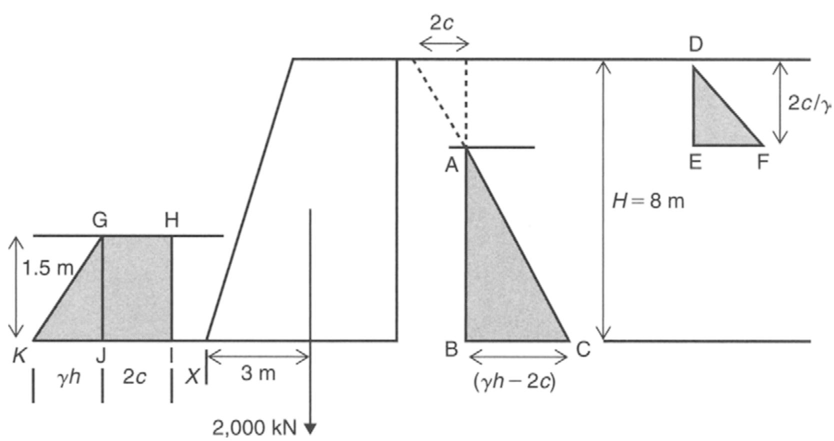


Figure 15.28 Force diagram including the passive earth pressure

Solution

STEP 1: Find the thickness of the cracked zone.

$$\text{height of the cracked zone DE} = 2c/\gamma = 2 \times 20/17.1 = 2.34\,\text{m}$$

STEP 2: Find the pressure at the bottom, BC.

$$\text{pressure at } B = (\gamma h - 2c) = (17.1 \times 8 - 2 \times 20) = 96.8\,\text{kN/m}^2$$

STEP 3: Find the pressure due to water.

$$\text{pressure at } E = \gamma_W \times h = 9.81 \times 2.34 = 22.9\,\text{kN/m}^2$$

STEP 4: Find the pressure in the passive side.

$$\text{pressure at K} = \text{KJ} + \text{JI} = \gamma h + 2c = 17.1 \times 1.5 + 2 \times 20 = 65.7\,\text{kN/m}^2$$

STEP 5: Find the lateral forces acting on the wall.

$$\text{area of triangle ABC} = 1/2 \times 96.8 \times (8 - 2.34) = 273.9\,\text{kN}$$

$$\text{area of triangle DEF} = 1/2 \times 22.9 \times 2.34 = 26.8\,\text{kN}$$

$$\begin{aligned}\text{area of triangle GKJ} &= 1/2 \times (\gamma \times h) \times 1.5\\ &= 1/2 \times (17.1 \times 1.5) \times 1.5\\ &= 19.2\,\text{kN}\end{aligned}$$

$$\begin{aligned}\text{area of rectangle GHIJ} &= 1.5 \times 2c\\ &= 1.5 \times 2 \times 20\\ &= 60\,\text{kN}\end{aligned}$$

$$\text{Total sliding force} = 273.9 + 26.8 - 19.2 - 60\,\text{kN} = 221.5\,\text{kN}$$

STEP 6: Find the resistance to sliding.

$$\begin{aligned}\text{resistance to sliding} &= \text{friction angle}\\ &\quad \times \text{weight of the retaining wall}\end{aligned}$$

$$\begin{aligned}\text{resistance to sliding} &= \tan 25^\circ \times 2{,}000\,\text{kN}\\ &= 0.466 \times 2{,}000 = 932.6\,\text{kN}\end{aligned}$$

$$\text{factor of safety against sliding} = 932.6/221.5 = 4.21$$

Usually, a factor of safety of 2.5 is desired. Hence, the factor of safety is sufficient.

STEP 7: Find the overturning moment.

The resultants of triangles ABC, DEF, and GKJ act at a distance 1/3 from the base.

$$\begin{aligned}\text{distance to the resulting force ABC from the bottom} &= (8 - 2.34)/3\\ &= 1.89\,\text{m}\end{aligned}$$

$$\begin{aligned}\text{distance to the resulting force DEF from the bottom} &= (8 - 2.34)\\ &\quad + 2.34/3 = 6.44\,\text{m}\end{aligned}$$

$$\text{distance to the resulting force GKJ from the bottom} = 1.5/3 = 0.5\,\text{m}$$

$$\text{distance to the resulting force GHJI from the bottom} = 1.5/2 = 0.75\,\text{m}$$

The overturning moment is obtained by obtaining moments around point X.

$$\text{overturning moment of ABC} = \text{force} \times \text{moment arm} = 273.9 \times 1.89 = 517.7\,\text{kN m}$$

$$\text{overturning moment of DEF} = \text{force} \times \text{moment arm} = 26.8 \times 6.44 = 172.6\,\text{kN m}$$

GKJ and GHJI are generating resisting moments.

$$\text{resisting moment of GKJ} = \text{force} \times \text{moment arm} = 19.2 \times 0.5 = 9.6\,\text{kN m}$$

$$\text{resisting moment of GHJI} = \text{force} \times \text{moment arm} = 60 \times 0.75 = 45\,\text{kN m}$$

$$\text{total overturning moment} = 517.7 + 172.6 - 9.6 - 45 = 635.7\,\text{kN m}$$

Resistance to overturning is provided by the weight of the retaining wall.

$$\text{resistance to overturning} = 2{,}000 \times 3 = 6{,}000\,\text{kN m}$$

$$\text{factor of safety against overturning} = 6{,}000/635.7 = 9.4$$

The resistance against overturning is sufficient.

Design Example 15.8

This example explores a sand layer above the clay layer in the active condition, with no groundwater considered. The top layer of the backfill is sand with friction angle, φ, of 30°. The density of the sand layer is 18 kN/m^3. The clay layer below the sand layer has a cohesion of 15 kPa and a density of 17.3 kN/m^3. The weight of the wall is 3,000 kN per meter, and the center of gravity is acting 2 m from the toe. The friction angle between concrete and the soil, δ, at the bottom of the retaining wall is found to be 25°. See Fig. 15.29.

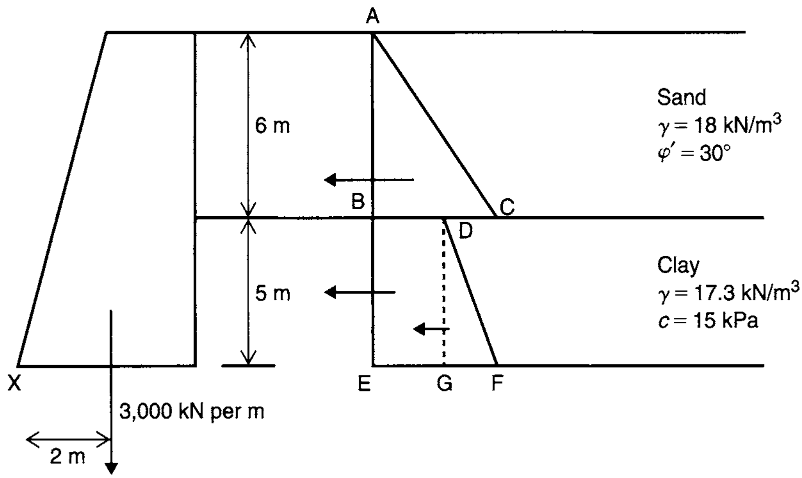


Figure 15.29 Sand layer underlain by a clay layer

Solution

First, the sand layer is considered.

STEP 1: Find the lateral earth pressure coefficient.

$$K_a = \tan^2(45 - \varphi/2)$$

$$K_a = \tan^2(45 - 30/2) = 0.33$$

STEP 2: Find the lateral earth pressure.

$$\begin{aligned}\text{lateral earth pressure at point B (sand layer) (BC)} &= K_a \times \gamma \times 6 \\ &= 0.33 \times 18 \times 6\,\text{kPa} \\ &= 35.6\,\text{kPa}\end{aligned}$$

Note that the lateral earth pressure in sand and clay at the same depth has different values.

Next, the clay layer is considered.

$$\begin{aligned}\text{lateral earth pressure at point B (clay layer) (BD)} &= \gamma \times h - 2c \\ &= 18 \times 6 - 2 \times 15 \\ &= 78\,\text{kPa}\end{aligned}$$

$$\text{lateral earth pressure at point E (clay layer) } \text{EF} = \text{EG} + \text{GF}$$
$$\text{EF} = \text{BD} + \text{GF}$$
$$\text{EF} = 78 + \gamma \times h$$
$$\text{EF} = 78 + (17.3 \times 5)$$
$$\text{EF} = 164.5\,\text{kpa}$$

STEP 3: Find the forces due to lateral earth pressure.

$$\text{area of triangle ABC} = 0.5 \times 35.6 \times 6 = 106.8\,\text{kN}$$
$$\text{area of rectangle BDEG} = 78 \times 5 = 390\,\text{kN}$$
$$\text{area of triangle DGF} = 0.5 \times \text{ GF } \times 5$$
$$= 0.5 \times (17.3 \times 5) \times 5\text{kN}$$
$$= 216.3\,\text{kN}$$
$$\text{total sliding force} = 106.8 + 390 + 216.3$$
$$= 713.1\,\text{kN}$$

STEP 4: Find the resistance against sliding.

$$\text{resistance against sliding} = \tan(\delta) \times \text{weight of the retaining wall}$$
$$= \tan(25) \times 3{,}000$$
$$= 1{,}399\,\text{kN}$$
$$\text{factor of safety against sliding} = 1{,}399/713.1 = 1.96$$

A factor of safety of 2.5 to 3.0 is desired. Hence, it will be necessary to increase the size of the wall.

STEP 5: Find the height to center of gravity of forces.

$$\text{height to center of gravity of triangle ABC} = 5 + 6/3 = 7\,\text{m}$$
$$\text{height to center of gravity of triangle DGF} = 5/3 = 1.67\,\text{m}$$
$$\text{height to center of gravity of rectangle BDEG} = 5/2 = 2.5\,\text{m}$$

STEP 6: Find the overturning moment around the toe (point X).

$$\text{overturning moment due to triangle ABC} = 106.8 \times 7$$
$$= 747.6\,\text{kN m}$$

$$\text{overturning moment due to triangle DGF} = 216.3 \times 1.67 = 361.2\,\text{kN m}$$

$$\text{overturning moment due to rectangle BDEG} = 390 \times 2.5 = 975\,\text{kN m}$$

$$\text{total overturning moment} = 747.6 + 361.2 + 975 = 2{,}083.8\,\text{kN m}$$

$$\text{total resisting moment} = 2 \times 3{,}000\,\text{kN m} = 6{,}000\,\text{kN m}$$

$$\text{factor of safety against overturning} = 6{,}000/2{,}083.8 = 2.88$$

Typically a factor of safety of 2.5 to 3.0 is desired.

15.7.4 Earth Pressure Coefficients for Cohesive Backfills

Sandy soil (noncohesive) is the most suitable type of soil that could be used behind retaining walls. Sandy soils drain freely, and groundwater can be drained away using drains. This reduces the pressure on the retaining wall. See Fig. 15.30.

An active earth pressure coefficient of K_a can be used for sandy soils.

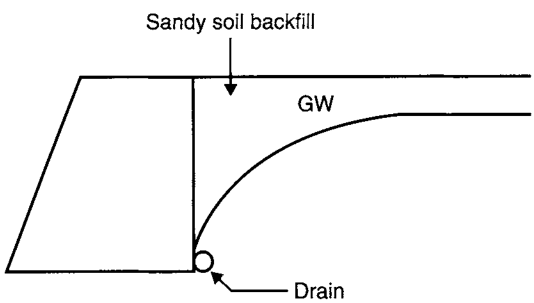


Figure 15.30 Drainage in retaining walls

15.7.5 Drainage Using Geotextiles

Drainage board can be used behind the wall to facilitate drainage. Filter fabric is used in front of the drainage board to filter any solid particles. See Fig. 15.31.

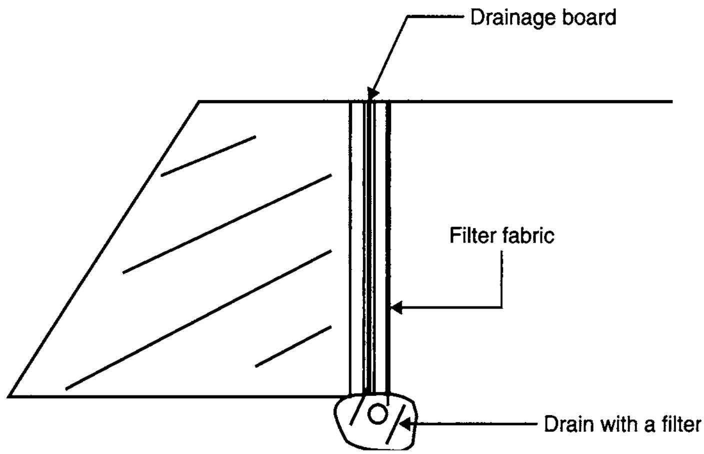


Figure 15.31 Drainage in retaining walls

15.7.6 Consolidation of Clayey Soils

Clay soils are not desired as a backfill material for retaining walls. In some instances, clay soils are used to reduce costs. The geotechnical engineer needs to investigate the consolidation properties of clay soils. Clayey material tends to consolidate and generate an additional thrust on the retaining wall. See Fig. 15.32.

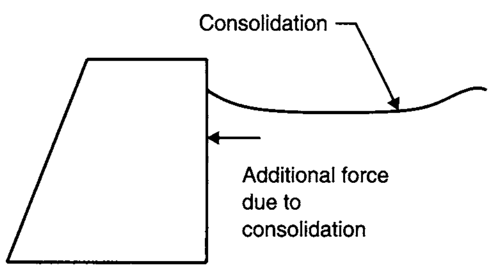


Figure 15.32 Thrust due to clay consolidation

How can the designer account for additional force due to clay consolidation? Complex computer programs are used to account for the additional thrust due to consolidation.

16

Gabion Walls

16.1 Introduction

Computations involved in gabion walls are no different from regular gravity earth retaining walls. The earth pressure forces are computed as usual, and the stability of the wall with respect to rotation and sliding is computed.

Gabion baskets are manufactured in different sizes. The typical basket is approximately 3 ft in size. Smaller baskets are easier to handle. At the same time, smaller baskets need more seams to be connected. Not all Gabion baskets are perfect cubes. Some baskets are made with elongated shapes. See Fig. 16.1.

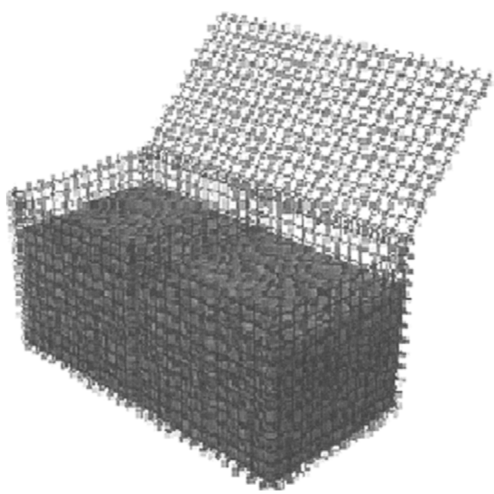


Figure 16.1 Gabion basket

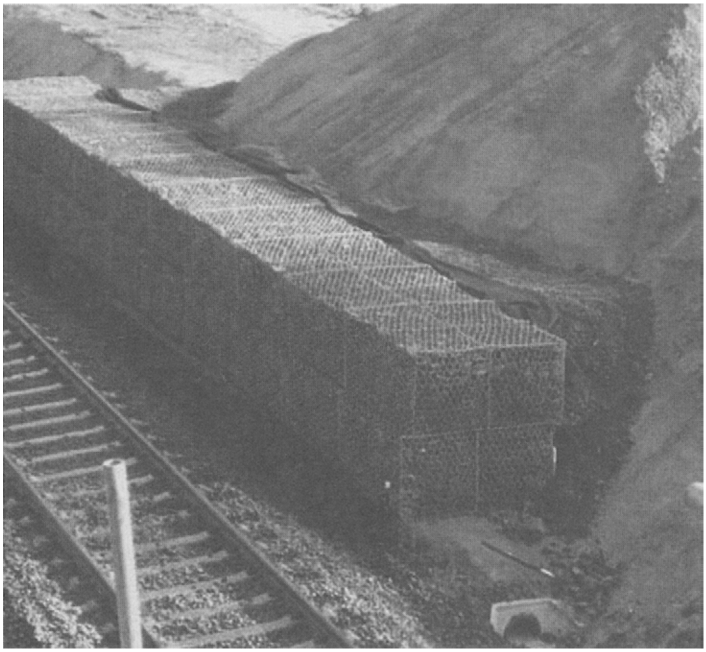


Figure 16.2 Gabion wall under construction

Gabion baskets can be connected to build a wall. See Fig. 16.2.

Design Example 16.1

A 15 ft high embankment has to be contained. Five foot Gabion baskets to be placed as shown in Fig. 16.3 have been proposed. Find the factor of safety of the gabion wall. The soil density, γ, is 120 pcf, the soil friction angle, φ, is 30°, the friction angle between the gabion baskets and the soil, δ, is 20°.

Assume all groundwater is drained. See Fig. 16.3.

Solution

STEP 1: Find the lateral earth pressure coefficient.

$$\begin{aligned} K_a &= \tan^2(45 - \varphi/2) \\ &= \tan^2(45 - 30/2) \\ &= 0.333 \end{aligned}$$

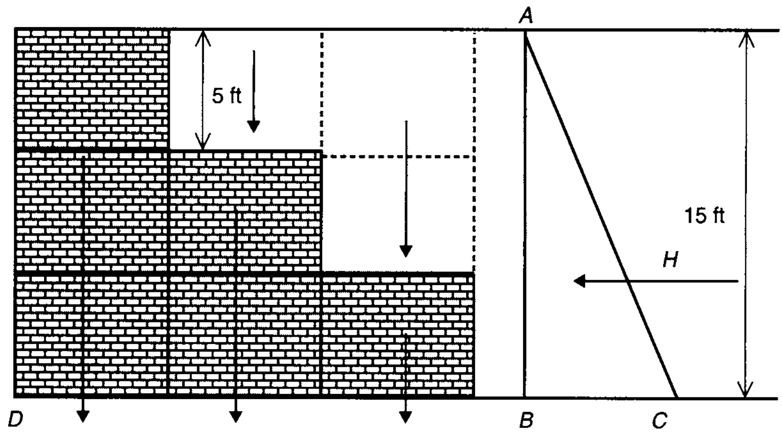


Figure 16.3 Gabion wall and forces

STEP 2: Compute the horizontal force due to earth pressure.

$$\text{horizontal force } (H) = \text{pressure due to soil}$$

$$BC = K_a \times \gamma \times h = 0.333 \times 15 \times (120) = 600 \text{ psf}$$

$$\gamma = 120 \text{ pcf and } h = 15 \text{ ft}$$

$$\text{area } ABC = 1/2 \times 15 \times BC = 1/2 \times 15 \times 600$$

$$= 4{,}500 \text{ lb}$$

$$\text{total horizontal force } (H) = 4{,}500 \text{ lb}$$

STEP 3: Compute the weight of the gabion wall.

Assume the density of stones to be 160 pcf. There are six gabion baskets.

$$W = \text{weight of gabion baskets} = 6 \times (5 \times 5) \times 160 = 24{,}000 \text{ lb}$$

There is soil sitting on the gabion baskets.

$$\text{weight of soil} = 3 \times (5 \times 5) \times 120 = 9{,}000 \text{ lb}$$

$$\text{density of soil} = 120 \text{ pcf}$$

$$\text{total weight of the gabion wall} = 24{,}000 + 9{,}000 = 33{,}000 \text{ lb}$$

STEP 4: Calculate the resistance against sliding.
See Fig. 16.4.

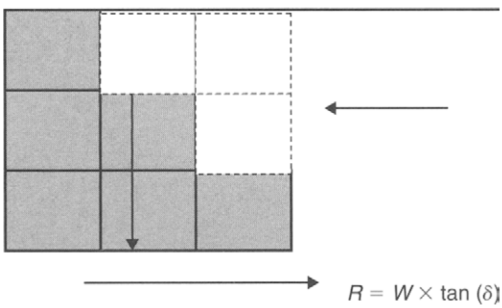


Figure 16.4 Forces acting on a gabion wall

$$\text{resistance against sliding} = \text{weight of the gabion wall} \times \tan(\delta)$$

$$R = 33{,}000 \times \tan(20^\circ)$$
$$= 12{,}011 \text{ lbs}$$

$$\text{factor of safety against sliding} = \frac{\text{resistance against sliding}}{\text{horizontal force}}$$
$$= 12{,}011/4{,}500$$
$$= 2.67$$

STEP 5: Calculate the overturning moment.
The resultant force acts 1/3 of the length of the triangle. Obtain moments around point D. See Fig. 16.3.

$$\text{overturning moment} = H \times 1/3 \times 15$$
$$= 4{,}500 \times 5$$
$$= 22{,}500 \text{ lb ft}$$

STEP 6: Calculate the resisting moment.
There are six rock blocks and three soil blocks. There are three blocks (rock) in the far left and one block at the far right. There are two blocks in the middle. See Fig. 16.5.

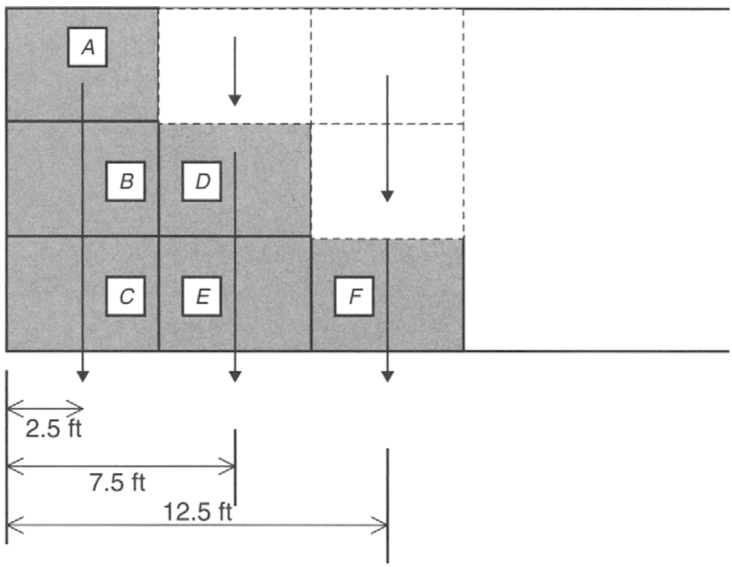


Figure 16.5 Weight due to gabion baskets

The resisting moment due to gabion baskets can be calculated as

resisting moment due to three blocks in the left ABC

$$= 3 \times (5 \times 5) \times 160 \times 2.5 = 30{,}000\,\text{lb ft}$$

$$\text{resisting moment due to two blocks } DE = 2 \times (5 \times 5) \times 160 \times 7.5 = 60{,}000\,\text{lb ft}$$

$$\text{resisting moment due to one block } F = 1 \times (5 \times 5) \times 160 \times 12.5 = 50{,}000\,\text{lb ft}$$

resisting moment due to soil sitting on top of gabion baskets

$$= 1 \times (5 \times 5) \times 120 \times 7.5 + 2 \times (5 \times 5) \times 120 \times 12.5$$

$$= 22{,}500 + 75{,}000$$

$$\text{resisting moment} = 97{,}500\,\text{lb ft}$$

STEP 7: Determine the factor of safety against overturning.

$$\text{factor of safety against overturning} = \frac{\text{resisting moment}}{\text{overturning moment}}$$
$$= 97{,}500/22{,}500$$
$$= 4.33$$

16.2 Log Retaining Walls

Timber logs are arranged to build log walls. These types of retaining walls are rare today, but can be used in wetland mitigation and for temporary structures. See Figs. 16.6 and 16.7.

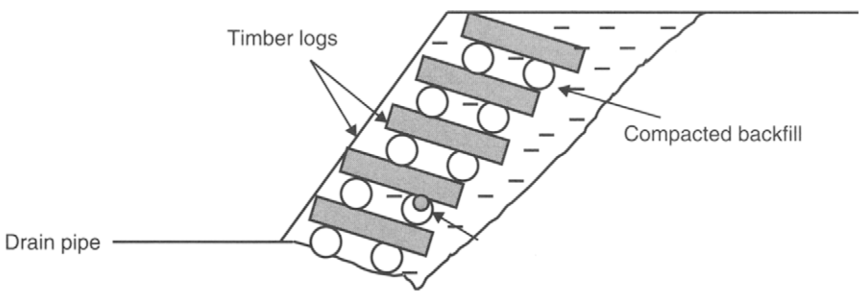


Figure 16.6 Log retaining wall (side view)

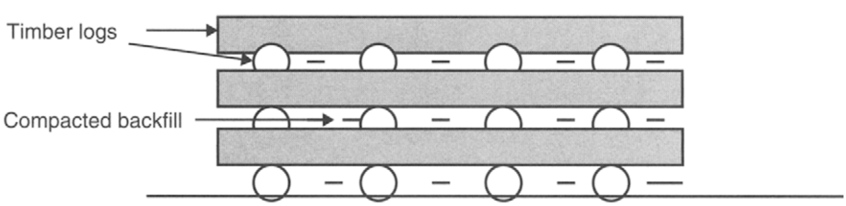


Figure 16.7 Log retaining wall (frontal view)

Log walls can be constructed cheaply, and are still widely used in areas where timber is common.

16.2.1 Construction Procedure of Log Walls

See Figs 16.8 and 16.9.

- Place the next layer of logs and backfill.
- Continue the process.

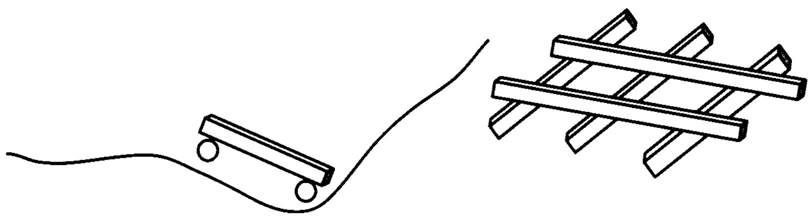


Figure 16.8 Placement of timber logs

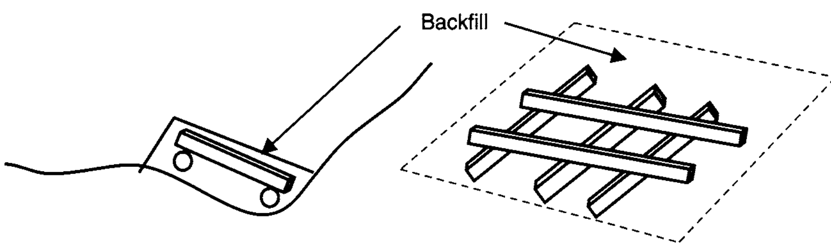


Figure 16.9 Placement of backfill between logs and compacting the soil

17

Reinforced Earth Walls

17.1 Introduction

Reinforced earth walls are becoming increasingly popular. Reinforced earth walls are constructed by holding the lateral earth pressure through metal strips. See Fig. 17.1.

The soil pressure in the reinforced earth walls is resisted by metal strips. See Fig. 17.2.

The following equations can be developed to compute the lateral earth force acting on a facing unit.

17.2 Equations to Compute the Horizontal Force on the Facing Unit, *H*

$$\sigma'_{\mathrm{v}} = \text{vertical effective stress}$$

$$\sigma'_{\mathrm{h}} = K_{\mathrm{a}} \times \sigma'_{\mathrm{v}} = \text{horizontal stress}$$

$$K_{\mathrm{a}} = \text{lateral earth pressure coefficient} = \tan^2(45 - \varphi/2)$$

$$\text{lateral soil pressure, } F = (K_{\mathrm{a}} \times \sigma'_{\mathrm{v}}) \times \text{ area of the facing unit}$$

17.3 Equations to Compute the Metal-Soil Friction, *P*

The metal-soil friction depends on the vertical effective stress acting on the strip, the area of the metal strip, and the friction angle between metal and soil.

$$\text{metal-soil friction, } P = 2 \times (\sigma'_{\mathrm{v}} \times \tan\delta) \times \text{area of the strip}$$

where

δ = friction angle between metal and soil
σ'_v = vertical effective stress
$\sigma'_v \times \tan\delta$ = horizontal friction force

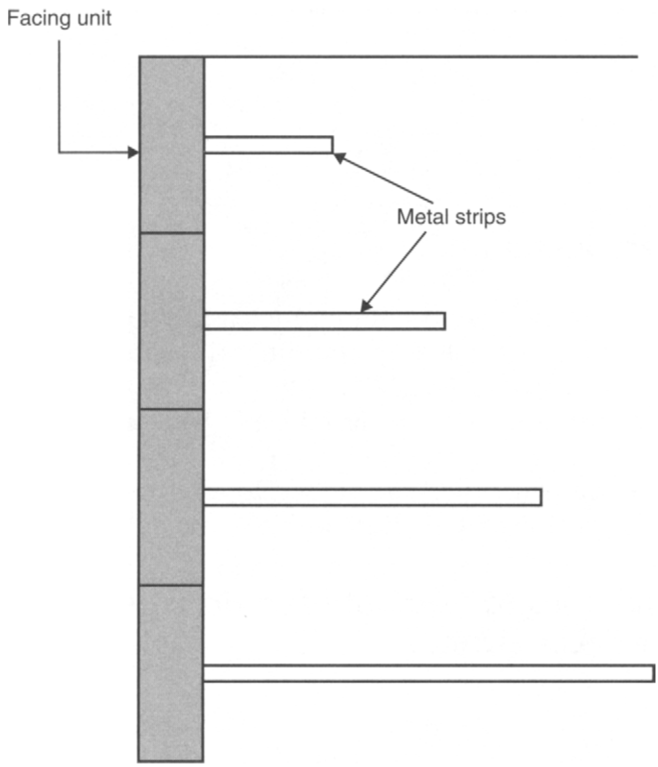


Figure 17.1 Reinforced earth wall

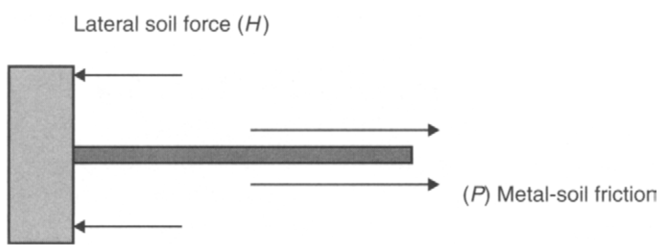


Figure 17.2 Forces acting on facing units and metal strips

The quantity is multiplied by 2, since there are two surfaces, top and bottom, in the metal strip. See Fig. 17.2.

$$\text{factor of safety (F.O.S.)} = P/H$$

Figure 17.3 Construction of metal strips (*Source*: California DOT)

Design Example 17.1

Find the length of metal strips *A*, *B*, and *C* in Fig. 17.4. The density, γ, of the soil is found to be 17.5 kN/m^3, and the friction angle, φ, of the soil is 25°. The facing units are 1 m × 1 m and the metal strips are 0.5 m wide. Find the length of the metal strips if the required factor of safety is 2.5.

Solution

STEP 1: Find the vertical effective stress at the center of the facing unit.

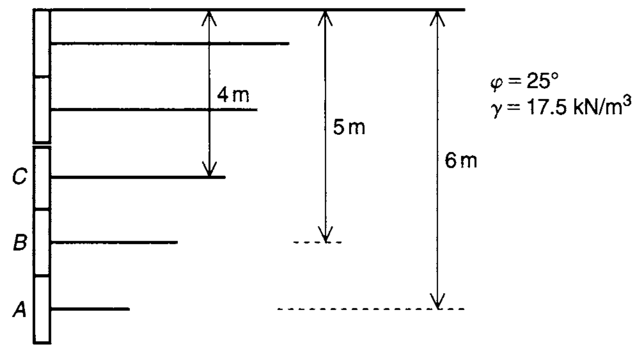


Figure 17.4 Reinforcing strips

For facing unit A, find the vertical effective stress at the center of the facing unit, σ'_v

$$\begin{aligned}\sigma'_v &= \gamma \times h\\ &= 17.5 \times 6\,\text{kN/m}^2\\ &= 105\,\text{kN/m}^2\end{aligned}$$

Find the lateral earth pressure coefficient, K_a

$$\begin{aligned}K_a &= \tan^2(45 - \varphi/2)\\ &= \tan^2(45 - 25/2)\\ &= 0.406\end{aligned}$$

Find the horizontal stress at the center of the facing unit (σ'_h)

$$\begin{aligned}\sigma'_h &= K_a \times \sigma'_v\\ &= 0.406 \times 105\,\text{kN/m}^2\\ &= 42.6\,\text{kN/m}^2\end{aligned}$$

$$\begin{aligned}\text{area of the facing unit, } A &= \text{width} \times \text{length}\\ &= 1\,\text{m} \times 1\,\text{m} = 1\,\text{m}^2\end{aligned}$$

$$\begin{aligned}\text{total horizontal force on the facing unit } &= \text{horizontal stress}\\ &\quad \times \text{area of the facing unit}\\ &= 42.6\,\text{kN}\end{aligned}$$

STEP 2: Find the metal-soil friction in facing unit A

$$\text{metal-soil friction, } P = 2 \times (\sigma'_V \times \tan\delta) \times \text{area of the strip}$$
$$= 2 \times 105 \times \tan 20 \times 0.5\,\text{L}$$
$$= 38.2\,\text{L}$$

required factor of safety (F.O.S.) = 2.5
L = length of strip

F.O.S. = metal soil friction/horizontal force on the facing unit

$$2.5 = 38.2\,\text{L}/42.6$$
$$\text{L} = 2.79\,\text{m}$$

The length of the metal strip within the active soil zone does not contribute to the strength. Hence the metal strip should extend 2.79 m beyond the active failure zone. See Figs. 17.5 and 17.6.

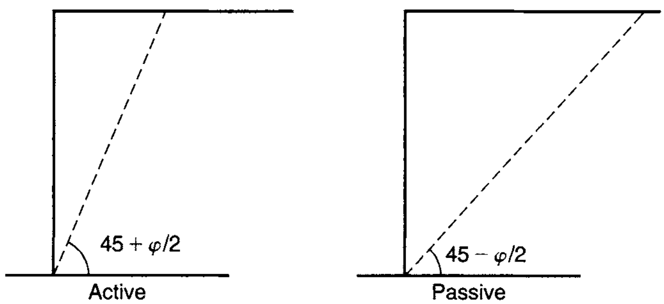


Figure 17.5 Active and passive failure planes

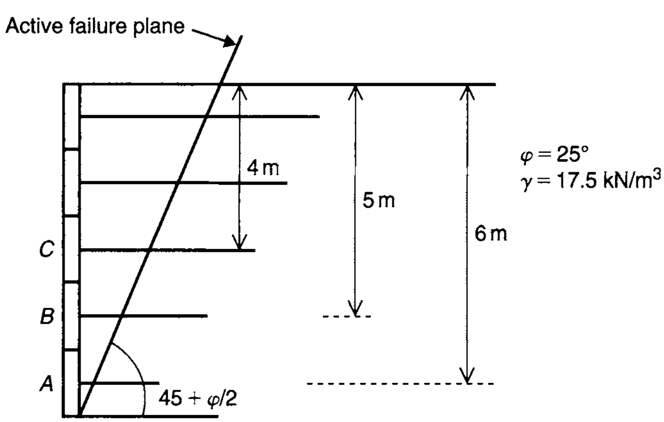


Figure 17.6 Metal strips and active failure plane

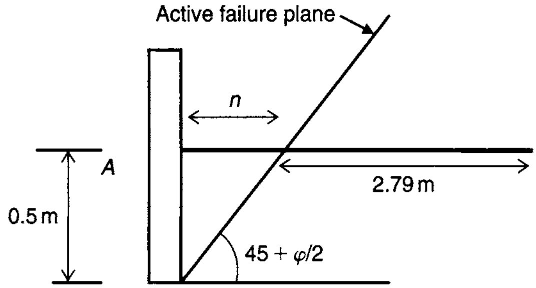


Figure 17.7 Facing unit A and the active failure plane

Figure 17.7 shows the facing unit A and the active failure plane.

The distance n can be computed since the distance to the center of the facing unit is 0.5 m.

$$n = 0.5 \times \tan(45 - \varphi/2)$$
$$= 0.5 \times \tan(45 - 25/2)$$
$$= 0.32\,\text{m}$$

$$\text{total required length of the strip} = 0.32 + 2.79 = 3.11\,\text{m}$$

The same procedure can be adopted to find the lengths of other metal strips.

18

Geotechnical Engineering Software

There are many geotechnical engineering software packages available on the market. In this chapter, an introduction to some of the more famous software will be discussed.

18.1 Shallow Foundations

18.1.1 SPT Foundation

SPT Foundation is a program available at www.geoengineer.org/sptprogram.html.

The program computes the bearing capacity of shallow foundations using the Terzaghi bearing capacity equation. The SPT blow count in the soil is used to obtain the friction angle to be used in the Terzaghi bearing capacity equation. The program uses the correlation provided by Hatakanda and Uchida (1996).

The program is capable of conducting the settlement of foundations, as well. The average SPT (N) value within the depth of influence below the footing is used for settlement computations. A portion of the program is available without charge. As per authors of the website, they use the method proposed by Burland and Burbidge (1984).

18.1.2 ABC Bearing Capacity Computation

Free programs from the following website are capable of conducting bearing capacity computations: http://www-civil.eng.ox.ac.uk/people/cmm/software/abc/.

ABC can be used to solve bearing capacity problems with many layers of soil. The program can be used to solve rigid foundations resting on a cohesive-frictional soil mass that is loaded to failure by a central vertical force. The program provides option for the cohesion to vary linearly with depth.

In reality, such situations may be rare. The program authors claim that the computations are done using a finite element grid using stress analysis without resorting to approximations. The program is fully documented and provided with a user manual. It is freeware at the present time, but will probably be commercially sold soon.

18.1.3 Settle 3D

Settle 3D is a program by www.rocscience.com for analysis of consolidation and settlement under foundations, embankments, and surface excavations. The program is capable of conducting 3D analysis of foundation settlement.

18.1.4 Vdrain—Consolidation Settlement

Vdrain is a free program available from www.cofra.com that can be used to calculate settlement and consolidation of soft soils.

18.1.5 Embank

Embank is a program that computes settlement under embankment loads. Embankments are common for bridge abutments. The program computes vertical settlement due to embankment loads. For the case of a strip symmetrical vertical embankment loading, the program superimposes two vertical embankment loads. For the increment of vertical stresses at the end of fill, the program internally superimposes a series of 10 rectangular loads to create the end of fill condition. This program can be downloaded free of charge from the FHWA website.

18.2 Slope Stability Analysis

18.2.1 Reinforced Soil Slopes (RSS)

RSS is a program capable of design and analysis of reinforced soil slopes (reinforced soil slopes). The software is based on the FHWA manual *Reinforced Soil Structures, Volume I—Design and Construction Guidelines* (FHWA-RD-89-043).

This program analyzes and designs soil slopes strengthened with horizontal reinforcement, as well as analyzing unreinforced soil slopes. The analysis is performed using a two-dimensional limit equilibrium method. The program is a predecessor of very popular STABL computer program from the 1990s (Federal Highway Administration, USA, http://www.fhwa.dot.gov/index.html).

18.2.2 Mechanically Stabilized Earth Walls (MSEW)

The program MSEW is capable of conducting design and analysis of mechanically stabilized earth walls. The program is available from http://www.msew.com/downloads.htm.

18.3 Bridge Foundations

18.3.1 FB Multipier

The FB MultiPier software package is capable of analyzing bridge piers connected to each other. Each pier is considered to be supported on piles or caissons. The settlement and load bearing capacity of foundations can be computed using this program. The authors state that this program conducts a nonlinear structural finite element analysis, and a nonlinear static soil model for axial and lateral soil behavior.

The product needs to be licensed prior to service, and can be downloaded from http://bsi-web.ce.ufl.edu/products/.

18.4 Rock Mechanics

Rock Mechanics is freeware available from Southern Illinois University Carbondale. The program can be downloaded from their website, http://www.engr.siu.edu/mining/kroeger/.

The program considers rock joints and water pressure to compute the stability. See Fig. 18.1.

18.4.1 Wedge Failure Analysis

Wedge failure is an important software package to tunneling engineers and rock slope stability computations. The formation of wedges due to joints needs to be analyzed. The free computer program available at

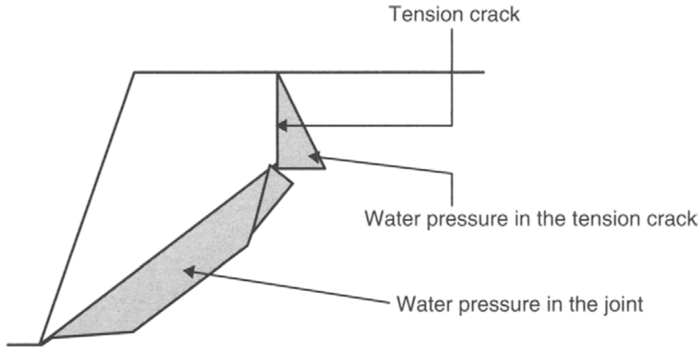


Figure 18.1 Rock slope stability

http://www.engr.siu.edu/mining/kroeger/ can be used to analyze the formation of wedges in a rock formation.

18.4.2 Rock Mass Strength Parameters

RocLab is a software program available from http://www.rocscience.com/ for determining rock mass strength parameters. The user can use the program to visualize the effects of changing rock mass parameters on the failure envelopes. Rocscience.com collaborates with E. Hoek, one of the most distinguished authorities in the rock mechanics field. Together they provide very valuable information on rock mechanics.

18.5 Pile Design

18.5.1 Spile

Spile is a versatile program used to determine the ultimate vertical static pile capacity. The program is capable of computing the vertical static pile capacity in clayey soils and sandy soils. The program is designed based on equations presented by Nordlund (1979), Meyerhof (1976), and Tomlinson (1985).

18.5.2 Kalny

The Kalny software package conducts pile group analysis. This program provides pile forces for regular and irregular pile groups for multiple load combinations in a single spreadsheet. The engineer can check governing forces for "corner" piles or review the forces on all piles in a two-dimensional fashion.

In certain situations, certain piles may have to be discarded due to doglegging or damage during driving.

18.6 Lateral Loading Analysis—Computer Software

When a lateral load (P) is applied as shown in Fig. 18.2, the following resistances would be developed.

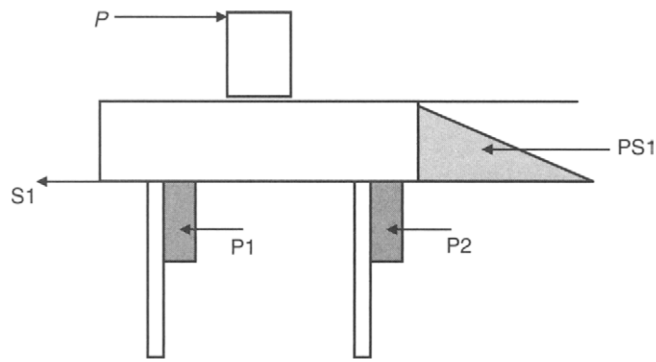


Figure 18.2 Lateral loading analysis

PS1 = passive soil resistance due to pile cap on one side of the pile cap
S1 = skin friction at the base of the pile cap
P1 and P2 = lateral soil resistance of piles

If the pile cap is connected to other pile caps with tie beams, there would be resistance due to tie beams, as well.

18.6.1 Lateral Loading Analysis Using Computer Programs

The input parameters to lateral loading analysis computer programs are twofold.

1. Pile parameters.
2. Soil parameters.

The pile parameters include

- Pile diameter.
- Center to center spacing of piles in the group.
- Number of piles in the group.
- Pile cap dimensions.

18.6.2 Soil Parameters for Sandy Soils

Soil parameters should be provided to the computer for each strata.

- Strata thickness.
- φ' value of the strata.
- Coefficient of subgrade reaction, k.

Note that the φ' value of sandy soil can be calculated using the following equation (Peck et al., 1974).

$$\varphi' = 53.881 - 27.6034 \times e^{-0.0147N}$$

where

N = average SPT value of the strata.

Note that the coefficient of subgrade reaction, k, can be obtained using Table 18.1. Similarly, soil parameters for other strata also need to be provided.

Table 18.1 Coefficient of subgrade reaction (k) vs. N (SPT)

SPT (N)	8	10	15	20	30
k (kN/m^3)	2.67×10^{-6}	4.08×10^{-6}	7.38×10^{-6}	9.74×10^{-6}	1.45×10^{-6}

Source: Johnson and Kavanaugh (1968).

18.6.3 Soil Parameters for Clayey Soils

The soil parameters required for clayey soils are

- S_u, the undrained shear strength. S_u is obtained by conducting unconfined compressive strength tests.

- ε_c, the strain corresponding to 50% of the ultimate stress. If the ultimate stress is 3 tsi (tons per square inch), then ε_c is the strain at 1.5 tsi.
- k_s, the coefficient of subgrade reaction.

The coefficient of subgrade reaction for clay soils is obtained from Table 18.2.

Table 18.2 Coefficient of subgrade reaction vs. undrained shear strength

	Average undrained shear strength (tsf)		
	(0.5–1) tsf	(1–2) tsf	(2–4) tsf
k_s (static) lb/in.^{3}	500	1,000	2,000
k_s (cyclic) lb/in.^{3}	200	400	800

Source: Reese (1975).

18.7 Finite Element Method

Most geotechnical engineering software is based on the finite element method. The finite element method is considered to be the most powerful mathematical method that exists today to solve piling problems.

- Any type of soil condition could be simulated using a finite element method. See Fig. 18.3.
- Complicated soil profile is shown in Fig. 18.3. Nodes in each of the finite elements are given the soil properties of that layer, such as φ, γ (density), cohesion, SPT (N) value, and so on.
- The nodes of the elements in layer 1 are given the soil properties of layer 1. Similarly, the nodes in layer 2 will be given the soil properties of layer 2.
- Due to this flexibility, isolated soil pockets can also be effectively represented.

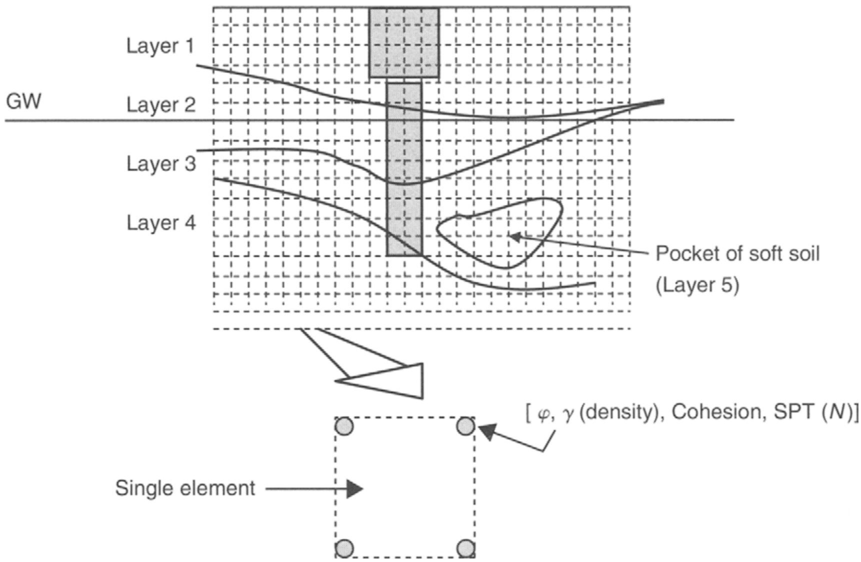


Figure 18.3 Finite element grid

18.7.1 Representation of Time History

- The capacity of a pile is dependent on the history of loading. A pile that was loaded gradually would have a higher capacity than a pile that was loaded rapidly.
- Assume that a developer is planning to construct a 10 story building in five years. In this case, the full building load on piles would gradually develop over a time period of five years.
- On the other hand, the developer could change the plan and decide to construct the 10 story building in two years. In this case, the full load on the piles would develop in two years. If the piles were to be fully loaded in two years, the capacity of the piles would be less than the first scenario.
- In such situations, finite element method could be used to simulate the time history of loading.

18.7.2 Groundwater Changes

The change of groundwater conditions also affects the capacity of piles. The change of groundwater level can be simulated easily, using the finite element method.

18.7.3 Disadvantages

The main disadvantage of the finite element method is its complex nature. In many cases engineers may wonder whether it is profitable to perform a finite element analysis.

18.7.4 Finite Element Computer Programs

Computer programs are available with finite element platforms. These programs can be used to solve a wide array of piling problems. The user is expected to have a working knowledge of finite element analysis to use these programs. More specialized computer programs are also available in the market. These programs do not require a knowledge of finite element analysis.

18.8 Boundary Element Method

- The boundary element method is a simplified version of the finite element method. In this method, only the elements at the boundaries are considered. See Fig. 18.4.

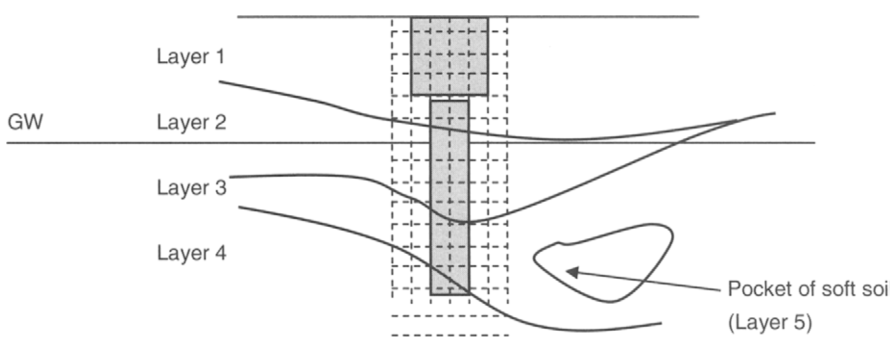


Figure 18.4 Boundary element method

- Only the elements at the soil-pile boundary are represented.
- In this method, the full soil profile is not represented. As shown in Fig. 18.4, the isolated soft soil pocket is not represented.
- On the other hand, fewer elements will make the computational procedure much simpler than the finite element method.

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19

Geotechnical Instrumentation

19.1 Inclinometer

A slight movement of soil has been observed prior to most landslides, earthquakes, and slope failures. Some believe that insects, horses, and pigs feel these slight movements. An inclinometer is designed to measure slight ground movements that occur prior to a slope failure. See Figs. 19.1 and 19.2.

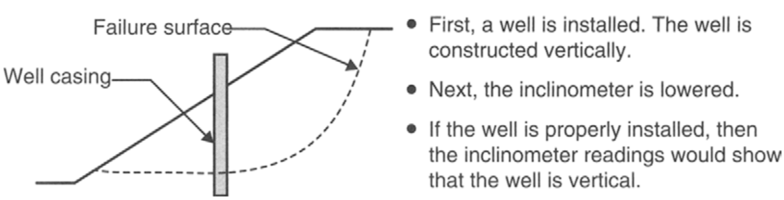

Figure 19.1 Inclinometer

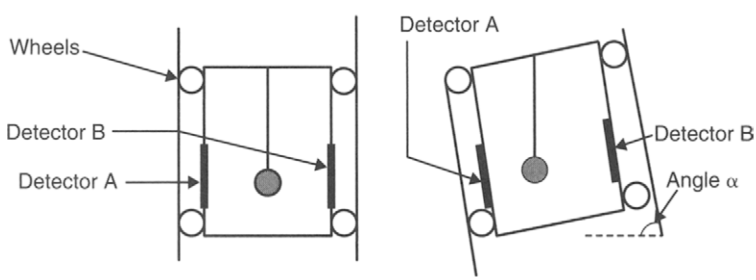

Figure 19.2 Working of the inclinometer

An inclinometer is shown in Fig. 19.2. The inclinometer is lowered into the well. The wheels are fixed at the side to guide through the well casing.

The pendulum has an electrical potential. When the inclinometer is vertical, both detectors (detector A and B) feel the same potential.

When the inclinometer is inclined as shown on the right in Fig. 19.2, by an angle of α, the pendulum is closer to detector A than to detector B. Hence detector A would record a higher reading than detector B. This electrical potential difference can be utilized to obtain the angle of inclination (or angle α).

19.1.1 Procedure

- The well is installed.
- Most inclinometers are approximately 2 ft long. The inclinometer is inserted into the well and a reading (angle φ) is taken from 0 ft to 2 ft. If the well casing is vertical, the reading should be 90°.
- Next, the inclinometer is inserted 2 ft more and another reading is taken. The process is continued to the bottom of the well. The obtained readings would look like those shown in Table 19.1.

Table 19.1 Inclinometer readings

Depth (ft)	0–2	2–4	4–6	6–8	8–10	10–12	12–14
Angle α	90.0	90.0	90.2	90.3	90.3	90.4	90.1
Depth (ft)	14–16	16–18	18–20	20–22	22–24	24–26	26–28
Angle α	90.5	90.3	90.5	90.1	90.2	90.7	90.3

- The next set of readings were taken a day or two later. If the soil had moved, the inclinometer would record different readings. A typical set of readings is shown in Table 19.2.

Table 19.2 Second set of inclinometer readings

Depth (ft)	0–2	2–4	4–6	6–8	8–10	10–12	12–14
Angle α	90.0	90.0	90.1	90.2	88.3	88.1	87.3
Depth (ft)	14–16	16–18	18–20	20–22	22–24	24–26	26–28
Angle α	87.2	89.1	88.5	88.3	89.1	89.0	89.2

Therefore, at a depth of 8 to 10 ft, the inclinometer has bent, since the angle shows a deviation from 90°. See Fig. 19.3.

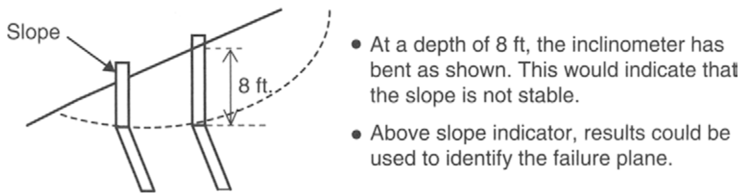

Figure 19.3 Measuring soil movement

It is possible to identify the slip circle by installing many inclinometers.

19.2 Tiltmeter

The operation of the tiltmeter is similar to a level. In a regular level, an air bubble is used to identify the tilt. If the air bubble is at the center, then the tilt is zero. Tiltmeters are filled with an electrically sensitive liquid instead of water. When the tiltmeter tilts, the bubble moves. The electrical signal coming from the tiltmeter is dependant upon the location of the bubble. Tiltmeters are used to measure the tilt in buildings, bridges, tunnels, and so on.

19.2.1 Procedure

- Tilt plates are attached to the building wall, as shown in Fig. 19.4.

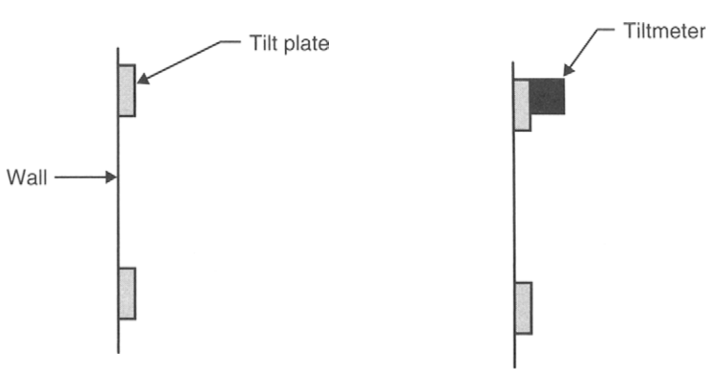

Figure 19.4 Tilt plates

- A tiltmeter is temporarily fixed to the plate. The tilt is measured. The tiltmeter is removed and then moved to measure the tilt in the next plate. One tiltmeter can be used to measure the tilt of many tilt plates. See Fig. 19.5.

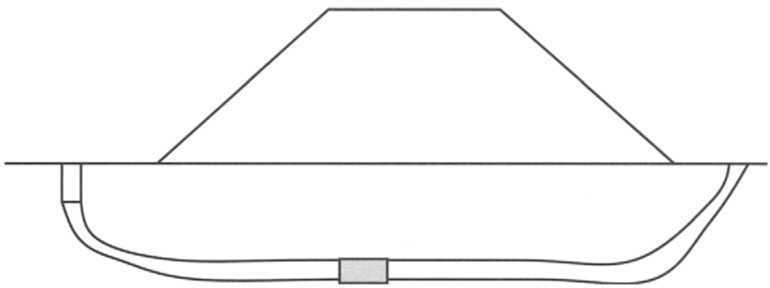

Figure 19.5 Horizontal tiltmeters

Horizontal tiltmeters can be installed to investigate soil movement due to foundations or embankments.

20

Unbraced Excavations

20.1 Introduction

Excavations less than 15 ft deep can be constructed without any supporting system under most soil conditions. Excavations constructed without a supporting system are known as unbraced excavations. The excavations need to be properly sloped in unbraced excavations.

20.1.1 Unbraced Excavations in Sandy Soils (Heights Less than 15 ft)

Design Example 20.1

Find the recommended permanent sloping angles for the soil types shown in Fig. 20.1.

Solution

Use Table 20.1 to obtain β. Table 20.1 does not recommend a sloping angle for such a situation. Bracing is needed.

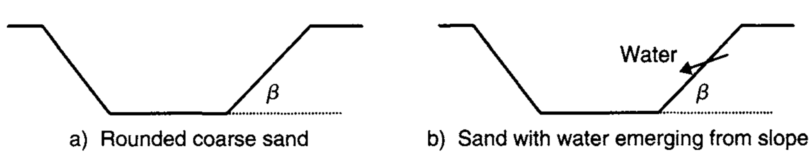

Figure 20.1 Unbraced excavations

Table 20.1 Sloping angles for unbraced excavations in sands

Soil type	Temporary slope	Permanent slope
	β(deg)	β(deg)
1. Gravel with boulders	53	
2. Sandy gravel	39	34
3. Angular sand	39	34
4. Rounded coarse sand	34	30
5. Rounded fine sand	30	27
6. Sand with water emerging from slope	22–16	

Source: Koerner (1984).

20.1.2 Unbraced Excavations in Cohesive Soils (Heights Less than 15 ft)

High plastic silts and clays are considered to be cohesive soils. See Table 20.2.

Table 20.2 Sloping angles for unbraced excavations in clay

Soil type	Plasticity index	Depth of excavation (ft)	β(deg)
Clayey silt	<10	0–10 ft	39
Clayey silt		10–15 ft	32
Silty clay	10–20	0–15 ft	39
Plastic clay	>20	0–10 ft	39

Source: Koerner (1984).

Design Example 20.2

Find the recommended sloping angles for the soil types shown in Fig. 20.2.

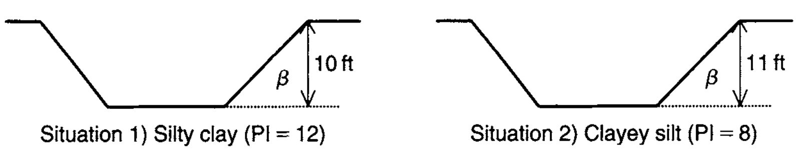

Figure 20.2 Design Example 20.1 illustration

Solution

Situation 1: Use Table 20.2 to obtain $\beta = 39°$.
Situation 2: Use Table 20.2 to obtain $\beta = 32°$.

Reference

Koerner, R. 1984. *Construction and geotechnical methods in foundation engineering.* New York: McGraw-Hill.

21

Raft Design

21.1 Introduction

Rafts, also known as mat foundations or floating foundations, are constructed in situations where shallow foundations are not feasible. The foundation engineer has the choice of picking either rafts or piles, depending on the situation. Most engineers usually prefer to select piles, not necessarily due to merit, but due to familiarity of piles compared to rafts.

21.2 Raft Design in Sandy Soils

Three prominent foundation engineers, Peck, Hanson, and Thorburn (1974), proposed the following method to design raft foundations.

The following equation can be used to find the allowable pressure in a raft.

$$q_{\text{allowable Raft}} = 0.22 \times N \times C_{\text{W}} + \gamma \times D_{\text{f}}$$

where

$q_{\text{allowable Raft}}$ = allowable average pressure in the raft, given in tsf

N = average SPT (N) value to a depth of $2B$, where B is the lesser dimension of the raft

$$C_{\text{W}} = 0.5 + 0.5D_{\text{W}}/(D_{\text{f}} + B)$$

where

D_W = depth to groundwater measured from the ground surface (ft)
D_f = depth to bottom of the raft measured from the ground surface (ft)
γ = total density of the soil (measured in tons per cubic feet)

Note that the units should match the units of $q_{\text{allowable Raft}}$ that are in tsf. Hence, the total density, γ, should be in tcf.

Design Example 21.1

Find the capacity of the raft shown in Fig. 21.1. The following information is given. The raft dimensions are 100 ft × 100 ft. The average SPT (N) value of the soil is 15. The total density of soil is 115 pcf.

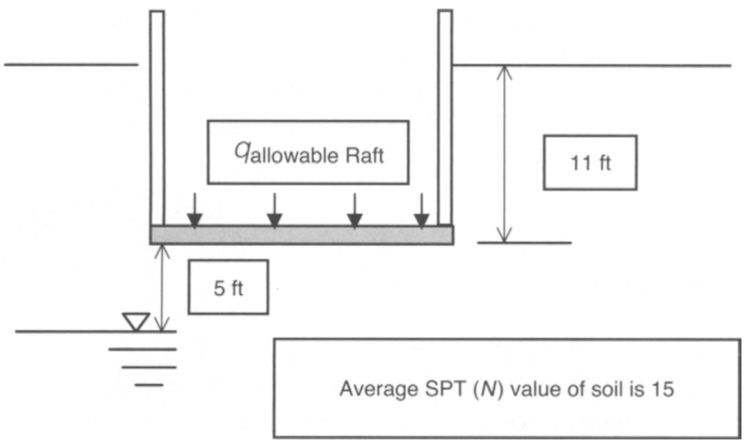

Figure 21.1 Design Example 21.1 illustration

Solution

STEP 1: Write down the equation for the allowable bearing capacity of the raft, $q_{\text{allowable Raft}}$, allowing a settlement of 2 in.

$$q_{\text{allowable Raft}} = 0.22 \times N \times C_W + \gamma \times D_f$$

STEP 2: Find the correction factor for groundwater, C_W.

$$C_W = 0.5 + 0.5D_W/(D_f + B)$$

where

D_W = depth to groundwater measured from the ground surface = 16 ft
D_f = depth to bottom of the footing measured from the ground surface = 11 ft
B = width of the raft = 100 ft

$$C_W = 0.5 + 0.5 \times 16/(11 + 100) = 0.572$$

STEP 3: Use the equation in step 1 to find $q_{\text{allowable Raft}}$.

$$q_{\text{allowable Raft}} = 0.22 \times N \times C_W + \gamma \times D_f$$

where

$N = 15$
$C_W = 0.572$
$\gamma = 115$ pcf = 0.0575 tcf (where γ needs to be converted to tcf since the equation is unit sensitive)
$D_f = 11$ ft

$$q_{\text{allowable Raft}} = 0.22 \times N \times C_W + \gamma \times D_f$$
$$q_{\text{allowable Raft}} = 0.22 \times 15 \times 0.572 + 0.0575 \times 11 = 2.52\ \text{tsf}$$

STEP 4: Calculate the total load that can be carried by the raft.
total load that can be carried by the raft = 2.52 × (100 ft × 100 ft) tons
= 25,200 tons

Reference

Peck, R. B., Hanson, W. E., and Thornburn, T. H. 1974. *Foundation engineering.* New York: John Wiley & Sons.

22

Rock Mechanics and Foundation Design in Rock

22.1 Introduction

Very high load foundations, caissons, and piles are carried down to the rock layer to increase the bearing capacity. Rock can provide a much higher bearing capacity than soil. Unweathered rock can have a bearing capacity in excess of 60 tsf. On the other hand, weathered rock or rocks subjected to chemical attack could be unsuitable for foundation use.

22.2 Brief Overview of Rocks

When a rock is heated to a very high temperature, it melts and becomes lava. Then, lava coming out from a volcano cools down and becomes rock. The earth's diameter is measured to be approximately 8,000 mi, and the bedrock is estimated to be only 10 mi deep. If the 10 mi thick bedrock is scaled down to 1 in., then the earth would be a 66 ft diameter sphere. The earth has a solid core with a diameter of 3,000 mi, and the rest is all lava, known as the mantle. See Fig. 22.1.

Occasionally, lava comes out of the earth during volcanic eruptions and cools down and becomes rock. This type of rock is known as igneous rock.

Some of the common igneous rocks are:

- Granite.
- Diabase.

- Basalt.
- Diorite.

See Fig. 22.2.

Volcanic eruptions are not the only process of rock origin. Soil particles constantly deposit on lake beds and on the ocean floor. Many million years later, these depositions solidify and convert into rock. Such rocks are known as sedimentary rocks. See Fig. 22.3.

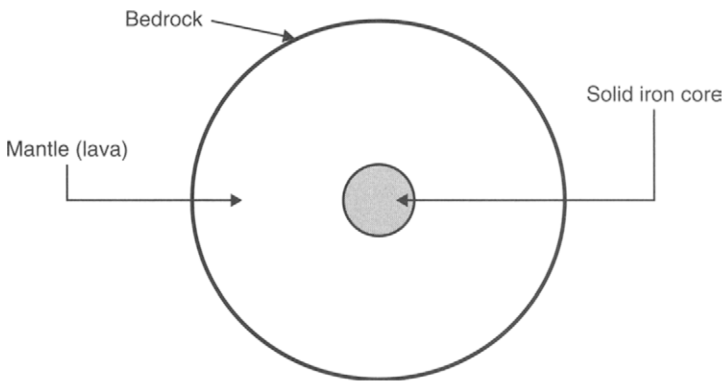

Figure 22.1 Earth

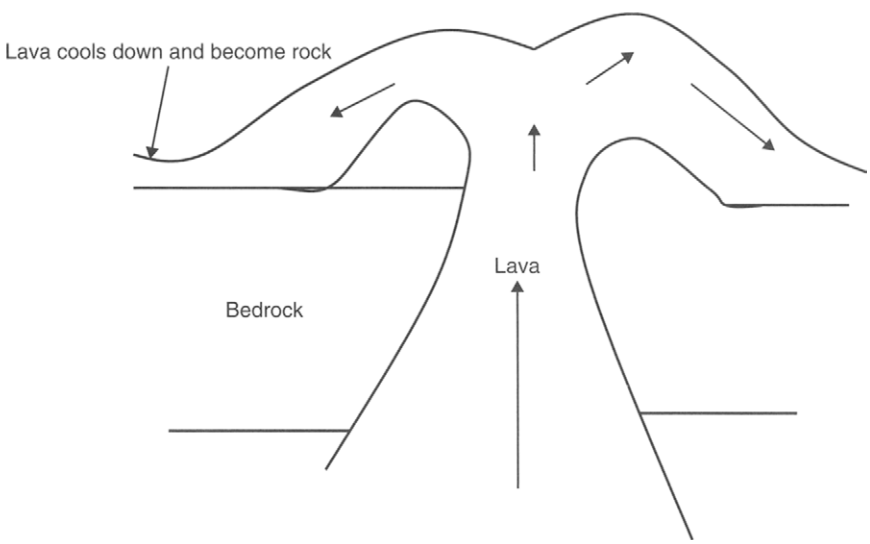

Figure 22.2 Volcanic eruptions and lava flow

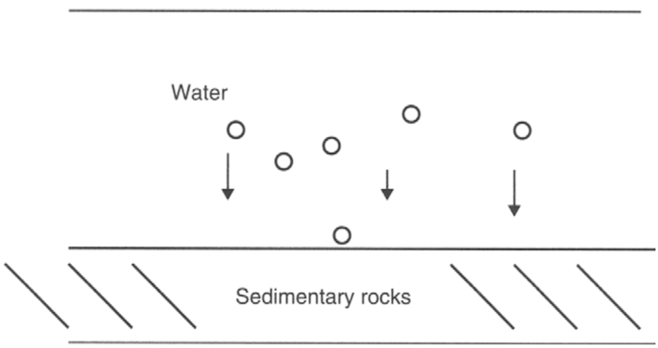

Figure 22.3 Sedimentary process

Some of the common sedimentary rocks are:

- Sandstone.
- Shale.
- Mudstone.
- Limestone.
- Chert.

Other than these two rock types, geologists have discovered another rock type. The third rock type is known as metamorphic rocks. When a tadpole becomes a completely different creature, a frog. We call this process *metamorphasis*. Similarly, either sedimentary rocks or igneous rocks could change and becomes a completely new type of rock, metamorphic rock.

How could a sedimentary rock or an igneous rock change into a new rock type? This happens when a rock is subjected to extreme pressure or temperature. Unthinkably large pressures occur in plate tectonic movements. Other events such as earthquakes, meteors, and volcanic eruptions also generate large pressures and temperatures. See Fig. 22.4.

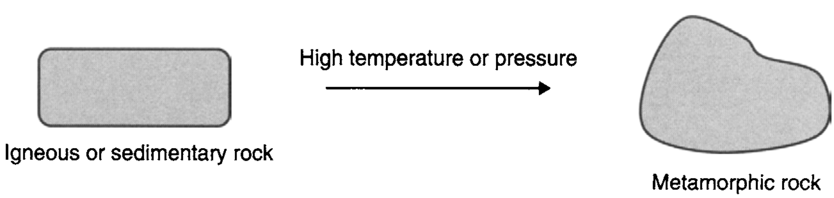

Figure 22.4 Formation of metamorphic rocks

Some of the common metamorphic rocks are:

- Gneiss.
- Schist.
- Marble.
- Slate.

Gneiss is usually formed when granite or similar igneous rock is subjected to heat or pressure. In many instances, shale is the parent rock of slate, and limestone is the parent rock of marble.

22.3 Rock Joints

The identification of rock type is important to the foundation or tunneling engineer. Yet in most situations, the suitability of a rock for foundation use is dependent upon rock joints. A rock joint is basically a fracture in the rock mass. Most rocks consist of joints. Joints could occur in the rock mass due to many reasons.

- **Earthquakes** Major earthquakes could shatter the bedrock and create joints.
- **Plate Tectonic Movements** Continents move relative to each other. When they collide, the bedrock folds and joints are created.
- **Volcanic Eruptions** Volcanic eruptions can cause earthquakes.
- **Excessive Heat** Generation of excessive heat in the rock can cause joints.

Depending on the location of the bedrock, the number of joints in a core run could vary. Some core runs contain few joints, while other core runs contain dozens of joints.

22.3.1 Joint Set

When a group of joints are parallel to each other, that group of joints is called a joint set. See Fig. 22.5.

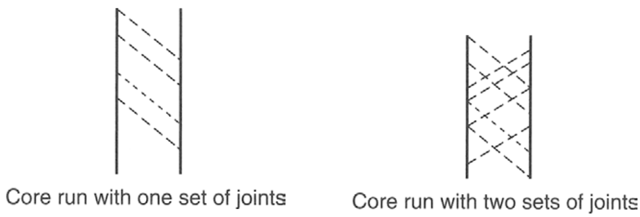

Figure 22.5 Joint sets

22.3.2 Foundations on Rock

The foundation shown in Fig. 22.6 can be considered to be unstable. If the foundation is already constructed, the engineer should consider the following factors that could reduce the friction in joints.

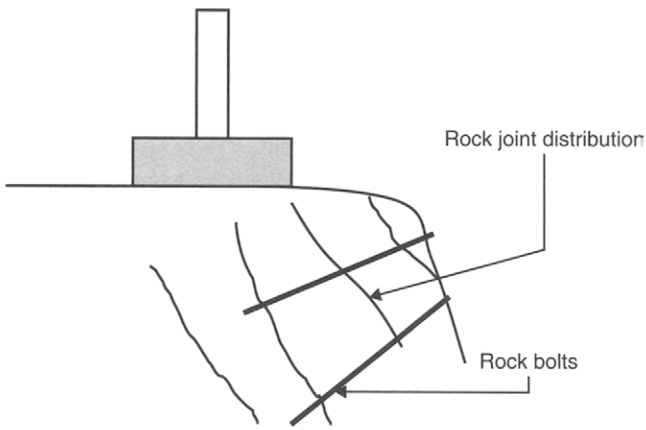

Figure 22.6 Foundation placed on rock with unstable joints

- **Joint Smoothness** If the joints are smooth, then the friction between joints can be considered to be minimal.
- **Water in Joints** If water is flowing through joints, the friction in the joints can be reduced.
- **Presence of Clay Particles in Joints** It is important to investigate the presence of clay particles in joints. Clay particles are deposited in joints due to groundwater movement. Clay particles in joints tend to reduce joint friction.

- **Earthquakes** If the building is located in an earthquake-prone region, the risk of failure will be high. In this case, rock bolts can be installed to stabilize the rock.

In Fig. 22.7, the rock joints are stable and rock bolts may not be necessary.

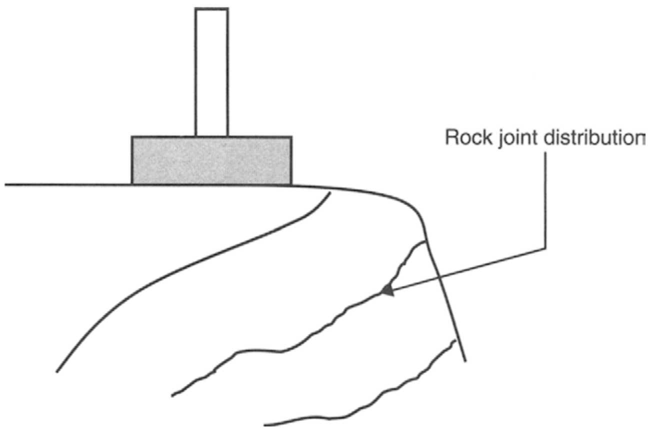

Figure 22.7 Foundation placed on rock with stable joints

22.4 Rock Coring and Logging

Information regarding rock formations is obtained through rock coring. Rock coring is a process whereby a rotating diamond cutter is pressed to the bedrock. Diamond, the hardest of all materials, cuts through the rock and a core is obtained. During the coring process, water is injected to keep the core bit cool, and to remove the cuttings.

A typical rock core is 5 ft in length. Rock cores are safely stored in core boxes made of wood.

See Figs. 22.8 and 22.9.

A typical core box can store four rock cores, each with a length of 5 ft. Core recovery is measured and noted for each rock core, in units of inches.

$$\text{recovery percentage} = \text{core recovery}/60 \times 100$$

For example, if the core recovery is 40 in., then the recovery percentage would be 66.7%.

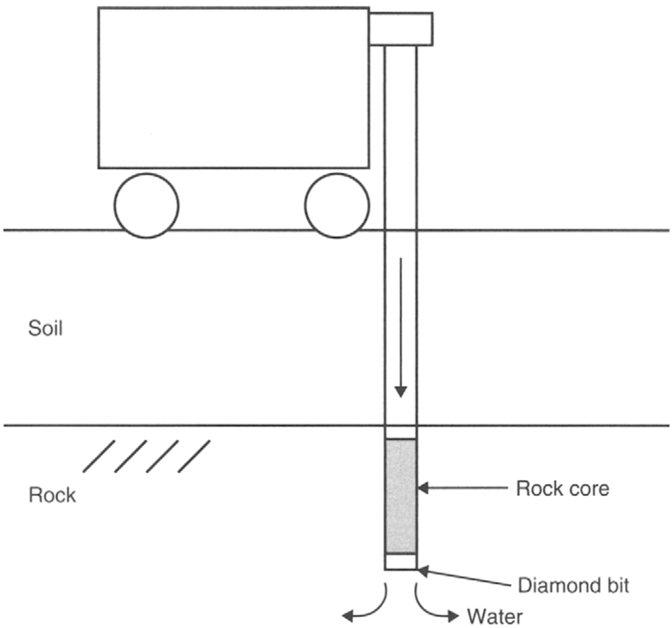

Figure 22.8 Rock coring

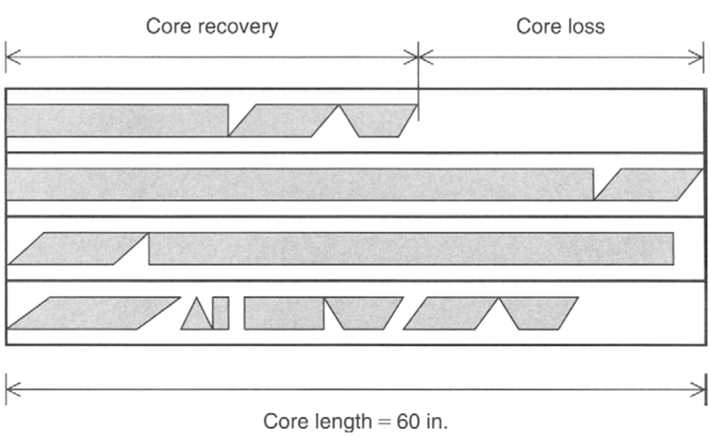

Figure 22.9 Core box

22.4.1 Rock Quality Designation (RQD)

Rock quality designation is obtained through the following process.

- Arrange all rock pieces as best as possible to simulate the ground conditions.
- Measure all rock pieces greater in length than 4 in.

RQD is given by

$$\text{RQD} = \text{total length of all rock pieces greater than 4 in.}/60 \times 100$$

Design Example 22.1

Lengths of rock pieces in a 60 in. rock core are measured to be 2, 3, 1, 4.5, 5.5, 2, 6, 8, 3, 2, 15.

Find the core recovery percentage and RQD.

Solution

STEP 1: Find the recovery percentage.

The total length of all the pieces is added to be 52 in.

$$\begin{aligned}\text{recovery percentage} &= 52/60 \times 100\\ &= 86.7\%\end{aligned}$$

STEP 2: Find all the pieces greater than 4 in.

4.5, 5.5, 6, 8, 15

$$\text{total length of pieces greater than 4 in.} = 39 \text{ in.}$$

$$\begin{aligned}\text{RQD} &= 39/60 \times 100\\ &= 65\%\end{aligned}$$

22.4.2 Joint Filler Materials

Some joints are filled with matter. This filler material can give the geotechnical engineer valuable information. Typical joint filler materials are

- **Sands** Sands occur in joints where there is a high energy flow of water (or high velocity).
- **Silts** Silts indicate that the flow is less energetic.
- **Clays** Clay inside joints indicates stagnant water in joints.

Joint filler material information would be very useful in interpreting Packer test data. It is known that filler material could clog up joints and reduce the flow rate with time.

22.4.3 Core Loss Information

It has been said by experts that core loss information is more important than rock core information. The core loss location may not be obvious in most cases. The coring rate, the color of the return water, and the arrangement of the core in the box can be used to identify the location of core loss. See Fig. 22.10.

Core loss occurs in weak rock or in highly weathered rock.

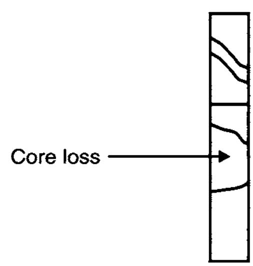

Figure 22.10 Core loss

22.4.4 Fractured Zones

A fracture log would be able to provide fractured zones. Fracture logs of each boring can be compared to check for joints.

22.4.5 Drill Water Return Information

The engineer should be able to assess the quantity of the returning drill water. Drill water return can vary from 90% to 0%. This information can be very valuable in determining weak rock strata. Typically, drill water return is high in sound rock.

22.4.6 Water Color

The color of the returning drill water can be used to identify the rock type.

22.4.7 Rock Joint Parameters

See Fig. 22.11 for rock joint parameter information.

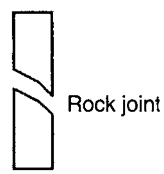

Figure 22.11 Rock joint

- **Joint Roughness** The joint surface can be rough or smooth. Smooth joints can be less stable than rough joints.
- **Joint Alteration** Note all alterations to the joint, such as color, filling of materials, and so on.
- **Joint Filler Material** Some joints are filled with sand, while other joints are filled with clay. Smooth joints filled with sand may provide additional friction compared to the smooth joint alone. However, rough joints filled with clay may reduce the friction in the joint compared to the rough joint if the clay were not present.
- **Joint Stains** Joint stains should be noted. Stains can be due to groundwater and various other chemicals.

22.4.8 Joint Types

As shown in Fig. 22.12, two types of joints are extensional joints and shear joints. Extensional joints are joints that formed due to tension or pulling apart.

Shear joints are joints that formed due to shearing.

Figure 22.12 Joint types

22.5 Rock Mass Classification

Who is the better athlete?

Athlete *A*: Long jump: 23 feet, runs 100 m in 11 s, high jump 6.5 feet
Athlete *B*: Long jump: 26 feet, runs 100 m in 13 s, high jump 6.4 feet

Athlete *A* is very weak at the long jump, but very strong in the 100 m dash. Athlete *B* is very good at the long jump, but not very good at running 100 m. Athlete *A* has a slight edge in high jump.

It is not easy to determine which athlete is better. Due to this reason, the International Olympic Committee came up with a marking system for the decathlon. The best athlete is selected based on the Olympic marking system.

A similar situation exists in rock types. Consider the following example.

Design Example 22.2

A geotechnical engineer has the choice to construct a tunnel in either rock type *A* or rock type *B*.

Rock type *A*: Average RQD = 60%, joints are smooth, joints are filled with clay
Rock type *B*: Average RQD = 50%, joints are rough, joints are filled with sand

Rock type *A* has a higher RQD value. On the other hand, rock type *B* has rougher joints. The smooth joints in rock type *A* are not favorable for geotechnical engineering work. The joints in rock type *A* are filled with clay, while the joints in rock type *B* are filled with sand. Based on this information, it is not easy to select a candidate, since both rock types have good properties and bad properties. Early engineers recognized the need for a classification system to determine the better rock type. Unfortunately, more than one classification system exists. These systems are called *Rock Mass Classification Systems*.

Popular Rock Mass Classification Systems are:

- Terzaghi Rock Mass Classification System (rarely used).
- Rock Structure Rating (RSR) method by Wickham (1972).
- Rock Mass Rating system (RMR) by Bieniawski (1976).
- Rock Tunneling Quality Index by Barton et al. (1972) (better known as the *Q* system).
- Recently, the *Q* system has gained popularity over the other systems.

22.6 Q system

$$Q = \mathrm{RQD}/J_n \times J_r/J_a \times J_w/\mathrm{SRF}$$

where

Q = rock quality index
RQD = rock quality designation
J_n = joint set number
J_r = joint roughness number
J_a = joint alteration number
J_w = joint water reduction factor
SRF = stress reduction factor

22.6.1 Rock Quality Designation (RQD)

To obtain the RQD, select all the rock pieces longer than 4 in. from the rock core. Measure the total length of all the individual rock pieces greater than 4 in. This length is given as a percentage of the total length of the core. Consider the case where the length of the core is 60 in. The total length of all the pieces longer than 4 in. is 20 in. In this case,

$$\text{RQD} = 20/60 = 0.333 = 33.3\%$$

where

RQD (0–25%) = very poor
RQD (25–50%) = poor
RQD (50–75%) = fair
RQD (75–90%) = good
RQD (90–100%) = excellent

22.6.2 Joint Set Number, J_n

One step in judging joints is to find the number of joint sets. When a group of joints have the same dip angle and a strike angle, that group is known as a joint set. In some cases, many joint sets exit. Assume there are eight joints in the rock core with following dip angles: 32°, 67°, 35°, 65°, 28°, 64°, 62°, 30°, and 31°. It is clear that there are at least two joint sets. One joint set has a dip angle at approximately 30°, while the other joint set has a dip angle of approximately 65°.

The Q system allocates the following numbers:

zero joints, $J_n = 1.0$
one joint set, $J_n = 2.0$
two joint sets, $J_n = 4.0$
three joint sets, $J_n = 9.0$
four joint sets, $J_n = 15.0$

A higher J_n number indicates a weaker rock for construction.

22.6.3 Joint Roughness Number, J_r

When subjected to stress, smoother joints will slip and failure will occur before failure in rougher joints. For this reason, joint roughness plays a part in rock stability. See Fig. 22.13.

Slippage occurs along a smoother joint at a lower load, P.

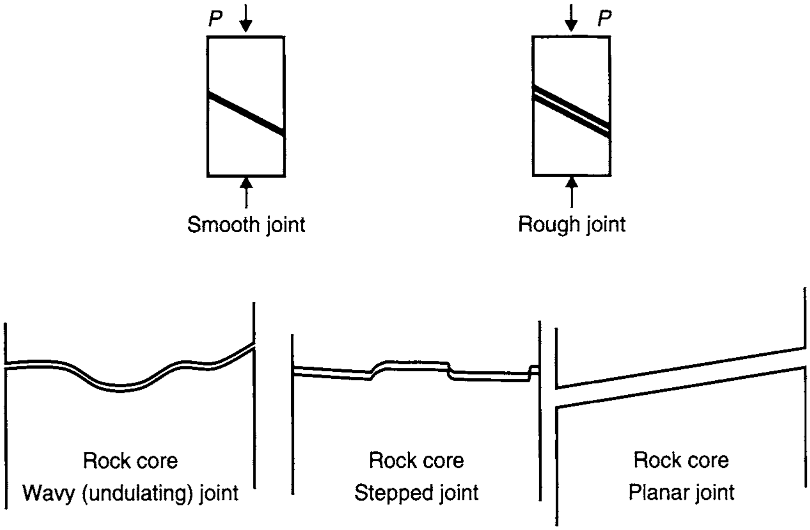

Figure 22.13 Smooth and rough joints

The following procedure explains how to obtain the joint roughness number.

STEP 1: There are three types of joint surface profiles.

- Wavy (undulating) joint surface profiles.
- Stepped joint surface profiles.
- Planar joint surface profiles.

No joint surface is either 100% planar, 100% stepped, or 100% wavy. Select the type that best describes the joint surface. See Fig. 22.13.

STEP 2: Feel the joint surface and categorize it into one of the types below.

- **Rough** If the surface feels rough.
- **Smooth** If the surface feels smooth.
- **Slickensided** Slickensided surfaces are very smooth and slick. Slickensided surfaces occur when there is a shear movement along

the joint. In some cases, the surface may be polished. In this case, polished (shining) patches can be observed. These polished patches indicate shear movement along the joint surface. The name slickensided was given to indicate slick surfaces.

STEP 3: Use Table 22.1 to obtain J_r.

Rock joints with stepped profiles provide the best resistance against shearing. From Table 22.1, a smooth joint with a stepped profile would be better than a rough joint with a planar profile.

Table 22.1 Joint roughness coefficient (J_r)

Joint profile	Joint roughness	J_r
Stepped		
	Rough	4
	Smooth	3
	Slickensided	2
Undulating		
	Rough	3
	Smooth	2
	Slickensided	1.5
Planar		
	Rough	1.5
	Smooth	1.0
	Slickensided	0.5

Source: Hoek, Kaiser, and Bawden (1995).

22.6.4 Joint Alteration Number, J_a

Joints are altered over time. Joints are altered from material filling inside them. In some cases, filler material can cement the joint tightly. In other cases, filler material can introduce a slippery surface, creating a much more unstable joint surface. See Table 22.2.

It is easy to recognize a tightly healed joint. In this case, use $J_a = 0.75$. If the joint has not undergone any alteration other than surface stains, use $J_a = 1.0$. If there are sandy particles in the joint, then use $J_a = 2.0$. If there is clay in the joint, then use $J_a = 3.0$. If clay in the joint can be considered to be low friction, then use $J_a = 4.0$. For this purpose, the clay types existing in joints need to be identified.

Table 22.2 Joint alteration number, J_a

Description of filler material	J_a
Tightly healed with a nonsoftening impermeable filling seen in joints (quartz or epidote)	0.75
Unaltered joint walls; no filler material seen (surface stains only)	1.0
Slightly altered joint walls; nonsoftening mineral coatings are formed; sandy particles, clay, or disintegrated rock seen in the joint	2.0
Silty or sandy clay coatings, small fraction of clay in the joint	3.0
Low friction clay in the joint (e.g., kaolinite, talc, and chlorite are low friction clays)	4.0

Source: Hoek, Kaiser, and Bawden (1995).

22.6.5 Joint Water Reduction Factor, J_w

The joint water reduction factor, J_w is a measure of the water in a joint. The J_w factor cannot be obtained from boring data. A tunnel in the rock needs to be constructed to obtain J_w. Usually, data from previous tunnels constructed in the same formation is used to obtain J_w. Another option is to construct a pilot tunnel ahead of the real tunnel.

See Table 22.3.

22.6.6 Defining the Stress Reduction Factor (SRF)

The SRF (stress reduction factor) is an indication of weak zones in a rock formation. The SRF cannot be obtained from boring data. A tunnel in the rock needs to be constructed to obtain the SRF as in the case of J_w. Usually, data from previous tunnels constructed in the same formation is used to obtain SRF. Another option is to construct a pilot tunnel ahead of the real tunnel.

All rock formations have weak zones. A weak zone is a region in the rock formation that has a low RQD value. Weak zones may have weathered rock or a different rock type.

22.6.7 Obtaining the Stress Reduction Factor (SRF)

The following guidelines are provided to obtain the stress reduction factor.

1. More than one weak zone occurs in the tunnel. In this case use SRF = 10.0.

Table 22.3 Joint water reduction factor, J_W

Description	Approximate water pressure (kg/cm2)	J_W
Excavation (or the tunnel) is dry; no or slight water flow into the tunnel	Less than 1.0	1.0
Water flows into the tunnel at a medium rate; water pressure 1.0–2.5 kgf/cm^2; joint fillings get washed out occasionally due to water flow	−2.5	0.66
Large inflow of water into the tunnel or excavation; the rock is competent and joints are unfilled (water pressure 1.0–2.5 kgf/cm^2)	2.5–10.0	0.5
Large inflow of water into the tunnel or excavation, the joint filler material gets washed away (water pressure 2.5–10 kgf/cm^2)	Greater than 10	0.33
Exceptionally high inflow of water into the tunnel or excavation (water pressure >10 kgf/cm^2)	Greater than 10	0.1 to 0.2

Source: Hoek, Kaiser, and Bawden (1995).

2. There is a single weak zone of rock with clay or chemically disintegrated rock (excavation depth <150 ft). Use SRF = 5.0.
3. There is a single weak zone of rock with clay or chemically disintegrated rock (excavation depth >150 ft). Use SRF = 2.5.
4. There is more than one weak zone of rock without clay or chemically disintegrated rock. Use SRF = 7.5.
5. There is a single weak zone of rock without clay or chemically disintegrated rock (excavation depth <150 ft). Use SRF = 5.0.
6. There is a single weak zone of rock without clay or chemically disintegrated rock (excavation depth >150 ft). Use SRF = 2.5.
7. Loose open joints are observed. Use SRF = 5.0.

Design Example 22.3

The average RQD of a rock formation was found to be 60%. Two sets of joints have been identified. Most joint surfaces are undulated (wavy) and rough. Most joints are filled with silts and sands. It has been reported that a medium inflow of water has occurred during the construction of past tunnels. During earlier construction, a single weak zone containing clay was observed at a depth of 100 ft. Find the Q value.

Solution

$$Q = \mathrm{RQD}/J_n \times J_r/J_a \times J_w/\mathrm{SRF}$$

The known values are as follows.

RQD = 60%. Since there are two sets of joints, $J_n = 4$. From Table 22.1, for undulating, rough joints, $J_r = 3$. From Table 22.2, since most joints are filled with silts and sands, $J_a = 2$. From Table 22.3, $J_w = 0.66$. From the text above, SRF = 5. Hence

$$Q = (60/4) \times (3/2) \times (0.66/5) = 2.97$$

References

Barton, N., et al., "Norwegian method of tunnelling", *World Tunnelling*, June 1972.

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Hoek, E., Kaiser, P. K., and Bawden, W. F. 1995. *Support of underground excavations in hard rock*. Rotterdam, The Netherlands: Balkema.

Wickham, S., et al., "Support determination based on geologic predictions", Proceedings of Rapid Excavation and Tunneling Conference, AIMW, NY 1972.

23

Dip Angle and Strike

23.1 Introduction

The dip angle is the angle that a set of joints extends against the horizontal plane. Each set of joints has a dip angle and a strike. The strike is the direction that the horizontal plane and dip direction intersect (see Fig. 23.1).

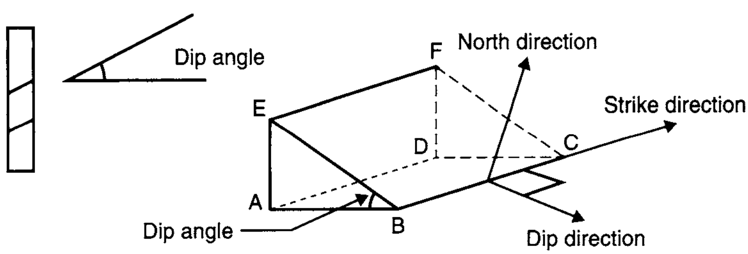

Figure 23.1 Dip angle and strike

- **Joint Plane** *EBCF* is the joint plane in Fig. 23.1
- **Dip Angle** The angle between the joint plane (*EBCF*) and the horizontal plane (*ABCD*). The dip angle is easily measured in the field.
- **Strike** The strike line is the horizontal line *BC* as shown in Fig. 23.1.
- **Strike Direction** The right hand rule is used to obtain the strike direction.

1. Open your right hand and face the palm down. Mentally lay the palm on the joint plane.
2. Point four fingers (except the thumb) along the downward direction of the slope.
3. The direction of the thumb indicates the strike direction.
4. The clockwise angle between the strike direction and the north direction is called the strike angle. The *ABCD* plane and north direction are in the same horizontal plane.

23.1.1 Dip Direction

The dip direction is the direction of the downward slope. The dip direction is different from the dip angle. Strike and dip directions are perpendicular to each other.

The dip angle and strike angle are written as shown below.

35/100

The first value above indicates the dip angle (not the dip direction). The second value indicates the strike direction measured clockwise from the north. The dip angle is always written with two digits, while the strike direction is always written with three digits. A joint plane with a dip angle of 40° and a 70° strike angle is written as 40°/070°.

23.2 Oriented Rock Coring

23.2.1 Oriented Coring Procedure

STEP 1: A knife is installed in the core barrel to create a mark in the rock core. This mark is known as the scribe mark.

STEP 2: The knife is installed in such a manner that a line drawn through the scribe mark and the center of the core points toward the north direction (line *AB*). The driller uses a magnetic compass prior to coring to locate north. Then the driller locates the knife. See Fig. 23.2.

STEP 3: Draw a horizontal line going through point C, at the joint (line *CD*). Both lines (line *AN* and line *CD*) are in horizontal planes.

STEP 4: Measure the clockwise angle between line *AN* (which is the north direction) and line *CD*. This angle is the strike angle of the joint.

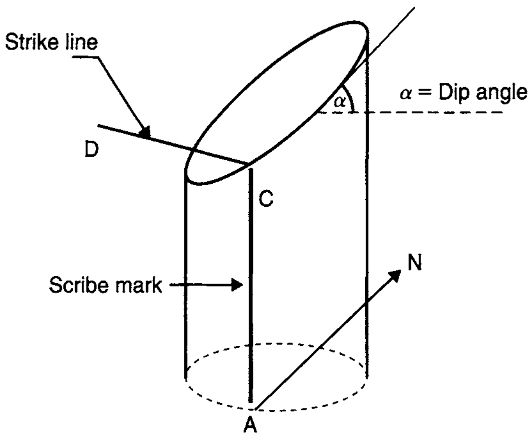

Figure 23.2 Oriented coring procedure

23.2.2 Oriented Coring Procedure (Summary)

- The driller finds the north direction using a compass. Then driller places the knife along the north direction. The line connecting the knifepoint (point A) and the center of the core will be the north-south line (line *AN*).
- Rock coring is conducted. A scribe mark will be created along the rock core. Note that the rock core does not rotate during coring.
- After the rock core is removed from the hole, a horizontal line is drawn along the plane of the joint (line *CD*). The clockwise angle between line *AN* and line *CD* is the strike angle.

23.3 Oriented Core Data

Figure 23.3 shows how to construct an oriented core.

- Oriented coring produces a dip angle and a strike angle for each joint.
- A joint can be represented by one point known as the pole.

A pole is constructed by the following method.

STEP 1: The joint plane is drawn across the sphere *A*.
STEP 2: A perpendicular line is drawn (line *B*) to the joint plane.

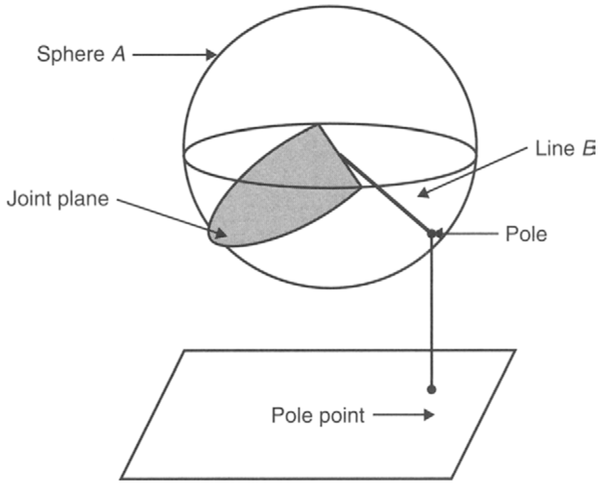

Figure 23.3 Illustration of pole and pole point

STEP 3: The point where line B intersects the sphere is known as the pole.
STEP 4: A vertical line is dropped from the pole to obtain the pole point.

Theoretically, there are two poles for any given joint, one in the lower hemisphere and one in the upper hemisphere. The pole in the lower hemisphere is always selected.

As you can see, it is not easy to obtain the pole point for a given joint. For that purpose, one needs a sphere and has to go through a lot of trouble. There are charts available to obtain the pole point for any given joint.

24

Rock Bolts, Dowels, and Cable Bolts

24.1 Introduction

Nomenclature changes from area to area. Generally, rock bolts (also known as rock anchors) are either nonstressed or prestressed. Nonstressed anchors are also known as rock dowels. When there is a slight movement in the rock mass, rock dowels get tensioned and provide a resisting force.

Cable bolts are a group of wires used to create a cable. Cable bolts are used for high capacity applications.

Rock anchors are used for shallow foundations, retaining walls, bridges, and tunnels.

The four basic types of rock anchors are

1. Mechanical rock anchors or rock bolts (either nonstressed or prestressed).
2. Grouted (either nonstressed or prestressed).
3. Resin anchors.
4. Nonstressed anchors, known as rock dowels.

24.1.1 Applications

See Figs. 24.1 and 24.2.

Rock anchors are widely used in retaining walls when rock is close enough for the anchors to be used. See Fig. 24.3.

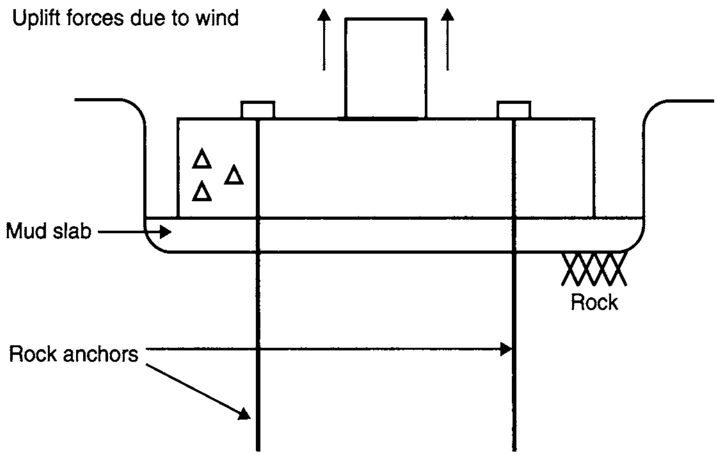

Figure 24.1 Rock anchors to resist uplift forces in a shallow foundation

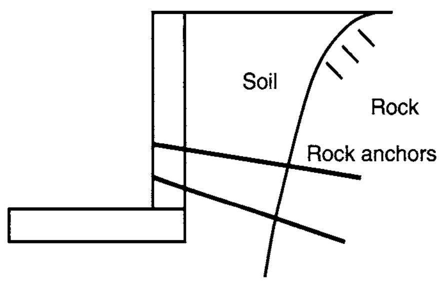

Figure 24.2 Rock anchors in a retaining wall

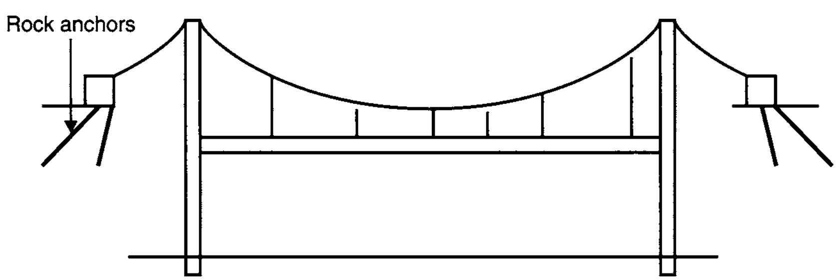

Figure 24.3 Rock anchors in a bridge

24.2 Mechanical Rock Anchors

The most popular mechanical anchor system is expansion shell anchors. To use expansion shell anchors, a hole is drilled in the rock, and the rock bolt assembly is inserted. Then a tensile force, *P*, is applied. When the force is applied, the anchor tries to move to the right. See Fig. 24.4.

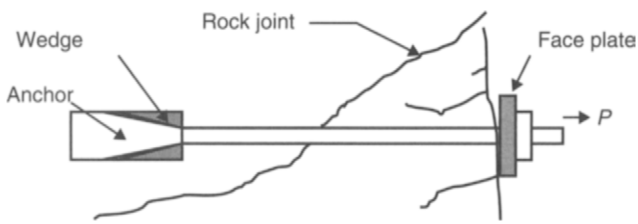

Figure 24.4 Mechanically anchored rock anchors

The movement of the anchor expands the two wedges and locks the bolt into the rock. A typical installation procedure for mechanical anchors is as follows.

STEP 1: Drill a hole in the rock.
STEP 2: Insert the mechanical anchor.
STEP 3: Activate the mechanical wedge assembly at the end to attach the anchor to the rock. Rock anchors have a wedge assembly at the end that expands when rotated.
STEP 4: Grout the hole to avoid corrosion in the anchor.

Hoek et al. (1995) recommend that the initial tension applied be 70% of the total capacity of the rock bolt. See Figs. 24.5, 24.6, and 24.7.

24.2.1 Mechanical Anchor Failure

The main reason for rock bolt failure is corrosion. The corrosion of rock bolts can be avoided by grouting the hole. Grouting of the hole is very important when there is water in the rock. Most permanent rock bolts are grouted. Grouting may not be necessary for temporary rock bolts. It should be mentioned that groundwater in some regions can be acidic and can accelerate the corrosion process.

24.2.2 Design of Mechanical Anchors

The failure surface of mechanical anchors is a cone. See Fig. 24.8. When a mechanical anchor is subjected to failure, a failure cone develops. The forces acting against cone failure are the weight of the cone and rock cohesion along the cone surface.

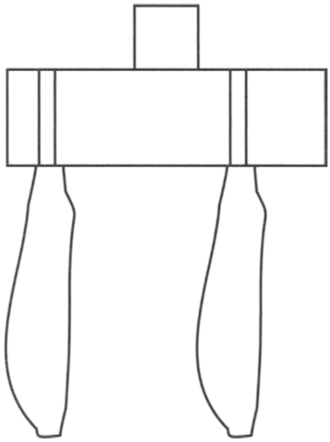

Figure 24.5 Step 1: Drill a hole

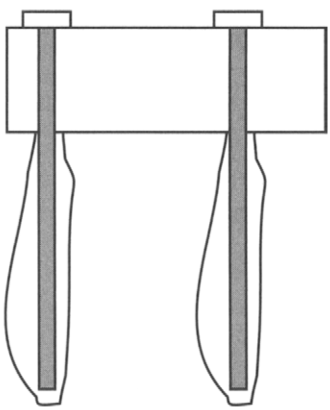

Figure 24.6 Step 2: Insert anchors

resistance to failure = weight of the cone + cohesion along the surface of the cone

The surface area and volume of cones are given by following equations.

$$\text{volume of a cone} = 1/3 \times (\pi \times r^2) \times D$$

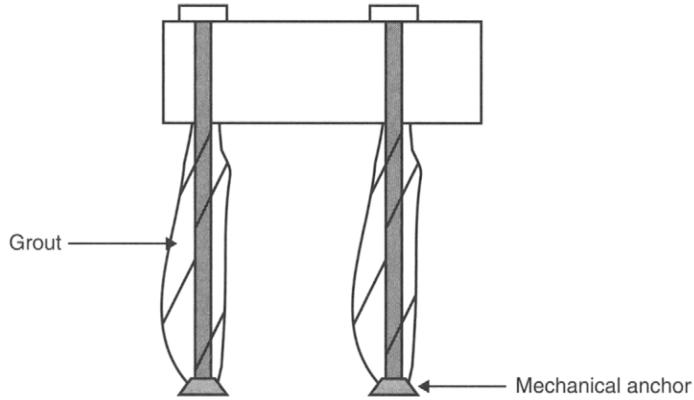

Figure 24.7 Mechanical anchor installation

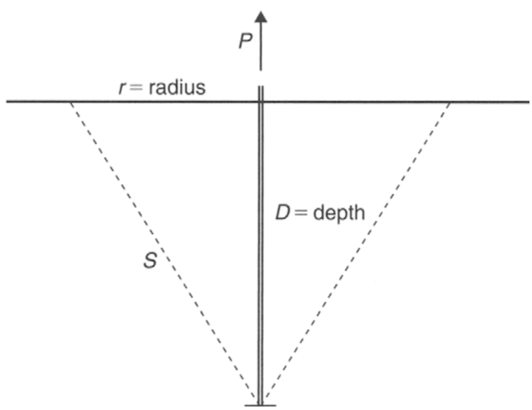

Figure 24.8 Failure cone of rock anchors

$$\text{surface area of a cone} = (\pi \times r) \times S$$

$$S = (r^2 + D^2)^{1/2}$$

The weight of the cone can be found if the density of the rock is known.

Design Example 24.1

Find the failure force of the mechanical anchor shown in Fig. 24.9. The following information is obtained. Assume the angle of the stress triangle to be 30°. The length of the rock anchor is 2.5 m. Ignore the weight of the rock. Assume that the rock cohesion is 300 kPa. Assume that the rock density is 24 kN/m^3.

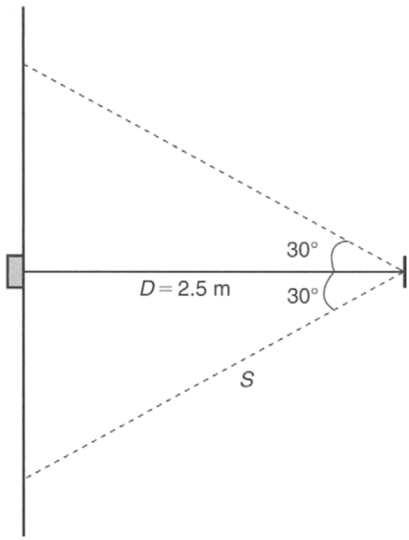

Figure 24.9 Failure mechanism of the rock anchor

Solution

$$\text{radius} = r = 2.5 \times \tan 30°$$

$$r = 1.44$$

$$S^2 = r^2 + D^2 = 1.44^2 + 2.5^2$$

$$S = 2.88 \text{ m}$$

$$\text{surface area of the cone} = (\pi \times r) \times S$$

$$= (\pi \times 1.44) \times 2.88 = 13.01 \text{ m}^2$$

$$\text{cohesive force} = 13.01 \times \text{rock cohesion}$$

$$= 13.01 \times 300$$

$$= 3{,}900 \text{ kN (436 tons)}$$

24.2.3 Grouting Methodology for Mechanical Rock Anchors

Both standard and hollow rock bolts are anchored to the rock using a cone and wedge mechanism. The bolt tension is held by the anchor. Hence, the primary purpose of grouting is to provide corrosion protection. For this reason, the strength of grout is not a factor in mechanically anchored rock bolts. The other purpose of grouting is to hold the rock bolt in place so that any ground vibrations will not cause the mechanical anchor to loosen.

Mechanically anchored rock bolts can be either grouted or ungrouted. For most temporary applications, rock bolts are not grouted. To make the grouting process easier, some rock bolt manufacturers have produced rock bolts with a central hole. The hole in the middle is used for the grouting process.

24.2.4 Tube Method

Tubes can be inserted into the hole to grout the rock bolt, as shown in Fig. 24.10. One tube is inserted to pump in the grout. This tube is known as the grout injection tube. Another tube is used to remove trapped air. This tube is known as the breather tube.

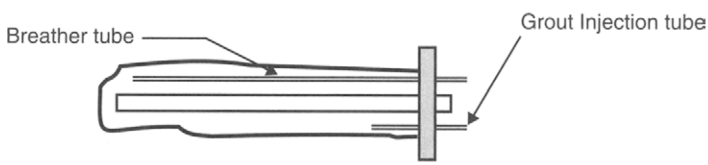

Figure 24.10 Grout injection tube and breather tube system; note that the wedge and cone mechanism is not shown

In theory, grout is injected through one tube and entrapped air is removed through the other tube. In practice, this mechanism can create problems. Often, the tubes get entangled. Another complication is that two holes need to be drilled through the face plate. To avoid these problems, hollow rock bolts are used.

24.2.5 Hollow Rock Bolts

Hollow rock bolts were designed to make the grouting process more reliable. In hollow rock bolts, there is a hole in the center of the rock bolt. This hole is used for the grouting process and entangling tubes are eliminated. Hollow rock bolts may have the same cone and wedge anchor mechanism. See Fig. 24.11.

When the rock bolt is downward facing, grout is inserted through the hole at the center and a short tube is inserted outside of the bolt to remove trapped air.

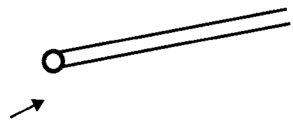

Figure 24.11 Hollow rock bolt

When the rock bolt is upward facing, a short tube (inserted outside of the bolt) is used for the grouting process and the central hole is used as a breathing tube.

24.3 Resin Anchored Rock Bolts

One major disadvantage of mechanically anchored rock bolts is that the mechanical anchor can loosen when there are vibrations in the rock due to blasting, train movement, traffic movement, or construction activities. Further, it is difficult to obtain a good mechanical anchor in weak and weathered rock. Shale, mudstone, and highly weathered low RQD rocks are not good candidates for mechanically anchored rock bolts. These difficulties could be avoided by using resin anchored rock bolts. See Fig. 24.12.

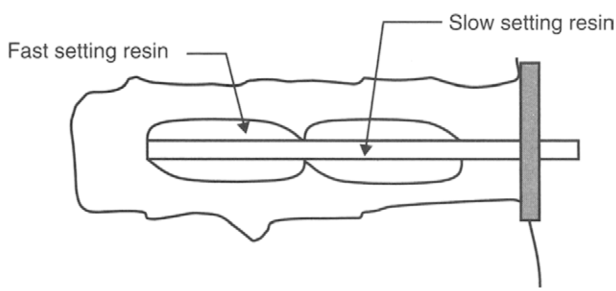

Figure 24.12 Resin anchored rock bolts

Two resin cartridges are inserted as shown in Fig. 24.12. When the resin cartridges are broken (by spinning the rock bolt), the fast-setting resin solidifies first. The fast-setting resin anchors the rock bolt into the rock. The tensile force is applied at this point, and the slow-setting resin has not yet solidified. The slow-setting portion is still in a liquid state. When tension is applied, the rod is free to extend within the slow-setting resin area. After the tension is applied and locked in, the slow-setting resin solidifies. For temporary applications, slow-setting resin may not be necessary. The purpose of slow-setting resin is to provide corrosion protection.

24.3.1 Disadvantages

- In fractured rock, the resin can seep into the rock and leave a little resin inside the hole.
- Some rocks contain clay seams. Resins may not be able to provide a good anchor in this type of rock.
- Some researchers have expressed concern regarding the long-term corrosion protection ability of resins. Further groundwater chemicals may react with the resin and compromise the resin's effectiveness.

24.3.2 Advantages

- Resins work well in fractured rocks.
- Installation is very fast compared to other methods.

24.4 Rock Dowels

Unlike prestressed rock bolts, rock dowels are not tensioned. Hence, rock dowels do not apply a positive force to the rock. Rock dowels are called into action when there is a movement in the rock. The simplest form of rock dowel is a grouted steel bar inserted into the rock. Together, these are known as grouted dowels. Other types include split set stabilizers and swellex dowels.

The three main types of rock dowels are

1. Cement grouted dowels.
2. Split set stabilizers.
3. Swellex dowels.

24.4.1 Cement Grouted Dowels

See Fig. 24.13.

Note that if the rock moves along the joint, the dowel becomes tensioned.

24.4.2 Split Set Stabilizers

These rock dowels are different from grouted dowels. What happens when a steel rod is pushed into a small hole (with a smaller diameter

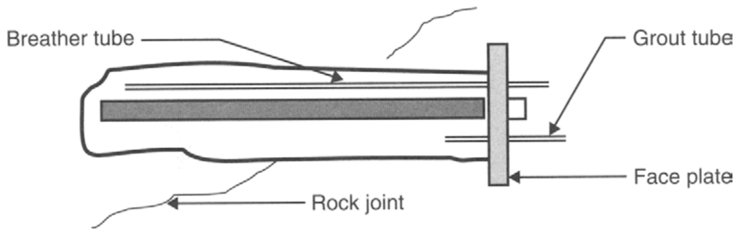

Figure 24.13 Cement grouted dowels

than the steel bar) that has been drilled into the rock? The steel bar is compressed and will be tightly entrapped inside the hole. Split set stabilizers are designed based on this principle. See Fig. 24.14.

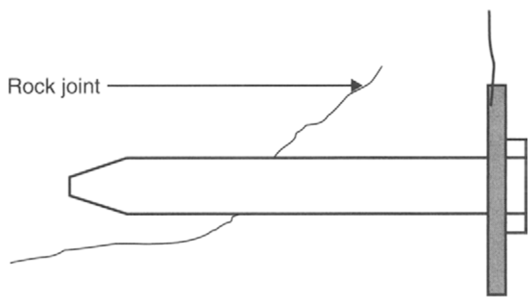

Figure 24.14 Split set stabilizers

Note that a hole smaller than the split set dowel is drilled. Then the dowel is pushed into the rock. A split set stabilizer with a diameter of 33 mm can resist a force of 10.9 tons. The recommended drill hole size is 31 mm. (Note that the hole diameter is smaller than the dowel.)

24.4.3 Advantages and Disadvantages

These dowels are very quick and easy to install, since there is no grouting involved. The main disadvantage is corrosion. The cost factor also may play a part in their selection.

24.4.4 Swellex Dowels

Swellex dowels have a central hole. After installation into rock, a high pressure water jet is sent in.

Note that the hole is drilled in the rock, then the swellex dowel is inserted. Next, a high pressure water jet is sent in. The dowel expands and hugs the rock, and this causes it to stay in place.

Corrosion is a major problem for swellex dowels.

24.5 Grouted Rock Anchors

Grouted rock anchors are nonstressed anchors. In the case of grouted rock anchors, the bonding between the rock and the anchor is achieved by grout. See Fig. 24.15.

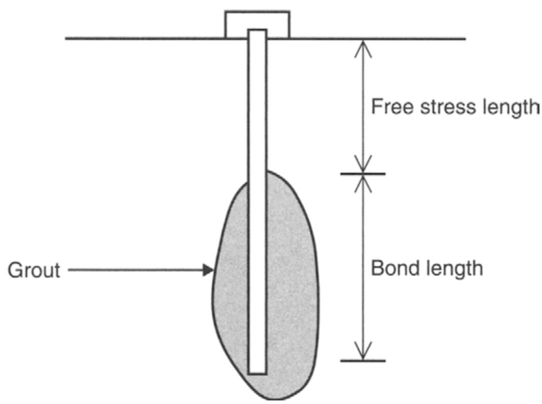

Figure 24.15 Grouted rock anchor

The anchor's strength derives from the bond strength between the rock and the grout.

The installation procedure of grouted anchors is as follows.

STEP 1: Drill a hole to the desired length.
STEP 2: Install the rock anchor.
STEP 3: Grout the hole.
STEP 4: Wait sufficient time for the grout to harden before applying the load.

24.5.1 Failure Triangle for Grouted Rock Anchors

Two anchors of same length are shown in Fig. 24.16. The stress development cones in grouted anchors are smaller than the comparable length for mechanical anchors, since grouted anchors need a bond length. See Table 24.1.

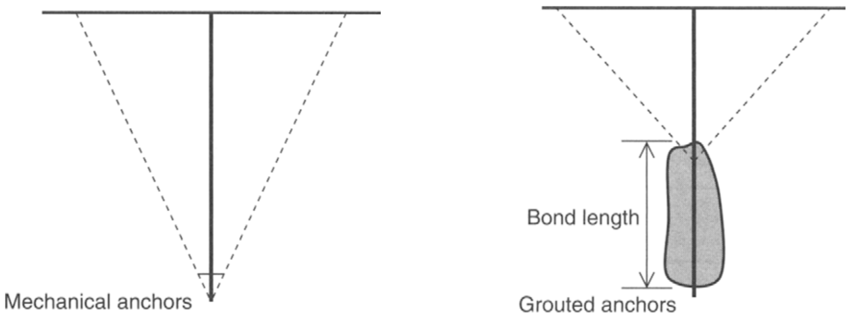

Figure 24.16 Stress cones for mechanical anchors and grouted anchors with similar length

Table 24.1 Rock-grout bond strength

Rock type	Rock-grout bond strength (MPa)	Rock-grout bond strength (psi)
Granite	0.55–1.00	80–150
Dolomite/limestone	0.45–0.70	70–100
Soft limestone	0.35–0.50	50–70
Slates, strong shales	0.30–0.45	40–70
Weak shales	0.05–0.30	10–40
Sandstones	0.3–0.60	40–80
Concrete	0.45–0.90	70–130
Weak rock	0.35–0.70	50–100
Medium rock	0.70–1.05	100–150
Strong rock	1.05–1.40	150–200

Source: Wiley (1999).

24.6 Prestressed Grouted Rock Anchors

Most rock anchors installed today are prestressed. Prestressed anchors are installed and subjected to a stress.

The following is the prestressing procedure.

STEP 1: Augur the hole.
STEP 2: Place the anchor.
STEP 3: Grout the desired length.
STEP 4: Wait until the grout is set.

STEP 5: Stress the rock anchor to the required stress.
STEP 6: Lock in the stress with a bolt.

See Figs. 24.17, 24.18, and 24.19.

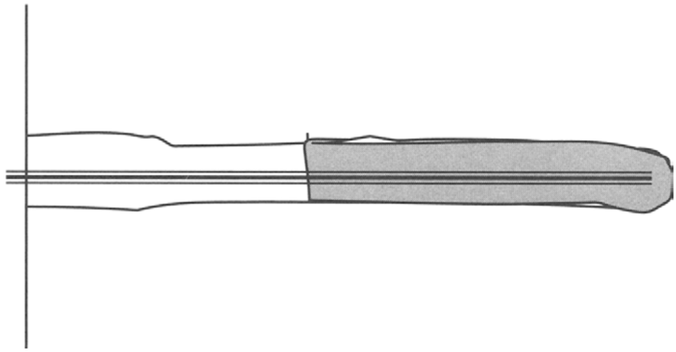

Figure 24.17 Augur a hole, place the anchor and grout

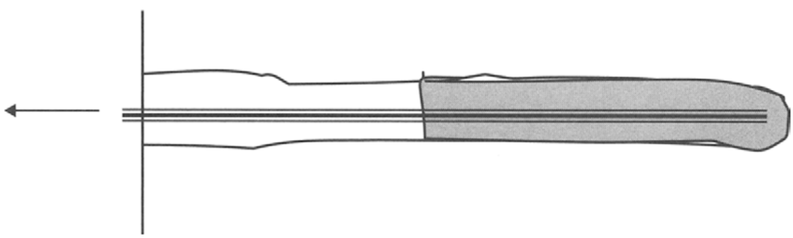

Figure 24.18 Wait till the grout is set and apply the stress using a jacking mechanism

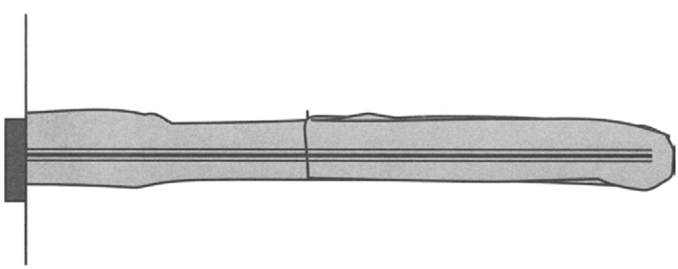

Figure 24.19 Lock in the stress and grout the rest of the anchor

Prestressed anchors have a number of advantages over nonstressed anchors.

24.6.1 Advantages of Prestressed Anchors

- Prestressed anchors will not move due to changing loads. In contrast, nonstressed anchors can move, and the grout-anchor bond can be broken. See Fig. 24.20.

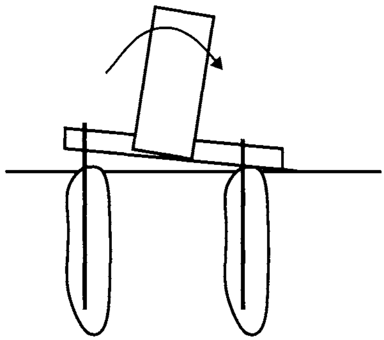

Figure 24.20 Nonstressed anchor lifted due to wind load

- The working load in a prestressed anchor is less than the applied prestress. In the case of nonstressed anchors, the load varies. This could give rise to fatigue in the steel and finally failure can occur.

24.6.2 Anchor-Grout Bond Load in Nonstressed Anchors

This section explores the load in the nonstressed anchor in Fig. 24.21.

1. The outside load in the nonstressed anchor is 10 kN. The bond load will be equal to 10 kN.
2. The outside load is increased to 20 kN. The bond load will change to 20 kN. The anchor-grout bond load changes with the applied load in nonstressed anchors. This creates elongation in the anchor and damages the grout.

Due to the change in load in the anchor rod, the rod will move with the changing load.

24.6.3 Anchor-Grout Bond Load in Prestressed Anchors

This section explores the load in the prestressed anchor in Fig. 24.22.

1. The anchor is prestressed to 20 kN and locked. No outside load is applied. The bond load is 20 kN.

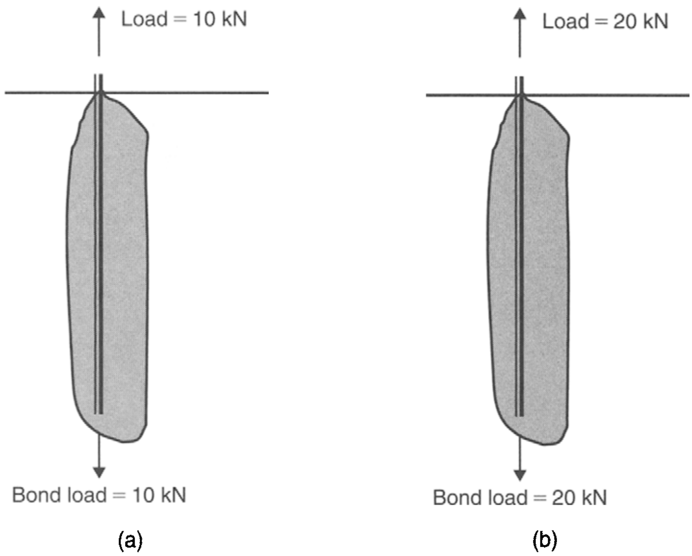

Figure 24.21 Load on nonstressed anchors

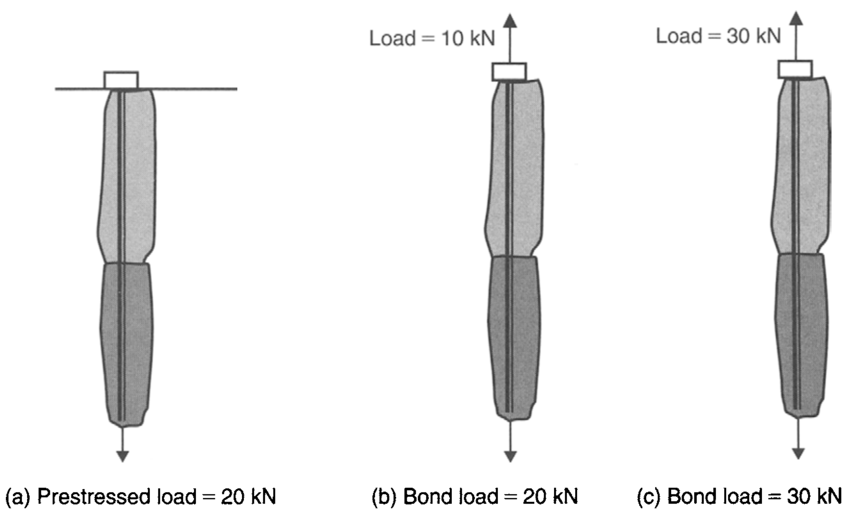

Figure 24.22 Load on prestressed anchors

2. A 10 kN outside load is applied. The bond load will remain at 20 kN. The rod will not move.
3. The outside load is increased to 30 kN. The bond load will now change to 30 kN, and the rod will move against the grout.

There will not be any elongation in the rod until the outside load has surpassed the prestressed load.

Design Example 24.2

Find the bond length required for the rock anchor in the retaining wall in Fig. 24.23. The rock grout bond strength was found to be 1.1 MPa and the grout hole is 0.3 m in diameter.

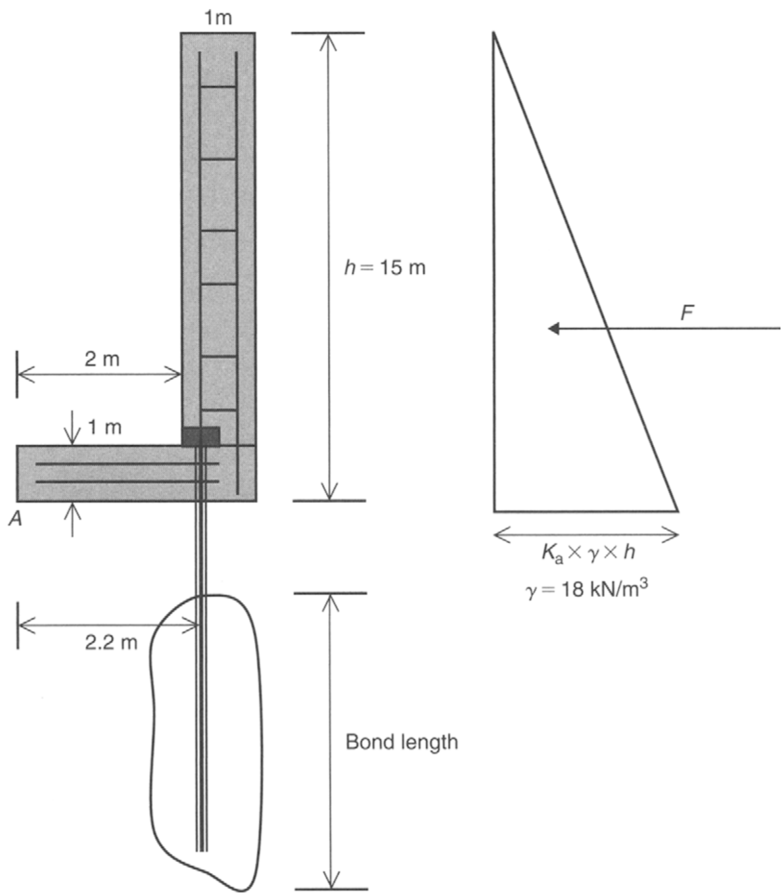

Figure 24.23 Retaining wall held by rock anchors

Solution

$$\text{lateral earth pressure at the bottom of the retaining wall} = K_a \times \gamma \times h$$
$$= 0.33 \times 18 \times 15$$
$$= 89.1\ \text{kN/m}^2$$

$$\text{total lateral force, } F = \text{area of the pressure triangle}$$
$$= 15/2 \times 89.1$$
$$= 668.25\ \text{kN}$$

$$\text{overturning moment around point A} = 668.25 \times 15/3$$
$$= 3{,}341.25\ \text{kNm}$$

$$\text{required factor of safety} = 2.5$$

$$\text{required resisting moment} = 2.5 \times 3{,}341.25$$
$$= 8{,}353\ \text{kNm}$$

resisting moment = resisting moment due to weight of the retaining wall + resisting moment due to pin pile

The resisting moment due to the weight of the retaining wall is due to two parts.

resisting moment due to weight of the retaining wall = resisting moment due to vertical portion of the wall + resisting moment due to horizontal portion of the wall

resisting moment due to vertical portion of the wall = weight × distance to the center of gravity

resisting moment due to vertical portion of the wall = $(1 \times 15) \times 23.5 \times 2.5$ kNm = 881.25 kNm

density of concrete = 23.5 kN/m^3

resisting moment due to horizontal portion of the wall = $(2 \times 1) \times 23.5 \times 1$ kNm = 47 kNm

$$\text{total resistance due to weight of the retaining wall} = 881.25 + 47$$
$$= 928.25 \text{ kNm}$$

$$\text{required resisting moment} = 8{,}353 \text{ kNm}$$

$$\text{resisting moment required from pin piles} = 8{,}353 - 928.25$$
$$= 7{,}424.75 \text{ kNm}$$

$$\text{resisting moment from pin piles} = P \times 2.2 \text{ kNm}$$

P = uplift capacity of pin piles

$$P \times 2.2 = 7{,}424.75$$

$$P = 3{,}375 \text{ kN}$$

where

bond length of pin piles = L
pin pile diameter = 0.3 m

$$P = (\pi \times \text{diameter}) \times \text{length} \times (\text{rock} - \text{grout bond strength})$$

$$P = 3{,}375 = (\pi \times 0.3) \times \text{length} \times 1{,}100 \text{ kPa}$$

$$\text{bond length required} = 3.25 \text{ m}$$

References

Wiley, D. C. 1999. *Foundations on rock*. London: Taylor and Francis.
Hoek, E., Kaiser, P. K., and Bawden, W. F. 1995. *Support of underground excavations in hard rock*. Rotterdam, The Netherlands: Balkema.

25

Soil Anchors

Soil anchors are mainly of two types.

- Mechanical soil anchors.
- Grouted soil anchors.

25.1 Mechanical Soil Anchors

Mechanical soil anchors are installed in retaining walls, shallow foundations, and slabs that are subjected to uplift. The end of the soil anchor consists of an anchor mechanism. The soil anchor is drilled into the soil. Then the anchor is pulled to activate the anchor mechanism. See Figs. 25.1, 25.2, and 25.3.

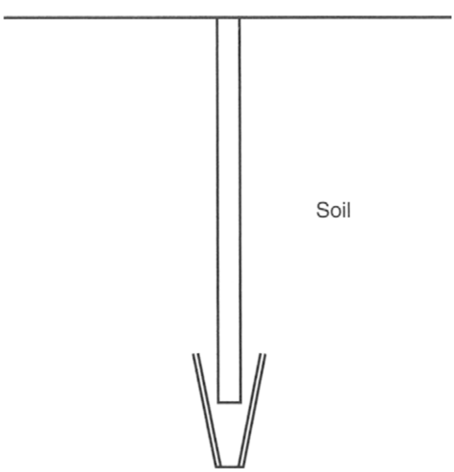

Figure 25.1 Drill the soil anchor

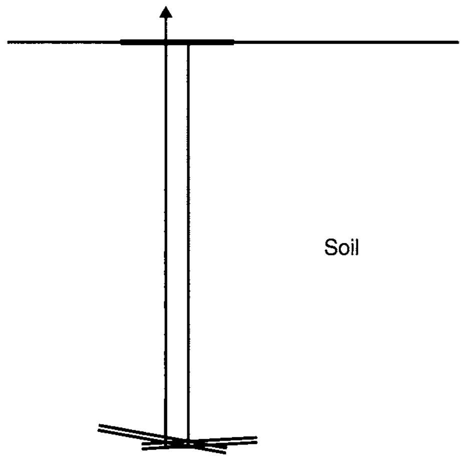

Figure 25.2 Pull the anchor to activate the anchor mechanism

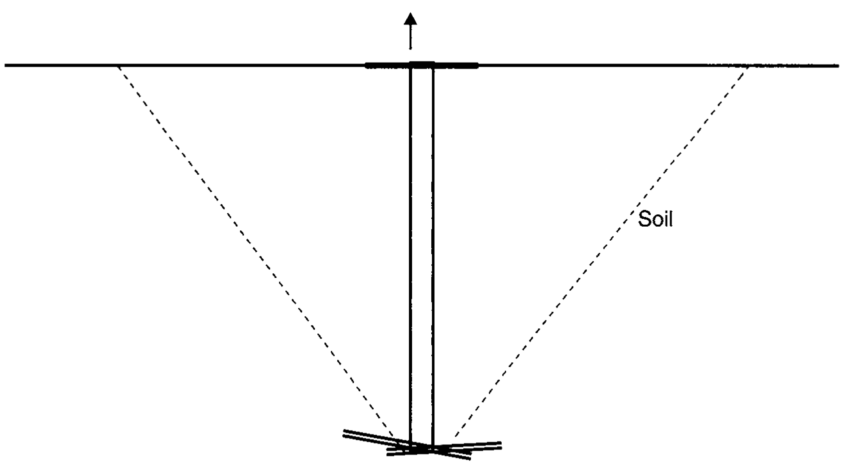

Figure 25.3 Failure cone for mechanical soil anchors

25.2 Grouted Soil Anchors

The principle of grouted soil anchors is the same as for grouted rock anchors. Grouted soil anchors could be nonstressed or prestressed.

The installation procedure for nonstressed grouted soil anchors is given below.

Step 1: Drill a hole.
Step 2: Insert the soil anchor.
Step 3: Grout the hole.

The installation procedure for prestressed grouted soil anchors is slightly different.

Step 1: Drill a hole.
Step 2: Insert the soil anchor.
Step 3: Grout the desired length of the hole.
Step 4: Wait until the grout is set.
Step 5: Apply the desired prestress.
Step 6: Grout the rest of the hole.

See Table 25.1.

Table 25.1 Ultimate soil-grout bond stress

Soil type	**Ultimate bond stress for pressure grouted anchors (psi)***
Cohesive soils (straight shaft)	
Soft silty clay	5–10
Stiff clay (medium to high plasticity)	5–10
Very stiff clay (medium to high plasticity)	10–25
Stiff clay (medium plasticity)	15–30
Very stiff clay (medium plasticity)	20–40
Cohesionless soil (straight shaft)	
Fine to medium sand (medium dense to dense)	12–55
Medium to coarse sand with gravel (medium dense)	16–95
Medium to coarse sand with gravel (dense to very dense)	35–100
Silty sand	25–50
Dense glacial till	43–75
Sandy gravel (medium dense to dense)	31–100

Source: Williams Form Engineering Corp. (modified by author) http://www.williamsform.com/.
*1 psi = 6.894 kPa.

Design Example 25.1

Find the grouted length required for a soil anchor installed in dense fine to medium sand. The soil anchor is needed to hold 6 tons, and the diameter of the grouted hole is 10 in. See Fig. 25.4.

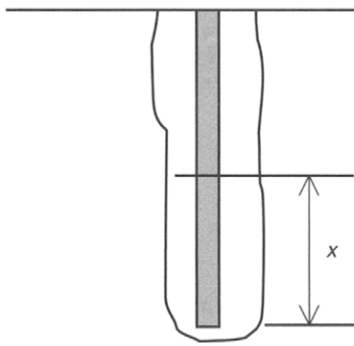

Figure 25.4 Design Example 25.1 illustration

Solution

Assume the bond length required is x.

From Table 25.1, the bond strength for dense fine to medium sand is 12 to 55 psi. Assume the bond strength to be 12 psi.

$$\text{total bond strength} = \text{perimeter area of the bond} \times \text{bond strength}$$

$$(\pi \times D) \times \text{bond length} \times \text{bond strength} = 6 \text{ tons}$$

$$(\pi \times 10) \times L \times 12 = 6 \times 2{,}000 \text{ lb}$$

$$L = 31.85 \text{ in.}$$

Assume a factor of safety of 2.0. Hence the bond length required is 63.7 in.

An actual photograph of augering for horizontal soil anchors is shown in Fig. 25.5.

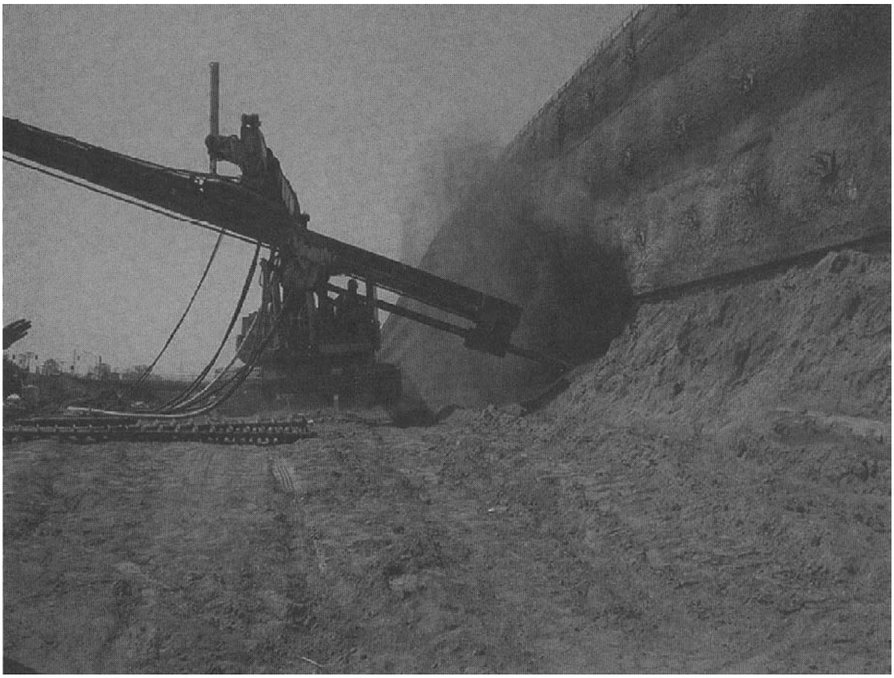

Figure 25.5 Augering for horizontal soil anchors (*Source*: California DOT)

26

Tunnel Design

26.1 Introduction

Tunnel design is an interdisciplinary subject, involving geotechnical engineering, geology, geochemistry, and hydrogeology. In this chapter, the main concepts of tunnel design will be presented.

Tunnels are becoming increasingly common, due to newly available equipment such as tunnel boring machines, road headers, relatively safe blasting techniques, and so on. Tunnel boring machines (TBMs) are widely used today due to their ability to construct tunnels at record speeds. See Fig. 26.1.

The most important part of a tunnel boring machine is its cutter head. The cutter head is composed of a collection of disc cutters that would be pressed into the rock. When the machine rotates, the disc cutters gradually cut the rock into pieces. Rock pieces that are cut are then collected in a bucket known as the muck bucket. Muck is removed from the tunnel by trains or by conveyor belt. The gripper, as the name indicates, grips the tunnel sides so that the boring machine can be pushed hard into the rock. See Figs. 26.2 and 26.3.

26.2 Roadheaders

Roadheaders are not as powerful as tunnel boring machines and can be used for small size tunnels. See Fig. 26.4.

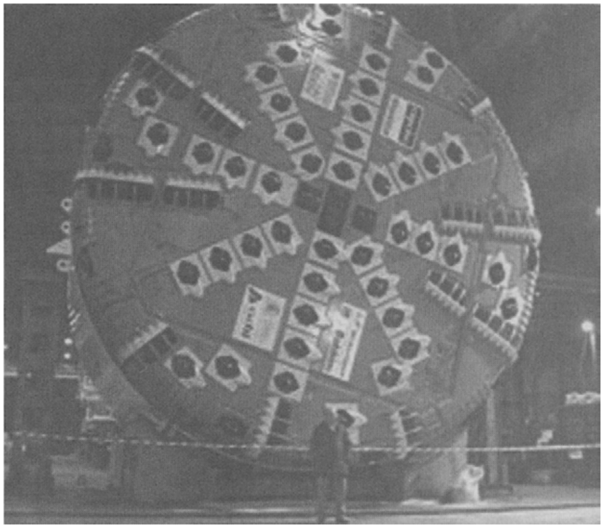

Figure 26.1 Robbins tunnel boring machine. (*Source*: http://www.therobbinscompany.com)

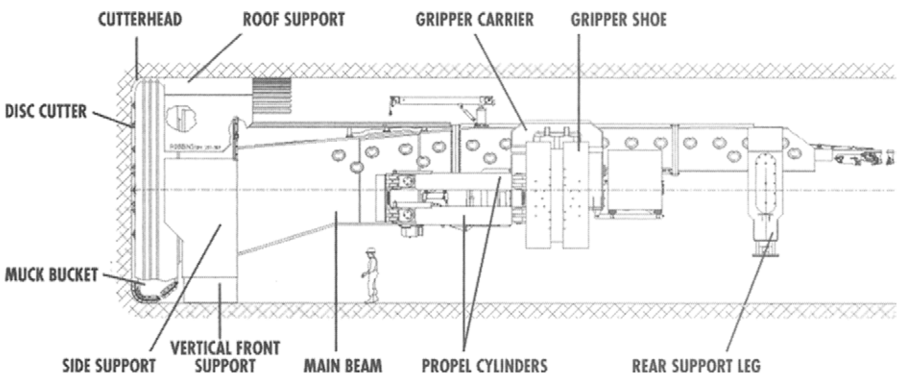

Figure 26.2 Detailed view of the tunnel boring machine. (*Source*: http://www.therobbinscompany.com)

26.3 Drill and Blast

Drilling and blasting was the earliest method of tunnel construction. In this technique, holes are drilled into the rock, packed with explosives, and blasted. See Fig. 26.5.

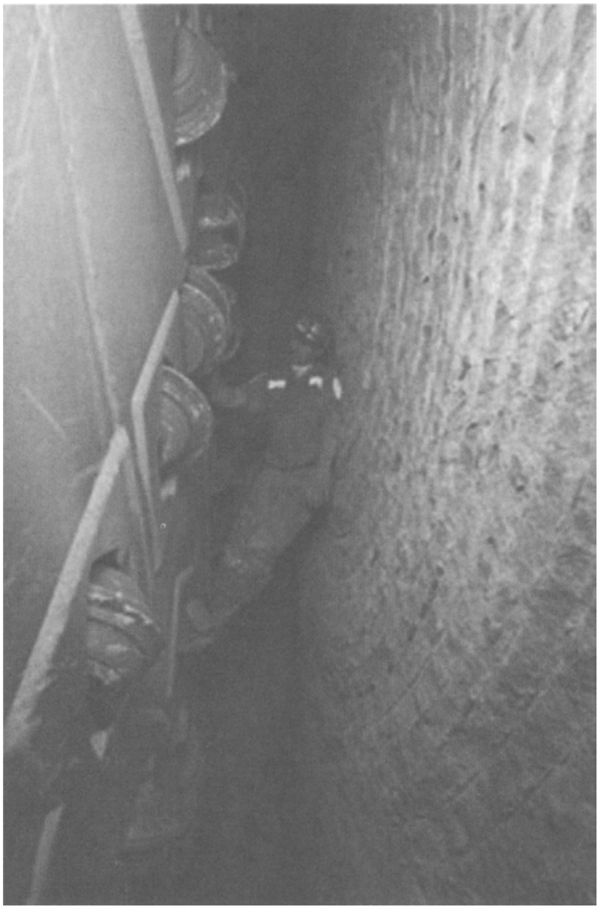

Figure 26.3 Disc cutters of a tunnel boring machine

26.4 Tunnel Design Fundamentals

Tunnel excavation in solid rock may not need much roof support to hold the rock. On the other hand, weak or weathered rock may need heavy support mechanisms. See Fig. 26.6.

In Fig. 26.6, the following significant items are important to consider during the design phase.

From point A to point B, the tunnel liner system has to support the soil overburden. A heavy tunnel support system is necessary in this region.

Figure 26.4 Roadheader in action. (*Source*: http://www.antraquip.net)

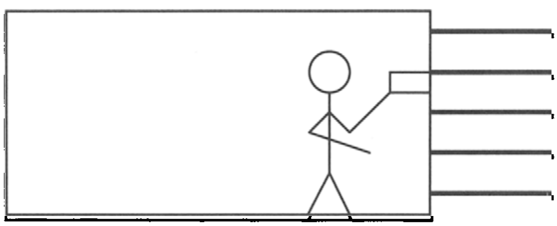

Figure 26.5 Drill and blast technique

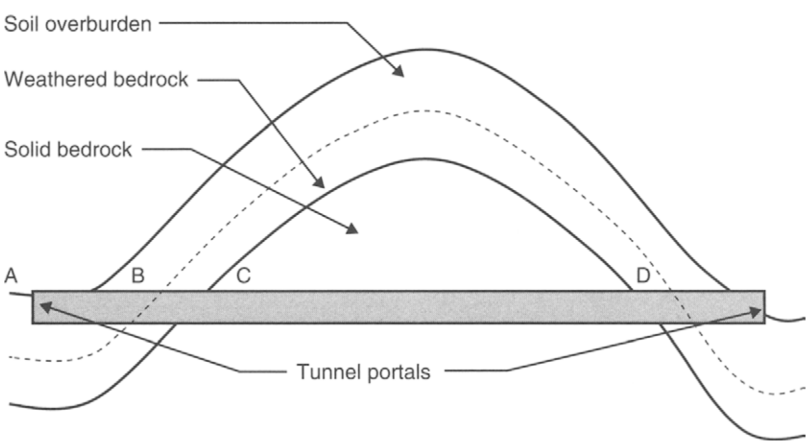

Figure 26.6 Typical tunnel through a mountain

From point B to point C, the top few feet of the bedrock is usually weathered. This region needs more tunnel support than point C to point D, since weathered bedrock exerts a higher loading than solid bedrock.

From point C to point D, tunneling through solid bedrock may depend on the hardness of the rock. The tunnel support system in this region would be minimal compared to tunneling through solid bedrock.

26.4.1 Literature Survey

1. **State Geological Surveys** Most states have conducted geological surveys of the state. These documents are available from the geological survey division of the state.
2. **Well Logs and Previous Borings** Well log data can be obtained from local well drillers for a nominal fee. Well logs normally contain rock stratification data, water bearing zones, high strength rock stratums, and weak rock stratums.
3. **Information on Nearby Tunnels** If there are tunnels nearby, design information on these tunnels would be of immense help. The author has satisfactorily obtained design information on nearby tunnels from corporations. The State Department of Transportation is another source for previous tunnel design work.
4. **Geological and Geotechnical Engineering Studies** Regional university libraries contain information on various geological and geotechnical engineering studies.

26.4.2 Subsurface Investigation Program for Tunnels

Borings

AASHTO (American Association of State Highway and Transportation Officials) recommends borings to a depth of 50 ft below the proposed tunnel bottom. In many cases, drilling 50 ft below the tunnel bottom is not warranted. Most tunnel projects would have many rounds of borings. The spacing of borings should be based on information requirements.

Piezometers

Piezometers should be installed to obtain groundwater data. The piezometers should be screened through the tunnel cross section.

Overburden piezometers should also be installed to obtain overburden aquifer information. There are many cases where water from overburden aquifers can leak into the tunnel during construction. See Fig. 26.7.

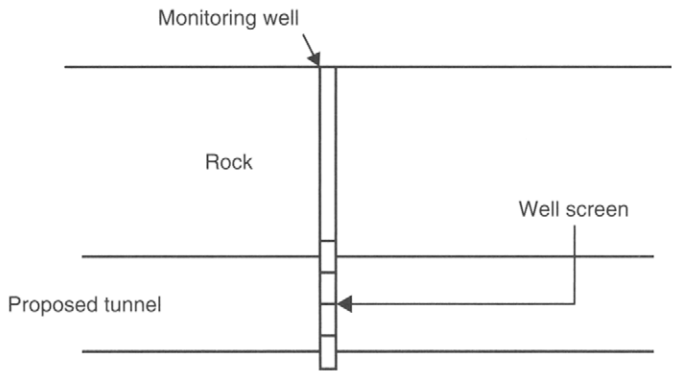

Figure 26.7 Well screens should be within the tunnel section

Geophysical Survey

Geophysical surveys are typically conducted on selected borings. Information such as elastic modulus, Poisson's ratio of rock, density, porosity, and weak zones can be obtained from a geophysical survey.

Geophysical surveys are conducted by sending a seismic signal and measuring the return signal. If large percentage of the signal returns, then the rock is hard and free of fractures. On the other hand, if most of the signal is lost, it is probably due to cracks and fissures in the rock.

Two types of geophysical techniques are available, single hole and double hole. See Fig. 26.8.

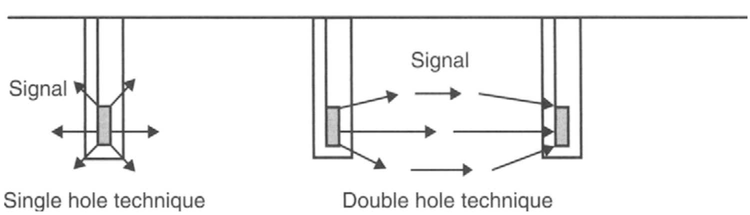

Figure 26.8 Geophysical methods

Packer Tests

Packer tests are conducted to measure the permeability of rock layers. An inflatable object made of rubber is inserted into the rock and pressurized. The pressure drop during the process is measured. If the rock is highly fractured, a large pressure drop will be noted. See Figs. 26.9 and 26.10.

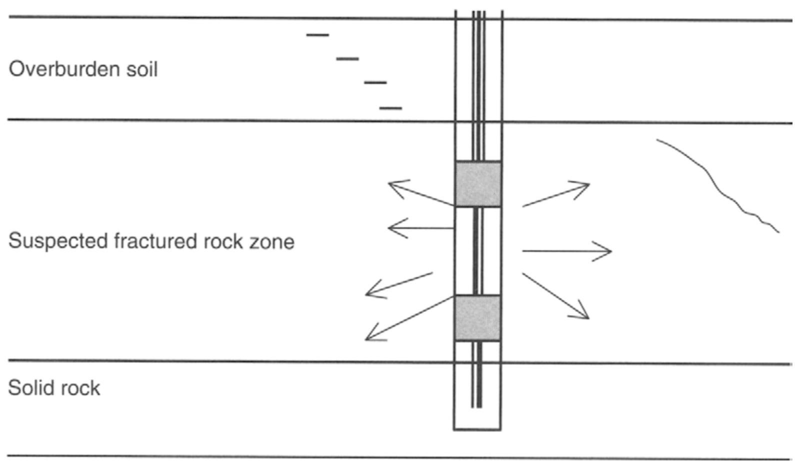

Figure 26.9 Packer inserted and pressurized

Packer tests should be conducted at the tunnel roof elevation, the tunnel cross section, and the tunnel bottom.

Usually during the boring program, engineers will be able to locate fractured rock zones. This can be achieved by analyzing the rock core logs and RQD values. See Fig. 26.11.

Groundwater plays a major role during the construction phase of the tunnel. By conducting packer tests at different locations, engineers can assess the impact of groundwater.

26.4.3 Laboratory Test Program

Laboratory tests are conducted to investigate the strength, hardness, swelling properties, tensile strength, and other properties of rock that may influence the design process.

26.4.4 Unconfined Compressive Strength Test

The unconfined strength test is performed by placing a rock sample in the testing apparatus and crushing it to failure. This test provides the cohesion of intact rock. Unconfined compressive strength is a very important test, since many correlations have been developed using unconfined compressive strength test values. See Fig. 26.12.

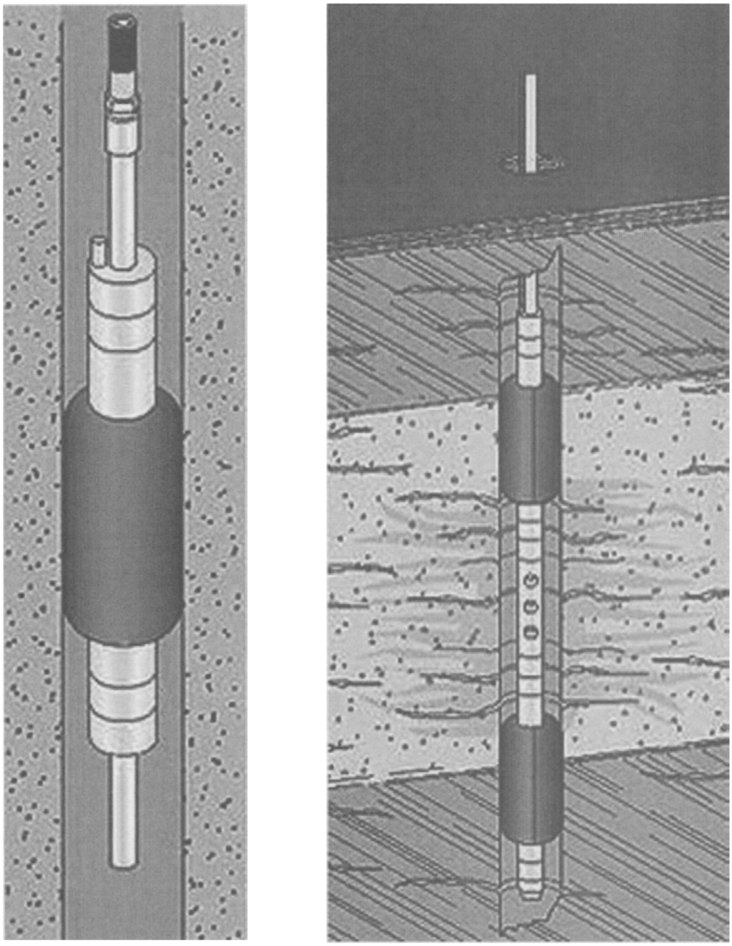

Figure 26.10 Inflatable packers. (*Source*: Hany Equipment http://www.haeny.com/mit/e_mit_gerate_detail.html#7)

26.4.5 Mineral Identification

Identification of minerals becomes very important in soft rock tunneling. The occurrence of minerals such as pyrite indicate the existence of caustic or acidic groundwater. In this case, the tunnel concrete could be subjected to chemical degradation due to acidic groundwater (Merritt, 1972). See Fig. 26.13.

Minerals such as montmorillonite, chlorite, attapulgite, and illite indicate that the rock could be subjected to swelling in the presence of moisture.

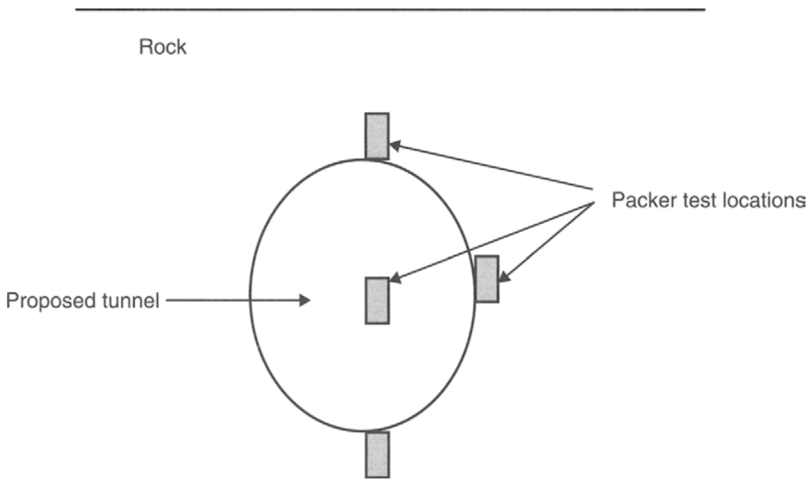

Figure 26.11 Typical Packer test locations

Figure 26.12 Unconfined compressive strength tests of rock. (*Source*: http://www.huntergeotechnics.com.au)

26.4.6 Petrographic Analysis

Petrography is the description and systematic classification of rocks by examination of thin sections. Petrographic analysis would include determining such properties as grain size, texture, color, fractures, and abnormalities. See Fig. 26.14.

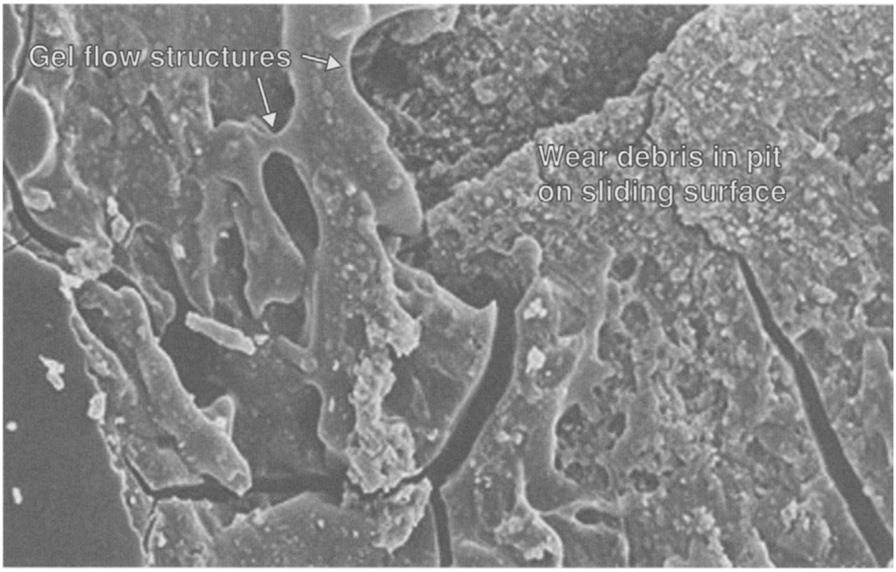

Figure 26.13 Scanning electron microscopic image of rock minerals

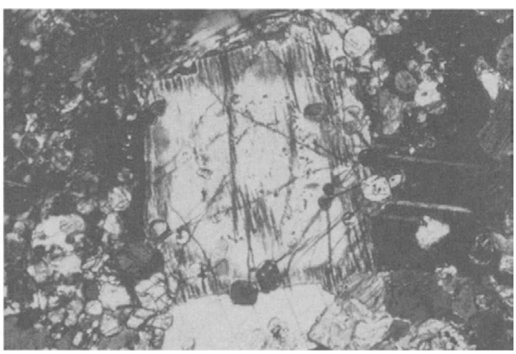

Figure 26.14 Thin section used for petrographic analysis

26.4.7 Tri-Axial Tests

Tri-axial tests are conducted to investigate the shear strength of rock under confining pressure. Usually, tri-axial tests are done on soft rocks.

26.4.8 Tensile Strength Test

Rocks just above the tunnel roof will be subjected to tensile forces. Hence, it is important to investigate the tensile strength of the rock stratum, especially above tunnel roofs.

26.4.9 Hardness Tests

The Schmidt hammer test is the most popular method to investigate the hardness of rocks. Hardness of rock is important for tunnel boring machine selection.

26.4.10 Consolidation Tests

Consolidation tests are done to investigate consolidation characteristics of soft rocks. Usually clay—shale, mudstone, and argillaceous soft rocks—can consolidate under pressure. Consolidation tests can be conducted to investigate the settlement properties of soft rock.

26.4.11 Swell Tests

Many soft rocks swell when exposed to moisture. Swelling is very common in rocks with clay minerals. Water tends to flow from areas of high stress to areas of low stress. When a tunnel is excavated, soil pressure in adjacent rocks is relieved. Rock units with relatively low stress concentrations will absorb water and swell. See Fig. 26.15.

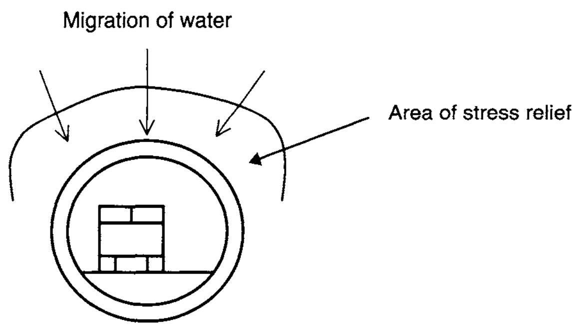

Figure 26.15 Large stresses are exerted on tunnel lining due to swelling of soft rock

26.5 Tunnel Support Systems

All modern tunnels are protected by tunnel support systems. In the past, tunnels built in hard, sound rock were left unsupported. Today, tunnel support systems typically consist of steel arches and concrete segments. Concrete segments are becoming increasingly popular in the tunneling industry. See Figs. 26.16 and 26.17.

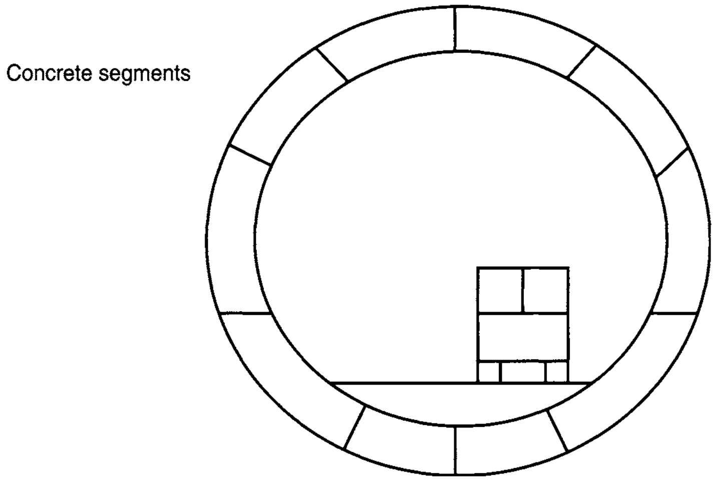

Figure 26.16 Tunnel built using concrete segments

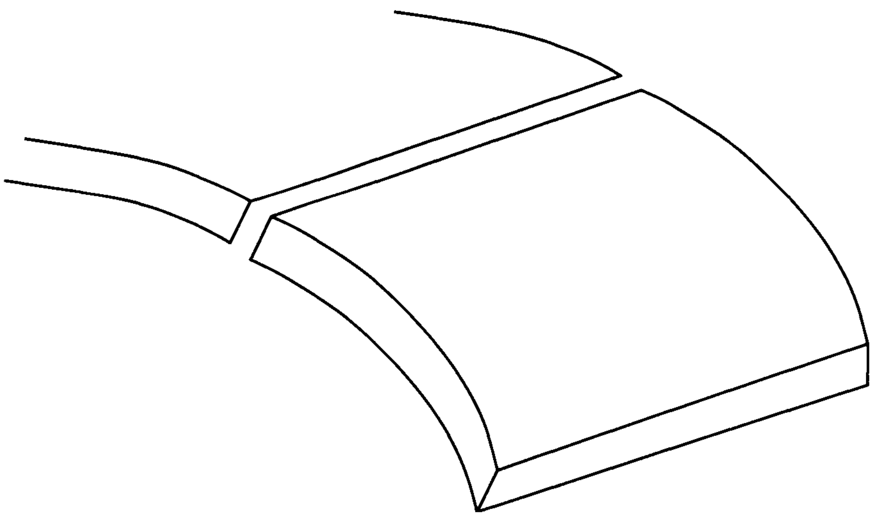

Figure 26.17 Concrete segments

Steel ribs are also commonly used for tunnel support systems. H-beams, in combination with concrete, are installed to provide support for the tunnel. See Figs. 26.18 and 26.19.

26.5.1 Shotcrete

Shotcrete and rock bolts are another support mechanism that is widely used today to support tunnels.

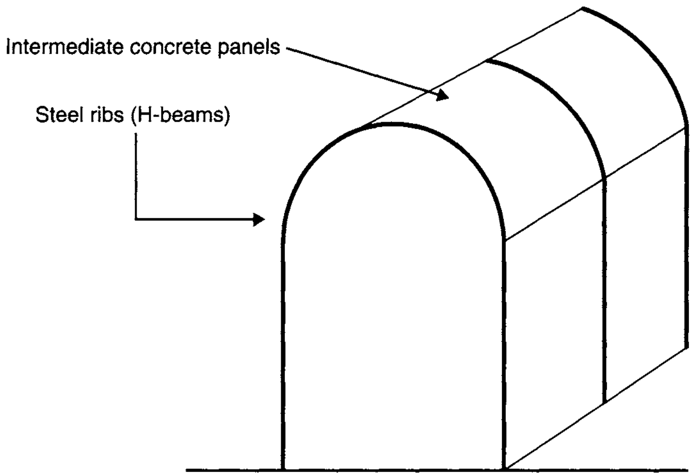

Figure 26.18 Steel rib supported tunnel

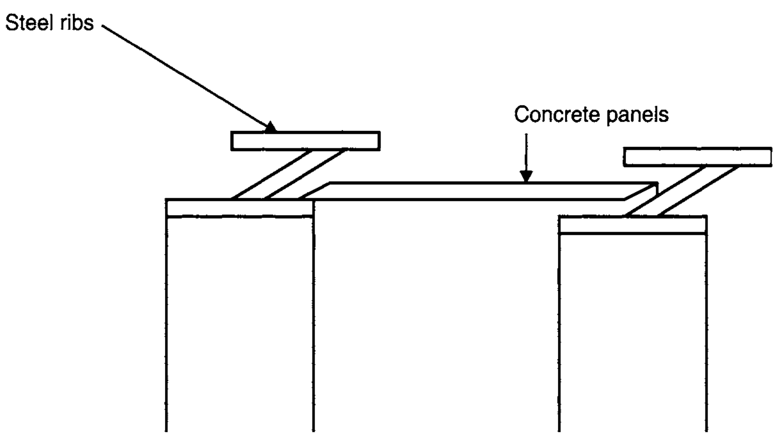

Figure 26.19 Steel supports (H-sections) and concrete panels

Shotcrete is nothing but a mixture of cement, sand, and fine aggregates. Cement, sand, and fine aggregates are mixed and shot into the rock surface. Two types of techniques are available.

26.5.2 Dry Mix Shotcrete

Some tunnel linings are constructed using rock bolts and shotcrete. See Figs. 26.20 and 26.21.

Dry mix shotcrete:

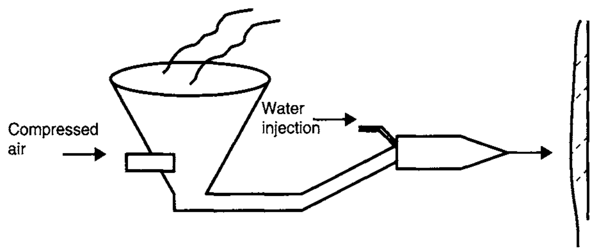

Figure 26.20 Shotcrete preparation

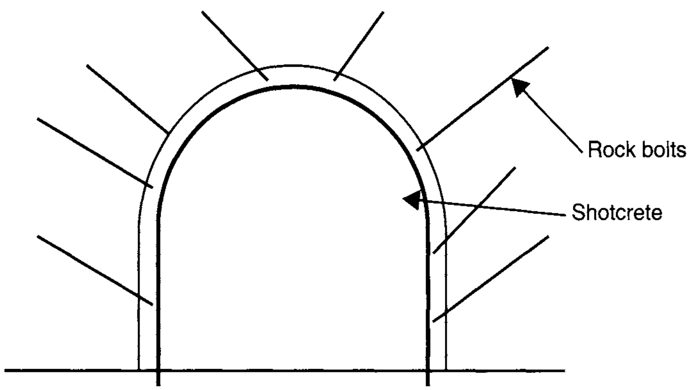

Figure 26.21 Rock bolts and shotcrete in a tunnel

26.6 Wedge Analysis

During tunnel excavations, rock wedges could form due to the intersection of joints, as can be seen in Fig. 26.22. Wedge analysis needs to be performed for all excavations in hard rock. Wedges are formed by the intersection of joints. When the right combination of joints is present at a given location, a wedge is formed. From boring data, it is possible to find the average joint spacing between joints.

- **Identification of Potential Wedges** The manual identification of potential wedges based on rock coring data is a formidable task.
- **Computer Programs** It is not realistic to perform wedge analysis manually based on coring data. For this reason, many engineers

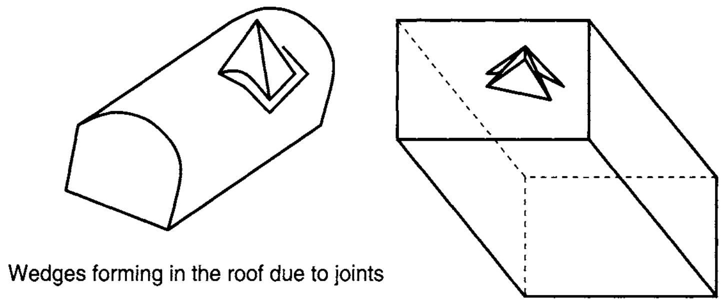

Figure 26.22 Wedges formed due to rock joints

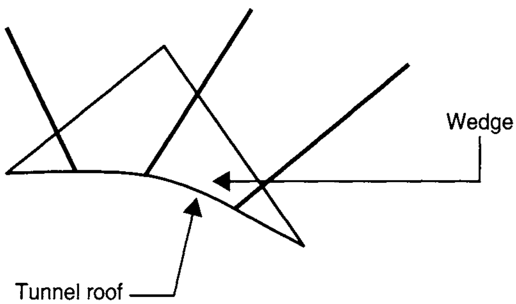

Figure 26.23 Rock bolts used to stabilize a wedge

use computer programs to perform wedge analysis. The UNWEDGE computer program by the Rock Engineering Group is very popular among tunnel engineers. See Fig. 26.23.

References

Hoek, E., Kaiser, P. K., and Bawden, W. F. 1995. *Support of underground excavations in hard rock.* Rotterdam, The Netherlands: Balkema.

Merritt, Frederick S. *Simplified Concrete Masonry Planning and Building,* Second Edition, McGraw-Hill Book Co. Inc., NY 1972.

27

Short Course on Seismology

27.1 Introduction

A general understanding of seismology is needed to design foundations to resist seismic events. The earth is a dynamic system. Under the bedrock (also known as the earth's crust) there is a huge reservoir of lava. Earthquakes and volcanoes occur when the pressure of the lava is released. Ancient peoples found out that earthquakes can be predicted by observing pendulums.

Figure 27.1 shows the normal movement of a pendulum, back and forth, drawing a straight line. Figure 27.2 shows the same pendulum

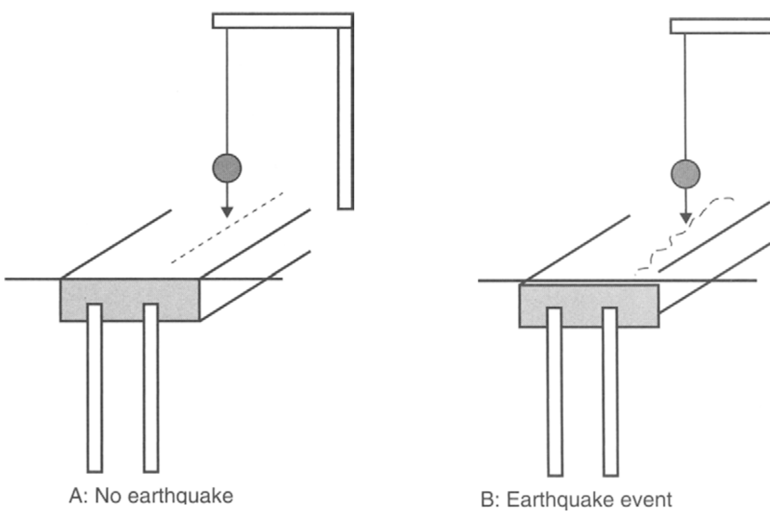

Figure 27.1 Movement of a pendulum during an earthquake

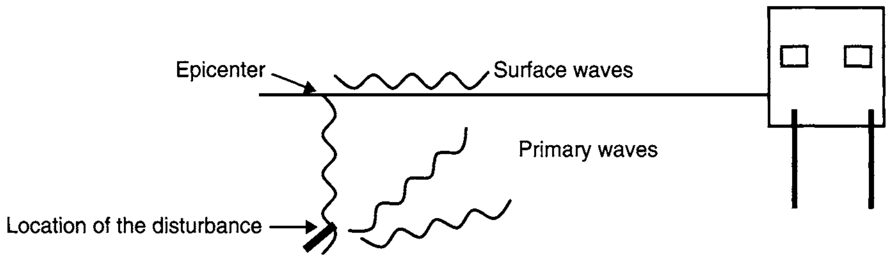

Figure 27.2 Seismic waves

drawing a wavy line during an earthquake event. Seismographs are designed using this principle.

Due to the movement of pile caps during earthquakes, piles are subjected to additional shear forces and bending moments. Earthquakes occur due to disturbances occurring inside the earth's crust. Earthquakes produce three main types of waves.

1. **P-Waves (Primary Waves)** P-waves are also known as compressional waves or longitudinal waves.
2. **S-Waves (Secondary Waves)** S-waves are also known as shear waves or transverse waves.
3. **Surface Waves** Surface waves are shear waves that travel near the surface.

27.1.1 Faults

Faults are a common occurrence on earth. Fortunately, most faults are inactive and will not cause earthquakes. A fault is a fracture where one block of earth moves relative to another.

27.1.2 Horizontal Fault

In a horizontal fault, one earth block moves horizontally relative to the other. The movement is horizontal. See Fig. 27.3.

27.1.3 Vertical Fault (Strike Slip Faults)

In vertical faults, one block moves in a downward direction in relation to another. See Fig. 27.4.

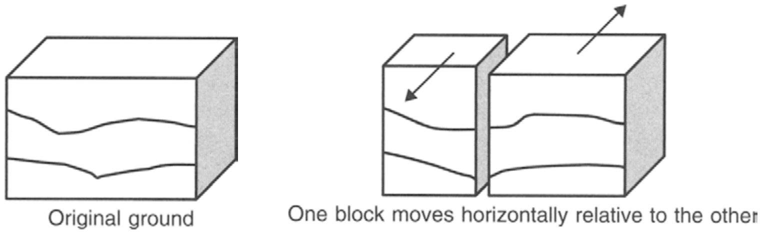

Figure 27.3 Movement in a horizontal fault

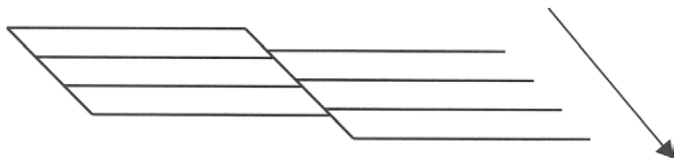

Figure 27.4 Movement in a vertical fault

27.1.4 Active Fault

An active fault is defined as a fault that has an average historic slip rate of 1 mm/yr or more during last 11,000 years (IBC).

27.2 Richter Magnitude Scale (*M*)

The Richter magnitude scale is a logarithmic scale that follows the relation

$$M = \log A - \log A_0 = \log(A/A_0)$$

where

M = Richter magnitude scale
A = maximum trace amplitude during the earthquake
A_0 = standard amplitude

Here, a standard value of 0.001 mm is used for comparison. This corresponds to a very small earthquake.

Design Example 27.1

Find the Richter magnitude scale for an earthquake that recorded an amplitude of (1) 0.001 mm; (2) 0.01 mm; and (3) 10 mm.

Solution

1. $A = 0.001\,\text{mm}$

$$M = \log(A/A_0) = \log(0.001/0.001) = \log 1 = 0$$
$$M = 0$$

2. $A = 0.01\,\text{mm}$

$$M = \log(A/A_0) = \log(0.01/0.001) = \log 10 = 1$$
$$M = 1$$

3. $A = 10\,\text{mm}$

$$M = \log(A/A_0) = \log(10/0.001) = \log 10{,}000 = 4$$
$$M = 4$$

See Table 27.1 for a list of the largest earthquakes ever recorded.

Table 27.1 Largest earthquakes recorded

Location	Date	Magnitude
Chile	1960	9.5
Prince William, Alaska	1964	9.2
Aleutian Islands	1957	9.1
Kamchatka	1952	9.0
Ecuador	1906	8.8
Rat Islands	1965	8.7
India-China border	1950	8.6
Kamchatka	1923	8.5
Indonesia	1938	8.5
Kuril Islands	1963	8.5

Source: USGS Website (*www.usgs.org* United States Geological Survey).

27.2.1 Peak Ground Acceleration

The peak ground acceleration is a very important parameter for geotechnical engineers. During an earthquake, soil particles accelerate. The acceleration of soil particles can be either horizontal or vertical.

27.2.2 Seismic Waves

The following partial differential equation represents seismic waves.

$$G\left(\partial^2 u/\partial x^2 + \partial^2 u/\partial z^2\right) = \rho\left(\partial^2 u/\partial t^2 + \partial^2 v/\partial t^2\right)$$

where

G = shear modulus of soil
∂ = partial differential operator
u = horizontal motion of soil
v = vertical motion of soil
ρ = density of soil

Fortunately, geotechnical engineers are not called upon to solve this partial differential equation.

A seismic waveform described by the above equation would create shear forces and bending moments on piles.

27.2.3 Seismic Wave Velocities

The velocity of seismic waves is dependent upon the soil/rock type. Seismic waves travel much faster in sound rock than in soil. See Table 27.2.

Table 27.2 Seismic wave velocity

Soil/rock type	Wave velocity (ft/sec)
Dry silt, sand, loose gravel, loam, loose rock, moist fine grained top soil	600–2,500
Compact till, gravel below water table, compact clayey gravel, cemented sand, sandy clay	2,500–7,500
Weathered rock, partly decomposed rock, fractured rock	2,000–10,000
Sound shale	2,500–11,000
Sound sandstone	5,000–14,000
Sound limestone and chalk	6,000–20,000
Sound igneous rock (granite, diabase)	12,000–20,000
Sound metamorphic rock	10,000–16,000

Source: Peck, Hanson, and Thornburn (1974).

27.3 Liquefaction

Sandy and silty soils tend to lose strength and turn into a liquid-like state during an earthquake. This phenomenon, called liquefaction, occurs due to the increase of pore pressure during an earthquake event in the soil caused by seismic waves. See Fig. 27.5.

Figure 27.5 Foundation failure due to liquefaction

The liquefaction of soil was thoroughly studied by I. M. Idris at the Bolten Seed conference during the 1970s. As one would expect, the liquefaction behavior of soil cannot be expressed in one simple equation. Many correlations and semi-empirical equations have been introduced by researchers. For this reason, Professor Robert W. Whitman convened a workshop in 1985 on behalf of the National Research Council (NRC). Experts from many countries participated in this workshop, and a procedure was developed to evaluate the liquefaction behavior of soils.

It should be mentioned that only sandy and silty soils tend to liquefy. Clay soils do not undergo liquefaction.

27.3.1 Impact Due to Earthquakes

Imagine a bullet hitting a wall as in Fig. 27.6. The extent of damage to the wall due to the bullet depends on number of parameters. The bullet properties that affect the amount of damage are

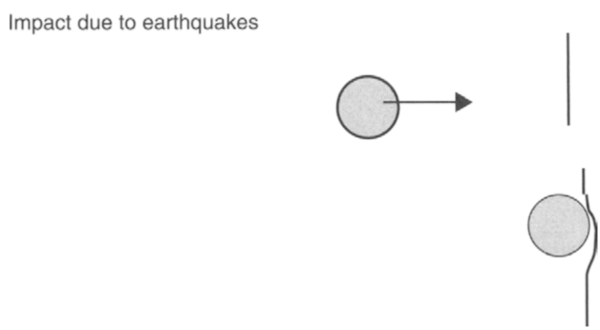

Figure 27.6 Bullet hitting a wall

1. Velocity of the bullet.
2. Weight of the bullet.

3. Hardness of the bullet material.

The wall properties that affect the amount of damage are

1. Hardness of the wall material.
2. Type of wall material.

A bullet hitting a wall is a single event, mainly affected by the parameters listed above. In contrast, the phenomenon of liquefaction is affected by many parameters, such as the earthquake, the soil type, and the soil's resistance to liquefaction. These will be discussed in detail in the sections that follow.

27.3.2 Earthquake Properties

- Magnitude of the earthquake.
- Peak horizontal acceleration at the ground surface, $a_{\max}$.

27.3.3 Soil Properties

- Soil strength (measured by Standard Penetration Test, SPT, value).
- Effective stress at the point of liquefaction.
- Content of fines (where fines are defined as particles that pass through the #200 sieve).
- Earthquake properties that affect the liquefaction of a soil are amalgamated into one parameter known as the Cyclic Stress Ratio, CSR.

$$\text{cyclic stress ratio, CSR} = 0.65 \times (a_{\max}/g) \times (\sigma/\sigma') \times r_d \qquad (27.1)$$

where

$a_{\max}$ = peak horizontal acceleration at the ground surface
σ = total stress at the point of concern
σ' = effective stress at the point of concern
r_d = stress reduction coefficient

The stress reduction coefficient parameter accounts for the flexibility of the soil profile.

$$r_d = 1.0 - 0.00765\ Z \text{ for } Z < 9.15\,\text{m} \tag{27.2}$$

$$r_d = 1.174 - 0.0267\ Z \text{ for } 9.15\,\text{m} < Z < 23\,\text{m} \tag{27.3}$$

where

Z = depth to the point of concern in meters

27.3.4 Soil Resistance to Liquefaction

As a rule of thumb, any soil that has an SPT value higher than 30 will not liquefy.

As mentioned earlier, resistance to liquefaction of a soil depends on its strength measured by SPT value. Researchers have found that resistance to liquefaction of a soil depends on the content of fines as well.

The following equation can be used for a clean sand. A clean sand is defined as a sand with less than 5% fines.

$$\text{CRR}_{7.5} = 1/[34 - (N_1)_{60}] + (N_1)_{60}/135 + 50/[10 \times (N_1)_{60} + 45]^2 - 2/200 \tag{27.4}$$

where

$\text{CRR}_{7.5}$ = soil resistance to liquefaction for an earthquake with a magnitude of 7.5 Richter

A correction factor needs to be applied to any other magnitude. That process is described later in the chapter. Equation 27.4 can be used only for sands. Also, the content of fines should be less than 5%. A correction factor has to be used for soils with a higher content of fines. That procedure is described later in the chapter.

$(N_1)_{60}$ is the standard penetration value corrected to a 60% hammer and an overburden pressure of 100 kPa. Note that above equations were developed in metric units.

The procedure to obtain $(N_1)_{60}$ is as follows.

$$(N_1)_{60} = N_m \times C_N \times C_E \times C_B \times C_R \tag{27.5}$$

where

N_m = SPT value measured in the field
C_N = overburden correction factor = $(Pa/\sigma')^{0.5}$

where

Pa = 100 kPa
σ' = effective stress of soil at point of measurement
C_E = energy correction factor for the SPT hammer

for donut hammers, $C_E = 0.5$ to 1.0
for trip type donut hammers, $C_E = 0.8$ to 1.3

C_B = borehole diameter correction

for borehole diameter of 65 mm to 115 mm use $C_B = 1.0$
for borehole diameter of 150 mm use $C_B = 1.05$
for borehole diameter of 200 mm use $C_B = 1.15$

C_R = rod length correction

Rods attached to the SPT spoon exert their weight on the soil. Longer rods exert a higher load on soil and in some cases the spoon sinks into the ground due to the weight of the rods, even without any hammer blows. Hence, a correction is made to account for the weight of rods.

for rod length <3 m, use $C_R = 0.75$
for rod length 3 m to 4 m, use $C_R = 0.8$
for rod length 4 m to 6 m, use $C_R = 0.85$
for rod length 6 m to 10 m, use $C_R = 0.95$
for rod length 10 m to 30 m, use $C_R = 1.0$

Design Example 27.2

Consider a point at a depth of 5 m in a sandy soil where fines <5%. The total density of the soil is 1,800 kg/m^3. The groundwater is at a depth of 2 m. The corrected $(N_1)_{60}$ value is 15. The peak horizontal acceleration

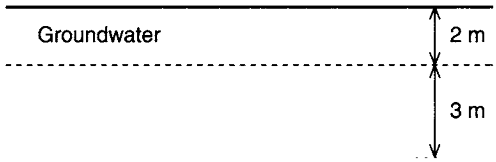

Figure 27.7 Soil profile

at the ground surface, a_{max}, was found to be 0.15 g for an earthquake of magnitude 7.5. Check to see whether the soil at a depth of 5 m would liquefy under an earthquake of magnitude 7.5. See Fig. 27.7.

Solution

STEP 1: Find the cyclic stress ratio, CSR.

$$\text{CSR} = 0.65(a_{max}/g) \times (\sigma/\sigma') \times r_d \tag{27.1}$$

where

a_{max} = peak horizontal acceleration at the ground surface = 0.15 g
σ = total stress at the point of concern
σ' = effective stress at the point of concern
r_d = stress reduction coefficient

Recall that the stress reduction coefficient is a parameter that accounts for the flexibility of the soil profile.

$$r_d = 1.0 - 0.00765\,Z \text{ for } Z < 9.15\,\text{m} \tag{27.2}$$

where

Z = depth to the point of concern in meters

$$r_d = 1.174 - 0.0267\,Z \text{ for } 9.15\,\text{m} < Z < 23\,\text{m} \tag{27.3}$$

$$\sigma = 5 \times 1{,}800 = 9{,}000\,\text{kg/m}^2$$
$$\sigma' = 2 \times 1{,}800 + 3 \times (1{,}800 - 1{,}000) = 6{,}000\,\text{kg/m}^2$$

where

density of water = 1,000 kg/m^2

Since the depth of concern is 5 m (which is less than 9.15 m), use Equation 27.2 to find r_d.

$$r_d = 1.0 - 0.00765\,Z \text{ for } Z < 9.15\,\text{m}$$
$$r_d = 1.0 - 0.00765 \times 5 = 0.962$$

Hence

$$\text{CSR} = 0.65 \times (0.15) \times 9{,}000/6{,}000 \times 0.962 = 0.1407$$

STEP 2: Find the soil resistance to liquefaction using the previous Equation 27.4.

$$\text{CRR}_{7.5} = 1/[34 - (N_1)_{60}] + (N_1)_{60}/135 + 50/[10 \times (N_1)_{60} + 45]^2 - 2/200 \tag{27.4}$$

The $(N_1)_{60}$ value is given as 15. Hence,

$$\text{CRR}_{7.5} = 0.155$$

Since the soil resistance to liquefaction (0.155) is larger than the CSR value (0.1407), the soil at 5 m depth will not undergo liquefaction for an earthquake of magnitude 7.5.

27.3.5 Correction Factor for Magnitude

As you are aware, Equation 27.4 (for $\text{CRR}_{7.5}$) is valid only for earthquakes of magnitude 7.5. A correction factor is proposed to account for magnitudes different from 7.5. The factor of safety (F.O.S.) is given by

$$\text{CRR}_{7.5}/\text{CSR}$$

where

$\text{CRR}_{7.5}$ = resistance to soil liquefaction for a magnitude of 7.5 earthquake
CSR = cyclic stress ratio (which is a measure of the impact due to the earthquake load)

The factor of safety for any other earthquake is given by following equation.

$$\text{Factor of Safety (F.O.S.)} = (\text{CRR}_{7.5}/\text{CSR}) \times \text{MSF} \tag{27.5}$$

where

MSF = magnitude scaling factor

The MSF is given in Table 27.3.

Table 27.3 Magnitude scaling factors

Earthquake magnitude	MSF suggested by Idris (1990)	MSF suggested by Andrus and Stokoe (2000)
5.5	2.2	2.8
6.0	1.76	2.1
6.5	1.44	1.6
7.0	1.19	1.25
7.5	1.00	1.00
8.0	0.84	–
8.5	0.72	–

Source: Idris (1990) and Andrus and Stokoe (2000).

The results of the 1985 NRC (National Research Council) conference gave freedom to engineers to select either of the values suggested by Idris (1985) or by Andrus and Stokoe (2000). The Idris values are more conservative. In noncritical buildings such as warehouses, engineers may be able to use Andrus and Stokoe values.

Design Example 27.3

The $CRR_{7.5}$ value of a soil is found to be 0.11. The CSR value for the soil is computed as 0.16. Will this soil liquefy for an earthquake of magnitude 6.5?

Solution

STEP 1: Find the factor of safety.

$$\text{Factor of Safety (F.O.S.)} = (CRR_{7.5}/CSR) \times MSF$$

where

MSF = magnitude scaling factor for an earthquake of 6.5
= 1.44 (Idris, 1985)

$$\text{F.O.S.} = (0.11/0.16) \times 1.44 = 0.99 \text{ (Soil would liquefy.)}$$

Use the MSF given by Andrus and Stokoe (2000).

$$\text{F.O.S.} = (0.11/0.16) \times 1.6 = 1.1 \text{ (Soil would not liquefy.)}$$

27.3.6 Correction Factor for Content of Fines

Equation 27.4 was developed for clean sand with fines content less than 5%. A correction factor is suggested for soils with higher fines content.

$$\text{CRR}_{7.5} = 1/[34 - (N_1)_{60}] + (N_1)_{60}/135 + 50/[10 \times (N_1)_{60} + 45]^2 - 2/200 \tag{27.4}$$

The corrected $(N_1)_{60}$ value should be used in Equation 27.4 for soils with higher fines content.

The following procedure should be followed to find the correction factor.

- Compute $(N_1)_{60}$ as in the previous case.
- Use the following equations to account for the fines content, referred to below as FC.

$$(N_1)_{60C} = a + b \times (N_1)_{60}$$

where

$(N_1)_{60C}$ = corrected $(N_1)_{60}$ value

The values of a and b are given as follows.

$$a = 0 \text{ for FC } <5\% \tag{27.6}$$

$$a = \exp[1.76 - 190/\text{FC}^2] \text{ for } 5\% < \text{FC} < 35\% \tag{27.7}$$

$$a = 5.0 \text{ for FC } >35\% \tag{27.8}$$

$$b = 1.0 \text{ for FC } <5\% \tag{27.9}$$

$$b = [0.99 + (FC^{1.5}/1{,}000)] \text{ for } 5\% < \text{FC} < 35\% \tag{27.10}$$

$$b = 1.2 \text{ for FC } >35\% \tag{27.11}$$

Design Example 27.4

The $(N_1)_{60}$ value for soil with 30% fines content was found to be 20. Find the corrected $(N_1)_{60C}$ value for that soil.

Solution

STEP 1: Find the $(N_1)_{60C}$ value.
$(N_1)_{60C} = a + b\ (N_1)_{60}$

For FC = 30%

$$a = \exp[1.76 - 190/\mathrm{FC}^2] = \exp[1.76 - 190/30^2] = 4.706 \qquad (27.7)$$

For FC = 30%

$$b = [0.99 + (\mathrm{FC}^{1.5}/1{,}000)] = 1.154 \qquad (27.10)$$

Hence

$$(N_1)_{60C} = 4.706 + 1.154 \times 20 = 27.78$$

Design Example 27.5

Consider a point at a depth of 5 m in a sandy soil where the fines content is 40%. The total density of the soil is 1,800 kg/m^3. The groundwater is at a depth of 2 m, and the density of water, γ_w, is 1,000 kg/m^3. The corrected $(N_1)_{60}$ value is 15. For this problem, all the correction parameters C_N, C_E, C_B, and C_R are applied, except for the fines content. The peak horizontal acceleration at the ground surface, a_{max}, was found to be 0.15 g for an earthquake of magnitude 8.5. Check to see whether the soil at a depth of 5 m would liquefy under this earthquake load.

Solution

STEP 1: Find the cyclic stress ratio using Equation 27.1.

$$\mathrm{CSR} = 0.65(a_{max}/g) \times (\sigma/\sigma') \times r_d \qquad (27.1)$$

$$\sigma = 5 \times 1{,}800 = 9{,}000\ \mathrm{kg/m^2}$$
$$\sigma' = 2 \times 1{,}800 + 3 \times (1{,}800 \times 1{,}000) = 6{,}000\ \mathrm{kg/m^2}$$

Since the depth of concern is 5 m (which is less than 9.15 m) use Equation 27.2

$$r_d = 1.0 - 0.00765\ Z \text{ for } Z < 9.15\,\text{m} \quad (27.2)$$

$$r_d = 1.0 - 0.00765 \times 5 = 0.962$$

Hence

$$CSR = 0.65 \times (0.15) \times 9{,}000/6{,}000 \times 0.962 = 0.1407$$

STEP 2: Provide the correction factor for the fines content.
For soils with 40% fines, $a = 5$ and $b = 1.2$ (Eqs. 27.8 and 27.11).

$$(N_1)_{60C} = a + b \times (N_1)_{60}$$

$$(N_1)_{60C} = 5 + 1.2 \times 15 = 23$$

STEP 3: Find the soil resistance to liquefaction using Equation 27.4.

$$CRR_{7.5} = 1/[34 - (N_1)_{60}] + (N_1)_{60}/135 + 50/[10 \times (N_1)_{60} + 45]^2 - 2/200 \quad (27.4)$$

The $(N_1)_{60C}$ value is found to be 23 (from Step 2).
Hence $CRR_{7.5} = 0.25$ (from Eq. 27.4)

$$\text{Factor of Safety (F.O.S.)} = (CRR_{7.5}/CSR) \times MSF$$

where

MSF = magnitude scaling factor

Obtain the MSF from Table 27.3.

MSF = 0.72 for an earthquake of 8.5 magnitude (Idris values)
CSR = 0.1407 (see Step 1)

$$\text{F.O.S.} = (0.25/0.1407) \times 0.72 = 1.27$$

The soil would not liquefy.

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28

Geosynthetics in Geotechnical Engineering

28.1 Geotextiles

Geotextiles are geosynthetic products. While clothes are woven out of natural fibers such as cotton and silk, geotextiles are woven out of synthetic fibers. Synthetic fibers are manufactured using various polymers. Since geotextiles are woven, they are relatively porous. Geotextiles are mainly used for the following purposes (see Fig. 28.1).

Figure 28.1 Geotextiles (*Source*: www.mirraequipment.com)

1. Separation.
2. Filtration.
3. Drainage.
4. Reinforcement.
5. Containment of contaminants.

28.2 Geomembranes

Geomembranes are made of thin plastic sheets. They are much more impervious than geotextiles. For this reason, geomembranes have been used in the landfill industry. Landfill liners, leachatte collection systems, and landfill covers are made of geomembranes. See Fig. 28.2.

Figure 28.2 Geomembrane used for a landfill liner (*Source*: http://www.hallaton.com)

28.3 Geosynthetic Clay Liners (GCLs)

GCLs are manufactured by sandwiching a bentonite clay layer between two geotextiles or geomembranes. See Fig. 28.3.

Figure 28.3 Geosynthetic clay liners (GCLs)

28.4 Geogrids, Geonets, and Geocomposites

Geogrids are mainly used for reinforcing purposes. See Fig. 28.4. The purpose of the geonets is to facilitate the drainage of water. Geocomposites are manufactured by combining two geotextiles (or geomembranes).

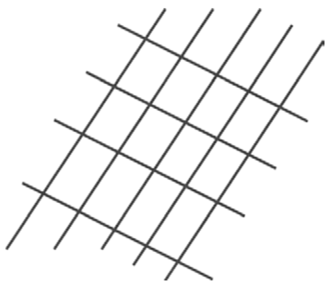


Figure 28.4 Geogrids

Common combinations used by the industry are the geotextile and the geogrid, the geonet and geotextile, and the geonet and the geomembrane. For the geotextile and geogrid combination, the geotextile separates one soil from another, while the geogrid provides the strength.

The combination of the geonet and geotextile is also used by industry. The geonet provides drainage while the geotextile provides separation and strength. See Fig. 28.5. The geonet and geomembrane combination serves this same purpose.

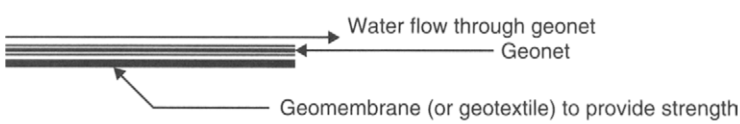


Figure 28.5 Geonet and geomembrane composite

29

Slurry Cutoff Walls

Slurry cutoff walls are designed mainly for seepage control. Recently, slurry cutoff walls have been designed around landfills to stop contaminant migration. See Fig. 29.1.

A slurry cutoff wall can be constructed as shown in Fig. 29.1 to stop contaminant migration. For the slurry cutoff wall to be effective, it should penetrate into an impermeable clay layer.

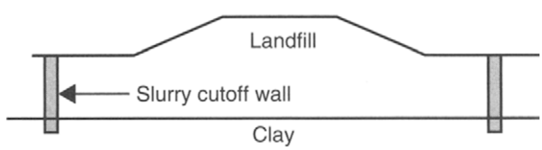


Figure 29.1 Slurry cutoff wall to stop contaminant migration

29.1 Slurry Cutoff Wall Types

Two types of slurry cutoff walls are popular.

- Soil-bentonite walls (SB walls).
- Cement-bentonite walls (CB walls).

29.2 Soil-Bentonite Walls (SB Walls)

A trench is excavated and filled with a mixture of soil, bentonite powder, and water. Normally, soil coming out of the trench is used. Theoretically, any type of soil can be used. The more fines (i.e., fine particles smaller than sieve #200) the soil has, the lower the permeability is. If the clay is in a lumpy form, the permeability will not be low, even though clay particles are finer than sieve #200. See Fig. 29.2.

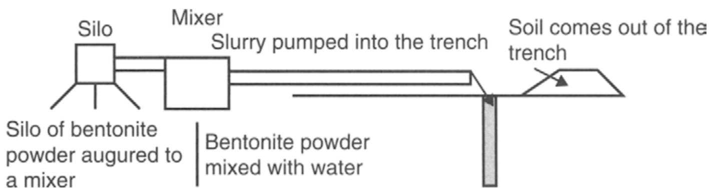


Figure 29.2 Slurry cutoff wall setup

The slurry is pumped straight to the trench to keep the trench open. A backhoe takes the slurry out of the trench and mixes it with the soil from the trench. The density of a soil-bentonite slurry is approximately 120 pcf.

The permeability of soil-bentonite walls depends on the type of soil. If the soil contains more sandy material, then the permeability would be higher. The bulging of the trench due to the high density of the soil-bentonite mixture is considered to be one of the problems of soil-bentonite walls. Cement-bentonite walls are used to avoid bulging.

29.3 Cement-Bentonite Walls (CB Walls)

A slurry is produced using cement and bentonite. The slurry is pumped into the trench to harden. Soil is not used. The density of a cement-bentonite slurry is approximately 70 pcf.

29.4 Trench Stability for Slurry Cutoff Walls in Sandy Soils

Design Example 29.1

Check the stability of a 25 ft deep trench. The groundwater is 5 ft below the ground surface, and the slurry level is maintained 2 ft below the ground surface. See Fig. 29.3.

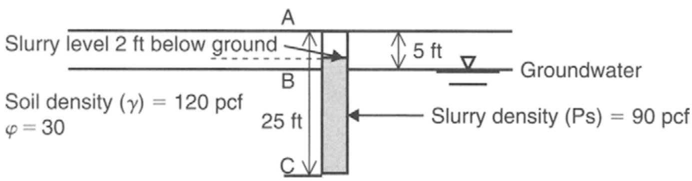


Figure 29.3 Slurry cutoff wall

Solution

STEP 1: Determine the trench sidewall stability.

The trench sidewall stability at point B is calculated as follows.

$$\text{vertical effective stress at point B in the surrounding soil} = 5 \times 120 = 600\,\text{psf}$$

$$\text{horizontal stress} = K_a \times \text{vertical effective stress} = 0.33 \times 600 = 198\,\text{psf}$$

$$K_a = \tan^2(45 - \varphi/2) = 0.333$$

$$\text{vertical stress in the slurry} = 3 \times 90 = 270\,\text{psf}$$

The slurry level is 2 ft below the ground level. The horizontal stress in the slurry is therefore 270 psf. For slurries and water,

$$K_a = 1$$

The pressure is the same in all directions.

The stress in the slurry (270 psf) is greater than the horizontal stress due to the soil (198 psf). Hence the trench will not cave in at point B. The factor of safety against caving in at point B is

$$270/198 = 1.4$$

The trench sidewall stability at point C is calculated as follows.

$$\begin{aligned}
&\text{vertical effective stress at point C in surrounding soil}\\
&\qquad = 5 \times 120 + 20 \times (120 - 62.4)\\
&\qquad = 1{,}752\,\text{psf}\\
\text{horizontal stress due to soil} &= K_a \times \text{vertical effective stress}\\
&= 0.33 \times 1{,}752 = 578\,\text{psf}\\
\text{vertical stress in the slurry} &= 23 \times 90 = 2{,}070\,\text{psf}\\
\text{horizontal stress in the slurry} &= 2{,}070\,\text{psf}
\end{aligned}$$

The factor of safety against trench cave in at point C can be calculated as

$$2070/578 = 3.58$$

STEP 2: Trench bottom stability.

The stability of the trench bottom also needs to be ensured. See Fig. 29.4.

$$\begin{gathered}
\text{vertical stress in surrounding soil at trench bottom}\\
= 1{,}752\,\text{psf (found earlier)}\\
\text{horizontal stress due to surrounding soil, } \sigma_{s1}, = K_a \times 1{,}752\\
= 0.33 \times 1{,}752 = 578\,\text{psf}\\
\text{stress in the slurry, } Ps = 2{,}070\,\text{psf}\\
\text{vertical stress in the soil just below the slurry} = 2{,}070\,\text{psf}
\end{gathered}$$

The vertical effective stress in the soil at the trench bottom just below the slurry can be calculated as

$$2{,}070 - \text{pore water pressure} = 2{,}070 - 20 \times 62.4 = 822\,\text{psf}$$

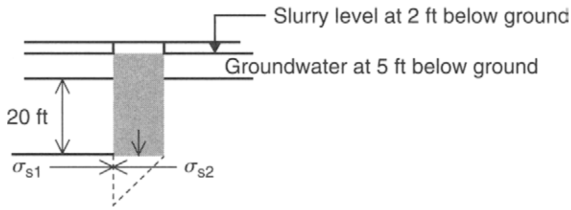


Figure 29.4 Trench bottom stability in a slurry cutoff wall

Note that the pore water pressure in the soil has to be reduced to obtain the vertical effective stress.

The horizontal stress due to the surrounding soil, σ_{s1}, necessary to push the trench bottom up must be greater than the passive pressure of the soil in the trench bottom underneath the soil. Next we calculate the passive stress of the soil underneath the slurry

$$\sigma_{s2} = K_p \times \text{vertical effective stress}$$

$$K_p = \tan^2(45 + \varphi/2) = 3.0$$

$$\sigma_{s2} = 3.0 \times 822 = 2{,}466\,\text{psf}$$

If σ_{s1} is greater than 2,466 psf, the trench bottom would be pushed up and would fail.

$$\sigma_{s1} = 578\,\text{psf}$$

$$\text{factor of safety against trench bottom failure} = 2{,}466/578 = 4.26$$

30

Pile Foundations

30.1 Introduction

Piles are used to transfer the load to deeper, more stable layers of soil. Large structures were built in Egypt, Asia, and Europe prior to the advent of piles. Ancient engineers had no choice but to dig deep down to the bedrock. The second century Sri Lankan structure known as Jethavana has a shallow foundation going 252 ft deep, right down to the bedrock. Today, without piles, large structures would be economically prohibitive. See Fig. 30.1.

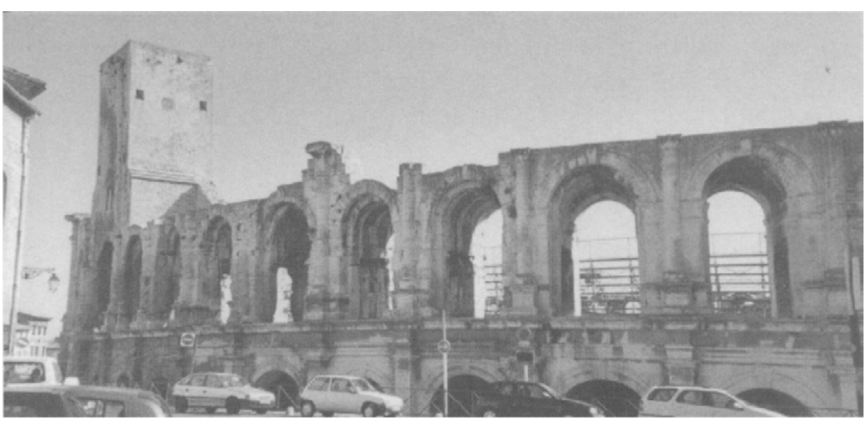
The first undisputed evidence of the use of piles is in the Arles circus arena built in 148 AD in France by Roman engineers. The wooden piles were probably driven by drop hammer mechanisms using human labor. See Fig. 30.2.

30.2 Pile Types

All piles can be categorized as displacement piles and nondisplacement piles. Timber piles, closed end steel pipe piles, and precast concrete piles displace the soil when driven into the ground. These piles are categorized as displacement piles.

Some piles displace soil during installation by a small degree (i.e., H-piles, open end steel tubes, hollow concrete piles).

Figure 30.1 Jethavana structure with a 252 ft deep shallow foundation



30.2.1 Displacement Piles

Displacement piles fall into two categories, large displacement piles and small displacement piles. The four main types of large displacement piles are timber piles, precast concrete piles (including reinforced and prestressed concrete piles), closed end steel pipe piles, and jacked down solid concrete piles.

The five main types of small displacement piles are tubular concrete piles, H-piles, open end pipe piles, thin shell type piles, and jacked down hollow concrete cylinders.

30.2.2 Nondisplacement Piles

Nondisplacement piles come in the following types.

- Steel casing is withdrawn after concreting (alpha piles, delta piles, Frankie piles, Vibrex piles).
- Continuous flight auger drilling and concrete placement (with or without reinforcements).
- Auguring a hole and placing a thin shell and concreting.
- Drilling or auguring a hole and placing concrete blocks inside the hole.

30.3 Timber Piles

To have a 100 ft long timber pile, one needs to cut down a tree of 150 ft or more. Timber piles are cheaper than steel or concrete piles. However, timber piles decay due to living microbes. Microbes need two ingredients to thrive: oxygen and moisture. For timber piles to decay, both oxygen and moisture must be present. Below the groundwater level, there is ample moisture but very little oxygen.

Timber piles submerged in groundwater will not decay. Oxygen is needed for the fungi (wood decaying microbes) to grow. Below groundwater level, there is no significant amount of air in the soil. For this reason, very little decay occurs below the groundwater level.

Moisture is the other ingredient needed for the fungi to survive. Above the groundwater level, there is ample oxygen.

In states such as Nevada, Arizona, and Texas, there is very little moisture above the groundwater level. Since microbes cannot survive without water, timber piles in these states can last for a long period of time.

This is not the case for states in the northern part of the United States. A significant amount of moisture will be present above groundwater level due to snow and rain. Hence, both oxygen and moisture are available for the fungi to thrive. Timber piles will decay under such conditions. Creosoting and other techniques should be used to protect timber from decay above groundwater level.

When the campanile of the Venetian church, San Marco, was demolished due to structural defects in 1902, the wood piles driven in 900 AD were in good condition. These old piles were reused to construct a new campanile in place of the old one. Venice is a city with a very high groundwater level, and the piles there have been underwater for centuries.

Engineers should consider possible future construction activities that could trigger lowering of the groundwater level. See Figs. 30.3 and 30.4.

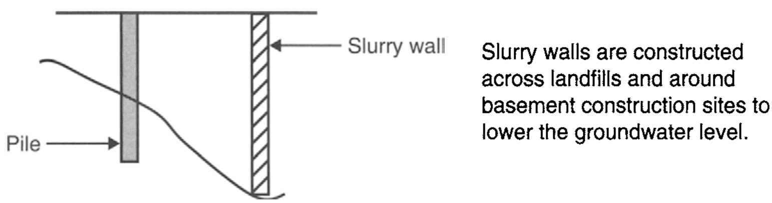


Figure 30.3 Slurry walls are constructed to lower the groundwater level

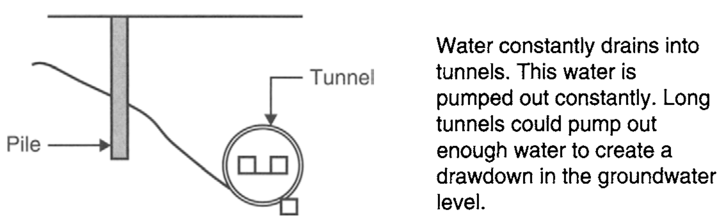


Figure 30.4 Water must be pumped out of tunnels, possibly enough to draw down the groundwater level

30.3.1 Timber Pile Decay: Biological Agents

There are many forms of biological agents that attack timber piles. Timber is an organic substance, and nature will not let any organic substance go to waste.

Fungi

Fungi belong to the plant kingdom. The main distinction of plants from animals is their ability to generate food on their own. On the other hand, all food types of animals come from plants. On that account, fungi differ from other plants. Fungi is not capable of generating its own food since it lacks chlorophyll, the agent that allows plants to generate organic matter using sunlight and inorganic nutrients. For this reason, fungi have to rely on organic matter for a supply of food.

Fungi need organic nutrients (the pile itself), water, and atmospheric air to survive.

Wood piles subjected to fungi attack can be identified by a swollen, rotted, and patched surface. Unfortunately, many piles lie underground and are not visible. For this reason, piles that could be subjected to environmental conditions favorable to fungi growth should be treated.

Marine Borers

Marine borers belong to two families, mollusks and crustaceans. Both groups live in seawater and brackish water. Waterfront structures usually need to be protected from these creatures. See Fig. 30.5.

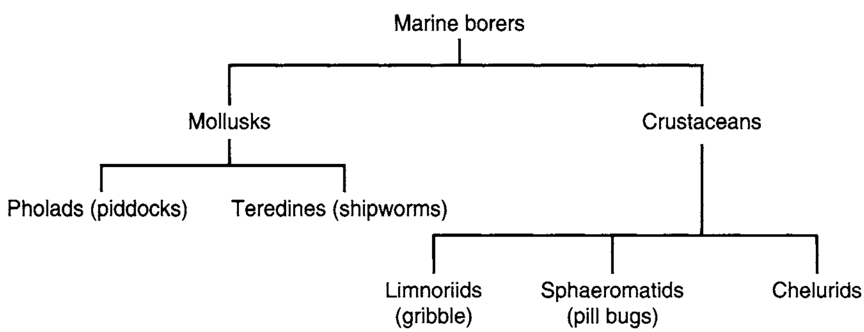


Figure 30.5 Species of marine borers

- **Pholads (Piddocks)** These are sturdy creatures that could penetrate the toughest of wood. Pholads can hide inside their shells for prolonged periods of time.
- **Teredines (Shipworms)** Teredines are commonly known as shipworms, due to their wormlike appearance. They typically enter the wood at the larval stage and grow inside the wood.
- **Limnoriids** Limnoria belong to the crustacean group. They have seven legs and can grow to a size of 6 mm. They are capable of digging deep into the pile and damage the pile from within, without any outward sign.

- **Sphaeromatids** Sphaeromatids create large size holes, of approximately 1/2 in. size, and they can devastate wood piles and other marine structures.
- **Chelurids** Chelurids are known to drive out limnoriids from their burrows and occupy them.

30.3.2 Preservation of Timber Piles

Three main types of wood preservatives are available.

1. Creosote.
2. Oil born preservatives.
3. Water born preservatives.

These preservatives are usually applied in accordance with the specifications of AWPA, the American Wood Preserver's Association.

Preservatives are usually applied under pressure. Hence the term "pressure treated" is used. The process of pressure treating is shown in Fig. 30.6. In Fig. 30.6a, the timber piles are arranged inside a sealed chamber. In Fig. 30.6b, the timber piles are subjected to a vacuum. Due to the vacuum, the moisture inside the piles is removed. In Fig. 30.6c, after applying the vacuum, the chamber is filled with preservatives. The preservative is subjected to high pressure until a prespecified volume of liquid is absorbed by the wood.

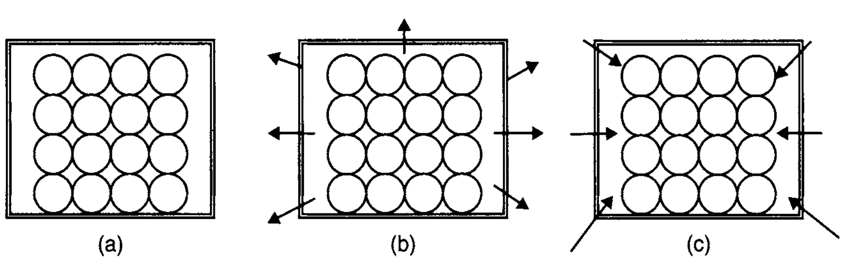


Figure 30.6 Creosoting of timber piles

30.3.3 Shotcrete Encasement of Timber Piles

Shotcrete is a mixture of cement, gravel, and water. High strength shotcrete is reinforced with fibers. Shotcrete is sprayed onto the top portion of the pile anywhere the pile could possibly be above groundwater level. See Fig. 30.7.

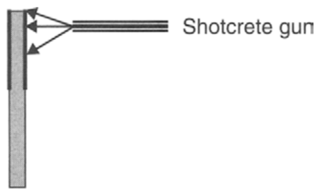


Figure 30.7 Shotcrete encasement

30.3.4 Timber Pile Installation

Timber piles need to be installed with special care. Timber piles are susceptible to brooming (crushing the fibers at the head of the pile) and damage. Any sudden decrease in driving resistance should be investigated.

30.3.5 Splicing of Timber Piles

Splicing of timber piles should be avoided if possible. Unlike steel or concrete piles, timber piles cannot be spliced effectively. The usual practice is to provide a pipe section (known as a sleeve) and bolt it to two piles. Figure 30.8 shows the splicing process. In Fig. 30.8a, the timber piles are tapered prior to splicing. In Fig. 30.8b, the sleeve (or the pipe section) is inserted. In Fig. 30.8c, the bottom pile is inserted. In Fig. 30.8d, the pipe section is bolted to the two piles.

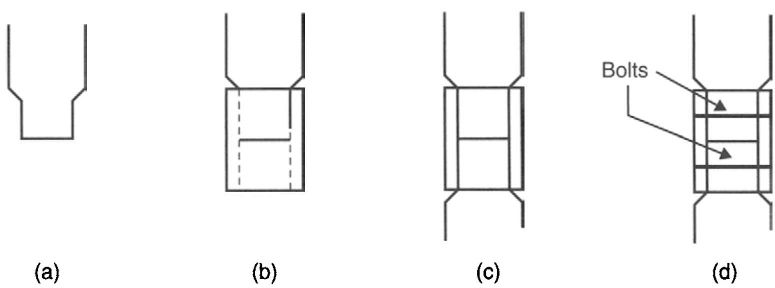


Figure 30.8 Splicing of timber piles

Sleeve joints are approximately 3 to 4 ft in length. The bending strength of the joint is much lower than the pile itself. The splice strength can be increased by increasing the length of the sleeve.

Most building codes require no splicing to be conducted on the upper 10 ft of the pile, since the pile is subjected to high bending stresses at upper levels. If splicing is absolutely required for timber piles in the upper 10 feet of the pile, it is recommended to construct a composite pile with the upper section filled with concrete. This type of construction is much better than splicing. See Fig. 30.9.

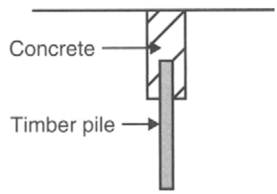


Figure 30.9 Concrete-timber composite pile

Sleeves larger than the pile may get torn and damaged during driving, and should be avoided. If this type of sleeves is to be used, the engineer should be certain that the pile sleeve would not be driven through hard stratums.

There is always a danger of uplift piles when splices are made. Timber splices are extremely vulnerable to uplift (tensile) forces and should be avoided. Other than sleeves, steel bars and straps are also used for splicing.

30.4 Steel H-Piles

Timber piles cannot be driven through hard ground. Steel H-piles are essentially end bearing piles. Due to limited perimeter area, H-piles cannot generate much frictional resistance.

Corrosion is a major problem for steel H-piles. Corrosion is controlled by adding copper into the steel. H-piles are easily spliced. H-piles are ideal for highly variable soil conditions. H-piles could bend under very hard ground conditions. This is known as "doglegging," and the pile installation supervisor needs to make sure that the piles are not out of plumb. H-piles can get plugged during the driving process. See Fig. 30.10. If the H-pile is plugged, end bearing would increase due to larger area. On the other hand, skin friction would become smaller due to the lesser wall area.

When H-piles are driven, both analyses should be done (unplugged and plugged) and the lower value should be used for the design. The following is a summary of unplugged and plugged H-piles.

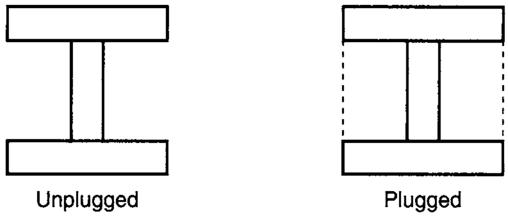


Figure 30.10 H-piles (plugging of soil)

- **Unplugged** Low end bearing, high skin friction.
- **Plugged** Low skin friction, high end bearing.

The splicing of H-piles is shown in Fig. 30.11.

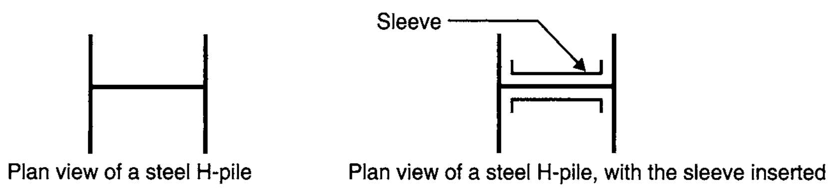


Figure 30.11 Splicing of H-piles

STEP 1: The sleeve is inserted into the bottom part of the H-pile, as shown in Fig. 30.11, and bolted to the web.
STEP 2: The top part of the pile is inserted into the sleeve and bolted.

30.4.1 Guidelines for Splicing (International Building Code)

The International Building Code (IBC) states that splices shall develop not less than 50% of the pile bending capacity. If the splice occurs in the upper 10 ft of the pile, an eccentricity of 3 in. should be assumed for the column load. The splice should be capable of withstanding the bending moment and shear forces due to a 3-in. eccentricity.

30.5 Pipe Piles

Pipe piles are available in many sizes, and 12-inch diameter pipe piles have a range of thicknesses. Pipe piles can be driven either open end or closed end. When driven open end, the pipe is cleaned with a jet of water.

30.5.1 Closed End Pipe Piles

Closed end pipe piles are constructed by covering the bottom of the pile with a steel plate. In most cases, pipe piles are filled with concrete. In some cases, pipe piles are not filled with concrete, to reduce the cost. If pipe piles were not filled with concrete, then a corrosion protection layer should be applied. If a concrete filled pipe pile is corroded, most of the load carrying capacity of the pile would remain intact due to concrete. On the other hand, empty pipe pile would lose a significant amount of its load carrying capacity. Pipe piles are a good candidate for batter piles.

The structural capacity of pipe piles is calculated based on concrete strength and steel strength. The thickness of the steel should be reduced to account for corrosion. (Typically reduced by 1/16 in. to account for corrosion). See Fig. 30.12.

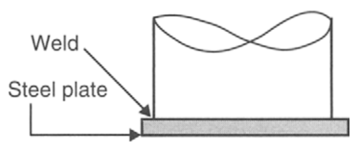


Figure 30.12 Closed end pipe pile

A pipe pile is covered with an end cap. The end cap is welded as shown in Fig. 30.13.

In the case of closed end driving, soil heave can occur. There are occasions where open end piles also generate soil heave. This is due to the plugging of the open end of the pile with soil. Pipe piles are cheaper than steel H-piles or concrete piles.

30.5.2 Open End Pipe Piles

Open end pipe piles are driven, and soil inside the pile is removed by a water jet. See Fig. 30.14. Open end pipe piles are easier to drive through hard soils than closed end pipe piles. See Fig. 30.15.

The ideal situations for open end pipe piles are as follows.

- Soft layer of soil followed by a dense layer of soil. See Fig. 30.16.
- Medium dense layer of soil followed by a dense layer of soil. See Fig. 30.17.

Telescoping piles are shown in Fig. 30.18. Due to the smaller diameter of the telescoping pipe pile, the end bearing capacity of the pile is

Figure 30.13 Pipe pile construction. (*Source*: http://www.dot.ca.gov)

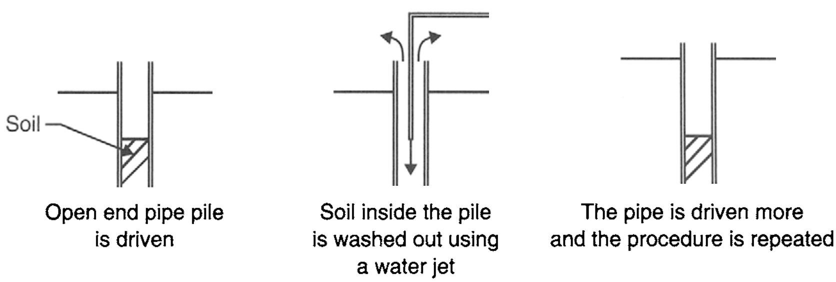


Figure 30.14 Driving of open end pipe piles

reduced in this type of pile. To accommodate the loss, the length of the telescoping pile should be increased.

30.5.3 Splicing of Pipe Piles

Pipe piles are spliced by fitting a sleeve. The sleeve would fit into the bottom section of the pile as well as the top section. See Fig. 30.19.

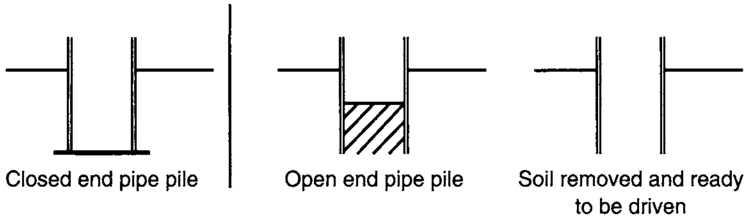
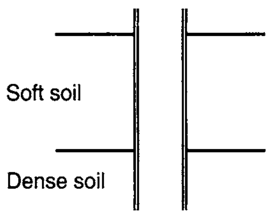
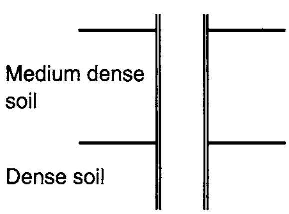


Figure 30.15 Driving of closed end pipe piles



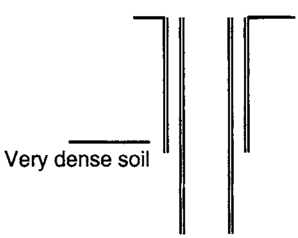
- Site condition: Soft layer of soil followed by a hard layer of soil.
- Open end pipe piles are ideal for this situation. After driving to the desired depth, soil inside the pipe is removed and concreted.
- Closed end pipe piles also could be considered to be ideal for this type of situation.

Figure 30.16 Pile in soft soil followed by dense soil



- Site condition: Medium dense layer of soil followed by a hard layer of soil.
- Open end pipe piles are ideal for this situation.
- Closed end pipe piles may not be a good choice, since driving closed end pipe piles through medium dense soil layer may be problematic.
- It is easier to drive open end pipe piles through a dense soil layer than closed end pipe piles.

Figure 30.17 Pile in medium dense soil underlain by dense soil



- Usually telescoping is conducted, when very dense soil is encountered.
- In such situations, it may not be possible to drive a larger pipe pile.
- Hence, a small diameter pipe pile is driven inside the original pipe pile.

Figure 30.18 Telescoping to improve driving ability

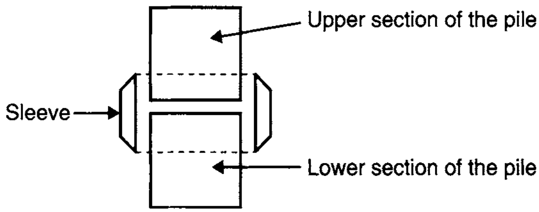


Figure 30.19 Splicing of pipe piles

30.6 Precast Concrete Piles

Precast concrete piles could be either reinforced concrete piles or prestressed concrete piles. See Fig. 30.20.

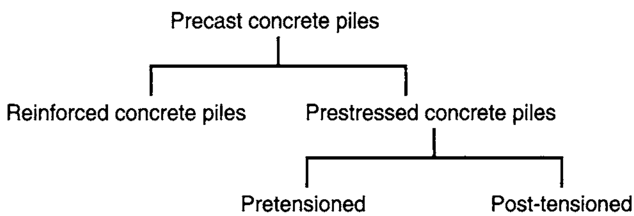


Figure 30.20 Precast concrete piles

30.7 Reinforced Concrete Piles

Reinforced concrete piles are constructed by reinforcing the concrete, as shown in Fig. 30.21.

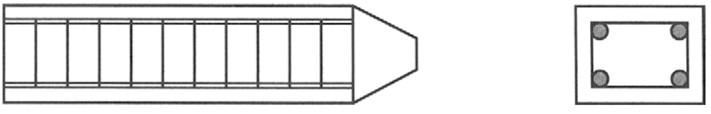


Figure 30.21 Reinforced concrete piles

30.8 Prestressed Concrete Piles

The pretensioning procedure for prestressed concrete piles is shown in Fig. 30.22. The post-tensioning procedure for prestressed concrete piles is shown in Fig. 30.23. The IBC specifies a minimum lateral dimension (diameter or width) of 8 in. for precast concrete piles.

Figure 30.22 Prestressed concrete piles

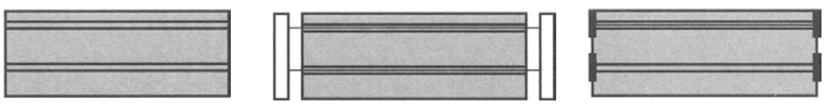


Figure 30.23 Post-tensioned concrete piles

30.8.1 Reinforcements for Precast Concrete Piles

As per the IBC, longitudinal reinforcements should be arranged in a symmetrical manner. Lateral ties should be placed every 4 to 6 in.

30.8.2 Concrete Strength (IBC)

As per IBC, the 28 day concrete strength (f_c') should be not less than 3,000 psi.

30.8.3 Hollow Tubular Section Concrete Piles

Most hollow tubular piles are post-tensioned to withstand tensile stresses. Hollow tubular concrete piles can be driven either closed end or open end. A cap is fitted at the end for closed end driving. These piles are not suitable for dense soils. In addition, in hollow tubular section concrete piles, splicing is expensive.

It is a difficult and expensive process to cut off these piles. It is very important to know the depth to the bearing stratum with reasonable accuracy.

30.9 Driven Cast-in-Place Concrete Piles

STEP 1: Steel tube is driven first.
STEP 2: Soil inside the tube is removed by water jetting.
STEP 3: The reinforcement cage is set inside the casing.
STEP 4: The empty space inside the tube is concreted while removing the casing.

30.10 Selection of Pile Type

Once the geotechnical engineer has decided to use piles, the next question is, which piles? There are many types of piles available, as discussed earlier. Timber piles are cheap, but difficult to install in hard soil. Steel piles may not be good in marine environments, due to corrosion.

Some possible scenarios are discussed below.

CASE 1: Granular soil with boulders underlain by medium stiff clay. See Fig. 30.24.

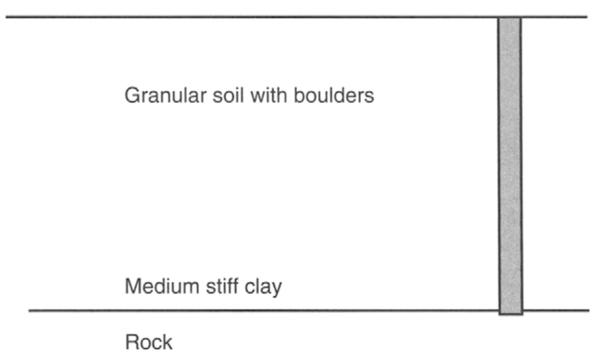


Figure 30.24 Pile in multilayer soil strata

Timber piles are not suitable for this situation, due to the existence of boulders in the upper layers. The obvious choice is to drive the piles all the way to the rock. The piles could be designed as end bearing piles.

If the decision was taken to drive piles to the rock for the configuration in Fig. 30.24, H-piles will be ideal. Unlike timber piles or pipe piles, H-piles can go through boulders.

On the other hand, the rock may be too far for the piles to be driven. If the rock is too deep for piles, a number of alternatives can be envisioned.

- Drive large diameter pipe piles and place them in medium stiff clay.
- Construct a caisson and place it in the medium stiff clay.

In the first case, driving large diameter pipe piles through boulders could be problematic. It is possible to excavate and remove boulders if the boulders are mostly at shallower depths.

In the case of H-piles, one has to be extra careful not to damage the piles.

Caissons placed in the medium stiff clay are a good alternative. The settlement of caissons needs to be computed.

CASE 2: Granular soil with boulders, underlain by medium stiff clay, further underlain by soft clay.
See Fig. 30.25.

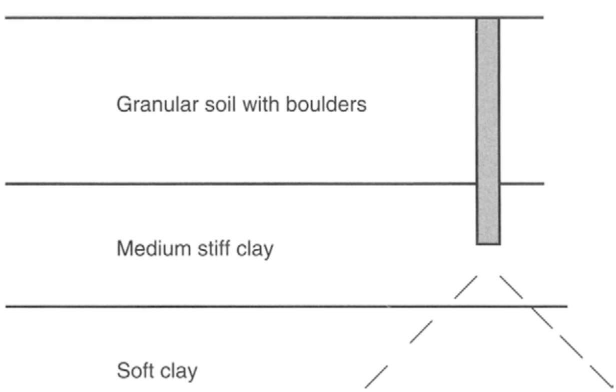


Figure 30.25 Pressure distribution in a pile

Piles cannot be placed in soft clay. It is possible to place piles in the medium stiff clay. In that situation, the piles need to be designed as friction piles. The pile capacity mainly comes from end bearing and skin friction. End bearing piles, as the name indicates, obtain their capacity mainly from the end bearing. On the other hand, friction piles obtain their capacity from skin friction.

If the piles were placed in the medium stiff clay, stresses would reach the soft clay layer below. The engineer needs to make sure that settlement due to compression of soft clay is within acceptable limits.

CASE 3: Soft clay with high groundwater, underlain by medium stiff clay, underlain by soft clay.
See Fig. 30.26.

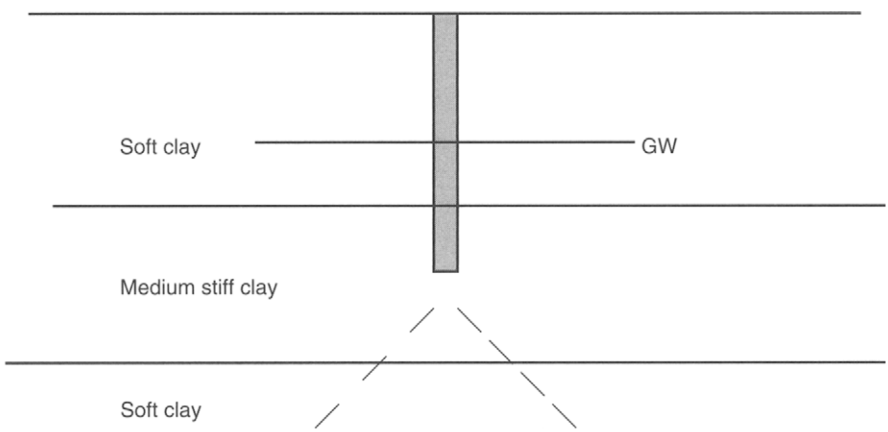


Figure 30.26 Pile with groundwater effects considered

Timber piles could be placed in the medium stiff clay. In this situation, the soft clay in upper layers may not pose a problem for driving. Where the timber piles are above groundwater, they should be protected.

Pipe piles also could be used in this situation. Pipe piles cost more than timber piles. One of the main advantages of pipe piles over timber piles is that they can be driven hard. At the same time, large diameter pipe piles may be readily available, and higher loads can be accommodated by fewer piles.

H-piles are ideal candidates for end bearing piles, whereas the Case 3 situation calls for friction piles. Hence, H-piles may not be suitable for the Case 3 situation.

31

Pile Design in Sandy Soils

A modified version of the Terzaghi bearing capacity equation is widely used for pile design. The third term, or the density term, in the Terzaghi bearing capacity equation is negligible in piles and hence usually ignored. The lateral earth pressure coefficient, K, is introduced to compute the skin friction of piles.

$$P_{\text{ultimate}} = \underbrace{(\sigma'_t \times N_q \times A)}_{\text{End bearing term}} + \underbrace{(K \times \sigma'_v \times \tan\delta \times A_p)}_{\text{Skin friction term}}$$

where

P_{ultimate} = ultimate pile capacity
σ'_t = effective stress at the tip of the pile
N_q = bearing factor coefficient
A = cross sectional area of the pile at the tip
K = lateral earth pressure coefficient (Table 31.4, covered later, will be used for K values)
σ'_v = effective stress at the perimeter of the pile (σ'_v varies with depth; usually, the σ'_v value at the midpoint of the pile is obtained)
$\tan\delta$ = friction angle between pile and soil (Table 31.3, covered later, will be used for δ values)
A_p = perimeter area of the pile

For round piles,

$$A_p = (\pi \times d) \times L$$

where

d = diameter
L = length of the pile

31.1 Description of Terms

31.1.1 Effective Stress, σ'

When a pile is driven, the effective stress of the existing soil around and below the pile changes. Figure 31.1 shows the effective stress prior to driving a pile.

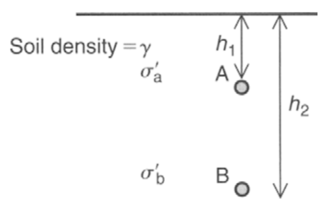


Figure 31.1 Effective stress at different depths

$$\sigma'_a = \gamma \times h_1$$
$$\sigma'_b = \gamma \times h_2$$

The effective stress after driving the pile is shown in Fig. 31.2. When a pile is driven, the soil around the pile and below the pile is compacted.

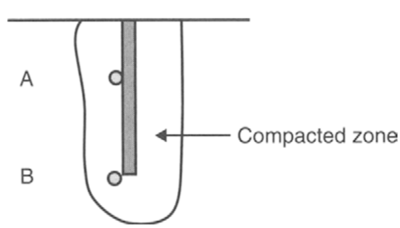


Figure 31.2 Effective stress near a pile

Due to the disturbance of soil during the pile driving process, σ'_{aa} and σ'_{bb} cannot be accurately computed. Usually, an increase in effective stress due to pile driving is ignored.

31.1.2 Bearing Capacity Factor, N_q

Many researchers have provided techniques to compute bearing capacity factors. The end bearing capacity is a function of friction angle, dilatancy of soil, and relative density. All these parameters are lumped into the bearing capacity factor, N_q. Different methods of obtaining the N_q value will be discussed.

31.1.3 Lateral Earth Pressure Coefficient, K

Prior to discussing the lateral earth pressure coefficient related to piles, it is necessary to investigate lateral earth pressure coefficients in general. See Fig. 31.3.

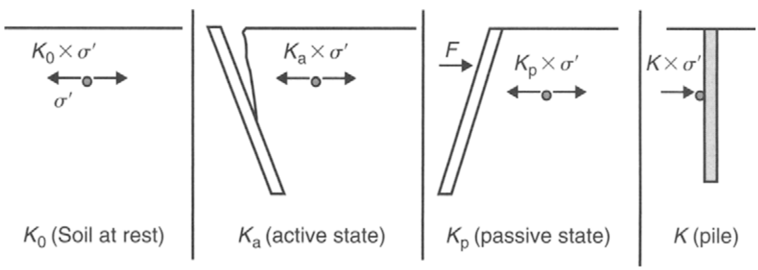


Figure 31.3 Lateral earth pressure coefficients

31.1.4 In Situ Soil Condition, K_0

For the at rest condition, the horizontal effective stress is given by $K_0 \times \sigma'$. K_0 is the lateral earth pressure coefficient at rest, and σ' is the vertical effective stress.

31.1.5 Active Condition, K_a

In the active condition, the soil is exerting the minimum horizontal effective stress, since soil particles have room to move. K_a is the active earth pressure coefficient. K_a is always smaller than K_0.

31.1.6 Passive Condition, K_p

In the passive condition, soil exerts the maximum horizontal effective stress, since the soil particles have been compressed. K_p is the passive earth pressure coefficient. K_p is always greater than K_0 and K_a.

31.1.7 Soil Near Piles, K

Soil near a driven pile is compressed. In this case, soil is definitely exerting more horizontal pressure than the in situ horizontal effective stress, K_0. Since K_p is the maximum horizontal stress that can be achieved, K should be between K_0 and K_p.

$$K_a < K_0 < K(\text{pile condition}) < K_p$$

Hence $K = (K_0 + K_a + K_p)/3$ can be used as an approximation.
Equations for K_0, K_a, and K_p are

$$K_0 = 1 - \sin\varphi$$
$$K_a = 1 - \tan^2(45 - \varphi/2)$$
$$K_p = 1 + \tan^2(45 + \varphi/2)$$

31.1.8 Wall Friction Angle, Tan δ

The friction angle between the pile material and the soil, δ, decides the skin friction. This friction angle varies with the pile material and soil type. Various agencies have conducted laboratory tests and have published δ values for different pile materials and soils.

31.1.9 Perimeter Surface Area of Piles, A_p

See Fig. 31.4.

$$\text{perimeter surface area of a circular pile} = \pi \times D \times L$$
$$\text{perimeter surface area of a rectangular pile} = A \times L$$

Skin friction acts on the perimeter surface area of the pile.

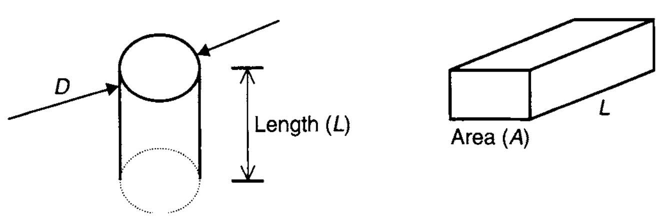


Figure 31.4 Perimeter surface area of piles

31.2 Equations for End Bearing Capacity in Sandy Soils

There are number of methods available to compute the end bearing capacity of piles in sandy soils.

31.2.1 API Method

The API method (American Petroleum Institute, 1984) uses the following equation for end bearing capacity.

$$q = N_{\mathrm{q}} \times \sigma'_{\mathrm{t}}$$

where

q = end bearing capacity of the pile (units same as for σ'_{t})
σ'_{t} = effective stress at pile tip

The maximum effective stress allowed for the computation is 240 kPa or 5.0 ksf. The value of N_{q} depends on the soil.

N_{q} = 8 to 12 for loose sand
N_{q} = 12 to 40 for medium dense sand
N_{q} = 40 for dense sand

The sand consistency (loose, medium, or dense) can be obtained from Table 31.2, covered below.

31.2.2 Martin et al. (1987)

Martin et al. (1987) have put forth two equations for end bearing capacity.

$$\text{SI units: } q = c \times N(\mathrm{MN/m^2})$$

$$\text{fps units: } q = 20.88 \times c \times N(\mathrm{ksf})$$

q = end bearing capacity of the pile

N = SPT value at pile tip (blows per foot)

c = 0.45 (for pure sand)

c = 0.35 (for silty sand)

Design Example 31.1

The SPT(N) value at the pile tip is 10 blows per foot. Find the ultimate end bearing capacity of the pile assuming that pile tip is in pure sand and the diameter of the pile is 1 ft.

Solution

$$\text{fps units: } q = 20.88 \times c \times N(\text{ksf})$$

$$q = 20.88 \times 0.45 \times 10 = 94\,\text{ksf}$$

$$c = 0.45 \text{ (for pure sand)}$$

$$\text{total end bearing capacity} = q \times \text{area}$$

$$\text{total end bearing capacity} = q \times \pi \times (d^2)/4$$

$$= 74\text{ kip} = 37\text{ tons } (329\,\text{kN})$$

31.2.3 NAVFAC DM 7.2 (1984)

$$q = \sigma'_t \times N_q$$

where

q = end bearing capacity of the pile (units same as σ'_v)
σ'_t = effective stress at pile tip

31.2.4 Bearing Capacity Factor, N_q

Table 31.1 shows that the N_q value is lower in bored piles. This is expected. During pile driving, the soil just below the pile tip becomes compacted. Hence, it is reasonable to assume a higher N_q value for driven piles.

Table 31.1 Friction angle vs. N_q

φ	26	28	30	31	32	33	34	35	36	37	38	39	40
N_q (for driven piles)	10	15	21	24	29	35	42	50	62	77	86	12	145
N_q (for bored piles)	5	8	10	12	14	17	21	25	30	38	43	60	72

Source: NAVFAC DM 7.2 (1984).

If water jetting is used, φ should be limited to 28°. This is due to the fact that water jets tend to loosen the soil. Hence, higher friction angle values are not warranted.

31.2.5 Kulhawy (1984)

Kulhawy reported the values from Table 31.2 for the end bearing capacity using pile load test data.

Table 31.2 Depth and end bearing capacity of piles in different soils

Depth		Saturated loose sand		Dry loose sand		Saturated very dense sand		Dry very dense sand	
ft	m	tsf	MN/m^2	tsf	MN/m^2	tsf	MN/m^2	tsf	MN/m^2
20	6.1	10	0.95	50	4.8	60	5.7	140	13.4
40	12.2	25	2.4	60	5.7	110	10.5	200	19.2
60	18.3	40	3.8	70	6.7	160	15.3	250	23.9
80	24.4	50	4.8	90	8.6	200	19.2	300	28.7
100	30.5	55	5.3	110	10.5	230	22.0	370	35.4

Source: Kulhawy (1984).

Design Example 31.2

Find the end bearing capacity of a 3 ft (1 m) diameter caisson using the Kulhawy values from Table 31.2. Assume the soil is saturated at the tip level of the caisson and the average SPT (N) value at the tip level is 10 blows/ft. See Fig. 31.5.

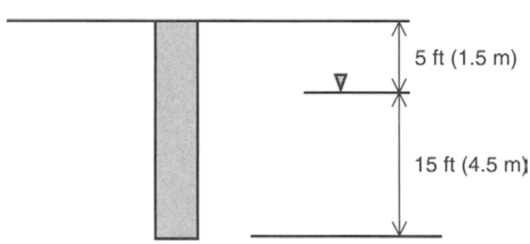


Figure 31.5 End bearing capacity in sand

Solution

An average N value of 10 blows per foot can be considered as loose sand. Hence, the soil below the pile tip can be considered to be loose sand. The depth to the bottom of the pile is 20 ft.

From Table 31.2, the ultimate end bearing capacity for saturated loose sand at 20 ft is 10 tsf (0.95 MN/m^2).

$$\text{cross sectional area of the caisson} = (\pi \times D^2/4) = 7.07\,\text{sq ft}\ (0.785\,\text{m}^2)$$

$$\text{ultimate end bearing capacity} = \text{area} \times 10\,\text{tons} = 70.7\,\text{tons}\ (0.74\,\text{MN})$$

It should be pointed out that some of the values in Table 31.2 provided by Kulhawy are very high. For example, a caisson placed in very dense dry soil at 20 ft gives 140 tsf.

It is recommended to use other methods to check the values obtained using tables provided by Kulhawy.

31.3 Equations for Skin Friction in Sandy Soils

Numerous techniques have been proposed to compute the skin friction in piles in sandy soils. These different methodologies are briefly discussed in this chapter.

31.3.1 McClelland (1974): Driven Piles

McClelland (1974) suggested the following equation for driven piles.

$$S = \beta \times \sigma'_v \times A_p$$

where

S = skin friction
σ'_v = effective stress at midpoint of the pile
S = total skin friction
σ'_v changes along the length of the pile
Hence, σ'_v should be taken at the midpoint of the pile.
A_p = perimeter surface area of the pile
See Fig. 31.6.

$$\text{perimeter surface area of a circular pile, } A_p = \pi \times D \times L$$

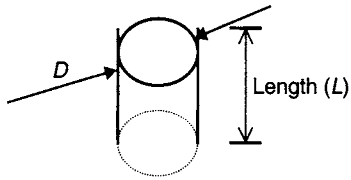


Figure 31.6 Skin friction acting on perimeter of the pile

where

$\beta = 0.15$ to 0.35 for compression
$\beta = 0.10$ to 0.25 for tension (for uplift piles)

31.3.2 Meyerhoff (1976): Driven Piles

Meyerhoff (1976) suggested the following equation for driven piles.

$$S = \beta \times \sigma'_{\mathrm{V}} \times A_{\mathrm{P}}$$

where

S = skin friction
σ'_{V} = effective stress at the midpoint of the pile
A_{P} = perimeter surface area of the pile
$\beta = 0.44$ for $\varphi' = 28°$
$\beta = 0.75$ for $\varphi' = 35°$
$\beta = 1.2$ for $\varphi' = 37°$

31.3.3 Meyerhoff (1976): Bored Piles

Meyerhoff (1976) suggested following equation for bored piles.

$$S = \beta \times \sigma'_{\mathrm{V}} \times A_{\mathrm{P}}$$

where

S = skin friction of the pile
σ'_{V} = effective stress at the midpoint of the pile
A_{P} = perimeter surface area of the pile
$\beta = 0.10$ for $\varphi = 33°$
$\beta = 0.20$ for $\varphi = 35°$
$\beta = 0.35$ for $\varphi = 37°$

31.3.4 Kraft and Lyons (1974)

Kraft and Lyons (1974) suggested the following.

$$S = \beta \times \sigma'_v \times A_p$$

where

S = skin friction of the pile
σ'_v = effective stress at the midpoint of the pile

$$\beta = c \times \tan(\varphi - 5)$$

where

$c = 0.7$ for compression
$c = 0.5$ for tension (uplift piles)

31.3.5 NAVFAC DM 7.2 (1984)

NAVFAC DM 7.2 (1984) suggests the following.

$$S = K \times \sigma'_v \times \tan\delta \times A_p$$

S = skin friction of the pile
σ'_v = effective stress at the midpoint of the pile

where

K = lateral earth pressure coefficient
δ = pile skin friction angle

31.3.6 Pile Skin Friction Angle, δ

See Table 31.3. δ is the skin friction angle between pile material and surrounding sandy soils. Usually, smooth surfaces tend to have less skin friction compared to rough surfaces.

31.3.7 Lateral Earth Pressure Coefficient, K

See Table 31.4. The lateral earth pressure coefficient is lower in uplift piles compared to regular piles. Tapered piles tend to have the highest K value.

Table 31.3 Pile type and pile skin friction angle

Pile type	δ
Steel piles	20°
Timber piles	3/4 φ
Concrete piles	3/4 φ

Source: NAVFAC DM 7.2 (1984).

Table 31.4 Pile type and lateral earth pressure coefficient

Pile type	K (piles under compression)	K (piles under tension, uplift piles)
Driven H-piles	0.5–1.0	0.3–0.5
Driven displacement piles (round and square)	1.0–1.5	0.6–1.0
Driven displacement tapered piles	1.5–2.0	1.0–1.3
Driven jetted piles	0.4–0.9	0.3–0.6
Bored piles (less than 24 in. diameter)	0.7	0.4

Source: NAVFAC DM 7.2 (1984).

31.3.8 Average *K* Method

The earth pressure coefficient K can be averaged from K_a, K_p, and K_0.

$$K = (K_0 + K_a + K_p)/3$$

The equations for K_0, K_a, and K_p are as follows.

$$K_0 = 1 - \sin\varphi \quad \text{(earth pressure coefficient at rest)}$$

$$K_a = 1 - \tan^2(45 - \varphi/2) \quad \text{(active earth pressure coefficient)}$$

$$K_p = 1 + \tan^2(45 + \varphi/2) \quad \text{(passive earth pressure coefficient)}$$

Design Example 31.3

This example covers a single pile in a uniform sand layer for the case where no groundwater is present. The diameter of the round steel pipe pile is 0.5 m (1.64 ft), and it is 10 m (32.8 ft) long. The pile is driven into a sandy soil stratum as shown in Fig. 31.7. Compute the ultimate bearing capacity of the pile.

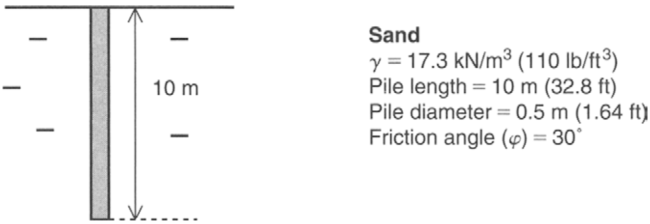


Figure 31.7 Single pile in sandy soil

Solution

STEP 1: Compute the end bearing capacity.

$$Q_{\text{ultimate}} = (\sigma_t' \times N_q \times A) + [K \times \sigma_p' \times \tan\delta \times (\pi \times d) \times L]$$

End bearing term Skin friction term

where

end bearing term $= \sigma_t' \times N_q \times A$

$$\sigma_t' = \text{effective stress at the tip of the pile}$$

$$\sigma_t' = \gamma \times \text{depth to the tip of the pile}$$

$$= 17.3 \times 10 = 173\,\text{kN/m}^2\ (3{,}610\,\text{psf})$$

Find N_q using Table 31.1. For a friction angle of 30°, $N_q = 21$ for driven piles.

$$\text{end bearing capacity} = \sigma_t' \times N_q \times A$$

$$= 173 \times 21 \times (\pi \times d^2/4) = 713.3\,\text{kN}\ (160\,\text{kip})$$

STEP 2: Compute the skin friction.

$$\text{skin friction term} = K \times \sigma_p' \times \tan\delta \times (\pi \times d) \times L$$

Obtain the K value.

From Table 31.4, for driven round piles, the K value lies between 1.0 and 1.5. Hence, assume $K = 1.25$. Obtain the σ'_p (effective stress at the perimeter of the pile). The effective stress along the perimeter of the pile varies with the depth. Hence, obtain the σ'_p value at the midpoint of the pile. The pile is 10 m long. Hence, use the effective stress at 5 m below the ground surface.

$$\sigma'_p(\text{midpoint}) = 5 \times \gamma = 5 \times 17.3 = 86.5\,\text{kN/m}^2 (1.8\,\text{ksf})$$

Obtain the skin friction angle, δ. From Table 31.3, the skin friction angle for steel piles is 20°. Find the skin friction of the pile.

$$\begin{aligned}
\text{skin friction} &= K \times \sigma'_p \times \tan\delta \times (\pi \times d) \times L \\
&= 1.25 \times 86.5 \times (\tan 20^\circ) \times (\pi \times 0.5) \times 10 \\
&= 618.2\,\text{kN } (139\,\text{kip})
\end{aligned}$$

STEP 3: Compute the ultimate bearing capacity of the pile.

$$\begin{aligned}
Q_{\text{ultimate}} &= \text{ultimate bearing capacity of the pile} \\
Q_{\text{ultimate}} &= \text{end bearing capacity} + \text{skin friction} \\
Q_{\text{ultimate}} &= 713.3 + 618.2 = 1{,}331.4\,\text{kN } (300\,\text{kip})
\end{aligned}$$

Assume a factor of safety of 3.0. Hence, the allowable bearing capacity of the pile can be calculated as

$$\begin{aligned}
Q_{\text{ultimate}}/\text{F.O.S.} &= 1{,}331.4/3.0 \\
\text{allowable pile capacity} &= 443.8\,\text{kN } (99.8\,\text{kip})
\end{aligned}$$

Note that 1 kN is equal to 0.225 kip. Hence, the allowable capacity of the pile is 99.8 kip.

Design Example 31.4

This example concerns a single pile in a uniform sand layer, for the case where there is groundwater present. A 0.5 m diameter, 10 m long round concrete pile is driven into a sandy soil stratum as shown in Fig. 31.8. The groundwater level is located 3 m below the surface. Compute the ultimate bearing capacity of the pile.

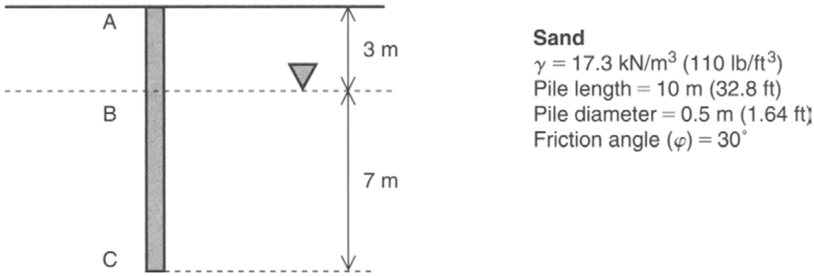


Figure 31.8 Single pile in sand (groundwater considered)

Solution

STEP 1: Compute the end bearing capacity.

$$Q_{\text{ultimate}} = \underbrace{(\sigma'_t \times N_q \times A)}_{\text{End bearing term}} + \underbrace{[K \times \sigma'_v \times \tan\delta \times (\pi \times d) \times L]}_{\text{Skin friction term}}$$

$$\text{end bearing term} = \sigma'_t \times N_q \times A$$

where

σ'_t = effective stress at the tip of the pile

$$\sigma'_t = 17.3 \times 3 + (17.3 - \gamma_w) \times 7$$

where

γ_w = density of water = 9.8 kN/m^3

$$\sigma'_t = 17.3 \times 3 + (17.3 - 9.8) \times 7 = 104.4\,\text{kN/m}^2\ (2{,}180\,\text{psf})$$

Find N_q using Table 31.1. For a friction angle of 30°, $N_q = 21$ for driven piles.

$$\begin{aligned}\text{end bearing capacity} &= \sigma'_t \times N_q \times A\\ &= 104.4 \times 21 \times (\pi \times 0.5^2/4)\\ &= 403.5\,\text{kN (96.8 kip)}\end{aligned}$$

STEP 2: Compute the skin friction (A to B).

The computation of skin friction has to be done in two parts. First find the skin friction from A to B and then find the skin friction from B to C.

$$\text{skin friction term} = K \times \sigma'_v \times \tan\delta \times (\pi \times d) \times L$$

Obtain the K value. From Table 31.4, for driven round piles, the K value lies between 1.0 and 1.5. Hence, assume that $K = 1.25$.

Obtain the σ'_v (effective stress at the perimeter of the pile). The effective stress along the perimeter of the pile varies with the depth. Hence, obtain the σ'_p value at the midpoint of the pile from point A to B. The length of the pile section from A to B is 3 m. Hence, find the effective stress at 1.5 m below the ground surface.

Obtain the skin friction angle, δ. From Table 31.3, the skin friction angle for steel piles is 20°.

$$\text{skin friction} = K \times \sigma'_v \times \tan\delta \times (\pi \times d) \times L$$

where

σ'_v = effective stress at midpoint of section A to B (1.5 m below the surface)

$$\sigma'_v = 1.5 \times 17.3 = 25.9\,\text{kN/m}^2\ (0.54\,\text{ksf})$$

And since

$$K = 1.25$$
$$\delta = 20^\circ$$

The skin friction (A to B) can be calculated as

$$\begin{aligned}\text{skin friction (A to B)} &= 1.25 \times (25.9) \times (\tan 20^\circ) \times (\pi \times 0.5) \times 3 \\ &= 55.5\,\text{kN (12.5 kip)}\end{aligned}$$

Find the skin friction of the pile from B to C.

$$\text{skin friction} = K \times \sigma'_v \times \tan\delta \times (\pi \times d) \times L$$

$$\sigma'_v = \text{effective stress at midpoint of section B to C}$$
$$= 3 \times 17.3 + (17.3 - 9.8) \times 3.5$$
$$= 78.2\,\text{kN/m}^2\ (1,633\,\text{psf})$$

Since

$$K = 1.25$$
$$\delta = 20^\circ$$

The skin friction (B to C) can be calculated as

$$\text{skin friction (B to C)} = 1.25 \times (78.2) \times \tan(20^\circ) \times (\pi \times 0.5) \times 7$$
$$= 391\,\text{kN}\ (87.9\,\text{kip})$$

STEP 3: Compute the ultimate bearing capacity of the pile.

$$Q_{\text{ultimate}} = \text{ultimate bearing capacity of the pile}$$
$$Q_{\text{ultimate}} = \text{end bearing capacity} + \text{skin friction}$$
$$Q_{\text{ultimate}} = 403.5 + 55.5 + 391 = 850\,\text{kN}$$

Assume a factor of safety of 3.0. Hence, the allowable bearing capacity of the pile can be calculated as

$$Q_{\text{ultimate}}/\text{F.O.S.} = 850.1/3.0 = 283\,\text{kN}$$
$$\text{allowable pile capacity} = 283\,\text{kN}\ (63.6\,\text{kip})$$

Note that 1 kN is equal to 0.225 kip.

Design Example 31.5

This example concerns multiple sand layers with no groundwater present. A 0.5 m diameter, 12 m long round concrete pile is driven into a sandy soil stratum as shown in Fig. 31.9. Compute the ultimate bearing capacity of the pile.

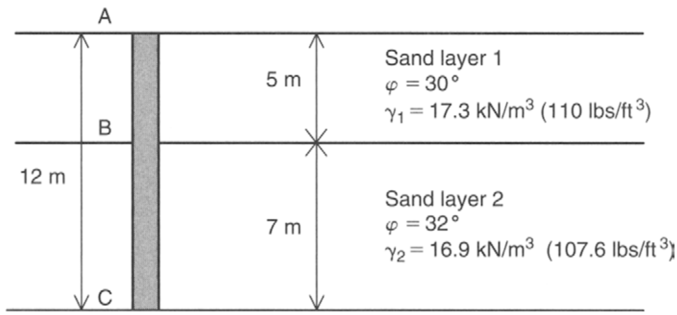


Figure 31.9 Single pile in multiple sand layers

Solution

STEP 1: Compute the end bearing capacity.

$$Q_{\text{ultimate}} = (\sigma'_t \times N_q \times A) + (K \times \sigma'_v \times \tan\delta \times A_p)$$

End bearing term Skin friction term

$$\text{end bearing term} = \sigma'_t \times N_q \times A$$

where

σ'_t = effective stress at the tip of the pile

$$\sigma'_t = \gamma_1 \times 5 + \gamma_2 \times 7$$
$$\sigma'_t = 17.3 \times 5 + 16.9 \times 7$$
$$= 204.8\,\text{kN/m}^2\ (4.28\,\text{ksf})$$

Find N_q using Table 31.1. Use the friction angle of the soil where the pile tip rests. For a friction angle of 32°, $N_q = 29$ for driven piles. The diameter of the pile is given as 0.5 m.

The φ value of the bottom sand layer is used to find N_q, since the tip of the pile lies on the bottom sand layer.

$$\text{end bearing capacity} = \sigma'_t \times N_q \times A$$
$$= 204.8 \times 29 \times (\pi \times d^2/4) = 1{,}166.2\,\text{kN}\ (262.2\,\text{kip})$$

STEP 2: Compute the skin friction.

The skin friction of the pile needs to be done in two parts.

- Skin friction of the pile portion in sand layer 1 (A to B)
- Skin friction of the pile portion in sand layer 2 (B to C)

Compute the skin friction of the pile portion in sand layer 1 (A to B).

$$\text{skin friction term} = K \times \sigma'_p \times \tan\delta \times A_p$$

$$A_p = (\pi \times d) \times L$$

Obtain the K value. From Table 31.4, for driven round piles, the K value lies between 1.0 and 1.5. Hence, assume $K = 1.25$.

Obtain the σ'_v (average effective stress at the perimeter of the pile). Obtain the σ'_v value at the midpoint of the pile in sand layer 1.

$$\begin{aligned}\sigma'_v(\text{midpoint}) &= 2.5 \times (\gamma_1)\\ &= 2.5 \times 17.3\,\text{kN/m}^2\\ &= 43.3\,\text{kN/m}^2\ (0.9\,\text{ksf})\end{aligned}$$

Obtain the skin friction angle, δ. From Table 31.3, the skin friction angle, δ, for concrete piles is $3/4\ \varphi$.

$$\delta = 3/4 \times 30° = 22.5°$$

This is true because the friction angle of layer 1 is 30°.

$$\begin{aligned}\text{skin friction in sand layer 1} &= K \times \sigma'_v \times \tan\delta \times (\pi \times d) \times L\\ &= 1.25 \times 43.3 \times (\tan 22.5°) \times (\pi \times 0.5) \times 5\\ &= 176.1\,\text{kN}\ (39.6\,\text{kip})\end{aligned}$$

STEP 3: Find the skin friction of the pile portion in sand layer 2 (B to C).

$$\text{skin friction term} = K \times \sigma'_v \times \tan\delta \times (\pi \times d) \times L$$

Obtain the σ'_v (effective stress at the perimeter of the pile). Obtain the σ'_v value at the midpoint of the pile in sand layer 2.

$$\begin{aligned}\sigma'_v(\text{midpoint}) &= 5 \times \gamma_1 + 3.5 \times \gamma_2\\ &= 5 \times 17.3 + 3.5 \times 16.9\,\text{kN/m}^2\\ &= 145.7\,\text{kN/m}^2\ (3{,}043\,\text{psf})\end{aligned}$$

Obtain the skin friction angle, δ.

From Table 31.3, the skin friction angle, δ, for concrete piles is $3/4\,\varphi$.

$$\delta = 3/4 \times 32^\circ = 24^\circ$$

This is true because the friction angle of layer 2 is 32°.

$$\begin{aligned}\text{skin friction in sand layer 2} &= K \times \sigma'_p \times \tan\delta \times (\pi \times d) \times L\\ &= 1.25 \times 145.7 \times (\tan 24^\circ) \times (\pi \times 0.5) \times 7\\ &= 891.6\,\text{kN}\ (200.4\,\text{kip})\end{aligned}$$

$$\begin{aligned}P_{\text{ultimate}} &= \text{end bearing capacity} + \text{skin friction in layer 1}\\ &\quad + \text{skin friction in layer 2}\\ \text{end bearing capacity} &= 1{,}166.2\,\text{kN}\\ \text{skin friction in sand layer 1} &= 176.1\,\text{kN}\\ \text{skin friction in sand layer 2} &= 891.6\,\text{kN}\\ P_{\text{ultimate}} &= 2{,}233.9\,\text{kN}\ (502.2\,\text{kip})\end{aligned}$$

Note that the bulk of the pile capacity comes from the end bearing. After that comes the skin friction in layer 2 (bottom layer). The skin friction in the top layer is very small. One reason for this is that the effective stress acting on the perimeter of the pile is very low in the top layer. This is due to the fact that effective stress is directly related to depth.

Hence

$$\begin{aligned}\text{allowable bearing capacity of the pile} &= P_{\text{ultimate}}/\text{F.O.S.}\\ &= 2{,}233.9/3.0\,\text{kN}\\ \text{allowable pile capacity} &= 744.6\,\text{kN}\ (167.4\,\text{kip})\end{aligned}$$

Design Example 31.6

This example concerns multiple sand layers with groundwater present. A 0.5 m diameter, 15 m long round concrete pile is driven into a sandy soil stratum as shown in Fig. 31.10. The groundwater level is 3 m below the surface. Compute the ultimate bearing capacity of the pile.

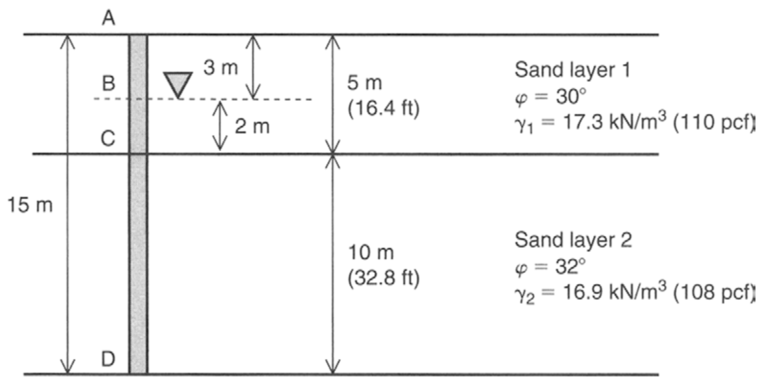


Figure 31.10 Single pile in sand (groundwater considered)

Solution

STEP 1: Compute the end bearing capacity.

$$Q_{\text{ultimate}} = \underbrace{(\sigma'_t \times N_q \times A)}_{\text{End bearing term}} + \underbrace{(K \times \sigma'_v \times \tan\delta \times A_p)}_{\text{Skin friction term}}$$

$$\text{end bearing term} = \sigma'_t \times N_q \times A$$

where

σ'_t = effective stress at the tip of the pile

$$\begin{aligned}\sigma'_t &= \gamma_1 \times 3 + (\gamma_1 - \gamma_w) \times 2 + (\gamma_2 - \gamma_w) \times 10 \\ &= 17.3 \times 3 + (17.3 - 9.8) \times 2 + (16.9 - 9.8) \times 10 \\ &= 137.9\,\text{kN/m}^2\ (2{,}880\,\text{psf})\end{aligned}$$

Find N_q using Table 31.1. For a friction angle of 32°, $N_q = 29$ for driven piles.

The φ value of the bottom sand layer is used to find N_q, since the tip of the pile lies on the bottom sand layer.

$$\text{end bearing capacity} = \sigma_t' \times N_q \times A$$
$$= 137.9 \times 29 \times (\pi \times d^2/4) = 785.2\,\text{kN}\ (176.5\,\text{kip})$$

STEP 2: Compute the skin friction.
The skin friction of the pile needs to be computed in three parts.

- Skin friction of the pile portion in sand layer 1 above groundwater (A to B).
- Skin friction of the pile portion in sand layer 1 below groundwater (B to C).
- Skin friction of the pile portion in sand layer 2 below groundwater (C to D).

STEP 3: Compute the skin friction of the pile portion in sand layer 1 above the groundwater level (A to B).

$$\text{skin friction term} = K \times \sigma_v' \times \tan\delta \times (\pi \times d) \times L$$

Obtain the K value. From Table 31.4, for driven round piles, the K value lies between 1.0 and 1.5. Hence, assume $K = 1.25$.

Obtain the σ_v' (effective stress at the perimeter of the pile). Obtain the σ_v' value at the midpoint of the pile in sand layer 1, above the groundwater level (A to B).

$$\sigma_v'\ (\text{midpoint}) = 1.5 \times (\gamma_1) = 1.5 \times 17.3\,\text{kN/m}^2$$
$$= 26\,\text{kN/m}^2\ (543\,\text{psf})$$

Obtain the skin friction angle, δ. From Table 31.3, the skin friction angle, δ, for concrete piles is $3/4\,\varphi$.

$$\delta = {}^3\!/_4 \times 30^\circ = 22.5^\circ$$

This is true because the friction angle of layer 1 is 30°.

$$\text{skin friction in sand layer 1 (A to B)} = K \times \sigma_v' \times \tan\delta \times (\pi \times d) \times L$$

$$= 1.25 \times (26) \times (\tan 22.5°) \times (\pi \times 0.5) \times 3$$
$$= 63.4\,\text{kN (14.3 kip)}$$

STEP 4: Find the skin friction of the pile portion in sand layer 1 below the groundwater level (B to C).

$$\text{skin friction term} = K \times \sigma'_v \times \tan\delta \times (\pi \times d) \times L$$

Obtain the σ'_v (effective stress at the perimeter of the pile).

$$\sigma'_v\ (\text{midpoint}) = 3 \times \gamma_1 + 1.0(\gamma_1 - \gamma_w)$$
$$= 3 \times 17.3 + 1.0 \times (17.3 - 9.8)\,\text{kN/m}^2$$
$$= 59.4\,\text{kN/m}^2\ (1{,}241\,\text{psf})$$

$$\text{skin friction in sand layer 1 (B to C)} = K \times \sigma'_p \times \tan\delta \times (\pi \times d) \times L$$
$$= 1.25 \times 59.4 \times (\tan 22.5°)$$
$$\times (\pi \times 0.5) \times 2$$
$$= 96.6\,\text{kN (21.7 kip)}$$

STEP 5: Find the skin friction in sand layer 2 below the groundwater level (C to D).

$$\text{skin friction term} = K \times \sigma'_v \times \tan\delta \times (\pi \times d) \times L$$

Obtain the σ'_p (effective stress at the midpoint of the pile):

$$\sigma'_v\ (\text{midpoint}) = 3 \times \gamma_1 + 2 \times (\gamma_1 - \gamma_w) + 5 \times (\gamma_2 - \gamma_w)$$
$$= 3 \times 17.3 + 2.0 \times (17.3 - 9.8) + 5 \times (16.9 - 9.8)\,\text{kN/m}^2$$
$$= 102.4\,\text{kN/m}^2\ (2.14\,\text{ksf})$$

From Table 31.3, the skin friction angle, δ, for concrete piles is $3/4\,\varphi$.

$$\delta = 3/4 \times 32° = 24°$$

This is true because the friction angle of layer 2 is 32°.

$$\begin{aligned}\text{skin friction in sand layer 2 (C to D)} &= K \times \sigma_v' \times \tan\delta \times (\pi \times d) \times L \\ &= 1.25 \times 102.4 \times (\tan 24^\circ) \\ &\quad \times (\pi \times 0.5) \times 10 \\ &= 895.2\,\text{kN (201.2 kip)}\end{aligned}$$

$$\begin{aligned}P_{\text{ultimate}} = &\ \text{end bearing capacity} \\ &+ \text{skin friction in layer 1 (above the groundwater level)} \\ &+ \text{skin friction in layer 1 (below the groundwater level)} \\ &+ \text{skin friction in layer 2 (below the groundwater level)}\end{aligned}$$

$$\begin{aligned}\text{end bearing capacity} &= 785.2\,\text{kN} \\ \text{skin friction in layer 1 (above the groundwater level)(A to B)} &= 63.4\,\text{kN} \\ \text{skin friction in layer 1 (below the groundwater level)(B to C)} &= 96.6\,\text{kN} \\ \text{skin friction in layer 2 (below the groundwater level)(C to D)} &= 895.2\,\text{kN}\end{aligned}$$

$$\begin{aligned}\text{total} &= 1{,}840\,\text{kN} \\ P_{\text{ultimate}} &= 1{,}840.4\,\text{kN (413.7 kip)}\end{aligned}$$

In this case, the bulk of the pile capacity comes from end bearing and the skin friction in the bottom layer. Hence, the allowable bearing capacity of the pile can be determined by

$$P_{\text{ultimate}}/\text{F.O.S.} = 1{,}840.4/3.0$$

$$\text{allowable pile capacity} = 613.5\,\text{kN (137.9 kip)}$$

31.3.9 Pile Design Using Meyerhoff Equation: Correlation with SPT (N)

End Bearing Capacity

Meyerhoff proposed the following equation based on the SPT (N) value to compute the ultimate end bearing capacity of driven piles. The

Meyerhoff equation was adopted by DM 7.2 (NAVFAC DM 7.2, 1984) as an alternative method to static analysis.

$$q_{ult} = 0.4\ c_N \times N \times D/B$$

where

q_{ult} = ultimate point resistance of driven piles (tsf)
N = standard penetration resistance (blows/ft) near pile tip
$c_N = 0.77 \log 20/p$
p = effective overburden stress at pile tip (tsf)

The effective stress p should be more than 500 psf. It is very rare for effective overburden stress at the pile tip to be less than 500 psf.

D = depth driven into granular (sandy) bearing stratum (ft)
B = width or diameter of the pile (ft)
q_1 = limiting point resistance (tsf), equal to $4N$ for sand and $3N$ for silt

If the average SPT (N) value at the tip of the pile is 15, then the maximum point resistance for sandy soil is $4N = 60$ tsf.

31.3.10 Modified Meyerhoff Equation

Meyerhoff developed the equation in the previous section using many available load test data and obtaining average N values. Pile tip resistance is a function of the friction angle. For a given SPT (N) value, different friction angles are obtained for different soils.

For a given SPT (N) value, the friction angle for coarse sand is 7 to 8% higher compared to medium sand. At the same time, for a given SPT (N) value, the friction angle is 7 to 8% lower in find sand compared to medium sand. For this reason, the author proposes the following modified equations.

$$q_{ult} = 0.45 c_N \times N \times D/B \text{ tsf} \quad \text{(coarse sand)}$$

$$q_{ult} = 0.4 c_N \times N \times D/B \text{ tsf} \quad \text{(medium sand)}$$

$$q_{ult} = 0.35 c_N \times N \times D/B \text{ tsf} \quad \text{(fine sand)}$$

Design Example 31.7

Find the tip resistance of the 2 ft (0.609 m) diameter pile shown in Fig. 31.11 using the Meyerhoff equation. The SPT (N) value at the pile tip is 25 blows per foot.

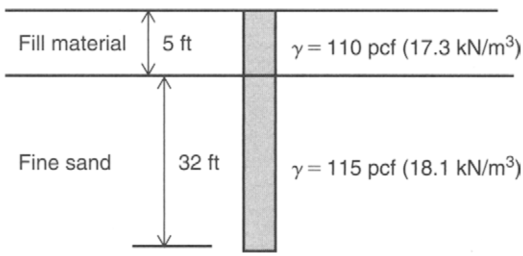


Figure 31.11 Use of Meyerhoff equation

Solution

Determine the ultimate point resistance for driven piles for fine sand.

$$q_{\text{ult}} = 0.35 c_{\text{N}} \times N \times D/B \text{ tsf (fine sand)}$$
$$c_{\text{N}} = 0.77 \log 20/p$$

where

$p =$ effective overburden stress at pile tip (tsf)

$$p = 5 \times 110 + 32 \times 115 = 4{,}230 \text{ psf} = 2.11 \text{ tsf } (0.202 \text{ MPa})$$
$$c_{\text{N}} = 0.77 \log(20/2.11) = 0.751$$
$$D = \text{depth driven into bearing stratum} = 32 \text{ ft } (9.75 \text{ m})$$

The fill material is not considered to be a bearing stratum.

$$B \text{ (width or diameter of the pile)} = 2 \text{ ft } (0.61 \text{ m})$$
$$q_{\text{ult}} = 0.35 c_{\text{N}} \times N \times D/B \text{ (fine sand)}$$

$$q_{ult} = 0.35 \times 0.751 \times 25 \times 32/2$$
$$= 105 \text{ tsf } (10.1 \text{ MPa})$$

$$\text{maximum allowable point resistance} = 4N \text{ for sandy soils}$$
$$4 \times N = 4 \times 25 = 100 \text{ tsf}$$

Hence

$$q_{ult} = 100 \text{ tsf } (9.58 \text{ MPa})$$
$$\text{allowable point bearing capacity} = 100/\text{F.O.S.}$$

Assume a factor of safety of 3.0.
Hence,

$$q_{allowable} = 33.3 \text{ tsf } (3.2 \text{ MPa})$$
$$\text{total allowable point bearing capacity} = q_{allowable} \times \text{tip area}$$
$$= q_{allowable} \times \pi \times (2^2)/4$$
$$= 105 \text{ tons } (934 \text{ kN})$$

31.3.11 Meyerhoff Equations for Skin Friction

Meyerhoff proposes the following equation for skin friction.

$$f = N/50 \text{ tsf}$$

where

f = unit skin friction (tsf)
N = average SPT (N) value along the pile

Note that, as per Meyerhoff, the unit skin friction f should not exceed 1 tsf. The author proposes the following modified equations to account for soil gradation.

$$\text{for coarse sand, } f = N/46 \text{ tsf}$$
$$\text{for medium sand, } f = N/50 \text{ tsf}$$
$$\text{for fine sand, } f = N/54 \text{ tsf}$$

Design Example 31.8

Find the skin friction of the 2 ft diameter pile shown in Fig. 31.12 using the Meyerhoff equation. The average SPT (N) value along the shaft is 15 blows per foot.

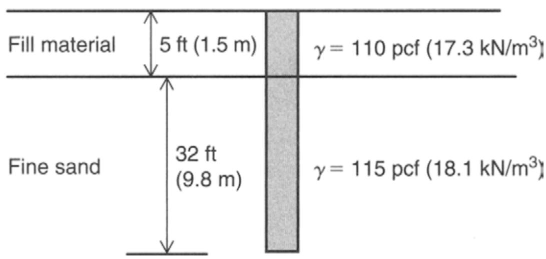


Figure 31.12 Skin friction using Meyerhoff equation

Solution

Ignore the skin friction in the fill material. For fine sand,

$$\text{unit skin friction } (f) = N/54 \text{ tsf}$$
$$= 15/54 \text{ tsf} = 0.28 \text{ tsf } (26.8 \text{ kN/m}^2)$$
$$\text{total skin friction} = \text{unit skin friction} \times \text{perimeter surface area}$$
$$\text{total skin friction} = 0.28 \times \pi \times D \times L$$
$$= 0.28 \times \pi \times 2 \times 32 = 56 \text{ tons}$$
$$\text{allowable skin friction} = 56/\text{F.O.S.}$$

Assume a factor of safety of 3.

$$\text{allowable skin friction} = 56/3.0 = 18.7 \text{ tons}$$

31.4 Critical Depth for Skin Friction (Sandy Soils)

The vertical effective stress, σ', increases with depth. Hence, skin friction should increase with depth indefinitely. In reality, skin friction

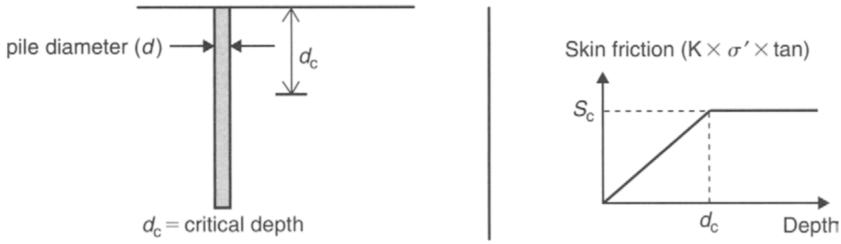


Figure 31.13 Critical depth for skin friction

will not increase with depth indefinitely. It was believed in the past that skin friction would become a constant at a certain depth. This depth was called critical depth. See Fig. 31.13.

- As shown in Figure 31.13, the skin friction was assumed to increase until the critical depth, and then maintain a constant value.

d_C = critical depth
S_C = skin friction at critical depth ($K \times \sigma'_c \times \tan\delta$)
σ'_c = effective stress at critical depth

The following approximations were assumed for the critical depth.

critical depth for loose sand $= 10 \times d$ (where d is the pile diameter or the width)

critical depth for medium dense sand $= 15 \times d$

critical depth for dense sand $= 20 \times d$

This theory does not explain the recent precise pile load test data. According to recent experiments, it is clear that skin friction will not abruptly become a constant, as was once believed.

31.4.1 Experimental Evidence for Critical Depth

Figure 31.14 shows a typical variation of skin friction with depth in a pile. As can be seen, experimental data does not support the old theory of a constant skin friction below the critical depth. The skin friction tends to increase with depth and just above the tip of the pile attain its maximum value. The skin friction drops rapidly after that. Skin friction does not increase linearly with depth as was once believed.

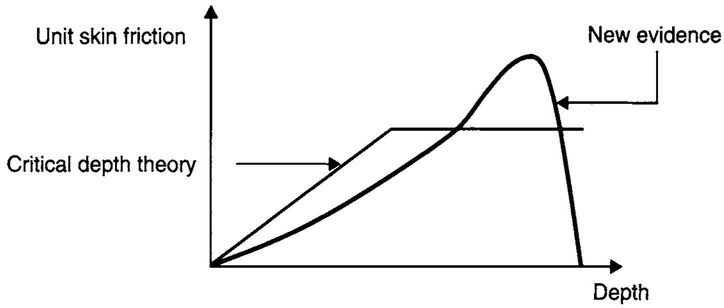


Figure 31.14 Skin friction vs. depth

It should be noted that no satisfactory theory exists at the present to explain the field data. Due to lack of a better theory, the critical depth theory of the past is still being used by engineers.

31.4.2 Reasons for Limiting Skin Friction

The following reasons have been put forward to explain why skin friction does not increase with depth indefinitely as suggested by the skin friction equation.

$$\text{unit skin friction} = K \times \sigma' \times \tan\delta$$

$$\sigma' = \gamma \times d$$

1. The above K value is a function of the soil friction angle (φ'). The friction angle tends to decrease with depth. Hence the K value decreases with depth (Kulhawy et al., 1983).
2. The above skin friction equation does not hold true at high stress levels due to readjustment of sand particles.
3. There is a reduction of local shaft friction with increasing pile depth (Randolph et al., 1994). See Fig. 31.15.

Assume that a pile was driven to a depth of 10 ft and that the unit skin friction was measured at a depth of 5 ft. Then assume the pile was driven further to a depth of 15 ft and the unit skin friction was measured at the same depth of 5 ft. It has been reported that the unit skin friction at 5 ft has a smaller value in the second case.

According to Fig. 31.15, the local skin friction reduces when the pile is driven further into the ground.

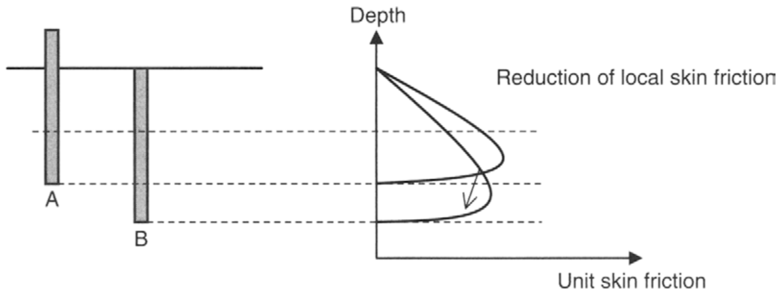


Figure 31.15 Depth vs. unit skin friction

As per NAVFAC DM 7.2 (1984), limiting the value of skin friction and end bearing capacity is achieved after 20 diameters within the bearing zone.

31.5 Critical Depth for End Bearing Capacity (Sandy Soils)

Pile end bearing capacity in sandy soils is related to effective stress. Experimental data indicates that end bearing capacity does not increase with depth indefinitely. Due to the lack of a valid theory, engineers use the same critical depth concept adopted for skin friction for end bearing capacity as well. See Fig. 31.16.

As shown in Fig. 31.16, the end bearing capacity was assumed to increase to the critical depth.

d_c = critical depth
Q_c = end bearing at critical depth
σ'_c = effective stress at critical depth

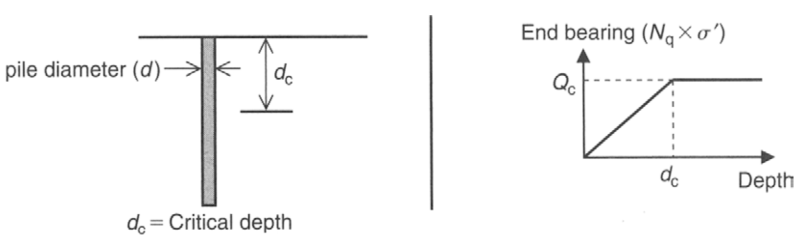


Figure 31.16 Critical depth for end bearing capacity

The following approximations were assumed for the critical depth.

critical depth for loose sand = $10 \times d$ (where d is the pile diameter or the width)

critical depth for medium dense sand = $15 \times d$

critical depth for dense sand = $20 \times d$

Is the critical depth for the end bearing the same as the critical depth for skin friction?

Since the critical depth concept is a gross approximation that cannot be supported by experimental evidence, the question is irrelevant. It is clear that there is a connection between end bearing capacity and skin friction, since the same soil properties act in both cases such as effective stress, friction angle, and relative density. On the other hand, it should be noted that the two processes are vastly different in nature.

31.5.1 Critical Depth

Design Example 31.9

Find the skin friction and end bearing capacity of the pile shown in Fig. 31.17. Assume that the critical depth is achieved at 20 ft into the bearing layer (NAVFAC DM 7.2, 1984). The pile diameter is 1 ft, and other soil parameters are as shown in Fig. 31.17.

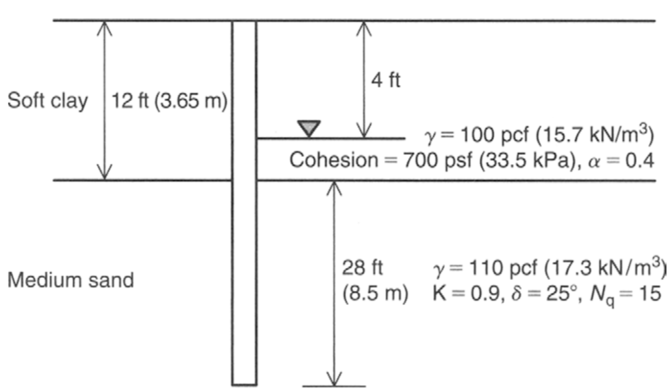


Figure 31.17 Critical depth in medium sand

Solution

The skin friction is calculated in the overburden. In this case, skin friction is calculated in the soft clay. Then the skin friction is calculated in the bearing layer (medium sand) assuming the skin friction attains a limiting value after 20 diameters (critical depth). See 31.18.

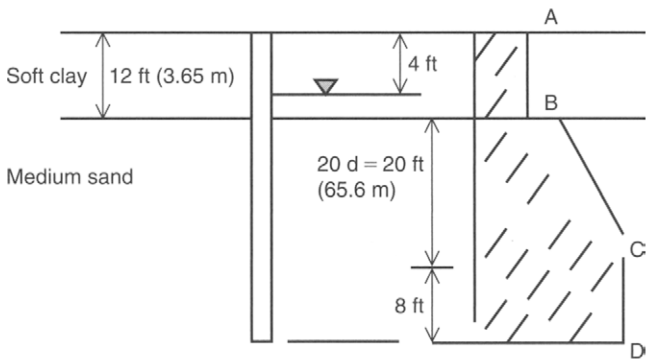


Figure 31.18 Development of skin friction

STEP 1: Find the skin friction from A to B.

$$
\begin{aligned}
\text{skin friction in soft clay} &= \alpha \times c \times \text{perimeter surface area} \\
&= 0.4 \times 700 \times \pi \times d \times L \\
&= 0.4 \times 700 \times \pi \times 1 \times 12 \\
&= 10{,}560 \text{ lb } (46.9 \text{ kN})
\end{aligned}
$$

STEP 2: Find the skin friction from B to C.

Critical depth is at point C, 20 ft below the soft clay

$$\text{Skin friction in sandy soils} = S = K \times \sigma'_v \times \tan\delta \times A_p$$

S = skin friction of the pile
σ'_v = average effective stress along the pile shaft

$$\text{Average effective stress along pile shaft from B to C} = (\sigma_B + \sigma_C)/2$$

σ_B = effective stress at B
σ_C = effective stress at C

To obtain the average effective stress from B to C, find the effective stresses at B and C and obtain the average of those two values.

$$\sigma_B = 100 \times 4 + (100 - 62.4) \times 8$$
$$= 700.8 \text{ lb/ft}^2 \text{ (33.6 kPa)}$$
$$\sigma_C = 100 \times 4 + (100 - 62.4) \times 8 + (110 - 62.4) \times 20$$
$$= 1{,}452.8 \text{ lb/ft}^2 \text{ (69.5 kPa)}$$

$$\text{average effective stress along pile shaft from B to C} = (\sigma_B + \sigma_C)/2$$
$$= (700.8 + 1452.8)/2 = 1076.8 \text{ lb/ft}^2$$
$$\text{skin friction from B to C} = K \times \sigma'_v \times \tan\delta \times A_p$$
$$= 0.9 \times 1{,}076.8 \times \tan(25^\circ) \times (\pi \times 1 \times 20)$$
$$= 28{,}407 \text{ lb}$$

STEP 3: Find the skin friction from C to D.

Skin friction reaches a constant value at point C, 20 diameters into the bearing layer.

$$\text{skin friction at point C} = K \times \sigma'_v \times \tan\delta \times A_p$$
$$\sigma'_v \text{ at point C} = 100 \times 4 + (100 - 62.4) \times 8$$
$$+ (110 - 62.4) \times 20$$
$$= 1{,}452.8 \text{ lb/ft}^2$$
$$\text{unit skin friction at point C} = 0.9 \times 1{,}452.8 \times \tan 25^\circ$$
$$= 609.7 \text{ lb/ft}^2 \text{ (29 kPa)}$$

The unit skin friction is constant from C to D. This is due to the fact that skin friction does not increase after the critical depth.

$$\text{skin friction from } C \text{ to } D = 609.7 \times \text{surface perimeter area}$$
$$= 609.7 \times (\pi \times 1 \times 8) \text{ lb}$$
$$= 15{,}323.4 \text{ lb (68.2 kN)}$$

Summing up,

$$\text{skin friction in soft clay (A to B)} = 10{,}560 \text{ lb}$$
$$\text{skin friction in sand (B to C)} = 28{,}407 \text{ lb}$$
$$\text{skin friction in sand (C to D)} = 15{,}323 \text{ lb}$$
$$\text{total} = 54{,}290 \text{ lb (241 kN)}$$

STEP 4: Compute the end bearing capacity.

The end bearing capacity also reaches a constant value below the critical depth.

$$\text{end bearing capacity} = q \times N_q \times A$$

where

q = effective stress at pile tip
N_q = bearing capacity factor (given as 15)
A = cross-sectional area of the pile

If the pile tip is below the critical depth, q should be taken at critical depth. In this example, the pile tip is below the critical depth, which is 20 diameters into the bearing layer. Hence, q is equal to the effective stress at the critical depth (point C).

$$\begin{aligned}
\text{effective stress at point C} &= 100 \times 4 + (100 - 62.4) \times 8 \\
&\quad + (110 - 62.4) \times 20 \\
&= 1{,}452.8 \text{ lb/ft}^2 \\
\text{end bearing capacity} &= q \times N_q \times A \\
&= 1{,}452.8 \times 15 \times (\pi \times d^2/4) \text{ lb} \\
&= 17{,}115 \text{ lb} \\
\text{total ultimate capacity of the pile} &= \text{total skin friction} + \text{end bearing} \\
&= 54{,}290 + 17{,}115 = 71{,}405 \text{ lb} \\
&= 35.7 \text{ tons (317.6 kN)}
\end{aligned}$$

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32

Pile Design in Clay Soils

32.1 Introduction

There are two types of forces acting on piles. See Fig. 32.1.

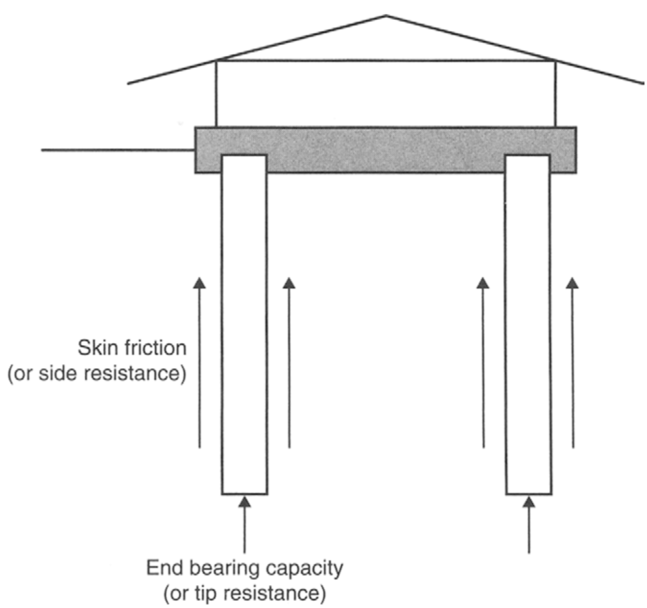

Figure 32.1 Skin friction in clay soil

1. End bearing acts on the bottom of the pile.
2. Skin friction acts on the sides of the pile.

To compute the total load that can be applied to a pile, one needs to compute the end bearing and the skin friction acting on the sides of the pile.

The modified Terzaghi bearing capacity equation is used to find the pile capacity.

$$P_u = Q_u + S_u$$
$$P_u = 9 \times c \times A_c + \alpha \times c \times A_p$$

where

P_u = ultimate pile capacity
Q_u = ultimate end bearing capacity
S_u = ultimate skin friction
c = cohesion of the soil
A_c = cross-sectional area of the pile
A_p = perimeter surface area of the pile
α = adhesion factor between pile and soil

As per the above equation, clay soils with a higher cohesion have a higher end bearing capacity. The end bearing capacity of piles in clayey soils is usually taken to be $9 \times c$.

32.1.1 Skin Friction

Assume a block is placed on a clay surface. Now if a force is applied to move the block, the adhesion between the block and the clay resists the movement. If the cohesion of clay is c, then the force due to adhesion would be $\alpha \times c$ (where α is the adhesion coefficient). See Fig. 32.2.

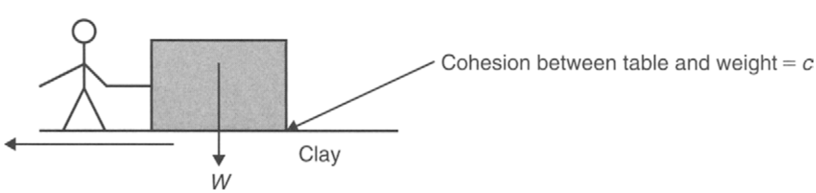

Figure 32.2 Cohesive forces

α = adhesion coefficient depending on pile material and clay type
c = cohesion

Highly plastic clays would have a higher adhesion coefficient. Typically, it is assumed that adhesion is not dependent on the weight of the block. This is not strictly true as explained later.

Now let's look at a pile outer surface. See Fig. 32.3.

$$\text{ultimate skin friction} = S_u = \alpha \times c \times A_p$$

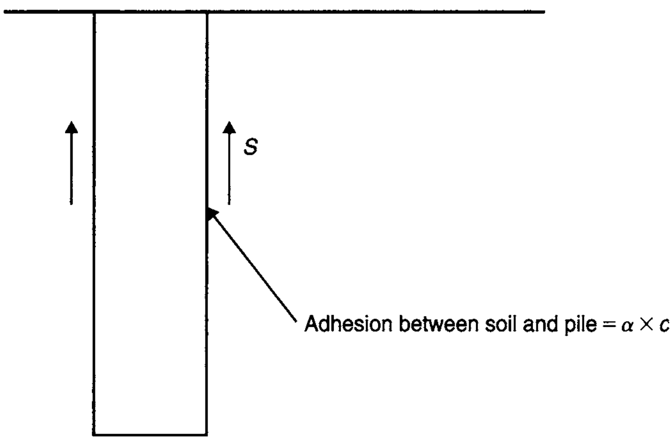

Figure 32.3 Adhesion

Usually it is assumed that ultimate skin friction is independent of the effective stress and depth. In reality, the skin friction is dependent on the effective stress and cohesion of soil.

The skin friction acts on the perimeter surface of the pile.

For a circular pile, the surface area of the pile is given by

$$\pi \times D \times L$$

where

$\pi \times D$ = circumference of the pile
L = length of the pile

$$\text{perimeter surface area of a circular pile, } A_p = \pi \times D \times L$$

32.2 End Bearing Capacity in Clay Soils, Different Methods

32.2.1 Driven Piles

Skempton (1959)

The equation proposed by Skempton is widely being used to find the end bearing capacity in clay soils.

$$q = 9 \times c_u$$

where

q = end bearing capacity
c_u = cohesion of soil at the tip of the pile

Martin et al. (1987)

$$q = c \times N \text{ MN/m}^2$$

where

$c = 0.20$
N = SPT value at pile tip

32.2.2 Bored Piles

Shioi and Fukui (1982)

$$q = c \times N \text{ MN/m}^2$$

where

$c = 0.15$
N = SPT value at pile tip

NAVFAC DM 7.2 (1984)

$$q = 9 \times c_u$$

where

q = end bearing capacity
c_u = cohesion of soil at the tip of the pile

32.3 Skin Friction in Clay Soils (Different Methods)

The equations are based on undrained shear strength (cohesion)

$$f_{\mathrm{ult}} = \alpha \times S_{\mathrm{u}}$$

where

f_{ult} = ultimate skin friction
α = skin friction coefficient
S_{u} = undrained shear strength or cohesion

$$S_{\mathrm{u}} = Q_{\mathrm{u}}/2$$

where

Q_{u} = unconfined compressive strength

These types of equations ignore effective stress effects. See Fig. 32.4.

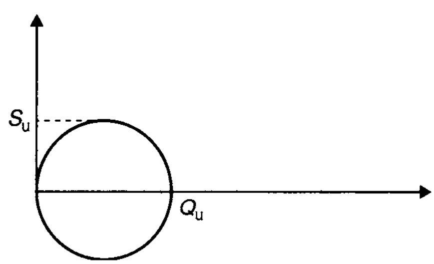

Figure 32.4 Mohr's circle diagram

32.3.1 Driven Piles

American Petroleum Institute (API)

API (1984) provides the following equation to find the skin friction in clay soils.

$$f = \alpha \times c_{\mathrm{u}}$$

f = unit skin friction
c_{u} = cohesion

$$\alpha = 1.0 \text{ for clays with } c_u < 25\,\text{kN/m}^2\ (522\,\text{psf})$$
$$= 0.5 \text{ for clays with } c_u > 70\,\text{kN/m}^2\ (1{,}460\,\text{psf})$$

Interpolate for the α value for cohesion values between 25 kN/m^2 and 70 kN/m^2.

Per the API method, the skin friction is solely dependant on cohesion. Effective stress changes with the depth and the API method disregard the effective stress effects in soil.

NAVFAC DM 7.2

$$f = \alpha \times c_u \times A_p$$

S = skin friction
c_u = cohesion
A_p = perimeter surface area of the pile

See Table 32.1. As in the API method, the effective stress effects are neglected in the DM 7.2 method.

Table 32.1 Pile type, cohesion, and adhesion

Pile type	Soil consistency	Cohesion range (kN/m^2)	α
Timber and concrete piles			
	Very soft	0–12	0–1.0
	Soft	12–24	1.0–0.96
	Medium stiff	24–48	0.96–0.75
	Stiff	48–96	0.75–0.48
	Very stiff	96–192	0.48–0.33
Steel piles			
	Very soft	0–12	0.0–1.0
	Soft	12–24	1.0–0.92
	Medium stiff	24–48	0.92–0.70
	Stiff	48–96	0.70–0.36
	Very stiff	96–192	0.36–0.19

Source: NAVFAC DM 7.2 (1984).

32.3.2 Bored Piles

Fleming et al. (1985)

$$f = \alpha \times c_u$$

f = unit skin friction
c_u = cohesion

$$\alpha = 0.7 \text{ for clays with } c_u < 25\,\text{kN/m}^2 \text{ (522 psf)}$$

$$\alpha = 0.35 \text{ for clays with } c_u > 70\,\text{kN/m}^2 \text{ (1,460 psf)}$$

Note that the α value for bored piles is chosen to be 0.7 times the value for driven piles.

The equations are based on the vertical effective stress

$$f_{ult} = \beta \times \sigma'$$

where

f_{ult} = ultimate skin friction
β = skin friction coefficient based on effective stress
σ' = effective stress

These types of equations ignore cohesion effects.

Burland (1973)

$$f = \beta \times \sigma'_v$$

where

f = unit skin friction
σ'_v = effective stress

$$\beta = (1 - \sin\varphi') \times \tan\varphi' \times (\text{OCR})^{0.5}$$

where

OCR = overconsolidation ratio of clay

The Burland method does not consider the cohesion of soil. One could argue that the OCR is indirectly related to cohesion.

32.3.3 Equation Based on Both Cohesion and Effective Stress

A new method was proposed by Kolk and Van der Velde (1996) that considers both cohesion and effective stress to compute the skin friction of piles in clay soils.

Kolk and Van der Velde Method (1996)

The Kolk and Van der Velde method considers both cohesion and effective stress.

$$f_{\text{ult}} = \alpha \times S_u$$

where

f_{ult} = ultimate skin friction
α = skin friction coefficient

In this case, the skin friction coefficient, α, is obtained using the correlations provided by Kolk and Van der Velde. The parameter α is based on both cohesion and effective stress.

S_u = undrained shear strength (cohesion)

The parameter α in the Kolk and Van der Velde equation is based on the ratio of undrained shear strength and effective stress. A large database of pile skin friction results was analyzed and correlated to obtain the α values. See Table 32.2.

According to Table 32.2, α decreases when S_u increases, and α increases when σ' (effective stress) increases.

Design Example 32.1

Find the skin friction of the 1 ft diameter pile shown in Fig. 32.5 using the Kolk and Van der Velde method.

Table 32.2 Skin friction factor

S_u/σ'	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
α	0.95	0.77	0.7	0.65	0.62	0.60	0.56	0.55	0.53	0.52	0.50	0.49	0.48
S_u/σ'	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5	3.0	4.0
α	0.47	0.42	0.41	0.41	0.42	0.41	0.41	0.40	0.40	0.40	0.4	0.39	0.39

Source: Kolk and Van der Velde (1996).

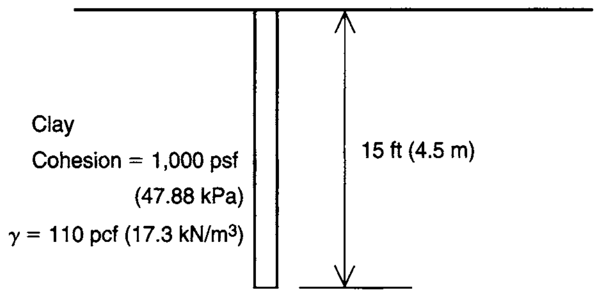

Figure 32.5 Skin friction in clay

Solution

The Kolk and Van der Velde method depends on both effective stress and cohesion. The effective stress varies with the depth. Hence, it is necessary to obtain the average effective stress along the pile length. In this case, obtain the effective stress at the midpoint of the pile.

$$\text{effective stress at the midpoint of the pile} = 110 \times 7.5\,\text{psf}$$
$$= 825\,\text{psf (39.5 kPa)}$$
$$\text{cohesion} = S_u = 1{,}000\,\text{psf (47.88 kPa)}$$
$$S_u/\sigma' = 1{,}000/825 = 1.21$$

From Table 32.2,

$$\alpha = 0.5 \text{ for } S_u/\sigma' = 1.2$$

Hence use

$$\alpha = 0.5$$
$$\text{ultimate unit skin friction} = \alpha \times S_u = 0.5 \times 1{,}000$$
$$= 500\,\text{psf (23.9 kPa)}$$
$$\text{ultimate skin friction of the pile} = 500 \times (\pi \times d \times L)$$
$$= 500 \times \pi \times 1 \times 15\,\text{lb}$$
$$= 23{,}562\,\text{lb (104.8 kN)}$$

32.4 Piles in Clay Soils

32.4.1 Skin Friction in Clay Soils

As mentioned in previous chapters, attempts to correlate skin friction with undrained shear strength were not very successful. A better correlation was found with skin friction and the S_u/σ' ratio, S_u being the undrained shear strength and σ' being the effective stress.

What happens to a soil element when a hole is drilled? See Fig. 32.6. The following changes occur in a soil element near the wall after the drilling process.

- The soil element shown in Fig. 32.6 is subjected to a stress relief. The undrained shear strength is reduced due to the stress relief.
- The reduction of undrained shear strength reduces the skin friction as well.

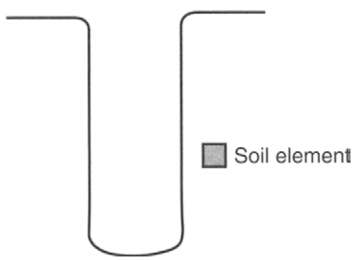

Figure 32.6 Drilling in soil

32.4.2 Computation of Skin Friction in Bored Piles

The same procedure used for driven piles can be used for bored piles, but with a lesser undrained shear strength. The question is, how much reduction should be applied to the undrained shear strength for bored piles?

The undrained shear strength may be reduced by as much as 50% due to the stress relief in bored piles. On the other hand, the measured undrained shear strength is already reduced due to the stress relief that occurred when the sample was removed from the ground. By the time the soil sample reaches the laboratory, the soil sample has undergone stress relief and the measured value already indicates the stress reduction. Considering these two aspects, a reduction of 30% in the undrained shear strength would be realistic for bored piles.

Design Example 32.2

This example explores a single pile in a uniform clay layer. Find the capacity of the pile shown in Fig. 32.7. The length of the pile is 10 m, and the diameter of the pile is 0.5 m. The cohesion of the soil is found to be 800 psf. Note that the groundwater level is at 2 m below the surface.

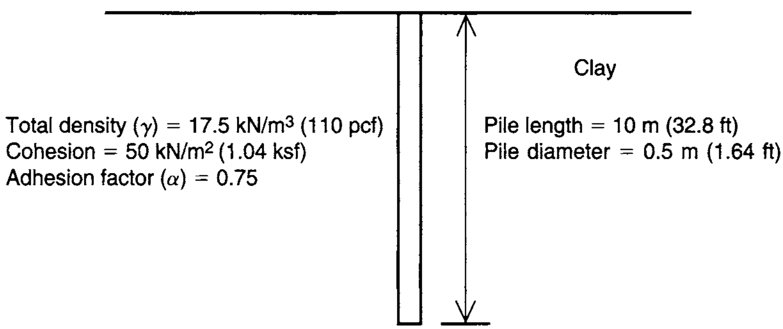

Figure 32.7 Single pile in clay soil

Solution

STEP 1: Find the end bearing capacity.

$$\text{end bearing capacity in clay soils} = 9 \times c \times A$$

$c = \text{cohesion} = 50\,\text{kN/m}^2$
$N_c = 9$

$A = \pi \times D^2/4 = \pi \times 0.5^2/4\,\text{m}^2 = 0.196\,\text{m}^2$

ultimate end bearing capacity $= 9 \times 50 \times 0.196 = 88.2\,\text{kN}$ (19.8 kip)

STEP 2: Find the skin friction.

$$\text{ultimate skin friction} = S_u = \alpha \times c \times A_p$$

$$\text{ultimate skin friction} = 0.75 \times 50 \times A_p$$

$$A_p = \text{perimeter surface area of the pile} = \pi \times D \times L$$

$$= \pi \times 0.5 \times 10\,\text{m}^2$$

$$A_p = 15.7\,\text{m}^2$$

$$\text{ultimate skin friction} = 0.75 \times 50 \times 15.7 = 588.8\,\text{kN (132 kip)}$$

STEP 3: Find the ultimate capacity of the pile.

$$\text{ultimate pile capacity} = \text{ultimate end bearing capacity} + \text{ultimate skin friction}$$

$$\text{ultimate pile capacity} = 88.2 + 588.8 = 677\,\text{kN (152 kip)}$$

$$\text{allowable pile capacity} = \text{ultimate pile capacity/F.O.S.}$$

Assume a factor of safety of 3.0.

$$\text{allowable pile capacity} = 677/3.0 = 225.7\,\text{kN (50.6 kip)}$$

Note that the skin friction was very much higher than the end bearing capacity in this situation.

Design Example 32.3

This example explores a single pile in a uniform clay layer with groundwater present. Find the allowable capacity of the pile shown in Fig. 32.8. The pile diameter is given as 1 m and the cohesion of the clay layer is 35 kPa. The groundwater level is 2 m below the surface. Find the allowable capacity of the pile.

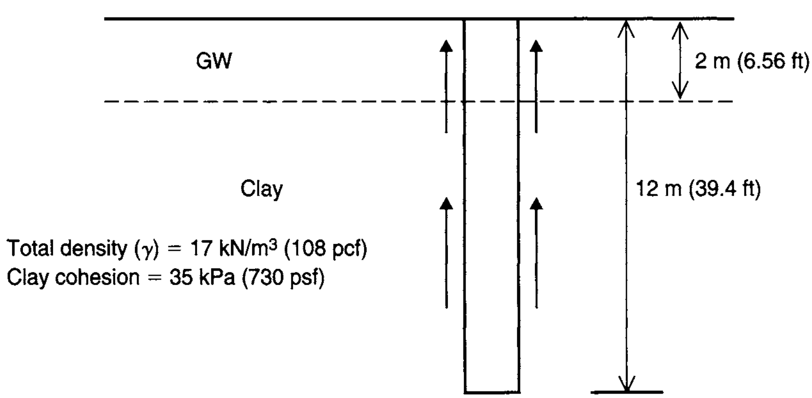

Figure 32.8 Skin friction (groundwater considered)

Solution

Unlike the case for sands, the groundwater level does not affect the skin friction in clayey soils.

STEP 1: Find the end bearing capacity.

$$\text{end bearing capacity in clay soils} = 9 \times c \times A$$

$$c = \text{cohesion} = 35\,\text{kN/m}^2$$

$$N_c = 9$$

$$A = \pi \times D^2/4 = \pi \times 1^2/4\,\text{m}^2 = 0.785\,\text{m}^2$$

$$\text{ultimate end bearing capacity} = 9 \times 35 \times 0.785$$

$$= 247.3\,\text{kN (55.5 kip)}$$

STEP 2: Find the skin friction.

$$\text{ultimate skin friction} = S_u = \alpha \times c \times A_p$$

The adhesion factor, α, is not given. Use the method given by API.

$$\alpha = 1.0 \text{ for clays with cohesion} < 25\,\text{kN/m}^2$$

$$\alpha = 0.5 \text{ for clays with cohesion} > 70\,\text{kN/m}^2$$

Since the cohesion is $35\,\text{kN/m}^2$, interpolate to obtain the adhesion factor, α.

$$(x - 1.0)/(35 - 25) = (x - 0.5)/(35 - 70)$$

$$(x - 1.0)/10 = (x - 0.5)/-35 - 35x + 35 = 10x - 5$$

$$45x = 40$$

$$x = 0.89$$

Hence α at $35\,\text{kN/m}^2$ is 0.89.

$$\text{ultimate skin friction} = 0.89 \times 35 \times A_p$$

$$A_p = \text{perimeter surface area of the pile} = \pi \times D \times L$$

$$= \pi \times 1.0 \times 12\,\text{m}^2$$

$$A_p = 37.7\,\text{m}^2$$

$$\text{ultimate skin friction} = 0.89 \times 35 \times 37.7\,\text{kN}$$

$$\text{ultimate skin friction} = 1{,}174.4\,\text{kN (264 kip)}$$

STEP 3: Find the ultimate capacity of the pile.

$$\text{ultimate pile capacity} = \text{ultimate end bearing capacity} + \text{ultimate skin friction}$$

$$\text{ultimate pile capacity} = 247.3 + 1{,}174.4 = 1{,}421.7\,\text{kN}$$

$$\text{allowable pile capacity} = \text{ultimate pile capacity/F.O.S.}$$

Assume a factor of safety of 3.0.

$$\text{allowable pile capacity} = 1{,}421.7/3.0 = 473.9\,\text{kN (106.6 kip)}$$

Design Example 32.4

In this example, the skin friction is calculated using the Kolk and Van der Velde method. A 3 m sand layer is underlain by a clay layer with cohesion of 25 kN/m^2. Find the skin friction of the pile within the clay layer. Use the Kolk and Van der Velde method. The density of both sand and clay are 17 kN/m^3. The diameter of the pile is 0.5 m. See Fig. 32.9.

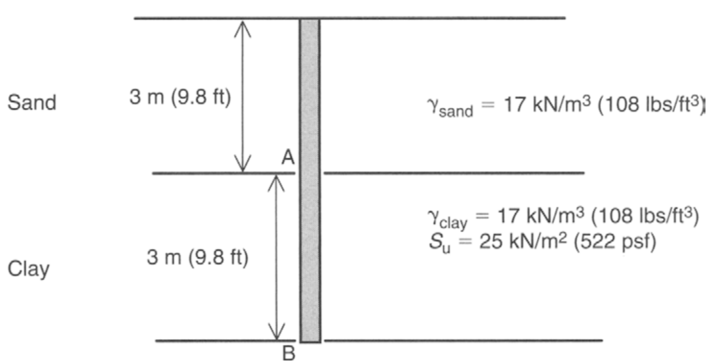

Figure 32.9 Pile in multiple layers

Solution

Find the skin friction at the top of the clay layer and the bottom of the clay layer. Obtain the average of the two values.

STEP 1: Find S_u/σ'.

At point A:

$$\sigma' = 3 \times 17 = 51\,\text{kN/m}^2\ (1{,}065\,\text{lb/ft}^2)$$

$$S_u = 25\,\text{kN/m}^2$$

$$S_u/\sigma' = 25/51 = 0.50$$

From Table 32.2, $\alpha = 0.65$.

At point B:

$$\sigma' = 6 \times 17 = 102\,\text{kN/m}^2$$

$$S_u = 25\,\text{kN/m}^2$$

$$S_u/\sigma' = 25/102 = 0.25$$

From Table 32.2, $\alpha = 0.86$.

STEP 2: Find the total skin friction.

Ultimate skin friction at point A:

$$f_{ult} = \alpha \times S_u = 0.65 \times 25 = 16\,\text{kN/m}^2$$

Ultimate skin friction at point B:

$$f_{ult} = \alpha \times S_u = 0.86 \times 25 = 21\,\text{kN/m}^2$$

Assume the average of two points to obtain the total skin friction.

$$\text{average} = (16 + 21)/2 = 18.5\,\text{kN/m}^2$$

$$\begin{aligned} \text{total skin friction} &= 18.5 \times (\text{perimeter}) \times \text{length} \\ &= 18.5 \times (\pi \times 0.5) \times 3 \\ &= 87\,\text{kN}\ (19.6\,\text{kip}) \end{aligned}$$

See Table 32.3 for a summary of the equations.

Table 32.3 Summary of equations

	Sand*	Clay*
Pile end bearing capacity	$N_q \times \sigma_v \times A$	$9 \times c \times A$
Pile unit skin friction	$K \times \sigma_v \times \tan\delta \times A_p$	$\alpha \times c \times A_p$

*A = bottom cross-sectional area of the pile; A_p = perimeter surface area of the pile; σ_v = vertical effective stress; N_q = Terzaghi bearing capacity factor; c = cohesion of the soil; K = lateral earth pressure coefficient; α = adhesion factor; δ = soil and pile friction angle.

32.5 Case Study: Foundation Design Options

D'Appolonia and Lamb (1971) describe the construction of several buildings at MIT. The soil conditions of the site are given in Fig. 32.10. Different foundation options were considered for the building.

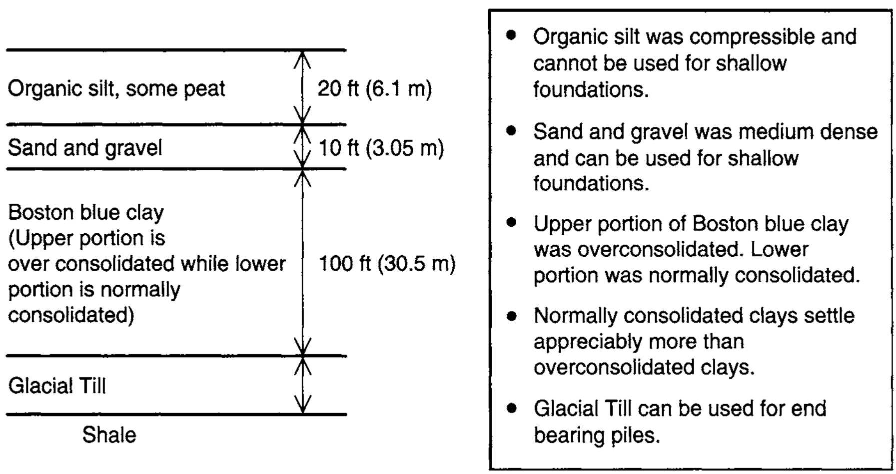

Figure 32.10 General soil conditions

32.5.1 General Soil Conditions

Note that all clays start as normally consolidated clays. Overconsolidated clays had been subjected to higher pressures in the past than existing in situ pressures. This happens mainly due to glacier movement, fill placement, and groundwater change. On the other hand, normally consolidated soils are presently experiencing the largest pressure they have ever experienced. For this reason, normally consolidated soils tend to settle more than overconsolidated clays.

32.5.2 Foundation Option 1: Shallow Footing Placed on Compacted Backfill

See Fig. 32.11.

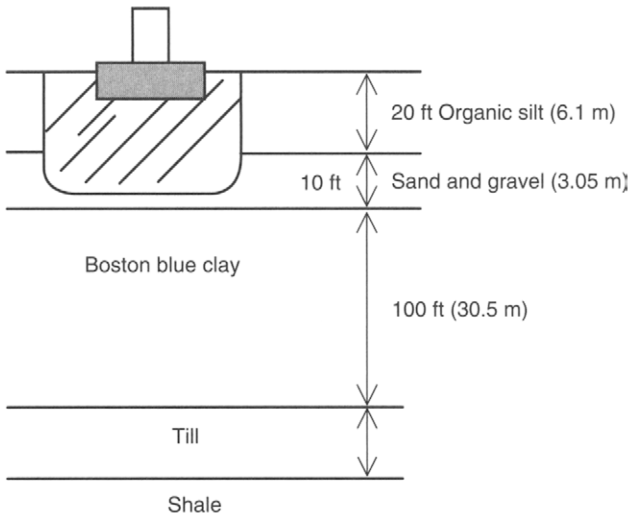

Figure 32.11 Shallow footing option

The construction procedure for foundation option 1 was as follows.

- Organic silt was excavated to the sand and gravel layer.
- Compacted backfill was placed and the footing was constructed.

This method was used for light loads.

It has been reported that Boston blue clay would settle by more than 4 in. when subjected to a stress of 400 psf (Aldrich, 1952). Hence, engineers had to design the footing so that the stress on the Boston blue clay was less than 400 psf.

32.5.3 Foundation Option 2: Timber Piles Ending on Sand and Gravel Layer

See Fig. 32.12.

The construction procedure for foundation option 2 was as follows.

- Foundations were placed on timber piles ending in the sand and gravel layer.

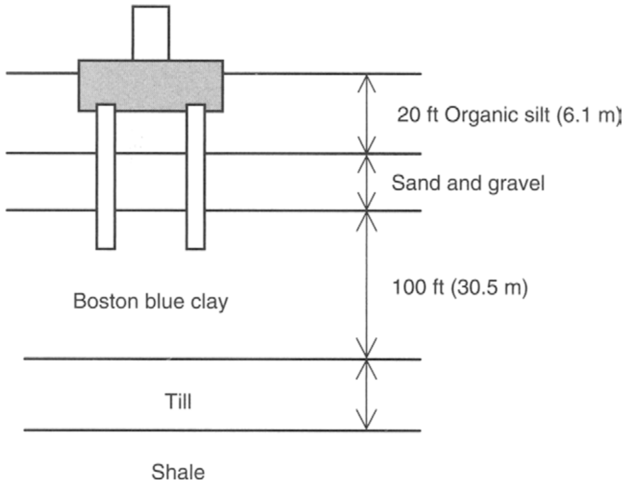

Figure 32.12 Short pile option

- Engineers had to make sure that the underlying clay layer was not stressed excessively due to piles.

This option was used for light loads.

32.5.4 Foundation Option 3: Timber Piles Ending in Boston Blue Clay Layer

See Fig. 32.13.

The construction procedure for foundation option 3 was as follows.

- Pile foundations were designed on timber piles ending in Boston blue clay.
- This method was found to be a mistake, since huge settlement occurred due to consolidation of the clay layer.

32.5.5 Foundation Option 4: Belled Piers Ending in Sand and Gravel

See Fig. 32.14.

For foundation option 4, foundations were placed on belled piers ending in the sand and gravel layer. It is not easy to construct belled piers in sandy soils. This option had major construction difficulties.

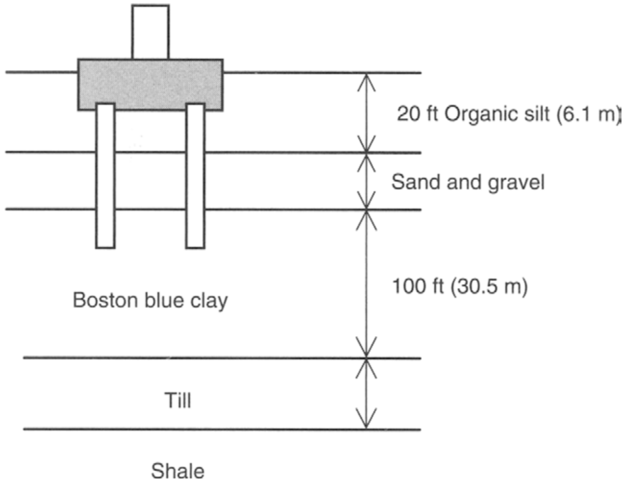

Figure 32.13 Medium length piles

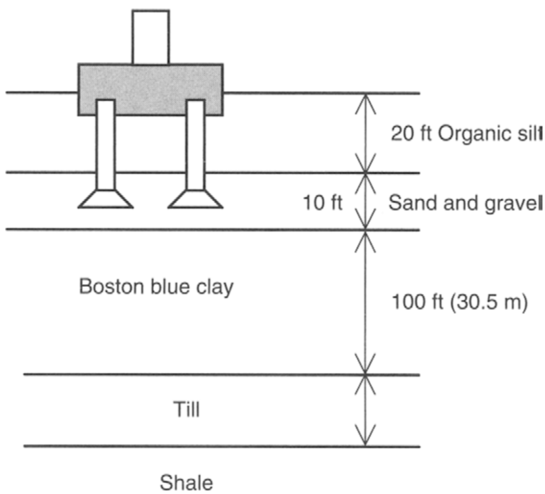

Figure 32.14 Short belled caissons

32.5.6 Foundation Option 5: Deep Piles Ending in Till or Shale

See Fig. 32.15.

The construction procedure for foundation option 5 was as follows.

- Foundations were placed on deep concrete pipe piles ending in till.
- These foundations were used for buildings with 20 to 30 stories.

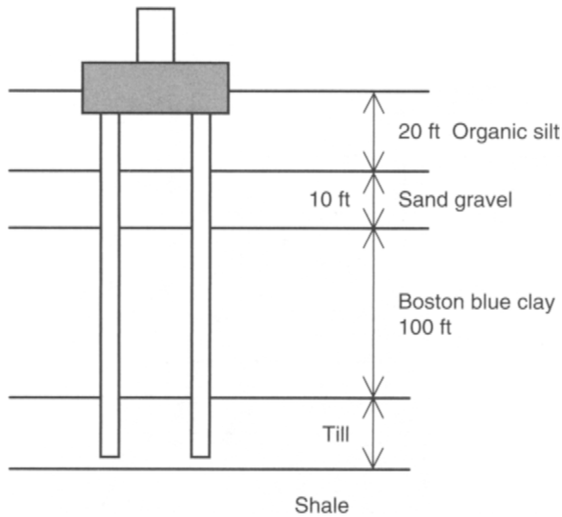

Figure 32.15 Deep piles

- Their performance was found to be excellent. Settlement readings in all buildings were less than 1 inch.
- Closed end concrete filled pipe piles were used. These piles were selected over H-piles due to their lower cost.
- During pile driving, adjacent buildings underwent slight upheaval. After completion of driving, these buildings started to settle. Great settlement in adjacent buildings within 50 ft of the piles was observed.
- Measured excess pore water pressures exceeded 40 ft of water column 15 ft away from the pile. Excess pore pressures dropped significantly after 10 to 40 days.
- Due to the upheaval of buildings, some piles were preaugured down to 15 ft, prior to driving. Preauguring reduced the generation of excess pore water pressures. In some cases, excess pore water pressures were still unacceptably high.
- Piles in a group were not driven at the same time. After one pile was driven, sufficient time was allowed for the pore water pressures to dissipate prior to driving the next pile.
- Another solution was to drive the pipe piles open end, and then clean out the piles. This was found to be costly and time-consuming.

32.5.7 Foundation Option 6: Floating Foundations Placed on Sand and Gravel (Rafts)

See Fig. 32.16.

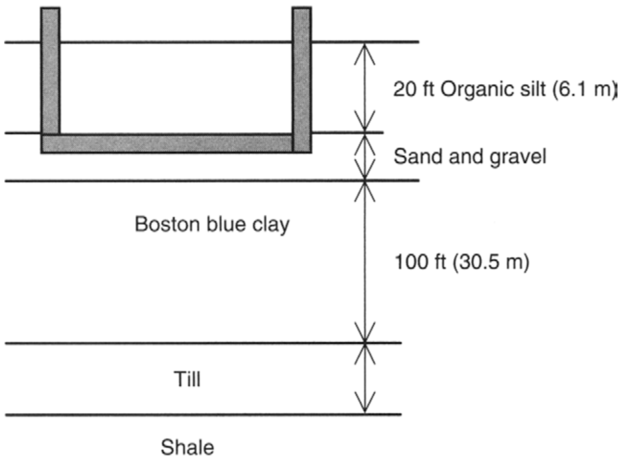

Figure 32.16 Floating foundation

The construction procedure for foundation option 6 was as follows.

- Mat or floating foundations were placed on sand and gravel.
- The settlement of floating foundations was larger than deep piles (option 5).
- The settlement of floating foundations varied from 1.0 to 1.5 in. The performance of rafts was inferior to deep piles. The average settlement in raft foundations was slightly higher than deep footings.
- There was a concern that excavations for rafts could create settlement in adjacent buildings. This was found to be a false alarm, since adjacent buildings did not undergo any major settlement due to braced excavations constructed for raft foundations.
- When the cost of driving deep piles was equal to the cost of raft foundations, it is desirable to construct raft foundations, since rafts would allow for a basement.
- A basement may not be available in the piling option, unless it is specially constructed with additional funds.

See Table 32.4.

Table 32.4 Settlement of buildings

Foundation type	Bearing stratum	Number of buildings	Average number of stories	Settlement
Timber piles	Boston blue clay	27	1–6	1–16 in.
Belled caissons	Sand and gravel	22	1–8	1–3 in.
Raft foundations	Sand and gravel	7	6–20	1–1.5 in.
Deep piles	Shale or till	5	6–30	0.5 in.

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33

Design of Pin Piles: Semi-Empirical Approach

33.1 Theory

Just a few decades ago, no engineer would have recommended any pile with less than a 9 in. diameter. Today, some piles are as small as a 4 in. diameter.

These small diameter piles are known as pin piles. They are also known by other names, such as mini piles, micro piles, GEWI piles, pali radice, root piles, and needle piles. See Fig. 33.1. The five parts of Fig. 33.1 are, briefly, as follows.

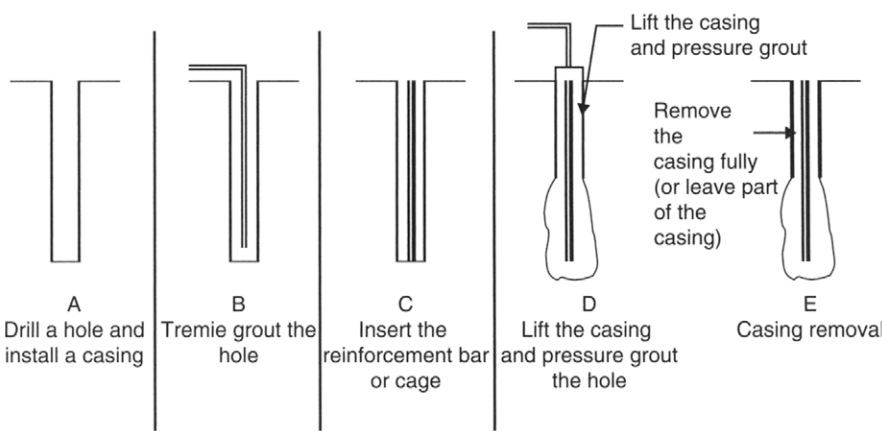

Figure 33.1 Pin pile procedure

A. Drill a hole using a roller bit or an augur. A casing is installed to stop soil from dropping into the hole. If the hole is steady, a casing may not be necessary.

B. Tremie grout the hole.

C. Insert the reinforcement bar or bars. In some cases more than one bar may be necessary.

D. Lift the casing and pressure grout the hole.

E. Fully remove the casing. Some engineers prefer to leave part of the casing. This would increase the strength and the cost.

33.1.1 Concepts to Consider

This section goes over Figure 33.1 in more detail.

Fig. 33.1*A*: Drilling the Hole

Auguring could be a bad idea for soft clays, since augurs tend to disturb the soil more than roller bits do. This would decrease the bond between soil and grout and would lower the skin friction. Drilling should be conducted with water. Drilling mud should not be used. Since casing is utilized to stop soil from falling in, drilling mud is not necessary. Furthermore, if drilling mud is used, it travels into the pores of the surrounding soil. This is particularly bad for the tremie grouting process. For proper bonding, the tremie grout must spread into the soil pores. When drilling mud occupies the pores, the tremie grout will not be able to spread into the soil pores. This would reduce the bond between grout and soil.

Fig. 33.1*B*: Tremie Grouting

After the hole is drilled, tremie grout is placed. Tremie grout is a cement/water mix. Tremie grout typically has a water content ratio of 0.45 to 0.5 by weight.

Fig. 33.1*C*: Placement of Reinforcement Bars

A high strength reinforcement bar (or number of bars wrapped together) or steel pipes can be used at the core. High strength reinforcement bars specially designed for pin piles are available from

Dywidag Systems International and William Anchors (leaders in the industry). These bars can be spliced easily, so that any length can be accommodated.

Fig. 33.1*D*: Lift the Casing and Pressure Grout

During the next step, the casing is lifted and the hole is pressure grouted. The pressure should be adequate to force the grout into the surrounding soil, to provide a good soil-grout bond. At the same time, the pressure should not be large enough to fracture the surrounding soil. A failure of the surrounding soil would drastically reduce the bond between grout and soil. This would decrease the skin friction. Typically grout pressure ranges from 0.5 MPa (10 ksf) to 1.0 MPa (where 1 MPa = 20.9 ksf).

Ground heave is another important aspect of pressure grouting. There are many instances where ground heave can occur during pressure grouting. This aspect needs to be considered during the design phase. If there are nearby buildings, action should be taken to avoid any grout flow into these buildings.

Fig. 33.1*E*: Remove the Casing

At the end of pressure grouting, the casing is completely removed. Some engineers prefer to leave part of the casing intact. Obviously, this would increase the cost. The casing would increase the rigidity and the strength of the pin pile. The casing would increase the *lateral resistance* of the pile significantly. Further, it is guaranteed that there is a pin pile with a diameter not less than the diameter of the casing. The diameter of a pin pile could be reduced due to soil encroachment into the hole. In some cases it is possible for the grout to spread into the surrounding soil and create an irregular pile. See Fig. 33.2.

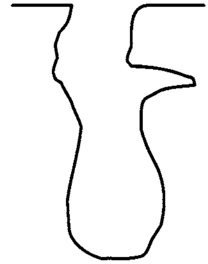

Figure 33.2 Irregular spread of grout

The grout could spread in an irregular manner, as shown in Fig. 33.2. This could happen when the surrounding soil is loose. The main disadvantage of leaving the casing in the hole is the additional cost.

33.2 Design of Pin Piles in Sandy Soils

The ultimate unit skin friction in gravity grouted pin piles is given by the Suzuki equation (Suzuki et al., 1972)

$$\tau = 21 \times (0.007N + 0.12)\ \text{ksf}$$

where

N = SPT value
τ = unit skin friction (in units of ksf)

Note that the above equation was developed based on empirical data.

Design Example 33.1

Compute the design capacity of the pin pile shown in Fig. 33.3 (diameter 6 in.). The surrounding soil has an average SPT (N) value of 15.

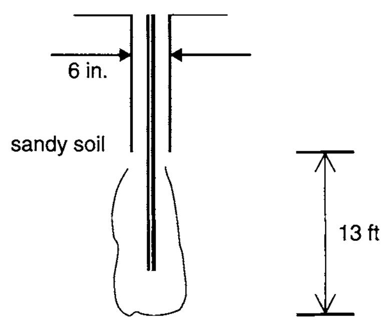

Figure 33.3 Pin pile in sandy soil

STEP 1: Find the ultimate skin friction.
Using the Suzuki equation for ultimate skin friction

$$\tau = 21 \times (0.007N + 0.12)\ \text{ksf}$$

For an N value of 15

$$\tau = 21 \times (0.007 \times 15 + 0.12)\ \text{ksf}$$

$$\tau = 4.725\ \text{ksf}$$

The diameter of the pile is 6 in., and the length of the grouted section is 13 ft.

STEP 2: Find the skin friction below the casing.

$$\text{skin friction below the casing} = \pi \times (6/12) \times 13 \times 4.725\,\text{kip}$$
$$= 96.5\,\text{kip} = 48.2\,\text{tons}$$

Note that most engineers ignore the skin friction along the casing. The end bearing capacity of pin piles is not significant in most cases, due to low cross sectional area. If the pin piles are seated on a hard soil or rock stratum, the end bearing needs to be accounted for.

Assume a factor of safety of 2.5. Therefore, the allowable capacity of the pin pile can be calculated as

$$48.2/2.5 = 19\,\text{tons}$$

Note that the Suzuki equation was developed for pin piles grouted using gravity. For pressure grouted pin piles, the ultimate skin friction can be increased by 20 to 40%.

Another equation was proposed by Littlejohn (Littlejohn, 1970)

$$\tau = 0.21N\,\text{ksf}$$

where

N = SPT value

For $N = 15$,

$$\tau = 0.21 \times 15 = 3.15\,\text{ksf}$$

The Suzuki equation yielded

$$\tau = 4.725\,\text{ksf}$$

For this case, the Littlejohn equation provides a conservative value for skin friction. If the Littlejohn equation was used for the above pin pile, then the ultimate skin friction and the allowable skin friction are

$$\text{ultimate skin friction} = \pi \times (6/12) \times 13 \times 3.15 = 64.3\,\text{kip}$$
$$\text{allowable skin friction} = 64.3/2.5 = 25\,\text{kip} = 12.5\,\text{tons}$$

The Suzuki equation yielded 19 tons for the same pin pile. See Fig. 33.4.

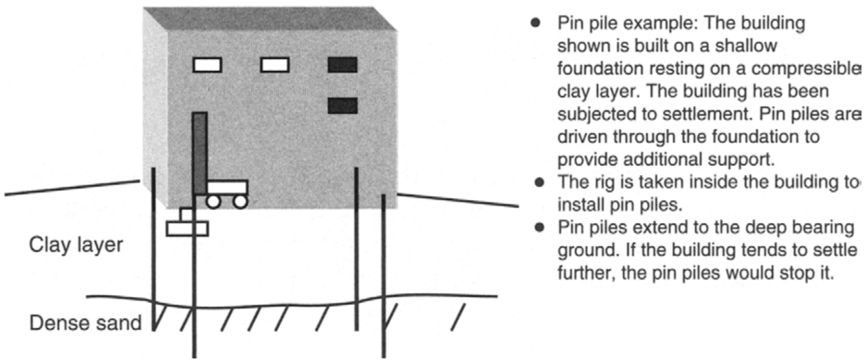

Figure 33.4 Pin piles located in dense sand

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Littlejohn, G. S. 1970. Soil anchors. *Ground Engineering Conference, Institution of Civil Engineers* (pp. 33–44).

Suzuki, I., Hirakawa, T., Morii, K., and Kanenko, K. 1972. Developments Nouveaux dans les Foundations de Plyons pour Lignes de Transport THT du Japon. *Conference Internationale de Grandes Reseaux Electriques a Haute Tension,* paper 21-01.

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34

Neutral Plane Concept and Negative Skin Friction

34.1 Introduction

During the pile driving process, the soil around the pile becomes stressed. See Fig. 34.1. Due to the high stress generated in the surrounding soil during pile driving, water is dissipated from the soil, as shown in Fig. 34.1a.

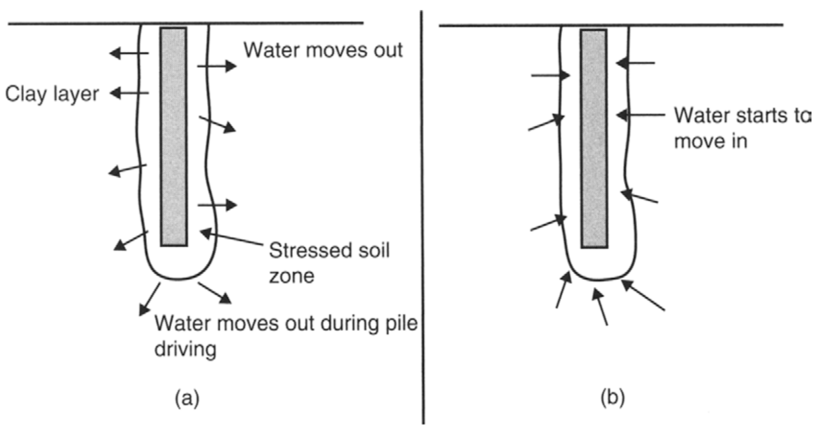

Figure 34.1 Water movement near a pile: (a) During pile driving, (b) After driving is completed

After pile driving is completed, water starts to come back to the previously stressed soil region around the pile, as shown in Fig. 34.1b. Due to the migration of water, the soil around the pile consolidates and settles. It should be mentioned that the settlement of soil surrounding the pile is very small, yet large enough to exert a downward force on the pile. This downward force is known as residual compression.

Due to the downward force exerted on the pile, the pile starts to move downward. Eventually the pile comes to an equilibrium and stops. At equilibrium, the upper layers of soil exert a downward force on the pile, while the lower layers of the soil exert an upward force on the pile. See Fig. 34.2.

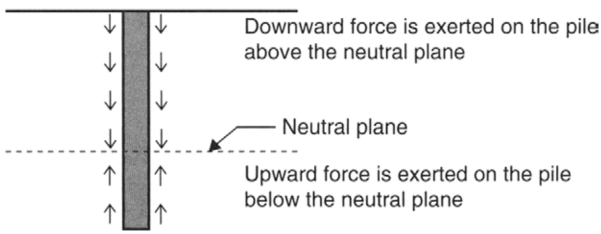

Figure 34.2 Neutral plane

The direction of the force reverses at the neutral plane.

34.1.1 Soil and Pile Movement Above the Neutral Plane

Both the soil and the pile move downward above the neutral plane. However, the soil moves slightly more in the downward direction than the pile does. Hence, relative to the pile, the soil moves downward. This downward movement of soil relative to the pile exerts a downward drag on the pile.

34.1.2 Soil and Pile Movement Below the Neutral Plane

Both the soil and the pile move downward below the neutral plane, as in the previous case. But this time, the pile moves slightly more in the downward direction relative to the soil. Hence, relative to the pile, the soil moves upward. This upward movement of the soil, relative to the pile, exerts an upward force on the pile.

34.1.3 Soil and Pile Movement at the Neutral Plane

Both the soil and the pile move downward at the neutral plane, as in the previous cases. But at the neutral plane, both the soil and the pile move downward by the same margin. Hence, the relative movement between the soil and the pile is zero. At the neutral plane, no force is exerted on the pile.

34.1.4 Location of the Neutral Plane

The exact location of the neutral plane cannot be estimated without elaborate techniques involving complicated mathematics. For floating piles, the neutral plane is taken to be 2/3 of the pile length. See Fig. 34.3.

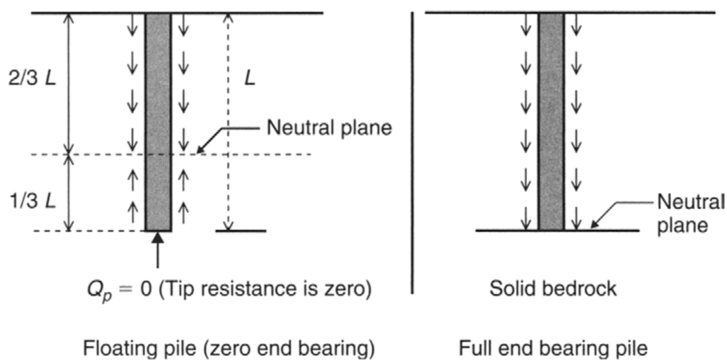

Figure 34.3 Neutral plane location

In the case of full end bearing piles (piles located on solid bedrock), the neutral plane lies at the bedrock surface. For an upward force to be generated on the pile, the pile has to move downward relative to the surrounding soil. Due to the solid bedrock, the pile is incapable of moving downward. Hence, the neutral plane lies at the bedrock surface.

34.2 Negative Skin Friction

Negative skin friction is the process where the skin friction due to soil acts in a downward direction. When the skin friction acts in a downward direction, the pile capacity decreases. Negative skin friction (also known as down drag), is a major problem in some sites.

34.2.1 Causes of Negative Skin Friction

One cause of negative skin friction is the placement of fill. Fill needs to be placed to gain required elevation for the floor slab. Additional fill consolidates the clay soils underneath. The settling clays then drag the pile down.

Another cause of negative skin friction is a change in groundwater level. When the groundwater level in a site goes down, the buoyancy force tends to diminish. Hence, the effective stress in clay increase, causing the clay layer to settle. The settling clay layer generates a negative skin friction on the pile. See Fig. 34.4.

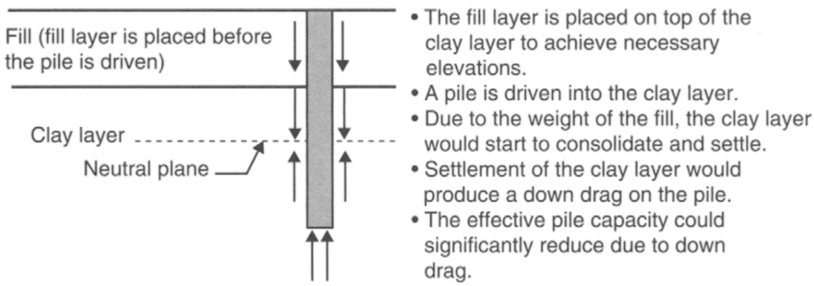

Figure 34.4 Negative skin friction

34.2.2 Summary

In most situations, fill has to be placed to achieve necessary elevations. The weight of the fill tends to consolidate the clay layer. Generally, the piles are driven after placing the fill layer. The settlement of the clay layer (due to the weight of the fill) induces a down drag force on the pile. If the piles were driven after full consolidation of the clay layer has occurred, no down drag would occur. Unfortunately, full consolidation may not be completed for years, and many developers will not wait that long to drive piles.

34.3 Bitumen Coated Pile Installation

Negative skin friction can effectively be reduced by providing a bitumen coat around the pile. See Fig. 34.5.

34.3.1 How Bitumen Coating Works Against Down Drag

See Fig. 34.6 for an examination of how a bitumen coating works against down drag.

The typical situation is shown in Fig. 34.7.

Note that due to larger fill load acting on the left half of the building, negative skin friction acting on pile *A* would be greater than on pile *B*.

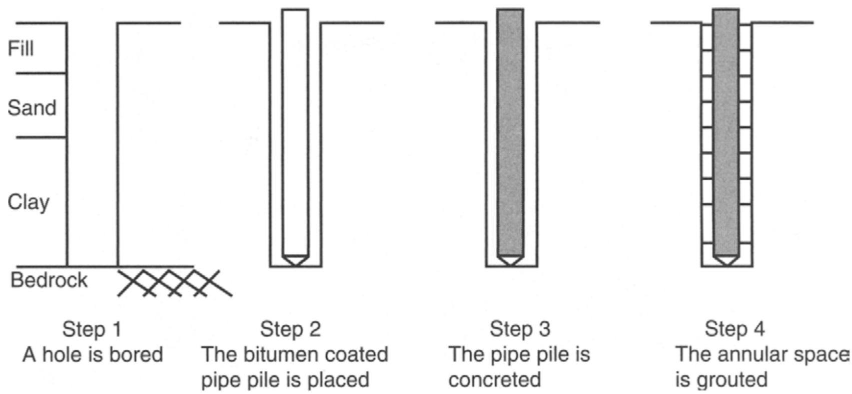

Figure 34.5 Bitumen coated pile installation

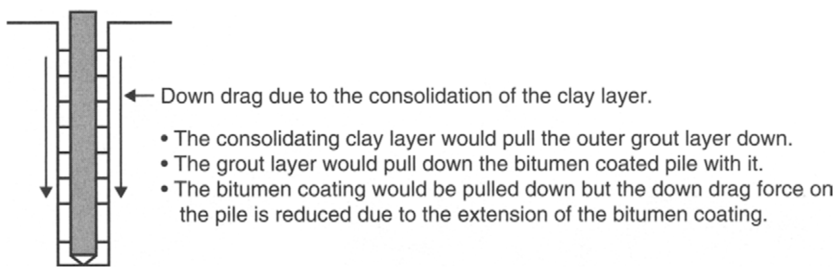

Figure 34.6 Bitumen coated pile mechanism

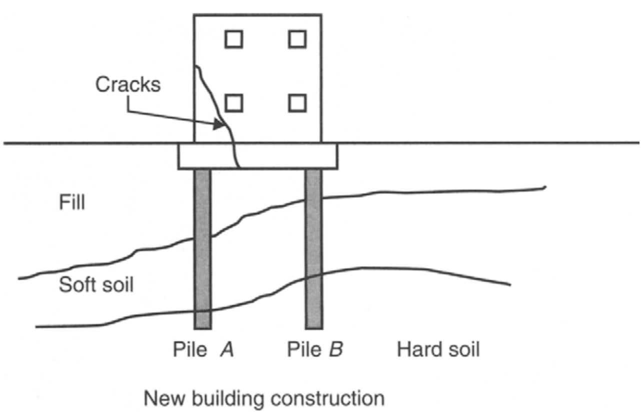

Figure 34.7 Negative skin friction due to fill

Hence, pile *A* would undergo larger settlement than pile *B*. Tension cracks could form as shown in Fig. 34.7 due to differential settlement.

35

Design of Caissons

35.1 Introduction

The term "caisson" is normally used to identify large bored concrete piles. Caissons are constructed by drilling a hole in the ground and filling it with concrete. A reinforcement cage is placed prior to concreting. The diameter of caissons could be as high as 15 ft.

The following is a list of other commonly used names to identify caissons.

1. Drilled shafts.
2. Drilled caissons.
3. Bored concrete piers.
4. Drilled piers.

A bell can be constructed at the bottom to increase the capacity. Caissons with a bell at the bottom are known as belled piers, belled caissons, or underreamed piers

35.2 Brief History of Caissons

It was clear to many engineers that large-diameter piles were required to transfer heavy loads to deep bearing strata. The obvious problem was the inability of piling rigs to drive these large-diameter piles.

The concept of excavating a hole and filling it with concrete was the next progression. The end bearing of the caisson was further improved by constructing a bell at the bottom of the hole.

Early excavations were constructed using steel rings and timber lagging. This was an idea borrowed from the tunneling industry. See Fig. 35.1.

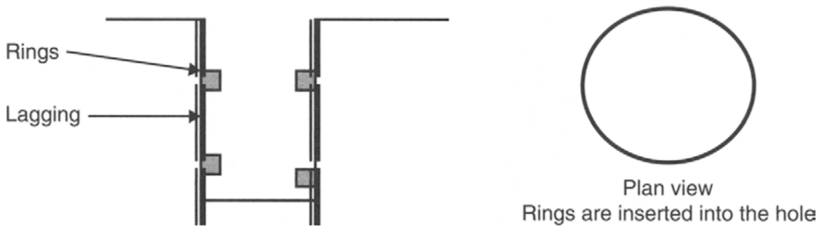

Figure 35.1 Ring structures

35.2.1 Machine Digging

With time, hand digging was replaced by machine digging. Two types of machines are in use.

- Auger types.
- Bucket types.

Auguring is the most popular method to excavate for caissons.

35.3 Caisson Design in Clay Soil

The equations for caissons in clay soils are similar to those of piles.

35.3.1 Different Methods

Reese et al. (1976) developed the relationship

$$\text{ultimate caisson capacity} = \text{ultimate end bearing capacity} + \text{ultimate skin friction}$$

$$P_u = Q_u + S_u$$

where

P_u = ultimate capacity of the caisson
Q_u = ultimate end bearing capacity
S_u = ultimate skin friction

The end bearing capacity, Q_u, is found by

$$Q_u = 9 \times c \times A$$

where

c = cohesion
A = cross-sectional area

The skin friction is

$$\text{skin friction} = \alpha \times c \times A_p$$

where

α is obtained from Table 35.1.
c = cohesion
A_p = perimeter surface area

The following notes apply to Table 35.1.

Table 35.1 α value and limiting skin friction

Caisson construction method	α (for soil to concrete)	Limiting unit skin friction (f, kPa)
Uncased caissons		
1. Dry or using lightweight drilling slurry	0.5	90
2. Using drilling mud where filter cake removal is uncertain (see Note 1 in text)	0.3	40
3. Belled piers on about same soil as on shaft sides		
3.1. By dry method	0.3	40
3.2. Using drilling mud where filter cake removal is uncertain	0.15	25
4. Straight or belled piers resting on much firmer soil than around shaft	0.0	0
5. Cased caissons (see Note 2 in text)	0.1 to 0.25	

Source: Reese et al. (1976).

- **Note 1—Filter Cake** When drilling mud is used, a layer of mud gets attached to the soil. This layer of mud is known as the filter cake. In some soils, this filter cake gets washed away during concreting. In some situations, it is not clear whether the filter cake will be removed during concreting. If the filter cake does not get removed during concreting, the bond between the concrete and the soil will be inferior. Hence, a low α value is expected.
- **Note 2—Cased Caissons** In the case of cased caissons, the skin friction is between the soil and the steel casing. α is significantly low for the steel-soil bond compared to the concrete-soil bond.

35.3.2 Factor of Safety

Due to the uncertainties of the soil parameters such as cohesion and friction angle, a factor of safety should be included. Suggested factor of safety (F.O.S.) values in the literature range from 1.5 to 3.0. The factor of safety of a caisson affects the economy of a caisson significantly. Assume that the ultimate capacity was found to be 3,000 kN and a factor of safety of 3.0 is used. In this case, the allowable caisson capacity would be 1,000 kN. On the other hand, a factor of safety of 2.0 would give an allowable caisson capacity of 1,500 kN, or a 50% increase in the design capacity. A lower factor of safety value would result in a much more economical design. Yet on the other hand, the failure of a caisson can never be tolerated. In the case of a pile group, the failure of one pile will probably not have a significant effect on the group. Caissons stand alone and typically are not used as a group.

When selecting the factor of safety, the following procedure is suggested.

- Make sure enough borings are conducted so that soil conditions are well known in the vicinity of the caisson.
- Sufficient unconfined strength tests should be conducted on all different clay layers to obtain an accurate value for cohesion.
- Sufficient borings with SPT (N) values must be made to obtain the friction angle of sandy soils.

If the subsurface investigation program is extensive, the lower factor of safety of 2.0 could be justified. If not, the factor of safety of 3.0 may be the safe bet. On the other hand, as a middle ground, a factor of safety 3.0 for skin friction and a factor of safety of 2.0 for end bearing can be used.

35.3.3 Weight of the Caisson

Unlike the case for piles, the weight of the caisson is significant and needs to be considered for the design capacity.

$$P_{\text{allowable}} = Q_u/\text{F.O.S.} + S_u/\text{F.O.S.} - W + W_s$$

where

Q_u = ultimate end bearing capacity
S_u = ultimate skin friction
$P_{\text{allowable}}$ = allowable column load
W = weight of the caisson
W_s = weight of soil removed

See Fig. 35.2.

Logically, the weight of the soil removed during construction of the caisson should be subtracted from the weight of the caisson.

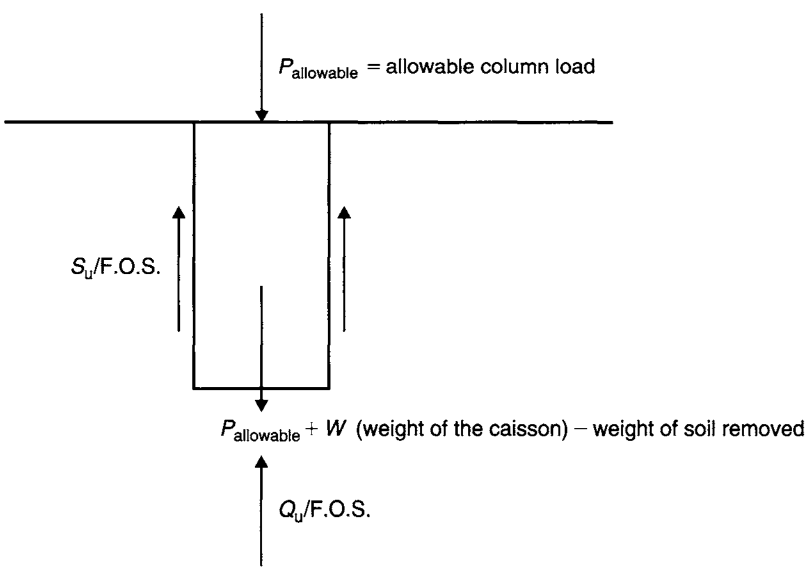

Figure 35.2 Forces on caissons

Research by Reese and O'Neill (1988) suggests ignoring the skin friction at the top 5 ft (1.5 m) of the caisson in clay since load tests suggested that the skin friction in this region is negligible. For the skin friction to mobilize, there must be a slight relative movement between the caisson and the surrounding soil. The top 5 ft of the caisson does not have enough relative movement between the caisson and the surrounding soil. It has been reported that clay at the top 5 ft could desiccate and crack and result in negligible skin friction. At the same time, the bottom of the shaft would undergo very little relative displacement in relation to the soil as well. Hence Reese and O'Neill suggest ignoring the skin friction at the bottom distance equal to the diameter of the caisson as well.

See Fig. 35.3. In Figure 35.3, D is the diameter of the shaft. Also, ignore the skin friction in the top 5 ft (1.5 m) and the bottom distance equal to the diameter of the shaft D.

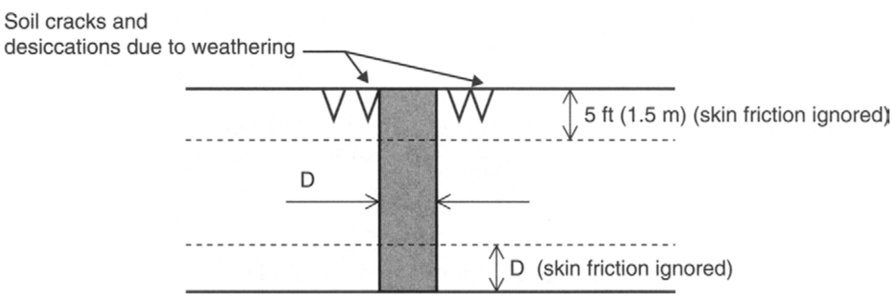

Figure 35.3 Desiccations on top of the caisson

35.3.4 AASHTO Method

AASHTO (American Association of State Highway and Transportation Officials) proposes the following method for caisson design in clay soils.

$$P_u = Q_u + S_u$$

where

P_u = ultimate capacity of the caisson
Q_u = ultimate end bearing capacity
S_u = ultimate skin friction

The end bearing capacity, Q_u, is calculated as

$$Q_u = 9 \times c \times A$$

where

c = cohesion
A = cross-sectional area

The skin friction is calculated as

$$\text{skin friction} = \alpha \times c \times A_p$$

where the adhesion factor, α, is obtained from Table 35.2, and where

c = cohesion
A_p = perimeter surface area

Table 35.2 Adhesion factor and maximum allowable skin friction

Location along drilled shaft	Value of α	Maximum allowable unit skin friction
From ground surface to depth along drilled shaft of 5 ft	0	0
Bottom 1 diameter of the drilled shaft	0	0
All other portions along the sides of the drilled shaft	0.55	5.5 ksf (253 kPa)

Source: AASHTO (American Association of State Highway and Transportation Officials).

It should be mentioned that the AASHTO method has only one value for α.

Design Example 35.1

This example explores caisson design in a single clay layer. Find the allowable capacity of a 2 m diameter caisson placed 10 m below the surface. The soil was found to be clay with a cohesion of 50 kPa. The caisson was constructed using the dry method. The density of the soil is 17 kN/m^3. The groundwater level is 2 m below the surface. See Fig. 35.4.

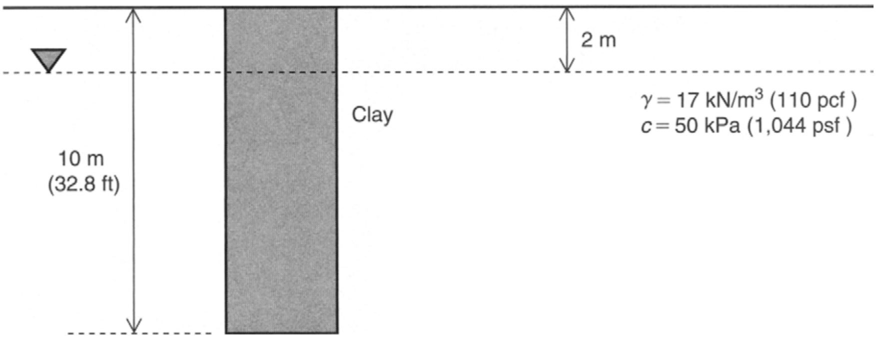

Figure 35.4 Caisson in clay

Solution

STEP 1: Find the end bearing capacity.

$$P_u = Q_u + S_u$$

end bearing capacity, $Q_u = 9 \times c \times A$

$$= 9 \times 50 \times (\pi \times d^2)/4$$

$$= 9 \times 50 \times (\pi \times 2^2)/4 = 1{,}413.8\,\text{kN}\,(318\,\text{kip})$$

STEP 2: Find the skin friction.

The cohesion of soil and the adhesion factors may be different above the groundwater level in the unsaturated zone. Unless the cohesion of the soil is significantly different above the groundwater level, such differences are ignored.

$$\text{skin friction} = \alpha \times c \times A_p$$

where

$\alpha = 0.5$ (from Table 35.1)

The caisson was constructed using the dry method.

$$A_p = \pi \times d \times L = \pi \times 2 \times L$$

The total length of the shaft is 10 m. Ignore the top 1.5 m of the shaft and the bottom D of the shaft for skin friction. The diameter of the shaft is 2 m.

effective length of the shaft = 10 − 1.5 − 2 = 6.5 m (21.3 ft)

$$A_p = \pi \times d \times L = \pi \times 2 \times 6.5 = 40.8\,\text{m}^2$$

$$\text{cohesion, } c = 50\,\text{kPa}$$

$$\text{skin friction} = 0.5 \times 50 \times 40.8\,\text{kN}$$

$$= 1{,}020\,\text{kN (229 kip)}$$

STEP 3: Find the allowable caisson capacity.

$$P_{\text{allowable}} = S_u/\text{F.O.S.} + Q_u/\text{F.O.S.} - \text{weight of the caisson}$$

$$+ \text{ weight of soil removed weight of the caisson}$$

$$W = \text{volume of the caisson} \times \text{density of concrete}$$

Assume the density of concrete to be 23.5 kN/m^3.

$$\text{weight of the caisson, } W = (\pi \times D^2/4 \times L) \times 23.5\,\text{kN}$$

$$= (\pi \times 2^2/4 \times 10) \times 23.5\,\text{kN}$$

$$= 738.3\,\text{kN (166 kip)}$$

Assume a factor of safety of 3.0 for skin friction and 2.0 for end bearing load.

$$P_{\text{allowable}} = S_u/\text{F.O.S.} + Q_u/\text{F.O.S.} - \text{weight of the caisson}$$

$$+ \text{ weight of soil removed}$$

$$P_{\text{allowable}} = 1{,}413.8/2.0 + 1{,}020/3.0 - 738.3 = 308\,\text{kN}$$

In this example, the weight of the removed soil was ignored.

Note that the $P_{\text{allowable}}$ obtained in this case is fairly low. Piles may be more suitable for this situation. If there is stiffer soil available at a lower depth, the caisson can be placed in a much stronger soil. Another option is to consider a bell.

Design Example 35.2

This example explores a caisson design in multiple clay layers. Find the allowable capacity of a 1.5 m diameter caisson placed at 15 m below the surface. The top layer was found to be clay with a cohesion of 60 kPa and the bottom layer was found to have a cohesion of 75 kPa. The top clay layer has a thickness of 5 m. The caisson was constructed using drilling mud where it is not certain that the filter cake will be removed during concreting. The density of soil is 17 kN/m^3 for both layers and the groundwater level is 2 m below the surface. See Fig. 35.5.

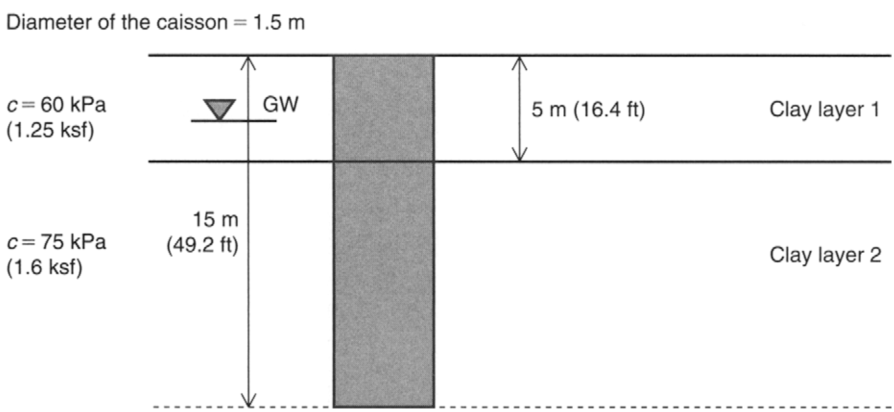

Figure 35.5 Caisson in multiple clay layers

Solution

STEP 1: Find the end bearing capacity.

$$P_u = Q_u + S_u$$

$$\text{end bearing capacity, } Q_u = 9 \times c \times A$$
$$= 9 \times 75 \times (\pi \times d^2)/4$$
$$= 9 \times 75 \times (\pi \times 1.5^2)/4$$
$$= 1{,}192.8\,\text{kN (268 kip)}$$

STEP 2: Find the skin friction in clay layer 1.

$$\text{skin friction} = \alpha \times c \times A_p$$

where

$\alpha = 0.3$ (from Table 35.1)

Assume that the filter cake will not be removed.

$$A_P = \pi \times d \times L = \pi \times 1.5 \times L$$

Ignore the top 1.5 m of the shaft and the bottom D of the shaft for skin friction. Find the effective length of the shaft in clay layer 1

$$5 - 1.5 = 3.5\,\text{m}$$

$$A_P = \pi \times d \times L = \pi \times 1.5 \times 3.5 = 16.5\,\text{m}^2$$

$$\text{cohesion, } c = 60\,\text{kPa}$$

$$\text{skin friction} = 0.3 \times 60 \times 16.5\,\text{kN}$$

$$= 297\,\text{kN (67 kip)}$$

STEP 3: Find the skin friction in clay layer 2.

$$\text{skin friction} = \alpha \times c \times A_P$$

where

$\alpha = 0.3$ (from Table 35.1)

Assume that the filter cake will not be removed.

$$A_P = \pi \times d \times L = \pi \times 1.5 \times L$$

Ignore the bottom D of the shaft. The effective length of the shaft in clay layer 2 is

$$10 - \text{D} = 10 - 1.5 = 8.5\,\text{m}$$

$$A_P = \pi \times d \times L = \pi \times 1.5 \times 8.5 = 40.1\,\text{m}^2$$

$$\text{cohesion, } c = 75\,\text{kPa}$$

$$\text{skin friction} = 0.3 \times 75 \times 40.1\,\text{kN}$$

$$= 902.2\,\text{kN (203 kip)}$$

$$\text{total skin friction} = 297 + 902.2 = 1{,}199.2\,\text{kN}$$

STEP 4: Find the allowable caisson capacity.

$$P_{\text{allowable}} = S_u/\text{F.O.S.} + Q_u/\text{F.O.S.} - \text{weight of the caisson} + \text{weight of soil removed}$$

$$\text{weight of the caisson, } W = \text{volume of the caisson} \times \text{density of concrete}$$

Assume density of concrete to be 23.5 kN/m^3.

$$\begin{aligned}\text{weight of the caisson, } W &= (\pi \times D^2/4 \times L) \times 23.5\,\text{kN}\\ &= (\pi \times 1.5^2/4 \times 15) \times 23.5\,\text{kN}\\ &= 622.9\,\text{kN}\end{aligned}$$

Assume a factor of safety of 3.0 for skin friction and 2.0 for the end bearing load.

$$P_{\text{allowable}} = S_u/\text{F.O.S.} + Q_u/\text{F.O.S.} - \text{weight of the caisson}$$

$$P_{\text{allowable}} = 1{,}199.2/3.0 + 1{,}192.8/2.0 - 622.9 = 373.2\,\text{kN (84 kip)}$$

The weight of the soil removed is ignored in this example.

35.4 Meyerhoff Equation for Caissons

35.4.1 End Bearing Capacity

Meyerhoff proposed the following equation based on the SPT (N) value to compute the ultimate end bearing capacity of caissons. The Meyerhoff equation was adopted by DM 7.2 as an alternative method to static analysis.

$$q_{\text{ult}} = 0.13\, c_N \times N \times D/B$$

where

q_{ult} = ultimate point resistance of caissons (tsf)
N = standard penetration resistance (blows/ft) near pile tip

$$c_N = 0.77 \log 20/p$$

where

p = effective overburden stress at pile tip (tsf)

Note that p should be more than 500 psf. It is very rare for the effective overburden stress at the pile tip to be less than 500 psf.

D = depth driven into granular (sandy) bearing stratum (ft)
B = width or diameter of the pile (ft)
q_1 = limiting point resistance (tsf), equal to $4N$ for sand and $3N$ for silt

35.4.2 Modified Meyerhoff Equation

Meyerhoff developed the above equation using many available load test data and obtaining average N values. Pile tip resistance is a function of the friction angle. For a given SPT (N) value, different friction angles are obtained for different soils.

For a given SPT (N) value, the friction angle for coarse sand is 7 to 8% higher compared to medium sand. At the same time, for a given SPT (N) value, the friction angle is 7 to 8% lower in fine sand compared to medium sand. Hence, the following modified equations are proposed.

$$\text{for coarse sand, } q_{\text{ult}} = 0.15\, c_{\text{N}} \times N \times D/B \,\text{tsf}$$

$$\text{for medium sand, } q_{\text{ult}} = 0.13\, c_{\text{N}} \times N \times D/B \,\text{tsf}$$

$$\text{for fine sand, } q_{\text{ult}} = 0.12\, c_{\text{N}} \times N \times D/B \,\text{tsf}$$

Design Example 35.3

Find the tip resistance of the 4 ft diameter caisson shown in Fig. 35.6 using the modified Meyerhoff equation. The SPT (N) value at the caisson tip is 15 blows per foot.

Solution

Pile capacity comes from tip resistance and skin friction. In this example, only the tip resistance is calculated.

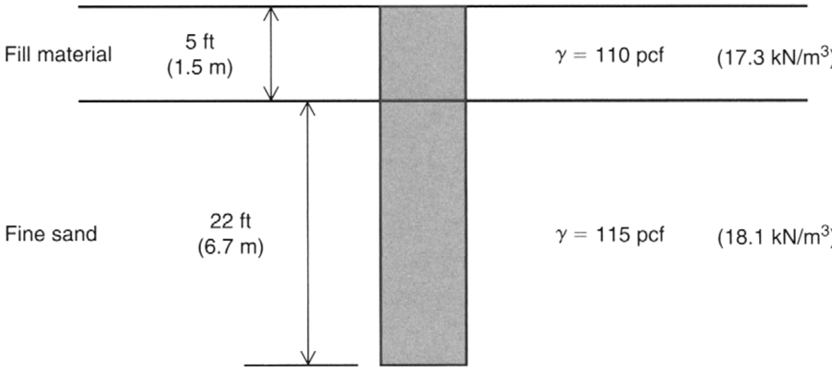

Figure 35.6 Caisson in fine sand

Find the ultimate point resistance for driven piles for fine sand.

$$q_{tip} = 0.12\, c_N \times N \times D/B \text{ tsf (for fine sand)}$$

$$c_N = 0.77 \log 20/p$$

$$p = \text{effective overburden stress at pile tip (tsf)}$$

$$p = 5 \times 110 + 22 \times 115 = 3{,}080 \text{ psf} = 1.54 \text{ tsf (147 kPa)}$$

$$c_N = 0.77 \log(20/1.54) = 0.86$$

$$D = \text{depth into bearing stratum} = 22 \text{ ft (6.7 m)}$$

Fill material is not considered to be a bearing stratum.

$$B = 4 \text{ ft (width or diameter of the pile)}$$

$$q_{tip} = 0.12\, C_N \times N \times D/B \text{ (fine sand)}$$

$$q_{tip} = 0.12 \times 0.86 \times 15 \times 22/4 = 8.5 \text{ tsf (814 kPa)}$$

$$\text{maximum allowable point resistance} = 4N \text{ tsf for sandy soils}$$

$$4 \times N = 4 \times 15 = 60 \text{ tsf}$$

Hence

$$q_{tip} = 8.5 \text{ tsf}$$

$$\text{allowable point bearing capacity} = 8.5/\text{F.O.S.}$$

Assume a factor of safety of 3.0. Hence, the total allowable point bearing capacity can be determined.

$$q_{\text{tipallowable}} = 2.84\,\text{tsf}\ (272\,\text{kPa})$$

$$Q_{\text{tipallowable}} \times \text{ tip area} = q_{\text{tipallowable}} \times \pi \times (4^2)/4$$

$$= 36\,\text{tons}\ (320\,\text{kN})$$

Only the tip resistance was computed in this example.

35.4.3 Meyerhoff Equation for Skin Friction

Meyerhoff proposes the following equation for skin friction for caissons.

$$f = N/100\,\text{tsf}$$

where

f = unit skin friction (tsf)
N = average SPT (N) value along the pile

Note that as per Meyerhoff, the unit skin friction f should not exceed 1 tsf.

The author has modified the Meyerhoff equations to account for soil gradation.

$$\text{for coarse sand, } f = N/92\,\text{tsf}$$

$$\text{for medium sand, } f = N/100\,\text{tsf}$$

$$\text{for fine sand, } f = N/108\,\text{tsf}$$

Design Example 35.4

Find the skin friction of the 4 ft diameter caisson shown in Fig. 35.7 using the Meyerhoff equation. The average SPT (N) value along the shaft is 15 blows per foot. Ignore the skin friction in the fill material.

Solution

Only the skin friction is calculated in this example.

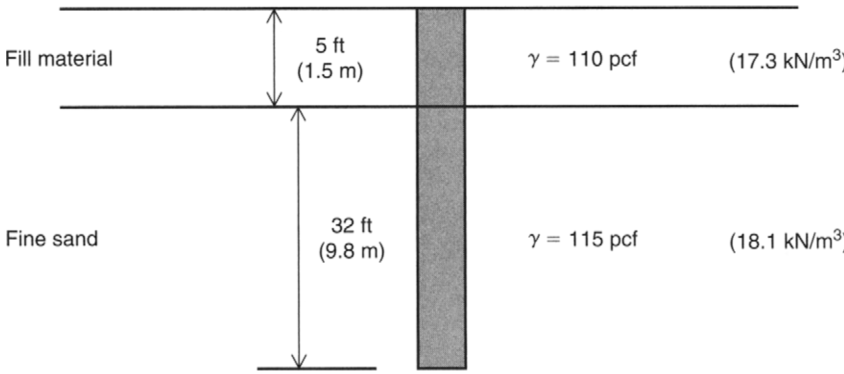

Figure 35.7 Caisson design using Meyerhoff method

Calculate the skin friction.
For fine sand the unit skin friction can be calculated as

$$
\begin{aligned}
f &= N/108\,\text{tsf} \\
&= 15/108\,\text{tsf} \\
&= 0.14\,\text{tsf}
\end{aligned}
$$

$$
\begin{aligned}
\text{total skin friction} &= \text{unit skin friction} \times \text{perimeter surface area} \\
&= 0.14 \times \pi \times D \times L \\
&= 0.14 \times \pi \times 4 \times 32 = 56\,\text{tons (498 kN)}
\end{aligned}
$$

$$\text{allowable skin friction} = 56/\text{F.O.S.}$$

Assume a factor of safety (F.O.S.) of 3.0.

$$\text{allowable skin friction} = 56/3.0 = 18.7\,\text{tons (166 kN)}$$

35.4.4 AASHTO Method for Calculating End Bearing Capacity

AASHTO adopts the method proposed by Reese and O'Neill (1988). The ultimate end bearing capacity in caissons placed in sandy soils is given by:

$$
\begin{aligned}
Q_u &= q_t \times A \\
q_t &= 1.20\,N\,\text{ksf}\ (0 < N < 75)
\end{aligned}
$$

where

N = standard penetration test value (blows/ft)

Alternately, in metric units

$$q_t = 57.5\,N\,\text{kPa}\ (0 < N < 75)$$

A = cross-sectional area at the bottom of the shaft

For all N values above 75,

$$q_t = 4{,}310\,\text{kPa (in metric units)}$$
$$q_t = 90\,\text{ksf (in psf units)}$$

35.5 Belled Caisson Design

Belled caissons are used to increase the end bearing capacity. Unfortunately, one loses the skin friction in the bell area since experiments have shown that the skin friction in the bell to be negligible (Reese 1976). In addition Reese and O'Neill (1988) and AASHTO suggest excluding the skin friction in a length equal to the shaft diameter above the bell. See Fig. 35.8.

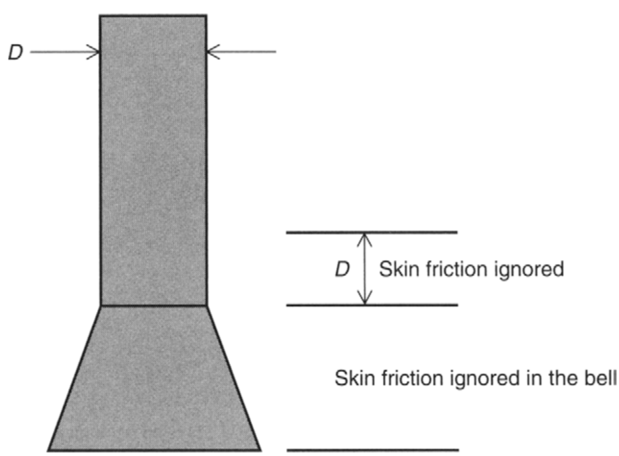

Figure 35.8 Belled caisson

Belled caissons are usually placed in clay soils. It is almost impossible to create a bell in sandy soils. On rare occasions, with special equipment, belled caissons can be constructed in sandy soils.

To restate, the skin friction in the bell and one diameter of the shaft, D, above the bell is ignored. The ultimate capacity of belled caissons is given by the following equation.

ultimate capacity of the belled caisson = ultimate end bearing capacity + ultimate skin friction

$$P_{\mathrm{u}} = Q_{\mathrm{u}} + S_{\mathrm{u}}$$

$$Q_{\mathrm{u}} = 9 \times c \times \text{bottom cross-sectional area of the bell}$$

where

c = cohesion

$$\text{area of the bell} = \pi \times d_{\mathrm{b}}^2/4$$

where

d_{b} = bottom diameter of the bell

$$S_{\mathrm{u}} = \alpha \times c \times \text{perimeter surface area of the shaft (ignoring the bell)}$$

$$S_{\mathrm{u}} = \alpha \times c \times (\pi \times d \times L)$$

d = diameter of the shaft
L = length of the shaft portion
α = 0.55 for all soil conditions unless determined otherwise by experiment (AASHTO)

Design Example 35.5

Find the allowable capacity of the belled caisson shown in Fig. 35.9. The diameter of the bottom of the bell is 4 m and the height of the bell is 2 m. The diameter of the shaft is 1.8 m and the height of the shaft is 10 m. The cohesion of the clay layer is 100 kN/m^2. The adhesion

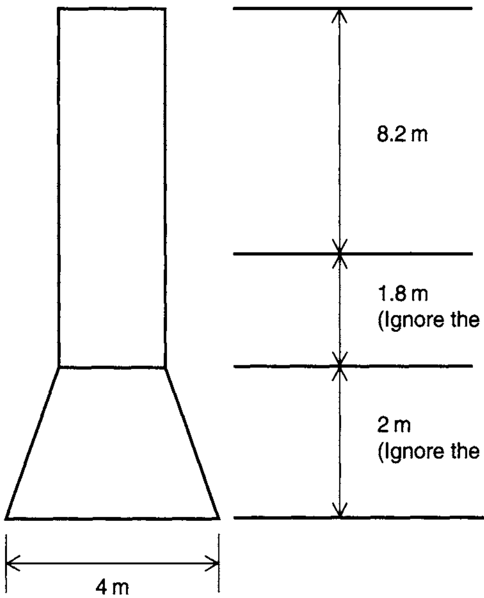

Figure 35.9 Skin friction in belled caisson

factor, α was found to be 0.55. Ignore the skin friction in the bell and one diameter of the shaft above the bell.

Solution

STEP 1: Find the ultimate caisson capacity, P_u.

$$P_u = Q_u + S_u - W$$

$$\begin{aligned} Q_u &= \text{ultimate end bearing capacity} \\ &= 9 \times c \times (\text{area of the bottom of the bell}) \\ &= 9 \times 100 \times (\pi \times 4^2/4) = 11{,}310\,\text{kN}\ (2{,}543\,\text{kip}) \end{aligned}$$

S_u = ultimate skin friction
W = weight of the caisson

STEP 2: Find the ultimate skin friction, S_u.

$$\begin{aligned} S_u &= \alpha \times c \times (\pi \times d \times L) \\ &= 0.55 \times 100 \times (\pi \times 1.8 \times 8.2) \\ &= 2{,}550\,\text{kN}\ (573\,\text{kip}) \end{aligned}$$

The height of the shaft is 10 m, and a length equal to one diameter of the shaft above the bell is ignored. Hence, the effective length of the shaft is 8.2 m.

STEP 3: Find the weight of the caisson.
Assume the density of concrete to be 23 kN/m^3.

$$\text{weight of the shaft, } W = (\pi \times d^2/4) \times 10 \times 23$$
$$= (\pi \times 1.8^2/4) \times 10 \times 23\,\text{kN} = 585.3\,\text{kN (131 kip)}$$

Find the weight of the bell.

$$\text{average diameter of the bell, } d_a = (1.8 + 4)/2 = 2.9\,\text{m}$$

Use the average diameter of the bell, d_a, to find the volume of the bell.

$$\text{volume of the bell} = \pi \times d_a^2/4 \times h$$

where

h = height of the bell

$$\pi \times 2.9^2/4 \times 2 = 13.21\,\text{m}^3$$
$$\text{weight of the bell, } W = \text{volume} \times \text{density of concrete} = 13.21 \times 23$$
$$= 303.8\,\text{kN (68.3 kip)}$$

STEP 4: Find the ultimate caisson capacity and the allowable caisson capacity.

$$\text{ultimate caisson capacity, } P_u = Q_u + S_u - \text{weight of the caisson} + \text{weight of soil removed}$$
$$\text{allowable caisson capacity} = 11{,}310/\text{F.O.S.} + 2{,}550/\text{F.O.S.} - 585.3 - 303.8$$

Assume a factor of safety of 2.0 for the end bearing and 3.0 for skin friction. Since the weight of the caisson is known fairly accurately, no safety factor is needed. The weight of the removed soil is ignored in this example.

allowable caisson capacity $= 11{,}310/2.0 + 2{,}550/3.0 - 585.3 - 303.8$
$= 5{,}615\,\text{kN}\,(1{,}262\,\text{kip})$

Design Example 35.6

Find the allowable capacity of the shaft in the previous example, assuming a bell was not constructed. Assume the skin friction is mobilized over the full length of the shaft. See Fig. 35.10.

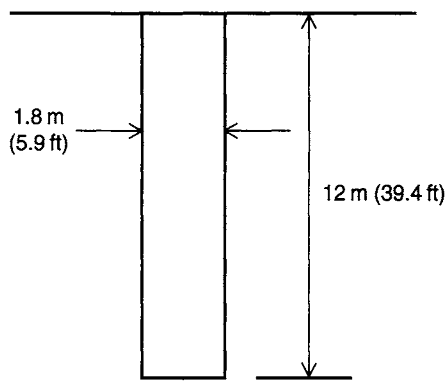

Figure 35.10 Caisson without a bell

Solution

STEP 1: Find the ultimate end bearing capacity, Q_u.

$$Q_u = 9 \times c \times (\pi \times 1.8^2/4)$$

Since there is no bell, the new diameter at the bottom is 1.8 m.

$$Q_u = 9 \times 100 \times (\pi \times 1.8^2/4) = 2{,}290\,\text{kN (515 kip)}$$

STEP 2: Find the ultimate skin friction, S_u.

$$S_u = \alpha \times c \times (\pi \times 1.8) \times 12$$

The new height of the shaft is 12 m, since a bell is not constructed.

$$S_u = 0.55 \times 100 \times (\pi \times 1.8) \times 12 = 3{,}732\,\text{kN (839 kip)}$$

Note that an assumption is made that the skin friction of the shaft is mobilized along the full length of the shaft for 12 m.

STEP 3: Find the weight of the shaft.

$$W = \text{weight of the shaft} = (\pi \times d^2/4) \times L \times \text{ density of concrete}$$

$$W = \text{weight of the shaft} = (\pi \times 1.8^2/4) \times 12 \times 23 = 702.3\,\text{kN (158 kip)}$$

STEP 4: Find the allowable caisson capacity.

$$\text{allowable caisson capacity} = Q_u/\text{F.O.S.} + S_u/\text{F.O.S.} - \text{weight of caisson} + \text{weight of soil removed}$$

Assume a factor of safety of 2.0 for the end bearing and 3.0 for skin friction. Since the weight of the caisson is known fairly accurately, no safety factor is needed.

$$\text{allowable caisson capacity} = 2{,}290/2.0 + 3{,}732/3.0 - 702.3\,\text{kN} = 1{,}686.7\,\text{kN}$$

Note that the weight of the removed soil is ignored in this example.

In the previous example, we found the allowable capacity with the bell to be 5,615 kN, significantly higher than the straight shaft.

Design Example 35.7

Find the allowable capacity of the belled caisson shown in Fig. 35.11. The diameter of the bottom of the bell is 4 m and the height of the bell is 2 m. The diameter of the shaft is 1.8 m and the height of the shaft is 11.8 m. The cohesion of the clay layer is 100 kN/m^2. The adhesion factor, α, was found to be 0.5. Ignore the skin friction in the bell and one diameter above the bell.

Solution

STEP 1: Find the ultimate caisson capacity, P_u.

$$P_u = Q_u + S_u - \text{weight of caisson} + \text{weight of soil removed}$$

$$\begin{aligned} Q_u &= \text{ultimate end bearing capacity} \\ &= 9 \times c \times (\text{area of the bottom of the bell}) \\ &= 9 \times 100 \times (\pi \times 4^2/4) \\ &= 11{,}310\,\text{kN (2,542 kip)} \end{aligned}$$

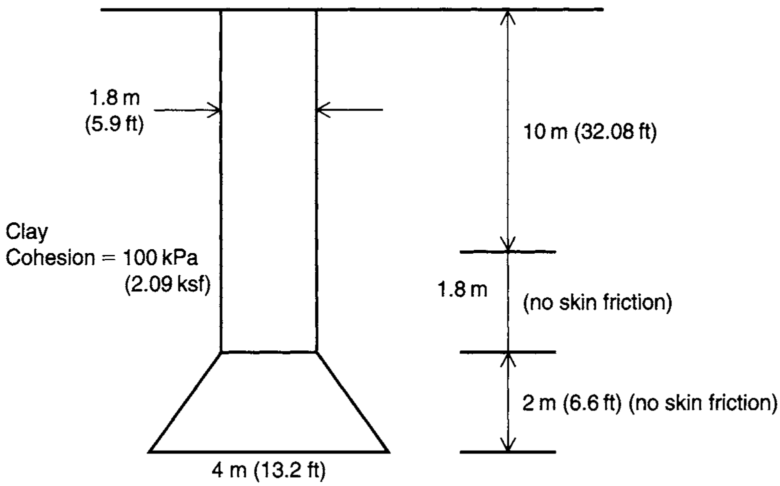

Figure 35.11 Belled caisson in clay

where

S_u = ultimate skin friction

STEP 2: Find the ultimate skin friction, S_u.

$$S_u = \alpha \times c \times (\pi \times d \times L)$$
$$= 0.5 \times 100 \times (\pi \times 1.8 \times 10)$$
$$= 2{,}827\,\text{kN}\ (636\,\text{kip})$$

The skin friction for 1.8 m (length equal to diameter of the shaft) is ignored.

STEP 3: Find the weight of the caisson.
Assume the density of concrete is 23 kN/m^3.

$$\text{weight of the shaft} = (\pi \times d^2/4) \times 11.8 \times 23$$
$$= (\pi \times 1.8^2/4) \times 11.8 \times 23\,\text{kN} = 690.6\,\text{kN}\ (155\,\text{kip})$$

Find the weight of the bell.

average diameter of the bell, $d_a = (1.8 + 4)/2 = 2.9\,\text{m}\ (9.5\,\text{ft})$

Use the average diameter of the bell, d_a to find the volume of the bell.

$$\text{volume of the bell} = \pi \times d_a^2/4 \times h$$

where

h = height of the bell

$$\pi \times 2.9^2/4 \times 2 = 13.21\,\text{m}^3$$

$$\begin{aligned}\text{weight of the bell} &= \text{volume} \times \text{density of concrete}\\ &= 13.21 \times 23 = 303.8\,\text{kN}\end{aligned}$$

STEP 4: Find the ultimate caisson capacity, P_u, and the allowable caisson capacity.

$$P_u = Q_u + S_u - \text{weight of caisson} + \text{weight of soil removed}$$

$$\begin{aligned}\text{allowable caisson capacity} &= 11{,}310/\text{F.O.S.} + 2{,}827/\text{F.O.S.}\\ &\quad -690.6 - 303.8\end{aligned}$$

Note that the weight of the removed soil was ignored in this example.

Assume a factor of safety of 2.0 for the end bearing and 3.0 for skin friction. Since the weight of the caisson is known fairly accurately, no safety factor is needed.

$$\begin{aligned}\text{allowable caisson capacity} &= 11{,}310/2.0 + 2{,}827/3.0 - 690.6 - 303.8\\ &= 5{,}602\,\text{kN (1,259 kip)}\end{aligned}$$

35.6 Caisson Design in Rock

35.6.1 Caissons Under Compression

See Fig. 35.12.

Note that most of the load is taken by skin friction in rock. In most cases, the end bearing load is less than 5% of the total load. In other words, 95% or more of the load will be carried by skin friction, developed in rock.

35.6.2 Simplified Design Procedure

In the next example, the simplified design procedure is first explained. In this procedure, the composite nature of the section is ignored.

Design Example 35.8

Design a concrete caisson with a W-section (steel) at the core to carry a load of 1,000 tons. Assume the skin friction to be 150 psi and the

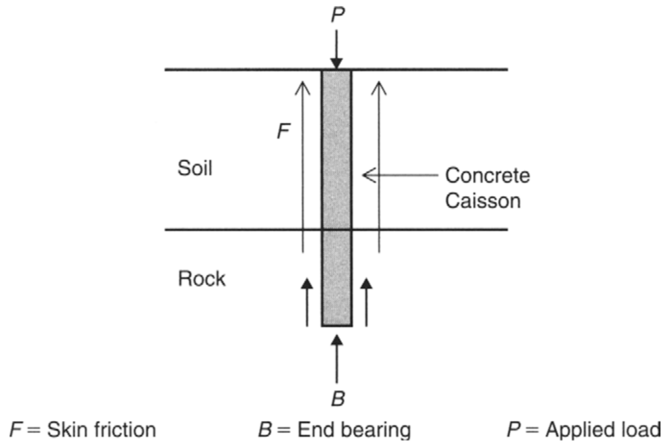

Figure 35.12 Caisson in rock

end bearing to be 200 psi. See Fig. 35.13. The following parameters are given. The ultimate steel compressive strength is 36,000 psi, and the ultimate concrete compressive strength is 3,000 psi.

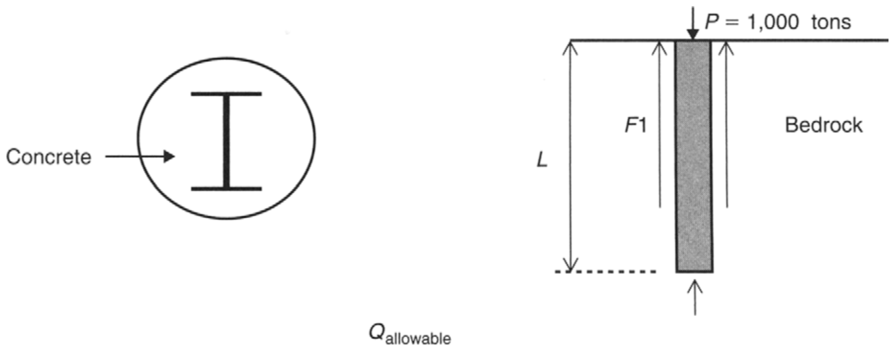

Figure 35.13 Concrete caisson with a W-section

STEP 1: Structural design of the caisson.

Assume a diameter of 30 in. for the concrete caisson. Since E (the elastic modulus) of steel is much higher than for concrete, the major portion of the load is taken by the steel. Assume 90% of the load is carried by the steel.

$$\text{load carried by steel} = 0.9 \times 1{,}000\,\text{tons} = 900\,\text{tons}$$

$$\text{allowable steel compressive strength} = 0.5 \times 36{,}000 = 18{,}000\,\text{psi}$$

(It should be noted that in the New York City Building Code, Table 11.3 of that code recommends a factor of safety of 0.5. Check your local building code for the factor of safety value.)

$$\text{steel area required} = \frac{(900 \times 2{,}000)}{18{,}000} = 100\,\text{sq in.}$$

Check the manual of steel construction for an appropriate W-section.

Use W 14 × 342. This section has an area of 101 sq in. The dimensions of this section are given in Fig. 35.14.

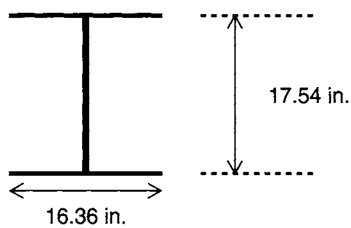

Figure 35.14 I-beam

STEP 2: Check whether this section will fit inside a 30 in. hole.

The distance along the diagonal can be calculated using the Pythagorean theorem.

$$(16.36^2 + 17.54^2)^{1/2} = 23.98\,\text{in.}$$

This value is smaller than 30 in. Hence the section can easily fit inside a 30 in. hole.

STEP 3: Compute the load carried by concrete.

$$\begin{aligned}\text{concrete area} &= \text{area of the hole} - \text{area of steel}\\ &= 706.8 - 101 = 605.8\end{aligned}$$

$$\begin{aligned}\text{allowable concrete compressive strength} &= 0.25\\ &\times \text{ultimate compressive strength}\\ &= 0.25 \times 3{,}000 = 750\,\text{psi}\end{aligned}$$

(It should be noted that in the New York City Building Code, Table 11.3 of that code recommends a factor of safety of 0.25. Engineers should refer to local building codes for the relevant factor of safety values.)

$$\text{load carried by concrete} = \text{concrete area} \times 750\,\text{psi}$$
$$= 605.8 \times 750\,\text{lb}$$
$$= 227.6\,\text{tons}$$
$$\text{load carried by steel} = 900\,\text{tons (computed earlier)}$$
$$\text{total capacity of the caisson} = 900 + 227.6 = 1{,}127.6\,\text{tons} > 1{,}000\,\text{tons}$$

Note that the designer can start with a smaller steel section to optimize the above value.

STEP 4: Compute the required length, L, of the caisson.
The skin friction is developed along the perimeter of the caisson.

$$\text{total perimeter of the caisson} = \pi \times (\text{diameter}) \times (\text{length}) = \pi \times D \times L$$
$$\text{total skin friction} = \pi \times 30 \times L \times \text{unit skin friction of rock (in this case 150 psi)}$$
$$\text{total skin friction} = \pi \times 30 \times L \times 150\,\text{lb} \quad (L \text{ should be in units of inches})$$

Design the caisson so that 95% of the load is carried by skin friction.

$$0.95 \times 1{,}000\,\text{tons} = 950\,\text{tons}$$

Hence, the total load carried through skin friction is 950 tons.

$$\text{total skin friction} = \pi \times 30 \times L \times 150 = 950\,\text{tons} = 950 \times 2{,}000\,\text{lb}$$
$$\text{length of the caisson required, } L = 134\,\text{in.} = 11.1\,\text{ft}$$

Design Example 35.9

The following parameters are given. See Fig. 35.15. The caisson diameter is 4 ft. The compressive strength of steel is 36,000 psi.

$$E_r/E_c = 0.5$$

where

E_r = elastic modulus of rock
E_c = elastic modulus of concrete

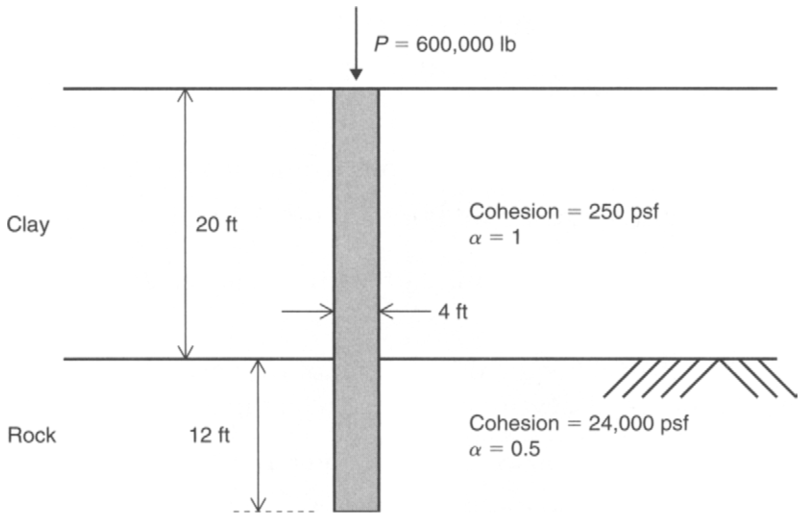

Figure 35.15 Caisson in rock (example)

cohesion of the bedrock = 24,000 psf
adhesion coefficient, α, for rock = 0.5
adhesion coefficient, α, for clay = 1.0

Solution

STEP 1: Compute the ultimate end bearing capacity.
q_u = ultimate end bearing strength of the bedrock = $N_c \times$ cohesion
where

$$N_c = 9$$

Hence

$$q_u = 9 \times \text{cohesion} = 216{,}000\,\text{psf}$$

$$\text{ultimate end bearing capacity, } Q_u = \text{area} \times q_u$$

$$Q_u = (\pi \times 4^2/4) \times 216{,}000 = 1{,}357\,\text{tons}$$

$$\text{allowable end bearing capacity, } Q_{\text{allowable}} = 1{,}357/3 = 452\,\text{tons}$$

STEP 2: Compute the ultimate skin friction.
ultimate unit skin friction per unit area within the rock mass, $f = \alpha \times c = \alpha \times 24,000$

where

adhesion coefficient, $\alpha = 0.5$

Hence

$$\text{ultimate unit skin friction, } f = 0.5 \times 24{,}000 = 12{,}000\,\text{psf}$$

Assuming a factor of safety of 3.0, the allowable unit skin friction per unit area within the rock mass is

$$f_{\text{allowable}} = 12{,}000/3.0 = 4{,}000\,\text{psf} = 2\,\text{tsf}$$

STEP 3: Find the skin friction within the soil layer.

The skin friction generated within the soil layer can be calculated as in a pile.

$$\text{soil skin friction, } f_{\text{soil}} = \alpha \times c$$

where

α = adhesion factor

$$f_{\text{soil}} = 1.0 \times 250 = 250\,\text{psf}$$

$$\begin{aligned}\text{skin friction mobilized along the pile shaft within the clay layer} \\ &= f_{\text{soil}} \times \text{perimeter} \\ &= 250 \times (\pi \times d) \times 20 = 62{,}800\,\text{lb}\end{aligned}$$

$$\text{allowable skin friction} = 62{,}800/3.0 = 20{,}900\,\text{lb} = 10\,\text{tons}$$

A factor of safety of 3.0 is assumed.

$$\begin{aligned}\text{load transferred to the rock, } F = P - 20{,}900 &= 600{,}000 - 20{,}900 \\ &= 579{,}100\,\text{lb}\end{aligned}$$

Note that it is assumed that the allowable skin friction within the soil is fully mobilized.

STEP 4: Determine the load transferred to the rock.

The load transferred to the rock is divided between the total skin friction, F_{skin}, and the end bearing at the bottom of the caisson, Q_{base}.

$$f_{\text{skin}} = \text{unit skin friction mobilized within the rock mass}$$

$$F_{\text{skin}} = \text{total skin friction} = f_{\text{skin}} \times \text{perimeter area within the rock mass}$$

q_{base} = end bearing stress mobilized at the base of the caisson

$$Q_{base} = q_{base} \times \text{area of the caisson at the base}$$

STEP 5: Find the end bearing, Q_{base}, and skin friction, F_{skin}, within the rock mass.

The end bearing ratio, n, is defined as the ratio between the end bearing load of the rock mass, Q_{base}, and the total resistive force mobilized, $Q_{base} + F_{skin}$, within the rock mass.

$$\text{end bearing ratio, } n = Q_{base}/(Q_{base} + F_{skin})$$

where n is obtained from Table 35.3.

Q_{base} = end bearing load generated at the base
F_{skin} = total skin friction generated within the rock mass
L = length of the caisson within the rock mass
a = radius of the caisson = 2 ft
E_r/E_c = elastic modulus of rock/elastic modulus of concrete = 0.5 (given)

$$\text{total load transferred to the rock mass} = 579{,}100 \text{ lb}$$
$$= Q_{base} + F_{skin} \text{ (see step 3)}$$

$L = 12$ ft and $L/a = 12/2 = 6$.
The highest value in Table 35.3 for $L/a = 4$. Hence, use $L/a = 4$.

From Table 35.3 for L/a of 4 and E_r/E_c of 0.5, the end bearing ratio, $n = 0.12$.

$$n = 0.12 = Q_{base}/(Q_{base} + F_{skin})$$
$$0.12 = Q_{base}/579{,}100$$

Table 35.3 End bearing ratio (n)

$E_r/E_c = 0.5$		$E_r/E_c = 1.0$		$E_r/E_c = 2.0$		$E_r/E_c = 4.0$	
L/a	n	L/a	n	L/a	n	L/a	n
1	0.5	1	0.48	1	0.45	1	0.44
2	0.28	2	0.23	2	0.20	2	0.16
3	0.17	3	0.14	3	0.12	3	0.08
4	0.12	4	0.08	4	0.06	4	0.03

Source: Osterberg and Gill (1973).

Hence

$$Q_{\text{base}} = 579{,}100 \times 0.12 = 69{,}492\,\text{lb} = 35\,\text{tons}$$
$$Q_{\text{allowable}} = 452\,\text{tons (see step 1)}$$

$Q_{\text{allowable}}$ is greater than the end bearing load, Q_{base}, generated at the base.

$$F_{\text{skin}} = \text{load transferred to the rock} - \text{end bearing load}$$
$$F_{\text{skin}} = 579{,}100 - 69{,}492 = 509{,}608\,\text{lb} = 255\,\text{tons}$$

F_{skin} should be less than $F_{\text{allowable}}$.

$$F_{\text{allowable}} = f_{\text{allowable}} \times \text{perimeter of the caisson within the rock mass}$$
$$f_{\text{allowable}} = 2\text{ tsf (see Step 2)}$$

Since a length, L, of 8 ft was assumed within the rock mass

$$F_{\text{allowable}} = 2\,\text{tsf} \times (\pi \times 4) \times 8 = 201\,\text{tons}$$
$$F_{\text{skin}} = \text{skin friction generated} = 255\,\text{tons (see above)}$$

$F_{\text{allowable}}$ is less than the skin friction generated. Hence, it will be necessary to increase the pile diameter or length of the pile.

References

Osterberg, J. O., and Gill, S. A. 1973. Load transfer mechanism for piers socketed in hard soils or rock. *Proceedings of the Canadian Rock Mechanics Symposium* (pp. 235–261).

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36

Design of Pile Groups

36.1 Introduction

Typically, piles are installed in a group and provided with a pile cap. The column will be placed on the pile cap so that the column load is equally distributed among the individual piles in the group. See Fig. 36.1.

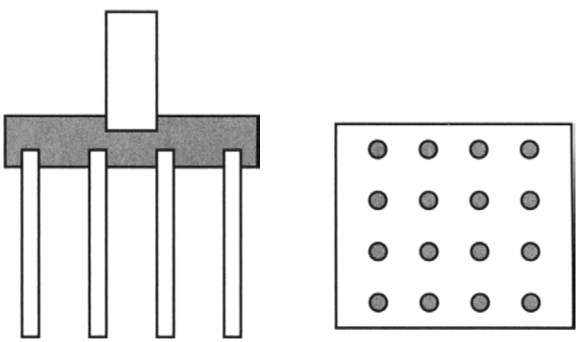

Figure 36.1 Pile group

The capacity of a pile group is obtained by using an efficiency factor.

$$\text{pile group capacity} = \text{efficiency of the pile group} \times \text{single pile capacity} \times \text{number of piles}$$

If the pile group contains 16 piles, the capacity of a single pile is 30 tons, and the group efficiency is found to be 0.9, then the group capacity is 432 tons.

$$\text{pile group capacity} = 0.9 \times 30 \times 16 = 432 \text{ tons}$$

It is clear that a high pile group efficiency is desirable. The question is, how can the group efficiency be improved? The pile group efficiency is dependent on the spacing between piles. When the piles in the group are closer together, the pile group efficiency decreases. When the piles are placed farther apart, the efficiency increases. When the piles are placed far apart, the size of the pile cap must be larger, increasing the cost of the pile cap.

36.2 Soil Disturbance During Driving

What happens in a pile group?

When piles are driven, the soil surrounding the pile will be disturbed. Disturbed soil has a lesser strength than undisturbed soil. Some of the piles in the group are installed in partially disturbed soil, causing them to have a lesser capacity than others. Typically, the piles in the center are driven first. See Fig. 36.2.

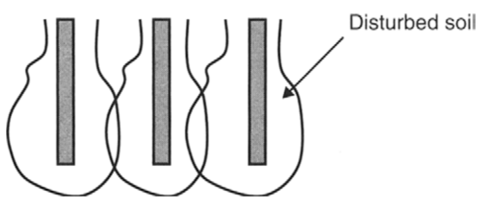

Figure 36.2 Disturbed soil due to driving

Soil disturbance caused by one pile impacts the capacity of adjacent piles. The group efficiency can be improved by placing the piles at a larger spacing. In clay soils, shear strength will be reduced due to disturbance.

36.3 Soil Compaction in Sandy Soil

When driving piles in sandy soils, the surrounding soil will be compacted. Compacted soil tends to increase the skin friction of the piles. Pile groups placed in sandy soils may have a group efficiency greater than 1.0. Soil compaction due to pile driving will be minimal in clay soils. In sandy soils, there is freedom for particle movement. Hence, compaction is easier.

36.3.1 Pile Bending

When driving piles in a group, some piles may be bent due to soil movement. This effect is more pronounced in clayey soils. See Fig. 36.3.

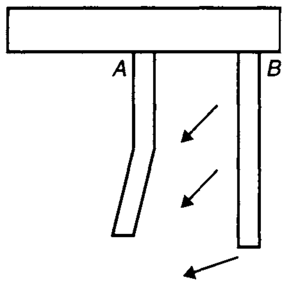

Figure 36.3 Pile bending due to hard driving

Assume pile *A* is driven first and pile *B* is driven next. Soil movement caused by pile *B* can bend pile *A* as shown in Fig. 36.3. This, in return, would create a lower group capacity.

36.3.2 End Bearing Piles

Piles that rely mainly on end bearing capacity may not be affected by other piles in the group. See Fig. 36.4. Piles that rely mainly on end bearing capacity are generally those ending in very strong bearing stratum or in rock. When piles are not dependent on the skin friction, a group efficiency of 1.0 can be used. Various guidelines for computing group capacity are given in the next section.

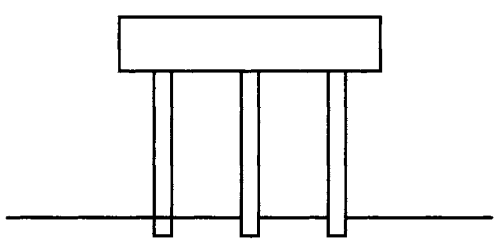

Figure 36.4 End bearing piles

36.3.3 AASHTO (1992) Guidelines

The American Association of State Highway and Transportation Officials, AASHTO, provides guidelines for three situations.

1. Pile group in cohesive soils (clays and clayey silts).
2. Pile group in noncohesive soils (sands and silts).
3. Pile group in strong soil overlying weaker soil.

Tables 36.1 and 36.2 have been constructed using the AASHTO guidelines for pile group efficiency in cohesive soils.

Table 36.1 Pile group efficiency for clayey soils*

Pile spacing (center to center)	Group efficiency
3*D*	0.67
4*D*	0.78
5*D*	0.89
6*D* or more	1.00

Source: AASHTO (1992).
*D = diameter of piles.

Table 36.2 Pile group efficiency for sandy soils*

Pile spacing (center to center)	Group efficiency
3*D*	0.67
4*D*	0.74
5*D*	0.80
6*D*	0.87
7*D*	0.93
8*D* or more	1.00

Source: AASHTO (1992).
*D = diameter of piles.

Design Example 36.1

This example explores pile group capacity. The pile group is constructed in clayey soils as shown in Fig. 36.5. Single pile capacity was computed to be 30 tons, and each pile is 12 in. in diameter. The center to center distance of the piles is 48 in. Find the capacity of the pile group using the AASHTO method.

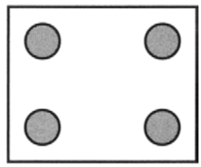

Figure 36.5 Pile group with four piles

Solution

$$\text{pile group capacity} = \text{efficiency of the pile group} \times \text{single pile capacity} \times \text{number of piles}$$

$$\text{center to center distance between piles} = 48 \text{ in.}$$

Since the diameter of piles is 12 in., the center to center distance is $4D$. From Table 36.1,

$$\text{efficiency of the pile group} = 0.78$$

$$\text{pile group capacity} = 0.78 \times 30 \times 4 = 93.6 \text{ tons}$$

Pile Group Capacity When Strong Soil Overlies Weaker Soil

Usually piles end in strong soils. In some cases, there could be a weaker soil stratum underneath the strong soil strata. In such situations, settlement due to the weaker soil underneath has to be computed. Settlement due to weak soil should be acceptable. If not, the number of piles in the pile group has to be increased. See Fig. 36.6.

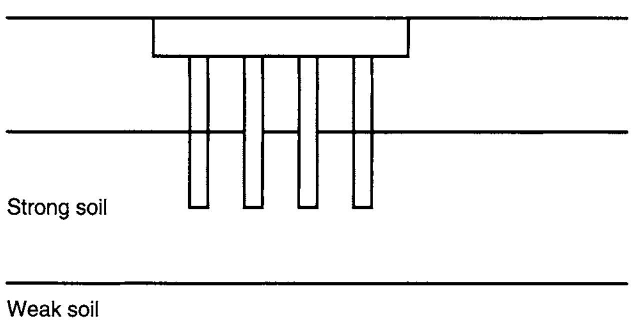

Figure 36.6 Pile group in strong soil overlying weak soil

Pile Spacing (Center to Center Distance): International Building Code Guidelines

In no case should the minimum distance be less than 24 in.

For circular piles, the minimum center to center distance is twice the average diameter of the butt.

For rectangular piles, the minimum center to center distance is three-fourths of the diagonal for rectangular piles.

For tapered piles, the minimum center to center distance is twice the diameter at one-third of the distance of the pile measured from the top of the pile. See Fig. 36.7.

$$\text{pile spacing} = 2D_1 \text{ or more}$$

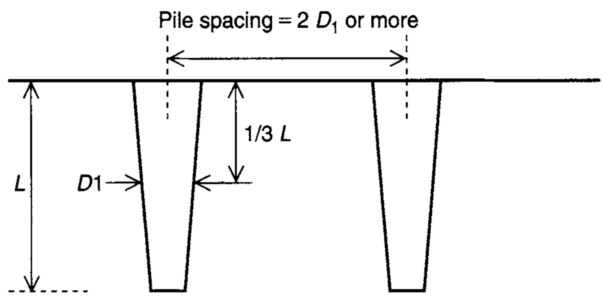

Figure 36.7 Pile spacing

Reference

American Association of State Highway and Transportation (AASHTO). 1992. Available at www.transportation.org.

Appendix: Conversions*

fps units	SI units
Length	
1 ft = 0.3048 m	1 m = 3.28084 ft
1 in. = 2.54 cm	
Pressure	
1 ksf = 1,000 psf	1 Pascal = 1 N/m^2
1 ksf = 47,880.26 Pa	1 MPa = 20.88543 ksf
1 ksf = 0.04788 MPa	1 MPa = 145.0377 psi
1 ksf = 47.88 kPa	1 kPa = 0.020885 ksf
1 psi = 6,894.757 Pa	1 kPa = 0.1450377 psi
1 psi = 6.894757 kPa	1 bar = 100 kPa
1 psi = 144 psf	
Area	
1 ft^2 = 0.092903 m^2	1 m^2 = 10.76387 ft^2
1 ft^2 = 144 in^2	

continued

*Density of water: 1 g per cubic centimeter = 1,000 g per liter = 1,000 kg/m^3 = 62.42 pounds per cubic foot (62.42 pcf).

continued

fps units	SI units
Volume	
1 ft^3 = 0.028317 m^3	1 m^3 = 35.314667 ft^3
1 gal = 8.34 lbs	
Density	
1 lb/ft^3 = 157.1081 N/m^3	1 kN/m^3 = 6.3658 lb/ft^3
1 lb/ft^3 = 0.1571081 kN/m^3	
Weight	
1 kip = 1,000 lb	1 kg = 9.80665 N
1 lb = 0.453592 kg	1 kg = 2.2046223 lb
1 lb = 4.448222 N	1 N = 0.224809 lb
1 ton (short) = 2,000 lb	1 N = 0.101972 kg
1 ton = 2 kip	1 kN = 0.224809 kip

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