

EARTHQUAKE RESISTANT ENGINEERING STRUCTURES VI

EDITOR
C.A. Brebbia

 WIT PRESS

Earthquake Resistant Engineering Structures VI

WIT*PRESS*

WIT Press publishes leading books in Science and Technology.

Visit our website for the current list of titles.

www.witpress.com

WIT*eLibrary*

Home of the Transactions of the Wessex Institute.

Papers presented at ERES VI are archived in the WIT eLibrary in volume 93 of

WIT Transactions on The Built Environment (ISSN 1743-3509).

The WIT electronic-library provides the international scientific community with immediate and permanent access to individual papers presented at WIT conferences.

Visit the WIT eLibrary at www.witpress.com.

SIXTH WORLD CONFERENCE ON
EARTHQUAKE RESISTANT ENGINEERING STRUCTURES

ERES VI

CONFERENCE CHAIRMAN

C.A. Brebbia

Wessex Institute of Technology, UK

INTERNATIONAL SCIENTIFIC ADVISORY COMMITTEE

Y. Ariga	M. Haroun	J.M. Nichols
T. Attard	A. Kappos	C.W. Roeder
D. E. Beskos	H. Kawakami	M. Saiidi
S. Dristos	P. Komodromos	E.J. Sapountzakis
G. Dundulis	G.G. Manolis	O. Sircovich Saar
M. Elgawady	G.C. Manos	C.C. Spyrakos
K. Fuchida		

Organised by

Wessex Institute of Technology, UK

Sponsored by

WIT Transactions on The Built Environment

WIT Transactions on The Built Environment

Transactions Editor

Carlos Brebbia

Wessex Institute of Technology
Ashurst Lodge, Ashurst
Southampton SO40 7AA, UK
Email: carlos@wessex.ac.uk

Editorial Board

E Alarcon

Universidad Politecnica de Madrid
Spain

S A Anagnostopoulos

University of Patras
Greece

H Antes

Technische Universitat Braunschweig
Germany

D E Beskos

University of Patras
Greece

F Butera

Politecnico di Milano
Italy

J Chilton

University of Nottingham
UK

M C Constantinou

State University of New York at Buffalo
USA

A De Naeyer

Universiteit Ghent
Belgium

J Dominguez

University of Seville
Spain

M N Fardis

University of Patras
Greece

L Gaul

Universitat Stuttgart
Germany

M Iguchi

Science University of Tokyo
Japan

W Jager

Technical University of Dresden
Germany

C Alessandri

Universita di Ferrara
Italy

E Angelino

A.R.P.A. Lombardia
Italy

D Aubry

Ecole Centrale de Paris
France

J J Bommer

Imperial College London
UK

P G Carydis

National Technical University of Athens
Greece

S Clement

Transport System Centre
Australia

G Degrande

Katholieke Universiteit Leuven
Belgium

W P De Wilde

Vrije Universiteit Brussel
Belgium

F P Escrig

University of Seville
Spain

C J Gantes

National Technical University of Athens
Greece

Y Hayashi

Nagoya University
Japan

L Int Panis

VITO Expertisecentrum IMS
Belgium

C M Jefferson

University of the West of England
UK

- D L Karabalis**
University of Patras
Greece
- W Jager**
Technical University of Dresden
Germany
- W B Kratzig**
Ruhr Universitat Bochum
Germany
- J W S Longhurst**
University of the West of England,
UK
- L Lundqvist**
Unit for Transport and Location Analysis
Sweden
- G D Manolis**
Aristotle University of Thessaloniki
Greece
- F M Mazzolani**
University of Naples "Federico II"
Italy
- G Oliveto**
Università di Catania
Italy
- A S Papageorgiou**
Rensselaer Polytechnic Institute
USA
- A M Reinhorn**
State University of New York at Buffalo
USA
- C W Roeder**
University of Washington
USA
- M Saiidi**
University of Nevada-Reno
USA
- S A Savidis**
Technische Universität Berlin
Germany
- Q Shen**
Massachusetts Institute of Technology
USA
- P D Spanos**
Rice University
USA
- H Takemiya**
Okayama University
Japan
- E Taniguchi**
Kyoto University
Japan
- M A P Taylor**
University of South Australia
Australia
- E Kausel**
Massachusetts Institute of Technology
USA
- A N Kounadis**
National Technical University of Athens
Greece
- A A Liolios**
Democritus University of Thrace
Greece
- J E Luco**
University of California at San Diego
USA
- M Majowiecki**
University of Bologna
Italy
- G Mattrisch**
DaimlerChrysler AG
Germany
- K Miura**
Kajima Corporation
Japan
- E Oñate**
Universitat Politècnica de Catalunya
Spain
- G G Penelis**
Aristotle University of Thessaloniki
Greece
- F Robuste**
Universitat Politècnica de Catalunya
Spain
- J M Roeset**
Texas A & M University
USA
- F J Sanchez-Sesma**
Instituto Mexicano del Petróleo
Mexico
- J J Sendra**
University of Seville
Spain
- A C Singhal**
Arizona State University
USA
- C C Spyarakos**
National Technical University of Athens
Greece
- I Takewaki**
Kyoto University
Japan
- J L Tassoulas**
University of Texas at Austin
USA
- R Tremblay**
Ecole Polytechnique
Canada

R van der Heijden

Radboud University
Netherlands

A Yeh

The University of Hong Kong
China

R Zarnic

University of Ljubljana
Slovenia

R van Duin

Delft University of Technology
Netherlands

M Zador

Technical University of Budapest
Hungary

This page intentionally left blank

Earthquake Resistant Engineering Structures VI

Editor

C.A. Brebbia

Wessex Institute of Technology, UK

WIT*PRESS* Southampton, Boston



Editor:

C.A. Brebbia

Wessex Institute of Technology, UK

Published by

WIT Press

Ashurst Lodge, Ashurst, Southampton, SO40 7AA, UK

Tel: 44 (0) 238 029 3223; Fax: 44 (0) 238 029 2853

E-Mail: witpress@witpress.com

<http://www.witpress.com>

For USA, Canada and Mexico

Computational Mechanics Inc

25 Bridge Street, Billerica, MA 01821, USA

Tel: 978 667 5841; Fax: 978 667 7582

E-Mail: infousa@witpress.com

<http://www.witpress.com>

British Library Cataloguing-in-Publication Data

A Catalogue record for this book is available
from the British Library

ISBN: 978-1-84564-078-1

ISSN: 1746-4498 (print)

ISSN: 1743-3509 (on-line)

*The texts of the papers in this volume were set
individually by the authors or under their supervision.
Only minor corrections to the text may have been carried
out by the publisher.*

No responsibility is assumed by the Publisher, the Editors and Authors for any injury and/or damage to persons or property as a matter of products liability, negligence or otherwise, or from any use or operation of any methods, products, instructions or ideas contained in the material herein.

© WIT Press 2007

Printed in Great Britain by Athenaeum Press Ltd.

All rights reserved. No part of this publication may be reproduced, stored in a retrieval system, or transmitted in any form or by any means, electronic, mechanical, photocopying, recording, or otherwise, without the prior written permission of the Publisher.

Preface

This book contains selected papers presented at the 6th International Conference on Earthquake Resistant Engineering Structures (ERES) which took place in Bologna, Italy in 2007. This meeting is one of the successful series of conferences organised by the Wessex Institute of Technology (WIT). The series started in Thessaloniki (1996), followed by Catania (1999), Malaga (2001), Ancona (2003) and Skiathos (2005).

The meeting provides a forum for the discussion of the latest developments in innovative design and construction of new earthquake resistant structures as well as the retrofitting of existing buildings. The success of the ERES Conference is closely linked to the innovation and quality of the presentations. It continues to attract promising young researchers as well as familiar names in the field of earthquake engineering. This combination is the main reason why the ERES meetings continue to bring to the attention of the international scientific community original high quality papers.

The importance of conferences like ERES is that they allow rapid dissemination of the latest research before the lengthy process of appearing in learned journals is undertaken. The WIT proceedings – which are produced in time for the conference – are immediately followed by the archiving of all papers in the Transactions of Wessex Institute Library where they are permanently and widely available (www.witpress.com). The Library contains all WIT conference papers since 1993 and attracts nearly a quarter of a million abstract downloads per year. The importance of this archive can not be overemphasised as it is essential for researchers and practitioners to have rapid access to the latest developments, particularly in fields such as earthquake engineering.

The ERES/07 papers appearing in the present book have been divided into the following sections:

- Earthquake resistant design
- Bridges
- Seismic isolation
- Passive protection devices and seismic isolation
- Self-centering systems
- Site effects and geotechnical aspects
- Seismic behaviour and vulnerability

- Lifelines
- Monitoring and testing
- Retrofitting
- Structural dynamics

The Editor appreciates that the task of editing this volume would not have been possible without the generous cooperation of the members of the International Scientific Advisory Committee and other colleagues to whom he is indebted for reviewing the papers. He is also grateful to all authors for their excellent contributions.

The Editor
Bologna, Italy
2007

Contents

Keynote contribution

A road map for seismic prevention of damage <i>M. Maugeri & S. Grasso</i>	XIX

Section 1: Earthquake resistant design

Vulnerability functions and the influence of seismic design parameters on initial costs for buildings provided with hysteretic energy-dissipating devices <i>J. García-Pérez, M. Zenteno & O. Díaz</i>	3
Seismic behavior over-resistance effects on buildings <i>J. A. Avila</i>	13
Design of reinforced concrete buildings according to the new NEHRP provisions <i>O. A. Mohamed & P. Khamwan</i>	23
Static and dynamic analytical and experimental analysis of 3D reinforced concrete panels <i>K. Numayr & R. Haddad</i>	33
Designing aspects of bridges placed in active seismic areas <i>V. Herak Marović, P. Marović & Ž. Nikolić</i>	43
Behaviour of coupling beams having vertical slits at the ends <i>S. B. Yuksel</i>	53
Principal stresses behaviour of a steel plate shear wall concerning buckling modes <i>P. Memarzadeh, M. Azhari & M. M. Saadatpour</i>	63

Earthquake architecture as an expression of a stronger architectural identity in seismic areas <i>T. Slak & V. Kilar</i>	73

Section 2: Bridges

Aspects of testing a large-scale two-span bridge model on multiple shake tables <i>N. Johnson, M. Saiidi & D. Sanders</i>	85
Seismic devices for bridges <i>D. Mestrovic & G. Grebenar</i>	95

Section 3: Seismic isolation

(Special session by P. Komodromos and M. C. Pochas)

Seismic isolation and energy dissipation: worldwide application and perspectives <i>A. Martelli</i>	105
Study of the seismic response of reinforced concrete isolated elevated water tanks <i>V. I. Fernández-Dávila, F. Gran & P. Baquedano</i>	117
Modeling of the structural impact of seismically isolated buildings <i>P. Polycarpou, L. Papaloizou, P. Komodromos & M. C. Phocas</i>	129

Section 4: Passive protection devices and seismic isolation

Aseismic study of a building with the efficiency-enhanced damping system <i>S. S. Ke, W. S. Li & B. J. Shih</i>	141
Introducing orthogonal roller pairs as an effective isolating system for low rise buildings <i>M. Hosseini & K. Kangarloo</i>	151

Section 5: Self-centering systems

(Special session by M. Elgawady)

Seismic response three-dimensional analyses of ten-story steel frames with column uplift <i>M. Midorikawa, T. Azuhata & T. Ishihara</i>	165

Shaking table test on seismic response of reduced-scale models of multi-story buildings allowed to uplift <i>T. Ishihara, T. Azuhata, K. Noguchi, K. Morita & M. Midorikawa</i>	175
Self-centering behavior of unbonded precast concrete shear walls <i>B. Erkmen & A. E. Schultz</i>	185
Displacement ductility demand and strength reduction factors for rocking structures <i>M. Trueb, Y. Belmouden & P. Lestuzzi</i>	195

Section 6: Site effects and geotechnical aspects

The 2006 Yogyakarta earthquake – a preliminary study of deaths <i>J. M. Nichols</i>	207
Local seismic amplification analysis in the industrial area of Sulmona, Central Italy <i>A. Rinaldini, A. Grillo & A. Marino</i>	215
Dynamic response of a large landslide during a strong earthquake <i>R. Meriggi & M. Del Fabbro</i>	225
Liquefaction potential evaluation for a site <i>S. Mittal & M. K. Gupta</i>	235

Section 7: Seismic behaviour and vulnerability

Seismic risk assessment of the Ignalina NPP refuelling machine <i>R. Bausys, G. Dundulis, R. Kacianauskas, D. Markauskas, S. Sliaupa, E. Stupak & S. Rimkevicius</i>	247
Comparing static linear and nonlinear analyses of safe rooms in a poor performance masonry building <i>M. Mazloom</i>	259
Empirical fragility curves for Peruvian school buildings <i>A. Muñoz, M. Blondet, R. Aguilar & M.-A. Astorga</i>	269
Evaluation of lateral load pattern in pushover analysis <i>S. I. Javadein & R. Taghinezhad</i>	279

3-D reproduction analyses for actual earthquake behaviors of existing dams <i>Y. Ariga</i>	289

Seismic hazard expression in risk assessment <i>X.-X. Tao, Z.-R. Tao & P. Li</i>	299

Section 8: Lifelines

Seismic reliability and cost evaluation for a hospital lifeline network system <i>K. Fuchida</i>	309

Human life saving lifelines and cost-effective design of an exclusive water supply system for fires following earthquakes <i>S. Takada & Y. Kuwata</i>	319

Section 9: Monitoring and testing

Shaking table tests on shallow foundations <i>J. Estaire & V. Cuéllar</i>	331

Development of a digitally-controlled single-axis earthquake shake frame for masonry walls testing <i>M. J. Guzman & S. L. Lissel</i>	343

Determination of seismic transport effects on buildings <i>D. Makovička & D. Makovička Jr.</i>	353

Section 10: Retrofitting

Towards a European code for seismic assessment and strengthening of existing buildings <i>S. Dritsos</i>	365

Flexural retrofitting of reinforced concrete bridge pier type cross-sections with carbon fiber reinforcing plastics <i>G. C. Manos & V. Kourtides</i>	375

Evaluating the retrofitting process for Imam (Soltani) Mosque monument after Silakhor Plan earthquake damage (31 March 2006) <i>H. R. Vosoughifar</i>	387

Effect of connection procedures on the behaviour of RC columns strengthened with RC layers and jackets <i>A. P. Lampropoulos, O. T. Tsioulou & S. E. Dritsos</i>	399

Seismic assessment of buildings by rapid visual screening procedures <i>P. Kapetana & S. Dritsos</i>	409

Section 11: Structural dynamics

Three-dimensional seismic damage simulation of wooden houses using a rigid body-spring method <i>H. Kawakami, E. A. Tingatinga & H. Y. Chang</i>	421

Controlling nonlinear vibrations in steel structures using an evolutionary gain formulation to optimally satisfy performance objectives <i>R. Dansby & T. Attard</i>	431

Dynamic analysis of plates stiffened by parallel beams <i>E. J. Sapountzakis & V. G. Mokos</i>	443

Dynamics in the practice of structural design: the problems of implementation <i>O. S. Saar</i>	453

Effect of impulsive force on earthquake response of rocking structural systems <i>T. Azuhata, T. Ishihara & M. Midorikawa</i>	459

Author Index	469

This page intentionally left blank

Keynote contribution

by

M. Maugeri and S. Grasso

This page intentionally left blank

A road map for seismic prevention of damage

M. Maugeri & S. Grasso

Department of Civil and Environmental Engineering, Catania, Italy.

Abstract

The seismic prevention of damage is a challenge for the third millennium. In spite of the tremendous work on analytical and experimental studies to reduce seismic risk, many cities in the world are prone to seismic risk and the dead and loose are increasing exponentially in the past century.

The scientific, social and political awareness of Italian community leads the Italian Department of Civil Protection to funding the Research Project on earthquake damage scenarios for a high risk area in the Mediterranean. The area chosen was the mid-sized city of Catania, located in the Sicily island, situated in the central part of the Mediterranean area.

The road map for the evaluation of seismic prevention of damage follows the subsequent parts: the earthquake-source characterisation and seismic action evaluation at the bedrock for the scenario earthquakes (section 2); the site effects evaluation (section 3); the vulnerability analysis of physical environment for evaluating the risk related to cavities, landslides and liquefaction (section 4); the soil-structure interaction analysis for shallow foundation and retaining walls (section 5); the vulnerability analysis of monuments and r.c. buildings (section 6); the seismic structural improvement of r.c. buildings (section 7); vulnerability analysis of urban road and infrastructures (section 8).

The road map followed for the seismic prevention of damage could be considered a pilot project for detailed earthquake scenarios analyses and for seismic prevention of damage in many cities, characterised as the city of Catania by severe site amplification phenomena, landslides, liquefaction, presence of cavities, presence of many monuments and buildings not designed to resist against earthquakes. The seismic structural improvement of these buildings must be based on site dependent response spectra, evaluated from the characterisation of the earthquake source. The seismic structural improvement is needed for the sustainable development of many cities prone to seismic risk.

Keywords: earthquake source modelling, site effects and microzoning, environmental vulnerability, building vulnerability, seismic retrofitting of buildings



1 Introduction

The seismic prevention of damage is a multidisciplinary task involving a research team, made by geologists, geophysics, geotechnical, structural and transportation engineering and town planning. The road map followed for seismic prevention of damage could be considered as a guideline for the multidisciplinary task of assessment of the seismic hazard, the evaluation of the seismic risk, the prevention of seismic risk for new constructions and the mitigation of seismic risk in the existing constructions, including monuments and cultural heritage.

The road map for the evaluation of seismic prevention of damage follows the subsequent parts (Maugeri [1]): the earthquake-source characterisation and seismic action evaluation at the bedrock for the scenario earthquakes (section 2); the site effects evaluation (section 3); the vulnerability analysis of physical environment for evaluating the risk related to cavities, landslides and liquefaction (section 4); the soil-structure interaction analysis for shallow foundation and retaining walls (section 5); the vulnerability analysis of monuments and r.c. buildings (section 6); the seismic structural improvement of r.c. buildings (section 7); vulnerability analysis of urban road and infrastructures (section 8).

Section 2 is related to source characterisation and seismic action, which represents the biggest uncertainty in the overall process for design and retrofitting of buildings. In particular are discussed: the scenario earthquake given by the two destructive earthquakes of 1693, the modelling of the moderate earthquake of December 13, 1990 and the seismic response evaluation from micro-tremors and from numerical analyses.

Section 3 is related to site effects evaluation, which shows a great spatial variability in the urban area of Catania, due to the geological and geo-lithological features, as well as the non-linear behaviour of the soil; site effects have been evaluated in particular in some test areas and in the areas where cultural heritage are located.

Section 4 is related to vulnerability of physical environment, using Geographical Information Systems (GIS) technique. Among the vulnerability of physical environment related to potential landslides, potential liquefaction, presence of cavities etc., a response analysis of the Monte Po hill is presented, as well as the survey of the cavities, which are relevant for the city of Catania. Section 5 is related to soil-structure interaction and to shallow foundation and retaining wall according to the suggestion of the Eurocode 8 Design of Structures for Earthquake Resistance.

Section 6 is related to the vulnerability analysis of buildings, by means of assessments and simulations using data from the Geographical Information System (GIS). An innovative approach is to link the vulnerability analysis to the seismic performance of RC buildings.

Section 7 is related to the seismic structural improvement of monuments and RC buildings. The Mediterranean cities are characterised by the presence of many monuments, which represent a world cultural heritage and must be preserved. Also the Mediterranean cities are characterised by buildings designed



without any seismic provision, which must be improved to resist against earthquakes.

Section 8 is related to the early alarm and vulnerability of infrastructures systems and urban road. The heavy alarm can be given by the first Italian seafloor observatory, placed offshore the Catania coasts, connected by an underwater electro-optical cable to communicate with the surface communicator. The early alarm could be useful for many purposes as for the interruption of pipeline for gas distribution to avoid fire during and after earthquakes. Among the infrastructures, also the functionality analysis of the urban road network in seismic emergency are considered.

2 Source characterisation and seismic action

The seismic action is generally prescribed by probabilistic approach, considering the probability of exceeding a certain level during the life of the constructions. In Italy the probabilistic evaluation of the seismic action is based on the probability of exceeding of 10% of exceeding in 50 years, considering only the earthquakes with a return period less than 475 years. This criterion is not a conservative one, because generally speaking the life of the construction exceeds 50 years; in fact in many Italian medieval cities the buildings are standing from more than 500 years.

The Code of Federal Regulations (United States 1991) requires new municipal solid waste landfills to be designed either for a maximum horizontal acceleration taken from a published seismic map for a 10 percent probability of exceedance (90 percent probability of non exceedance) in a 250-year exposure period or on the basis of a site specific analysis. The related return period for the map-based acceleration is 2375 years. The criterion of a site specific analysis is not specified in the regulation, but rather is left up to the individual states and may be probabilistic or deterministic. Because of the lower uncertainty, the return period for a site specific analysis may be less than 2375 years.

Increasing the historical knowledge of past earthquakes and changing the probabilistic criteria, the probabilistic approach leads to update the seismic action, which has been changed in the past in Italy (Fig. 1), so, buildings designed to resist to the given seismic action at the time of construction earthquakes could be not resisting at the updated seismic action.

Alternatively to the probabilistic approach, the road map for the evaluation of seismic damage takes into consideration the deterministic approach based on the evaluation of source mechanism. The scenario earthquake considered is the January 1693 events, as the biggest one; the February 1818 earthquake as medium earthquake and the December 1990 as a low earthquake (Fig. 2). The geological, geophysical and laboratory investigations performed provide additional constraints to the surface geological setting of the area as well as to the parameterization of the physical models. Numerical simulations have been applied with the twofold aim of estimating strong ground motion scenarios for different earthquake hypotheses and evaluating the effectiveness of 1-D non-linear and 2-D and 3-D linear methods for the estimation of the local response.



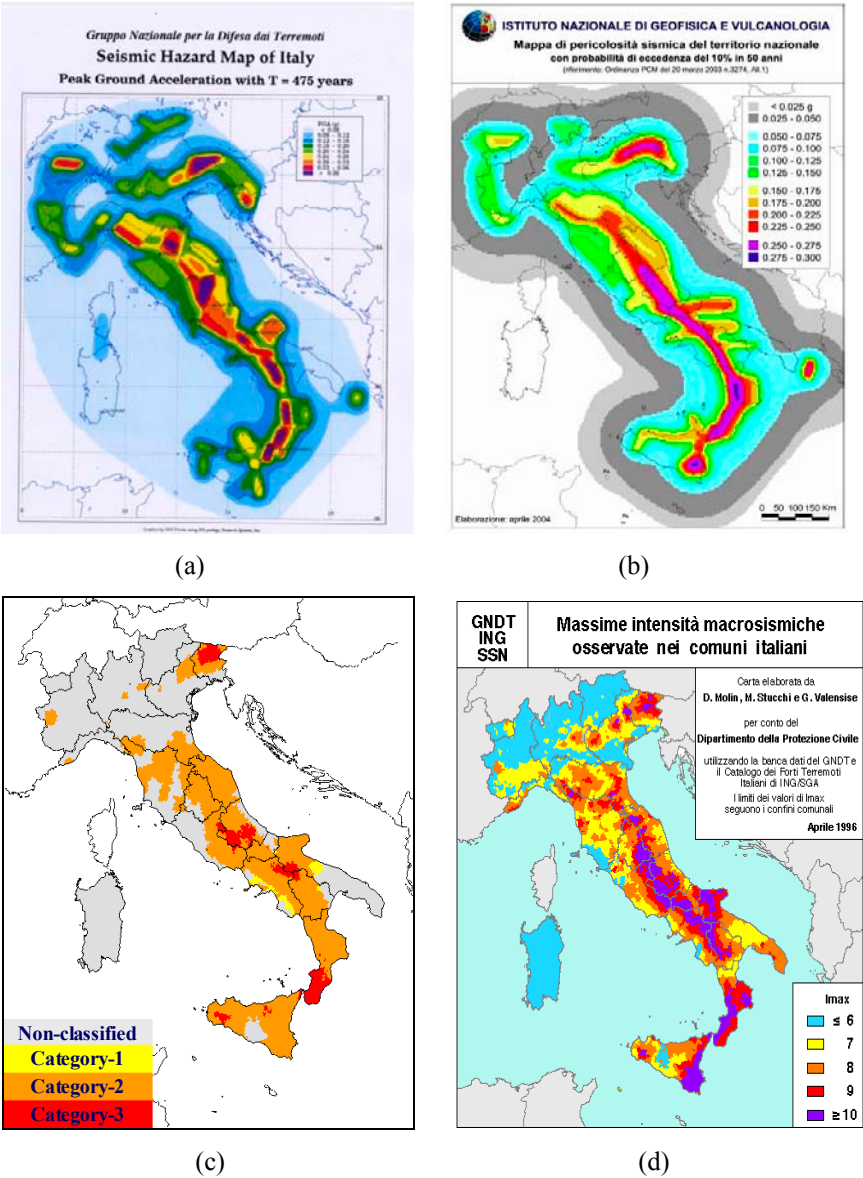


Figure 1: Seismic action: (a) Seismic hazard map of Italy in terms of acceleration proposed by the GNDT on 1984. (b) Seismic hazard map of Italy in terms of acceleration attached to the Ordinance O.P.C.M. 3274/2003 according to the Civil Defence of Italy. (c) National Seismic Regulation of Italy in terms of acceleration (category 2 for Catania area equal to 0.25g. (d) Maximum intensity felt in Italy (intensity ≥ 10 for Catania area).

The use of advanced methods capable of generating of synthetic seismograms can give a valuable insight into the evaluation of a seismic ground motion scenario.

The reference event is the catastrophic earthquake (M7+) that struck Eastern Sicily on January 11, 1693, assumed as a level I scenario event in the Project. The earthquake is linked to the Ibleo-Maltese fault system, which is the major seismogenic structure of Eastern Sicily.

The approach (Priolo [2]) solves the 2-D full-wave propagation through laterally heterogeneous media, and therefore are well suited to provide accurate synthetic seismograms, and analyse the effect of the medium heterogeneity and local conditions on the ground motion. The approach uses a 2-D Chebyshev spectral element method. The ground motion is simulated along few selected transects (Fig. 3), where a realistic geological structure is defined, including the fine local details. A simplified extended source model is adopted. The results consist of detailed estimates of the main parameters that define the ground motion, and include some synthetic accelerograms at the surface and at a given depth (Fig. 4).

The reference medium earthquake is the February 20, 1818 M=6.2 earthquake, whose epicentre was close to the northern part of the present settlement of the city. Also this earthquake is considered a tectonic earthquake and is associated to the northern continuation of the Ibleo-Maltese fault system. Despite of its medium magnitude, the 'Catanese' earthquake has to be accounted for the seismic hazard assessment of Catania, because of its vicinity to the city.

The near-fault strong ground motion is computed through a hybrid stochastic-deterministic method (EXWIM). This method simulates rupture propagation along finite fault and solves the 3-D full-wave propagation in inelastic media with a vertically heterogeneous structure (Laurenzano et al. [3]). In order to evaluate an exhaustive scenario, different slip distributions and hypocenters are considered. The structural model assumed is representative of the Eastern Sicily area, however local site conditions are taken into account at each site in a simplified way (i.e. the VS_{30} value). The ground motion is computed for a regular grid of receivers sampling the urbanized area of Catania (Fig. 5).

The results consist of three-component waveforms, acceleration and displacement response spectra and other relevant parameters used to describe the ground motion (Fig. 6).

The seismicity of the Ibleo-Maltese escarpment is rather anomalous, since it generates a very low number of low to medium earthquakes. Consequently, there are only few events that can be used to validate numerical simulations of 1693 (strong) and 1818 (medium) earthquakes. The December 13, 1990, M = 5.8 earthquake is actually the only medium size event occurred along the northern segments of the Ibleo-Maltese system, which was recorded instrumentally. This earthquake is associated to a rupture of the transcurrent segment of the Ibleo-Maltese fault, and it was recorded by the ENEA-ENEL accelerometric network. Another reason of interest for modelling this earthquake is the fact that the seismogram recorded by the Catania ENEA-ENEL station shows anomalously large ground accelerations.



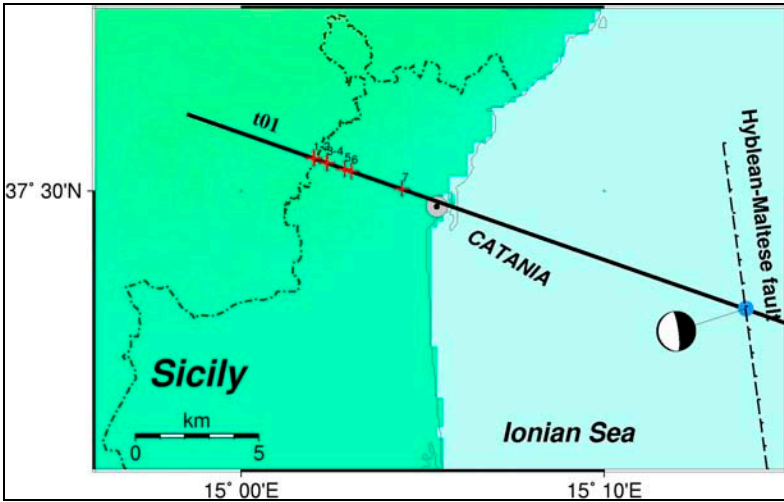


Figure 3: Base map of the study area, showing the transept position and the sites location. The blue circle shows the position assumed for the reference earthquake of January, 11, 1693.

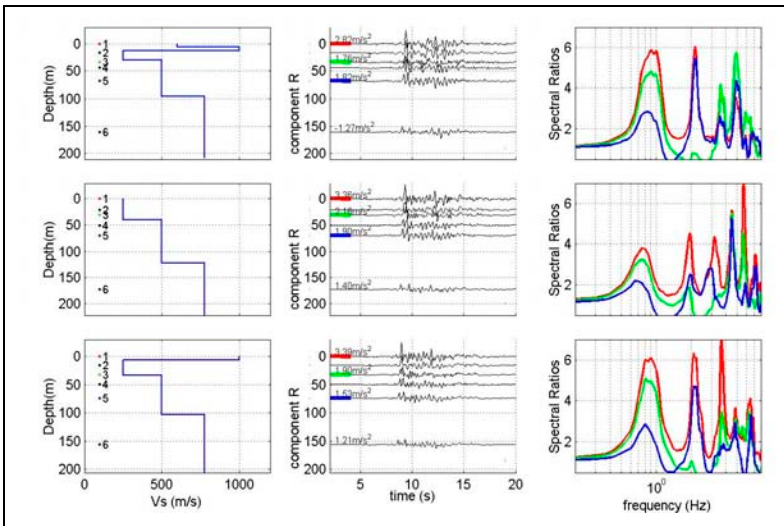


Figure 4: Site response at sites n. 1, 3, and 5 computed for a seismic moment distribution characterised by one dominant asperity. VS profile (left), acceleration time histories of the radial component (centre), and spectral ratios between the accelerations computed at different depths (i.e., receivers 1 (red colour, at ground surface), 3 (green, at $z \gg 35$ m) and 5 (blue, at $z \gg 70$ m), respectively) and receiver 6, located within the bedrock at depth of about 170 m.

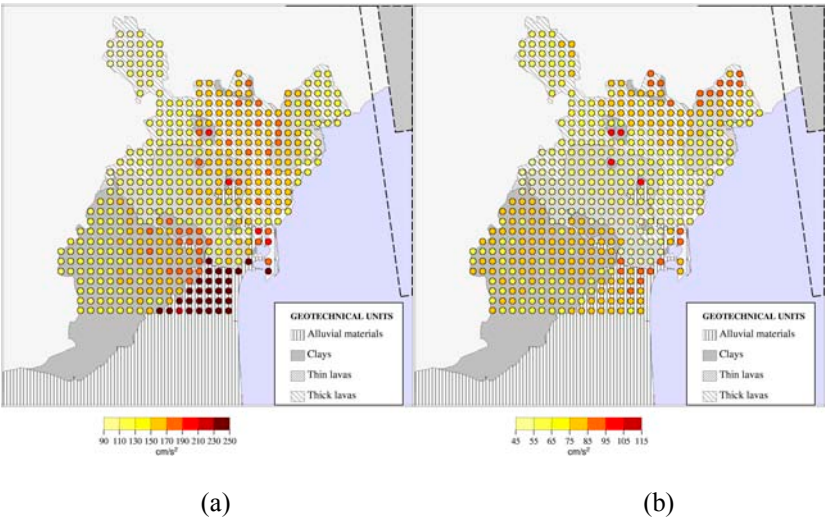


Figure 5: The average PGA scenario for 1818 earthquake, which consists of the mean value obtained considering the three fault sizes (small, medium and big), three slip distributions, two rake models (constant and variable) and all nucleation points, computed at each receiver. (a) Horizontal component; (b) vertical component (After Laurenzano et al. [3]).

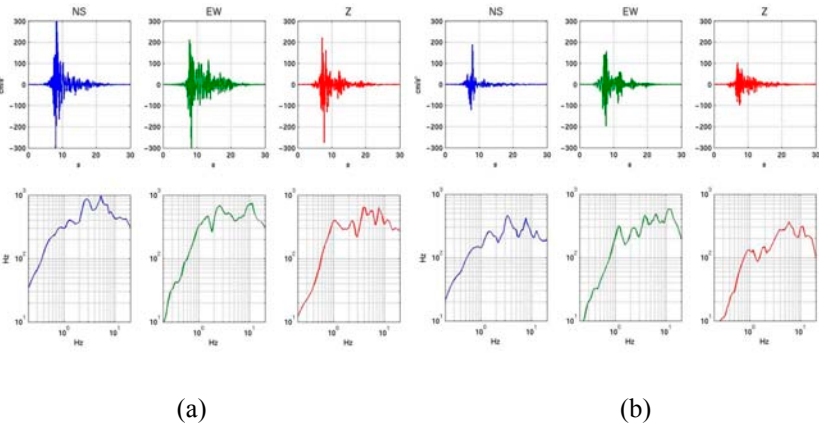


Figure 6: The seismograms and acceleration response spectra for medium size fault, variable rake distribution and up-dip rupture, computed at receivers located close each other for two different soil conditions. (a) fine alluvium of the plain; (b) lava (After Laurenzano et al. [3]).

The ground acceleration recorded at Catania (2.43 m/s^2) has amplitudes about 2,5 times larger than that recorded at Sortino (1.003 m/s^2), although both stations are at about the same epicentral distance (about 30 km) (see Fig. 2). These anomalies are attributed (e.g.: Di Bona et al. [4]) to both local site effects and the presence of strong crustal heterogeneities. The use of a method — the 2-D Chebyshev spectral element method — which solves the seismic full-wave propagation through a complex geological structure, is then of maximum interest to verify this kind of hypothesis (Laurenzano and Priolo [5]). In Fig. 7 it is possible to see the comparison between recorded and computed velocity and acceleration seismograms.

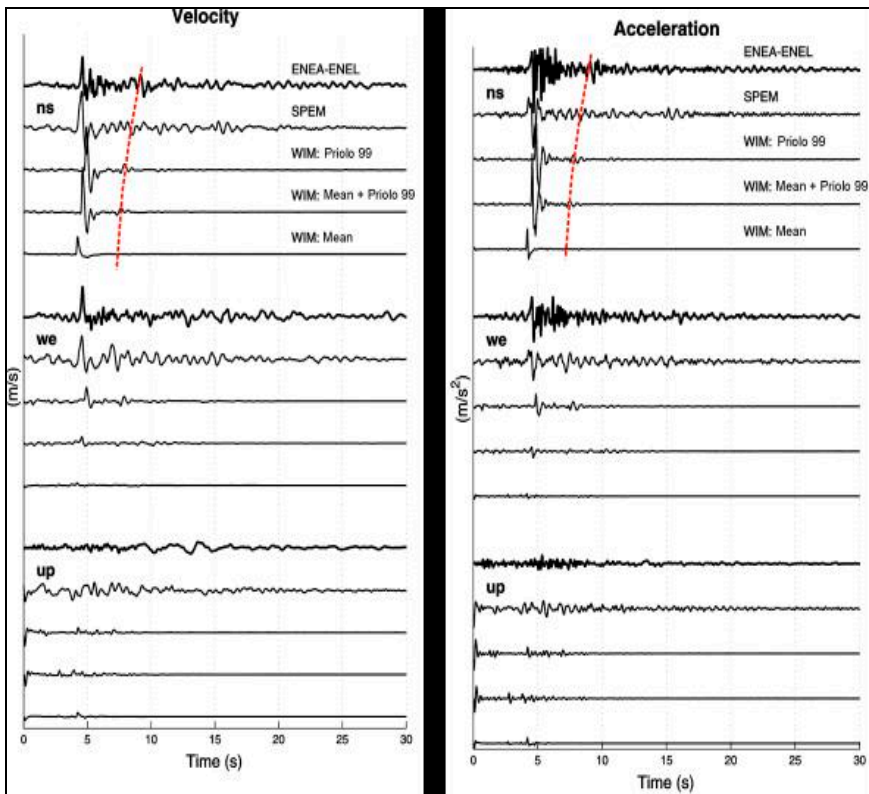


Figure 7. Three component velocity and acceleration seismograms recorded by the Catania ENEA-ENEL accelerometric station (thick lines) and computed numerically (thin lines) for different plane layer models. (WIM: Mean) mean regional structure; (WIM: Mean + Priolo [6]) deep structure of the mean regional model and shallow structure of the 2D model; (WIM: Priolo [6]) the best plane layer approximation of the 2D structure; (SPEM) 2D model (After Laurenzano and Priolo [5]).



3 Site effects and seismic microzoning

Site effects evaluation for seismic microzoning is based on the geological model of the city of Catania and the geotechnical site characterisation including soil non linearity evaluation.

A detailed geological survey of part of the historical downtown has been carried out, therefore improving the previous geological map of Catania (Monaco et al., [7]). This study provides a more detailed mapping of the surface formations, and the results have been used to define the shallow layers of the ground motion simulation models, employed for the evaluation of site effects.

For geotechnical model a site characterisation empirical correlations between the shear waves velocity and geotechnical soil properties and direct in-situ measurements of shear waves velocity were made at specific test sites were used. Down-Hole (D-H), Cross Hole (C-H), Seismic Dilatometer Marchetti Test (SDMT) tests were performed at different test sites, geo-settled by GIS (Fig. 8). On the basis of empirical correlations and direct measurements of shear waves velocity, a geotechnical model is reported in Fig. 9.

For geotechnical characterisation static and dynamic laboratory tests were performed. The dynamic resonant column tests (RCT) and cyclic loading torsional shear tests (CLTST) were performed to detect soil non-linearity. In Fig. 10 are reported the shear modulus and damping ratio for the Plaja Beach test site for sandy soil (Cavallaro and Maugeri [10]). Similar evaluation has made also for clayey soil (Cavallaro et al. [11]).

The specific laboratory test for detecting soil non-linearity behaviour is a key point for evaluating local response and site effects, because as stated in paragraph 2.6, 1-D non-linear soil response is preferable to 2-D linear soil response in the flat epicenter area. The correct evaluation of local soil response is in turn a key point for the microzonation study reported in the following paragraph.

The site response of seven sites located in Catania has been evaluated through both 1-D and 2-D numerical simulations. The main goals of the study were: 1) to analyse how the wave-field is modified during its passage through the sequence of the shallowest soil layers of a 2-D model, 2) to compare the effect of different definitions of “bedrock seismic input” on 1-D simulations, and 3) to evaluate the range of applicability of non-linear 1-D and linear 2-D approaches (Laurenzano et al., [8] in the case of strong ground motion.

The 2-D spectral element method was used for the 2-D simulations. The investigated sites are located along transect T_{01} (Fig. 3). The shallow structure of the model has been defined in detail at the seven study sites using all the available geotechnical data. Seismograms have been computed (Fig. 4) at several depths, starting from the ground surface, in order to study the wave field propagation through about one hundred meter of surface soils.

The 1-D method, which is commonly used in engineering practice, takes into account the detailed shear waves soil profile of surface layers, including soil non-linearity. The seismic response at the ground surface has been evaluated



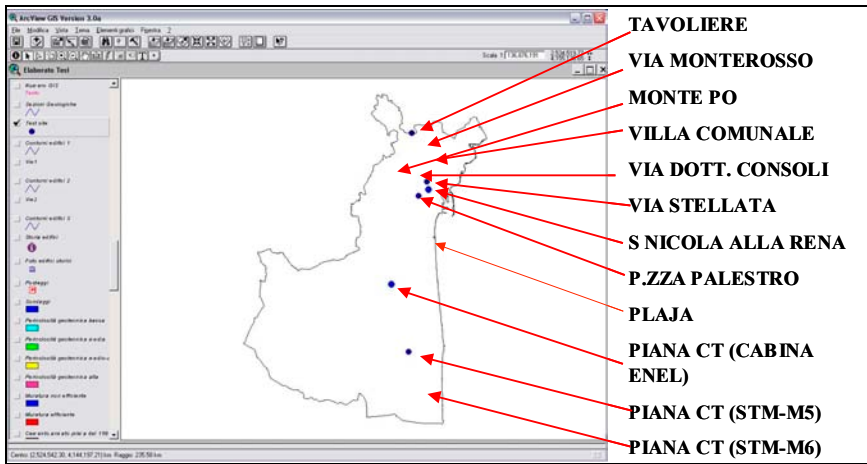


Figure 8: Test sites where direct measurements of V_s were performed: Cross-Hole test was performed at Piana CT (STM-M5); Down-Hole test was performed at Piana CT (STM-M5), Tavoliere, via Monterosso, via Dottor Consoli, via Stellata, San Nicola alla Rena, Piazza Palestro, Cabina ENEL; Seismic Marchetti Dilatometer Test (SDMT) was performed at Monte Po, Villa Comunale, Paja, Piana CT-STM-M6.

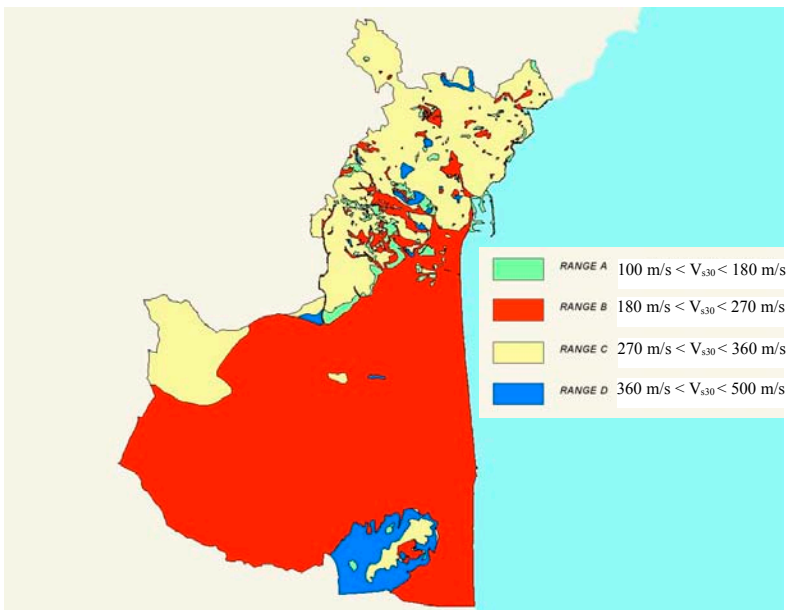
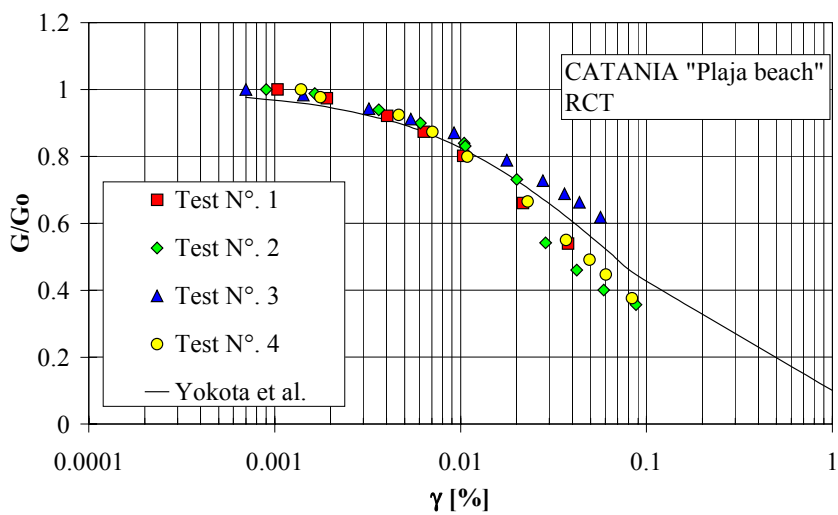
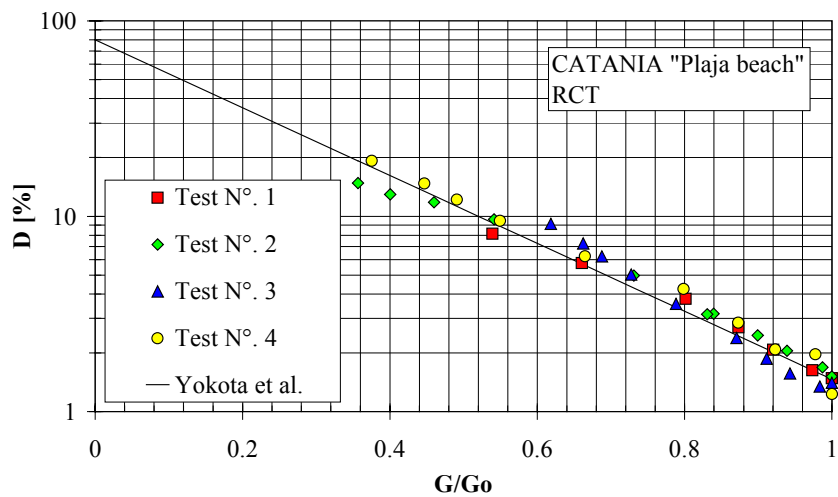


Figure 9: Geophysical map of the city of Catania based on shear waves velocity empirical correlations and measurements at the test sites.



a) G/G_0 - γ curves from RCT for "Plaja beach" site.



b) D - G/G_0 curves from RCT for "Plaja beach" site

Figure 10: Soil non linearity for site response analysis (after Cavallaro and Maugeri [10]).



defining the input motion through both a conventional approach (i.e., scaled recorded accelerograms at bedrock) and the synthetic accelerograms given by the 2-D code at a given depth (see section 2). The results show that the seismic input provided by the deterministic 2-D simulations, which reaches the value of about 0.5 g, is considerably larger than the probabilistic one, and it has the effect of producing large non-linear behaviour within the soil column. Hence, 1-D non-linear modelling has to be preferred to the 2-D linear one in the epicentre area, whenever the soil structure can be approximated by a 1-D model.

The geophysical map of shear waves velocity was geo-settled, as well as the 1100 borings considered and the related soil profiles on which the 1-D site response were performed.

According to the Manual for Zonation on Seismic Geotechnical Hazards (ISSMGE [9]), the Grade 3 microzonation for ground movement of the city of Catania was performed. The geo-settled map of the microzonation of the whole city of Catania is reported in Fig. 11. The map shows the variability of peak ground acceleration due to the soil heterogeneity. It is also to be stressed that the expected peak ground acceleration evaluated by the deterministic approach and local site response given by the map is greater than that reported in Fig. 1a, based on probabilistic approach.

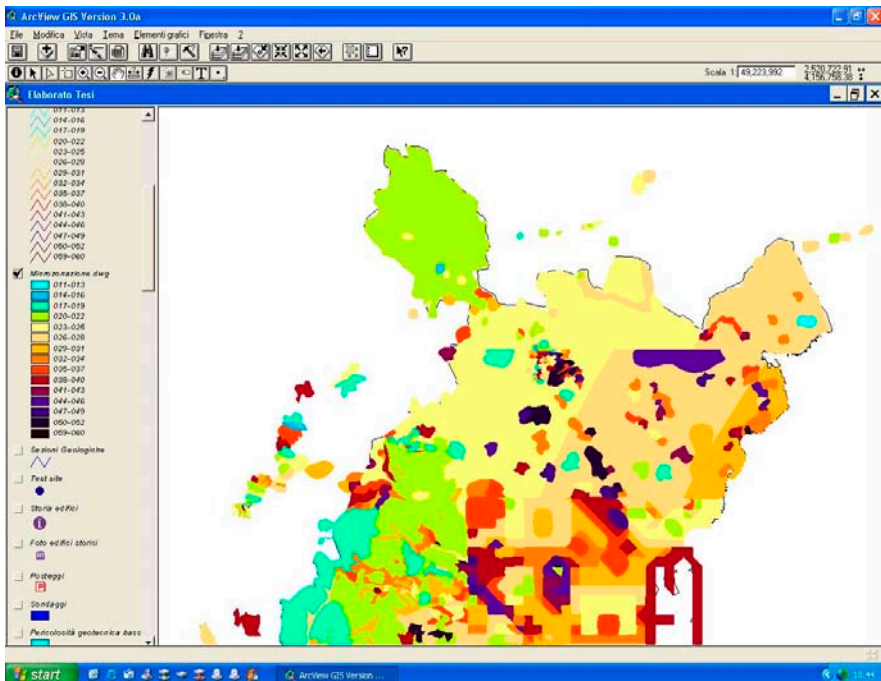


Figure 11: The geo-settled microzonation map in terms of peak ground acceleration based on deterministic evaluation for the January 11, 1693 strong scenario earthquake.

4 Vulnerability of seismic environment by landslides, liquefaction and cavity collapse

Seismic vulnerability is generally linked with the analysis of vulnerability of building, while it is also very important to analyze the stability of soils where the buildings are founded. This new concept leads to the analysis of the vulnerability of physical environment.

The main goals for the vulnerability of physical environment are: modeling of the vulnerability of slopes to the Scenario Earthquake and application of the model one representative landslides behaviour, i.e. the Monte Po landslide, located in the urban area of Catania; modeling of liquefaction including instability due to lateral spreading; survey of the cavities under the Catania area and implementation of a database of detected cavities.

To detect the slope stability hazard two new models have been developed of which one for clay slope for which soil stability is affected by strength cyclic degradation (Biondi and Maugeri [12]) and one for saturated sand slope for which soil stability is affected by pore pressure build-up (Biondi et al. [13]). The model referred to clay slope has been applied for based displacement analysis of the Monte Po landslide in Catania (Figs. 12 and 13).

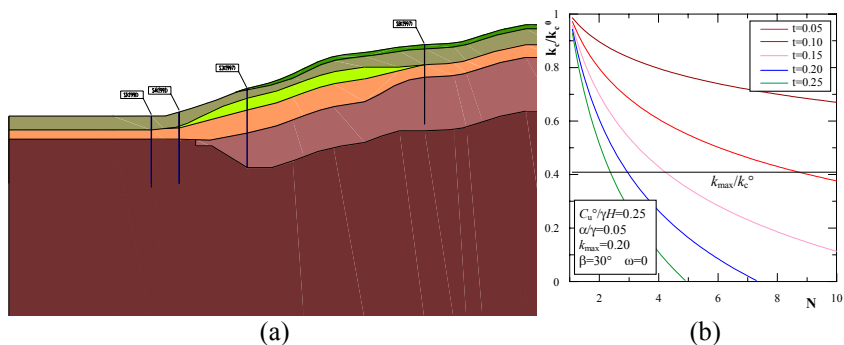


Figure 12: a) Schematic section of the Monte Po slope assumed in the analysis; b) Reduction of slope critical acceleration caused by the soil cyclic degradation.

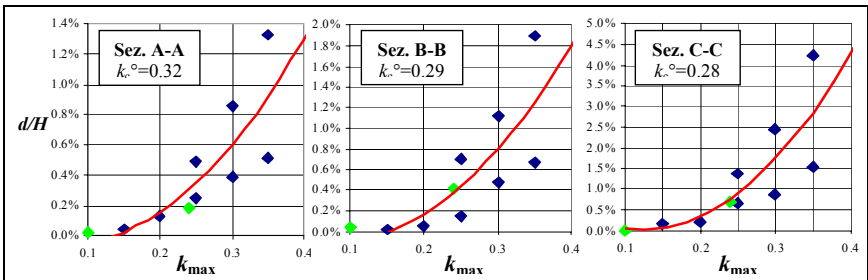


Figure 13: Results of the displacement response analysis of the Monte Po slope.

The new model has been applied to the potential landslides in the city of Catania; the geo-settled microzoning map of landslides hazard is reported in Fig. 14.

The main landslide hazard is linked with the Monte Po landslide (No. 4 in the map), for which the stabilization work has been made. At high risk are also the Santa Sofia landslides (No. 2a and No. 2b), for which the stability analyses have been made. At medium risk level is the Acquicella landslide (No. 6), for which the stability analysis has been made. At low risk can be classified all the remaining landslides.

The model referred to the saturated sand slope has been applied to the shore line of Catania city where flow failure with lateral spreading can be expected because of liquefaction phenomena. The analysis of potential liquefaction (Grasso and Maugeri [14]) and of the lateral spreading leads to the geo-settled microzoning map of the liquefaction hazard (Fig. 15). All the Shore-line, along Catania beach, has been classified at high hazard, including part of the Harbor. At low hazard is the area near Librino and Pigno.

Survey of the cavities under the Catania area and implementation of a database of detected cavities have been made. The following cavities have been studied in detail: Casa di Sant'Agata, cavity Piazza A. Di Benedetto, cavities via Lavandaie, Pozzo Gammazita, cavity Piazza Currò, Cripta of S. Agostino Church (Bonaccorso et al. [15]). In the city of Catania the cavities represent a high risk for foundation stability of some buildings (Fig. 16).

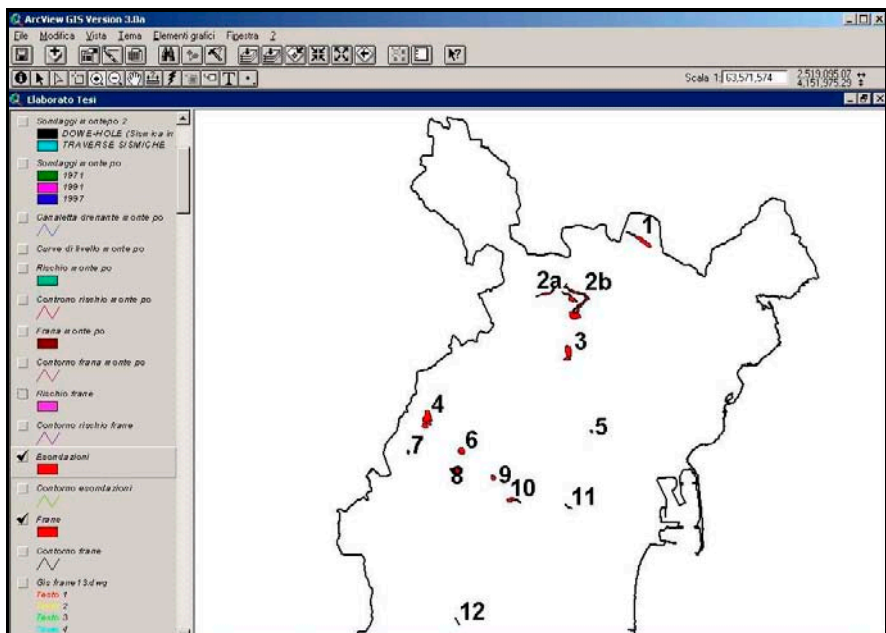


Figure 14: GIS localisation of the landslide hazard areas in the central part of Catania.



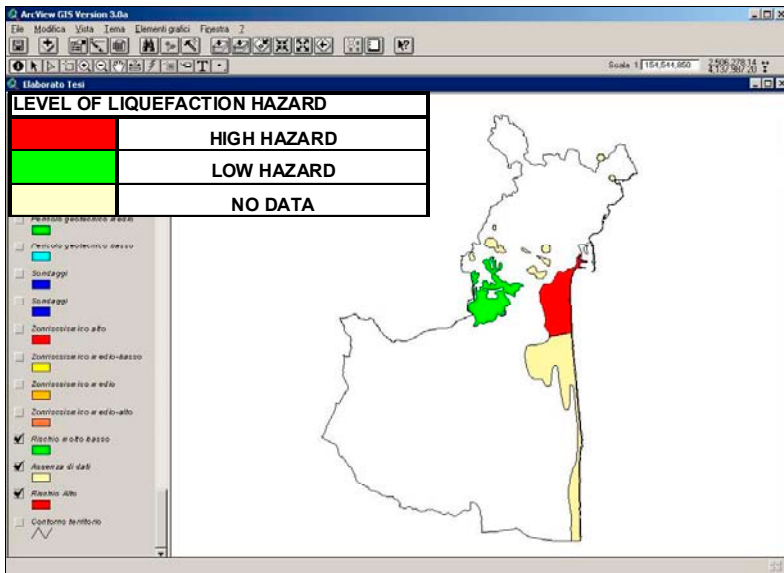


Figure 15: Map of the liquefaction potential of the city of Catania.

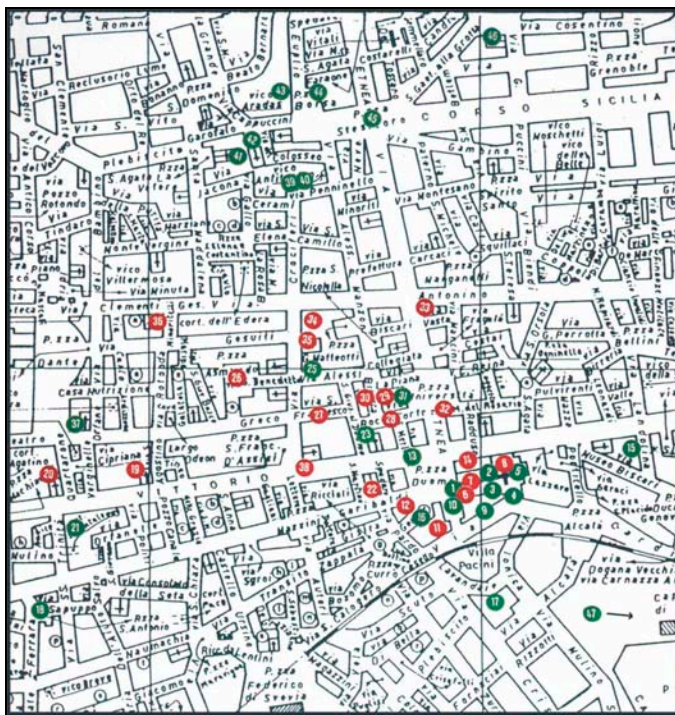


Figure 16: Map of Cavities located at the Catania city centre.

5 Geotechnical structures and soil-structure interaction

After analyzing the vulnerability of physical environment (landslides, liquefaction and cavity collapse, before analyzing the building vulnerability, the vulnerability of geotechnical structures, including foundation and retaining walls, as well as soil-structure interaction must be analyzed.

A new model for analyzing retaining wall stability has been developed and application of the model for evaluating the factor of safety has been performed (Caltabiano et al. [16]). The risk analysis of road infrastructure system is related to the stability of retaining walls; in some cases in the city of Catania the retaining wall is supporting a building; the model developed takes into account different typology of surcharge applied on the backfill due to buildings, vehicles etc.

A new model for bearing capacity analysis taking into account inertial forces not only in the foundation but also in the soil, according to the suggestion of Eurocode EC8, has been developed [17]. The model has been applied to the foundation analysis of some masonry and R.C. buildings built in Catania with no seismic design (Fig. 17). The analysis of foundation stability is based on the results of the test sites investigation. The results achieved show that the existing foundation must be improved to resist against seismic forces. The reinforcement of foundation will be considered in the Code of Practice concerning the assessment and strengthening of reinforced concrete buildings. Soil-foundation-structure interaction has been analyzed by shaking table tests for a frame of a R.C. building [18].



Figure 17: Location at the Catania city centre of some masonry old buildings built between 1917 and 1931, for which the foundation stability analysis has been carried out.



6 Vulnerability of monuments and buildings

The main goals for the vulnerability and seismic structural improvement of buildings to prevent damage are: assessment of the construction typology, identification tests and evaluation of vulnerability and earthquake resistance of monumental buildings; evaluation of building vulnerability and earthquake resistance for the most common construction typology of R.C. buildings; evaluation of critical acceleration, for limiting state serviceability vulnerability for most common construction typology of R.C. buildings.

About the vulnerability of Monuments, the vulnerability models and scenarios for the Churches have been analysed by Cavaleri et al. [19]. Concerning masonry ancient Churches, an improved survey based on points and penalties has been proposed by Zingone et al. [20]. The propose methodology has been already applied to survey and evaluate a set of 10 churches. Among these Churches, the most vulnerable one in order to carry out specific study has been selected. Further assessment of the selected monument based on diagnostic tests and numerical simulation have been made for Saint Nicola alla Rena Church (Valente and Zingone [21]) damaged by the moderate 1990 earthquake. The results of the case study of Saint Nicola alla Rena have been organized so that the following steps have been stressed: a) definition of a reference analytical model based on data revealed by means of the surveying forms; b) design of the vibrational test in situ, by means of numerical simulations, to acquire the dynamic characteristics, subjecting the systems to different types of dynamic loads; c) execution of the test in situ and acquisition of the dynamic characteristics in terms of accelerations; d) development of an accurate analysis in the time and frequency domains of the acquired responses and definition of both flexural and torsional modal shapes (Figure 18).

About the vulnerability of R.C. buildings, Cosenza et al. [22] have been carried out the seismic assessment of two reinforced concrete buildings, representative of the most common r.c. typologies of the Catania city. The studied outlined a high vulnerability for the buildings: the collapse will occur for PGA values ranging between 0.10 and 0.15 g, whereas the expected PGA values are about equal to 0.3 – 0.4. However, as outlined by Cosenza et al. [23], the studies carried out by different research groups (Verderame et al. [24]), (Decanini and Mollaioli [25]) showed that the use of different models result inconsistent even if concerning the same building.

Vulnerability of R.C. buildings can be made by simplified procedures based on vulnerability score or by detailed analysis based on resistance evaluation of the buildings against earthquakes. The considered building has been built in the years '70 and never completed. Because of this a degradation phenomena caused a decreasing of strength to be evaluated. With this aim the procedure developed by Oliveto et al. [26] and [27], has been applied. The results obtained show a high degree of vulnerability for the expected seismicity at the site. The evaluation of seismic resistance and vulnerability of existing buildings are strictly correlated because preliminary must be evaluated the strength against earthquake and the consequent vulnerability of it.



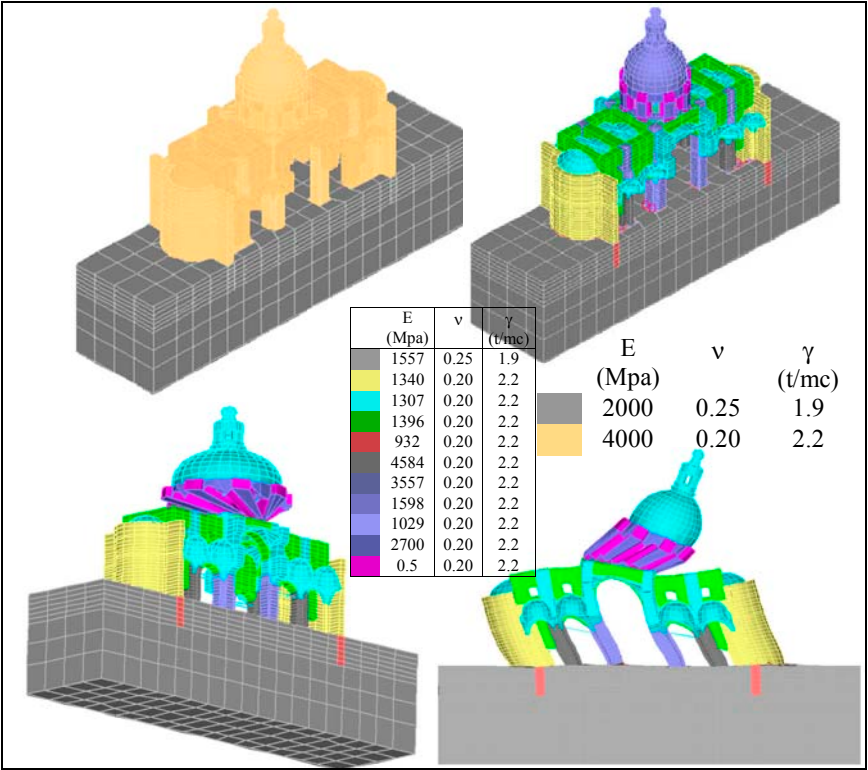


Figure 18: Numerical model dynamically identifying the San Nicola alla Rena Church in Catania.

7 Strengthening of reinforced concrete buildings to prevent damage

The main goals for strengthening the reinforced concrete buildings are: remedial works for most common construction typology of R.C. buildings with traditional and innovative techniques; Code of Practice for the improvement of the most common typology of R.C. buildings; transfer of the Code of Practice to the Municipality and other Institution; transfer of the Code of Practice to the Engineers and to the Technicians; transfer to the Municipality office a Land Information System (LIS) database of all the results obtained by the Research Project; criteria for priority on the remedial works execution.

Among the various systems for structural improvement for the considered building, the base seismic isolation is particularly suitable. However even with the base isolation, some buildings can show some vulnerability. In this case a structural strengthening is also needed. As an example, for a building base-isolated some shear walls were modelled and designed by Calio and Marletta [28] and [29].

The seismic structural improvement was then consisting of base isolation (Fig. 19) and of structural strengthening. The study shows an interesting behaviour predicted by the model. Because of the structural strengthening due to shear walls, the vibration periods of the building became less, and then bigger seismic forces were acting on the building. So the increasing of the resistance due to the shear walls was not enough to compensate the increasing of actions. However, because of base isolation, the vibration periods of the building became higher as well as the damping; consequently the seismic action was decreasing and the building behaves as linear system.

For the assessment and strengthening of reinforced concrete buildings, a code of practice will be developed and disseminated among professional engineering. The Code of Practice (Nisticò et al. [30]) concerns inspection, assessment and strengthening provisions. The inspection provisions concern all the activities needed to define building geometry and mechanical properties of concrete and reinforcement. Among the recently proposed seismic assessment guidelines the ATC 40 proposal seems to be better tailored for the Italian scenario, safe that some adjustment is needed.

The proposed intelligent Data Bank [30] is an integrated software expert system [31] for the seismic vulnerability evaluation. The system provides an expert interface and a vulnerability analyzer. The expert interface assists the surveyor in the geometric and mechanical description of reinforced concrete buildings; the vulnerability analyzers will assist the engineers in the planning and estimation of the interventions for seismic risk management. Finally, the “local” risk analysis could be connected to the “global” analysis of the city through a G.I.S. interface. Further the Code of Practice will drive, if needed, the engineer in the selection of the best retrofitting strategy by means of heuristic rules and comparative numerical simulation.



Figure 19: Squat columns in the original building before (a) and after (b) the insertion of the seismic isolators (After Calìo and Marletta [28]).

8 Vulnerability of infrastructure systems

The vulnerability of a city is more than the summation of the vulnerability of a single building because a city is a fragile system composed by different sub-systems such as: the road system; the lifelines system and the urban buildings system.

The vulnerability of the roads have been related to vulnerability of landslides (see section 4) and retaining walls (see section 5). The vulnerability of roads due to landslides is reported in Fig. 20 (Biondi et al. [37]). The vulnerability of bridges was investigated by Calvi and Pavese [32], Martinelli and Perotti [33], Nisticò and Monti [34]. In some cases it is very relevant to analyze the functionality of road network during and after an earthquake. As regards road infrastructure system, an original methodology for the risk analysis of the functionality of the urban infrastructures system during earthquakes has been developed and applied to the risk analysis of a specific urban area of Catania by Cafiso et al. [35 and 36]. The analysis of road system vulnerability is made on the basis of: geometrical characteristics of roads and buildings prone to its; vulnerability of these buildings; the exposure to traffic flows. The map of the vulnerability of road systems is reported in Fig. 21.

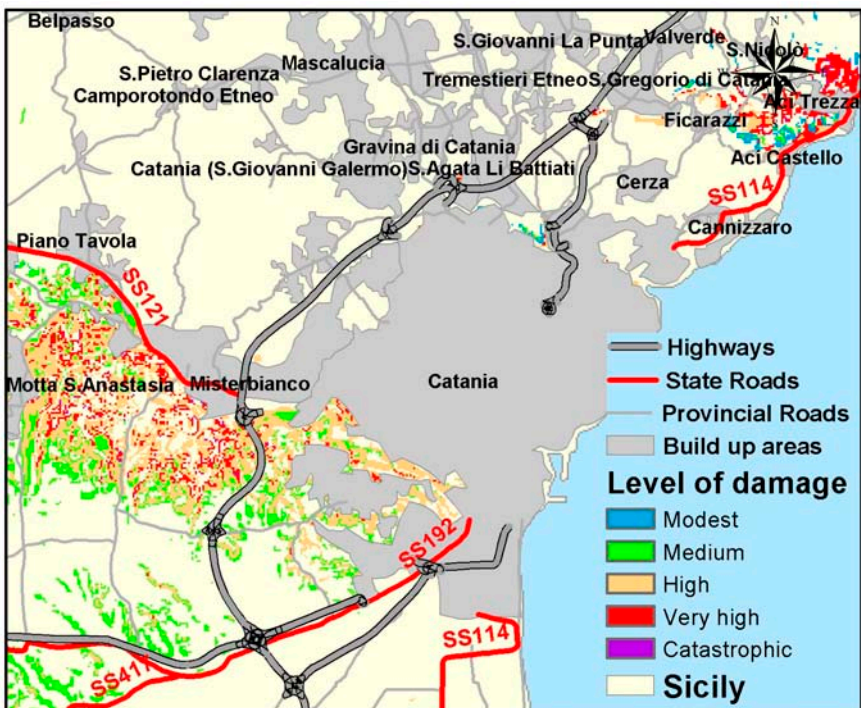


Figure 20: Map of earthquake-induced damages on slopes: detail for the Catania area.



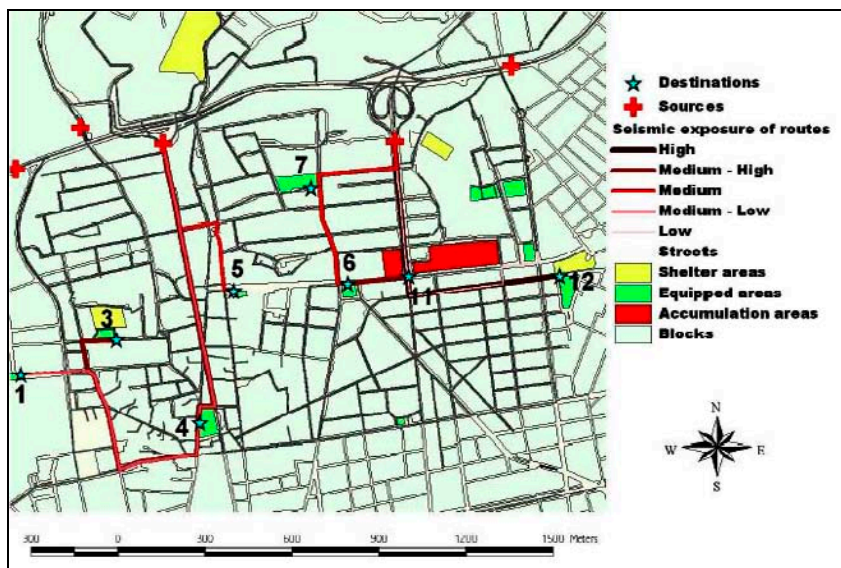


Figure 21: Indirect seismic exposure of the selected routes, after Cafiso et al. [35].

Some studies were carried out in the past to assess the response of buried pipes to lateral ground movements with the aim to establish pipe failure risk. The analyses of soil-pipe interaction in slope with earthquake-induced movements is developed using the discrete element method to evaluate deformations and stresses in pipelines crossing unstable slopes.

The distribution of displacement, lateral deflection and bending moment along the pipe are calculated for the prevision of unacceptable conditions for pipelines and to prevent seismic hazard in a risk analysis, as reported by Casamichele et al. [38] in Figs. 22 and 23. Seismic hazard assessment for pipelines crossing unstable slopes has been also performed by Casamichele et al. [39].

The seismic vulnerability of the urban system of Catania is considered as a set of relationships between built areas and void areas for connection. 1.346 void space are considered, consisting of streets and squares. The prevailing causes for the exposure of the population (in each empty urban space) caused by the activities practised in the built areas have been defined. To this aim the main typologies of economic activities have been determined and specific forms of evaluation have been defined. The points are assigned to the five categories of judgement (year of construction of the manufacture where the activity is located, number of consumers/hour, function of the road, presence of analogous activities within the radius of 300 m, general vulnerability), with maximum value of 50, which is also the index of maximum risk.

As regards the evaluation of the general vulnerability of the urban framework of Catania, the following factors have been considered: organisation of the

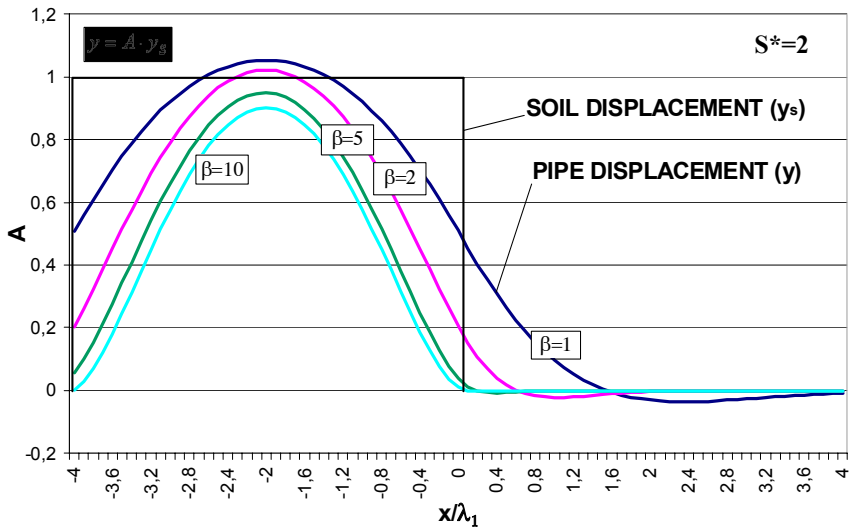


Figure 22: Analysis of lifelines hazard: Soil and pipe displacement.

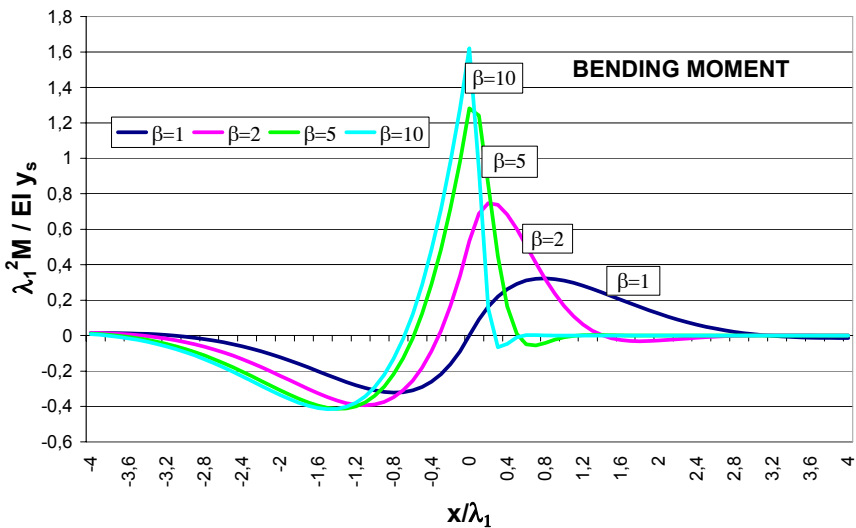


Figure 23. Analysis of lifelines hazard: elasto-plastic evaluation of bending moment for the load on the pipe $p=0.04$ and soil displacement $y_s=0.05$ m for different values of stiffness ratio parameter $\beta = \lambda_1/\lambda_2$, where λ_1 is the vertical characteristic length for the pipeline in the unstable zone and λ_2 is the characteristic length for the pipeline in the stable zone.

vertical structures (the presence of the connections between orthogonal walls); nature of the vertical structures (employed materials and their conditions); position of the building; type of foundations; distribution of the resistance elements; regularity of the project; presence of appendices or projections; state of fact and evident interventions of amelioration or maintenance carried out; lack of joints. The seismic vulnerability of urban system is linked not only with the vulnerability of buildings (see section 6) but also with the functionality of road network and interruption of economic activities. Also the exposition of the population due to economic activities is considered for the evaluation of the seismic risk (Campo [40]).

9 Conclusions

The road map followed for seismic prevention of damage could be considered a multidisciplinary task, involving geologists, seismologists, geotechnical engineers, structural engineers, transportation engineers and urban planners for the assessment of the seismic hazard, for the evaluation of the seismic risk of physical environment, for the prevention of seismic risk for new constructions and for the mitigation of seismic risk in the existing constructions, including monuments and cultural heritage.

The key-point for the evaluation of the seismic risk is the evaluation of the seismic hazard. The evaluation of seismic action for which the probabilistic or deterministic approaches can be used. Increasing the historical knowledge of past earthquakes and changing the probabilistic criteria, the probabilistic approach leads to update the seismic action, which has been changed in the past in Italy, so, buildings designed to resist to the given seismic action at the time of construction earthquakes could be not resisting at the updated seismic action.

Alternatively to the probabilistic approach, the road map for the evaluation of seismic damage takes into consideration the deterministic approach based on the evaluation of source mechanism. The scenario earthquake considered is the January 1693 events, as the biggest one; the February 1818 earthquake as medium earthquake and the December 1990 as a low earthquake.

Local site response has been evaluated with 2-D and 1-D model. An innovative approach was that of evaluating the synthetic accelerograms not only at the surface but also at the bedrock. Using the last as input, PGA up to 0.5g was evaluated. The site response with 1-D models is highly influenced by soil non-linearity. From results so far obtained, the 1-D non-linear model is preferable to 2-D linear model in the epicentral and flat area. A microzonation for ground movements of the city of Catania is presented. The results show great variability of the local site response due to the variability of soils (clay, sand and lava rock).

As far as concern the vulnerability of physical environment due to landslides, two innovative models have been developed: one for clay slope and one for saturated sand slope. The models have been applied to the landslide and liquefaction hazard evaluation in the city of Catania. Among the element of



vulnerability of physical environment, the peculiar presence of cavity in the city of Catania has been also taken into account.

An innovative model has been developed for foundation stability evaluation including soil inertia effect, according to the suggestions of the Eurocode EC8. An original model has been developed for the evaluation of the seismic stability of the earth retaining walls, which is a relevant topic for the physical vulnerability of the road infrastructure system.

As far as concern the vulnerability of buildings and monuments, an innovative procedure has been used for the numerical model dynamically identified of the Church S. Nicolò l'Arena. Innovative procedures have been also proposed for strength analysis of an existing building and for the strengthening of buildings to resist to an earthquake. As an application is shown a building reinforced by base-isolation and shear walls.

An innovative procedure has been developed also for the evaluation of the functionality of the road system during and after an earthquake. An innovative approach is also to evaluate the vulnerability of lifelines system. Finally an innovative aspect is represented by the evaluation of the seismic vulnerability of the urban system, by the analysis of the vulnerability of urban building aggregates and the analysis of the number of population exposed to the seismic risk.

In conclusion the work has been developed methodology aspects and innovative models related to: deterministic evaluation of seismic action, site effect evaluation, microzonation and vulnerability of physical environment evaluation (landslides, liquefaction, cavities), vulnerability and strengthening of test buildings, vulnerability of road system, lifelines system and urban buildings aggregates.

References

- [1] Maugeri M. (2005). "Seismic Prevention of Damage: a case history in a Mediterranean City". M. Maugeri Editor, WIT Press Southampton, 408 p.
- [2] Priolo E., (2000). "2-D Spectral Element Simulation of the Ground Motion for a Catastrophic Earthquake". In the Catania Project – Earthquake Damage Scenarios for a High Risk Area in the Mediterranean. Faccioli and Pessina Editors. GNDT, Rome, 2000, 225 p.
- [3] Laurenzano G., Priolo E., Kline P. and Vuan A. (2004). Near fault earthquake scenarios for the February 20, 1818 M=6.2 'Catanese' event. Proc. Of the Conf. Risk analysis 2004, Rhodes-29 September 2004.
- [4] Di Bona, M., Cocco, M., Rovelli, A., Berardi, R., and Boschi, E. (1995). "Analysis of strong-motion data of the 1990 Eastern Sicily earthquake", *Ann. Geofis.*, **38**, 283-300.
- [5] Laurenzano G., and Priolo E. (2005). Numerical Modelling of the December 13, 1990, M=5.8 Eastern Sicily Earthquake. A Case History: the city of Catania (Italy). Editor M. Maugeri. WIT Press, Southampton.
- [6] Priolo, E. (1999). "2-D spectral element simulations of destructive ground shaking in Catania (Italy)", *J. of Seismology*, **3** (3), 289-309.



- [7] Monaco C., Catalano S., De Guidi G., Tortorici L. (2004). The Geological map of the urban area of Catania (Eastern Sicily). In: Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy). Editor M. Maugeri. WIT Press, Southampton.
- [8] Grasso S., Laurenzano G., Maugeri M. and Priolo E. (2004). Seismic response in Catania by different methodologies. In: Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy). Editor M. Maugeri. WIT Press, Southampton.
- [9] ISSMGE (1999). Manual for Zonation on Seismic Geotechnical Hazards (Revised Version). The Technical Committee No. 4 for Earthquake Geotechnical Engineering of the ISSMGE, published by the Japanese Geotechnical Society of SMGE.
- [10] Cavallaro A., Maugeri M. "Non linear behaviour of sandy soil for the city of Catania". In: Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy). Editor M. Maugeri. WIT Press, Southampton.
- [11] Cavallaro A., Maugeri M., Lo Presti D.C.F. and Pallara O., 1999. Characterising Shear Modulus and Damping from in Situ and Laboratory Tests for the Seismic Area of Catania. *Proceeding of the 2nd International Symposium on Pre-failure Deformation Characteristics of Geomaterials*, Torino, 28 - 30 September 1999, pp. 51 - 58.
- [12] Biondi G. Maugeri M. (2004). Seismic response analysis of Monte Po hill (Catania). In: Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy). Editor M. Maugeri. WIT Press, Southampton.
- [13] Biondi G., Cascone E., Maugeri M., Motta E. (2002). "Earthquake effects on the displacements of the liquefiable slopes". Topic: 2.2 - "Dam and Slopes". 12-th European Conference on Earthquake Engineering, Londra, 12-16 September 2002.
- [14] Grasso S., Maugeri M. 2006. "Using Kd and Vs From Seismic Dilatometer (SDMT) for Evaluating Soil Liquefaction". Proc. of the Second International Conference on the Flat Dilatometer, Washington, April 2 – 5, 2006.
- [15] Bonaccorso R., Grasso S., Lo Giudice E., Maugeri M., 2004. "Cavities and hypogaeal structures of the historical part of the city of Catania". In: Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy). Editor M. Maugeri. WIT Press, Southampton.
- [16] Caltabiano, S., Cascone, E., Maugeri, M. (2004). Seismic factor of safety evaluation for earthquake retaining walls. In: Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy). Editor M. Maugeri. WIT Press, Southampton.
- [17] Maugeri, M., Novità, D. (2004). Evaluation of the dynamic bearing capacity of a masonry building by means of a characteristics line method. In: Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy). Editor M. Maugeri. WIT Press, Southampton.



- [18] Massimino, M.R., Maugeri, M. (2004). Shaking table test and numerical modelling of dynamic soil structure interaction. In: *Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy)*. Editor M. Maugeri. WIT Press, Southampton.
- [19] Cavaleri, L., Lagomarsino, S., Podestà, S., Zingone, G. (2000). Vulnerabilità models and Damage Scenarios for the Churches. In *The Catania Project. Earthquake damage scenarios for a high risk area in the Mediterranean*. Faccioli E. and Pessina V. (Eds), CNR-GNDT, Roma, 2000, pp. 205-212.
- [20] Zingone, G., Cavaleri, L., Cucchiara, C. (1999). Seismic vulnerability of calcarenite ashlar churches. *Acts of the Workshop on Seismic performance of Monument, Lisboa, Portugal, November, 12-14*.
- [21] Valente G., Zingone G. (2004). Methodology and techniques for seismic protection of the monumental patrimony. In: *Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy)*. Editor M. Maugeri. WIT Press, Southampton.
- [22] Cosenza, E., Manfredi, G., Verderame, G. (2000). Seismic assessment of R.C. structures: case studies in Catania. In *The Catania Project: Earthquake Damage Scenarios for a High Risk Area in the Mediterranean*. Faccioli E. and Pessina V. (Eds), CNR-GNDT, Roma, 2000, pp. 161-167.
- [23] Cosenza, E., Manfredi, G. and Verderame, G. M. (2002) Seismic assessment of gravity load designed r.c. frames: critical issues in structural modelling, *Journal of Earthquake Engineering*, vol. 6, Special Issue 1, 101-22.
- [24] Verderame, G.M., Polese, M., Cosenza, E. and Manfredi, G. (2000). *Analisi di Vulnerabilità Sismica di un edifici in cemento realizzato nella città di Catania antecedentemente alla normativa sismica, Il comportamento sismico di edifici in c.a. progettati per carichi verticali – Applicazioni all’edilizia di Catania*”, CNR-GNDT, 201 pp.
- [25] Decanini, L.D. and Mollaioli, F. (2000) *Analisi di Vulnerabilità Sismica di Edifici in Cemento Armato Pre-Normativa, Il comportamento sismico di edifici in c.a. progettati per carichi verticali – Applicazioni all’edilizia di Catania*”, CNR-GNDT, 201 pp.
- [26] Oliveto G., Calì I., Marletta M., 2001. Resistenza di un edificio in c.a. realizzato nella città di Catania antecedentemente all’entrata in vigore della legge sismica. In: E. Cosenza (editor), *Comportamento sismico di edifici in cemento armato progettati per carichi verticali: applicazioni all’edilizia della città di Catania*, CNR-GNDT, Esagrafica. Roma, ISBN: 88-88151-02-8.
- [27] Oliveto G., Calì I., Marletta M., 2002. Seismic resistance and vulnerability of reinforced concrete buildings not designed for earthquake action. In: G. Oliveto (editor), *Innovative Approaches to Earthquake Engineering*. WIT Press, Southampton (UK), ISBN: 1-85312-885-6.
- [28] Calì I., Marletta M., 2004. Seismic resistance of reinforced concrete buildings with shear walls. In: *Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy)*. Editor M. Maugeri. WIT Press, Southampton



- [29] Calìo I., Marletta M., Vaccaro S., 2004. Seismic resistance of existing reinforced concrete buildings retrofitted by base isolation. In: *Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy)*. Editor M. Maugeri. WIT Press, Southampton.
- [30] Braga F., Negri M., Nisticò N., Tanzillo M., 2004. A systematic approach concerning the assessment and strengthening of reinforced concrete buildings of the Catania city. In: *Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy)*. Editor M. Maugeri. WIT Press, Southampton.
- [31] Padula A., 2004. A shell for the construction of knowledge bases aimed at assessing the behaviour of rc buildings under seismic actions. In: *Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy)*. Editor M. Maugeri. WIT Press, Southampton.
- [32] Calvi G.M., Pavese A. (2000). Seismic assessment of bridges in the Catania area: seismic assessment of bridges piers. In *The Catania Project: Earthquake Damage Scenarios for a High Risk Area in the Mediterranean*. Faccioli E. and Pessina V. (Eds), CNR-GNDT, Roma, 2000, pp. 188-192.
- [33] Martinelli L., Perotti F. (2000). Seismic assessment of bridges in the Catania area: detailed analysis of a typical overcrossing. In *The Catania Project: Earthquake Damage Scenarios for a High Risk Area in the Mediterranean*. Faccioli E. and Pessina V. (Eds), CNR-GNDT, Roma, 2000, pp. 193-197.
- [34] Nisticò N., Monti G. (2000). Seismic assessment of bridges in the Catania area: damage scenario. In *The Catania Project: Earthquake Damage Scenarios for a High Risk Area in the Mediterranean*. Faccioli E. and Pessina V. (Eds), CNR-GNDT, Roma, 2000, pp. 198-203.
- [35] Cafiso S., Condorelli A., Mussumeci G., 2004. Functional analysis of the urban road network in seismic emergency. A GIS application on Catania city. In: *Seismic Prevention of Damage for Mediterranean Cities. A Case History the city of Catania (Italy)*. Ed. M. Maugeri. WIT Press Southampton.
- [36] Cafiso, S., Condorelli, A., Cutrona, G., Mussumeci G., 2004. A Seismic Network Reliability Evaluation on GIS Environment - A Case Study on Catania Province. *Risk Analysis 2004*, Rodi.
- [37] Biondi G., Condorelli A., Maugeri M., Mussumeci G., 2004. Earthquake-Triggered Landslides hazard in the Catania Area. *Risk Analysis 2004*, Rodi.
- [38] Casamichele P., Maugeri M., Motta E., 2004. Seismic hazard assessment for pipelines crossino unstable slopes. In: *Seismic Prevention of Damage for Mediterranean Cities. A Case History: the city of Catania (Italy)*. Editor M. Maugeri. WIT press, Southampton.
- [39] Casamichele P., Maugeri M., Motta E., 2004. Numerical analysis of buried pipes subjected to lateral soil movements. *Risk Analysis 2004*, Rodi.
- [40] Campo G., 2001. The seismic vulnerability of the urban framework of Catania: scenarios and interventions. In: *Verso una città sicura*. Eds. Maugeri M. and Grasso S. (in Italian).



Section 1

Earthquake resistant design

This page intentionally left blank

Vulnerability functions and the influence of seismic design parameters on initial costs for buildings provided with hysteretic energy-dissipating devices

J. García-Pérez, M. Zenteno & O. Díaz

*Instituto de Ingeniería, Edificio 2, Mecánica Aplicada, UNAM,
Ciudad Universitaria, Mexico*

Abstract

This paper is intended to attain both the influence of seismic design parameters on initial cost and seismic vulnerability functions for reinforced concrete buildings provided with hysteretic energy-dissipating devices. In order to obtain this, an established methodology for earthquake resistant design is applied to different types of buildings.

In the optimization process, in order to attain optimum design values, it is necessary to have both initial cost functions, as well as costs due to earthquakes. Initial cost functions are described in terms of design parameters, usually the seismic design coefficient or the vibration period. The influence of seismic design parameters on the initial cost is first studied, and then functions relating costs of the structures to the design parameters are obtained. In order to do this, we analyze different types of reinforced concrete buildings where each one is represented by a reinforced concrete frame composed of beams and columns, with hysteretic energy-dissipating devices installed as braces. The structures studied are hypothetical buildings built at a soft site in the Valley of Mexico with a different number of stories. Cost analyses obtained for these systems are compared with those attained for a conventional frame just composed of beams and columns. Vulnerability functions (drift-seismic intensity) are obtained from those structures studied here. These vulnerability functions together with the cost analyses performed are used to find the cost of damage-seismic intensity relations. The results show that the use of systems with energy-dissipating devices gives a better cost-benefit behavior when the system is under high seismic intensities. Moreover, these results are appropriate for performing long-term cost-benefit analyses.

Keywords: energy dissipation, costs, vulnerability curves, reinforced concrete buildings.



1 Introduction

The current seismic design of reinforced concrete buildings is based on the development of structural capacity to dissipate energy due to the action of the acting loads, through elastic or inelastic deformations, thereby developing more efficient levels of structural control every day. One way to attain this structural control is by the so called passive control consisting of increasing structural damping, modifying the natural period of vibration or combining both structural properties. One of the devices applied to a structure requiring this type of control is known as energy dissipating device (EDD). Here we use the type of EDD installed as an external brace to the reinforced concrete frame, linked by stiffeners working in tension or compression. These devices are manufactured in a factory, thus maintaining appropriate quality control, and providing stable hysteretic behavior under high cycles of deformations. The foregoing provides high capacity for dissipating energy. Failure conditions of these elements are obtained at laboratories by studying their load deformation capacity.

On the other hand, in order to attain optimum design parameters for buildings erected in seismic zones, the current optimization philosophy requires balancing the total expected present cost of a structure, including the initial cost and maintenance costs, as well as losses due to damage and failure. Regarding initial costs in terms of the base shear design coefficient, we find some expressions in the literature developed by Whitman et al [1], Grandori [2], Ferrito [3], Rosenblueth [4], Vargas and Jara [5], and García-Pérez [6]. Some studies have also been done in obtaining initial costs functions in terms of the natural period of vibration of the structure (Reyes [7], Esteva et al [8], García-Pérez et al [9]). Despite the studies carried out so far, some expressions are still necessary for initial costs in terms of design parameters, especially for structures with energy dissipating devices. Here we find some expressions in terms of the basal shear coefficients, as well as the period of vibration for both conventional structures and structures with EDDs. Then the initial cost expressions are used to obtain functions describing probable damages to the structures in terms of the intensity of the seismic motion causing these damages.

2 Buildings with energy dissipating devices

The purpose of seismic design is to provide each structure with characteristics allowing for developing optimum behavior in terms of their design economy, when subjected to the action of earthquakes occurring during their life-cycle. Satisfactory structural behavior is expected under low intensity seismic events, since story displacements are controlled to minimize damage in nonstructural elements. Under strong seismic excitations, structural collapse through damage of some structural elements must be avoided by controlling their deformations.

A balance between economy and structural safety must be pursued, although allowable limits of linear behavior are exceeded in some members, but without reaching failure. Therefore, the structures proposed here are ductile spatial frames able to resist lateral forces through their stiffness and energy dissipating



devices. Ductile behavior of structures is closely linked to damage of their structural elements, thus research is directed at mixed systems comprised of reinforced concrete and EDDs, trying to concentrate dissipated energy mostly by reducing deterioration and degradation of mechanical and dynamical properties of conventional structural elements.

Seismic design criteria for reinforced concrete ductile frames established in the Federal District Building Code and its Complementary Technical Norms (RCDF) [10], consider that all structural elements under high intensity seismic excitation undergo inelastic deformations, when absorbing and dissipating a fraction of the total energy acting in the whole structure. These deformations are concentrated in specific regions of frames, such as the zones of maximum internal moments. Seismic design criteria are intended to avoid structural collapse by designs based on weak beams and strong columns, ensuring that under highly seismic excitations larger inelastic deformations will occur at the end of beams and not in the columns.

2.1 Methodology in buildings with EDDs

In order to design the buildings with EDDs studied here a methodology developed by Campos [11] is used. This methodology is based on a performance design criterion using allowable ductilities and a group of parameters that influence greatly on the behavior of the structural systems. These parameters are related each other during the different steps of the analysis and design.

The energy induced on a structure depends on different factors which are directly related to soil motion, damping, stiffness and strength, among other things. On the other hand, structural response under seismic excitation can be improved by either decreasing input seismic energy or by increasing dissipating energy. The latter may be reached by introducing viscous or hysteretic damping. The methodology developed by Campos [11] consists of obtaining stiffness and strength for each story without exceeding both allowable deformations and ductilities as well as developing the ductility established for the design.

2.2 Variables used in the design process

In the design methodology adopted, design variables denominated as control variables are those design parameters defining mechanical properties of the structural system, as well as maintenance and repairing politics. Lateral stiffness of the dissipating device k_d related to total stiffness K in each building story is denoted by $r_k = k_d / K$. Yielding displacement of the dissipating device δ_{yd} related to yielding displacement of the conventional frame δ_{yc} is represented by $\varphi = \delta_{yd} / \delta_{yc}$. Relationship between lateral strength of the dissipating device R_d to total lateral strength R of each story is given by $r_R = R_d / R = \varphi r_k / 1 + (\varphi - 1)r_k$. If we consider the same maximum lateral displacement of the story for both conventional frames and frames with EDDs, it is found that the relationship between ductilities under lateral displacement of a story for each case is given



by $\varepsilon = \mu_d / \mu_c = 1/\varphi$. From the definition of control variables, the following equations were obtained: $\alpha = r_k / 1 - r_k = k_d / k_c$, $\beta = \varphi\alpha = R_d / R_c$, where k_c and R_c are the stiffness and lateral strength of the story for the conventional frame. Expressions for total stiffness and strength are given by $K = k_c + k_d = k(1 + \alpha)$; $R = R_c + R_d = R_c(1 + \beta)$. In order to apply a superposition between the conventional frame and the system with EDDs it is necessary that $\mu = \delta_{\max} / \delta_y = \delta_{\max} K / R$, where δ_y and $\delta_{\max}$ are the yielding and maximum relative displacements of the story of the frame with EDDs. A relationship between elastic and inelastic spectral ordinates is expressed as $Q(\mu, T) = S_e(T) / S(\mu, T) = c_e / c_i$, where T stands for the structural period, and c_e and c_i denote the elastic and inelastic seismic design coefficient.

3 Seismic design procedure

3.1 Design spectrum

The design spectrum used here is an average spectrum normalized to the spectral intensity of a family of five accelerograms, with statistical properties similar to those of the record obtained for the September 19, 1985 earthquake at SCT site in Mexico City (García-Pérez et al [9]). Normalization of this spectrum consists in equating intensities of the elastic spectra of the earthquakes simulated to those intensities of the earthquake recorded. The spectral intensity of each normalized spectrum accelerations is defined by $(1/2\pi) \int_0^{T_1} S_a(T, \xi) dT$, where $S_a(T, \xi)$ is the spectral acceleration ordinate corresponding to structural period T , with damping $\xi = 0.05$ and $T_1 = 3.5$ sec. for earthquakes occurring in soft soil.

3.2 Procedure

Here we adopt the design process developed by Campos [11] consisting of a preliminary step or design and a final design. In the first step, values for variables α and β are computed taking into account the most appropriate level of ductility, and also maximum story displacements $\delta_{\max}$ (1.2 % of the height story) are determined. Then an estimation of the structural period is done by taking it as ten per cent of the total number of stories, and the corresponding elastic (c_e) and inelastic (c_i) coefficients are obtained with the aid of the design spectrum. Seismic forces and corresponding design shear force R_e are determined by means of a seismic static analysis, and the reduced design shear force R is obtained as $R = R_e / Q = R_e / \mu$. With these data, values of the control variables are proposed as a percentage of stiffness and strength that the EDD must provide. Then a relationship between ductilities is found such that it will be our limit parameter



of comparison between the structural design of the conventional frame and the frame with EDDs. It is also considered that under lateral loads, each story has two significant displacements, namely, a relative horizontal displacement δ_H , and a rotation as a rigid body θ resulting from elongation and contraction of columns, thus the story deformation is given by $\delta = \delta_H - \theta h$. Structural elements of the frame with EDDs are pre-designed in terms of stiffness and strength required according to the percentage that each system must provide (frame and EDD), thus successive iterations are performed until finding more appropriate dimensions giving displacements similar to those allowed. To complete the first step, a comparison between stiffness of the stories is made by coefficient $C_k = k_a / k_c$, which must stay within the permissible limit given by $|C_k - 1| \leq \epsilon$, where k_a is the new lateral stiffness of the story.

After determining dimensions of structural elements, beams, columns, and EDDs, we proceed to design them, and finally the relative displacements in each story are checked such that allowable limits indicated in the RCDF [10] are complied.

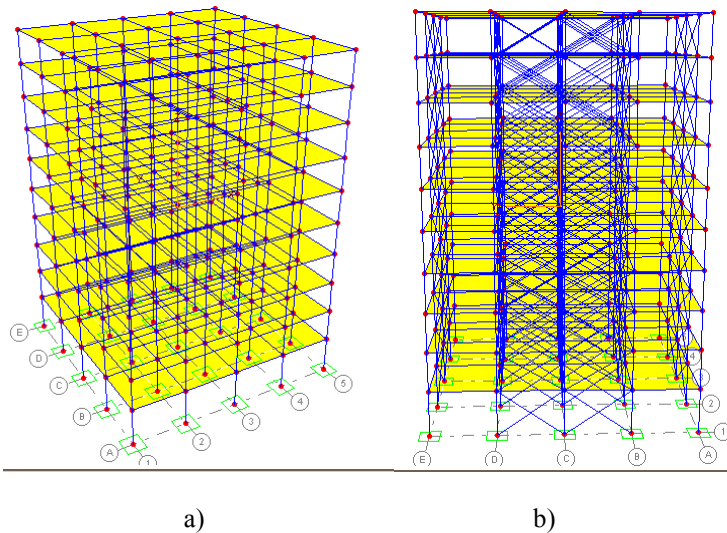


Figure 1: a) Conventional frame; b) frame with EDDs.

4 Types of structures studied

A group of regular buildings of five, ten, fourteen, and twenty stories were designed in order to find their initial cost functions. The structures are regular symmetrical frames in both directions, with four bays five meters wide each. The first story is 3.5 meters high while all upper stories are 3 m. high. Compression strength of the concrete $f'_c = 250 \text{ kg/cm}^2$ and yield strength of the



steel $f_y = 4200 \text{ kg/cm}^2$ are used. Floor systems are based on reinforced concrete slabs supported by beams and behaving as a rigid diaphragm. Columns are designed for uniaxial bending, as long as we are using bi-dimensional frames. A model of the buildings used is shown in fig. 1 for conventional frames and frames with EDDs, respectively.

Structural analysis of the buildings is performed under a model for plane frames comprised of rigid nodes connecting to flexible bars of finite stiffness. Their behavior resembles that of a real building by taking into account deformations by axial load in columns, as well as deformations due to bending and shear in beams and columns. Loads and load combinations applied to each structural system are determined according to the RCDF [10], resulting a dead load of 560 and 420 kg/m^2 for roof and story in this case, respectively, as well as a live load for gravitational loads of 250 and 100 kg/m^2 for roof and story, and a live load for earthquakes of 180 and 70 kg/m^2 .

4.1 Design of energy dissipating devices

The energy dissipating devices employed here are steel diagonals A-36 that stiffen the frame and are located diagonally according to the array previously determined, as shown in fig. 1b. These dissipating devices lead to stiffness K_D which together with stiffness of the concrete frame K_C provides the required design stiffness of the structural system. Cross sectional areas of the diagonals are computed by $A = k_D L / (E_D D \cos^2 \theta)$ where L is the length of the diagonal, E_D is the material modulus of elasticity, D the number of diagonals in each story, and θ is the angle between the diagonal and the horizontal line. Once the preliminary cross section is obtained, the design forces of the EDDs can be computed. The diagonal elements work under an axial load to both compression and in tension. Designing the connection assumes that the design force induced by seismic motion acts in the direction of the diagonal.

4.2 Values of some variables

In order to find the stiffness and strengths of frames and EDDs under study, values of 0.5 are proposed for α and β which lead to $K_d = 0.33$, $K_C = 0.67$, $R_C = 0.67$ and $R_d = 0.33$. Now the relationship of yield displacements between EDDs and the corresponding story is given by $\beta / \alpha = 1$, such that the relationship between lateral strength of the EDD and total strength of the structural system of each story is given by $r_k = R_d / R = \psi r_k / [1 + (\psi - 1)r_k] = 0.33$. The ductility relationship between the two systems, that is, EDD and conventional frame, is given by $\varepsilon = \mu_d / \mu_c = 1 / \psi$ which becomes 1 if it is assumed that $\mu_d = \mu_c = 3$. Superposition of both systems requires that $\mu = \delta_{\max} / \delta_y = \delta_{\max} k / R$.



5 Results

5.1 Initial cost functions

After designing conventional buildings and buildings with EDDs, a cost analysis is performed to know their total cost. The analysis takes into account only structural costs (beams, columns and EDDs), but not costs of nonstructural elements, finishing or indirect costs.

In a previous paper by García-Pérez et al [9], initial cost functions in terms of seismic design coefficients were presented, and four types of reinforced concrete buildings were analyzed. Each one of the types corresponds to a period of vibration. Here, we analyze buildings with five, ten, fourteen and twenty stories. For each building type, we designed different structures with different periods of vibration, thereby allowing for initial cost curves in terms of the period of vibration for five, ten, fourteen and twenty stories, as shown in fig. 2. General expressions for initial cost functions in terms of seismic design parameters are still underway.

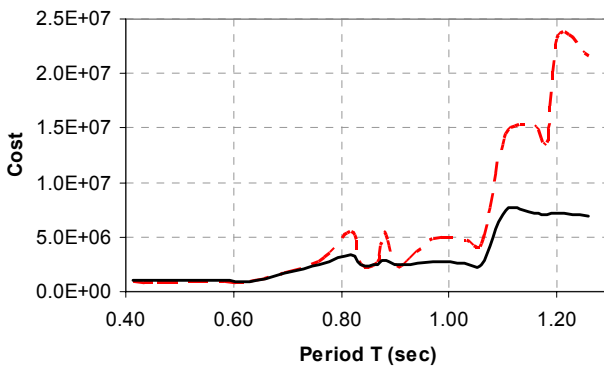


Figure 2: Initial cost in terms of period of vibration for conventional frames (dash line) and frames with EDDs (continuous line).

5.2 Vulnerability functions

Seismic vulnerability functions are computed here by following the methodology explained in García-Pérez et al [9], where the vulnerability function in terms of the economic consequences is expressed by the following equation as: $\delta_E(y) = \delta_E(y|S)(1 - p_F(y)) + \delta_{EF}p_F(y)$, where $\delta_E(y)$ is the expected value of the damage cost due to an earthquake of intensity y , $\delta_E(y|S)$ is the expected value of such cost, but it is conditioned to the survival of the system denoted by S , to the intensity y . δ_{EF} is the cost of collapse and $p_F(y)$ is the probability of occurrence of collapse. A detailed explanation of the computation of these functions is given in García-Pérez et al [9].



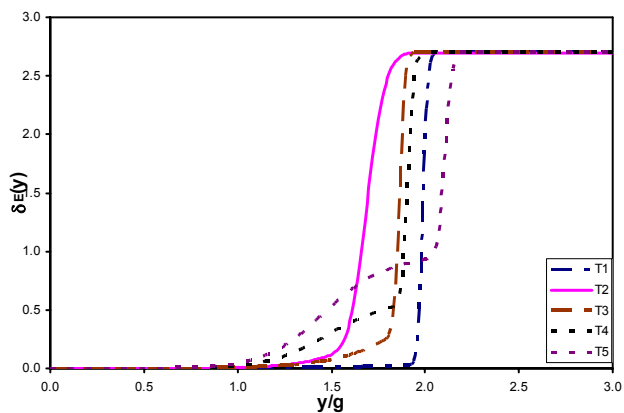


Figure 3: Vulnerability functions for conventional frames with 5 stories.

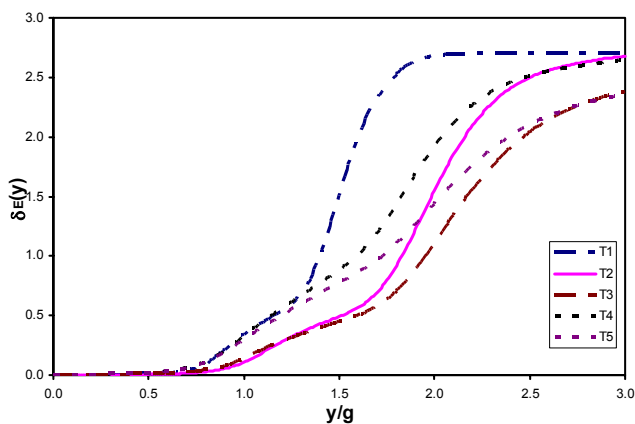


Figure 4: Vulnerability functions for conventional frames with 14 stories.

Figures 3 and 4 show vulnerability functions obtained for conventional buildings of five and fourteen stories, respectively. The results show that for both cases there is no clear trend of the variation of these functions in terms of the period of vibration for the same number of stories. In buildings with five stories the systems with a smaller period of vibration show that the cost of damage increases quickly within a small interval of intensities, reaching the total failure relatively faster once the damage has occurred. As the period of vibration increases, the damage cost with the intensity increases very slowly, although the damage is presented at smaller intensities. On the other hand, in those systems with large periods of vibration, damage in the dividing walls seems to exert certain influence at the beginning. In structures with fourteen stories, the system with a smaller period of vibration, together with the two systems of larger period,



presents a large increment of damage at lower intensities than those for intermediate systems. Here again, the dividing walls contribute significantly to the damage when it begins. The wide dispersion in vulnerability functions obtained for the systems with the same number of stories, but for different periods of vibration, must be studied in great detail, to determine if the dispersion is due to the fitting of the functions used to compute the vulnerability functions, since we have used very simple approximations in order to solve the problem. Those cases for structures with EDDs are currently under study.

6 Concluding remarks

Initial cost curves were obtained in terms of design parameters for both conventional structures and structures with energy dissipating devices. Systems with five, ten, fourteen and twenty stories were analyzed, and different periods of vibration were considered for each one of these structures. The expressions obtained were used in computing the vulnerability functions of the systems which show wide dispersions due perhaps to the use of approximations in the fitting process. General expressions in terms of design parameters and vulnerability functions for systems with energy dissipating devices are still left for future research.

References

- [1] Whitman, R.V., Biggs, J.M., Brennan, J., Cornel C.A., de Neufville, R. & Vanmarcke, E, Summary of methodology and pilot application. *Seismic decision Analysis Report No. 9*, MIT Dept of Civil Engineering, Cambridge, MA, Oct, 1973.
- [2] Grandori, G., Seismic zoning as a problem of optimization, *Proc Second International Conference on Structural Safety and Reliability*, Munich, pp 613-624, 1977.
- [3] Ferrito, J.M., Economics of seismic design for new buildings, *Journal of Structural Engineering*, ASCE **110(12)**, pp 2925-2937, Dec, 1984.
- [4] Rosenblueth, E., What should we do with structural reliabilities, *Reliability and Risk Analysis of Statistics and Probability in Soil and Structural Engineering*, Waterloo, Ontario, pp 24-34, May, 1987.
- [5] Vargas, E. & Jara, J. M., Influencia del coeficiente sísmico de diseño en el costo de edificios con marcos de concreto, *Memorias VIII Congreso Nacional de Ingeniería Sísmica, & VII Congreso Nacional de Ingeniería Estructural*, Acapulco, Gro, pp D30-39, Nov, 1989, (In Spanish)
- [6] García-Pérez, J., Seismic zoning for initial- and total-cost minimization, *Earthquake Engineering & Structural Dynamics*, 29, pp 847-865, 2000.
- [7] Reyes, C., The service limit state in the seismic design of buildings, PhD thesis, School of Engineering, National University of Mexico, 1999, (In Spanish)



- [8] Esteva, L., Díaz, O., García-Pérez, J., Sierra, G., & Ismael, E., Life-cycle optimization in the establishment of performance-acceptance parameters for seismic design, *Structural Safety*, **24** (2-4), pp. 187-204, 2002.
- [9] García-Pérez, J., Zenteno, M., & Díaz, O., Initial cost and seismic vulnerability functions for buildings with energy-dissipating devices, *First International Conference on Safety and Security Engineering*, Rome, Italy, WIT Press, **82**, pp. 161-170, Jun, 2005.
- [10] Federal District Building Code and its Complementary Technical Norms (RCDF), *Diario Oficial de la Federación*, México, DF, 2004, (in Spanish)
- [11] Campos, D., Optimization criteria for the design of buildings with hysteretic energy-dissipating devices, PhD thesis, National University of Mexico, 2005, (in Spanish)



Seismic behaviour: over-resistance effects on buildings

J. A. Avila

*Institute of Engineering, National University of Mexico, Mexico and
Faculty of Engineering, National University of Mexico, Mexico*

Abstract

The damage levels observed in the field, due to the 1985 earthquakes, is compared in a building in Mexico City with the analytical predicted behavior with, and without, the available over-resistance effects. The results were compared to those obtained from the conventional seismic analysis and to the observed damage behavior after the earthquake. Even the structures behaviour, located in soft soil in Mexico City, are qualified as adequate; there were some problems in some buildings, especially those between 7 to 17 levels. The structures behavior shows that these count with a certain over-resistance range that has been indirectly included and that was possibly the reason that a great number of buildings have not collapsed, even though they suffered severe damage. Elastic and inelastic time-history analyses are made. The soil-structure interaction and the P- Δ effects are included in the analysis. The analytical periods are compared to those experimentally obtained. A very good congruency between the analytically predicted behavior and the observed damage level after the earthquake is obtained. The direction and the stories with maximum damage match with the direction and stories with maximum deformations obtained from the analysis. The structural element resistances determined in a nominal way result were quite low compared to their real average values. It is noticed that the structure has a superior lateral resistance capacity compared to that given in the conventional design.

1 Introduction

Even the behavior of structures located in the soft zone in Mexico City, and subject to the 1985 earthquakes, are qualified as satisfactory; there were some



problems in some of them, especially in the 7 to 17 levels of buildings. The structural behavior shows that these structures fall within a certain over-resistance range that has been given and that it was possibly the reason that a great number of buildings have not collapsed, even though they showed severe damage. Considering the observed behavior in many buildings, the necessity of studying in detail the available over-resistance effects was merged in order to widely explain the seismic-resistance behavior participation of such structures. The inelastic response of a structure that suffered damage in the 1985 earthquake was analyzed in this work prior to the SCT-EW record of the 19th September 1985 earthquake. The results were compared to those obtained from the conventional seismic analysis and to the damage behavior observed after the earthquake.

2 Elastic response

2.1 Building description

The earthquake resistant system was based on frames in the longitudinal direction. The short direction head axis had four shear walls, and the internal axis had only frames (see fig. 1). The foundation was semi-compensated with a 6.425 meters deep rigid box, a foundation beam grid and friction piles of 22 meters in length. The building was constructed between 1970 and 1971. During the project, the structure was considered type A (important).

2.2 Damage description

There was only longitudinal direction damage between the ground level and level 6. The evidence of plastic hinges in the frame beam extremes in this direction was evident. Plastic hinges were observed in the base of the columns located in the ground level as well as diagonal fissures in some 3-4 and 5-6 stories columns.

2.3 Over-resistance effects

The over-resistance sources studied were: 1) slab steel (additional to the beam); 2) hardening effect because of the reinforcement steel strain (EPD); 3) average real stress in steel and concrete; 4) slab participation in the beam positive flexural moment resistance; 5) concrete core confinement. Table 1 shows the flexural moment resistances calculated values of a beam type, with and without over-resistances. In order to appreciate the differences between values, case 1 was taken as a base; fig. 2 shows the flexural moment-curvature curves for each one of the considered cases. The given confinement by the transversal reinforcement steel does not practically produce any section resistance increase; nevertheless, a confined section is capable of resisting a much bigger deformation than one without the confinement. Notice that the deformation capacity is independent of the steel stress-strain model, or if the slab participation is considered or not. The hardening zone consideration by reinforcement steel strain, was one of the most important; the results in a



resistance level show significant increase, according to case 1. This type of comparison is also made in columns with flexural moment-axial force interaction diagrams; the differences obtained presented a similar pattern found in beams.

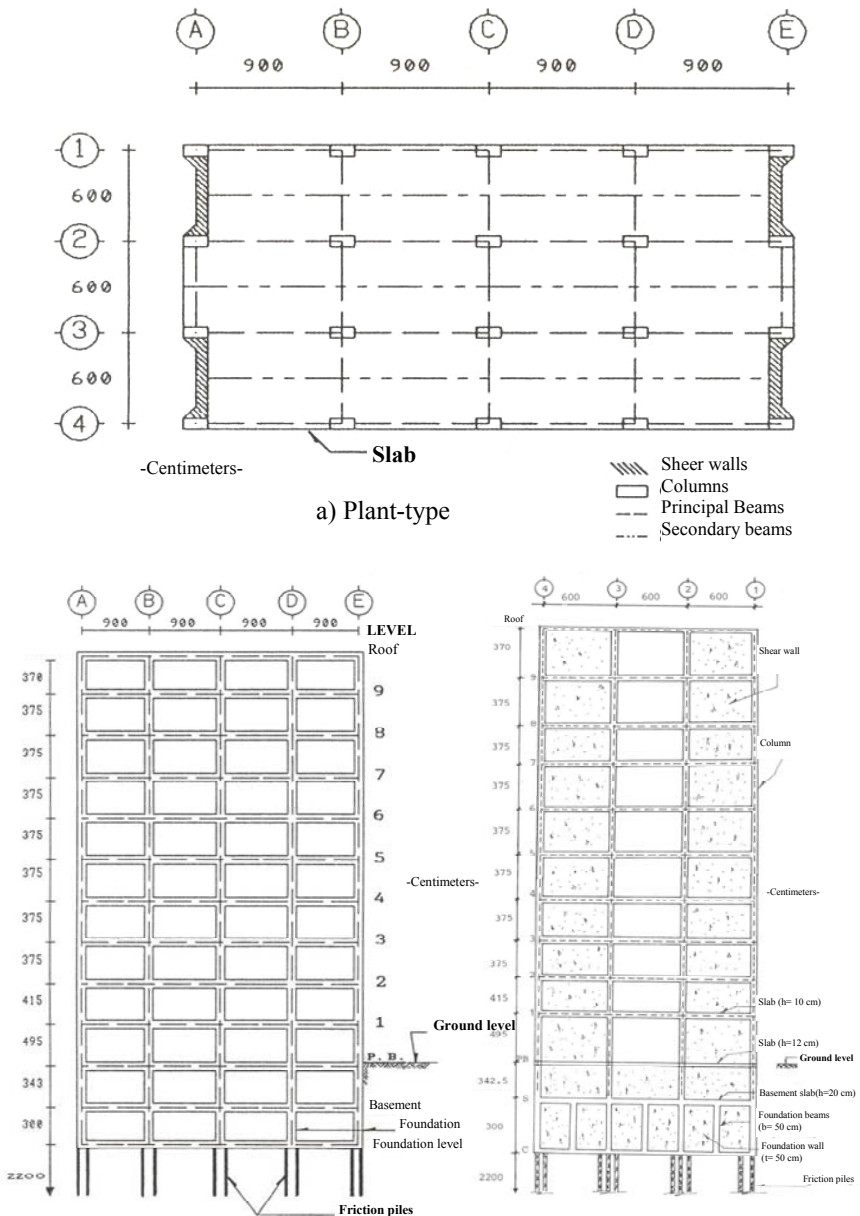


Figure 1: Structural plant-type and cuts (dimensions in meters).



Table 1: Flexural moment resistances calculated values of a beam type, with and without over-resistances.

	1	2	3	4
MR+ (t-m)	83.95	84.09	109.32	109.98
MR- (t-m)	146.40	147.51	152.83	156.18

	5	6	7	8
MR+ (t-m)	84.15	84.26	111.94	112.17
MR- (t-m)	177.09	178.75	174.09	178.96

	9	10	11	12
MR+ (t-m)	89.83	123.20	101.68	132.12
MR- (t-m)	177.09	178.96	209.51	213.15

CASE	CONSIDERATIONS
1	Rectangular beam, without slab reinforcement; S/C; EPP; E/N
2	Rectangular beam, without slab reinforcement; C/C; EPP; E/N
3	Rectangular beam, without slab reinforcement; S/C; EPD; E/N
4	Rectangular beam, without slab reinforcement; C/C; EPD; E/N
5	Rectangular beam, with slab reinforcement; S/C; EPP; E/N
6	Rectangular beam, with slab reinforcement; C/C; EPP; E/N
7	Rectangular beam, with slab reinforcement; S/C; EPD; E/N
8	Rectangular beam, with slab reinforcement; C/C; EPD; E/N
9	"T" beam, with slab reinforcement; C/C; EPP; E/N
10	"T" beam, with slab reinforcement; C/C; EPD; E/N
11	Rectangular beam, with slab reinforcement; C/C; EPP; E/P
12	Rectangular beam, with slab reinforcement; C/C; EPD; E/P

S/C - Without confinement

EPP - Perfectly elastic-plastic

E/N - Nominal stresses

C/C - With confinement

EPD - Elastic-plastic with hardening

E/P - Average real stresses



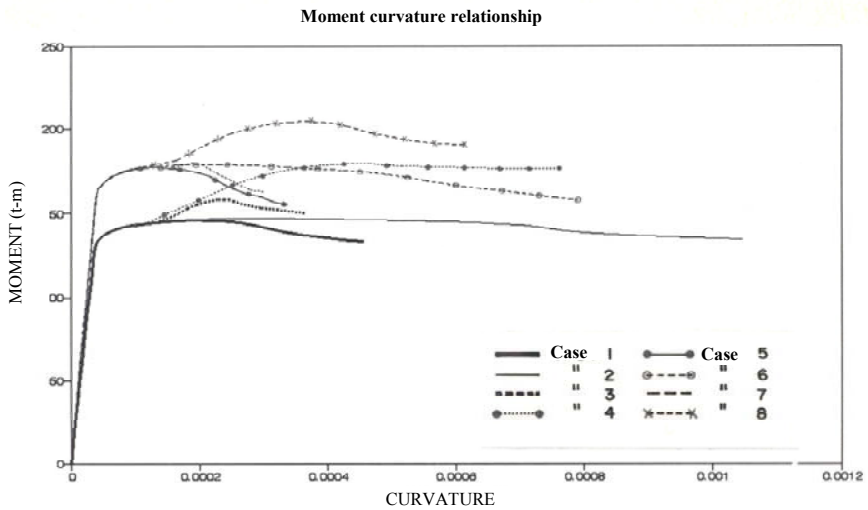


Figure 2: Moment curvature relationships of a beam type.

2.4 Vibration periods

The longitudinal direction vibration periods for the fixed base condition in ground level (PB) and in slab foundation level are practically the same, 1.67 and 1.69 seconds, respectively; when including the influence of the soil-structure interaction effects an increase of little more than the 10% was obtained, getting to 1.84 seconds (see table 2). The measured period (2.1 seconds) shows a great flexibility. The difference between can be attributed to the damage suffered in this direction for the lateral stiffness lost, regarding the maximum damage direction. The transversal direction vibration periods variation for the two fixed base types are practically nil, with 1.00 second for both conditions. Nevertheless, the period difference between the fixed base condition and the condition in which the soil-structure interaction effects were taken, gives significant results because of the increase of 30%. Comparing this last and the measured result, they are nearly the same which is congruent so that this direction does not present damage.

3 Inelastic responses

3.1 Studied cases

The selected cases characteristics, from a total of 21 inelastic step-by-step analyzed cases, were: A (without confinement, EPB model, rectangular beam, V3%, C1.5%); B (without confinement, Takeda model, rectangular beam, V3%, C1.5%); C (with confinement, Takeda model, rectangular beam, V3%, C1.5%); D (with confinement, EPB model, rectangular beam, V3%, C1.5%); E (with confinement, Takeda model, "T" beam, V3%, C1.5%); F (with confinement,



EPB model, “T” beam, V3%, C1.5%); G (with confinement, EPB model, rectangular beam, V3%, C1.5%, EP); H (with confinement, Takeda model, rectangular beam, V3%, C1.5%, EP). EPB: elastic-plastic bilinear hysteretic model, V3% and C1.5%: 3% and 1.5% slopes given to the program to take notice the deformation hardening effect in beams and columns, respectively, and EP: reinforcement steel and concrete average real stresses.

Table 2: Vibration periods of longitudinal and transversal directions.

Mode	Base condition		
	Fixed in ground level	Fixed in foundation level	Soil-structure interaction
1	1.67	1.69	1.84
2	0.55	0.56	0.63
3	0.33	0.33	0.42

Note: T1 (measured period) = 2.1 seconds.

a) Longitudinal direction

Mode	Base condition		
	Fixed in ground level	Fixed in foundation level	Soil-structure interaction
1	1.00	1.01	1.31
2	0.27	0.27	0.51
3	0.21	0.21	0.26

Note: T1 (measured period) = 1.3 seconds.

b) Transversal direction

3.2 Maximum horizontal displacements

Using the Takeda hysteretic model (B, C, E and H cases) the lateral displacements result are bigger. The inelastic response is greatly diminished according to the elastic one; in fig. 3, A case with roof lateral displacement histories in longitudinal direction are compared, with inelastic and elastic behavior, respectively. The inelastic displacements tended to reduce a little more of 50% in the maximum accelerations range. The displacement histories amplitudes and behavior pattern for the other cases gives result very similar to A case. Fig. 4 compares the roof lateral displacement histories of the longitudinal direction (important damage frames) and transversal (no-damage concrete walls and frames) of the building in order to show the seismic-resistance behavior differences in both directions. The response in the short direction is quite smaller, which indicates the great available lateral stiffness by the shear walls presence in the head axis. During the analysis in this direction there were no yields detected.



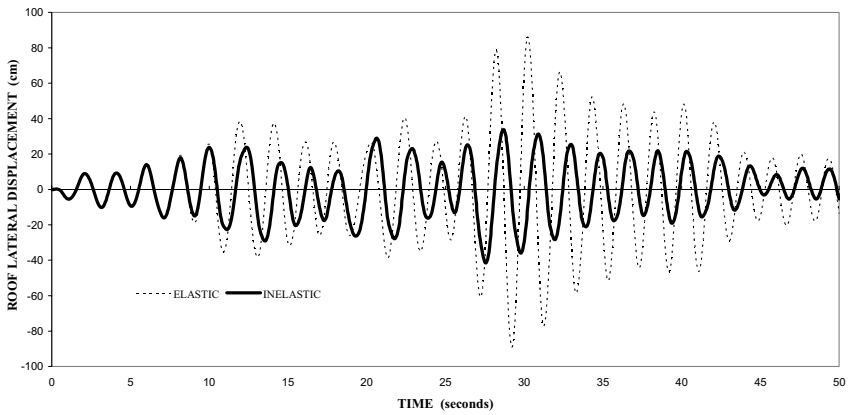


Figure 3: Roof lateral displacement histories in longitudinal direction, A case, inelastic and elastic behaviour.

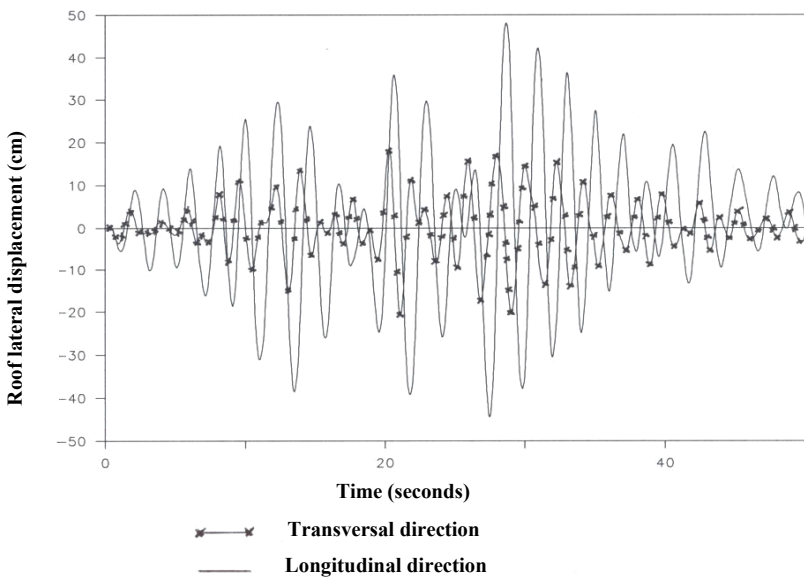


Figure 4: Roof lateral displacement histories of the longitudinal direction (important damage frames) and transversal (no-damage).



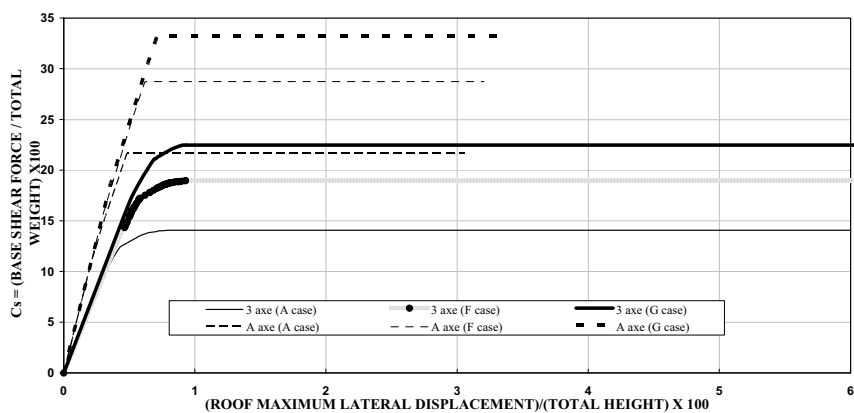


Figure 5: Base shear force–roof lateral displacement relations, non-linear static analysis, axes A-E (transversal direction), 3 axe (longitudinal direction) and, A, F and G cases.

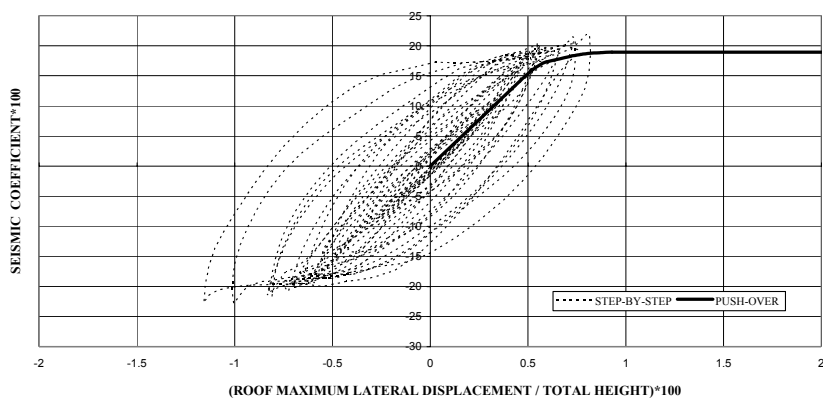


Figure 6: Base shear force–roof lateral displacement relations, G case, inelastic step-by-step and static (Pushover) analysis, longitudinal direction

3.3 Base shear force–roof lateral displacement relations

In this study non-linear static analysis were made taking the structure in both directions, up to its collapsed condition, for a determined failure mechanism (see fig. 5). The results correspond to 3 axis (longitudinal direction), and to the axis A-E (transversal direction), for comparative purposes. The seismic loads distribution type was the result of a spectral modal dynamic analysis previously



made. The employed resistances in the analysis were those obtained in A, F and G cases; the gravitational load effects were included in the responses. For the longitudinal direction, seismic coefficients of 0.14, 0.19 and 0.23 were obtained, for A, F and G cases respectively. In the step-by-step inelastic analysis, the results were 0.15, 0.22 and 0.25. Regarding the transversal direction results under lateral load monotonically increase, the over-resistance effects employed were clearly shown, from 0.22 for the nominal case, it was an increase of 0.29 for the case in which the confinement, the slab participation for positive bending moment and nominal stress were considered, and reached 0.3 for the last case in which the confinement was used, rectangular beams an average stresses. In fig. 6 the base shear force-roof lateral displacement relations for the G case are compared, obtained from the inelastic dynamic (step-by-step) and static (Pushover) analysis, longitudinal direction. In the horizontal axis we have the roof displacements divided between the building total height and in the vertical axis the seismic coefficients are presented, both responses in a percentage. In the inelastic static analysis the structure lateral resistance is lightly underestimated, because the second slope effect was not considered; nevertheless these results give a very good idea of such property. Including the G case over-resistances, the inelastic excursion cycle numbers diminishes in an important way and the seismic coefficient increases to 50%, according to the A case results. The differences between the two analysis types are due mainly to the hypothesis in which every one of the employed computer programs is supported.

3.4 Local ductility maximum demands in beams and columns

Fig. 7 shows the observed damage distribution, as well as the global distribution of the plastic hinges in A, E and H cases. The results for E case present bigger similitude to the physically observed case; A case is presented for being the case in which the conventional criteria for the resistance calculation is supported. H case resulted very similar to E case. For the three cases, A, E and H, the local ductility maximum demands " μ_L " were also calculated by level, for beams and columns. The developed maximum demands in beams were concentrated in the first level, and in the inferior extremes in columns of ground level; in columns the values that result are small and not very important. The structural element resistances determined in a nominal way resulted in their being quite lower than their average real values.

4 Conclusions

In general there was a good congruency between the calculated behavior and the observed damage level after the earthquake. The vibration periods showed that the structure presented a great flexibility in one of its directions. The direction and the stories with maximum damage match with the direction and stories with maximum deformations obtained from the analysis. It is noticed that the structures have a superior lateral resistance capacity regarding to those given in the conventional design; calculating the inelastic seismic responses with the



nominal resistances could get us to a greatly overestimated non-linear behavior value, global and locally. The mechanism that tends to be formed in each case, independently of the resistance type, matches with the design philosophy “weak beam-strong column”, the most part of plastic hinges are formed in the beam extremes.

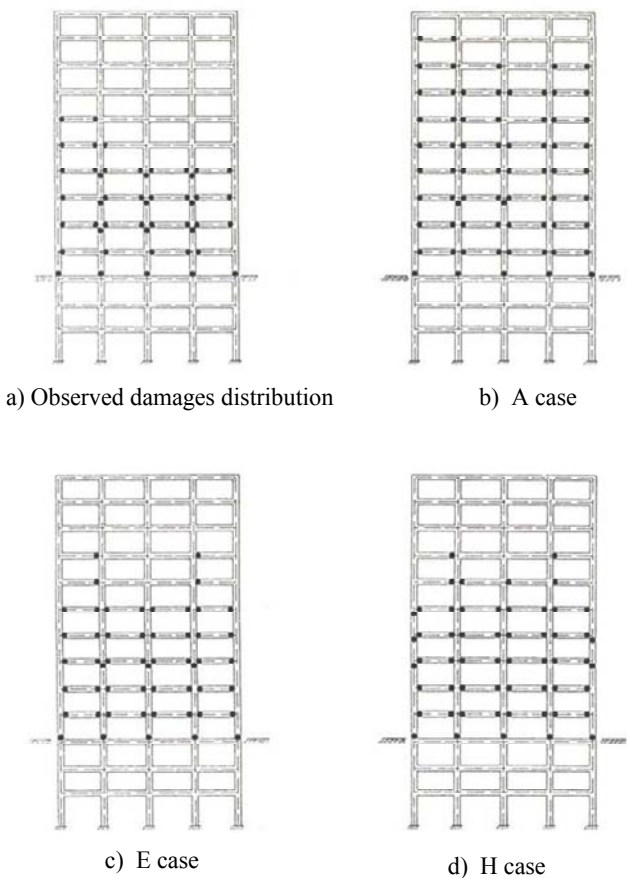


Figure 7: Observed damage distribution and global distribution of the plastic hinges in A, E and H cases.



Design of reinforced concrete buildings according to the new NEHRP provisions

O. A. Mohamed & P. Khamwan

*Department of Civil, Biomedical and Environmental Engineering,
University of Hartford, West Hartford, USA*

Abstract

This paper examines the seismic analysis and design requirements for earthquake resistant reinforced concrete buildings according to the recent NEHRP recommended provisions, also known as FEMA 450. To demonstrate the implementation of the NEHRP provisions, a case study reinforced concrete building is analyzed and designed. A number of key modeling and design considerations are examined such as: (1) the effects of upper and lower limits imposed by the provisions on design fundamental period; (2) the variation of drift along the height of the building in a structure that contains a dual lateral force resisting system in one direction and a moment resisting frame in another direction, compared to the limiting NEHRP value; (3) comparison of the torsional irregularity limit in the provisions to finite element computations for dual lateral resisting system as well as special moment resisting frame system; (4) the effects on structural response of the interaction of shear-wall and special moment resisting concrete frames.

Keywords: seismic design, NEHRP provisions, dual systems, torsional irregularity.

1 Introduction

The Building Seismic Safety Council (BSSC) of The National Institute of Building Science (NIBS) established the National Earthquake Hazard Reduction Program (NEHRP) more than 25 years ago with the objective of minimizing earthquake hazards and the damage and injury they might cause. The United States Federal Emergency Management Agency (FEMA) has been supporting BSSC through a number of contracts to publish and update documents, such as



FEMA 450 [1], on the best practices for design of buildings and other structures to mitigate the hazards of earthquakes. Although FEMA 450 is not a general building code, its provisions are incorporated in whole or in part by legally binding codes such as the International Building Code (IBC) [2] and recognized load estimation standards such as ASCE 7 [3] published by the American Society of Civil Engineers. The recommendations of the previous NEHRP provisions are discussed and analyzed elsewhere [4, 5]. The next NEHRP update is scheduled for publication in 2008.

This paper explores the analysis and design recommendations for reinforced concrete buildings on the recently published NEHRP provisions [1]. A case study reinforced concrete building is analyzed and designed using the Equivalent Lateral Force (ELF) method described in section 5.2 of the NEHRP recommended provisions which will be referred to in this paper as *the provisions*. The use of the ELF method is limited to structures with regular mass/stiffness/strength properties and when the lateral motions and torsional motion are not strongly coupled. Despite these restrictions, many structures are designed according the ELF method. Furthermore, to determine if a more accurate method, such as nonlinear time history analysis, is necessary, certain quantities will need to be determined and evaluated based on the ELF method.

2 Description of case study

The case study is a reinforced concrete building consisting of 13-stories above grade and one story below grade. The plan view of a typical floor is shown in fig. 1. The lateral force resisting system in the North-South (N-S) direction consists of four special moment resisting frames with Response Modification Factor, $R = 8$. Lateral force resistance in the East-West (E-W) direction is provided by a dual system consisting of special moment frames and the shear walls with $R = 8$. The response modification factor is intended to account for damping, overstrength, and ductility present in the system at large displacement. Analysis was done using ETABS software produced and marketed by Computers and Structures, Inc., USA. Site class is C, which represents very dense soil and soft rock with shear wave velocity between 360 m/s and 760 m/s. Seismic Use Group is 'I' representing structures other than essential facilities or those having substantial public hazard.

Spectral response acceleration parameter at short periods is $s_s = 1.65$ and at one second is $s_1 = 0.68$. The design spectral response acceleration parameter at short periods is calculated to be $s_{Ds} = 1.1$ and at one second is $s_{D1} = 0.589$. The ELF method uses eqn. (1) to calculate fundamental mode base shear force, V , for sustained weight, W .

$$V = CW \quad (1)$$

The structure was designed for the seismic forces described above in addition to typical, dead, live, and wind loads to satisfy the requirements of the provisions and ACI 318 [6].



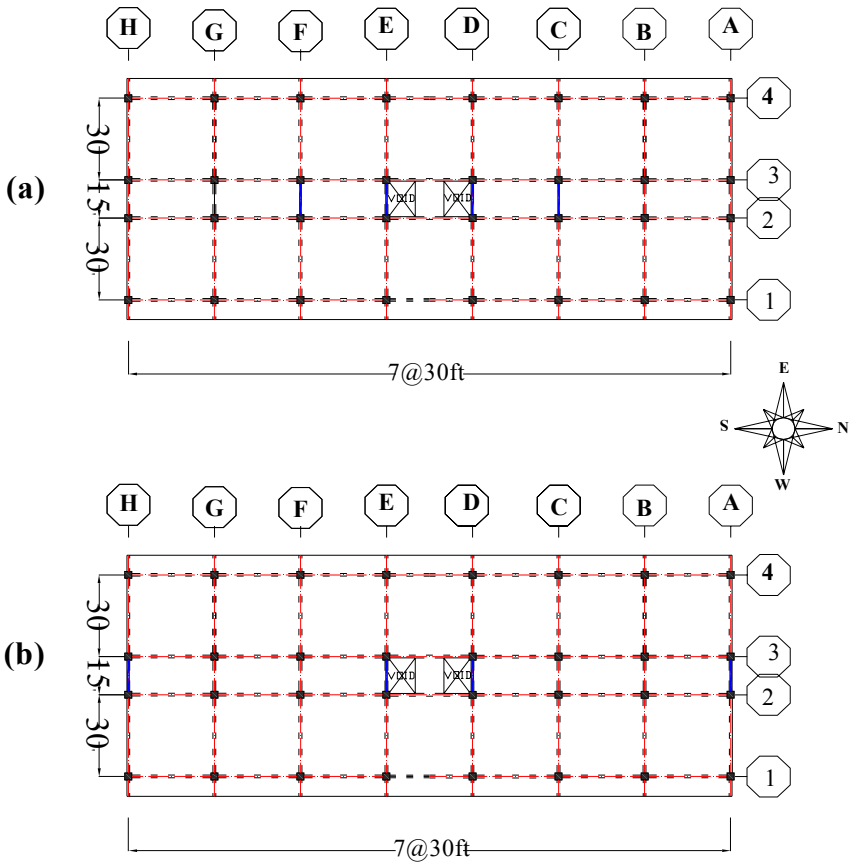


Figure 1: Case study, (a) plan view of the reinforced concrete building (b) alternative shear-walls locations to reduce torsional amplification.

3 Lower and upper limits on design fundamental period

The provisions permit the fundamental period to be approximated by eqn. (2),

$$T_a = C_r h_n^x \quad (2)$$

where,

C_r & x : coefficients that depend on lateral force resisting type, from Table 5.2-2 of the provisions.

h_n : building height in feet (meters).

Based on eqn. (2), the special moment resisting frames in N-S direction may be designed for a period $T_a = 1.51$ seconds and the dual system in the E-W direction may be designed for $T_a = 0.88$ seconds.



A finite element analysis shows that the fundamental period in the N-S is $T_a = 1.89$ seconds and the fundamental period in the E-W direction $T_a = 1.64$ seconds. Clearly, using *the provisions* approximate periods, which was derived from a lower-bound regression analysis of a number of buildings, produces a conservative design for the system in the N-S direction and even more conservative design in the E-W direction where the dual system is used.

Although the use of more accurate methods for determination of fundamental periods is recommended in *the provisions*, the value used in design is limited by eqn. (3).

$$T_{upper} = C_u C_{cr} h_n^x \quad (3)$$

The N-S special moment resisting frames upper limit on fundamental period is $T_{upper} = 2.11$ seconds, which is higher than the finite element calculated value of $T_a = 1.51$ seconds. The E-W dual system upper limit on fundamental period is $T_{upper} = 0.825$ seconds, which is lower than the finite calculated value of $T_a = 0.88$ seconds. Therefore, in this case study, *the provisions* impose a penalty on the dual system, which leads to a rather conservative design, while no such penalty is imposed on the special moment resisting frame. According to section 5.2.2 of *the provisions*, the designer is allowed to use analysis-based fundamental period that is no more than 40% to 70% of the approximate fundamental period estimated by eqn. (2). This depends on the magnitude of the design spectral response acceleration parameter at a period of one second, S_{D1} .

4 Drift resistance by special moment frames compared to dual systems

Drift calculated using elastic analysis based on the ELF method should be magnified by certain prescribed factors to account for the fact that during a design seismic event, a structure may deform beyond the elastic limit. The amplification factor for the dual system in the E-W direction is $C_d = 6.5$ and for the special moment frame in the N-S direction is $C_d = 5.5$, according to Table 4.3-1 of *the provisions*. All members contributing to lateral resistance in the E-W and N-S directions were assumed to be cracked and the upper limit on fundamental period described in section 3 of this paper was not considered as permitted by *the provisions*. The calculated and magnified drifts are shown in fig. 2, together with the maximum permissible drift. For this case study structure, which belongs to Seismic Use Group I, the maximum permissible drift at any level is 2% of the building height below that level. This structure meets drift limitations in both directions.

The magnified drifts shown in the N-S moment resisting frames tends to curve up near the top story while the magnified drifts in the E-W dual system tends to flatten toward the top story. The flattening of the drift curve in the E-W is caused by the interaction between the shear walls and the moment resisting frames. As a result of shear-wall frame interaction exhibited by the drift curve, the in-plane shears in the frames do not differ significantly from story to another.



This helps keep shear design uniform, which could potentially contribute to reducing the labour part of the construction cost.

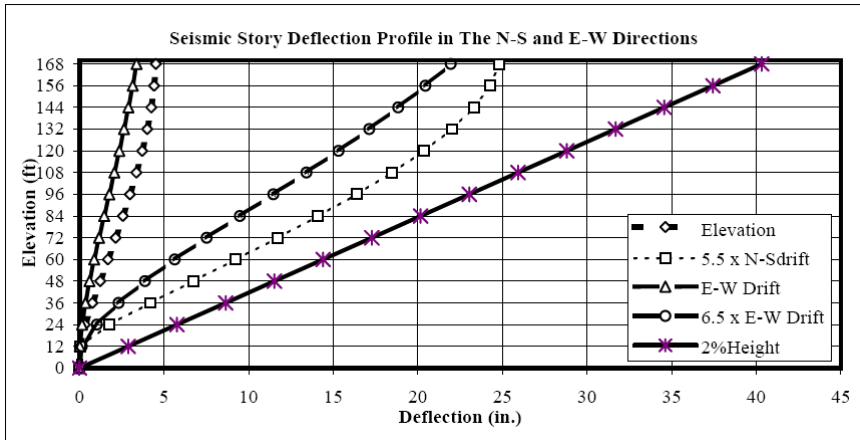


Figure 2: Calculated and magnified drifts in the E-W and N-S directions, compared to the maximum permissible drift.

5 Dynamic amplification due to torsional effects

A structure that is irregular in plan could experience amplification of lateral deformation due to torsion during a seismic event. Section 5.2.4.3 of *the provisions* requires that inherent torsional moment of the structure, M_t , and accidental torsion moment, M_{ta} , be magnified by the dynamic amplification factor is given by eqn. (4), when the lateral forces in members are calculated. This condition applies to structures that fall in Seismic Design Categories, C, D, E, and F.

$$A_x = \left(\frac{\delta_{\max}}{1.2\delta_{\text{avg}}} \right)^2 \quad (4)$$

$\delta_{\max}$ = the maximum displacement at Level x , and

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x .

The mass and lateral force resisting elements in the E-W and N-S directions are distributed such that $M_t = 0$, however, M_{ta} caused by lateral loads applied at 5% of the dimension of the structure from the centre of mass, must be considered per section 5.2.4.2 of *the provisions*. According to eqn. (4), dynamic



amplification is not necessary when $\frac{\delta_{\max}}{\delta_{\text{avg}}} \leq 1.2$. Analysis of the structure shown in fig. 1(a) produces the ratio $\delta_{\max} / \delta_{\text{avg}}$ shown on fig. 3. Clearly, dynamic amplification of accidental torsion is not necessary in the N-S direction. However, in the E-W direction, where the aspect ratio of the building in plan view is large, dynamic amplification of accidental torsion is necessary. This is the case even though the shear walls are stiffening the building in the E-W direction resulting in reduced deformations compared to the N-S direction as shown in fig. 2.

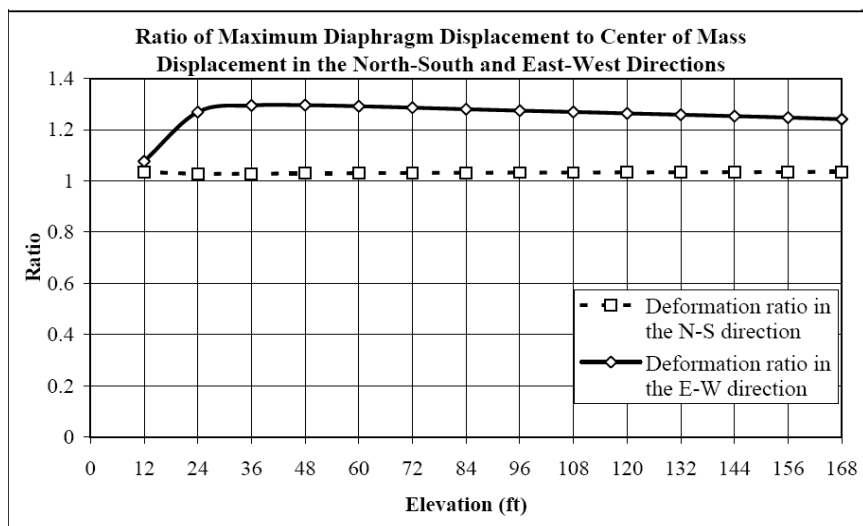


Figure 3: Ratio of maximum diaphragm displacement to centre of mass displacement in the E-W and N-S directions for the shear walls configuration shown in fig. 1(a).

As an alternative to amplification of accidental torsion in the E-W direction as suggested by fig. 3, the two outer shear walls are removed and placed at the extreme ends of the building parallel to the E-W direction, between grid lines 3 and 4, as shown in fig. 1(b). Analysis of the structure in fig. 1(b) shows that placing the outer shear walls at the extreme ends of the building increases Torsional resistance and reduces the diaphragm deformation ratio such $\frac{\delta_{\max}}{\delta_{\text{avg}}} < 1.2$, as shown in fig. 4. Therefore, dynamic amplification of accidental torsion in the E-W direction will not occur.



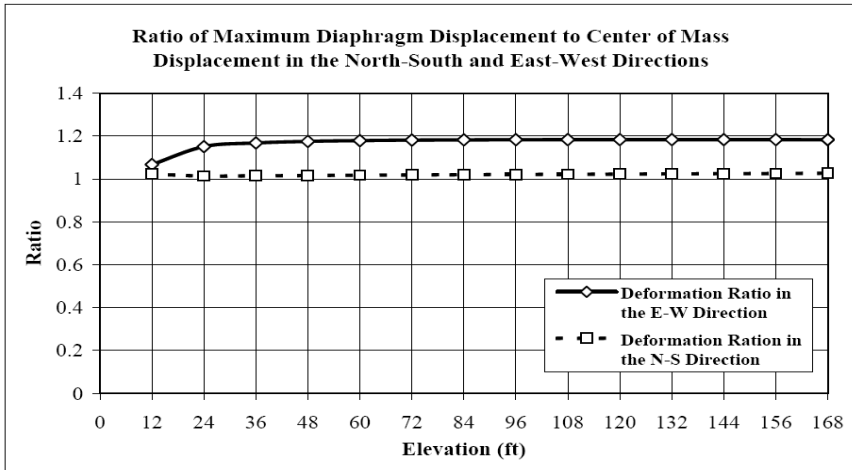


Figure 4: Ratio of maximum deformation to centre of mass deformation in the E-W and N-S directions is reduced below the dynamic amplification limit for the structure in fig. 1(b).

6 Interaction between shear-walls and moment resisting frames

The dual system of shear-walls and special moment resisting frames located in the E-W direction of the structure has positive effects in the structural response. As shown in fig. 5, the frames that did not include shear walls, namely, Frame A and Frame B, attracted less lateral shear compared to Frame C, which contained a shear-wall. The balance of the shear force in the plane of Frame C was attracted by the shear-wall. However, beams in the frames that do not contain shear-walls in the E-W direction produces in-plane shear forces that do not differ greatly from story to another [5]. This provides opportunity for economic design with reduced/variation in shear design of beams.

Section 5.2.5 of the provisions requires that the structure at each story level be capable of resisting overturning moments calculated according to eqn. (5).

$$M_x = \sum_{i=x}^n F_i (h_i - h_x) \quad (5)$$

where,

h_i & h_x : height from base to level i or x .

F_i : the portion of the seismic base shear, V , introduced at level i .

For the dual system in the E-W direction, the interaction of shear walls with moment frames reduces overturning moments in the frames due, in part, to the out-rigger effects of the E-W spanning beams on each side of the shear-wall. As shown in fig. 6, Frame C, located in the plane that contains a shear-wall experienced reduced overturning moments compared to Frames A and B which



lie in planes that do not contain shear-walls. However as demonstrated elsewhere [5], frames that do not contain shear-walls but located in load direction that contains shear walls, also gain the benefit of reduced overturning moments compared similar frames located on a load direction that contains only moment resisting frames without any shear walls.

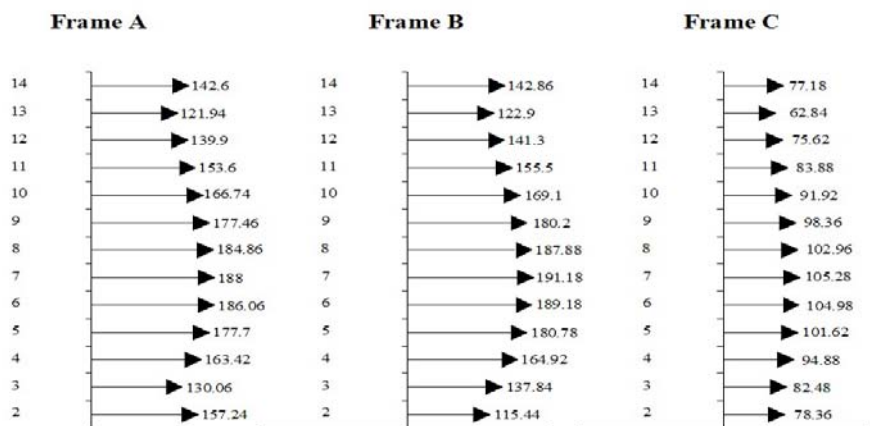


Figure 5: Frame shear forces in the E-W direction. Frames A and B do not contain shear-walls. Frame C contains shear-walls.

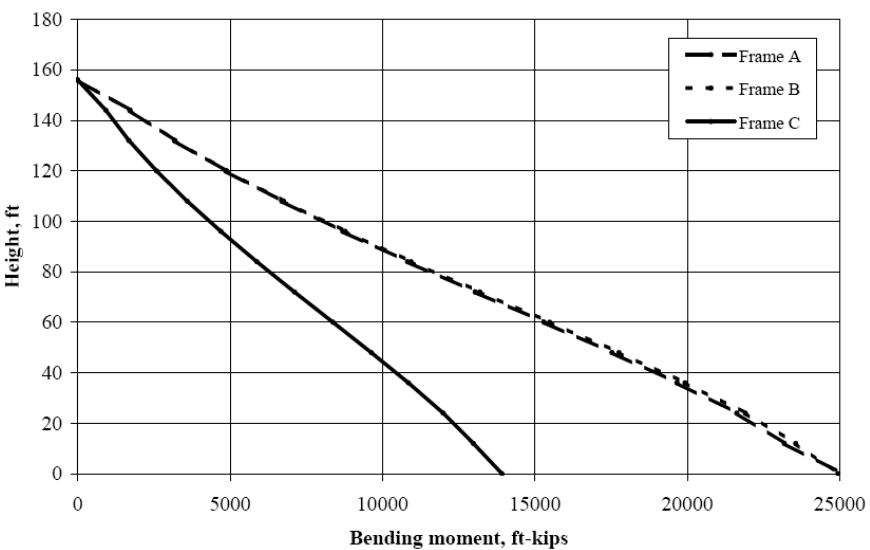


Figure 6: Overturning moments in the E-W direction. Frames A and B experienced higher overturning moments than Frame C.



7 Summary and conclusions

- Even structures that are regular both with respect to distribution of stiffness and mass may still experience excessive drift due to accidental torsion when the plan aspect ratio is large. *The provisions* require that dynamic effects of torsion to be considered in the calculation of loads distributed to structural elements.
- It is possible to reduce drift caused by excessive accidental torsion by placing the lateral force resisting elements judiciously away from the geometric centroid of the floors.
- Dual lateral force resisting systems consisting of shear walls and special moment frames produces lateral deflection profile that is relatively flat instead of the typical curves characteristic of cantilever shear walls or shear dominated curves of special moment frames. As a result, the in-plane shear forces of the frames that do not include shear walls do not differ significantly along the height of the structures. This can lead to relatively uniform shear design.
- Dual lateral force resisting system consisting of shear walls and special moment frames experience lower overturning moments at the special moment frames containing the shear walls, especially with long span girders because of outrigger effects.

References

- [1] United States Federal Emergency Management Agency. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and other Structures – FEMA 450*, Prepared by the National Institute of Building Science, Washington DC, USA, 2003.
- [2] International Code Council. *2006 International Building Code*, Washington DC, 2006.
- [3] American Society of Civil Engineers. *ASCE 7-05 - Minimum Design Loads for Buildings and other Structures*, Reston, VA.
- [4] Mohamed, O. A. Exploration of the FEMA368 Guidelines for the Seismic Design of Reinforced Concrete Buildings. *Proc. of ERES - fifth International Conference on Earthquake Resistant Engineering Structures*, Eds. C.A. Brebbia, D.E. Beskos, G.D. Manolis, and C.C. Spyarakos, WIT Press, Southampton, UK, pp. 765-774, 2005.
- [5] United States Federal Emergency Management Agency. *Guide Application of 2000 NEHRP Recommended Provisions, FEMA 368*, Multihazard Building Design Summer Institute, Emmitsburg, Maryland, 2003.
- [6] American Concrete Institute. *Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)*. Farmington Hills, MI, 2005.



This page intentionally left blank

Static and dynamic analytical and experimental analysis of 3D reinforced concrete panels

K. Numayr & R. Haddad

Civil Engineering Department,

Jordan University of Science and Technology, Irbid, Jordan

Abstract

A three-dimensional panel system, which was offered as a new method for construction in Jordan using relatively high strength modular panels for walls and ceilings, is investigated in this paper. The panel consists of two steel meshes on both sides of an expanded polystyrene core and connected together with a truss wire to provide a 3D system. The top face of the ceiling panel was pored with regular concrete mix, while the bottom face and both faces of the wall panels were cast by shotcreting (dry process). To investigate the structural performance of this system, an extensive experimental testing program for ceiling and wall panels subjected to static and dynamic loadings was conducted. The load-deflection curves were obtained for beam and shear wall elements and wall elements under transverse and axial loads, respectively. Static and dynamic analyses were conducted, and the performance of the proposed structural system was evaluated and compared with a typical three dimensional reinforced concrete frame system for buildings of the same floor areas and number of floors. Compressive strength capacity of a ceiling panel is determined for gravity loads, while flexural capacity is determined under the effect of wind and seismic loading. It was found that the strength and serviceability requirements could be easily satisfied for buildings constructed using the three-dimensional panel system. The 3D panel system is superior to that of conventional frame system in its dynamic performance, due to its high stiffness to mass ratio.

Keywords: three-dimensional, panel, static, dynamic, concrete, shotcrete, gravity, wind, seismic load.



1 Introduction

The structural systems adopted since the beginning of this century were based on either skeleton and/or shear wall systems, both proved to provide safety and integrity for the constructed facilities [1]. However, the former is more popular and usually constructed using one or two of the materials, steel, wood, and/or concrete. The use of any of these materials depends on its availability, seismic activity of the region where the structure is to be built, and the dominant weather conditions. This is why, wood and steel are mostly used in the construction industry in the United States, and Europe, whereas, reinforced concrete is extensively used in the Middle East and the rest of the third world countries.

Construction using the above materials and structural systems requires considerable time and is relatively expensive, especially for low-income people. To overcome this, new alternative systems and/or construction techniques were proposed to cut down both construction cost and time. Reinforced concrete panels [2-4], precast shear walls [5-6], and pre-stressed beams, and slabs [7], were used to reduce construction time, as well as construction cost especially in large projects that demands massive production of these elements. In addition, owing to better quality control during concrete casting these elements would be stronger and more durable than those cast on site. Nevertheless, the spreads of poverty, especially in the third world countries demand safe, serviceable, and low cost construction systems. This had researchers and engineers search for innovative ideas to deal with the problem. In Jordan, for example, low-income housing was established on using cheap local construction materials, blocks and reinforced concrete, to construct small units. Yet, these units failed in providing safety and serviceability. Therefore, the government demolished high percentage of these units although were in service for less than twenty years.

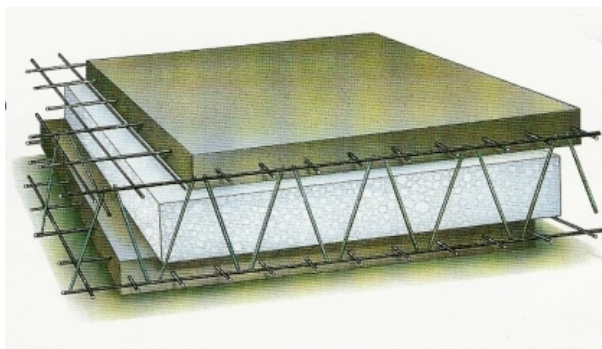


Figure 1: Typical panel showing steel meshes in concrete layers, steel truss, and polystyrene core.

Recently, a three-dimensional system was offered as a new method for construction using relatively high strength modular panels for walls and ceilings. These consist of two steel meshes on both sides of an expanded polystyrene core



and connected together with a truss wire to provide a three-dimensional system. Details of typical ceiling and wall panels are shown in Fig. 1. To investigate the structural performance of this system, an extensive experimental testing program for ceiling and wall panels was conducted. This included, two points loading of ceiling and wall beams, one and two points loading of shear wall elements, and axially distributed loading of shear wall elements. The load-deflection curves were obtained, through measuring the mid-span deflection (for beam panels) and mid-span and lateral deflections for shear wall panel elements loaded axially. Both static and dynamic analyses were conducted, and the dynamic performance of the proposed structural system was evaluated and was compared with that of a typical skeleton system, both used to construct similar floor numbers and areas.

2 Static and dynamic analysis

Static and dynamic analysis were performed for wall and the ceiling panels in a shear building with floor area of 256 m^2 as shown in Fig. 2. The dead load on each floor is estimated based on the assumption that the following materials exist over the ceiling panel: tiles of 25 mm, cement mortar of 25 mm, and sand layer of 50 mm. Hence, the overall dead load was found to be 4.3 kN/m^2 . A live load of 1.96 kN/m^2 for private residents is used. The own weight of a wall panel is found to be 1.96 kN/m^2 . The top part of the ceiling panel was cast using 60 mm relatively low slump concrete, which was consolidated by rodding, while the bottom part of ceiling panel of 45 mm thickness and both layers of the wall panels of about 40 mm thickness were cast by shotcreting (dry process). Concrete strengths (f'_c) for regular and shotcrete concretes were, conservatively, assumed to be about 25 MPa, and 17 MPa, respectively. The steel yielding strength (f_y) was assumed of about 414 MPa. The moduli of elasticity for steel and concrete are $E_s = 200000 \text{ MPa}$, and $E_c = 4700\sqrt{f'_c} \text{ MPa}$, respectively.

The structural capacities of the ceiling and the wall panels are determined according to the design methods specified in the American Concrete Institute "Building Code Requirements for Reinforced Concrete ACI 318," [8]. Flexural and shear capacity, in addition to deflection, of ceiling panel is calculated under the effect of gravity loads for a typical $4\text{ m} \times 4\text{ m}$ slab in a residential building. For a wall panel, the compressive capacity is determined for gravity loads, while flexural capacity is determined under the effect of a uniformly distributed wind load of 1 kN/m^2 . Both the ceiling and wall panels are assumed, conservatively, simply supported. The calculated external moment, shear, and/or thrust loads for ceiling and/or wall panels are listed in Table 1. Also the moment, shear, and/or thrust structural capacity of a one meter-width cross section for the ceiling panels are listed in Table 1. The deflection of a ceiling panel is calculated and compared to experimental and the allowable values.

The dynamic analysis was carried out on the two systems shown in Fig. 2, namely, the panel-supported floor Fig. 2(a), and skeleton-supported floor Fig. 2(b). The dynamic analysis for one, three and seven story shear buildings was carried out and the results are listed in Table 2. In the dynamic analysis of



the 3-D system, three cases were considered: a) lateral walls only; b) longitudinal walls only; and c) both longitudinal and lateral walls. The analysis of the seven floor system was carried out considering single panels or double panels for the walls of the lower three floors. In analyzing the skeleton-supported floor system, the columns were assumed to be 300x300 mm, reinforced with 8 ϕ 14mm rebars with 2 ϕ 10 mm ties spaced at 200 mm. The overall calculated dead load is 900 kg/m² plus a live load of 200 kg/m² were considered in the calculation of the lump masses.

3 Experimental evaluation of the 3D system

3.1 Materials

Pozzolanic Portland cement, manufactured by Jordan Cement Factory, limestone crushed coarse and fine aggregate, and natural sand were used to prepare concrete mixture used to cast the top layer of the slab panel. The bottom layer of slab panels and both layers of wall panels were shotcreted using a concrete mixture prepared using the same above cement with a mixture of crushed limestone fine aggregate and natural sand (Suweileh sand) at mass ratio of 2:1. The gradations of coarse limestone aggregate and the mixture of fine limestone and the natural sand were proportioned so as to meet the ASTM requirements [9]. The (specific gravity (SSD) and absorption) of coarse limestone aggregate, fine limestone aggregate, and natural sand were determined according to the ASTM test methods C 127 and 128 [9]. These were found to be (2.61, and 1%), (2.54, 4%), (2.6, 0.8%), respectively. The compacted and loose unit weights of coarse limestone aggregate, determined according to ASTM test method C 29 [9], were 1540, 1320 kg/m³, respectively.

3.2 Concrete mix design

The American Concrete Institute (ACI 211.1) method of mix design for normal weight concrete was used to proportion the concrete mixture, [8]. The cylinder specified strength was 30 MPa at 28-days, which is equivalent to cube strength of 38 MPa. The margin of strength taken in the design of the concrete mixes was chosen so that the proportion of strengths less than the specified strength is less than 5% and that the standard deviation used to determine the margin is assumed to be 4.0 MPa. Therefore a target strength of 44.6 MPa was used in the mix proportioning. The shotcrete mixture was proportioned at cement to fine particles mass ratio of 1 to 5, respectively.

3.3 Panels casting and concrete strength evaluation

The slab panels were placed horizontally. Then, the top layer was cast using regular concrete, during which the concrete was consolidated by an electric vibrator. After that, the bottom layer was cast by shotcreting (dry process) using the mix specified for this procedure. As for the wall panels, both layers were shotcreted after the panel was positioned vertically. The panels were covered by



burlap and were kept moist until time of testing. Six standard cube specimens (150 mm) were cast from the concrete mix to obtain the compressive strength at 28 days of curing. Cores were obtained from a special panel (0.6 x 1 x 0.1 m), fabricated and cast from the same shotcrete mix and cured for 28 days, were tested for compressive strength. Other cores were obtained from the top layer of slab panel after being cured for 28 days. The compressive strength for concrete cubes averaged 43.4 MPa, whereas equivalent cube strength of obtained cores averaged 19 MPa.

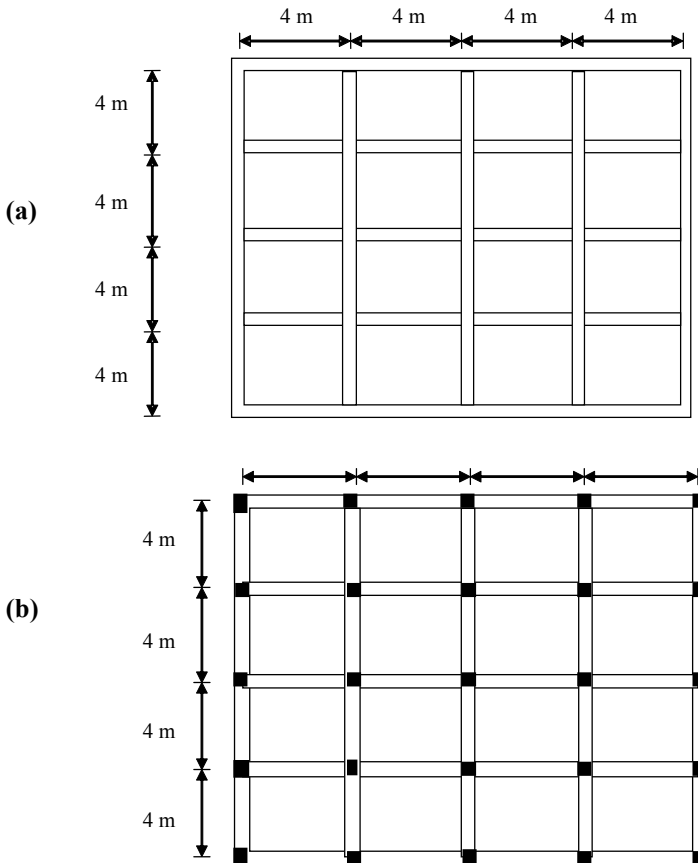


Figure 2: Floor plans for (a) 3-D panel system, (b) skeleton system.

4 Summary and discussion of results

A summary of the results of the static and dynamic analysis in addition to the experimental ones is presented herein to give a better insight of the structural



performance of the 3-D system compared to the traditional skeleton system. Table 1 presents ultimate and reduced nominal capacity values of ceiling panel subjected to transverse gravity loads. It also presents ultimate and reduced nominal axial load capacity of a wall panel. The ceiling panel satisfies shear requirement ($V_u < \phi V_n$) as part of one or two way slabs. It doesn't satisfy moment ($M_u < \phi M_n$) as part of one way slab, and therefore the section needs modification, for instance increasing the area of steel or the depth of the section. It satisfies serviceability requirement for deflection. This suggests that the ceiling panel cannot be used with unsupported length of 4.0 m as a part of one-way slab. The maximum unsupported length for the panel to be used in one way slab so that the section moment capacity is not exceeded is found to be about 2.7 m. The thrust, and moment capacity of the wall panels are much higher than external thrust, and moment, respectively. This indicates that the proposed wall panels as high as 3m and that the two-way ceiling panels spanned as long as 4 m can be used with a high margin of safety.

Table 1: Static analysis of ceiling and wall panels of the 3DS.

Panel Type	Type	Calculated			Allowable		
		Mu (kN.m)	ULC (kN)	δ (m)	ϕM_n (kN.m)	ULC (kN)	δ (m)
CP	OWS	18.76	18.76	0.0123	9.32	54.1	0.017
	TWS	5.40	9.38	0.0062	9.32	54.1	0.017
WP	NA	1.46	111.4	NA	8.32	711	NA

ULC: Shear for ceiling and axial force for wall panels; δ : deflection; OWS: one-way slab; TWS: two-way slab; NA: not applicable.

For a one-story shear building, the lumped mass of one typical panel, one continues wall and ceiling through the building, which includes half the mass of nine wall panels of (4 m) long each and (3m) high in addition to the dead load and live load masses of four ceiling panels, (4mx4m) each excluding openings of windows and doors in interior and exterior walls is about 50 tons compared to that of the skeleton system of about 70 tons. The natural frequency is about 6.15 radians/sec when considering lateral walls only.

From response spectra of elastic system for 1940 El Centro earthquake, assuming 5% damping, the maximum relative displacement is about 0.1 m. for one meter width of wall panel, the ultimate shear force (V_u) and the ultimate moment (M_u) are 18.4 kN and 27.6 kN-m respectively. The reduced nominal shear capacity of the section ($\phi V_n=46.7$ kN) is satisfactory, while, the reduced nominal moment capacity ($\phi M_n=9.35$ kN-m) is not satisfactory. Therefore, lateral walls alone are not capable of resisting lateral loads produced by low intensity earthquake or by slow wind speeds. Table 2 presents values for the natural frequency, horizontal displacement, shear and moment for different number of floors using suggested 3-DS and using skeleton system. It can be noticed that the natural frequencies for the 3-DS are much larger than these for



Table 2: Calculated natural frequency, horizontal displacement, shear, and moment for different numbers for floors constructed using suggested panel system (Case C), (3DS), and Skelton system, (SK).

Floor	No	ω (radians/sec)		U (m)		V (kN)		M (kN-m)	
		3DS	SK	3DS	SK	3DS	SK	3DS	SK
1	1	353.37	24.91	0.91E-5	0.01254	14.41	109.48	21.61	164.22
3	1	150.92	10.636	0.66E-4	0.0406	104.4	354.24	156.6	531.71
	2	417.24	29.406	0.12E-3	0.0721	81.78	277.14	122.67	415.71
	3	590.94	41.649	0.15E-3	0.0882	41.65	144.54	62.48	216.81
7 Single	1	69.275	4.882	0.26E-3	0.0307	413.7	268.08	620.55	402.12
	2	204.52	14.414	0.52E-3	0.0584	394.54	244.4	591.81	366.61
	3	330.41	23.261	0.74E-3	0.08202	357.53	216.84	536.28	325.26
	4	440.04	31.013	0.94E-3	0.10167	304.60	194.42	456.89	291.64
	5	529.71	37.333	0.00108	0.1175	238.04	171.8	357.09	257.71
	6	595.57	41.975	0.00119	0.1289	160.60	134.08	240.86	201.11
	7	635.61	44.797	0.00124	0.1346	75.3	72.29	112.90	108.43
7 Double*	1	85.88		0.12E-3		380.35		570.4	
	2	232.62		0.24E-3		366.21		549.8	
	3	364.1		0.35E-3		340.01		509.9	
	4	498.14		0.54E-3		301.23		452.3	
	5	597.17		0.69E-3		242.85		364.5	
	6	643.73		0.80E-3		167.31		251.0	
	7	837.68		0.85E-3		79.12		119.0	

*: The first three floors are constructed using double panels; ω : natural frequency; U: deflection; V: external Shear; M: external moment.

horizontal displacement response to El-Centro earthquake of the 3-DS is much less than that of the skeleton system. For instance the seventh floor horizontal displacement of the 3-D double panel system (the lower three floors are constructed with double panel walls) is 0.85 mm (1.24 mm for the single panel walls for the lower three floors), while it is 134.6 mm for the seventh floor of the skeleton system. The shear and bending moment capacity of the walls in the 3-DS one and the three story buildings are satisfactory using single panels. However, the single panel walls are not satisfactory for the seven-story building and the walls of the lower three stories should be constructed of double panels so that the building is capable of resisting moderate-to-high intensity earthquake. In this case and for 3m effective length of wall, the ultimate shear and the ultimate moment at the base of the first story of the seven story building are: $V_u = 1.1 \times 1.3 \times 380.35 = 544$ kN, $M_u = 1.1 \times 1.3 \times 570.4 = 817$ kN-m, where 1.1, 1.3 are the axial and lateral load factors respectively. It should be noted here that the values in



table 2 resulted from applied service load. The reduced shear and moment capacities are ($\phi V_n=580$ kN) and ($\phi M_n=980$ kN-m) respectively. The shear and moment values for one column of the skeleton system are: $V_u=383$ kN, $M_u=575$ kN-m, $\phi V_n=200$ kN and $\phi M_n=60$ kN-m which clearly indicates that the columns provided are not adequate to resist lateral loads produced by low intensity earthquake or by slow wind. It is also clear in this table that the 3DS is superior to the skeleton system in resisting lateral loads.

Table 3: Ceiling and wall panel specimens subjected to static loading.

Panel	Name	No.	Dimensions (cm)				P_{max} (kN)
			a	l	b	h	
Ceiling	C1B28	1	35	105	36	20	17
	C2B28	2	34	102	35	20	23
	C3B28	3	34	102	36	20	22
	C4B28	4	36	108	56	20	33
	C5B28	5	36	108	56	20	31
	C5B28	6	35	105	57	20	32
Wall	W1B7	1	30	90	40	18	12
	W2B7	2	30	90	40	18	13
	W3B7	3	30	87	40	18	13
	W4B28	4	30	87	40	18	17.5
	W5B28	5	30	87	40	18	18
	W6B28	6	29	87	40	18	17
	W7F28	7	29	87	18	40	135
	W8F28	8	29	87	18	40	132
	W9F28	9	29	87	18	40	123
	W1S	1	30	60	18	18	32.5
	W2S	2	30	60	18	18	21
	W3S	3	30	60	18	18	22.5
			e	b	t	h	
	W1V	1	0	40	17	96	233
	W2V	2	0	38	17	98	290
	W3V	3	0	40	17	98	310
	W1VE	1	0.275	120	18	280	450

C: Ceiling; W: Wall; B: Beam Element; F: Two points loading on shear wall element; S: One Point Loading on shear wall element; V: Axial distributed loading on shear wall element; VE: Axial loading on shear wall element with eccentricity; 7: seven days of curing; 28: twenty eight days of curing.

Table 3 presents experimental flexural results for ceiling and wall panels in addition to experimental axial compression results for wall panels (Fig. 3). The load-deflection diagrams, obtained for different wall and ceiling panels, showed typical elasto-plastic behavior, which was demonstrated in linear behavior up

until the yielding point of reinforcing steel before became nonlinear up to failure load. The maximum transverse load ($P_{\max}$) for beam specimens was for ceiling panels C4B28, C5B28, and C6B28 at about 32 kN with corresponding shear and moment of 17.14 kN and 6.17 kN-m, respectively. The nominal shear and moment capacities are 43 kN and 5.8 kN-m, respectively. It is clear that the moment at failure is slightly higher than the nominal moment capacity of the beam, while the shear at failure is much less than the shear capacity of the beam. Therefore the failure is due to bending, since the moment capacity is exceeded. The maximum experimental axial compression load for each of wall panel specimen W2V and W3V is about 300 kN, while the nominal axial load capacity of the wall is ($P_n=380$ kN). The failure at a lower value can be attributed to that the concrete layers are thin or having large slenderness ratio at one hand and the ability to behave as a composite section on the other hand.

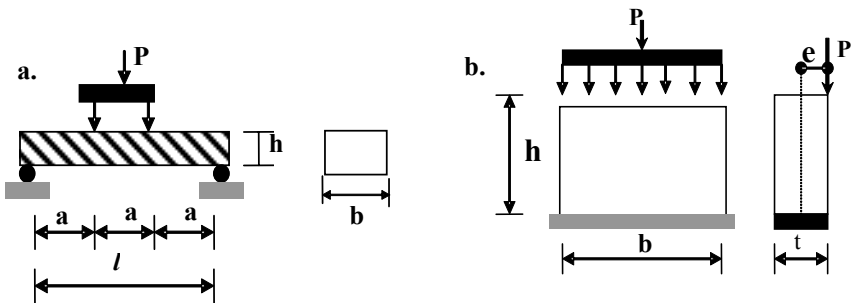


Figure 3: Experimental set up for (a) simply supported beam, (b) wall panel.

References

- [1] Nilson, A. H. Design of concrete structures, 12th edition, McGraw-Hill, Singapore, 1997.
- [2] Benayoune, A., Samad A. A. & Trikha, D.N., Abang, A. A., Ashrafov, A. A., Structural behavior of eccentrically loaded precast panels. *Construction and Building Materials*, **20** (9), pp. 713-724, 2006.
- [3] Benayoune, A., Samad, A. A., Abang, A. A. & Trikha, D. N., Response of pre-cast reinforced composite sandwich panels to axial loading. *Construction and Building Materials*, **21**(3), pp. 677-685, 2007.
- [4] Lan, S., Lok, T.-S. & Heng, L., Composite structural panels subjected to explosive loading. *Construction and Building Materials*, **19** (7), pp. 387-395, 2005.
- [5] Mo, Y.L. & Chan J., Behavior of reinforced-concrete-framed shear walls. *Nuclear Engineering and Design*, **55** (1), pp. 55-68, 1996.
- [6] Paulay T. & Priesley M. J., Seismic design of reinforced concrete and masonry Buildings, Wiley, New York, 1992.



- [7] Nilson, A. H., Design of prestressed concrete, 2nd edition, John Wiley & Sons Inc., Canada, 1987.
- [8] ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318M-05)," American Concrete Institute, Farmington Hills, Mich., USA, 2005.
- [9] ASTM Book of Standards, "Construction: Concrete and Aggregates (V. 04-05)," American Society for Testing Materials, Ann Arbor, MI, USA, 2005.



Designing aspects of bridges placed in active seismic areas

V. Herak Marović, P. Marović & Ž. Nikolić

*Faculty of Civil Engineering and Architecture,
University of Split, Croatia*

Abstract

The national territory of the Republic of Croatia is in a very active seismic area, so earthquake influence on bridge structure is often relevant for the choice of span number, the type of bridge superstructure, piers disposition and their height and stiffness, the connection between piers and superstructure or piers and foundations, dimensioning of elements and reinforcement, detail solutions, material consumption, etc. Mechanical resistance and overall bridge stability must be provided by appropriate design aspects taking care of maximal function, economics and aesthetic performances. The results of the bridge seismic analysis according to the new Eurocode 8/2 code are greater seismic forces and higher seismic capacity of the structure compared to the results of the previous codes. Seismic isolation with elastomeric bearings, placed between bridge superstructure and bridge substructure, is used as a common way to reduce the seismic action to the structure and to prevent structural damage. The response of the seismically isolated bridges is in many cases more complicated than the response of the conventionally designed structures because some parameters which are usually neglected in the analysis of the traditionally designed structures should be taken into account. This paper presents comparative analysis of the results obtained by two different methods proposed in Eurocode 8/2, i.e. the fundamental mode method and the response spectrum method for several seismic isolated viaducts at the Adriatic highway.

Keywords: viaduct, seismic area, isolated bridge, elastomeric bearings, fundamental mode method, response spectrum method.



1 Introduction

Structural conception of bridges is probably more strictly related to function, aesthetics and economics than in any other type of structures. Therefore, bridges give the impression of being simple structures whose seismic response could be easily predicted. Accordingly, seismic design of bridges in Croatia have received relatively little attention in the past, maybe because we have not been exposed to a single very strong event for a long time, or because of our inert behaviour.

Seismic calculation of bridge structures in active seismic areas is a significant part of the overall calculations with the aim of proving the mechanical resistance and stability. Aseismic bridge design is of special importance because its serviceability during and after the earthquake depends on it. The territory of the Republic of Croatia is in a very active seismic area (Figure 1) so an earthquake influence on bridge structure is often relevant for the choice of bridge type structure, computation model, element dimensions, material consumption, detail solutions and for the overall bridge mechanical resistance and stability.



Figure 1: Geological chart of the Republic of Croatia [1].

For the last half of the century we used very simplified, out-of-date and unharmonized regulations for the seismic calculation of bridge structures like “Rules on Temporary Technical Regulations for Construction of Structures in Seismic Areas” [2] dated 1964 and “Rules on Technical Standards for Design and Calculation of Engineering Structures in Seismic Areas” [3] dated 1990.



According to the mentioned regulations a greater reduction of inertial forces due to ground motion was permitted related to the modern European standards. Furthermore, verification of ductility was not requested although the ductility is a property which is important for dissipation of seismic energy. This approach raises a risk on bearing capacity and on serviceability of the bridge structure. Structural Eurocodes are based on the modern approach to the calculation of structures incorporating the idea of unification of the conditions for design and construction in European countries. The procedure of the seismic calculation of bridge structures according to Eurocode 8/2 is the result of modern theoretic and experimental analyses and construction experiences. The results of the bridge seismic analysis according to Eurocode 8/2 are greater seismic forces and higher seismic capacity of the structure compared to the results of the previous codes.

2 Basic principles of the bridge seismic design according to Eurocode 8/2

The calculation philosophy of the seismic resistant bridges according to the Eurocodes is based on the demand that, during the period of bridge exploitation after the occurrence of earthquake of the predicted intensity, the bridge must not collapse (ultimate limit state) and that the damage (serviceability limit state) must not influence the traffic. Eurocode 8/2 [4] gives recommendations for the seismic calculation of bridges with a description of basic principles and rules which follow the basic demands of the seismic calculations presented in Eurocode 8/1 [4]. These rules are destined for construction girder bridges supported by abutments and vertical or nearly vertical piers, arc and frame bridges, and are not recommended for suspension bridges, highly curved bridges, bridges with significant longitudinal grade and skew bridges. Eurocode 8/2 also incorporates some basic rules and principles for constructing special bridges and seismic protection of the bridges by the use of isolation devices for the purpose of reducing the seismic response.

In designing the seismic resistant structures according to the European standards aimed to assure integrity and serviceability of the bridge structure during the earthquake with foreseen intensity, special attention should be focussed on aseismic shaping of bridges. Namely, seismic conditions, especially in the areas of higher seismic intensity, are often the decisive factor for choosing the type of structure, the load-bearing system, the connections between superstructure and substructure, dimensioning of elements and reinforcement, material consumption, detailing, etc.

In seismic active areas the bridge superstructure should be designed as a continuous deck, i.e. as a statically highly indeterminate system. That means that the superstructure should have as few expansion joints as possible. As superstructure is leaned on substructure the stiffness of abutments and piers influence the seismic forces redistribution. The dispositions of the bridges with equal pier heights is more favourable because of a more even redistribution of the seismic forces on the supporting elements, i.e. the equalization of pier dimensions and the quantity of built in reinforcement and equable distribution of



stresses in the subsoil. Namely, the short very stiff piers as well as very high flexible piers should be avoided or expelled from the seismic forces acceptance system using flexible bearings. The first should be expelled due to the ability of accepting a greater part of the total seismic force, and the second due to the very high deformability. The ductile behaviour of bridge structure is ensured by the equalization of pier height and by making it possible to have a greater number of supporting elements to take part in the longitudinal and transverse bridge direction seismic forces acceptance with simultaneous opening of the plastic hinges in the majority of piers. The plastic hinges in piers (which are foreseen in the bottom parts) should be ensured according to the foreseen pier deformation by adequate reinforcement taking the damage into consideration which must not affect the traffic on the bridge. The eventuality of damage occurrence should be foreseen in easily accessible places due to the easy detection and repair. The opening of the plastic hinges in the bridge superstructure is not allowed. The plastic hinges will not open in the piers flexibly connected to the bridge superstructure and in the piers with the smaller stiffness compared with the other bridge piers. The bridge foundations should stay undamaged upon seismic actions.

The behaviour of the bridge during an earthquake can be designed by the adequate disposition of the elastomeric bearings upon which the bridge superstructure is leaned on abutments and piers. The flexibility of the elastomeric bearings (increasing its height) causes the prolongation of the fundamental period of the bridge and the reduction of the seismic force. At the same time, displacements of the structure are increased which causes a need for placing bigger and more expensive expansion joints or increases the number of bridge dilatations. To reduce the displacements of the structure it is possible to direct the dissipation of the seismic energy to the abutments and piers with seismic dampers. Furthermore, for leaning the superstructure on the substructure over the movable bearings it is necessary to assure the satisfactory width of the superstructure overlapping in order to prevent the falling of the bridge superstructure during extreme movements. In that case, the structure should be additionally assured by designing seismic boundary stone on the piers, i.e. by appropriate design and reinforcement of the breast abutment wall. The combination of all the aforementioned points would be the most effective in high seismic areas.

3 Design of seismic isolated bridges

3.1 Basic principle of seismic isolation

Seismic isolation is used as a common way to reduce the seismic action to the structure and to prevent the structural damage. Two systems can be used: isolators and dampers. Isolators are flexible devices which reduce the stiffness of the structure and the period of the structure becomes longer. Dampers reduce seismic load according to the principle of energy dissipation. Figure 2 shows the basic principle of seismic isolation [5].



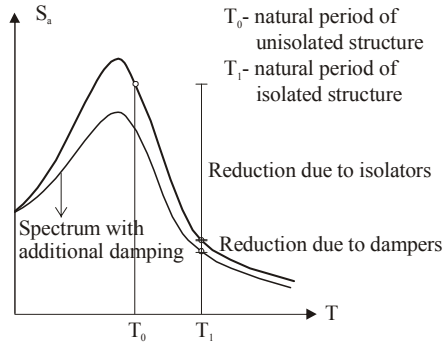


Figure 2: Basic principle of seismic isolation [5].

Elastomeric bearings are frequently used as isolators to lengthen a natural period of bridges, especially viaducts. They are situated between the superstructure and columns. They have a great bearing capacity and stiffness in the vertical direction and less shear stiffness in the horizontal direction which implies the reduction of the total structural stiffness in the longitudinal and transversal direction, as well as reduction of the seismic load. An earthquake causes large horizontal displacements and deformation of the bearings. Therefore, the choice of the bearings has significant influence on the obtained results.

3.2 Methods of analysis of seismically isolated viaducts

Several methods can be used for the analysis of seismically isolated viaducts. The type of analysis can be linear or non-linear, while the dynamic model is single-degree of freedom or multi-degree of freedom. Eurocode 8/2 proposes the following methods for analysis of bridges: fundamental mode method, response spectrum method, alternative linear methods (power spectrum analysis, time series analysis) and non-linear time domain analysis. Some examples of the engineering modelling of seismically isolated viaducts with the discussion of the influencing parameters can be found in Ref. [6].

The corresponding dynamic equation in the analysis of seismic isolated viaducts includes mass, damping and stiffness matrix, time, acceleration, velocity, displacement and load vectors. The change of the damping matrix, the stiffness matrix and the load vector over time depends on the applied accelerogram. The change of stiffness matrix in isolated viaducts depends, not only on accelerogram, but also on the changing of elastomeric bearings stiffness. This change depends on the force in elastomeric bearings. The damping matrix in isolation systems also additionally changes due to the velocity in the bearings. The use of non-linear models in seismic analysis of isolated bridges is necessary to obtain relevant results especially for complex bridges with large spans, the stiffness changes, dilatations, etc.

In spite of that, European codes have a tendency toward simplification of the analysis procedure. A linear method of analysis is more favourably received by



designers in relation to a non-linear dynamic analysis. The fundamental mode method and response spectrum method are preferred in engineering practice.

The fundamental mode method gives equivalent static seismic forces which are derived from the inertia forces corresponding to the fundamental natural period of the structure in the direction under consideration. The method includes simplifications regarding the shape of the first mode and the estimation of the fundamental period. The method can be applied in all cases in which the dynamic behaviour of the structure can be sufficiently approximated by a single dynamic degree of freedom model.

The response spectrum method can provide an acceptable approximation if the appropriate approximation of the elastomeric bearings is applied. However, the typical behaviour of elastomeric bearings is elastoplastic [5] so it is difficult to model their characteristics by a linear model. In addition, the modulus of elasticity is different in the vertical and horizontal directions, which should be considered in numerical modelling. The elasticity modulus can be expressed according to the literature [5, 7, 8].

The stiffness of bearings is a function of the shear modulus G . According to Eurocode 8/2 the shear modulus for normal laminated bearings is $G=1.2 \text{ N/mm}^2$ for $\varepsilon_s \leq 1.2$ and $G=1.6 \text{ N/mm}^2$ for $\varepsilon_s = 2.0$ where ε_s is the shear strain due to the total seismic displacement. The choice of shear modulus is not simple because whole seismic computation of the structure is performed for presumed magnitude. The verification of the shear modulus is performed during dimensioning of the bearings when the shear strains are computed. The difference in horizontal displacements of the structures for $G=1.2 \text{ N/mm}^2$ or $G=1.6 \text{ N/mm}^2$ can be 20%.

In the application of the response spectrum method on isolated bridges it is necessary to apply several approximations in modelling of elastomeric bearings which can influence the results.

4 Numerical examples

Comparative analysis of the results obtained by the fundamental mode method and the response spectrum method for several viaducts at the Adriatic highway [9–12] will be presented.

Viaducts consist of superstructure supported by abutments and piers. The reduction of the seismic action is performed with elastomeric bearings between superstructure and substructure. The behaviour factor of the structure is $q=1.0$ and subsoil class is A. The elasticity modulus of the concrete is 31500 MPa . The cross sections of all piers are equal. The cross sectional area is $A=2.64 \text{ m}^2$ and the moments of inertia are $I_x=2.8872 \text{ m}^4$ and $I_y=1.4512 \text{ m}^4$. The weight of the structure consists of the deck weight, the weights of piers and 20% of variable load at the viaduct. The mass of piers is $6.73 \text{ kNs}^2/\text{m}$. The total effective mass is less than or close to $1/5$ of the mass of the deck. The theoretical eccentricity between the centre of stiffness of the supporting elements and the centre of mass of the deck does not exceed 5% of the length of the deck. The analyzed viaducts



are on the limit-line between the application of the fundamental mode method and the response spectrum method according to their characteristics.

Longitudinal sections of the viaducts with design acceleration, characteristics of elastomeric bearings and the heights of piers are shown in Figure 3.

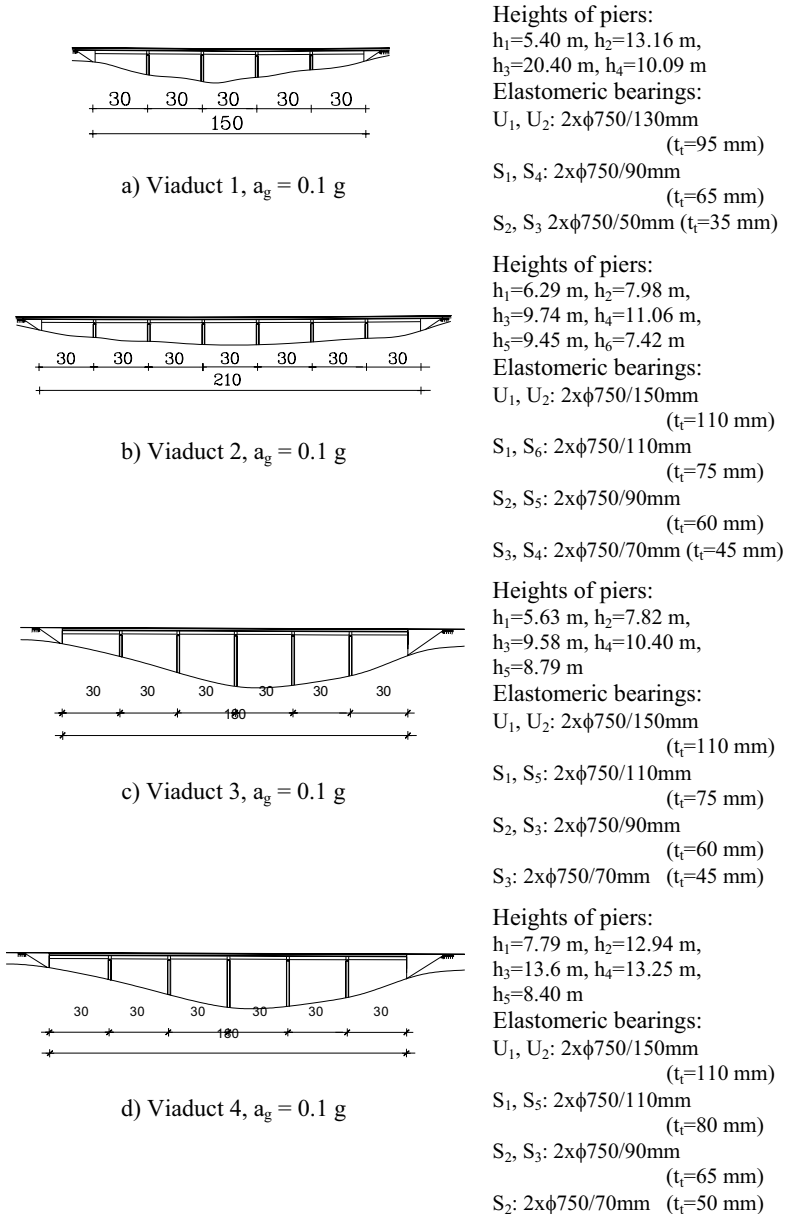
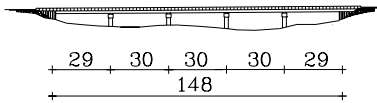
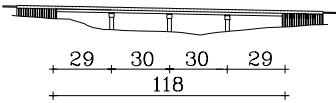


Figure 3: Main characteristics of the analyzed viaducts.

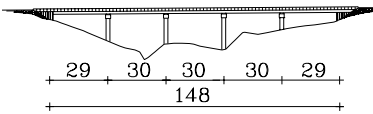




e) Viaduct 5, $a_g = 0.2 \text{ g}$



f) Viaduct 6, $a_g = 0.29 \text{ g}$



g) Viaduct 7, $a_g = 0.29 \text{ g}$

Heights of piers:

$$h_1=8.31 \text{ m}, h_2=10.96 \text{ m},$$

$$h_3=10.07 \text{ m}, h_4=12.69 \text{ m}$$

Elastomeric bearings:

$$U_1, U_2: 5 \times \phi 500/159 \text{ mm}$$

$$(t_i=115 \text{ mm})$$

$$S_1, S_4: 10 \times \phi 500/144 \text{ mm}$$

$$(t_i=104 \text{ mm})$$

$$S_2, S_3: 10 \times \phi 500/99 \text{ mm}$$

$$(t_i=71 \text{ mm})$$

Heights of piers:

$$h_1=13.28 \text{ m}, h_2=16.77 \text{ m},$$

$$h_3=14.48 \text{ m}$$

Elastomeric bearings:

$$U_1, U_2: 5 \times \phi 500/129 \text{ mm}$$

$$(t_i=88 \text{ mm})$$

$$S_1, S_3: 10 \times \phi 500/114 \text{ mm}$$

$$(t_i=77 \text{ mm})$$

$$S_2: 10 \times \phi 500/84 \text{ mm} (t_i=60 \text{ mm})$$

Heights of piers:

$$h_1=19.11 \text{ m}, h_2=21.23 \text{ m},$$

$$h_3=20.85 \text{ m}, h_4=10.44 \text{ m}$$

Elastomeric bearings:

$$U_1, U_2: 5 \times \phi 500/159 \text{ mm}$$

$$(t_i=110 \text{ mm})$$

$$S_1, S_4: 10 \times \phi 500/114 \text{ mm}$$

$$(t_i=77 \text{ mm})$$

$$S_2, S_3: 10 \times \phi 500/99 \text{ mm}$$

$$(t_i=71 \text{ mm})$$

Figure 3: (continued).

Table 1 shows the results in the longitudinal direction obtained by the fundamental mode method with the rigid deck model approach and response spectrum method. The fundamental mode is firstly computed for a rigid structure without elastomeric bearings, T_s . After that, the period based on the stiffness of elastomeric bearings T_{EB} is computed and finally the fundamental period of isolated viaducts is obtained with the expression $T_{FP} = \sqrt{T_s^2 + T_{EB}^2}$. For the computed period an ordinate of the design spectrum, $R(T) = a_g S \eta \beta_0 (T_C/T)^{k_1}$, and the equivalent seismic force, $F_{FP}(T) = R(T) \cdot M$, are calculated. The calculation of the stiffness of structure is based on the stiffness of piers $K_s = \sum K_i = 3EI \sum L / h_i^3$ where E is the modulus of elasticity, I is the moment of inertia, L is the span of the viaduct and h_i is the height of each pier. The horizontal stiffness of elastomeric bearings is given by the expression $k_h = GA / t_i$, where G is the shear modulus, A is the area of the elastomers and $t_i = \sum t_i$ is the total thickness of the elastomer layers.



The response spectrum method gives the first period T_{RS} and the total seismic force F_{RS} .

Analysis of the viaducts by the response spectrum method shows that the first mode activates a considerable part of the total mass of the viaduct. The first mode activates more than 90% of the total mass in all viaducts, while the first transversal mode activates more than 85% of the mass. The influence of the other transversal modes on the total seismic transversal force, as well as horizontal forces in elastomeric bearings and piers is negligible. The difference of the total seismic horizontal force in the longitudinal direction obtained by the response spectrum method and fundamental mode method for analysed viaducts is less than 11%. Similar results are obtained in the transversal direction. If we take into consideration that the simulation of the elastoplastic behaviour of elastomeric bearings in the linear response spectrum method is not possible, the obtained differences in practical engineering can be tolerated.

Table 1: Comparative analysis by fundamental mode method (FP) and response spectrum method (RS).

Viaduct	T_S (s)	T_{EB} (s)	T_{FP} (s)	F_{FP} (kN)	T_{RS} (s)	F_{RS} (kN)
1	0.380	1.170	1.226	3080	1.330	2722
2	0.394	1.370	1.426	6497	1.340	7132
3	0.368	1.340	1.390	5625	1.350	6064
4	0.553	1.464	1.565	5342	1.430	6076
5	0.548	1.113	1.241	9560	1.140	8638
6	0.909	0.929	1.300	9011	1.127	10320
7	1.010	1.078	1.477	13055	1.437	13710

5 Conclusion

As the national territory of the Republic of Croatia is in a very active seismic area, earthquake influence on bridge structure is often relevant for the choice of span number, the type of bridge superstructure, piers disposition and their height and stiffness, the connection between piers and superstructure or piers and foundations, dimensioning of elements and reinforcement, detail solutions, material consumption, etc.

So, in this paper we present comparative analysis of the results obtained by two different methods proposed in Eurocode 8/2, i.e. the fundamental mode method and the response spectrum method for several seismic isolated viaducts at the Adriatic highway.

The performed analyses show: (i) obtained seismic forces by these two methods are almost the same; the difference is within 10%; (ii) the fundamental mode method gives the results of sufficient accuracy although the viaducts are at the limit which Eurocode 8/2 recommend for use of the fundamental mode method.



Acknowledgements

The partial financial support, provided by the Ministry of Science, Education and Sports of the Republic of Croatia under the projects *Numerical and Experimental Models of Engineering Structures*, Grant No. 0083061, *Numerical and Experimental Investigations of Engineering Structures Behaviour*, Grant No. 083-0831541-1547 and *Non-linear Dynamic Analysis of Three-dimensional Reinforced Concrete Structures*, Grant No. 083-0831541-1532, is gratefully acknowledged.

References

- [1] *The Great Atlas of Croatia*, Mozaik knjiga, Zagreb, p. 399, 2002, (in Croatian)
- [2] Rules on Temporary Technical Regulations for Construction of Structures in Seismic Areas. Official Bulletin 39/64, 1964. (in Croatian)
- [3] Rules on Technical Standards for Design and Calculation of Engineering Structures in Seismic Areas. 1990. (in Croatian)
- [4] Eurocode 8. Design Provisions for Earthquake Resistance of Structures. European Committee for Standardization, ENV 1998-1 & ENV 1998-2, Brussels, 1994.
- [5] Naeim, F. & Kelly, J.M., *Design of Seismic Isolated Structures*, John Wiley & Sons, 2002.
- [6] Isaković, T. & Fischinger, M., Engineering modelling of seismically isolated viaducts. *Engineering Modelling*, 15(1-4), pp. 93-98, 2002.
- [7] Nikolić, Ž. & Herak Marović, V., Aspects of Seismic Bridge Design, *Proc. of the Int. Conf. on Bridges*, ed. J. Radić, Structural Engineering Conferences and Croatia Society of Structural Engineers: Zagreb, pp. 471-478, 2006.
- [8] Šimunić, Ž., Radić, J., Mekjavić, I. & Pavlović, B., Girder bridge durability analysis based on dynamic and static indicators. *Građevinar*, 53(2), pp. 61-81, 2001. (in Croatian)
- [9] Radnić, J., Herak-Marović, V., Nikolić, Ž., et al., Some structures on the highway Zagreb-Split-Dubrovnik from Zadar to Bisko, *Proc. of the Congress of HSGI*, ed. V. Simović, HSGI: Zagreb, pp. 517-531, 2004. (in Croatian)
- [10] Main Designs of Viaducts at the Adriatic Highway, Section Zadar 2-Benkovac, IGH PC Split, Split, 2001. (in Croatian)
- [11] Main Designs of Viaducts at the Adriatic Highway, Section Vrpolje-Prgomet, IGH PC Split, Split, 2002. (in Croatian)
- [12] Main Designs of Viaducts at the Adriatic Highway, Section Dugopolje-Bisko, IGH PC Split, Split, 2004. (in Croatian)



Behaviour of coupling beams having vertical slits at the ends

S. B. Yuksel

Department of Civil Engineering, Selcuk University, Turkey

Abstract

Architectural considerations and functional use result in door openings on the shear walls of tunnel form buildings, which cause coupled shear walls to be connected by short, deep and thin coupling beams. These coupling beams are subjected to higher shear forces and their thickness becomes generally less than 250mm for the tunnel form buildings, and much less than their counterparts in conventional reinforced concrete structures. It is simply not possible to design practically constructible coupling beams in the tunnel form buildings. In a coupled shear wall system, shear forces acting on the coupling beams can be reduced simply by the application of vertical separation joints (slits) at the ends of the coupling beams. As a design alternative, the use of slit connections at the ends of the coupling beams to be able to decrease the shear stiffness and shear forces was analytically investigated. Shear stiffness terms of common slit connected coupling beams (SCCBs) were derived by using plane stress finite elements. To be specific, extensive parametric study with respect to the geometry of a SCCB was carried out. Coupling beam heights, coupling beam lengths, slit heights and slit lengths were varied in an extensive parametric study to demonstrate their influences on the shear stiffness terms.

Keywords: coupling beams, coupled shear walls, finite element analysis, non-prismatic members, tunnel form buildings.

1 Introduction

Tunnel form (shear wall dominant) building system is an industrialized construction technique in which structural walls and slabs of the building are cast in one operation by using steel forms having accurate dimensions and plain surfaces [1]. In tunnel form construction, in situ concrete is poured into two half-



tunnel forms to form shear walls and floor slabs simultaneously [2]. When this process is repeated, usually in a 24hr cycle per floor, the residential units can be created with great rapidity. In general, all the floor plans become the same due to the same steel tunnel forms being utilized in all of the stories. A typical tunnel form system and its site applications are demonstrated in fig. 1.

Shear walls act as the primary gravity and lateral load carrying members, and may contain openings for functional use in tunnel form buildings. The sizes of the openings are determined by the functional use and the architectural restrictions on the shear walls; the dimensions of the coupling beams are defined in that way. The geometric limits result in deeper coupling beams in relation to their clear span above the door openings, and the thickness of these coupling beams are usually less than 250mm for the tunnel form buildings as can be seen in fig. 2. The dimensional constraints and high shear forces acting on these beams cause their design to become almost impossible according to the code specified reinforcement configurations.



Figure 1: Typical tunnel form systems at construction stage.



Figure 2: Typical deep coupling beams above the door opening and diagonal reinforcement without confining ties in a tunnel form building.



Coupling beams in tunnel form buildings are susceptible to high shear forces due to dimensional constraints. Although using the code specified diagonal layout with confining ties [3, 4] for the coupling beams of tunnel form buildings seems to be a solution, this detailing is generally avoided in practice (see fig. 2) due to constructional difficulties. Serious problems with construction and difficulties in manufacturing can occur during the application of diagonal layout with confining ties when the thickness of the wall is less than 250mm. Until now, the practical design application of thin coupling beams of tunnel form building structures has been limited.

As a design alternative, shear forces in coupling beams can be decreased by introducing vertical separation joints (slits) at each ends of coupling beams without violating the architectural requirements and functional use. Yuksel [5] performed more than 800 static and dynamic finite element analyses on 240 different coupled shear walls with SCCBs having different stories, to be able to generalize the seismic behavior of the coupled shear walls with SCCBs. The internal force distribution, the overall stiffness and the dynamic behavior of the coupled shear walls with SCCBs were investigated and it was proven that the shear forces in deep coupling beams decrease significantly due to the existence of slits at the ends.

The objective of this paper is to present the behavior of SCCBs with the aid of the finite element method. Parametric studies are performed to investigate the shear stiffness factors of SCCBs. The effects of slits due to their application at the ends of the coupling beams are investigated for typical SCCBs. Unless the detailed finite element modeling is utilized, the conventional methods become deficient to compute the stiffness factors due to abrupt change in the centroidal axis associated with the non-prismatic section (see fig. 3). Despite the robustness of the finite element modeling, the generation of the fixed-end forces from the nodal outputs of the detailed mesh still remains as an intricate task.

2 Application of vertical separation joints to reduce the shear stiffness of the coupling beams

The dimensions of the coupling beams are the effective parameters on the behavior of the coupled shear walls when they are subjected to lateral loads [6–8]. In particular, the coupled shear walls will react to the lateral loads due to the stiffness ratio of the coupling beams to the shear walls [9]. Apparently, reducing the height of the coupling beam section will decrease its stiffness and result in a diminishing effect on the internal shear forces of the coupling beams, yet there are generally height constraints for the coupling beams due to the architectural restrictions and functional use. However, the shear stiffness and the internal shear forces of the deep coupling beams can be reduced by introducing slits without changing the dimensions of the coupling beams and not violating the architectural and functional requirements. It is proven that the use of the vertical slits at the ends of the coupling beams potentially reduces the higher shear forces acting on these coupling beams [5].



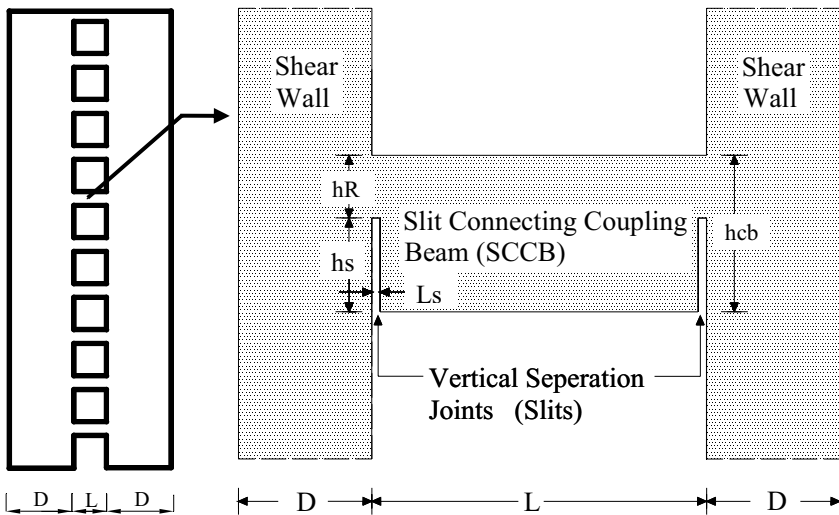


Figure 3: Application of slits at the ends of the coupling beams for coupled shear walls and the details of the slit connected coupling beams.

SCCBs can be classified as the special non-prismatic beams with varying slit heights and slit lengths at their ends. The geometric parameters of typical SCCBs are presented in fig. 3, where; h_{cb} = height of the coupling beam, L = length of the coupling beam, h_s = slit height at the beam-wall connections, L_s = slit length at the beam-wall connections, h_R = height of the coupling beam at the beam-wall connections, D = length of the individual shear walls forming the coupled shear wall system, b = shear wall or coupling beam thickness. Slit height ratio is defined as the ratio of the slit height to the total height of the coupling beam ($S = h_s / h_{cb}$). Slit length ratio is the ratio of the slit length to the coupling beam length ($\alpha = L_s / L$).

3 Parametric study and the finite element modelling of SCCBs

Coupling beams with symmetrical slits at their ends shown in fig. 3 are generated as the model structures for the analysis. Whole parts of the SCCBs were modeled using four-node shell elements with two translational degrees of freedom (d.o.f.) and one rotational d.o.f. per node. In order to have adequate accuracy, SCCBs were modeled using shell elements with dimensions of 10mm×10mm. The SAP2000 computer program [10] was used to develop the finite element models of the typical coupling beams with symmetrical vertical slits at their ends. A typical finite element model of SCCBs (given $b=0.2\text{m}$, $h_{cb}=0.9\text{m}$, $L=1\text{m}$) having the slit height of 450mm and slit length of 40mm was formed with 8560 shell elements and is shown at the left side of the fig. 4. The deflected shape of the same SCCB due to a vertical unit displacement at the left



end is illustrated in the right side of the fig. 4 for the shear stiffness analysis. In finite element analyses, the vertical unit displacements are represented by a set of prescribed nodal displacements [11]. Since the computation of the stress values or nodal forces is not sufficient for the calculation of the stiffness terms, the shear stiffness terms of SCCBs were calculated by using the nodal force outputs of the finite element analysis proposed by Bathe [12] and discussed in Horowitz [13].

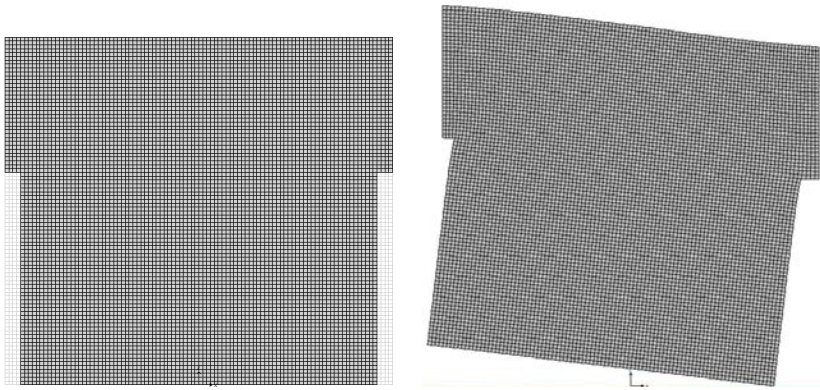


Figure 4: A typical finite element model of a SCCB using 8560 shell elements and its deflected shape due to vertical unit displacement at the left, while all other d.o.f.s are held for shear stiffness analysis.

In practice, the coupling beams above the door openings of the tunnel form buildings are generally constructed with 0.7~0.9 m heights and 0.8~1.2 m lengths for functional use and architectural considerations. The thickness of these coupling beams is generally less than 250mm for tunnel form buildings. For actual modeling, while the length of the coupling beams were varied as 0.8m, 0.9m, 1.0m, 1.1m, 1.2m, the thickness of the shear wall and the coupling beam was taken as the constant value of 0.2m. The depth of the coupling beams was taken as 0.75m and 0.90m for the parametric studies. The dimensions of the cross sections of the shear walls and the coupling beams are consistent with practical applications. The compressive strength of concrete was assumed to be 25MPa. The modulus of elasticity (E) and the Poisson's ratio (ν) were taken as 3×10^7 kN/m² and 0.2 respectively for all the analyses.

The slit heights ($h_s = 0.0\text{m}, 50\text{mm}, 100\text{mm}, 150\text{mm}$, etc) and the slit lengths ($L_s = 0.0\text{mm}, 10\text{mm}, 20\text{mm}, 30\text{mm}, 40\text{mm}$ and 50mm) were changed to achieve the values of the parameters, and the slab thickness was taken as 0.10m for all the analyses. Since the slits can only be extended up to the bottom of the floor slab whose thickness is taken as 0.10m, the maximum slit height can be 0.65m ($S = 0.866$) and 0.80m ($S = 0.888$) for 0.75m and 0.90m coupling beam heights, respectively. For each case, the vertical unit displacement was applied to each SCCBs to be able to determine the shear stiffness terms, and the outputs of finite element analysis results were scanned to compute the shear stiffness terms.



4 Effect of slits on the shear stiffness terms of coupling beams

The coupling beams that are deep in relation to their clear span undergo significant shear deformations. Thus, the effect of shear deformation in deep coupling beams is of greater importance than in conventional beams. Many classical books on structural analysis [14] give the stiffness influence coefficients of a prismatic beam element including transverse shear deformation. The shear stiffness term is expressed as in eqn. (1).

$$k_{11} = \frac{1}{1 + 2g} \times \frac{12EI}{L^3} \quad (1)$$

where E is the modulus of elasticity of the material, I is the gross moment of inertia of the section about the bending axis, L is the beam length, g is the dimensionless shear constant defined as $g = (6fEI)/(GA L^2)$, f is the shape factor (1.2 for rectangular cross sections), G is the shear modulus of the material and A is the cross sectional area of the beam section. Because a SCCB is non-prismatic, eqn. (1) does not accurately represent its shear stiffness. A detailed analysis is carried out to determine the magnitude of the reductions made on the shear stiffness terms in the presence of vertical separation joints of various slit height ratios, slit length ratios, coupling beam lengths and coupling beam heights. The shear stiffness term of a beam element is the shear force required to produce a vertical unit displacement at one end while all other d.o.f.s are set to zero (see fig. 4). The effect of shear deformations was taken into account in deriving the stiffness terms of SCCBs.

The shear stiffness terms including transverse shear deformations of coupling beams without any slits are calculated for different coupling beam lengths by using eqn. (1). The values obtained by eqn. (1) are compared with those obtained by the finite element analyses. The comparisons present better agreement with the maximum observed deviation of 3.9% for the shear stiffness term of the coupling beam having the length of 0.8m. The deviation in the stiffness terms decreases as the coupling beam heights decrease or the coupling beam lengths increase.

The effect of slit heights on the shear stiffness term (given as $b=0.2\text{m}$ and $h_{cb} = 0.90\text{m}$) is presented in the left graph of fig. 5 for different coupling beam lengths ($L = 0.8\text{m}, 0.9\text{m}, 1.0\text{m}, 1.1\text{m}, 1.2\text{m}$). The slit height has a significant effect on the reduction of the stiffness terms of the coupling beams. For a given specific coupling beam length, as the height of the slits increases, the reduction in the shear stiffness terms increases at an increasing rate. The relationship between the slit height ratio and the reduction in shear stiffness terms is nonlinear. A typical plot of the shear stiffness terms of SCCBs (given as $b=0.2\text{m}$ and $h_{cb} = 0.90\text{m}$) versus the slit height ratios is presented in the right graph of fig. 5 for different slit lengths ($L_s = 0\text{mm}, 10\text{mm}, 20\text{mm}, 30\text{mm}, 40\text{mm}$ and 50mm). However, it should be noted that, for a given specific coupling beam length and slit height, as the length of the slits increases, the reduction in shear stiffness terms increases. The decrease in shear stiffness terms according to the



slit lengths is in negligible level. Fig. 6 represents the variation of the shear stiffness terms (the cross sectional dimension is given as $0.2\text{m} \times 0.75\text{m}$) as the functions of slit heights for 0.8m and 1m lengths of SCCBs where the slit lengths vary ($L_s = 0\text{mm}, 10\text{mm}, 20\text{mm}, 30\text{mm}, 40\text{mm}$ and 50mm). The decrease in the shear stiffness terms for the given slit height ratios and coupling beam lengths do not change considerably for different coupling beam heights.

The shear stiffness terms of SCCBs ($k_{11(\text{SCCB})}$) are normalized with respect to the shear stiffness terms of the prismatic coupling beams ($k_{11(\text{SCCB})} / k_{11}$). The shear stiffness terms of the coupling beams without any slit connections (k_{11}) are calculated using eqn. (1). Table 1 presents the normalized shear stiffness terms of 1m length SCCBs (given $b=0.2\text{m}$ and $h_{cb} = 0.90\text{m}$) with respect to slit height ratio (S) for different slit length ratios (α). Also, the normalized shear stiffness terms of SCCBs having dimensions of $b=0.2\text{m}$ and $h_{cb} = 0.90\text{m}$ are given in Table 2 with respect to the slit height ratios (S) for different SCCB lengths ($L = 0.8\text{m}, 0.9\text{m}, 1.0\text{m}, 1.1\text{m}, 1.2\text{m}$). It is proven in Table 1 and Table 2 that the shear stiffness terms for SCCBs are not constant, as they depend on the relation between slit height ratios (S), slit length ratios (α) and the coupling beam lengths (L). The variation of the shear stiffness terms is pronounced more for the slit height ratios than for the slit length ratios and the coupling beam lengths. For a given specific coupling beam length and slit height, as the length of slits increases, the reduction in shear stiffness terms increase in negligible level. The shear stiffness terms for given specific slit heights and slit lengths do not change considerably for different coupling beam lengths.

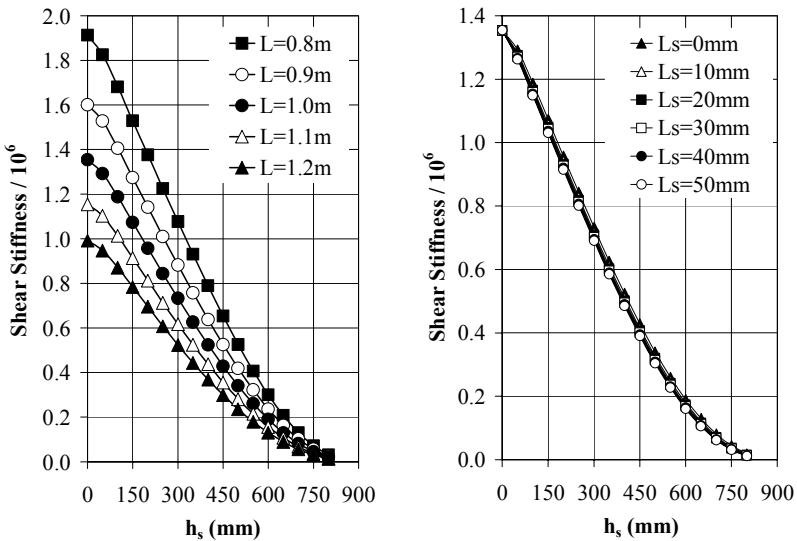


Figure 5: The variation of shear stiffness terms of SCCBs versus slit heights (h_s) for different coupling beam lengths (L) and slit lengths (L_s).



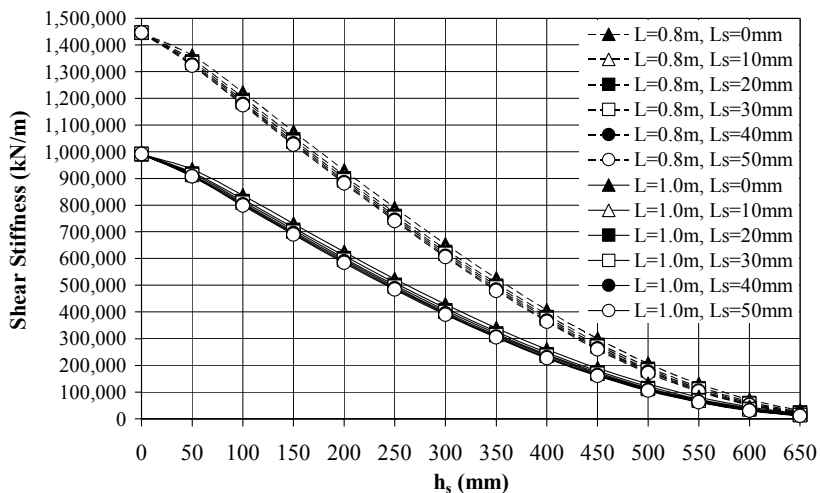


Figure 6: The variation of shear stiffness terms of SCCBs ($b=0.2\text{m}$, $h_{cb}=0.75\text{m}$) versus slit heights for different coupling beam lengths and slit lengths.

Table 1: Normalized shear stiffness factors ($k_{11}(\text{SCCB}) / k_{11}$) for 1m length SCCBs with respect to slit height ratios (S) for different slit length ratios (α).

S	α (slit length ratio)					
	0.00	0.01	0.02	0.03	0.04	0.05
0.00	1.000	1.000	1.000	1.000	1.000	1.000
0.06	0.955	0.946	0.942	0.938	0.935	0.933
0.11	0.878	0.866	0.860	0.856	0.852	0.848
0.17	0.793	0.781	0.774	0.769	0.765	0.762
0.22	0.707	0.695	0.689	0.684	0.679	0.676
0.28	0.623	0.611	0.605	0.600	0.595	0.592
0.33	0.542	0.530	0.523	0.518	0.514	0.510
0.39	0.463	0.451	0.445	0.440	0.435	0.432
0.44	0.388	0.377	0.370	0.365	0.361	0.358
0.50	0.317	0.306	0.300	0.296	0.292	0.288
0.56	0.252	0.242	0.236	0.232	0.228	0.225
0.61	0.193	0.184	0.178	0.174	0.171	0.168
0.67	0.141	0.133	0.128	0.124	0.121	0.118
0.72	0.096	0.090	0.086	0.083	0.080	0.078
0.78	0.061	0.055	0.052	0.049	0.047	0.046
0.83	0.033	0.029	0.027	0.025	0.024	0.023
0.89	0.014	0.012	0.011	0.010	0.009	0.008

Table 2: Normalized shear stiffness factors ($k_{11}(\text{SCCB}) / k_{11}$) for SCCBs having cross sectional dimensions of $b=0.2\text{m}$ and 0.9m with respect to slit height ratios (S) for different SCCB lengths (L).

S	$k_{11}(\text{SCCB}) / k_{11}$				
	L				
	0.8m	0.9m	1.0m	1.1m	1.2m
0.00	1.000	1.000	1.000	1.000	1.000
0.06	0.955	0.955	0.955	0.955	0.955
0.11	0.879	0.878	0.878	0.878	0.878
0.17	0.799	0.796	0.793	0.791	0.791
0.22	0.719	0.713	0.707	0.704	0.701
0.28	0.640	0.631	0.623	0.618	0.613
0.33	0.563	0.551	0.542	0.534	0.529
0.39	0.487	0.473	0.463	0.454	0.448
0.44	0.413	0.399	0.388	0.379	0.372
0.50	0.342	0.328	0.317	0.308	0.302
0.56	0.275	0.262	0.252	0.244	0.238
0.61	0.213	0.201	0.193	0.186	0.181
0.67	0.157	0.148	0.141	0.135	0.131
0.72	0.109	0.102	0.096	0.093	0.090
0.78	0.068	0.064	0.061	0.058	0.056
0.83	0.038	0.035	0.033	0.032	0.031
0.89	0.016	0.015	0.014	0.014	0.013

For rigorous finite element simulations on all stiffness terms of SCCBs, the interested reader is addressed to the work presented by Yuksel [5]. In that study also, an empirical formula is proposed for the equivalent beam model consisting of two noded prismatic beam elements representing SCCBs. The formulation includes the shear deformations and the shapes of the cross sections of SCCBs. The method is introduced in a simple format and coupled shear walls with SCCBs can easily be modeled by the equivalent frame method.

5 Summary and conclusions

The cross-sectional area of the coupling beam at the beam-wall connections is purposely reduced by slit application. A series of shear stiffness analysis of SCCBs is carried out with the aid of the finite element method. The results obtained from the finite element analyses indicate significant decreases in shear stiffness force acting on the deep coupling beams due to slit existence at the ends. The height of the slits at the ends of the coupling beams is a significant parameter of the shear stiffness terms. As the height of the slits increases, the reduction in shear stiffness terms increases. The behavior of the coupling beams can be adjusted by applying the appropriate amount of slits at the ends of the coupling beams. Therefore, the designer can decrease the shear stiffness and the



internal shear forces of the deep coupling beams effectively by introducing the appropriate amount of slits at the ends of the coupling beams without violating the architectural requirements.

References

- [1] Kalkan, E. & Yuksel, S.B., Prons and Cons of Multi-Story RC Tunnel Form Buildings. *The Structural Design of Tall and Special Buildings*, (in press).
- [2] Yuksel, S.B. & Kalkan, E., Behavior of Tunnel Form Buildings Under Quasi-Static Cyclic Lateral Loading. *Structural Engineering and Mechanics*, (in press).
- [3] ACI 318-05., *Building Code Requirements for Reinforced Concrete and Commentary*. American Concrete Institute; Farmington Hills, MI., 2005.
- [4] TSC. 1998., *Specifications for the Structures to be Built in Disaster Regions*. Ministry of Public Work and Settlement, Ankara, Turkey (Turkish Seismic Code 1998).
- [5] Yuksel, S.B., Slit Connected Coupling Beams For Tunnel Form Building Structures. *The Structural Design of Tall and Special Buildings*, (in press).
- [6] Chaallal, O., Gauthier, D. & Malenfant, P., Classification methodology for coupled shear walls. *Journal of Structural Engineering*, ASCE, **122(12)**, pp. 1453-1458, 1996.
- [7] Subedi, N.K., RC-coupled shear wall structures. II: Ultimate strength calculations. *Journal of Structural Engineering*, ASCE, **117(3)**, pp. 681-697, 1991.
- [8] Shiu, K.M., Takayanagi, T. & Corley, G., Seismic behaviour of coupled wall system. *Journal of Structural Engineering*, ASCE, **110(5)**, pp. 1051-1066, 1984.
- [9] Paulay, T. & Priestly, M.J.N., *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley & Sons: New York, 1992.
- [10] Computer and Structures Inc. (CSI). 2002. SAP2000 User's Manual. Berkeley, CA, June 2002. www.csiberkeley.com.
- [11] El-Mezaini, N., Balkaya, C. & Cıtipioğlu, E., Analysis of frames with nonprismatic members. *Journal of Structural Engineering*, ASCE. **117(6)**, pp. 1573-1592, 1991.
- [12] Bathe, K.J., *Finite Element Procedures*, Prentice Hall Publisher: NJ, USA, 1996.
- [13] Horowitz, B., Singularities in elastic finite element analysis. *Concrete International*, December: pp. 33-36, 1997.
- [14] Weaver W, Gere JM., *Matrix Analysis of Framed Structures*, Van Nostrand Reinhold: New York, 1990.



Principal stress behaviour of a steel plate shear wall concerning buckling modes

P. Memarzadeh, M. Azhari & M. M. Saadatpour

Department of Civil Engineering, Isfahan University of Technology, Iran

Abstract

When buckling occurs in the infill plate of a steel plate shear wall (SPSW), a diagonal tension field is formed through the plate. This paper investigates the influence of torsional stiffness of surrounding members (i.e. beams and columns) on the buckling coefficient and tension field behaviour of SPSW. The linear buckling equations in the sense of von-Karman have been solved in conjunction with various boundary conditions, by using the Ritz method. Also, in this research the effects of symmetric and anti-symmetric buckling modes on the behaviour of the tension field and buckling coefficient have been studied.

Keywords: steel shear wall, thin plate, shear buckling, symmetric, anti-symmetric, Ritz method, principal stresses.

1 Introduction

The steel plate shear wall is a lateral load resisting system consisting of an infill plate located within a frame. While performing experimental investigations on the thin aluminum shear panels of an aircraft, Wagner found out that in thin-webbed structures with stiff boundary members a diagonal tension field would be formed when buckling occurs. Then Wagner [1] developed the pure tension theory stating that the formation of the tension field is the primary mechanism for shear resistant. The incomplete tension field theory was later presented by Kuhn *et al.* [2]. On the basis of Kuhn's theory the shear resistance capacity is a combination of pure shear and inclined tension field.

Design engineers require the ability to assess inelastic structural response using conventional analysis software that is commonly available. An analytical model—termed the strip model—was developed by Thorburn *et al.* [3] to simulate the tension field behaviour, wherein the infill plate is modelled as a



series of tension-only strips at the same angle of inclination, θ , as the tension field. They derived the angle of inclination for the strips, θ , from the principle of Least Work as a function of axial stiffness of boundary members.

By including the effect of in-plane flexural stiffness of boundary members and employing the principle of Least Work, Timler and Kulak [4] derived another equation for θ in terms of axial and flexural rigidities of surrounding members.

The Canadian Steel Design Standard [5] suggests the application of the strip model as a design tool for steel plate shear wall (CAN/CSA 516-01) and the equation derived by Timler and Kulak [4] for the calculation of θ (clause 20.3.1). However, researchers are still searching for an increase in the precision of the prediction of the overall behavior of the shear wall.

This paper investigates the effect of different parameters on buckling loads as well as on the distribution and orientation patterns of the tension field principal stresses. These parameters include torsional stiffness of boundary members as well as symmetric and anti-symmetric buckling modes.

2 Theory

2.1 Modelling of SPSW

The surrounding members of the SPSW are modeled by the springs.

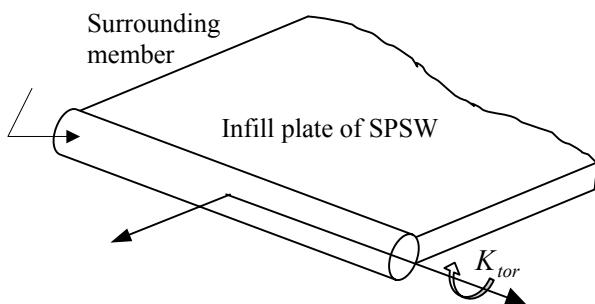


Figure 1: General scheme for a section of the model.

To define logical parameter for the amount of torsional stiffness of surrounding members, the non-dimensional stiffness parameter α is introduced as follows:

$$\alpha = \frac{K_{tor}}{D} \quad (1)$$



where K_{tor} is the unit length torsional stiffness of surrounding members and D is the flexural rigidity of plate. A model is defined for studying the effect of the stiffness parameter α shown in fig. 1. This way, a comparison between the effectiveness of different stiffness parameters of surrounding members is carried out.

2.2 Ritz method

This paper utilizes the Ritz method to analyze the buckling of infill plate of a SPSW under an applied in-plane shear loading (fig. 2). The geometric (V_p) and the elastic strain energy (U) are the variants used in the energy solution, and are given by the following equations:

$$V_p = -N_{xy} \int_A (w_{,x} w_{,y}) dA \quad (2)$$

where N_{xy} , A and w are the elastic shear buckling load, the area and the lateral buckling displacement of the plate, respectively. The comma denotes differentiation with respect to the corresponding co-ordinates.

$$U = \frac{D}{2} \int_A [(w_{,xx} + w_{,yy})^2 - 2(1 - \nu)(w_{,xx} w_{,yy} - w_{,xy}^2)] dA + U_s \quad (3)$$

in which U_s is the strain energy of the spring are defined by:

$$U_s = \int \left[\frac{K_{tor}}{2} \left(w_{,x}^2 \Big|_{x=-\frac{a}{2}} + w_{,x}^2 \Big|_{x=\frac{a}{2}} \right) \right] dy + \int \left[\frac{K_{tor}}{2} \left(w_{,y}^2 \Big|_{y=-\frac{b}{2}} + w_{,y}^2 \Big|_{y=\frac{b}{2}} \right) \right] dx \quad (4)$$

In the use of the Ritz method, an appropriate displacement function for w must be chosen. That used herein is the polynomial-based displacement function which consists of a boundary polynomial specifying the geometric and kinematic boundary conditions multiplied by a complete simple polynomial. This displacement function is written by:

$$w = \varphi_b(\xi, \eta) \sum_{q=0}^p \sum_{r=0}^q a_m \phi_m(\xi, \eta) \quad (5)$$

where p is the degree of a two-dimensional polynomial and a_m is the arbitrary Ritz coefficient.



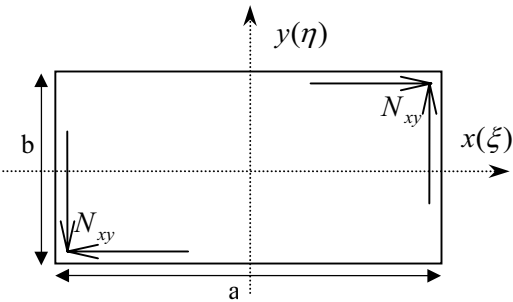


Figure 2: Isotropic plate under pure shear.

$\phi_m(\xi,\eta)$ is the m-th term of a two-dimensional polynomial as below (Smith *et al.* [6]):

$$\phi_m(\xi,\eta) = \xi^r \eta^{q-r} \tag{6}$$

in which $\xi = 2x/a, \eta = 2y/b$ is given by:

$$m = \frac{(q+1)(q+2)}{2} - r \tag{7}$$

The term $\varphi_b(\xi,\eta)$ is the boundary polynomial describing the boundary conditions defined by:

$$\varphi_b(\xi,\eta) = (\xi-1)^1(\xi+1)^1(\eta-1)^1(\eta+1)^1 \tag{8}$$

In the buckling analysis, the kinematic and geometric boundary conditions are specified when the boundary polynomial $\varphi_b(\xi,\eta)$ is multiplied by the corresponding internal interpolation polynomial.

2.3 Linear eigenvalue analysis

The total potential energy Π of the system is given by:

$$\Pi = U + V_p \tag{9}$$

Based on the principal of minimum potential energy, the total potential Π in eqn. (9) is minimized with respect to the unknown Ritz coefficient a_m . Because Π is a function of the product of Ritz coefficients $a_m a_n$, minimization by

formal differentiation leads to a set of simultaneous linear independent equations. The solution of these equations produced the eigenvalues (buckling loads) and substituting of the corresponding eigenvectors into the displacement function w in eqn. (5) as the Ritz coefficients gives the buckling modes.

2.4 Stress analysis

Since the buckling modes of a plate specify the proportional values of transverse deflections, the corresponding values of strains and stresses will be calculated proportionally. Using the transverse deflection w , the stresses in the mid-plane of plate can be written by:

$$\begin{aligned}\sigma_x &= \frac{E}{2} (w_{,x}^2 + \nu w_{,y}^2) \\ \sigma_y &= \frac{E}{2} (w_{,y}^2 + \nu w_{,x}^2) \\ \tau_{xy} &= G w_{,x} w_{,y}\end{aligned}\quad (10)$$

where G is the shear modulus of elasticity. Using the Mohr's circle, the state of stresses can be represented in the principal coordinates. Also the angle of inclination of the tension field can be calculated by determining the orientation of the principal stresses. Then, it is possible to plot the distribution and orientation patterns of the principal stresses in the tension field of a plate.

3 Numerical parametric studies

3.1 Shear buckling analysis

A computer program has been developed based on the von-Karman theory and the Ritz method. The numerical analyses were performed by the computer program. In these buckling analyses, the value of p was selected equal to 8. To compare the various buckling analyses, the non-dimensional buckling coefficient was employed as follows:

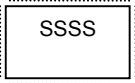
$$k_s = \frac{N_{xy} b^2}{\pi^2 D} \quad (11)$$

By plotting the various buckling mode shapes, it will be specified which modes are symmetric or anti-symmetric. The "first" symmetric and anti-symmetric modes are corresponding with the minimum values of the symmetric and anti-symmetric buckling loads, respectively. However, in this paper the word "first" is omitted for brevity. On the purpose of verifying the validity of buckling analyses, the results are compared with the available references. So, the stiffness



of spring is selected equal to zero or infinite for modelling simply support (S) or clamped edges (C), respectively. In table 1, the resulting buckling coefficients from the present analyses have been compared with those reported in references. As the table 1 shows, the results are in good agreement.

Table 1: Comparison the present results with those of available reference.

Stiffness Parameter	Boundary Conditions	a/b	k_s		
			Present		Timoshenko (1963)
			Symmetric	Anti- symmetric	
$\alpha = 0$		1.0	9.3254	11.5484	9.34
		1.5	7.0707	7.9591	7.10
		2.0	6.5464	6.5781	6.60
		3.0	5.9535	5.8465	5.90
$\alpha = \infty$		1.0	14.6515	17.1165	14.71
		1.5	11.4791	12.0293	11.50
		2.0	10.6527	10.5545	10.34
		3.0	9.8449	10.6985	-----

Typically, the symmetric and anti-symmetric buckling modes of plate are depicted in three-dimension views (fig. 3). Fig. 4 shows the effect of varying the stiffness parameters α on the symmetric and anti-symmetric buckling coefficients of plate. The following results can be concluded by attending to these figures:

- The symmetric and anti-symmetric buckling coefficients of a plate with aspect ratio equal or greater than 1.5 are close together.
- Although the symmetric buckling mode is often the critical mode of shear buckling, sometimes the anti-symmetric mode would be critical.
- Fig. 4 shows that the shear buckling mode of a plate would not be changed by varying the stiffness parameter α , because there is no intersection for curves in fig. 4.

3.2 Stress analysis

3.2.1 Principal stress distribution pattern (PSDP)

By comparing the PSDPs with the corresponding buckling modes, the areas where the amounts of principal stresses are peak, may be specified. Fig. 5 illustrates these comparisons for two extreme values of zero and infinite for the stiffness parameter α . This figure shows that the peak(s) of principal stresses occurs at the slope(s) of buckling mode shapes for both symmetric and anti-symmetric modes. Therefore, in symmetric buckling modes, the principal stresses peaks are being at both sides of the plate centre, while in anti-symmetric buckling this peak would be in centre of the plate. Also, fig. 5 shows that the PSDPs are symmetric for both symmetric and anti-symmetric buckling modes.



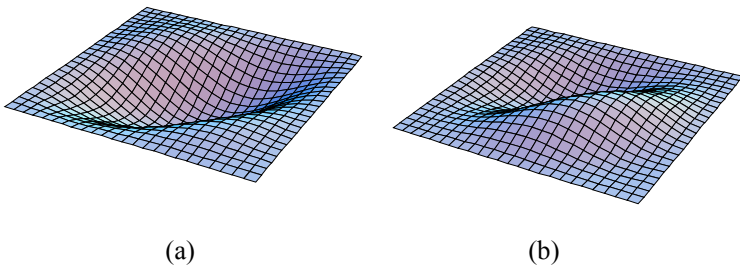


Figure 3: Symmetric (a) and anti-symmetric (b) shear buckling modes.

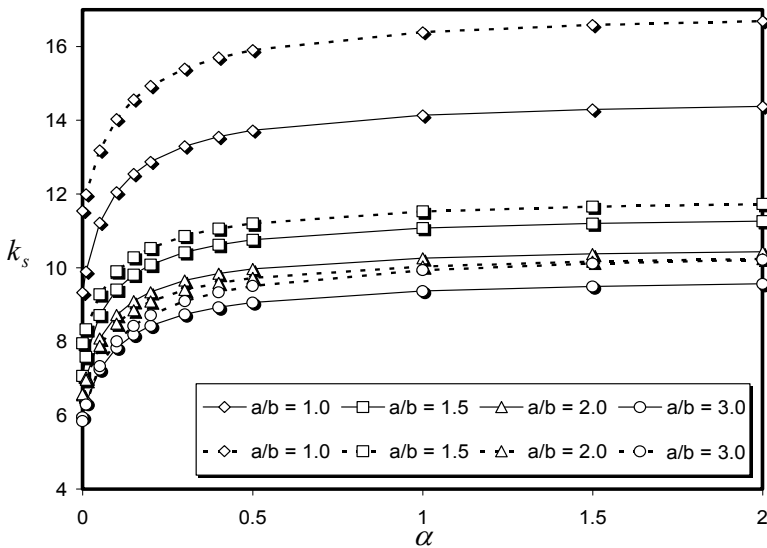


Figure 4: Shear buckling coefficient vs. α (line for symmetric and dashed for anti-symmetric buckling).

3.2.2 Principal stress orientation pattern (PSOP)

For showing some patterns simultaneously, it is advantageous that the patterns are putted together and combined as shown in fig. 6. The orientations of principal stresses can be determined at each point of the plate by using the Mohr's circle. Fig. 7 shows the combined PSOPs related to various values of stiffness parameters α . In this figure, the orientation of each depicted line represents the orientation of the related principal stress. By careful observation, it is realized that, there are areas in the plate where the orientations of related principal stresses will not be changed by varying the value of the stiffness parameter α .



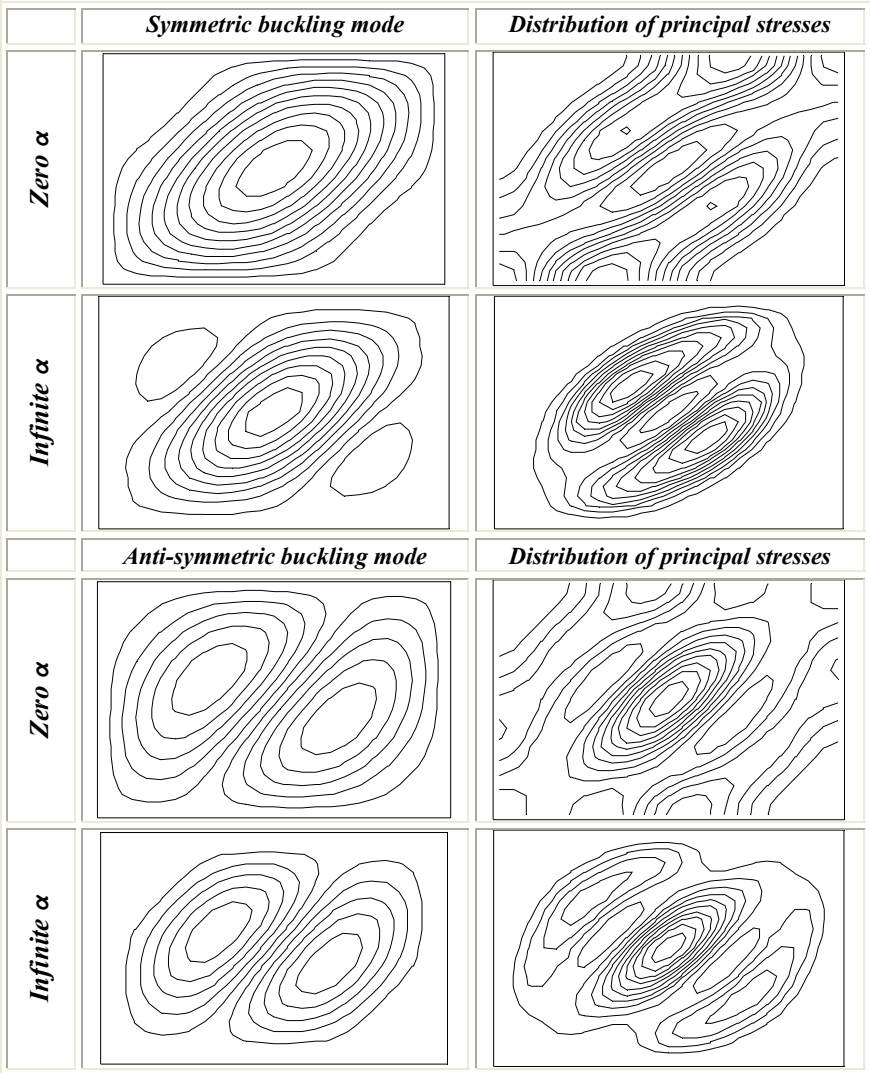


Figure 5: Symmetric and anti-symmetric buckling mode shapes and PSDPs for two extremes of $\alpha = 0$ and $\alpha = \infty$ (aspect ratio 1.5).

These areas of the plate in symmetric buckling are more extended than those in anti-symmetric buckling. Also, these areas have different distribution for the symmetric and anti-symmetric buckling modes.

Fig. 8 shows the combinations of PSOPs related to symmetric and anti-symmetric buckling modes. This figure reveals that the PSOPs are relatively different for the symmetric and anti-symmetric buckling modes.

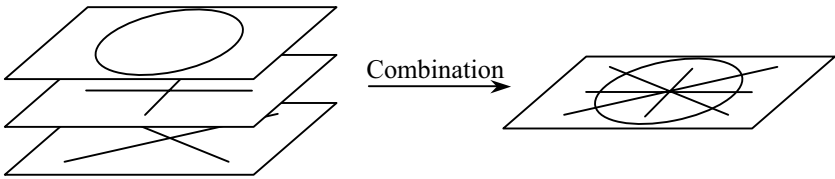


Figure 6: Scheme for combination of some patterns.

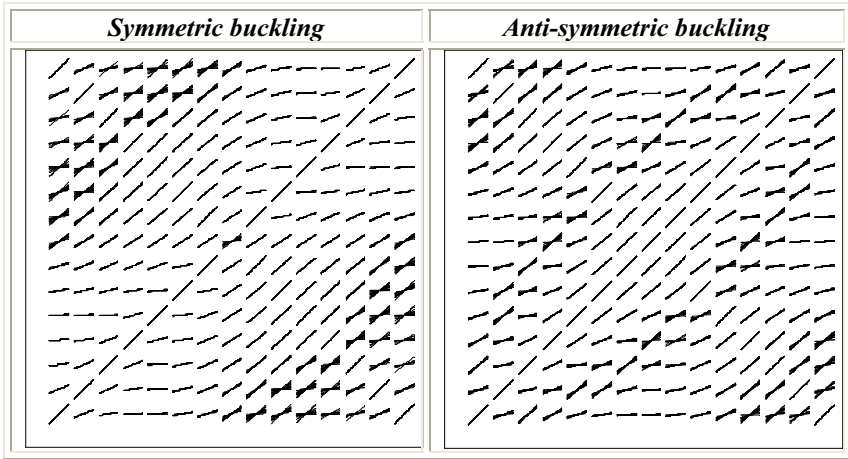


Figure 7: Combination of PSOPs related to various α for symmetric and anti-symmetric buckling modes.

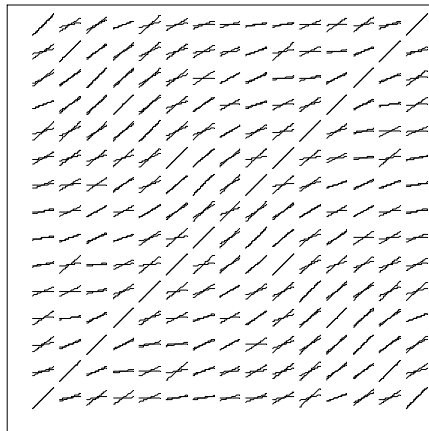


Figure 8: Combination of PSOPs related to symmetric and anti-symmetric buckling modes ($\alpha = 0$).



4 Conclusions

Initial imperfection of plates due to their fabrication causes that the plates do not experience the buckling bifurcation point. The type of postbuckling mode of a plate may be a function of its initial imperfection, especially for the plate with close buckling loads correspond with different buckling modes. This research reveals that buckling loads correspond with first symmetric and anti-symmetric buckling modes of a plate with an aspect ratio equal or greater than 1.5 are close together. This result specifies the important role of the initial imperfection of a plate in determining the postbuckling mode of the plate.

It is also shown that the orientation patterns of principal stresses correspond with the symmetric and anti-symmetric buckling modes of a plate are different, relatively. Since the angle of inclination of the tension field of a SPSW is an effective parameter on development of the strip model, so, this result may be vital in modifying the strip model. The role of initial imperfection of the plate in determining the type of buckling mode has not been included in any analytical models presented so far.

These studies also reveal that variation of amount of torsional stiffness of boundary members does not change the orientations of principal stresses in some areas of the plate. These areas where located in the slopes of the buckling mode shapes have relatively great principal stresses.

References

- [1] Wagner, H., Flat sheet metal girders with very thin webs, Part I – General theories and assumptions. *Technical Memo No. 604, National Advisory Committee for Aeronautics*, Washington, D.C, 1931.
- [2] Kuhn, P., Peterson, J.P., and Levin, L.R., A summary of diagonal tension, part I – Methods of analysis. *Technical Note 2661, National Advisory Committee for Aeronautics*, Washington, D.C, 1952.
- [3] Thorburn, L.J., Kulak, G.L., and Montgomery, C.J., Analyses of steel plate shear walls. *Structural Engineering Report No. 107*, University of Alberta, Canada, 1983.
- [4] Timler, P.A., Kulak, G.L., Experimental study of steel plate shear walls, *Structural Engineering Report No. 114*, Department of Civil Engineering, University of Alberta, Edmonton, Canada, 1983.
- [5] Canadian Standard Association, CAN/CSA S16-01, *Limit States Design of Steel Structures*, Toronto, Ontario, 2001.
- [6] Smith, S.T., Bradford, M.A., and Oehlers, D.J., Elastic buckling of unilaterally constrained rectangular plates in pure shear. *Engineering Structures*, **21**, pp. 443-453, 1999.



Earthquake architecture as an expression of a stronger architectural identity in seismic areas

T. Slak & V. Kilar

University of Ljubljana, Faculty of Architecture, Slovenia

Abstract

This paper discusses the term “earthquake architecture” as a result of intersection of design principles in architecture and earthquake engineering. It examines the hypothesis that the architectural design which reflects an earthquake threat might be an important source of stronger architectural identity typical for earthquake prone areas. The purpose of the paper is to encourage the development of new principles and forms of architectural design in these areas. Technology, codes and cooperation with earthquake engineers are not the only or satisfactory solutions for appropriate culturally respectful design of buildings and landscape in earthquake prone areas. The possibilities of architectural response to an earthquake threat are further analyzed. The paper describes earthquake engineering and architectural background of earthquake architecture and gives some examples of positive practise. The intensity of relations between the two fields is divided into different levels. Higher levels of intensity interfere more into the field of earthquake architecture. The given examples interpret various possible levels of cooperation within earthquake architecture.

Keywords: earthquake architecture, earthquake engineering, architecture, structures in architecture, building, earthquake resistant design.

1 Introduction

In the paper, the expression “earthquake architecture” is used to refer to a particular type of architecture which arises in earthquake prone areas, as a response to the requirements of earthquake engineering and is a consequence of combining earthquake engineering and architecture. The realization of a building without a suitable earthquake resistant structure is not possible today, however, it is possible to design a building in such a way that earthquake resistance is not expressed and structural influence on architecture is minimal. In such cases we



can speak of *concealed* ways of earthquake resistance of a building. On the other hand, architecture can respond in the concept itself, i.e. in two ways: *effectively*, with increased horizontal stiffness of a building or (in addition to that) *symbolically*, with metaphorical changes in design. Earthquake architecture is the “missing link” between earthquake engineering and architecture. It combines the best of both fields and establishes a new approach and quality in construction in earthquake prone areas, mainly in compliance with measures of architectural excellence.

The complex requirements of earthquake engineering directly influence the architectural composition and concepts in architecture, thus detailed examination of influences is the basis for any architectural activity in seismic areas. The modern methods for increasing earthquake resistance of buildings are based on the seismic codes, as well as on the usage of passive and/or active systems for damping and dissipation of earthquake energy. According to (Mezzi et al [9]) they enable a freer building design and more flexible solutions in architectural design in earthquake prone areas.

It has been noted that, by introducing more and more detailed standards and regulations, the principles of earthquake resistant design are becoming important determining factors of architectural design in earthquake prone areas. It seems reasonable to believe that architecture should always be local, i.e. designed in accordance with micro-location features of the area, and that it should in some way respond to the earthquake threat. Adjustment to the earthquake resistant construction requirements is often regarded as pressure on artistic freedom and a limitation in following trends coming from the areas of the developed world not prone to earthquakes (the Netherlands, Great Britain, Scandinavia, etc.). But the problem in question is not the limitations, but rather lack of knowledge and inability to develop a particular and, within frameworks of earthquake resistant construction, inventive architecture.

Our hypothesis is that, at the contemporary time of emphasised concern for sustainable and regional development and in searching for a new, particular expression in architecture, the response of architecture to earthquake threats can present an important source of stronger architectural identity typical of earthquake prone regions. In the paper the hypothesis is verified by a comparative analysis and intersection of concepts of modern earthquake resistant design and architectural concepts of composition and building design. Furthermore, the article analyses the basic characteristics of earthquake architecture and seeks and examines the areas of possible conflicts and constraints.

2 Concepts of modern earthquake resistant design

When designing a building in a seismic area, we have to comply with the regulations and recommendations given in building standards and codes. These demands have a decisive influence on the design of structural system of the object, which in turn interferes with the architectural concept. Earthquake engineering has developed a variety of ways for increasing earthquake resistance of buildings,



which present different concepts of building protection in line with generally established design philosophy in earthquake prone areas. Roughly, the ways of achieving suitable earthquake resistance of a building can be divided into the following four groups: A) tectonic construction, B) basic protection according to regulations, C) passive protection, D) active protection and developing systems.

A) Tectonic construction (regularity, symmetry, height limitation, etc.)

Classic, tectonic (also traditional) principles of regular construction are taken into consideration, which were in force in history before the establishment of building codes: mass is concentrated in the lower storeys, walls are massive (thick) and are getting thinner towards the top, regularity is ensured (symmetry, direct supporting, maximum floor plan dimension ratio 1:4), buildings have height limitations which depend on the materials used, layout shows high density of structure which warrants the shear transmission of horizontal forces into foundations. The structure is “designed” to remain elastic during a potential earthquake. In our case this term refers to emphasised and prevailing principles of regular construction. Examples of markedly non-tectonic construction are buildings with a soft ground floor, with the majority of mass in the upper floors, irregularly shaped, with larger overhangs, etc.

B) Basic protection according to codes (modern earthquake resistant construction, required combination of strength and ductility)

The basic protection according to modern building codes developed in the last decades, and as it is defined in this paper, presents nowadays a minimum level of earthquake resistant construction, which has to be taken into account when constructing new buildings and when adapting existing buildings in earthquake prone areas. It has to be emphasised that structural engineer, in contrast to an architect, is held liable for the adequacy of a structure design, which means that all systems used must comply with code requirements for safety and quality.

C) Passive protection (base isolation, energy dissipation systems)

This group includes various passive base isolation systems, which are usually combined with various types of passive energy dissipation systems or devices. These structural protective devices can be divided into two major groups: 1) *Seismic isolation* (elastometric or lead rubber bearings, sliding friction pendulum bearings and sliding bearings with restoring force) and 2) *Damping systems* (histeretical dampers, viscous dampers, tuned mass/liquid dampers, phase transformation dampers) (Constantinou et al [5]). These systems can be placed above the foundations or in critical areas along the entire structure.

D) Active protection (base isolation + semi-active and active damping systems) and systems in development

This is an upgrade to passive protection, which includes the use of the latest technologies, such as *semi-active and active energy dissipation systems* (mass/fluid dampers, bracing systems etc.), *computer controlled response of buildings* to earthquake simulation using electrorheological (ER) and magnetorheological (MR) dampers and *other smart variable stiffness and*



damping systems. The material properties of ER and MR materials can be changed in milliseconds by an applied low-power electric, or magnetic, field. At zero electric field, these materials are viscous liquids. At high fields they behave like viscoelastic-plastic solids. Members making use of ER or MR fluids can regulate very large forces with almost no external energy. (Yang [11]). One of the most promising developing technologies today in areas with frequent (regular) seismic activity is *Neuro-fuzzy logic systems* or *Fuzzy systems* (also Neural fuzzy models) (Kim et al [7]). It is an active, computer controlled system, which monitors earthquake activity in the location itself, and treats the building and its surroundings as a complex dynamic system. After processing information, it can in this way calculate the highest probability of earthquake direction and automatically “prepares” for an earthquake. After several earthquakes, the computer as a neuron network uses the “fuzzy logic” principle to predict the next earthquake. Neuro-fuzzy logic system enables a certain form of local seismic predictions, which are though to be the most accurate for the building in question, and is related to (semi-)active protection systems.

3 Concepts of architectural composition and design

Architectural composition and concepts have not changed much from antique, when first architectural theorist Vitruvius determined architecture by structure (*firmitas*), usefulness (*utilitas*) and aesthetics (*venustas*). Studying the architectural theory, we find these postulates in various forms throughout all history and it seems they have remained unchanged from their formation until today. Despite the differences in interpretation, none of the more serious definitions questions the status of architecture as art. The work of an architect has the characteristics of a cultural act and artistic achievement. With the development of architectural theory, the previously mentioned postulates have been complemented by numerous other detailed starting points and subdivisions, among which we most frequently come across spatial (urban) aspects, which are actually a matter of context. Architectural concepts, which arise through evaluation and ethics, are nowadays determined also by: location and urbanisation of the environment, the morphology of a building and its surroundings, context, the significance of an building with regard to purpose and/or importance, historical determination, building typology, the concept of architectural design, the elements of architectural design, the harmony of composition (ratios, relations) and other starting points, about which an architects forms an opinion, assesses the existing situation and carries out architectural intervention in the space. In doing so, the architect takes full responsibility for the space, which can be upgraded, neutralized, or deformed etc. by his intervention.

Thus architecture is not an idealised form, but a consequence of starting points offered by the site, when it is evaluated, read and analysed in the process of creation, and which, after all, represents the prevailing category for determining architecture. Structure and in our case earthquake design of a building is the necessity which ensures safety and stability of a building. Modern construction and earthquake engineering enables much more than in the past; therefore the need for architectural freedom has increased as well, and should be more accessible with the help of technology.



Architecture is perceived in different ways. It comprises the visual aspects of a building in space and the abstract perception of architecture not visible to the eye, but which can be comprehended through the use and sensual perception of the building. Regarding visual effects, the earthquake resistant load bearing structure can be *emphasised* or *hidden and concealed*. In his article *A post-biblical view* Lebbeus Woods clearly emphasises the importance of adequate construction in seismic areas: “Earthquakes as natural event are not inherently catastrophic. Destruction is not the ‘fault’ of earthquakes, but rather of the buildings, which, even in the regions regularly visited by earthquake, are not designed to work harmoniously with the violent forces periodically released.” (Woods in: Garcia [16]). It is this ability to harmonize the actual (structural) and architectural (aesthetic) response to earthquake forces which we ought to be searching for and appreciate in assessment.

4 Earthquake architecture

4.1 Definition

The broad expanse of the intersection of architecture with earthquake engineering is considered to be within the scope of the term *earthquake architecture*. The first mention of the phrase *earthquake architecture* occurs in the paper “Earthquake Engineering and Earthquake architecture” by Bob K. Reithermann. He noted that while ‘earthquake engineering’ was a common term for organisations and conferences, ‘earthquake architecture’ had an unaccustomed ring to it, and asked “Is there such a thing as *earthquake architecture*, and if so, what is it?” (Reitherman [10]). C. Arnold uses the phrase *earthquake architecture* to describe a degree of architectural expression of some aspect of earthquake action or resistance (Arnold [2]). The breadth of expressive possibilities ranges from metaphorical (visually expressed) uses of seismic issues, to the more straightforward exposure of seismic technology.

Nunotani Headquarter Building in Tokyo (Figure 1) is an extreme example of metaphor and symbolism used in an architectural response to seismic design. Its disjointed and displaced facade elements are intended to “represent a metaphor for the waves of movement as earthquake periodically compress and expand the plate structure of the region.” However, the fact remains that seismic issues have generated an innovative architectural design concept (Charleson and Taylor [3]).

4.2 How to achieve earthquake architecture?

Earthquake architecture can be defined as any visual or conceptual inter-connection between the concepts of earthquake engineering (section 2) and concepts of architecture (section 3). The inclusion of the requirements of earthquake resistant design in the process of creating and conceptualizing the architecture of a real building can be based on conceptual or visual level. Looking at it visually, we can speak of *hidden and concealed* ways of earthquake resistant architecture on the one hand, and *revealed* or *emphasised* on the other. From the conceptual point of view, earthquake architecture is realized only by



including the principles of earthquake engineering in the architectural concept itself, and in this way we achieve the highest level of cooperation through identification, where architecture is based entirely on the principles of earthquake engineering. Strategies for realizing the vision of a more widely accepted earthquake architectural approach inevitably depend on architects. Structural engineers need to be the catalysts for the vision to be caught and progressed (Charleson et al [4]). In the present paper, which presents the first steps of our research, we decided to analyse three different levels of including earthquake engineering in architecture:

- Level 1:** Earthquake resistance as a concept is inferior to architecture;
- Level 2:** Concepts of architecture and earthquake engineering are complementary;
- Level 3:** Earthquake resistant structure identifies architecture.



Figure 1: Example of symbolism and metaphor which architecture uses to react to earthquake threats: Nunotani Headquarter Building in Tokyo.

We have noted that there is not much earthquake architecture in earthquake prone areas. We can claim that a large number of buildings do not show architectural, i.e. visible or conceptual characteristics of earthquake architecture, or they use merely hidden ways of earthquake safe construction and earthquake engineering technology. In these cases the possibility of using earthquake architecture as a form of expression thus remains unrealized potential. Nonetheless, there is also a negative side to earthquake architecture, we might call it “anti-” or “non-earthquake” architecture. In this case the visual and abstract in architecture is achieved by contradicting earthquake reality, which negates (confrontation) or ignores (indifference) the requirements of earthquake design. At the worst, architecture can defy the rules of earthquake resistant construction with intentional mistakes in design. This negative side represents the conflict in the relationship between earthquake engineering and architecture, thus also within earthquake architecture itself. In this case legislation is the only guarantee that “anti-earthquake” architecture cannot be realised to the full extent in practice.



4.3 Examples of earthquake architecture

The three assumed levels of including earthquake engineering in architecture can be supported with the following examples.

Level 1: Earthquake resistance as a concept is inferior to architecture

The expressiveness of architecture is above structure, which as an inferior partner mainly provides safety and serves the architectural concept, which actually does not originate in earthquake design. An already conceptualized building, sometimes together with the structure, seeks confirmation in earthquake engineering and adapts minimally to the requirements of earthquake safety in further procedures. Advanced technologies can be used, structure is hidden behind facades and majority details are hidden. Two such examples are shown in Figure 2. Architecture achieves a high level of autonomy, sometimes at the expense of earthquake resistance of a structure. The influence of structure on architecture is thus minimal and mostly has an inferior role.

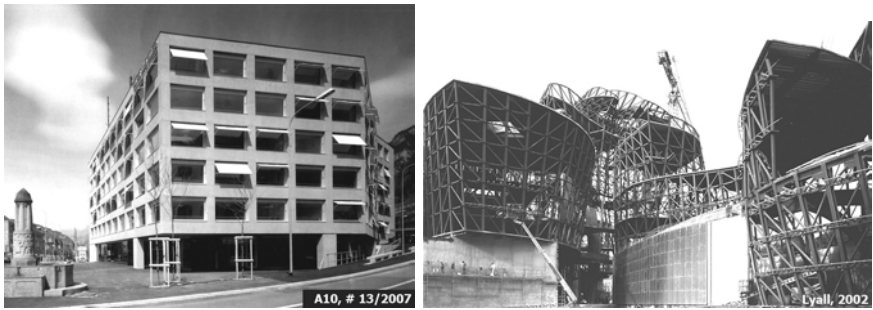


Figure 2: Structure adapted to the requirements of architecture: a building with “soft storey” (left) and the structure of a museum in Bilbao which simply follows the architectural idea which is completely formalistic and artistic. In the end, the entire structure is covered with façade (right).

Level 2: Concepts of architecture and earthquake engineering are complementary

Structure design is expressed and visible in the facades of buildings and the interior. Structure design is one of the motives of architecture and is also a logical consequence of building design. In this instance a high level of cooperation of both fields and mutual understanding are needed. The influence on architecture can be substantial; however, it can also be almost invisible or minimal, if it means the integration of structure into architectural design. A few examples where the cooperation between architecture and earthquake engineering was one of the guides in architecture design are presented in Figure 3.

Level 3: Earthquake resistant structure identifies the architecture

This level is based on using structure as the exclusive aesthetic norm, i.e. structure is the only articulated form which determines architecture. This



principle could be named (*earthquake resistant*) *structure as architecture* and enables a high intensity of development in both earthquake engineering and architecture (Lyll [8]). It is hardly possible to speak of influence on architecture, since this level is all about structure which *is* architecture (Figure 4). The author can be an engineer who uses structural design to also give a building its final form, or an architect with detailed knowledge of earthquake engineering, materials and structures.



Figure 3: Example of cooperation between architecture and earthquake engineering: Manantiales building, Chile (left), Wool House in Wellington (middle) and Union House in Auckland with added bracings (right).



Figure 4: Example of identification of architecture with seismic design: tectonic (trapezoidal) shape of Hancock Building in Chicago with visible bracings over the facade (left) and Dance centre Aix-en-Provence (right).

Concepts of earthquake protections in contemporary architecture also derive from ideas of bionics applicable to engineering and architecture. One of the most powerful tools nature has at its disposal to solve resistance problems in live organisms is force microfragmentation (Pioz in: Garcia [6]). The shift from metaphor of the machine to the metaphor of the organism is evident (Abley and Heartfield [1]). The aim of such an approach is to engage in a high level of

cooperation with engineers or use integral knowledge to design architecture which would be a synthesis of smart materials, form and structure. Some examples are given in Figure 5.

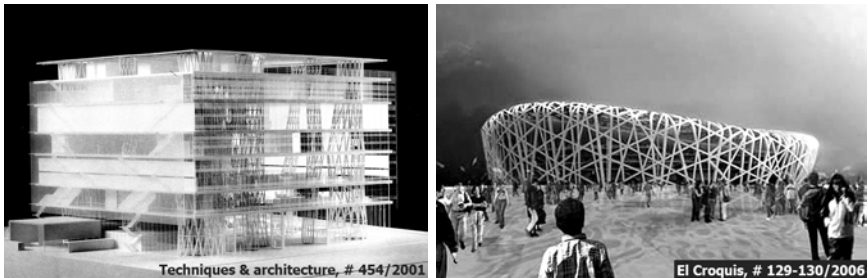


Figure 5: Example of “force microfragmentation”: Municipal multi-media library in Sendai and project of the Olympic stadium “Bird’s Nest” in China.

There are no clear divisions between the above mentioned levels of relations in earthquake architecture, which means that transitions from one level to another are sometimes possible in the process of architectural work in earthquake areas. With everything considered, it is important to distinguish between the actual effect architectural design has on horizontal resistance of a building and the symbolic or metaphorical reaction as a response of architecture – art to uncontrollable forces of an earthquake, which in some cases, due to irregularity and the desire to “provoke”, even causes weaknesses or conscious structural mistakes. In this case we speak of a negative version of relationship within earthquake architecture.

5 Conclusions

From the first preliminary results of review, analysis and evaluation of earthquake architecture we can make the following observations and conclusions:

- The response of architecture to earthquake threats can present an important source of a stronger architectural identity typical of earthquake prone regions.
- Earthquake architecture can be defined as any visual or conceptual inter-connection between the concepts of earthquake engineering and concepts of architecture.
- Looking at it visually, we can speak of *hidden and concealed* ways of earthquake resistant architecture on the one hand, and *revealed* or *emphasised* on the other. From the conceptual point of view, earthquake architecture is realized only by including the principles of earthquake engineering in the architectural concept itself.
- There is not much earthquake architecture in earthquake prone areas. Thus the possibility of using earthquake architecture as a form of expression remains unrealized potential.



- “Anti-” or “non-earthquake” architecture contradicts the earthquake reality by negation (*confrontation*) or ignorance (*indifference*) of the requirements of earthquake design. In this case the building code is the only tool that can prevent “anti-earthquake” architecture to be realised to the full extent in practice.
- Earthquake architecture is the “missing link” between earthquake engineering and architecture. It combines the best of both fields and establishes a new approach and quality in construction in earthquake prone areas, mainly in compliance with measures of architectural excellence.
- Further research and analyses of interconnections of architectural and earthquake resistant concepts within the field of earthquake architecture are planned to be conducted in the near future.

References

- [1] Abley, I., Heartfield, J., 2001: Sustaining architecture in the anti-machine age. Wiley-Academy, John Wiley & Sons Ltd. London.
- [2] Arnold, C., 1996: Architectural aspects of Seismic Resistant Design. Proceedings of the 11th World Conference on Earthquake Engineering.
- [3] Charleson, A.W., Taylor M., 2000: Towards an Earthquake Architecture, Proceedings 12th WCEE, NZ National Society for Earthquake Engineering.
- [4] Charleson, A.W., Taylor, M., Preston, J., 2001: Envisioning Earthquake Architecture in New Zealand, Proceedings of the Technical Conference of the New Zealand Society for Earthquake Engineering Annual Conference, Wairakei.
- [5] Constantionou, M., Soong, T. T., Dargush, G. F., 1998: Passive Energy dissipation systems for structural design and retrofit. MCEER, University of Buffalo, NY, USA.
- [6] Garcia, B., 2000: Earthquake Architecture, New construction techniques for earthquake disaster prevention. Loft Publications, Barcelona.
- [7] Kim, H., Roschke, P. N., Lin, P., Loh, C., 2005: Neuro-fuzzy model of hybrid semi-active base isolation system with FPS bearings and an MR damper. Science direct.
- [8] Lyall, S., 2002: Masters of Structure, Engineering Today’s Innovative Buildings. Laurence King Publishing Ltd, London.
- [9] Mezzi, M., Parducci, A. and Verducci, P., 2004: Architectural and Structural Configurations of Buildings with Innovative Aseismic Syst., Proc. of the 13. WCEE.
- [10] Reitherman, R., 1985: Earthquake Engineering and Earthquake Architecture. Part of the AIA “Workshop for Architects and Related Building Professionals” on Designing for Earthquakes in the Western Mountain States.
- [11] Yang, G., 2001. Large-scale magnetorheological fluid damper for vibration mitigation: modeling, testing and control. (dissertation) Graduate school of the University of Notre Dame, Indiana.



Section 2

Bridges

This page intentionally left blank

Aspects of testing a large-scale two-span bridge model on multiple shake tables

N. Johnson¹, M. Saiidi² & D. Sanders²

¹*Stantec Consulting, USA*

²*Department of Civil and Environmental Engineering,
University of Nevada, Reno, USA*

Abstract

A quarter scale model of a two-span reinforced concrete bridge was recently tested using the NEES multiple shake table system at the University of Nevada, Reno. The project was funded through a Network for Earthquake Engineering Simulation (NEES) demonstration grant. The prototype was designed using the provisions of the National Cooperative Highway Research Program document 12-49 for seismic design of highway bridges. The input shake table motions included the soil-foundation-structure-interaction effects. The bridge was designed for “life safety”. Test results demonstrated that the model met the performance objectives for both earthquakes. Additional analytical studies were conducted to evaluate the bridge model response for design spectra-compatible, synthetic ground motions.

Many important lessons were learned in the course of designing, constructing, testing, data interpretation, and extensive analytical studies that followed. These lessons demonstrated the system effects on individual piers and the structure, ramifications of multi-support excitation testing, performance under design earthquakes, effect of redundancy in the lateral loading system, and the effectiveness of existing analytical models in replicating the response. The presentation and the paper will provide the highlights of the experimental and analytical studies and a summary of important results and conclusions.

Keywords: shake table, bridge, earthquake, experimental testing, columns, reinforced concrete, NCHRP 12-49.



1 Introduction

A vast amount of experimental research has been concentrated on broadening technology to calculate the nonlinear response and understanding performance of highway bridges under earthquake loads. Past experiments have primarily focused on components of bridge systems to improve and validate modeling techniques, to test performance of new designs, or to evaluate old designs and to develop retrofit measures to improve response of existing structures that have insufficient details to adequately resist earthquake forces. However, due to limitations of earthquake testing facilities, and because system testing of bridges requires a large scale specimen, system tests have generally not been conducted.

The research that is presented in this document is part of a collaborative Pre-NEES study to investigate soil-foundation-structure-interaction (SFSI) of bridge systems. Data from this test has been used by research collaborators to integrate the bridge structural response into computer models to study SFSI. The major focus in this research was on the multiple shake table testing of a large scale reinforced concrete bridge system including the analytical modeling of bridges and investigation of bridge system response. Information in this paper presents highlights of the select portions of analytical and experimental studies of the shake table tests. Further information can be found in Johnson et al. [6].

2 Prototype design

Seismic detailing of the prototype was based on the Caltrans SDC [4] and NCHRP 12-49 *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges* [3]. General design of the prototype was based on the American Association of State Highway and Transportation Officials AASHTO LRFD bridge specifications [1]. The bridge specimen (fig. 1), which was composed of 11 major components, was designed to model the system interaction between three two-column bridge bents of varying heights. It was created at quarter scale to maximize the size of the specimen while remaining below the capacity of the shake tables.

The total height of the specimen to the top of the superstructure was 3.28 m; the total length was 20.5 m. Span lengths were 9.14 m and the columns of the three bents had clear heights of 1.83 m (bent 1), 2.44 m (bent 2), and 1.52 m (bent 3) with the tallest bent in the middle. The superstructure was composed of a solid slab that was post-tensioned in both the longitudinal and transverse direction of the bridge. It was designed to maintain generally un-cracked stiffness properties throughout the tests and its stiffness matched the stiffness of the prototype about both bending axes. Due to the scaling effect, masses of the quarter scale model provide a smaller axial stress than in the prototype scale. Some of the required axial load was provided by the self weight of the bridge model. The rest was provided by superimposed dead load that was attached to the top of the superstructure.



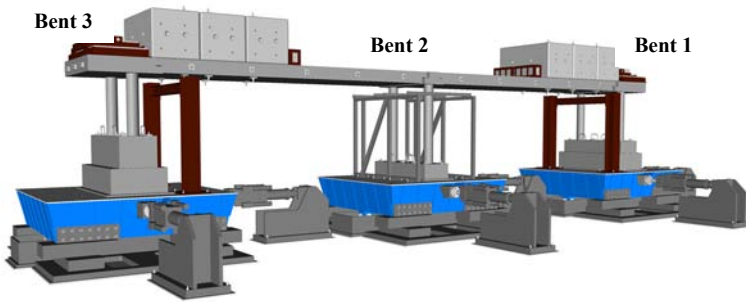


Figure 1: Rendering of bridge model on shake tables.

2.1 Seismic design

The demand in the NCHRP design code is site specific. Two performance levels are specified in NCHRP 12-49: life safety and operational. The prototype bridge was considered to be a non-essential (not a critical lifeline) bridge. Therefore the life safety performance level is the level that was considered for the design of the bridge model. Life safety is the minimum performance level that is allowed in the specifications and is intended to protect human life during and after a rare earthquake. For both the rare and expected design earthquakes, two performance level categories must be met for the life safety performance to be satisfied: service level and damage level.

For the *expected* earthquake, which is what is to be expected during the life of the bridge, the service level that must be satisfied is “immediate” use and the damage level is “minimal”. Normal bridge operation can take place after post-earthquake bridge inspection. The minimal damage level permits limited damage to the columns including narrow flexural cracking and slight inelastic response. The columns should be completely repairable under non-emergency conditions. For the *rare* earthquake, which is the maximum considered earthquake, the expected service level is “significant disruption” and the damage level is “significant”. Limited post-earthquake access may be possible, however, the bridge may need to be replaced. Cracking, reinforcement yield, and major concrete spalling may take place and replacement of the columns may be necessary. However, the bridge should not collapse.

2.2 Seismic detailing

The lateral reinforcement consisted of spiral steel that was continuous throughout the height of the columns. For comparison, the lateral steel was designed using four bridge design codes: NCHRP 12-49 [3], Caltrans SDC [4], AASHTO Standard Specifications [2], and AASHTO LRFD [1]. All of the design codes contain two basic design requirements, confinement reinforcement to increase plastic hinge rotation capacity, and shear reinforcement to prevent shear failure. The spiral reinforcement was first designed based on confinement requirements and then was checked to ensure sufficient shear capacity.



The design that was implemented in the shake table model was that resulting from NCHRP 12-49. Reinforcement to prevent longitudinal buckling, which could be considered as a type of confinement, controlled the design of all three columns. The amount of reinforcement that was included as spirals in all three sets of the columns provided a lateral reinforcement ratio of 0.009. For comparison, requirements for confinement controlled all three of the columns considering both of the AASHTO codes, and controlled the tallest column considering the Caltrans SDC. For the short and medium height columns, Caltrans lateral reinforcement requirements were controlled by shear.

Table 1: Shake table tests and achieved bend displacement ductility.

Test	Target motion (g)	Bent 1 μ_A	Bent 2 μ_A	Bent 3 μ_A
1-11	0.18	0.29	0.15	0.37
12	0.075	0.35	0.16	0.24
13	0.15	0.95	0.41	0.61
14	0.25	1.13	0.54	0.97
15	0.5	2.33	1.20	2.86
16	0.75	3.93	2.20	3.68
17	1	2.95	1.87	2.79
18	1.33	4.14	3.23	6.46
19	1.66	5.25	4.10	9.22
20	1	3.35	2.96	6.81

3 Shake table motions

Both low and high amplitude testing was conducted on the bridge model. Earthquake motions that were used were calculated based on the measured records at the Century City Country Club from the 1994 Northridge, California earthquake. The low amplitude tests included transverse coherent and incoherent, and biaxial coherent target motions (tests 1-11). Low amplitude motions were such that the longitudinal reinforcement in the columns did not yield. High amplitude tests (tests 12-20) were a transverse coherent motion that was applied in increments from a pre-yield demand (0.075g PGA) until bent failure (1.66g PGA) when the shortest of the bents failed in flexure from crushing of confined concrete and buckling of longitudinal reinforcement. After failure of the first bent, an additional 1g motion was applied to the bridge (test 20). The additional motion caused only limited additional damage.

4 Analytical models

The goal of the analytical modeling was twofold: the first was to determine the validity of contemporary analytical modeling in duplicating the response of the bridge throughout the range of damage states; the second was to develop a computer model to use for further study.



Both SAP2000 version9 [5] and Drain-3DX [8] were used in the analytical modeling. Standard methods using nonlinear fiber elements were employed to define column nonlinearity using the measured material properties of the bridge. The achieved shake table motions from high amplitude tests were input to the models in order to capture low amplitude through failure response. Because of the achieved motion incoherency, the measured accelerations from the shake tables were filtered and double integrated for input to the computer models.

There are two fundamental differences between the SAP2000 and Drain-3DX models. The first difference is in the nonlinear elements of the columns (described below). The second is in the method of integration used to solve the forces and displacements. The integration method used for SAP2000 was the Newmark beta method using average acceleration and a relative convergence tolerance of 10^{-7} . Drain-3DX uses a more direct non-iterative method to calculate the structural response through time. Rather than iterating force for convergence, the force error is applied to the next step. This method requires small time step, but is more stable than traditional iterative methods [8]. For similar convergence of results, the Drain-3DX model was approximately 22 times faster.

There are three primary differences between the nonlinear elements of the columns for SAP2000 and Drain-3DX. The first is that the fiber nonlinearity in the SAP2000 model was lumped at the center of each hinge length. For the Drain-3DX model the moment-curvature relationship from the fiber section was integrated over the hinge length which had a parabolic distribution of curvature. The second is that for fibers in the SAP2000 model, all materials were specified with strength degradation upon failure. In the Drain-3DX model, because strength loss is not permitted in the constitutive relationship for steel, strength degradation upon material failure was only specified for the concrete. Therefore, the drain model did not account for rupture of the longitudinal reinforcement. The final difference between the column elements is for the SAP2000 model, bond slip was specified as part of the steel material properties. For the Drain-3DX model, a specific zero-length fiber element that explicitly defined bond slip accounted for concrete gap opening in tension, slip of the reinforcement, and compression of concrete into the connection was included at the column ends.

It was concluded that the Drain-3DX results provided the best match to the measured structural response of the model due to a more refined distributed plasticity fiber element and an element that explicitly modeled reinforcement bond-slip. Because of the good correlation with the measured results and more efficient computation of the Drain-3DX model, it was used to conduct parametric studies of the bridge response.

5 Performance

5.1 Measured performance

The observed and measured response of the shake table bridge model led to two conclusions with regards to performance. The first was that the modeling



technique that was used for testing was successful. The second is that the columns, which were designed according to contemporary earthquake design code performed well.

The shortest of the columns, which were located in bent 3 and had an aspect ratio of only 2.5, showed no signs of shear distress from the pre yield state through failure in flexure other than minor shear cracking. The confinement reinforcement in bent 3 provided adequate confinement stress on the concrete and lateral support of the longitudinal reinforcement to delay buckling of the longitudinal reinforcement, so that the bent could reach a displacement ductility of 7.6 (8.9 using pushover calculated yield displacement) before failure.

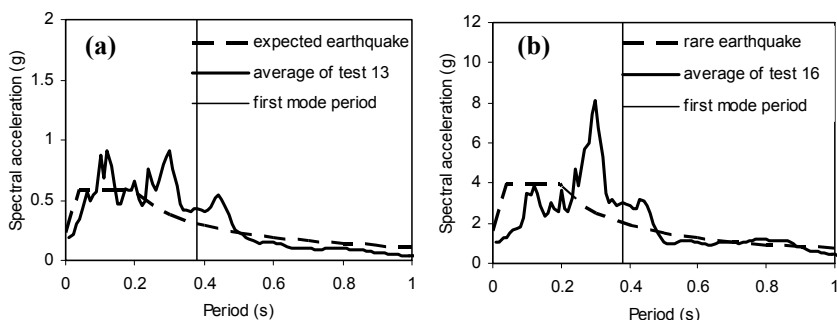


Figure 2: Shake table test motions compared to design spectra for (a) expected event and test 13, and (b) rare event and test 16.

To evaluate the measured response of the shake table bridge model with respect to the design spectra, the tests having achieved shake table accelerations that had approximately the same spectral acceleration at the calculated natural period of the bridge in the transverse direction were determined (fig 2). For test 13 (fig 2(a)), with spectral accelerations that were conservatively equivalent to the amplified expected design earthquake at the first two transverse model frequencies, the maximum displacement ductility was 0.95 in bent 1. Since none of the bents reached yielding during this test and damage was negligible, the service level performance objective of the NCHRP requirements for the expected earthquake, which was “immediate”, as well as the damage level performance objective for the expected earthquake, which was “minimal”, were both satisfied. The plastic rotational capacity for the immediate use performance level is 0.01 radians. Based on the curvature measurements from tests, the maximum rotations measured at the plastic hinge regions was only 50% of the immediate use capacity.

Test 16 (fig 2(b)) was shown by the response spectra to contain spectral accelerations that are conservatively equal to that of the rare design earthquake at the first two transverse modal frequencies of the bridge. All of the bents underwent yielding during test 16. The maximum bent displacement ductility demand was in bent 1, and was 3.31. The failure displacement demand, which was defined as the ratio of maximum displacement over failure displacement for



each bent was also the largest in bent 1 and was 0.41. No sign of loss in lateral force capacity was seen, and the column deflections were well below failure. Therefore, life safety performance requirements of the rare event, which were a service level of “significant disruption”, and a damage level of “significant”, were fulfilled by a relatively high margin. For NCHRP 12-49, the plastic rotational capacity for the life safety performance level can either be calculated from an equation or can be assumed to be 0.035 radians. Based on the curvature measurements from tests the largest ratio measured rotation demand over rotational capacity for the life safety requirement assuming is 0.820 for bent 1.

5.2 Analytical performance

Artificial motions that matched the rare and expected earthquake design spectra were calculated and were input to the Drain-3DX model of the bridge specimen to provide a more direct comparison of the bridge response due to design motions so that the response of the bridge could be compared to the performance criteria.

For the expected event, none of the bents underwent significant yielding. No significant hysteretic energy was dissipated in any of the bents. The largest displacement ductility demand was 1.35 for bent 1. The largest ratio of the calculated rotational demand over immediate use performance criteria capacity was 0.59. Since the bents underwent only limited inelastic response and the calculated rotation demands at the column ends were well below the immediate use performance capacity, the bridge performed well and conformed to the performance requirements of the expected design earthquake.

For the rare event, all of bents underwent significant yielding. The largest ratio of displacement demand divided by the calculated ultimate displacement capacity was 0.69 for bent 1. No sign of reaching the lateral force capacity was calculated, and the column deflections were well below failure displacement. Therefore, the life safety performance requirements of the rare event, which were a service level of “significant disruption” and a damage level of “significant”, were satisfied. The largest ratios of the calculated rotational demand over life safety performance criteria using the 0.035 radian capacity and code equation were 0.58 and 0.64 for bent 3, respectively.

6 System effects

6.1 System vs. individual response

To determine the system effect on the shake table model, the response of the complete bridge model and individual bents was studied analytically. The Drain-3DX analytical model was used to calculate the response for test motions 13 through 19 of the complete bridge and of the individual bents having tributary mass. This provided a comparison of bent response for component testing on a single shake table, with response from system testing on multiple shake tables. The damage index, which was developed by Park and Paulay [7],



was the primary method of comparison. It is an empirical measure of damage based on a combination of amount of dissipated hysteretic energy and the maximum displacement demand over ultimate displacement ratio. A damage index of zero indicates no damage; one indicates a high probability of failure.

After yielding, each of the three bents followed a general trend of system vs. individual damage. The system damage index demand in bent 1 exceeded the individual demand by as much as 41%. In bent 2, the system damage index demand was less than the individual demand by as much as 38%. The damage index demands for system and individual response were approximately the same for bent 3 with a maximum system demand difference of 8.4% after yielding.

6.2 Additional systems studied

Four additional systems having a constant stiffness index (sum of lateral column stiffness) were analyzed using the Drain-3DX model and compared to the system response of the shake table model. The aspect ratios of the columns were within the same range as those tested on the shake tables. The systems included a system with uniform column height, symmetric with a stiff center bent, a symmetric version of the test specimen, and an asymmetric system with a stiff center bent. For comparison, both displacement ductilities and damage indices were calculated for the system and individual bent response of the models subjected to both the rare and expected design motions.

Significant system effects were apparent in the bridges that were analyzed. The maximum system/individual damage ratio on the five systems for the expected earthquake motion, which placed demands on the columns in the systems near column yielding, was 1.57. The maximum system/individual ratio for the rare motion, which imposed demands far greater than yielding in the columns, was 1.32.

6.3 System redundancy

The failure of an interchange bridge at junction of I5-SR14 during the 1994 Northridge was caused by large variation among the column heights that led to high concentration of shear in one of the columns and its failure. This led to the conclusion that to avoid this type of behavior, the column heights need to be the same in the replacement bridge [9]. Many bridge designers tend to follow a design methodology for earthquake resistant bridges to design a bridge so that if possible it is symmetric and uniform to avoid irregular system response. Therefore it is desirable to design bridges that have uniform column height. As long as earthquake demands on the columns for this type of bridge do not fail the columns, the bridge remains intact. However, if the columns reach their failure displacement, then the entire system will fail due to lack of substructure redundancy.

A comparison was made between calculated ductility demands on the columns for the uniform height bridge and shake table bridge specimen for the test motions for the design motions (expected and rare), respectively. For the rare design motion, the maximum displacement ductility demands on the



specimen and uniform systems are 5.59 and 5.35, respectively. The specimen has a maximum displacement ductility demand that is approximately 5% larger than that of the uniform system. It should be noted that there was only a 5% increase in displacement ductility demand from the rare design earthquake for the non-symmetric system (in comparison to the uniform case), which was shown in tests to provide redundancy in column capacity after failure of the most critical bent. Although this study was limited it does suggest that providing redundancy by varying the column heights might be a better alternative than making the column heights the same.

7 Conclusions

The following are conclusions from the selected portions of experimental and analytical studies that were presented in this paper.

- (1) The amount of lateral reinforcement provided by the NCHRP 12-49 that is recommended for preventing global buckling of longitudinal reinforcement in the plastic hinge zone was adequate to prevent buckling until large displacement ductility was reached.
- (2) The flexural failure of the column with the smallest aspect ratio (2.5) showed that the Caltrans and NCHRP 12-49 seismic detailing requirements for shear reinforcement were adequate.
- (3) The bent with the shortest columns failed when the bridge was subjected to a 1.66 PGA ground motion. Although this bent had failed, the remaining two still provided sufficient redundancy and capacity to withstand a 1.0 g PGA motion that followed.
- (4) Available analysis tools using conventional methods were successful in estimating the nonlinear response of a concrete bridge structure with flexure dominated columns from the pre-yield state up to failure. The Drain-3DX model which explicitly included bond slip and incorporated a more efficient integration method better matched measured results and was therefore used for further study.
- (5) Analytical modeling using the design motions showed that the maximum column ductility demand for the rare design earthquake was merely 5% larger for the specimen than for a uniform height column system. This small increase is offset considerably by the increased redundancy of a system with variable height columns.
- (6) For the bridge that was tested in this study, system effects did not increase the amount of hysteretic energy dissipation or large displacement cycles on the columns for given values of achieved displacement. However, for specific motions, the system effect caused significant differences in damage to the bents. For post yield motions, the system effect on the damage indices ranged from a decrease of 39% to an increase of 41%.



Acknowledgements

This research was sponsored by the National Science Foundation through NEES award number CMS-0324326. The NSF program directors were Steven McCabe and Joy Pauschke. The study was part of a multi-institution project under the overall direction of Sharon Wood of the University of Texas, Austin. The authors are indebted to the dedicated support of Patrick Laplace and Paul Lucas of the UNR structures lab in the course of the shake table studies.

References

- [1] AASHTO LRFD Bridge Design Specifications. AASHTO, Washington D.C., 1998.
- [2] AASHTO Standard Specifications for Highway Bridges, 17th edition. AASHTO, Washington D.C., 2002.
- [3] ATC/MCEER Recommended LRFD Guideline for the Seismic Design of Highway Bridges (2001) Part 1: Specifications, MCEER-02-SP01, MCEER/ATC joint venture, NCHRP 12-49 Project Team., 2001.
- [4] Caltrans (California Department of Transportation), Caltrans Seismic Design Criteria Version 1.3. Engineering Service Center, Earthquake Engineering Branch, California, 2004.
- [5] CSI, Inc., "SAP2000 Linear and Nonlinear Static and Dynamic Analysis and Design of Three-Dimensional Structures," version 9. Berkeley, CA., 2005.
- [6] Johnson, N., Saiidi, M., and Sanders, D., "Large-Scale Experimental and Analytical Seismic Studies of a Two-Span Reinforced Concrete Bridge System", Civil Engineering Department, University of Nevada, Reno., 2006.
- [7] Park, R., and Paulay, T., "*Reinforced Concrete Structures*," Wiley Interscience, 1975.
- [8] Prakash, V., and Campbell, S. "Drain-3DX: Static and Dynamic Analysis of Inelastic 3D Structures", Department of Civil Engineering, University of California, Berkeley, 1994.
- [9] Saiidi, M., R. Moore, and A. Itani, "Seismic Performance of Reinforced Concrete Bridges With Un-Conventional Configurations," American Concrete Institute, *Structural Journal*, pp. 717-726, September, 2001.



Seismic devices for bridges

D. Mestrovic & G. Grebenar

Faculty of Civil Engineering, University of Zagreb, Croatia

Abstract

The paper summarizes earthquake calculation in accordance with Eurocode 8, ACI regulations and codes applied in Croatia (Europe). Basic data about earthquakes are given and an overview of seismic devices is presented. Calculations for Zeceve Drage (Croatia) bridge are made. Time history analysis was calculated for accelograms based on earthquakes occurring in Petrovac (1979), with magnitude of 6.8, and in Ulcinj (1979), with magnitude 5.3, (former Yugoslavia). New methods of structure protection with dampers were applied on the bridge. Spectrum analysis based on EC8/2 for ground acceleration of 0.19g for the past period of 500 years, and time history analysis for ground acceleration of 0.25g and for the past period of 1000 years was used. The damper effect on a bridge with two spans was tested in a laboratory of Civil Engineering Faculty of Zagreb University, Croatia.

Keywords: seismic devices, damper, elastomer bearing.

1 Introduction

When designing structures in seismically active regions, it is essential to know characteristics of ground motion. Regulations usually determine maximum effective ground acceleration. The earthquake ground shaking is usually presented in the form of a response spectrum of acceleration. That acceleration is actually an attempt to describe potentially destructive ground motion. Generally, acceleration tends to be equal with actual maximum ground acceleration resulting from an earthquake.

Site conditions of soil, such as its type and load bearing capacity, are also important. If short periods prevail, structure is rigid and founded on rocky soil, then such structure has small natural periods and earthquake would have catastrophic effects on this kind of structure. And if a structure is slender and founded on soft ground and subjected to earthquake with predominant longer



periods, the earthquake effects on the structure could be catastrophic. The earthquake effect in a particular region is determined by its intensity and strength. Earthquake is phenomenon of energy. To avoid damage leading to collapse of a structure, dissipation of energy should be as high as possible. This is achieved by ductility of particular elements of the structure, and recently, by modern devices.

2 Modern seismic devices

Earthquake effect on the structure represents action of inertia forces caused by ground motion. How great these forces will be depends on seismic excitation and natural periods of the structure. The force value can be limited by softening of structure, while a degree of damage is lowered by stiffening the structure. Since these two requirements are contradictory, the use of anti-seismic devices has increased, because their application ensures elastic behavior of structural elements. Although the new methods of structure protection were proposed almost a century ago, until today only a small number of examples were applied in practice. This can be explained, firstly, by difficulty in adapting the existing seismic regulations to actual execution of seismically resistant structures, and secondly, by shortage of appropriate devices which would enable safe and effective construction of seismically protective systems. Dimensioning of structure in case of earthquake refers to stiffening or softening. Stiffening can be permanent or temporary. Permanent stiffening is achieved by greater dimensions of a structure, and temporary stiffening by shock transmitters. Softening is achieved by isolating a structure and by dissipation of energy (T and Y strategy). T-strategy represents increase of natural periods, and Y strategy limits the forces transmitted between superstructure and piers. T-strategy is achieved by elastomers, as well as by highly dampening elastomers, and Y-strategy, by hysteresis or hydraulic dampers. Dissipating isolators combine the above. Bridges can be protected from earthquake by selection of proper equipment. Most significant pieces of equipment are: fixators, base isolators, isolators with dissipating effect, friction pendulum, seismic slide isolators, bearings with frequency converters, shock transmitters, dampers: hydraulic, hysteresis and dampers with tuned mass, and expansion joints. Fixators are foreseen to transmit a given force without displacement. They are known as fixed bearings. By contrast, elastomers enable floating support for superstructure, and are also called base isolators. The base isolators increase natural period of a structure, which result in reduction of acceleration during seismic attack. They have effect of a spring which makes a structure to return into its original position. Dissipating effect of isolators is achieved when elastomer is furnished with lead core which dissipates energy. Following types of bearings are developed especially for earthquake: friction pendulum, seismic slide isolator and bearing with frequency converters. Friction pendulum consists of bearing slab with sphere and one slide cladding made of polished stainless steel. Seismic steel slide isolator transmits vertical loading and ensures free horizontal flexibility. It has recentering capacity, and can have high dampening properties. Departure from



resonant area is achieved by bearings with frequency converters. Shock transmitters are important for transmission of horizontal force. They are hydraulic devices which decrease quick movement of superstructure as compared to substructure, which result in previously foreseen force. Shock transmitters produce temporary connections which become active only during dynamic excitations. Dissipation of energy is produced by dampers. The most important types are: hydraulic, steel and dampers with tuned mass. Figure 1 represents force-displacement relation for typical shock transmitter.

Viscous dampers are devices that enable displacements due to temperature changes, creep and shrinking, but do not create considerable forces, however, they dissipate great quantities of energy during sudden dynamic entrance of seismic energy, and that energy is transformed to heat. Steel hysteresis damper dissipates energy using property of steel fluctuation. Damper with tuned mass is installed at the structure point that has significant or highest vibration level. The device consists of moving/swinging mass, spring and damping element. Advantage of the device is that it shifts structure frequency from resonant area. Very important are expansion joints. They are designed in such a way to take seismic displacements, in addition to service displacements.

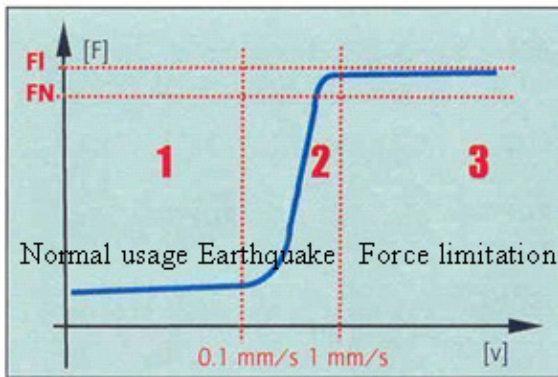


Figure 1: Force-displacement relation for shock transmitters.

3 Regulations

Majority of regulations take into account the following: seismicity factor, dynamic factor, factor dependent on soil category, factors of damping, structure, risk, and importance of a structure. The earthquake effect is described by effective maximum ground acceleration.



Table 1: Q factor.

Ductile member	Post-elastic behavior	
	Limited ductile	Ductile
<i>Reinforced concrete columns</i>		
Vertical columns - bending	1.5	3.5
Bent column - bending	1.2	2.0
Short strong column	1.0	1.0
<i>Steel columns</i>		
Vertical column - bending	1.5	3.0
Bent column - bending	1.2	2.0
Natural support - column	1.5	2.5
Eccentric support - column	-	3.5
<i>Abutments</i>	1.0	1.0
<i>Arches</i>	1.2	2.0

Table 2: R factor.

SUBSTRUCTURE	R
Wall-type pier	2.0
Reinforced concrete pile bents	
a) only vertical piles	3.0
b) one or more batter piles	2.0
Single columns	3.0
Steel or composite steel and concrete pile bents	
a) only vertical piles	5.0
b) one or more batter piles	3.0
Multiple column bents	5.0
CONNECTIONS	1.2
Superstructure to abutment	0.8
Expansion joints within a superstructure span	0.8

In order to avoid explicit non-linear analysis, and taking into account capacity of a structure to dissipate energy through ductile behavior of its members, and also by other mechanisms, a linear analysis is applied based on a response spectrum which is reduced as compared to the elastic spectrum. Because of that the reduced spectrum is called design spectrum. Design response spectrum is obtained from elastic by means of behavior factor “q”. Behavior factor “q” is approximation of value of the seismic forces which would affect a structure, if its response is completely elastic, with 5% of relative viscose damping, and of



minimum seismic forces that can be used for designing with conventional linear model and which would at the same time ensure a satisfactory response of a structure. Value for “ q ” is given in a table 1. Values given in the table can be applied only to accessible plastic hinges. When a superstructure leans against columns of various degrees of stiffness, then a lowest “ q ” value is selected for calculation purpose, while $q = 1.0$ is used for calculations of columns with elastomers. Seismic forces for given columns and connections are determined by dividing elastic forces by appropriate Modification Factor (R). Values for R are given in the table 2. For EC8/2 it should be noted that structure ductility is described by behavior factor “ q ”, and for ACI regulations modification factor R is used. Regulations valid in Croatia do not explicitly require checking of ductility, property essential for dissipation of seismic energy, which increases a risk of load bearing capacity and applicability of reinforced structures calculated according to such regulations.

4 The bridge Zeceve Drage

Zeceve Drage bridge is 940,8 m long. It is situated in a horizontal curve. The height difference between left and right abutment is approximately 23 m. Span structure is a box girder 12,5 m wide and 4 m high. The area of cross section above the support is 10,2 m², and in the field it is 9,4 m². The piers have rectangular cross section at the top, which changes into hollow, with 30 cm thick wall, expanding to 50 cm at the bottom. The bridge has 18 piers, the highest being approx. 53 m high. The spans are 50 m, except for the ones closer to the abutment, which are 40 m. Span structure was modeled using shell elements, piers are modeled as beams. Supports are modeled using spring elements. Connection pier-span structure was modeled with coupling elements. Material for span structure was concrete C45, piers are C35 and structural steel is S400 (according to Eurocode 2 regulations). All loads applied on bridge for purpose of seismic calculations are used according to Eurocode 8 regulations.

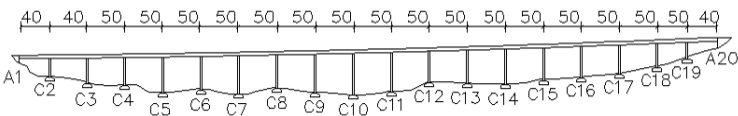


Figure 2: Longitudinal section of the bridge.

5 Results of analysis

Two analyses were used for earthquake calculation – spectrum analysis based on EC8/2 for ground acceleration of 0,19g for the past period of 500 years, and time history analysis for ground acceleration of 0,25g and for the past period of



1000 years. Spectral analysis was conducted for elastic and design spectrum, with $q = 1,5$ (limited ductile behavior). Time history analysis was calculated for accelograms of magnitudes 5.5, 6, 6.5 and 7, with distances of 0 and 15 km from epicenter, based on earthquakes occurring in Petrovac, in April 1979, with magnitude of 6.8, and Ulcinj, in April 1979, with magnitude 5.3, in the region of former Yugoslavia. Thirty (30) modes were used for calculations. The hydraulic viscous dampers of 1500, 2000 and 3000 kN were added in longitudinal direction connecting the abutments. The dampers of 2000 kN were selected because they enable elastic behavior of the piers. The table 3 represent results of forces in cross section at the piers bottom.

Table 3: Calculation results (shear forces in kN) of Zeceve Drage bridge - columns S8 to S10.

Column	S8	S9	S10
Elastic spectrum	4354	5597	2567
Design spectrum	2885	3709	1701
5.5M 0 km	231	298	141
5.5M 15 km	313	402	184
6.0M 0 km	460	592	276
6.0M 15 km	350	450	210
6.5M 0 km	442	567	264
6.5M 15 km	546	704	331
7.0M 0 km	1507	1936	889
7.0M 15 km	1005	1292	582
DAMPER 1500 kN 7.0M 0 km	843	1081	452
DAMPER 2000 kN 7.0M 0 km	657	840	356
DAMPER 3000 kN 7.0M 0 km	467	594	283

6 Laboratory testing

In laboratory the damper effect on a bridge with two spans and elastomer bearings was tested, and the results were compared with data obtained by calculations. For bridge with damper in longitudinal direction, the experiments confirmed results obtained by calculations.





Figure 3: Model of bridge with dampers.

Table 4: Frequencies and periods (results of experiments and calculations).

	Structure without damper		Structure with damper (50 N)	
axis	Frequency (Hz)		Frequency (Hz)	
	computed	experiment	computed	experiment
x	105.39	99	122.48	118
y	107.96	99	121.32	127

Table 5: Damping of bridge model.

Axis	Without damper	With damper (50N)
x	9.8%	14.89%
y	10.6%	12.47%

7 Conclusion

Selection of adequate equipment is necessary for the bridge protection. Today, dampers are being used very often. They reduce the seismic forces, increase damping of structure and enable uniform distribution of induced energy over the entire structure. This paper focuses on their greatest importance, reduction of the forces in piers by damping, during which they remain in elastic state. The bridge piers have elastic behavior, so that no significant structure damages occur during earthquake.



References

- [1] Chopra, A. K., Dynamics of Structures, Theory and Applications to Earthquake Engineering. New Jersey: Prentice Hall, 1995.
- [2] Clough, R.W., and Penzien, *Dynamics of Structures*, New York: McGraw – Hill, 1993.
- [3] Petersen, C., *Schwingungsdaempfer im Ingenieurbau*, Muenchen: Maurer Soehne GmbH & Co. KG, 2001.
- [4] Nizic, A., Seismic devices in bearing structures, M.Sc. Degree thesis, 2004
- [5] D. Cizmar, A. Nizic, D. Mestrovic, “Importance of dynamic characteristics of accelograms to structural response”, SECED Conference, 2005.
- [6] A. Mihanovic, “Dynamics of structures”, University of Split – Faculty of Civil Engineering, 1995.
- [7] SeismoSoft [2004] “SeismoSignal - A computer program for signal processing of strong-motion data” [online]. Available from URL: <http://www.seismosoft.com>.
- [8] S.R.A.C. inc., “COSMOS/M 2.6 Electronic Documentation”, ASTAR – Advanced Dynamics, p. 166-175, Los Angeles, 2000.
- [9] C. I. Huerta Lopez, Y. Shin, E. J. Powers, J. M. Roesset, “Time frequency analysis of earthquake records”, 12WCE2000, 2000.



Section 3
Seismic isolation
(Special session by
P. Komodromos and
M. C. Pochas)

This page intentionally left blank

Seismic isolation and energy dissipation: worldwide application and perspectives

A. Martelli

ENEA, Bologna, Italy

Abstract

At present there are approximately 5,000 structures in the world which are protected by seismic isolation (SI), energy dissipation (ED) and other modern seismic vibration passive control (SVPC) systems, such as shock transmitters (STs) and shape memory alloy devices (SMADs), and the number of such applications is increasing more and more. The conclusive influence of the features of the design rules used on the extension of application of the SVPC systems is evident. With regard to such an application, Japan has consolidated its worldwide leadership, with over 3,000 seismically isolated buildings in October 2006 and many others protected by ED systems. The Russian Federation remains second for the number of isolated buildings (550 in June 2005). Third, with 490 isolated buildings, is the P.R. China. In the USA, due to the very penalizing design code in force for SI of buildings, there are now only a few new applications of this kind (their overall number is approximately 200, although they are mostly quite important, half being retrofits). At present Italy (which is still worldwide leader as to SVPC application to bridges and viaducts) remains fifth for the number of isolated buildings already opened to activity: 43, besides 19 protected by ED or SMADs and 28 by STs. However, there is a significant increase of the number of Italian building applications of the SVPC systems completed in the last two years and of that of new projects: this occurred thanks to the new national seismic code, enforced in May 2003. As to other countries, of note are the growing use of SVPC in Taiwan and of SI of buildings in Armenia and New Zealand. Furthermore, important applications also began in Turkey, Greece, Portugal and Cyprus (many thanks to devices manufactured in Italy) and are going on in France (in particular in La Martinique island) and Chile. To be stressed are also the increasing use of SI for liquefied natural gas tanks and nuclear structures and the already significant application of the SVPC to cultural heritage, especially in Italy.

Keywords: passive control of vibrations, seismic isolation, energy dissipation, shape memory alloys, shock transmitters, seismic retrofit, buildings, bridges and viaducts, cultural heritage, liquefied natural gas tanks, nuclear reactors.



1 Introduction

At present, there are approximately 5,000 structures in the world that are protected by seismic isolation (SI), energy dissipation (ED) and other modern seismic vibration passive control (SVPC) systems, such as shock transmitters (STs) and shape memory alloy devices (SMADs); the number of these applications is increasing more and more. The conclusive influence of earthquake experience and availability and features of the design rules used in each country on the extension of application of the SVPC systems in such a country has been confirmed by the recent data. This paper very briefly summarizes the application of the aforesaid systems worldwide (based on the data provided to the authors by other *ASSISI* and GLIS members), by stressing the progress of such an application in Italy. Details on this subject may be found, for instance, in the book of Dolce *et al.* [1].



Figure 1: Ojiya City building that withstood the 2004 Mid Niigata quake without any damage; its rubber bearings and SDs.



Figure 2: The 87.4m high building, which was seismically isolated at Tokyo in 2000 (it was the first Japanese application of SI to high-rise buildings).



Figure 3: Sketch of the complex of twenty one 6- to 14-storey buildings, all erected on an isolated artificial ground at Sagamihara (Tokyo area) with LRBs, Sliding Devices (SDs) and Ball Bearings.

2 Application in Japan

Japan, thanks to the availability of an adequate specific code since 2000 and the free use of SI since 2001, has consolidated its worldwide leadership (with over 3,000 isolated buildings in October 2006), by continuing the extensive adoption of the SVPC systems which had been initiated after the excellent behavior of two isolated buildings near Kobe during 1995 Hygo-ken Nanbu earthquake and was later confirmed by all Japanese isolated buildings struck by subsequent



earthquakes (fig. 1). In the aforesaid country, where the first application of base SI dates 1985, the trend is now to isolate, on the one hand, even high-rise buildings (fig. 2) and sets of buildings supported by a common isolated reinforced concrete (r.c.) structure (the so called “artificial ground”, a solution which enables large savings of construction costs – see fig. 3) and, on the other hand, very small private houses. Furthermore, several Japanese buildings have been protected by various kinds of dampers: for instance, the applications of the Buckling-Restrained Braces (BRBs) were already over 250 in 2003.

Finally, the use of the SVPC systems recently increased in Japan for bridges and viaducts. The latter began there rather later than for buildings; it is being largely based on the use of High Damping Rubber Bearings (HDRBs) and Lead Rubber Bearings (LRBs) and considerably extended especially after the 1995 Hygo-ken Nanbu earthquake (by beginning obligatory for overpasses in Kobe).

3 Application in the Russian Federation

The Russian Federation remains second for the number of isolated buildings (550 in June 2005). The use of modern SI systems (namely HDRBs), similar to those adopted in the other countries, is now replacing that of the previous “low cost” isolators, which had been installed since the years 1970s. Recent Russian application includes retrofit of important historical buildings (figs. 4–6) and new designs concern even high-rise buildings.



Figure 4: The Irkutsk City Central Bank that was retrofitted with HDRBs.



Figure 5: National Drama Theatre at Gorno-Altai retrofitted with HDRBs and visco-elastic dampers (VEDs).



Figure 6: The Mihailo-Arkhangelskaya church at Irkutsk City, which was retrofitted by means of HDRBs.

4 Application in the People's Republic of China

Third at worldwide level as to the use of SVPC systems (with 490 isolated buildings, including 270 masonry ones, in June 2005) is the People's Republic of China, where there has been a significant increase of the number of applications for some years, in particular to dwelling buildings (figs. 7 and 8), and large works are going on, such as those concerning the 50 isolated buildings of the new residential center of Peking (fig. 9). In this country there are also very old buildings protected by rough SI systems, but the use of the modern ones began only in 1991.





Figure 7: R.c. dwelling building at Shantou that was the first Chinese application of HDRBs (1991). It withstood a significant earthquake with no damage in 1994.



Figure 8: The complex of 60 new masonry dwelling buildings that was erected in Western China with HDRBs in 1996.



Figure 9: The “Isolation House Building on Subway Hub”, the new residential centre of Peking formed by 50 7–9-storey buildings ($480,000\text{ m}^2$) supported by a unique 2-storey ($1500\text{ m} \times 2000\text{ m}$), containing all facilities and been isolated by means of HDRBs.

5 Application in the USA

In the USA, the application of the SVPC systems to bridges and viaducts and, for dampers, also to buildings, is still progressing satisfactorily.

However, in spite of the excellent behavior of some US important isolated buildings during the 1994 Northridge earthquake and long application experience (since 1985), there is now only a limited number of new applications of this kind, due to the very penalizing design code in force for the isolated buildings: according to recent information, the U.S. seismically isolated buildings are now “only” approximately 200, although they are mostly quite important and half of them are retrofits (figs. 10–12).





Figure 10: The University of Southern California Hospital at Los Angeles protected by LRBs and completed in 1991, that withstood the 1994 Northridge earthquake without any damage, contrary to other conventionally founded hospitals located in the neighbourhood (e.g. the Olive Hospital, which had just been repaired after suffering severe damage during the San Fernando earthquake).



Figure 11: San Francisco City Hall, erected in 1912, which was damaged by the 1989 Loma Prieta earthquake and was seismically retrofitted with 530 LRBs and 62 SDs in 2000.



Figure 12: The new 911 Emergency Communications Centre at San Francisco, protected by HDRBs in the years '90s so as to remain fully functional to 8.3 magnitude. This is the design earthquake level for all strategic buildings in California. In any case, also the existing public buildings shall be retrofitted so as to withstand very large quakes (e.g. of 8.0 magnitude). This imposes the use of SI, in spite of its large cost in the USA.

6 Application in Italy

At present Italy remains fifth for the number of isolated buildings already opened to activity: they are now 43 (fig. 13), in addition to 19 protected by ED systems or SMADs and 28 provided with STs. However, after many years of rather limited use of the SVPC systems (first due to the lack of design rules to the end of 1998, then because of their inadequacy and very complicated and time consuming approval process to May 2003), there is now a significant increase of the number of applications completed in the last biennium (the Italian isolated buildings were 25 in June 2005) and, especially, a large number of new applications in progress or designed (further 44 isolated buildings are already under construction or in an advanced design phase): this occurred thanks to the new Italian seismic code, enforced through Ordinance Nr. 3274/2003 of the Prime Minister (mostly as a consequence of the tragedy of San Giuliano di Puglia during the Molise and Puglia earthquake of October 31, 2002), which frees and simplifies the adoption of the SVPC systems. New applications concern not only strategic and public buildings, including hospitals and schools (figs. 14–18 and 20), but also dwelling buildings (fig. 19) and cultural heritage (figs. 21–25).



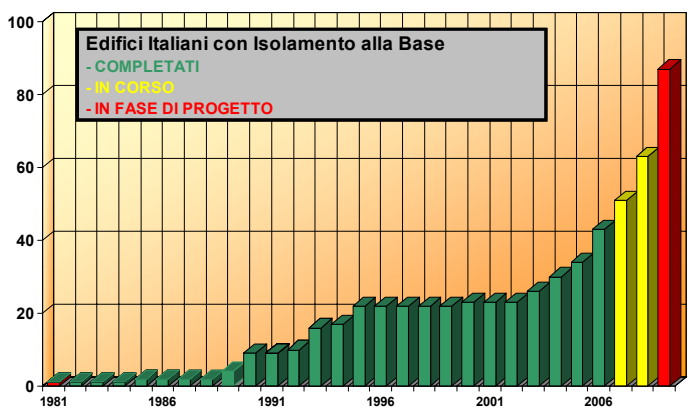


Figure 13: Cumulative number of Italian building applications of SI for the period 1981 to 2006. The very few applications from 1995 to 2003 were due to the absence of adequate design rules.



Figure 14: Views of the new wing of Gervasutta Hospital at Udine, which was the first Italian hospital structure to be protected by SI, after its completion and during construction with 56 HDRBs in 2005. All new Italian hospitals being or to be erected in seismic areas now include the SI for earthquake protection.

In addition, Italy remains the worldwide leader as regards the number and importance of bridges and viaducts protected by SVPC systems (they were over 150 already at the beginning of the years 1990s). It is worthwhile reminding that the first application of SI to Italian bridges and viaducts dates 1975 (it was to the Somplago viaduct, which survived the 1976 Friuli earthquake without any damage, contrary to most other structures similarly located in the epicentral area), while the first Italian isolated building was erected in 1981, namely 4 years before the first applications of this kind in Japan and the USA (it concerned a suspended steel-structure fire-command building in Naples that had been conventionally designed before the 1980 Campano-Lucano quake, when the site was not yet seismically classified, and allowed for not fully modifying the original design).





Figure 15: The fire station of the new Civil Defence Centre of Central Italy, completed at Foligno (Perugia) with 52 HDRBs and 5 SDs in 2005. The site of this centre was reclassified to zone 1 in 2003.

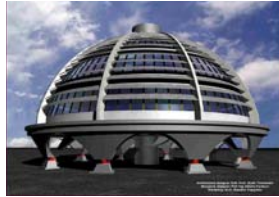


Figure 16: The control building of the new Civil Defence Centre being now erected at Foligno with 10 HDRBs of 1 m diameter (after completion, the Foligno centre will be formed by 13 isolated buildings).



Figure 17: The new Civic Room / Red Cross Headquarters of Gaggio Montano (Bologna, seismic zone 3), quite an irregular structure that was seismically isolated by means of 33 HDRBs and 4 SDs in 2006.



Figure 18: Plastic model of the two buildings of the new Francesco Jovine school being reconstructed at San Giuliano di Puglia, Campobasso (supported by an unique isolated slab), after the collapse of the previous primary school during the 2002 Molise and Puglia earthquake, and view of its SI system, formed by 61 HDRBs and 12 SDs, during construction in 2006-2007.



Figure 19: Plastic model of 6 dwelling buildings to be erected on the same artificial ground slab supported by 40 HDRBs and 12 SDs in the framework of the demolition / reconstruction project of the present very degraded Pontecitra 11-buildings complex at Marigliano (Naples). Erection of over 60 buildings on 16 slabs, supported 400–450 HDRBs and 360–380 SDs, has been planned.



Figure 20: View, during its construction in 2005, of the new building of the Polytechnical University of Marche at Ancona, protected by 86 BRBs.



Figure 21: Left: some of the 47 SMADs installed in the Upper Basilica of St. Francis at Assisi in 1998-99, during its restoration after the damages caused by the 1997-98 Umbria and Marche earthquake, to connect both tympana to the transept roof. Centre and right: two of the 34 STs installed inside the Upper Basilica, during the aforesaid restoration, to stiffen it.



Figure 22: The Cathedral of Santa Maria di Collemaggio at L'Aquila, a unique example of Romanic style in Abruzzo, and view of one of the elastic-plastic dampers (EPDs) installed in its roof after the walls had vibrated during the 1997-98 Marche and Umbria earthquake, in spite of the very large distance from the epicentre.

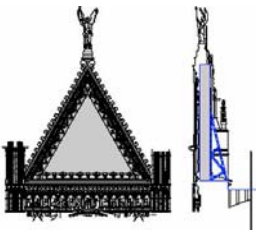


Figure 23: Left and centre: seismic improvement in progress for the Dome of Siena, by means of recentring viscous dampers (VDs), to avoid the overturning of the façade. Right: view of a VD during tests.

7 Application in other countries

As to other countries, of note are the present strong increase of the use of SI and other SVPC systems in Taiwan (after the 1999 Chi Chi earthquake and consequent modification of the seismic code, which now allows for the use of SI), the still growing number of isolated buildings in New Zealand (one of the lands of origin of the SVPC systems, in particular those based on lead technology) and, as a consequence of the 1988 Spitak earthquake, also the still developing Armenia, where even significantly high buildings are being seismically isolated and important retrofits have been performed with SI (fig. 27).



Figure 24: The Bronzes of Riace, each seismically isolated by means of a 3-stage HDRB system.



Figure 25: The David of Michelangelo, for which a SI project was undertaken by ENEA, ALGA and the University of Perugia.



Figure 26: The Iran Bastan Museum at Tehran (Iran), for which retrofit by means of SI is being designed in the framework of collaborations between Iran and Italy, which also involve IIEES, ICTP and Italian members of ASSISI and GLIS (the University of Reggio Calabria and ENEA).



Figure 27: The 16-storey "Our Yard" multifunctional complex, seismically isolated by means of Increased Damping Neoprene Bearings, during its construction at Yerevan (Armenia) in 2006.

Furthermore, important applications of the SVPC systems also began in Turkey (after the 1999 Kocaeli and Duzce earthquakes, during which the now seismically isolated new Ataturk Istanbul airport, being at that time conventionally constructed, was damaged, while the Bolu viaduct of the Istanbul-Ankara freeway was saved by Italian EPDs), Greece (fig. 29), Portugal (fig. 30) and Cyprus; they are going on in Canada, France (especially in La Martinique island), South Korea, Chile, Mexico and Indonesia and are beginning in other countries (e.g. Iran, see fig. 26). Many of the aforesaid applications (a large part of those in Taiwan, Turkey, Greece, Portugal, South Korea, Cyprus, Indonesia, etc.) make use of SVPC devices manufactured in Italy.

Some more recent Turkish isolated buildings have been protected with LRBs and Low Damping Rubber Bearings (LDRBs).



Figure 28: The Kocaeli University Hospital (Turkey), which was seismically isolated with the Friction Pendulum System (FPS) in 2006.



Figure 29: The International Broadcasting Centre at Athens, Greece, isolated with 292 Italian HDRBs in 2003.



Figure 30: The “La Luz” new hospital at Lisbon (Portugal), which was seismically isolated, together with a residence for old people in 2006; view of some of the 315 HDRBs installed at the buildings base, which were manufactured in Italy.



8 Application to the cultural heritage

It is also worthwhile mentioning the already significant application of the SVPC systems to cultural heritage, especially in Italy, including that to monumental structures (e.g. to the Upper Basilica of St. Francis at Assisi, severely damaged by the 1997-98 Marche and Umbria earthquake, see fig. 20), single masterpieces (e.g. the Bronzes of Riace and hopefully, in the near future, David of Michelangelo, see figs. 24 and 25), ceilings of archaeological excavations (e.g. those at Akrotiri, in the Greek Santorini island) and museums. With regard to the latter, the design for retrofitting the Iran Bastan Museum in Tehran with



seismic isolation is particularly important (fig. 26): this is being developed in the framework of collaborations between Iran and Italy, which also involve the International Institute of Earthquake Engineering and Seismology (IIEES) of Tehran, the Abdus Salam International Centre of Theoretical Physics (ICTP) of Trieste (Italy) and Italian members of *ASSISI* and GLIS, affiliated to the Mediterranean University of Reggio Calabria and ENEA.

9 Application to the industrial structures

Finally, to be stressed is the increasing use of SI in the industrial field, in particular for high risk plants such as Liquefied Natural Gas (LNG) tanks (e.g. in Turkey and the P.R. China, see figs. 31 and 32, after the first applications performed in Greece and South Korea some years ago) and nuclear structures. Besides the first application of SI to Japanese structures of this kind (the Nuclear Fuel Related Facility, see fig. 33), of note is that the construction of new isolated nuclear reactors has been planned to start soon, both in Japan (where design rules allowing to license them are already available) and in other countries, in particular in France, where SI has already been decided for the Jules Horowitz Reactor and ITER fusion plant, to be both built at the Cadarache Research Centre (characterized by 0.33 g peak ground acceleration). As to France, it is worthwhile reminding the SI applications to nuclear reactors and spent nuclear fuel storage pools performed at Cruas and La Hague in the years 1970s (in addition to those to civil structures), to allow for the use of standardized plant designs in areas characterized by seismic intensities larger than those considered in such designs.



Figure 31: The two 140,000 m³ LNG tanks of Egegaz at Aliaga, Turkey, protected by 112 LRBs and 241 LDRBs.



Figure 32: The two 160,000 m³ LNG tanks at Guandong (P.R. China), each with 360 HDRBs in 2006.



Figure 33: The Nuclear Fuel Related Facility, which was the first nuclear structure to be seismically isolated in Japan.

10 Conclusions

The state-of-the-art of the application of the modern seismic vibration passive control (SVPC) techniques has been shortly reported, especially for seismic isolation (SI) of buildings. Particular attention has been devoted to the use of the



SVPC systems in Italy, by showing the leadership it achieved in this field at European level, in spite of the problems suffered until four years ago (first due to the absence of specific design rules, then owing to their inadequacy and the too complicated approval process). It is worthwhile mentioning that the contributions provided by ENEA have been of fundamental importance for the development and application of the SVPC systems in Italy. The present excellent prospects for a wide extension of the use of such systems in our country, thanks to the new national seismic code and the seismic reclassification of the Italian territory, have been stressed. More generally, the key role plaid by the availability and features of specific design rules on the success of the aforesaid systems in the different countries has been cited.

References

- [1] Dolce, M., Martelli, A., and Panza, G. 2006. *Moderni Metodi di Protezione dagli Effetti dei Terremoti (Modern Methods for the Protection from Earthquake Effects)*, Special edition for the Italian Civil Defense Department, A. Martelli, ed., 21^{mo} Secolo: Milan, Italy (in Italian).



Study of the seismic response of reinforced concrete isolated elevated water tanks

V. I. Fernández-Dávila, F. Gran & P. Baquedano

*School of Civil Engineering at Civil Works, Central University,
Santa Isabel, Chile*

Abstract

This investigation carries out the seismic response of parametric elastic models of reinforced concrete isolated elevated water tanks. From the study of the physical and geometric variables that characterize elevated water tanks it was possible to define parametric models with the purpose of obtaining a wide representative family of structures. The parameters were grouped in the following form: a) elevated tank: ratio of heights, ratio of slenderness, ratio of diameters, ratio diameter-thickness, and ratio water mass-structure mass, b) isolation system: ratio of slenderness, horizontal and vertical stiffness, c) water: the water-structure interaction effect is modeled using the mechanical analogy proposed by Housner.

This special type of continuous structure, similar to an inverted pendulum, has been discretized according to the lumped mass criterion and the support structure of the tower was partitioned in ten one-dimensional elements. As seismic loads were applied the design spectrum of accelerations were used as recommended by the Chilean code NCh 2745 Of.2003, respectively.

The maximum responses were obtained for the lateral displacements, the shear forces and bending moments. The sensitivity analysis of the structural models of isolated elevated water tanks allowed us to observe that the maximum bending moments and the maximum shear forces are equivalent to the eighth part of the maximum responses obtained in a similar fixed-base elevated water tank, and that the relative lateral displacements are lower than 0.2%, reducing the deformations in the structure significantly.

Keywords: elevated water tanks, dynamic of structures, seismic loads, seismic base isolation, lateral displacements, shear forces, bending moments.



1 Introduction

Chile has suffered devastating seismic effects of great magnitude on many occasions, which resulted in serious consequences such as the loss of human lives and resources. On the basis of past experiences, the repetition of this phenomenon in the future must be thought of as a certain possibility, incurring the same catastrophic effects that have occurred in the past and maybe in higher proportions. Having taken this consideration into account, it is necessary to prepare to face new menaces of this nature, adopting ways to avoid or minimize the effects of earthquakes that could occur in the future [4, 9].

Elevated water tanks are industrial structures built for the purpose of maintaining the water supply. There are researches on this special kind of continuous structure that has its bases fixed and isolated [11]. The application of seismic isolation systems in other parts of the world has concentrated its efforts on the research of conventional structures such as buildings, that results in very attractive research about its application in this special kind of continuous structure generally considered as rigid [9]. Indeed, in the last years, the seismic isolation system has seen an increased application on buildings in countries that have high seismic risks (Japan, United States, Italy, Canada, New Zealand). Its effectiveness was proven during the occurrence of important earthquakes such as Northridge (USA, 1994) and the Kobe (Japan, 1995), due to the fact that these areas presented an important number of structures designed with frictional and elastomeric isolation systems [10].

The objective of this investigation is to study the seismic responses of this special kind of “compound structure” with the purpose of understanding the structural behaviour due to seismic action.

2 Methodology

2.1 Type of structure

A reinforced concrete elevated tank of drinkable water which had a flexible connection between the superstructure and the foundation, denominated seismic isolator, was analyzed. These mechanisms (table 1) work in an elastic range and consist basically of a collection of thin rubber plates interspersed with steel plates which are stuck to the rubber with an adhesive gum and then are subjected to a vulcanisation process. A resistant element of a low horizontal rigidity and high vertical rigidity was obtained as a result, succeeding to uncouple the structure from the seismic movements of the land. Twelve isolators that are equidistant to each other and located in the perimeter of the structure of support, were used (figs. 1, 2).

The kind of superstructure used is the elevated water tank made of reinforced concrete as the composite. This kind of structure presents a support base or shaft and in its higher area a tank or barrel, both elements are of transversal, circular section.



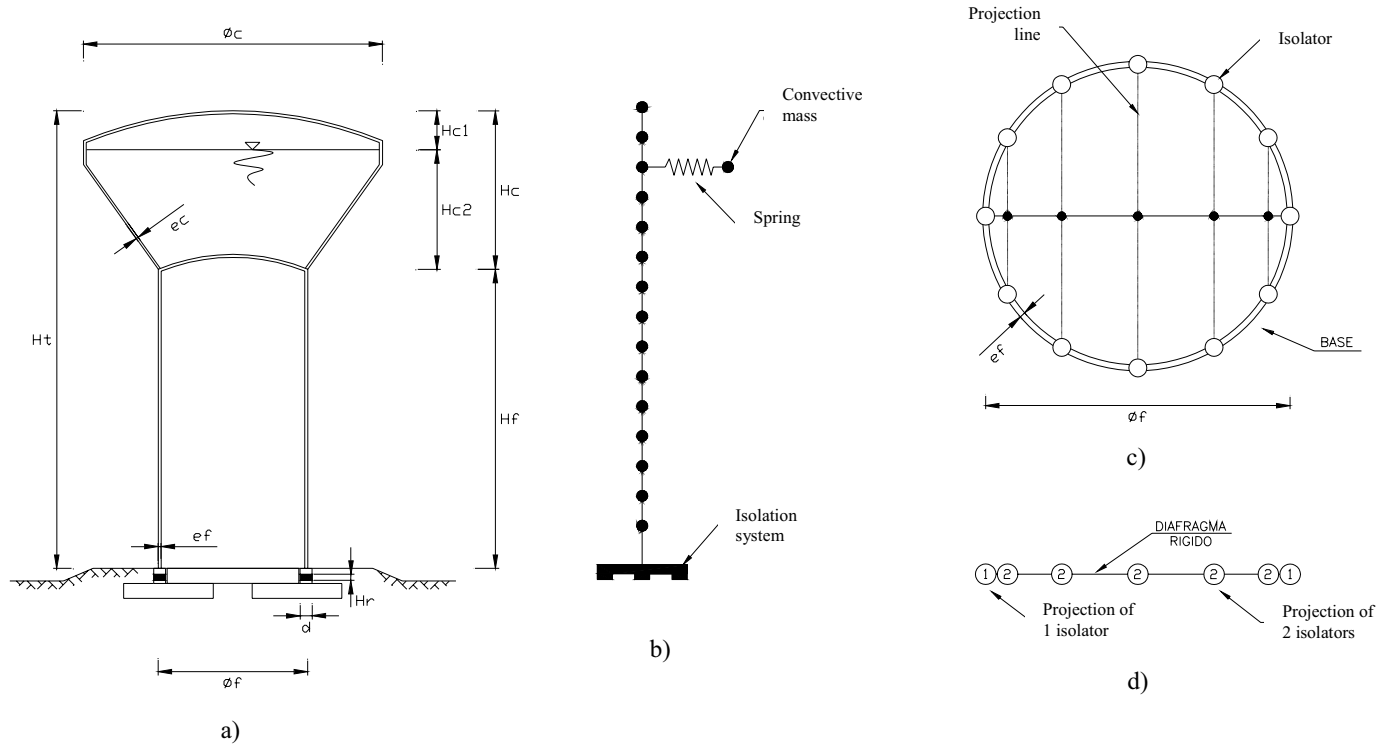


Figure 1: Elevated tank type "composite", and isolation system: a) Transverse section, b) Lumped masses model, c) Location of isolators in the base, d) Projection of isolators in elevation.

This choice was performed from a sensitivity analysis of the tanks of this kind constructed in the central area of Chile and its capacity to support great water masses inside. The kind of isolator considered on the research is the high damping isolator (HDR) [10], owing to the fact of its high capacity to dissipate the energy that comes from the seismic movement of the land, preventing this energy from being totally absorbed by superstructure.

For the sensitivity analysis, eight real tanks that fulfill the required geometry have been found. These tanks constitute the pattern database, identifying the more relevant geometric and physic relevant features (tables 2, 3) from the study of each one of them. Geometric properties were considered such as (fig. 1): H_t , H_c , H_f , which are the total heights of the tank and the structure of support, respectively; in addition, e_f , e_c , are the thicknesses of the structure of support and the tank; ϕ_f , ϕ_c , are the diameter of the structure of support and the tank; and H_{c1} and H_{c2} , are the fixed and variable height of the tank, respectively. The modeling of the tank, such as structures of the reversed pendulum kind, is shown in table 3 and consists of verifying more than 50% of the total weight which is found in the superior level [7].

Table 1: Characteristics of the isolators.

Description	Unit	Reinforced rubber	No reinforced rubber	Steel
IRHD ¹		45	65	100
σ_t	MN/m ²	28	21	420
σ_u	%	680	420	40
E	MN/m ²	1,9	5,9	210.000
G	MN/m ²	0,54	1,37	81.000
k	MN/m ²	1.000	1.200	176.000
ν		0,4997	0,4997	0,29
Resilience	%	80	60	100
V_s	m/s	37	37	5.000

¹International Rubber Hardness.

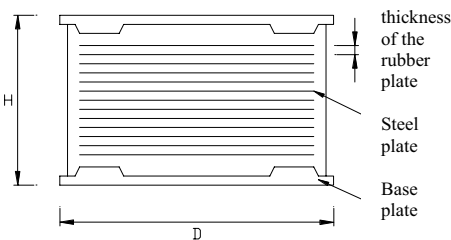


Figure 2: Elastomeric seismic isolator.



Table 2: Characteristics of the real tanks and seismic data.

Nº	Tank	Capacity (m ³)	H _t (m)	H _c (m)	H _f (m)	φ _c (m)	φ _f (m)	e _c (m)	e _f (m)	H _{c1} (m)	H _{c2} (m)	Seismic data		
												Zone	Soil	ξ (%)
1	Pontigo-Buin	2.000	31,3	10,1	21,2	23,8	12	0,2	0,25	2,1	8,0	2	II	5
2	Linderos	2.000	38,3	10,1	28,2	23,8	12	0,2	0,25	2,1	8,0	2	II	5
3	Paine	1.000	35,8	6,8	29,0	19,0	12	0,2	0,20	1,7	5,2	2	II	5
4	Los Tilos	1.500	29,8	8,8	21,0	19,0	12	0,2	0,20	3,6	5,2	2	II	5
5	Estadio-Estación Buin	1.500	32,8	8,8	24,0	19,0	12	0,2	0,20	3,6	5,2	2	II	5
6	Melipilla	500	30,3	5,3	25,0	12,9	9	0,2	0,20	1,6	3,7	3	III	5
7	El Monte	500	25,3	5,3	20,0	12,9	9	0,2	0,20	1,6	3,7	3	III	5
8	El Trébol	2000	38,3	10,1	28,2	24,2	12	0,2	0,25	2,3	7,8	2	II	5

Table 3: Weights of the elevated water tanks (kN).

Tank	W _{fuste}	W _{cuba}	W _t	W _f	W _{total}	W _{sup} = W _{cuba} + W _{H2O}	W _{sup} /W _{total} (%)
Pontigo-Buin	5.000	5.390	10.390	20.000	30.390	25.390	83,6
Linderos	6.650	5.390	12.040	20.000	32.040	25.390	79,3
Paine	5.470	3.410	8.880	10.000	18.880	13.410	71,0
Buin	3.960	3.980	7.940	15.000	22.940	18.980	82,7
Estadio Buin	4.520	3.980	8.500	15.000	23.500	18.980	80,8
Melipilla	3.530	1.750	5.280	5.000	10.280	6.750	65,6
El Monte	2.830	1.750	4.580	5.000	9.580	6.750	70,5
El Trébol	6.650	5.520	12.170	20.000	32.170	25.520	79,3

2.2 Fluid-structure interaction

The fluid-structure interaction was determined using the equivalent mechanical model proposed by professor Housner. Effectively, it proposes that the motion of the total mass of water can be represented in the following way: a) a solidary mass to the tank, called fixes or impulsive mass (M_0); and b) a mass that represents the phenomenon of surge of water, named movable or convective mass (M_1) and connected to the walls of the tank by total stiffness K [5]. Eqs. (1) to (6) allow us to evaluate the impulsive and convective masses, the stiffness of the spring, the water vibration period, and the location of these masses measured from the base of the tank.

$$\frac{M_0}{M_F} = \frac{\tanh\left(\frac{\sqrt{3}}{2} \cdot \frac{D}{H}\right)}{\frac{\sqrt{3}}{2} \cdot \frac{D}{H}}, \quad \frac{M_1}{M_F} = \frac{363}{512} \cdot \frac{\tanh\left(\sqrt{13.5} \cdot \frac{H}{D}\right)}{\sqrt{13.5} \cdot \frac{H}{D}} \quad (1, 2)$$



$$\frac{H \cdot K}{W_F} = \frac{45}{2} \cdot \left(\frac{M_1}{M_F}\right)^2 \cdot \left(\frac{H}{D}\right)^2, \quad T_a = 2\pi \cdot \sqrt{\frac{M_1}{K}} \tag{3, 4}$$

$$h_0 = \frac{3}{8} \cdot H \cdot \left[1 + \alpha \cdot \left(\frac{M_F}{M_0} - 1\right)\right], \quad h_1 = H \cdot \left[1 - \frac{\text{Cosh}\left(\sqrt{13.5} \cdot \frac{H}{D}\right) - \beta}{\sqrt{13.5} \cdot \frac{H}{D} \cdot \text{Senh}\left(\sqrt{13.5} \cdot \frac{H}{D}\right)}\right] \tag{5, 6}$$

where M_F , and W_F , are the total mass and weight of the water; α and β are dependent variables of the pressures of the walls; h_0 , and h_1 , are the heights of the impulsive and convective masses, both measured with respect to the bottom of the tank; T_a , fundamental period of vibration of the convective mass; H and D , are the height and diameter of the tank, respectively. The values considered for this study were $\alpha=0$, and $\beta=1$ [5], because the pressures of the water stored on the walls of the container are considered. In the present study the height H is equal to the height H_c , and the diameter D is equal to ϕ_c of the analyzed model (fig. 1a). In addition, W_f is equal to W_{H_2O} .

2.3 Parametric analysis of the structure

From the study of the most relevant elastic characteristics that determine the behavior of the eight elevated water tanks defined in the database, it was possible to select ten parameters of interest that, if combined suitably, allow us to represent an ample family of this type of structure [1, 3, 4, 9]. The parameters are as follows:

- (RH) Height ratio (tank – structure of support) = H_c/H_f
- (RD) Diameter ratio (tank – structure of support) = ϕ_c/ϕ_f
- (RR) Height – Diameter ratio = RH/RD
- (HD) Slenderness ratio = H_f/ϕ_f
- (DE_c) Diameter ratio – thickness in the tank = ϕ_c/e_c
- (DE_f) Diameter ratio – thickness in the structure of support = ϕ_f/e_f
- (RD_c) Diameter ratio-thickness = DE_c/DE_f
- Mass ratio = M_{H_2O}/M_t
- (RH_c) Height ratio in the cube = H_{c1}/H_c
- (RH_a) Slenderness ratio of isolator = H_f/d

Table 4: Values adopted for the parameters and number of studied cases.

Id	Tank					Isolator	Seismic			N° of total cases
	RR	HD	RD _c	RH _c	RM	RH _a	Soil	Zona	ξ (%)	
1	0,10	2,4	1,0	0,2	0,9	0,35	2			972
2	0,21	2,9	2,0	0,4	1,5	0,50	3	2	5	
3	0,36	3,2	2,7	---	2,0	1,00	---			

---Value does not exist.



Table 4 shows the geometric and seismic parameters considered in this study, as well as the values assigned to each one of them. These values were obtained from the analysis of sensitivity of the values adopted by each parameter of the eight structures of the database pattern. From this new database, a family of 972 elevated water tank structures could be generated.

2.4 Sensitive analysis

A sensitivity analysis was made in which the responses of a tank modeled by finite elements (MEF) and another one modeled by the criterion of lumped masses were compared (MC), in which the structure of support was discretized in 10 elements and the tank in five elements, both being of frame type [2]. The responses that were compared were the periods of vibration, the lateral displacements, the shear forces, and bending moments. The tank modeled by finite elements took control of elements type shells [2] of size $1 \times 1 \text{ m}^2$.

The maximum errors found were: 2,3% in the periods of vibration, 8.8% in the lateral displacements, 6.6% in the basal shear forces, and 4.3% at the bending moments.

It was observed that the responses determined with criterion MC are greater than the responses obtained by MEF. This comparison was made on an empty and a full water elevated tank, considering, in addition, situations of isolated base and fixed [1, 4].

2.5 Design spectrum

The seismic load that was used corresponded to the design spectrum of the NCh 2745 Of. 2003 code [8]. In this norm is the type of elastic spectrum, which must be reduced by the factor of reduction R that is indicated in the code of industrial structures NCh 2369 Of. 2002 [7]. This design spectrum (fig. 3) depends on as much the seismic zone as the type of ground on which the structure is founded.

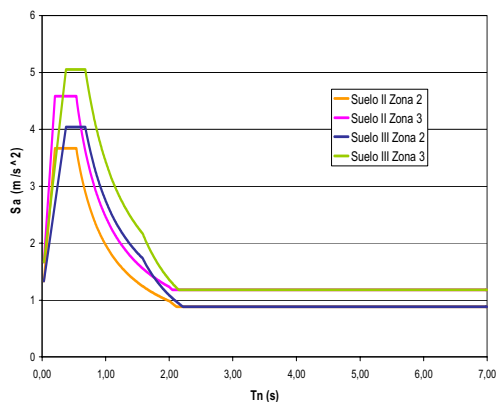


Figure 3: Design spectrum utilized.



3 Analysis of the results

The analysis of the 972 parametric models of the elevated water tanks was made using a computational tool called SAP2000 [2]. In order to obtain the maximum responses, the complete quadratic combination rule (CQC) was used [7]. On the basis of this method the total displacements were obtained, as well as the shear forces and bending moments.

The study considered that half of the models to be founded on soil type II and the rest on soil type III (Fig. 3) [7], with the purpose of comparing the interest seismic responses. The validity of this study is limited to the parameters that are adopted by the following dominions: $RR \in [0.10;0.36]$; $HD \in [2.4;3.2]$; $RDE [1.0;2.7]$; $RH_c \in [0.2;0.4]$; $RM \in [0.9;2.0]$; $RH_a \in [0.35;1.00]$; Soil type [2;3].

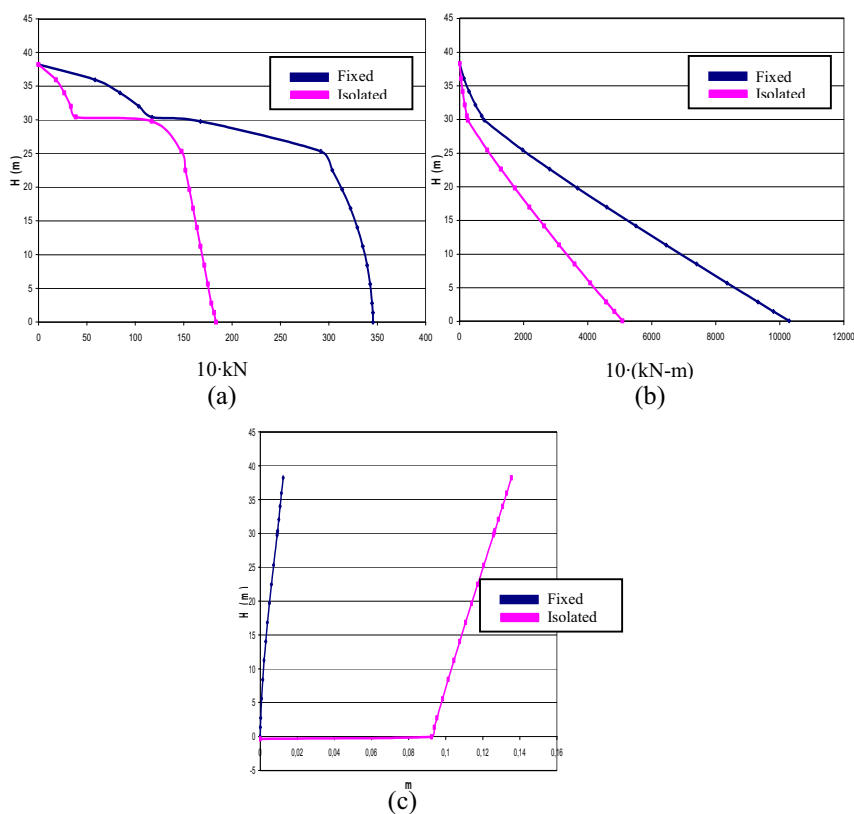


Figure 4: Seismic responses of the Pointigo-Buin tank: (a) shear forces, (b) bending moments and (c) lateral displacements.

The analysis of results shows the following:

- When comparing the tanks of fixed base with their similar of isolated base were verified that with the incorporation of the isolation device to the



shear forces (Fig. 4(a)), and the bending moments are reduced in a 50% (Fig. 4(b)).

- In the cases of fixed and isolated bases, it is demonstrated that the main cause for the abrupt increment of the magnitudes of the shear force is water movement due to seismic excitation.
- The lateral displacement experiments a strong increment of its magnitude in the zone of the isolator that borders 1000%, since the lateral stiffness of this one is considerably smaller for the stiff than it is for the structure of support (Fig. 4(c)).
- For the totality of the parametric models, the safety factors of buckling and rollover of the isolators were verified satisfactorily.

4 Conclusions

- When comparing elevated r/c tanks of fixed base with their similar of isolated base, it was verified that the incorporation of the isolation device reduces the shear force and the bending moments in 50%, and although the water stays as the fundamental period of vibration, the isolation system takes the second modal shape of vibration that in the case of the fixed tanks it belongs to the structure (figs. 5, 6).
- The Chilean code [6] indicates that the relative displacement in all the levels of the structure must be smaller than 2%. For the analysis of the database the tanks fulfill this requirement since the maximum relative displacement was of 1,2%. This means that the tank has a lateral displacement in the form of a rigid body.
- The incorporation of a system of isolation in the high tanks brings as a consequence that the structure of support presents compressive stress different to the tank that does not consider this flexible fusion that presents tensile effort additionally.

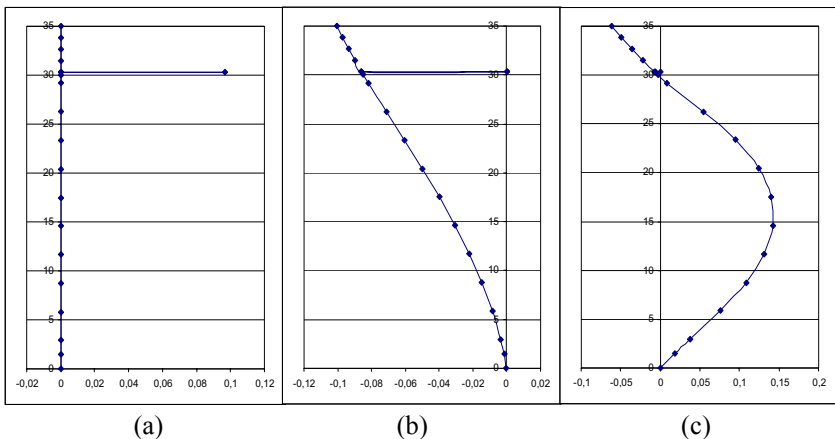


Figure 5: First three modal shapes of the tank N° 1 with fixed base and water full: (a) first mode, (b) second mode, (c) third mode.



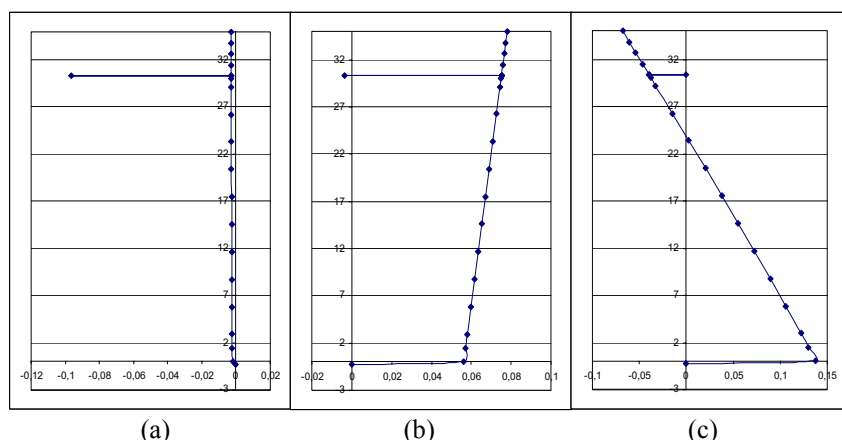


Figure 6: First three modal shapes of the tank N° 1 with isolated base and water full: a) first mode, (b) second mode; (c) third mode.

- d) The differences of the maximum responses found to the finite element analysis with the analysis of lumped masses using the expressions for the fluid-structure interaction [5] were of 2.3% in the periods of vibration, of 8.8% in the lateral displacements, 6.6% in the basal shear force, and 4.3% at the bending moments [1, 4].
- e) The geometric form that acquires the representative outline of the maximum responses of the elevated tanks with isolation is similar to the same structure without isolation. Therefore the seismic behavior of a structure fixes and an isolate is similar, varying only the maximum values.
- f) From this one study it is possible to obtain simplified expressions for the analysis of elevated water tanks with seismic isolation in his base [3].

Acknowledgment

The authors wish to thank the School of Civil Engineering at Civil Works of the Central University, for their support for this investigation.

References

- [1] Baquedano, P., Gran, F., Fernández-Dávila G., V.I. (2006) Methodology for the parametric analysis and seismic design of isolated elevated water tanks. 8th U.S. NCEE. SF, California. USA. April 18-22.
- [2] Computers & Structures, Inc. (2003) *SAP2000 Non linear version 8.2 Academic License*. Proyecto de Investigación N° 28. Universidad Central de Chile. Enero.
- [3] Fernández-Dávila G., V.I., Dünner D., R., Carrión P., L. (2005) Simplified Method for Seismic Analysis of Industrial Chimneys. *Structural Journal of ACI*. 102-S34, Vol. 102, Issue 3, Pp 347-353, May.



- [4] Fernández-Dávila G., V.I., Baquedano, P., Gran, F. (2005) Estudio de la Respuesta Sísmica de Estanques Elevados de Agua de Hormigón Armado con Aislación Sísmica en la Base. *IX Jornadas de Achisina*. Concepción, Chile. 16-19. Noviembre.
- [5] Housner, G.W. (1963) Dynamic Analysis of Fluids in Containers Subject to Acceleration. *Bull. Seismology Soc. Am.* 47 (1), 15-37.
- [6] INN (1996) *NCh 433 Of. 96. Diseño sísmico de edificios*. Instituto Nacional de Normalización, Santiago, Chile.
- [7] INN (2002) *NCh 2369 Of.2002 Análisis y diseño sísmico de estructuras industriales*. Instituto Nacional de Normalización, Santiago, Chile.
- [8] INN (2003) *NCh 2745 Of.2003 Análisis y diseño de Edificios con Aislación Sísmica*. Instituto Nacional de Normalización, Santiago, Chile.
- [9] Muñoz P., M., Fernández-Dávila G., V.I. (2002) Analysis and seismic design of elevated water tanks. *7th U.S. NCEE*. Boston, Massachusetts. USA. July 21 - 25
- [10] Naeim, F., Kelly, J.M. (1999) *Design of Seismic Isolated Structures: From Theory to Practice*. John Wiley & Sons, Berkeley, California, USA.
- [11] Shenton, H.W., Hampton, F.P. (1999) Seismic response of isolated elevated water tanks. *Journal of Structural Engineering*. Vol. 125. Issue 9, 965-976. September.



This page intentionally left blank

Modeling of the structural impact of seismically isolated buildings

P. Polycarpou, L. Papaloizou, P. Komodromos & M. C. Phocas
*Department of Civil and Environmental Engineering,
 University of Cyprus, Nicosia, Cyprus*

Abstract

Structural impact can be considered using methods that are based on either stereomechanical or force-based approaches. The force-based approach, which uses contact springs that are automatically formed during impact, is more suitable for simulations of multiple deformable bodies, such as colliding buildings. After making a comparison among the most common impact models of the force-based approach, a modified impact model is proposed as a variation of the linear viscoelastic impact model (Kelvin-Voigt). The modified viscoelastic impact model avoids tensile impact forces during detachment and enables the consideration of permanent plastic deformations due to poundings. The proposed impact model is used for simulations of poundings of seismically isolated buildings with adjacent structures, in order to assess the influence of potential structural impact on the effectiveness of seismic isolation. Poundings are assumed to occur at the isolation level between the seismically isolated building and the adjacent moat wall whenever the available seismic gap is exceeded due to a strong earthquake excitation. The simulations reveal that poundings may substantially increase floor accelerations, especially at the floor where impacts occur, and excite higher modes of vibration, increasing the interstory deflections.
Keywords: poundings, structural impact, seismic isolation, seismic gap.

1 Introduction

Seismic isolation introduces flexibility, or a sliding mechanism, at the isolation level of a relatively stiff building, shifting its fundamental period outside the dangerous for resonance range, or preventing the transmission of a shear force higher than a certain value, in order to reduce the induced seismic loads.



A practical constraint in the implementation of seismic isolation is the wide seismic gap that must be provided as a clearance around the building to facilitate the expected large relative displacements at the isolation level. Considering that there are often certain practical restrictions to the size of the available clearance around seismically isolated buildings, a reasonable concern is the possibility of poundings with adjacent structures during very strong earthquakes.

A critical aspect in numerical simulations of structural pounding is the impact model that is employed and the values of the associated parameters, which affect the computed results. In most research studies on structural pounding, force-based impact models are used, exerting impact forces to the colliding structures whenever their separation distances are exceeded. Anagnostopoulos [1], Jankowski [2], Muthukumar and DesRoches [3] and others have proposed various methodologies using either a linear or a non-linear impact spring together with an energy dissipation mechanism to model structural pounding. However, none of these impact models takes into account the remaining plastic deformations of the colliding structures.

Following a brief description of the simulation approach, the most commonly used impact models are assessed, leading to a proposed variation of the linear viscoelastic impact model. Subsequently, selected simulation results are presented with emphasis placed on the influence of the impact modelling and the values of the corresponding parameters.

2 Description of the problem

Poundings are assumed to happen between the moat wall and the base mat at the isolation level, which is the most common case of structural impact for a seismically isolated building due to the large relative displacements at the isolation level. The superstructure is modeled as a multi-degree of freedom system with shear-beam behavior and the masses lumped at the floor levels (fig. 1(a)).

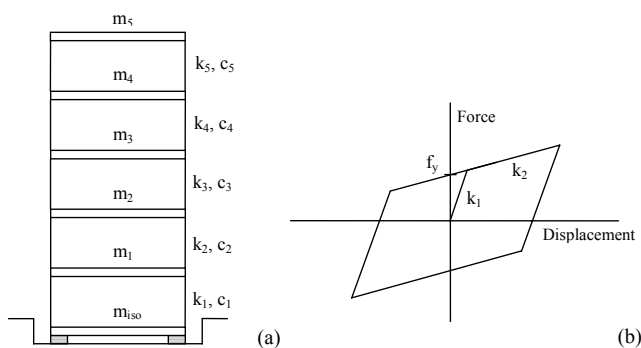


Figure 1: (a) Analysis model of the seismically isolated structure; (b) the bilinear model considered for the isolation system.

A bilinear behaviour is considered for the isolation system (fig. 1(b)), with additional viscous damping, while the superstructure is assumed to remain elastic



during earthquake excitations. The equations of motion are formed considering all inertia, damping and elastic forces, while the impact forces are taken into account only during poundings. Impact is considered assuming an impact spring and an impact dashpot in parallel, which exert impact forces to the colliding structures whenever their separation distances are exceeded. At each time step the equations of dynamic equilibrium are directly integrated using the Central Difference Method (CDM), computing the displacements and other response quantities at the following time step.

3 Impact modeling

Structural impact is considered using force-based methods, also known as penalty methods. These methods allow interpenetration between the colliding structures, which is justified by their deformability at the vicinity of the impact. Contact springs are automatically formed when an impact is detected, kept as long as the building remains in contact with the moat wall and removed as soon as the building is detached from the wall. The interpenetration depth is used together with the stiffness of the contact spring to estimate, according to the impact model, the contact forces that are applied to the structures, pushing them apart.

In this work, both linear and non-linear impact models are used, in order to investigate the effect of the impact model selection on structural response with pounding incidences. Specifically, the Kelvin-Voigt model and the Hertzian model with non-linear damping were selected, using the formulas provided by Anagnostopoulos and Jankowski, respectively, for the estimation of the impact-damping coefficient. In addition, an adjustment to the Kelvin-Voigt model is proposed.

3.1 Linear viscoelastic impact model

The linear viscoelastic impact model, also known as Kelvin-Voigt model, is one of the most commonly used in structural pounding and consists of a linear impact spring and a viscous impact dashpot. Whenever there is impact, the impact force at time t is provided by the expression:

$$F_{imp}(t) = k_{imp} \cdot \delta(t) + c_{imp} \cdot \dot{\delta}(t) \quad (1)$$

where k_{imp} is the stiffness of the linear impact spring, $\delta(t)$ is the interpenetration depth of the colliding bodies that overlap each other, c_{imp} is the impact-damping coefficient and $\dot{\delta}(t)$ is the relative velocity between the colliding structures at time t .

Anagnostopoulos [1] has provided the following analytical expressions that associate the impact-damping coefficient with the coefficient of restitution (COR) and the masses m_1 and m_2 , of the colliding bodies:

$$c_{imp} = 2 \cdot \xi_{imp} \sqrt{k_{imp} \frac{m_1 \cdot m_2}{m_1 + m_2}} \quad (2)$$



$$\xi_{imp} = -\frac{\ln(COR)}{\sqrt{\pi^2 + (\ln(COR))^2}} \quad (3)$$

where ξ_{imp} is the impact damping ratio ($0 < \xi_{imp} < 1$). The COR is defined as the ratio of the relative velocities between the colliding bodies after and before impact ($0 \leq COR \leq 1$). The derivation of the above formulas is based on the conservation of energy. However, this model exhibits an initial jump of the impact force values upon impact due to the damping term. Furthermore, the damping force causes negative impact forces that pull the colliding bodies together, during the unloading phase, instead of pushing them apart (fig. 2(a)). Nevertheless, as it is shown later, this simple impact model provides sufficiently accurate results for the overall structural response, given that proper values are used for the impact parameters.

3.2 Non-linear viscoelastic impact model

Another commonly used structural impact model uses a non-linear impact spring, based on Hertz's contact law. According to this model, it is assumed that the impact force increases exponentially with the interpenetration depth, usually with an exponent of 1.5. In order to include an energy dissipation mechanism, some researchers [2, 3] have incorporated a non-linear damper parallel to the non-linear spring during the approach phase of the contact (fig. 2(b)). In that case the impact force during the approach phase equals:

$$F_{imp}(t) = \hat{k}_{imp} \cdot \delta(t)^{1.5} + \hat{c}_{imp}(t) \cdot \dot{\delta}(t) \quad (4)$$

while during the restitution phase, the energy dissipation is omitted and the impact force equals:

$$F_{imp}(t) = \hat{k}_{imp} \cdot \delta(t)^{1.5} \quad (5)$$

According to Jankowski [2], the impact-damping coefficient $\hat{c}_{imp}(t)$ is provided by the following formula in terms of the impact damping ratio $\hat{\xi}_{imp}$ and the interpenetration depth $\delta(t)$:

$$\hat{c}_{imp} = 2 \cdot \hat{\xi}_{imp} \sqrt{\hat{k}_{imp} \cdot \sqrt{\delta(t)} \cdot \frac{m_1 \cdot m_2}{m_1 + m_2}} \quad (6)$$

The impact-damping ratio $\hat{\xi}_{imp}$ can be estimated using Jankowski's [4] formula:

$$\hat{\xi}_{imp} = \frac{9\sqrt{5}}{2} \frac{1 - COR^2}{COR(COR(9\pi - 16) + 16)} \quad (7)$$



The damping ratio, according to eq. (7), may take values greater than 1 and specifically approaches infinity for $COR = 0$ (perfectly plastic impact) [4], in contrary with the ξ_{imp} provided by eq. (3) for the linear viscoelastic impact model which takes values between 0 and 1.

3.3 Proposed impact model

In order to avoid the tensile impact forces that arise between the colliding structures at the end of the restitution period, due to the damping term, a minor adjustment is proposed for the linear viscoelastic model. In particular, when the impact force is about to change sign, the impact spring and dashpot are removed, considering that the building is detached from the moat wall. The wall is kept at its current position, assuming some remaining plastic deformations, which increase the corresponding available width of the seismic gap (fig. 2(c)).

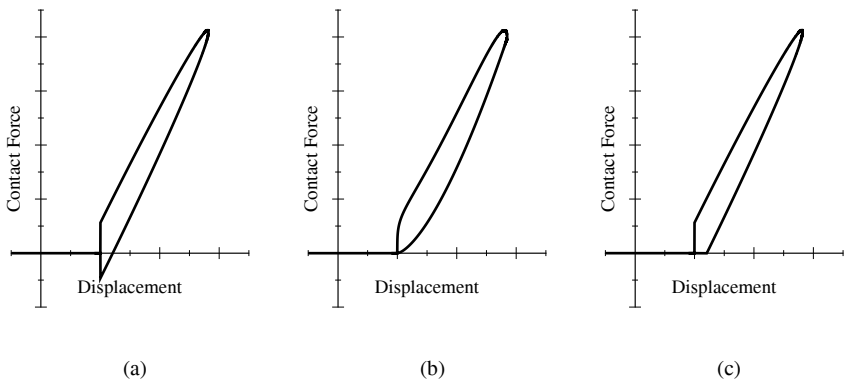


Figure 2: Impact models: (a) linear viscoelastic model; (b) non-linear viscoelastic model; and (c) the proposed modified linear viscoelastic model with permanent deformation.

Therefore, the equation that provides the impact force can be written as:

$$F_{imp}(t + \Delta t) = \begin{cases} k_{imp} \cdot \delta(t) + c_{imp} \cdot \dot{\delta}(t) & \text{when } F_{imp}(t) > 0 \\ 0 & \text{when } F_{imp}(t) \leq 0 \end{cases} \quad (8)$$

When using the force-based impact models, it is very important to appropriately determine a value for the impact stiffness, which depends on the mechanical properties of the material and the geometry of the contact surface of the colliding bodies. A wide range of diverse values has been used in the literature for different kinds of impact problems. Van Mier et al [8], who experimentally examined the case of impact between concrete bodies, concluded that the impact stiffness, considering a non-linear impact spring, should vary



from 40 to 80 kN/mm^{1.5} in order to match experimental results. However, those values may not represent properly the impact forces that are applied during poundings of a large-scale building against a wall or another structure.

4 Simulations and results

4.1 Example

A typical seismically isolated building is used in the simulations in order to examine the pounding effects and the case of using different impact models. The superstructure is assumed to have 5 floors, each with a lumped mass of 500 tons, while each story has horizontal stiffness of 1 GN/m. An additional mass of 500 tons is assumed to be lumped at the isolation level, while the bilinear properties of the isolation system were taken as follows (see fig. 1(b)): $k_1 = 200$ MN/m, $k_2 = 25$ MN/m, $f_y = 0.1 \times W_{\text{tot}}$, where W_{tot} is the total weight of the building. A damping ratio equal to 2% was assumed for the superstructure, while for the isolation system, in addition to the hysteretic energy dissipation, a 5% viscous damping ratio was considered. The fundamental period of the fixed-supported superstructure is equal to $T_{\text{fixed}} = 0.494$ sec.

The three previously described impact models (fig. 2) are used, to consider potential poundings of the isolated building with the moat wall. In particular, the linear viscoelastic model with impact stiffness equal to $k_{\text{imp}} = 1250$ kN/mm, the Hertzian model with non-linear damping using as impact stiffness the value of $\hat{k}_{\text{imp}} = 277.8$ kN/mm^{1.5} and the proposed modified viscoelastic model, which allows plastic deformations, with impact stiffness equal to $k_{\text{imp}} = 1250$ kN/mm, are used. These values were selected in order to obtain the same maximum impact force when the base mat hits the wall with a constant velocity of 1 m/sec. The maximum impact force was calculated using a finite element analysis, simulating the collision of a concrete slab against a retaining wall with a velocity of 1 m/sec. For all models the *COR* was taken equal to 0.7 and the masses of the colliding bodies equal to 500 tons and 1000 tons for the base mat and the moat wall, respectively. For each of these cases, dynamic analysis of the building is performed under the Northridge 74 Sylmar-Converter Station record ($\text{PGA} = 0.897g$), which is a relatively very strong excitation.

According to the simulation results, the total accelerations as well as the interstory deflections and, therefore, the story shear forces of the seismically isolated building may significantly increase due to poundings that occur when the available seismic gap is exceeded. Peak values of interstory deflections and absolute floor accelerations are plotted in fig. 3, considering two different widths of the seismic gap, specifically 18 cm and 25 cm, and compared with the corresponding values of the fixed-supported and base-isolated building without impact. In the case of poundings, interstory deflections and total floor accelerations become higher than the corresponding peak responses of the fixed-supported building (fig. 4). Due to poundings with the moat wall, the structure may experience maximum floor accelerations at the isolation level, instead at the top-floor of the building. It is evident that poundings may change the mode of



deformation of a seismically isolated building, exciting higher modes of the structure, instead of moving as an almost rigid-body according to its fundamental mode.

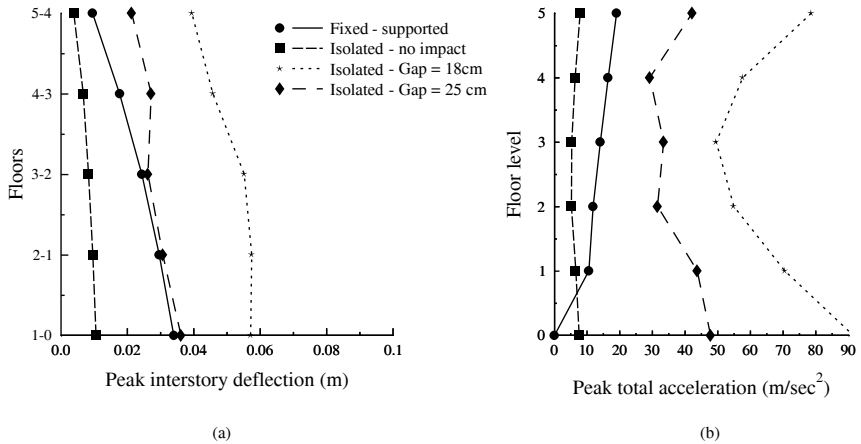


Figure 3: (a) Maximum interstory deflections; and (b) maximum absolute floor accelerations for the 5-story building under the Northridge earthquake.

The peak responses of the seismically isolated structure with the separation gap equal to 18 cm for the three impact models are presented in Table 1. In general, the differences are very small, as concerns the computed response.

Table 1: Peak responses of the 5-story structure under Northridge Earthquake with gap = 18 cm for the three different impact models.

Peak Response	Kelvin-Voigt	Hertzian Viscoelastic	Modified Kelvin-Voigt
Base floor displacement [cm]	22.189	21.310	22.227
Top floor displacement [cm]	44.003	43.611	44.050
Interstory deflection [cm]	5.738	5.769	5.737
Total acceleration (top floor) [m/sec ²]	78.856	75.451	78.553
Total acceleration (base floor) [m/sec ²]	91.866	102.660	91.338
Remaining plastic deformation [cm]	----	----	0.412



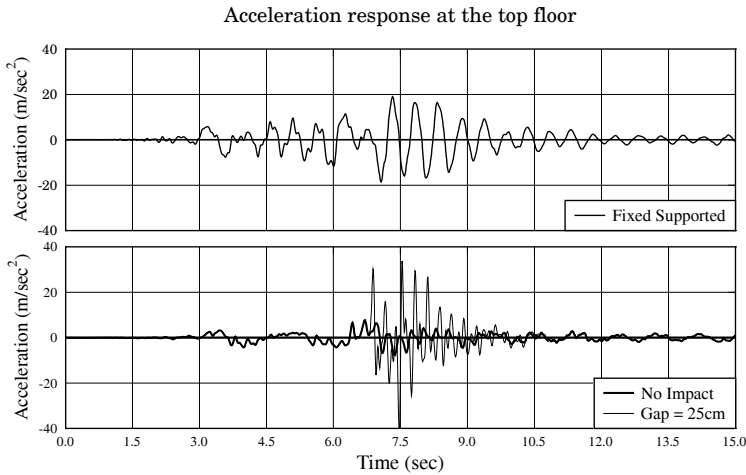


Figure 4: Acceleration time history response for Northridge earthquake.

4.2 Parametric analysis

The evaluation of the stiffness and damping parameters of the force-based impact models poses a major difficulty, as very limited experimental results are available to validate the proposed impact models. In order to examine the effect of the impact stiffness and the coefficient of restitution on the peak response of the seismically isolated building during poundings, a series of parametric studies has been performed.

In particular, fig. 5 shows the peak floor accelerations and interstory deflections of the 5-story seismically isolated building under the Northridge earthquake, assuming a seismic gap equal to 18 cm and considering the modified linear viscoelastic impact model to simulate poundings. Figure 6 shows the corresponding results using the Hertzian viscoelastic impact model. In general, for very low values of the impact stiffness, the response is increasing with k_{imp} for both linear and non-linear impact models. For higher values, the response remains almost insensitive to the variation of impact stiffness except for the acceleration response at the isolation level which substantially increases with this parameter.

The value of the coefficient of restitution also seems to affect the acceleration response at the isolation level, especially for the non-linear viscoelastic impact model. Specifically, for low values of COR (less than about 0.5) the damping ratio (eq. (7)) becomes larger than 1.0, rendering the impact highly overdamped and causing high local acceleration response during impact. In contrary, the corresponding plot for the linear viscoelastic impact model (fig. 5) shows that this effect of COR is not so pronounced. The rest of the response at the upper floors seems to be quite insensitive to the variation of the coefficient of restitution.



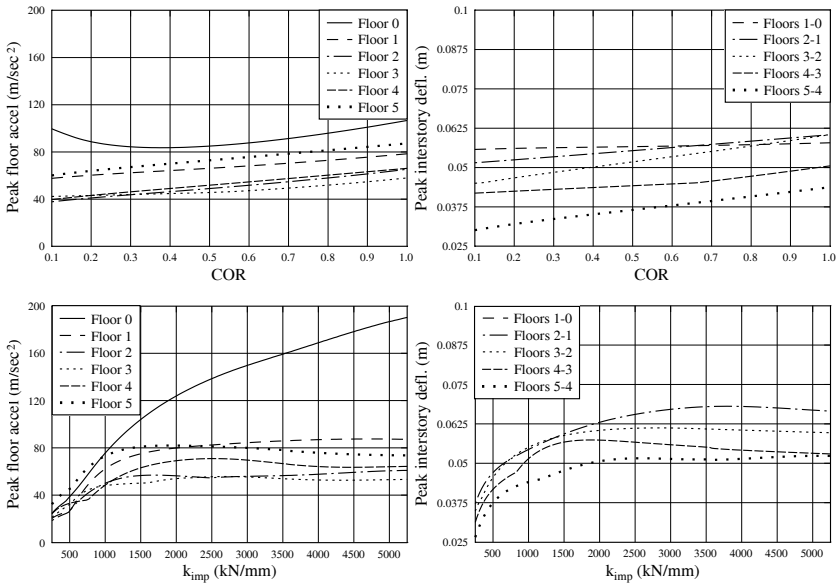


Figure 5: Influence of the coefficient of restitution (COR) and the impact stiffness (k_{imp}) on the peak floor accelerations and interstory deflections considering the modified linear viscoelastic impact model.

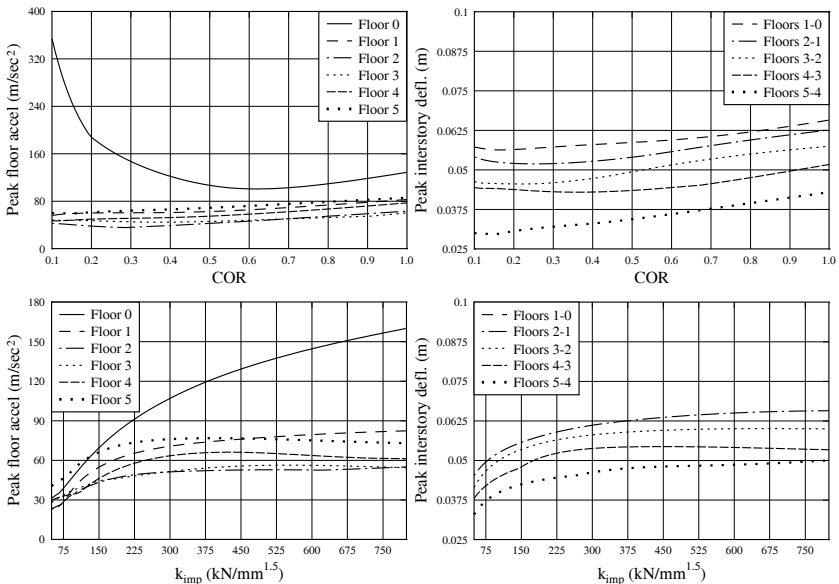


Figure 6: Effect of COR and k_{imp} on the peak floor accelerations and interstory deflections considering the Hertzian viscoelastic impact model.



5 Conclusions

Poundings substantially increase both floor accelerations and interstory deflections of seismically isolated buildings subjected to a strong seismic excitation. In the present study both the linear and the non-linear viscoelastic structural impact models are investigated and a modified impact model is proposed that takes into account remaining plastic deformations during poundings while avoiding tensile forces after detachment. The impact stiffness parameter, using both linear and non-linear impact models, seems to highly affect floor accelerations at the isolation level where impacts occur. The rest of the responses are slightly affected by the variation of impact stiffness after a certain value. Very low values for the coefficient of restitution increase substantially the peak floor accelerations at the level of impact when using the non-linear viscoelastic impact model.

Acknowledgements

The authors would like to thank the European Commission for funding the corresponding research proposal EIPOSIS#014591 (Earthquake-Induced Poundings of Seismically Isolated Structures) under the Marie Curie IRG/ENG action of the FP6.

References

- [1] Anagnostopoulos, S.A. Pounding of buildings in series during earthquakes. *Earthquake Engineering and Structural Dynamics* 1988, **16**, pp. 443-456.
- [2] Jankowski, R. Non-linear viscoelastic modelling of earthquake-induced structural pounding. *Earthquake Engineering and Structural Dynamics*, **34**, pp. 595-611, 2005.
- [3] Muthukumar, S. & DesRoches, R. A Hertz contact model with non-linear damping for pounding simulation. *Earthquake Engineering and Structural Dynamics*, **35**, pp. 811-828, 2006.
- [4] Jankowski, R. Analytical expression between the impact damping ratio and the coefficient of restitution in the non-linear viscoelastic model of structural pounding. *Earthquake Eng. and Struct. Dynamics*, **35**, pp. 517-524, 2006.
- [5] Tsai, H.C. Dynamic analysis of base-isolated shear beams bumping against stops. *Journal of Earthquake Engineering and Structural Dynamics*, **26**, pp. 515-528, 1997.
- [6] Malhotra, P.K. Dynamics of seismic impacts in base-isolated buildings. *Earthquake Engineering and Structural Dynamics*, **26**, pp. 797-813, 1997.
- [7] Matsagar, V.A., Jangid, R.S. Seismic response of base-isolated structures during impact with adjacent structures. *Engineering Structures*, **25**, pp. 1311-1323, 2003.
- [8] Van Mier, J.G.M., Puijssers, A.F., Reinhardt, H.W., Monnier, T. Load-Time Response of Colliding Concrete Bodies. *Journal of Structural Engineering* 1991; **117**:354-374.



Section 4

Passive protection devices and seismic isolation

This page intentionally left blank

Aseismic study of a building with the efficiency-enhanced damping system

S. S. Ke¹, W. S. Li¹ & B. J. Shih²

¹*National Science and Technology Center for Disaster Reduction, Taiwan*

²*Department of Civil Engineering,*

National Taipei University of Technology, Taiwan

Abstract

The objective of the present paper is to demonstrate the effectiveness of the efficiency-enhanced damping system (EDS) on the reduction of seismic vibrations of building structures with the device. Due to the limited damper stroke, the application of linear fluid dampers on buildings is restrained and dampers are usually installed as the diagonally auxiliary member. The EDS consists of linear fluid dampers and relatively rigid linking members to formulate a leverage mechanism for improving the capacity of damper efficiency by increasing the input velocity of the damper according to the arm ratio between the connecting lengths of the structural member and dampers. Through the adjustment of arm ratio, the input velocity of the damper will be magnified.

Two principal topics are focused on in this paper: first, to theoretically and numerically identify the feasibility and effectiveness of the EDS on the improvement of aseismic capability of buildings; second, to verify the accuracy of results and the related limitations between the theoretical analysis and numerical simulation through shaking table experiments. From the experimental observation and simulation results, the damping ratio of building structures will be largely increased and the seismic responses of the model building will be obviously reduced by adopting the EDS. As a consequence, the heavily damped structure systems do show a lot of potential for this application in civil engineering.

Keywords: efficiency-enhanced damping system, structure control, fluid damper, shaking table experiment, response spectrum analysis.



1 Introduction

It has been pointed out that the linear fluid damper is one of the promising energy dissipaters both in buildings and bridges for the following reasons [1]: (1) the low cost, (2) no external power supply is required, (3) the wide range of the operation temperature, (4) it is reliable and (5) it is almost maintenance free. In addition, due to the low axial stiffness when a damper is operated below its cut-off frequency and the damping coefficient remains constant, the dynamic analysis of a structure with linear fluid dampers remains linear. This makes it suitable for use in strengthening structures against ground motions.

According to the seismic design code in Taiwan, the relative story drift ratio should not exceed the 5/1000 of story height. For example, the maximum story drift will be 1.65 cm with 3.3m story height. If the behaviors of beam and column are confined in elastic range under earthquake, the allowable drift should be very limited. The efficiency and effectiveness of linear fluid damper will not be demonstrated, due to small stroke. To apparently increase the energy dissipation of damper, the EDS is proposed to meet the designed demand and provide a robust mechanism to expand further feasibility of the device.

The EDS consists of linear fluid dampers and relatively rigid linking members to formulate a leverage mechanism (shown in Figure 1) for improving the capacity of damper efficiency by increasing the input velocity of damper according to the arm ratio between the connecting lengths of structural member and dampers. Through the adjustment of arm ratio, the input velocity of damper will be magnified. The magnifying effect of EDS is simply shown in Figure 2 [2].

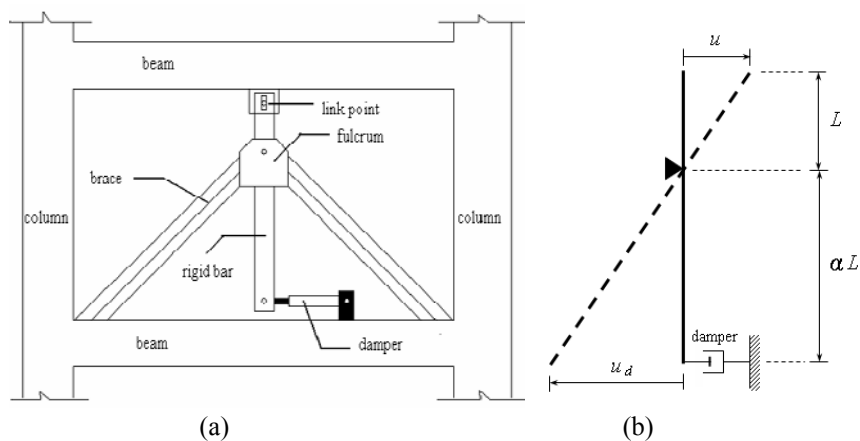


Figure 1: (a) Building with added damper and linking members. (b) Function mechanism of EDS.

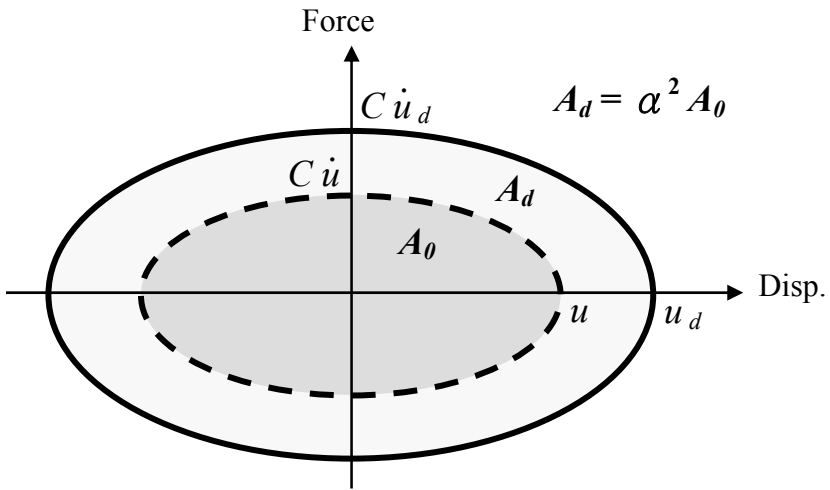


Figure 2: The conceptual illustration of EDS's efficiency.

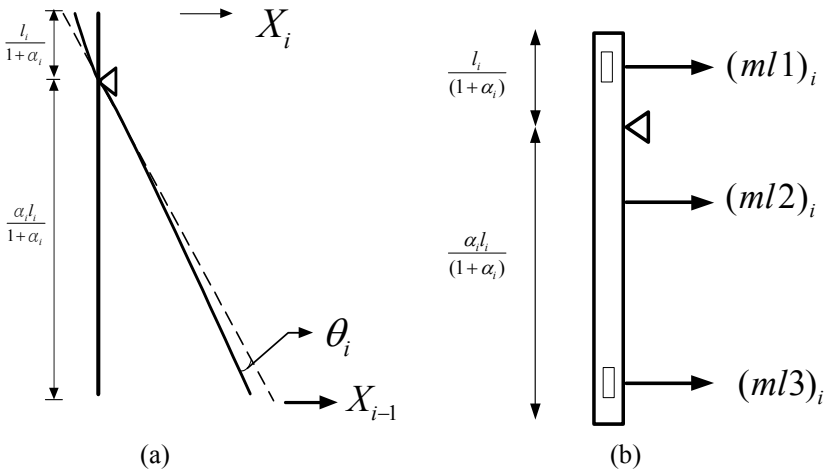


Figure 3: (a) Coordinates system of leverage mechanism. (b) Mass distribution of leverage.

2 Theoretical formulation

The derivation of EDS's equations of motion to formulate the relative deformations between upper and lower decks is depicted as the coordination system shown in figure 3 [3].



In equation (1) representing the kinematic energy of system, the components are composed of constitution from deck movement plus translational and rotational motion of leverage. For a simplified model, only the movements relative to leverage pivot are taken into account.

$$\begin{aligned}
 T = & \sum_{i=1}^N \left(\frac{1}{2} M_i \dot{X}_i^2 + \frac{1}{2} (ml2)_i \dot{X}_{i-1}^2 \right) \\
 & + \sum_{i=1}^N \frac{1}{2} (ml3)_i \left[\alpha_i (\dot{X}_i - \dot{X}_{i-1}) - \frac{2\alpha_i \dot{\theta}_i l_i}{(1+\alpha_i)} - \dot{X}_{i-1} \right]^2 \\
 & + \sum_{i=1}^N \frac{1}{2} (I_\alpha)_i \left[\frac{(1+\alpha_i)}{l_i} (\dot{X}_i - \dot{X}_{i-1}) - \dot{\theta}_i \right]^2 \quad (1)
 \end{aligned}$$

θ_i : the leverage rotation in i th floor

N : number of story

l_i : the leverage length in i th floor

$\dot{X}_i$: the relative velocity in i th floor

X_i : the absolute displacement in i th floor

$\dot{X}_0$: the absolute velocity at rigid base

X_0 : the absolute displacement at rigid base

$M_i = (m_c)_i + (m_s)_i + (ml1)_i$

m_c : column mass m_s : deck mass

α_i : the arm ration in i th floor

$(I_\alpha)_i = \frac{(\alpha_i^2 - \alpha_i + 1)ml_i^2}{3(\alpha_i + 1)^2}$: the moment inertia in i th floor

$ml1, ml2, ml3$: the leverage mass

From equation (2), the total potential energy includes collection from floor deck, damper and deformation of leverage.

$$\begin{aligned}
 V = & \sum_{i=1}^N \frac{1}{2} K_i (X_i - X_{i-1})^2 + \sum_{i=1}^N \frac{1}{2} (k_d)_i \left[\alpha_i (X_i - X_{i-1}) - \frac{2\alpha_i \theta_i l_i}{(1+\alpha_i)} \right]^2 \\
 & + \sum_{i=1}^N \frac{1}{2} k_i \left[\frac{l_i \theta_i}{(1+\alpha_i)} \right]^2 + \sum_{i=1}^N \frac{1}{2} \frac{k_i}{\alpha_i^3} \left[\frac{\alpha_i l_i \theta_i}{(1+\alpha_i)} \right]^2 \quad (2)
 \end{aligned}$$



K_i : the stiffness of i th floor

k_d : the stiffness of linear fluid damper

k_i : the stiffness of leverage arm in i th floor

The energy dissipation, Δw in equation (3), is the sum of damping from structural members and dampers. If leverage device is rigid enough to omit the deformation and rotational velocity, a multiplier, $\alpha^2 C_a$, means the folds to improve the performance of damper device.

$$\Delta w = - \sum_{i=1}^N C_i (\dot{X}_i - \dot{X}_{i-1})(X_i - X_{i-1}) - \sum_{i=1}^N (C_a)_i \left[\alpha_i (\dot{X}_i - \dot{X}_{i-1}) - \frac{2\alpha_i \dot{\theta}_i l_i}{(1 + \alpha_i)} \right] \cdot \left[\alpha_i (X_i - X_{i-1}) - \frac{2\alpha_i \theta_i l_i}{(1 + \alpha_i)} \right] \quad (3)$$

C_i : the damping in i th floor

C_a : the damping of linear fluid damper

By Lagrange equation, the motion equation of building structure with EDS could be derived in equation (4). The detailed content of each matrix can be consulted in the reference [3].

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + [K]\{U\} = -[M]\ddot{X}_0 \quad (4)$$

3 Numerical simulation

A 10-story numerical model with 5 and 4 bays in both directions is established to explore the seismic response by SAP2000, as shown in Figure 4. In the model, the connection joint at girder and leverage arm is hinge type with displacement constraints in X and Y directions. Other hinge type connectors, free rotating in Y axis, are located at anchorages of damper's both ends connecting to rigid leverage and braced seat [4].

In numerical case study, Chi-Chi Earthquake (1999, Taiwan) is chosen as the input ground motion with 0.33g PGA to verify the efficiency of EDS. One of the numerical cases, whose EDS's arm ratio (α) is adjusted to 3 by 24 damper (damping coefficient $C=350\text{KN/m/s}$), is compared with empty structure. The damping ratios for with- or without- EDS are 30% and 5% respectively. From numerical results, in figure 5, the maximum story drift ratio decreases from 8.62/1000 to 4.97/1000 and maximum share force from 24,428KN to 14,633KN.



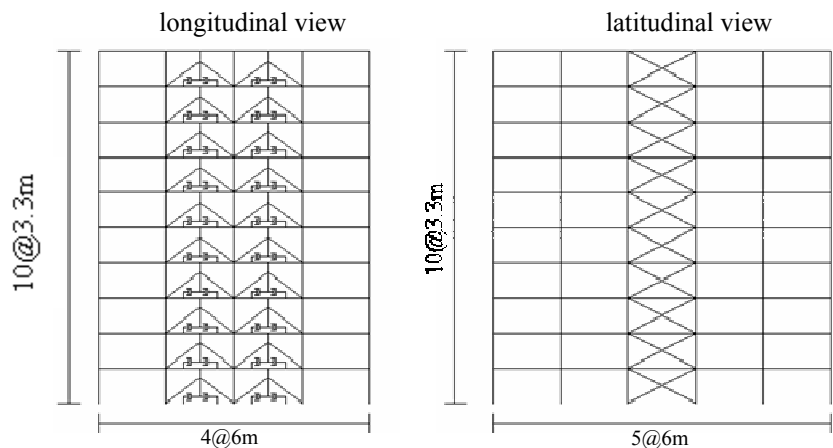


Figure 4: 10-story model with longitudinal and latitudinal views.

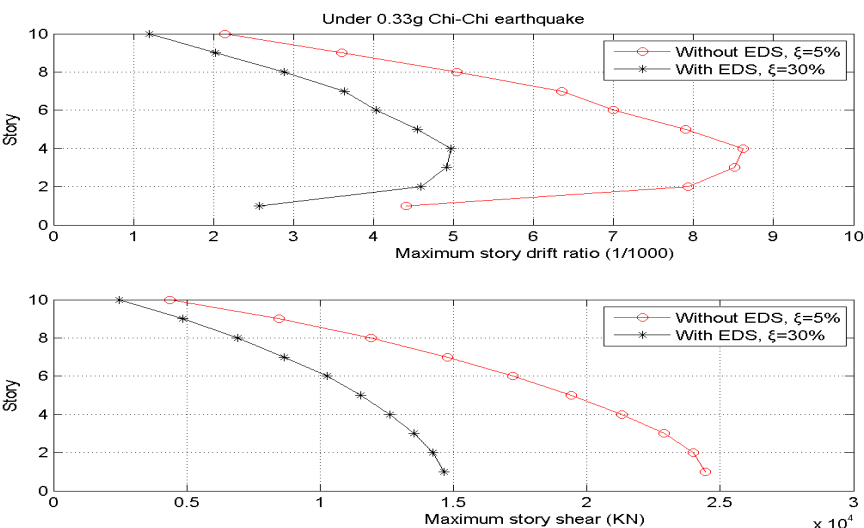


Figure 5: Story drift ratio and story shear of 10-story building model with/without EDS under 0.33g Chi-Chi earthquake.

4 Shaking table experiments

To verify the effectiveness in reductions of relative displacement and shear of the damping system, a three-story building model (full-scale steel structure) with EDS is set up for bi-axial shaking table tests. The building model is 4.5m in length (x-direction), 3m in width (y-direction), and 9m in height. The mass of the building model including two concrete blocks bolted in each story is 38-ton. The



1st and 2nd natural frequencies of building model are 1.17Hz (x-direction) and 3.67Hz (y-direction). Design and installation of EDS is shown in figures 6–7[5].

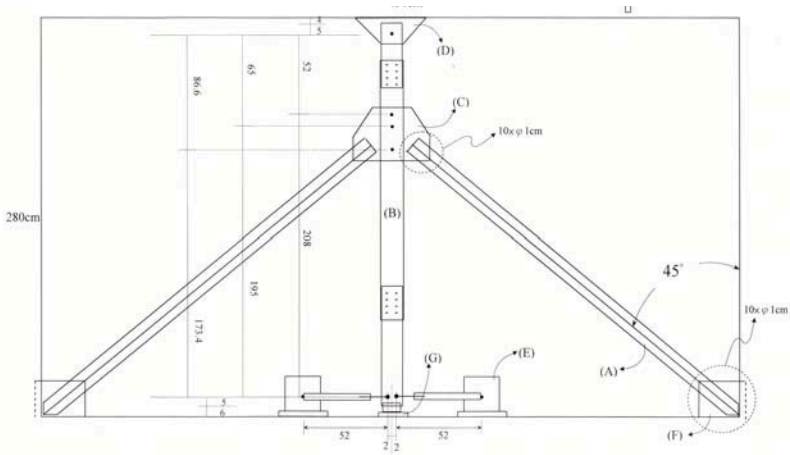


Figure 6: The sketch and dimensions of EDS. (Unit: cm.)



(a)



(b)

Figure 7: (a) Steel building model with EDS. (b) Installation of EDS.

During shaking table experiments, the applied damper has the dimensions of 66cm in length and 20cm in stroke. As compared with frequently used damper, the adopted one is relatively small. With the arm ratios of 2, 3, 4, the outcomes tell the limited number of damper in EDS will significantly enhance the damper ratio of 38-ton steel frame from 3% to 15%~35%. And the reduction ratio of relative displacement to first floor ranges from 62%~72%, depicted in figure 8. In figure 9, closeness between the test results and the numerical simulations is presented.



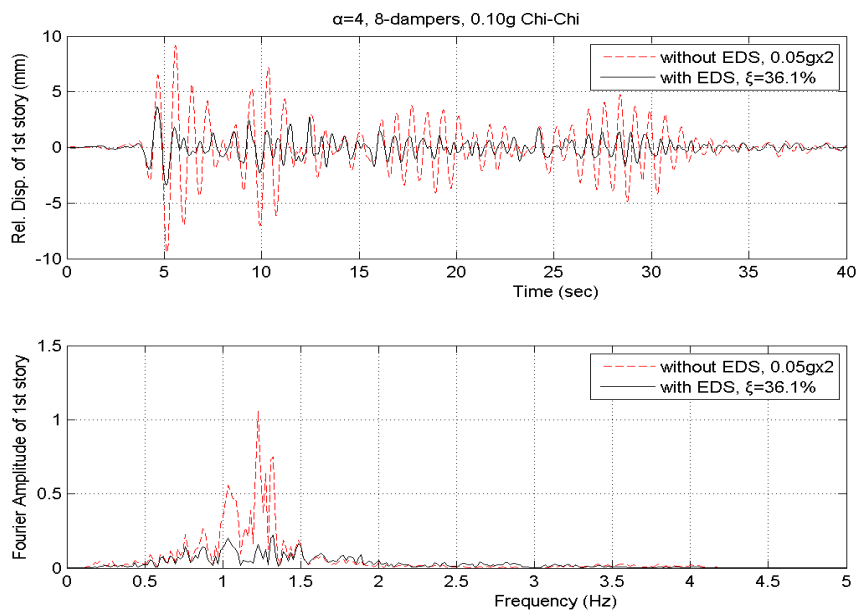


Figure 8: Responses of the building model with/without EDS under 0.1g Chi-Chi earthquake.

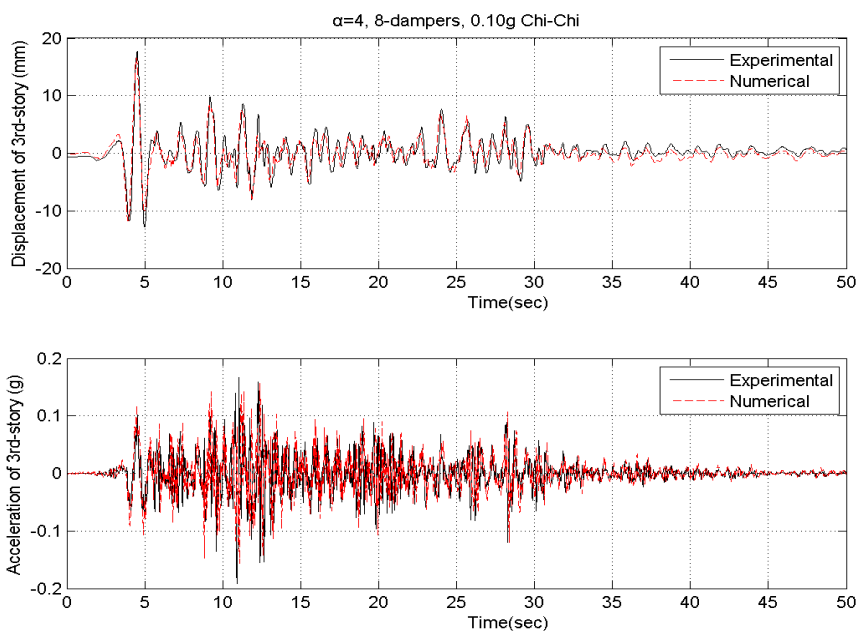


Figure 9: Numerical simulations and experimental results of the building model.



5 Conclusions

A new damping device system is proposed. Based on the results of the theoretical analysis, it is found that not only the maximum relative displacement but also the maximum base shear can be reduced significantly if the device system has been used on the building. In addition, the results of building model shaking table tests tell us that the aseismic capabilities of the building models are largely increased due to the energy dissipation characteristics of the device system. Further more the linear approach used in numerical analysis of the building model with EDS and corresponding results, also demonstrate the simplicity in analysis. In conclusion, it is suggested that this device system be applied in the design of new buildings as in the retrofitting of existing buildings with inadequate aseismic strength.

References

- [1] Shinozuka, M. et al, Passive and Active Fluid Dampers in Structural Applications, *US, China, Japan, Workshop on Structural Control*, pp.1-9, Shanghai, China, 1992.
- [2] Tang, J.P., Ke, S.S. & Ke, C.L., Aseismic Study of Heavily Damped Building Structure, *Journal of the Chinese Institute of Civil and Hydraulic Engineering*, 13(4), pp.793-804, 2001. (in Chinese)
- [3] Ke, S.S., Aseismic Study of the Building With the Efficiency-enhanced Damping System, *Ph.D. Thesis*, C.E. Dept., National Central University, Taiwan, 2004. (in Chinese)
- [4] Tang, J.P., Ke, S.S. & Wu, H.D., Seismic Retrofitting Study of the Building With the Efficiency-enhanced Damping System, *Structural Engineering*, Vol.19, No.3, pp.3-28, 2004. (in Chinese)
- [5] Tang, J.P., Ke, S.S. & Lai, H.K., The Building Equipped with the EDS Subjected to 2D Horizontal Shaking Table Tests and Analyses, *Structural Engineering*, Vol.18, No.1, pp.3-36, 2003. (in Chinese)



This page intentionally left blank

Introducing orthogonal roller pairs as an effective isolating system for low rise buildings

M. Hosseini¹ & K. Kangarloo²

¹*The International Institute of Earthquake Engineering and Seismology (IIEES), Tehran, Iran*

²*Graduate School, South Tehran Branch of the Islamic Azad University (IAU), and Member of Young Researchers Club, Tehran, Iran*

Abstract

In this paper a new isolating system is introduced which does not need sophisticated manufacturing techniques, and is not costly as other existing systems like lead-rubber bearing, or friction pendulum bearing systems. The proposed system consists of two pairs of orthogonal steel rollers, making possible the movement of the superstructure in all horizontal directions. Rollers move on a cylindrical steel bed, which gives a restoring capability to the system. The two rollers are connected together at both ends with two hinged plates. This makes the two rollers move together and have the same elevation in the cylindrical bed at any instant during the earthquake. The natural period of the system is almost independent of the superstructure mass, and is basically a function of r/R ratio in which r is the radius of the rollers and R is the radius of the cylindrical beds. To obtain the appropriate values of r and R to reach a specific value of the natural period of the isolated system, in addition to analytical hand calculations, some numerical Finite Element calculations have been performed. The calculations have been verified by laboratory tests. Results show that if the rollers and cylindrical beds are made of high-strength steel (MO40 alloy steel) the system can be used effectively buildings up to five stories.

Keywords: orthogonal rollers, base isolation, rolling and slope resistance, Hertz contact theory, transfer reaction, contact friction.



1 Introduction

Several isolating systems have been introduced for buildings so far, however, almost all of them need high technology on the one hand, which is not available in many of developing countries, or are still costly for those countries which have the required technology, and on the other hand, the isolators need to be replaced in the case of occurrence of a strong earthquake. Among various types of isolator, rollers have been shown to be to most usable ones, however, some difficulties like distortion of rollers and/or high rolling resistance under extensive loads have hindered their use. Accordingly, although the first applicable cases of these isolators have reported in the late 1980s, (Pham [1], Zastrow [2]), the studies on this type of isolators are ongoing (Tsai et al. [3]).

The idea of “two sets of mutually orthogonal free rolling rods under the basement of the structure” was proposed and tested in early- to mid 90s, (Lin et al. [4], Lin and Hone [5]), however, those cases were limited to light structures or single-story small models. Lin and his colleagues have added a soft spring to the basement to give a re-centering force to the rolling system and reduce the permanent displacement. A roller type isolation system has been also introduced for the individual showcases and individual works of art (Ueda et al. [6]). Their isolation system consists of two layers that form a XY-motion mechanism; each layer consists of wheels, and rails having a circular-linear-combined shape in the vertical cross-section to produce a restoring force. Ueda and his colleague have carried out some shake table tests were, showing good isolation performance of system. Although their system has shown high efficiency, because of using rails and wheels its load bearing capacity is low, and the main usage of their isolating system is in museums objects. Recently, another type of roller isolation system has been proposed by Uematsu et al. [7]. They have claimed that their system can be made of readily available materials and can be easily installed without heavy construction equipment or special skills, however, their system is again basically for light weight structures, and using it for buildings needs some modifications.

It is seen that although various roller isolating systems have been introduced, their capability for usage under high vertical loads in the range of vertical forces of multistory buildings columns is not so much developed yet. In this paper a somehow new isolating system is introduced which is suitable for using in multi-story buildings up to five stories. The specifications of this system are explained in the next section of the paper.

2 The proposed isolating system

The proposed system basically consists of two pairs of orthogonal steel rolling rods, which can move on separate cylindrical steel beds as shown in fig. 1.

It can be seen in fig. 1 that because of orthogonal setting of rollers pairs the top plate can move in all horizontal directions. The concave beds give a restoring capability to the system. The two rollers of each pair are connected together at both ends with two plates with hinge connections as shown in fig. 2.



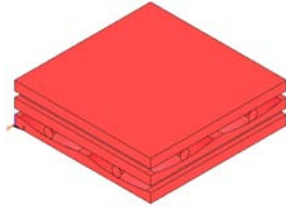


Figure 1: The proposed base isolation system with orthogonal rollers.

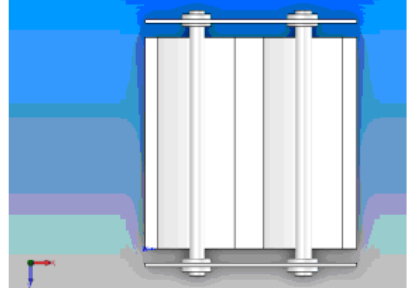
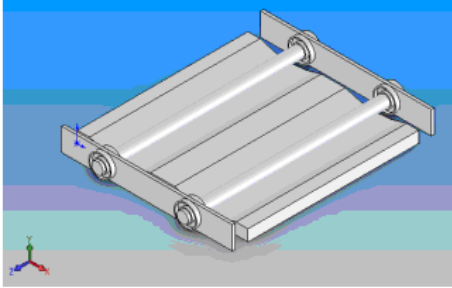


Figure 2: Plates connecting rollers for synchronized motion.

This makes the two rollers of each pair to move together and have the same elevation in the cylindrical beds at any instant during the earthquake, otherwise they may lose their parallel status because of seismic disturbances, particularly the vertical excitations of ground motions.

3 The system equation of motion

To drive the equation of motion for the isolated system, it is assumed that the superstructure moves on the isolating system as a rigid body with total mass of M as shown in fig. 3.

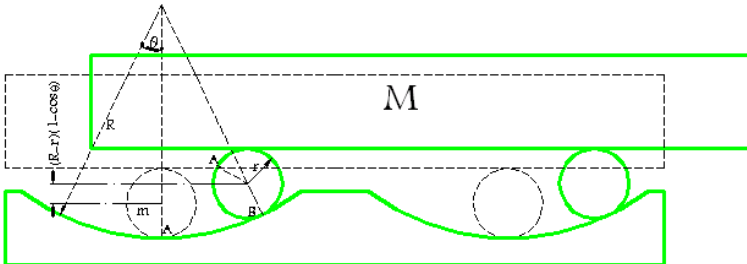


Figure 3: The rigid body model of the superstructure in the isolator.

The mass of rollers is negligible in comparison with M . If $\mu = \tan \theta$ (μ is coefficient of sliding friction and θ is the generalized coordinate for motion of



the system as shown in fig. 3), the rollers simply roll between surfaces without slippage. On this basis by using LaGrange equation, $\frac{d}{dt}(\frac{\partial T}{\partial \dot{\theta}}) - \frac{\partial T}{\partial \theta} + \frac{\partial V}{\partial \theta} = Q_{\theta}$, the differential equation of motion is obtained as:

$$\ddot{x} + \frac{g}{(R-r)}x = -\text{sgn}(\dot{x})\frac{b}{r}g - \ddot{u}_g \tag{1}$$

In eqn (1) x is the relative lateral displacement of the superstructure, R and r are respectively radius of bed and rollers, b is the coefficient of rolling resistan, explained in section 3, g is the gravity factor, and $\ddot{u}_g$ is the ground acceleration.

The period of the system, T_n , and its restoring force, p , are given by:

$$T_n = 4\pi\sqrt{\frac{(R-r)}{g}} \quad , \quad p = \frac{Mg}{(R-r)} \tag{2}$$

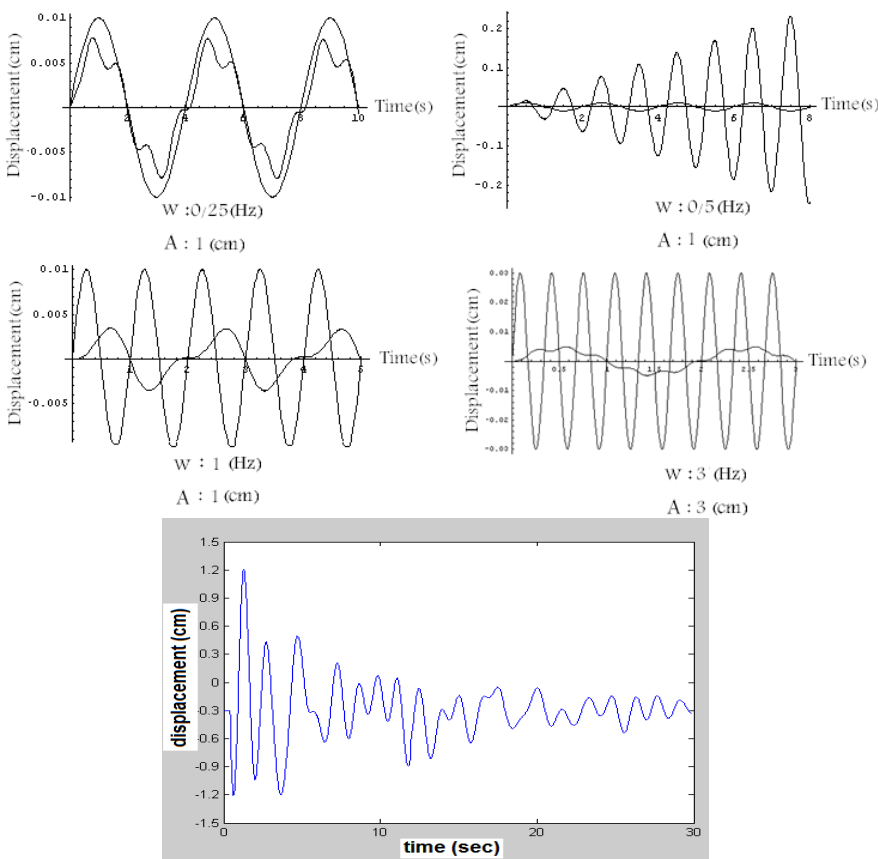


Figure 4: Responses of the system to harmonic and earthquake loadings.

As it is seen, the natural period of the system is independent of the superstructure mass. The main affecting parameters in the value of natural period of the isolated system are the radii of curvatures of rollers and bed. By assuming $r = 2$ cm and $R = 26$ cm, and solving the equation of motion for the harmonic force of $A \sin(\omega t)$, and also El Centro earthquake accelerogram (reduce to 50%) of the obtained responses are as shown in fig. 4.

It is seen that for $\omega = 0.5$ Hz the resonance response has occurred, since the period of the system, given by eqn (2) is 2.0 seconds in this case.

4 Rolling resistance

The major source of resistance during rolling motion is the deformations that occur when the two surfaces are compressed together. At the contact points, the roller flattens, while a small trench is formed in the surface (see fig. 5).

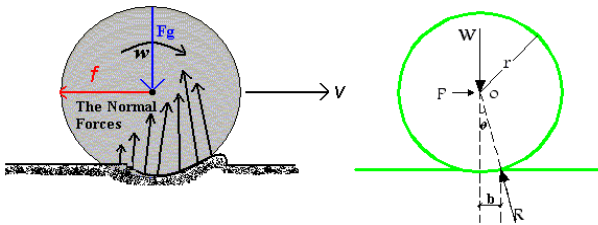


Figure 5: The actual forces acting between the rolling rod and the surface.

The overall rolling resistance or friction results in a force at the center of the wheel and is parallel to the surface of contact, and is represented eqns (3).

$$P.r = W.b \quad , \quad M_r = W.b.r \quad (3)$$

In eqn (3) b is called Rolling Resistance Parameter. It has the physical dimension of length and its value depends to several factors such as rotating speed, applied pressure or force, roughness of the surfaces, etc. that are not represented clearly. The value of b varies from 25 mm for steel wheel on the steel rail up to 125 mm or higher for the steel wheel on the ground.

5 Lateral stiffness of the system

Seismic design by using the Design Spectra needs two main parameters, the natural period and the damping coefficient. In the case of rolling systems these parameters are directly related to the lateral stiffness of roller bearings as:

$$T = 2\pi \sqrt{\frac{M}{K}} \quad , \quad \zeta = E / \left(4\pi \frac{KD^2}{2} \right) \quad (4)$$

where E is the energy dissipated by the bearing system in one sinusoidal cycle, D is the maximum lateral displacement, and K is the stiffness of the system ($K = F/D$). Force-displacement behavior of the roller bearing is shown in fig. 6.



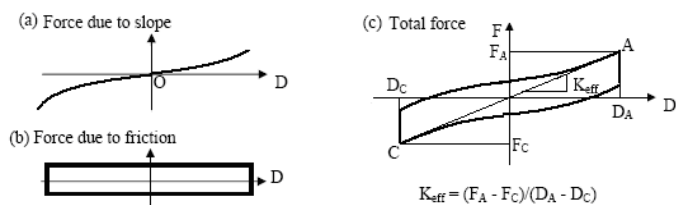


Figure 6: Force-displacement behavior of the roller bearing.

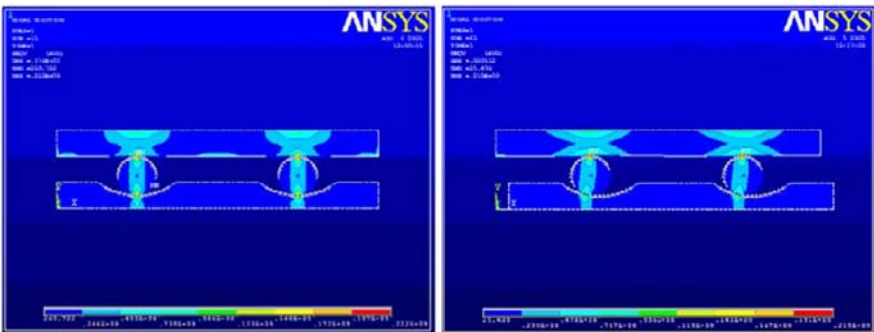


Figure 7: Von Mises stresses for the case of 1.0 cm lateral movement.

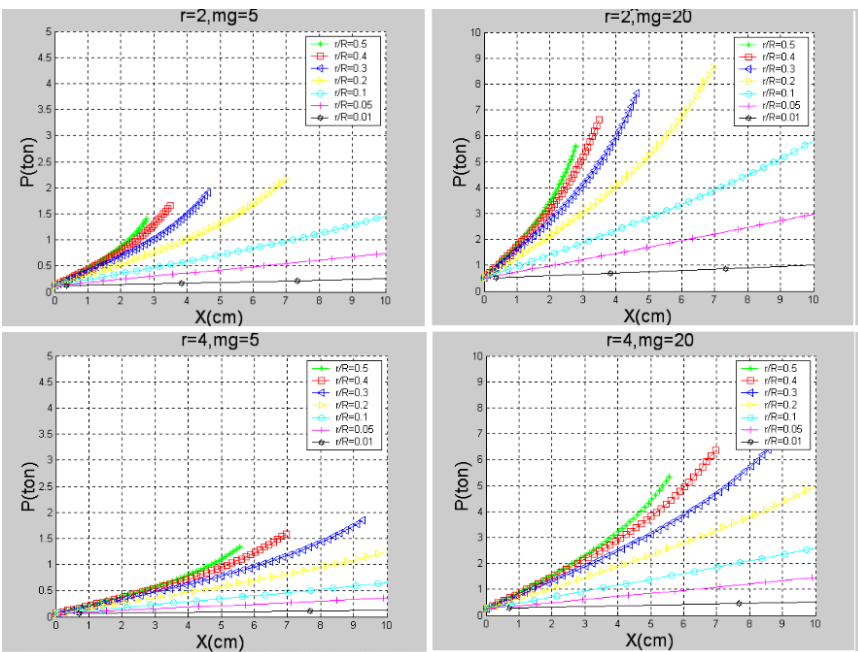


Figure 8: Force-displacement graphs of the system for different ratio of r/R .



For analytical stuffy of the lateral stiffness of the system, ANSYS program has been used making benefit from its large displacement option. Fig 7 shows the Von Misses stress values for the case of 1.0 cm lateral movement. The force-displacement graphs, obtained by analyses for different r/R values, varying from 0.01 to 0.5, using two values of 2 cm and 4 cm for r , and two values of 5.0 kgf and 20.0 kgf for the vertical load on the rollers are shown in fig. 8.

It is seen in fig. 8 that in all cases as the r/R value increases the system tends to nonlinear behavior, and shows a hardening state. The effect of higher vertical load on the lateral stiffness of the system is also observed in fig. 8, which is in good agreement with the calculated values obtained by eqn (4).

6 The vertical stiffness

When two elastic surfaces are pushed together a contact area is created with high stress in materials, for example, the settlement of a cylinder over another cylinder or a sphere over another sphere. These cases can be described by Hertz contact theory [9]. For two cylinders under a force F , as shown in fig. 9, the length of contact area and the peak pressure can be calculated by the following eqns:

$$a = \sqrt{\frac{2F(1-\nu^2) \left[\frac{1}{E_1} + \frac{1}{E_2} \right]}{\pi L \left[\frac{1}{d_1} + \frac{1}{d_2} \right]}}, \quad P_{\max} = \frac{2F}{\pi a L} \quad (5)$$

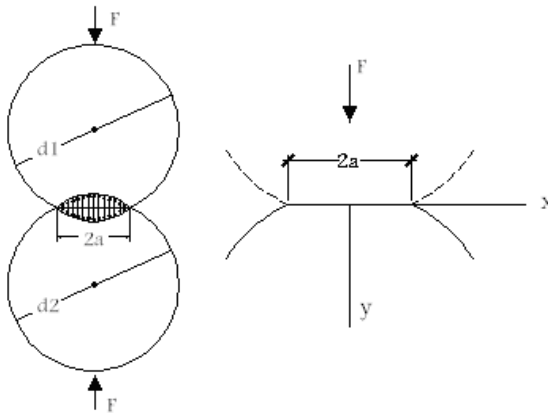


Figure 9: Hertz contact theory for two cylinders.

In eqns (5) ν is the Poisson ratio, and E_1 and E_2 are respectively the modulus of elasticity of the two materials, and L is the length of cylinders. In the case of a cylinder on a flat surface the radius of the flat surface is considered as ∞ in the formula, and in the case of a cylinder on the internal surface of another bigger cylinder, the radius of the bigger cylinder is used with the negative sign. In the latter case the max shear stress, created in the contact area is about 30% of the maximum normal stress, namely $0.3\sigma_{z,max}$, and occurs around $0.39a$ below the surface [9].

To find out the accuracy of finite element analyses a simple problem based on the above discussion has been solved by ANSYS program. Fig. 10 shows the results of this analysis in comparison with the analytical values obtained by Hertz contact theory [9]. As it is seen in fig. 10 there is a good agreement between numerical and analytical results. Of particular interest may be the maximum shear stress and its location which can be seen in the fig. clearly. Based on the good agreement between numerical and analytical results the calculations performed by using ANSYS program are verified, and the numerical results for other similar cases such as the case for calculating the lateral stiffness can be trusted. Accordingly, it is expected that the tests results shows good agreement with numerical results as well.

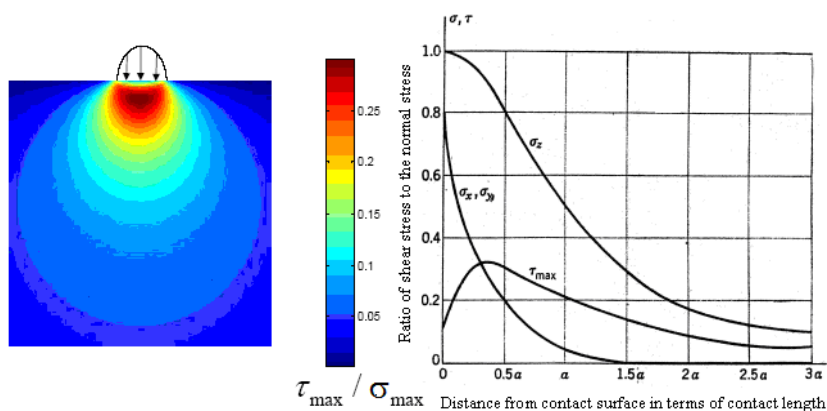


Figure 10: Comparing the numerical and analytical results of the contact problem.

7 The laboratory tests

For studying the performance of the proposed system in different directions and to find out the effect of superstructure's weight on the system performance, some shaking table and cyclic or pseudo-dynamic tests have been carried out. At first,



by using a rigid body model, as shown in fig. 11, behavior of the isolating system subjected to lateral excitations were studied.

Fig. 12 represents the response histories of the system in two orthogonal directions to the a series of harmonic excitations with frequencies of 0.25, 0.5, 1, 3, and 5 Hz, and also to the El Centro earthquake all applied to the system at an angle of 45 degrees with respect to the main axes of the rollers. The masses used in this test were 50 kg, 110 kg, and 400 kg, but no difference was observed in the results, as expected. It is seen in fig. 12 that in no case the displacement value has exceeded 5.0 cm, and in the case of earthquake the values are less than 1.0 cm. It can also be seen that the maximum displacements in two orthogonal directions are almost the same, although the time histories are not quite similar in those two directions.

During the first series of tests it was realized that the rollers can not remain parallel during their motion. It can be because of the imperfection in the production process of rollers and their beds, which have been resulted into some undesired motions as they roll in their bed. To overcome this difficulty two thin and narrow plate with hinge connections were added to the system (see fig. 2), and the next test were preformed on the improved system. In the next step of experimental studies, by using actuators, the sample model (made of high strength – MO40 steel alloy) was loaded with different vertical loads and at the same time was pushed and pulled laterally to see how the value of vertical load affect the lateral force-displacement of the system. The observed result, which can not be presented here due to lack of space, were in very good agreement with the numerical results shown in figs 6 and 7. The maximum applied vertical load was 700 kN, which in about the column force of a five story ordinary building.

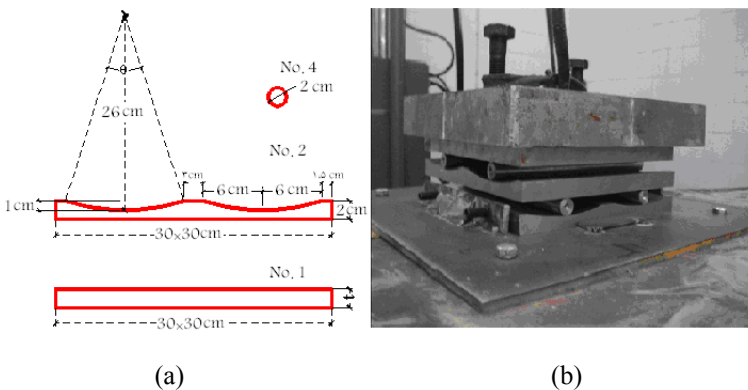


Figure 11: The first stage of the laboratory tests by rigid body model. (a) Geometry of the test model, (b) the test model on the shaking table.



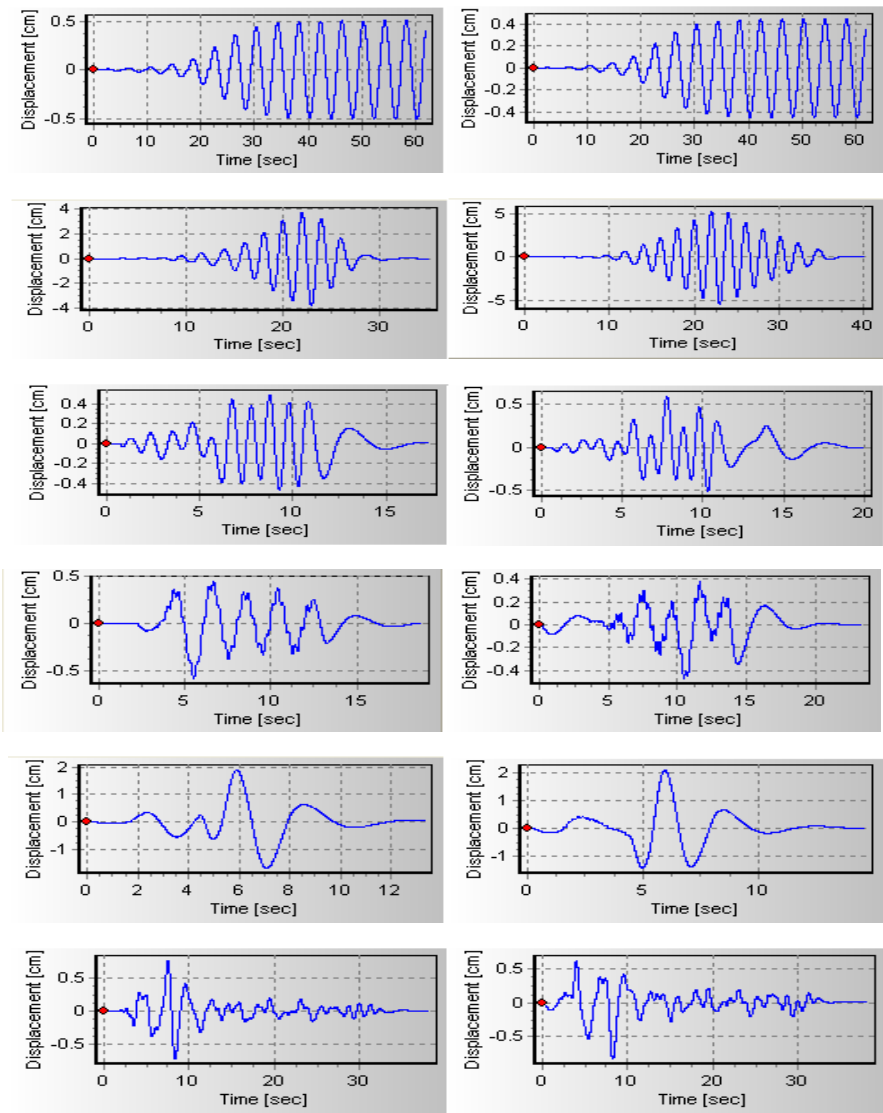


Figure 12: Response histories of the system to harmonic and earthquake loadings.

8 Conclusions

Based on the analytical, numerical and experimental results it can be said that:

- The system has good performance subjected to horizontal excitation in every direction.



- The natural period of the system can be easily controlled by the r/R value, and getting a period of 2.5 second or more is not difficult.
- The maximum lateral displacement of the system under earthquake excitations can be kept limited to a few centimeters by using higher values of r/R , provided that the dominant frequency of the earthquake is not low.

Finally, it should be noted that the dimensions of the base isolation systems are important, because there is always a desire for simplicity in installation and low cost. The range of horizontal motion of rollers should be at least equal to the spectral displacement used in the region for seismic design.

References

- [1] Pham, L. T., A base-isolation design using spherically ended rollers and telescopic shock absorbers, *Bulletin of the New Zealand National Society for Earthquake Engineering*. Vol. 21, no. 2, pp. 135-139, June 1988.
- [2] Zastrow, J. B., Rolling with the Big One: base isolation will cushion the jolting, *Northern California Real Estate Journal*. Vol. 4, no. 4, pp. 5-6. 20, Nov. 1989.
- [3] Tsai, Meng-Hao; Chang, Kuo-Chun; and Wu, Sih-Yi, Shaking Table Tests of a Scaled Bridge Model with Rolling-Type Seismic Isolation Bearings, *100th Anniversary Earthquake Conference*, 2006.
- [4] Lin, T. W., Chern, C. C. and Hone, C. C., Experimental study of base isolation by free rolling rods, *Earthquake Engineering & Structural Dynamics*, Vol. 24, no. 12, pp. 1645-1650, Dec. 1995.
- [5] Lin, T. W. and Hone, C. C., Base isolation by free rolling rods under basement, *Earthquake Engineering & Structural Dynamics*, Vol. 22, no. 3, pp. 261-273, Mar. 1993.
- [6] Ueda, Satoshi; Enomoto, Takao; and Fujita, Takafumi, Experiments and Analysis of Roller Type Isolation Device, *Proceedings of 13 WCEE*, 2004.
- [7] Uematsu, Takeyoshi; et al., Development of Compact Vibration Isolation Equipment Applicable to Existing Residences – Restoring Mechanism Utilizing Roller Bearings, *Proceedings of 13 WCEE*, 2004.
- [8] Liang, Z. Song, J. Wang, J. and Lee, G. C., A Sloping Surface Roller Bearing System for Seismic Isolation of Highway Bridges, *Technical Report, MCEER*, State University of New York at Buffalo, 2003.
- [9] Johnson, K. L., *Contact Mechanics*, Cambridge University Press, 1985.



This page intentionally left blank

Section 5
Self-centering systems
(Special session by
M. Elgawady)

This page intentionally left blank

Seismic response three-dimensional analyses of ten-story steel frames with column uplift

M. Midorikawa¹, T. Azuhata² & T. Ishihara²

¹*Division of Architectural and Structural Design,
Graduate School of Engineering, Hokkaido University, Japan*

²*National Institute for Land and Infrastructure Management, Japan*

Abstract

Previous studies have suggested that rocking vibration accompanied with uplift motion might reduce the seismic damage of buildings subjected to severe earthquake motions. In this paper, the three-dimensional seismic response of base-plate-yielding rocking systems with columns allowed to uplift is evaluated and compared with that of fixed-base systems by finite element numerical analyses. The study is carried out using ten-story, one-by-three bay steel frames of a base-plate-yielding rocking system. Base plates that yield due to tension of columns are installed at the base of each column. The earthquake ground motions are the JMA record of the 1995 Kobe Earthquake and a synthesized motion. The maximum input velocity is scaled to examine the structural response at 0.50 m/s. The main findings from this study are as follows 1) The base shear coefficients of the uplift model are reduced to 68% to 82% of the fixed-base model subjected to one-dimensional input motions in the horizontal direction and to 59% to 76% of the fixed-base model subjected to two-/three-dimensional input motions. 2) The horizontal roof displacements of the uplift model almost increase relative to the fixed-base model. The ratio of the uplift to fixed-base models is from 0.89 to 1.25 in the case of one-dimensional input motions, and from 0.78 to 1.30 in the case of two-/three-dimensional input motions. 3) While the girders of the fixed-base model yield in bending at the second to eighth floors, those of the uplift model yield in bending only at the second and third floors.

Keywords: *seismic response reduction, rocking vibration, steel frame, column uplift, yielding base plate.*



1 Introduction

It has been pointed out that the effects of rocking vibration accompanied with uplift motion may reduce the seismic damage of buildings subjected to strong earthquake ground motions [1, 2]. Based on these studies, structural systems have been developed which permit rocking vibration and uplift motion under appropriate control during major earthquake motions [3, 4].

A rocking structural system under development employs the yielding mechanism of base plates. When weak base plates yield due to tension of columns during a strong earthquake ground motion, the columns uplift and permit a building structure to rock. In this system, the yielding base plates dissipate some of the input seismic energy by the inelastic behaviour.

In this paper, the seismic response of a ten-story steel frame of base-plate-yielding rocking system is examined by the finite element analyses [5].

2 Analytical modelling and numerical analyses

A ten-story, one-by-three bay steel frame shown in Figure 1 was analyzed. The structure is modelled in two types of a three-dimensional frame; fixed-base model (nodal points of about 6000) and base-plate uplift (BPL) model (nodal points of about 7500).

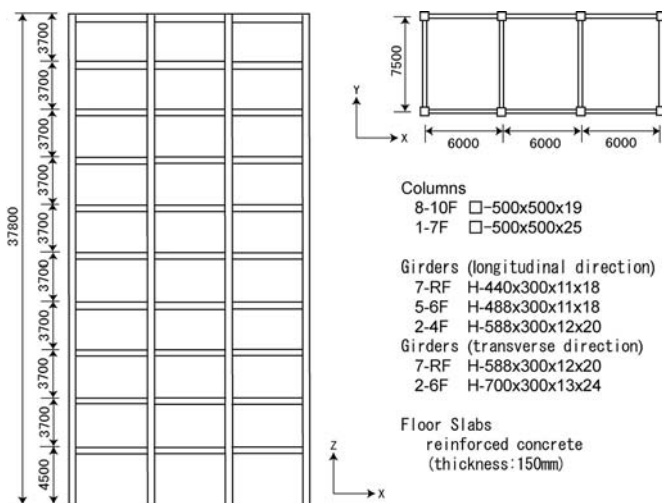


Figure 1: Ten-story steel frame (unit: mm).

The base plates and the columns of the first story are modelled using shell elements. The columns and girders at the second and upper stories are modelled using beam elements. The foundation beam is assumed to be rigid.



The yielding base plate shown in Figure 2 is fixed at each outside end of wing plate. Contact elements are employed between the base plates and the rigid foundation beam. The contact conditions such as the normal contact force and the tangential contact slip without friction are considered between the rigid foundation beam and the shell elements of base plates.

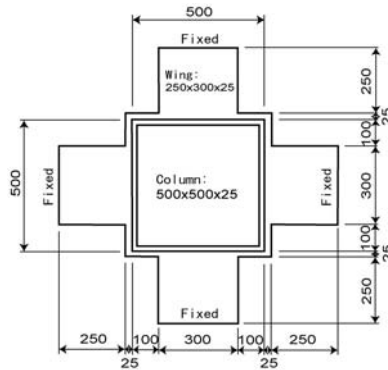


Figure 2: Plan of yielding base plate (unit: mm).

The base plate and the first story column are modelled with an elasto-plastic material considering a kinematic hardening rule with the Mises-Hencky yield condition. The characteristics values of steel are assumed; Young's modulus = 2.06×10^8 kN/m², post-yielding modulus = 2.06×10^6 kN/m², yield strength = 2.94×10^5 kN/m², Poisson's ratio = 0.3 and specific gravity = 7.8. The tri-linear moment-curvature relation is assumed in the columns and girders at the second and upper stories.

The reinforced-concrete floor slab of 150 mm thickness is modelled using two-dimensional stress elements that are connected to beam-to-column joints.

The weight of each floor is assumed to be 1150 kN. The masses of the analytical model are lumped at each nodal point of girders. The vertical components of masses are defined in order to capture vertical inertia effects associated with rocking. The vertical load corresponding to the lumped masses is applied to each node of the analytical model before starting the dynamic response analyses.

It is assumed that the viscous damping results from the initial stiffness-dependent effects. The critical damping ratio of 2%, that is stiffness-proportional type, is introduced to the first mode corresponding to the fixed-base model.

The numerical time integration in the analyses is the combined use of the Newmark method with constant acceleration and the Newton-Raphson method for equilibrium iteration within the time step of 0.01 second. The synthesized ground motion BCJ-L2 and the 1995 JMA Kobe record that are normalized in the maximum ground velocity of 0.50 m/s, are used as input for the dynamic response analyses. The duration is thirty seconds in the analyses. The JMA Kobe record is used in the analyses subjected to one-, two- and three-dimensional input motions, in which the NS and EW components are applied to the transverse and longitudinal directions of the analytical model, respectively.



3 Results and discussion

3.1 Pushover analyses

Figure 3 shows the relationships between the base shear and the roof displacement obtained from the pushover analyses for the fixed-base and BPL models. The base shear coefficient of the fixed-base and BPL models at the roof drift angle of 1/100 are 0.41 and 0.27 in the transverse direction, and 0.37 and 0.30 in the longitudinal direction, respectively. In the transverse direction, the base shear coefficient of the BPL model is 0.16 at base-plate uplift yielding, and 0.13 in the simple uplift model without base plates.

Although the base shear coefficient of the BPL model at the roof drift angle of 1/100 is much smaller than that of the fixed-base model, the increase of the base shear coefficient of the BPL model in the transverse direction is larger than the fixed-base model because of the hardening effects in the inelastic behaviour of yielding base plates.

Furthermore, the maximum responses from the dynamic analyses are plotted in Figure 3. There are some differences between the seismic and pushover analytical results because of the higher mode effects.

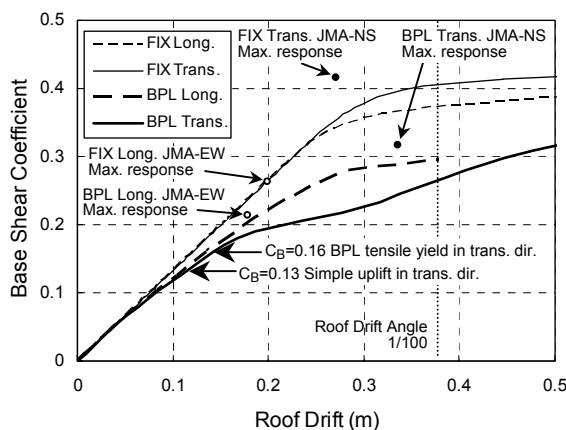


Figure 3: Base shear versus roof displacement.

3.2 Seismic response analyses

The natural periods of the fixed-base model are 1.26 seconds for the first mode in the longitudinal direction, 1.25 seconds for the second mode in the transverse direction and 1.04 seconds for the third mode in torsion, and those of the BPL model are 1.62 seconds for the first mode in the transverse direction, 1.42 seconds for the second mode in the longitudinal direction and 1.20 seconds for the third mode in torsion.

Figure 4 shows the time histories of the responses of the BPL model subjected to the three-dimensional input motion of the JMA Kobe record.



Figure 5 shows the corresponding time histories obtained from the analyses subjected to the one-dimensional input motion.

The maximum responses of the fixed-base and BPL models are summarized in Table 1. From this table, it is pointed out that:

- 1) The base shear coefficients of the BPL model are reduced to 68 to 82% of the fixed-base model subjected to one-dimensional input motions and to 59 to 76% of the fixed-base model subjected to two-/three-dimensional input motions.
- 2) The horizontal roof displacements of the BPL model almost increase relative to the fixed-base model. The ratio of the BPL to fixed-base models is from 0.89 to 1.25 in the case of one-dimensional input motions, and from 0.78 to 1.30 in the case of two-/three-dimensional input motions. However, the ratios in the case of two-/three-dimensional input motions are between 1.14 and 1.15 when comparing in a vector sum in the two horizontal directions, and is therefore within the values in case of one-dimensional input motions.

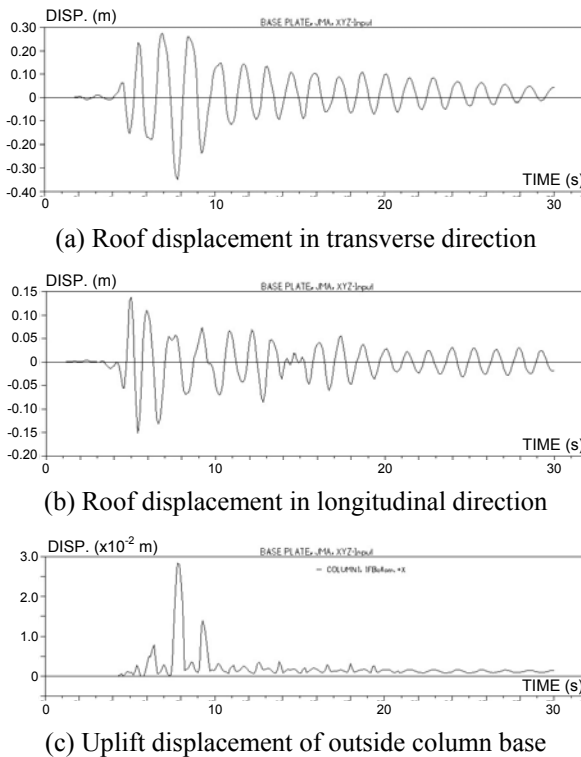


Figure 4: Time histories of displacement response of BPL model subjected to three components of JMA record.

- 3) The horizontal roof accelerations of the BPL model are reduced when compared to the fixed-base model. The ratio of the BPL to fixed-base models is from 0.72 to 1.01. On the contrary, the horizontal roof velocities are almost the same in two models.



- 4) The uplift displacement of the BPL model is 35 mm in maximum, and is approximately 1/170 of the span length.
- 5) The velocities at the base of the BPL model are from 150 to 300 mm/s.

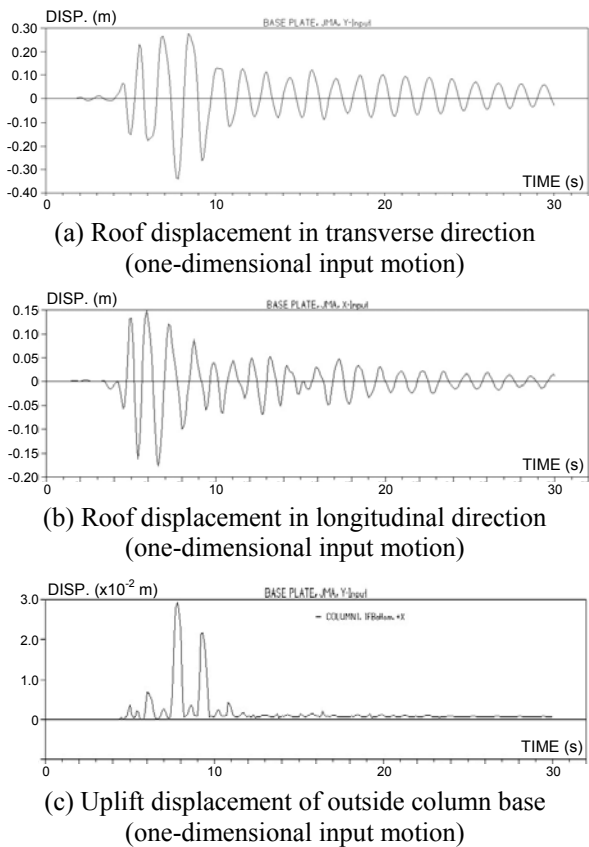


Figure 5: Time histories of displacement response of BPL model subjected to one component of JMA record.

- 6) The uplift displacement of the BPL model results in the remarkably large cumulative plastic strain in the wing plate of the base plate, whose maximum values are from 25 to 80% in the case of one-dimensional input motions and from 37 to 38% in the case of two-/three-dimensional input motions. The location of the maximum value is the column-side end of the wing plate of the base plate. According to the static loading test results of yielding base plates [6], the maximum cumulative plastic strain reaches 138% in the test base plate with thickness of 25 mm and over 88% to 163% in the test base plate with thickness of 19 mm. Consequently, the maximum cumulative plastic strain obtained from the analyses are kept within the ultimate capacity.



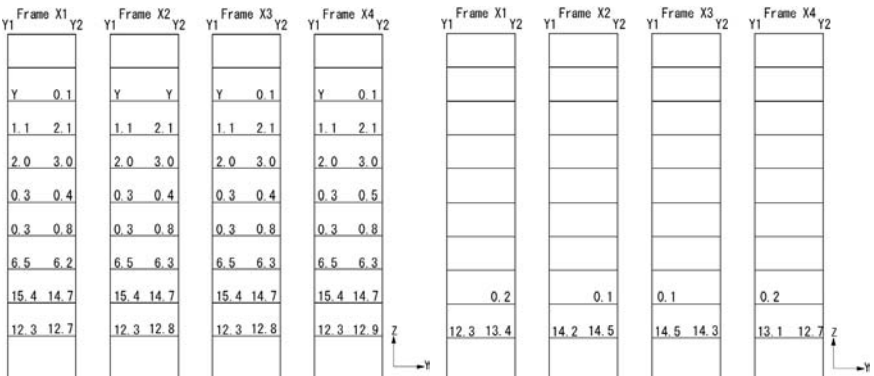


Table 1: Maximum response values of frame models.

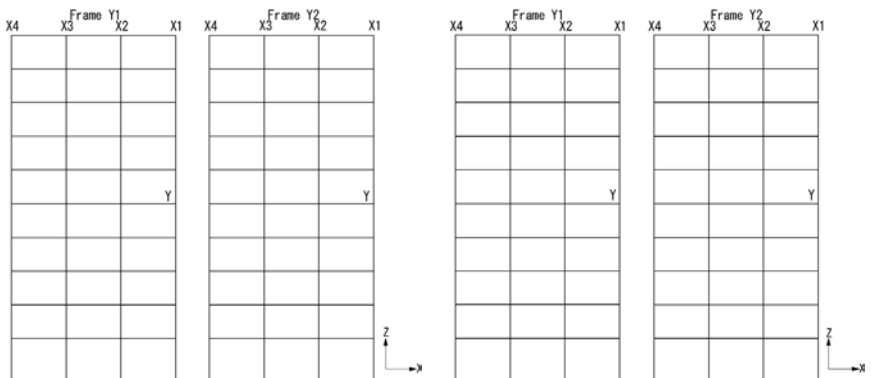
Input	Condition		Roof				Base			
	Base	Direction	Δ_h *1 (mm)	Δ_v *2 (mm)	Vel. (m/s)	Acc. (m/s ²)	δ_{up} *6 (mm)	V_{dn} *7 (mm/s)	C_b *8	$\Sigma \epsilon_b$ *9 (%)
BCJ	Fix	Trans.	333	6.26	1.48	9.91	-	-	0.431	-
BCJ	BPL	Trans.	356	28.2	1.46	7.15	28.9	261	0.291	80.5
BCJ	Fix	Long.	322	4.67	1.56	10.40	-	-	0.401	-
BCJ	BPL	Long.	354	19.8	1.45	8.48	24.8	213	0.311	69.5
JMA	Fix	Trans.	272	5.93	1.95	9.57	-	-	0.421	-
JMA	BPL	Trans.	339	29.9	1.53	9.68	29.4	296	0.319	25.2
JMA	Fix	Long.	198	3.33	1.15	6.29	-	-	0.262	-
JMA	BPL	Long.	177	6.56	1.04	5.46	4.09	37.7	0.214	1.30
JMA (2com) *3	Fix	Trans.	270	6.77	1.95	9.57	-	-	0.411	-
		Long.	(0.99) *5 193 (0.97)		(1.00) *5 1.15 (1.00)	(1.00) *5 6.18 (0.98)			(0.98) *5 0.249 (0.95)	
JMA (2com)	BPL	Trans.	347	36.5	1.56	9.40	32.3 [0.96] *10	175	0.311	36.9
		Long.	(1.02) 151 (0.85)		(1.02) 1.03 (0.99)	(0.97) 4.69 (0.86)			(0.97) 0.155 (0.72)	
JMA (3com) *4	Fix	Trans.	270	6.67	1.95	9.58	-	-	0.411	-
		Long.	(0.99) 193 (0.97)		(1.00) 1.15 (1.00)	(1.00) 5.94 (0.94)			(0.98) 0.250 (0.95)	
JMA (3com)	BPL	Trans.	350	31.5	1.56	9.43	35.1 [1.05]	148	0.306	35.0
		Long.	(1.03) 151 (0.85)		(1.02) 1.03 (0.99)	(0.97) 4.68 (0.85)			(0.96) 0.147 (0.69)	

Notes) *1: horizontal roof displacement, *2: vertical roof displacement, *3: horizontal two-component input motion, *4: three-component input motion, *5: ratio of the value in the case of two-/three-dimensional input motion to the one in the case of one-dimensional input motion. *6: uplift displacement at column base, *7: landing velocity at column base, *8: base shear coefficient of structure, *9: cumulative plastic strain in wing plate of base plate, and, *10: ratio of the uplift displacement in the case of two-/three-dimensional input motion to the sum of the uplift displacements in the transverse and longitudinal directions in the case of one-dimensional input motion.

In Table 1 the numerals in the parentheses in the columns of roof displacement, roof velocity, roof acceleration and base shear coefficient indicate the ratio of the value in the case of two-/three-dimensional input motion to the one in the case of one-dimensional input motion. And the numerals in the parentheses in the column of uplift displacement indicates the ratio of the uplift displacement in the case of two-/three-dimensional input motion to the sum of the uplift displacements in the transverse and longitudinal directions in the case of one-dimensional input motion. It is suggested that the response of a structure subjected to two-/three-dimensional input motions is readily predicted from the response of that subjected to one-dimensional input motions, because these ratios of the BPL model in Table 1 are almost equal or less than unity.



Fixed-base model BPL model
(a) Transverse direction



Fixed-base model BPL model
(b) Longitudinal direction

Figure 6: Cumulative plastic curvature ratios of girders in transverse direction subjected to three components of JMA record.

Figure 6 shows the cumulative plastic curvature ratios at girder ends of the fixed-base and BPL models subjected to the three-dimensional input motion of the JMA Kobe record. The cumulative plastic curvature ratio is defined as the ratio of cumulative plastic curvature to yield curvature of girder section. While the girders of the fixed-base model yield in bending at the second to eighth floors in the transverse direction, those of the BPL model yield in bending at the second to third floors in the transverse direction. In addition, the cumulative plastic curvature ratios are almost the same in the transverse direction and quite small in the longitudinal direction in two models.

Furthermore, although the sectional force of columns does not reach the full plastic moment, the peak local stress in compression goes beyond the yield strength at the bottom of columns at the first story.

4 Summary and conclusions

The reduction of the three-dimensional seismic response of base-plate-yielding rocking systems with columns allowed to uplift is evaluated and compared with that of fixed-base systems by finite element numerical analyses, using ten-story, one-by-three bay steel frames of base-plate-yielding rocking system.

The results of this study are summarized as follows:

- 1) The maximum base shears and horizontal roof accelerations in the seismic response of the structures with column uplift are effectively reduced in the base-plate-yielding rocking system from those of the fixed-base system. The base shear coefficients of the uplift model are reduced to 68% to 82% of the fixed-base model subjected to one-dimensional input motions in the horizontal direction and to 59% to 76% of the fixed-base model subjected to two-/three-dimensional input motions.
- 2) The maximum roof displacements in the seismic response of the rocking structures are not much different from the response values of the fixed-base systems, but almost increase relative to the fixed-base model. The ratio of the uplift to fixed-base models is from 0.89 to 1.25 in the case of one-dimensional input motions, and from 0.78 to 1.30 in the case of two-/three-dimensional input motions.
- 3) The energy dissipation of the yielding base plates is expected to be effective in reducing the response displacement of yielding-base-plate rocking systems. While the girders of the fixed-base model yield in bending at the second to eighth floors, those of the uplift model yield in bending only at the second and third floors.

Acknowledgements

The authors express their gratitude to Mr. M. Kawakami and M. Shoji of Kozo Keikaku Engineering Inc. for their excellent support in the analytical work. The authors also express their appreciation to Mr. T. Sudo for his assistance in preparing the data and figures. Part of this work is supported by the Ministry of



Education, Culture, Sports, Science and Technology (MEXT) of Japan under Grant-in-Aid for Scientific Research, Project No. 16360284 and 18560572.

References

- [1] Rutenberg, A., Jennings, P. C. & Housner, G. W., The response of Veterans Hospital Building 41 in the San Fernando Earthquake. *Earthquake Engineering and Structural Dynamics*, 10(3), pp. 359-379, 1982.
- [2] Hayashi, Y., Tamura, K., Mori, M. & Takahashi, I., Simulation analyses of buildings damaged in the 1995 Kobe, Japan, Earthquake considering soil-structure interaction. *Earthquake Engineering and Structural Dynamics*, 28(4), pp. 371-391, 1999.
- [3] Midorikawa, M., Azuhata, T., Ishihara, T. & Wada, A., Shaking table tests on rocking structural systems installed yielding base plates in steel frames. *Proc. STESSA 2003 (4th International Conference on Behaviour of Steel Structures in Seismic Areas)*, pp. 449-454, Naples, Italy, 2003.
- [4] Midorikawa, M., Azuhata, T., Ishihara, T. & Wada, A., Shaking table tests on seismic response of steel braced frames with column uplift. *Earthquake Engineering and Structural Dynamics*, 35(14), pp. 1767-1785, 2006.
- [5] ADINA R&D, Inc., Theory and modelling guide – ADINA. *Report ARD 02-7*, 2002.
- [6] Ishihara, T., Midorikawa, M. & Azuhata, T., Hysteresis characteristics of large-scale column base for rocking structural systems, *Journal of Constructional Steel*, 14, pp. 381-384, 2006. (in Japanese)



Shaking table test on seismic response of reduced-scale models of multi-story buildings allowed to uplift

T. Ishihara¹, T. Azuhata¹, K. Noguchi¹, K. Morita²
& M. Midorikawa³

¹National Institute for Land and Infrastructure Management, Japan

²Building Research Institute, Japan

³Hokkaido University, Japan

Abstract

The effects of rocking vibration accompanied by uplift motion may reduce seismic damage to buildings. Structural systems that are allowed to uplift can be recognized as one of the simplest “self-centring” systems utilizing potential energy of self-weight. To investigate the effect of uplift motion on seismic responses of buildings experimentally, we conducted parametric shaking table tests using reduced-scale specimens with multi-stories. In this paper, the results of the tests are reported and discussed.

Keywords: self-centering, seismic response reduction, rocking vibration, reduced-scale model, higher mode.

1 Introduction

It has been pointed out that structural systems of buildings during strong earthquakes have been subjected to foundation uplift [1, 2]. After the first study by Muto et al. [3], many studies dealing with foundation uplift in flexible systems have been conducted (e.g. [4–7]) and some of these researches are summarized in the appendix of ATC-40 [8]. The authors also studied experimentally and analytically from the point of view of utilizing transient uplift motion for reduction of seismic response (e.g. [9–11]). In the experiment, 5 story and 3 story reduced scale steel frame were used as specimen.

Structural system allowed to uplift can be recognized as one of the simplest “self-centring” system. This system utilizes potential energy of self-weight to



absorb the seismic input energy temporally. Because the reduction of seismic forces makes the structure easy to remain in its elastic range, no residual deformation may be caused in this system after an earthquake.

As pointed out in the literature, structures allowed to uplift have nonlinear behaviour and are subjected to impact forces after an excursion of an uplift motion, more experimental data should be needed to utilize transient uplift motion for seismic design of buildings to survive the severe earthquake without residual deformation.

Now we planed parametric shaking table tests with small scale models. Experimental parameters of the models can be natural period, number of stories, stiffness distribution along the height, etc. In this paper, part of the experimental results are reported and discussed.

2 Specimen and experimental procedures

2.1 Specimen

Figure 1 shows the specimen. The specimen is composed of units bolted each other in vertical direction. Each unit corresponds to each story of shear-type buildings with one bay. The height and span of the units are 218mm and 200mm respectively. The weight of the units is only about 17kg, light enough to handle by oneself. Each unit has two floor elements (steel plates, $t=9\text{mm}$) at its top and bottom. To make the natural period of the specimen enough to long to represent the dynamic behaviours of real buildings and to provide sufficient vertical stiffness and strength to sustain impact force at landing after an uplift motion, vertical resisting elements (VREs) and horizontal resisting elements (HREs) are arranged separately. As VREs and HREs, four steel flat bars (50×6) with butt hinges at both ends and four piano wires ($D=4\text{mm}$, $\sigma_y(0.2\%\text{offset})=1.2\text{kN/mm}^2$) are used respectively. Each unit is recognized as a one-directional link with elastic HREs.

To allow the specimen to uplift, pins with half cylindrical shape ($R=20\text{mm}$) are attached on the bottom plate of the lowest unit. Supports with a shallow V-shaped channel are attached on the footing beam as in the preceding test [9]. The specimen is just put on the supports, so the specimen is allowed to uplift without slippage between the specimen and the footing beam under earthquake excitation.

For fixed base condition, the bottom plate of the lowest unit is tightly bolted to the footing beam.

Table 1 shows the model properties. In this paper, the results of 3 models with 4, 6, and 8 stories are reported. The stiffness distribution along the height is uniform. Natural periods of models are as long as those of real buildings. In the table, "critical" means the initiation level of uplift, that is, the level when the overturning moment reaches the resisting moment due to self-weight. "Normalized overturning moment" means that the overturning moment is normalized by $MgB/2$, where M is total mass of the specimen, g is gravitational acceleration and B is span ($B=200\text{mm}$).



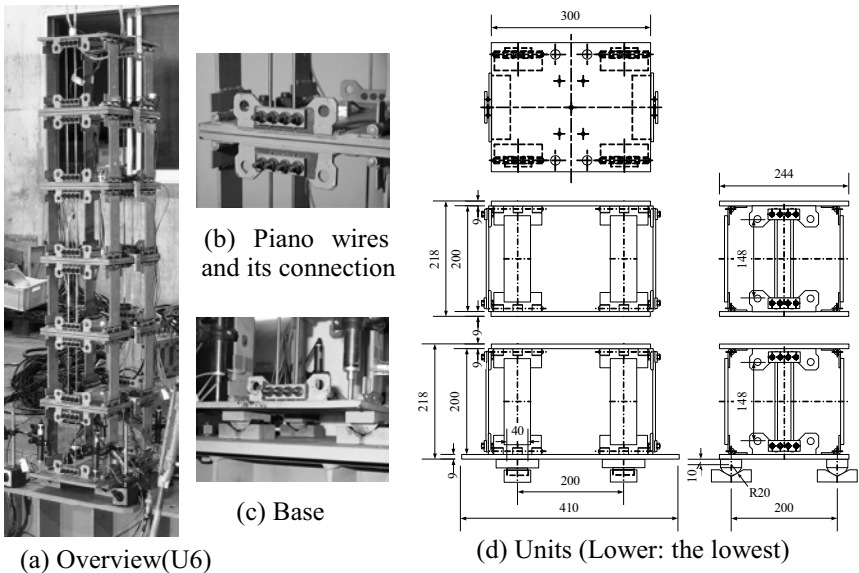


Figure 1: Reduced scale model.

Table 1: Model properties.

Model	Number of stories	H/B	Period(s)*	Damping ratio (%)*	Critical base shear coefficient**	Critical normalized overturning moment**
U4	4	4.41	0.35	9.9	0.198	0.950
U6	6	6.59	0.60	5.4	0.120	0.906
U8	8	8.77	0.88	4.6	0.083	0.853

* based on the results of free vibration tests under fixed base condition

** 1st mode approximation considering $P\Delta$ effect

2.2 Experimental procedure

The specimen is oscillated only in one horizontal direction. Earthquake excitation used in the shaking table test is 1940 El Centro NS component. The time scale is not changed but the input amplitude (I.A.) is selected at a wide range of intensities. Figure 2 shows pseudo velocity spectrum based on the acceleration measured on the shaking table.

The measured structural response quantities were all horizontal floor acceleration (more precisely, acceleration parallel to floors), all horizontal floor displacement, uplift displacement, vertical acceleration (more precisely, acceleration parallel to VREs) in the lowest story.



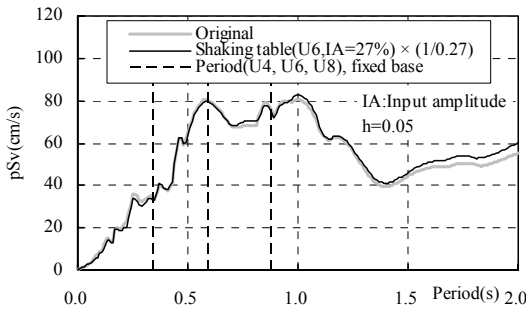


Figure 2: Pseudo velocity spectrum.

3 Test results and discussion

3.1 Calculation of responses

Before the results are shown, the methods of calculation of responses are explained. When the rotation θ of the base becomes large due to uplift, measured acceleration parallel to floors, a_i , includes the components due to gravity and vertical response acceleration, a_v .

$$a_i = (\ddot{u}_i + \ddot{y}_g) \cos \theta - (g + a_v) \sin \theta \approx \ddot{u}_i + \ddot{y}_g - (g + a_v) \theta \quad (1)$$

where $\ddot{u}_i$ is relative horizontal acceleration, $\ddot{y}_g$ is base (shaking table) horizontal acceleration. a_v is calculated by the measured acceleration parallel to the lowest unit's VREs considering its inclination. In equation (1), we assume that θ and a_v are independent of the height of floor because the specimen is designed as shear type structure. The absolute horizontal accelerations of floors are,

$$\ddot{u}_i + \ddot{y}_g \approx a_i + (g + a_v) \theta \quad (2)$$

Story shears and story moments are computed based on the absolute horizontal accelerations calculated by the right-hand side of equation (2) and the masses measured in advance. Overturning moment is calculated as the sum of story moments.

Restoring moment m_B due to self-weight considering the deformation of the specimen is as follows:

$$\frac{m_B}{MgB/2} = \mp 1 + \sum \left(\frac{M_i}{M} \frac{u_i}{B/2} \right) \quad (3)$$

where M_i is the mass of floor, u_i is the relative horizontal displacement of floor. The second term on the right-hand side in equation (3) is negligible in real scale buildings, but it should be included in the small scale and flexible test as shown in this paper.



3.2 Dynamic behaviour and maximum responses

Figure 3 shows the time histories of roof displacement, comparing those of fixed base condition under the same input amplitude. Although the displacements become larger than those of fixed base, elastic deformations of superstructures are smaller because almost over half of the displacements are caused by rigid rotation.

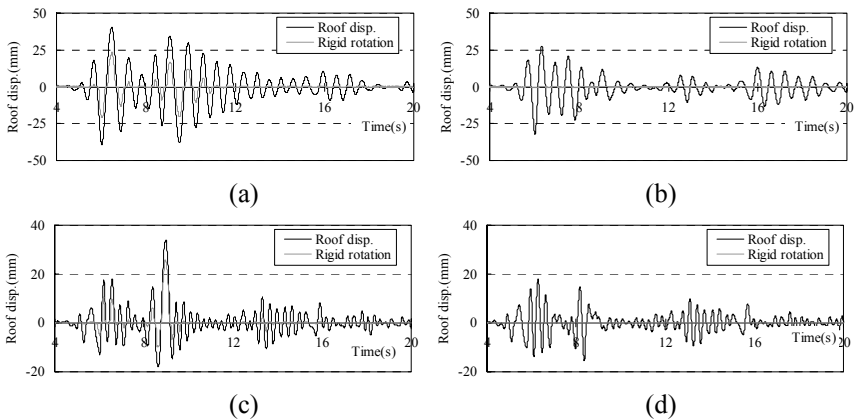


Figure 3: Time histories of roof horizontal displacement: (a) U6(I.A.=27.0%); (b) U6fix(I.A.=27.0%); (c) U4(I.A.=74.7%); (d) U4fix(I.A.=74.7%).

Figure 4 shows the time histories of absolute horizontal accelerations (see eq. (2)), story shear coefficients in the lowest (1st) stories (i.e. base shear coefficients), overturning moments and uplift displacements. Dot-dash lines and dotted gray lines show the critical base shear coefficients (see table 1) and restoring moment due to self-weight (see eq. (3)) respectively. Once a transient uplift motion starts, shorter periodic vibrations are clearly observed. Those are relatively larger compared to fixed base condition. These phenomena are recognized to be higher mode effect as pointed out mainly by analytical studies (e.g. [4, 6, 7, 11]).

Figure 5 shows dynamic load displacement relationships. Dotted lines show stiffness of the first mode in fixed base condition based on the measured natural period of the model and critical base shear coefficient. Maximum forces are reached just after lift-off. During an excursion of transient uplift motion, higher mode effects can be also observed in these graphs.

Figure 6 shows maximum base shear and overturning moment comparing those of fixed base condition. Horizontal dotted lines show critical values. The results show that allowing uplift reduces the seismic forces.



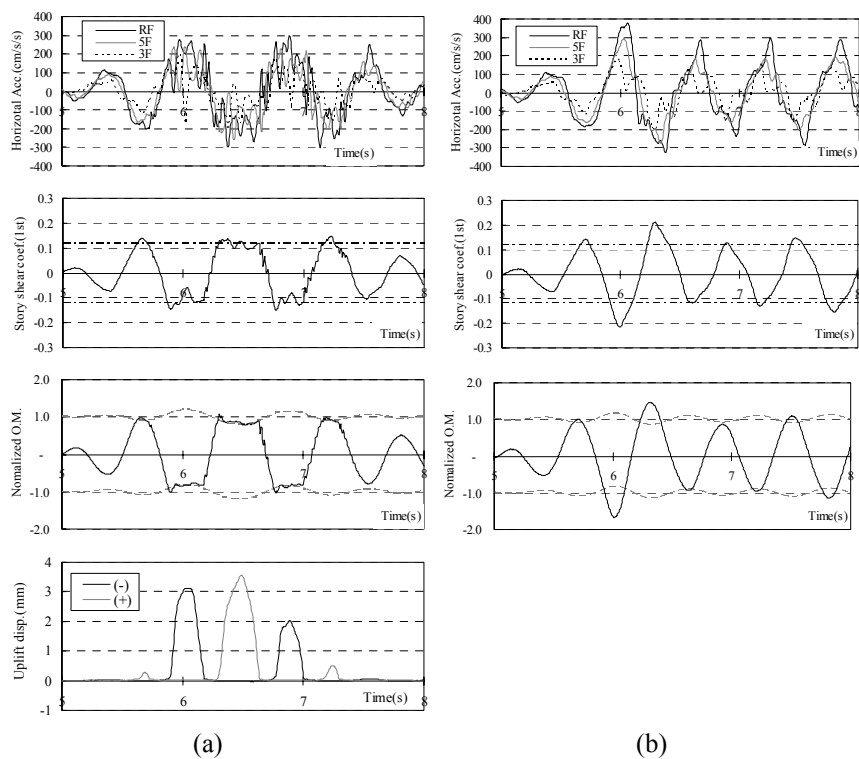


Figure 4: Time histories of responses, U6 (I.A.=27.0%): (a) U6; (b) U6fix.

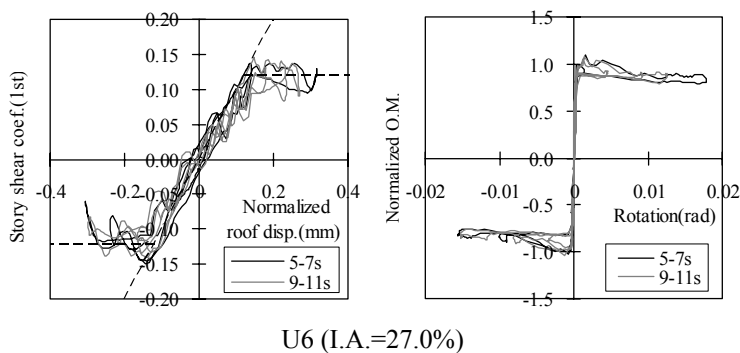


Figure 5: Dynamic load displacement relationship.



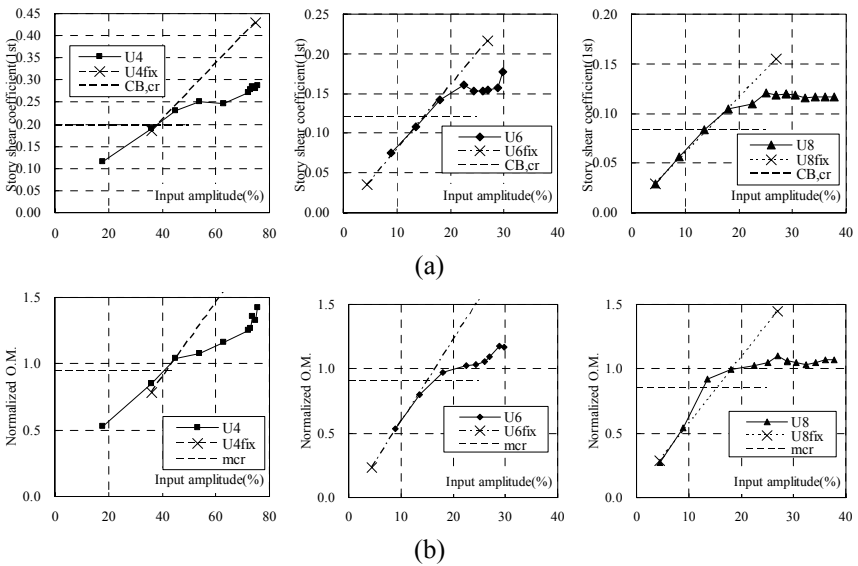


Figure 6: Maximum base shear and overturning moment: (a) Base shear coefficient; (b) normalized overturning moment.

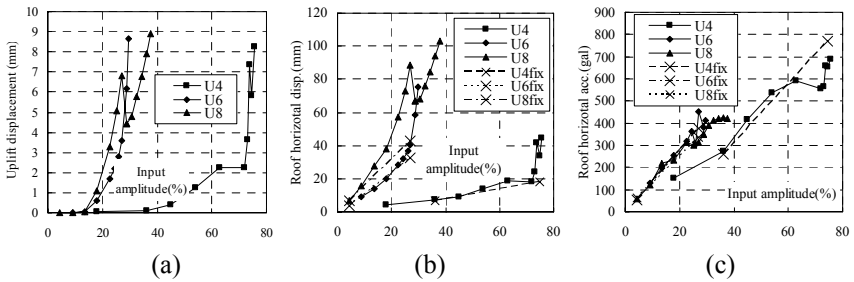


Figure 7: Maximum displacement and acceleration: (a) uplift displacement; (b) roof horizontal displacement; (c) roof horizontal acceleration.

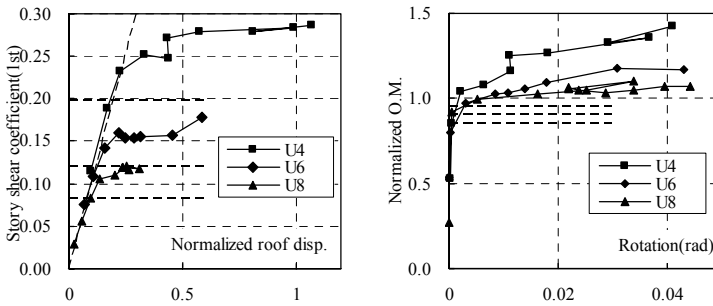


Figure 8: Maximum load displacement relationship.



Figure 7 summarizes the maximum displacements and accelerations. Uplift displacements are rapidly increased as the input amplitude increases. Roof horizontal displacements are also rapidly increased. Note that rigid rotations are dominant in roof displacements as mentioned above. Roof horizontal accelerations are almost as large as those of fixed base condition.

Figure 8 shows the maximum load displacement relationships. Dotted lines show the stiffness of fixed base condition and critical values. U4 is subjected larger forces than U6 and U8 due to the smaller aspect ratio H/B . In the range of the tests, maximum responses reach about 1.5 times larger values than the corresponding critical ones.

Figure 9 shows the distribution of responses along the height. In the figure, “Ai” shows the distribution defined as standard one in building standard law in Japan. As pointed out in the analytical basic study [11], transient uplift motion changes the distribution of normalized story shear coefficient into top heavy one. Normalized horizontal accelerations are almost as large as or larger than those of fixed base condition. Secondary systems in the buildings allowed to uplift would be affected by these relatively large acceleration. To evaluate the responses of secondary systems in uplifting buildings, further study is needed including the effect of shortening of vibration period mentioned above.

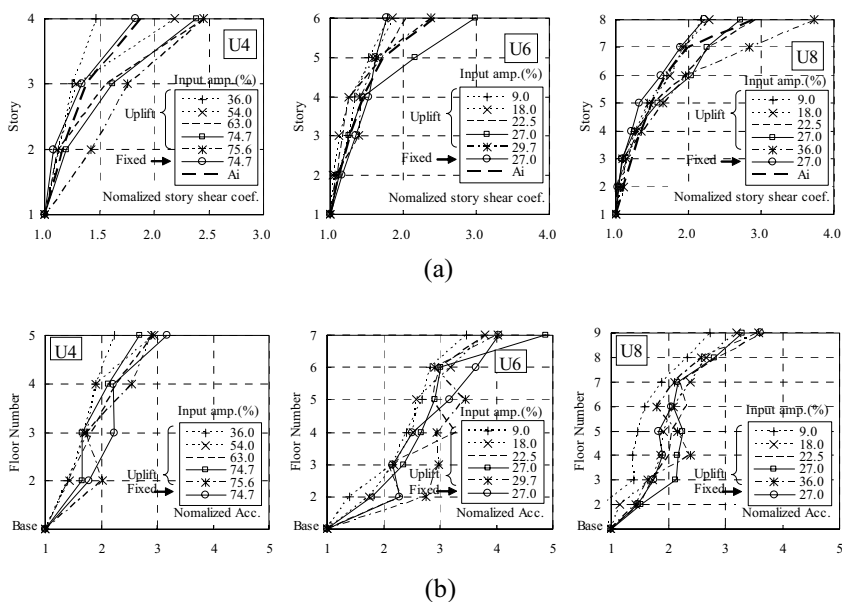


Figure 9: Normalized responses along the height: (a) normalized story shear coefficient; (b) normalized horizontal acceleration.

4 Conclusion

In this paper, part of the results of parametric shaking table tests conducted with small scale shear type building models allowed to uplift are reported. Conclusions are summarized as follows:

- 1) Seismic response reduction effect is confirmed.
- 2) Once a transient uplift motion starts, shorter periodic vibrations are clearly observed in the tests. These phenomena can be recognized to be higher mode effect as pointed out in the literature.
- 3) Transient uplift motion changes the distribution of normalized story shear coefficient into top heavy one.
- 4) Normalized horizontal accelerations of floors are almost as large as or larger than those of fixed base condition.

Acknowledgements

Part of this work is supported by the National Research Institute for Earth Science and Disaster Prevention (NIED) of Japan and the Ministry of Education, Culture, Sports, Science and Technology (MEXT) of Japan under Grant-in-Aid for Scientific Research, Project No. 18560572 and 16360284. The supports are gratefully acknowledged.

References

- [1] Rutenberg, A., Jennings, P.C., & Housner, G.W., The response of veterans hospital building 41 in the San Fernando earth-quake. *Earthquake Engineering and Structural Dynamics*, **10**, pp.359-379, 1982.
- [2] Hayashi, Y., Tamura, K., Mori, M. & Takahashi, I., Simulation analysis of buildings damaged in the 1995 Kobe, Japan, earthquake considering soil-structure interaction. *Earthquake Engineering and Structural Dynamics*, **28**, pp.371-391, 1999.
- [3] Muto, K., Umemura, H., & Sonobe, Y., Study of the overturning vibration of slender structures. *Proceedings of the Second World Conference on Earthquake Engineering*, **2**, pp.1239-1261, 1960.
- [4] Meek, J.W., Effect of foundation tipping on dynamic response. *Journal of Structural Engineering*, **101**(ST7), pp.1297-1311, 1975
- [5] Meek, J.W., Dynamic response of tipping core buildings. *Earthquake Engineering and Structural Dynamics*, **6**, pp.437-454, 1978.
- [6] Yim, S.C-S. & Chopra, A.K., Simplified earthquake analysis of multistory structures with foundation uplift, *Journal of Structural Engineering*, **111** (12), pp.2708-2731, 1985
- [7] Oliveto, G., Calio, I., & Greco, A., Large displacement behavior of a structural model with foundation uplift under impulsive and earthquake excitations. *Earthquake Engineering and Structural Dynamics*, **32**, pp.369-393, 2003



- [8] ATC-40 *Seismic evaluation and retrofit of concrete buildings*, volume 2 – Appendices, Appendix F: Supplemental information on foundation effects. Applied Technology Council, 1996.
- [9] Midorikawa, M., Azuhata, T., Ishihara, T. Matsuba, Y. & Matsushima, Y., Earthquake response reduction of buildings by rocking structural systems, *Proceedings of SPIE, Smart structures and materials*, pp.265-272, 2002.
- [10] Midorikawa, M., Azuhata, T., Ishihara, T. & Wada, A., Shaking Table Tests on Seismic Response of Steel Braced Frames with Column Uplift, *Earthquake Engineering and Structural Dynamics*, **35**(14), pp.1767-1785, 2006
- [11] Ishihara, T., Midorikawa, M. & Azuhata, T., Vibration characteristics and dynamic behaviour of multiple story buildings allowed to uplift, *Proceedings of SPIE, Smart structures and materials*, **6169**, pp.61691A-1-8, 2006



Self-centering behavior of unbonded precast concrete shear walls

B. Erkmen & A. E. Schultz
University of Minnesota, USA

Abstract

Concrete shear walls are a cost-effective way of providing lateral load resistance for structural systems located in seismic regions. If concrete shear walls are precast and rely on unbonded post-tensioned tendons for flexural reinforcement, then the structural damage observed in conventionally reinforced cast-in-place shear walls arising from tensile stress transfer can be avoided altogether. Over the past decade, it has been recognized that excellent seismic performance of precast concrete shear walls can be mobilized by utilizing post-tensioned unbonded vertical reinforcement in precast shear walls to create extensible connections that allow controlled rocking. Another important advantage of post-tensioned precast concrete shear walls, and one which has not been studied extensively, is their superior self-centering characteristic. The self-centering property of unbonded post-tensioned walls is generally attributed to the presence of the post-tensioning force. However, the experimental results presented in this study indicate that the post-tensioning force may completely die out during cyclic loading while the wall retains its self-centering characteristic. Moreover, analytical study, verified with experimental results, indicates that with proper design of end-anchorage for post-tensioned tendons, self-centering can be achieved even when the post-tensioning force dies out completely. The study summarized here investigates the self-centering ability of unbonded precast concrete shear walls, particularly the effects of post-tensioning force, tendon layout, and the end-anchorage detail.

Keywords: seismic performance, self-centering, unbonded tendons, precast concrete, shear walls, post-tensioning.



1 Introduction

Conventional concrete shear walls that are a part of monolithic structures are expected to undergo significant structural damage (flexural and shear cracking, toe crushing, and rebar fracture and buckling) and residual lateral displacement during seismic events. Thus, the economic impacts of the associated seismic damage can be significant. Precast concrete shear wall structures, on the other hand, are significantly different in terms of expected structural behavior. As such, different design philosophies have been developed for precast structures in the USA [1, 2] where precast concrete structural systems fall into two design categories. The first design one is “emulation construction”, in which precast structures are detailed to emulate monolithic reinforced concrete structural systems. The second alternative is “jointed construction”, in which precast members are interconnected predominantly by dry joints (i.e., requiring no concrete to be cast at the site).

In general, the non-emulative design philosophy is preferred as it allows certain joints between the precast members to undergo inelastic deformations without significant damage. This inherent characteristic can be used for seismic resistance. For precast shear wall, joints between panels may open and close, and undergo inelastic deformations without significant damage. These locations provide deformation capacity and, possibly, energy dissipation in precast structural systems [3].

Over the past decade, it has been recognized that the seismic performance of precast concrete structures can be improved if the flexural reinforcement is post-tensioned and placed inside ducts that are left ungrouted (i.e., unbonded). Due to the lack of bond between reinforcement and concrete, damage is not introduced in the concrete through bond stress transfer from the reinforcement [3, 4–7]. However, there is limited information related to their self-centering capability precast walls. Their self-centering capability has generally been attributed to the presence of post-tensioning force, which has led to concerns that the post-tensioning force may significantly decrease or completely die out during seismic loading as the wall rocks and the tendon is elongated. However, this relationship and its underlying causes have not been investigated explicitly, and a detailed investigation of the self-centering mechanism of precast shear walls is needed.

2 Background

Figure 1 shows a test specimen representing a typical post-tensioned precast concrete shear wall with tendons that are placed in ducts and left unbonded over wall height. During erection, the only wet concrete that is used is dry-pack grout between panels, footings and floor slabs. The only locations where wet concrete is placed during erection are the connection surfaces between wall panels, or between the panels and footing or floor slabs, where dry-pack grout is used to fill the gaps and provide uniform bearing stress transfer.



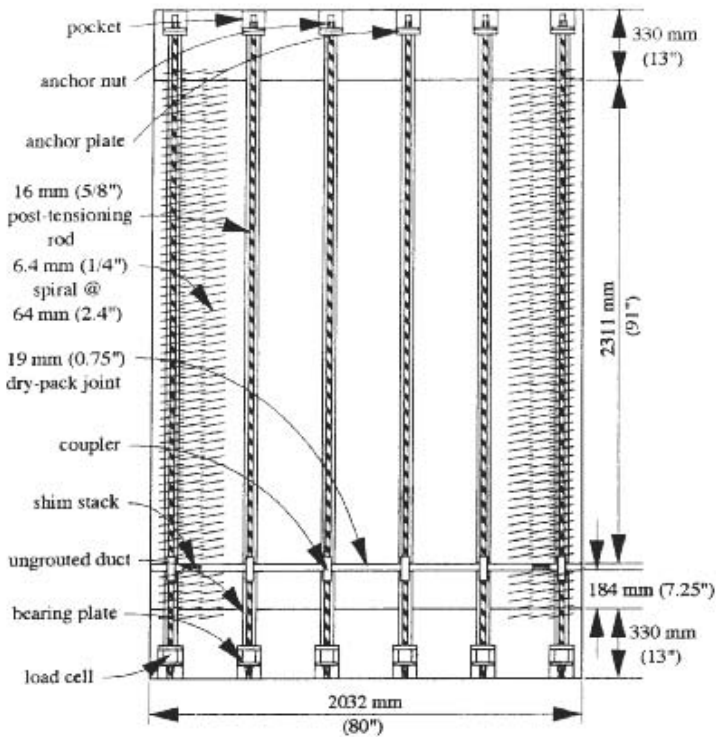


Figure 1: Test specimen of unbonded, post-tensioned precast shear wall.

Unbonded tendons are anchored to the wall panels only at anchorage and tensioning locations, of which feature eliminates bond stress transfer and the associated tensile cracking damage in the concrete. For the case of oversize ducts and straight tendons, friction losses are negligible, and the uniform strain distribution along the tendons delays tendon yielding and rupture. Under seismic loading, the loading and unloading branches of the force-displacement relation are close to each other by virtue of small amount of damage to the materials. And, upon unloading, little residual drift is observed.

An extensive research program conducted in the USA to utilize the concept of jointed precast concrete structures [8] has shown that unbonded, post-tensioned shear walls can be used as the primary lateral load carrying element in regions of high seismicity. This study culminated with a series of pseudo-dynamic tests of a large-scale (1:0.6) five-story precast concrete building. The behavior of the unbonded precast shear wall was excellent, with only minor non-structural damage in the loading direction that included shear walls. The residual drifts in the wall direction after design level excitation did not exceed 0.06% after sustaining a top drift of 1.8% of structure height.



3 Experimental investigation

Schultz et al. [3] conducted an experimental investigation, as part of the PRESSS program, investigate the characteristics of connection regions in jointed precast shear walls. One shear wall specimen (PTT), which featured unbonded, post-tensioned tendons (PTT) at a horizontal joint, was a 2/3-scale representation of the lowest two stories of a prototype precast concrete shear wall in a six story precast office building. Concrete with a compression strength of 34.5 MPa (5,000 psi) was used in conjunction with 1,124 MPa (163 ksi) post-tensioning bars, the latter which were spliced using standard couplers. The vertical bars were placed in oversized ducts and anchored to the walls at the top and at the foundation level. A 19mm (3/4 in.) thick layer of high-strength dry-pack mortar was placed between panels at horizontal joints.

Six tendons with a 16mm (5/8 in.) diameter were uniformly placed in the connection region, even though PRESSS recommendations suggest tendon placement near the middle of the wall to protect them from large tension strains. Uniform distribution was used to limit out-of plane movement. The bars were initially post-tensioned to 60% of tendon strength (i.e., 695 MPa, 95 ksi). Spiral reinforcement was provided at the edges of the panels to confine the concrete as large compression strains were expected in these regions due to wall rocking.

Specimen PTT was tested at the National Institute of Standards and Technology (NIST) using the Tri-directional Test Facility (TTF) [9] under quasi-static loading. In-plane horizontal drift and overturning moment were applied to specimen PTT, in addition to a constant vertical load for a net vertical compression stress equal to 689 Pa (100 psi).

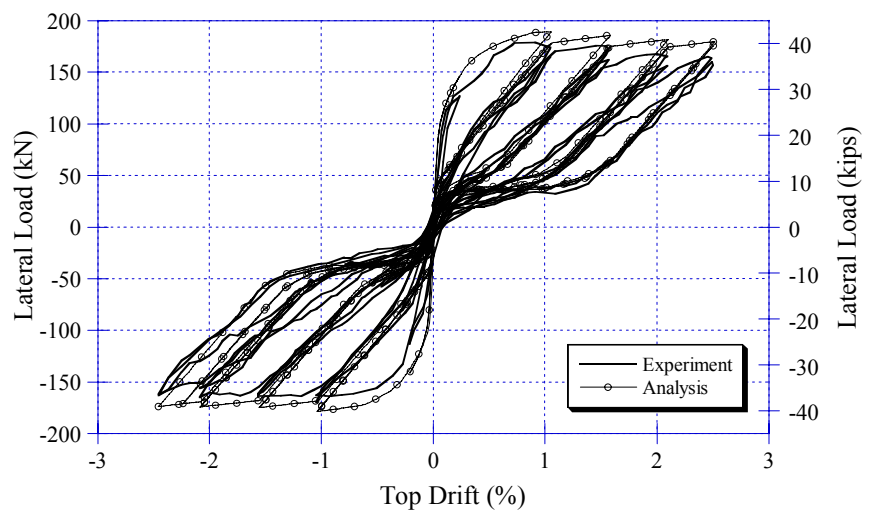


Figure 2: Specimen PTT experimental and analytical response.



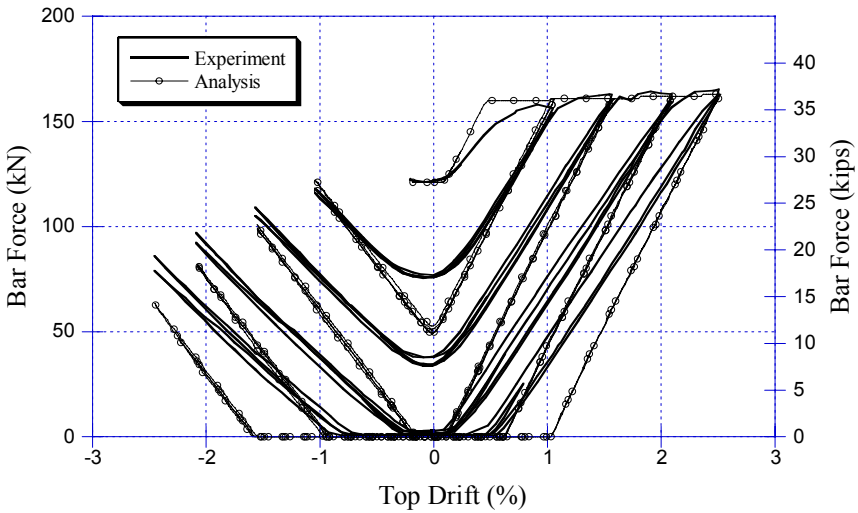


Figure 3: Bar force-drift relationship for tendon EI.

The overturning moment at the top of the specimen represented the continuity moment at an elevation equal to one-third of total wall height from the base of the prototype six-story shear wall.

The lateral force-drift response of specimen PTT (Fig. 2) was stable with small, but finite energy dissipation capacity and good self-centering capacity. High initial stiffness and linear behavior were observed until a gap began to open along the horizontal joint, after which the behavior became nonlinear. The stiffness of the wall, at a drift of approximately 0.2%, started to decay gradually due to both yielding of the reinforcement and gap opening at the joint region. The peak lateral load capacity of 178 kN (40 kips) was maintained throughout the loading history despite the fact that most tendons lost their post-tensioning force by the end of the loading history (Fig. 3). In spite of the decay in both stiffness and post-tensioning force, the wall preserved its self-centering ability during the test, with almost no residual displacement up to a maximum drift of 2.5%. The test was stopped because the stroke capacity of actuators was reached.

4 Analytical modeling

Nonlinear static analyses of specimen PTT were carried out using a model developed with the DRAIN-2DX program [10]. Kurama et al. [11] first described the use of DRAIN-2DX for the analysis of precast concrete shear walls with unbonded, post-tensioned tendons. Four types of elements (truss, concrete fiber, rigid link, and tension link) were used. Concrete fiber elements served to model the wall panels and they did not include reinforcement because the unbonded post-tensioning tendons were the only reinforcement placed continuously through the horizontal joint. Vertical and horizontal non-



prestressed reinforcement were not continuous beyond panel edges. Truss elements were used to model the post-tensioning tendons, and these elements were not connected (i.e., unbonded) to the concrete panels. A rigid link element was used at the top of the wall to connect the degrees of freedom of the truss elements (i.e., tendons) to those of the fiber elements (i.e., wall panels) and ensure compatibility. Tension link elements (rigid in tension and slack in compression) were used at the ends of each tendon to prevent them from developing compression. The tendons did not bear against any surface beyond the anchor plates to develop such compression resistance (Fig. 1).

The load sequence included an initial application at the top of the wall of a constant vertical load of 214 kN (48.0 kips), 13 kN (3.0 kips) of which was wall weight. Then a post-tensioning force of 121 kN (27.3 kips), which was slightly higher than the test value 118 kN (26.6 kips), was applied to each tendon. The post-tensioning force decreased to the test value upon application due to elastic shortening of the concrete. The experimental cyclic lateral drift and top moment histories were applied next to obtain the response of the wall (Fig. 2). No significant differences in stiffness, lateral load capacity and absorbed energy are observed in the computed response relative to that measured in the experiments. Most importantly, the self-centering behavior of the model shows good fidelity with the experimental results.

Figure 3 shows the computed and measured force versus lateral displacement relationships for the third tendon (EI), from the left edge of the specimen. The force-drift curve for tendon EI predicted with the DRAIN-2DX model is slightly stiffer than that obtained from the experiment, specifically in unloading branches where the specimen exhibited stiffness degradation with increasing drift. Figure 3 also indicates that the post-tensioning force vanished at a drift ratio equal to zero after the drift ratio had achieved a value of approximately 2.2%. This means that the remaining tendons lost their post-tensioning force at smaller drift values given their proximity to the edges of the panel. Yet, specimen PTT exhibited good self-centering behavior through all of the cycles at 2.5% drift, even though all of the post-tensioned force was lost for drift values larger than 2.2%.

5 Self-centering ability of post-tensioned shear walls

The superior self-centering behavior of unbonded, post-tensioned walls is, in general, attributed to the presence of a finite post-tensioning force. However, the experimental and analytical findings presented suggest that post-tensioning force is not the only source of self-centering behavior. Vertical compression is generated by permanent loads (e.g., wall weight plus any additional dead load), semi-permanent loads (e.g., live loads), and transient loads (e.g., loads associated with vertical seismic excitation). In the case of specimen PTT, wall weight and external compression provided 214 kN (48 kips) of constant vertical load which was present to resist lateral loading even when post-tensioning force vanished.



The effect of several structural design parameters on the self-centering response of specimen PTT are investigated below. These parameters include: (1) tendon end-anchorage detail; (2) initial tendon stress; and (3) tendon location.

5.1 Tendon end-anchorage detail

The tendons used for specimen PTT (Fig. 1) were anchored in pockets which did not allow bearing of the tendon ends, such that the bars could not develop compression once the post-tensioning force died out. Tendons with anchorages that cannot undergo compression are referred to as “compression-prevented” tendons, whereas tendons with anchorages that allow compression are referred to as “compression-allowed” tendons. Thus, a variation of the analytical model for specimen PTT was developed which did not include the “rigid-slack” link elements described earlier, such that the tendons were modeled as “compression-allowed” to investigate the effect of the end-anchorage detail on wall behavior. Figure 4 shows the force-displacement response predicted by the model with compression-allowed tendons.

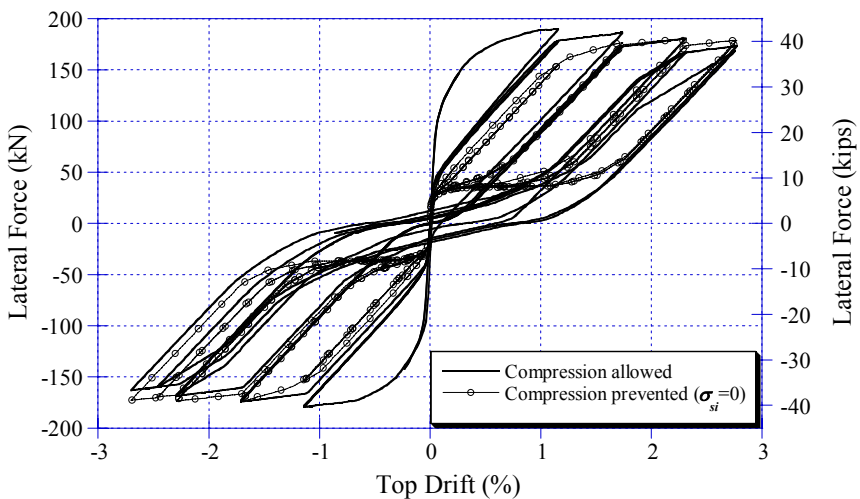


Figure 4: Predicted response of specimen PTT with tendons that develop compression, or tendons that do not develop compression (but have no initial prestress).

Despite the presence of the post-tensioning force, the predicted self-centering behavior of the wall with compression allowed tendons is inferior to that of the wall with compression prevented-tendons (Fig. 3). This difference is due to permanent elongation of the compression-allowed tendons, which take place at large drifts when the post-tensioning force has vanished.



5.2 Initial tendon stress

Another variation of the DRAIN-2DX model of specimen PTT was used to investigate the effect of the magnitude of post-tensioning stress on self-centering behavior. For that case, no post-tensioning force was applied to the tendons ($\sigma_{st}=0$) which were modelled as “compression-prevented”. Figure 4 shows the predicted lateral force-top drift response of the wall for such conditions.

The absence of initial post-tensioning force resulted in large reductions in initial stiffness and energy dissipation during cyclic loading. However, the model preserved its superior self-centering behavior despite the absence of post-tensioning force. The presence of axial load and compression-prevented tendons was sufficient to preserve the self-centering behavior of the wall under cyclic loading. Tendons that are initially unstressed, but snug tight (i.e., with no slack), develop force as soon as the horizontal joint opens under lateral loading.

5.3 Tendon location

Two additional variations of specimen PTT were modeled by varying tendon location. In the first model, the outermost tendons on both sides of the wall were removed, and the area of the remaining two tendons (i.e., centered tendons) was tripled. The modified wall had the tendons were concentrated over the middle of wall length. In the second model, the four tendons closest to the wall center were removed, and the area of the two outermost tendons (i.e., edge tendons) was tripled. Thus, the tendons were concentrated along the wall edges. In both cases the tendons were not allowed to develop compression.

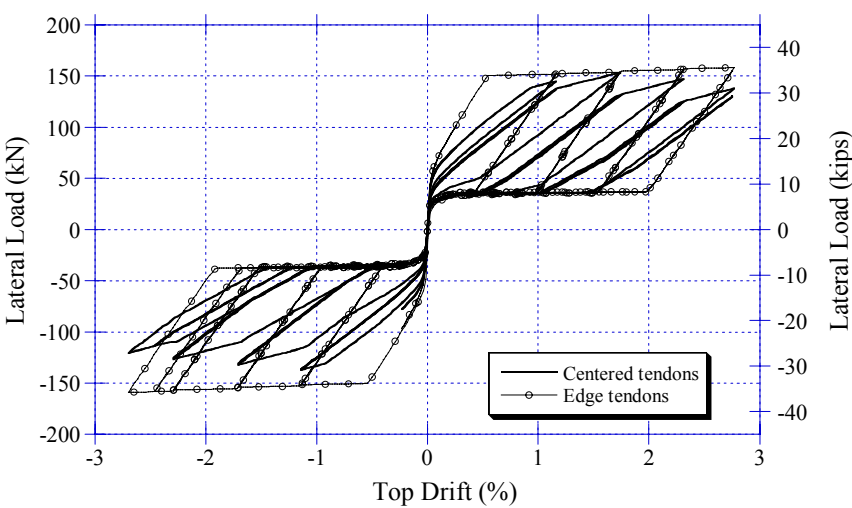


Figure 5: Predicted response of specimen PTT with compression-prevented tendons that are placed in the center of the wall, or along the edges.

The predicted lateral load-drift relationships for the “centered tendon” and “edge tendon” models of the wall are presented in Fig. 5. The stiffness and energy absorption capacities of “centered tendon” model are considerably smaller than those of the “edge tendon” model, or even the original model (Fig. 2). These differences are due to the reduction in tendon arm (i.e., distance from tendon to wall center) which decreases tendon strain change per unit of gap opening. However, the response of the walls indicates that moving the tendons towards the center or the edge of the wall does not affect self-centering behavior.

6 Conclusions

Based on the experimental observations cited and the results predicted by the analytical models developed in this study, the following conclusions are made:

1. The DRAIN-2DX modeling techniques described here can be used to predict the lateral load response of precast concrete shear walls with unbonded, post-tensioned tendons.
2. Unbonded, post-tensioned precast concrete shear walls rely on the total combination of post-tensioning force, wall weight and net external vertical compression load to develop resistance to lateral loads as well as self-centering ability.
3. The end-anchorage detail of unbonded tendons can significantly affect the hysteresis and self-centering behavior of unbonded walls.
4. Initial post-tensioning force has negligible effect on self-centering behavior of unbonded walls if the tendons are compression-prevented.
5. The distribution of tendons has negligible effect on self-centering behavior of the walls, but it may significantly affect the lateral stiffness and energy absorption capacity of the walls.

Acknowledgements

This work was supported in part by the Precast/Prestressed Concrete Institute (PCI) through a Daniel P. Jenny Fellowship and by the Department of Civil Engineering at the University of Minnesota through a Sommerfeld Fellowship. The authors wish to thank Professor Yahya Kurama from the University of Notre Dame for providing the modified DRAIN-2DX code.

References

- [1] *International Building Code*, International Code Committee, Whittier, CA, 2006.
- [2] *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450)*, 2003 Edition, Building Seismic Safety Council, Washington, DC, 2004.



- [3] Schultz, A., Cheok, G., and Magana, R., "Performance of Precast concrete Shear Walls", *Proc.*, 6th U.S. Nat. Conf. on Earth. Engrg., EERI, Oakland, CA, 1998.
- [4] Cheok, G. S., Stone, W. C., and Lew, H.S., "Seismic performance behavior of precast concrete beam-column joints," *Proc.*, Symp. on Struct. Engrg. in Nat. Haz. Mit., ASCE:Reston, VA, pp. 83-88, 1993
- [5] Priestley, M.J.N. and Tao, J.R.T., "Seismic Response of Precast Prestressed Concrete Frames with Partially Debonded Tendons", *PCI Journal*, 38(1), pp. 58-69, 1993.
- [6] Kurama, Y., Pessiki, S., Sause, R., Lu, L.-W., and El-Sheikh. M., "Analytical Modeling and Lateral Load Behavior of Unbonded Post-Tensioned Precast Concrete Walls", Rep. No. EQ-96-02, Dept. of Civil and Envir. Engrg., Lehigh University, Bethlehem, PA 1996, 191 pp.
- [7] Holden, T.J., "A comparison of the seismic performance of precast wall construction: emulation and hybrid approaches," Res. Rep. 2001-04, ISSN 0110-3326, University of Canterbury, Christchurch, New Zealand.
- [8] Priestley, M. J. N, Sritharan, S., Conley, J. R., and Pampanin, S., "Preliminary Results and Conclusions from the PRESSS Five-Story Precast Concrete Test Building" *PCI Journal*, 44(6), pp. 42-67, 1999.
- [9] Woodward, K., and Rankin, F., "The NBS TRI-Directional Test Facility", NBSIR 84-2879, U.S. Dept. of Comm., Nat. Bur. of Stds., Gaithersburg, MD, 1984.
- [10] Prakash, V., and Powell, G., "DRAIN-2DX Base program Description and User Guide; Version 1.10", Rep. No. UCB/SEMM-93/17, Dept. of Civil Engrg., Univ. of Calif., Berkeley, 1993.
- [11] Kurama, Y., Sause, R., Pessiki, S., Lu, L.-W., and El-Sheikh, M. "Seismic Design and Response Evaluation of Unbonded post-Tensioned Precast Concrete Walls", Res. Rep. No. EQ-97-01, Dept. of Civil and Envir. Engrg., Lehigh University, 1997, 184 pp.



Displacement ductility demand and strength reduction factors for rocking structures

M. Trueb¹, Y. Belmouden² & P. Lestuzzi²

¹ETHZ-Swiss Federal Institute of Technology Zurich, Switzerland

²EPFL-Ecole Polytechnique Fédérale de Lausanne, Switzerland

Abstract

This paper reports the main results of an extensive parametric study using numerical simulations and computing displacement ductility demand of non-linear single-degree of freedom (SDOF) systems and multi-degree of freedom (MDOF) systems for a set of 164 registered ground motions. The objective of this study is to propose values of strength reduction factors for rocking behavior for seismic analysis. In the first part focused on SDOF systems, non-linear seismic responses obtained with a hysteretic model simulating rocking are statistically compared with the ones related to established hysteretic models for ductile structures. Similar to established hysteretic models, results confirm that the frequency has little influence on the ductility demand if it is below 2 Hz and a substantial influence if it is above 2 Hz. Moreover, they show that the other parameters, especially the hysteretic behavior model, have only little influence on the displacement ductility demand. Surprisingly, displacement ductility demand is found to be practically independent of the additional viscous damping ratio. Finally, a relationship between displacement ductility demand and strength reduction factor for rocking systems is proposed. The second part shows that the results obtained for SDOF systems are also valid for MDOF systems.

Keywords: displacement ductility demand, strength reduction factor, non-linear structural response, rocking, earthquake, seismic analysis, hysteretic model.

1 Introduction

Intensive numerical investigations have already been performed to examine the relationships between strength reduction factors and non-linear behavior of structures subjected to earthquake ground motions (see [1] for a review of



significant investigations). The studies were generally focused on non-linear single-degree-of-freedom (SDOF) systems defined by different hysteretic models. However, the involved hysteretic models (elastoplastic, Clough, Takeda, etc.) were mostly related to seismic behavior with significant energy dissipation such as ductile reinforced concrete shear walls. Until recently, no systematic investigations were carried out for structures without hysteretic energy dissipation capacity such as slender unreinforced masonry shear walls that show very different seismic behavior. Other structures that show this type of behavior are precast post-tensioned reinforced concrete structures or concentrically braced steel structures with slender diagonal elements.

This paper presents the main results gained during the master thesis performed by the first author at the Swiss Federal Institute of Technology in Lausanne (EPFL). More complete description of this work may be found in [2]. The research project aims to answer the following question: under what conditions can the strength reduction factor for structures without hysteretic energy dissipation capacity be extended beyond the limited value of 1.5 accounting for overstrength only proposed by the construction codes?

2 Methodology

The methodology used in this study consists first of a systematic investigation of the non-linear response of SDOF systems subjected to a set of 164 earthquake recordings. Figure 1 illustrates the methodology schematically. The structural behavior is described by a hysteretic model developed for simulating non-linear behavior without hysteretic energy dissipation capacity and by two recognised hysteretic models as reference.

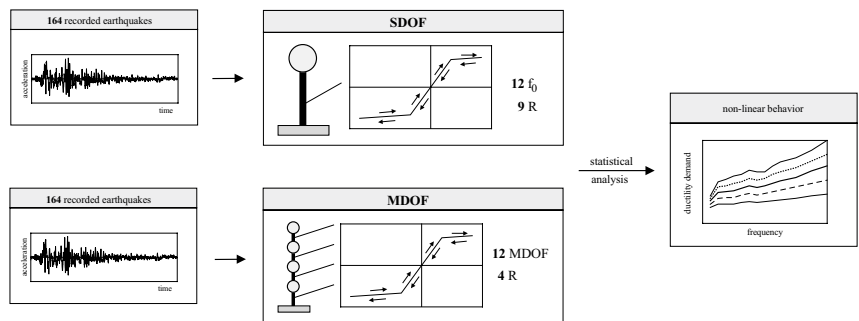


Figure 1: Schematic description of the followed methodology.

Statistical analysis of the seismic response is performed for twelve initial natural frequencies (f_0) representing the typical range of natural frequencies of buildings and for nine values of the strength reduction factor (R). The displacement ductility demand is considered to be a representative indicator for the non-linear seismic behavior. The investigations are later extended to MDOF systems. The motivation behind this second part of the investigations is to test if



the results obtained for SDOF systems hold true for MDOF systems representing buildings.

3 Ground motions

164 registered ground acceleration time histories are used. In order to consider earthquakes that may produce significant non-linearities in the structural behavior, only recordings with a magnitude larger than 5 were considered. Figure 2 shows the magnitude-epicentral distance relationship of the set of 164 selected recordings. The magnitudes range from 5.0 to 7.6, the epicentral distances range from 2 to 195 km and the peak ground accelerations (PGA) range from 0.61 to 7.85 m/s².

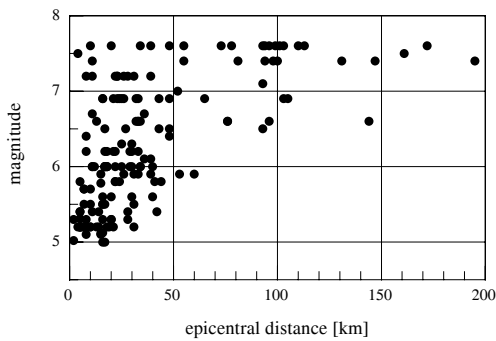


Figure 2: Magnitude-epicentral distance relationship of the 164 recordings.

4 Investigations with SDOF systems

According to the methodology illustrated in Figure 1, the following parameters are examined in the first part of the study with SDOF systems: the initial natural frequency, the strength reduction factor, the hysteretic energy dissipation capacity using three hysteretic models and the viscous damping ratio. The non-linear SDOF system is defined by the following parameters: the initial natural frequency (f_0), the strength reduction factor (R) and the hysteretic model.

Twelve initial natural frequencies covering the range of frequencies of usual buildings are evaluated. The natural frequencies range from $f_0=0.5$ Hz to 4.0 Hz in steps of 0.25 Hz. The following hysteretic models are used to compute the non-linear responses: a bilinear self-centring model (S-model), an elastoplastic-model and the modified Takeda-model. The force-displacement relationships defining the S-model and the modified Takeda model are plotted in Figure 3.

The bilinear self-centring hysteretic model is the simplest model to represent elements without or very little hysteretic energy dissipation capacity. It is called self-centring because it unloads such that there is no residual displacements when the external load is reduced to zero. Because of its shape, this model is



called “S-model”. The post-yield stiffness is defined as being a fraction of the initial stiffness.

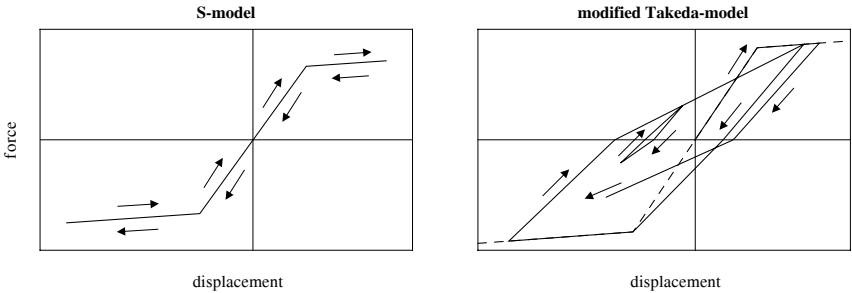


Figure 3: The S-model and the modified Takeda model.

The modified Takeda-model simulates well the features of ductile structures such as capacity designed reinforced concrete structures. The Takeda-model was initially proposed by Takeda et al. [3]. It was later modified by many researchers. The version used here is the one of Allahabadi and Powell [4].

4.1 Results with SDOF systems

Relative displacements are used to represent the dynamic non-linear response. Because the computations are repeated for each recording, 164 values are used to determine the average and standard deviation for each couple of strength reduction factor and initial fundamental frequency. The results for the displacement ductility demand are presented first, in terms of mean values and in terms of variability. Later section relates the impact of the damping ratio on the non-linear behavior.

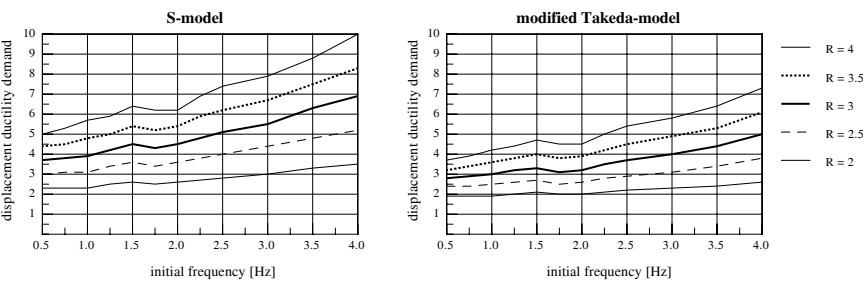


Figure 4: Mean values of the displacement ductility demand as a function of the initial frequency of SDOF system.

4.1.1 Mean values of displacement ductility demand

The displacement ductility demand (μ_A) is defined as the ratio of the peak non-linear displacement to the yield displacement. The displacement ductility demand varies strongly between different considered ground motions but mean values obtained from a large number of ground motions show clear tendencies.

Typical results are illustrated in Figure 4. The plotted results correspond to a post-yield stiffness equal to 10% of the initial stiffness for both hysteretic models. The plots show very similar tendencies. As expected, larger displacement ductility demands are related to S-model. However, the differences are not pronounced. Moreover, the general shape of the curves is conserved. The displacement ductility demand stays more or less constant for frequencies below 2 Hz and afterwards increases with increasing frequency.

4.1.2 Variability of displacement ductility demand

Besides mean values, variability is the main statistical characteristic of the displacement ductility demand. Typical results are illustrated in Figure 5 for one value of the strength reduction factor ($R=3$). In order to characterize the variability, the mean values (solid line) are plotted together with mean values plus one standard deviation and mean values minus one standard deviation (dotted lines) as a function of the initial frequency of the SDOF systems. Based on the plots of Figure 5, the comparison between the S-model and the modified Takeda-model shows that even if variability is significantly larger for the S-model, there are similarities in both hysteretic models. Variability stays approximately constant for frequencies below 2 Hz and significantly increases afterwards.

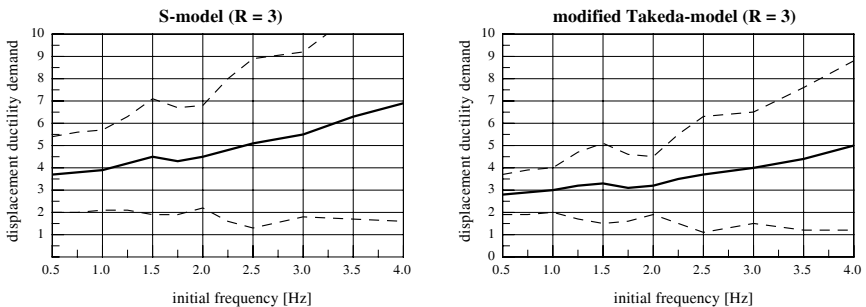


Figure 5: Variability of the displacement ductility demand as a function of the initial frequency of the SDOF system.

4.1.3 Viscous damping ratio

For the viscous damping ratio, the performed parametric study generated unexpected results. Figure 6 shows typical results. The displacement ductility demand stays approximately constant for all considered damping ratios except those between 0% and 1%. The displacement ductility demand is smaller than the obtained plateau for damping ratios between 0% and 1% and gradually increases in this range until it stabilises at a constant value. This phenomenon is independent of the initial frequency and the value of strength reduction factor. Obviously, the damping ratio reduces the elastic and the non-linear response by the same amount.



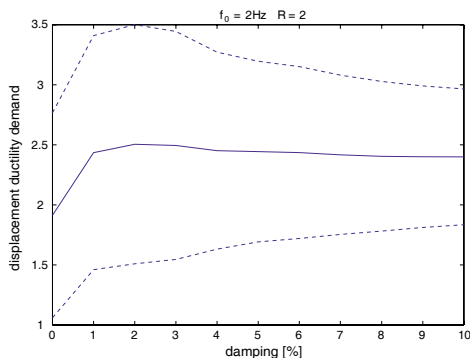


Figure 6: The impact of the viscous damping ratio is restricted to the range between 0% and 1%.

4.2 A simplified formulation for R - μ_Δ - T relationship

The main objective of the research project is to propose strength reduction factor-displacement ductility demand relationship for structures without hysteretic energy dissipation capacity. However, similar to the equal displacement rule, the formulation should remain as simple as possible. In brief, for structures without capacity of hysteretic energy dissipation, the study is focused on the improvement of the equal displacement rule for the frequency range below 2 Hz, particularly for strength reduction factors between 2 and 3. Figure 4 shows that the equal displacement rule ($\mu_\Delta=R$) leads to underestimating the results for all frequencies above 0.5 Hz. By contrast, the usual competing empirical rule of equal energy ($\mu_\Delta=R^2/2+1/2$) leads to largely overestimated results for strength reduction factors above $R=2$ (e.g. $\mu_\Delta=5$ for $R=3$). Consequently convenient relationship should lie between these two common empirical rules. As a boundary condition, the relationship should lead to $\mu_\Delta=1$ for $R=1$. Based on the results of the parametric study, a simplified formulation for R - μ_Δ - T relationships is proposed as follows:

$$\mu_\Delta = 3R/2 - 1/2; \quad T > 0.5 \text{ s.} \quad (1)$$

The proposed R - μ_Δ relationship is printed in Figure 7 and plotted together with the obtained results of Figure 4. The relationship (1) is set to be valid in terms of mean values for the frequency range below 2 Hz and for strength reduction factors between 2 and 3. The relationship should be adjusted if it is to be used for higher strength reduction factors. One suggested modification consists of removing the constant member. Note that for $R=2$, Equation (1) and the empirical equal energy rule lead to identical results ($\mu_\Delta=2.5$).



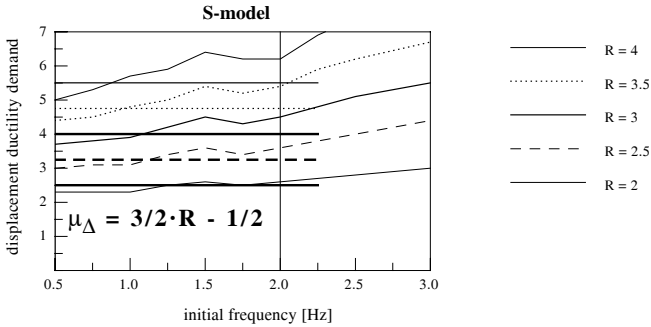


Figure 7: Proposed R - μ_{Δ} relationship in comparison with the results.

5 Investigations with MDOF systems

In order to verify the validity of the results obtained for SDOF systems for multistorey structural wall buildings, a second investigation is performed with MDOF systems. Non-linear responses are computed using the same database of 164 recordings. The same type of non-linear constitutive law according to the S-model is used for every storey of the MDOF system.

5.1 Definition of MDOF systems

Figure 8 shows an example of the structures which were used in this part of the study. The model represents a building with four stories. The mass of the building is modelled as a concentrated mass (M) at each story level and it is kept the same for every story. The slabs are considered infinitely rigid in their in-plane direction and no rotational degrees of freedom are introduced. Each story has one horizontal lateral displacement degree of freedom. All the stories are modelled with the S-model. This hypothesis is based on the assumption that the slabs are infinitely rigid and therefore every wall element between the slabs can undergo a rocking behavior with no coupling effect. All other failure mechanisms, such as sliding or shear, are excluded.

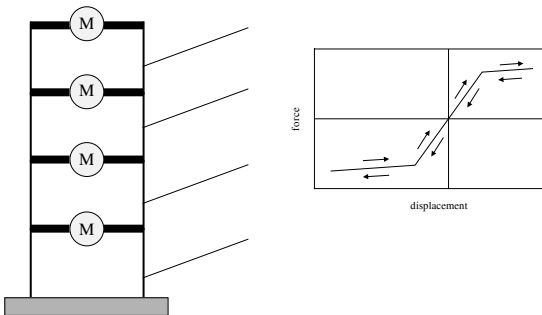


Figure 8: Sketch for a four-story structure used in the MDOF systems investigations.



The determination of the displacement ductility demand is carried out for a two story, a four story and a six storey building model. A parametric study is performed for four values of the initial story stiffness ($K=100$ N/m, 500 N/m, 1000 N/m and 2000 N/m) and for four values of the strength reduction factor ($R=1.5, 2.0, 3.0$ and 4.0). The total mass is equal to unity. The resulting fundamental frequency for all MDOF systems investigated is given in Table 1.

Table 1: Fundamental frequencies of the MDOF systems.

Initial stiffness [N/m]	Frequency [Hz]		
	2 DOF	4 DOF	6 DOF
100	1.4	1.1	0.9
500	3.1	2.5	2.1
1000	4.4	3.5	3.0
2000	6.2	5.0	4.2

5.2 Equivalent SDOF system

To ensure a relevant comparison of the results between MDOF and SDOF systems, equivalent SDOF systems are defined for each MDOF system. An equivalent SDOF system follows the same hysteretic model as the stories of the corresponding MDOF system (S-model). Thus both systems have the same initial fundamental frequency. However, the post-yield stiffness for the equivalent SDOF system should be calibrated to reproduce the same global behavior as the corresponding MDOF system. The equivalence is determined on the basis of push-over curves and leads to a modification (multiplication) of the hardening coefficient for equivalent SDOF systems (1.2 times for 2 DOF, 0.8 times for 4 DOF and 0.7 times for 6 DOF systems).

5.3 Displacement ductility demand

The computation of displacement ductility demand with MDOF systems is not as straightforward as with SDOF systems. It is important to distinguish between local and global ductility. The $R-\mu_\Delta-T$ relationships are expressed for global displacement ductility demands. For example, the equal displacement rule is formulated for the global displacement ductility demand of a structure. Therefore, the comparison of the displacement ductility demand between SDOF and MDOF systems needs to be done on the basis of the global displacement ductility demand. The global displacement ductility demand is defined as the peak non-linear displacement at the top of the building divided by the top displacement at the stage when the first element reaches its yield relative displacement. The global yield displacement is the peak linear elastic displacement of the top of the building divided by the corresponding strength reduction factor.



5.4 Results with MDOF systems

The displacement ductility demand is chosen as a representative value for the non-linear behavior. In virtue of the discussion above, the global ductility is used to compute the displacement ductility demand. The results are plotted in Figure 9 as a function of the fundamental frequencies of the examined structures.

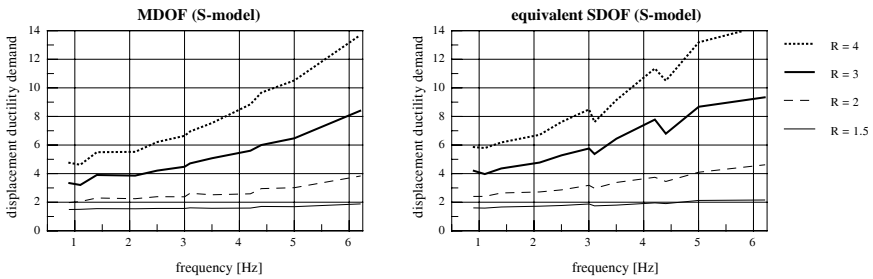


Figure 9: Mean values of the displacement ductility demands for MDOF systems and related equivalent SDOF systems.

The plots of Figure 9 show that the equivalent SDOF system (right) generally overestimates the displacement ductility demand when compared to the corresponding MDOF system (left). The difference lies between 10% and 15%. In the adopted methodology, some equivalent SDOF systems have a similar initial natural frequency (see Table 1) but a quite different post-yield stiffness ratio. This explains the abrupt drops in the force-displacement curves of the equivalent SDOF systems (Figure 9, right).

6 Summary and conclusions

In this paper, the seismic response of structures that show a non-linear rocking behavior such as slender unreinforced masonry shear walls or precast post-tensioned reinforced concrete elements is investigated. The displacement ductility demand is computed for a set of 164 registered ground motions. Statistical analyses are performed to characterize seismic performance. The obtained results reveal that hysteretic models without hysteretic energy dissipation capacity definitely do not lead to excessive displacement ductility demand. This is an important result that contradicts the widely held perception. It is often assumed that this kind of structural behavior is not an efficient mechanism to withstand strong earthquakes, even if it may be associated with significant deformation capacity. In the light of the presented results it is found that hysteretic energy dissipation capacity is not the unique characteristic of a good seismic behavior. The non-linear behavior due to the transition between initial stiffness and post-yield stiffness is the main favourable aspect that affects seismic behavior.



Note that since different yield displacements are considered for the definition of the non-linear systems, the results obtained for the displacement ductility demand may not be extended to those for the displacement demand.

Compared to non-linear SDOF systems, similar seismic behavior is also seen in MDOF systems. However, the SDOF system has a tendency to overestimate the displacement ductility demand of the corresponding MDOF system by about 15%.

The upper-limit value of 1.5, currently being recommended by the design codes for strength reduction factors of structures with limited hysteretic energy dissipation capacity considering only their overstrength is definitely too conservative. As long as the structural elements have a large displacement capacity, strength reduction factors up to 3 can be adopted. Note that additional attention should be paid to the fact that no other structural failure mechanism can take place and that strength degradation may be excluded. For frequencies below 2 Hz a prediction of the displacement ductility demand may be obtained by using the proposed $R-\mu_\Delta-T$ relationships. This conclusion is important for many cases. One example are slender unreinforced masonry elements subjected exclusively to the “rocking” failure mode.

References

- [1] Miranda E. and Bertero V., Evaluation of Strength Reduction Factors for Earthquake-Resistant Design. *Earthquake Spectra*. Vol 10, No. 2, pp. 357-379, 1994.
- [2] Trueb M., *Seismic behaviour of non-linear elements with little energy dissipation*. Master thesis, Ecole Polytechnique Fédérale de Lausanne (EPFL), Switzerland, 2005.
- [3] Takeda T., Sozen M. A. and Nielsen N. N., Reinforced concrete response to simulated earthquakes. *Journal of the Structural Division. Proceedings of the American Society of Civil Engineers (ASCE)*. Vol. 96, No. ST12, 1970.
- [4] Allahabadi R. and Powell G. H., *Drain-2DX User Guide*. Report No. UCB/EERC-88/06. College of Engineering, University of California, Berkeley, 1988.



Section 6

Site effects and geotechnical aspects

This page intentionally left blank

The 2006 Yogyakarta earthquake – a preliminary study of deaths

J. M. Nichols

Department of Construction Science, TAMU, Texas, USA

Abstract

The M_w 6.3 Yogyakarta, Java, Indonesia, Earthquake occurred on May 27th 2006 and killed more than 5,000 people and injured more than 36,000 people. The earthquake had a duration of 52 seconds, which is a long duration for the magnitude of the event, and left 600,000 people without shelter. The earthquake occurred near Mt. Merapi, which is an active volcano. The paper's purpose is to outline the existing knowledge about the earthquake and place this knowledge within the context of recent studies of the statistics of earthquake fatalities. The study of the earthquake deaths and injuries form part of an ongoing investigation into the development of methods to estimate fatalities in given earthquakes, and in particular the upper bounds to the fatalities observed in a special group of rare fatal earthquakes.

Keywords: Java earthquake, earthquake fatalities, Maximum earthquake deaths.

1 Introduction

An M_w 6.3 earthquake occurred near the city of Yogyakarta (20 km SSW) on the island of Java on May 27, 2006 resulting in than 5782 deaths, and 36,299 injuries [1, 2]. This death toll places the 2006 Yogyakarta earthquake into the group of rare earthquakes in the last two millennia that define the bounds of deaths in such events. These rare events, only seven in the 20th century, are critical to understanding the site factors that affect losses in earthquakes. This paper updates the information available at the end of the 20th century to revise the original mathematical model [3] developed to estimate earthquake deaths for a given magnitude.

Shiono [4] completed the seminal study on earthquake fatality rates in 1995 using the 1976 Tangshan earthquake as the basis for the analysis. The 1976



Tangshan earthquake estimated death toll was 242,000, with a peak fatality rate of 30 to 50% in the Felt X to XII area. This Chinese earthquake represents the largest fatal event of the 20th century. Shiono demonstrated a clear and unambiguous relationship between population fatality rates and the distance from the epicentre of the earthquake.

2 Literature review

Structural engineering standards developed in the latter part of the last century allow for the construction of modern houses, structures, and buildings that can withstand some, but not all, earthquakes [5]. Nichols et al [3] show in a study of the earthquake fatalities against earthquake magnitude that a bounding function could be established for the twentieth century earthquake fatality data. Figure 1 shows the original bounding function, plotted with the rare fatal events that provide the points to determine the bounding function for fatalities.

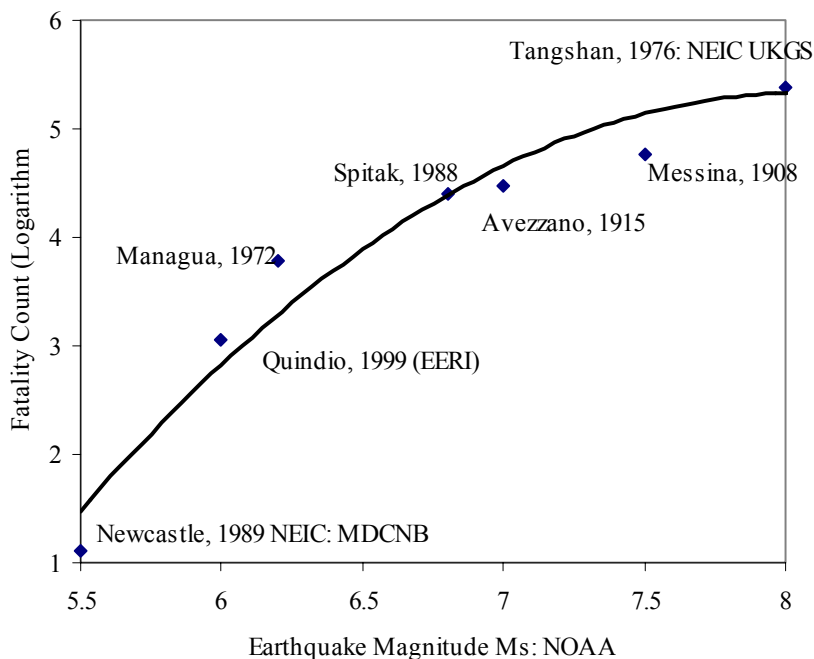


Figure 1: The original bounding function for the fatality count against earthquake magnitude.

The form of this original bounding function, based on data from the period before 2000 AD, was estimated using standard regression techniques (Eqn. (1)).

$$\log(\Xi_B(M)) = 9.335M - 0.577M^2 - 32.405 \quad (1)$$



The function has a regression coefficient of 0.95 for a fatality count of $\Xi_B(M)$ and an earthquake magnitude M . Earthquake magnitudes were determined from the USGS National Earthquake Information Center [6]. These magnitudes are M_s from the NOAA catalogue, except for 1976 Tangshan, and the 1989 Newcastle earthquakes that are coded UKGS and MDCNB respectively in the catalogue. The magnitude of the 1999 Quindío, Columbia, earthquake was based on the EERI report [7] and was coded M_l . The fatality counts, Ξ_R , for the remaining earthquakes of this last century are below the bounding function, $\Xi_B(M)$.

There are usually observable and simple reasons why these fatality counts are lower than the bounding function, e.g., earthquakes in remote regions, rates of attenuation, population densities, higher building construction standards, and timing of the event. However, given the post earthquake studies from the twentieth century it was evident that the closer the meizoseismal area of a large earthquake was to the center of a population, then the higher was the potential mortality rate [7–9].

The fatal meizoseismal area in the context of this paper is the area of damage that can cause death. In this case we are very specifically referring to areas enclosed by the line of building damage likely to cause death. Shiono's data [4] indicates that this limit of deaths is at about the iso-seismal delineating the felt intensities five and six. The damage outside this area is economically significant, but not generally fatal. The second recent observation is the increase in fatal earthquakes per annum during the 20th century from about 4 annually in 1900 to 16–20 annually in 2000.

3 2006 Yogyakarta earthquake details

Figure 2 shows the location of the major damage center in Java from the May 27th earthquake. Table 1 presents the earthquake details from the USGS report [2] for the event. The USGS report provides the following MMI intensity data, “felt (IX) at Bantul and Klaten, (VIII) at Sleman and Yogyakarta, (V) at Surakarta, (IV) at Salatiga and Blitar and (II) at Surabaya. Felt in much of Java. Also felt at Denpasar, Bali”

The critical data for the study is the distribution of losses in the event and the corresponding distribution of the population at the time of the event. The estimated population of Yogyakarta is 500,000, providing a raw estimate of the fatality rate across the city of about 1.2%. This fatality rate is comparable with the overall rate in the M 7.7 2001 Gujarat earthquake of 1.1%, even though the Java event is a significantly smaller earthquake. The other interesting feature is the relatively poor construction in India which is blamed for the large death toll, needs to be compared to the Indonesian building standards.

4 Earthquake consequences to the bounding function

The 27th May Java earthquake proved to be one of those earthquakes that cause fatalities as well as building damage, but at an overall fatality rate for the



magnitude of the earthquake that only occurs about once a decade. The interesting feature of the first 7 years of the 21st century is that two such events have occurred in this period. The first in Italy with deaths of 22 schoolchildren in an M 5.4 event and now the deaths of 5782 people in an M_w of 6.3 in Java.

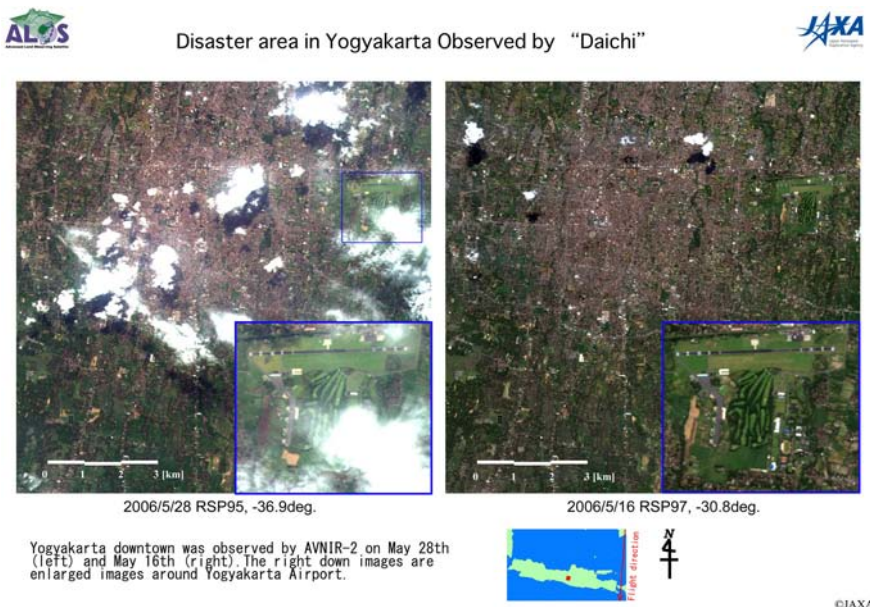


Figure 2: Disaster area in Java (Jaxa [10]).

Table 1: USGS Report on Event.

Description	USGS Official	Comments
Magnitude M_w	6.3	Strong
Date (UTC)	Friday May 26, 2006	
Time (UTC)	22:53:58	
Date (Local)	Saturday May 27, 2006	
Time (Local)	5:53:58 AM	
Location	7.962°S, 110.458°E	
Depth	10 (km)	Set by location program
Distances from Cities	(1.) 20 km SSE of Yogyakarta (2.) 455 km ESE of Jakarta	
Location uncertainty - horizontal	± 7.5 km	
Parameters	$N_{st} = 130$ $N_{ph} = 130$ $D_{min} = 220.2$ km $R_{mss} = 1.4$ seconds $G_p = 43^\circ$	Teleseismic moment magnitude



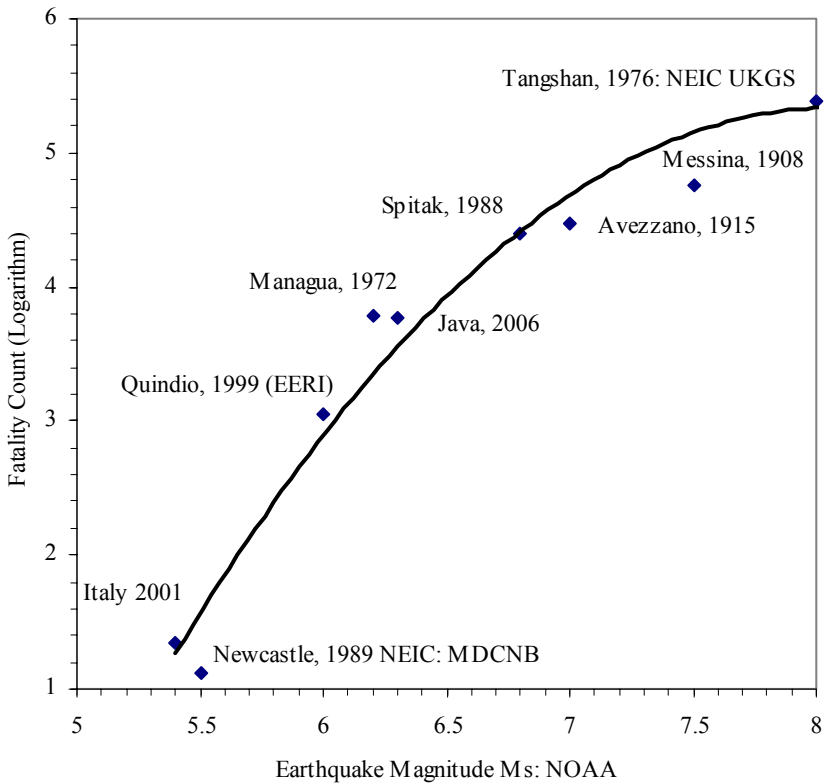


Figure 3: Revised fatality data and bounding function.

Figure 1 shows the plot of the pre-2000 data for high fatality rate earthquakes against earthquake magnitude. The recent Italian and Javanese earthquakes have been added to the simple database used to develop Figure 1. This simple database was analysed to determine the revised bounding function for peak fatality rates plotted against earthquake magnitude. Figure 3 shows the revised plot and revised bounding equation.

The form of this revised bounding function, based on data from the period before 2008 AD, was estimated using standard regression techniques and is given in Eqn. (2)

$$\log(\Xi_B(M)) = 9.2276M - 0.572M^2 - 31.884 \quad (2)$$

The function has a regression coefficient of 0.97 for a fatality count of $\Xi_B(M)$. The slight increase in the regression coefficient can be attributed to the increased number of points in the plot. The critical difference is the estimates of the fatalities for the two bounding functions. Table 2 lists a set of the fatality estimates for the two bounding functions for a range of earthquake magnitudes. The percentage differences between the two equations are presented in Table 2.



The differences show a slight fall at the upper end of the range, and a 15% increase in losses in the mid range event. The clear question is whether the 1908 Messina event should be considered a bounding function and this question will be the subject of further research.

Table 2: Changes in the bounding equation from 2000 to 2007 AD.

Earthquake Magnitude	Original Equation 2000 AD (Estimated fatalities)	Revised Equation 2007 AD (Estimated Fatalities)	Percentage Difference
5	1	1	129%
5.5	30	37	121%
6	681	782	115%
6.5	7,839	8,563	109%
7	46,452	48,540	104%
7.5	141,661	142,438	101%
8	222,331	216,371	97%

The May 27th 2006 Yogyakarta earthquake is clearly one of the largest fatal events for a given earthquake magnitude recorded in the last 100 years. This event will provide with further research a better picture of the impacts that the five factors have on the earthquake fatality rate for mid range fatal earthquakes. The five factors are a density function, a building and ground factor, an attenuation factor, a fatality rate factor and an aleatory uncertainty factor.

5 Conclusions

The Yogyakarta earthquake that occurred on the 27th May 2006 has an official death toll of 5782 people, with many more injured and left homeless. Recent research on fatal earthquakes has shown an increase in the number of fatal events from about four per annum in the year 1900 to about sixteen to twenty in the year 2000. A much smaller group of fatal earthquakes have particularly high fatality rates for the magnitude of the event. This group of high rate fatal earthquakes has had two new members added to the set since 2000 AD, which are the M 5.4 2001 Italian earthquake resulted in the death of 22 schoolchildren and the 2006 Yogyakarta earthquake. This paper presents the changes to the estimated fatality bounding function when plotted as a function of earthquake magnitude for these two additional rare fatal events. The mid range of the earthquake magnitudes has a fifteen percent increase in the estimated tolls. The upper end has a small drop of three percent, which is not considered statistically significant. The Yogyakarta earthquake requires further study to determine the factors that caused the high death toll.



Acknowledgements

The National Science Foundation funded this research under Grant Number CMMI-0703846 entitled “2006 Java Earthquake -- A Study of the Deaths and Injuries”

References

- [1] CNN.COM, <http://www.cnn.com/2006/WORLD/asiapcf/06/05/indonesia.quake/> (accessed 26th Feb 2007), June 6th 2006, 2006.
- [2] USGS Report on the 2006 Yogyakarta Earthquake <http://earthquake.usgs.gov/eqcenter/recenteqsww/Quakes/usneb6.php> (Accessed 9 Feb 2007), 2006.
- [3] Nichols, J.M., Lopes De Oliveira, F., And Totoev, Y.Z., The development of a synthetic fatality function for use in the economic analysis of the rehabilitation and repair of structures, *StrucDam Conference*, Brazil, 2000.
- [4] USGS, Current Earthquake Information http://neic.usgs.gov/neis/bulletin/01_EVENTS/01_EVENTS.html (Accessed March 28, 2001), 2001.
- [5] EERI, *The Quindío, Columbia Earthquake of January 25, 1999*, www.eeri.org, 1999.
- [6] Holmes, W.T., (2000), The 1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, *Earthquake Spectra*, **16(1)**, pp. 101-14, 2000.
- [7] Shiono, K., (1995) Interpretation of published data of the 1976 Tangshan, China Earthquake for the determination of a fatality rate function, *Structural Engineering / Earthquake Engineering*, **11(4)**, pp. 155s-163s, 1995.
- [8] Ward, S.N., and Valensise, G. R., Fault parameters and slip distribution of the 1915 Avezzano, Italy, earthquake derived from geodetic observations, *BSSA*, **79(3)**, pp. 690-710, 1982
- [9] Algermissen, S.T., *A study of earthquake losses in the San Francisco Bay Area Data and Analysis*, US Dept of Commerce, NOAA: SF, 220, 1972.
- [10] JAXA, Yogyakarta earthquake satellite imagery http://www.jaxa.jp/press/2006/05/20060528_daichi_e.html (accessed 26th Feb 2007), 2006.



This page intentionally left blank

Local seismic amplification analysis in the industrial area of Sulmona, Central Italy

A. Rinaldini, A. Grillo & A. Marino

Department of Productive Activities and Interaction with the Environment, ISPESL, Italy

Abstract

This paper deals with local seismic amplification analysis in the industrial district of the Sulmona basin (Central Italy) using both Nakamura's HVSR technique and 1-D numerical simulation computed with Shake91 code. An extensive geognostic database allows one to make out the geometries of the subsurface layers up to a depth of 50 m and cross-hole tests led to characterization of the dynamic proprieties of terrenes, namely the shear wave velocities values. Noise measurements were collected in the 0.1-10 Hz frequency domain and data were used to calibrate the 1-D simulations, performed using as source a seismic record of $M=5.5$. The results obtained point out that local site effects are present in the central and eastern parts of the basin. Moreover, the comparison between the 1-D simulation spectra and the *spectra of project* provided in the Italian norms enhances the fact that in these areas the expected response is underestimated in the increased frequency range of buildings.

Keywords: geognostic investigations, dynamic properties, H/V spectral ratios, resonance frequencies, 1-D simulation, industrial activities.

1 Introduction

Intramontane basins are a peculiar physiographic feature of the Apennine range in Central Italy. With flat extensions up to hundreds of km^2 , these endoreic structures of tectonic origin are characterized by a high seismicity and facilitate the development of important urban settlements and productive activities into the Apennine chain.

In this paper are presented the results of background noise measurements and 1-D simulations performed in the area of Sulmona. The main goal was to assess



the local seismic response using different methodologies in an area where a potential seismic hazard exists caused by moderate-to-strong earthquakes originated in the Apennines.

2 Geological setting

The Sulmona basin is the easternmost of the great tectonic intramontane depressions that characterize the inner part of the Central Apennines. With a rectangular shape, the basin follows the patterns of the major tectonic structures active in the area [1, 2]. The mountain ranges that delimit the plain are formed by calcareous successions deposited in different structural and paleo-geographical domains, fig.1; the basin is filled up with fluvial-lacustrine and continental deposits aged from Pliocene to Quaternary, with a thickness of about 500 meters.

2.1 Structural geology

The plain is delimited by Meso-Cenozoic limestone ridge-lines that since the late Miocene have been shortened, folded and thrust toward NE by a complex sequence of compressional phases (Parotto and Praturlon [3]). Post-orogenic extensions took origin during the late Pliocene, followed by a distensive tectonic that dissected the ancient fold-and-thrust structures and reactivated as normal faults many of the old surfaces of weakness, originating a half-graben structure deepening eastward.

The main structural feature is constituted by the Apennine trend fault system that delimits the plain in the N-E sector, active until the Holocene (Miccadei *et al* [2]).

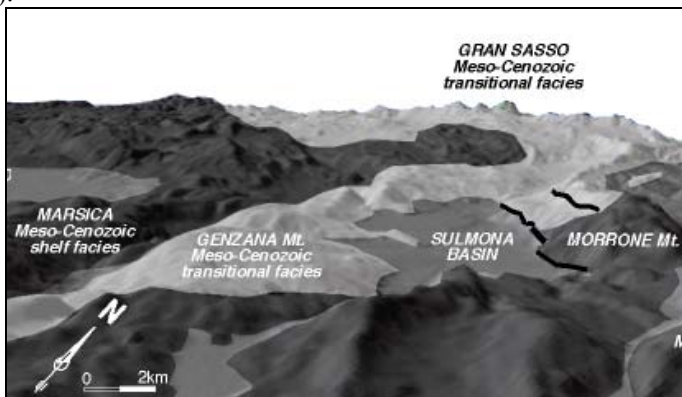


Figure 1: Three-dimensional model of the Sulmona basin surrounded by Mesozoic ridges of different paleogeographic origin.

2.2 Seismicity

The studied area is part of a highly seismic region of Central Italy and the available historic records are quite detailed [4, 5]. Earthquakes with epicenters in



the neighborhood of the town of Sulmona had a maximum intensity of VII MCS; yet Boschi *et al* [5] report that some major seismic events occurred on a regional scale, *e.g.* the 1706 Maiella (IX-X MCS) and the 1915 Fucino (VIII-IX MCS) earthquakes, shattering the area with loss of lives and wide building damage.

3 Subsurface characterization of the Sulmona area

The interpretation of the H/V spectral ratios and transfer functions represents a fundamental task in the seismic characterization of a sediment site. To reach this goal, it is necessary the knowledge of thicknesses, geotechnical properties and shear-waves velocities of the subsurface layers.

A geotechnical characterization of the subsoil in the industrial district of Sulmona was thus accomplished, using a geognostic database that was developed both collecting new data from in situ prospectings, and acquiring informations provided by previous investigations.

3.1 In-situ investigations

The computing of the shear-waves velocities of subsurface layers was performed carrying out cross-hole (C-H) prospectings in 3 couples of boreholes located in and out the industrial district, fig. 2. Data from drillings were also used to integrate and validate the informations stored in the geognostic database.

Laboratory analyses on undisturbed core samples defined both the main static and dynamic geotechnical properties of the collected materials.

3.1.1 Stratigraphies

Two boreholes (S. Croce and La Torre) were drilled in continuous coring outside the industrial district, near the Mt. S. Cosimo ridge and reached depths of 58.5 and 54 meters respectively. In S. Croce site the lithologic succession is mainly constituted by thinly stratified silty-sands and silty-clays. In La Torre area the stratigraphy is formed by gravel layers interbedded with silty-clays and clays.

The sounding named Agricoltura was drilled in the eastern side of the industrial district, reaching a depth of 40 meters. Here the stratigraphic column is constituted by gravels that intercalate at various depths with silty-sands and silty-clays layers. Over-consolidated clays follow up to the bottom of the borehole.

3.1.2 C-H investigations

This prospecting has the advantage to investigate undisturbed materials and to operate on the rock mass scale. S-waves measurements were acquired using a three-directional geophone firmly fixed to the hole covering by means of a packer inflated with compressed air.

A hole hammer was the energizing source used. The hammer was lowered into one of the holes and fixed at the desired depth with extendable jaws controlled from outside. During data acquisition, step by step, source (S) and sensor (G) were situated at the same height, allowing one to investigate the horizontal paths of shear waves, fig.3.



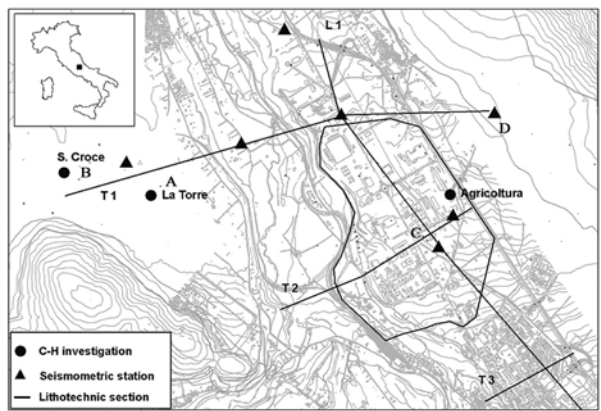


Figure 2: Location of in-situ prospectings carried out in the Sulmona industrial district. Both positions of lithotechnical cross-sections and synthetic stratigraphies are also shown.

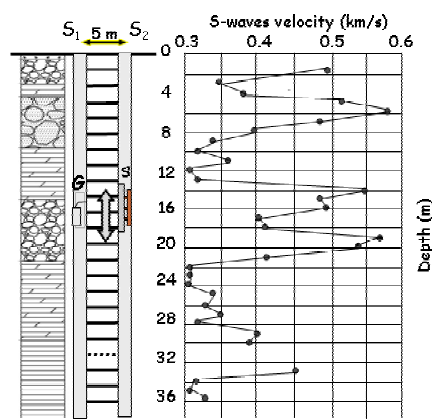


Figure 3: Shear waves velocity values measured in Agricoltura site.

3.1.2.1 Results Shear wave velocities measured in all the investigated sites show for gravels and coarse sands mean velocity values of about 450 m/s; for silty-clays and clays values range from 350 to 400 m/s.

3.2 Geognostic database

A detailed framework of the geometries and the physical-mechanical properties of buried layers was obtained using a geognostic database developed from 122 stratigraphic columns and 40 geognostic tests, half of them covering the industrial area. Many data were furnished by the geologist Mancini [6], others were collected during previous investigations (Rinaldini *et al* [7]).



Soils with similar geotechnical proprieties were unified in homogeneous lithotechnical units and 4 cross-sections were drawn tracing the arrays of the seismometric measurements. This led to the reconstruction of the geometry of terrenes up to a depth 50 m; the dataset provided also the quick identification, for each lithotechnical unit, of the main physical-mechanical properties affecting the seismic response.

4 Background noise measurements

A campaign of field investigations focused on the study of possible site effects was carried out around the industrial area of Sulmona. Background noise measurements were performed applying Nakamura's [8] technique, which is the ratio of the horizontal-component noise spectrum and that of the vertical component (HVSr).

Seismic arrays were set in the basin in continuous recording for the eventual acquisition of strong-motion data. As proposed in other studies [9–11] was placed in the site of Roccasale a remote reference-site station, situated in the eastern edge of the basin. Yet nor weak nor strong motion data were recorded during the measurements.

4.1 Field data

During early March 2005 three arrays of noise measurements, each composed by 5 seismic stations, were performed in free-field disposed along the basin both longitudinally and transversally. Each seismic station was formed by a K2 seismometer, a GPS and three SS1 velocity sensors of 1 Hz frequency.

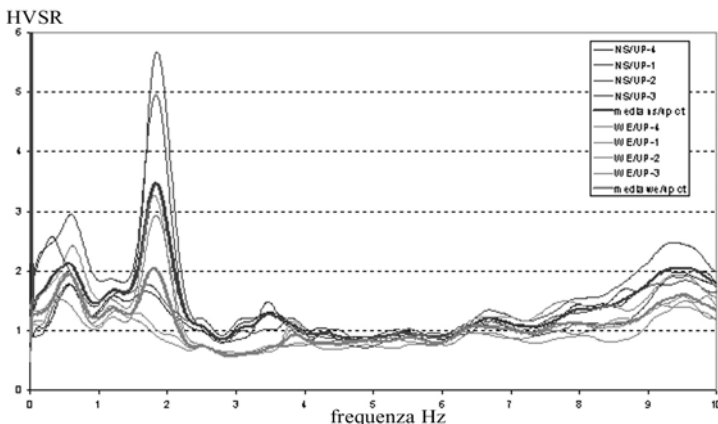


Figure 4: H/V spectral ratio measurements in Piano la Torre. Two peaks are reported, for each component, in the 0.6 Hz and 2 Hz frequency range.



Background noise measurements were collected in the 0.1–10 Hz range, computing, for each component, the means of the Fourier spectra obtained in acquisition windows 40 sec-long. The H/V spectral ratios acquired during the day were compared and mediate for each component, fig. 4.

4.2 Results

HVSR analyses showed an amplification peak of about 0.6 Hz in all the stations located in the central sector of the basin, except for Park Hotel station where were not detected peaks under 3 Hz. In the easternmost margin of the basin (Fonte Amore site) were not recorded peaks under 1 Hz.

The spectral amplitudes measured on sediment sites are higher than that of the rock site. However, the Fourier spectra analysis showed that during the diurnal records, for all the stations and for each component, is present a sharp peak of 2 Hz. The absence of such peaks during night times lead to suppose a human noise origin caused by traffic and industrial activities.

5 1-D numerical simulation

5.1 Basics

Along their paths towards the surface, shear waves modify their frequencies and amplitudes if pass from a speedy bedrock into a soft soil constituted by a succession of plain-parallel layers. This phenomenon originates site-effects on the ground that can be analyzed computing the ratio between the Fast Fourier spectrum (FFT) on the surface of the selected layer and that of the same component on the bedrock.

5.2 Methodology

The 1-D numerical simulations were aimed at analyzing the elastic behaviour of the ground under seismic shaking conditions. According to a linear-equivalent approach [12, 13], that generally fit with the geologic conditions present in the studied area, was used a mono-dimensional model and adopted the hypothesis of plain-parallel layers.

5.2.1 Data input

The amplification functions (AF) of the ground were obtained using the SHAKE-1D calculation code, applying 4 separate rules of decay G/G_0 and a dumping factor of 5% in the hypothesis of PGA values equal to 0.09 g. These PGA values were also used to elaborate the spectra of project proposed in the Italian Technical Norms for the Constructions [14].

All numerical simulations obtained were calibrated on the results of the H/V spectral ratio measurements.

5.2.1.1 Geologic information The geognostic database allowed to make out 5 synthetic stratigraphic columns, namely La Torre (A), S. Brigida, Hotel S. Croce



and La Vigna (B), Park Hotel (C), Fonte Amore (D) and Raiano (E) on the sites where were located the HVSR stations. Each layer of the stratigraphic columns was thus characterized in terms of the dynamic properties, as shown in table 1.

5.2.1.2 Strong motion source The Raiano seismic station was used as bedrock and the input parameter for the strong motion simulation was the record of the 1984 S. Donato Valcomino earthquake ($M=5.5$), measured in the Atina station (FR), that belongs to the National Accelerometric Network (NAN).

Table 1: Dataset showing the main dynamic characteristics of site B.

B stratigraphy (S. Brigida, Hotel S. Croce, La Vigna)							
layer	lithology	S-waves	density	height	depth	D	G/G ₀
		m/s	t/m ³	m	m	%	
I	silt	350	1.90	1.5		5	2
II	gravel	450	1.80	11	12.5	5	1
III	silty-clay	350	1.90	12	24.5	5	3
IV	gravel	450	1.80	4.5	29	5	1
V	silty-clay	350	2.00	18	47	5	3
VI	silt	400	2.10	103	150	5	3
bedrock	limestone	1000	2.40			1	

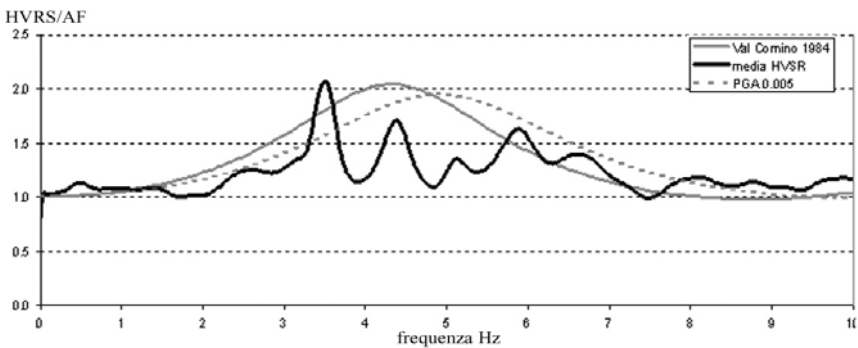


Figure 5: Diagram showing the H/V spectral ratio and the AF values in Park Hotel site.

5.3 Data output

The amplifying functions obtained from the strong motion simulation enhance the presence of frequencies peaks in the 0.6 Hz and the 2 Hz range. These values are respectively referable to the thickness of the deposits and to the second vibration mode of the ground. The amplification factor for such frequencies is respectively about 2.5 and 1.8. These results are quite similar in all the investigated sites, except for the Park Hotel stratigraphy.



5.3.1 Park Hotel site

The amplifying function enhances a resonance value included in a frequency range from 3 Hz to 5 Hz, probably caused by the local stratigraphy. In fact, the 1-D simulation and the HVSR data indicate the lack of a sharp rigidity contrast between the sedimentary cover and the seismic bedrock, constituted in this site by flysh. The correspondent value of the amplification factor is about 2, fig 5.

5.4 AF spectral analyses

The AF spectra obtained from the 1-D simulation were compared with the spectra of project proposed in Technical Norms for the Constructions, and for each stratigraphy was adopted the related soil-type, as shown in table 2.

Table 2: Parameters of the Technical Norms and related stratigraphies.

Sites	Soil	V_{S30} (m/s)	S	T_B	T_C	T_D	ξ %	a_g
A	C	411	1.25	0.15	0.50	2.00	5	0.35
B	C	411	1.25	0.15	0.50	2.00	5	0.35
C	C	455	1.25	0.15	0.50	2.00	5	0.35
D	E	452	1.25	0.15	0.50	2.00	5	0.35
E	C	400	1.25	0.15	0.50	2.00	5	0.35

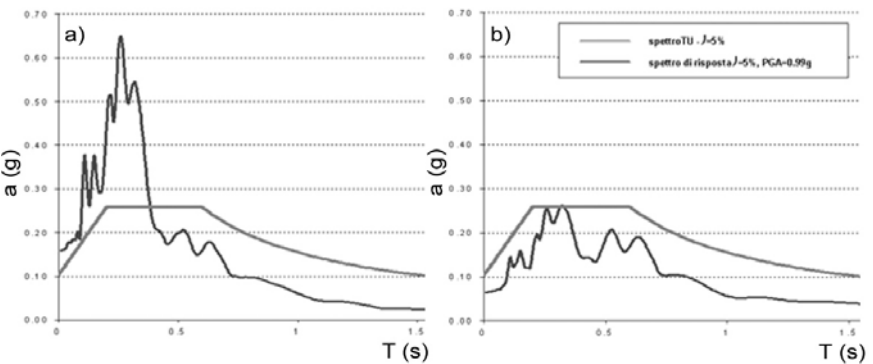


Figure 6: Comparison between the obtained 1-D spectrum of response and the elastic spectrum of project in: a) Fonte Amore, b) Raiano.

In the A, B and E sites the obtained AF fits with the spectrum of project; otherwise, in C and D the spectra obtained in the 1-D simulation are higher than the spectra of project for periods of 0.3 - 0.4 s, as shown in figures 6a) and 6b). The C and D stratigraphies are respectively related to a seismic substratum that lacks in a high rigidity contrast, and to an area where the thickness of soft soils reaches the maximum depth.



6 Conclusions

This study deals with the assessment of the local seismic response in the Sulmona basin, applying two different methodologies. In the former, based on the HVSR technique, results showed amplification peaks of about 0.6 Hz in the stations located in central sector of the basin, except for Park Hotel site. A 2 Hz peak was also measured in all the seismometric stations during the diurnal records, probably due to anthropic factors.

The latter technique consisted in a numerical simulation performed under strong motion conditions, using the SHAKE 91 software. As input parameters were used the dynamic proprieties of soils and the seismic source was the 1984 S. Donato Valcomino earthquake ($M=5.5$), measured in the NAN station of Atina. The amplification functions of the ground, obtained with SHAKE-1D, were calibrated with the HVSR investigations data.

The 1-D simulation results, compared with the spectra of project provided in the actual Italian norms, enhanced the fact that the expected response, in the interested frequency range of buildings, is underestimated for the stations of Park Hotel and Fonte Amore. These are located respectively in the central sector of the plain, where the seismic substratum lacks a high rigidity contrast, and in the eastern margin of the basin where the lithologic succession reaches the maximum thickness.

References

- [1] Ciccacci, S., D'Alessandro, L., Dramis, F. & Miccadei, E., Geomorphological evolution and neotectonics of the Sulmona intramontane basin (Abruzzi, Apennine, Central Italy). *Zeitschrift für Geomorphologie*, **118**, pp. 27–40, 1999.
- [2] Miccadei E., Paron, P & Piacentini T., The SW escarpment of the Montagna del Morrone (Abruzzi, Central Italy): geomorphology of a faulted-generated mountain front. *Geografia Fisica e Dinamica Quaternaria*, **27(1)**, pp. 55–87, 2004.
- [3] Parlotto, M. & Praturlon A., Geological summary of central Apennines (Chapter 3). *Structural Model of Italy*, ed. C.N.R., Roma, pp. 257–311.
- [4] Postpischil, D., Catalogo dei terremoti italiani dall'anno 1000 al 1980. *Quaderni della Ricerca Scientifica*, **114(2B)**, pp. 239, 1985.
- [5] Boschi, E., Ferrari, G., Gasperini, P., Guidoboni, E., Smriglio, G. & Valensise G., *Catalogo dei forti terremoti in Italia dal 461 a.C. al 1980*, ING-SGA: Bologna, pp. 543–549, 1995.
- [6] Mancini, A., Personal communication, 19 April 2006, Consortium for the Sulmona industrial development, Sulmona, Italy.
- [7] Rinaldini, A., Pecci, M., Marino, A., Bellagamba, S. & Ciucci, M., Sviluppo di un database geologico per l'analisi dei rischi naturali nelle attività della Piana di Sulmona (L'Aquila). *Atti Quarto Convegno valutazione e gestione del rischio negli insediamenti civili e industriali*, Pisa, 110.pdf, 2004.



- [8] Nakamura, Y., A method for dynamic characteristics estimation of subsurface using microtremors on the ground surface. *Quarterly Reports of the Railway Technical Research Institute Tokyo*, **30**, pp. 25–33, 1989.
- [9] Borchardt, R.D., Estimates for site response spectra for design (methodology and justification). *Earthquake spectra*, **10**, pp. 617–653, 1994.
- [10] Lachet, C., Bouchon, M., Theodulidis, N. & Bard, P.Y., Horizontal to vertical spectral ratio and geological conditions. *Proc. of the X Europ. Conf. On Earthquake Engineering*, Balkema: Rotterdam, pp. 285–289, 1995.
- [11] Bard, P.Y., Effects of surface geology on ground motion: recent results and remaining issues. *Proc. of the X Europ. Conf. On Earthquake Engineering*, Balkema: Rotterdam, pp. 305–323, 1995.
- [12] Schnabel, P.B., Lysmer, J. & Seed, H.B., *Shake a computer program for earthquake response analysis of horizontally layered sites. User's manual*, Earthquake engineering research center, University of California: Berkley, 1972.
- [13] Idriss, J. & Sun, J.I., *SHAKE91- A computer program for conducting equivalent linear seismic response analysis of horizontally layered soil deposits*. Dep. Of Civil and Environmental Engineering, University of California: Davis, 1992.
- [14] Gazzetta Ufficiale della Repubblica Italiana (eds). *Norme Tecniche per le Costruzioni*, Testo Unitario, n. 222 del 23/9/2005



Dynamic response of a large landslide during a strong earthquake

R. Meriggi & M. Del Fabbro

Department of Georesources and Territory, Udine University, Italy

Abstract

This paper reports the stability analysis results of a slope, located in the north-eastern Alps of the Friuli Venezia Giulia (Italy), subjected to an earthquake of equal magnitude to that which shook the area in 1976. The soil mass involved in the landslide was greater than 1 million m³ and caused heavy structural damage, especially in the village of Salars. The failure surface mostly develops inside the shale formation present below the detrital cover. The geotechnical properties have been measured by laboratory tests and geophysical investigations, accompanied by the monitoring of deep movements, water table variations and weather conditions. Soil investigations and displacement monitoring point out a generalised situation close to instability confirmed by the results of the pre-seismic stability analysis. Both simplified and advanced methods have been used to analyse the slope stability conditions. Dynamic slope behaviour has been analysed by means of a finite element analysis and the results have allowed the displacements, accumulated during the paroxysmic phase, to be estimated using Newmark's method; the calculated displacements have also been compared to those obtained by statistical correlations proposed by other authors. Moreover the increments of pore water pressures have been evaluated using correlations with shear stress increments along the sliding surface; these new values of pore water pressure have subsequently been used to estimate the post-seismic slope stability conditions and only a slight reduction of the safety factor was observed. This is due to high confinement pressures existing along the failure surface. The theoretical displacement accumulated in dynamic conditions has resulted in nearly twice that measured annually and may therefore cause further damage to, or the collapse of, buildings already damaged by the natural evolution of the landslide movement.

Keywords: dynamic stability analysis, safety factor, displacement.



1 Introduction

The paper reports the results of stability analyses of the slope on which the village of Salars lies, in the north-eastern Alps of Friuli Venezia Giulia (Italy) (Fig. 1). The site is located on the right bank of the Margò stream and, since 1960, has been subjected to large landslide movements involving small villages situated downstream from Salars, causing damage to and the collapse of some buildings, compelling the inhabitants to abandon them and move house. In the last forty-five years major movements have occurred in the area involved and the stability situation has progressively worsened. This study therefore aimed to evaluate slope behaviour during and immediately after a seismic event equivalent, in magnitude and intensity, to the strong earthquake which struck the area in 1976. Both simplified and advanced methods have been used to perform the dynamic analysis: the former are very useful and common in professional practice, but may lead only to a rough estimate of displacements, while the latter must be used in order to obtain more realistic permanent deformations. Besides, pore water pressures, developed in the soil mass during the seismic event, depend on the size and distribution of dynamic shear stresses, which may be calculated with a good degree of accuracy by FEM analysis.

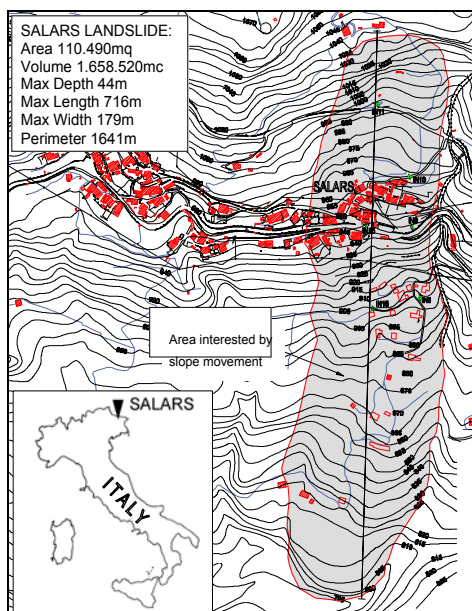


Figure 1: Location of the Salars landslide.

The lithological and geotechnical properties of the soils involved in the landslide were measured using a large number of boreholes, together with geophysical and laboratory tests; deep slope movements, groundwater table



variations and weather conditions were monitored over a period of about four years. In particular 24 boreholes were drilled up to a depth of between 35m and 75m, both core destruction drilling and continuous core borings with collecting of remoulded and undisturbed samples for the laboratory analysis. Lefranc and Lugeon permeability tests were performed inside the boreholes, and 20 inclinometers and 19 Casagrande piezometers subsequently installed. Several cross-hole, down-hole and VSP tests allowed the buried shapes inside the soil mass to be identified; six seismic refraction bases of 230m in length were also performed. The stratigraphical section based on the information obtained from the borings is shown in Fig. 2; three different lithological units are distinguishable:

- detrital and morainic cover, sometimes with erratic blocks, of a thickness varying from one to ten metres; the cover mainly originates from alteration of the underlying shale;
- very weathered grey and light-brown shale bank of variable thickness and schistose structure;
- from compact to strongly fractured siltstone. Siltstone is mainly grey-blackish, with schistose structure and calcite veins, alternating with dark grey shale layers 50cm thick.

The failure surface position and deep displacement were monitored by inclinometers from 1995 to 1999. Mean values of cumulated displacements during the observation period are between 90mm and 150mm, with the highest value, 240mm, being measured by inclinometer n. 10, located in the mid-upper part of the landslide (Fig. 2). According to the method suggested by IUGS/WGL [1], the landslide may be classified as a slow movement, with a displacement rate ranging between 22.5mm/yr and 37.5mm/yr. The evolution of displacements has not undergone much modification, even after the construction of 5 drainage wells in the second half of 1996 in an attempt to stabilize the landslide.

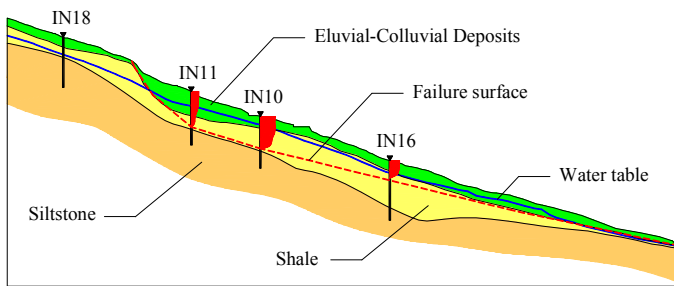


Figure 2: Stratigraphical section of the landslide soil mass.

The failure surface (Fig. 2) mainly develops inside shale and crosses the detrital cover only near the toe and the main scarp. The landslide movement is prevalently translational and develops locally at the interface with the siltstone



bedrock. The main morphologic landslide parameters are shown in Fig. 1; they involve an instable soil mass greater than 1 million cubic meters, a length greater than 700m, a mean width of 170m and a mean depth of 40m. The detrital and shale cover that forms the landslide has a wide variation of permeability, $10^{-7} \text{ m/sec} < k < 10^{-4} \text{ m/sec}$, even for soils which belong to the same lithotypes and present at similar depths. The piezometric measurements have pointed out that the water level remains almost constant even after heavy rains. Along the main landslide axis, the water table (Fig. 2) lies at a depth of about 10m below the topographic surface near the crown, becomes deeper until about 20m from the village and surfaces at the toe of the slope. Study of the monthly rainfall regime shows a remarkable difference in intensity between the rainy (April-November) and dry seasons (December-March). Mean annual rainfall is about 1510mm, with 112 rainy days (r.d.) per year and a mean rainfall intensity of 13.5mm/r.d. Laboratory tests have been done on both remoulded and undisturbed samples collected from the landslide mass and these allowed material classification, plus the measurement of peak and residual shear strength by means of direct shearbox and CU triaxial tests. Shale formation and detrital cover have similar particle size and plasticity characteristics, so that it's not possible to clearly distinguish the two lithotypes. They are mostly formed by rock fragments in a sandy-silty-clayey matrix, inactive and with low plasticity. The mean values of index properties are shown in Table 1.

Table 1: Main geotechnical properties of analysed soils.

Clay (%)	Silt (%)	Sand (%)	Gravel (%)	γ_d (kN/m ³)	γ_s (kN/m ³)
10	17	28	45	19.41	21.67
W_L (%)	W_P (%)	PI	CF (%)	AI	e
23.9	17.4	6.5	10	0.65	0.445

The peak shear strength was measured with both direct shear and CU triaxial tests, and is characterized by friction angle values ranging between $22^\circ < \phi < 35^\circ$ and effective cohesion values close to zero. The residual shear strength, measured after 5 back and forth travels in the direct shearbox, differs for the two formations present inside the landslide mass. The shear strength values used in the stability analysis are $\phi'_R = 20^\circ$ for the detrital cover and $\phi'_R = 24.5^\circ$ for the shale formation. Elastic characteristics of soils present in the landslide mass were obtained indirectly from the results of geophysical tests: their mean values are $G_0 = 178 \text{ MPa}$ and $E_0 = 481 \text{ MPa}$ for the detrital cover, while $G_0 = 1336 \text{ MPa}$ and $E_0 = 3608 \text{ MPa}$ have been used for shale. A value of $\nu = 0.35$ has been assumed for both formations. In order to analyse the behaviour of the landslide mass during a seismic event, a real earthquake record was utilised; this event represents the strong earthquake which shook the area on the 6th of May 1976, with a Richter magnitude of $M_s = 6.4$ registered by the closest seismological station (Fig. 3).



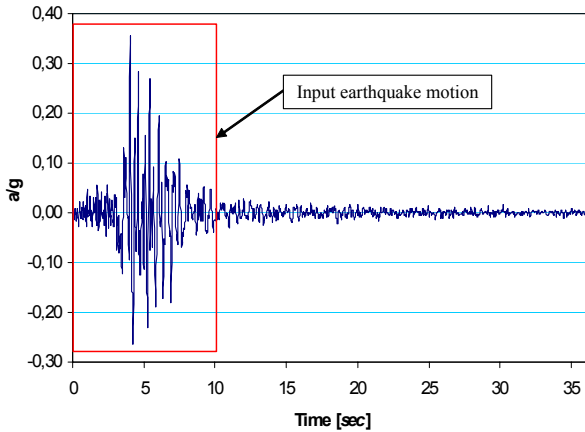


Figure 3: Design earthquake record.

The seismic wave propagated in an N-S direction. The seismic signal lasted 36.53s, with a peak acceleration of 0.357g after 4.02s. The acceleration data corresponding to the first 10 seconds have been used in the dynamic analysis (Fig. 3); after that instant the seismic action produces negligible shear stresses with respect to those obtained during the maximum earthquake record intensity. The main parameters used to represent the seismic action in a synthetic form are the Arias intensity [2] and the seismic destructiveness potential factor (Saragoni [3]), equal to $I_A=0.75\text{m/s}$ and $P_D=0.062\text{m}\cdot\text{s}$ respectively (Grimaz [4]).

2 Landslide stability analysis

The landslide stability analysis was performed in static and dynamic conditions using both limit equilibrium and finite element methods. In particular, the evaluation of landslide behaviour during the seismic event was conducted using an uncoupled dynamic method, computing the increase in pore water pressures caused by dynamic loads separately. The discretization of the complex geometry was obtained by means of an unstructured meshing to accurately model the real aspects of the slope. The failure surface does not cross the siltstone layer and was represented in the analysis as bedrock. The boundary conditions along the interface between shale and siltstone were imposed by means of null displacements in vertical and horizontal directions, presuming the absence of differential displacements between the two materials under both dynamic and static actions. A pre-seismic analysis was done to verify the fitness of FEM to represent the slope stability conditions; as deduced by the evolution of deep and superficial displacements, these conditions are close to limit equilibrium. The state of stress and strain of the slope in static conditions were computed by an FE analysis (Geostudio [5]) using a linear elastic model in terms of effective stress, assigning shear strength and stiffness values to the previously indicated modelled materials. Pore water pressures inside the slope were evaluated introducing the



water table position obtained by piezometric measurements in the model. The results of FEM analysis, stress-strain state inside the slope and shear stress acting along the failure surface, were elaborated again using an LE program to obtain a safety factor in static condition. The soil mass was divided into slices and a safety factor for each one was computed as a ratio between available shear force and the mobilized one, forces achieved by integration of stresses along the base of slices. The probabilistic analysis was performed with the Montecarlo method, assigning a mean residual friction angle of $\phi'_r=24.5^\circ$, a standard deviation of ± 0.4 to the shale material and 2000 trial runs. The calculation pointed out a mean value of safety factor $FS=1.054$ with an associated probability of failure $P_f=0.3\%$; these values seem to represent the real equilibrium condition of the slope quite well. Using the same statistical variation hypotheses for the residual friction angle ϕ_r of the shale, equilibrium limit stability analysis was also conducted with the Morgenstern-Price method and the main results, $FS=1.051$ and $P_f=0.3\%$, match those obtained by FEM very well. The comparison of the mobilized shear stresses along the sliding surface, calculated with the two methods, is shown in Figs. 4(a) and 4(b).

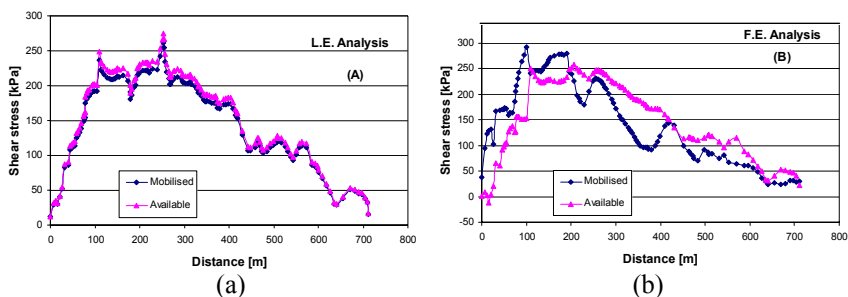


Figure 4: Shear stresses along the failure surface: (a) LE analysis, (b) FEM analysis.

Dynamic analysis was used to evaluate the stability conditions of the slope during the seismic event, permanent displacements and pore water pressure generated by cyclic shear stresses. The first step was the pre-seismic static analysis in order to evaluate stress-strain states inside the landslide mass. The next step was uncoupled advanced analysis using time-acceleration data of the first 10 seconds of the earthquake record (Fig. 3). During seismic action the soil behaviour is assumed to be non-linear elastic, assigning the initial shear modulus G_0 and damping D , and also their variation with cyclic shear strain γ . The G modulus reduction ratio was evaluated by means of the relationship proposed by Ishibashi and Zhang [6]:

$$\frac{G}{G_0} = k \cdot \left(\sigma'_m \right)^m \quad (1)$$

where the parameters k and m depend on plasticity index $PI=6.5$ and the response of cyclic shear strain γ . The mean normal effective stress, evaluated at step 0,



was assumed as $\sigma'_m=50\text{kPa}$ for detrital cover and $\sigma'_m=200\text{kPa}$ for shale. The trend of damping ratio was calculated using the equation proposed by Ishibashi and Zhang [6]:

$$D = 0.333 \frac{1 + \exp(-0.0145 \cdot PI^{1.3})}{2} \cdot \left[0.586 \cdot \left(\frac{G}{G_o} \right)^2 - 1.547 \frac{G}{G_o} + 1 \right] \quad (2)$$

The cumulated displacements during the paroxysmal phase, calculated by means of Newmark's method [7], were computed with an LE stability program (Geostudio [5]), using the total stress state and the corresponding strain, obtained by FE analysis. At first the critical acceleration value for the landslide mass was calculated automatically and then, for each temporal step, the mean acceleration which develops along the whole failure surface. The estimate of landslide permanent displacements was obtained by means of a double integration of the excess acceleration, neglecting the values that led to counterslope movements. Critical acceleration and total accumulated displacement (Fig. 5(a)) were $a_c=0.09156\text{m/s}^2$ and $S=0.052\text{m}$ respectively.

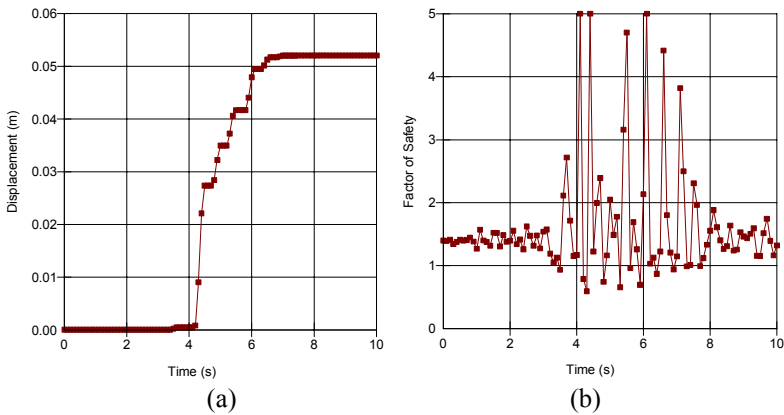


Figure 5: (a) Cumulative slope movement vs. time; (b) Safety factor vs. time.

This displacement value was compared with the one obtained by statistical correlations which use some helpful parameters to represent the intensity of seismic action, as follows:

$$S_n = (0.292 + 0.0762 I_A)^2 \quad (\text{Luzi and Pergalani [8]}) \quad (3)$$

$$\log S_{0(av)} = 1.46 \log I_A - 6.642 a_c/g + 1.546 \quad (\text{Jibson [9]}) \quad (4)$$

$$S_{0(av)} = 0.011 P_D^{0.977} (a_c/g)^{-1.338} \quad (\text{Crespellani et al. [10]}) \quad (5)$$



The displacements calculated for the slope subjected to the 1976 earthquake are summarised in Tab.2; the value determined with the equation proposed by Luzi and Pergalani [8] is the one closer to the dynamic analysis value.

Table 2: Displacements evaluated by statistical correlations.

Author	Displacement (m)
Luzi and Pergalani [8]	0.12
Jibson [9]	0.20
Crespellani et al. [10]	0.34

The safety factor (Fig. 5(b)) resulted as less than one only in the temporal interval where the acceleration values are maximum (from 3.5 seconds until 8 seconds approximately). The higher values of safety factor referring to the same temporal interval are caused by the elevated inertial forces that develop when the acceleration imposed on the soil mass by the earthquake is in the opposite direction to the movement. In the short time intervals where $FS \gg 1$, the function of displacement versus time assumes a constant value; that means no displacements have been cumulated in these periods. The mean value of safety factor associated to the whole dynamic action resulted as $FS=1.318$, higher than that calculated in static conditions. The evaluation diversity and hypothetical stability increase expressed by this factor depend on the different rheological models adopted for the two analyses: in the static case a linear elastic behaviour was modelled, while in the dynamic analysis the behaviour is non-linear and non-conservative. Post-seismic stability of a landslide depends on the increase of pore water pressures and on the decay of shear resistance, both phenomena related to the stress-strain state induced by the cyclic action. The earthquake-induced pore pressure ratio was computed by equations proposed by Coumoulos and Bouckovalas [11] and Egglezos and Bouckovalas [12]:

$$\Delta u_{max}^* = \frac{\Delta u(N)}{\sigma'_{vo}} = \frac{2}{\pi} \sin^{-1} \left\{ N_{eq}^{1/2a} \cdot \sin \left[\frac{\pi}{2} \cdot \Delta u_I^* \right] \right\} \quad (6)$$

$$\Delta u_I^* = C_1 \cdot \left(\tau_d^* \right)^{C_2} \cdot D_r^{C_3} \quad (7)$$

where $a = 0.7$ (Seed and Booker [13]). To compute the induced pore pressure ratio generated during the first cycle, Δu_I^* , constant numerical values proposed by Coumoulos and Bouckovalas [11] were used, $C_1=2.6$, $C_2=2.78$ and $C_3=-4$, for relative density $D_r=0.65$, while the values of the cyclic shear stress ratio $\tau_d^* = \tau_d / \sigma'_{vo}$ were directly obtained from the dynamic analysis results. In order to evaluate the influence on the increase of pore pressures, the number of equivalent cycles was calculated by two methods. Biondi et al. [14] have proposed the following formula:

$$\ln N_{eq} = -3,8370 + 2.67 \cdot \ln M - 0.3436 \cdot \ln a_{max} \quad (8)$$



that, for $M=6.4$ and $a_{\max}=0.357g$, leads to a value of $N_{eq}=2$. Instead the graphical correlation proposed by Seed et al. [15], based only on the earthquake magnitude, indicates a more prudent mean value of $N_{eq}=6$. The solution of (6) with the last value of N_{eq} provides pore pressures increments of less than 6kPa along the failure surface. Post-seismic stability analysis was done using the Morgenstern-Price method, changing the pore pressure increments into a water table equivalent height. In the post-seismic phase the safety factor resulted as $FS^{ps}=1.049$, a value which, together with a small probability failure increment $P_f=0.35\%$, shows that slope stability is not particularly influenced by the reduction in available shear strength in this phase. For the mean earthquake-induced pore pressure ratio $\Delta u^*_{\max}=0.0041$ computed with (6) along the failure surface, the post-seismic safety factor FS^{ps} resulted as being in good agreement with that estimated by the simplified equation proposed by Biondi & Maugeri [16] for an infinite slope, $FS^{ps}=FS(1-\Delta u^*_{\max})=1.047$. At a parity of other factors, the slight increase in pore pressure generated by seismic action depends on the normalised dynamic shear stress, $\tau^*_d=\tau_d/\sigma'_{vo}$, which is not particularly elevated due to high confinement pressures, σ'_{vo} acting along the failure surface.

3 Conclusions

The Salars landslide is caused by the high pore pressures induced by an aquifer fed by underground flows coming from the upper zone of the slope, characterized by high permeability and fissured rock mass. Even after heavy rains the piezometric level remains almost unaltered and this may explain the constant increasing trend of displacements measured by inclinometers. The two analyses performed with different methods have indicated very similar safety factors: $FS=1.051$ for LE method and $FS=1.054$ for FE method, both associated to the same probability of failure (0.3%). The theoretic displacement accumulated by the soil mass, assimilated to a rigid block and subjected to a seismic event equivalent to the strong earthquake of 1976, resulted as about 5cm, nearly twice that measured annually in static condition. Such displacement may cause further damage to or collapse of the buildings already damaged by the landslide geostatic evolution. The displacements obtained by several authors' statistical correlations resulted as higher than that calculated analytically. The slight increase in pore pressure generated by dynamic action doesn't substantially alter the slope stability conditions during the post-seismic phase; in fact its safety factor is $FS^{ps}=1.049$, related to a slight increase in probability of failure, $P_f=0.35\%$. It can be concluded that the stability conditions of the slope don't seem to reduce greatly on the occasion of high intensity seismic events, because of both high soil mass inertia and high confinement pressures acting on the failure surface, which prevent the development of high pore pressures.

References

- [1] IUGS/WGL, A suggested method for describing the rate of movement of a landslide. *IAEG Bull.*, 52,75-78, 1995.



- [2] Arias A., A measure of earthquake intensity. In *"Seismic design for nuclear power plants"*, MIT Press, Cambridge, Massachusetts, 438-468, 1970.
- [3] Saragoni G.R., Response spectra and earthquake destructiveness. *Proc. IV U.S. National Conference on Earthquake Engineering*, Palm Springs, Florida. EERI, 35-43, 1990.
- [4] Grimaz S., Caratterizzazione dei suoli di fondazione ai fini della definizione dell'azione sismica di progetto. ENAIP, Udine. Italy, 2005.
- [5] Geostudio, Software Manual. GEOSLOPE Int., Calgary, Canada, 2004.
- [6] Ishibashi I., Zhang X., Unified dynamic shear moduli and damping ratios of sand and clay. *Soils and Foundations*, 33(1), 182-191, 1993.
- [7] Newmark N., *Effects of earthquakes on dams and embankments*. Géotechnique, 15(2), 139-160, 1965.
- [8] Luzi L., Pergalani F., Analisi di stabilità di situazioni tipo connesse con fenomeni franosi in condizioni statiche e dinamiche di un'area campione. *Ingegneria Sismica*, anno XI, n.2, 10-32. 1996.
- [9] Jibson R., Predicting earthquake-induced landslide displacements using Newmark's sliding block analysis. *Transportation Research Record 1411*, Transportation Research Board, Washington D.C., pp.9-17, 1994.
- [10] Crespellani T., Madiati C., Vannucchi G., Earthquake destructiveness potential factor and slope stability. *Géotechnique*, 48(3), 411-419, 1998.
- [11] Coumoulos H., Bouckovalas G.D., Analytical relationships for earthquake-induced pore pressure in sands. *Research Report*, National Technical University of Athens, 1996.
- [12] Egglezos D.N., Bouckovalas G.D., Analytical relationships for earthquake-induced pore pressure in sand, clay and silt. *Proc. of the 11th European Conference on Earthquake Engineering*, Paris, 1998.
- [13] Seed H.B., Booker J.R., Stabilization of potentially liquefiable sand deposits using gravel drains. *Journal of Geotechnical Engineering*, ASCE, 103(7), 757-768, 1977.
- [14] Biondi G., Cascone E., Maugeri M., Number of uniform stress cycles equivalent to seismic loading. *Proc. 11th Int. Conf. on Soil Dyn.& Earth. Eng. and 3rd Int. Conf. on Earth. Geotec. Eng.*, Berkeley, California, Vol.2, 705-712, 2004.
- [15] Seed H.B., Idriss I.M., Makdisi F., Banerjee N., Representation of irregular stress time histories by equivalent uniform stress series in liquefaction analysis. *Report n. EERC 75-29*, Earthquake Engineering Research Center, University of California, Berkeley, 1975.
- [16] Biondi G. & Maugeri M., A modified Newmark type-analysis according to EC-8 requirements for seismic stability analysis of natural slopes. *Proc. of the Int. Workshop "Geotechnical Evaluation and Application of the Seismic Eurocode EC8"*, Athens, 2006.



Liquefaction potential evaluation for a site

S. Mittal¹ & M. K. Gupta²

¹*Department of Civil Engineering, IIT Roorkee, India*

²*Department of Earthquake Engineering, IIT Roorkee, India*

Abstract

Many previous works have attempted to predict the occurrence of liquefaction in the field. A comprehensive work has been done on small samples under triaxial and simple shear conditions. A few investigations have been done on large size samples and also on vibration tables. In the present study, the large size sample tests have been conducted in field and the prediction of liquefaction occurrence has been studied. To study the effect of side wall on the test specimen in large size samples, some tests have been conducted in a field pit also. A case study has also been discussed that how the bridge survived seismic shocks at the time of actual occurrence of earthquake the foundation of which was designed based on the suggested measures discussed in this paper.

Keywords: liquefaction, progressive failure, potential evaluation, embankment design, vibration table, shake table.

1 Introduction

If soil is saturated at the instant of collapse, the weight of soil particles is temporarily transferred from the points of contact with their neighbours into the water (Terzaghi and Peck [13]). Dynamic triaxial and simple shear tests on small sample were conducted in USA, Japan and U.K. (Castro [2]), while large size sample tests (vibration table studies) were carried out in India and USSR (Gupta and Prakash [7]). Cyclic loading triaxial compression tests on small sample and also vibration table tests on large size sample have thrown considerable light on the factors inducing liquefaction of saturated sands and both provide methods to predict liquefaction potential which uses data obtained in these tests (Gupta [5]). But the results are affected by the method of test and test equipment used and also on method of analysis. Hence no uniform agreement could be achieved till today. Seed and Idriss [11, 12] have also



performed considerable research on the phenomenon of liquefaction with cyclic loading triaxial compression tests and simple shear tests (Seed and Lee [9], Lee and Seed [10]). When the strains become excessive (generally about 20%) the sample has been considered to have failed. It has been reported that these tests suffer from draw back of redistribution of void ratio during the test and also simulation of field behaviour (Castro and Poulos [3], Gupta and Prakash [7]). Also one of the important aspects of liquefaction is progressive failure (Gupta [5]). This is not simulated in tests under small sample triaxial or simple shear tests. Also a comprehensive study on large size sample on a horizontal vibration table to predict possibility and extent of liquefaction has been done at Indian Institute of Technology (I.I.T.), Roorkee, India during past 4 decades (Mittal [8], Agrawal [1]). The method of predicting liquefaction potential developed by Gupta [5] uses the test data obtained on large size sample under horizontal sinusoidal vibrations and takes care of progressive nature of liquefaction, while all other available methods do not consider this. Although the vibration table tests in a tank suffer from the draw back that there is reflection of waves from sides of the tank, yet these types of tests have predicted the field behaviour with reasonable accuracy. The present study was carried out to explore the confidence level in these studies by comparing laboratory tests on soil sample deposited in tank and with other tests carried out on natural deposit in field. This study indicates that the tests conducted in tank in laboratory represent the field conditions more realistically and hence may be considered as standard method of determination of liquefaction potential at sites.

1.1 Laboratory tests on large size sample

The studies of liquefaction potential assessment were extended by using dead weight surcharge (Gupta [5]) on vibration table. The test set up consists of a horizontal shake table which can be excited sinusoidally at different frequencies up to a maximum acceleration of 2 g. A steel tank of size 1050 mm x 600 mm x 400 mm (LxBxH) is mounted on the shake table (Fig. 1). The vibration table was designed in such a manner that it could take any acceleration level which is possible in any earthquake. For preparing the saturated sand deposit in it, the tank was filled with a known quantity of water (140 kg.) and a known weight of dry sand (300 kg) was poured from a constant height with the help of a funnel (Fig. 2). The excess water which was overlying the soil sample in the tank was then removed by siphoning and weighed to compute the initial relative density of sand deposit. This deposit was vibrated at a desired frequency and required number of revolutions to obtain desired relative density (Fig. 3). For tests under initial surcharge condition to represent a sample at depth, the surcharge was applied with pre -cast concrete blocks attached rigidity to a steel plate (Fig. 4). The precast concrete blocks were used in the present study as the earlier studies conducted on vibration table with air pressure surcharge (Gupta and Prakash [7]) had many limitations (the air pressure surcharge simulated only initial stress conditions in field). The laboratory test on the sample is considered to represent soil conditions in field at one particular depth. The tests were performed with varying amounts of dead weight surcharges. This method has been seen to be a



realistic approach. The points in favour of this method are as follows: (1) the sample is prepared and consolidated under anisotropic conditions; (2) deformation occurs under plane strain conditions; (3) visually observed behavior of loose sands on vibration table is similar to what has been observed during an earthquake; (4) it is possible to trace actual pore water pressure distribution in a large mass of saturated sand during liquefaction; and (5) progressive nature of development of liquefaction during an earthquake is taken care of in the analysis which uses the vibration table test data.



Figure 1: A view of shake table.



Figure 2: Rainfall technique of sample preparation.

The method suffers from the following limitations: (1) the shear stress can not be rigorously controlled in these tests as could be done in small sample in triaxial or simple shear apparatus; (2) uniform shear stress is not developed throughout the sand sample; and (3) grouping earthquake into suitable number of cycles requires an engineering judgment. To gain more confidence in use of steel tank for determination of liquefaction potential, a study has been done in a test pit also made in the natural soil. The details of this study and the comparison between test pit data and steel tank data are discussed in the subsequent paragraph.

2 Tests conducted

2.1 Soil used and its properties

In all the tests, the soil used was locally available Solani river sand. The grain size analysis of sand is shown in Fig. 5. Mean grain size of the sand is 0.15 mm



and its uniformity coefficient is 1.9. According to Indian Standard Classification, the sand belongs to SP Group. The other properties of sand are shown below in Table 1.

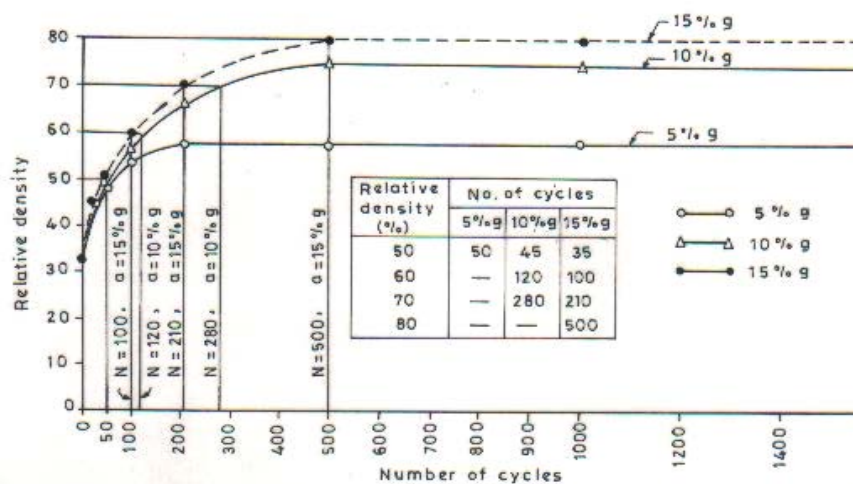


Figure 3: Results showing relative density (D_{ri}) versus no. of cycles.

Table 1: Properties of sand used.

Specific gravity G_s	: 2.59
Maximum void ratio (e_{max})	: 0.84
Minimum void ratio (e_{min})	: 0.44

2.2 Vibration table

The test set up was same as used by Gupta [5] and also discussed in above lines. The tests were performed on the artificially saturated sand samples prepared as described above to study the effect of: (a) sample size (b) acceleration (c) initial relative density and (d) over burden pressure. To study the effect of sample size, the tests were conducted in 3 different sizes of tanks keeping other parameters constant. The tank sizes adopted were 1050 x 600 x 400 mm³, 800 x 600 x 400 mm³ and 600 x 600 x 400 mm³. It was found that higher tank size gives maximum increase in pore pressure, and the difference in rise in pore pressure between tanks of 1050 x 600 x 400 mm³ and 600 x 600 x 400 mm³ is marginal (Fig. 6). Hence, the tank of size as 1050 x 600 x 400 mm³ may be considered as reasonable size for liquefaction study. Figure 7 shows the plot between pore pressure and acceleration for different initial relative densities of the sample and a particular surcharge condition of 2.3 kN/m². One of the curves shows

increasing trend of pore pressure with the increase in acceleration up to a certain value of acceleration only. Then it starts decreasing. It means that at this level of pore pressure, the soil mass had completely liquefied. It is also noticed in the tests that the pore pressure is increased with increase in acceleration and the complete liquefaction did not occur up to an acceleration of 50%. The sand is seen to have liquefied during upward flow of water. Thus it can be stated that complete liquefaction occurred first at some depth below the surface and upper layers were liquefied subsequently. Figure 8 shows the plot between pore pressure and relative density with acceleration as 20% and 50% for surcharge as 2.3 kN/m^2 . The pore pressure line indicating the complete liquefaction is also drawn. It can be observed from this figure that with increase in initial relative density, the increment trend in excess pore pressure decreases and there is no rise in pore pressure after 90% relative density. In case of higher accelerations also, no increase in pore water pressure is observed beyond 90% sample density. A few tests were also tried for the sample density as 92%. At this density, the negative pore pressures were observed at acceleration of 60%g. Thus it can be stated that with the increase in intensity of acceleration and at higher relative densities, the dilation of soil takes place. It can be possible because the sand particles are supposed to be in tension, and also the interlocking of particles is broken and the expansion of soil mass in volume takes place. This phenomenon was observed in zero surcharge condition also. Therefore, in such circumstances, it is obvious that liquefaction will not occur and there will be increase in apparent effective stress. Castro [2] also reported that during the test on medium dense to dense sands under cyclic loading conditions in triaxial tests, dilation of sample is observed. He further concluded that if the sand is at relative density of more than 50% it may not liquefy (Green and Ferguson [4]).



Figure 4: Method of surcharge.

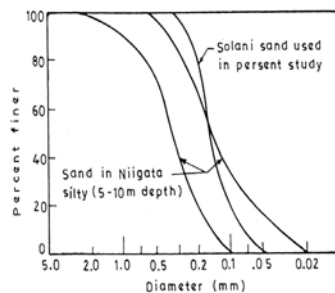


Figure 5: Grain size distribution curve application.



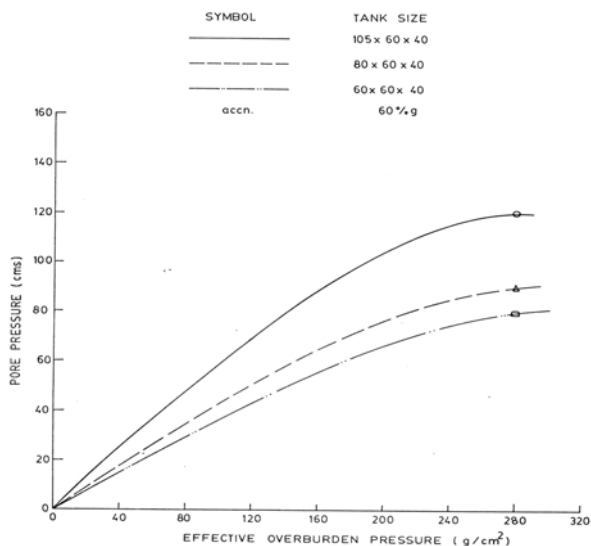


Figure 6: Effect of tank size.

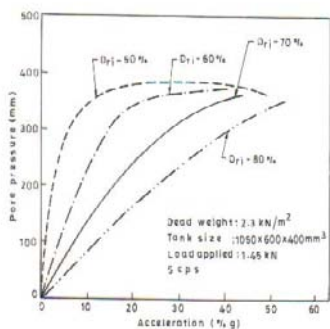


Figure 7: Pore pressure versus relative density.

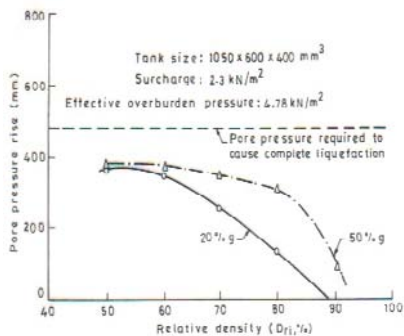


Figure 8: Pore pressure versus acceleration.

The tests were conducted under three different overburden pressure, e.g. 2.3 kN/m², 17 kN/m² and 25 kN/m². Figure 9 shows the plot between excess



pore pressure and effective overburden pressure for acceleration of 50%g for different initial relative densities. Line AB shows complete liquefaction. For any point on this line, excess pore water pressure is equal to initial effective overburden pressure. Zone of complete liquefaction is also shown. From this figure, it is evident that pore pressure increases with increase in surcharge. But it can be anticipated that with further increase in overburden pressure, there will be no further rise in pore pressure. At this stage, the inter granular stresses are large to provide enough resistance against shear of sand particles and causing less pore pressure increase.

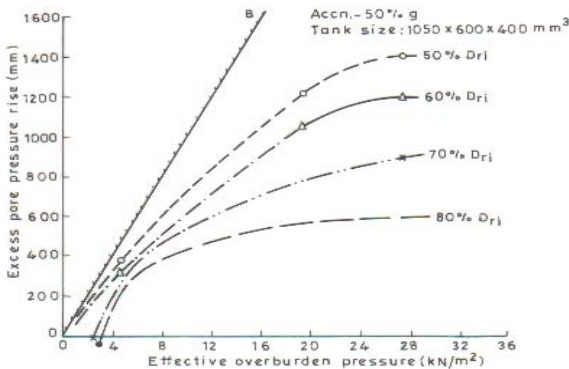


Figure 9: Pore pressure versus overburden pressure.

3 Field pit tests

The test consists of a pit in natural ground in which the sand sample is prepared. A mechanism to impart vibration to the deposit, an acceleration recording device and a pore water pressure measuring device were also provided. The test pit was 700 mm long, 550 mm wide and 400 mm deep (Fig. 10). The pit is made in the field and all the side walls are of the local soil. In order to make the pit impervious a thin coating of clay is done on all the four sides as well as the bottom. Further a very thin polythene membrane is stuck over the clay coating to check the mixing of clay with sand and to further check any seepage from the pit. The vibrations were imparted by exciting a concrete block with the help of an oscillator and a motor mounted over it (Fig. 11). The block is made of concrete at one side of pit and top of the block is at ground level. The size of block is 700 mm x 550 mm x 400 mm deep. The oscillator consists of shaft in pair, geared together and driven by a motor. Equal eccentric masses are mounted on the shaft at equal eccentricity (e) from the center of the shaft. The gears are so arranged that the shafts rotate in opposite directions. When the masses are in same horizontal plane, the horizontal component of centrifugal force cancels out, and the oscillator produces pure vertical sinusoidal vibrations. On the other hand



if they are in same vertical plane, pure horizontal sinusoidal vibrations are produced. For variation of frequency in the test a D.C. motor was used in these investigations. For producing desired acceleration, the eccentricity is kept fixed and speed of motor is increased till the desired acceleration is achieved. The accelerations of 5%, 10%, and 30%g were achieved at frequency of 3 cps, 5 cps and 10 cps respectively. The accelerations are recorded with the help of an acceleration pick up. The signal is amplified through a universal amplifier and recorded on pen recording oscillograph. Increase in pore pressure during vibration is measured and recorded. The measurement is done with the help of a simple glass tube piezometer placed in the pit through a rubber tube 3 mm in diameter. At the end of the tube a porous stone is fixed to check the choking of tube with sand. There is a time lag in recording the increase in pore pressure with the generation of pore pressure. Thus maximum pore pressure recorded in the simple glass tube piezometer involves an error. The values recorded in this method are corrected as suggested by Gupta [5].

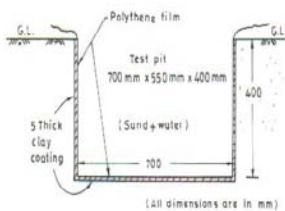


Figure 10: Sectional layout of test pit.

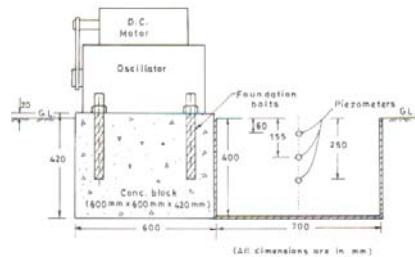


Figure 11: Test set up and vibration producing device.

4 Case study: design of road embankment

Liquefaction possibility was determined for bridge site using the laboratory and field pit test method described in this paper. The vibration table tests were conducted on soil sample obtained from the site. The site was located in Assam, India.

The study indicates that during an expected earthquake the soil layers in about 15 m depth would liquefy (Fig. 12) and a dense effective over burden pressure of about 5 t/m² may make the deposit safe against liquefaction (Fig. 13). Based on this information, the guide bund and approach road embankments were designed with suitable beams. Fig. 14 shows a typical section of the embankment. During the earthquake of Aug. 6, 1988 (magnitude = 6.8 with duration of 2 minutes), the road embankment and guide bunds designed by proposed technology performed satisfactorily.

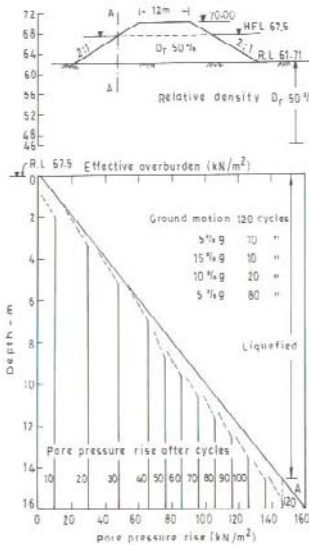


Figure 12: Liquefaction analysis of soil deposit.

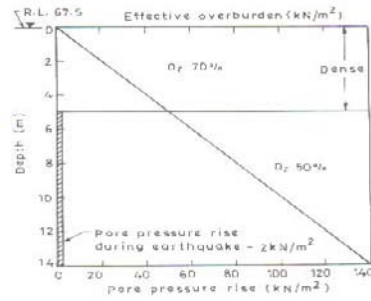


Figure 13: Liquefaction analysis of soil deposit with 5 m dense overburden.

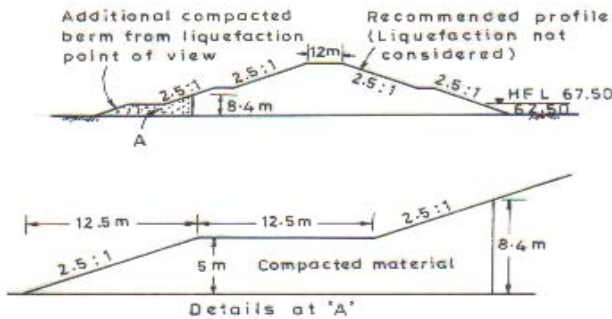


Figure 14: Typical section of approach road.

5 Conclusions

Based on the above study, following conclusions are drawn. (1) The size of tank (1050 mm x 600 mm x 400 mm) for evaluation of liquefaction potential from large sample tests gives maximum increase in pore pressure hence it can be recommended as a reasonable size of tank. (2) With increase in relative density, the increment in pore pressure decreases under vibration and chances of liquefaction are reduced. The sand may not liquefy if it is at relative density of more than 90%. In such circumstances, there is the reduction in shear strength



under vibration in the sand particles on account of reduction in friction between sand particles. (3) Dead weight overburden pressure affects the liquefaction behaviour in a characteristic manner. The sand has been observed to have liquefied at some acceleration with increasing overburden pressure initially till a certain overburden pressure is reached. However, with further increase in overburden pressure beyond this value, the increase in pore pressure decreases and chances of liquefaction are reduced. (4) The dead weight surcharge device as adopted in the present investigation can be taken to represent the field conditions with sufficient degree of confidence. (5) There is a threshold value of acceleration below which the sand remains stable and beginning of pore water pressure starts only at acceleration larger than this. (6) The method proposed in the present study can be used to evaluate liquefaction potential at any site.

References

- [1] Agrawal, S.K. (1990), "Large size sample test in field for study of liquefaction of sands", *ME Dissertation*, University of Roorkee, India.
- [2] Castro, G. (1975), "Liquefaction and cyclic mobility of saturated sands", *ASCE, Vol. 100, GT6*, June 1975, pp.551-569.
- [3] Castro, G. & Poulos, S.J. (1977), "Factors affecting liquefaction and cyclic mobility" *ASCE, Vol. 103, GT* pp 501-516
- [4] Green P.A. and Ferguson, S. (1971), "On liquefaction phenomenon by Prof. A. Casagrande, Report of Lecture", *Geotechnique (London), Vol. XXI, No.3*, pp. 197-202.
- [5] Gupta, M.K. (1977), "Liquefaction off sands during earthquakes", *PhD Thesis*, University of Roorkee, India.
- [6] Gupta, M.K. and Prakash, S. (1978), "Investigations on liquefaction of sands", *Indian Geotechnical journal, Vol. 10, No. 4*, pp.332-347
- [7] Gupta, M.K. & Prakash, S. (1980), "Investigations on Liquefaction of sands", *Journal of Indian Geotechnical Society, Vol. 10, No. 4* 2.
- [8] Mittal, S. (1988), "Vibration table studies for prediction of liquefaction a critical study", *ME Dissertation*, University of Roorkee, Roorkee, India.
- [9] Seed, H.B. and Lee, K.L. (1966), "Liquefaction of saturated sands during cyclic loading", *Journal of Soil mechanics and Foundation division, ASCE, Vol. 92, No. SM1*, pp 47-70
- [10] Lee, K.L. and Seed, H.B. (1967), "Cyclic stress condition causing liquefaction of sand", *Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 93, No.SM3*, pp.83-108.
- [11] Seed, H.B. and Idriss, I.M. (1967), "Analysis of soil liquefaction, Niigata Earthquake", *Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 93, No.SM3*, pp.83-108.
- [12] Seed, H.B. and Idriss, I.M. (1970), "*A simplified procedure for evaluating soil liquefaction potential*", University of California, Berkeley, Earthquake Engg. Research Centre, Report No.EERC 70-9
- [13] Terzaghi, K. and Peck, R.B. (1948), "*Soil mechanics in engineering practice*", John Wiley and Sons Inc, New York



Section 7

Seismic behaviour and vulnerability

This page intentionally left blank

Seismic risk assessment of the Ignalina NPP refuelling machine

R. Bausys¹, G. Dundulis², R. Kacianauskas¹, D. Markauskas¹,
S. Sliupa³, E. Stupak¹ & S. Rimkevicius²

¹*Vilnius Gediminas Technical University, Lithuania*

²*Lithuanian Energy Institute, Lithuania*

³*Institute of Geology & Geography, Lithuania*

Abstract

The Ignalina NPP operates a RBMK type reactor. An important characteristic of the RBMK reactor is its online refuelling. The refuelling is carried out using the refuelling machine at a time of reactor operation. The refuelling machine is situated in the hall located above the reactor. The refuelling machine is supported and moved by a girder crane. The dropping down of the refuelling machine bears high risk to its integrity and reactor core functioning. One of risk scenarios considers impact of an earthquake on the plant. As a part of it, it is essentially important to carry out a global seismic assessment of the refuelling machine.

The seismic assessment of the support of refuelling machine is presented in this paper. The response of the reactor building to an earthquake was considered in the seismic analysis. The recently updated free field response spectra for the Ignalina NPP site were applied. The 3D thin-walled FE model of the reactor building was used for analysis. The 5% damping was applied in calculation of the secondary response spectra of the refuelling machine. The response spectra of the acceleration and displacement of the supports of the refuelling machine were examined. The modelling was run for three different positions of the refuelling machine, i.e. at an initial position (waiting mode), at a position of unloading fuel assemblies and at a position of transportation.

Keywords: nuclear power plant, seismic analysis, finite element method, in-structure spectra.



1 Introduction

The Ignalina Nuclear Power Plant operates two graphite moderated, boiling water, multichannel RBMK-1500 reactors. This is the most advanced version of the RBMK design series (actually the only two of this type that were built). “RBMK” is a Russian acronym for “Channelized Large Power Reactor” [1]. It differs from the other reactor types. The refuelling of the RBMK type reactors is carried out under normal operation and nominal reactor power. Refuelling at full reactor power is accomplished by means of the refuelling machine. The refuelling machine performs operational and safety-related tasks. The safety tasks are as follows:

- unloading fuel assemblies with damaged cladding;
- on –load refuelling of channels with a leaking upper seal;
- replacing fuel assemblies with process plugs in channels with flow disturbances.

The refuelling machine is supported and moved by a girder crane. The refuelling machine schematically is presented in Figure 1.

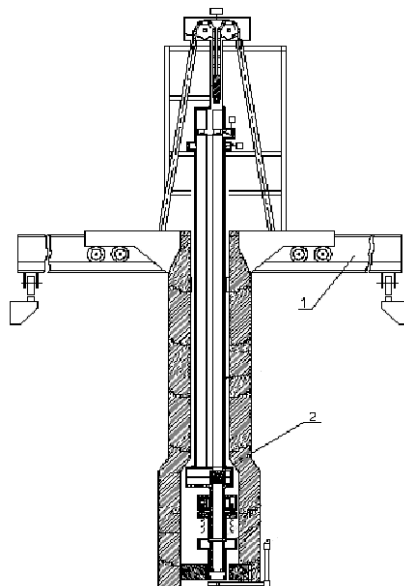


Figure 1: General view of refuelling machine, 1 – crane, 2 – refuelling machine.

The refuelling machine is located in the reactor hall. The refuelling machine service areas are as follows:

- a. the refuelling machine storage area;
- b. a practice and a calibration area (this area is used for adjustment and testing of the refuelling machine);



- c. the spent fuel reception area;
- d. a repair area for replacement of damaged fuel caskets.

During the earthquake, the refuelling machine may be destabilised and drop down causing damage to the refuelling machine and reactor core. The global seismic evaluation of the refuelling machine is important in assessment of the reactor safety. The safe operation of the refuelling machine is important not only during the reactor operation, but also when it is stopped. The unloading of the fuel from shutdown reactor takes several years.

The Ignalina NPP is located in the western part of the East-European platform, at the transition between the two large-scale structural features, i.e. the Baltic depression and the Mazury-Belarus high [2–4]. A number of tectonic faults were identified by detailed geological-geophysical mapping in the Ignalina NPP area that may potentially induce the seismic event.

The seismic dynamic stability analysis of the Ignalina NPP building is performed using the BRIGADE/Plus code [5] in the present study. The response spectra of the support of the refuelling machine were calculated separately for the refuelling machine storage area, spent fuel reception area and transportation area. The design basis earthquake of the Ignalina NPP, the modelling of the reactor building and the modelling of the refuelling machine are described in three chapters of the paper. In the next chapter the results of the seismic assessment of the refuelling machine are discussed. The conclusions are made in the final chapter based on the performed analysis.

2 Design basis earthquake of INPP

The Baltic region is considered as an aseismic or of very low seismic activity. However, available data indicate that rather strong earthquakes took place in a past. The oldest historical record dates back 1303, when the strong earthquake shook Prussia (Kaliningrad District) and destroyed most of the timber houses. The seismic activity is distributed unevenly in the Baltic region. The maximum activity is confined to the central Latvia characterized by most intense faulting of the earth's crust cored by the largest-scale Liepaja-Saldus fault zone crossing the country from the east to the west [2, 3].

The closest to Ignalina NPP seismic event took place in Daugavpils area 35 km to the North-East of the Ignalina NPP in 1908.12.29. The intensity is $I_0=6-7$ (noise resembling gun-shot, apertures on the surface 3-4 inches wide, and fractures in building walls), estimated $M_L=4.6$, the hypocentral depth is of 10 km. The epicentre is confined to the fault trending North West-South East. Furthermore, this event is related to the central part of the recent uplift attaining 4 mm/a, which is the highest value registered in the Baltic region [6]. Ignalina NPP is located on the South - West flank of this neotectonic uplift. A number of the different-scale tectonic lineaments were identified in the Ignalina NPP area. A few faults are essentially distinct in the sedimentary cover and underlying crystalline basement. Furthermore, these major tectonic features show recent activity of the horizontal and vertical movements [6]. Therefore, the potential



near-field seismic events should be considered in the seismic risk assessment of the plant.

The Design Basis Earthquake was recently updated for the Ignalina NPP site [2] following the IAEA recommendations [7] and specifically the USSR Standards (PN AE-G-5-006-87). In present analysis the Design Basis Earthquake of the Ignalina NPP was assumed as strong as $I_0 = 7.5$ Hz. The free-field response spectra of the Ignalina NPP were derived using the deterministic approach [2], the peak ground acceleration was calculated as high as 166 cm/s^2 . These spectra were used as the input seismic load in the seismic analysis of the reactor building.

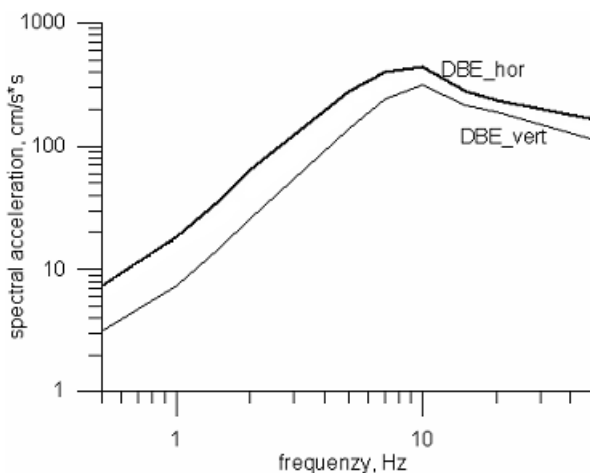


Figure 2: Free-field response spectra of Ignalina NPP Design Basis Earthquake (5% damping, horizontal and vertical motion components).

3 Ignalina NPP building model

The Ignalina NPP unit 2 consists of five buildings [1]. They are located close to each other, but have separate foundations. Therefore these buildings are considered separately in the seismic analysis. The reactor building containing the refuelling machine was selected for structural seismic analysis. The cross-section of the reactor building with main components is presented in Figure 3.

The reactor building contains an RBMK-1500 reactor (pos. 1) with a main circulation circuit (pos. 3, 4, and 5). The accident localisation, in which the main circulation circuit is located, is adjacent to the reactor core. The hall above the reactor is a large workspace housing the refuelling machine (pos. 2). The spent-fuel storage pond is situated in an adjacent hall, but separated from the reactor hall. The reactor compartment consists of a rectilinear structure, the horizontal cross-section of which is $90 \text{ m} \times 90 \text{ m}$ and a height of about 63 m.



The FE model of the Ignalina NPP reactor building was created using the Brigade/Plus pre-processor [5]. Element type S4R [8] is used in the model of the reactor building walls and slabs. Element type S4R is a fully integrated, general-purpose, finite-membrane-strain shell element.

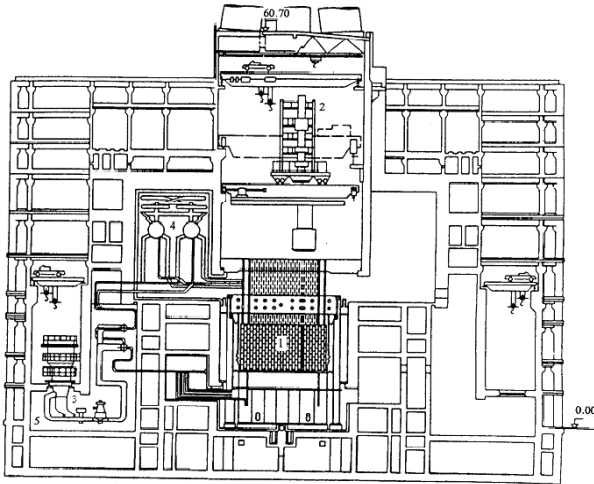


Figure 3: Cross-section of the reactor building with main components: 1 - reactor, 2 - refuelling machine, 3 - main circulation pump, 4 - separator drum, 5 - MCP pipelines.

The monolithic reinforced concrete construction and prefabricated parts of the main compartments of reactor building were included in the model. The prefabricated parts of the auxiliary compartments of reactor building were removed in the model. The following main equipment located in the reactor building and prefabricated concrete parts of the auxiliary compartments of the reactor building represent concentrated masses in the FE model of Ignalina NPP building (see Figure 3):

- mass of the reactor (pos. 1);
- mass of Main Circulation Pumps (pos. 3);
- mass of Drum Separators (DS) and piping located in the DS room (pos. 4);
- mass of the water in the condensing pools;
- mass of the removed prefabricated reinforced concrete structures of building.

The FE model of the Ignalina NPP reactor building is presented in Figure 4. This is a fixed-based model for dynamic analysis.

The masses of the removed prefabricated reinforced concrete wall structures are applied in a form of consistence masses, while masses of equipment are modelled as lumped masses and associated rotational inertia masses.



The concrete properties used for the seismic analysis were obtained from building norms [9]. The value of Young's modulus of reinforced concrete is increased by factor 1.2 due to dynamic effects.

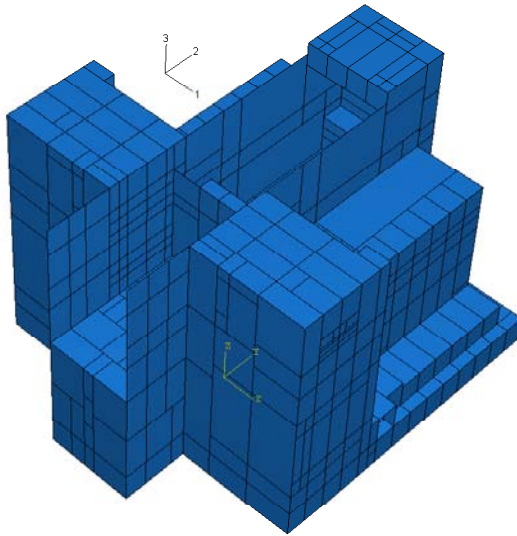


Figure 4: Geometrical model of the Ignalina NPP building.

4 Modelling of the refuelling machine

Refuelling machine is considered as dynamic subsystem, structure of which should be considered independently on the structure of the building. This model assumes uncoupled dynamic behaviour of machine and the building. In framework of this assumption, the mass of the machine is considered as an external mass affecting dynamic behaviour of the building structure, while evaluation of the seismic loads within machine structure would be considered as a secondary problem.

The working area of the refuelling machine, locations of it in the model and location of in-structure points are presented in Figure 5. Mass of the refuelling machine including mass of the moving crane is 450 000 kg [10]. It is modelled as lumped mass. Vertically, the mass is located at the level $z = 36.0$ m of the moving crane (Figure 5(a)). Horizontally, crane moves in the South–North direction. The working area is restricted by position $y = 42.0$ m and by position $y = 90.0$ above the reactor hall by baseline walls (Figure 5(b)). The lumped mass is attached in the centre of the crane beam and defined by the coordinate $x = 0$ in the FE model (24/2, Figure 5(a) and (b)).

The seismic assessment of refuelling machine is carried out in refuelling machine storage area, spent fuel reception area and transportation area. Therefore three positions of the refuelling machine were fixed for modelling purposes as following (Figure 5(b)):



- at storage position ($y = 54$ m);
- at spent fuel reception position ($y = 78$ m);
- at transportation position - selected position between storage and spent fuel reception areas ($y = 66$ m).

Assessment of the seismic risk is focussed to the behaviour of the crane rail. The accelerations of the rail serve the base for evaluation of dynamic loads of the machine structure, while implements are important for evaluation of integrity and maintenance of the building–crane–machine system. The corresponding horizontal (East–West) in-structure response spectra were of key interest.

The location coordinates of the crane rail supports on the wall structure are $z = 36$ m and $x = \pm 12.0$ m (Figure 5(a) and b)). Four characteristic points along the machine working area on the wall structure are chosen for modelling purposes. Location coordinates of the analysed points 1, 2, 3, and 4 are respectively $y_1 = y-6$, $y_2 = y-3$, $y_3 = y+3$ and $y_4 = y+6$ (Figure 5(b)), where y is the coordinate of the refuelling machine position.

The mass of the refuelling machine was added in the discrete rigid point that is located in the mass centre of this machine and was connected with the wall of the reactor compartment by rigid constraints. Constraints are attached to the wall in a form of the line segment at the level of the crane rail $z = 36$ m. The segment length is taken to be with of the crane vehicle and is equal to 12.0 m. The graphic illustration of the model is presented in Figure 6.

5 Results and discussion

The analysis was performed in the frequency domain and the floor response spectra at the supports of refuelling machine were determined. Vertical and horizontal (East–West direction) floor response spectra were calculated using free-field ground response spectra. The horizontal floor response spectra at two positions (storage and spent fuel reception) of refuelling machine are presented in Figure 7. The highest acceleration values are obtained for 5 Hz frequency equalling 3.1g. The influence of the position of refuelling machine on the maximum acceleration is illustrated in Figure 8. The highest acceleration was identified at the storage position ($y=48$ m) of refuelling machine.

The horizontal (East–West) displacements at two positions of refuelling machine (storage and spent fuel reception) are presented in Figure 9. The maximum value of the displacement was defined for 5 Hz and equals to 0.037 m. The influence of the position of refuelling machine on the displacement was evaluated. The maximum displacement of supports of refuelling machine was obtained for the storage position ($y=48$ m, Figure 10). The differences of the displacement of supporting points of the girder crane of refuelling machine reaches 0.074 m ($2 \cdot 0.037$). The distance between girders of crane is 22.16 m (see Figure 5(a)). The possibility of dropping of the refuelling machine is small in case of the seismic event.



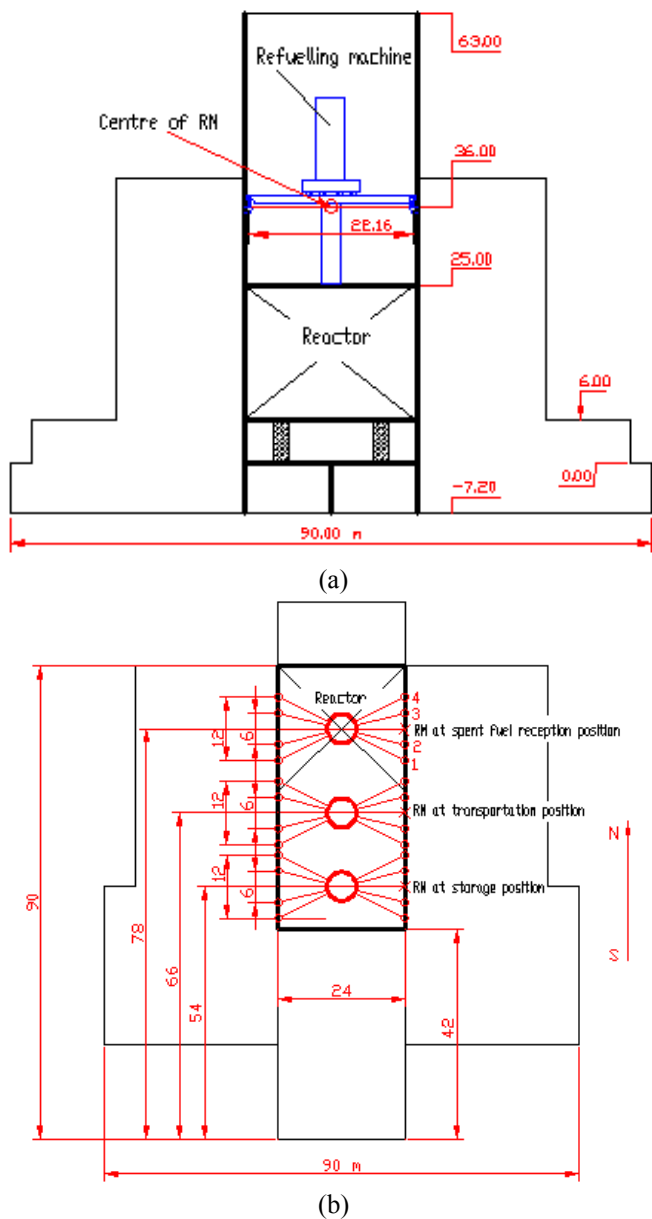


Figure 5: Schematic illustration of the reactor building with RM: (a) cross-section of the reactor building through axis of reactor, (b) layout of the building above of reactor at level 36 m.



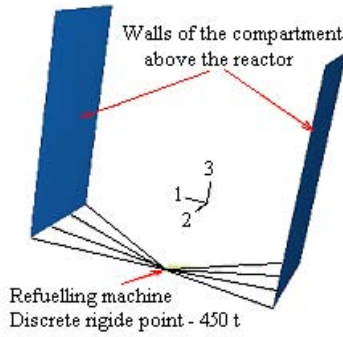


Figure 6: Modelling of mass of the refuelling machine.

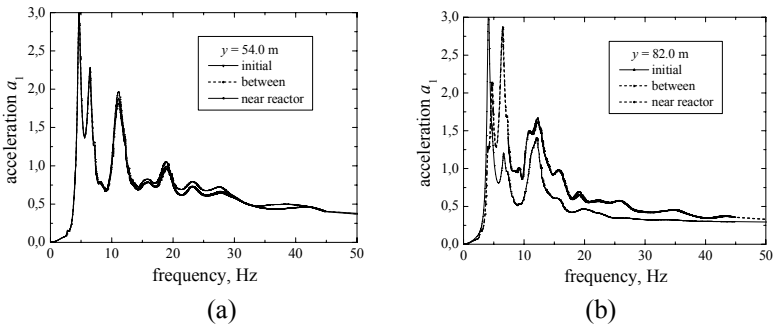


Figure 7: Horizontal acceleration floor response spectra at refuelling machine support: (a) at storage position, (b) at spent fuel reception position.

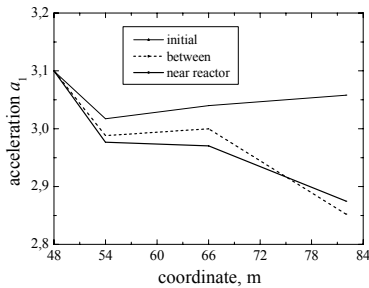


Figure 8: Variation of the horizontal acceleration floor response spectra with refuelling machine support position.



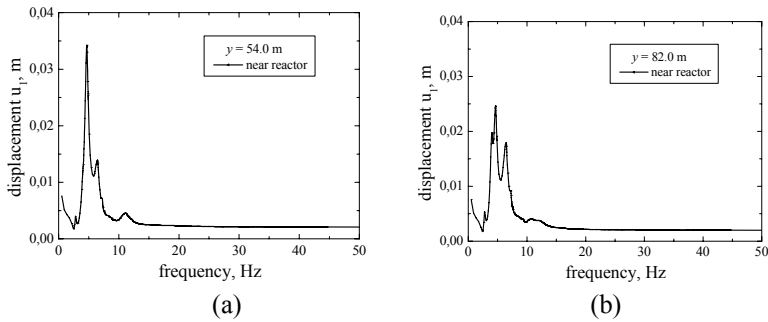


Figure 9: Horizontal displacement floor response spectra at refuelling machine support: (a) at storage position, (b) at spent fuel reception position.

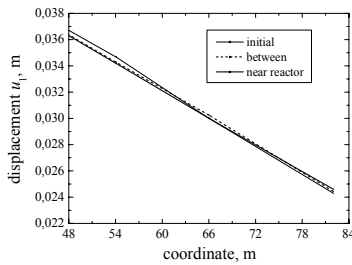


Figure 10: Variation of the horizontal displacement floor response spectra with refuelling machine support position.

6 Concluding remarks

The detailed 3D thin-walled finite element model of Ignalina NPP was applied for the seismic analysis of the refuelling machine. The in-structure response spectra of the accelerations and displacements of the supporting points of the refuelling machine are calculated using free-field ground response spectra of the Design Basis Earthquake. These response spectra were defined for three positions of the refuelling machine - at the storage position, at the spent fuel reception position and at the transportation position. The highest acceleration and displacement values were obtained for the storage position of the refuelling machine.

The differences of the displacement of supporting points at girder crane of the refuelling machine can reach 0.074 m. The possibility of the collapse of the refuelling machine is small in case of the seismic event.



Acknowledgments

The study was supported by Lithuanian Science Foundation. The authors also would like to express gratitude to the administration and technical staff of Ignalina NPP for providing information regarding operational procedures and operational data.

References

- [1] Almenas, K., Kaliatka, A. and Uspuras, E., *Ignalina RBMK-1500. A source Book, extended and updated version*, Ignalina Safety Analysis Group, Lithuanian Energy Institute, 1998.
- [2] S. Sliupa, R. Kacianauskas, D. Markauskas, G. Dundulis and E. Uspuras. Design Basis Earthquake of the Ignalina Nuclear Power Plant. *Geology*, 2 (54), pp. 19-30, 2006.
- [3] Sliupa, S., Impact of last glaciation on stress regime and fault activity of the Baltic region. *Geologija*, **39**, pp. 20-34, 2002.
- [4] Marcinkevicius, V., Report of integrated geological-hydrogeological and engineering geology mapping at the scale of 1:50 000 of Druksiai area. Vilnius, Geological Survey of Lithuania, 1995.
- [5] BRIGADE/Plus Version 1.2. User's Manual, Scanscot Technology AB, 2003.
- [6] Zakarevicius A. Investigation of the recent movements earths crust in the territory of the Lithuania. Summary of research report presented for habilitation. Vilnius, Technika, 1999, 35 p.
- [7] International Atomic Energy Agency. Seismic Design and Qualification for Nuclear Power Plants. Safety Guide No. NS-G-1.6, IAEA, Vienna, 2003.
- [8] ABAQUS Version 6.4. Analysis User's Manual, ABAQUS, Inc., 2003.
- [9] Norms and Rules for Buildings SNiP 2.03.01-84. (in Russian).
- [10] RBMK-1500 technical description. Ignalina NPP, 1985.



This page intentionally left blank

Comparing static linear and nonlinear analyses of safe rooms in a poor performance masonry building

M. Mazloom

Department of Civil Engineering, Shahid Rajaei University, Iran

Abstract

The idea of safe rooms has been developed for decreasing the earthquake casualties in masonry buildings. The information obtained from previous ground motions occurring in seismic zones expresses the lack of enough safety of these buildings against earthquakes. For this reason, an attempt has been made to create some safe areas inside the existing masonry buildings, which are called safe rooms. The practical method for making these safe areas is to install some prefabricated steel frames in some parts of the existing structure. These frames do not carry any service loads before an earthquake. However, if a devastating earthquake happens and the load bearing walls of the building are destroyed, some parts of the floors, which are in the safe areas, will fall on the roof of the installed frames and the occupants who have sheltered there will survive. This paper presents the performance of these frames located in a destroying three storey masonry building with favorable conclusions. In fact, the experimental pushover diagram of the safe room located at the ground-floor level of this building is compared with the analytical results and it is concluded that pushover analysis is a good method for seismic performance evaluation of safe rooms. Also this experimental diagram shows that the strength and displacement capacity of the steel frame are adequate to accommodate the distortions generated by seismic loads and aftershocks properly.

Keywords: earthquake, masonry building, casualties, safe room, vibration.

1 Introduction

Brick masonry has been used as a load bearing material for centuries. In gravity structures constructed by this material, the level of gravity stresses are low and



the factor of safety against compression failure is high [1]. Moreover, there is no need for high technology to construct them; as a result, they are not expensive. These advantages of masonry buildings persuade some people to construct and utilize them. But masonry structural elements, which can only bear small tensile stresses, can not resist earthquake effects [2–7].

Any strengthening of an existing structure varies from case to case depending on the specific situation which needs to be considered [8]. Also, solutions to strengthening problems require a high degree of individual attention to detail and there is a wide range of expensive choices for design and construction method. In other words, strengthening of the existing structures is expensive and time consuming.

Safe room is the name of a new method, which is neither expensive nor time consuming, for lowering earthquake casualties in masonry buildings. In this method some safe rooms will be prepared inside the building and the existing load carrying system of the structure does not strengthen. To check this suggestion, the performance of some safe rooms located in a demolishing three storey masonry building is presented in this paper. Baker et al. have introduced a similar shelter for saving the human lives against bomb explosions [9]. This shelter, which is like a table, is capable of accommodating a family of two adults and two children in such a way that if the house collapses completely, due to a near miss from a large bomb, the occupants will not be crushed by the derbies and they will be able to escape or be rescued in a short time.

In an ideal world there would be no debate about the proper method of demand prediction and performance evaluation of the steel frames of safe rooms at low performance levels. Clearly, inelastic time history analysis that predicts with sufficient reliability the forces and cumulative deformation demands in every element of the structural system is the final solution. The implementation of this solution requires the availability of a set of ground motion records that account for the uncertainties and differences in severity, frequency characteristics, and duration due to rupture characteristics and distances of the various faults that may cause motions at the site. Moreover, it requires the adequate knowledge of element deformation capacities with due regard to deterioration characteristics that define the limit state of acceptable performance.

It should be worked towards this final solution, but it is also needed to recognize the limitations of today's states of knowledge and practice. Recognizing these limitations, the task is to perform an evaluative process that is relatively simple, but captures the essential features that significantly affect the performance goal. In this context, the accuracy of demand prediction is desirable, but it may not be essential, since neither seismic input nor capacities are known with accuracy. Using inelastic pushover analysis for the steel frames of safe rooms, which is the subject of this paper, serves this purpose provided its limitations and pitfalls are fully recognized.

2 Static nonlinear analysis

The purpose of static nonlinear or pushover analysis is to evaluate the expected performance of a structural system by estimating its strength and deformation



demands in design earthquakes by means of a static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest. The evaluation is based on an assessment of important performance parameters, including global drift, interstorey drift, inelastic element deformations, deformations between elements, and element and connection forces for elements and connections that cannot sustain inelastic deformations. The inelastic pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces occurring when the structure is subjected to inertia forces that no longer can be resisted within the elastic range of structural behaviour.

Pushover analysis is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis. The followings are some examples of such response characteristics [10]: the realistic force demands in potentially brittle elements, such as axial force demands in columns, force demands in brace connections, moment demands in beam-to-column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers; estimates of the deformation demands for elements that have to deform inelastically in order to dissipate the energy imparted to the structure by ground motions; consequences of the strength deterioration of individual elements on the behaviour of the structural system; identification of the critical regions in which the deformation demands are expected to be high and that they have to become the focus to thorough detailing; identification of the strength discontinuities in plan or elevation that will lead to changes in the dynamic characteristics in the inelastic range; estimates of the interstorey drifts that account for strength or stiffness discontinuities and p-delta effects; verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections, the stiff nonstructural elements of significant strength, and the foundation system.

Static pushover analysis has no rigorous theoretical foundation. It is based on the assumption that the response of the structure can be related to the response of an equivalent single degree-of-freedom (SDOF) system. This implies that the response is controlled by a single mode, and that the shape of this mode remains constant throughout the time history response. Because each safe room contains a SDOF structural system, pushover analysis can be used for its seismic performance evaluation. It should be emphasized that both assumptions above are correct in this simple structure. However, those assumptions are incorrect in multi degree-of-freedom (MDOF) structures but pilot studies carried out by several investigators have indicated that these assumptions lead to rather good predictions of the maximum seismic response of MDOF structures, provided their response is dominated by a single mode [11–13].

3 Experimental work

The constructed three storey masonry building contained two $3 \times 4 \text{ m}^2$ rooms. The façade of the building can be seen in figure 1. The steel frames were installed in



the southern rooms of all the floors. It should be noted that all the installed frames were concentric.

Jack arch masonry slabs were utilized for constructing the entire three floors of the masonry building. There is a comprehensive research about the seismic behaviour of jack arch masonry slabs and their poor seismic performance has been highlighted there [3]. To overcome this shortcoming, some horizontal bracings were used for connecting the bottom flanges of the beams. These bracings improved the rigidity of the masonry slabs. After finishing the construction work, the rooms were decorated and some statues were located there.



Figure 1: The three storey masonry building.

To impose the devastating horizontal forces to the building, some steel hooks were welded to its lintels. Then, some cables connected these hooks to three heavy vehicles. The angles between the moving directions of these three vehicles were about 120° . To create some vibrations in the demolishing building, the second and third vehicles started moving with a very small delay from the first and second one respectively.

Figure 2 shows the building at the time of destruction. It is clear that the lateral loads were very serious and devastating. According to figure 3, similar to the buildings located very near an active fault, all parts of the masonry building collapsed. However, it can be seen that the safe rooms carried the shock and impact loads generated by the collapsing masonry building properly and also the statues located inside the safe areas did not crush by the derbies.

To control the safe rooms against vibrations caused by aftershocks, a lateral drift of 20 cm was imposed to the frame of the first floor. This horizontal displacement was within the elastic limit of the structure. The amount of the



applied force was 2200 kgf, which was about 10% of the total weight of the remaining system. Afterwards, the end part of the cable was cut; consequently, a free vibration happened in the remaining structure. This steel structure reacted properly and after finishing the free vibration, the structural system returned to its initial position with no destruction or overturning.



Figure 2: The demolishing masonry building.



Figure 3: Operation of the safe rooms after collapsing the load bearing walls.

4 Seismic performance evaluation

In this part of the research, the lateral force-horizontal displacement relationship of the safe room located at ground-floor level was obtained to assess its seismic performance. For this purpose a pullback test was conducted and the incremental lateral displacements were imposed to the steel frame and the applied force in each step was measured. In figure 4, the total base shear in each step is plotted against the accompanied roof level lateral drift. In this figure, the results of static-linear and pushover analyses can be observed too. According to this figure



the stiffness coefficients (k) obtained from static linear analysis, pushover analysis and laboratory tests were 181 kgf/cm, 91.9 kgf/cm and 120 kgf/cm respectively. The natural period T calculated according to the k values above were 2.19 sec, 3.07 sec and 2.89 sec respectively. Also the average k value of static linear analysis and pushover analysis (k_{avr}) was $(181+91.9)/2=136.45$ kgf/cm. The T value obtained from k_{avr} was 2.52 sec.

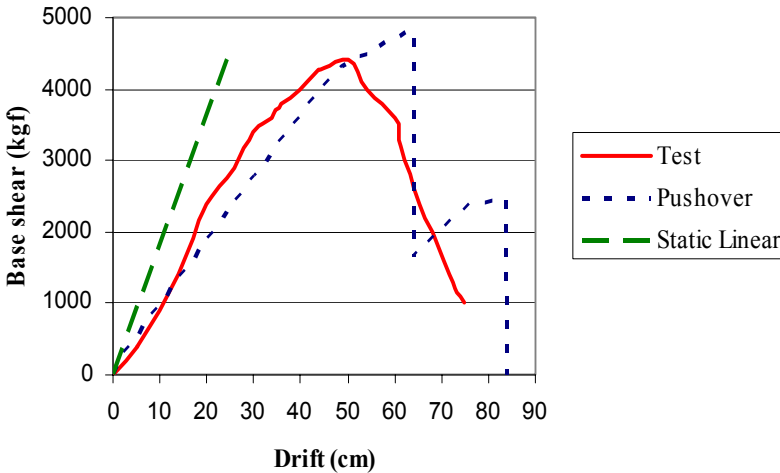


Figure 4: Pushover diagram of the investigated structure.

Standard No. 2800-05 [14] gives the following equation for measuring the lateral seismic load applied to the structure.

$$F=[(A.B.I)/R].w \quad (1)$$

where A is design base acceleration, B is response coefficient, I is importance coefficient, R is performance coefficient and w is vibrating weight of the building. In this standard B is related to the natural period of vibration of the building, kind of the ground, and the seismic zones. Table 1 shows the calculated lateral loads of the steel frame located at ground-floor level in an area of high seismicity and the ground kind 2. It can be seen that with reference to the experimental T value the calculated lateral load F is 1188 kgf. This load, which is not the lateral force resistance capacity of the frame, is the seismic load imposed to the structure according to standard No. 2800-05. It should be the best predicted lateral load because the experimental T value, which is the real T value, is utilized for calculating the response coefficient B . Of course, considering a slightly higher amount of applied load to the structure for its analysis and design creates a small margin of safety for the system against unprecedented happenings at the time of earthquake. It can be seen from Table 1 that pushover analysis underestimates and static linear analysis overestimates this value. The best prediction may be according to the T value obtained from k_{avr} , which shows the



calculated lateral load F is 1320 kgf. This value is about 11% higher than the one obtained from experimental T value. In other words, an acceptable margin of safety will exist here if the T value obtained from k_{avr} is utilized for estimating the imposed lateral seismic load F .

According to the experimental pushover graph of figure 4 the maximum force applied to the system was 4400 kgf, which was about 20% of the total weight of the remaining structure. This lateral force was more than two times of all the analytical seismic forces shown in Table 1 and also more than the calculated conservative value of $F=3850$ kgf according to standard No. 2800-05. These results pronounce the adequacy of strength capacity of the steel structure to accommodate the seismic loads.

Incorporation of all important structural response characteristics in the prediction of the SDOF displacement demand implies the ability to represent the load-deformation response of the structure with appropriate hysteretic characteristics. According to figure 4 the elastic SDOF displacement demand can be computed as $D_e=F/K$. The computed elastic displacement demands can be seen in Table 1. It is clear that the results of pushover analysis, experimental work, Standard No. 2800-05 and average analytical value were 12.1, 9.9, 21.3 and 9.7 centimeters respectively. These elastic displacement demands are the base lines for predicting the inelastic displacement demands, which need to be accomplished with due consideration given to the yield strength and hysteretic characteristics of the SDOF system. Both effects of yield strength and hysteretic characteristics can be accounted for through cumulative modification factors applied to the elastic displacement demands. It is worth noting that much information has been generated on the effect of yield strength on SDOF seismic demands [15-22]. In this research, the equation suggested by standard No. 2800-05 is utilized for calculating the inelastic displacement demands of the investigated safe room according to the elastic results. This equation is:

$$D_{ine}=0.7R.D_e \quad (2)$$

where R is performance coefficient, D_e is elastic displacement demand and D_{ine} is inelastic displacement demand. According to this equation, once the R -factor is known, the SDOF inelastic displacement demand can be computed. Standard No. 2800-05 suggests that the R -factor of ordinary moment resisting steel frames is equal to 5. Table 1 gives the inelastic displacement demands of the safe room according to the assumption above. It is clear that the maximum analytical amount of inelastic displacement demand is 42.4 cm. The experimental pushover graph of figure 4 shows the drift at the time of ultimate lateral load and its maximum value were 50 cm and 75 cm respectively. In other words, all the analytical inelastic displacement demands were lower than the amount of roof displacement at the time of ultimate lateral force. Also standard 2800-05 gives the conservative amount of 74.6 cm for inelastic deformation demand, which is almost equal to the maximum experimental drift of 75 cm. These results pronounce the adequacy of displacement capacity of the steel structure to accommodate the distortions generated by seismic loads and aftershocks properly.



Table 1: Calculated seismic forces and displacement demands.

	Different Methods				
	Static Linear Analysis	Pushover Analysis	Test Result	Standard No. 2800-05	Average Analytical Value
Force F (kgf)	1430	1114	1188	3850	1320
$D_e = F/K$ (cm)	7.9	12.1	9.9	21.3	9.7
$D_{inc} = 0.7R_e$	27.7	42.4	34.7	74.6	34

5 Conclusions

The idea of a safe room is a practical solution for lowering the earthquake casualties in poor performance masonry buildings located at earthquake-prone areas. Because each safe room contains a SDOF structure, pushover analysis is a very good solution for evaluating its seismic performance. The experimental pushover diagram of the investigated structural system of the safe room located at the ground-floor level of a destructed 3 storey masonry building showed that the ductility, lateral stability and strength capacity of the structural system were quite satisfactory. Also the structures of the experimented safe rooms carried a considerable free vibration properly. Therefore, the structural system of the investigated safe rooms was capable of accommodating the distortions generated by seismic loads and aftershocks properly.

References

- [1] Bakhteri, J. and Sambasivam, "Mechanical behaviour of structural brick masonry: an experimental evaluation," *Proceedings of the 5th Asia-Pacific Structural Engineering and Construction Conference*, Johor Bahru, Malasia, August, 2003, 305-317.
- [2] Rangelova, F. "Earthquake and blast shock loading on masonry veneer structures", *5th Asia-Pacific Conference on Shock & impact loads on Structures*, Changsha, Hunan, China, November, 2003, 323-327.
- [3] Maheri, M.R. and Rahmani, H. "Static and seismic design of one-way and two-way jack arch masonry slabs", *Engineering Structures*, 2003, 25, 1639-1654.
- [4] Henderson, R.C., Fricke, K.E., Jones, W.D. and Beavers, J.E. "Summary of a large- and small-scale unreinforced masonry infill test program", *Journal of Structural Engineering*, December, 2003, 1667-1675.



- [5] Memari, A.M., Burnett, E.F.P. and Kozy, B.M. "Seismic response of a new type of masonry tie used in brick veneer walls", *Construction and Building Materials*, 16, 2002, 397-407.
- [6] Taghdi, M., Bruneau, M. and Saatcioglu, M. "Analysis and design of low-rise masonry and concrete walls retrofitted using steel strips", *Journal of Structural Engineering*, September, 2000, 1026-1032.
- [7] Barbieri, A., Mantegazza, G. and Gatti, A. "Behaviour of masonry walls subject to shear stresses and reinforced with FRCM", *2nd Specialty Conference on the Conceptual Approach to Structural Design*, Milan, Italy, July, 2003, 257-264.
- [8] Ivanyi, G. and Buschmeyer, W. "Conceptual design in strengthening of concrete bridges", *2nd Specialty Conference on the Conceptual Approach to Structural Design*, Milan, Italy, July, 2003, 521-527.
- [9] Baker, J.F., Horne, M.R. and Heyman, J., *The Steel Skeleton*, Volume II, *Plastic Behaviour and design*, The Cambridge University Press, 1956.
- [10] Krawinkler, H., and Seneviratna, G. D. P. K. "Pros and cons of a pushover analysis for seismic performance evaluation", *Engineering Structures*, 20 (4-6), 1998, 452-464.
- [11] Lawson, R. S., Vance, V. and Krawinkler, H. "Nonlinear static pushover analysis – why, when and how?", *5th US Conference of Earthquake Engineering*, Vol. 1, Chicago, IL, 1994, 283-292.
- [12] Miranda, E. "Seismic evaluation and upgrading of existing buildings", Ph.D. dissertation, Department of Civil Engineering, University of California, Berkley, CA, 1991.
- [13] Fajfar, P. and Fischinger, M. "N2 – a method for non-linear seismic analysis of regular structures", *9th World Conference of Earthquake Engineering*, Vol. 5, Tokyo-Kyoto, Japan, 1988, 111-116.
- [14] Standard No. 2800-05, *Iranian Code of Practice for Seismic Resistant Design of Buildings*, 3rd Edition, Building and Housing Research Center, PN S 253, 2005.
- [15] Fajfar, P. and Krawinkler, H., *Nonlinear seismic analysis and design of reinforced concrete buildings*, Elsevier, London, 1992.
- [16] Krawinkler, H. and Rahnama, M. "Effects of soft soils on design spectra", *10th World Conference on Earthquake Engineering*, Vol. 10, Madrid, Spain, 1992, 5841-5846.
- [17] Miranda, E. and Bertero, V.V. "Evaluation of strength reduction factors for earthquake-resistant design", *Earthquake Spectra, EERI*, 1994, 10 (2), 357-379.
- [18] Nassar, A.A. and Krawinkler, H. "Seismic demands for SDOF and MDOF systems", *John A. Blume Earthquake Engineering Center, Report No. 95*, Department of Civil Engineering, Stanford University, 1991.
- [19] Nassar, A.A., Krawinkler, H. and Osteraas, J.D. "Seismic design based on strength and ductility demands", *10th World Conference on Earthquake Engineering*, Vol. 10, Madrid, Spain, 1992, 5861-5866.
- [20] Newmark, N.M. and Hall, W.J. "Earthquake spectra and design", *EERI Monograph Series*, 1982.



- [21] Rahn timer, M. and Krawinkler, H. "Effects of soft soils and hysteresis models on seismic design spectra", *John A. Blume Earthquake Engineering Center, Report No. 107*, Department of Civil Engineering, Stanford University, 1993.
- [22] Vidic, T., Fajfar, P. and Fischinger, M. "Consistent inelastic design spectra: strength and displacement", *Earthquake Engineering and Structural Dynamics*, 1994, 23.



Empirical fragility curves for Peruvian school buildings

A. Muñoz, M. Blondet, R. Aguilar & M.-A. Astorga

Department of Civil Engineering, Catholic University of Peru, Peru

Abstract

This paper describes an estimation of economic losses in Peruvian educational buildings for different levels of seismic action. Opinions of experts were processed to generate seismic intensity versus damage relationships. Economic losses were then expressed through fragility curves and damage matrices. For frequent earthquakes (50-year return period) a 50% loss would be expected in adobe buildings and 20% loss in confined masonry structures built before 1997. During severe earthquakes (500-year return period) only confined masonry structures built after 1997 would be repairable at a cost of around 40% of their original cost.

Keywords: fragility curves, damage matrices, 1997 Peruvian seismic code, seismic loss, types of Peruvian school buildings.

1 Introduction

After earthquakes, Peruvian authorities allocate large quantities of resources to recover the educational infrastructure affected. The recovery usually consists in repairing the least affected buildings or retrofits and reinforcing the worst affected ones.

Experience has shown that damage to buildings could be significantly reduced by strengthening work done before earthquakes. It is not possible for the Peruvian government to handle a national strengthening program, so it is necessary to develop a plan to reduce the infrastructures' seismic risk gradually.

As a contribution to this national plan, we have identified the most representative buildings and we have made an estimation of the seismic behaviour using tools to quantify losses.



2 Educational buildings in Peru

In Peru there are approximately 41000 public schools constructed at different periods of time, with different architecture and materials. There are buildings of reinforced concrete, masonry, wood, “adobe”, “sillar” and even mixed constructions such as clay-sillar masonry or quinchá-adobe constructions.

According to the Ministry of Education’s report [1], the materials most typically used for school buildings are adobe, concrete-masonry and wood.

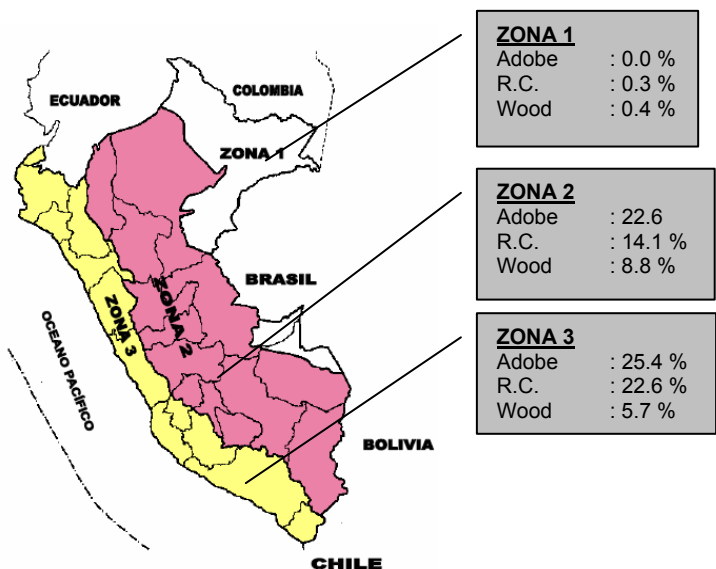


Figure 1: Peruvian school distribution according to the prevailing construction material.

Figure 1 shows the school distribution according to the prevailing material in the three seismic zones established by the Peruvian seismic design code [2].

Adobe buildings represent 48% of the total buildings and these are located in the highest Peruvian seismic zones. Reinforced concrete or masonry buildings represent 37%, and almost all of them are located in the highest seismic zones.

With the help of the “National Institute for Education and Health Infrastructure – INFES” (in charge of Peruvian school buildings between 1993 and 2003) we have identified five types of buildings as the most representative in the country.

2.1 780 Modern building

This type has been built since 1997 following the Peruvian seismic code, which in that year significantly raised the rigidity and lateral resistance requirements. The typical building has a rectangular plan, with one, two or three stories and



7.80m x 7.80m square classrooms. In the longitudinal direction, the building has two frames with big columns; and in the transverse direction, confined masonry walls. The roofs are lightened unidirectional slabs 0.20m thick.



Figure 2: 780 Modern buildings.

2.2 780 Pre SDC-1997 building

This type had been built prior to 1997 (Figure 3). Its architecture is similar to the 780 modern building. In the longitudinal direction the building has two frames with weak columns and in the transverse direction confined masonry walls. The roofs are lightened unidirectional slabs 0.20m thick. This type of building has suffered significant damage in past earthquakes. The problems were mainly due to the limited lateral rigidity that triggers the “short column” problem.



Figure 3: 780 pre Seismic Design 1997 Code buildings.

2.3 Big school building

These buildings were built 50 years ago (Figure 4). They have two or three stories with approximately 10m-long classrooms. Its plan area is more than double that of the 780 buildings. In the longitudinal direction the building has



three edges of columns and in the transverse direction confined masonry walls. The roofs are lightened unidirectional slabs 0.20m thick.



Figure 4: Big school buildings.

2.4 Adobe building

These buildings vary greatly in dimensions and quality of materials. The Government does not have a typical module. For our purposes, we have considered as the typical building a single story rectangular plan with two or three classrooms (Figure 5). The walls are orthogonal to each other and are 0.40m thick. The roof is light and tilted.

Adobe buildings

2.5 High Educational Pre 1997-SDC building

These are two- to five-story buildings (Figure 6) with rectangular plan similar to the big school buildings. The structural system consists of three reinforced concrete frames in the longitudinal direction and confined masonry walls in the transverse direction. The roofs are lightened unidirectional slabs 0.20m thick.



Figure 5: Adobe buildings.





Figure 6: High educational pre 1997- SDC buildings.

3 Seismic loss estimation

The seismic intensity has been expressed using the Mercalli Modified (MM) scale and also using the peak ground acceleration. The relation between these two indicators is presented in Table 1.

Table 1: Relation between seismic intensity and PGA.

IMM	PGA	Description
V	<0.05	Felt by some people.
VI	0.05 – 0.10	Felt by everybody.
VII	0.10 – 0.20	Negligible damage in well design-built buildings.
VIII	0.20 – 0.35	Slight damage in well design-built buildings.
IX	0.35 – 0.50	Serious damage in special designed structures.
X	> 0.50	Building's destruction.
XI		Few buildings remain standing.
XII		Total destruction and changes in the landscape.

In order to quantify earthquake damage we use the ratio between the loss value and the replacement value. This ratio, eqn. (1), is called Damage Factor (DF).

$$\text{Damage Factor (DF)} = \frac{\text{Lost Value(LV)}}{\text{Replace Value(RV)}} \quad (1)$$

Limits have been established for the DF in ATC13 [3] and seven damage levels (DL) have been defined as shown in Table 2.



Table 2: Damage levels.

DL	DF limits	Description
1. None	0	No damage
2. Very light	0 - 0.01	Light, limited and located damage, repairs not required.
3. Light	0.01 - 0.1	Significant and located damage (few elements) repairs not required.
4. Moderate	0.1 - 0.3	Significant located damage, repairs required.
5. High	0.3 - 0.6	Extensive damage, major repairs required.
6. Severe	0.6 – 0.99	Extensive major damage, demolishing and repairs required.
7. Collapse	1	Total destruction.

The intensity-damage relation was handled using probability density functions. Using these relations, fragility curves and damage matrixes have been obtained in order to quantify losses.

There is no statistical information available about the seismic behavior of Peruvian school buildings, so we gathered expert opinions to obtain basic information to create probability functions.

4 Delphi method

We collected and handled the experts’ opinions using the Delphi method [4]. We described the structural characteristics of school buildings and designed a form to obtain the damage estimation under several seismic severity levels. For each level we asked for the DL and the possible values for mean, maximum and minimum damage factor (MDF, Vmin, and Vmax respectively – Figure 7).





Porcentaje de Daños en Edificaciones Educativas Peruanas

Por favor anote los valores que en su opinión describen la relación Daño-Intensidad para las edificaciones educativas peruanas.

Gran Unidad Escolar																
Intensidades	V - VI			VII			VIII			IX			X - XI			
Daño	ND	Min (%)	VMP (%)	Max (%)	ND	Min (%)	VMP (%)	Max (%)	ND	Min (%)	VMP (%)	Max (%)	ND	Min (%)	VMP (%)	Max (%)
Confiability																

Figure 7: Form to collect experts’ opinions.

We communicated with the group of experts by internet and also by printed and digital means. The internet direct link to access the web page is <http://www.pucp.edu.pe/secc/civil/dsrep/>. This web page contains information about the Delphi method, principal characteristics of school buildings, a small image gallery and the results of the work.

5 Density functions, fragility curves and damage matrices

Damage distribution was modeled using the Beta Function because of its simplicity and adaptability to the information of the group of experts (Figure 8). We assumed that the estimation of the mean damage factor (MDF) corresponds to the mean value of the Beta Distribution and that 90% of the occurrence's probability is between the extreme values V_{max} and V_{min} [3].

With the mean values obtained from the group of experts, damage probability distribution functions (p) were created for each intensity and for each type of Peruvian school building.

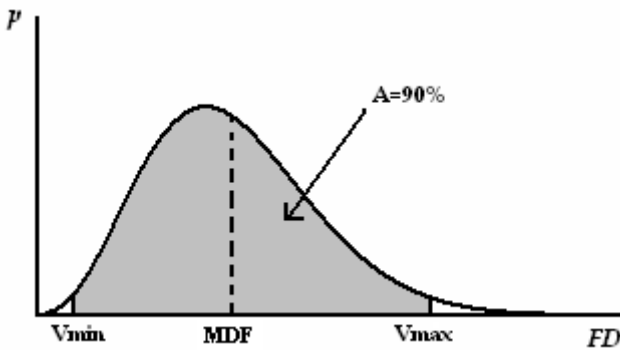


Figure 8: Beta function assumed to represent the damage distribution.

Fragility curves were determined computing the exceeding probabilities (E.P) corresponding to the extreme value of each damage limit in each seismic intensity, eqn. (2). For the damage matrices it was necessary to determine the occurrence's probability (q) of each damage state in each intensity, eqn. (3).

$$E.P = 1 - P = 1 - \int_0^{d_i} p \, dDF \quad (1)$$

$$q = \int_{d \min}^{d \max} p \, dDF \quad (2)$$



As a result of this procedure, we obtained fragility curves and damage matrices corresponding to the five types of Peruvian school buildings. Figure 9 and Table 3 show the results for the 780 modern building.

Using these tools, losses could be represented as the product of the mean damage factor and the construction value. Figure 11 corresponds to 50 “780 Pre 1997-SDC” buildings whose construction value was US\$ 4 800 000. In this type of buildings, for intensities greater than VIII+, the damage factor exceeds 60%. From this limit on, we must consider total loss [5] and the cost of the loss will be the same as the initial construction value.

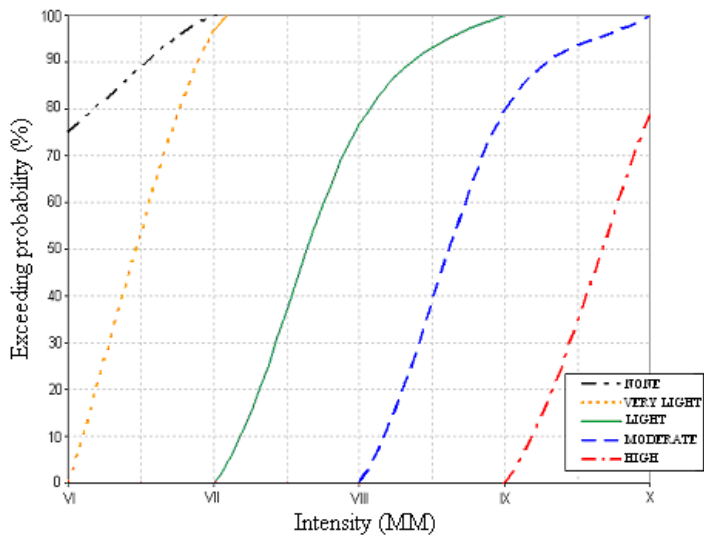


Figure 9: Fragility curves for 780 modern building.

Table 3: Damage matrix for 780 modern building.

Damage Level	Central damage factor	Damage probability (%)				
		VI	VII	VIII	IX	X
None	0	25.0	0	0	0	0
Very light	0.005	75.0	3.3	0	0	0
Light	0.05	0.1	96.6	23.3	0	0
Moderate	0.20	0	0.1	76.6	20.2	0
High	0.45	0	0	0.1	79.8	20.9
Severe	0.80	0	0	0	0	79.1
Collapse	1.00	0	0	0	0	0
Mean damage factor (MDF)		0.38	4.86	16.53	39.95	72.69



6 Conclusions

For the two major Peruvian seismic zones, we have identified five types of school buildings as the most representative ones. One type corresponds to adobe buildings, three types are reinforced concrete-masonry (RC-M) buildings built before 1997 and the last ones are bigger RC-M buildings built after 1997.

If we consider that irreparable damage is when the mean damage factor (MDF) is higher than 60%, results show that adobe buildings would be irreparable from VII MM of intensity, while RC-M buildings built before 1997 would be irreparable from VIII+ MM. New RC-M buildings would be irreparable from X MM.

For frequent earthquakes (50 years of return period), results show that the MDF in adobe buildings would be 45%; in RC-M buildings built before 1997 around 20%; and in new RC-M buildings just 5%.

For odd earthquakes (500 years of return period) results show that the MDF in adobe buildings would be 95%; it would be about 65% for RC-M buildings built before 1997; and 38% for new RC-M buildings.

The number of RC-M school buildings built following the 1997 Peruvian seismic design code represents only 2% of the total number of school buildings. Results show that only these buildings would be repairable after a severe seismic event.

References

- [1] Ministerio de Educación del Perú, 2003. Cifras de la educación 1998-2003. Lima, Peru.
- [2] Ministerio de vivienda, construcción y saneamiento. 2003. Norma técnica de diseño sismorresistente-NTE 030. Lima, Peru.
- [3] ATC 13 (Applied Technology Council), 1985. Earthquake Damage Evaluation Data for California. Redwood City, California, USA.
- [4] Linstone A, Turof M. 1975. The Delphi Method: Technique and Applications. Massachusetts, USA.
- [5] Federal Emergency Management Agency. 1988. FEMA 154: Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook. Washington, USA.



This page intentionally left blank

Evaluation of lateral load pattern in pushover analysis

S. I. Javadein¹ & R. Taghinezhad²

¹*Department of Civil Engineering, Islamic Azad University, Bandar Anzali, Iran*

²*Department of Civil Engineering, Islamic Azad University, Torbat Heydariye, Iran*

Abstract

The objective of this study is to evaluate the performance of the frame structures for various load patterns and a variety of natural periods by performing pushover and nonlinear dynamic time history analyses. 3, 5, 7, 9 and 13-story moment steel frame structures are used in the analyses and the load distributions for pushover analyses are chosen as triangular, IBC ($k=2$) and rectangular. These frames have five different natural periods. Even though the nonlinear dynamic time history analysis is the best way to compute seismic demands, FEMA-356 and ATC-40 proposes the use of nonlinear static procedure or pushover analysis. The five frame structures have been analyzed using the nonlinear program SAP2000. This paper is also intended to compare the results of pushover and nonlinear dynamic time history analyses. To evaluate the results from the pushover analyses for three load patterns and also five natural periods, nonlinear dynamic time history analyses are performed. Earthquake ground motions recorded at 3 stations during various earthquakes are used in the analyses. The ground motion records used in this study include TABAS, NAGHAN and ELCENTRO. Pushover and nonlinear time history analyses results are compared to choose the best load distribution for a specific natural period for this type of frame structure.

Keywords: pushover analysis, nonlinear time history, load patterns, moment-resisting frame.



1 Introduction

Only the life safety and collapse prevention in general earthquake resistant design phenomena are explicitly prevented in seismic design codes. The design is generally based on evaluating the seismic performance of structures. It is required to consider inelastic behavior while evaluating the seismic demands at low performance levels. FEMA-356 [1] and ATC-40 [2] use pushover analysis as nonlinear static analysis but nonlinear time history analysis has more accurate results on computing seismic demands. The purposes in earthquake-resistance design are:

- (a) to prevent non-structural damage in minor earthquakes, which may occur frequently in life time.
- (b) to prevent structural damage and minimize non-structural damage in moderate earthquakes which may occur occasionally.
- (c) to prevent collapsing or serious damage in major earthquakes which may occur rarely. Designs are explicitly done only under the third condition.

The objective of this study is to evaluate the performance of the frame structures for various load patterns and variety of natural periods by performing pushover and nonlinear dynamic time history analyses. 3, 5, 7, 9 and 13-story moment steel frame structures are used in the analyses and the load distributions for pushover analyses are chosen as triangular (IBC, $k=1$), (IBC, $k=2$) and rectangular, where k is the an exponent related to the structure period to define vertical distribution factor IBC [3]. The five frame structures have been analyzed using nonlinear program SAP2000 [4] and the results have been compared by recorded response data. Both nonlinear static pushover analysis and nonlinear dynamic time history analysis are performed. The correlations between these nonlinear analyses are studied. The performance of the buildings subjected to various representative earthquake ground motions is examined. Finally, pushover and nonlinear time history analyses results are compared to choose the best load distribution (pattern) for specific natural period for these types of steel moment frame structures.

2 Ground motion data

The nonlinear response of structures is very sensitive to the structural modeling and ground motion characteristics. Therefore, a set of representative ground motion records that accounts for uncertainties and differences in severity, frequency and duration characteristics has to be used to predict the possible deformation modes of the structures for seismic performance evaluation purposes. For this study, it is considered as 3 different data used in the nonlinear dynamic time history analyses, given in the Table 1. The peak ground accelerations are in the range 0.348 to 0.722g, where g is acceleration due to gravity.



Table 1: Ground motion data used in the analyses.

Record	Duration	Pga
NAGHAN	5.04	0.722g
TABAS	25.04	0.933g
ELCENTRO	53.8	0.348g

3 Description of the frame structure

Four steel moment frames with 3, 5, 7, 9 and 13-story were utilized to cover a broad range of fundamental periods. The moment steel frame structures are shown in Figure 1. The case study frames were designed for the AISC-ASD2001 [5] and Iranian 2800 Code-ver.3 [6]. Two dimensional models of case study frames were prepared using SAP2000 considering the necessary geometric and strength characteristics of all members that affect the nonlinear seismic response. Rigid floor diaphragms were assigned at each story level and the seismic mass of the frames were lumped at the mass center of each story. Gravity loads consisting of dead loads and 25% of live loads were considered in pushover and nonlinear time history analyses. The columns are assumed as fixed on the ground. Yield strength of the steel reinforcements is 2400 kg/cm^2 . Also the cross section of all beams and columns in these frames are IPE and IPB-shapes respectively. Tree vibration analyses were performed to determine elastic periods and mode shapes of the frames. The dynamic properties of the case study frames are summarized in Table 2.

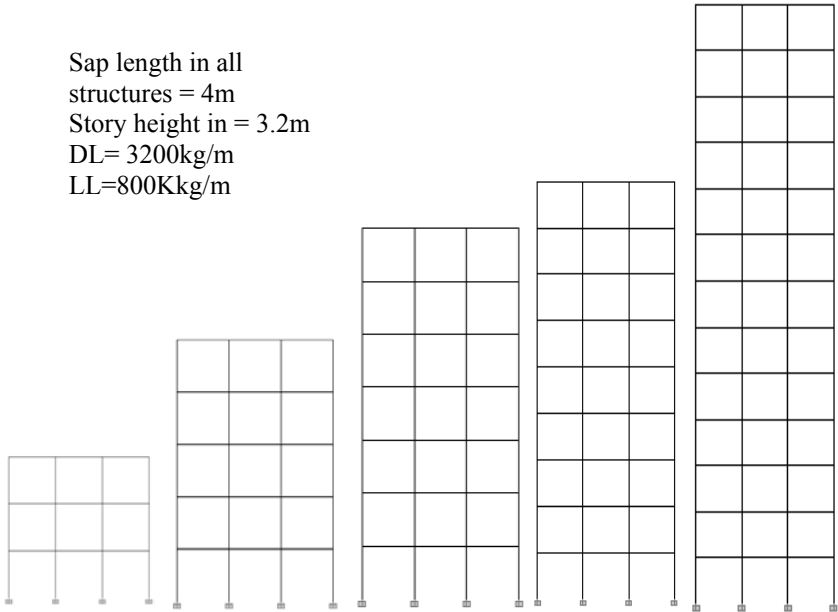


Figure 1: Diagram of analyzed 3, 5, 7, 9 and 13-story moment steel frames.



The first, second and third natural periods of the structures are given in Table 2.

Table 2: Dynamic properties of case study frames.

Frame	Period (sec.)		
	T1	T2	T3
3 Story	1.07	0.40	0.23
5 Story	1.34	0.52	0.30
7 Story	1.57	0.63	0.37
9 Story	2.58	0.82	0.45
13 Story	2.98	0.88	0.50

4 Nonlinear static pushover analysis of frame structures

The static pushover procedure has been presented and developed over the past twenty years by various researches. The method is also described and recommended as a tool for design and assessment purpose for the seismic rehabilitation of existing building and represents a main component of the Spectrum Capacity Analysis Method (ATC-40) [2]. It is clear from recent discussion that this approach is likely to be recommended in future codes.

For low performance levels, to estimate the demands, it is required to consider inelastic behavior of the structure. Pushover analysis is used to identify the seismic hazards, selection of the performance levels and design performance objectives. In pushover analysis, applying lateral loads in patterns that represent approximately the relative inertial forces generated at each floor level and pushing the structure under lateral loads to displacements that are larger than the maximum displacements expected in design earthquakes (Li [7]). The pushover analysis provides a shear vs. displacement relationship and indicates the inelastic limit as well as lateral load capacity of the structure. The changes in slope of this curve give an indication of yielding of various structural elements. The main aim of the pushover analysis is to determine member forces and global and local deformation capacity of a structure. The information can be used to assess the integrity of the structure. After designing and detailing the moment steel frame structures, a nonlinear pushover analysis is carried out for evaluating the structural seismic response. For this purpose the computer program SAP2000 has been used. Three simplified loading patterns; triangular (IBC, $k=1$), IBC ($k=2$) and rectangular, where k is an exponent related to the structure period to define vertical distribution factor, are used in the nonlinear static pushover analysis of 3, 5, 7, 9 and 13-story steel frame structures. Load criteria are based on the distribution of inertial forces of design parameters. The simplified loading patterns as uniform distribution, triangular distribution and IBC distribution. These loading patterns are the most common loading parameters. Vertical distribution of seismic forces:



$$F_x = C_{vx} V \quad (1)$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (2)$$

C_{vx} = Vertical distribution factor.

V = Total design lateral force or shear at the base of structure.

w_i and w_x = The portion of the total gravity load of the structure h_i .

h_x = The height from the base.

k = An exponent related to the structure period.

In addition these lateral loadings, frames are subjected live loads and dead weights. P- Δ effects have been taken into the account during the pushover analyses. The lateral force is increased for 3, 5, 7, 9 and 13-story steel frames until the structures collapsed. Beam and column elements are used to analyze the frames. The beams are assumed to be rigid in the horizontal plane. Inelastic effects are assigned to plastic hinges at member ends. Strain-hardening is neglected in all elements. Bilinear moment-rotation relationship is assumed for both beam and column members. The results of the pushover analyses in 3, 5, 7, 9 and 13-story steel frames are presented in Figures 2 and 3 respectively.

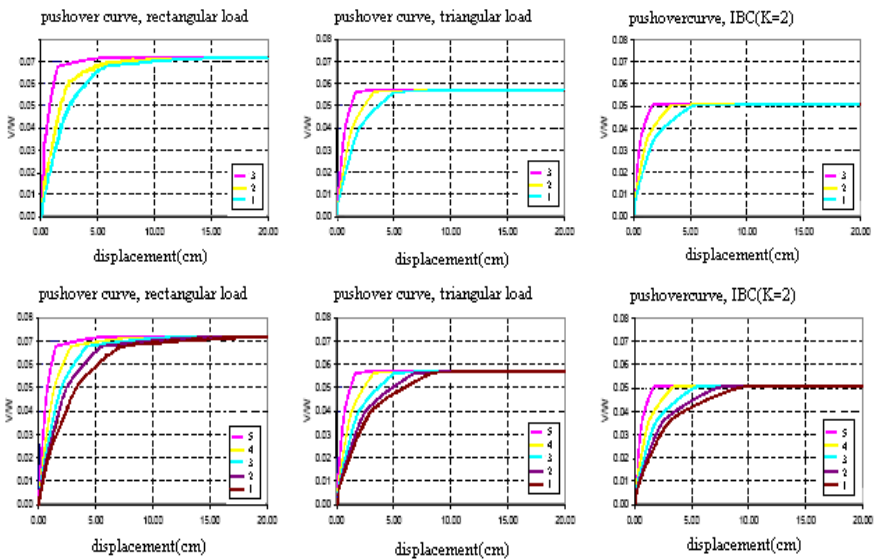


Figure 2: Pushover curves of 3 and 5-story steel frame for three different load patterns.



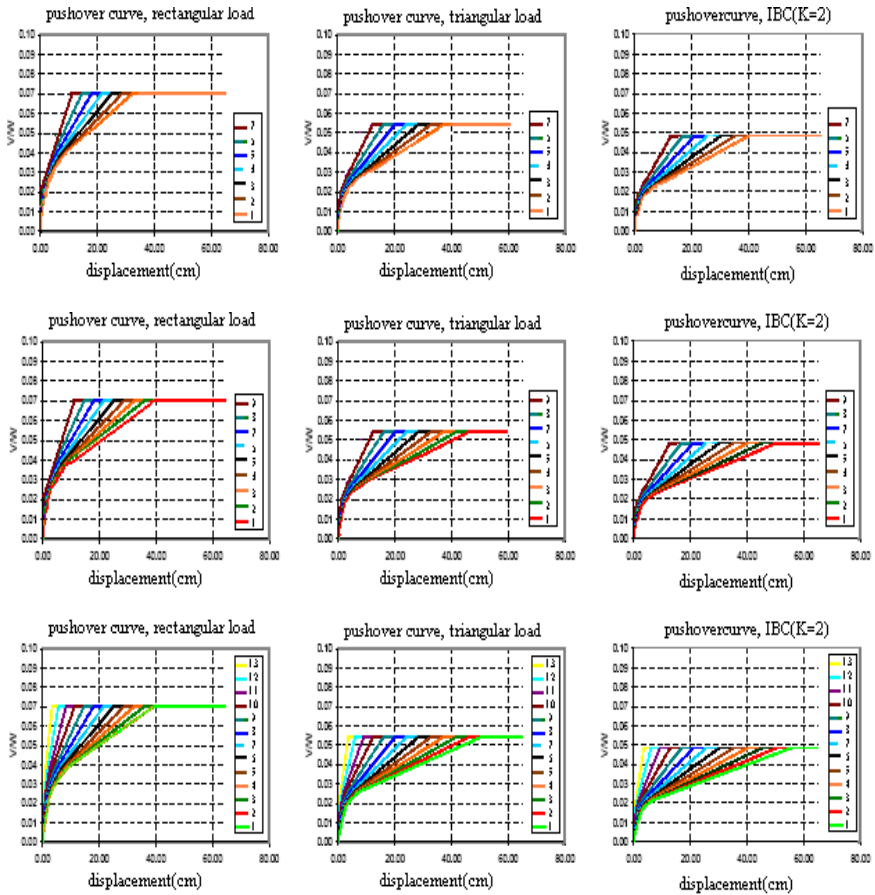


Figure 3: Pushover curves of 7, 9 and 13-story steel frame for three different load patterns.

The pushover curves are shown for three distributions, and for each frame structures. The curves represent base shear-weight ratio versus story level displacements for uniform, triangular and IBC load distribution. Shear V was calculated by summing all applied lateral loads above the ground level, and the weight of the building W is the summation of the weights of all floors. Beside these, these curves represent the lost of lateral load resisting capacity and shear failures of a column at the displacement level. The changes in slope of these curves give an indication of yielding of various structural elements, first yielding of beam, first yielding of column and shear failure in the members. By the increase in the height of the frame structures, first yielding and shear failure of the columns is experienced at a larger roof displacements and rectangular distribution always give the higher base shear-weight ratio comparing to other load distributions for the corresponding story displacement (horizontal displacement).



5 Nonlinear dynamic time history analysis of frame structures

After performing pushover analyses, nonlinear dynamic time history analyses have been employed to the five different story frame structures. These frames are subjected live and dead weights. Also $P-\Delta$ effects are under consideration as in pushover analysis. For time history analysis $P-\Delta$ effects have been taken into the account. Finite element procedure is employed for the modeling of the structures during the nonlinear dynamic time history analyses. SAP2000 has been used for nonlinear time history analysis and modeling. The model described for pushover analyses has been used for the time history analyses. Mass is assumed to be lumped at the joints. The frames are subjected to 3 earthquake ground motions, which are recorded during TABAS, NAGHAN and ELCENTRO for the nonlinear dynamic time history analyses (Figure 4).

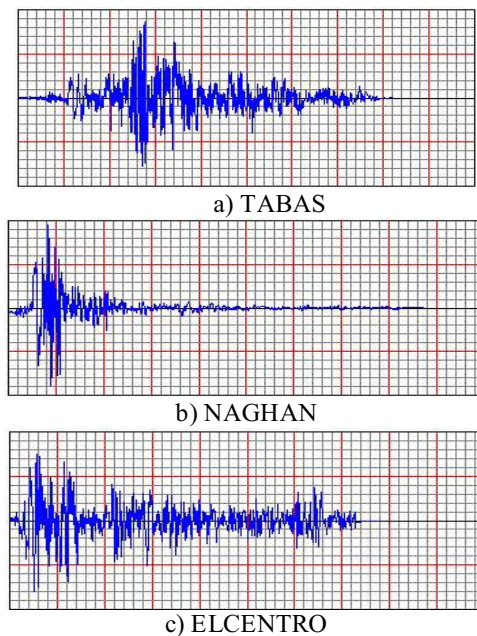


Figure 4: Acceleration-time histories of ground motion records.

These data are from different site classes as I, II, III and IV. The selected earthquake ground motions have different frequency contents and peak ground accelerations. The ground motion data are chosen from near-field region to evaluate the response of the frame structures in this region and comparison of them with pushover analyses results.

The results of nonlinear time history analysis for 3, 5, 7, 9 and 13-story steel frame structure are presented in Figure 5.



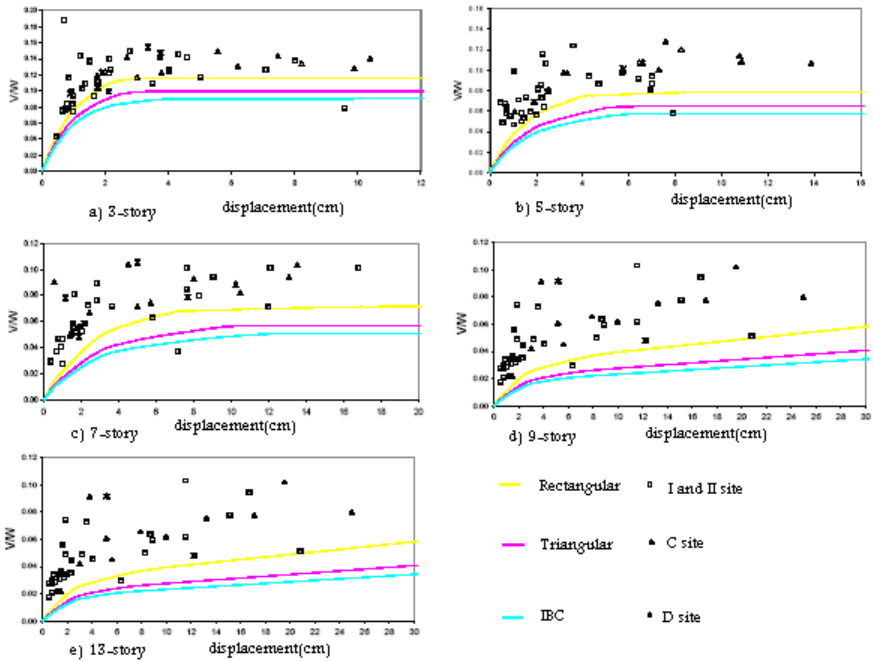


Figure 5: Pushover and nonlinear time history results of 3, 5, 7, 9 and 13-story.

Pushover and nonlinear time history analyses results are compared to for specific natural period for five different frame structures and for each load distributions; rectangular, triangular and IBC ($k=2$).

6 Conclusion

After designing and detailing the moment steel frame structures, a nonlinear pushover analysis and nonlinear dynamic time history analysis are carried out for evaluating the structural seismic response for the acceptance of load distribution for inelastic behavior. It is assumed for pushover analysis that seismic demands at the target displacement are approximately maximum seismic demands during the earthquake. According to Figures 2 and 3, for higher story frame structures, first yielding and shear failure of the columns is experienced at the larger story displacements and rectangular distribution always give the higher base shear-weight ratio comparing to other load distributions for the corresponding story displacement. As it is presented in Figure 5, nonlinear static pushover analyses for IBC ($k=2$), rectangular, and triangular load distribution and nonlinear time history analyses results for the chosen ground motion data (all of them are near-field data) are compared. Pushover curves do not match with nonlinear dynamic time history analysis results especially for higher story moment steel frame structures (9 and 13-story frame structures). The pushover analyses results for



rectangular load distribution estimate maximum seismic demands during the given earthquakes more reasonable than the other load distributions, IBC ($k=2$), and triangular.

References

- [1] FEMA (2000b). "Prestandard and Commentary for the Seismic Rehabilitation of Buildings", Report FEMA-356, Federal Emergency Management Agency, Washington, DC, U.S.A.
- [2] ATC-40, "Seismic evaluation and Retrofit of Concrete Buildings", Vol.1, Applied Technology Council, Redwood City, CA, 1996.
- [3] IBC, International Building Code, International Conference of Building Officials, Whittier, California, 2000.
- [4] Computers and Structures Inc. (CSI), SAP2000 Three Dimensional Static and Dynamic Finite Element Analysis and Design of Structures V7.40N, Berkeley, California.
- [5] AISC, Manual of Steel Construction: Load and Resistance Factor Design, 3rd Edition, American Institute of Steel Construction, Chicago, IL, 2001.
- [6] Iranian Code of Practice for Seismic Resistant Design of Building, Standard No. 2800-5, 3rd Edition, Building and Housing Research Center, 2005.
- [7] Li, Y.R. Non-Linear Time History And Pushover Analyses for Seismic Design and Evaluation. PhD Thesis, University of Texas, Austin, TX. 1996.



This page intentionally left blank

3-D reproduction analyses for actual earthquake behaviors of existing dams

Y. Ariga

Chigasaki Research Institute, Electric Power Development Co., Japan

Abstract

Results evaluated by dynamic analysis method will be significantly affected by the values of dynamic property. Therefore, the dynamic property values should be quantitatively evaluated based on actual earthquake phenomena in order to realize reliable dynamic analysis. I have developed 3-D nonlinear dynamic analysis method for coupled dam-joints-foundation-reservoir system and in order to verify the validity of the method developed, I have made 3-D reproduction analyses for actual earthquake behaviors of existing dams. By these reproduction analyses, the dynamic property values of dam and foundation have been evaluated quantitatively. Furthermore, the efficiency of the analytical method developed has been proved. The 3-D reproduction analysis for the actual earthquake behavior of an existing dam is necessary to verify the efficiency of the dynamic analysis method. Effective utilization of the earthquake motions observed is important for realizing accurate and reliable evaluation for seismic safety of structures.

Keywords: earthquake safety, 3-D dynamic analysis, verification, earthquake observation, dynamic property.

1 Introduction

Confirmation and securing of dam safety against large earthquakes is very important subject in earthquake countries. With the rapid improvement of numerical analysis techniques, a 3-D dynamic analysis method has come to be applied for earthquake safety evaluation of existing dams. The stresses and strains calculated by the dynamic analysis procedure will be largely changed according to the dynamic property values. Among the dynamic properties, the dynamic shear modulus (or shear wave velocity) and the damping factor are the



most influential. Therefore, the dynamic shear modulus and the damping factor should be evaluated carefully and quantitatively based on the actual earthquake phenomena. In this study, the dynamic shear modulus and the damping factor of dam and foundation were evaluated quantitatively, and the validity of the 3-D dynamic analysis method proposed was verified based on the reproduction analyses for actual earthquake behaviors of existing dams. The purpose of 3-D reproduction analysis for the actual earthquake behaviour of existing dam is as follows:

- quantitative evaluation of dynamic property values;
- identification of 3-D analytical model;
- verification of reliability of 3-D dynamic analysis method.

2 Development of 3-D nonlinear dynamic analysis method

A dynamic interaction between dam and foundation (Miura, [2]), a dynamic reduction effect on dam by reservoir water, a radiation of wave energy from the boundary of foundation, a dissipation of wave energy from the boundary of reservoir, a non-linear effect of dam material against strong earthquake motions (Hatano [3]), a discontinuous behaviors of joints, and so forth should be considered quantitatively and properly in order to realize an accurate evaluation for earthquake safety of dams. Taking these matters into account, I have developed a 3-D nonlinear dynamic analysis method for a coupled dam–joints–foundation–reservoir system (Ariga [4]). One of the typical examples of 3-D analytical model for a coupled dam–joints–foundation–reservoir system is shown in fig. 1.

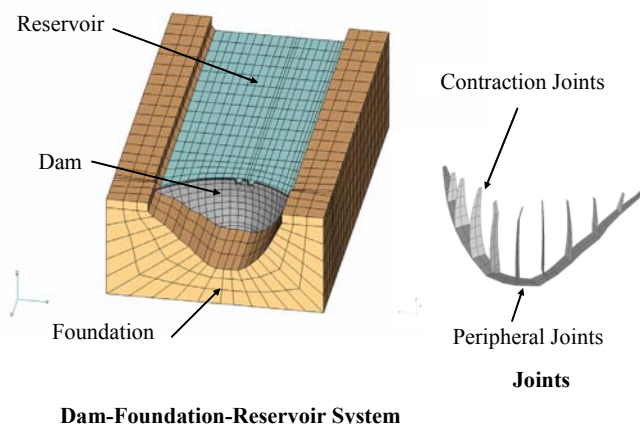


Figure 1: Typical 3-D model for dam–joints–foundation–reservoir system.

The contraction and peripheral joints are generally arranged for preventing the cracks due to the change of temperature, etc. So, it is considered that the



discontinuous behaviors of joints will have significant effects on the dynamic response of dam against very strong earthquake motion. A 3-D joint element is applied for modeling the joints or cracks. As for the reservoir, the wave equation is dispersed by the finite difference method.

3 Procedure for 3-D reproduction analysis of existing dams

The basic flow of 3-D reproduction analysis is shown in fig. 2. The dynamic property values can be identified by reproducing the actual earthquake behaviors of existing dams. The dynamic shear modulus and the damping factor can be evaluated by adjusting the analytical results to the earthquake observation results. The dynamic shear modulus can be evaluated by fitting the predominant frequencies of transfer function between dam base and dam crest. The damping factor can be evaluated by fitting the maximum amplitude of motions.

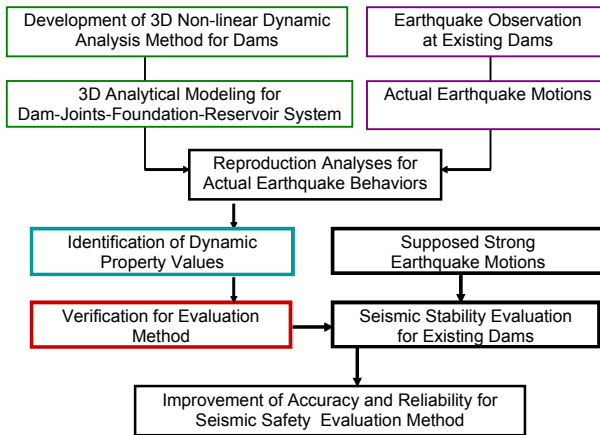


Figure 2: Verification and improvement for 3-D dynamic analysis method.

4 Reproduction analyses for existing concrete dams

4.1 Existing dams analyzed

The 3-D reproduction analyses were made about the Nukabira Dam [4] (hereafter the NK Dam) during the 1993 Kushiro-oki Earthquake, the Shintoyone Dam [5] (hereafter the ST Dam) during the 1997 near-field earthquake, the Ikehara Dam [6] (hereafter the IK Dam) during the 1995 Hyogoken-nanbu Earthquake, and the Tagokura Dam (hereafter the TG Dam) during the 2004 Niigataken-chuetsu Earthquake. By these reproduction analyses, the dynamic shear modulus and the damping factor were evaluated quantitatively, and the efficiency and validity of the 3-D dynamic analysis method proposed was proved finally.



4.2 Transformation of motions from the observed point to the input boundary

In regard to 3-D reproduction analysis, it is necessary to transform the motions observed at the dam into the input motions at the bottom boundary of the 3-D analytical model. I have devised the procedure for transforming the observed motions into the input motions. The input motions at the bottom boundary can be regenerated by utilizing the transfer function between the earthquake observation point and the bottom boundary of the model, as shown in fig. 3. In the de-convolution, each component of motions is converted one by one. And, in the 3-D reproduction analysis, three components of motions are input simultaneously.

4.3 Reproduction analysis for the TG Dam

The shape of the TG Dam and the arrangement of the seismometers are shown in fig. 4. The 3-D analytical model for the TG Dam is shown in fig. 5. The model was made as the 3-D coupled dam–foundation–reservoir system. The dam and foundation is meshed with the finite elements, and the reservoir is meshed with the finite difference grids. As for the boundary conditions, the rigid boundary is applied for the bottom boundary, and the viscous boundary is applied for the lateral boundaries. The water depth of the reservoir was set to be the same condition when the earthquake occurred.

The dynamic property values of the TG Dam identified by the 3-D reproduction analysis for actual earthquake behavior are shown in table 1.

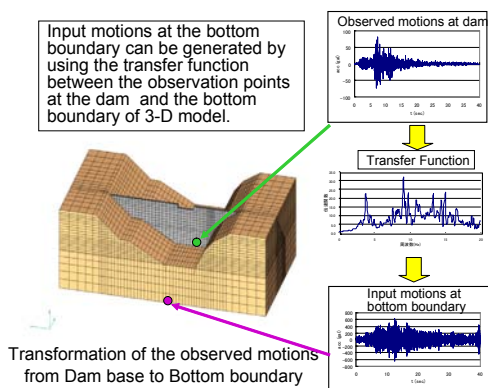


Figure 3: Transform of motions from the observed point to the input boundary.



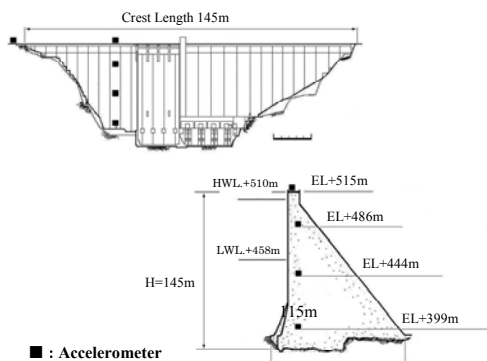
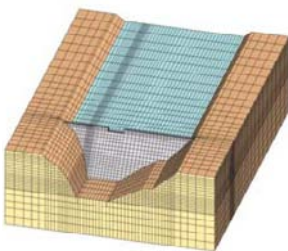


Figure 4: Location of seismometers at the TG Dam.



TG Dam : Concrete Gravity Dam
Dam height: 145m, Crest length: 462m
Dam volume: 1950000m³

Figure 5: 3-D analytical model.

Table 1: Dynamic property values identified for the TG Dam.

Item	Density	Poisson's ratio	Dynamic shear modulus	Shear wave Velocity	Damping Factor
Dam	2.4 g/cm ³	0.20	9600 N/mm ²	1980 m/s	5.0%
Rock	2.6 g/cm ³	0.25	8000 N/mm ²	1740 m/s	5.0%

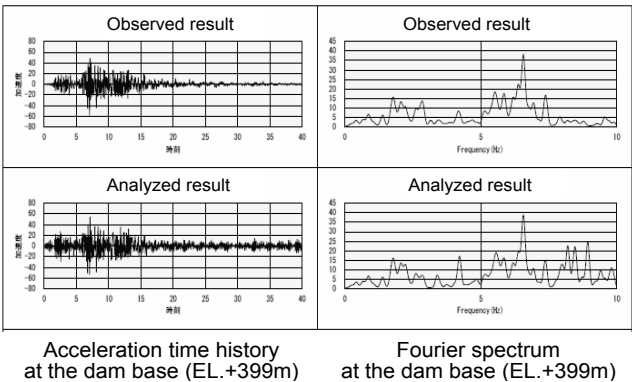


Figure 6: Comparison of acceleration time history at the dam base of the TG Dam.

The comparison between the observed results and the reproduction analysis results regarding the acceleration time history and the Fourier spectra at the dam base (EL.+399m) of the TG Dam is shown in fig. 6. Similarly, the comparison



about the acceleration time history at the dam crest is shown in fig. 7. As for the dam base (EL.+399m), the reproduction analysis results agree with the observation results well. As for the dam crest, the reproduction analysis results became slightly larger than the observation results.

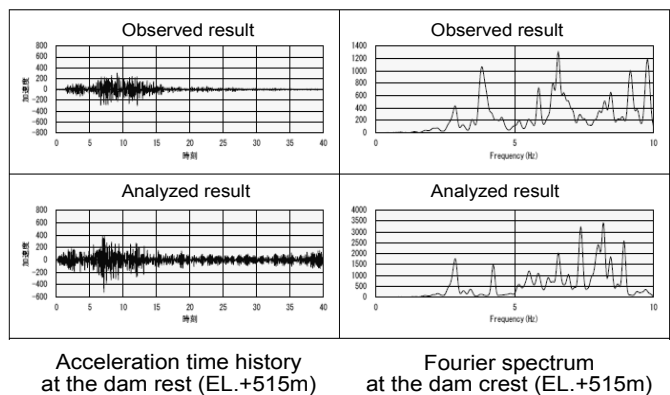


Figure 7: Comparison of acceleration time history at the crest of the TG Dam.

4.4 Reproduction analysis for the IK Dam

The shape of the IK Dam and the arrangement of the seismometers are shown in fig. 8. The 3-D analytical model for the IK Dam is shown in fig. 9. The dynamic property values of the IK Dam identified by the 3-D reproduction analysis are shown in table 2.

As the representative results, the comparison between the observed result and the analyzed result about the acceleration time history at the crest center is shown in fig. 10.

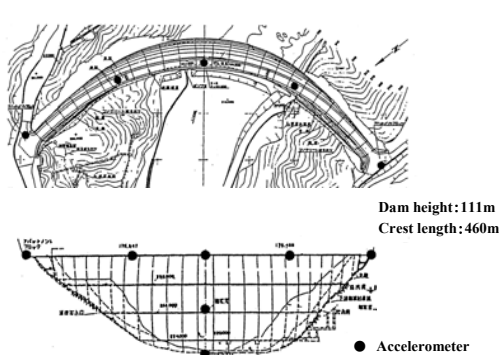


Figure 8: Location of seismometers at the IK Dam.

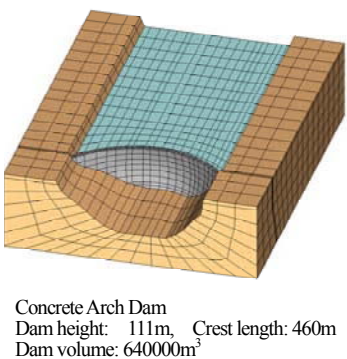


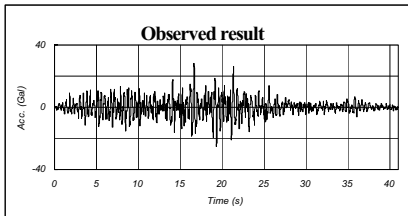
Figure 9: 3-D analytical model.



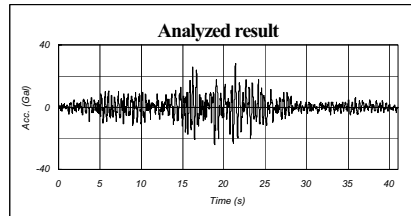
Table 2: Dynamic property values identified for the IK Dam.

Item	Density	Poisson's ratio	Dynamic shear modulus	Shear wave Velocity	Damping Factor
Dam	2.3 g/cm ³	0.20	13500 N/mm ²	2400 m/s	2.9%
Rock	2.55 g/cm ³	0.25	11700 N/mm ²	2120 m/s	4.0%

The acceleration time history at the dam can be reproduced well. The damping factor of the IK Dam was supposed according to the time of the motion, namely the damping factor for the time of 0 - 13 (sec) was set to be 2.6%, 2.9% for 13 - 20 (sec), and 3.1% for 20 - 41 (sec). It is considered that the reproducibility can be improved by applying this procedure. As for the waveform of the time history, a peculiar difference between the observed results and the analyzed results was not recognized.



(1) Observed time-history



(2) Analyzed time-history

Figure 10: Comparison of acceleration time-history at the crest center of the IK Dam. (a) Observed history, (b) analyzed time-history.

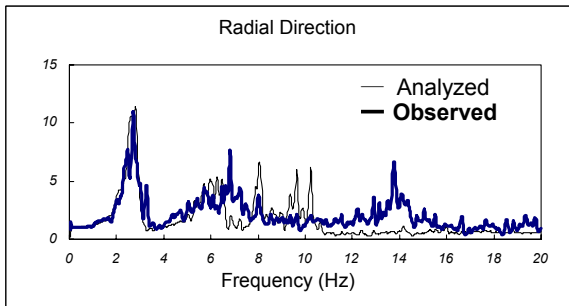


Figure 11: Comparison of transfer function in the radial direction between the dam center and the dam base of the IK Dam.

The comparison in regard to the spectral function, or the ratio of Fourier spectrum between the crest center and the dam base) is show in fig. 11. In regard to the frequency domain lower than 4 Hz, especially as for the natural frequency



(2.8 Hz) of the IK Dam, the analyzed result agreed with the observed result comparatively well.

4.5 Dynamic property values identified by the reproduction analyses

Table 3 shows the dynamic property values identified by the 3-D reproduction analyses. As the results, the S-wave velocity of the TG Dam, the IK Dam, the NK Dam and the ST Dam were evaluated to be 1980 m/s, 2400 m/s, 2120 m/s and 2110 m/s, respectively. Similarly, the damping factor of the TG Dam, the IK Dam, the NK Dam and the ST Dam were evaluated to be 5%, 2.9%, 5%, and 5%, respectively. In these cases, as the amplitude of earthquake motions are not so large, the actual earthquake behaviors can be reproduced by the linear analysis. In case of very strong earthquake motions, the nonlinear dynamic analysis taking the non-linearity of material will be required.

Table 3: Dynamic property values identified by the reproduction analyses.

Dam Type		Concrete Gravity Dam		Concrete Arch Dam	
Dam Name		Tagokura	Numabira	Ikehara	Shintoyone
Earthquake	Date	2004.10.23	1993.1.15	1995.1.17	1997.3.16
	Name	Niigataken-Chuetsu	Kushiro-oki	Hyogoken-nanbu	Near Toyohashi
	Magnitude	M6.8	M7.8	M7.2	M5.8
	Epicenter Distance	37 km	110 km	106 km	35 km
D A M	Density	2.4 g/cm ³	2.4 g/cm ³	2.3 g/cm ³	2.35 g/cm ³
	Dynamic Shear Modulus	9,600 N/mm ²	11,032 N/mm ²	13,500 N/mm ²	10,700 N/mm ²
	S-wave Velocity	1980 m/s	2120 m/s	2400 m/s	2110 m/s
	Damping Factor	5 %	5 %	2.9 %	5 %
	Max. Acc. at Dam Crest	454.9 gal	77.4 gal	82.3 gal	709 gal
	Max. Acc. at Dam Base	102.5 gal	27.5 gal	11.6 gal	68.5 gal
	Natural Frequency	3.9 Hz	5.2 Hz	2.8 Hz	5.2 Hz
	Dam Height	145 m	76 m	111 m	116.5 m
	Crest Length	462 m	293 m	460 m	311 m
B A S E	Density	2.6 g/cm ³	2.6 g/cm ³	2.55 g/cm ³	2.60 g/cm ³
	Dynamic Shear Modulus	8,000 N/mm ²	9,380 N/mm ²	11,700 N/mm ²	9,600 N/mm ²
	S-wave Velocity	1740 m/s	1880 m/s	2120 m/s	1900 m/s
	Damping Factor	5 %	5 %	4.0 %	5 %

The damping factor described here means a material damping factor, that is a hysteretic damping, because the radiation of wave energy from the boundary of foundation to the free field can be naturally considered in the 3-D dynamic analysis. If the radiation damping from the foundation to the free field is not considered, an additional damping factor should be taken into account.

It is considered that the values of the S-wave velocity and the damping factor were slightly changed according to the dam, because of the differences about the shape and size of dam, the acceleration level of earthquake motion, the dynamic interaction between dam and foundation.



In executing the 3-D reproduction analyses, it was comparatively easy to reproduce in regard to the NK Dam, the ST Dam and the IK Dam. However, it was comparatively hard to reproduce for the TG Dam. It is considered that such difference may be caused by the appropriateness of the location and arrangement of seismometers.

5 Conclusions

In order to realize an accurate and reliable evaluation for seismic safety of existing dams, a dynamic interaction between dam and foundation, a reduction effect on a dynamic response of dam by reservoir water, a radiation of wave energy from the boundary of foundation to the free field, a non-linear effect of dam material, a discontinuous behaviors of contraction joints and peripheral joints against very strong earthquake motions, and so forth should be considered quantitatively and properly.

Taking these matters into account, I have developed a 3-D nonlinear dynamic analysis method for a coupled dam – joints – foundation – reservoir system.

The dynamic deformation property values have significant effects on the dynamic stresses and strains calculated by the dynamic analysis, so the values of the dynamic shear modulus and the damping factor should be quantitatively evaluated based on the actual earthquake motions.

An efficiency and validity of the dynamic analysis procedure be verified based on the actual earthquake phenomena.

I have made the 3-D reproduction analyses for actual earthquake behaviors of the NK Dam, the IK Dam, the ST Dam and the TG Dam, and evaluated the values of dynamic shear modulus and the damping factor of these dams quantitatively and practically.

And I have verified the efficiency and validity of the 3-D non-linear dynamic analysis method which I have developed in this study based on these reproduction analyses.

When the acceleration level of earthquake motion is not so large, the earthquake behavior can be reproduced by the linear dynamic analysis. But, when the acceleration level is very large, the nonlinear dynamic analysis taking not only the non-linearity of dam material but also the discontinuous effects of joints will be required [7].

The 3-D dynamic analysis method is necessary to evaluate the earthquake safety quantitatively. If the earthquake observation data are obtained, the 3-D reproduction analysis for the actual earthquake behavior is effective to verify the validity of the dynamic analysis method [8].

In order to improve the disaster prevention performance of existing dams, the feedback of seismic safety evaluation to the earthquake countermeasures is necessary. A smooth and quick confirmation of dam safety will be strongly required after very large earthquake. The organic fusion of the earthquake observation data and the 3-D dynamic analysis enables to produce new information which is useful for the earthquake disaster prevention and mitigation [9].



References

- [1] Ariga, Y., 2001. Study on quantitative evaluation of dynamic property of dams by 3-D reproduction analyses, Thesis for doctorate of Saitama University
- [2] Miura, F., H. Okinaka, 1989. Dynamic analysis method for 3-D soil-structure interaction systems with the viscous boundary based on the principle of virtual work, Proc. of JSCE, No.404/I-11, pp.395-404
- [3] Hatano, T., 1968. Theory of Failure of Concrete and Similar Brittle Solid on the Basis of Strain, Proc. of JSCE, No.153 , pp.31-39
- [4] Ariga Y., S. Tsunoda, H. Asaka, 2000. Determination of dynamic properties of existing concrete gravity dam based on actual earthquake motions, The 12th World Conference on Earthquake Engineering (12WCEE), No.334, p.1-8.
- [5] Ariga Y., Cao Z., Watanabe H., 2003. Seismic Stability Assessment of An Existing Arch Dam Considering the Effects of Joints, Proceedings of the 21st International Congress on Large Dams, Q.83-R.33, p.553-576.
- [6] Ariga Y., H. Watanabe, 2004. Reproduction Analysis of Real Behavior of Existing Arch Dam during the 1995 Hyogoken-Nanbu Earthquake, The 13th World Conference on Earthquake Engineering (13WCEE), No.405, p.1-10.
- [7] Ariga, Y., Quantitative evaluation method for dynamic tensile strength of dam concrete by combining shaking table test and 3-D reproduction analysis, Proceedings of the First International Conference on Advances in Experimental Structural Engineering(AESE2005), Vol.1, pp.409-416.
- [8] Ariga, Y., 2006. Verification of 3-D seismic safety evaluation method for existing dams by reproduction analysis for actual earthquake behavior, First European Conference on Earthquake Engineering and Seismology (1st ECEES), No.1214, pp.409-416
- [9] Ariga, Y., Y. Fujinawa, and M. Hori, 2006. Development of immediate evaluation method for earthquake safety of existing dams, 100th Anniversary Earthquake Conference – commemorating the 1906 San Francisco Earthquake, No.196, pp.1-11.



Seismic hazard expression in risk assessment

X.-X. Tao^{1,2}, Z.-R. Tao² & P. Li¹

¹*Harbin Institute of Technology, People's Republic of China*

²*Institute of Engineering Mechanics,
China Earthquake Administration, People's Republic of China*

Abstract

Seismic risk assessment, consisting of seismic hazard analysis and vulnerability evaluation, is really a very comprehensive assessment as the first and fundamental step in the earthquake disaster prevention process. In China, seismic hazards and vulnerabilities of many cities have been assessed in the last dozen years. In general, the assessments are completed separately and are combined together for loss estimation. A contrast in the final estimation is pointed out in this paper. The seismic hazard is expressed as the exceeding or occurring probability of an earthquake action, such as intensity I or acceleration A . It is an integrative effect from all earthquakes in surrounding potential source areas with various magnitudes. The vulnerability is expressed by the probabilities of damage states of structures given the earthquake action. The loss is estimated through combining this hazard, vulnerability, and loss rates of all damage states. However, it is impossible that all of the earthquakes occur at the same time, and the high-intensity area can never cover the whole metropolis. To make it clear, a numerical example is presented in the paper. The scenario earthquake method is a solution, which is consistent with the regional seismic environment and is determined from the regional attenuation relationship of ground motion. The caused shaking, damage and loss distribution of population, buildings and infrastructures can be further estimated easily. In this way, the overestimation of loss in metropolis is avoided. A case study is demonstrated in this paper as an example.

Keywords: *scenario earthquake, seismic hazard, losses evaluation.*



1 Introduction

Seismic risk is the possibility or chance of impact to human community due to occurrence of an earthquake, including damage, economic loss, victims and injuries, and the gross of the loss, thus risk statements must be given in quantitative terms, Franz Sauter [1]. It describes the effects and consequences of a devastating earthquake in details to help designers, managers and rescuers plan earthquake prevention and reduction operations. As well known, seismic risk assessment consists of seismic hazard analysis and vulnerability evaluation. In general, the two parts are completed by seismological team and engineering team separately. To combine these two together, seismic hazard expression must be improved, especially for application in financial instrument such as insurance, cat bond. Some suggestions are presented in this paper.

2 Seismic hazard curve

In the seismic risk assessment procedure adopted for many cities of China, the expected total loss of buildings in a given time period can be calculated by:

$$E(L) = \sum_S \sum_{k=1}^5 P_S(D_k)(L_S(D_k) + W_S(D_k)) \quad (1)$$

where, k is for the 5 damage states (None, Slight, Moderate, Extensive and Complete); $L_S(D_k)$ is the loss of S^{th} type buildings being in k^{th} damage state; $W_S(D_k)$ is the indoor property loss of S^{th} type buildings being in k^{th} damage state; $P_S(D_k)$ is the probability of S^{th} type buildings being in k^{th} damage state. The former two depend on the corresponding unit cost, unit indoor property and the total construction area of S^{th} type buildings, and loss ratio of S^{th} type buildings in k^{th} damage state. The latter is referred to as the engineering seismic risk, can be described as eq. (2).

$$P_S(D_k) = \sum_{I=6}^9 P_S(D_k|I) \cdot P(I) \quad (2)$$

where, $P_S(D_k|I)$ is the conditional probability of S^{th} type buildings being in k^{th} damage state given intensity I , so-called vulnerability of S^{th} type buildings, which is evaluated from damage experience, analysis of a certain number of buildings and sometimes from results of experiments; $P(I)$ is the possibility of intensity I occurrence, and can be derived from $P(I \geq i)$, so-called seismic hazard. The latter depends on the regional seismic environment and attenuation relationship of ground motion, and is generally referred to as seismic hazard curve. In nature, earthquake intensity is a sequential classified variable, so $P(I \geq i)$ is not really a continuous curve. Of course, intensity I in eq. (2) can be



substituted with other ground motion parameter Y , by this way, the symbol “ Σ ” can be changed into “ $\int$ ”, then $P(Y > y)$ is a continuous curve. Intensity I is preferred to other parameters for vulnerability evaluation, since matrix of $P_S(D_k|I)$ is mainly from statistical data of earthquake damages in the past, in which earthquake intensity is an essential parameter.

3 Seismic hazard underestimation at low intensity range

In general, $P(I)$ is calculated by:

$$P(I) = P(I \geq i) - P(I \geq i + 1) \quad (3)$$

However, the result of the subtraction will not be reasonable, if I is low. For example, the result of case study in a region with low seismicity is shown in Table 1. One can see from the table there must be something wrong, since $P_{50}(I = 3)$ is less than $P_{50}(I = 4)$.

Table 1: The occurring probability of low intensity in 50 years.

Intensity	III	IV	V	VI
$P_{50}(I \geq i)$	0.9289	0.6627	0.2855	0.0583
$P_{50}(I = i)$	0.2662	0.3772	0.2272	0.0533

In nature, $P(I)$ must be a monotone decreasing function, i.e. $P(I)$ must be greater than $P(I + 1)$, even if intensity is low. Deal with the procedure of seismic hazard, one can understand that $P(I \geq i)$ is contributed by earthquakes in many potential source areas with various magnitudes and various occurring times. It means that $P(I \geq i)$ consists of not only $P(I = i)$ and $P(I \geq I + 1)$, but also $P(I = i \text{ and } I > i)$. The later cannot be ignored in seismic zone with strong activity for intensity less than VII and the evaluated period is long, like 50 years or 100 years. The general seismic hazard assessment assumes that the occurrence of earthquake is independent each other, therefore a solution is that the exceeding probability in a short time period t can be firstly calculated from the hazard in long period T as follows.

$$P_t(I \geq i) = 1 - [1 - P_T(I \geq i)]^{\frac{t}{T}} \quad (4)$$

Obviously, $P(I = i \text{ and } I > i)$ can be ignored when the period is short enough. In general, one month is short enough for a region with generic seismic



activity; it should be shorten to days for region with high activity. By this way, $P_t(I = i)$ can be calculated by eq. (5).

$$P_t(I = i) = P_t(I \geq i) - P_t(I \geq i + 1) \tag{5}$$

The occurring probability of intensity I in long time period T , can be obtained in reverse of eq. (4).

$$P_T(I = i) = 1 - [1 - P_t(I = i)]^{\frac{T}{t}} \tag{6}$$

The result of the case study is improved as shown in Table 2. One can see from the table that $P(I = i)$ is now monotone decreasing, and the probabilities in Table 1 are underestimated very much for intensity III and IV, also underestimated for intensity V, even VI. The similar underestimation may happen for higher intensity in region with high seismicity.

Table 2: The occurring probability of low intensity in 50 years.

Intensity	III	IV	V	VI
$P_{50}(I = i)$	0.7899	0.5277	0.2404	0.0535

4 Seismic hazard overestimation at high intensity range

In general, high intensity area is not very large, since the fast attenuation at epicentre area. Following is a set of intensity attenuation relationships for North China.

$$\begin{aligned} I &= 6.046 + 1.480M - 4.792 \log(R_a + 25), & \sigma_a &= 0.49 \\ I &= 2.617 + 1.435M - 3.318 \log(R_b + 7), & \sigma_b &= 0.56 \end{aligned} \tag{7}$$

where I is intensity, are distances along major and minor axes respectively, σ_a and σ_b are regression variances of the two formulas, M is magnitude. For $M=6, 7, 8$ and $I=VIII, IX, X$, R_a, R_b and the areas of the ellipses corresponding to them can be calculated, and listed in Table 3.

Table 3: R_a, R_b and the corresponding areas for given M and I .

Intensity Magnitude	VIII	IX	X
6	2.9/2.4/22	-	-
7	31.8/18.4/1837	10.1/5.7/181	-
8	90.6/61.8/17582	46.5/27.4/3997	19.2/10.2/614

There are three numbers in each grid, the left is for R_a in km, the middle is for R_b in km, and the right is for the area in km^2 .

The area of the whole city in above case study is 22161km^2 , none of high intensity areas in the table reaches this value, i.e. never cover the whole city, even intensity VIII from an earthquake with very infrequent magnitude 8. In conclusion, $P(I)$ in eq. (2) cannot be adopted in risk assessment for the occurring probability of I to the whole city, otherwise the expected loss must be overestimated. The authors believe a solution is scenario earthquake method.

5 A hazard expression - scenario earthquake

Exceeding probability, hazard curve are not familiar to public, not only the original people, but also engineers, planners and decision makers. To express seismic hazard clearly to public, scenario earthquake has been adopted since 1990s. It is consistent with the regional seismic environment and is determined from the regional attenuation relationship of ground motion. By means of the procedure developed by the authors, the scenario earthquake can cause the same intensity with the intensity on the hazard curve with given exceeding probability, with a magnitude less than the upper bound magnitude of the potential source area with most contribution to the probability, and a distance comparative with the potential source area.

The potential source areas in the case study are shown in fig.1.

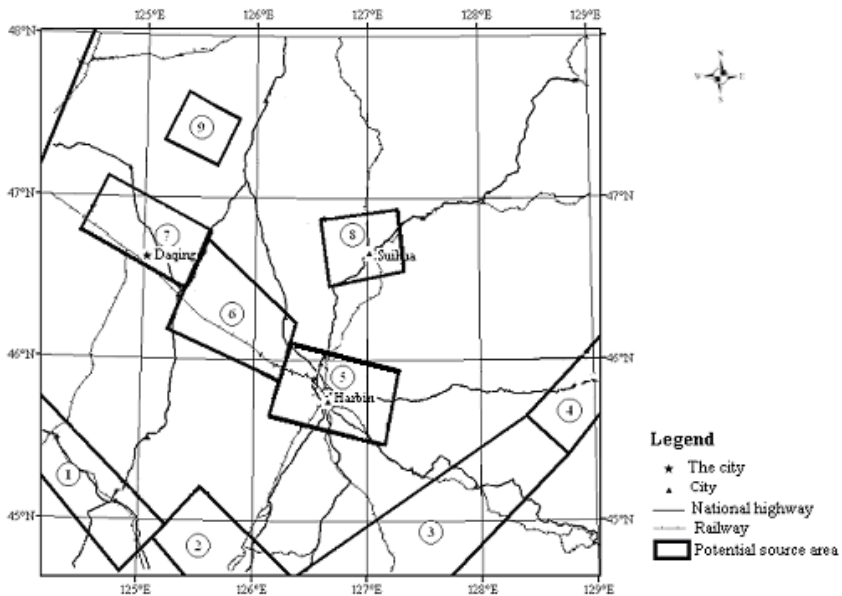


Figure 1: Potential source areas and scenario earthquake of the case study.



GIS based systems for damage evaluation are developed for many cities and regions worldwide. The system is a powerful tool to perform spatial analysis and mapping of losses due to a scenario earthquake. The damaged areas of buildings, damaged length of highways, bridges or pipelines, damaged number of electric power or communicate facilities in 5 damage states from scenario earthquake can be assessed very fast. Damage results of the case study from scenario earthquakes with magnitudes 6 and 5 respectively corresponding occurring probabilities 0.005 and 0.0000426 in 50 years, are shown in fig. 2 and fig. 3. In each figure, (a) is for building damage, (b) for road network damage, (c) for electric power system damage, and (d) for water supply system, respectively.

From the figures, one can see that the highest intensities are VII and VIII, but the areas with these intensities are very limited. Furthermore, the system can estimate the gross loss, death and injuries, and their spatial distribution quickly by spatial operating capacity of GIS. The results of this case are listed in table 4.

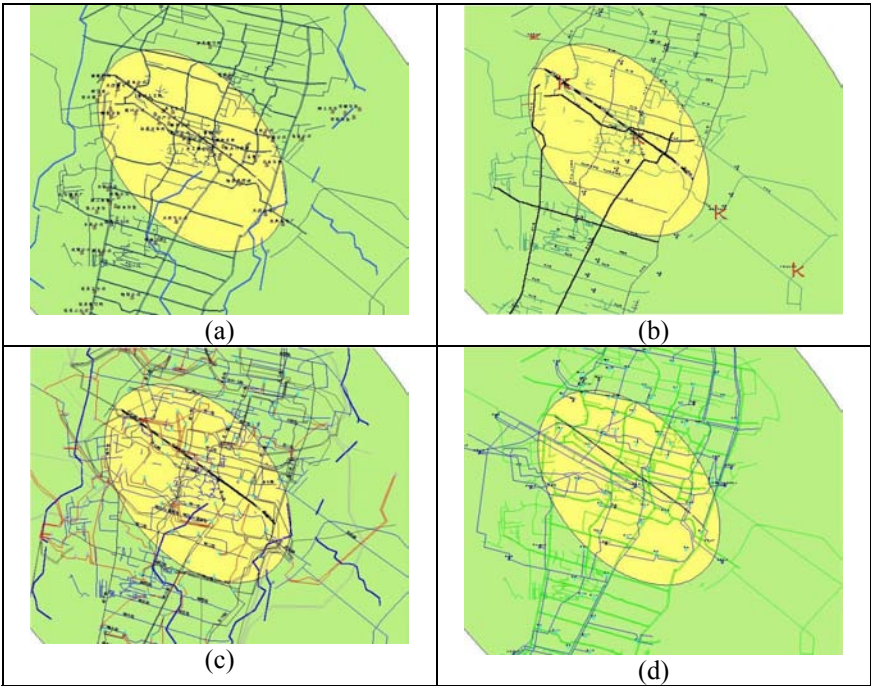


Figure 2: Damage caused by an earthquake with magnitude 5 in the case study.

Table 4: Gross loss, death, injuries and homeless.

Scenario E.Q.	Loss (Million RMB)	Death	Injuries	Homeless
5.0	48.1	0	0	509
6.0	1124.8	101	404	94709



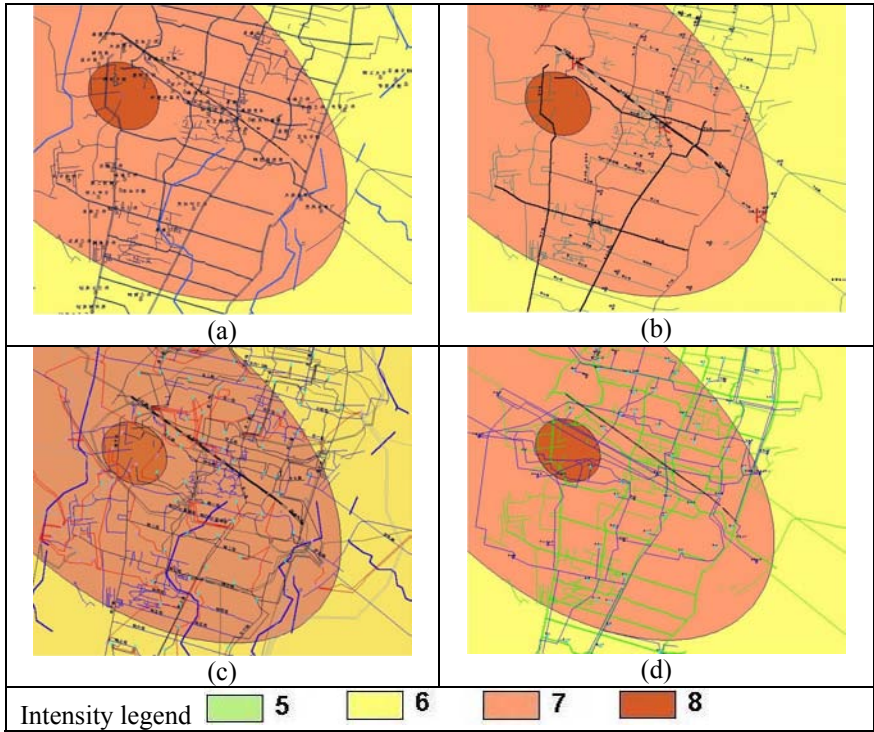


Figure 3: Damage caused by an earthquake with magnitude 6 in the case study.

6 Conclusions

Two problems in seismic hazard expression are pointed out in this paper for risk assessment. One at low intensity range corrected by formula (4) to (6). Another is solved by scenario earthquake method. The latter must be assisted by a GIS base system. For example, the damage estimation of buildings, road network system, electric power system, water supply system are shown for a case study, and then the gross loss, death, injuries and homeless are also listed for demonstration.

Acknowledgements

This research is supported partly by the Earthquake Science Foundation under Contract No.606027, the Heilongjiang Natural Science Foundation under Contract No.G2005-13, and by the Science Foundation (No. TDXX-0504) from the Key Lab. on Information Science and Engineering of the Chinese Railway Ministry.



References

- [1] Franz Sauter, EERI Committee on Seismic Risk, Glossary of terms for probabilistic seismic-risk and hazard analysis. *Earthquake Spectra*, 1, pp.33–40, 1984.
- [2] Anton Zaicenco and Vasile Alkaz, Urban Seismic Risk Studies with Utilization of GIS. *Presentation on NATO Advanced Research Workshop*, 2005.
- [3] B. Tucker, et al., The Quito, Ecuador, earthquake risk management project: An evaluation, *Proc. of 5th Int. conf. on seismic zonation*, pp178-1804, 1995, Nice.
- [4] R. K. McGuire, Scenario earthquake for loss studies based on risk analysis, *Proc. of 5th Int. conf. on seismic zonation*, pp1325-1333, 1995, Nice.
- [5] M. Erdik and J. Swift-Avci, Development of earthquake hazard and damage scenarios, *Proc. of 5th Int. conf. on seismic zonation*, pp2153-2165, 1995, Nice.
- [6] L. Xie, X. Tao et al. A GIS based earthquake losses assessment and emergency response system for Daqing oil field, *Proc. of 12th World Conf. on earthquake engineering*, Paper No. 0091, 2000, Auckland.



Section 8

Lifelines

This page intentionally left blank

Seismic reliability and cost evaluation for a hospital lifeline network system

K. Fuchida

*Department of Civil & Architectural Engineering,
Yatsushiro College of Technology, Kunamoto, Japan*

Abstract

This paper presents an evaluating method for seismic reliability and costs of a lifesaving lifeline connecting to a hospital during an earthquake. By using the reliability analysis method for a lifeline network based on the Monte Carlo method, reliability of a virtual model and a real one of Yatsushiro-city water supply pipeline network is evaluated. Numerical computations are performed for models with various conditions of ground and pipes. As a result, in a case of seismic intensity 5, the reliability of a pipeline connecting to a hospital near the sea side is low. In a case of seismic intensity 6, all nodes connecting to hospitals have very low reliability and this suggests that other methods of preparing for earthquakes are needed except improving method of conditions of grounds or pipes.

Keywords: lifeline, reliability, network, cost.

1 Introduction

Severe damages of lifeline systems during earthquake much affect the social activity and urban life of citizens. It is very important that aseismic investment for lifesaving lifeline systems is performed in view of fast recovery of economical and social works in urban area after earthquakes.

This study aims to investigate the seismic reliability and costs of a lifesaving lifeline connecting to a hospital during earthquake. The cost is evaluated by the sum of aseismic investment (mainly pipeline reinforcement and ground improvement), restoration cost and system-down cost with the system reliability, and then discuss the effective investment by minimizing the total cost of every pre-investment and post-repairing cost. For the assumption model of water



supply pipeline network and real one in Yatsushiro city, the relations between reliabilities and conditions of pipeline and ground are investigated.

2 Analytical method

The flow of the reliability analysis in this study is shown in Figure 1(Akiyoshi [1]). In the first step, the network of water supply pipelines is modeled into the node-link discrete system and the failure probability of each link is evaluated by the Quantification theory I. The seismic intensity distribution for future earthquakes in the objective area is induced by using the past occurrence records of earthquakes in the respective area. The probability of the occurrence of earthquakes within a period is evaluated.

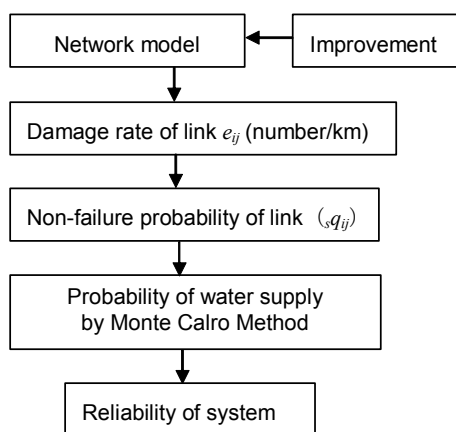


Figure 1: Flow of reliability analysis.

The failure of pipeline is dependent on the seismic intensity, the ground characteristics and the pipeline characteristics. By unifying the failure data of past earthquakes, which are Miyagi-ken-oki (1978), Nihonkai-chubu (1983), Kushiro-oki (1993) and Hyogo-ken-nanbu (1995), with the Quantification theory I, the equation of regression for the failure ratio of water supply pipeline is derived (Kubo [2] and Hino [3]) in terms of the category weights which are shown in Table 1. In this process the occurrence of liquefaction is estimated by the method of JRA [4].

In the second step of this analysis, the reliability for each link of a water supply system is computed by Monte Carlo simulation technique (Tamura [5]) as the corresponding failure probability which is based on the Quantification theory I, in terms of earthquake intensity, stiffness of ground, scale of liquefaction and type of pipes. In this process, the connectivity of the system is defined as the rate of existing path among the pair of nodes, and the probability of system connectivity is also evaluated by the Monte Carlo simulation.



Table 1: Results of quantification theory analysis.

Item	Category	Category weight
Seismic intensity	V (80~250gal)	0.301
	VI (250~400gal)	2.331
	VII (>400gal)	23.292
Ground type	1	1.000
	2	12.610
	3	12.800
Liquefaction	Remarkable	1.000
	Slightly	0.732
	None	0.137
pipeline type	Cast iron	0.517
	Ductile cast iron	0.065
	Steel	0.045
	Asbestos cement	0.676
	PVP	0.046
Diameter of pipeline	50~125mm	1.000
	150~300mm	0.446
	350~600mm	0.052
	>700mm	0.012

In the next step of this study, the effects of aseismic investment on the reliability of the lifeline system, the loss of damage and the restoration cost of the system are investigated as the flow shown in Figure 2. Aseismic investment for the lifeline is assumed to include the initial cost for strengthening of pipes and liquefaction-preventing ground improvement. In this study the total cost (TC:in eq.(1)) is used as the index for optimal aseismic investment, where the total cost is expressed as the sum of the restoration cost (RES), the industrial loss due to system-down (SDC) and the reinforcement cost (RC) as

$$E(TC) = E(RES) + E(SDC) + RC \quad (1)$$

where

$$E(RES) = \sum_{i=1}^n DN_i \times RES_i \quad (2)$$

$$E(SDC) = CP \times PROB_i(SW) \times DAY_i \quad (3)$$

$$RC = \sum_{i=1}^n RP_i + \sum_{i=1}^n RG_i \quad (4)$$

and $E()$ means expected value, DN_i : number of failures of i-th damaged link, RES_i : restoration cost of i-th reinforced link, CP : total cost of industrial



production per day in Yatsushiro city, $PROB_i(SW)$: probability of suspension of water of i-th reinforced link, DAY_i : dates for restoration of i-th reinforced link, RP_i and RG_i : reinforcement cost of pipeline and ground, respectively. The unit prices and computations used here are followings (Takada [6]):

- (1) improvement of ground (SCP): 3000(yen) / m (depth) / (number of pile)
- (2) improvement of pipeline: 100000(yen) / m (length)
- (3) restoration cost: 10million (yen) / number of restoration
- (4) industrial production: 1.4 billion (yen) / day
- (5) water charge: 40 (yen) / person / day
- (6) water supply population: 36800 people

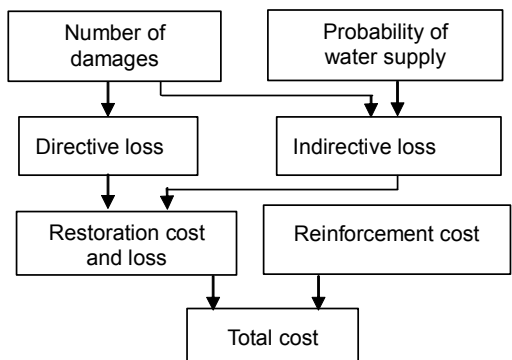


Figure 2: Flow of cost evaluation.

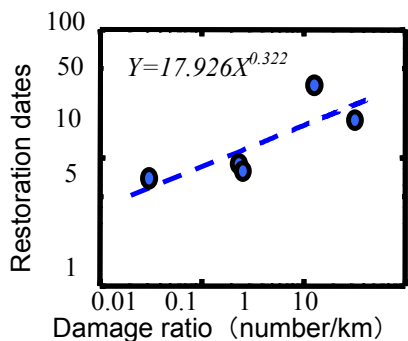


Figure 3: Relation between restoration rate and damage ratio.

The dates of restoration are related by the regression analysis with damage ratio as shown in Figure 3 (Hino [3]). The order of priority for aseismic reinforcement is determined by the algorithm of the shortest-path assignment based on the Dijkstra's method (Civil Planning Research Committee [7]). Two patterns of the restoration strategy are assumed for performing the aseismic reinforcement, in which one is in order of least costed links and the other is in order of much damaged links.



3 Results and considerations

Numerical computations for the aseismic investment and the reliability for the lifeline systems are conducted for the water supply pipeline system models in assumption case and Yatsushiro city.

Figure 4 shows the assumption model of water supply pipeline system for evaluation the relations between the seismic reliability and aseismic conditions. This area is divided to the mesh of 0.5km x 0.5km and the conditions of pipes and the ground in one mesh are assumed to be same. Two types of the aseismic reinforcement are assumed, in which one is the reinforcement of pipeline segment and the other is the reinforcement of the ground by improvement method against liquefaction.

Figure 5 and Figure 6 show the relations between reliability and improvement numbers of pipe and ground, for respectively. In both Figures 5 and 6, the initial conditions of the grounds and pipes are seismic intensity 6, ground classification type 2, slightly liquefaction, cast iron pipe and diameter of pipe 150mm. In the case of improvement of pipe type from cast iron pipe to steel one in Figure 5, reliability increases from 0.2 to 0.5. In the case of improvement of the ground from type 2 to type 3 in Figure 6, the reliability of the system increases from 0.2 to 0.9. The result of Fig.6 means that the ground improvement is effective for increasing the reliability of the system.

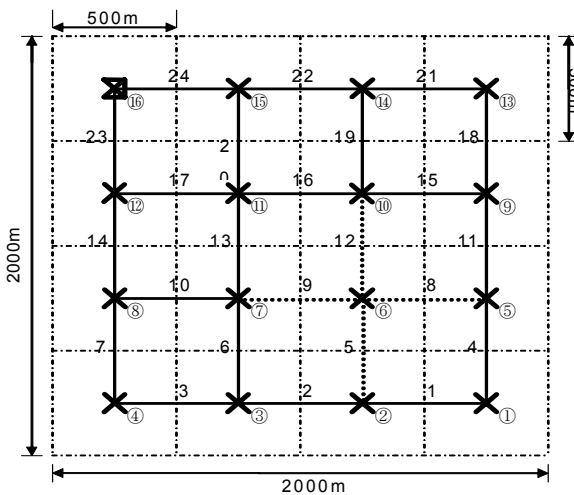


Figure 4: Assumption model of network.

Figure 7 shows the relation between the cost and reliability (the water supply rate) as the corresponding failure probability within 100 years. In Figure 7 it needs the total cost about 1.6 billion Yen for the water supply rate of about 50%, which means that the aseismic investment is not effective.



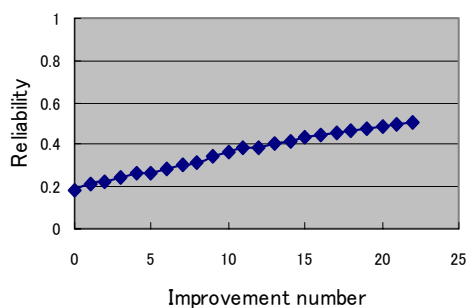


Figure 5: Reliability versus improvement number of pipe.

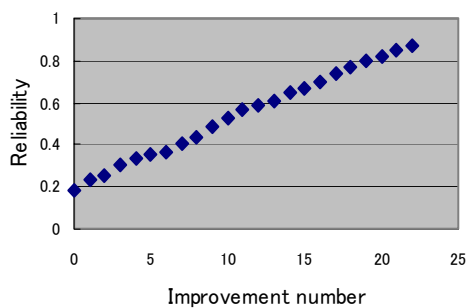


Figure 6: Reliability versus improvement number of ground.

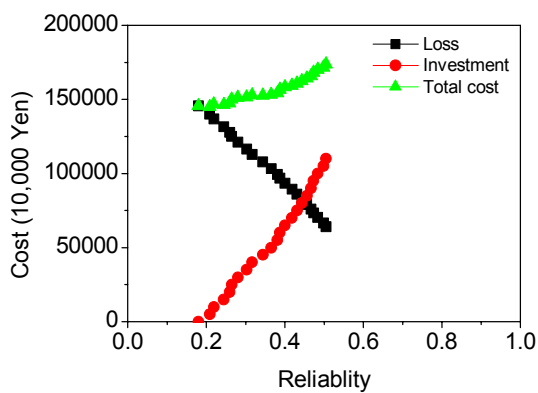


Figure 7: Relation between cost and reliability.



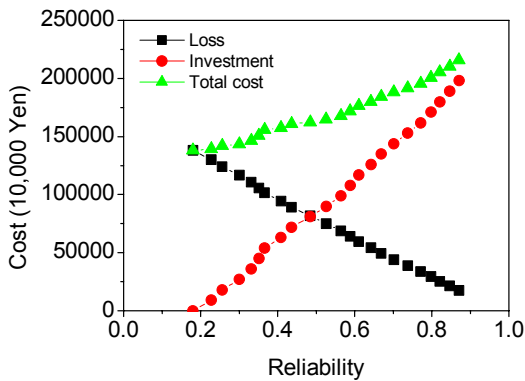


Figure 8: Relation between cost and reliability.

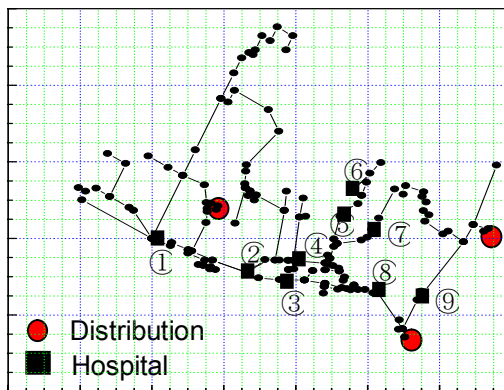


Figure 9: Water supply model of Yatsushiro city.

Figure 8 shows the relations between the total cost and reliability (water supply rate) for the case of improvement of ground type as same as in the case of Figure 6. The total cost increases with reliability and maximum cost reaches about 2.1 billion Yen for the reliability about 0.9, which means that a little investment is effective for increasing the water supply rate.

Figure 9 shows the water supply pipeline model in Yatsushiro city in which this area expanded in 12km (EW direction) $\times$ 8km (NS direction), and the pipeline system is modeled for the distribution lines of the diameter larger than 50mm as 146 links, 138 nodes and 3 distribution basins. This area is divided to the mesh of 500m $\times$ 500m. The conditions of the ground in one mesh are assumed to be same.



Table 2 shows the reliabilities of nodes which are connected to the nine hospitals in Yatsushiro city for the cases of seismic intensity 5 and 6. In Table 2, the reliability means the water supply rate from the basin to the node. The average value of the reliability in whole Yatsushiro city is about 0.787 and 0.203 for the seismic intensity 5 and 6, respectively.

In the case of seismic intensity 5 the reliability 0.642 of the hospital A (No.1) is lower than the averaged value of Yatsushiro city, which means that the hospital A locates near the shore line and the ground is possible to be liquefied. The reliabilities of the hospital G and H which locate near the mountain are high. In the case of seismic intensity 6 the reliabilities of the almost hospitals are very low values. This suggests that another countermeasure is needed except physical ones like reinforcement of pipeline and ground improvement.

Table 2: Reliability of hospital lifeline.

Hospital	Link	Node	Reliability	
			Intensity 5	Intensity 6
① A	49	16	0.642	0.021
② B	63	60	0.860	0.058
③ C	72	67	0.935	0.058
④ D	77	70	0.931	0.160
⑤ E	87	82	0.879	0.101
⑥ F	91	85	0.790	0.048
⑦ G	97	93	0.918	0.132
⑧ H	124	117	0.937	0.239
⑨ I	136	126	0	0

4 Conclusions

In this study, the relations between the system reliability, conditions of ground and pipe, and the aseismic investment are investigated for the assumption model of the water supply pipelines, then the water supply pipelines in Yatsushiro city are modelled as network and the reliabilities of hospital lifeline in Yatsushiro city are investigated. Conclusions are summarized as follows:

- 1) Reliabilities of network depend on the pipe types and ground conditions and have low values for cast iron or asbestos cement pipe, middle or small diameter pipe, and ground possible to be liquefied.
- 2) It is difficult to increase the reliability of the network system by the pre-earthquake investment for improving pipeline. It is possible to increase the reliability of the system and decrease the total cost of it by the pre-earthquake investment for improving ground.
- 3) For the hospital lifeline network in Yatsushiro city, the reliability of the hospital locating near shore line is low in the case of seismic intensity 5 and the



reliability of all hospital decrease in the case of seismic intensity 6. Thus another countermeasure except physical ones is needed in seismic intensity more than 6.

References

- [1] Akiyoshi, T., Fuchida, K. & Maeda, S., An Estimation Method on Aseismic Investment for Lifeline Systems, *Proc. of 10th Earthquake Engineering Symposium*, Vol.3, pp.3187-3192, 1998 (in Japanese).
- [2] Kubo, S., Estimation of Seismic Damage and Restoration of Lifeline System, Master Thesis for Kumamoto Univ., 1984 (in Japanese).
- [3] Hino, A., Optimization of rational aseismic investment for lifeline, Master Thesis for Kumamoto Univ., 2000 (in Japanese).
- [4] Japan Road Association, *Specifications for highway bridges, Part V Earthquake-resistant design*, JRA, 1996 (in Japanese).
- [5] Tamura, C. and Kawakami, H., Seismic risk analysis of underground lifeline system by use of Monte Carlo method, *Proc. of JSCE*, No.311, pp.37 48, 1981 (in Japanese).
- [6] Takada, S., Direct and indirect economic losses of Kobe water systems during the 1955 Hyogoken-Nanbu Earthquake, *Proc. of Third China-Japan-US Trilateral Symposium on Lifeline Earthquake Engineering*, pp.291-300, 1998.
- [7] Civil Planning Research Committee, *Analysis and planning of traffic network – newest theory and application* - Text of Civil Planning Course, JSCE, pp.33 38, 1987 (in Japanese).



This page intentionally left blank

Human life saving lifelines and cost-effective design of an exclusive water supply system for fires following earthquakes

S. Takada & Y. Kuwata

Department of Civil Engineering, Kobe University, Japan

Abstract

This paper investigates the effects of the lifeline malfunction on the loss of human lives during the 1995 Kobe earthquakes. Firstly those effects were reviewed for firefighting and hospital waters relating to water supply lifelines. Those lives which might have been saved if the water lifelines had worked shortly after the earthquake are analyzed based on the records of firefighting operation and have been counted as approximately 32 to 45 people. Next, an exclusive water supply system for firefighting introduced into fire fragile areas after the earthquakes is designed giving basic procedures based on cost-effectiveness.

Keywords: lifeline, firefighting.

1 Introduction

Lifeline is categorized into four systems; energy supply system, water supply and treatment system, transportation system and information system, which are essential infra-structures for social activity, especially in urban livelihood. As a lesson from the 1995 Kobe earthquake, we learned that human lives might be lost as a result of the lifeline malfunctions in an emergency situation after earthquakes; for examples, traffic congestion of telecommunication system in the search and rescue (SAR) activity, no firefighting water from hydrant, traffic congestion in transporting earthquake injuries and the lifeline malfunction in emergency medical care centers.

This paper investigates how the lifeline malfunction affected the loss of human lives during the 1995 Kobe earthquakes. Effects related to water supply lifelines were reviewed. Especially on the stage of fires following earthquake,



those who might be saved if water lifeline had worked shortly after the earthquake are counted based on the records of firefighting activity. Then, an exclusive water supply system for firefighting introduced into fire fragile areas after earthquakes is designed giving basic procedures based on cost-effectiveness.

2 Loss of human lives related to water supply lifelines

2.1 Firefighting water

More than 300 fires occurred following the 1995 Kobe earthquake causing the deaths of approximately 550 people among 6,434 earthquake casualties. Causes of ignition are thought to be the turning of stove, gas leakage and the round area-block restoration of electric power without checking individual house condition. Many factors such as traffic congestion hampering the transportation of fire engines and no water from hydrants, which are installed in the drinking water supply system, led to the urban conflagration. There is duty for the fire department to prepare fire protection environment; however the lack of water from hydrants attached to drinking water supply systems operated by Kobe City Water Bureau made the fire grow in a large area.

Figure 1 depicts 52 locations of the fires causing burnt deaths in Kobe city during the earthquake [1]. The circle size indicates the number of casualties. Large victims were concentrated in the west part of Kobe City.



Figure 1: Victims by fires during the 1995 Kobe earthquake.



2.2 Water in medical care centre

A lot of hospitals had major damage to their building and medical facilities and lost function of emergency medical care in the 1995 Kobe earthquake. Here a case for the main hospital called Kobe City General Hospital is shown in detail. The hospital located on an artificial island, Port Island in Kobe bay is one of the biggest hospitals in Kobe City and normally accepts 2,535 ambulatory patients and 959 hospitalized patients per a day. Main structure of the building with 12 stories constructed in 1980 had slight earthquake damage, thanks to the seismic countermeasures for liquefaction. However, hospital lifelines and interior equipments could not be of use.

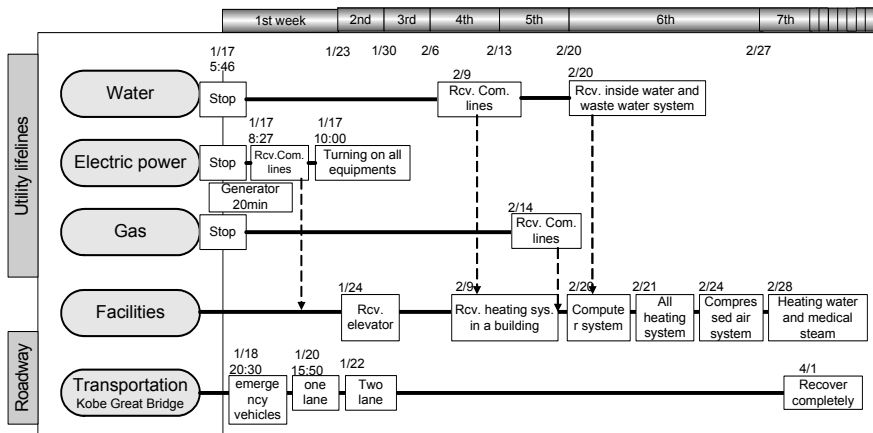


Figure 2: Restoration process of hospital lifelines in Kobe City General Hospital during the 1995 Kobe earthquake.

There were several breaks and leaks in the municipal water pipelines from distributing water reservoirs via the Kobe Great Bridge to the hospital, by destructive strong ground motion and liquefaction. In the hospital, an elevated water tank for drinking (80m^3) and its piping were damaged, and an elevated water tank for miscellaneous use (60m^3) was cracked. Due to water leaked from these tanks, the water supply system to pump water up to the elevated tanks was automatically switched on, and water in a receiving tank located on the ground drained. There was no useful water inside the hospital. Immediately after the earthquake, water companies and Self Defense Force delivered water tanks with 20 tons a day, which was not enough compared with daily use of 700 to 900 tons. Although the pipeline outside the hospital was repaired in 3 weeks after the earthquake, repair of the elevated water tanks needed another 1 week. When the inside water supply system was completely repaired, most medical equipments and machineries restarted gradually as shown in Figure 2. In this case, the outage of water gave a significant impact on hospital sanitary environment and medical



care operations. Many hospitalized patients were transported to other medical care centers and the hospital could not accepted ambulatory patients. The number of people who were affected by the hospital malfunction has not been made clear.

3 Estimation of human lives saved from fires – if the water had come at hydrants

3.1 Case study for human lives saved from fires

52 locations of the fires causing deaths can be divided into 7 cases in conditions of fire spread and response throughout the detailed analyses on fire growth and firefighting operation [1–4], as listed in Table 1. With respect to 6 fire locations of Case 1 listed in Table 1, the people who would have been saved if the water had come at hydrants are listed in Table 2. We believe 32 to 45 people might have been saved out of 95 burnt people.

Table 1: Case study on fires causing burnt deaths in relation to fire spread and fire fighting activity.

Case	Fire spread and response	No. of fires	Burnt people
1	Possible to save people if water had come at hydrants	6	95
2	Insuppressible against aggressive fire growth	9	110
3	Suppressible against fire but impossible to save people	5	20
4	Insuppressible against fire due to the delay of fire engine arrival	6	195
5	Burnt without fire engine arrival	5	65
6	Burnt only in one house	4	4
7	Insufficient data to analyze	17	40
Total		52	529

Table 2: Estimated human lives saved in the burnt areas.

Burnt district	Burnt people	Possible to be saved
A	8	1 to 4
B	25	7 to 12
C	15	2 to 4
D	40	22
E	5	0 to 2
F	2	0 to 1
Total	95	32 to 45



3.2 Case study for saved people from fires

Followings are explanations on the analytical procedure to obtain the number of people who would be saved under the assumption that the water at the hydrants had come during the earthquake [5]. The burnt district A in Table 2 is shown as an example.

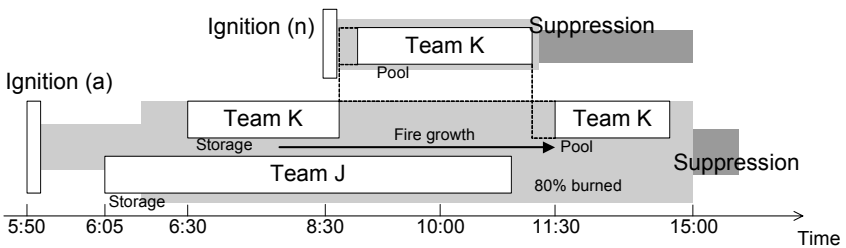


Figure 3: Fire operation timeline at district A.

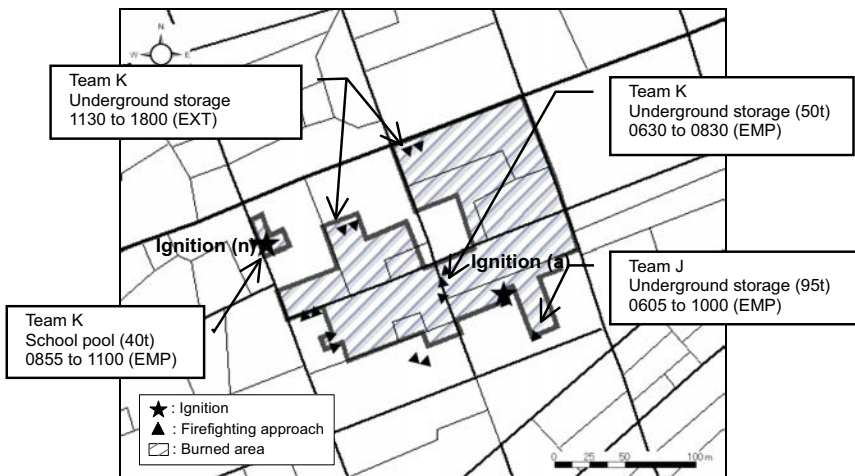


Figure 4: Burnt districts and fire fighting activity.

The timeline from ignition, fire spread and firefighting activities up to suppression is shown in Figure 3. In the district A, 25 people were burnt to death in the fire of ignition (a), which was ignited at 5:50 a.m. on January 17, 1995 and suppressed at 15:30 on the day with the final burnt area of approximately 30,000 m². Figure 4 shows the burnt area and approaches of firefighting activities with water usages such as pool and underground storage. Two firefighting teams with fire engines and personnel were dispatched to this

area by Kobe City Fire Bureau, and arrived at the district A at 6:05 a.m., 15 minutes after the ignition. One team (called Team J) carried out the firefighting activity up to 10:00 a.m., when the underground storage of water got empty. The other team (called Team K) also did from 6:30 a.m. to 8:30 a.m. until the empty of the other storage, and then moved to the other fire of ignition (n), using water in the pool at an elementary school from 8:55 a.m. to 11:00 a.m. At 11:30 a.m. Team K returned to the operation at the fire of ignition (a), using water of the pool.

The fact is that both of the teams could not continue the activity after 10:00 a.m. because of the lack of firefighting water from hydrants although the fire was still spreading. Assumption made herein is that the fire spread from the ignition location to the northeastern area could not be suppressed, but that to the western area could be reduced resulting in half of the burnt area, if the firefighting water had been reached at hydrants considering the operation timeline of the two teams. Then the total 25 burnt people could be decreased to 13 to 18 people considering the burnt area at the time of storage was empty and the uncertainty of their addresses. Analyses for other 5 burnt districts in the Case 1 have been made as well.

4 Design of exclusive fire fighting water supply system

Several countermeasures are prepared in order to maintain firefighting water after earthquakes. Kobe City Fire Bureau currently increases large underground storages of the water from the lessons of urban conflagrations during the 1995 Kobe earthquakes. On the other hand, AWW (Auxiliary Water Supply System) in San Francisco and DFPS (Dedicated Fire Protection System) in Vancouver are water supply systems dedicated for firefighting constructed after the huge earthquakes [6]. They play the role of back-up function for water from hydrants attached to drinking water supply systems.

Here an exclusive water supply system dedicated for firefighting (here after called EWSS) is introduced to a part of Kobe City as one of countermeasures for fires following earthquakes. Figure 5 shows the basic concept designing the EWSS. The design can be done by following steps: (1) determination of a fire hazard area for the system installed, (2) estimate of water volume required for firefighting, (3) selection of water source, (4) hydrants allocation, and (5) design of pipeline system considering hydraulic reliability and cost benefit.

Figure 6 shows water supply districts with percentage of anti-seismic pipelines improved after the Kobe earthquake [7]. The target area for EWSS installed is determined considering factors on inflammable wooden houses, weak pipelines and locations of disaster related public organizations [8].

The required firefighting water is counted by predicting the number of ignitions and fire spread under the most severe conditions of seasons and hours based on the experiences in the previous earthquakes in Japan as well as in the Kobe earthquake. Table 3 shows the parameters used for estimating the required water volume against the ignitions of fire type. One of reservoirs of the drinking water supply system, currently out of use, is selected as the water source of



EWSS after comparing cost and performance with the case of water sources by the existing water transmission tunnel.

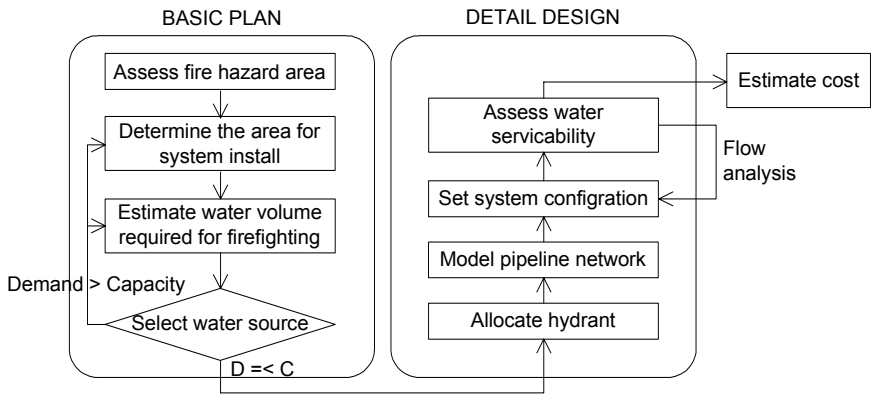


Figure 5: Design of exclusive fire fighting water supply system.



Figure 6: Water supply districts with percentage of anti-seismic pipeline.



Table 3: Required firefighting water against an ignition of fire type

	Fire within house	Conflagration
Burnt area (m ²)	120	6,800
Burnt perimeter (m)	53	476
Average perimeter for fire engine (m)	8	15
Number of hydrants for water use	3.2	15.6
Watering time (min.)	79	1,328
Required water volume for firefighting (m ³)	157	10,624

A pipeline network model with hydrants has been proposed based on hydraulic flow analyses, which gives appropriate pipe diameters to satisfy the pressure of firefighting water. The proposed pipeline network at minimum cost is shown in Figure 7. The cost for the EWSS install is 90.5 billion US\$ as listed in Table 4.

The verification of the cost effectiveness of the proposed EWSS was discussed from the view points of other countermeasures such as the construction of large underground storages of water under several scenarios of earthquake. Please refer other our papers [9].

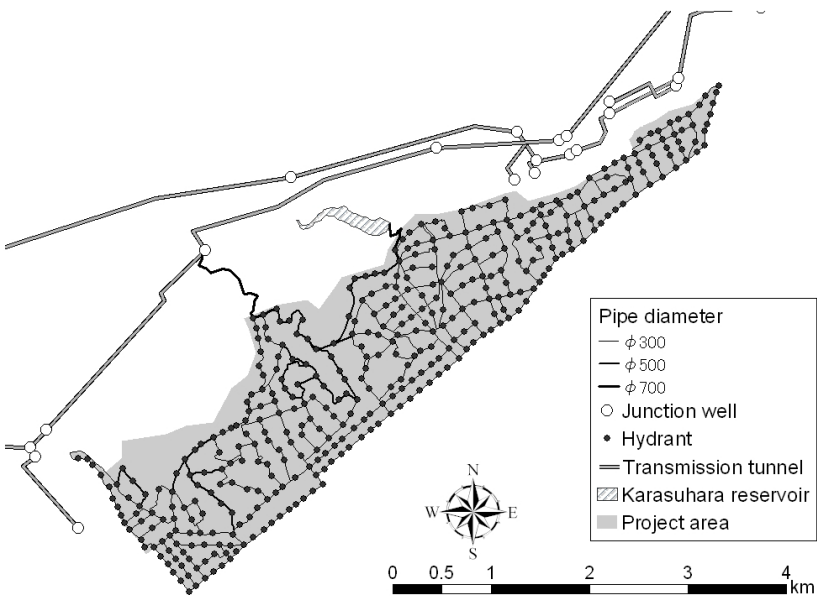


Figure 7: Proposed model of EWSS.



Table 4: Cost estimated for the EWSS install.

	Unit price (US\$/m)	Pipe length (m) or pump head (m)	Cost (US\$)
Pipeline $\phi 300$	1,080	62,634	67.8
$\phi 500$	1,667	8,534	14.3
$\phi 700$	2,333	1,482	3.5
Subtotal		72,651	85.5
Pump	83,333	60	5.0
Total			90.5

5 Conclusive remarks

This paper discussed the importance of human life safety related to lifelines during earthquakes. Effects due to malfunction of the water supply lifelines are reviewed on firefighting water and hospital lifelines for emergency medical cares. Followings can be concluded as the results of this paper.

Those who would be saved if water lifeline had worked well shortly after the 1995 Kobe earthquake are analyzed based on the records of firefighting activity and counted as approximately 32 to 45 human lives.

A cost effective design procedure and an application for an exclusive firefighting water supply system (EWSS) have been proposed considering earthquake fire hazard and hydraulic efficiency.

References

- [1] Japan Association for Fire Science and Engineering (eds.), *Investigating report on fires in the 1995 Kobe earthquake*, 1996 (Jp).
- [2] Kobe City Water Bureau, *Report on firefighting activity during the Hanshin-Awaji earthquake disaster*, No.1, Gyosei, 1996 (Jp).
- [3] Yasuno, K. & Hayakawa, T., *Fire and water supply system*, ed. Japan Water Research Center, 2001 (Jp).
- [4] Lifeline Network, Kansai (ed.), *Lessons from Hanshin-Awaji Earthquake*, Proc. of Symposium on Disaster Prevention and Mitigation, pp.141-146, 1997 (Jp).
- [5] Takada, S., Gonsoku, Y. & Kuwata, Y., Fire spreading and casualty due to outage of water at hydrants, *Journal of Japan Association for Earthquake Engineering*, 5(2), pp.1-15, 2005 (Jp with Eng abstract).
- [6] Scawthorn, C., Fire following earthquakes (chapter 29). *Earthquake Engineering Handbook*, eds. Wai-Fah Chen & Charles Scawthorn (eds.), CRC press, 2003.
- [7] Kobe City Water Bureau, Personal communication, 2005.
- [8] Kobe City Disaster Prevention Council, Kobe City Urban Planning for Disaster, 2005 (Jp).
- [9] Takada, S. Kuwata, Y. & Gonsoku, Y., Cost-effectiveness analysis for instruction of dedicated water supply system for firefighting, *Journal of Japan Association for Earthquake Engineering* (submitted).



This page intentionally left blank

Section 9

Monitoring and testing

This page intentionally left blank

Shaking table tests on shallow foundations

J. Estaire & V. Cuéllar

Laboratorio de Geotecnia, CEDEX, M° de Fomento, Madrid, Spain

Abstract

In this paper the results of shaking table tests on shallow foundations, using dry Hostun RF sand, are presented. These tests were performed in a six-degree of freedom seismic simulator formed by 3 m long square table, able to move 10 t mass with up to 1-g accelerations. On that simulator a rigid box of great dimensions (3 m long, 1,1 m wide and 1,2 m high) was placed and filled with some 6 t of sand. During the filling of the box, some accelerometers were installed inside the sand to measure accelerations suffered by the granular deposit. The shallow foundations were modelled with three metallic blocks that transmitted different loads to the ground. The tests were designed taking into account the usual scale factors.

In the thirty tests performed, the shaking table imposed a wavy acceleration at the bottom of the box with different amplitudes and frequencies.

The result analysis was focused on the amplification of acceleration during its propagation in vertical direction through the sand deposit and the vertical and horizontal movements of the foundation blocks during the dynamic excitation.

Keywords: model test, shallow foundations, earthquake resistant, sand, settlement.

1 Introduction

Earthquakes are still nowadays natural phenomena of a great destructive potential, so there is a great interest in achieving a better knowledge of the behaviour of soils and the structures founded on them under dynamic conditions.

Shallow foundations have had a seismic behaviour that can be qualified as good and adequate, so they have not received much attention in the past. However, failure cases reported in some buildings in Ciudad de Mexico, founded on soft clay, during the Michoacán earthquake in 1985, Romo and Auvinet [1],



changed this way of thinking and made it necessary to consider the problem of the dynamic behaviour of shallow foundations.

2 Testing method followed

The reduced scale tests were conducted in the six-degree of freedom shaking table belonging to CEDEX (Madrid, Spain). This seismic simulator is formed by a 3 m long square table moved by several actuators whose movements are servocontrolled, Navarro [2]. On this shaking table a rigid box was installed, consisting of a metallic structure with lateral walls, made of 5 mm wide transparent metacrilate. The box was 3 m long, 1,1 m wide and 1,2 m high and had a capacity of 4 m³. The weight of the empty structure was 380 kg and filled with around 6 t of Hostun RF sand, Flavigny et al [3].

This sand is a uniform sand of medium size, as the diameter of its grains is between 0,65 and 0,16 mm, the mean diameter (D_{50}) is about 0,35 mm and the coefficient of uniformity ($C_u = D_{60}/D_{10}$) is around 1,8. The maximum and minimum densities obtained in laboratory were 15,68 kN/m³ and 12,65 kN/m³.

The sand was poured through a 50-litre hopper in 20 cm layers. Once each layer was levelled, the whole system was vibrated during one minute with acceleration at the base of the table of 40 Hz frequency and 0,25 g amplitude. The sand was thus laid with an average relative density of 90%. Above the different sand layers, a one cm thick layer of coloured sand was poured beside the transparent metacrilate walls to control the evolution of sand settlements in depth. Figure 1 shows the box filled with sand situated on the shaking table, with the displacement transducers situated on the surface.



Figure 1: Photograph of the box, filled with sand, on the shaking table.



The situation of the measurement apparatus can be seen in Figure 2. Some LVDT transducers were installed on the metallic blocks, which simulated shallow foundations, to measure their vertical and horizontal displacements. The horizontal movement of the whole box was also controlled. The vertical movement of the sand deposit, at a point away from the influence zone of the foundation, was measured in order to keep a record of the evolution of the density of the sand deposit during the dynamic excitation.

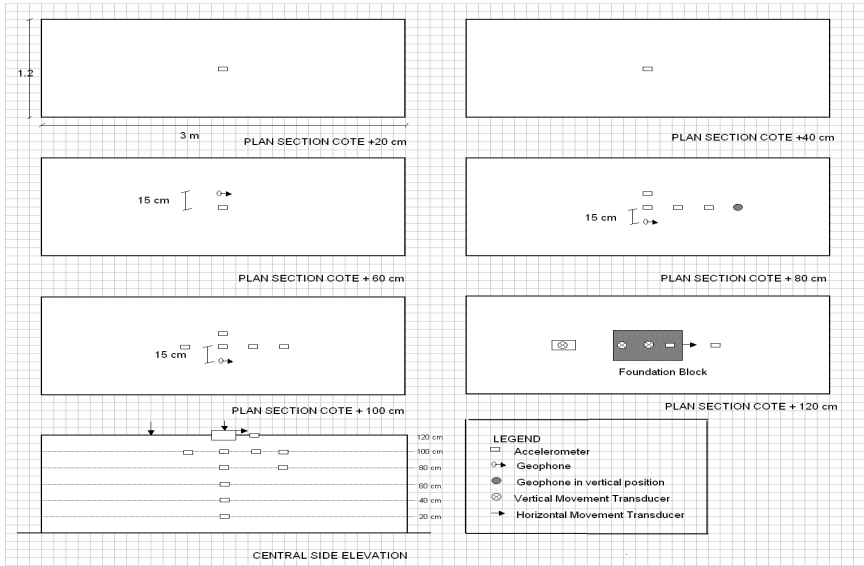


Figure 2: Position of the accelerometers and displacement transducers.

3 Tests performed

The tests were performed placing on top of the sand deposit a metallic block of 25 cm long, 20 cm wide and 15 cm high, buried 5 cm deep, which simulated a shallow foundation. Three metallic blocks (named Z-A, Z-B, Z-C) weighing 1,5; 20 and 40 kg, respectively, were used.

The foundation blocks used in the tests and the sand deposit that they were placed on were designed to simulate real situations of shallow foundations resting on dry sandy layers. According to the scale factors applicable to the tests, Iai [4], the geometry and the loads of these tests make it possible to simulate a great number of real cases. For instance, if a scale factor of 10 is used, the resultant foundation dimensions (2,5 x 2,0 x 1,5 m) and the vertical stresses transmitted to the ground (80 kPa) by the heaviest block are usual values in daily practice. The interpretation of the test results must be done taking into account the corresponding scale factor to displacements, velocities and accelerations.



With the measurement system implemented in the box, 30 tests were performed, in which, for each of the three foundation blocks, the frequency of the imposed signal of accelerations in the base of the box (5 and 10 Hz) and the amplitude (0,1 – 0,2 – 0,35 – 0,5 and 0,6g) were changed.

It was decided to perform tests with amplitudes of imposed acceleration superior to 0,35g, which is the limit value derived from the fluidisation theory by Richards et al [5] for fine granular material and a value scarcely exceeded in similar tests performed on shaking tables by different research groups as Fukutake et al [6], Taylor [7], Shamoto et al [8] and Maugeri et al [9]. The aim of the tests performed with high accelerations was, on one hand, to test the validity of such theory and, on another hand, to analyse the behaviour of a sand deposit in such a “fluidisation” situation, when sand stops behaving as a frictional material to begin showing a behaviour similar to a fluid.

The tests lasted 20 seconds, so 100 cycles were imposed, in the tests with a frequency of 5 Hz, and 200 cycles in those with 10 Hz of frequency. Figure 3 shows one of the signals, in terms of accelerations, imposed on the shaking table to perform the tests. It can be seen that, at the beginning and at the end of the imposed signal, the acceleration increases and decreases, respectively, in a ramp during two cycles, to make easier the task of the shaking table actuators.

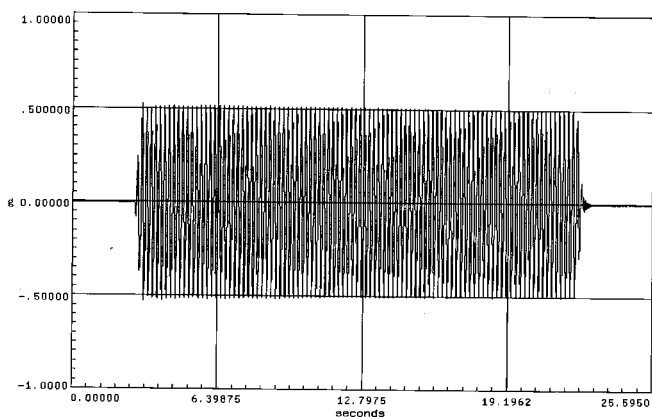


Figure 3: Signal imposed by the shaking table at the base of the box.

4 Results obtained

4.1 Amplification of horizontal acceleration in vertical direction

This section studies the amplification of horizontal accelerations on the shaking table, during its vertical propagation until reaching the top sand surface and the influence of the presence of the metallic blocks. Figure 4 shows the records of horizontal accelerations measured in the box base and near the surface.



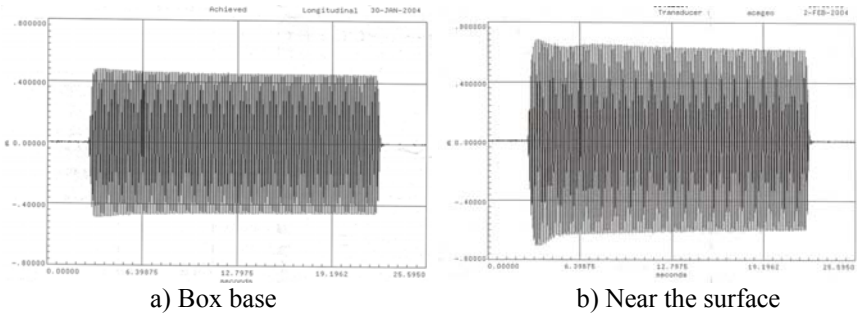


Figure 4: Records of horizontal accelerations at different depths in the box.

The average values of the measured acceleration amplification have been represented for each group of tests performed in Figure 5. The average curve obtained in free field tests performed in the same conditions, Estaire and Cuéllar [10], was also included. The amplification was quantified in percentage value, comparing the maximum amplitude measured at each depth with the amplitude of the acceleration imposed by the shaking table at the base of the box.

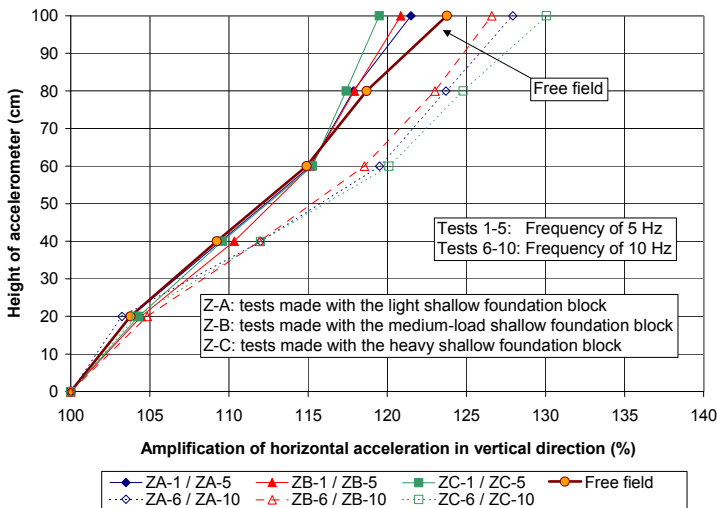


Figure 5: Average acceleration amplification in vertical direction.

The principal aspects that can be highlighted from the results are:

- In all the tests, the acceleration amplitude increases when the signal moves vertically along the sand.
- Larger amplifications were obtained in the tests performed with higher frequency. This difference is more noticeable as the acceleration amplitude imposed at the bottom of the box increases.



- c.- The amplification measured in the accelerometers situated at 100 cm high is lower in the tests performed with a foundation block, if compared with the free field tests performed with the same frequency signal of 5 Hz. This difference increases, as the load imposed by the foundation block is greater. This aspect can be analysed more carefully in Figure 6, in which the amplification measured in the accelerometer nearest to the foundation block was represented for all the tests.

The analysis of Figure 6 makes it possible to highlight the following experimental aspects:

- a.- In all the cases, independently of the frequency, the signal amplification increases parallel to the increase in the amplitude of the acceleration signal imposed. This fact can be checked through the parabolic curves that adjust the experimental results.

$$\text{Amplification}_{(in \%)} = 115 + 100.a_{(in g)}^{3,5} \dots \text{for 5 Hz frequency}$$

$$\text{Amplification}_{(in \%)} = 115 + 100.a_{(in g)}^2 \dots \text{for 10 Hz frequency}$$

- b.- In all the cases, the amplification is greater in the tests performed with the acceleration signal of highest frequency (10 Hz). This greater amplification can be due to the fact that frequency of 10 Hz is nearer to the sand deposit natural frequency, which can be quantified in about 40 Hz, taking into account that the shear wave propagation velocity in vertical direction is about 190 m/s, as interpreted from shear propagation test results.

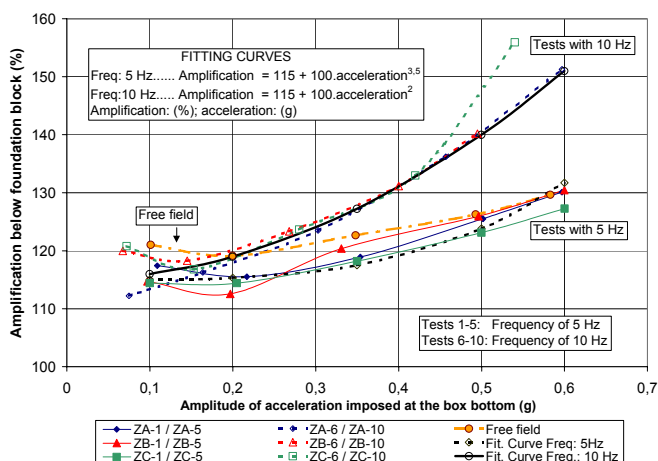


Figure 6: Amplification in the ground below the foundation block.

- c.- Furthermore, it can be seen that the difference between the results obtained with the two frequencies (5 and 10 Hz) is greater when the amplitude of the imposed acceleration signal increases.
- d.- The difference in the amplification measured below the foundation block, between the free field situations and when a block is placed on



top, decreases when the amplitude of the imposed acceleration signal increases.

4.2 Vertical movements of foundation blocks

The vertical movements of the shallow foundations were also analysed. Figure 7 shows two of the vertical movement records, representatives of all the tests. In some of the tests there was almost no remaining settlement, as all the vertical movement had a wavy recoverable nature. On the contrary, in other tests the settlement experienced by the foundation block when the test finished can be clearly seen, although during the dynamic excitation part of the vertical movement was wavy and recoverable.

The results obtained in all the tests have been represented in Figures 8 and 9. The curves that best fitted the results have been also included in both figures.

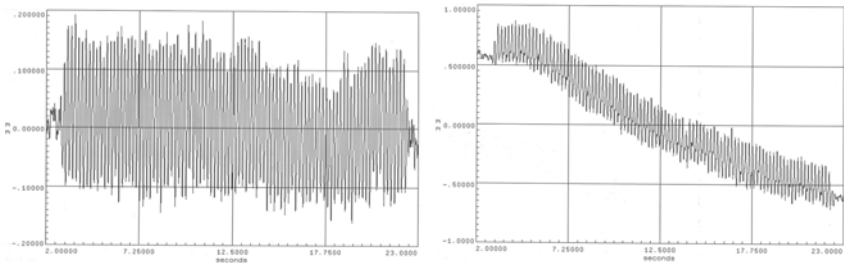


Figure 7: Vertical movement records of the foundation blocks.

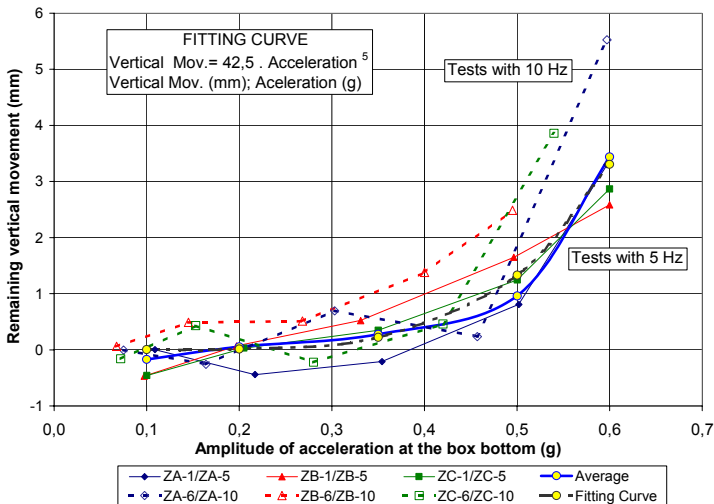


Figure 8: Remaining vertical movement measured in the foundation blocks.



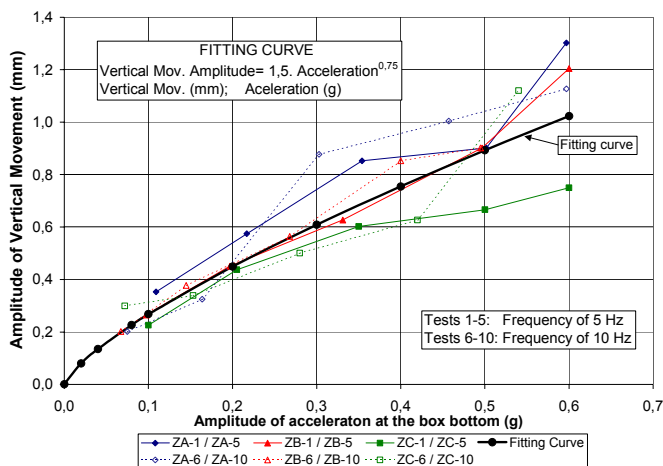


Figure 9: Average amplitude of the vertical movement of the blocks.

The analysis of the vertical movements obtained in the tests makes it possible to state the following:

- In all the tests with an imposed acceleration amplitude inferior to 0,35g, the observed behaviour can be qualified as almost elastic: the remaining vertical movements measured were less than 0,7 mm and the vertical movement amplitude was superior to the vertical remaining movement.
- On the contrary, for the greatest amplitudes used in the tests (0,5 and 0,6g) the remaining settlements are bigger: about 2 mm in the tests made with amplitude of 0,5 and between 2,5 and 5,5 mm, for the tests made with amplitude of 0,6g. Besides, in these cases, the amplitude of the vertical movement was lower than the remaining settlement.
- The measured movements do not allow the establishment of a clear different behaviour in relation to the foundation block load.
- The movements measured in the tests performed with the imposed acceleration signal of highest frequency (10 Hz) are, in general, greater than the ones recorded in 5 Hz frequency tests.
- The vertical movements increases exponentially with the imposed acceleration amplitude, as can be checked numerically with the fitting curve:
- The vertical movement amplitude grows parabolically with the imposed acceleration amplitude as is reflected in the fitting curve.

4.3 Horizontal movements of foundation blocks

During the tests, the horizontal movement of the foundation blocks in the direction of the imposed acceleration, relative to the box walls, was measured with a displacement transducer. Two of the records of such movements can be



seen in Figure 10. These two records are representative of the two types of movements obtained in the different tests: those in which the block had a wavy and recoverable horizontal movement and those in which there was a final remaining sliding.

The test results are represented in Figures 11 and 12.

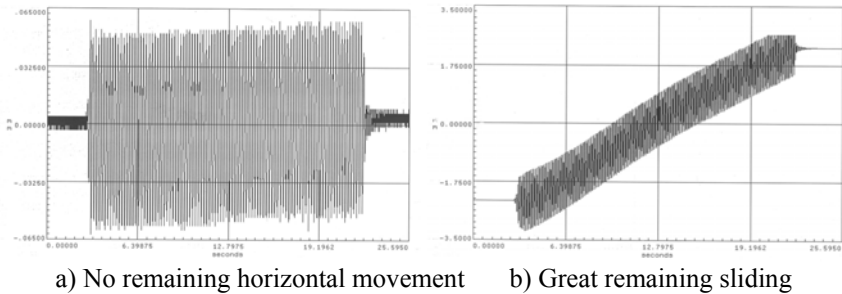


Figure 10: Record of relative horizontal movement of the foundation blocks.

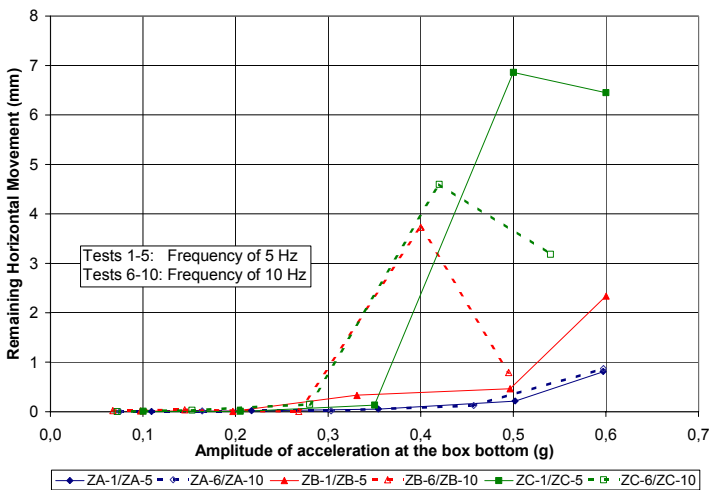


Figure 11: Remaining horizontal movement measured in the foundation blocks.

The following comments can be made as a result of the analysis of the horizontal movement data obtained in the tests:

- a.- From a general point of view, the tests made it possible to prove that, in all the cases, the remaining horizontal movement of the foundation blocks, and the amplitude of that movement, increase considerably when the imposed acceleration amplitude is superior to 0,35g. This value is similar to the one deduced from the “fluidisation theory” by Richards et al [5] for a sandy material with an angle of friction of about 35°.



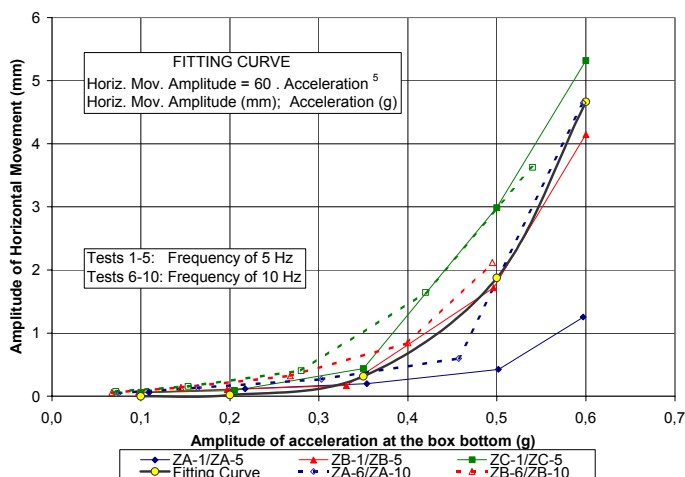


Figure 12: Amplitude of the horizontal movement measured in blocks.

- b.- The results seem to indicate that the greater the mass of the foundation block used, the greater the horizontal movement, the other factors being equal.
- c.- The horizontal movement amplitude increases exponentially with the amplitude of the acceleration imposed at the bottom of the box.
- d.- In all the tests, except in the four with the greatest remaining horizontal movement, the horizontal movement is superior to the value of the remaining sliding of the foundation block. This fact indicates that the nature of the horizontal movement was, in general, wavy, elastic and recoverable.

5 Conclusions

- a.- There exists a great difference in the acceleration amplifications measured in the tests in relation to the imposed acceleration frequency. The presence of the foundation block modifies the accelerations in the zone of the ground affected by the load.
- b.- The measured accelerations seem to indicate that when the imposed acceleration amplitude is either 0,5 or 0,6g, the general behaviour of the foundation can be considered unstable, as indicated by the fluidisation theory by Richards et al [5].
- c.- The values of the remaining vertical movement of the foundation blocks, measured in the tests performed with accelerations of amplitude superior to a certain critical value (0,35g, in this case) can be high, even for high relative densities in the foundation ground. In these cases there has been no evidence of additional densification in non-loaded zones.



- d.- This fact indicates that shallow foundations can have settlements incompatible with a good behaviour of the founded structure.
- e.- The great remaining horizontal movements when the acceleration amplitude is superior to a certain critical value (0,35g, in this case) indicate that shallow foundations can exceed the sliding resistance at its base. In this case, the failure of the foundation can occur due to the corresponding ultimate limit state of sliding, a situation which can be more frequent when the vertical applied loads are low, as in bridge abutments.

Acknowledgements

The authors wish to acknowledge D. Francisco Navarro and his team for their dedication and effort during the performance of the shaking table tests.

References

- [1] Romo, M. & Auvinet, G., Seismic Behaviour of Foundations on cohesive soft soils (Chapter III). *Recent Advances in Earthquake Eng. and Struct. Dynamics*. Ouest Editions, Nantes, pp 311-328, 1992.
- [2] Navarro, F., Simulador sísmico de seis grados de libertad. Técnicas de compensación analógicas y numéricas. *Ingeniería Civil*, **100**, 1995.
- [3] Flavigny, E., Desrue, S. J. & Palayer, B., Note technique: le sable d'Hostun R.F. *Rev. Franc. Géotech.*, **No 53**, pp. 67-70, 1990.
- [4] Iai, S. & Sugano, T., Soil-structure interaction studies through shaking table test. *Earthquake Geotech. Eng.* Lisboa. Balkema, pp. 927-940, 1999.
- [5] Richards, R., Budhu, M. & Elms, D.G., Seismic Fluidisation and Foundation Behaviour. *Second Int. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*. St. Louis, Missouri, Paper No. 5.11, 1991.
- [6] Fututake, K., Ohtsuki, A., Sato, M. & Shamoto, Y., Analysis of saturated dense sand-structure system and comparison with results from shaking table. *Earthq. Eng. and Structural Dynamics*, Vol. 19, pp. 977-992, 1990.
- [7] Taylor, C.A., Dar, A.R. & Crewe, A.J., *Shaking table modelling of seismic geotechnical problems*. 10th Europ. Conf. on Earthq. Eng., 1994.
- [8] Shamoto, Y., Sato, M. & Zhang, J., Simplified estimation of earthquake induced settlements in saturated sand deposits. *Soils and Foundations*, Vol. 36, No. 1, pp. 39-50. March. 1996.
- [9] Maugeri, M., Musumeci, G. & Novita, D., Shaking table test to failure of a shallow foundation subjected to an eccentric load (Chapter 4). *Report No. 6: Soil Dynamics and Foundation Structures*. ECOEST2, 2001.
- [10] Estaire, J. & Cuéllar, V., Shaking table dynamic tests of a granular deposit. *V ERES*, pp. 451-460. Thessaloniki (Greece), May, 2005.



This page intentionally left blank

Development of a digitally-controlled single-axis earthquake shake frame for masonry walls testing

M. J. Guzman & S. L. Lissel

Department of Civil Engineering, University of Calgary, Canada

Abstract

For the evaluation of masonry specimens' response under seismic forces, a 2-actuator, single-axis shake frame was developed at the University of Calgary. The system works with hydraulic power and contains specialized hardware and software for management and control. These include MTS Multipurpose TestWare for the control of the Hydraulic System and National Instruments SCXI and LabVIEW for the data acquisition. Tests using the El Centro earthquake (1940) with just the shake frame mass, and with additional mass (two 200 mm concrete masonry walls 1 m long x 1.6 m high), indicated the satisfactory functionality of the shake frame. In addition, a comparison between the masonry walls' behaviour under earthquake loads and a finite element (SAP2000) model was carried out and is presented here.

Keywords: shake frame, shake table, masonry, seismic load simulation.

1 Introduction

Structures are designed and built to withstand a variety of load conditions keeping in mind safety and economics. In the case of earthquake loads, some structures are more vulnerable than others. Depending on which material the structures are made of, their response and resistance to seismic forces will vary. Masonry structures, for example, are able to resist earthquake loads as long as they are properly designed but, to achieve that, studies and experimentation must be done. Since masonry is not a homogeneous material, the dynamic behaviour of masonry structures is a complex matter. The interaction between masonry units, mortar, grout and the reinforcement create a significant number of variables that influence the response of masonry to dynamic loading;



deformation, damage pattern or failure modes will also be affected by the vibrations induced in the structure by the earthquake.

Seismic Simulation Tables, also known as Shake Tables, are generally used for experimentation on the dynamic behaviour of any type of structure. Using single-axis or multiple-axis excitation, these machines are able to accurately reproduce, in a laboratory environment, the acceleration time history recorded during an earthquake. In the global market they can be purchased at a price of US\$150,000 to over US\$1,000,000, depending on the complexity of their design. Hence, the cost of these devices is of significant influence in the experimentation process. In consideration of these factors, for the dynamic testing of masonry building components, in this case walls and wall intersections, a low-cost testing apparatus was designed and built at the University of Calgary.

Three main stages were involved in the development and construction of the testing apparatus: a) design process; b) construction process; and c) testing functionality of the apparatus.

2 Shake frame

2.1 Design and construction

The Shake Frame is a Single Degree Of Freedom (SDOF) seismic simulation apparatus for dynamic testing of masonry building components, in this case walls and wall intersections. Based on a general design by Hagel et al. [1], the main body of the frame consists of four W360x162 steel I-sections, creating a frame of 2 x 3 m between respective beam centerlines. The robustness of the beams was selected so they could handle the internal forces and bending with negligible deformations. Because the frame was going to be attached to two hydraulic actuators parallel to each other but 2 m apart, torsion could be expected if there was a difference in force between them or if, in a worse case scenario, one of them stops while the other is still working. Therefore, bracing was designed to withstand the torsion. HSS 89x89x9.5 sections were selected for this purpose (Figure 1).

Steel plates at the ends of the beams and the braces allow the use of bolted connections, so the frame can be easily modified or dismantled. All the connections were designed following the specifications in the Canadian standard; the connections between the frame and the braces were designed for shear, and connections between the reaction frames and the floor were designed as slip critical connections.

One of the considerations for the design was that the apparatus should travel with as little friction as possible; therefore, a Ball Bearing and Rail System was used for these purposes. Eight 750 mm long Roller Rail pieces and sixteen Roller Runner Blocks (two on each rail) were selected as the support and guidance system for the Shake Frame. A combination of different sizes of plates, and the application of a self-levelling grout with a thickness of approximately 6 mm between the plates, were used to attach the frame to the strong floor and compensate for alignment and levelling issues of the guide rails, allowing nearly frictionless displacement of the frame (Figure 2).



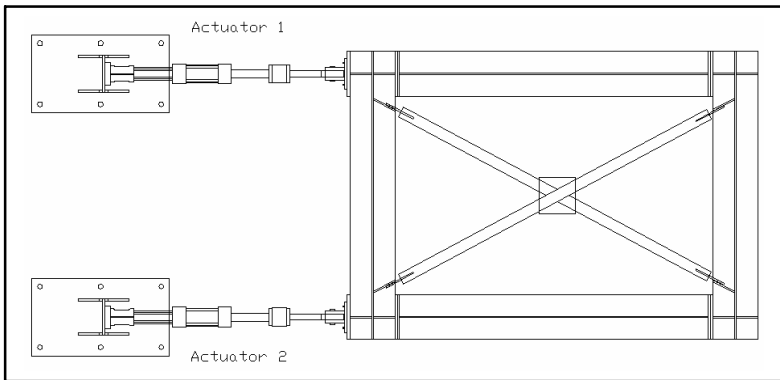


Figure 1: Shake frame plan view.

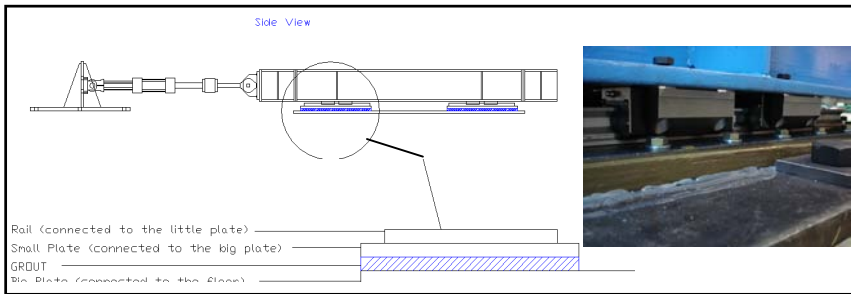


Figure 2: Plate arrangement for alignment compensation; and rail system.

The displacement, force capacity and speed of the Shake Frame are determined by the system's actuators, Ammanagi et al. [2]. These devices can provide a maximum load of 150 kN each, a maximum displacement of ± 125 mm, and they can operate without difficulty at the system design frequency of up to 4 Hz.

2.2 Hydraulic system

To induce an earthquake signal, or a signal of any kind, to the shake frame, two hydraulic actuators were used. The actuators, the hydraulic power system that activates them, and the MTS Systems Corporation software that controls them will be referred to here as the hydraulic system. The hydraulic system produces and controls the movement of the shake frame. The hydraulic power system consists of a Hydraulic Power Unit (HPU), an accumulator, two Hydraulic Service Manifolds (HSMs), and two Actuators. The HPU produces the high-pressure hydraulic fluid for the system operation. At high pressure, the hydraulic fluid is directed out of the HPU to the accumulator, which helps dampen



pressure line fluctuations in the hydraulic system. From the accumulator, the hydraulic fluid goes to the HSMs. These are hydraulic pressure and flow regulation devices that provide an independent control of the hydraulic pressure applied to a test station. The manifolds allow each station to be turned on and off, and set to a low-pressure level, independently from each other and from the HPU. Finally, the hydraulic fluid reaches the actuators, which will put in motion the shake frame in force or displacement control. A FlexTest GT Test Controller is used to control the actuators and the HPU. This is a multi-channel, multi-station control system that combines transducer signal conditioners, servovalve drivers, hydraulic pump and service manifold controls; and, a system control software, to manage servo devices in closed loop testing applications.

3 Shake frame functionality

3.1 The quake

The Imperial Valley 1940-05-19 04:37 earthquake had a magnitude of 6.95 on the Richter scale; the epicenter was located at 32.7601 latitude, -115.416 longitude, and at a depth of 8.8 km.

Using the MTS MultiPurpose TestWare (MPT) software, the displacement time history data from the Imperial Valley 5/19/40-04:39, El Centro Array #9, 180 (USGS Station 117), was programmed in the Hydraulic System. The maximum acceleration, velocity and displacement are 0.313 g, 298 mm/s and 133.2 mm respectively. Due to limitations on the maximum displacement capacity of the actuators, the earthquake input signal was modified. The signal's maximum displacement was changed to 125 mm (Figure 3).

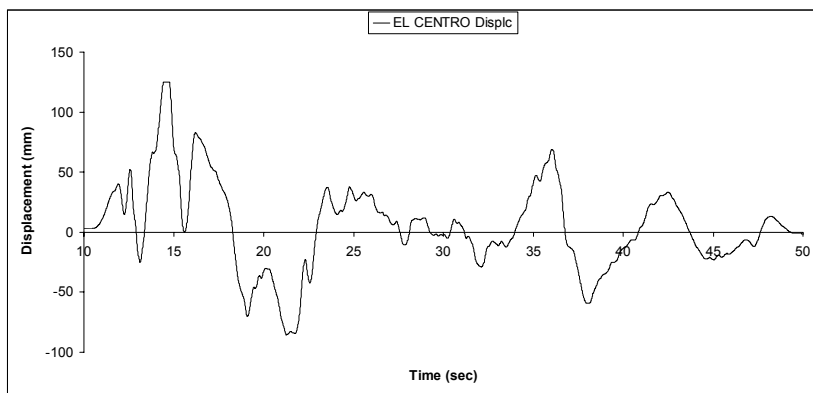


Figure 3: Input displacement signal, El Centro (1940) modified.



3.2 Test one

For the first test, to establish the functionality of the frame, the El Centro earthquake displacement signal was applied to the frame with no additional loading on the system.

To collect force, displacement and acceleration data from the shake frame, a variety of devices were used. The MTS actuators are complete with load cells and Linear Variable Differential Transformers (LVDTs) that allow the measurement of the force and the displacement produced by them. In addition, a Resistive Potentiometer was connected to the frame to obtain displacement measurements.

Through means of the load cells, the MultiPurpose TestWare software indicated that the load differential between the actuators was in the range of 5 to 15 kN. The LVDTs indicated that the displacement errors relative to each other were minimal, and that the accuracy of each LVDT for the applied signal relative to the input signal was ± 5 mm. Upon further investigation, it was observed that the severity of the errors and differentials depended on the characteristics of the input signal; i.e. when using a sine wave signal with low frequency and short stroke, the errors in displacement and the load differentials were small compared to an input signal with the same frequency and stroke but with a ramp shape, or the same sine wave with a higher frequency.

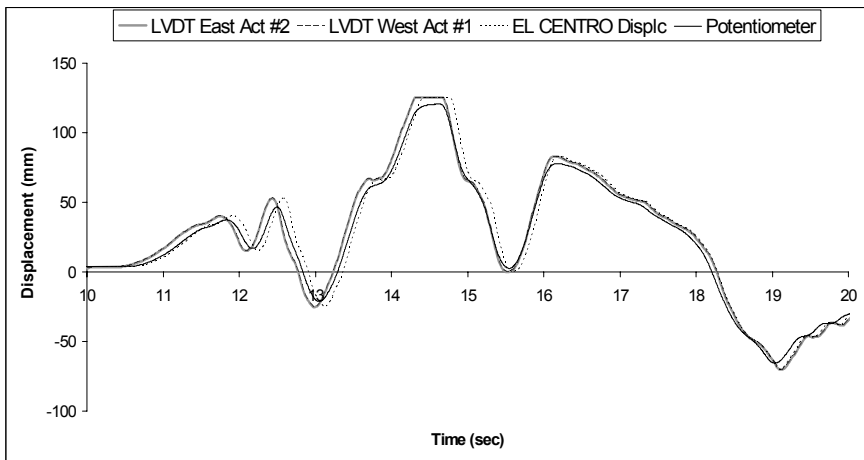


Figure 4: Displacement signals from LVDTs, potentiometers and El Centro input.

According to the LVDT for Actuator 1, the maximum displacement, registered during the simulated earthquake, was 125.24 mm and the maximum displacement measured by the LVDT in Actuator 2 was 125.34 mm. The difference between these two is 0.10 mm. From this, it can be inferred that the actuators work simultaneously (synchronized) and with a minimal displacement differential (Figure 4).



A comparison of the plotted data from the LVDTs and the El Centro earthquake input signal shows that the shake frame is displacing exactly as it is programmed to (Figure 4). However, in the Station Manager of the MTS software there are some features (Meters) that allow the user to monitor the system signal; and, by setting them to monitor the Running Max/Min Displacement Error it was observed that the maximum error registered during the program was 5 mm. Furthermore, by analysing the plotted data from a Resistive Potentiometer connected to the frame (Figure 4), it was noticed that the maximum displacement reached during the running of the program was 120 mm, while the system was programmed to have a maximum displacement of 125 mm; a difference of 5 mm. This number is consistent with the MTS Meters output data.

Figure 5 shows a comparison between the wall's acceleration, the derived acceleration and the location of the maximum displacement. Using KISTLER K-Beam Accelerometers to collect acceleration data from the shake frame, and applying filters (0.2 Hz High Pass and 15 Hz Low Pass) to reduce noise and unwanted vibrations, it was noticed that the acceleration signal from the shake frame and the one derived from the input displacement data were almost the same. There are two points (at 4 and 4.31 s) where the acceleration increases significantly as a result of the adjustments to the displacement signal.

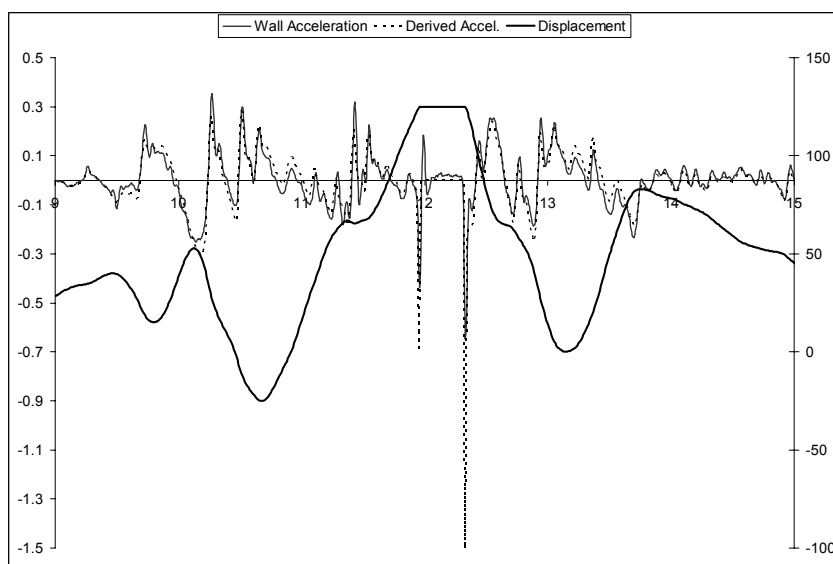


Figure 5: The wall's acceleration, derived acceleration and displacement over the first 6 s.



3.3 Second test

Finally, to verify the functionality of the shake frame when additional mass is applied, two concrete masonry walls were built in place, on the side beams of the shake frame. The walls were 1 m long x 1.6 m high and 200 mm thick, with a total mass of approximately 500 kg (both walls). They were attached to the frame using two threaded rods grouted only in the first course. No other reinforcement was used in the walls. They were tested after 28 days.

Using a videogrammetry system, Tait et al. [3], to measure displacement by means of video cameras and fixed visual targets, displacement data from the shake frame was obtained. A comparison of the input displacement signal, the potentiometer (unloaded frame), and the videogrammetry (loaded frame) shows a minimal difference in relation to each other. As observed in the first test, there is a 5 mm maximum error compared with the input data, but the loaded and unloaded signals are practically the same (Figure 6 and 7) indicating satisfactory performance of the system.

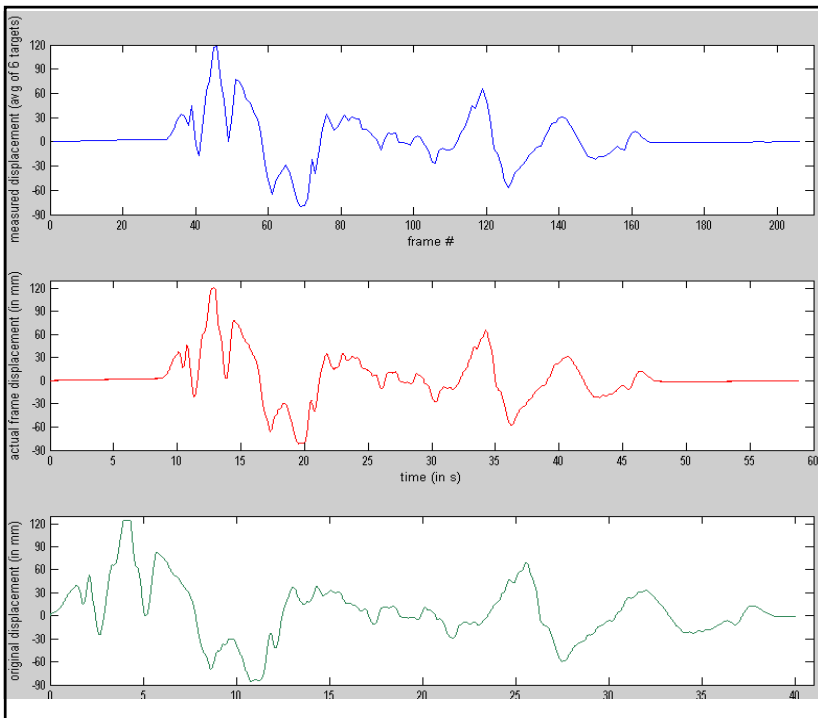


Figure 6: Videogrammetry, potentiometer and original input signals (Tait et al. [3]).



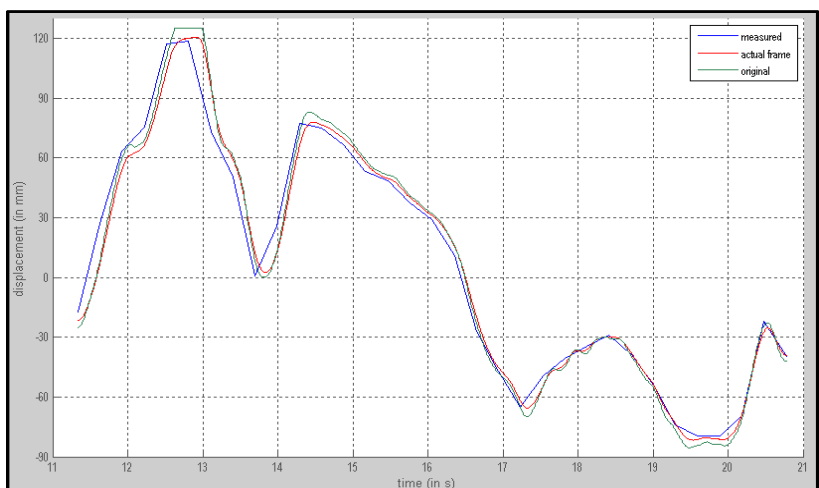


Figure 7: The original input, potentiometer and videogrammetry signals over the first 10 s (Tait et al. [3]).

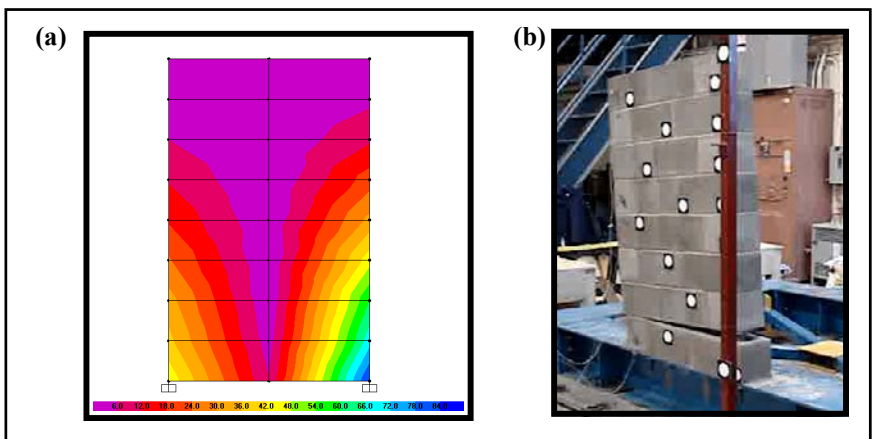


Figure 8: (a) Distribution of stresses inside the wall (SAP2000); (b) wall failure.

A Finite Element model of the masonry walls was created using SAP2000. The walls were simulated by means of Shell Elements; 16 elements of 0.50 x 0.20 m and 200 mm thick. Applying the acceleration signal from the unloaded shake frame on the SAP model, and establishing a comparison of the acceleration data obtained from the walls, at 0.10, 0.90 and 1.50 m high, and the acceleration from the SAP model at similar heights, indicated that the model was an accurate representation of the walls.



Figure 8(a) shows the inside tensile stress distribution in one of the walls. High stresses are detected at the lower part of the model, the closest to the bottom of the wall the higher the stresses. This corresponds with the failure mode of the walls, bond failure at the bed joints (Figure 8(b)).

4 Conclusions

The maximum error registered by the different sensors used in the experiments was 5 mm. This represents 4% of the maximum input displacement (125 mm).

The readings obtained by the LVDTs in the actuators indicated that the actuators worked synchronized and according to the specifications.

Although the displacement signal was modified due to the limits of the actuators, the acceleration derived from that signal varied compared very well to the original El Centro earthquake acceleration signal. Nevertheless, the shake frame accurately reproduced the input signal in both of the experiments (unloaded and loaded conditions).

The finite element model, developed for further masonry dynamic tests, was proven to reasonably predict the behaviour of masonry elements when subjected to dynamic loading.

References

- [1] M.D. Hagel, S.L. Lissel, and T.G. Brown, *Design of a seismic simulation frame for testing of masonry structures*, Proceedings of the 13th International Brick and Block Masonry Conference Amsterdam, July 4-7, 2004.
- [2] S. Ammanagi, V. Poornima, A. Sera, and R. Sunder, *Development of a digitally-controlled 3-axis earthquake shake table*, Current Science, Jan 2005.
- [3] M. Tait, I. Couloigner, M. J. Guzman, and S. L. Lissel, *Vision base deformation monitoring of a masonry wall under simulated earthquake conditions*, EUSIPCO 2007.



This page intentionally left blank

Determination of seismic transport effects on buildings

D. Makovička¹ & D. Makovička Jr.²

¹*Czech Technical University in Prague,
Klokner Institute, Czech Republic*

²*Static and Dynamic Consulting, Czech Republic*

Abstract

Building structures in the vicinity of above-ground or underground transport lines are loaded by vibrations excited by the passage of motor vehicles or trains. These vibrations propagate as technical seismicity effects through the soil to the foundations of buildings in the vicinity of their source. Due to its tuning, the building structure usually amplifies the effects of technical seismicity. These vibration levels may have an impact on people working or living in the buildings, or on sensitive equipment installed in the buildings.

This paper deals with an experimental determination of the transport actions generated by various types of vehicles (trucks, trams, trains) in different conditions (roads, railways). The purpose of the paper is to formulate actions specific for various types of transport conditions as a basis for the design of structures.

Keywords: technical seismicity, transport effect, vibration measurement.

1 Introduction

The character of the vibrations generated by transport depends particularly on vehicle weight, driving speed, how the vehicles move and in what way and direction. Another parameter is the “evenness” of the vehicle trajectory, in terms of whether it concerns the quality of the pavement surface or the horizontal and vertical railway alignment, the way in which the rails are fastened, the composition of the pavement courses, etc. The magnitude of the vibrations is influenced not only by the vibration parameters at the source but also by the composition of the environment on the way from the source to the threatened



building structure, in particular the composition of the geological environment and its mechanical properties, i.e. stiffness, wave propagation velocity, distance-dependent damping, etc. Last but not least, the magnitude of the vibration may be amplified or damped by the building structure itself and by its foundations, in particular the frequency tuning of the threatened structure.

As a rule, the intensity of vibrations propagating through the soil is higher than the intensity of vibrations due to acoustic phenomena. As far as safety is concerned, this level of vibrations generated by standard traffic is of no significance for current buildings, with the exception of historical or decrepit structures. Major cracks may originate due to the passage of very heavy vehicles or the operation of construction plant (such as vibration rollers) on new construction sites in the proximity of existing buildings.

Before transport-generated vibrations begin to cause damage to threatened structures a more serious problem may arise, i.e., the impact of vibrations on the people dwelling within these structures. Vibrations of this type usually exceed the safety limits specified by hygienic standards (see Fig. 1) well before cracks and fissures originate in the structure.

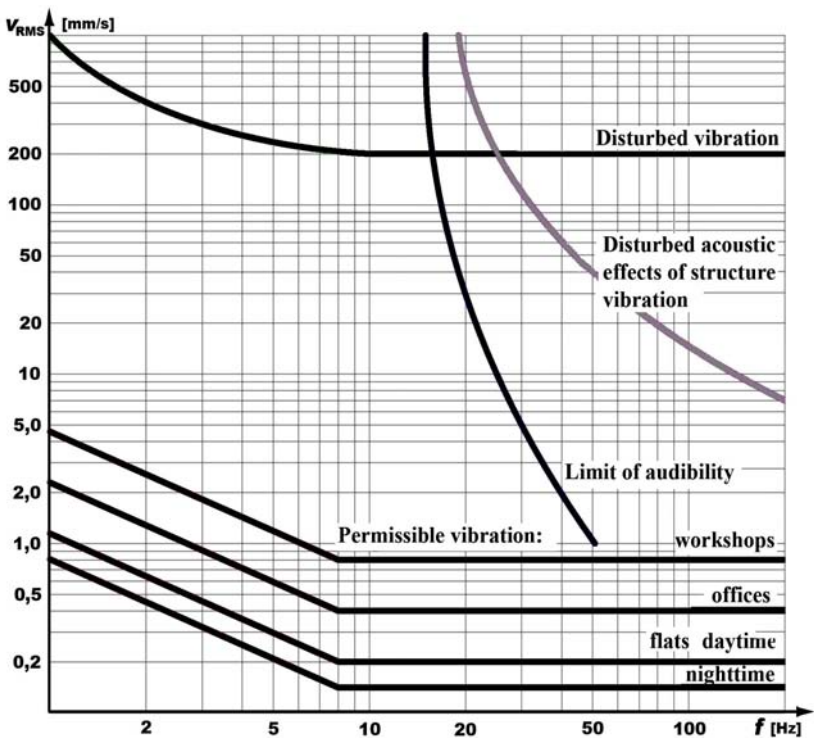


Figure 1: Comparison of hygienic limit values [4] for disturbed and permissible vibration.



2 Road transport

To ascertain the laws and common principles governing transport-generated vibrations a number of measurements of vibrations were carried out in the proximity of roads and in (usually low-rise) buildings in their environs. Selected maximum values measured at various locations are given in Table 1.

Table 1: Comparison of the acceleration amplitude level of transport-generated vibrations on various locations.

Vibration source	Location	Acceleration interval a_{peak} [mm/s ²]		Measurement distance from source [m]	Measurement location
Tramway	Older main road, Prague	18	22	12	Footway curb
Heavy truck		12	13	5	Footway curb
Subway	Prague 4, Pankrác	90	120	8	Building foundations above subway
	Prague 5, Radlice	30	65	2	Station platform
		15	17	3	Tunnel lining
	Prague 5, Hůrka	121	690	15	Pile head
Heavy truck	New main road	12	16	3	Local road
		13	14	8	Staircase
		21	23	8	House, 1st floor
	Local village road	14	19	8	House, ground floor
		30	36	8	House, 1st floor
		19	23	1	Road curb
		12	18	10	Staircase
Train	Railway and parallel main road outside the city	9	9	500	Test foundation in non-built-up area
Heavy truck		2	8	500	
Bus		2	6	500	

These measured vibration maxima are documented with a few selected vibration histories and their FFT spectra. Fig. 2 (Local village road in Table 1) shows the measured vibration levels in close proximity to the carriageway during the passage of a fully-loaded truck travelling at 20–30 km/h along a local bituminous road with relatively small surface irregularities up to 10 mm. Measurements were also performed on the ground floor of a new family house, without a basement, next to the road, in the close proximity of the front wall, and finally also on the 1st floor level of the same house above the ground floor measuring site. These measurements confirm the well-known fact that any building structure amplifies the vibrations from the road on its own natural frequencies. The transmission of traffic-generated vibrations through the subsoil to the building structure must be analyzed in detail, as a rule, although this vibration amplitude level is very low for the first group of limit states.



Another example involves a two-storey family house in the proximity of a new very high quality main road during the passage of heavy international transport trucks (TIR) the house at approx. 40–50 km/h. The measurements were made both on the footway next to the carriageway and on the ground floor and the 1st floor levels of the family house in the proximity of the outside walls. A comparison of the measured vibration levels (Table 1) shows obvious vibration amplitude amplification inside the house. This vibration amplification concerns the frequency components corresponding with the natural frequencies of the specific building, naturally different from the preceding case.

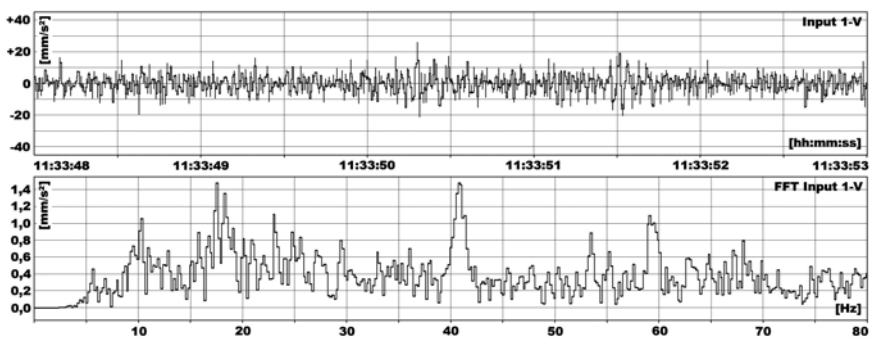


Figure 2: Time duration and FFT spectrum of vertical vibration of the action of a truck on a local village road.

The next example concerns the vibrations measured on the curb of an older good-quality urban road in Prague carrying road and tram transport, generated by the passage of heavy freight transport at 50–60 km/h and the passage of trams at 40–50 km/h (see Table 1). These measurements show that the amplitude level of transport-generated vibrations on a good quality carriageway, whether of a well-maintained older road or of a new road, is comparable.

Table 2: Comparison of effective vibration acceleration levels (mm/s²) produced by a heavy truck and a bus driving at different speeds along an uneven pavement.

Site	25 km/h		50 km/h	
	Bus	Track	Bus	Track
Ground near front façade	20.5	19.9	64.5	33.2
Outside wall at ground level	11.2	10.1	30.9	15.7
Ground floor, centre	20.3	20.8	62.9	30.1
1st floor, centre	35.0	37.3	96.2	46.7
Note: The bus was provided with air-cushioning, the truck with steel leaf springs				



For a comparison with foreign experience, Table 2 (from [3]) presents the results of Canadian measurements of a heavy truck and a bus of identical weight category along an uneven carriageway. European conditions are characterized by the diagram in Fig. 3 for various types of passage of heavy trucks and tractors along standard carriageways [1]. A comparison of the two foreign materials with our experience reveals that the foreign results concern mostly uneven, possibly unpaved road surfaces. In the case of urban or newer higher class roads, these vibrations are lower.

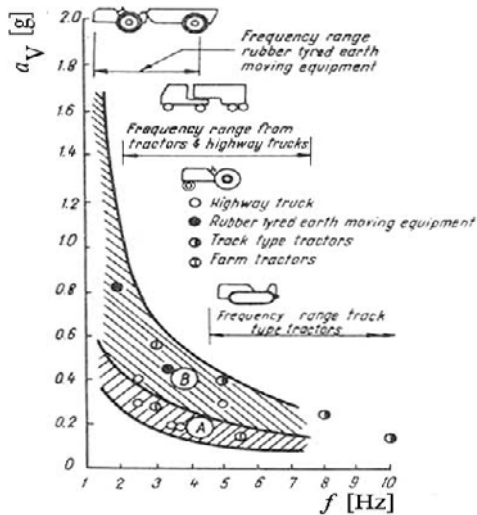


Figure 3: Characteristic vibration of various tracks crossing.

Nevertheless, a comparison of the results of all above-mentioned measurements enables us to assume that the estimates of the effect of transport on building structures are approximate and variable. It is therefore recommended to improve the accuracy of the estimates, particularly for more important structures or buildings of extensive plan or height, especially housing estates, hotels, offices and school buildings, by measurements in the specific conditions of the given site.

Generally speaking, we can conclude that the frequencies of the vibrations produced by road transport propagating through the ambient environment to the nearby buildings vary approximately between 5 and 25 Hz. Their amplitudes vary between 0.005 and 2 m/s² in terms of acceleration, and between 0.05 and 25 mm/s in terms of velocity.

The dominant vibration frequencies and amplitudes of building excitation depend on a number of factors, in the first place on the pavement, vehicle weight and design, its speed and the manner of driving (e.g. stopping), type, composition, consolidation and moisture content of the road subsoil and the route of the propagation of the vibrations to the building, road distance from the building, season (upper ground stratum frozen, dry, sodden), etc. These factors



are mutually interdependent and it is impossible to define simple relations among them.

For example, the influence of vehicle travel speed is connected with pavement evenness. Generally speaking, the more uneven the pavement surface, the greater the effect of vehicle speed on the excited vibration amplitudes. Similarly vehicle drive, e.g., vibrations generated by stopping, also depends on initial vehicle speed and pavement evenness. In the case of low velocities and a good quality pavement surface, the manner of vehicle stopping is practically insignificant in comparison with high-speed roads or uneven pavements. Table 2 reveals that the effects of the heavy truck and of the bus at a speed of 25 km/h are comparable, while at 52 km/h the truck effects are almost double those of the bus.

The amplitudes of the excitation vibrations and their dominant frequencies also depend on subsoil type and consolidation. The stiffer the subsoil and the lower the wave propagation damping in the subsoil, the higher the vibration amplitudes transferred to the threatened structure. During vibration propagation through the subsoil the natural frequencies of soils – the strata covering the base rock – become significant. In the conditions of the Czech Republic, the usual soil cover on top of the bedrock is 2–4 m in thickness, in which case the natural frequencies of the soil on top of the bedrock may approach the natural frequencies of the buildings (their walls and floors). In that case the transmission of transport-generated vibrations to the building structures is increased by the resonance effect.

3 Rail transport

The seismic load produced by surface or subsurface rail transport manifests itself, as in the case of road transport, as a kinematic load on the building foundations in the proximity of these transport routes [2]. Similarly as in road transport, the characteristics of the excitation vibrations on individual sites may differ significantly according to vehicle type, travel route, driving method, etc. The character of the excitation vibrations generated by rail transport corresponds with the passage of a train across track irregularities and the transverse sway of the vehicles due to the leeway between the wheel and the rail. The measured histories of these effects usually make it possible to identify the number of wagons in a train, the effect of the number of axles manifesting itself on the measurement records during the passage of the train past the measurement site by individual groups of amplitudes and subsequent fading.

The rate of differences in the measured vibrations produced by the passage of underground trains is revealed by a comparison of a few selected records and their frequency spectra in Prague conditions.

Fig. 4(a) shows a record of the measurements on Line C of the Prague Underground Railway in Pankrác. Fig. 4(b) shows a record of the measurements on a newer section of Line B in Radlice. For the sake of comparison both sites are included in Table 1 together with the vibrations measured on Line B near Hůrka station, where the track emerges from the tunnel to the bridge structure,



where the magnitude of the vibrations is also influenced by a change in the stiffness of the track subbase. The amplitudes of the vibrations generated by the passage of underground trains along the tunnel are approximately in the ratio of 2 : 1 (see Table 1), though the frequency structure of the vibrations is entirely different. The vibrations measured on the sites next to the underground lines passing approximately at the same depth below ground level (shallow lines) manifest themselves by different dominant frequencies – from 35 to 65 Hz in case of the older Line C, and from 48 to 80 Hz in case of the newer section of Line B. The structure of the tunnel tube and of the permanent way obviously exercises a dominant influence on the character of the excitation frequencies.

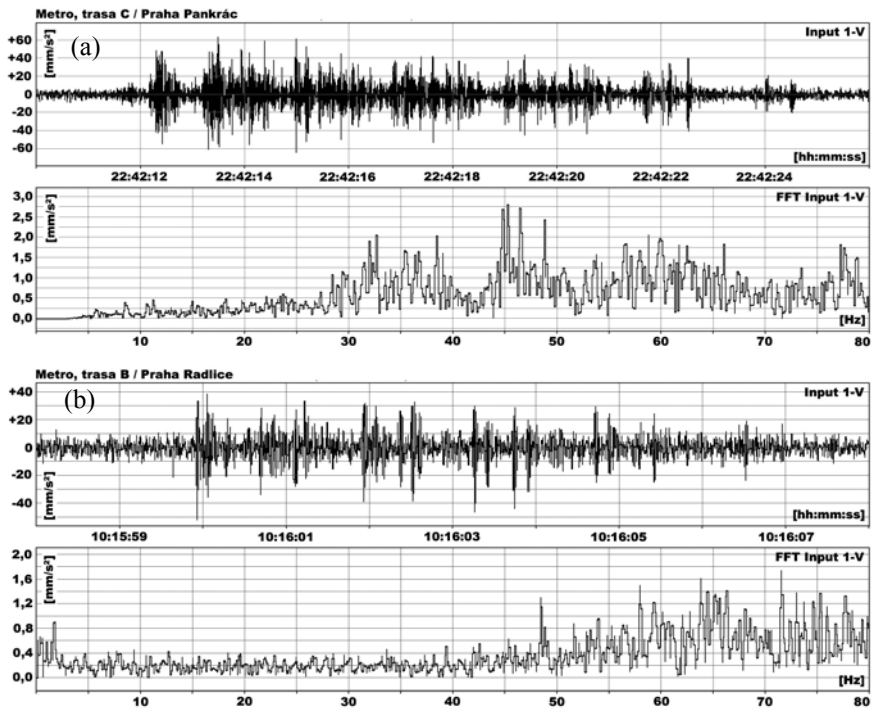


Figure 4: Time duration and FFT spectrum of vertical vibration of the action of a train, measured on a building foundation on subway Line C (a) and on a station platform on Line B (b).

In the course of the transmission of vibrations from the deep underground lines to the ground surface, the amplitude excitation level is usually reduced. For instance, the construction of the Hilton Hotel in Prague was preceded by vibration measurements on the level of the underground tunnel lining between stations and on the test foundation on the free site. An example of the frequency spectra of the measured vertical vibrations is shown in Fig. 5. A comparison of the response level at the tunnel structure level and at ground level reveals that the



frequency structure of the excitation signal in the course of its passage through the geological environment changes due to strata thickness, distance from the source, etc. The spectrum of the test foundation measurements also contains the frequencies of the location of this foundation on the subsoil (Fig. 5 shows the peak of the natural frequency of the foundation vibrations on the subsoil at a rate of some 69 Hz). This frequency is insignificant for the assessment of the excitation vibration of the future building.

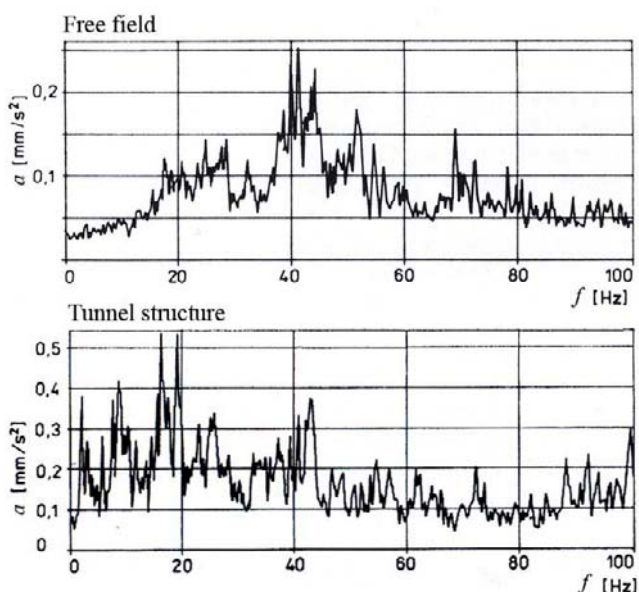


Figure 5: Comparison of measured spectra of free field and tunnel structure vibration.

When considering the effects of the movement of a train along new lines on the magnitude of the excited vibrations, the train stopping at the station results in an approximately ninefold increase in the vibration amplitudes in comparison with a smooth train run through the station.

When monitoring vibrations in the rail track and in the tunnel tube structure, the rail track vibrations need not always be higher than those of the tunnel or station structure; it depends on the spectrum of the natural frequencies of the respective tunnel structure (by analogy with the vibrations transmission of vibrations from the pavement to the nearby buildings in the case of road transport). As a result, the rail track vibrations may be increased or reduced in the frequencies corresponding with the natural tunnel frequencies.

Generally speaking, we can conclude that in Prague conditions effective vibrations (effective vibration acceleration) due to train travel on the underground railway are of the order of tenths of mm/s^2 , if these vibrations propagate through the geological environment over greater distances and from major depths. Momentary peak amplitudes of vibration acceleration vary within



units to tens of mm/s^2 . The dominant frequency components of the vibrations vary from 40 Hz upwards. In building foundations connected firmly with the underground railway structure e.g. through RC structures, concrete grout escaped from the foundations, base rock, an increase in the amplitudes of the excitation vibrations from this boundary is obvious.

4 Influence of landscaping on vibration transmission

Most of the total energy from shallow or surface sources (whether transport or stationary sources, such as machine foundations, etc.) is transmitted to the ambient structures in the form of surface waves of the ground or of various material subsoil strata. The main part, approx. 67%, consists in Rayleigh surface waves, with 26% falling to shear waves and 7% to longitudinal waves.

Due to the surface waves, vibration propagation to building structures is decisively influenced by the form of connection of the carriageway and the threatened building by a paved surface (pavement, concrete surfaces) or by an unpaved surface (lawns, gardens).

The effect of this type of connection on vibration propagation from the carriageway to the building structures is shown by the yielded experimental results of the measurements of the effect of a vibration roller on ambient buildings, firmly connected with the road by paved footways or separated from the footway by a 5 m green strip. In the case of paved surfaces, the vibrations were approximately twice as high (Fig. 6).

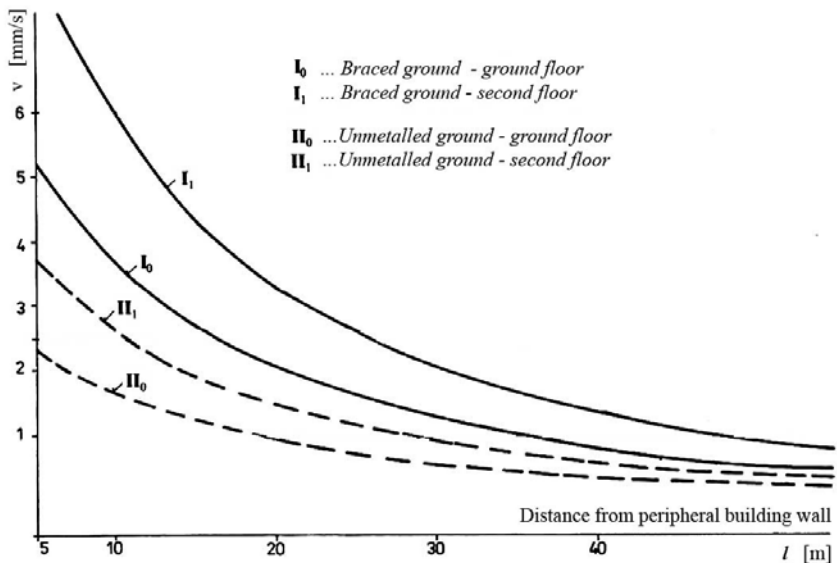


Figure 6: Measured vibration velocity in a building for various distances of the roller from a two-storey masonry structure.



5 Conclusion

The purpose of this paper was to compare the effects of surface and subsurface transport on the building structures in their vicinity, and an informative survey of their amplitude and frequency characteristics, including possible modifications to these excitation vibrations due to the response of the structure to their effect. The problem is highly complicated, and does not always permit an explicit answer without a detailed analysis, because both the excitation character and the response of the structure depend on a number of parameters not only at the source and at the threatened structure but also en route from the source to the threatened structure.

For these reasons, and in accordance with foreign experience it seems most adequate to measure the vibration level at the source, en route or at the threatened structure. If the structure is under construction, it is possible to make the measurements at its foundations or in their close proximity. The measurements will be followed by an assessment of the response of the threatened structure, and possibly by a proposal of measures aimed at reducing the vibrations of the whole structure or a part of it.

The response values measured on individual sites need not be significantly similar; on the contrary, they may differ considerably. The vibration values given here may, consequently, be considered as a first approach to actual values of a specific structure in specific conditions.

Acknowledgements

This research was supported as a part of GA projects No. 103/06/1521 and CZ.04.3.07/3.2.01.3/3323 for which the authors would like to thank the Grant Agency of the Czech Republic, the European Social Fund, the Capital City of Prague and the State Budget of the Czech Republic.

References

- [1] Major, A., *Dynamics in Civil Engineering*. Akadémiai Kiadó, Budapest 1980.
- [2] Makovička, D. & Makovička, D., Jr., Response analysis of a building loaded by technical seismicity propagating from a tube railway structure, *Earthquake Resistant Structures V*, ed. Brebbia, C.A., WIT Press, Southampton, 2005, pp. 675-684.
- [3] Osama Hunaidi, Traffic Vibrations in Buildings, *National Research Council of Canada*, No. 39, June 2000.
- [4] ISO 2631-2:1989, Evaluation of human exposure to whole-body vibration – Part 2: Continues and shock-induced vibration in buildings (1 to 80 Hz).



Section 10

Retrofitting

This page intentionally left blank

Towards a European code for seismic assessment and strengthening of existing buildings

S. E. Dritsos

Department of Civil Engineering, University of Patras, Patras, Greece

Abstract

In Europe, actions towards preparing a code document regarding assessment and strengthening of existing reinforced concrete structures are already present in the draft document Part 3 of EC8. However, for the case of strengthening by the addition of new reinforced concrete, specific provisions, to check the capacity of the connections between contact surfaces, are missing.

Structural design of strengthened concrete elements can be placed into the framework of the currently known processes of design, which are used for new constructions, supplemented by a crucial investigation at the interface between the contact surfaces, to ensure that failure in each strengthened element precedes failure at the interface, between the old and the new material. For that reason, shear forces and shear resistances at the interfaces, between the old and the new element, must be examined. An evaluation of the shear force that develops between the contact surfaces can be obtained in a similar way as for steel and concrete composite structural elements. The main mechanisms that contribute to the shear resistance at the interface are: (a) concrete-to-concrete adhesion, (b) concrete-to-concrete friction, (c) connecting action from steel bars placed across the interface between the old and the new concrete and (d) bent steel bars welded between the bars of the old and the new concrete. The total shear resistance between contact surfaces can be found by summing the individual shear resistances that are mobilised by each individual mechanism for a common interface slip. To prevent a brittle failure at the interface, a minimum amount of steel shear connectors in the form of dowels or bent steel bars must be provided.

Keywords: aseismic code, assessment, buildings, design, reinforced concrete, repair, retrofitting, strengthening.



1 Introduction

Various methods and techniques are used in practice to enhance the seismic capacity of reinforced concrete (R.C.) structures (fib [9]; Dritsos [5, 6]; Tsonos [16]; CEB Bul. N° 162 [3]). However, analytical tools to manipulate the subject are rare and the absence of a specific design code, regarding retrofitting of existing old structures makes a complex and difficult problem (Tassios [12]; Tsonos [15]; Apostolopoulos [1, 2]).

In Europe, actions towards preparing a code document regarding assessment and strengthening of existing R.C. structures, has already present a draft document of EC8-Part 3 [7] revising the existing Part 1.4 of EC8 [8]. However, for the case of strengthening by the addition of new reinforced concrete, specific provisions, to check the capacity of the connections between contact surfaces, are missing.

In the following, supplemental relevant material, regarding the above issue, is provided for possible use in the final code edition.

2 Control of a sufficient connection between contact surfaces

Structural design of strengthened concrete elements can be placed into the framework of the presently known processes of design that are used for new constructions, supplemented by a crucial investigation at the interface between the contact surfaces, to ensure that, failure in each strengthened element precedes failure at the interface, between the old and the new material. (Tassios [13]; Chronopoulos [4]; EC 8 [8]; GRECO [10]; Dritsos [6]).

Load transfer mechanisms between the old and new materials must be capable of transferring the tensile, compressive and shear stresses that develop at the interface.

As far as interface tensile stresses are concerned, the transfer can be guaranteed if the developed stresses are lower than the tensile strength of the weakest concrete. If not, an appropriate quantity of reinforcement or anchor bars crossing to the contact surface should be provided, as specified later in this paper.

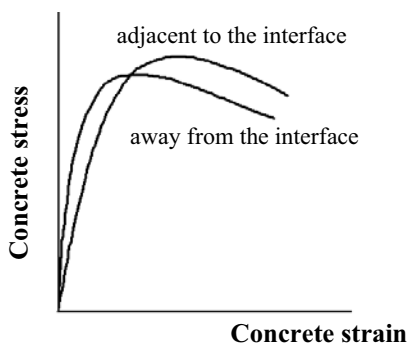


Figure 1: Stress against strain diagram for interface concrete.



Regarding concrete-to-concrete direct compression, a full continuity compression transfer can be expected at the interface if adequate treatment measures have been performed on the old concrete surface (such as roughening). However, as shown in figure 1 (CEB Bul. N° 162 [3]), a lower modulus of elasticity should be considered for concrete adjacent the interface, as higher deformations develop due to the mechanical treatment of the existing concrete and contact and compaction imperfections. Obviously, the interface compressive strength can be considered to equal the lowest compressive strength of the contact materials.

In order to guarantee a sufficient connection between contact surfaces, the check for safety at the ultimate limit state can be expressed symbolically by the following equation of safety:

$$S_d \leq R_d \quad (1)$$

where: S_d is the design action effect and R_d is the design resistance. This control will include checking the shear force and the shear resistance at the interface between the old and the new element. That is to say, the following relationship must be satisfied:

$$V_{Sd}^{interface} \leq V_{Rd}^{interface} \quad (2)$$

where: $V_{Sd}^{interface}$ is the shear force acting at the interface and $V_{Rd}^{interface}$ is the shear resistance at the interface.

Obviously, a guaranteed connection that avoids premature failure would be desirable. This would be because it represents the critical factor for the effectiveness of the intervention and would ensure an acceptable degree of reliability for calculations.

If failure between the contact surfaces precedes failure of the strengthened element, the load bearing capacity of the connection will determine the load bearing capacity of the strengthened element. In addition, the load bearing capacity of the strengthened element cannot be considered smaller than that of the original unstrengthened element.

The control between contact surfaces along the whole length of the strengthening structural element should be based on average values of $V_{Sd}^{interface}$ and $V_{Rd}^{interface}$ corresponding to various segments of length l_{i-j} (i and j for successive segments) into which the element has been divided. That is to say:

$$V_{Sd(i-j)}^{interface} \leq V_{Rd(i-j)}^{interface} \quad (3)$$

The length of each successive segment should not be greater than twice the height of the cross section of the element. However, the process can be facilitated if the lengths of segments are also fixed at characteristic cross sections. As such, sections dividing an element should be placed at the following locations: (a) at the largest positive or negative bending moment, (b) at the supports, (c) at positions of point loads, (d) where there are abrupt changes in cross section and (e) at the ends of cantilevers.



2.1 Shear forces acting at the interface

An evaluation of the shear force that develops between the contact surfaces can be obtained by analysing each segment of the strengthened element assuming monolithic behaviour (by approximately calculating the shear stress at the interface using mechanics theory). Alternatively, the more accurate calculation method that is applied for steel and concrete composite structural elements could be used. Figure 2 schematically illustrates the shear force that develops between contact surfaces.

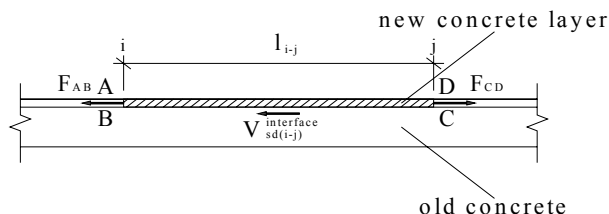


Figure 2: Shear force at the interface.

If a structural element has been strengthened with the new layer of concrete, the size of the shear resistance between the contact surfaces, for a segment length of l_{i-j} , can be determined by considering the equilibrium of forces in the new concrete segment ABCD of figure 2. That is:

$$V_{Sd(i-j)}^{interface} = V_{Sd}^{BC} = F_{AB} - F_{CD} \quad (4)$$

A process of section analysis can be used to determine the magnitudes of the forces F_{AB} and F_{CD} . That is, by taking sections through the whole element at positions i and j respectively and determining the internal tensile or compressive forces corresponding to layer sections AB or CD.

2.2 Interface shear resistance

Four mechanisms contribute to the shear resistance at the interface. These are concrete-to-concrete adhesion, concrete-to-concrete friction, the connecting action from either steel bars placed across the interface between the old and the new concrete or bent steel bars welded between the bars of the old and the new concrete. These four mechanisms can be subdivided into the two groups of unreinforced and reinforced interfaces, depending on whether or not additional steel is placed across the interface or welded between the bars of the old and the new concrete. In general, the shear resistance developed at the interface depends on the amount of slippage at the interface.

2.2.1 Unreinforced interface shear resistance

The two mechanisms acting at an unreinforced interface are adhesion and friction. Figure 3 (CEB Bul. N^o. 162 [3]), presents a plot of the mobilised shear resistance (τ) against interface slip (s) and it can be seen that the maximum adhesion values are achieved for low interface slip values (in the region of



0.02 mm), while friction becomes important for much higher slippages. Therefore, the maximum resistances from adhesion and friction do not coincide and cannot be considered to act together.

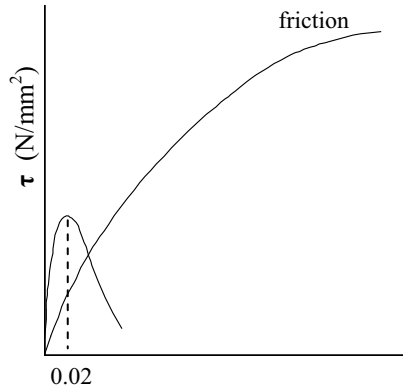


Figure 3: Mobilised shear against slip.

The main parameters affecting the adhesion at the interface are the roughness and treatment of the joint surface and the tensile strength of the weaker concrete. The Greek Retrofitting Code (GRECO [10]) accept the following design values for the magnitude of adhesion at concrete interfaces:

- 0.25 f_{ck} for smooth interface conditions
- 0.75 f_{ck} for rough interface conditions
- 1.00 f_{ck} if a resin bonding agent is used at the interface
- 1.00 f_{ck} if the additional concrete is shotcrete

where: f_{ctk} is the characteristic value of the tensile strength of the weaker concrete.

The parameters that affect concrete-to-concrete friction are the size and shape of the aggregates if exposed (large angular aggregates are better) and the surface roughness of the original column (rougher surfaces have greater areas of surface contact). Additional parameters include the concrete compressive strength, the external normal compressive stress (a higher normal stress gives a higher shear stiffness) and if the loading is cyclic or not (cyclic loading quickly deteriorates the contact surfaces giving a larger slip or a lower shear response). Representations that model concrete-to-concrete friction (τ_f) can be found in the literature. In an analytical work presented by Tsoukantas and Tassios [17] the following formula was proposed:

$$\left(\tau_f / \tau_{fu} \right)^4 - \left(\tau_f / \tau_{fu} \right)^3 = 0.3s - 0.03 \quad (5)$$

Using the above equation it can be found that the shear resistance due to friction (τ_f) reaches a maximum when the relative slip is in the region of 1.75 mm. Moreover the maximum value of the design concrete-to-concrete shear resistance due to friction (τ_{fu}) can be calculated from the following equation:



$$\tau_{fu} = 0.4(f_c^2 * \sigma_c)^{1/3} \quad (6)$$

where: f_c is the compressive strength of the weaker concrete and σ_c is the interface compressive stress.

2.3 Reinforced interfaces

2.3.1 Clamping action of transversal reinforcement

When a steel bar crosses the interface between old and new concrete, an additional action that may occur is clamping action. This action would take place when the surface of the old concrete has been roughened, or shotcrete has been placed and if the steel bar is adequately anchored. As it is demonstrated in figure 4 (Tassios and Vintzeleou [14]) when a shear stress is applied, a slip is produced and the contact surface between the old and the new concrete must open as one surface rides up over the other due to the roughness.

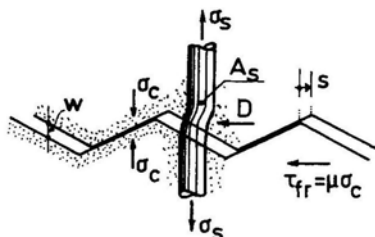


Figure 4: A qualitative description of friction resistance τ_f due to clamping action.

Therefore, a tensile stress is activated in the steel bar, which in turn produces a corresponding compressive stress, or clamping action, and a frictional resistance is mobilised. Equation 6 can be modified in order to take into account the additional frictional resistance mobilised by clamping action, as follows:

$$\tau_{fu} = 0.4(f_c^2 * (\sigma_c + \rho_d f_y))^{1/3} \quad (7)$$

where: ρ_d is the total cross sectional area of the shear connectors divided by the cross sectional area between the contact surfaces and f_y is the yield stress of the transversal bars.

2.3.2 Dowel action of interface reinforcement

Transverse resistance of steel bars crossing the contact interface (fig. 5), is commonly referred as dowel action. Parameters that affect dowel action include the concrete strengths of the new and the old concrete, the yield stress of the dowel (f_y), the diameter of the dowel (d_b) and the amount of dowels placed. The maximum interface resistance is obtained only if the dowels are adequately embedded in the old and the new concrete at depths of at least 8 times the dowel diameter. In addition, measures should be taken to avoid failure due to placing dowels too close to the edge of the concrete.



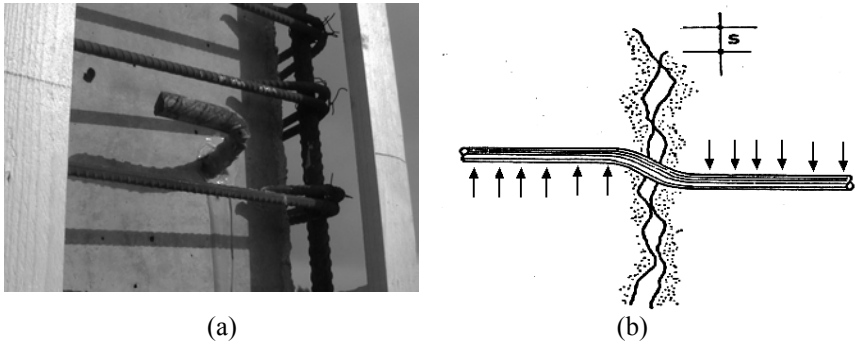


Figure 5: (a) Use of dowels in concrete jackets, (b) Dowel action.

At least 3, 5 or 6 times the dowel diameter are respectively required from the edge of the original element or the top or base of the original element or jacket if a partial jacket is placed. In an analytical work presented by Vintzeleou and Tassios [18] the following model was proposed concerning one dowel bar:

$$\text{For small slip values: } (V/V_u) = 200 * \sqrt{f_c/f_y} * (s/d_b) \leq 0.4 \quad (8)$$

$$\text{For higher slip values: } (V/V_u) = \frac{4}{3} * \sqrt[4]{s/d_b} \leq 1.0 \quad (9)$$

where: V is the magnitude of the mobilized dowel resistance due to a slippage s , V_u is the maximum value of V and d_b is the dowel diameter.

The maximum value of the design shear resistance from dowel action (V_u) can be calculated from the following equation (Rasmussen [11]; Vintzeleou and Tassios [18]):

$$V_u = 1.3 * d_b^2 * \sqrt{f_c * f_y} \quad (10)$$

If earthquake action is expected, it would be conservative to remove the value of 1.3 from equation (10) (GRECO [10]).

2.3.3 Action of welded bent steel bars

A practice that is commonly used and has a good reputation, is to weld bent steel bars between the reinforcement of the old concrete and the new concrete (fig.6(a)). When there is relative slip between the old and the new concrete, a part of the force in the old bar is transferred to the new bar via the bent bar. Figure 6b conservatively demonstrates the mechanism (CEB Bul. N^o. 162 [3]; Tassios [12]). When there is slippage (s) at the interface, one of the angled legs of the bent bar is elongated by a length of $s/(\sqrt{2})$ while the other angled leg is shortened by the same length. Therefore, the respective tensile or compressive strains (ϵ_{sb}) and stresses (σ_{sb}) are:

$$\epsilon_{sb} = \frac{s/\sqrt{2}}{\sqrt{2}h_s} = \frac{s}{2h_s} \quad \text{and} \quad \sigma_{sb} = E_s \frac{s}{2h_s} \leq f_{yb} \quad (11)$$



where: h_s is the distance between the centrelines of the outer arms of the bent steel bar (fig.6b), E_s is the modulus of elasticity for the steel bar and f_{yb} is the characteristic value of the yield strength of the bent steel bar.

By considering the equilibrium of forces, the force that can be transferred to the new reinforcement (T_s), expressing in other words the shear capacity of the interface, can be derived:

$$T_s = A_{sb} * E_s (s / \sqrt{2} h_s) \leq T_{sy} = \sqrt{2} A_{sb} f_{yb} \quad (12)$$

where: A_{sb} is the cross sectional area of the bent bar and T_{sy} is the force required to yield the weaker longitudinal bar.

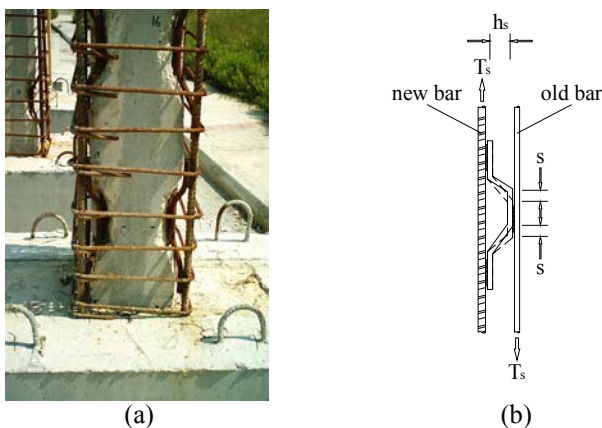


Figure 6: (a) Use of bent bars in concrete jackets, (b) bent bar model.

3 Conclusions and proposal for design

The total shear resistance between contact surfaces can be found by summing the individual shear resistances that are mobilised by each individual mechanism for a common interface slip. Figure 7a presents a plot of the superposition of slippage from all the mechanisms discussed above for the transfer of shear stress at the interface obtained from available literature (CEB Bul. N° 162 [3]; Tsoukantas and Tassios [17]; GRECO [10]) and represents typical experimental results. In order to simplify calculations, bilinear diagrams of the type OAB of figure 7(a) could be applied. Elastic simplifications, as in curves OA_1 or OB of figure 7(b), could be used to facilitate the analysis. More precise results could be obtained by using elasto-plastic diagrams such as curve OA_1BB_1 of figure 7(b). In general, the remaining shear resistance (τ_{res}) could be considered as insignificant.

For structural elements that resist seismic actions, it may be useful (and it would simplify calculations) if the mechanisms of adhesion and friction were ignored and only the shear resistance from dowels or other shear connectors is taken into consideration. In other elements that do not resist seismic action (for example concrete slabs), it could be considered that shear connectors are



required only when, in some region of the structural element, the shear stress between the contact surfaces exceeds the shear strength from adhesion or friction.

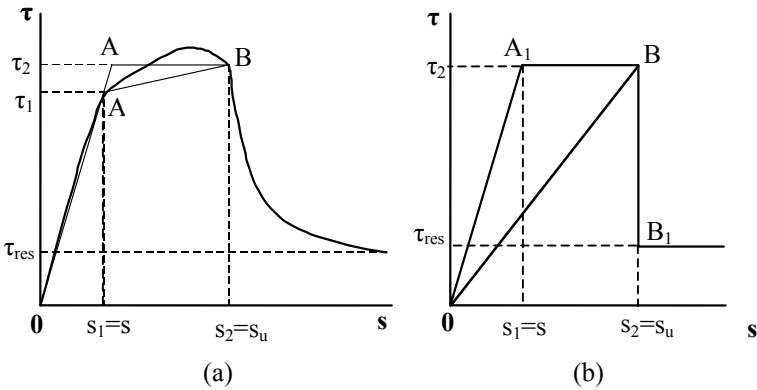


Figure 7: Representations of the longitudinal transfer of shear stress: a) typical experimental result and bi-linear simplification and b) elastic and elasto-plastic simplifications.

In order to prevent a brittle failure at the interface, a minimum amount of steel shear connectors in the form of dowels or bent steel bars are required for concrete-to-concrete connections. The required percentage can be calculated in a similar way to that of determining the minimum shear reinforcement in monolithic elements and the following relationship has been proposed (Dritsos [6]; GRECO [10]):

$$\rho_d \geq \max (0.18 f_{ctm}/f_{yk}, 0.12\%) \quad (13)$$

where: f_{ctm} is the average tensile strength of stronger concrete and f_{yk} is the characteristic yield strength of the steel shear connectors or bent down bars.

References

- [1] Apostolopoulos, C. and Michalopoulos, M., "The Impact of Corrosion on the Mechanical Behavior of Steel Undergoing Plastic Deformation". *Materials and Corrosion*. Vol. 58. N^o. 1, p.p. 5-12, 2007.
- [2] Apostolopoulos, C., "Mechanical Behavior of Corroded Reinforcing Steel Bars S500s Tempcore Under Low Cycle Fatigue". *Construction and Buildings Materials and Corrosion*, 2006, (in print).
- [3] CEB Bul. N^o. 162, "Assessment of Concrete Structures and Design Procedures for Upgrading (Redesign)". Bulletin D'Information, *Comite Eurointernational du Beton*, Paris, 1983.
- [4] Chronopoulos, M. P., "Guidelines and Practical Rules for Redesign of Repaired/Strengthened Reinforced Concrete Elements". Proceedings of the 7th Greek Congress on Concrete, Patras, Greece. Vol. 2, pp. 201-210. *Technical Chamber of Greece*, 1985 (in Greek).



- [5] Dritsos, S. E., "Seismic Retrofit of Buildings: A Greek Perspective". *Bulletin of the New Zealand Society for Earthquake Engineering*, Vol. **38**, N° 3, pp. 165-181, 2005a.
- [6] Dritsos, S. E., "Repairs and Strengthening of Reinforced Concrete Structures". *University of Patras*, Patras, Greece., 2005b (in Greek).
- [7] EC 8, "Eurocode 8: Design of Structures for Earthquake Resistance. Part 3: Assessment and Retrofitting of Buildings". Draft N° 5. prEN 1998-3: 2004 (E). *CEN Technical Committee CEN/TC250*, Brussels, 2004.
- [8] EC 8, "Eurocode 8: Design Provisions for Earthquake Resistance of Structures: Part 1-4: Strengthening and Repair of Buildings". prENV 1998-1-4: 1995. *CEN Technical Committee CEN/TC250*, Brussels, 1995.
- [9] fib, "Seismic Assessment and Retrofit of Reinforced Concrete Buildings". State-of-art Report, Bulletin 24. *Federation International du Beton*, Lausanne, 2003.
- [10] GRECO, "Greek Retrofitting Code". Second draft version by the *Greek Organization for Seismic Planning and Protection*, Greek Ministry for Environmental Planning and Public Works, Athens, Greece, 2005 (in Greek).
- [11] Rasmussen, B. H., "The Carrying Capacity of Transversely Loaded Bolts and Dowels Embedded in Concrete". *Bygningssstatistiske Meddelelser*, Vol. **34**, N° 2, 1963.
- [12] Tassios, T. P., Postgraduate studies course notes on: Theory for design of repaired and strengthened structures. School of Civil Engineering, *National Technical University of Athens*, Athens, Greece, 2004 (in Greek).
- [13] Tassios, T. P., "Physical and Mathematical Models for Redesign of Damaged Structures". *Proceedings of the IABSE Symposium: Strengthening of Building Structures Diagnosis and Therapy*, Venice, pp. 30-52, 1983.
- [14] Tassios, T., Vintzeleou, E., "Concrete-to-Concrete Friction". *Journal of Structural Engineering*, Vol. **113**, N° 4, paper N° 21442, 1987.
- [15] Tsonos, A. G., "Seismic Rehabilitation of Reinforced Concrete Joints by the Removal and Replacement Technique". *European Earthquake Engineering*, Vol. **3**, pp. 29-43, 2001.
- [16] Tsonos AG., "Lateral Load Response of Strengthened Reinforced Concrete Beam-to-Column Joints". *ACI Structural Journal*, Vol. **96**, N° 1, pp. 46-56, 1999.
- [17] Tsoukantas S. G. and Tassios T. P., "Shear Resistance of Connections between Reinforced Concrete Linear Elements". *ACI Structural Journal*, Vol. **86**, N° 3, pp. 242-249, 1989.
- [18] Vintzeleou, E. N. and Tassios, T. P., "Mathematical Models for Dowel Action under Monotonic and Cyclic Conditions". *Magazine of Concrete Research*, Vol. **38**, N° 134, pp. 13-22, 1986.



Flexural retrofitting of reinforced concrete bridge pier type cross-sections with carbon fiber reinforcing plastics

G. C. Manos & V. Kourtides

*Laboratory of Strength of Materials and Structures,
Department of Civil Engineering, Aristotle University,
Thessaloniki, Greece*

Abstract

Results and conclusions are presented from an experimental investigation with identical column-specimens, which were constructed with prototype materials, having a height of 1600mm and a cross section 300mm by 200mm. They represent models of a part of a bridge-pier near its base, with or without partial confinement of Carbon Fibre Reinforcing Plastic (CFRP) layers. They were subjected to compressive loads as this type of stress field is expected to develop at the base of such vertical members under combined vertical loads and seismic actions, where undesired compression failure may develop. The retrofitting of this type of reinforced concrete cross sections, with h/b ratio larger than 1.5, is aimed at prohibiting, up to a point, such compression failure. This type of partial confinement may also be applied to retrofitting similar vertical structural members with non-accessible sides. With the successful application of this partial confinement, an increase of almost 50% was observed in the compression capacity of the test specimens. Moreover, the deformability of these specimens was substantially increased, demonstrating the effectiveness of this type of partial confinement. It was also demonstrated from the experimental sequence that critical factors for the effectiveness of this partial CFRP confinement were the type of anchorage of the CFRP layers on the body of the cross-section and the number of CFRP layers.

Keywords: carbon fibre reinforcing plastics, retrofitting, bridge pier, reinforced concrete.



1 Introduction

The upgrading of reinforced concrete (R/C) cross-sections, with one side rather longer than the other ($h/b > 1.5$), by partial application of CFRP (Carbon Fibre Reinforcing Plastic) confinement is investigated here (Figures 1 and 2). This partial application of CFRP confinement is aimed at the retrofitting of bridge-pier type R/C cross-sections in order to prohibit, up to a point, the development of premature compressive failure at the base of the pier due to combined compression and flexure from seismic loads (Figure 1, Kawashima [1]). The performance of such structural elements was studied extensively in the past (Pinto [4]). This type of partial confinement may also be applied to upgrading vertical structural members with non-accessible sides. Design guidelines for rectangular FRP jackets applied on rectangular columns have been proposed with the limitation that the cross-sections have aspect ratio $h/b < 1.5$ (Tsonis [5]). For higher aspect ratios it is recommended designing a circular or oval jacket. However, it is expected that for rectangular cross sections with aspect ratios larger than 1.5 the radius of a circular or rectangular jacket will be too large and will result in ineffectual confinement and will prove costly and impractical. For this reason it is desirable to investigate alternative schemes for increasing the confinement of rectangular cross-sections with relatively large aspect ratio without resorting to complete circular or oval jackets. Such a scheme is studied here using CFRP layers that do not extend all around the cross-section (Figure 2 “*partial confinement*”). To compensate for the fact that the CFRP layers do not enclose the cross-section entirely, anchorage of these layers must be provided, as shown schematically at the bottom of figure 2. To this end, a laboratory investigation was carried out to study the effectiveness of such partial confinement together with alternative anchorage schemes. As will be explained, this effectiveness was tested by subjecting the specimens only to compressive loads. Despite this limitation, as will be demonstrated from the results of this investigation, the most significant aspects of the critical factors for this “*partial confinement*” scheme were brought to light.

2 Test specimens

The initial cross-section, which formed the basis of the tested specimens, had an aspect ratio h/b equal to 2.5. This is a rectangular cross section of a bridge pier model structure, which was tested both at the laboratory and at the Volvi-Greece European Test Site in the framework of the European project Euro-Risk (Manos et al. [2, 3]). This cross section was intentionally designed to develop flexural mode of failure at the base of the pier; moreover, it was desirable to find ways to retrofit such specimens by prohibiting premature compression failure at the base by means of partial CFRP confinement. The effectiveness of the partial CFRP confinement is studied by subjecting the tested specimens only to pure compressive loads. This type of stress field is expected to develop in the base of such vertical members under combined vertical loads and seismic actions, where undesired compression failure may develop (figure 1). In order to limit the



maximum level of compressive loads required to bring to failure such a cross-section with the loading arrangements available, the tested specimens had a cross section (figure 2) of 200mm by 300mm instead of 200mm by 500mm of the initial cross-section for the bridge pier specimens tested both at the laboratory and at the test site under combined compression and flexure Manos et al. [2, 3]). Moreover, in order for the tested specimens to form compression failure at the same part of the cross-section where such failure would develop at the base of the initial bridge pier model (figure 1), one part of the tested cross-section was left identical to the initial cross-section (the one that is marked in figure 3 as weak) whereas the remaining part was strengthened both with longitudinal and in particular, with transverse reinforcement (the one marked in figure 3 as strong). In this way, with the compression capacity of the weak part being smaller than that of the strong part, the compression failure was expected to develop at the weak part. This proved to be correct during the experiments, as will be shown in the following sections. The CFRP partial confinement was applied at the weak part, as is shown at the right hand side of figure 3 with the anchor bolts being applied at the part of the CFRP layers attached to the strong part. By studying the resulting bearing capacity and mode of failure under compression of the tested specimens (with or without partial confinement) the effectiveness of such a repair scheme could be demonstrated and classified as listed in Table 1.

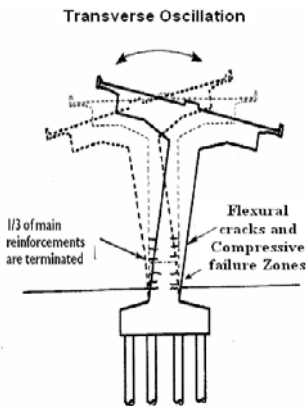


Figure 1: Bridge pier compression failure mode.

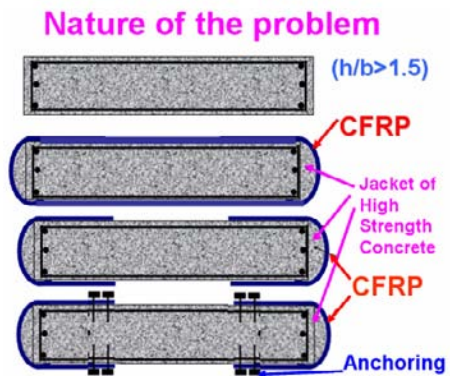


Figure 2: Partial confinement provided by CFRP layers anchored with bolts.

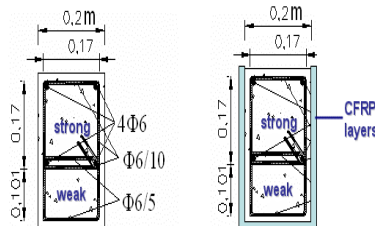


Figure 3: Test specimens without and with partial CFRP confinement.



Table 1: Effectiveness of the partial confinement.

Compression failure mode	Capacity Increase *	Effectiveness
Weak part	Small increase	Low
Weak part	Considerable increase	Considerable
Both weak and strong parts	Substantial increase	Very effective

* (compared to the unconfined specimens)

2.1 Construction of test specimens

Ten identical specimens were constructed and eight of them were used in the current experimental sequence (see Table 2). All specimens were reinforced in the same way and were cast at the same time with the same mixture aiming for similar plain concrete strength values. The total height of the specimens was 1600mm. They had a mid-part height of 580mm that was left to develop the compression failure. The cross section of this mid-part was the one shown in figure 3, including the two distinct parts (the weak and the strong). The two edges (top and bottom) of the specimens, with a height of 510mm each, were confined during the experiment with strong steel brackets covering these parts from all sides thus prohibiting any compressive failure developing at those two edges (see figure 4). The partial CFRP confinement was attached only on the three sides of these specimens covering the weak part of the cross-section and leaving the fourth side (of the strong part) free without any CFRP layers (figure 3).

Table 2: Test specimen with their corresponding concrete strength.

Virgin Specimens	CFRP Confinement	Repaired Specimens	Plain Concrete Strength (Mpa)*
Specimen 1 Test 1	No	Test 2 3+(2) CFRP	28.0
Specimen 1a Test 1	3 CFRP layers	Test 2 3 GFRP	25.8
Specimen 3 Test 1	No	Test 2 3+(2) CFRP	27.6
Specimen 3a Test 1	5 CFRP layers	Test 2 5 CFRP	27.6
Specimen 4 Test 1	5 CFRP layers		27.7
Specimen 4a Test 1	5 CFRP layers		27.7
Specimen 5 Test 1	5+(2) CFRP layers	Test 2, 5+2 CFRP Test 3, 5+2 CFRP Test 4 7 CFRP	27.6
Specimen 5a Test 1	No	Test 2, 7 CFRP Test 3 7+4 CFRP	25.8



There were 8 virgin specimens, namely 1, 1a, 3, 3a, 4, 4a, 5, and 5a (Table 2). These specimens were tested in their virgin state in which some of them were without partial CFRP confinement whereas the rest had the partial CFRP confinement applied to them from the beginning. The second column of table 2 indicates the partial confinement condition of the virgin specimens. The testing sequence of these virgin specimens is signified as Test 1. Most of these specimens were repaired after they had reached their limit state during their previous test. In all the repaired specimens the CFRP partial confinement was applied. This sequential test number of the repaired specimens is signified as Test 2 (for the 1st repair), Test 3 (for the 2nd repair) etc. (see column 3 of Table 2). In the same column the number of CFRP layers used in the partial confinement for the repaired specimens is also indicated. The total number of specimens, virgin and repaired, was seventeen. In Table 2 the unconfined concrete compressive stress is also listed, found from cylinders with diameter 150mm and 300mm height; these cylinders were obtained during the casting of each virgin specimen.

The anchorage of the CFRP layers was applied along the two long sides, which were attached to the sides of both the weak and the strong parts of the cross-section (figure 3). The main load that was applied was axial compression, although in limited specimens the axial compression was combined with bending, which is not reported here. From the observed behaviour, the effectiveness of the applied partial confinement could be deduced. As shown in table 1, this judgment was based on the level of the bearing capacity combined with the type of compression failure that was formed (at the weak or strong part). Moreover, the observed behaviour of the various parts of the test specimens, such as the CFRP layers and their anchorage, helped to identify the factors that bear an adverse or beneficial influence on these aspects of the behaviour.



Figure 4: Confining steel brackets and partial CFRP confinement at the mid-part.



Table 3: Summary of test results together with the basic specimen characteristics.

Spec. No	Virgin	Repaired	CFRP layers	Anchors	Average Stress at failure (Mpa)	Failure Mode
1 Test 1	Yes	No	No	No	41.12 (146.9%)*	Weak part
1 Test 2	No	Yes/ EMACO	3 (+2)	yes	41.69 (148.9%)	Bolts
1a Test 1	Yes	No	3	Yes	45.78 (177.4%)	Pull out Anchors-1
3 Test 1	Yes	No	No	No	40.79 (147.7%)	Weak part
3 Test 2	No	Yes	3 (+2)	Yes	45.78 (165.9%)	anchorage
3a Test 1	Yes	No	5	Yes	47.09 (170.6%)	anchorage
3a Test 2	No	Yes/ EMACO	5	Yes	42.51 (154.0%)	anchorage
4 Test 1	Yes	No	5	Yes weak	46.60 (168.2%)	anchorage
4a Test 1	Yes	No	5	Yes weak	45.53 (164.4%)	anchorage
5 Test 1	Yes	No	5 (+2)	Yes strong	53.96 (195.5%)	Steel bracket
5 Test 2	Yes	-	5 (+2)	Yes strong	55.26 (200.2%)	Steel bracket
5 Test 3	-	Top / EMACO	5 (+2)	Yes Strong	53.96 (195.5%)	CFRP mid-height
5 Test 4	No	Yes/ EMACO	7	Yes Strong	58.86 (213.3)	Strong stirrups
5a Test 1	Yes	No	No	No	40.88 (158.4%)	Weak part
5a Test 2	No	Yes	7	Yes Strong	57.23 (221.8%)	CFRP mid-height
5a Test 3	No	Yes	7 +4	Yes Strong	60.17 (233.2%)	Strong part

* As % of the corresponding plain concrete strength.

2.2 Instrumentation to obtain the average stress-strain behaviour

Apart from monitoring the compressive load, the deformations of the mid-part were also continuously recorded throughout each experiment with displacement measurements taken at each side of the cross-section. Eight displacement sensors (two at each side) were employed to record the deformations of the mid-part. Although the deformation of this mid-part was far from uniform, as could be seen from the obtained displacement measurements of the weak and strong parts



(figure 5), the average axial displacement, which was found by averaging the measured displacement values at all four sides of each specimen, is mostly used here as an indication of the deformability of each specimen. By dividing this average axial displacement by the height of the mid-part an average axial strain could also be obtained in this way. The following discussion of the observed behaviour of each specimen is based on diagrams of average axial stress versus average axial strain found from the previously described averaging process. More detailed study on the obtained non-uniform deformability for each specimen will be carried out at a future stage. An additional measurement that was obtained during the experimental sequence was the axial strain that developed at the CFRP layers of the partial confinement of the mid-part. These CFRP strain measurements are an additional indication of the effectiveness of the partial confinement.

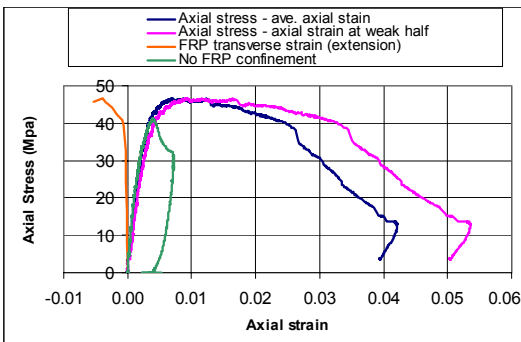


Figure 5: Spec. 4, Test 1 (5 layers CFRP) comparison spec. 1 Test 1 (no confinement).



Figure 6: Virgin specimen without confinement.

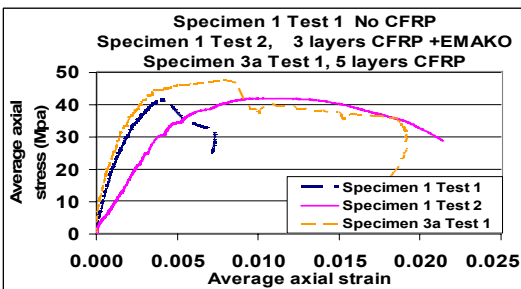


Figure 7: Partial confinement of low effectiveness.



Figure 8: Failure of anchor bolt.





Figure 9: Failure of anchor bolts.
Low effectiveness of
partial confinement.



Figure 10: Tensile failure of
CFRP layers.
Considerable
effectiveness of
partial confinement.

3 Discussion of test results

3.1 Partial confinement of low effectiveness

In figure 7 the obtained behaviour of specimen 1 (Test 2) with 3 CFRP layers and specimen 3a (Test 1) with 5 CFRP layers is compared with specimen 1 test 1 (no partial confinement). Specimen 1 test 2 was formed from specimen 1 test 1 by repairing the failed specimen 1 test 1 with special (low shrinkage) concrete as well as with 3 layers of CFRP forming the partial confinement. This repaired specimen failed in compression with almost the same capacity as the previously-tested virgin test with no partial confinement (figure 6), but with larger deformability. The effectiveness of the partial confinement is low and it is due to the failure of the anchor bolts of the applied confinement. In figure 7 the observed behaviour of specimen 3a Test 1 is also included. This is a virgin specimen that had a 5-CFRP layer partial confinement. Despite the increase in the CFRP layers, the observed effectiveness of the partial confinement is low as was for specimen 1 test 2, again because of the failure of the anchor bolts.

The failed virgin specimen without the partial confinement is shown in figure 6 whereas figure 8 and 9 depicts the failure of the anchor bolts for the “low effectiveness” partial confinement. In figure 11 the observed behaviour of specimen 4 test 1 is depicted. This was a virgin specimen with 5 CFRP layers partial confinement. In this case, a certain alteration was applied in the anchor bolts, by increasing their length. However, this was not sufficient to improve accordingly the effectiveness of the partial confinement, which was again linked to the failure of the anchor bolts.



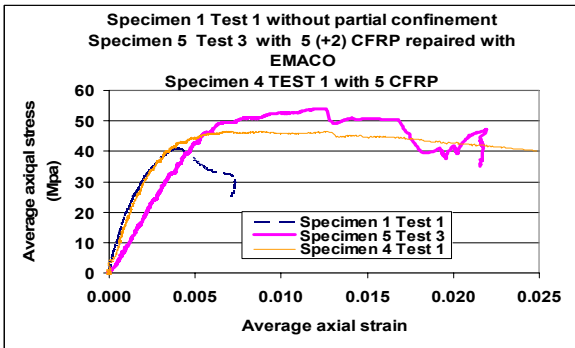


Figure 11: Partial confinement of considerable effectiveness.



Figure 12: Strengthening of the stirrups for the strong part.

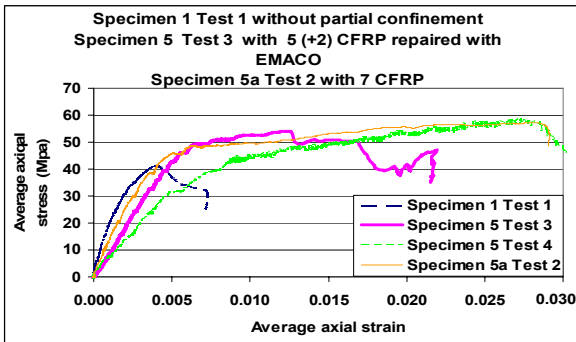


Figure 13: Very effective partial confinement.



Figure 14: Failure of the CFRP layers of the mid-part.

3.2 Partial confinement of considerable effectiveness

In figure 11 the obtained behaviour of specimen 5 (Test 3) with 5 (+2) CFRP layers is compared to specimen 1 test 1 (no partial confinement). Specimen 5 test 3 was formed by repairing a previously failed virgin specimen with special (low shrinkage) concrete as well as with 5 layers of CFRP forming the partial confinement. An additional two (+2) CFRP layers were applied at the part of the section where the anchor bolts were placed. This repaired specimen failed in compression with a modest increase (31%) in its capacity when compared with the capacity of the virgin unconfined specimen. The effectiveness of the partial confinement in this case was classified as considerable. This was due to an alteration in the anchoring of the partial confinement which proved to be



relatively successful. The limit state for this specimen commenced with the tensile failure of the CFRP layers at the central zone and was accompanied, as expected, by a consequent compressive failure of the neighbouring weak part of the section. This is depicted in figure 10 where the anchor bolts, which were left intact, are also shown. As mentioned in section 3.1, the comparison in figure 11 is extended to include specimen 4 (Test 1), in which the partial confinement exhibited low effectiveness due to the failure of the anchor bolts.

3.3 Very effective partial confinement

In figure 13, the behaviour of specimens 5 (Test 4) and 5a (Test 2) is compared with the behaviour of specimens 1 test 1 (with no partial confinement) and 5 (Test 3) discussed in section 3.2. Specimens 5 (Test 4) and 5a (Test 2) were formed by repairing previously failed specimens with partial confinement of 7 layers of CFRP. Moreover, all the anchoring of their partial confinement was made with bolts going through the whole width of the strong part of the repaired section (see Table 3 and figure 3). In order to avoid the compression failure of the strong part the effectiveness of the closed hoops was enhanced as shown in figure 12 by welding. As can be seen in figure 13, a substantial increase (40%), in the bearing capacity as well as in the deformability, resulted from the described partial confinement for these two specimens. Their behaviour was in this way better than the behaviour of specimen 4 (Test 1) which was classified before as one of considerable effectiveness of the partial confinement (section 3.2). The anchor bolts, which were left intact, are also shown in figure 14 together with the failure of the CFRP layers of the mid-part.

4 Conclusions

1. The undesired compression failure expected to develop in the base of vertical members with reinforced concrete cross sections having h/b ratio larger than 1.5 under combined vertical loads and seismic actions is studied through specially formed specimens subjected to uniform compression. The retrofitting of such specimens with partial CFRP confinement is aimed at prohibiting, up to point, such compression failure. This type of partial confinement may also be applied to upgrading vertical structural members with non-accessible sides.

2. From the results of the experimental investigation with identical specimens, with or without this type of partial CFRP confinement, the successful application of such partial confinement was demonstrated. An increase of almost 50% was observed in the compression bearing capacity of some of the tested specimens. Moreover, the deformability of these specimens was substantially increased, demonstrating the effectiveness of this type of partial confinement.

3. It was found from the experimental sequence that critical factors for this increase were the type of anchorage of the CFRP partial confinement and the number of CFRP layers. Successful anchoring of the CFRP layers allowed this partial confinement to become effective and to permit the use of a larger number



of CFRP layers. In the present study alternative anchoring schemes were tried with limitations imposed by the geometry of the model cross-section. Similar limitations imposed by the geometry and the reinforcement of the cross-section will also dictate the design of such an anchoring scheme for a prototype cross-section. Further investigation on the performance of such prototype anchoring arrangements may be necessary.

Acknowledgements

This work has been partially supported by the European Union, Project EVG1-CT-2001-00040. The project is funded by the RESEARCH DG of the European Commission within the context of the Environment Program “Global Change and Natural Disasters” and is here gratefully acknowledged.

References

- [1] Kawashima, K. (2000), Seismic performance of RC bridge piers in Japan: an evaluation after the 1995 Hyogo-ken nanbu earthquake, *Prog. Struct. Engng Mater.* 2000; 2: 82–91.
- [2] Manos, G.A., Kourtides, V., Yasin, B. & Soulis, V.J. (2004), Dynamic and Earthq. Response of Model Structures at the Volvi – Greece European Test Site, *13th WCEE*, Vancouver,, Canada,, No.787.
- [3] Manos, G., Renault, P. & Sextos, A. (2005), Investigating the design implications of the influence between neighboring model structures at the Euroseistest site, *Proceedings of EUROLYN 2005*.
- [4] Pinto A. V., editor (1996), Pseudodynamic and Shaking Table Tests on R.C. Bridges. *ECOEST PREC*8* Report No. 8, November 1996.
- [5] Tsonis, G (2004) “Seismic Assessment and Retrofit of Existing Reinforced Concrete Bridges”, Ph.D. Thesis, Politecnico di Milano.



This page intentionally left blank

Evaluating the retrofitting process for Imam (Soltani) Mosque monument after Silakhor Plan earthquake damage (31 March 2006)

H. R. Vosoughifar

*Civil Engineering Department, Faculty of Technology,
Islamic Azad University, Tehran South Branch, Tehran, Iran*

Abstract

In this paper the retrofitting process for Imam Mosque after Silakhor Plan Earthquake (31 March 2006) was evaluated. Retrofitting the historical structures or monuments has been a challenge for experts and authorities for years while no basic and integrated measure has been taken in this regard and with Bam earthquake and destruction of historical Arg-e-Bam and Seilakhor plain earthquake and damage incurred by historical places and heritage they are threatened to be destroyed. The earthquake tremors in Borujerd caused damage of various degrees in the historic and cultural buildings. In the city of Borujerd, the most significant damage included the damage of the minarets in the Jame' Mosque, the collapse of false ornamental stalactite ceiling in the Imam (Soltani) Mosque and partial collapse of the beehive dome in the Imamzadeh Ja'far Shrine in the Lorestan province. The main damage in the Imam Mosque included collapse of the false ceiling including the rich stalactite ornament on the south *eivan* and cracking on the flanking parts of the north *eivan*. Destruction has resulted from the execution of a big concrete beam over the doorway that changed the behaviour of the same and made it rigid. In some cases incorrect retrofitting would lead to structural damage at the time of an earthquake. The concrete tie-beam on the south *eivan* of Imam Mosque has aggravated the impact of an earthquake on the structure as, during construction, the keystone of the original arch has also been weakened. This tie-beam should be replaced by a more resilient system of reinforcement. Moreover, terms of reference for cooperation between restoration and retrofitting of the monuments should be drafted specifically for each monument according to the particular conditions prevailing in each case.

Keywords: retrofitting, Imam Mosque, restoration, earthquake.



1 Introduction

In recent years many monuments have been damaged by earthquakes in Iran. The residents of the ancient city of Bam in southeast Iran slept as 26 December 2003 began. By 5:26 am, the city lay in ruins, shattered by an earthquake that lasted just 10 s and measured 6.5 on the Richter scale, devastating more than 90% of the city centre and historic buildings [1]. An important cultural loss was the almost total destruction of the well-known historic citadel Arg-e-Bam. This monument, declared by UNESCO as a World Heritage Site, is the biggest mud-brick structural complex in the world. The structure was well intact before the 2003 Bam earthquake [10]. The historic monument of Arge Bam, parts of which date back 2000 years, was severely damaged. With an area of 220,000 m², it includes 25 distinct monuments comprising residential, social, educational and commercial buildings, a military camp, mosques, bazaar, school, prison, sports centre, ice house and the governor section, surrounded by 2,000 m of walls up to 18 m high [15]. Many researchers have analyzed this monument and they wrote a lot of scientific papers about historic buildings and earthquakes. This earthquake and its effect on Arge Bam created a new viewpoint in Iran for monument disaster management including pre disaster, in event management and post event disaster management. In pre event disaster management retrofitting and restoration are introduced. Structural preservation of historic buildings in seismic areas has evolved to become one of the important and relatively new issues in earthquake engineering. It encompasses the identification of the existing structural system and materials used [6] in the construction, including zones of previous repair [2], weakness, cracking and other structural discontinuities, linear and non-linear dynamic analysis, ambient vibration testing, soil and foundation investigations and the strong instrumentation of the monument [8]. On the basis of all these experimental and analytical investigations, alternatives of structural interventions towards the improvement of its structural worthiness can be formulated.

2 Damage index

There are numerous types of damage scales with various attributes, qualities, difficulties or advantages. The seismic damage index is based on buildings damaged or destroyed by earthquakes. By identifying the damage index of a monument structure, in addition to a correct understanding from real behaviour of the structure, the required criterion for retrofitting would be given. Researchers have given many relations for determination of the damage index. Giving an index for failure is a subject that has attracted researchers' attention for more than three decades. For this purpose and knowing the failure indices of a structure we may understand the structure behaviour in a correct way and apply it to regulate its risky margins. On the other hand to control the current condition of a structure the knowledge of its failure method would be necessary for giving an improvement plan. In other words, finding a damage index in a structure



make it clear that to what level the given structure would resist against side forces like earthquakes.

The background of activities that have taken place for determination of the damage index goes back to the early years of the 1970s. In 1972, Whitman showed using the ground movement intensity and damage of buildings upon the ratio of expenses and repair [21]. In 1979 and upon two qualitative criterion, final deformation and coefficients of effects, another method was given by Bertero and Brokken [4]. In 1985, Park and Ang gave a newer method upon maximum possible deformation of a member and final deformation with their combination with the maximum absorbed energy [17]. In 2000, Iemura and Mikami showed that the damage index should be considered before structure analysis and during the application of structural limitations [11]. They gave a new relation based on the Park and Ang relation [17] and the level of ductility [11]. In 2001, Honglin et al., gave a modern method based on collected data from a GIS system. This method was an innovation upon which the damage index was evaluated in an area in a qualitative manner. In the same year, Bozorgnia and Bertero gave two separate indices of structure damage for structures. Such relations have been clearly compiled with the performance-based design [5]. In 2003 Reinhorn and Valles defined a damage index upon which the fatigue is directly incorporated in calculations [19]. In 2003, Papadopoulos et al., with a simple and accurate method introduced an exact method for calculation of the damage index which is quicker and simpler than prior methods [16]. In 2005, Colombo and Negro gave a method for calculation of the damage index, which has been used independently from material [7]. Lourenc and Roque performed an investigation about the possibility of using simplified methods of analysis and simple indexes as indicators for fast screening and decided to prioritize deeper studies in historical masonry buildings and assess vulnerability to seismic actions. These indexes are based mostly on the in plan dimensions and height of the buildings. The simplified methods indicate that, in Portugal, the average in plan area of earthquake resistant walls and average height are independent of the seismicity. This puzzling feature can be related to the short memory of the ancient builders and the fact that major earthquakes in Portugal have rather long return periods (over 200 years) [13].

3 Typical damage in monuments

An analysis of the damage survey of historical masonry buildings for the Umbria–Marche earthquake [9] shows that the problem is generalized and that structural typologies, as well as associated type and distribution of damage, are fairly recurring. Vulnerability may be reduced through retrofitting/protection to better resist the seismic demand.

Anti-seismic action requires the knowledge of seismic site response, the definition of the seismic load (a rather challenging issue) and the knowledge of the characteristics of existing buildings. This is a gigantic task, requiring large funds and considerable large time-span, but several efforts have been made to create damage scenarios and to prioritize retrofitting works, e.g. [3, 12].



A review of the historical sources in Iran, supported by present day knowledge of earthquake engineering, shows that, as far as the earthquake damage is concerned, the slender, free standing members of a complex are in the first line of an earthquake. Minarets, wind towers and high portals of *eivans* and entrance halls are amongst the weakest members. There are numerous references to the collapse of minarets and high portals in past earthquakes. There are in fact very few old minarets, which have not undergone extensive restoration or reconstruction in the areas of higher seismic activities. Remnants of partially toppled minarets, particularly those integrated within the building, are frequently seen in the old mosques. The collapsed minarets of the grand mausoleum of *Sultanieh* in Zanjan, the *Jami* mosque in Ashtarjan, *Jami* mosque of Kerman and the formerly *Shah* Mosque in Mashhad are some examples. It should be noted that the response of a tall slender structure, such as a minaret, during an earthquake depends primarily on the frequency contents of the ground shaking. A low frequency, distant shock may easily topple a minaret or a slender tower, but a high frequency local shock may affect the main building more than the minaret. An example of this behavior is the minaret of *Bagh-i Qushkhane* in Isfahan, the only section of a large building still remaining. High portals of entrance halls and *eivans* have also performed weakly in earthquakes. The upper parts of these portals are effectively, free standing slender elements, susceptible to low frequency ground shaking, in a direction perpendicular to the portal. Collapse of the upper part of the high portal of *Jami* mosque in Gunabad (built in 13th century AD) during the devastating earthquake of Aug 31st 1968 ($M = 7.4$) is a more recent event [14].

4 Silakhor Plan earthquake and monuments

On 31 March 2006, a series of earthquakes with the strongest shock measuring 6 on the Richter scale (according to Iran Geophysics Centre) struck south-western Iran and affected the cities and villages around Borujerd Lorestan Province. The seismic jolt caused extensive damage in many villages and the city of Borujerd. There are about 40 cultural heritage properties which have sustained damage of various degrees in the earthquake-stricken area. The earthquake epicentre was in Darb-e-Astaneh a remote village about 40 kilometres west of the city of Dorud. The series of seismic shocks however affected a vast populated rural and urban area: some 25 villages and the city of Borujerd were severely damaged. Telephone lines, electricity and gas supplies had been cut in some areas. The city of Borujerd dates to the ancient history of Iran. Containing approximately 40 cultural heritage properties, it boasts most of the cultural building property in the whole Lorestan province. The earthquake tremors in Borujerd caused damage of various degrees in the historic and cultural property of the province. In the city of Borujerd, the most significant damage includes the damage of the minarets in the Jame' Mosque, the collapse of false ornamental stalactite ceiling in the Imam (Soltani) Mosque and partial collapse of the beehive dome in the Imamzadeh Ja'far Shrine. In the Lorestan province, Imamzadeh Khaled Ibn Ali, Hojatieh School, Ghaleh and Rangineh Mosques were also damaged. Many of the old



historic houses were also damaged in the historic fabric of borujerd. However, rapid urbanization and building activities during the past three decades have caused soaring land prices and created the tendency to convert the cultural and historic property into modern apartment buildings accessed by wide streets. Lack of vehicular access in the old fabric, lack of proper maintenance, misuse or inappropriate upkeep and use, and Government policies favouring modern building types and technologies, all contributed to deterioration of the historic fabric. Accordingly, much of the old historic fabric was already demolished before the earthquake.

4.1 Imam (Soltani) Mosque

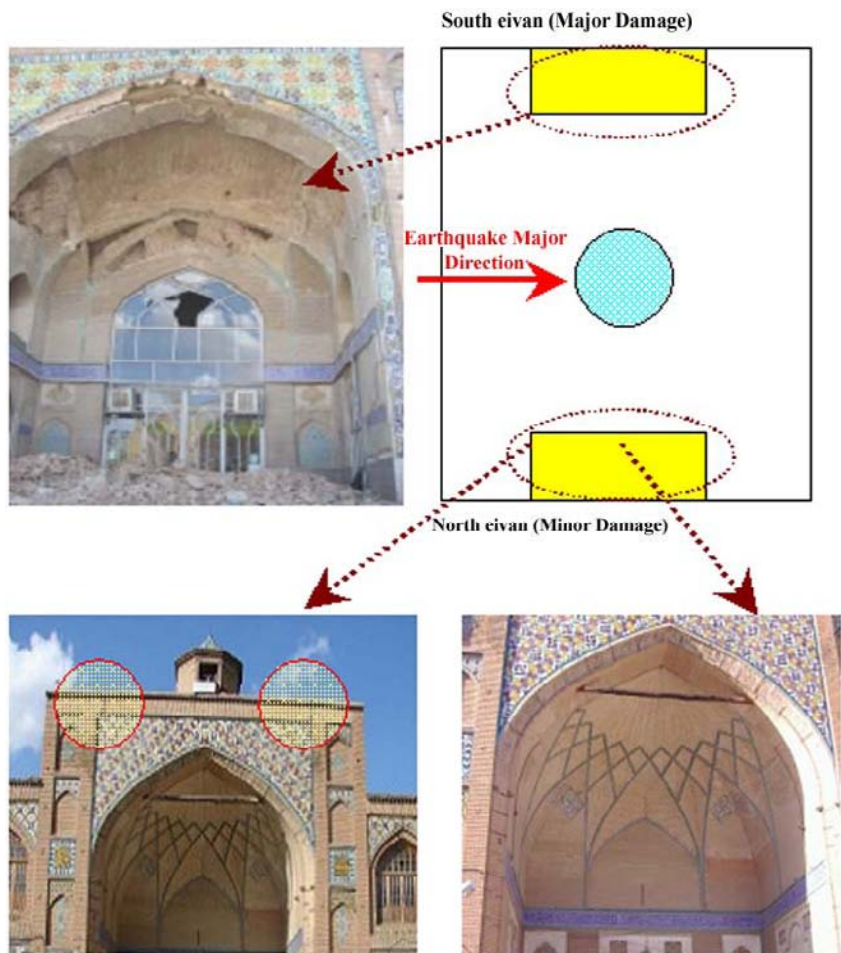
Soltani Mosque of Borujerd was known as Masjed Shah in the Pahlavi Dynasty and today it is called Masjed-e Imam Khomeini. This mosque registered in the National Heritage List (ID number 394), the Imam (formerly Soltani) Mosque has been built in the early Qajar period (circa 1830 AD). Designed after the Imam (formerly Shah) Mosque of Tehran, the monument has been designed as a combination of a Mosque and a theological school with 16 small rooms (*hojreh*) to accommodate theological students. The faience ornament used in this monument is among the unique samples of Qajar tiles. The older mosque was probably built in 10th century A.D. Soltani means related to Sultan which refers to Fath Ali Shah Qajar who ordered the rebuilding of this building.

4.2 Damage in Imam Mosque

The main damage in the Imam Mosque includes the collapse of the false ceiling including the rich stalactite ornament. The south *eivan* and of the north *eivan*. This damage is shown in figure 1.

In the retrofitting process increased stiffness creates a negative effect for the building's seismic performance. Other intervention strategies may aim at producing other types of change in the structural behavior, such as increasing the energy dissipation capacity, by means of specific devices, or decreasing the inertia forces, for instance by means of base isolation. It is therefore important that the choice of a seismic upgrading strategy considers all the changes in structural behavior it may induce. Moreover, it is very important to know how the solutions adopted influence the seismic resistance of different collapse mechanisms [20]. Destruction shown in the figure 2(a) has resulted from the execution of a big concrete beam over the doorway that changed the behaviour of the same and made it rigid. As is seen, in some cases incorrect retrofitting would lead to structural damage at the time of earthquake. Retrofitting methods should be based on improvement of elasticity of the structure and those methods that increase the rigidity of structure are not suitable solutions for seismic reconstruction. Figure 2(b) shows for performance that this tie beam arch keystone was destroyed. This matter and the tie beam impact increased damage in the north *eivan*. In the repair process, conductors and consultants must consider seismic load and they must design and repair with special brick with a lock system.





(a) the solid concrete tie-beam



(b) Damage in Arch keystone

Figure 2: The solid concrete tie-beam over the main arch in the south *eivan*.



The concrete tie-beam on the south *eivan* of Imam Mosque has aggravated the impact of the earthquake on the structure as during the construction the keystone of the original arch has also been weakened. This tie-beam should be replaced by a more resilient system of reinforcement.

Figure 3 shows the effect of the concrete tie beam on dynamic behaviour of south *eivan* of Imam Mosque. This beam prevents flexible behaviour and this has caused more damage in the south *eivan* than the north *eivan*.



Figure 3: Effect of concrete tie beam on dynamic behaviour.

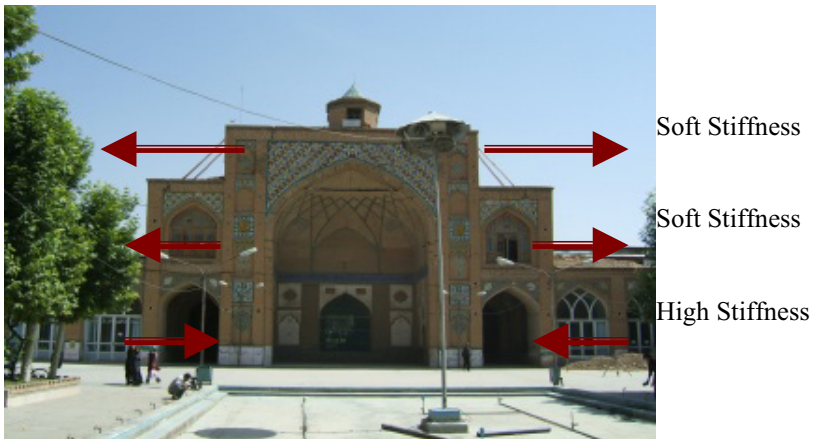


Figure 4: The north *eivan* after the earthquake.

Figure 4 shows north *eivan* behaviour against earthquake. This *eivan* had soft, flexible and suitable dynamic behaviour.

The earthquake has damaged the intermediary structures between the vaults and the finished floor opening the way for penetration of water. Internal decorations in particular the *mihrab* inscribed in the Imam Mosque are more exposed to water damage. Figure 5 shows water seepage in this monument.





Figure 5: Water damage after earthquake.

5 Retrofitting process

The subject of retro-fitting and seismic retrofitting of historical and heritage buildings is a very new subject in Iran. The knowledge of the repair and conservation of these buildings is old, and sufficient experience exists in this area. However, combining the two branches of knowledge and experience in the fields of seismic retrofitting, conservation, and repair, especially for old historical ornamental buildings made of masonry and adobe material is a new subject with little theoretical and field experience all over the world and especially in Iran.

An iterative method is proposed for the seismic assessment of old masonry buildings. In each stage damage in the structural elements or connections between elements due to collapse (brittle behavior) or yielding (ductile behavior) are identified and the structural system changed accordingly [18].

Besides the seismic intensity at collapse, the method allows the identification of the weakest links and connections in the structure and the identification of its expected collapse mechanism, which are relevant information to the design of seismic retrofitting solutions. The proposed method is conservative, as it does not account for the energy dissipation capacity, which is likely to be underestimated by means of using an equivalent linear damping coefficient, and overestimates the effects of the seismic action, as it does not account for its duration. It cannot be applied to regular block masonry, as it cannot simulate the behavior of the interfaces and the geometrical non-linearity. In general case retrofitting process is shown in figure 6.

Given the type of mortar used in the monuments penetration of water can cause several kinds of damage: changing colour; damage to ornament; and increasing structural weaknesses at earthquake time. In Imam Mosque the damp-proof measures to protect the building against the humidity are ineffective and have further aggravated the situation. There is a need to replace the bituminous mats by an appropriate method in line with the traditional water insulation techniques. There is a need to provide temporary light-weight roofing (corrugated sheets) for emergency protection of the roofs against the coming autumn precipitation. This might be used as a general approach for all parts of



roof, if time constrains encourages it. Contractors of Imam Mosque involved in repairing the roofing, need to be warned not to dump debris on the roofs. This may cause new visible or invisible damage.

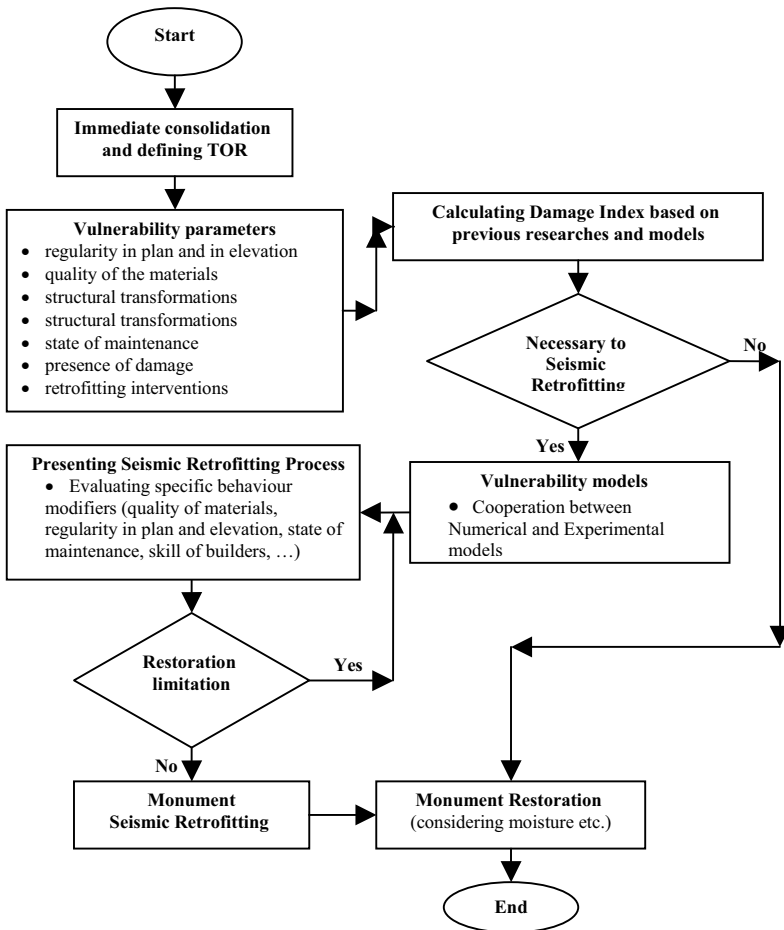


Figure 6: Retrofitting process in monuments.

6 Results

• Terms of References (TOR) for restoration and consolidation of the monuments should be drafted specifically for each monument according to the particular conditions prevailing in each case. The ToRs for Imam Mosque should specifically include the following services:

- a. Calculating quality and quantity seismic damage index
- b. Reducing major risk of structure
- c. Investigating dangerous and structural cracks



- d. Reducing stiffness of tie beam
- e. Damp-proof insulation
- Protected core zone. Each monument has a protected area. In the case of Imam Mosque, the new roofing of Bazaar, with corrugated sheets had destroyed the original fabric of Bazaar and has damaged the urban landscape and assaulted the historic buildings along the bazaar.
- In Imam Mosque severe damage has been incurred by the main rib arches (*tavizeh*) at the front façade of *eivans*. In addition to the earthquake forces the weakness of structures due to the poor use of bricks (jack-arch or *zarbi*, instead of vaulting or *rumi*) had aggravated the situation.
- Seismic strengthening projects are specialized works, which need a thorough knowledge and experience in this field. It is not a type of work that any novice can easily participate in. Therefore, the qualifications of the engineers and contractors should be evaluated and approved by the Committee of Experts as discussed in Item 3 above, and the projects should be assigned to qualified organizations. Of course, the door should remain open for future engineers and contractors to gradually enter this field.
- Tie beam on dynamic behaviour of south *eivan* of Imam Mosque prevents flexible behaviour and this has caused more damage in south *eivan* than north *eivan*. Therefore mistakes in the retrofitting process will cause increased damage at earthquake time.

References

- [1] Akbari M.E., Farshad A.A., Asadi-Lari M., “The devastation of Bam: an overview of health issues 1 month after the earthquake”, Journal of the royal institute of public health, 118, 403–408, 2004.
- [2] Bakolas A, Biscontin G, Zendri E. Characterization of mortars from traditional buildings in seismic areas. PACT Revue du groupe europeen d’etudes pour les techniques physiques, chimiques, biologiques et mathematiques appliquees a l’archecologie 56,17-29, 1998.
- [3] Barbat AH, Ye’pez F, Canas JA. Damage scenarios simulation for seismic risk assessment in urban zones. Earthquake Spectra ,12(3):371–94, 1996.
- [4] Bertero, V. and Brokken, S., “Infills in Seismic Resistant Buildings”, Journal of Struct. Eng., ASCE, 109 (6), 1337 – 61, 1983.
- [5] Bozorgnia, Y. and Bertero, V., “Improved Shaking and damage Parameters for Post-Earthquake applications”. Proceedings of the SMIP01 Seminar of Utiliration of Strong-Motion Data, Los Angeles, California, September 12, PP. 1-22, 2001.
- [6] Cakmak AS, Erdik M, Moropoulou A. A joint program for the protection of the Justinian Hagia Sophia. In: Proceedings of the Fourth International Symposium on the Conservation of Monuments in the Mediterranean Basin, Rhodes, vol. 4, 1997.
- [7] Colombo, A. and Negro, P. “A damage index of generalized applicability”, Eng. Strct. 27, 1164 –1174, 2005.



- [8] Durukal E, Cakmak AS, Yuzugullu O, Erdik M. Assessment of the earthquake performance of Hagia Sophia. PACT Revue du groupe europeen d'etudes pour les techniques physiques, chimiques, biologiques et mathematiques appliquees a l'archeologie , 56: 49-57, 1998.
- [9] G.N.D.T. Seismic damage and vulnerability of the churches in Umbria. CD-Rom. Presidency from the Ministry Council [in Italian], 1998.
- [10] Hossein Sadeghi, S.M. Fatemi Aghda, Sadaomi Suzuki, Takeshi Nakamura, "3-D velocity structure of the 2003 Bam earthquake area (SE Iran): Existence of a low-Poisson's ratio layer and its relation to heavy damage", Tectonophysics 417, 269–283, 2006.
- [11] Iemura, H. and Mikami, T., "Demand Spectra of yield strength and ductility factor to satisfy the required seismic performance objectives." proceeding of International workshop on Annual commemoration of chi-chi Earthquake, Volume II – Technology Aspect, PP. 501 –508, 2000.
- [12] Langa K, Bachmanna H. On the seismic vulnerability of existing buildings: a case study of the city of Basel. Earthquake Spectra, 20(1):43–66, 2004.
- [13] Lourenc P.B., Roque J.A., "Simplified indexes for the seismic vulnerability of ancient masonry buildings", Construction and Building Materials 20, 200–208, 2006.
- [14] Maheri M. R., "Seismic Vulnerability of post-Islamic Monumental Buildings in Iran, a Review of Historical Sources", Fifth International Conference of Seismology and Earthquake Engineering, Tehran, Islamic Republic of Iran, 2007(accepted paper).
- [15] Motamed J., Adlparvar M., AlHussaini A., Vosoughifar H. R., Walls A., "Reconstruction and Retrofit of the Historical Monument of Arge Bam after Earthquake Damage", The first international congress on seismic retrofitting", Tehran, Iran, 2006.
- [16] Papadopoulos, P. and Mitsopoulou, A. and Athanatapulou, A. "Failure Indices for RLC Building Structures" , published by Elsevier Science Ltd. 12th European Conference on Earthquake Engineering., 2003.
- [17] Park, Y. – J., and Ang, A. H., "Seismic damage analysis of reinforced Concrete buildings". J. Struct. Eng., III(4) , 740 – 757, 1985.
- [18] Rafaela Cardoso, Mário Lopes, Rita Bento, "Seismic evaluation of old masonry buildings. Part I: Method description and application to a case-study", Engineering Structures 27, 2024–2035, 2005.
- [19] Reinhorn, A. M., Valles, R. E., and Lysiak, M., "Simplified Inelastic Response Evaluation Using Composite Spectra", Earthquake Spectra, in review, 2003.
- [20] Rita Bento, Mário Lopes, Rafaela Cardoso, "Seismic evaluation of old masonry buildings. Part II: Analysis of strengthening solutions for a case study", Engineering Structures 27, 2014–2023, 2005.
- [21] Whitman RV, Reed JW, Hong ST. *Earthquake damage probability matrices*. Proceedings of the Fifth WCEE, Rome, Italy, 1972.



This page intentionally left blank

Effect of connection procedures on the behaviour of RC columns strengthened with RC layers and jackets

A. P. Lampropoulos, O. T. Tsioulou & S. E. Dritsos

Department of Civil Engineering, University of Patras, Greece

Abstract

This study examines the behaviour of reinforced concrete (RC) columns strengthened with either a RC jacket or an additional RC layer.

The main object of this work was to evaluate the behaviour of the strengthened specimens for different connection procedures used to bond the old and new concrete. For the column strengthened with a concrete jacket, the effect of the roughness of the interface on the behaviour of the strengthened specimen was examined. For the column strengthened using an additional RC layer, the presence of shear connectors between the old and new reinforcement was also examined to provide adequate connection between the two concrete members. The strength degradation of the interface because of the cyclic loading was taken into account. Using the results of the analyses, monolithic coefficients (these are special coefficients that correlate the behaviour of the strengthened specimens to the respective monolithic) were calculated for the different connection procedures examined.

Keywords: concrete, column, strengthening, jacket, finite element, interface, friction, cohesion, shear connectors.

1 Introduction

Strengthening of RC columns using additional concrete layers and jackets perimetric to the initial element is a quite common technique used for the strengthening of RC members. However, there are many uncertainties about the behaviour of the strengthened elements. The finite element method was used to examine the behaviour of columns strengthened with RC jacket or additional RC



layers. The ATENA software was used in this study, as it was previously found to be appropriate for the accurate prediction of the behaviour of RC elements, even in the post peak region of the load deflection curve (Lampropoulos and Dritsos [6]).

A major issue in modeling composite elements, such as the above columns, is the simulation of the interface between the old and new concrete. Thus, appropriate contact elements have been used in this study to simulate the interface. The effect of cyclic loading on the behaviour of the interface is also important and, thus, a methodology, proposed by Lampropoulos and Dritsos [5], for the strength degradation of the interface using the finite element method was used to accurately predict the behaviour of the strengthened specimens under earthquake loading.

Using the results obtained from the analyses of the strengthened specimens, monolithic factors have been calculated. Monolithic factors are special correction factors that can be used to define the behaviour of the strengthened element by comparing it with the behaviour of the respective monolithic element. Monolithic factors for the stiffness, resistance, deformation and ductility, are defined as the ratio of the stiffness, strength, deformation and ductility of the strengthened element over the stiffness, strength, deformation and ductility of the respective monolithic element. These factors can be calculated at the characteristic points of the load–deflection curve (i.e. yield, maximum load capacity, failure).

2 Geometry and material properties

In this study, a RC column strengthened with RC jacket and another one strengthened with additional RC layer were examined. In addition, different interface conditions were examined.

The cross sectional dimensions of the original column were 250 by 250 mm and its height was 1800mm. The thickness of the jacket was 75 mm and its height was 1300 mm. The concrete strength of the initial column was 27 MPa and for the jacket was 55.8 MPa. The longitudinal reinforcement of the initial column was 4 bars of 14 mm diameter (steel grade S220), and stirrups 8 mm diameter steel (grade S220) with a spacing of 200 mm. In the jacket, the longitudinal reinforcement was 4 bars of 20 mm diameter (steel grade S500), and the stirrups were 10 mm diameter (steel grade S500) with a spacing of 100 mm. The reflected value for the axial load in the original column was 0.4 and a horizontal displacement was applied at the top of the column. The column was fixed to a strong footing (fig. 1).

The strengthened column with the additional layer had exactly the same geometry and the same material properties with the one with the concrete jacket. The reinforcement of the additional layer was 2 longitudinal bars of 20mm diameter and steel grade S500. The loading conditions were the same to the strengthened specimen with the concrete jacket, described above. Figure 1 presents the finite element models used for the analyses.



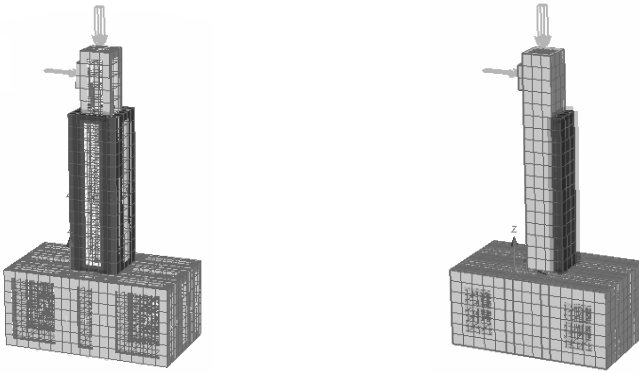


Figure 1: Finite element models for the column strengthened with jacket and with additional layer.

3 Analytical work

As mentioned above, the ATENA finite element software was used to perform the analyses. The stress-strain curve proposed by CEB-FIP Model Code 1990, CEB-fib [2], was used to simulate the behaviour of concrete in compression, (fig. 2).

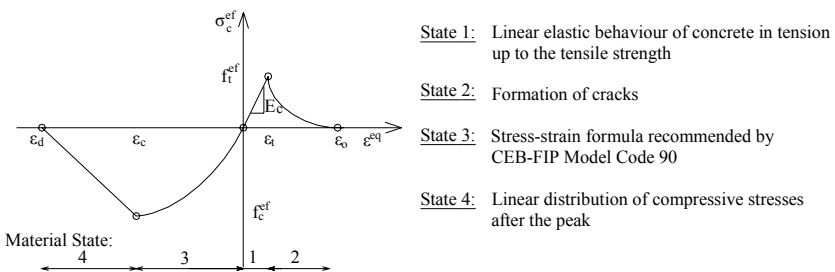


Figure 2: The stress-strain behaviour of concrete.

The element, used to simulate the bar reinforcement, was a link element with bilinear stress-strain behaviour with strain-hardening. This element is capable of including relative slip with the concrete element using three different reinforcement bond models (Cervenka et al [3]). For the analyses carried out, the model proposed by the CEB-FIP Model Code 1990 was used.

The interface between the old and new concrete was simulated using special contact elements. These elements were considered to be fixed at the footing and free at the top. A contact pair, consisting of two special contact elements was used to simulate the behaviour of the interface. The first contact element was a



target surface that modelled the surface of the new concrete, while the second was a contact surface that modelled the surface of old concrete. The model, used to define the behaviour of the contact elements, is presented in figure 3.

The behaviour at the interface can be simulated using the contact elements and appropriate values for the coefficients of friction (μ) and cohesion (c). However, the behaviour of the interface is affected by the cyclic loading. To include the effect of the cyclic loading on the strength degradation of the interface, the procedure proposed in a study by Lampropoulos and Dritsos [5] was used for the cyclic loading presented in figure 4a. According to this procedure, the ratio of the reduced coefficient of friction to the initial coefficient of friction (in the first loading cycle) was calculated for each load cycle (fig. 4b). The lower value for the coefficient of friction is considered to be 0.4. After this value, it is considered to remain constant.

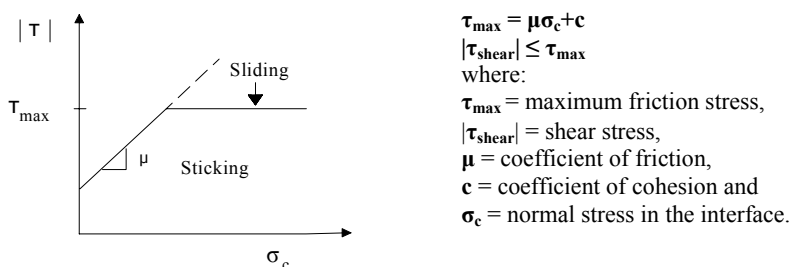


Figure 3: Shear stress against normal stress distribution at the interface.

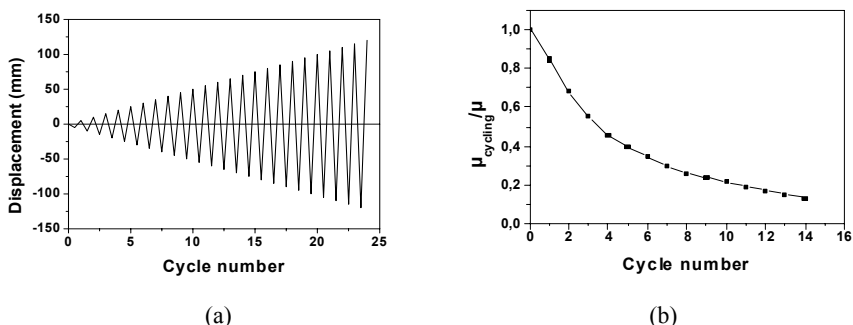


Figure 4: (a) Cyclic loading, (b) reduction ratio for the coefficient of friction.

For the column strengthened with the perimetric RC jacket, three different analyses were performed. The following values were used for the coefficients of friction and cohesion: a) $\mu=0.4$ - $c=1$ MPa [Specimen RMJ₁], b) $\mu=1$ - $c=1$ MPa [Specimen RMJ₂] and c) $\mu=1.55$ - $c=1$ MPa [Specimen RMJ₃]. The values for the coefficient of friction - for the specimens RMJ₂ and RMJ₃ - were reduced during the analysis by using the ratio shown in figures 4b (fig. 5). The cohesion was considered to be equal to 0.5 MPa after the first loading cycle and equal to zero



after the second loading cycle. A monolithic specimen with the same geometry to the strengthened was also examined [Specimen RMJ_{monolithic}].

For the strengthened model with the additional layer in the compressive side of the original column, three analyses with the following coefficients of friction and cohesion were carried out: a) $\mu=0.4$ - $c=1$ MPa [Specimen RML₁], b) $\mu=1$ - $c=1$ MPa [Specimen RML₂] and c) $\mu=1.55$ - $c=1$ MPa [Specimen RML₃]. It was assumed that the model for the reduction of the coefficient of friction (presented in figure 4b) can also be applied for the case of RC elements strengthened with an additional layer. The coefficient of friction was reduced using the ratio shown in figure 4b; the values used during the analysis are depicted in figure 5. The cohesion was considered to be equal to 0.5 MPa, after the first loading cycle, and equal to zero, after the second loading cycle.

Two additional analyses were performed, one with the original column [Specimen RM] and another one with a monolithic column that had the same geometry as the strengthened specimen [Specimen RML_{monolithic}]. According to the response of the strengthened column, it was evaluated that the strength of the composite specimens was much lower than the one of the respective monolithic specimen. To increase the strength of the composite specimens, special shear connectors were used between the old and new reinforcement [Specimen RML_{SC}], (fig. 6). Four connectors, 10 mm in diameter and steel grade S220, were placed along the height of the column to connect the old with the new reinforcement. The values for the coefficients of friction and cohesion were the same as the values used for specimen RML₂. Figure 6 shows a view of the shear connector between the old and new reinforcement.

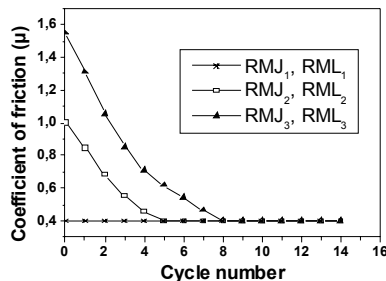


Figure 5: Values for the coefficient of friction in each loading cycle for the models examined.

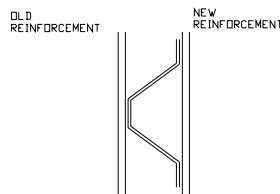


Figure 6: Shear connectors between the old and new reinforcement.



4 Results

The load-deflection results of the analyses are presented in this section. The load–deflection curves were converted to idealized two-linear curves. In order to establish a bilinear idealization, the rule of equal energy under the capacity curve has been adopted in a similar way to that described in ATC 40 [1] and GRECO [4], so that the total energy up to the maximum force is the same for both the load-deflection curve and the bilinear idealization. The failure load, P_u , is defined as the lateral load that is 20% less than P_{max} and the failure displacement, δ_u , corresponds to this failure load.

According to the idealized curves, the characteristic points for the yield, maximum load capacity and failure of the specimen were defined. The ductility (displacement at failure / displacement at yield) was also calculated. The results for the strengthened column with the perimetric jacket are shown in figure 7.

Using the results presented in figure 7, monolithic factors for the displacement and strength at yielding ($k_{\delta y}$, k_{Fy}), maximum load ($k_{\delta max}$, k_{Fmax}) and failure ($k_{\delta u}$, k_{Fu}) were calculated. Monolithic factors for the stiffness at yielding (k_{ky}) and failure (k_{ku}) and for ductility k_{duct} were also calculated. These values are presented in table 1.

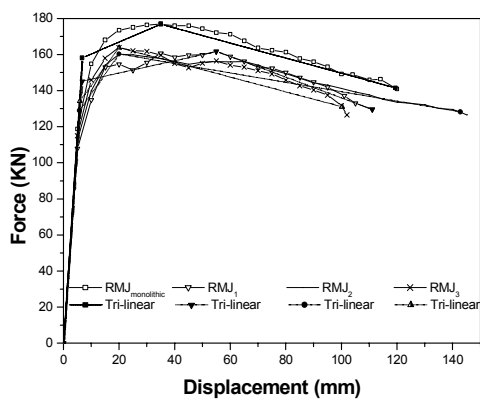


Figure 7: Load-deflection curves for the specimens RMJ_{monolithic}- RMJ₁- RMJ₂- RMJ₃.

Table 1: Values for the monolithic factors.

Specimen	$k_{\delta y}$	k_{Fy}	$k_{\delta max}$	$k_{\delta u}$	$k_{Fmax/u}$	k_{ky}	k_{ku}	k_{duct}
RMJ ₁	1.01	0.92	1.57	0.93	0.91	0.91	0.98	0.92
RMJ ₂	0.86	0.81	0.57	1.20	0.91	0.95	0.76	1.39
RMJ ₃	0.88	0.85	0.57	0.84	0.93	0.97	1.11	0.96

According to the results of the strengthened specimens, presented in table 1, it can be seen that there was a slight increase of the maximum load and the load at failure in specimen RMJ₃ in relation to the other two strengthened models



RMJ₁, RMJ₂. The load at yielding obtained for specimen RMJ₁ was larger than that evaluated for the other two strengthened specimens. Furthermore, the displacement at failure and as a result the ductility of the specimen RMJ₂ were found to be the largest values calculated for the strengthened specimens. According to the load-deflection curve and up to a deflection equal to 30mm, it is obvious that there was an increment in the load capacity for the strengthened specimen when larger values for the coefficient of friction were used. The results using the characteristic points at yield, at maximum load and at failure seem to be more complicated because the load-deflection curve was converted to idealized tri-linear and because of the fact that the coefficient of friction was reduced during the analyses of the specimens RMJ₂ and RMJ₃. However, it is obvious that there were not significant differences between the results obtained for the different values of the coefficient of friction. This is because the effect of cyclic loading on the strength of the interface was taken into account. As a result, the values for the coefficient of friction, for the different models examined, became very similar and exactly the same after the first loading cycles (fig. 5). The results obtained for the strengthened specimens with the additional layer in the compressive side are presented in figure 8.

It is obvious from figure 8 that when an additional layer is used to strengthen the original column, roughening the interface between the old and new concrete cannot provide adequate connection and, hence, the use of shear connectors is vital. From the same results plotted in figure 8, the values for the load and deflection at the characteristic points of yielding, maximum load and failure were calculated. These values were then used to calculate the monolithic coefficients (Table 2).

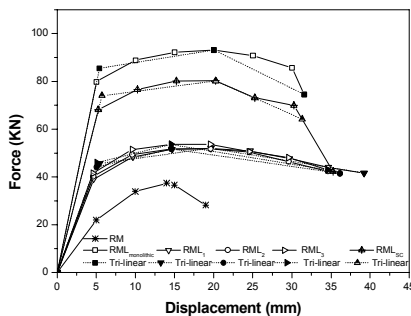


Figure 8: Load-deflection curves for the column strengthened with additional layer.

As in the case of the original column strengthened with an RC jacket, there were not significant differences in the results obtained for the different values of the coefficient of friction and cohesion. However, by using shear connectors in the interface between the old and new concrete, the strength of the specimen was increased significantly. To justify this behaviour, the sliding in the interface was examined. Two analyses, with and without shear connectors (Specimens



RMLCFC_{SC} and RMLCFC respectively) were performed. The coefficient of friction was considered to be equal to 1 and cohesion equal to 1 MPa; these values were kept constant during the analysis. The effect of cyclic loading on the strength degradation of the interface was omitted to simplify the investigation. The load-deflection curves and the sliding at a point on the compressive side of the interface - along the height of the column - are shown in figure 9.

Table 2: Monolithic factors calculated for specimens strengthened with an additional layer.

Specimen	$k_{\delta v}$	k_{Fv}	$k_{\delta max}$	$k_{\delta u}$	$k_{Fmax/u}$	k_{kv}	k_{ku}	$k_{duct.}$
RML ₁	1.01	0.53	0.98	1.24	0.56	0.53	0.45	1.23
RML ₂	0.94	0.52	0.73	1.14	0.56	0.55	0.49	1.22
RML ₃	0.94	0.54	0.73	1.09	0.57	0.57	0.52	1.16
RML _{SC}	1.05	0.86	1.01	0.99	0.86	0.81	0.87	0.94

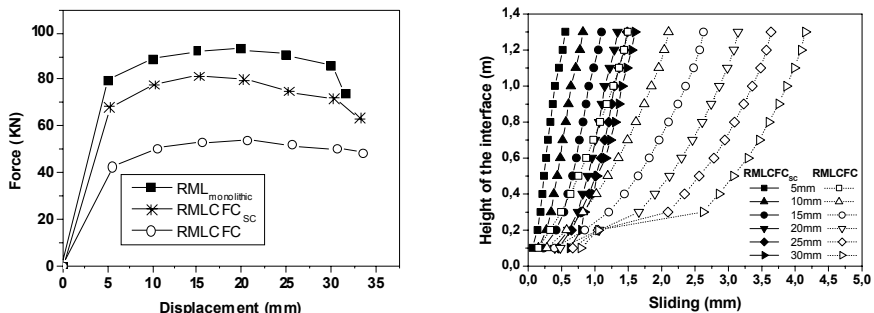


Figure 9: Load-deflection curves and sliding at the interface for the specimens RMLCFC_{SC} and RMLCFC.

According to figure 9, it is obvious that the values for the sliding in the interface for the specimen RMLCFC are much larger than the respective values for the specimen RMLCFC_{SC}.

Figure 10, compares the monolithic factors calculated for the strength and stiffness at yielding and failure, for specimens strengthened using an additional RC layer and jacket. The strength and stiffness' monolithic factors (at yielding and failure), calculated for the columns strengthened with a perimetric jacket, are larger than the values determined for the columns strengthened in their compressive side with an additional concrete layer.

5 Conclusions

According to the results presented in this work, the following conclusions can be drawn.



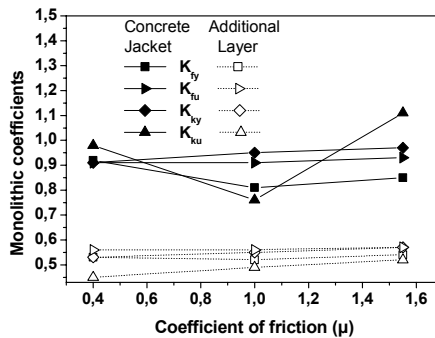


Figure 10: Monolithic factors using concrete jacket and additional layer.

- When the effect of the cyclic loading on the strength degradation of the interface is taken into account, the increase of the coefficient of friction, which simulates the strengthening of the interface, does not have a serious influence on the behaviour of the strengthened specimens.
- When RC jacket is constructed perimetric to the original column, the roughening of the interface between old and new concrete can ensure a quite adequate connection between the two members.
- Strengthening with additional RC layer requires shear connectors at the interface. If there are not shear connectors, then the strength of the composite specimen is much lower than that of the respective monolithic specimen, even for a well roughened interface.
- It was found that when an RC column is strengthened with an RC layer and there no shear connectors between the old and new reinforcement, there are large values for the sliding in the interface between the layer and the original column. The use of shear connectors, results in the significant reduction of the values of the sliding.
- According to the strength and stiffness' monolithic factors calculated for specimens strengthened with an additional RC layer and jacket, it was determined that the higher values were obtained when a perimetric jacket was used.

Acknowledgement

The contribution of Dr. Kyriacos Neocleous to the preparation of this manuscript is greatly acknowledged.

References

- [1] ATC-40, Seismic evaluation and retrofit of concrete buildings, Applied Technology Council, Vol 1, California, USA, 1996.



- [2] CEB-fib, CEB-fib Model Code 1990, Comite Eurointernational du Beton, Thomas Telford, London, 1993.
- [3] Cervenka, V. Jendele, L. & Cervenka, J., ATENA Program Documentation Part 1 Theory, Prague, 2005.
- [4] GRECO, Draft Version “Greek Retrofitting Code” by the Greek Organization for Seismic Planning and Protection, Greek Ministry for Environmental Planning and Public Works, Athens, 2005 (In Greek).
- [5] Lampropoulos, A. & Dritsos, S., Numerical Prediction of Behaviour of Strengthened R.C. Columns under Cyclic Loading, Proc of 11th International Conference on Structural Faults and Repair, Edinburgh, UK, 2006.
- [6] Lampropoulos, A. & Dritsos, S., Prediction of the Behaviour of Strengthened R.C. Columns using Finite Elements, 15th Concrete Conference TEE, Alexandroupoli, Greece, 2006 (In Greek).



Seismic assessment of buildings by rapid visual screening procedures

P. Kapetana & S. Dritsos

Department of Civil Engineering, University of Patras, Patras, Greece

Abstract

Recently, several pre-earthquake screening methods have been developed in order to rapidly evaluate the vulnerability profile of the existing building stock, which has been constructed before or after the adoption and enforcement of seismic codes. The objective of these methods is to identify, inventory and rank all high-risk buildings in a specified region so that a strategy of priority based interventions to buildings can be formed. Major parameters that have affects on the seismic risk are the seismicity of the location, vulnerability and importance of the building structure. The most known rapid visual screening methods have been developed in countries of high seismic risk such as the USA, Greece, New Zealand, India and Canada and they are briefly described in this paper. Furthermore, these methods are applied to a sample of 456 reinforced concrete buildings, located in Athens, whose structural characteristics and levels of damage by the 1999 Athens earthquake are known. In particular, 93 buildings collapsed, 201 sustained severe damage, 69 moderate and 93 buildings sustained light damage. By the methods' implementation, eight different scores have been determined for each building, according to the scoring systems of the applied methods. The results of those applications are used to evaluate the methods' reliability in identifying potentially seismically hazardous reinforced concrete buildings. The obtained results indicate that the implementation of the Greek method results in the most reasonable connection between damage severity and structural scores for all levels of damage, while the Greek method is represented to be the most efficient in terms of both predicting the damage level and leading to the reliable formation of a high-priority set of buildings.

Keywords: pre-earthquake, rapid, visual, screening, seismic, vulnerability, assessment, building.



1 Introduction

Worldwide, several empirical screening methods have been developed which can help to rapidly evaluate the vulnerability profile of large number of different types of buildings. Pre-earthquake screening of buildings is already used in many earthquake prone countries to identify the most potentially seismically hazardous buildings and prioritizing the ones that would warrant a more detailed analysis. Pre-earthquake screening methods can be divided into two primary categories. The first one concerns methods whose implementation requires both rapid visual screening of the building and determination of a structural score and the second one concerns methods whose implementation requires measuring out some dimensions of the construction and performing some simple structural calculations. The most known rapid visual screening methods that have been worldwide proposed are: the method of the U.S.A by the Federal Emergency Management Agency (FEMA), the Greek method developed by the Earthquake Planning and Protection Organization (OASP), Rapid Evaluation Method by the New Zealand Society for Earthquake Engineering (NZSEE), India's method by the Indian Institute of Technology and the used methodology in Canada developed by the National Research Council's (NRC) Institute for Research in Construction. To the second category belong: the Japanese method developed by the Japanese Building Disaster Prevention Association (JBDPA), the Turkish method developed by the Structural Engineering Research Unit, the Initial Evaluation Process by the New Zealand Society for Earthquake Engineering and the Italian method by the National Earthquake Defense Group (GNDT). The methods used and applied in this paper belong to the first category, while the main aim is to ascertain which rapid visual screening method seems to be the most credible.

2 Rapid visual screening methods

Rapid visual screening entails assessing buildings to ascertain their level of seismic risk following a simplified procedure whose main objective is to determine if the buildings should or should not be subject to a more detailed investigation. Particularly, these procedures include completing special Data Collection Forms concerning structural and non-structural characteristics of the construction and determining a Structural Score according to which construction gets ranked.

2.1 The method of the U.S.A. by the Federal Emergency Management Agency

The procedure for Rapid Visual Screening (RVS) was first proposed in the U.S.A and was given in "Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook" in 1988. The procedure was further modified in 2002 to incorporate latest technological advancements and lessons from earthquake disasters in the 1990s. Even though this RVS procedure was



originally developed for typical constructions in the U.S.A, it has been widely used in many other countries after suitable modifications. According to the last edition, the first step of the RVS process includes identification of the primary structural lateral load-resisting system and classification of the building in one of 15 structural type categories, according to the building's material. By this classification, the building gets a basic score, which then gets modified due to probable vulnerability attributes concerning the building's shape (irregularities, high rise, soft story, torsion, short columns, large heavy cladding) and soil conditions. Should the final Structural Score be lower than 2, then a more detailed analysis is required. The inspection, data collection and decision-making process typically occurs at the building site and is expected to take around 30 minutes for each building (FEMA, [4]). It should be noted that, within the framework of this paper, the method of the first edition will be called FEMA-88 and that of the second one FEMA-02.

2.2 The Greek method by the Earthquake Planning and Protection Organization

The Greek method developed by OASP in 2000 is based on the first edition of the FEMA 154 Handbook and will be called OASP-0. The method provides a standard rapid visual screening procedure to identify both the primary structural lateral-load-resisting system and structural materials of the building. By this identification, the building gets classified in one of 18 structural types and it is awarded an Initial Structural Hazard Score. Then, this score will be modified by identifying both the seismic zone and three significant structure characteristics (weak story, short columns and regular arrangement of the masonry) that affect the building's seismic response to arrive at the Basic Structural Hazard Score. Finally, this score will be modified by identifying some modifiers related to the observed performance attributes to arrive at the Final Score. Buildings having a Final Score of 2 or less should be investigated in more detail (OASP, [3]). However, since hazard scores and score modifiers are in question, two alternative scoring scenarios have been proposed in order to identify potentially hazardous buildings more accurately. The first one is based on the OASP-0 method and denoted will be in the following OASP-R and the second one is based on the second edition of the FEMA 154 Handbook and will be denoted FEMA-G.

2.3 Rapid evaluation method by the New Zealand Society for Earthquake Engineering

The Rapid Evaluation, proposed in 1996, largely follows the process presented in the first edition of the FEMA 154 Handbook with the significant variation away from presenting a relatively abstract score, which reflects the logarithm of probability of a damage state, to presenting a score, which has a loose and very conservative relationship to a damage ratio. Differences between FEMA and NZSEE methods concern the number of structural types and the score modifiers considered, while the request for a more detailed evaluation of the building



comes from a graph, which is a function of the building gross area and the final structural score (Brundson et al, [1]). In the following the methodology by the NZSEE will be denoted as NZ-96.

2.4 Seismic vulnerability assessment of buildings in India

The procedure for Rapid Visual Screening used in India is similar to that developed by the FEMA in 2002 after suitable modifications. The modifications concern both soil and structural types and the values of the considered modifiers. Particularly, there are three soil and ten structural types considered (no tilt-up or reinforced masonry buildings are included). Generally, the final score $S < 0.7$ indicates high vulnerability requiring further evaluation and retrofitting of the building (Sinha and Goyal, [6]).

2.5 Seismic screening of buildings in Canada

The widely used methodology in Canada is given in the “Manual for Screening of Buildings for Seismic Investigation” in 1993 and it was developed by the NRC. Its purpose is to establish numerically a Seismic Priority Index (SPI)-a ranking-which results from the addition of a Structural Index and a Non-Structural Index. Major factors in determining the screening score are the building location, soil conditions, type and use of the structure, obvious building irregularities, the presence or absence of non-structural damages, building age and the building importance and occupancy characteristics. Should SPI be greater than 20 the priority is evaluated to be high (CCIEP, [2]).

3 Implementation of alternative rapid visual screening methods

As it has been mentioned, within the framework of this paper, it is attempted to ascertain the efficiency of the presented rapid visual screening methods in identifying potentially hazardous buildings. Thus, a database has been used, which had been created shortly after the 1999 Athens earthquake (Karabinis, [5]). The database consists of 456 reinforced concrete buildings, located in N.W. of Athens and constructed between 1950 and 2000, whose structural characteristics and levels of damage by the 1999 Athens earthquake are known. In particular, 93 buildings collapsed, 201 buildings sustained severe damage, 69 buildings moderate damage and 93 buildings sustained light damage. The seismicity level of the area is moderate (seismic zone II), while soil conditions are of type A (rock or rigid clay). According to the scoring systems of the presented methods (OASP-0, OASP-R, FEMA-02, FEMA-G, FEMA-88, INDIA, NZ-96, CANADA), eight different structural scores have been determined for each building and the results of the methods' implementation are given below.



3.1 Percentages of buildings per set of structural scores

Figures 1-4 illustrate the correlation between percentages of buildings and structural scores, as they have been determined by the scoring systems of the applied methods, for all damage categories (collapsed buildings, buildings with severe, moderate and light damage). In addition, both the trendlines of the graphs, which are graphic representations of trends in data series and come from linear regression analysis, and the corresponding coefficients of determination R^2 .

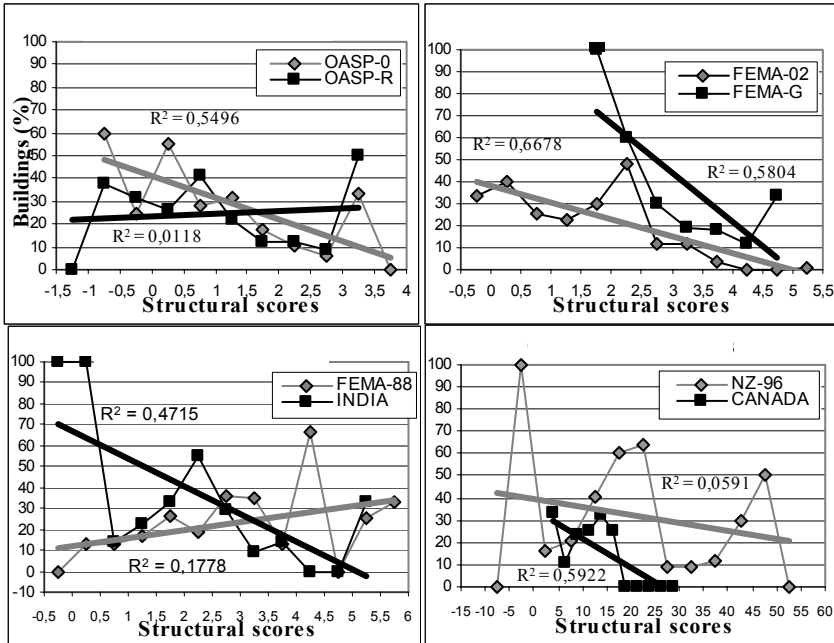


Figure 1: Collapsed buildings.

From the graphs of Fig.1, it is observed that, concerning the category of collapsed buildings, the implementations of OASP-0, FEMA-02, FEMA-G and India's method result in a reasonable connection between the number of the collapsed buildings and structural scores, because as structural scores increase percentages of collapsed buildings decrease. On the contrary, the implementations of OASP-R and FEMA-88 methods result in an unreasonable connection between damage severity and structural scores, because as structural scores increase percentages of collapsed buildings increase. The same conclusion is obtained for the NZ-96 and Canada methods, since in the above scoring systems high values of scores indicate high vulnerability.

From Figure 2, it can be seen that FEMA-02, Indian, FEMA-G and Canada methods are ineffective in predicting severe damages, while FEMA-88's precision in representing possible severe damage is the highest one.



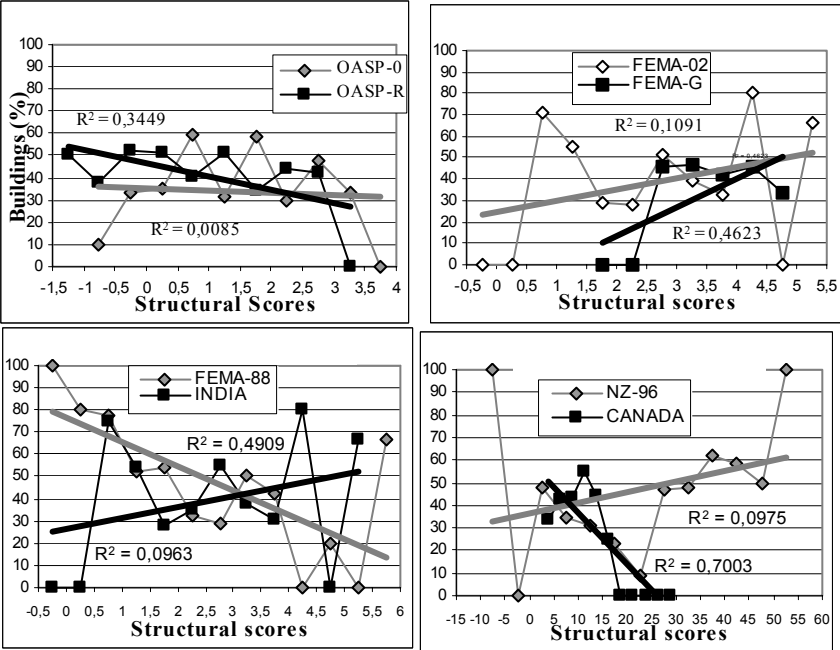


Figure 2: Buildings with severe damage.

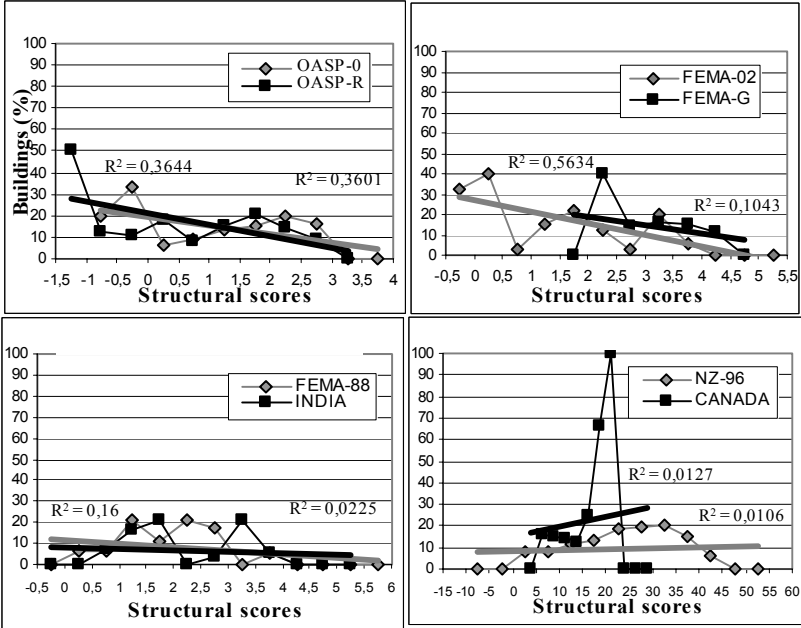


Figure 3: Buildings with moderate damage.



Figure 3 represents that buildings with moderate damage are identified as such by all rapid visual screening methods, with FEMA-02 predicting possible moderate damage more effectively than the other methods do.

Concerning buildings with light damage, it can be seen that FEMA-02 and Canada methods are not effective in identifying buildings safe in earthquake, while the other methods are presented to be quite reliable in predicting potential light damage.

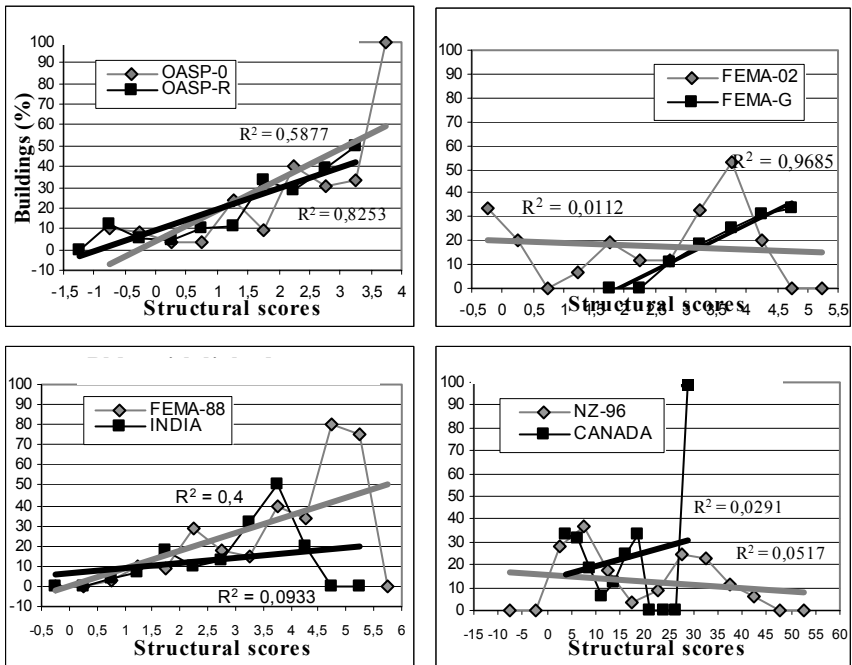


Figure 4: Buildings with light damage.

3.2 Averages of structural scores per building damage category

Figure 5 shows the averages of structural scores per building damage category for all rapid visual screening methods. Collapsed buildings are indicated with “C”, buildings with severe damage with “S”, buildings with moderate damage with “M” and buildings with light damage with “L”. It is observed that for OASP-0, OASP-R, FEMA-02 and the Indian method, there is a reasonable connection between values of averages, because as damage gets more severe so averages decrease. A similar tension is observed for FEMA-G and FEMA-88 methods. However, the results are not that satisfying, because, concerning FEMA-88 method, collapsed buildings have an average of structural scores greater than buildings with severe damage do and, concerning FEMA-G method, buildings with severe damage have an average of structural scores greater than buildings with moderate damage do. Finally, among NZ-96 average values an



unreasonable connection is presented, while Canada averages seem to be in agreement with damage severity, apart from buildings with moderate damage that have an average greater than buildings with severe damage do.

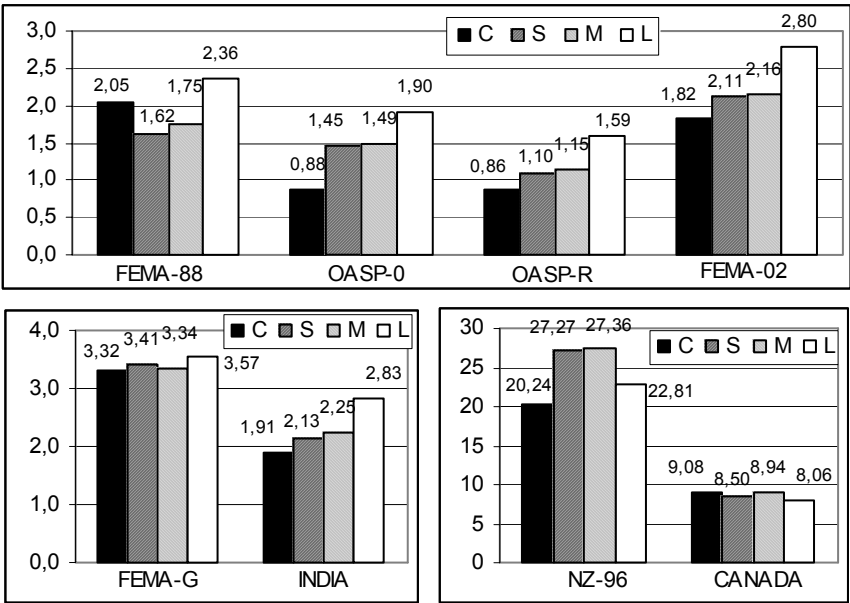


Figure 5: Averages of structural scores per building damage category.

Table 1: Methods’ effectiveness.

Method	BUILDING DAMAGE CATEGORIES							
	Collapse		Severe damage		Moderate damage		Light damage	
	[1]	[2]	[1]	[2]	[1]	[2]	[1]	[2]
OASP-0	✓	✓	✓	✓	✓	✓	✓	✓
OASP-R	✗	✓	✓	✓	✓	✓	✓	✓
FEMA-02	✓	✓	✗	✓	✓	✓	✗	✓
FEMA-G	✓	✓	✗	✗	✓	✓	✓	✓
FEMA-88	✗	✗	✓	✓	✓	✓	✓	✓
INDIA	✓	✓	✗	✓	✓	✓	✓	✓
NZ-96	✗	✗	✓	✗	✓	✗	✓	✓
CANADA	✗	✓	✗	✓	✓	✗	✗	✓

In order to reach conclusions relative to methods’ efficiency, the results obtained by reviewing Figures 1-5 were summarized and they are presented in Table 1. Columns indicated with (FEMA, [4]) concern the evaluation results

obtained from paragraph 3.1, while those with concern the evaluation results from paragraph 3.2. Method's effectiveness is indicated with "✓" and ineffectiveness with "✗".

It is observed that OASP-0 method is the only one effective in identifying reinforced concrete buildings both safe and not safe in earthquake in both evaluation tasks.

3.3 Efficiency measure of collapse prediction

An alternative criterion for the methods' efficiency would be the comparison of the percentages of buildings that did actually collapse in several high priority sets.

Table 2: Efficiency measures.

Subset \ Method	10% of bldgs =46	20% of bldgs =92	50% of bldgs=228
	$E_{C,10}$	$E_{C,20}$	$E_{C,50}$
OASP-0	0.57	0.46	0.33
OASP-R	0.46	0.42	0.28
FEMA-02	0.39	0.38	0.29
FEMA-G	0.28	0.27	0.26
FEMA-88	0.11	0.18	0.27
INDIA	0.48	0.34	0.29
NZ-96	0.12	0.26	0.13
CANADA	0.16	0.27	0.29

As these percentages indicate efficiency towards collapse prediction, the efficiency measure of collapse prediction, E_C , can be adopted for their reference. Obviously, the higher this percentage is, the more efficient the method can be considered. Table 2 lists the efficiency measures of collapse prediction that correspond to 10%, 20% and 50% high priority subsets for all methods, denoted as $E_{C,10}$, $E_{C,20}$ and $E_{C,50}$ respectively. It is observed that OASP-0 method is characterized by the highest measures in all subsets examined, while Indian, OASP-R and FEMA-02 methods follow. Unlike, FEMA-88 and NZ-96 methods are characterized, in general, by the lowest measures. However, when 50% high priority set is examined, it can be seen that, except from NZ-96, measure values of all methods are very close. Suppose that there would be available budget for detailed analysis only for 10% high priority set of buildings that are $0.10 \times 456 = 46$ buildings. If those buildings were to be selected prior to the earthquake strike by chance, without using the results of some rapid visual screening procedure, then in this set would be included $0.10 \times 93 = 9.3$ buildings that would finally collapse. Unlike, if buildings' selection had been based on the results obtained by using OASP-0 method, then $0.57 \times 46 = 26$ buildings would be prevented from collapsing. It should be noted that a measure of $93/456 = 0.2$ or less would not be acceptable, because that would indicate that the method's



efficiency in collapse prediction is not reliable and that selecting buildings for further investigation by chance would prevent an equal or greater number of buildings from collapse.

4 Conclusions

Assessing the results from the implementation of rapid visual screening methods, the following conclusions are reached: (a) A reasonable correlation between structural scores and collapse probability appears to exist only when scoring systems of OASP-0, FEMA-02, Indian and FEMA-G methods are used, (b) The averages of structural scores per building damage category have a reasonable connection with damage severity only when OASP-0, OASP-R, FEMA-02 and the Indian method are implemented. In addition, OASP-0 method appears to have the best scoring difference between averages of collapsed buildings and buildings with little damage, (c) OASP-0, OASP-R, Indian and FEMA-02 methods are characterized by the highest efficiency measures of collapse prediction when 10%, 20% and 50% high priority subsets are examined, with the OASP-0 measure being the highest of all. However, for 50% priority subsets, values of measures are almost the same, apart from that of New Zealand.

The reached conclusions above come from a limited number of data, related to the seismic response of existing buildings in earthquake. Thus, in order to propose the most reasonable rapid evaluation procedure, the assessment of additional data is required.

References

- [1] Brundson, D., Holmes, S., Hopkins, D., Merz, S., Jury, R. & Shephard, B., Rapid Evaluation (Chapter 4). *The Assessment and Improvement of the Structural Performance of Earthquake Risk Buildings*. Report. Draft for General Release for the Building Industry Authority by the New Zealand Society for Earthquake Engineering, pp. 22-35, 1996.
- [2] Canadian Office of Critical Infrastructure Protection and Emergency Preparedness (CCEIP), *Seismic Hazard, Building Codes and Mitigation Options for Canadian Buildings*, www.ociepc-bpiepc.ca
- [3] Earthquake Planning and Protection Organization (OASP). *Provisions for Pre-Earthquake Vulnerability Assessment of Public Buildings (Part A)*, Athens, Greece, 2000. (In Greek).
- [4] Federal Emergency Management Agency (FEMA). *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook*, ATC: Redwood City, California, 1988, 2nd Edition, 2002.
- [5] Karabinis, A. *Rating of the First Level of Pre-Earthquake Assessment*, Final Report, Earthquake Planning and Protection Organization, Athens, Greece, 2004. (In Greek).
- [6] Sinha, R. & Goyal, A. *A National Policy for Seismic Vulnerability Assessment of Buildings and Procedure for Rapid Visual Screening of Buildings for Potential Seismic Vulnerability*, Published by the Indian Institute of Technology, Dept. of Civil Engineering, Bombay, 2002.



Section 11

Structural dynamics

This page intentionally left blank

Three-dimensional seismic damage simulation of wooden houses using a rigid body-spring method

H. Kawakami¹, E. A. Tingatinga² & H. Y. Chang¹

¹*Geoshere Research Institute, Saitama University, Japan*

²*Graduate School of Science and Engineering, Saitama University, Japan*

Abstract

In Japan, the majority of modern residential and a fraction of commercial structures are of woodframed construction. The vast number of wooden houses is critical to the infrastructure of the country and their continued good performance is integral to the overall economic welfare. However, the casualties and damage caused by the 1995 Kobe earthquake suggest that these structures are most vulnerable to strong motion earthquakes. Therefore, in order to guarantee the safety of the general public in the event of future earthquakes, it is necessary to study the mechanisms of collapse of these built structures and to provide a way to identify their weak points for the benefit of retrofiting. To address the above issues, a new methodology was developed for the seismic performance assessment of wooden houses. This methodology identifies local failures such as column buckling and connection fracture, which may induce the global system to collapse. In this study, a three-dimensional rigid body-spring method, which can describe the inelastic behaviour of a structure and simulate the progressive collapse process, was employed. The sequence of the analysis and results in the form of computer animations offer a real-time assessment of the structural integrity of buildings during earthquakes.

Keywords: wooden houses, collapse, damage, buildings, rigid body-spring method, simulation.

1 Introduction

The evolution in computer hardware has had remarkable impact on computing science and engineering design. Desktop and portable computers nowadays are



operating at tremendous speed, have huge memory resources and multiple processing units, thereby permitting very fast computations never conceived before. Taking advantage of this advancement, several researches have been conducted to simulate various natural phenomena and to analyze engineering problems that were previously infeasible. For the past two decades computer experiments utilizing multimedia and realistic computer animation reinforce, if not replace, traditional expensive experiments.

Recent efforts to simulate earthquake response due to strong motion earthquakes attracted many researchers in the field of architecture, structural and earthquake engineering. One of the most familiar accelerogram used is that of the Kobe earthquake in Japan which occurred in January 17, 1995 and caused enormous damage and destruction of structures and deaths in Kobe and in nearby areas. It claimed more than 6,000 deaths, injured more than 35,000 people and caused about 10 trillion yen worth of damage in Kobe and in nearby areas. The death toll was reported to be mostly caused by the collapse of buildings and breakdown of other civil engineering facilities (Otani [1]). Some were due to landslides, overturning of furniture, fires that broke out resulting from rupturing of gas pipes, etc. However, investigation revealed that most deaths were due to collapse of traditional wooden houses.

In order to guarantee the safety of the general public in the occurrence of future earthquakes, it is necessary to study the mechanisms of collapse of each wooden house and building.

This study presents a three-dimensional simulation of structure collapse during strong motion earthquakes, such as the 1995 Kobe earthquake, using Rigid Body-Spring Method (RBSM). Various modes of collapse of a wooden structure modeled as an assembly of rigid bodies connected by inelastic links at their ends will be presented. In modeling structural components, a link configuration is suggested to take into account structural damping and inelastic behavior.

The main objectives of this study are to simulate the earthquake response of wooden houses, to understand the process in which they collapse, and then to identify the weak point of the structure. Specifically, it aims to

1. simulate collapse mechanisms by modeling structural elements using rigid body-spring method implementing a configuration of inelastic links;
2. investigate how the collapse progresses from the local failures of the framing members.

2 Nonlinear analysis using assembly of rigid bodies

Originally proposed by Kawai [2], the basic approach of RBSM is to divide the given structure into appropriate number of rigid elements connected by spring systems. The displacements are completely described by the positions and rotations of the rigid bodies while the deformation energy of the structure is stored in the spring system.



In this paper, nonlinear analysis of structures will be carried out by introducing nonlinear springs to take into account large displacements and failure of structures during strong motion earthquakes.

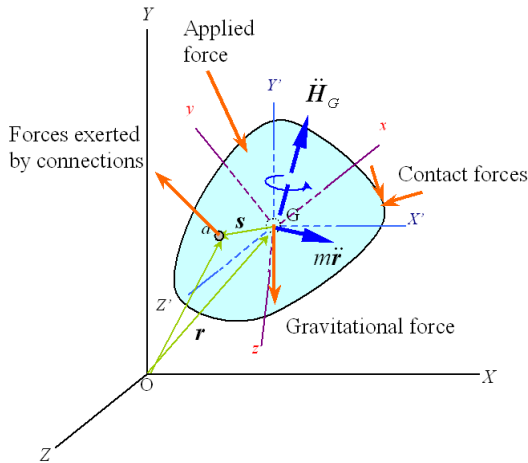


Figure 1: Applied and effective forces on a rigid body and coordinate systems used in computer animation.

2.1 Position and orientation of bodies [2–4]

In rigid body assemblies, various coordinates systems must first be well understood (fig.1). A rigid body in space is positioned with respect to the inertial coordinate system OXYZ by a vector $\mathbf{r}$ attached to its mass center. Points that define the shape of the body or points where links are attached, say point $\mathbf{a}$, are defined by a vector $\mathbf{s}$ in terms of local coordinates (with respect to Gxyz). If needed, the global coordinates of this point $\mathbf{S}$ are computed as

$$\mathbf{S} = \mathbf{R}\mathbf{s} + \mathbf{r} \quad (1)$$

where $\mathbf{R}$ is a rotation matrix that transforms coordinates from Gxyz frame to OXYZ frame.

Orientation of the body can also be described using Hamilton's quaternion $\mathbf{q}$ [4] so that Gxyz is obtained by rotating OXYZ about an axis $\mathbf{u}$ by an angle γ .

$$\mathbf{q} = \left[\cos \frac{\gamma}{2}, \mathbf{u} \sin \frac{\gamma}{2} \right] \quad (2)$$

2.2 Governing equations [2–4]

To animate various systems using rigid bodies, appropriate forces must be taken into account. Forces that arise due to relative positioning of objects (e.g., contact, collision), object's velocity, connections (e.g., springs, dampers), and user-



specified vector fields (e.g., gravity, other external forces) must be exerted on bodies properly. These forces induce linear and angular accelerations depending on the mass and mass distribution of the body, respectively.

The two fundamental equations used to analyze motion of rigid bodies in space are

$$\sum \mathbf{F}(t) = m\ddot{\mathbf{r}}(t) \quad (3)$$

$$\sum \mathbf{M}_G(t) = \dot{\mathbf{H}}_G(t) \quad (4)$$

where $\ddot{\mathbf{r}}(t)$ is the acceleration of the center of mass and $\dot{\mathbf{H}}_G(t)$ is the rate of change of the angular momentum about the mass center of the rigid body. If x , y and z axes coincide with the principal axes of inertia of the rigid body, eqn (4) reduces to the well known Euler's equations of motion and is expressed in terms of the inertia tensor $\sum \mathbf{M}_G(t) = \mathbf{I}(t)\dot{\boldsymbol{\omega}}(t)$.

The above equations can be rewritten by first assuming

$$\dot{\mathbf{r}}(t) = \mathbf{v}(t) \quad (5)$$

$$\dot{\mathbf{q}}(t) = \frac{1}{2}\boldsymbol{\omega}(t) * \mathbf{q}(t) = \mathbf{g}(\mathbf{q}(t), \boldsymbol{\omega}(t)) \quad (6)$$

where $\boldsymbol{\omega}(t) * \mathbf{q}(t)$ denotes a shorthand of the multiplication of two quaternions $[0, \boldsymbol{\omega}(t)]$ and $\mathbf{q}(t)$.

At any instant, the state of a rigid body is stored in a vector $\mathbf{x}(t)$ consisting of its position, orientation, and its linear and angular velocities (Baraff [4]). Mathematically collected as,

$$\mathbf{x}(t) = \begin{Bmatrix} \mathbf{r}(t) \\ \mathbf{q}(t) \\ \mathbf{v}(t) \\ \boldsymbol{\omega}(t) \end{Bmatrix}_{13 \times 1} \quad (7)$$

Using eqns (5) through (7), we can rewrite eqns (3) and (4) as the time derivative of the state vector

$$\dot{\mathbf{x}}(t) = \begin{Bmatrix} \mathbf{v}(t) \\ \mathbf{g}(\mathbf{q}(t), \boldsymbol{\omega}(t)) \\ \sum \mathbf{F}(t)/m \\ \mathbf{I}^{-1}(t) \sum \mathbf{M}(t) \end{Bmatrix} \quad (8)$$

This system of first-order differential equations are sufficient to perform physically based animation of rigid bodies. Knowing the current state of the rigid bodies and the derivative information at any time, a differential equation solver can now be used to compute the state vector at a subsequent time. It is important to note that the sum of forces and sum of moments about rigid body's mass center include contact forces and collision forces (or impulses) when it moves relative to another rigid body. This of course entails the use of efficient collision



detection routines. Since collision detection, response and contact handling are well documented in computer animation and robotics, it will not be discussed in detail in this paper.

2.3 Earthquake ground motions

Throughout the scope of this paper, the earthquake accelerogram used was that observed at Kobe Marine Metrological Observatory during of the 1995 Kobe earthquake as shown in fig. 2(a). The maximum accelerations in EW-, NS-, and UD-directions are 6.0 m/s^2 at 5.5 s , 8.2 m/s^2 at 5.5 s , and 3.3 m/s^2 at 4.7 s , respectively. Figure 2(b) shows the corresponding displacement-time histories.

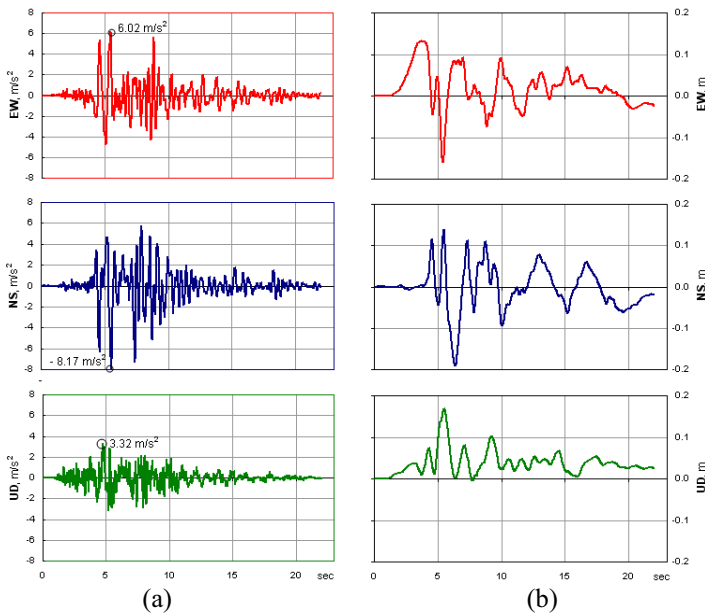


Figure 2: Time history of (a) ground acceleration and (b) ground displacement of the 1995 Kobe earthquake used in analyses.

2.4 System of non-linear springs and dashpots

A new link consisting of a spring and a damper, parallel to one another is introduced. Forces exerted to points on connected bodies is the vector sum of spring and damper components. The stress-strain behavior of materials, as modeled by springs, is idealized by the straight lines in fig. 3. In this model the restoring force in tension is proportional to the strain up to ε_{YT} with maximum yield restoring force F_{YT} . The second straight line represents the strain-hardening characteristic until point C, when the restoring force reaches its



ultimate value F_{UT} . For different set of parameters ε_{YC} , F_{YC} , ε_{UC} , and F_{UC} , the behavior in compression can be modeled. If the strain in the link exceeds its maximum value ε_{UT} or ε_{UC} , no subsequent forces are exerted to the connected bodies. The link is marked DELETED to aid analysis of link failure. A set of links are positioned to model axial, shear and bending deformations.

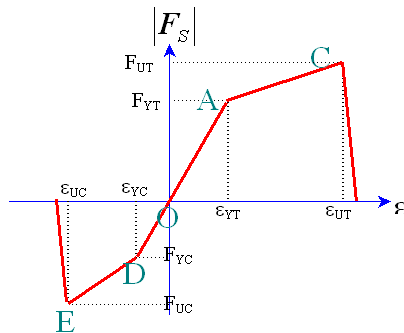


Figure 3: Force exerted by link on rigid bodies.

3 Application to wooden houses [6]

3.1 Dynamic collapse of conventional wooden houses

Using the numerical method presented above, earthquake responses of typical wooden houses in Japan, as shown in fig. 4, were computed. In modeling, the dead load of the floor slab, beams, columns, walls and roofs, and the live loads were estimated. The stress-strain curves for the material of the structural members were modeled. The model structures were then subjected to doubly-amplified waves of the 1995 Kobe earthquake in fig. 2.

Figures 4(a)-(d) show various collapse mechanisms of wooden houses including (a) collapse due to the soft first story, (b) tumbling type collapse, (c) failure at the second floor due to the amplification of the vibration at the upper floor, and (d) collapse of intermediate floor.

Even though the soft first story type of collapse is often stressed, the failure mechanism depends on the design and physical layout of the structure, i.e., strength and distributions of columns, beams and walls. For example, fig. 5 shows the response sequence of the three-storey wooden house in fig. 4(d). During the strong motion earthquake, the house deformed beyond the limit of linearly elastic behavior, and the second floor began to twist at around 4 s, and collapsed at about 6 s.

3.2 Reliability of simulated response

Most wooden houses in Japan are generally composed of frame units made of columns and beams, and walls. Therefore, in order to establish the reliability and correctness of collapse simulation of an entire house, it is obligatory to show the



agreement of simulated and experimental results for such structural units. Figure 6 shows frames with different types of reinforcements. A monotonically increasing force was then applied horizontally to the upper beam of each frame, and the response was computed using the same program that was used in figs. 4 and 5.

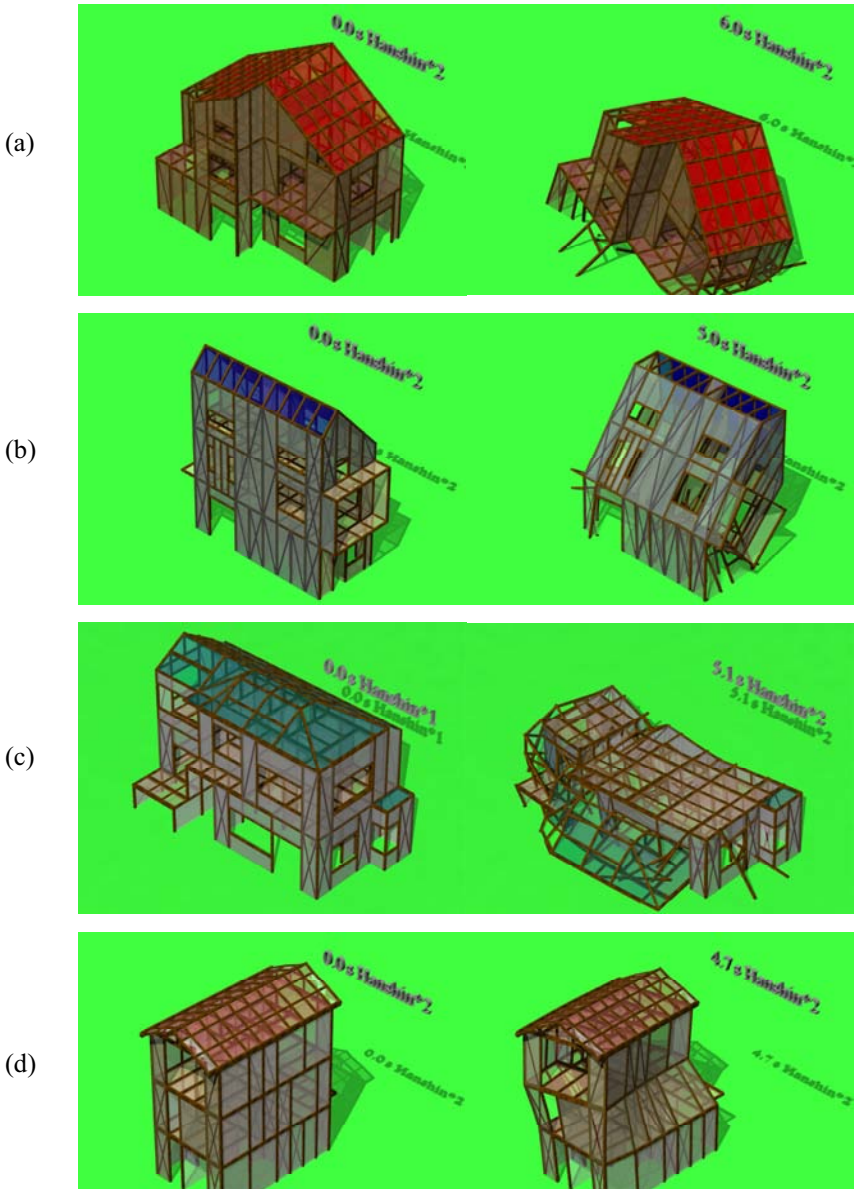


Figure 4: Collapse mechanisms of typical wooden houses.



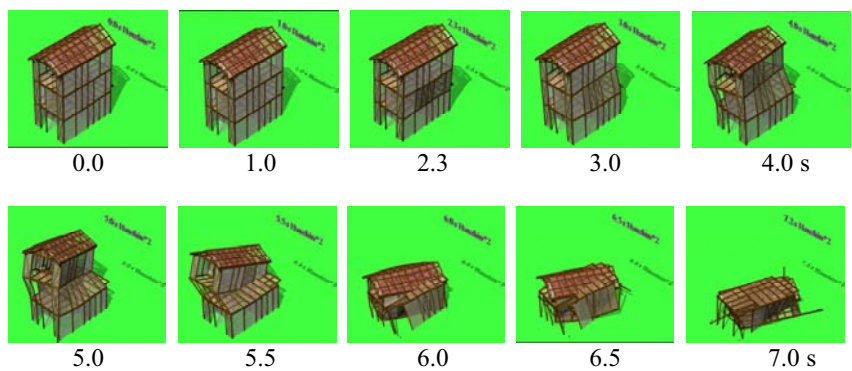


Figure 5: Progressive collapse sequence of a three-storey wooden house.

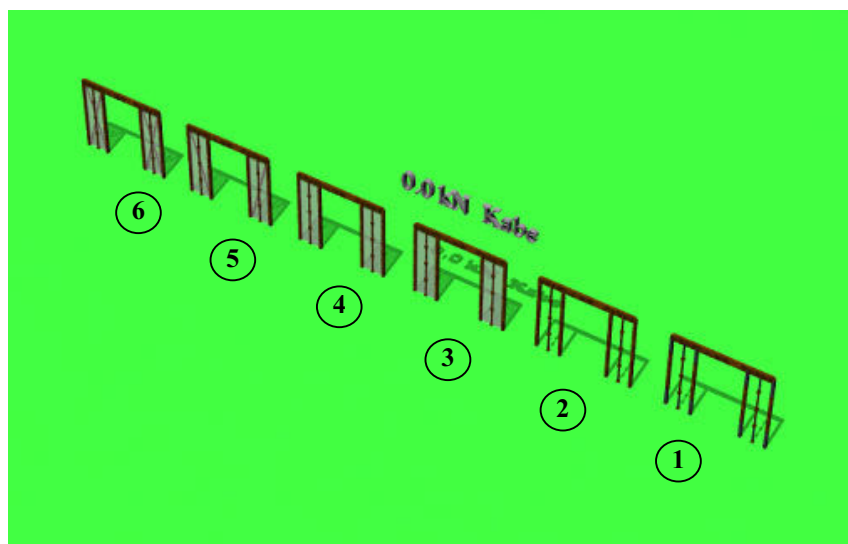


Figure 6: Samples of wooden frame units. From right to left: wooden frames with (1) no end connection plate; (2) end connection plates; (3) plaster board; (4) siding board; (5) two single braces, plaster board and siding board; and (6) two double braces, plaster board and siding board.

Based on the obtained response of the upper beam, the relationship between the displacement and force was plotted by solid lines in fig. 7 for each frame unit (1) through (6). The experimental relationships obtained by Miyoshi et al. [5] were also plotted by dashed lines in fig. 7. The numerical and the experimental curves agree well including the rigidity in small strain, the maximum strength, and the weakening process, and this shows the reliability of the simulation in this paper.



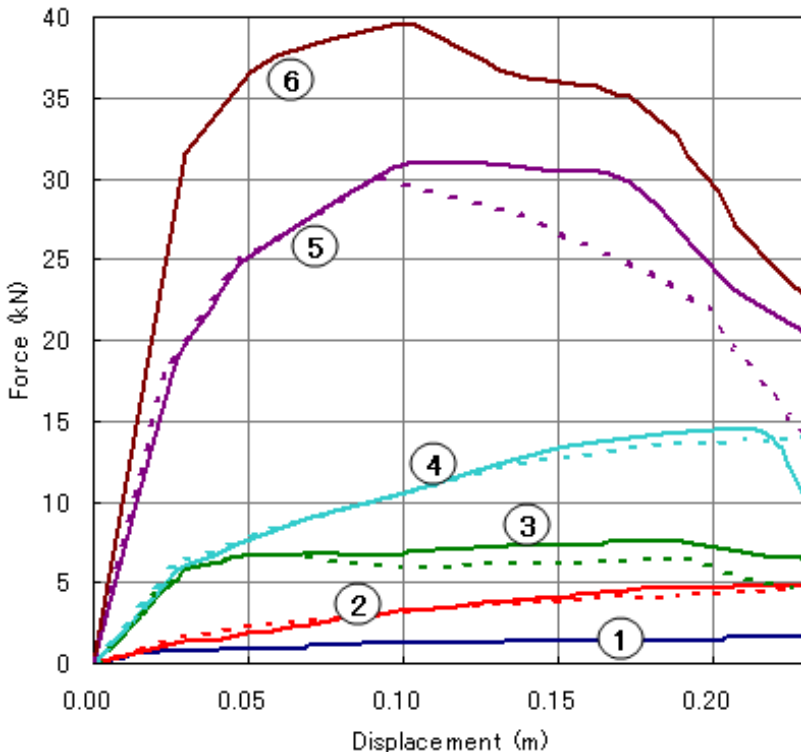


Figure 7: Force-displacement relationships of wooden frames.

4 Conclusions

This research attempted to simulate seismic collapse of wooden houses subjected to the 1995 Kobe earthquake using the Rigid Body-Spring Method (RBSM), and the following conclusions were drawn:

1. The simplified model of wooden houses is capable of demonstrating to some extent various collapse behavior during strong motion earthquakes. More accurately, the link system used to characterize plastic hinges can simulate local failure that causes the entire house to collapse during strong motion earthquakes.
2. The method provides a way to identify the weak point of a structure thus allowing engineers to perform retrofitting analysis easily so as to suggest ways to improve the seismic performance of built wooden houses.

References

- [1] Otani, S., Disaster mitigation engineering -the Kobe earthquake disaster-, *JSPS Seminar on Engineering in Japan*, Royal Society, London, 1999.



- [2] Kawai, T., New element models in discrete structural analysis, *Japan Society of Naval Architects, Japan*, 141, pp. 174-180, 1977.
- [3] Hamilton, W. R., On quaternions, *Proceedings of the Royal Irish Academy*, 3, pp. 1-16, 1847.
- [4] Baraff, D., Dynamic Simulation of Non-Penetrating Rigid Bodies, *Ph.D. Thesis*, Cornell University, 1992.
- [5] Miyoshi, K., Ohashi, Y., Takahashi, K., Watahiki, M. & Nakano, I., Static loading test and shaking table test of walls for wooden houses, Part 1: static loading test, *Summaries of Technical Papers of Annual Meeting, Architectural Institute of Japan*, pp. 199-200, 2001 (in Japanese).
- [6] Kawakami, H., www.saitama-u.ac.jp/kawakami/ (in Japanese).



Controlling nonlinear vibrations in steel structures using an evolutionary gain formulation to optimally satisfy performance objectives

R. Dansby & T. Attard

*Department of Civil and Geomatics Engineering and Construction,
California State University, Fresno, USA*

Abstract

An evolutionary gain formulation is implemented within a nonlinear quadratic control algorithm framework used to minimize the performance index of a structural steel system that is subjected to various earthquake ground motions. The control architecture is formulated using a numerical integration scheme that solves the nonlinear responses of a degrading system and formulates an optimal gain matrix that is used to control building displacement demands by satisfying the desired performance-objectives per time-step. The performance-objectives are defined for various 'damage-safe' and elastic demands to show the versatility of the proposed control solution. The results of the evolutionary gain approach are compared to more conventional LQR techniques. Towards this end, a Control NONlinear time-history analysis (CONON) program was developed to simulate the responses of kinematically strain-hardened systems and to compute the optimal semi-active device output forces per time-step as part of the control solution that implements the proposed evolutionary gain. The minimization of the cost function is independent of the weighing matrices of the system, thus alleviating any need to compute these terms per time step. Instead, an iterative Riccati matrix is determined per time-step and used to generate the evolutionary gain. The results are compared by examining several hysteresis plots of the steel system against other feedback-based methods. The proposed system implementation shows a marked increase in the ability to control the desired target response and meet acceptable performance goals.

Keywords: performance-based analysis, evolutionary control, state-space analysis, nonlinear analysis.



1 Introduction

Research in performance-based engineering continues to be active wherein the design and implementation of various damage-mitigation systems for seismic protection is investigated [1]. Through structural control, the lateral force-resisting system in a building is enhanced by using performance objectives to define a *permissible and safe* response level – such as not allowing a particular amount of lateral deflection to be exceeded on a given story. One means of satisfying performance criteria has been through the implementation of semi-active devices [2, 3] that generate *reaction* forces in response to imposed demands. The objective of these systems is to offset responses in buildings that exceed pre-defined performance levels by dissipating imparted earthquake energy that would otherwise be absorbed by the structure.

Passive base isolation systems have been adopted by agencies, such as FEMA [4], and building codes officials, such as the IBC [5], for the seismic protection of buildings. Except for certain near-field, pulse-type earthquake motions with low frequencies [6, 7], base isolation systems have shown very adequate ability to dissipate the low energy associated with high-frequency far-field motions especially in low-rise ‘stiff’ buildings [8]. For low-energy motions, passive base isolation uses a soft and flexible base to decrease a building’s natural frequency and filter out the high frequency ground motion components. However, one added advantage that semi-active systems provide over passive systems [9] is that vibrations can be controlled in real-time (through a closed-loop control scheme) that enables the structure to respond in a ‘safe-mode’ of vibration at all times by dissipating the large magnitudes of earthquake energy often observed in near-field excitations. In fact, various semi-active stiffness devices have been developed to overcome limitations of passive systems, including variable stiffness systems wherein braces engage and disengage during an earthquake resulting in a change in the vibration properties of a building [10]. To overcome the suddenness of the engaging/ disengaging nature of the stiffening system, a semi-active device was developed that transitioned smoothly between states thus avoiding high-frequency resonance [11]. Variable damping devices have also been developed and applied as part of a ‘hybrid’ system [8] that uses passive base isolation to avoid structural resonance and dissipate high-energy from near-field excitations. MR fluid dampers [12] have been used as variable damping devices and also in combination with variable stiffness devices in smart base-isolated systems that are able to vary their stiffness and damping continuously and remain in a low energy non-resonant state during an earthquake. In this light, the control solution used in this study is applied through a semi-active system that is controlled using an optimal nonlinear centralized algorithm that is based on linear quadratic theory [13]. The control solution uses an evolutionary gain matrix that adapts the Ricatti matrix per time-step, or in a steady-state snapshot. The procedure has been integrated in a fully-automated program called CONON – Control NONlinear Time-History Analysis – that was developed to simulate



the controlled responses of buildings under earthquake events through the feedback mechanism in semi-active systems.

2 Nonlinear control and inelastic performance-objectives

A simulated random plastic analysis of single- and multi-story buildings under stationary earthquake input by Attard and Mignolet [14] expectedly revealed that the induced structural damage could not exceed certain thresholds even under near-resonant conditions unless an increase in the amount of energy was supplied through the ground motion. Conversely, only minimal damage was observed in non-resonant buildings where there were high levels of earthquake-energy imparted to the structures. While passive base isolation and (passive) viscous damping provide viable and relatively inexpensive solutions toward controlling such potential damages, near-field earthquakes (Northridge in 1994, Landers in 1992, Chi-Chi in 1992, and Turkey in 1999) in the direction of the fault-plane rupture have tremendous damage-inducing potential because of the high-energy content contained in the ground motion. The implementation of a hybrid control system that is composed of semi-active dampers that dissipate this high energy and variable stiffening elements that smoothly decrease the stiffness (and frequency) of the building to avoid earthquake resonance seems to provide a 'best of both worlds' solution.

The objectives of this study are to control a building's responses consistent with elastic and marginally-inelastic performance objectives using a semi-active control architecture to reduce damages and large inter-story drifts in the main structural components.

2.1 Optimal nonlinear control through an evolutionary gain

The equation of motion for a multi-degree of freedom shear frame excited by a ground acceleration is given in equation (1).

$$\mathbf{M}\ddot{\mathbf{x}}(t) + \mathbf{C}\dot{\mathbf{x}}(t) + \mathbf{F}_R(\mathbf{t}) = -\mathbf{M}\ddot{\mathbf{x}}_g(t) \quad (1)$$

The mass and damping matrices are defined as $\mathbf{M}$ and $\mathbf{C}$, respectively, where $\mathbf{x}(t)$ is the relative displacement vector of the building. The horizontal ground acceleration applied to the base of the building is given as $\ddot{\mathbf{x}}_g(t)$. The function $\mathbf{F}_R(t)$ is used to define the stiffness of the system, which changes when any member of the building responds inelastically – such as when a member reaches its elastic limit during loading, unloading, or re-loading on the force vs. deflection hysteresis curve. This will occur when a member's cross section just starts to yield. The simulation of the plastic excursions that are experienced by members provides information on the extent of damages in an uncontrolled system, which justifies the need to implement a control system [1]. In satisfying performance objectives that are (minimally) greater than a member's yield capacity, system responses that venture into the inelastic state because of



incurred time-delays can be accurately analyzed, and the semi-active devices can be designed to be able to generate any additional output reaction forces in that event. The generating output control force that semi-active devices are required to generate for inelastic systems increases because of the dissipated hysteretic energy of the structural member that the device now needs to compensate for to control the response. Therefore, while it is undesirable to utilize semi-active devices in inelastic systems, time-delays or inter-modal coupling in multi-story buildings can realistically cause some members to become inelastic. To compensate for such limitations, an effective control solution is developed herein using an evolutionary gain to maximize the force output of the semi-active system such that it is not restricted by typical 'weighing matrices.'

The control algorithm that is developed for this purpose utilizes an optimal Ricatti matrix that is computed during *each* time-step and is subsequently used to determine an evolutionary gain that relates the control force output. As such, a new control force is computed based on a new gain during each time step, wherein subsequent changes to the overall stiffness and damping properties of the system can potentially cause one or more structural components to surpass their elastic yield limits. Therefore, the algorithm checks all the degrees of freedom of the building during each time step to verify that each performance is satisfied. In order to incur inelastic behavior in structural members because of time-delays that would not allow a device to react quickly enough to elastically control a response, inelastic target performance levels are assumed. As such, the stiffness and damping of the structure can alter the state-space terms used to minimize the cost function that is given in equation (2).

$$J_k = \int_{t_k}^{t_k + \Delta t} \frac{1}{2} [z_k^T(t) Q z_k^T(t) + f_{c,k}^T(t) R f_{c,k}^T(t)] dt \quad (2)$$

Here, J_k is the cost function that is minimized over each time-step, k , over the time interval Δt using the computed state-space responses, $z(t)$, and control forces, $f_{c,k}(t)$, at each time-step. The state-space responses are defined in continuous form as

$$z(t) = \begin{Bmatrix} x(t) \\ \dot{x}(t) \end{Bmatrix}_{(2 \times DOF) \times 1} \quad (3)$$

where $x(t)$ is the elastic displacement relative to the ground, and DOF is the system's number of degrees of freedom. The matrices Q and R are weighing matrices that monitor the performance of the system in terms of the desired controlled responses and the applied control force, respectively. Including a control force component on the right side of equation (1) and separating the elastic and inelastic components, the equation of motion of the system can be expressed as



$$M\ddot{\mathbf{x}}(t) + C\dot{\mathbf{x}}(t) + K\mathbf{x}(t) = -M\ddot{\mathbf{x}}_g(t) - \sum_{i=1}^{plex} \alpha_i K \mathbf{x}_{ie,i}(t) + D\mathbf{f}_c(t) \quad (4)$$

where $\mathbf{D}$ is a location vector of the control forces in the building, and $\mathbf{K}$ is the elastic stiffness of the system. The factor α is a proportionality term used to assess the (discretized) inelastic stiffness over the time span of each plastic excursion, where $plex$ is the total number of excursions. The corresponding inelastic displacement over each plastic excursion is defined as $\mathbf{x}_{ie,i}(\mathbf{t})$, which is calculated using equation (5) or (6) as

$$\mathbf{x}_{ie,i}(\mathbf{t}) = \mathbf{x}_{tot}(\mathbf{t}) - \mathbf{x}(\mathbf{t}) - \sum_{j=1}^{i-1} \mathbf{x}_{ie,j}(\mathbf{t}) \quad \text{where } i \geq 2 \quad (5)$$

$$\text{and } \mathbf{x}_{ie,1}(\mathbf{t}) = \mathbf{x}_{tot}(\mathbf{t}) - \mathbf{x}(\mathbf{t}) \quad (6)$$

where $\mathbf{x}(\mathbf{t})$ was previously defined as the elastic displacement. In state-space form, equation (4) can be expressed as

$$\dot{\mathbf{z}}(\mathbf{t}) = \mathbf{A}\mathbf{z}(\mathbf{t}) + \mathbf{H}\ddot{\mathbf{x}}_g(\mathbf{t}) + \sum_{i=1}^{plex} \mathbf{F}_{c,ie} \mathbf{x}_{ie,i}(\mathbf{t}) + \mathbf{B}\mathbf{f}_c(\mathbf{t}) \quad (7)$$

where

$$\mathbf{A} = \begin{bmatrix} \mathbf{0} & \mathbf{I} \\ -\mathbf{M}^{-1}\mathbf{K} & -\mathbf{M}^{-1}\mathbf{C} \end{bmatrix}_{(2 \times \text{DOF}) \times (2 \times \text{DOF})} \quad (8)$$

$$\mathbf{H} = \begin{Bmatrix} \mathbf{0} \\ -\mathbf{I} \end{Bmatrix}_{(2 \times \text{DOF}) \times 1} \quad (9)$$

$$\mathbf{F}_{c,ie} = \begin{Bmatrix} \mathbf{0} \\ -\alpha_i \mathbf{M}^{-1}\mathbf{K} \end{Bmatrix}_{(2 \times \text{DOF}) \times 1} \quad (10)$$

$$\mathbf{B} = \begin{Bmatrix} \mathbf{0} \\ \mathbf{M}^{-1}\mathbf{D} \end{Bmatrix}_{(2 \times \text{DOF}) \times 1} \quad (11)$$

In equations (8) – (11), $\mathbf{I}$ is the identity matrix, $\mathbf{0}$ is a vector of zeros, $\mathbf{H}$ is a location vector that excites all the lateral *DOFs* using the ground accelerations, and $\mathbf{B}$ is a location vector of the control forces used to control the inelastic



responses which are distinguished by the components $\mathbf{F}_{c,ie}$. When the structure displaces inelastically (and nonlinearly), the reduced stiffness, $\alpha\mathbf{K}$, reflects the incurred state of damage that is to be controlled. Equation (2) reflects the minimization of the cost function with respect to $\mathbf{z}(\mathbf{t})$ at each discrete time step by using a non-steady-state Ricatti matrix solution where

$$P_{k+1} = Q + (e^{A\Delta t})^T P_k (I + GR^{-1}G^T P_k)^{-1} e^{A\Delta t} \quad (12)$$

where the convergence of $\mathbf{P}$ per time step occurs when $(\mathbf{P}_{k+1} - \mathbf{P}_k \rightarrow 0)$. In equation (12), $e^{A\Delta t}$ is defined as the continuous state-space equation of motion and $\mathbf{G}$ is defined as

$$G = A^{-1}(e^{A\Delta t} - I)B \quad (13)$$

Finally, the evolutionary gain matrix can be expressed per time step as

$$Gain(ev)_{k+1} = -(G^T P_{k+1} G + R)^{-1} G^T P_{k+1} e^{A\Delta t} \quad (14)$$

where the control force, $f_{c,k+1}$, is a function of the changing states-space response and the evolutionary gain, $Gain(ev)$, at each time step.

$$f_{c,k+1}(t) = [Gain(ev)]z_{k+1} \quad (15)$$

Equation (15) assumes a zero time delay. The minimization of the *total* state-space response, $\mathbf{z}(\mathbf{t})$, using equation (2) is then given in equation (16) as

$$J = \sum_{k=1}^{TimeCount} \int_{t_k}^{t_k+\Delta t} \frac{1}{2} [z_k^T(t) Q z_k^T(t) + f_{c,k}^T(t) R f_{c,k}^T(t)] dt \quad (16)$$

where *TimeCount* is used in CONON to indicate the number of discrete time steps used in the numerical analysis.

3 Numerical example

The procedure using the evolutionary gain, $Gain(ev)$, for controlling inelastic responses as discussed above was applied to a single-degree-of-freedom model consisting of a mass of 87.5kN-s²/m supported by two – 3.65m columns (SI, W310x74 - W12x50) with a total elastic stiffness of 160.8kN/cm ($\sigma_y = 248\text{MPa}$; modulus, $E = 200\text{GPa}$). The elastic natural frequency of this system was found to be 2.16Hz, and a 5% damping ratio was assumed to be present. The stiffness



degradation was characterized by $\alpha_1 = 0.25$ in equations (4), (7), and (10) where the yield deflection, Δ_y , was calculated as 1.79cm and a value of 1 was used for $plex$ in equations (4) and (7). Further, this system was subjected to the El Centro excitation (S00E component) of the 1940 Imperial Valley Earthquake - (EC). The response of the SDOF system was marched in time in CONON starting from zero initial conditions using the Newmark-Beta scheme with linear acceleration and a time step, Δt , of 0.02s.

Shown in Fig. 1(a) is the controlled force-displacement hysteresis for the inelastic SDOF system described above using the evolutionary gain to control demands and satisfy a performance of $1.5^x\Delta_y$, or 2.68cm. Fig. 1(a) shows the uncontrolled response (no control is applied), and the SDOF system in both cases is excited using the EC ground record. Fig. 1(b) shows the controlled hysteresis determined using a 'Constant Gain' formulation where $z(t)$ was calculated over the entire time history. The Ricatti and Gain formulations in equations (12) and (14), respectively, remained constant in the minimization of equation (17).

$$J = \int_0^{\text{TimeCount}} \frac{1}{2} [z(t)Qz(t) + f_c(t)Rf_c^T(t)] dt \quad (17)$$

A comparison of the 'Controlled' and the 'Constant Gain' plots in Figs. 1(a) and 1(b), respectively, reveals a distinct likeness. An advantage of the proposed evolutionary gain formulation is that it does not directly utilize the weighing **Q** and **R** matrices to arrive at the final hysteresis. A change in **Q** and **R** generates the hysteresis shown in Fig. 1(b) that is labeled as 'Constant Gain (Q and R).' Not only did this control system fail to meet the desired performance of 2.68cm, but it also necessitated a control force output that was nearly 3-times as large as that used in the evolutionary gain formulation. This was a result of the additional inelastic energy that the SDOF system lost as it became further damaged. Consequently, the semi-active device tried to restore this loss with a large output. This was not the case with the controller that utilized the evolutionary gain since the displacement converged to the desired performance on *each* time step if the performance was exceeded. A hysteresis using a 'Constant Gain - Delta Z' formulation is also shown in Fig. 1(b), which was computed using $z(t) - z(t)_{\text{target}}$ in equation (2) where $z(t)_{\text{target}}$ was the desired performance of $1.5^x\Delta_y$. For this formulation, the semi-active device provides a small amount energy and control force output to the structure because of the small net value of $z(t) - z(t)_{\text{target}}$.

Shown in Fig. 2(a) are four hysteresis plots when a target displacement demand of $1.0^x\Delta_y$ is used. These plots indicate 1) that some damage occurs in the system (possibly) because of a reaction time-delay of the semi-active device in responding to the imposed demands, 2) that the evolutionary gain and constant gain are once again nearly identical – see the two dark center hysteresis plots in Fig. 2(a) – with similar control force outputs, and 3) that a change in **Q** and **R** causes the 'Constant Gain' system to deviate from the desired performance, which also results in larger control force outputs. In comparing the control force



outputs that were calculated in the inelastic range, the ‘Constant Gain (Q & R)’ system generates about 4-times as much control force as the control system that was formulated using the evolutionary gain; in the elastic range, the former generates about 3-times as much force, which could be costly.

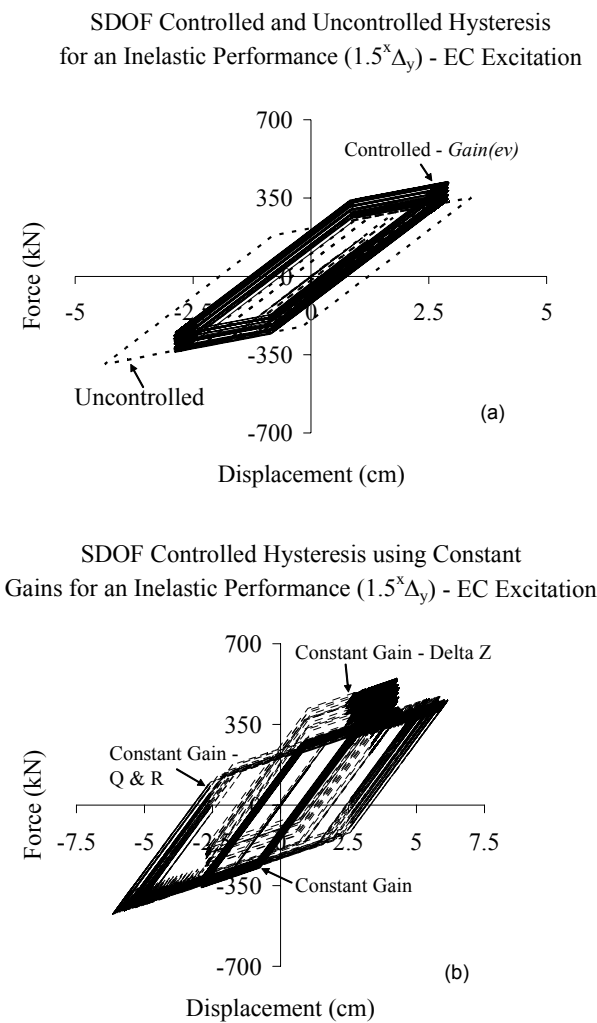
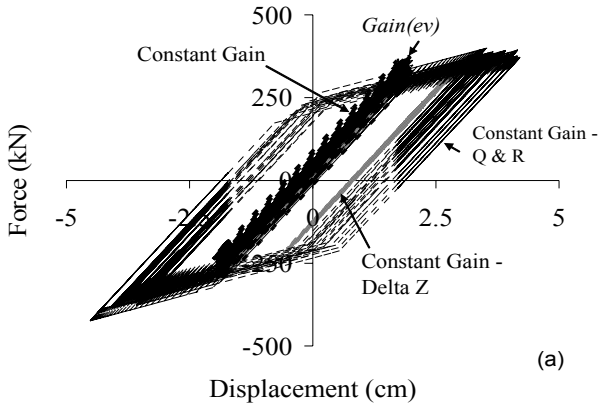


Figure 1: (a) Controlled (evolutionary gain) and uncontrolled hysteresis; (b) controlled hysteresis using a constant gain based on $\Delta z(t)$, a constant gain based on $z(t)$, and a constant gain that uses another set of **Q** and **R** matrices.



SDOF Controlled Hysteresis for an Elastic Performance
($1.0^x\Delta_y$) - EC Excitation



SDOF Controlled Hysteresis for an Elastic Performance
($0.8^x\Delta_y$) - EC Excitation

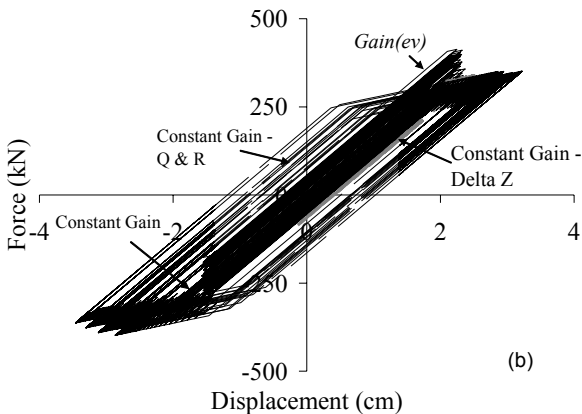


Figure 2: Plots showing the evolutionary gain hysteresis, a constant gain (with delta Z), a constant gain, and another constant gain using different Q and R matrices. The hysteresis are developed in the elastic range for (a) $1.0^x\Delta_y$ and (b) $0.8^x\Delta_y$.

The results in Fig. 2(b) are similar to those in Fig. 2(a), and show that even for a performance 20% below that of yield, the system still behaves inelastically. This magnifies the importance of developing inelastic control algorithms in structural control frameworks. As a comparison, the control model for evolutionary gain was analyzed using two other records as input: 1) a scaled



1994 Northridge record (Alhambra – Fremont School), near-field excitation – sNR; and 2) modulated (non-stationary) Gaussian white noise with Kanai-Tajimi spectra of parameters $\omega_g = 3\text{Hz}$, $\xi_g = 0.6$, and $G_0 = 0.07$, nonstationary record – (mKT). Figs. 3(a) and (b) show the controlled hysteresis loops for a ‘minimal’ inelastic performance objective (sNR) and for an elastic objective (mKT) where there was some damage to the system caused by delays. In both cases, the controller using the evolutionary gain satisfactorily met the performance objectives.

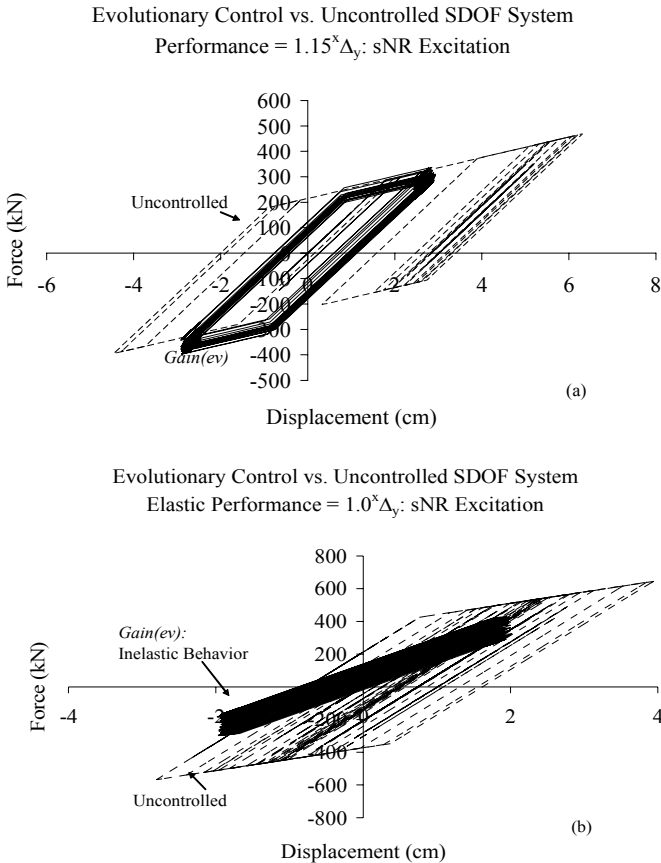


Figure 3: (a) Northridge excitation on a minimally inelastic performance; (b) modulated white noise applied to an elastic performance.

4 Conclusions

An optimal nonlinear control solution is proposed for inelastic and elastic systems using an evolutionary gain formulation to compute the required force output generated by semi-active devices per time step. The procedure discretizes



the structural system's stiffness and separates the elastic and inelastic components based on any plastic excursions experienced by the kinematically strain-hardened system. The algorithm, CONON – Control NONlinear Time-History Analysis – was developed to simulate and control inelastic responses in buildings using the proposed evolutionary gain. The code uses an efficient subroutine to expeditiously converge to the desired performance objectives. It was found that the procedure uses a minimal amount of output control force to adequately converge to the desired elastic and inelastic performance conditions (time-delays notwithstanding) for various near-field, non-stationary, and far-field earthquake excitations without relying on weighing matrices that might inconsistently meet performance criteria and result in excessive output forces.

References

- [1] Ohtori, Y., Christenson, R.E., and Spencer, Jr., B.F. (2004). "Benchmark control problems for seismically excited nonlinear buildings," *Journal of Engineering Mechanics*, 130(4), 366-385.
- [2] Spencer, Jr., B.F., and Nagarajaiah, S. (2003). "State of the art of structural control," *Journal of Structural Engineering*, 129(7), 845-856.
- [3] Yang, G. (2001). "Large-scale magnetorheological fluid damper for vibration mitigation: modeling, testing, and control," Ph.D. Dissertation, Department of Civil Engineering and Geological Sciences, University of Notre Dame, Notre Dame, IN.
- [4] Federal Emergency Management Agency (FEMA) (2001). "NEHRP recommended provisions for seismic regulations for new buildings and other structures. Part 1: Provisions." *FEMA 368*, Washington, DC.
- [5] International Code Council, *2000 International Building Code*, Falls Church, VA., 2000.
- [6] Makris, N. (1997). "Rigidity-plasticity-viscosity: can electrorheological dampers protect base-isolated structures from near-source ground motions?" *Earthquake Engineering & Structural Dynamics*, 26, 571-591.
- [7] Kelly, J.M., (1999), "The current state of base isolation in the United States," *Proc., 2nd World Conference on Structural Control*, Kyoto, Japan, Vol. 1, 1043-1052.
- [8] Madden, G., Symans, M., and Wongprasert, N. (2002). "Experimental verification of seismic response of building frame with adaptive sliding base-isolation system," *Journal of Structural Engineering*, 128(8), 1037 – 1045.
- [9] Attard, T. (2007). "Controlling all Inter-Story Displacements in Highly-Nonlinear-Moment-Resisting Frames Using Optimal Passive Damping," *Journal of Structural Engineering*, ASCE. Accepted for Publication.
- [10] Spencer, B.F. and Nagarajaiah, S. (2003). "State of the art of structural control," *Journal of Structural Engineering*, 129(7), 845-856.
- [11] Varadarajan, N. and Nagarajaiah, S. (2004). "Wind response control of building with variable stiffness tuned mass damper using empirical mode



- decomposition / hilbert transform,” *Journal of Engineering Mechanics*, **130**(4), 451-458.
- [12] Gavin, H.P., Alhan, C. and Oka, N. (2003). “Fault Tolerance of Semi-Active Seismic Isolation,” *Journal of Structural Engineering*, **129**(7), 922-930.
- [13] Franklin, G.F., powell, J.D., and Emami-Naeini, A. (2002). “Feedback Control of Dynamic Systems,” Prentice-Hall, Upper Saddle River, New Jersey.
- [14] Attard, T., and Mignolet, M. (2005). “Evolutionary Model for Random Plastic Analyses of Shear-Frame Buildings Using a Detailed Degradation Model,” *9th International Conference on Structural Safety and Reliability*, ICOSSAR2005, Rome, Italy



Dynamic analysis of plates stiffened by parallel beams

E. J. Sapountzakis & V. G. Mokos

*School of Civil Engineering, National Technical University,
Athens, Greece*

Abstract

In this paper a general solution for the dynamic analysis of plates stiffened by arbitrarily placed parallel beams of arbitrary cross section subjected to an arbitrary dynamic loading is presented. According to the proposed model, the stiffening beams are isolated from the plate by sections in the lower outer surface of the plate, taking into account the arising tractions in all directions at the fictitious interfaces. The aforementioned integrated tractions result in the loading of the beams as well as the additional loading of the plate. Their distribution is established by applying continuity conditions in all directions at the interfaces. The analysis of both the plate and the beams is accomplished on their deformed shape taking into account second-order effects. The method of analysis is based on the capability to establish a flexibility matrix with respect to a set of nodal mass points, while a lumped mass matrix is constructed from the tributary mass areas to these mass points. Both free and forced damped or undamped transverse vibrations are considered and numerical examples with great practical interest are presented. The discrepancy in the obtained eigenfrequencies using the presented analysis (which approximates better the actual response of the plate-beams system since it permits the evaluation of the shear forces at the interfaces in both directions) and the corresponding ones ignoring the inplane forces and deformations justify the analysis based on the proposed model.

Keywords: reinforced plate with beams, nonuniform torsion, warping, ribbed plate, slab-and-beam structure, vibrations, dynamic analysis.



1 Introduction

Structural plate systems stiffened by beams are widely used in buildings, bridges, ships, aircrafts and machines. In this paper a general solution for the analysis of plates stiffened by arbitrarily placed parallel beams is presented. The adopted structural model is a refined one of that proposed by Sapountzakis and Katsikadelis in [1]. Six boundary value problems with respect to the plate transverse deflection, to the plate inplane displacement components, to the beam transverse deflections, to the beam axial deformation and to the beam nonuniform angle of twist are formulated and solved using the Analog Equation Method (AEM) [2], a BEM based method. The essential features and novel aspects of the present formulation compared with previous ones are summarized as follows.

- i. The stiffened plate is subjected to an arbitrary dynamic loading, while both the number and the placement of the parallel stiffening beams are also arbitrary (eccentric beams are also included).
- ii. The influence of the transverse traction component at plate-beams interfaces is taken into account. A nonuniform variation of the distribution of the transverse shear interface force is taken into account by applying compatibility equations on points in the transverse direction. Thus, the adopted model permits the evaluation of the shear connectors in both directions.
- iii. Displacement continuity conditions at the interfaces are applied along all three axes of the coordinate system, leading to the formulation of a system of equations involving two nonlinear functions, namely the longitudinal and transverse inplane shear forces at the interfaces.
- iv. The eccentricities of both the centroid and the shear center axes with respect to the midline of the plate – beam interface are also included.
- v. The nonuniform torsion in which the stiffening beams are subjected is taken into account by solving the corresponding problem and by comprehending the arising twisting and warping in the corresponding displacement continuity conditions.
- vi. Terms arising from the internal variable axial loading of both the plate and the beams coming from the longitudinal and transverse inplane shear forces at the interfaces are taken into account.
- vii. Damping resistance is also included.

2 Statement of the problem

Consider a thin plate of homogeneous, isotropic and linearly elastic material with modulus of elasticity E and Poisson ratio ν , having constant thickness h_p and occupying the two dimensional multiply connected region Ω of the x, y plane bounded by the piecewise smooth $K+1$ curves $\Gamma_0, \Gamma_1, \dots, \Gamma_{K-1}, \Gamma_K$, as shown in Fig.1. The plate is stiffened by a set of $i = 1, 2, \dots, I$ arbitrarily placed parallel beams of homogeneous, isotropic and linearly elastic material with modulus of



elasticity E_b^i and Poisson ratio ν_b^i , which may have either internal or boundary point supports. For the sake of convenience the x axis is taken parallel to the beams. The stiffened plate is subjected to the lateral load $g = g(\mathbf{x}, t)$, $\mathbf{x} : \{x, y\}, t \geq 0$. For the analysis of the aforementioned problem a global coordinate system Oxy for the analysis of the plate and local coordinate ones $O_i x_i y_i$ and $O_i \tilde{x}_i \tilde{y}_i$ corresponding to the centroid and shear center axes of each beam are employed as shown in Fig.1.

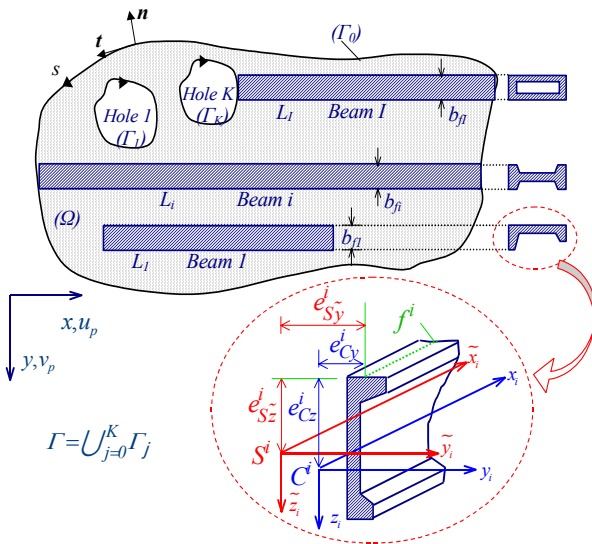


Figure 1: Two-dimensional region Ω occupied by the plate.

The solution of the problem at hand is approached by a refined model of that proposed by Sapountzakis and Katsikadelis in [1]. According to this model, the stiffening beams are isolated from the plate by sections in the lower outer surface of the plate, taking into account the arising tractions at the fictitious interfaces (Fig.2). Integration of these tractions along the width of the i -th beam results in line forces per unit length, which are denoted by q_x^i , q_y^i and q_z^i encountering in this way the influence of the transverse component q_y , which in the aforementioned model [1] was ignored. The aforementioned integrated tractions result in the loading of the i -th beam as well as the additional loading of the plate. Their distribution is unknown and can be established by imposing displacement continuity conditions at the interfaces along x_i , y_i and z_i local axes following the procedure developed in this investigation.



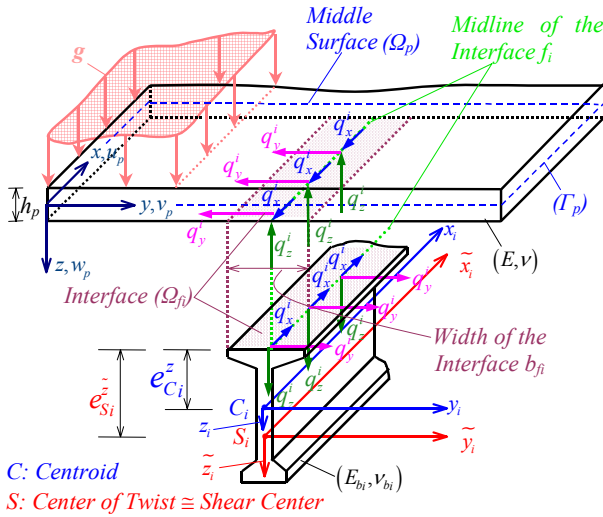


Figure 2: Isolation of the beams from the plate.

On the basis of the above considerations the response of the plate and of the beams may be described by the following initial boundary value problems.

(a) *For the plate.* The plate undergoes transverse deflection and inplane deformation. Thus, for the transverse deflection the equation of equilibrium employing the linearized second order theory can be written as

$$D\nabla^4 w_p + \rho_p \ddot{w}_p + c_p \dot{w}_p - \left(N_x \frac{\partial^2 w_p}{\partial x^2} + 2N_{xy} \frac{\partial^2 w_p}{\partial x \partial y} + N_y \frac{\partial^2 w_p}{\partial y^2} \right) =$$

$$g - \sum_{i=1}^I \left(q_z^i + \frac{\partial m_{py}^i}{\partial x} + \frac{\partial m_{px}^i}{\partial y} - q_x^i \frac{\partial w_p}{\partial x} - q_y^i \frac{\partial w_p}{\partial y} \right) \delta(y - y_i) \quad \text{in } \Omega \quad (1)$$

the corresponding boundary conditions as

$$\alpha_{p1}w_p + \alpha_{p2}R_{pn} = \alpha_{p3} \quad \beta_{p1}\frac{\partial w_p}{\partial n} + \beta_{p2}M_{pn} = \beta_{p3} \quad \text{on } \Gamma \quad (2a,b)$$

and the initial conditions as

$$w_p(\mathbf{x}, 0) = w_{p0}(\mathbf{x}) \quad \dot{w}_p(\mathbf{x}, 0) = \bar{w}_{p0}(\mathbf{x}) \quad (3a, b)$$

where $w_p = w_p(x, y)$ is the time dependent transverse deflection of the plate; $D = Eh_p^3 / 12(1 - \nu^2)$ is its flexural rigidity; $N_x = N_x(\mathbf{x}, t)$, $N_y = N_y(\mathbf{x}, t)$, $N_{xy} = N_{xy}(\mathbf{x}, t)$ are the membrane forces per unit length of the plate cross section; $m_{py}^i = q_x^i h_p / 2$; $m_{px}^i = q_y^i h_p / 2$; $\rho_p = \rho h_p$ is the surface mass density of the plate with ρ being the volume mass density; c_p is the plate flexural

damping constant; $w_{p0}(\mathbf{x})$, $\bar{w}_{p0}(\mathbf{x})$ are the initial deflection and the initial velocity of the points of the middle surface of the plate; $\delta(y - y_i)$ is the Dirac's delta function in the y direction; M_{pn} and R_{pn} are the bending moment normal to the boundary and the effective reaction along it, respectively. Finally, a_{pi} , β_{pi} ($i = 1, 2, 3$) are functions specified on the boundary Γ .

Since linearized plate bending theory is considered, the components of the membrane forces N_x , N_y , N_{xy} are given as

$$\begin{aligned} N_x &= C \left(\frac{\partial u_p}{\partial x} + \nu \frac{\partial v_p}{\partial y} \right) & N_y &= C \left(\nu \frac{\partial u_p}{\partial x} + \frac{\partial v_p}{\partial y} \right) \\ N_{xy} &= C \frac{1-\nu}{2} \left(\frac{\partial u_p}{\partial y} + \frac{\partial v_p}{\partial x} \right) \end{aligned} \quad (4a,b,c)$$

where $C = Eh_p / (1 - \nu^2)$; $u_p = u_p(\mathbf{x}, t)$ and $v_p = v_p(\mathbf{x}, t)$ are the displacement components of the middle surface of the plate arising from the line body forces q_x^i , q_y^i ($i = 1, 2, \dots, I$). These displacement components are established by solving independently the plane stress problem, which is described by the following quasi-static (inplane inertia forces are ignored) boundary value problem (Navier's equations of equilibrium)

$$\nabla^2 u_p + \frac{1+\nu}{1-\nu} \frac{\partial}{\partial x} \left[\frac{\partial u_p}{\partial x} + \frac{\partial v_p}{\partial y} \right] - \frac{1}{Gh_p} \sum_{i=1}^I q_x^i \delta(y - y_i) = 0 \quad (5a)$$

$$\nabla^2 v_p + \frac{1+\nu}{1-\nu} \frac{\partial}{\partial y} \left[\frac{\partial u_p}{\partial x} + \frac{\partial v_p}{\partial y} \right] - \frac{1}{Gh_p} \sum_{i=1}^I q_y^i \delta(y - y_i) = 0 \text{ in } \Omega \quad (5b)$$

$$\gamma_{p1} u_{pn} + \gamma_{p2} N_n = \gamma_{p3} \quad \delta_{p1} u_{pt} + \delta_{p2} N_t = \delta_{p3} \quad \text{on } \Gamma \quad (6a,b)$$

in which $G = E / 2(1 + \nu)$ is the shear modulus of the plate; N_n , N_t and u_{pn} , u_{pt} are the boundary membrane forces and displacements in the normal and tangential directions to the boundary, respectively; γ_{pi} , δ_{pi} ($i = 1, 2, 3$) are functions specified on the boundary Γ .

(b) For each beam. Each beam undergoes transverse deflection with respect to z_i and y_i axes, axial deformation along x_i axis and nonuniform angle of twist along $\tilde{x}_i$ axis. Thus, for the transverse deflection with respect to z_i axis the equation of equilibrium employing the linearized second order theory can be written as

$$E_b^i I_y^i \frac{\partial^4 w_b^i}{\partial x_i^4} + \rho_b \ddot{w}_b^i + c_b^i \dot{w}_b^i - N_b^i \frac{\partial^2 w_b^i}{\partial x_i^2} = q_z^i - q_x^i \frac{\partial w_b^i}{\partial x_i} + \frac{\partial m_{by}^i}{\partial x_i} \quad \text{in } L_i \quad (7)$$



the corresponding boundary conditions as

$$a_{1i}^z w_b^i + a_{2i}^z R_z^i = a_{3i}^z \quad \beta_{1i}^z \theta_y^i + \beta_{2i}^z M_y^i = \beta_{3i}^z \quad \text{at the beam ends} \quad (8a,b)$$

and the initial conditions as

$$w_b^i(x, 0) = w_{b0}^i(x) \quad \dot{w}_b^i(x, 0) = \bar{w}_{b0}^i(x) \quad (9a,b)$$

where $w_b^i = w_b^i(x_i, t)$ is the time dependent transverse deflection of the i -th beam with respect to z_i axis; I_y^i is its moment of inertia with respect to y_i axis; $N_b^i = N_b^i(x_i, t)$ is the axial force at the x_i centroid axis; ρ_b is the surface mass density of the beams; $m_{by}^i = q_x^i e_{Cz}^i$; c_b^i is the i -th beam flexural damping constant; $w_{b0}^i(x)$, $\bar{w}_{b0}^i(x)$ are the initial deflection and the initial velocity of the points of the neutral axis of the i -th beam with respect to z_i axis; a_{ji}^z , β_{ji}^z ($j = 1, 2, 3$) are coefficients specified at the boundary of the i -th beam; θ_y^i , R_z^i , M_y^i are the slope, the reaction and the bending moment at the i -th beam ends, respectively.

The $v_b^i = v_b^i(x_i)$ transverse deflection with respect to y_i axis must satisfy the following quasi-static (transverse inertia forces with respect to y_i axis are ignored) boundary value problem

$$E_b^i I_z^i \frac{\partial^4 v_b^i}{\partial x_i^4} - N_b^i \frac{\partial^2 v_b^i}{\partial x_i^2} = q_y^i - q_x^i \frac{\partial v_b^i}{\partial x_i} - \frac{\partial m_{bz}^i}{\partial x_i} \quad \text{in } L_i, i = 1, 2, \dots, I \quad (10)$$

$$a_{1i}^y v_b^i + a_{2i}^y R_y^i = a_{3i}^y \quad \beta_{1i}^y \theta_z^i + \beta_{2i}^y M_z^i = \beta_{3i}^y \quad \text{at the beam ends} \quad (11a,b)$$

where I_z^i is the moment of inertia of the i -th beam with respect to y_i axis; $m_{bz}^i = -q_x^i e_{Cy}^i$; a_{ji}^y , β_{ji}^y ($j = 1, 2, 3$) are coefficients specified at its boundary; θ_z^i , R_y^i , M_z^i are the slope, the reaction and the bending moment at the i -th beam ends. Since linearized beam bending theory is considered the axial deformation u_b^i of the beam arising from the arbitrarily distributed axial force q_x^i ($i = 1, 2, \dots, I$) is described by solving independently the following quasi-static (axial inertia forces are neglected) boundary value problem

$$E_b^i A_b^i \frac{\partial^2 u_b^i}{\partial x_i^2} = -q_x^i \quad \text{in } L_i, i = 1, 2, \dots, I \quad (12)$$

$$a_{1i}^x u_b^i + a_{2i}^x N_b^i = a_{3i}^x \quad \text{at the beam ends} \quad (13)$$

where N_b^i is the axial reaction at the i -th beam ends given as



$$N_b^i = E_b^i A_b^i \frac{\partial u_b^i}{\partial x_i} \quad (14)$$

Finally, the nonuniform angle of twist with respect to $\tilde{x}_i$ shear center axis has to satisfy the following quasi-static (torsional and warping inertia moments are ignored) boundary value problem

$$E_b^i I_w^i \frac{\partial^4 \theta_x^i}{\partial \tilde{x}_i^4} - G_b^i I_x^i \frac{\partial^2 \theta_x^i}{\partial \tilde{x}_i^2} = -q_y^i e_{S_z}^i + q_z^i e_{S_y}^i \quad \text{in } L_i, i = 1, 2, \dots, I \quad (15)$$

$$a_{1i}^{\tilde{x}} \theta_x^i + a_{2i}^{\tilde{x}} M_x^i = a_{3i}^{\tilde{x}} \quad \beta_{1i}^{\tilde{x}} \frac{\partial \theta_x^i}{\partial \tilde{x}_i} + \beta_{2i}^{\tilde{x}} M_w^i = \beta_{3i}^{\tilde{x}} \quad \text{at the beam ends} \quad (16a,b)$$

where $\theta_x^i = \theta_x^i(\tilde{x}_i)$ is the variable angle of twist of the i -th beam along the $\tilde{x}_i$ shear center axis; $G_b^i = E_b^i / 2(1 + \nu_b^i)$ is its shear modulus; I_w^i , I_x^i are the warping and torsion constants of the i -th beam cross section, respectively $a_{ji}^{\tilde{x}}$, $\beta_{ji}^{\tilde{x}}$ ($j = 1, 2, 3$) are coefficients specified at the boundary of the i -th beam; M_x^i is the twisting moment and M_w^i is the warping moment due to the torsional curvature at the boundary of the i -th beam.

Eqns. (1), (5a), (5b), (7), (10), (12), (15) constitute a set of seven coupled partial differential equations including ten unknowns, namely w_p , u_p , v_p , w_b^i , v_b^i , u_b^i , θ_x^i , q_x^i , q_y^i , q_z^i . Three additional equations are required, which result from the displacement continuity conditions in the direction of x_i , y_i and z_i local axes at the midline of each (i -th) plate – beam interface. These conditions can be expressed as

$$w_p - w_b^i = e_{S_y}^i \theta_x^i \quad \text{in the direction of } z_i \text{ local axis} \quad (17)$$

$$u_p - u_b^i = \frac{h_p}{2} \frac{\partial w_p}{\partial x} - e_{C_z}^i \frac{\partial w_b^i}{\partial x_i} - e_{C_y}^i \frac{\partial v_b^i}{\partial x_i} + \left(\phi_S^P \right)_{f_i} \frac{\partial \theta_x^i}{\partial \tilde{x}_i} \quad \text{in } x_i \text{ local axis} \quad (18)$$

$$v_p - v_b^i = -\frac{h_p}{2} \frac{\partial w_p}{\partial y} - e_{S_z}^i \theta_x^i \quad \text{in the direction of } y_i \text{ local axis} \quad (19)$$

where $\left(\phi_S^P \right)_{f_i}$ is the value of the primary warping function with respect to the shear center S of the beam cross section at the midline of the f_i (i -th) interface.



In all the aforementioned equations the values of all the eccentricities e_{Cz}^i , e_{Cy}^i , e_{Sz}^i , e_{Sy}^i and of the primary warping function $\varphi_S^P(\tilde{y}_i, \tilde{z}_i)$ should be set having the appropriate algebraic sign corresponding to the local beam axes.

It is worth here noting that the coupling of the aforementioned equations is nonlinear due to the terms including the unknown q_x^i and q_y^i interface forces.

3 Solution procedure

The numerical solution of the aforementioned problem is achieved employing the method presented by Katsikadelis and Kandilas [3]. According to this method the domain Ω occupied by the plate is discretized by establishing a system of M nodal points on it, corresponding to M mass cells, to which masses are assigned according to the lumped mass assumption. Subsequently, the stiffness matrix, the damping matrix as well as the load vector with respect to these nodal points are established employing the Analog Equation Method [2], a BEM based method. This procedure leads to the typical equation of motion for the stiffened plate

$$[m]\{\ddot{w}\} + [c]\{\dot{w}\} + [k]\{w\} = \{g\} \quad (20)$$

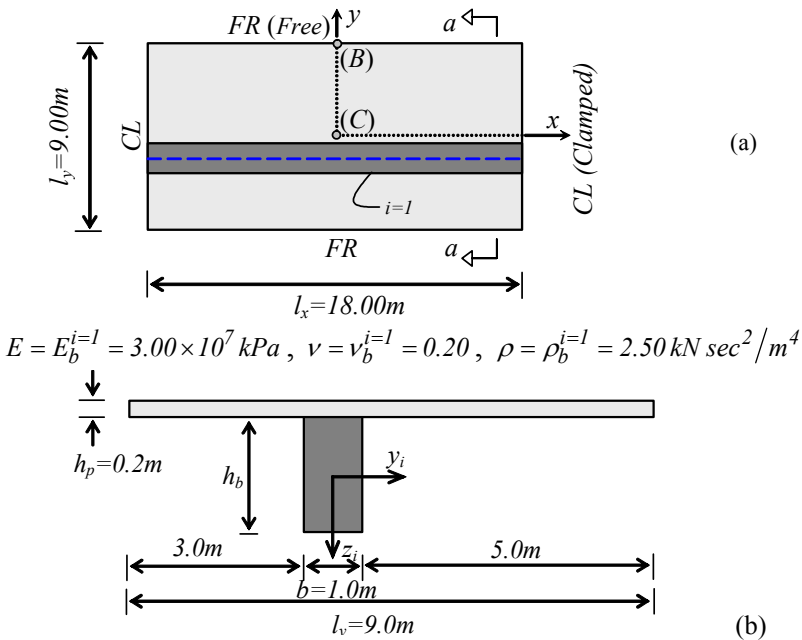


Figure 3: Plan view (a) and section a-a (b) of the stiffened plate.

4 Numerical examples

A rectangular plate stiffened by an eccentrically placed rectangular beam as shown in Fig.3 has been studied. In Table 1 the first four eigenfrequencies taking into account or ignoring the interface forces are presented as compared with those obtained from FEM solutions.

Table 1: $\Omega_n = \omega_n \sqrt{\rho}$ for various beam heights of the stiffened plate.

Ω_n	AEM with q_x, q_y (Present study)	AEM without q_x, q_y	Shell-Beam FE (SAP 2000)	Shell FE (NASTRAN)	Solid FE (NASTRAN)
No beam					
1	–	22.1365	22.1434	22.1092	22.1159
2	–	37.6026	37.1742	37.0602	37.1034
3	–	61.419	61.0651	60.9110	60.9295
4	–	85.2215	84.1991	83.9974	84.0378
Beam height 50cm					
1	36.42768	35.0639	40.8344	41.5054	44.4339
2	73.06466	49.0746	58.8534	59.3171	64.5881
3	82.96732	81.4040	85.9298	86.1819	90.4215
4	122.3293	108.9031	114.3645	113.3465	126.3263
Beam height 100cm					
1	41.6788	37.8244	48.2654	48.3007	52.6324
2	84.6763	76.65077	86.4500	86.0006	94.2175
3	89.0430	85.9815	89.4149	89.2065	98.2054
4	135.9904	128.5372	135.5596	135.2461	151.5446
Beam height 150cm					
1	47.7668	41.5472	50.3380	50.1915	55.0737
2	92.05131	84.1049	90.0815	89.8198	95.0838
3	93.50865	89.7506	95.9265	95.2206	112.6931
4	146.0206	138.0287	138.5487	138.0804	152.9531
Beam height 200cm					
1	51.2582	45.5581	51.1244	50.9369	56.1035
2	93.2207	89.6741	90.3146	90.0342	95.4062
3	101.8806	91.7293	99.4490	98.6612	118.3639
4	151.0629	144.7945	139.5643	139.0592	153.1325

5 Concluding remarks

The proposed model permits the study of a stiffened plate subjected to an arbitrary loading, while both the number and the placement of the parallel



stiffening beams are also arbitrary (eccentric beams are also included). The accuracy of the results compared with solid FE is remarkable.

References

- [1] Sapountzakis, E.J. & Katsikadelis, J.T., Analysis of Plates Reinforced with Beams, *Computational Mechanics*, 26, pp. 66-74, 2000.
- [2] Katsikadelis, J.T., The Analog Equation Method. A Boundary – only Integral Equation Method for Nonlinear Static and Dynamic Problems in General Bodies, *Theor. and Appl. Mech.*, 27, pp.13-38, 2002.
- [3] Katsikadelis, J. T. & Kandilas, C.B., A Flexibility Matrix Solution of the Vibration Problem of Plates Based on the Boundary Element Method, *Acta Mechanica*, 83, pp.51-60, 1990.



Dynamics in the practice of structural design: the problems of implementation

O. S. Saar

Sircovich Saar Civil Engineering, Jerusalem, Israel

Abstract

This paper stresses the difficulties practising engineers are facing in order to keep pace with the continuously evolving knowledge on structural dynamics, and suggests possible ways to make this knowledge accessible to them towards implementation in the practice of structural design.

Keywords: problems of implementation, practising engineer, design situation for statics or dynamics, designers approach, online journal.

1 Introduction

It is very well known that a remarkably great number of engineers in the practice of structural design, all over, are confronted with difficulties when dealing with a dynamic design situation. This paper is intended to focus attention on this anomaly and to outline some possible implications.

2 The problems of implementation

Structural dynamics is, no doubt, the most complex subject that underlines the broad knowledge required from an engineer in order to perform the practice of structural design. Because of that, engineers are trying continuously to enlarge their professional knowledge by looking for sources of additional learning on the issue, such as technical books, journals, publications, descriptions of particular dynamic events or any information that can enrich their understanding or provide them with guiding tools for practical application. But, going through those sources reveals itself to be a very hard, sometimes impossible task, since sooner or later they are faced, unfortunately, with familiar concepts that had already lost a clear quantitative meaning such as damping, hysteresis, impulse, or with



mathematical elaborations that are difficult to follow, since they are not a part of their daily occupation.

For a vast number of engineers in design offices, dynamic design situations that depart from the Codes or are not included there at all, are sporadic events, as opposed to static design situations, which comprise the bulk of their work.

The main difference between a static and a dynamic design situation, for a structural engineer, can be described through a comparison between two design situations of a simple, statically determinate beam in the elastic stage, namely when under a static compared to a dynamic load; in the static design situation, calculations and design can be performed directly. By contrast, in the dynamic design situation, the external load, which varies with time, imparts on the structure dynamic forces those themselves vary with time; an interdependence is therefore created between the external load and the geometrical-mechanical characteristics of the beam and calculations and design become more complicated than those typical of a static design situation. In some cases, calculations can reach highly sophisticated levels, requiring a special mathematical training that is uncommon in the practice of structural design.

In the present computer age it is quite normal for the engineer to rely on the correct use of computers for a solution that satisfies the requirements of safety and performance. Practising engineers should not have to deal with the mathematical formulation and solution of the differential equations of equilibrium. In order to deal with problems of structural design they should have mainly a clear and thorough understanding of the dynamic issue.

Mathematical treatment of a dynamic case is indeed the right way to find the response of a structure in a particular dynamic design situation; this makes the issue closer to the activity of an expert than to that of an engineer facing a practical problem to solve.

Because of that a clear distinction should be made, when considering dynamics in structural design, between those regarding academic activities and those to be applied by the engineer in the daily practice. In the following table a comparison is made of a similar work performed by a structural engineer and by an academic:

<u>Structural Designer</u>	<u>Expert's Approach</u>
1. Geometrical-mechanical definition of the structure.	1. Physical and mathematical modeling; formulation of the dynamic equation of motion.
2. Input of structure's model and its loads into a computer program.	2. Mathematical solution of the equation of motion.
3. Interpretation of graphic and numerical output as required for the design of the structure.	3. Formulation of structure's response function; elaboration of response curves.



3 The practising engineer

Practising structural engineers develop with time a certain ‘feel’ for structures in their daily work, which is actually the result of an extensive practice together with a deep and correct professional knowledge. To develop such an “intuition” for a dynamic design situation, the engineer should pay special attention to other factors, in addition to the stiffness k , which is characteristic of a static design. These are the damping ratio ξ , and mass M ; but since those three factors are interacting, the engineer finds it very hard to trail along a trial and error path.

The engineer tries usually to focus his/her attention, whenever possible, on the physical meanings of concepts related to dynamics: the period “ T ” and the natural frequency “ $f=1/T$ ” for example, have very clear physical meaning, but then arises the question what is the physical meaning of “ Ω ”, the natural circular frequency of a structure? Does it represent, by chance, the circular rotating motion of a mass in a rotating machine? If so, how is it connected, for example, with the movement of a mass in a reciprocating machine? He can try perhaps to give a physical explanation saying that Ω measures indeed the vibrating movement of the structure as a motion along a mathematical “ 2π ” circle. In this manner Ω will represent the number of hypothetical circles per second.

For academia the natural circular frequency belongs to the beautiful construction of the world of mathematics that makes possible to develop and achieve the present and continuously evolving knowledge in structural dynamics; in practice, “ $\Omega=2\pi f$ ” is measured in radians per seconds.

The world of design of the structural engineer is full of questions when facing a dynamic situation which is not a regular structure subjected to earthquake or wind loads; that design situation is covered within the Norms; but, how should he/she proceed if for example the span of a bridge is in the transition category limits, or when designing a building with irregular plan, or when some construction details correctly designed in his office were changed, unintentionally, at the construction site, or when he/she would like to ameliorate the response of a structure by introducing limited changes in the foundation?

Actual codes do not always provide an adequate treatment to a variety of dynamic events that may require consideration in the process of design, such as external dynamic loads produced by human activity (dancing, running, skipping, etc.), operating machinery (turbine, printing press, forging, etc.), road/rail traffic, construction activities (piling, blast excavation, heavy compaction, etc.), collisions of cars or airplanes, collapse of ground foundations, etc.

Proper consideration of dynamic effects on structures such as overstressing, vibrations and fatigue of structural materials is becoming more important with the modern tendency to design slender and elegant structures. Those structures are the result of conscious professional efforts to stress the engineer’s capabilities, the use of stronger materials, and the implementation of sophisticated building technologies.



Among the consequences that the engineer should be aware of are not only building damages, but also physiological and psychological effects on people, degradation of machine performances, and more. These effects cannot always be quantified in the output of computer calculations, but the engineer is expected to take adequate preventive measures whenever possible.

Unfortunately, for some of these effects there are neither satisfactory answers in the Codes nor sound and strong applicable rules or reliable guidelines. True, there are dozens of periodic publications on dynamics, but none of these again is oriented to the daily practice of structural design; this creates an intolerable situation for the practising engineer who can not find with relative ease, the answers he/she is looking for.

4 The practice of structural design

The need for easy answers is pertinent to the practice of structural design: the latter is a highly absorbent and intensive daily work, with a variety of dominant activities, besides the proper dedication to statical calculations and design.

A description of the practice of structural design would include first:

- *a preliminary technical phase* of coordination with other professionals of the design team, consultations with contractors and budget considerations.
- *a creative phase* of conceptual elaboration of a structural solution, including structural materials, basic dimensions and site construction considerations.
- *a phase of preliminary statical calculations*, based on already accorded geometry.
- *a phase of preliminary appreciation of costs* including discussions with owner and architect.

Only then does the right time arrive for *comprehensive statical calculations and preparation of construction drawings and detailing*, followed by *computations and special remarks for tender*.

This varied and intensive list of activities, performed during a long period of time, is still to be complemented with *top site construction supervision* which will include, most probably, corrections of construction errors, which are not so easy to elaborate.

5 Didactic tools

Along the last fifty years, authors expressed their concern about the difficulties to make structural dynamics accessible, both to students and engineers in the daily practice.

“students find the mathematical manipulation so intriguing that they fail to develop the physical understanding essential for good design”; Biggs [1].

“In developing this book, much emphasis has been placed on making structural dynamics accessible to students and professional engineers because many find this subject to be difficult”; Chopra [2].



More recently the book “Dynamics in the Practice of Structural Design” [3] came about as a result of the author’s close awareness, as a professional, of the world of the practising engineer. It is therefore emphatically user-friendly.

Since structural dynamics is a subject of evolving knowledge, practising engineers understandably need continuous updating. They are conscious of this need and avidly seek such knowledge. Hence it is very important that information/learning material be made accessible to them.

Conferences like ERES are very important, rich and highly instructive in the topics they embrace. They should be complemented with other didactic tools that will be then made available to practising engineers, wherever they are, on a periodic basis.

The academic community should stand behind this platform since the multi-facet work of the practising engineer and the enormous responsibility he carries on his shoulders, entitles him to expect their support.

References

- [1] An Introduction to Structural Dynamics. John M. Biggs 1964. McGraw-Hill, Inc.
- [2] Dynamics of Structures. Anil K. Chopra 1995. Prentice Hall
- [3] Dynamics in the Practice of Structural Design. Oscar Sircovich Saar 2006. WIT Press



This page intentionally left blank

Effect of impulsive force on earthquake response of rocking structural systems

T. Azuhata¹, T. Ishihara¹ & M. Midorikawa²

¹*National Institute for Land and Infrastructure Management, Japan*

²*Hokkaido University, Japan*

Abstract

To reduce seismic damage of steel building structures, a rocking structural system which employs the yielding mechanism of base plates has been suggested by the authors. When weak base plates yield due to column tension during a strong earthquake ground motion, the columns uplift and enable the building structure to rock. The performance of the suggested system for earthquake response reduction has previously been successfully demonstrated by the use of shaking table tests on a three story half scale braced frame. However, these tests also established that considerable vertical impulsive force occurs at the column bases when the uplifting columns touch down to their original position. The objective of this study is to investigate the influence of this impulsive force on earthquake response of real scale rocking structures. Earthquake response analyses are carried out on a steel model frame with yielding base plates. The frame has ten stories and one bay. The height and the width are 37.8 m and 7.5 m, respectively. The viscous damping is assumed to be proportional to the initial stiffness. The critical damping ratio of 2.0% is introduced to the first mode. The analysis results showed that vertical response acceleration on the frame is largely and instantly amplified by the impact effect when the uplifting column touches down. However, the impulsive vertical force does not damage the column, because it damps very quickly under the assumption of stiffness-proportional viscous damping.

Keywords: rocking structural system, uplift, impulsive force, seismic damage reduction, yielding base plate, steel building structure.



1 Introduction

Past studies have pointed out that the effects of rocking accompanied with uplift motion may reduce the seismic damage to buildings subjected to strong earthquake ground motions [for example, 1-3]. Based on these studies, structural systems have been developed that permit rocking vibration and uplift motions under appropriate control during strong earthquake ground motions [4-10]. The past shaking table tests by the authors on a three story half scale braced frame verified that the rocking structural system, which employs the yielding mechanism of the base plates, can reduce earthquake responses of the frame [7-10]. However, these tests also showed that the considerable vertical impulsive force occurs at the column bases by the impact effect when the uplifting columns touch down to their original position. When applying the rocking structural systems to real buildings, we need to clear the influence of this impulsive vertical force. In this study, earthquake response analyses are carried out on a real scale model frame which is applied the rocking structural systems to and the influence of the impulsive vertical force on its earthquake responses is investigated.

Table 1: Section of structural members.

	Floor	Section
Column	8-10	□-500x500x19
	1-7	□-500x500x25
Beam	7-RF	H-588x300x12x20
	2-6	H-700x300x13x24

Table 2: Yield strength of steel.

Beam under 2F	588 (kN/mm ²)
Other members	294 (kN/mm ²)

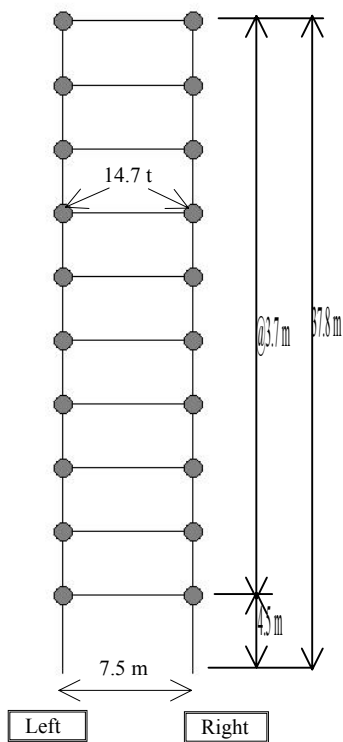


Figure 1: Model frame.

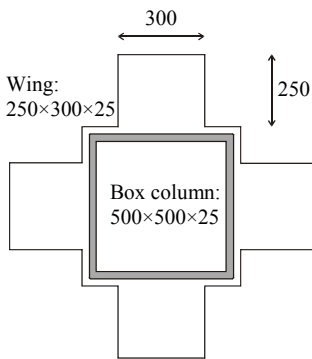


Figure 2: Plan of base plate with four thin wings.



2 Earthquake response of rocking structural system

2.1 Model frame and analytical procedure

The model steel frame used in this study is shown in Fig. 1. The frame has ten stories and one bay. The height and the width are 37.8m and 7.5m, respectively. The total weight is 2881.2kN. To permit the frame uplift motion, the yielding base plate shown in Fig. 2 is attached at each column base on the first floor. The sections of the structural members are shown in Table 1. And yield strength of steel is shown in Table 2. Because the yielding base plate cannot fix the rotation of the column base, the beam under the second floor bends easily. Thus higher strength steel is used only for it. Fig. 3 shows the numerical model for the base part of the frame. To represent uplift motion, two types of springs are attached at each column base. The force-deformation relationships of the springs are shown in Fig. 3 (b)(c). As for base plates, the relationship shown in Fig. 3(b) is modeled based on the static test results [11, 12]. The characteristic values of the base plate are shown in Table 3. The bending force-deformation relationship of edge parts of columns and beams is a normal-bilinear type. The M-N interaction is considered to evaluate response of columns.

Mode shapes and natural periods of the model frame are shown in Fig. 4. According to our past study, shaking table test results including the impact effect on the test frame that was permitted uplift motion can be represented by the FEM dynamic response analysis method assuming the viscous damping is proportional to the initial stiffness [10]. Thus the analysis procedure in this study is also based on the same assumption about the viscous damping. The critical damping ratios for three representative modes are shown in Table 4. The ratio introduced to the first mode is 2%. The predominant mode in vertical is the sixth mode with the natural period of 0.106s. The critical damping ratio for this mode is 29.48%.

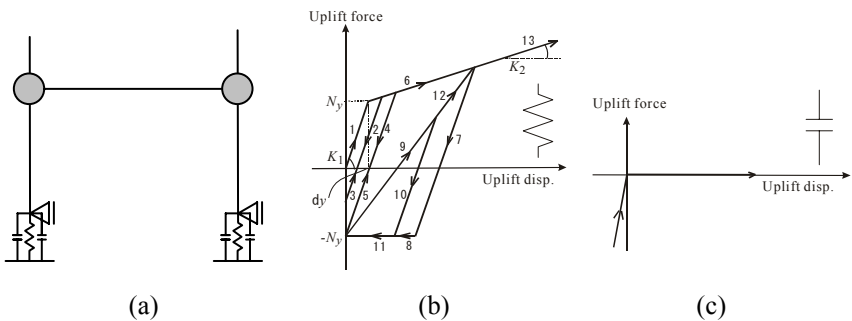


Figure 3: Numerical model for base part of frame. (a) The base of the rocking system, (b) base plate, (c) ground contact.

Table 3: Characteristic values of base plate.

Q_y	K_1	δ_y	K_2/K_1
441 (kN)	146 (kN/mm)	3.0 (mm)	0.2



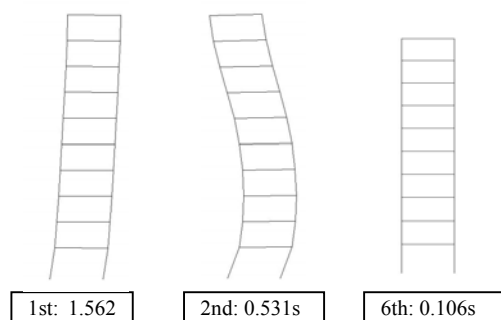


Figure 4: Mode shapes and natural periods.

Table 4: Viscous damping ratios.

	T (s)	h (%)
1st	1.562	2.00
2nd	0.531	5.88
6th	0.106	29.48

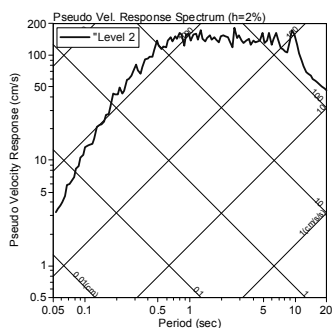


Figure 5: Tripartite response spectrum of input motion.

The input ground motion is an artificial ground motion (BCJ L2), which is used for structural design of high-rise buildings in Japan. The time duration is 120s and the peak velocity is 0.5m/s. The linear response spectrum for 1-DoF systems with critical damping ratio $h=2\%$ is shown in Fig. 5. For calculating the earthquake responses, a step by step time history response analysis method is used. The time interval of numerical integration is 0.001s.

2.2 Analysis results

In Fig. 6, the maximum roof drift angle and base shear of the test frame with yielding base plates that permitted uplift motion (BPY model) are compared with those of the same frame whose bases are fixed (F model). Solid lines show the corresponding results of static pushover analyses for the both models. The Ai-distribution, which is regulated by Japanese building seismic code, is used as the lateral force distribution for the static analyses. By permitting the frame uplift motion, the maximum base shear of the test frame can be reduced although the maximum roof drift angle is increased a little. Fig. 7 shows the damage aspect of the test frame with the bases fixed. Plastic hinges occur in some beams. In contrast, the analysis result for the test frame permitted uplift motion shows all structural members keep elastic except the yielding base plates.



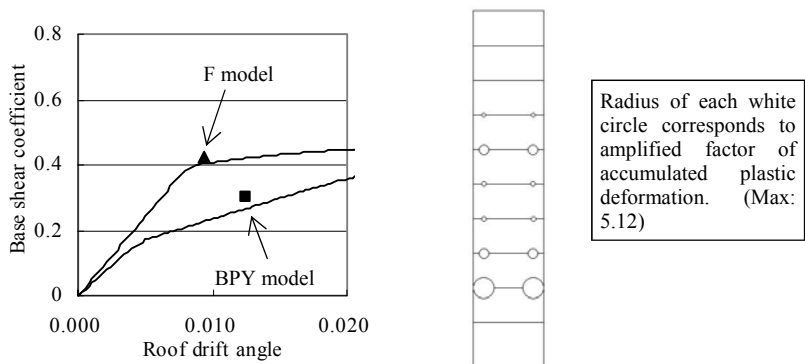


Figure 6: Roof drift angle and base shear coefficient.

Figure 7: Damage aspect of F model.

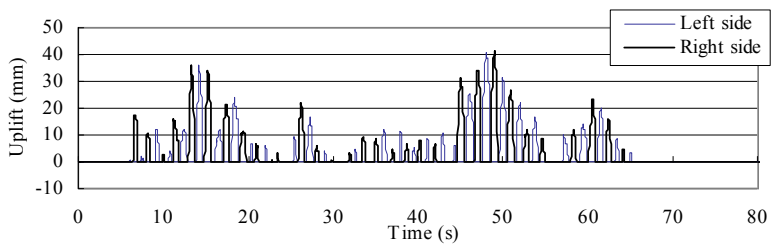


Figure 8 Time history of uplift displacement.

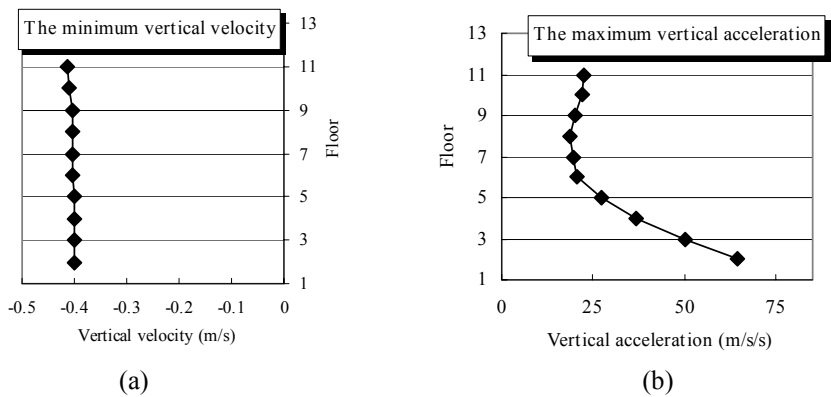


Figure 9: Peak vertical responses on each floor: (a) vertical velocity, (b) vertical acceleration.

Fig. 8 shows the time history of uplift displacements of the BPY model. The maximum value is about 40mm. This means the maximum rigid rotational angle



of the frame is about 1/189. Fig. 9 shows the peak vertical velocity and acceleration on the each floor of the BPY model. These are observed in the right side. Fig 10 and Fig. 11 show time history of vertical forces at the column bases of the F model and the BPY model, respectively. Comparing Fig. 10 with Fig. 11, we can judge both compressive (minus) and tensile (plus) forces of the BPY model fall below those of the F model, although the peak vertical responses of the BPY model are largely amplified as shown in Fig. 9.

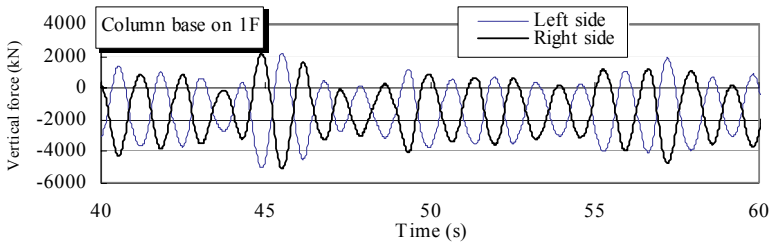


Figure 10: Time history of vertical force at base of F model.

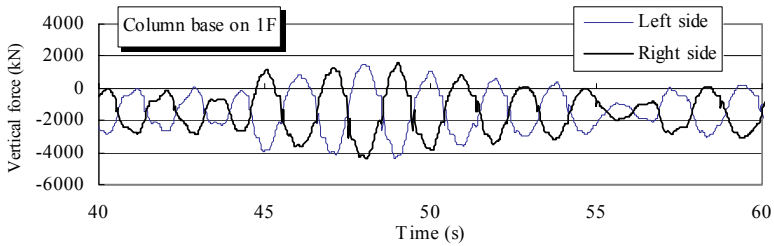


Figure 11: Time history of vertical force at base of BPY model.

3 Discussion

Impulsive vertical force N_{IMP} at the column base of the rocking structural system with one bay is calculated by the following equation [9].

$$N_{IMP} = \Delta N_{COM} + \Delta N_{TEN} \quad (1)$$

where: ΔN_{COM} : Vertical force variation in the compressive side, ΔN_{TEN} : Vertical force variation in the tensile side.

For the fixed base model, the impulsive vertical force by eq. (1) is always zero that can be easily understood by seeing Fig. 10.

The calculation results for the BPY model are shown in Fig. 12. The considerable impulsive forces are presented in this figure. By the way, the external vertical force $_{R,L}F_{Vi}$ by the vertical inertia effect in the left or right side of the each floor is calculated by the following equation.



$${}_{R,L}F_{Vi} = \sum_{i=n}^N m_n \ddot{z}_n \quad (2)$$

where, m_n : mass of node, that is 14.7t in this study (see Fig. 1), $\ddot{z}_n$: vertical response acceleration on the node in the right or left side of each floor.

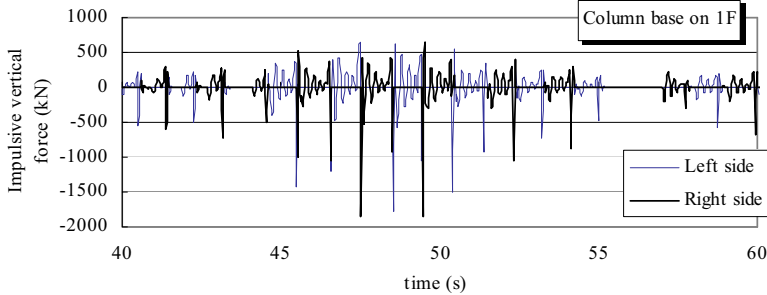


Figure 12: Time history of impulsive vertical force at base of BPY model.

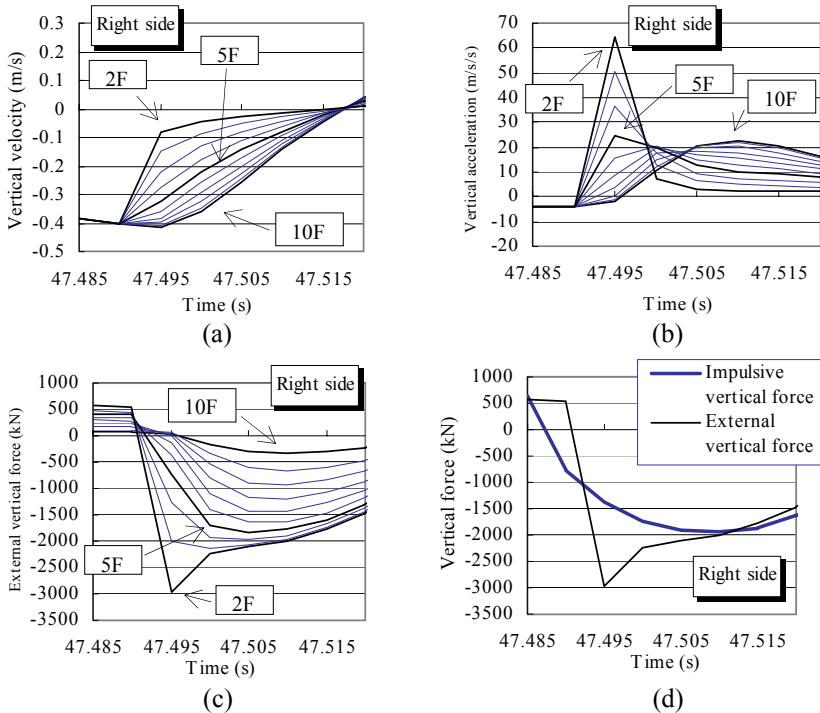


Figure 13: Time histories of vertical responses in 47.485 – 47.520s: (a) vertical response velocity, (b) vertical response acceleration, (c) external vertical force; and (d) comparison between impulsive vertical force and external vertical force at base.



The external vertical force at the base calculated by Eq. (2) using the peak vertical acceleration values shown in Fig. 9 exceed the maximum of the impulsive vertical force shown in Fig. 12 largely. This result means that the vertical response accelerations on the frame may not reach their maximum simultaneously. In Fig. 13, time histories of vertical response velocity, acceleration, external vertical force by Eq. (2) and impulsive vertical force by Eq. (1) on the right side of the frame are shown focusing the time duration, 47.485-47.52s, when the peak impulsive vertical force is observed in Fig. 12.

Table 5: Viscous damping ratios for supplemental analysis.

	T (s)	h (%)
1st	1.562	3.00
2nd	0.531	3.00
6th	0.106	12.03

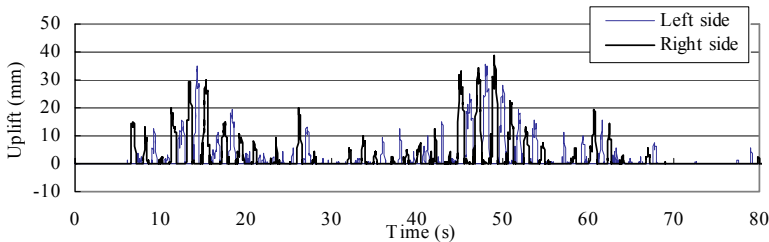


Figure 14: Time history of uplift displacement (supplemental analysis).

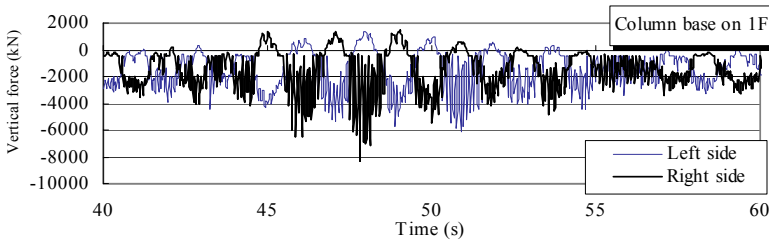


Figure 15: Time history of vertical force at base of BPY model (supplemental analysis).

Just before the column touches down, the vertical response velocity reaches its peak, that is about 0.40m/s on all floors as shown in Fig. 13(a). After that, the velocities diminish to zero. The velocity of the second floor damps most quickly. Fig. 13(b) and Fig. 13(c) show the phase difference between the responses of the lower floors and those of the higher floors. As shown in these figures, the vertical acceleration and external force are amplified largely only on the lower floors in the beginning. We can see that these amplified values do not directly



correspond with the impulsive vertical force in Fig. 13(d). It is supposed that these are moderated by the effect of viscous damping in vertical.

To investigate the effect of the viscous damping in vertical, a supplemental analysis is carried out. The viscous damping type for the analysis is changed to the Rayleigh type. The critical damping ratios for the representative modes are shown in Table 5. Time histories of uplift and vertical force at the bases are shown in Fig. 14 and Fig. 15, respectively. We can see that the damping type affects on the peak uplift little, comparing Fig. 14 with Fig. 8. In contrast, when using the Rayleigh type viscous damping for the analysis, the impulsive vertical force obviously affects the response of the vertical force at the base. It means that we need further studies on the viscous damping in vertical including the soil viscous damping under the base to evaluate more precisely the influence of the impulsive forces on responses of the rocking structures.

4 Conclusion

The effect of the impulsive vertical force on earthquake responses of a real scale model frame that permitted uplift motion was investigated. When the uplifting columns touched down to their original position, the striking impulsive vertical forces were observed. However, they hardly affect the peak vertical forces at the bases, because these forces damped very quickly under the assumption of stiffness-proportional viscous damping.

The results of the supplemental analysis in which the Rayleigh type of the viscous damping was used shown that striking impulsive vertical force arose more distinctly on the vertical force at bases. It means the further study on the viscous damping in vertical including soil viscous damping under the base is required to evaluate more precisely the influence of the impulsive forces on the response of the rocking structures.

Acknowledgement

Part of this work is supported by the Ministry of Education, Culture, Sports, Science and Technology (MEXT) of Japan under Grant-in-Aid for Scientific Research, Project No. 18560520 and 16360284.

References

- [1] Meek, J. W., Effects of foundation tipping on dynamic response. *ASCE, Vol.101, No.ST7*, pp. 1297-1311, 1975.7.
- [2] Rutenberg, A., Jennings, P. C. & Housner, G. W., The response of Veterans Hospital Building 41 in the San Fernando Earthquake. *Earthquake Engineering and Structural Dynamics*, 10(3), pp. 359-379, 1982.
- [3] Hayashi, Y., Tamura, K., Mori, M. & Takahashi, I., Simulation analyses of buildings damaged in the 1995 Kobe, Japan, Earthquake considering



- soil structure interaction. *Earthquake Engineering and Structural Dynamics*, 28(4), pp. 371-391, 1999.
- [4] Clough, R. W. & Huckelbridge, A. A. Preliminary experimental study of seismic uplift of a steel frame. *Report No.UBC/EERC-77/22. EERC, University of California, Berkeley, CA*, 1977.
 - [5] Iwashita, K., Kimura, H., Kasuga, Y. & Suzuki, N. Shaking table test of a steel frame allowing uplift. *Journal of Structural and Construction Engineering*. AIJ 561, pp. 47-54, 2002. (In Japanese)
 - [6] Kasai, K., Kanda, M. & Okuma, K. Real example for a passively controlled building with stepping column: analysis and full-scale damper experiment. *Proceeding of Passive Control Symposium 2001*. Structural Engineering Research Center, Tokyo Institute of Technology, pp. 235-249, 2001. (In Japanese)
 - [7] Midorikawa, M., Azuhata, T., Ishihara, T., Matsuba, Y., Matsushima, Y. & Wada, A., Earthquake response reduction of buildings by rocking structural systems., *Proc. SPIE, Smart Structures and Materials 2002*, pp. 265-272, 2002.
 - [8] Midorikawa, M., Azuhata, T., Ishihara, T. & Wada, A., Shaking table tests on rocking structural systems installed yielding base plates in steel frames. *Proc. of STESSA2003*, pp. 449-454, 2003.6.
 - [9] Midorikawa, M., Azuhata, T., Ishihara & Wada, A. Dynamic behavior of steel frames with yielding base plates in uplift motion for seismic response reduction, *Journal of Structural and Construction Engineering*. AIJ 572, pp. 97-104, 2003. (In Japanese)
 - [10] Midorikawa, M., Azuhata, T., Ishihara, T. & Wada, A., Shaking Table Testson Seismic Response of Steel Braced Frames with Column Uplift. *Earthquake Engineering and Structural Dynamics*, 35(14), pp. 1767-1785, 2006.
 - [11] Ishihara, T., Midorikawa, M., Azuhata, T. & Wada, A., Hysteresis characteristics of column base for rocking structural systems with base plate yielding, *Journal of Construction Steel*, vol. 11, pp. 51-56, 2003.11. (In Japanese)
 - [12] Ishihara, T., Midorikawa, M. & Azuhata, T., Hysteresis Characteristics of Large-scale column Base for Rocking Structural Systems, *Journal of Constructional Steel*, 14, pp. 381-384, 2006. (In Japanese)



Author Index

Aguilar R.	269	Javadein S. I.	279
Ariga Y.	289	Johnson N.	85
Astorga M.-A.	269		
Attard T.	431	Kacianauskas R.	247
Avila J. A.	13	Kangarloo K.	151
Azhari M.	63	Kapetana P.	409
Azuhata T.	165, 175, 459	Kawakami H.	421
		Ke S. S.	141
Baquadano P.	117	Khamwan P.	23
Bausys R.	247	Kilar V.	73
Belmouden Y.	195	Komodromos P.	129
Blondet M.	269	Kourtides V.	375
		Kuwata Y.	319
Chang H. Y.	421		
Cuéllar V.	331	Lampropoulos A. P.	399
		Lestuzzi P.	195
Dansby R.	431	Li P.	299
Del Fabbro M.	225	Li W. S.	141
Díaz O.	3	Lissel S. L.	343
Dritosos S. E.	365, 399, 409		
Dundulis G.	247	Makovička D. Jr.	353
		Makovička D.	353
Erkmen B.	185	Manos G. C.	375
Estaire J.	331	Marino A.	215
		Markauskas D.	247
Fernández-Dávila V. I.	117	Marović P.	43
Fuchida K.	309	Martelli A.	105
		Maugeri M.	XIX
García-Pérez J.	3	Mazloom M.	259
Gran F.	117	Memarzadeh P.	63
Grasso S.	XIX	Meriggi R.	225
Grebenar G.	95	Mestrovic D.	95
Grillo A.	215	Midorikawa M.	165, 175, 459
Gupta M. K.	235	Mittal S.	235
Guzman M. J.	343	Mohamed O. A.	23
		Mokos V. G.	443
Haddad R.	33	Morita K.	175
Herak Marović V.	43	Muñoz A.	269
Hosseini M.	151		
Ishihara T.	165, 175, 459	Nichols J. M.	207
		Nikolić Ž.	43

Noguchi K..... 175

Numayr K. 33

Papaloizou L. 129

Phocas M. C..... 129

Polycarpou P..... 129

Rimkevicius S..... 247

Rinaldini A. 215

Saadatpour M. M. 63

Saar O. S. 453

Saiidi M. 85

Sanders D..... 85

Sapountzakis E. J..... 443

Schultz A. E..... 185

Shih B. J..... 141

Slak T..... 73

Sliaupa S..... 247

Stupak E..... 247

Taghinezhad R..... 279

Takada S. 319

Tao X.-X..... 299

Tao Z.-R 299

Tingatinga E. A..... 421

Trueb M. 195

Tsioulou O. T..... 399

Vosoughifar H. R..... 387

Yuksel S. B..... 53

Zenteno M. 3



WITPRESS ...for scientists by scientists

Seismic Prevention of Damage:

A Case Study in a Mediterranean City

Edited by: M. MAUGERI, Università di Catania, Italy

The recent destructive earthquakes of Kocaeli and Duzce in Turkey exemplify the high seismic risk of the Mediterranean area, close to the contact between various tectonic plates and therefore frequently shaken by earthquakes of medium or high intensity.

The scientific, social and political awareness of the seismic risk prompted the Italian Department of Civil Protection to fund a Research Project on earthquake damage scenarios for a high-risk area in the Mediterranean. A subsequent Research Project, focussed on detailed scenarios and actions for seismic prevention of damage, was financed by the "Istituto Nazionale di Geofisica e Vulcanologia" (INGV).

A multidisciplinary team researched the analysis and mitigation of seismic risk in Catania, a mid-sized Mediterranean city subjected to medium-high earthquake risk. This book summarises the results of the two-year project, which involved geology, geophysics, geotechnical, structural and transportation engineering and urban planning.

The book includes material that will be valuable to many specialists including environmental, civil, earthquake and civil defence engineers, geologists, geophysicists and town planners, and will be a useful guideline for seismic hazard assessment and seismic risk evaluation, prevention and mitigation.

Series: Advances in Earthquake Engineering, Vol 14

ISBN: 1-84564-004-7 2005 424pp
£139.00/US\$250.00/€208.50

Earthquake Resistant Engineering Structures V

*Edited by: C.A. BREBBIA, Wessex Institute of Technology, UK,
D.E. BESKOS, University of Patras, Greece, G.D. MANOLIS, Aristotle University of Thessaloniki, Greece and
C.C. SPYRAKOS, National Technical University of Athens, Greece*

Earthquake mitigation remains an engineering topic of paramount scientific importance, as demonstrated by the tragic events following the Sumatra earthquake and tsunami of December 24th, 2004 that reverberated across the globe and generated world-wide sympathy for the victims and their plight.

Given the inestimable loss of life and huge financial cost associated with major earthquakes in densely populated areas, it becomes clear that earthquake mitigation is an important challenge for the global engineering community and a major social priority for national governments in seismically-prone countries.

This book contains over 70 papers from the fifth in the popular international conference series on the subject. A wide list of topics include: Geophysical and Geotechnical Aspects; Seismic Behavior and Vulnerability; Structural Dynamics; Seismic Isolation and Control; Monitoring and Testing; Risk Reduction and Earthquake-resistant Design.

WIT Transactions on The Built Environment, Vol 81

ISBN: 1-84564-018-7 2005 840pp
£294.00/US\$470.00/€441.00

All prices correct at time of going to press but subject to change.

WIT Press books are available through your bookseller or direct from the publisher.



WITPRESS ...for scientists by scientists

Earthquake Geodynamics

Seismic Case Studies

Edited by: E.L. LEKKAS, University of Athens, Greece

Containing contributions from renowned geoscientists with first-hand experience, this book discusses various aspects of earthquake geodynamics in seismically active regions. The chapters include lessons learnt from earthquakes that have struck urban areas during the past two decades and cover a wide variety of topics including the analysis of seismic sequences, the study of surficial deformation and geophysical techniques for microzonation purposes.

Partial Contents: The 1999 Earthquake in Izmit - The Study of Actualistic Strike-Slip Tectonic Forms; Contribution of GIS to Analysis of the 7 September 1999 Athens Earthquake; The 26 July 2001 Skyros (North Aegean Sea, Greece) Earthquake; Low-Strain Techniques Used for Microzoning Studies in Soft Rock Areas in Greece; Parameters of Intensity Distribution in the Izmit and Düzce (Turkey) Earthquakes.

Series: Advances in Earthquake Engineering, Vol 12

ISBN: 1-85312-996-8 2004 200pp
£80.00/US\$128.00/€120.00

Find us at
<http://www.witpress.com>

We are now able to supply you with details of new WIT Press titles via

E-Mail. To subscribe to this free service, or for information on any of our titles, please contact the Marketing Department, WIT Press, Ashurst Lodge, Ashurst, Southampton, SO40 7AA, UK

Tel: +44 (0) 238 029 3223

Fax: +44 (0) 238 029 2853

E-mail: marketing@witpress.com

Earthquake Resistant Engineering Structures IV

Edited by: G. LATINI, Università Politecnica delle Marche, Italy and C.A. BREBBIA, Wessex Institute of Technology, UK

Of interest to all specialists working on earthquake resistant engineering structures, this book consists of papers from the fourth international conference on this subject.

WIT Transactions on The Built Environment, Vol 72

ISBN: 1-85312-984-4 2003 400pp
£132.00/US\$211.00/€198.00

Innovative Approaches to Earthquake Engineering

Edited by: G. OLIVETO, Università degli Studi di Catania, Italy

"...takes you right to the front lines of this ever-evolving field."

CIVIL ENGINEERING

Embracing a wide spectrum of problems and providing an extensive view of modern seismic engineering, this volume features invited chapters from leading experts.

Partial Contents: Design of Damage Controlled Structures; Vulnerability of Historical and Monumental Buildings, Significant Ground Motion Parameters and Evaluation of Seismic Performance.

Series: Advances in Earthquake Engineering, Vol 10

ISBN: 1-85312-885-6 2002 336pp
£108.00/US\$167.00/€162.00



WITPRESS ...for scientists by scientists

Stochastic Structural Dynamics in Earthquake Engineering

G.D. MANOLIS, Aristotle University, Thessaloniki, Greece and
P.K. KOLIOPOULOS, Technological Education Institute, Serres, Greece

"...a fine textbook."

APPLIED MECHANICS REVIEWS

Designed specifically for earthquake practitioners, whilst also accessible to graduate students and researchers, this text contains many examples and exercises with solutions.

Series: Advances in Earthquake Engineering, Vol 8

ISBN: 1-85312-851-1 2001

296pp+CD-ROM

£145.00/US\$225.00/€217.50

Dynamic Structural Design

Inverse Problem Approach

I. TAKEWAKI, Kyoto University, Japan

"...a timely contribution to the subject...it gives insight to how the methodology can be applied to the design of structures in the context of performance-based design. The book is a valuable resource for researchers and engineers interested in learning and applying the inverse method in seismic resistant design of structures."

JOURNAL OF STRUCTURAL ENGINEERING

Series: Advances in Earthquake Engineering, Vol 6

ISBN: 1-85312-745-0 2000 292pp

£98.00/US\$149.00/€147.00

Wave Motion in Earthquake Engineering

Edited by: E. KAUSEL, Massachusetts Institute of Technology, USA and **G.D. MANOLIS**, Aristotle University, Thessaloniki, Greece

Features invited contributions from researchers whose work has recently been the focus of attention in journals and at conferences.

Partial Contents: Wave Propagation in Complex Geological Structures and their Effects on Strong Ground Motion; Spatial Variability of Seismic Motions Recorded over Extended Ground Surface Areas; The Thin-Layer Method in Seismology and Earthquake Engineering.

Series: Advances in Earthquake Engineering, Vol 5

ISBN: 1-85312-744-2 2000 376pp

£145.00/US\$225.00/€217.50

Seismic Isolation for Earthquake-Resistant Structures

P. KOMODROMOS, Massachusetts Institute of Technology, USA

"...a useful reference in practical design of base-isolated structures."

APPLIED MECHANICS REVIEWS

A thorough exploration which can be used to minimize earthquake induced loads, and to mitigate/reduce resulting damage in low-to medium-rise buildings.

Series: Advances in Earthquake Engineering, Vol 7

ISBN: 1-85312-803-1 2000 224pp

£79.00/US\$122.00/€118.50



WITPRESS ...for scientists by scientists

Scattering of Seismic Waves

Applications to the Mexico City Valley

E. REINOSO, Universidad Nacional Autonoma de Mexico, Mexico

"...a valuable guide."

ZENTRALBLATT FÜR MATHEMATIK

This book deals with the application of boundary element technology to the study of scattering of seismic waves from irregular topographies and buried valleys. The author presents numerical refinements that allow the practical computation of two- and three-dimensional problems and proves that these are accurate and efficient. The refinements are then applied to reproduce amplifications observed in the Mexico City Valley.

Series: Advances in Earthquake Engineering, Vol 11

ISBN: 1-85312-833-3 2002 220pp
£79.00/US\$122.00/€118.50

All prices correct at time of going to press but subject to change.

WIT Press books are available through your bookseller or direct from the publisher.

WIT eLibrary

Home of the Transactions of the Wessex Institute, the WIT electronic-library provides the international scientific community with immediate and permanent access to individual papers presented at WIT conferences. Visitors to the WIT eLibrary can freely browse and search abstracts of all papers in the collection before progressing to download their full text.

Visit the WIT eLibrary at
<http://library.witpress.com>

Computer Analysis and Design of Earthquake Resistant Structures: A Handbook

Edited by: D.E. BESKOS and S.A. ANAGNOSTOPOULOS, University of Patras, Greece

"...a recommended text for academic institutions or organizations that are doing extensive research in earthquake engineering or designing structures in earthquake-prone regions."

E-STREAMS

Series: Advances in Earthquake Engineering, Vol 3

ISBN: 1-85312-374-9 1997 936pp
£195.00/US\$295.00/€292.50

The Kobe Earthquake

Geodynamical Aspects

Edited by: C.A. BREBBIA, Wessex Institute of Technology, UK

Contains scientific reports and analyses of the Kobe earthquake from Japanese researchers.

Series: Advances in Earthquake Engineering, Vol 1

ISBN: 1-85312-430-3; 1-56252-345-7
(US, Canada, Mexico) 1996 160pp
£85.00/US\$138.00/€127.50

WITPress

Ashurst Lodge, Ashurst, Southampton, SO40 7AA, UK.

Tel: 44 (0) 238 029 3223

Fax: 44 (0) 238 029 2853

E-Mail: witpress@witpress.com

