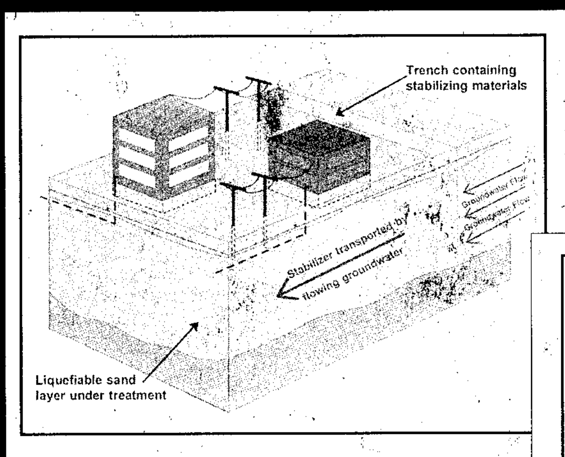
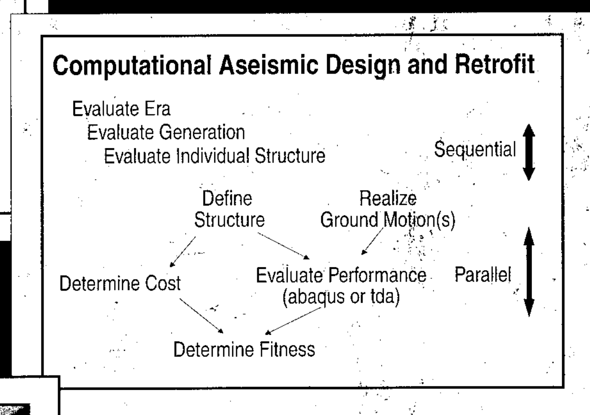
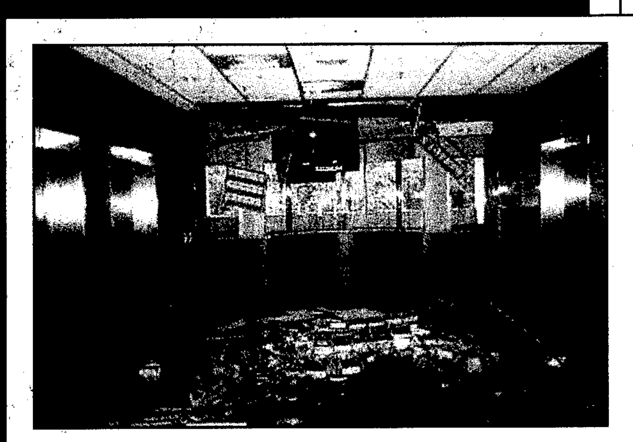
Proceedings of the Second MCEER Workshop on Mitigation of Earthquake Disaster by Advanced Technologies (MEDAT-2)







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Technical Report MCEER-01-0002
July 23, 2001

This workshop was held November 30 – December 1, 2000 at the Holiday Inn Emerald Springs Hotel, Las Vegas, Nevada, and was supported primarily by the Earthquake Engineering Research Centers Program of the National Science Foundation under Award Number EEC-9701471.

MCEER
MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
A National Center of Excellence in Advanced Technology Applications

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NOTICE

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Passive treatment for mitigation of liquefaction risk, taken from "Passive Site Remediation for Mitigation of Liquefaction Risk," by Patricia M. Gallagher and James K. Mitchell, pages 29-36.

Overall framework for computational aseismic design and retrofit taken from "Computational Aseismic Design and Retrofit with Application to Passively Damped Structures," by Gary F. Dargush and Ramesh Sant, pages 175-186.

Interior nonstructural damage to Shiu-Tuan Hospital following the 1999 Chi-Chi, Taiwan earthquake; photograph courtesy of G. Yao, National Cheng Kung University and the National Center for Research on Earthquake Engineering, Taiwan.

MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

A National Center of Excellence in Advanced Technology Applications

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Edited by
Michel Bruneau[1] and Daniel Inman[2]

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This workshop was held November 30 - December 1, 2000 at the Holiday Inn Emerald Springs Hotel, Las Vegas, Nevada, and was supported primarily by the Earthquake Engineering Research Centers Program of the National Science Foundation.

16. Abstract (limit 200 Words)

This workshop on Mitigation of Earthquake Disaster by Advanced Technologies was held in Las Vegas, 11/30-12/01, 2000. The workshop consisted of all plenary two-day sessions and was the second in a new series of MCEER workshops on advanced technologies. The workshop gathered 35 experts to explore the state-of-the-art and practice in advanced technologies for the seismic evaluation/retrofit of health care facilities with a particular emphasis on material and technologies for mitigating the risk of soil liquefaction , structural damage and non-structural damage. Separate technical sessions each offered 1) An overview presentation of the desirable seismic performance objectives of hospitals and retrofit strategies currently under investigation by MCEER; 2) Presentations illustrating examples of research conducted within that framework; 3) Presentations highlighting technologies that are or could meet the criteria stated in (1) and how they are used in non-earthquake engineering applications; 4) A broader overview of state-of-the-art in various technologies applicable to the earthquake engineering problem at hand; 5) Panel sessions for the researchers assigned to one of the three specified topics, where participants discussed how new technologies could be implemented; and 6) A summary session to report findings and recommendations reached during the workshop for each technical block.

17. Document Analysis a. Descriptors

Earthquake Engineering. Health care facilities. Hospitals. Retrofitting. Nonstructural damage. Structural damage. Liquefaction. Damping. Semi-active control systems. Structural control. Piping systems. Critical systems.

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Workshop Description

The workshop on *Mitigation of Earthquake Disaster by Advanced Technologies* (MEDAT), was held under sponsorship of the Multidisciplinary Center for Earthquake engineering Research (MCEER) and the National Science Foundation (NSF), in Las Vegas on November 30th and December 1st, 2000. The workshop (MEDAT-2) consisted of all plenary two-day sessions and was the second in a new series of MCEER-sponsored workshops involving advanced technologies.

The workshop gathered a small, select multidisciplinary group of approximately 35 experts (earthquake engineers and researchers from other fields of advanced technology) to exchange and explore how innovative applications of advanced technologies (non-destructive inspection, health monitoring advanced materials, innovative devices, etc.) could be used for earthquake disaster mitigation. More specifically, it explored the state-of-the-art and state-of-practice in the use of advanced technologies for the seismic evaluation and retrofit of health care facilities with a particular emphasis on material and technologies that would be useful to mitigate the risk of:

- Soil liquefaction
- Structural damage
- Non-structural damage

To address these topics, separate technical blocks were scheduled and structured as follows:

Focus	Technologies currently researched by MCEER	Other Advanced Technologies
Overview	(1)	(4)
Examples	(2)	(3)
Panel Discussion	(5)	
Summary Session and Recommendations	(6)	

For each technical block, the numeral above corresponded to (in order of presentation):

1. An overview presentation of the desirable seismic performance objectives of hospitals and the retrofit strategies currently under investigation by MCEER.

2. Several presentations that illustrated examples of research projects conducted within that framework.
3. Several presentations that illustrated examples of technologies that are or could meet the criteria stated in (1) and how they are used in other (i.e. non-earthquake engineering) applications.
4. A broader overview of state-of-the-art in various technologies that could also be applied to the earthquake engineering problem at hand, with or without further developments.
5. Panel sessions for the researchers assigned to one of the three specified topics, where participants discussed how new technologies could be implemented, and (if possible) elaborated research strategies for this implementation.
6. A summary session to report findings and recommendations reached during the workshop for each technical block. These brief reports are included in Appendix A of the proceedings.

Presentations made at the workshop are available on our web site at *http://mceer.buffalo.edu/publications/sp_pubs/medat2/default.asp*. They are in either PowerPoint or PDF format, and complement the 30 papers in this volume.

Contents

Overview of MCEER's Vision, Mission and Strategic Plan 1
Michel Bruneau

Technical Block 1: General Overview of Geotechnical Issues 9
Chairs: Ricardo Dobry and James Mitchell

Centrifuge-Based Evaluation of Pile Foundation Response to Lateral Spreading and Mitigation Strategies 11
Ricardo Dobry, Tarek Abdoun and Thomas D. O'Rourke

Liquefaction Remediation in Silty Soils 23
S. Thevanayagam and W. Jia

Passive Site Remediation for Mitigation of Liquefaction Risk 29
Patricia M. Gallagher and James K. Mitchell

Mitigation of Liquefaction Hazards 37
Juan I. Baez

Performance of Soil Reinforcement Systems in Earthquake Zones 47
Sherif Hanna and Ilan Juran

Foundation Liquefaction Retrofit Work - Abstracts 63
Ignatius Po Lam

Technical Block 2: Advanced Technologies for Structural Retrofit 65

General Overview of Earthquake Engineering Issues for MCEER Hospital Project 67
Michel Bruneau

Technical Block 2A: Materials and Damage Monitoring 75
Chairs: Michel Bruneau and Jayanth Kudva

Engineered Cementitious Composites for the Retrofit of Critical Facilities 77
Keith Kesner and Sarah L. Billington

Polymer Matrix Composite (PMC) Infill Walls for Seismic Retrofit 87
Amjad J. Aref and W.Y. Jung

Contents

Pattern Recognition for Structural Health Monitoring — 99
Charles R. Farrar and Hoon Sohn

The Development of a Wireless Modular Health Monitoring System for Civil Structures — 105
Jerome P. Lynch, Kincho H. Law, Erik G. Straser, Anne S. Kiremidjian and Tom W. Kenny

Damping in Structural Applications — 109
Darel E. Hodgson and Robert C. Krumme

Seismic Control Devices Using Low-Yield-Point Steel — 115
Yasushi Maeda, Tanemi Yamaguchi, Toru Takeuchi and Takumi Ikebe

Use of FRP Composite Materials in the Renewal of Civil Infrastructure in Seismic Regions — 123
Vistasp M. Karbhari

Structural Composites with ECC — 137
Gregor Fischer and Victor C. Li

Technical Block 2B: Damping and Semi-Active Systems — 145
Chairs: T.T. Soong and A.S. Whittaker

Damping and Semi-Active Systems: A State-of-the-Art Report — 147
T.T. Soong

Passive Seismic Control of Building Structures — 157
Andrew Whittaker

Enabling Technology Testbed Project: Design of a Semi-Active Seismic Protective System for a LA Building — 165
G.C. Lee, M. Tong, Yihui Wu and Shubin Ruan

Computational Aseismic Design and Retrofit with Application to Passively Damped Structures — 175
Gary F. Dargush and Ramesh Sant

Contents

The Seismic Safety Program for Hospital Buildings in California - Part 1: Seismic Performance Requirements for New Hospital Buildings and Part 2: The Seismic Retrofit Program for Existing California Hospitals 187
C.V. Tokas and K. Schaefer

Recent Trends of Damage Controlled Structures in Japan 203
Akira Wada, Yihua Huang and Mamoru Iwata

Critical Facilities in New York State: General Comments 213
Thomas M. Jung

Technical Block 3: Advanced Technologies for Nonstructural Retrofit 217
Chairs: Mircea Grigoriu and Daniel Inman

General Overview of Earthquake Engineering Issues for Non-structural Systems 219
Micea Grigoriu, Christopher Roth and Ehab Mostafa

Static Axial Behavior of Some Typical Restrained and Unrestrained Pipe Joints 229
M. Maragakis, R. Siddharthan and Ronald Meis

Seismic Vulnerability and Retrofit of Nonstructural Components 237
T.T. Soong

Seismic Protection of Some Nonstructural Components in Hospitals 245
M.P. Singh and Rildova

Part I – Seismic Retrofit of Critical Piping Systems (Above Ground Piping) and Part II – Seismic Design of Critical Piping Systems (Above Ground Piping) 253
George Antaki

Recycled Plastics: Characteristics and Seismic Application 267
M. Ala Saadeghvaziri and K. MacBain

Smart Materials Technologies for Bolted-joints in Civil Systems 277
Gyuhae Park, Daniel E. Muntges and Daniel J. Inman

Contents

Appendix A: Workshop Discussions 285

Technical Block 1: Geotechnical Issues, Summary and Discussion 287
James K. Mitchell and Ricardo Dobry

Technical Block 2A: Materials and Damage Monitoring 289
Michel Bruneau

Technical Block 2B: Passive and Active Control Systems in Health Care Facilities 291
T.T. Soong and Andrew S. Whittaker

Technical Block 3: Nonstructural Retrofit 293
Daniel J. Inman

Appendix B: List of Workshop Presentations 297

Appendix C: Workshop Information 301

Agenda 303

Participants 307

Overview of MCEER's Vision, Mission and Strategic Plan

by Michel Bruneau
Deputy Director, Multidisciplinary Center for Earthquake Engineering Research and Professor, Dept. of Civil, Structural, and Environmental Engineering, University at Buffalo

Abstract

The vision of the Multidisciplinary Center for Earthquake Engineering Research (MCEER) is to help establish earthquake resilient communities. Its mission is to discover, nurture, develop, promote, help implement, and in some instances pilot test, innovative measures and advanced and emerging technologies to reduce losses in future earthquakes in a cost-effective manner. This paper presents an abridged version of MCEER's Strategic Plan that was developed to fulfill this vision and mission. The full version of this Plan is available on MCEER's web site, at: http://mceer.buffalo.edu/aboutMCEER/strategicPlan/default.asp .

MCEER is an NSF-funded Research Center that integrates fundamental research, enabling technology research, facilitating technologies for implementation, and test beds. MCEER also works with the entire earthquake loss reduction community, which consists of practicing engineers and other design professionals, policy makers, regulators and code officials, facility and building owners, governmental entities, and other stakeholders who have responsibility for loss reduction decision making, to ensure that research results are implemented to improve safety and advance earthquake loss reduction for government, private industry, and the public-at-large.

Vision

Economic losses from urban earthquakes in the last decade have risen dramatically. Recent damaging U.S. quakes, such as the 1989 Loma Prieta and the 1994 Northridge events, recorded losses in the tens of billions of dollars. Japan's 1995, Kobe earthquake, magnitude 6.9, resulted in losses exceeding $120 billion. A similar moderate-size earthquake striking a major U.S. metropolitan area, or a repeat of historic large-scale events that hit San Francisco (1906) and Memphis (1811, 1812) could cause extensive loss of life and injury, as well as widespread damage, with direct economic losses (damage to the built environment, building contents, inventory and ensuing business disruption) and indirect losses (supply shortages and other ripple effects to economic sectors not sustaining direct damage) comparable to those of Kobe. An event of that magnitude would result in tremendous human losses and economic hardship to affected communities. Earthquake losses will rise at an escalating rate in future years unless major loss reduction programs are undertaken.

Experience shows, however, that relatively new buildings and infrastructure, designed and constructed according to the current state-of-practice in earthquake engineering, perform significantly better than older ones. Indeed, the largest threat to society lies in the seismically vulnerable infrastructure designed and constructed at a time when earthquake-resistant design had not yet matured. Considering the enormous inventory of such structures nationwide, it is neither financially nor politically possible to upgrade all existing structures up to the level of performance considered acceptable by today's standards. In fact, only a small percentage of all buildings nationwide will have been retrofitted when the next large earthquake strikes, and the affected population will likely suffer a large number of injuries due to collapse or damage of buildings and residential units.

In MCEER's judgment, the best way to achieve the stated vision of earthquake resilient communities in the short term is to invest in two focused system-integrated endeavors: the rehabilitation of critical infrastructure facilities that society will need and expect to be operational following an earthquake, more specifically hospitals and lifelines; and the improvement of emergency response and crisis management capabilities to ensure efficient response and prompt recovery following earthquakes (see figure 1).

Health facilities and lifelines constitute critical infrastructural elements that must remain operational following a major earthquake. While many other types of buildings house various types of emergency services (e.g. fire stations), hospitals contain complex and tightly integrated structural and non-structural systems that pose unique seismic retrofit challenges. Because hospitals are so complex, many of the techniques developed to retrofit hospitals can be transferred to achieve similar seismic performance in other types of less complicated critical facilities. Furthermore, not only would hospital structural retrofit strategies be adaptable to other structures, but many of the approaches taken to retrofit hospital water reservoir, internal distribution pipe networks, and other equipment, can be adapted to reduce the seismic vulnerability of industrial facilities and protect against toxic spills during earthquakes.

MCEER works toward its vision helping to establish earthquake resilient communities through an interdisciplinary and coordinated program that emphasizes fundamental and applied research combined with sustained and systematic education, outreach, and implementation efforts.

Mission

The mission of the Center is to discover, nurture, develop, promote, help implement, and in some instances pilot test, innovative measures and advanced and emerging technologies to reduce losses in future earthquakes in a cost-effective manner. Advanced technology research at MCEER includes innovative applications of:

- *engineered systems and materials,*
- *scientific methodologies, and,*
- *concepts and analytical approaches*

that have not traditionally been used in earthquake engineering and loss reduction problems, with an emphasis on applications that require a multidisciplinary effort.

The mission of the Center is predicated on the premise that the future of earthquake engineering and loss reduction lies in advanced and emerging technologies and associated innovative measures. More specifically, because the aforementioned critical facilities and emergency response activities constitute the most essential (and most severely strained) support systems

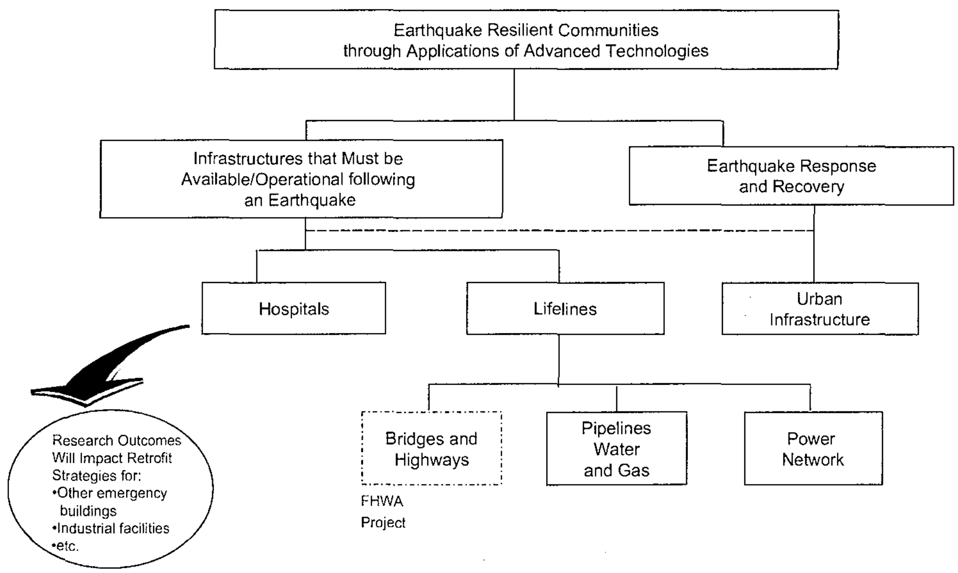

Figure 1: Schematic of MCEER's Strategic Plan

following an earthquake, MCEER believes that new and emerging technologies must be used to ensure that these systems will be able to function at their expected performance level in future earthquakes, and has made it its research mission to investigate how various advanced technologies can be implemented to achieve this objective. Accordingly, MCEER's research program focuses on investigating how these technologies can be adapted and implemented to reduce earthquake hazards while providing a higher level of performance than is possible with conventional techniques, as well as on demonstrating how these new technologies can enhance emergency preparedness and recovery.

The advanced and emerging technologies considered by the Center include, but are not limited to:

- site remediation technologies,
- structural control and simulation,
- high performance materials,
- condition assessment technologies, including technologies for estimating both potential and actual earthquake losses, and
- decision support systems.

Many different advanced technologies within each of these broader categories provide the tools necessary to overcome barriers and achieve the broader objectives outlined above. More specifically, for hospitals, this includes the development of retrofit strategies to ensure post-earthquake hospital serviceability, retrofit design for the equipment and main structural systems using new materials, technologies and concepts, as well as geotechnical research to ensure survival in spite of soil liquefaction. It also requires a close interaction between social scientists, economists and engineers to identify technically sound seismic rehabilitation strategies that can be justified based on cost-benefit analyses and that also ensure a post-earthquake level of service that meets public expectations.

Within this structure, advanced technologies become the tools that allow the Center to achieve its vision. Investigators assigned to individual research projects investigate how specific advanced technologies can be developed and implemented to solve the specific problems relevant to each of the first three program areas. Figure 1 illustrates how the three research programs emerge from the common vision.

MCEER Systems Approach

Users of MCEER research consist of the various decision-makers, knowledge providers, and groups and individuals who adopt loss reduction measures and undertake actions to increase the earthquake resistance of critical facilities while containing direct and indirect earthquake losses. These societal actors include (but are not limited to) practicing engineers and other design professionals, policy makers, regulators and code officials, facility and building owners, governmental entities, and other stakeholders who have responsibility for loss reduction decision making (Figure 2) These constitute the "loss reduction market" (LRM), which has a number of significant characteristics. However, due to space constraints, these cannot be summarized here, but are available at the web address provided in the abstract of this paper.

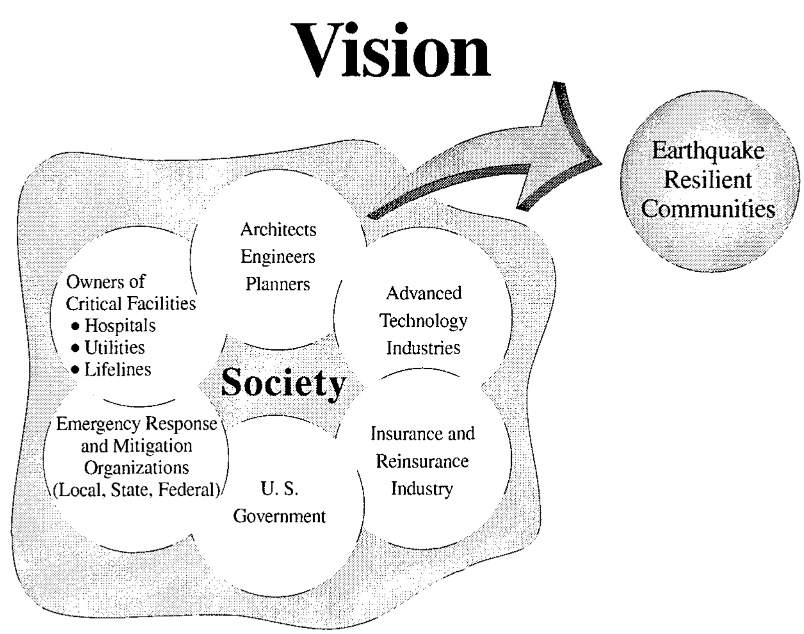

Figure 2: Impact on society from direct users of MCEER research.

MCEER conceptualizes the research process in open-system terms. That is, the Center conducts research with an understanding of the characteristics, needs, and requirements of the broader society. The Center does not see itself as creating knowledge for its own sake, but rather seeks to see that knowledge applied in the wider societal environment. As noted earlier, that environment consists of various stakeholder groups that have multiple and often conflicting values and interests. There are a wide range of possible approaches to managing the earthquake threat, ranging from investment in pre-event mitigation through reliance on post-event response and the provision of post-disaster aid as strategies containing losses. Stakeholders in different organizations, communities, and regions of the country vary in the emphasis they place on these alternative solutions. Key actors also differ in the extent to which they are willing to tolerate the uncertainties associated with research findings and recommended solutions. Equally important, they are likely to differ considerably in their expectations concerning acceptable levels of seismic performance for elements in the built environment and in the levels of risk and vulnerability they consider acceptable. Overall, however, the market is very sensitive to the costs associated with different loss-reduction approaches, which creates pressure for demonstrating that potential solutions to earthquake-related problems are cost-effective.

MCEER's research also begins with a recognition that efforts to achieve higher levels of earthquake resistance are constrained by numerous barriers, including the sheer complexity of the earthquake problem and the difficulties inherent in developing reliable research findings and credible policy recommendations; the low priority assigned to earthquake loss reduction in many areas of the country; financial barriers associated with adopting and implementing loss reduction solutions, and relatedly, the difficulties inherent in demonstrating the cost-effectiveness of loss reduction measures; lack of clarity with respect to legal and regulatory authorities; and various other knowledge, political, perceptual, and economic barriers. MCEER therefore recognizes that if loss reduction efforts are to be successful, a wide array of alternative products, technologies and strategies will be needed, to allow market participants the latitude and flexibility to select among an array of different loss-reduction options.

The manner in which MCEER seeks to provide the new knowledge that can help overcome these barriers is illustrated in Figures 3 and 4, the former providing more details on the nature of the loss reduction market and the long-term implementation path, the latter focusing more on the activities within MCEER's control, and further details of the operating context.

As shown in the figures, the technologies, tools, and strategies MCEER develops must be appropriate for the societal environment in several respects. First, they must take into account that communities and regions differ in levels of hazardousness and vulnerability, as well as in levels of awareness and receptiveness to loss-reduction measures. Second, they must be responsive to the complexity of the loss reduction market, in terms of needs, economic and political interests, priorities, conceptions of acceptable risk, and familiarity with the earthquake problem. Third, they must be geared toward taking advantage of both research and implementation opportunities presented by earthquake events, since earthquakes and other disasters often serve as catalysts for change, particularly when they stimulate champions or policy entrepreneurs to place loss reduction on the policy agenda. Other changes in the societal environment, such as the passage of new laws or the adoption of new codes and standards, can also encourage stakeholders to adopt and implement loss reduction measures.

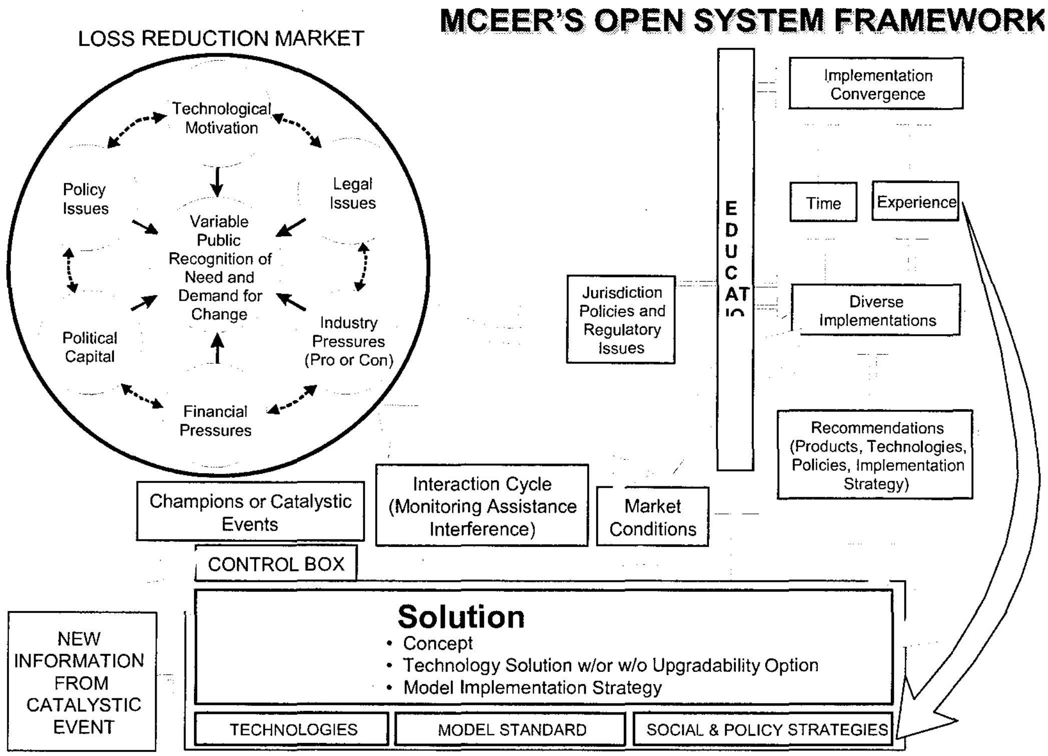

Figure 3: MCEER's Open System Framework

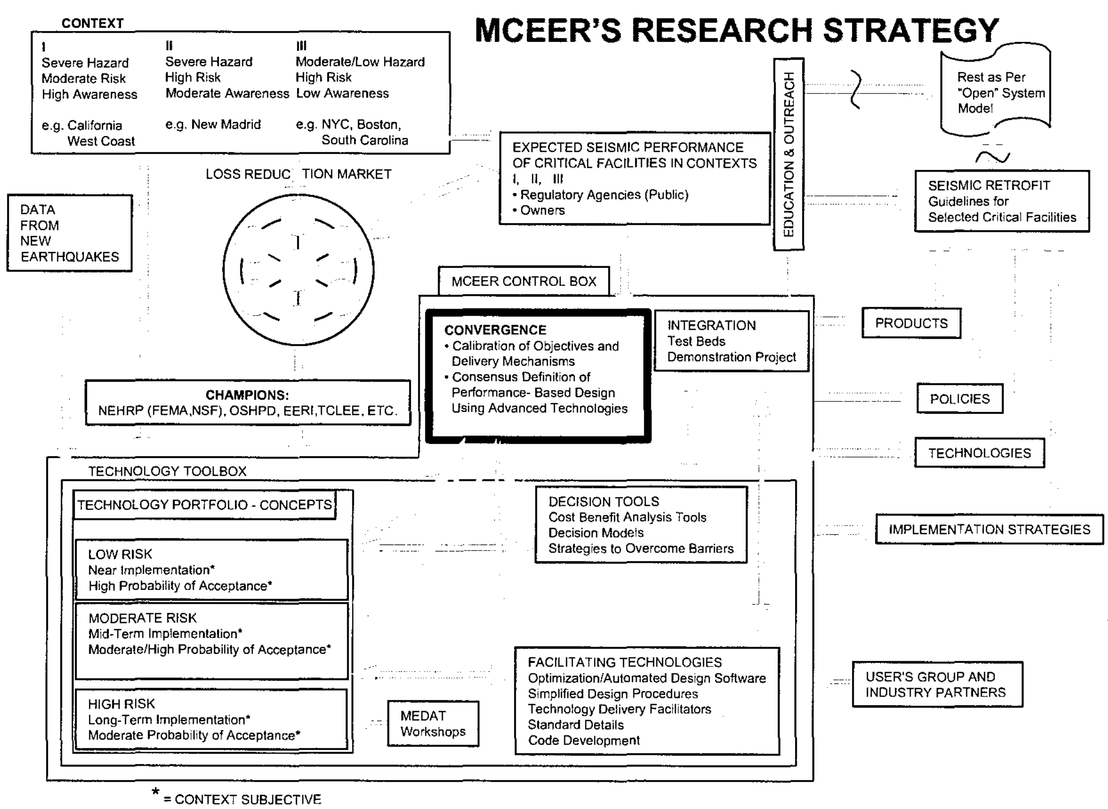

Figure 4: MCEER's Research Strategy

As illustrated in Figure 4, to best address the needs of of the larger system, MCEER has developed an integrated research program that consists of the following elements:

1. A "technology portfolio" consisting of a range of candidate approaches. MCEER uses the term "portfolio" to suggest that some of the technologies being investigated are riskier than others from the point of view of effectiveness and implementation potential. The Center acknowledges that there is some probability that some of the technologies investigated by MCEER will prove not to be cost-effective, or that they will encounter too much societal resistance to be implemented in the near term. This "portfolio' is, in other words, diversified in terms of both potential risks (e.g., failure to demonstrate proof-of-concept or cost effectiveness, failure to demonstrate implementation potential) and potential payoffs. The logic behind the development of the technology portfolio is that, among all the technologies that are investigated, many will prove cost-effective and will be adopted and implemented.
2. As a group, the technologies in the portfolio will provide the necessary diversity to tackle complex loss reduction challenges, together with the flexibility to rapidly adjust research directions when earthquake events or other changes in the societal environment alter stakeholder receptivity or attitudes about what constitute acceptable solutions to various aspects of the earthquake problem. To ensure that candidate technologies are continually added to the portfolio, MCEER is conducting a series of workshops on the theme of "Mitigating Earthquake Disasters Through Advanced Technologies" (MEDAT), which are designed to identify new and emerging technologies that can be usefully applied to enhancing levels of seismic safety.
3. Facilitating technologies that, based on findings from MCEER's research, appear to offer the greatest promise for addressing key loss-reduction challenges. Examples of facilitating technologies include optimization and automated design software, simplified design procedures, other technology delivery facilitators, standard details, and seismic codes.
4. A range of tools that provide support for rehabilitation, response, and recovery decision making. These decision tools and their associated technologies include advanced methods for earthquake loss estimation and post-event damage assessment, cost-benefit methodologies that can be used to support mitigation decision-making, and response and recovery decision support systems. As part of this phase of its research program, MCEER is also investigating strategies for overcoming barriers to the adoption and implementation of loss reduction technologies. Decision tools and facilitating technologies will be combined in testbeds and demonstration projects that provide a focus for the work of multidisciplinary teams.
5. Project activities that focus on bringing about a convergence between engineering and societal perspectives on the performance of critical facilities. It has historically been very difficult to bridge gaps between the results of engineering analyses that attempt to predict the performance of structures and systems and the ways in which stakeholders such as regulatory agencies, facility owners, and the general public define acceptable performance levels. MCEER will address this need through research activities aimed at reconciling these often divergent views.
6. Research outcomes that result from these multidisciplinary investigations. Those outcomes include products and devices, policies and guidelines (e.g. seismic retrofit guidelines for critical facilities), new methodological approaches, technologies whose effectiveness has been demonstrated, and implementation strategies. Implementation activities are encouraged

through the Center's education and outreach efforts. At the same time, MCEER recognizes that, given variations in earthquake vulnerability, commitment to earthquake loss reduction, and the costs associated with adoption relative to benefits, there will also be variation in which loss reduction measures are judged most appropriate for different societal settings.

Throughout this process, constant monitoring and interaction takes place at various levels (as shown in Figures 3 and 4). This ensures responsiveness and resource reallocation to capitalize on opportunities created by positive changes in receptiveness of the loss reduction market. Likewise, assessments of the effectiveness of pioneering implementations made as a result of early research outcomes, combined with the benefit of time and experience, allow researchers to identify previously unrealized shortcomings in knowledge, new needs, and better potential solutions, and define additional research needs.

Over time, benefiting from the experience of various tentative implementations, convergence towards a single model of implementation is foreseen. Typically however, as with any endeavor dealing with the public, this is a lengthy process. In this regard, education and outreach play a catalytic role to accelerate adoption of the new technologies. MCEER's implementation strategy provides many important linkages between the research and all interested parties (public agencies, code committees, users of the technology, etc.). These linkages are described in more detail in the Strategic Plan (see the web address provided in the abstract of this paper).

Conclusions

This paper provides a brief overview of MCEER's vision, mission, and Strategic Plan, in general terms, with emphasis when appropriate to the seismic evaluation and retrofit of hospital structures. It also illustrates MCEER's Systems Approach that drives its research agenda.

As indicated here, the workshops on the Mitigation of Earthquake Disasters using Advanced Technology (MEDAT) serve an important role in identifying new and emerging technologies that may not yet have been considered in earthquake engineering.

Technical Block 1: General Overview of Geotechnical Issues

Chairs: Ricardo Dobry and James Mitchell

Centrifuge-Based Evaluation of Pile Foundation Response to Lateral Spreading and Mitigation Strategies
Ricardo Dobry, Tarek Abdoun and Thomas D. O'Rourke

Liquefaction Remediation in Silty Soils
S. Thevanayagam and W. Jia

Passive Site Remediation for Mitigation of Liquefaction Risk
Patricia M. Gallagher and James K. Mitchell

Mitigation of Liquefaction Hazards
Juan I. Baez

Performance of Soil Reinforcement Systems in Earthquake Zones
Sherif Hanna and Ilan Juran

Foundation Liquefaction Retrofit Work - Abstracts
Ignatius Po Lam

Centrifuge-Based Evaluation of Pile Foundation Response to Lateral Spreading and Mitigation Strategies

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Introduction

The effects of liquefaction on deep foundations are very damaging and costly. Permanent lateral ground deformation or lateral spreading is a main source of distress to piles, either alone or in combination with inertial superstructural forces and moments arising during shaking and acting on a soil already weakened by rising water pore pressures. Cracking and rupture of piles at shallow and deep elevations, rupture of pile connections, and permanent lateral and vertical movements and rotations of pile heads and pile caps with corresponding effects on the superstructure have been observed (Fig. 1). This has affected buildings, bridges, port facilities and other structures in Japan, the U.S. and other countries including the 1989 Loma Prieta, CA and the 1995 Kobe, Japan, earthquakes (Hamada and O'Rourke, 1992; O'Rourke and Hamada, 1992; Tokimatsu et al., 1996; Dobry and Abdoun, 2001).

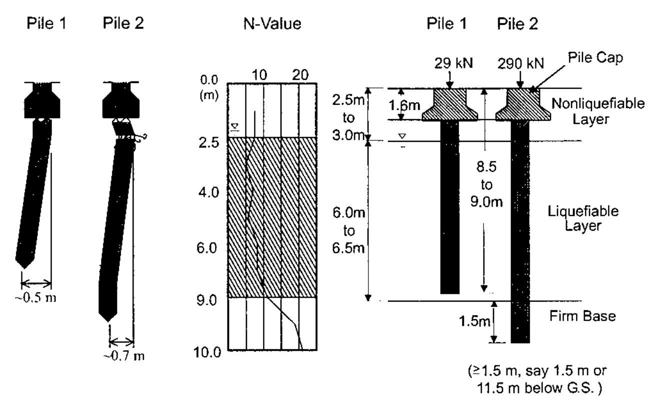


Figure 1. Damage to pile foundations due to lateral spreading under NFCH building, 1964 Niigata earthquake, Japan (Hamada, 1992, 2001)

Examination and analysis of case histories have revealed important aspects of the phenomenon and highlighted its complexity. It is essentially a kinematic soil-structure interaction process involving large ground and foundation permanent deformations, with the deep foundation and superstructure responding pseudostatically to the lateral permanent displacement of the ground.

While in some cases the top of the foundation displaces laterally a distance similar to that in the free field, like in Fig. 1 where both the ground and foundation moved horizontally about 1 m, in others it moves much less due to the constraining effect of the superstructure, or of the deep foundation's lateral stiffness including pile groups and batter piles. The foundation may be exposed to large lateral soil pressures, including especially passive pressures from the nonliquefied shallow soil layer riding on top of the liquefied soil. In some cases this soil has failed before the foundation with negligible bending distress and very small deformation of the foundation head and superstructure (Berrill et al., 1997); while in others the foundation has failed first in bending (Fig. 1) and/or has experienced excessive permanent deformation and rotation at the pile heads. The observed damage and cracking to piles is often concentrated at the upper and lower boundaries of the liquefied soil layer where there is a sudden change in soil properties, or at the connection with the pile cap (Fig. 1). More damage tends to occur to piles when the lateral movement is forced by a strong nonliquefied shallow soil layer (end-bearing pile No. 2 in Fig. 1), than when the foundation is freer to move laterally and the forces acting on them are limited by the strength of the liquefied soil (floating Pile No. 1 in Fig. 1).

Lateral spreading has been identified as a major hazard to pile foundations of hospital buildings, and centrifuge modeling as a key tool to identify and quantify mechanisms, calibrate analyses and evaluate retrofitting strategies for pile foundations. Figure 2 shows the 100 g-ton RPI geotechnical centrifuge used for this research, which is located at the RPI campus in Troy, NY. This centrifuge, originally commissioned in 1989 with support from NCEER, has in-flight earthquake simulation capability allowing base shaking to be applied to the base of the model. It was recently selected by NSF together with other earthquake engineering experimental sites throughout the U.S. to form the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES, www.eng.nsf.gov/nees). Additional information on the centrifuge equipment used in this research, results from other projects and the basic principles of centrifuge modeling, can be found at the RPI Web site (www.ce.rpi.edu/centrifuge), which also has useful links to other relevant Web sites; see also summary articles by Dobry et al. (1995) and Dobry and Abdoun (1998, 2001). In addition to the centrifuge experiments themselves done at RPI, this centrifuge-based research has included other analytical, laboratory, case history review and retrofitting strategy components, conducted either at Cornell University or in close cooperation between the RPI and Cornell teams. The RPI-Cornell joint centrifuge-based research on lateral spreading effects on piles started in 1995 with support from NCEER and NSF and has continued since then with current support from both MCEER and NSF. The technical discussion below is divided in three parts: case of pile bending response to lateral spreading controlled by the pressure of the liquefied soil, case of response controlled by shallow nonliquefied soil layer, and pile retrofitting strategies and results.

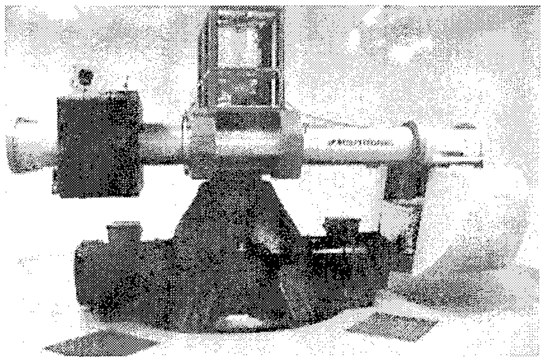


Figure 2. 100 g-ton geotechnical centrifuge with in-flight shaking capability at RPI

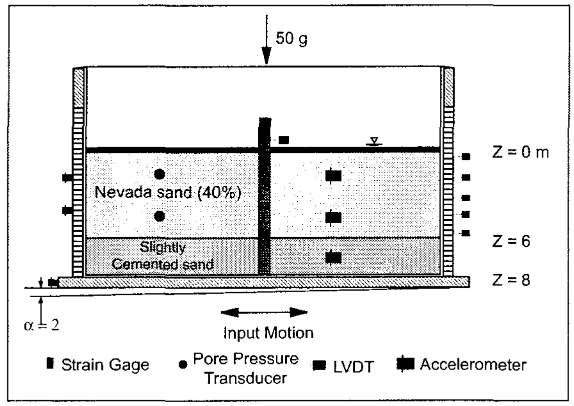


Figure 3. Lateral spreading pile centrifuge model in two-layer soil profile (Abdoun, 1997)

Pile Bending Response Controlled by the Liquefied Soil: Figure 3 shows centrifuge pile Model 3, simulating the bending response of a pile foundation subjected to the lateral pressure of a liquefied soil due to lateral spreading. These and other experiments were conducted using the rectangular, flexible-wall laminar box container sketched in Fig. 3. This laminar box is comprised of a stack of up to 39 rectangular aluminum rings separated by linear roller bearings, arranged to permit relative movement between rings with minimal friction. In Model 3 as well as in all other lateral spreading spreading experiments, the laminar box and the shaker under it are inclined a few degrees to the prototype horizontal direction to simulate an infinite mild slope and provide the shear stress bias needed for a lateral spread. The flexibility of this box container is demonstrated by the large permanent deformations and strains attained in the experiments (Fig. 5).

In the test of Fig. 3, the soil profile consists of two layers of fine Nevada sand saturated with water: a top liquefiable layer of relative density, D_r = 40% and 6 m protoype thickness, and a bottom slightly cemented nonliquefiable sand layer having a thickness of 2 m. The prototype single pile is 0.6 m in diameter, 8 m in length, has a bending stiffness, EI = 8000 kN-m^2, and is free at the top. The pile model is instrumented with strain gages to measure bending moments along its length, and a lateral LVDT at the top to measure the pile head displacement. The soil is instrumented with pore pressure transducers (piezometers) and accelerometers, as well as with lateral LVDTs mounted on the rings of the flexible wall to measure soil deformations in the free field. A prototype input accelerogram consisting of 40 sinusoidal cycles of a peak acceleration of 0.3 g was applied to the base, which liquefied the whole top layer in a couple of cycles and induced a permanent lateral ground surface displacement in the free field of about 0.8 m.

Results of this experiment are shown in Figs. 4-5. As soon as the top sand layer liquefied at the beginning of shaking, it started moving laterally downslope throughout the shaking, with the maximum displacement at all times measured at the ground surface, and with this surface ground displacement increasing monotonically with time to its final value D_H = 0.8 m at the end of shaking. The maximum bending moment along the pile at any given time occurred at the interface between the two soil layers, that is at a depth of about 6 m. Figure 4 shows the time history of this prototype bending moment for Model 3, measured at z = 5.75 m; the plot reveals that the moment increased to a maximum M_{max} = 110 kN-m at a time, t ≈17 sec, with the moment decreasing afterwards despite the continuation of shaking and the continuous increase of the soil deformation in the free field. The pile head displacement in the same figure also reached a maximum at about 17 sec and decreased afterwards. Clearly at this time the liquefied soil reached its maximum strength and applied a maximum lateral pressure to the pile, with the soil flowing around the pile, exhibiting a smaller strength and applying a smaller pressure afterwards; as a result, the model pile bounced back and the bending moments decreased. The two photos in Figure 6 - taken after the centrifuge tests - illustrate this flow of liquefied soil around the pile in other two models where colored sand had been placed around the pile.

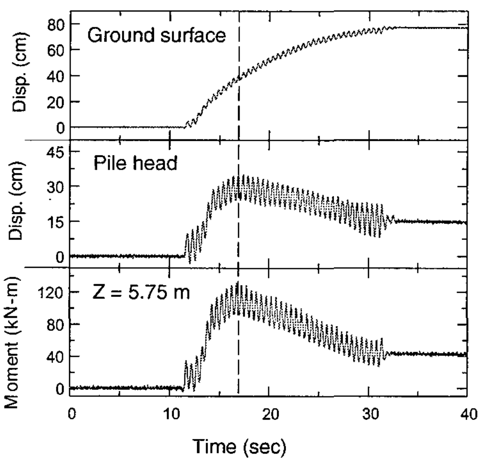


Figure 4. Prototype lateral displacement of soil and pile and ground surface, and pile bending moment at a depth of 5.75 m in model of Figure 3 (Abdoun, 1997)

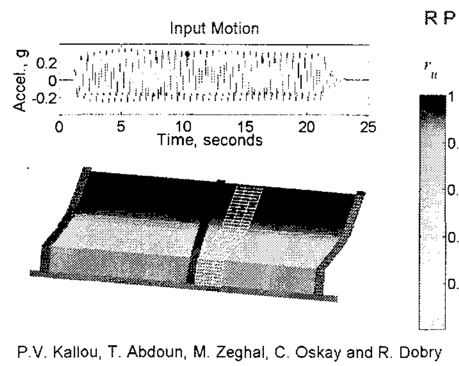


Figure 5. Frame taken out of visualization of two-layer centrifuge model of Figure 3, produced from the recorded data (Kallou et al., 2001; to see whole visualization, visit *http://www.ce.rpi.edu/centrifuge*)

Figure 5 summarizes the state of the system during a repeat of Model 3, at the time when the pile head displacement and the bending moment at a depth of about 6 m attained their maximum values. This is a frame taken from the visualization of the experiment produced from the measurements (the whole visualization may be viewed at the RPI centrifuge Web site). The displaced shape of the box container indicates the lateral spreading in progress, with concentration of permanent shear straining in the lower part of the liquefied soil; this box shape was obtained from the lateral LVDTs placed on the side walls. This distorted shape is also copied as a white mesh to the right side of the pile for direct comparison between ground and pile displacements as well as to visualize the larger movement of the liquefied soil flowing around the pile, compared with the displacement of the pile itself. The blue color in the upper part of the loose sand layer indicates complete liquefaction as measured by the piezometers, while the green color in the lower part of the layer indicates lower excess pore pressure due to dilative cyclic stress-strain response of the liquefied sand in that part of the shaking cycle. At other times corresponding to different parts of the shaking cycle the whole layer is blue and hence completely liquefied.

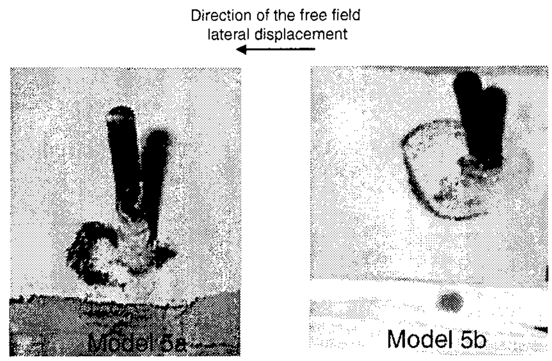
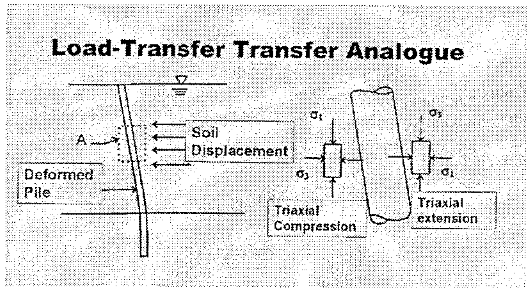


Figure 6. Photos showing flow of liquefied sand around the pile in the downslope direction in two-layer centrifuge models (Abdoun, 1997). The photos were taken after the test in models where colored sand had been placed in a circular ring around the pile.

In addition to Model 3 summarized in Figs. 3-5, similar centrifuge tests of a single pile with a pile cap, with densification around the pile to simulate pile driving, and with 2x2 pile groups indicated that, while M_{max} still occurs at a depth of about 6 m sometime during the shaking, the value of M_{max} increases with the area of pile foundation exposed to the soil lateral pressure and decreases in the pile groups due to the contribution to moment of the axial forces in the piles (frame effect). Simple limit equilibrium calculations with a constant assumed maximum pressure of the liquefied soil along the pile, p_l , indicate that values of p_l of the order of 10 kPa explain well all measured trends and values of M_{max} in this series of centrifuge tests.

The physical origin and basic mechanisms determining the behavior of the liquefied soil, including the lateral pressure on pile foundations and values such as p_l and M_{max} measured in these centrifuge tests, are not yet well understood and are the subject of intense research. The Cornell team has proposed the explanation sketched in Fig. 7, with p_l and M_{max} controlled by the peak undrained shear strength of the saturated sand loaded in the extension mode (Goh and O'Rourke 1999; Goh, 2001). Based on p-y curves generated analytically from triaxial extension tests conducted at Cornell using the same Nevada sand and relative density of the centrifuge tests, nonlinear Beam-on-Winkler - Foundation (BWF) analyses of centrifuge Model 3 were able to predict closely the measured bending response (Fig. 8).



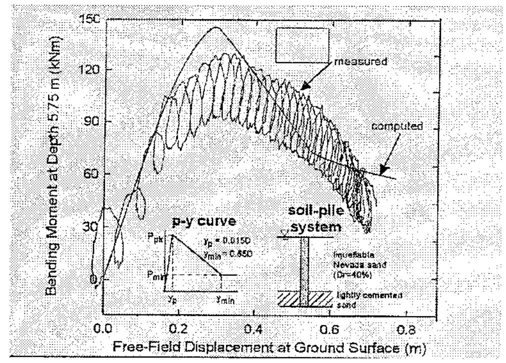


Figure 7. Concept used to develop undrained triaxial extension model for the lateral loading of liquefied soil on the pile (Goh and O'Rourke, 1999; Goh, 2001)

Figure 8. Comparison between predicted and measured pile bending moment in centrifuge model of Fig. 3 at the lower boundary of liquefied soil using triaxial extension undrained loading approach (Goh and O'Rourke, 1999; Goh, 2001)

Pile Bending Response Controlled by Shallow Nonliquefied Layer: Figure 9 shows centrifuge Model 2, where a strong shallow nonliquefied soil layer increases significantly the bending response of the pile foundation to lateral spreading. The shallow top layer consists of a 2-m thick (in prototype units), free draining, slightly cemented sand. Model 2m, not shown here, is entirely similar to Fig. 9 but with a mass added above ground to evaluate the combined effects of lateral spreading and inertial loading.

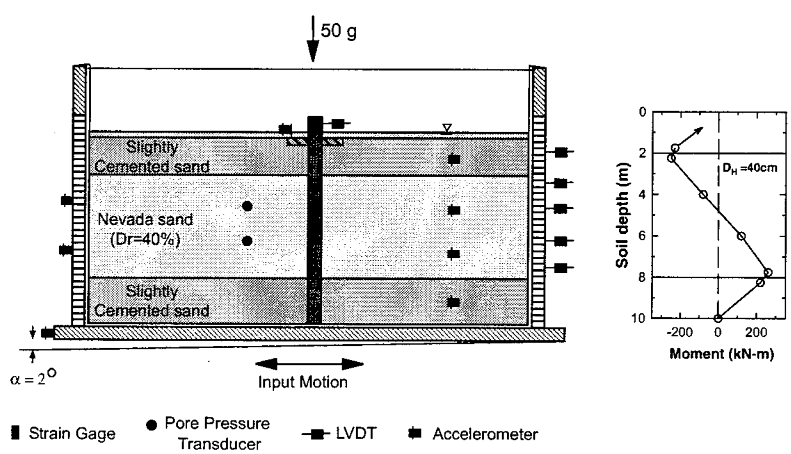


Figure 9. Lateral spreading pile centrifuge model in three-layer soil profile (Abdoun, 1997)

Figures 10-11 summarize the main characteristics of the bending response of Model 2 (only lateral spreading), which is also typical of other pile models tested in this 3-layer soil profile. The same as in Model 3 discussed before, the 6-m thick noncemented sand layer liquefied early in the shaking after which the lateral spreading increased monotonically, reaching a value D_H = 0.8 m at the end of shaking (Fig. 11). The pile bending moments in the top 2 m first increased with time of shaking and then decreased after passive failure of the top nonliquefiable layer against the pile (Fig. 10); while the bending moments near the bottom increased monotonically and never decreased, as the bottom nonliquefiable layer did not fail. The values of maximum bending moments at 2 m and 8 m are close to 300 kN-m, much greater than those measured in 2-layer tests such as shown in Fig. 3, which did not exceed 170 kN-m even when a pile cap was added. The shapes of the bending moment profiles at various times presented in Fig. 10 indicate that the deformed shape of the pile had a double curvature caused by the top and bottom soil layers loading the pile in opposite directions. This double curvature was confirmed by the fact that when the top soil layer failed, the pile head and cap "snapped" in the downslope direction (Fig. 11), showing that at very shallow depths the pile was pushing the soil rather than the other way around. Both the passive failure of the top layer and the moment concentrations at the top and bottom boundaries of the liquefied layer indicated by the figures are consistent with the experience from earthquake case histories. These moment concentrations are also predicted by theory (e.g., Meyersohn, 1994; Meyersohn et al., 1992; Debanik, 1997). Another interesting aspect of Figs. 10-11 is that the bending moments vary linearly within the liquefied layer, suggesting that they are essentially controlled by the loading of the top and bottom layers, with the pressure of the liquefied soil being negligible. The values of M_{max} at z = 2 m and z = 8 m are higher than the corresponding values of M_{max} at z = 6 m for the 2-layer soil profiles, such as in Fig. 4, which were controlled by the strength of the weaker liquefied soil. The authors have successfully calibrated a limit equilibrium method to predict M_{max} in some of these 3-layer pile centrifuge models, after incorporating basic kinematic considerations to allow for the change in pile curvature (and hence of the sign of the passive soil pressure on the pile) within the top nonliquefied soil layer.

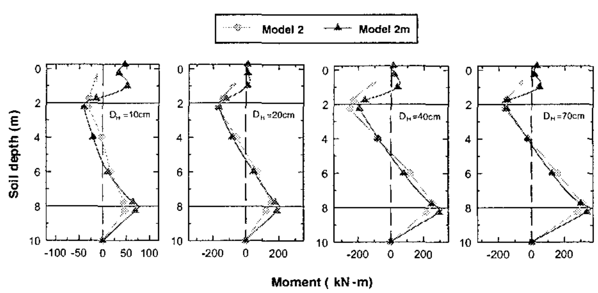


Figure 10. Measured bending moment response along pile in lateral spreading centrifuge models without (Model 2) and with (Model 2m) inertial loading (Wang, 2001)

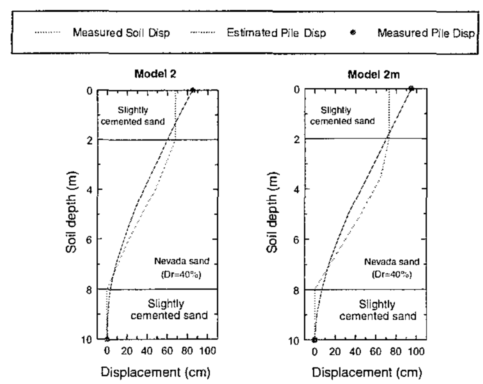


Figure 11. Snapping of pile in downslope direction in centrifuge models without (Model 2) and with (Model 2m) intertial loading (Wang, 2001)

The comparisons in Figs. 10-11 between Models 2 and 2m reveal interesting aspects of the role played by superstructural inertia in the lateral spreading process. For depths greater than 2 or 3 m, the effect of lateral spreading predominates and the inertial loading due to the mass can be ignored. However, at shallow depths of less than 2 m, that is in the top nonliquefiable layer, the bending moments of the two centrifuge models are very different, with those of Model 2m changing rapidly with time due to the combined effect of inertia and lateral spreading. However, even in Model 2m the maximum moments still tend to concentrate at the upper and lower boundaries of the liquefied layer. Despite the rapid change in shallow bending moments due to the mass, when the top soil layer failed in passive in Model 2m, the pile head and cap "snapped" in the downslope direction, exactly the same as in Model 2 (Fig. 11), showing that the soil failure mechanism was still controlled by lateral spreading.

Another factor which has been studied in the centrifuge for the 3-layer soil model is the influence of the superstructural stiffness that field case histories has shown to be important. This has been done by the addition of lateral and rotational springs above ground connected to the pile head, such as spring k in Fig. 12 (Ramos, 1999). As expected, the analysis of these centrifuge results has required significant kinematic considerations and parameters, even when simple limit equilibrium calculations are conducted. On the other hand, some aspects of the analysis become simpler compared with the case of k = 0 (Fig. 3), in that if the value of k is large enough, there is no double curvature of the pile at very shallow depths, and no "snapping" of the pile in the downslope direction as in Fig. 11. That is, the constraining effect of spring k forces the lateral pressure of the nonliquefied layer on the pile to act in the same downslope direction at all depths between 0 and 2 m.

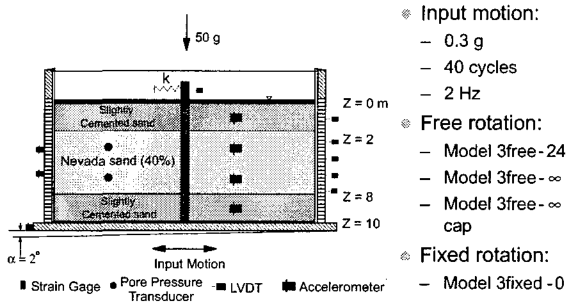


Figure 12. Lateral spreading pile centrifuge model incorporating effect of superstructural stiffness (Ramos, 1999)

Pile Retrofitting Strategies and Results: Both case histories and centrifuge models have shown the great importance of the shallow nonliquefiable soil in increasing the bending response of the pile foundation. Therefore, a promising rehabilitation approach of existing foundations is to replace the shallow soil in a trench around piles and pile cap by a frangible material that will yield under constant lateral soil forces (Fig. 13a). This would decrease both bending moments and foundation deformations while allowing the ground lateral spreading to take place without interference from the foundation. As this retrofitting scheme also decreases the lateral resistance of the foundation to inertial loading, the desired frangible material selected, while yielding to static force should remain resilient under the transient inertial loading. Alternatively, the trench surrounding the foundation with frangible material may be located at some distance from the foundation so as to increase the resistance to inertial loading (Fig. 13b).

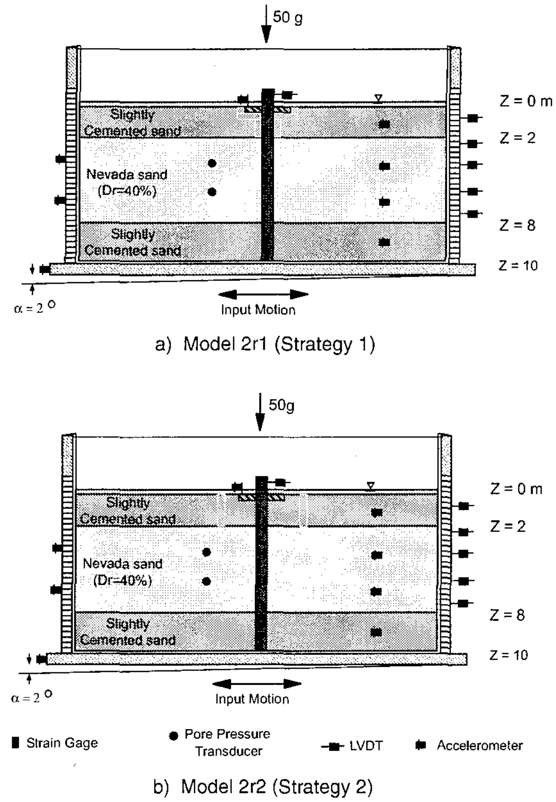


Figure 13. Lateral spreading pile centrifuge models to evaluate retrofitting strategies (Wang, 2001)

A series of centrifuge models of a single pile with pile cap in the 3-layer soil profile were conducted using the retrofitting setups of Fig. 13, labeled respectively Strategy 1 and Strategy 2. These experiments are listed in Table 1, which include also the benchmark nonretrofitted Models 2 and 2m, already discussed. Models 2rl, 2mrla and 2mrlb were done with Strategy 1, without and with a mass above ground, and Models 2r2 and 2mr2 were conducted with Strategy 2. In both cases, a soft clay was placed in a trench either directly around or at some distance from the foundation. In future tests the use of an artificial frangible material with higher resistance to transient loading is planned (Wang, 2001).

Figures 14-15 illustrate measurements and observations obtained from Models 2rl and 2r2. The free field lateral ground displacements during shaking in centrifuge tests without and with pile foundation retrofitting were essentially the same (Fig. 11), consistent with the assumption that they represent truly free field response. Figure 14 compares the bending moment response without and with retrofitting. As expected, there is a dramatic reduction in the moments in the top 2 m of pile in contact with the nonliquefiable soil. The maximum moment there was close to 300 kN-m in Model 2 and becomes about 10 kN-m after retrofitting. A smaller reduction is also observed for the maximum bending moment at the lower boundary of the liquefied layer, at about 8 m depth. Similarly, the pile head displacements at the end of the tests were reduced by a factor of two by retrofitting (from 85 to 40-50 cm, with D_H = 70 to 80 cm for the soil in the free field).

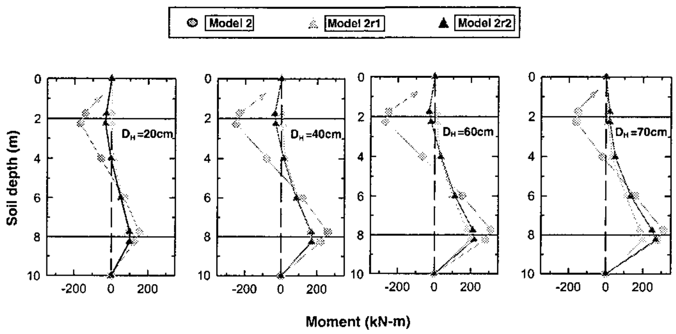


Figure 14. Measured bending moment response along pile in lateral spreading centrifuge models without (Model 2) and with (Models 2r1 and 2r2) foundation retrofitting (Wang, 2001)

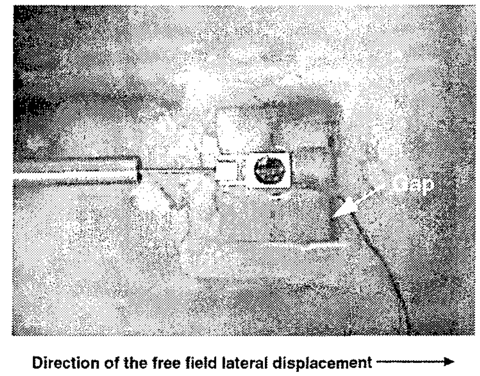


Figure 15. Plan view of retrofitted pile cap and ground after the test, Model 2r1 (Wang, 2001)

The photo of Model 2rl in Fig. 15, taken after the test, illustrates the corresponding "crunching" of the soft clay against the pile cap in the upslope side, and opening of a gap downslope between soil and foundation. However, the counterpart to this reduction in permanent bending response to lateral spreading of the pile foundation was an increase of transient pile accelerations and displacements, especially in the tests incorporating inertial loading (Models 2mrla,b and 2mr2, not shown), due to the reduced lateral ground support in the top 2 m of the foundation; future tests will address this problem.

Table 1: Program of centrifuge tests to evaluate retrofitting strategies 1 and 2 (Wang, 2001)

Test No.	Cap	Mass	Retrofitting	Comments
2	Yes	No	No	
2rl	Yes	No	Yes	
2r2	Yes	No	Yes	
2m	Yes	Yes	No	
2mrla	Yes	Yes	Yes	
2mrlb	Yes	Yes	Yes	Repeat of 2mrla
2mr2	Yes	Yes	Yes	

Conclusions and Future Research: Case histories during earthquakes have shown the significance of lateral spreading in causing damage to deep foundations and supported structures during earthquakes. The complexity of the problem requires use of centrifuge physical modeling to clarify mechanisms, quantify relations and calibrate analysis and design procedures. Centrifuge results so far have clarified the deep foundation response, have shown significant agreement with field experience, and are being used to calibrate limit equilibrium and Beam-on-Winkler-Springs (p-y) analytical methods. Specifically, the importance of the shallow nonliquefiable soil layer riding on top of the liquefied soil in increasing foundation bending response has been clarified. Retrofitting strategies are being evaluated in the centrifuge, aimed at mitigating the effect of lateral spreading associated the pressure of this shallow layer while preserving needed lateral resistance to inertial loading. Additional work is needed to understand and quantify the response of nonretrofitted and retrofitted pile foundations, with centrifuge model experiments combined with case studies and theory, toward improving the state-of-practice of seismic design and retrofitting of deep foundations against liquefaction.

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Liquefaction Remediation in Silty Soils

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Abstract
An electrokinetic permeation technique for injection of cementitious grouting materials into low permeable silty soils for liquefaction mitigation and foundation retrofitting is discussed. Preliminary experimental data shows that such a technique is feasible. Further research and exploration is needed to further develop and refine this technique.

Introduction
Loose saturated granular deposits, when subjected to rapid shear loading, experience a rapid increase in pore pressure and temporary loss of strength, which may lead to liquefaction, a stage, when soil looses almost all the strength and behaves like a liquid. Such a stage leads to lateral ground spreading, densification and vertical ground settlements, and ground instability. This in turn causes foundation distress and instability of buildings and other superstructure. Soil types prone to liquefaction are loose sands, and silty sands/sandy silts containing mostly non-plastic silt. Only rarely clays, except for sensitive clays, do experience loss of strength. Past observations of such damages are reminders of the need to develop and transfer prudent *liquefaction/ground damage potential screening techniques* and *site remediation measures* for prevention of liquefaction and/or strengthening of foundations supporting critical facilities such as hospitals, lifelines, and transportation systems founded on such soils.

Current liquefaction screening techniques primarily rely on past field observations of liquefied/non-liquefied sites containing liquefiable soils and intuitive extrapolation of research experience with clean sands. Screening correlations have been developed between the measured normalized penetration resistance (SPT, CPT), or shear wave velocity (v_{s1}) at the liquefied/non-liquefied sites and cyclic stress ratio (CSR) induced by an earthquake at those sites during past earthquakes. A demarcation line separating the data points corresponding to the liquefied sites from those from the non-liquefied sites is considered to represent the cyclic resistance ratio (CRR) of a soil deposit at a site as a function of the in situ penetration resistance or shear wave velocity of that deposit at that site. For a new site, CRR thus discerned from the above relationship using the in situ data from that site is compared with the anticipated cyclic stress ratio (CSR) from a future earthquake at that site to identify whether or not that site would liquefy. Similar techniques are also available for liquefaction induced ground damage potential assessment. Such methods require further research and refinements for extrapolations to all sites, especially those containing silts, with confidence.

The main subject of this paper pertains to site remediation for liquefaction prevention.

Current soil improvement techniques to prevent liquefaction hazards (Table 1) aim to increase cyclic resistance of a liquefiable deposit by one or more of the following: soil densification, drainage, reinforcement, and cementation/solidification by grouting or deep mixing techniques. The applicability of these techniques depends on the soil type, site accessibility, allowable site disturbance, and cost-benefit considerations. All of the above techniques are generally applicable for (clean) sands. When the silt content increases beyond about 15 %, the above techniques become difficult to implement except for soil mixing techniques. The difficulty of implementing densification/drainage and permeation grouting techniques arises due to the low permeable nature of such soils.

When the site accessibility or site disturbance is a concern, in cases such as existing critical health care facilities, permeation grouting technique remains the most viable option, while others become less attractive, if the site contains high permeable sands. The problem is more compounded when such sites contain liquefiable silty soils, leaving little or no cost-effective choice for soil remediation.

This paper presents preliminary results from a recent study aimed to address the latter. This study utilizes direct current to inject solidifying/binding materials into low permeable silty soils and thereby increase the resistance to liquefaction by means of filling the voids by these materials as well as cementation of silt particles. Due to space limitations, only a brief outline of the concept, experimental setup, and results are presented.

Table 1: Soil Improvement Methods for Liquefaction Prevention

Technique	Soil Treated			Accessibility (Existing Structures)	Typical Cost
	Sand	Silty sand	Silt		
Permeation Grout	Yes (Fines ≤5%)	?	No	Yes	MFC - $130/m^3$ Silicate - $250/m^3$ + $50-$100/m
Compaction Grout	Yes	Yes	Marginal	Yes	$20/m^3$ + $50-$100/m
Soil Mixing	Yes	Yes	Yes	No?	$100-$200/m^3$
Jet Grout	Yes	Yes	Yes	No?	≥$320/m^3$
Electro- Kinetic Injection	?	Yes	Yes	Yes	Matl. + Power
Passive Grouting (Mitchell et al.)	Yes	?	?	Yes	-
Stone Column & Vibro-Densification	Yes	?	?	No?	-

MFC – microfine cement, ? = uncertain

Traditional Permeation Grouting

The purpose of permeation grouting is to fill voids as well as facilitate cementation/bonding of particles. This increases density of the soil as well as strength. The potential for loss of contacts and collapse of the soil structure during seismic loading is minimized. Thus liquefaction, the associated lateral spread, and vertical deformation are minimized. The load carrying capacity of the soil beneath foundation is improved minimizing foundation distress.

During permeation grouting, usually a pair of binding agents, a grout and a reactant (hardener), is pumped at high pressure into liquefiable zones via drill holes, typically spaced at 1 to 2m apart. The grout and the hardener react to make a gel or other agents that fill the voids and facilitate the formation soil particle bonds. The time required for the hardener and the grout to fully react varies from minutes to several hours depending on the grout concentrations and the associated chemistry. This limitation and other pumping limitations usually restrict the soil types that can be grouted to highly permeable sands with a permeability range of about 10^{-1} to 10^{-3} cm/s, depending on the grout characteristics. Silty soils often have a coefficient of permeability of the order of 10^{-5} cm/s or less. Such soils cannot be grouted successfully by means of hydraulic pumping.

Electrokinetic injection of grout components into such soils is an innovative idea that appears attractive for such soils. It can be implemented in the field in a way similar to permeation grouting, but using dc current as the means of introducing the grout/hardeners into the ground.

Electrokinetic Grouting

Fig.1 shows a schematic picture of the experimental setup used in this study. When a d.c voltage gradient is applied across a saturated soil, the following phenomena may occur. (i) pore fluid flow from anode to cathode (called electroosmosis), (ii) transport of positively charged dissolved ions from anode to cathode (called electromigration), and (iii) transport of negatively charged dissolved ions from cathode to anode (called electromigration). Past experience with other low permeable (clayey) soils indicate that the electroosmotic fluid flow velocity is of the order of 10^{-5} cm/s or larger and the ionic transport velocity is of the order of 10^{-4} cm/s or larger per 1V/cm voltage gradient. Such experience indicates that it may be possible to use direct electric current to introduce the grout and hardening agents into low permeable silty soils by judiciously introducing the various grout components near the cathode or anode region. By controlling the concentrations and sequence of introduction the various components the hardening time may also be controlled. Preliminary experiments were conducted to assess the potential feasibility of this idea.

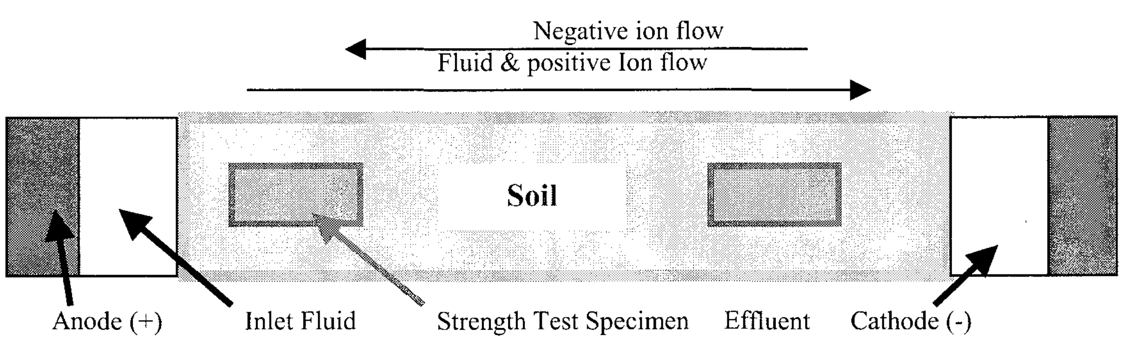


Fig.1: Electrokinetic Grouting Experiment – Schematic Diagram

Preliminary Experiments

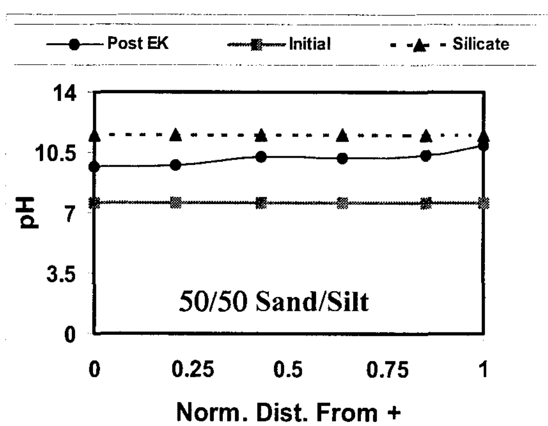
Fig.1 shows the experimental set-up for the first batch of experiments. In these experiments soil columns were prepared by mixing 50% sand and 50% silt by weight saturated with water. The initial electric conductivity and pH of the soil were measured. In the first test series (TEK-19), the inlet tank at the anode (+) was filled with sodium silicate grout (50% silicate, 50% water) and the tank at the cathode (-) contained water. The soil columns were placed horizontally and a d.c voltage gradient of 1.5 V/cm was applied for 3 days using graphite electrode. A control-test (TEK-20) was also conducted using the same setup subjected to the same electric voltage gradient but containing water at the anode inlet tank. In each case, fluid flow rate and electric current were monitored versus time. Following electric treatment, soil samples were recovered from various locations from anode to cathode and were tested for pH and conductivity. Two cylindrical specimens were also recovered and dried under room temperature. The dried specimens were subjected to unconfined compression test. For comparison purposes, conductivity and pH measurements were also made on samples of a sand/silt mix prepared by mechanically mixing it with 50% silicate.

Results

Conductivity and pH: Figs.2a-b show the pH and conductivity data before and after E-K treatment for test TEK-19. Also shown in these figures are the relevant data for the same 50/50 sandy silt mixed mechanically with sodium silicate. The pH and conductivity values for the E-K-treated soil are higher than the initial values. They are less than the values corresponding to the soil mixed with silicate mechanically.

Figs.3a-b show the pH and conductivity data for the *control test* (TEK-20). Generally the post E-K soil pH decreased while the conductivity increased only slightly. The reason this is due to hydrolysis of water producing H^+ ions at the anode. These H^+ ions are then transported into the soil causing a reduction in pH and a slight increase in conductivity.

In contrast, in TEK-19, the increase in pH is due to intrusion of silicate into the soil by electroosmosis as well as neutralization of the H^+ ions by the sodium silicate present in test TEK-19. The intruded silicate also increases the conductivity of the soil.



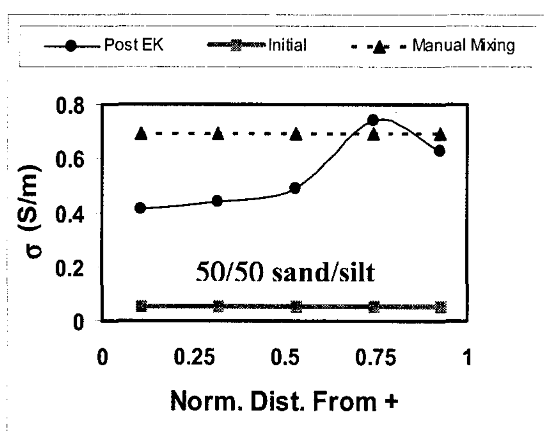


Fig.2 pH and conductivity data – Test TEK-19

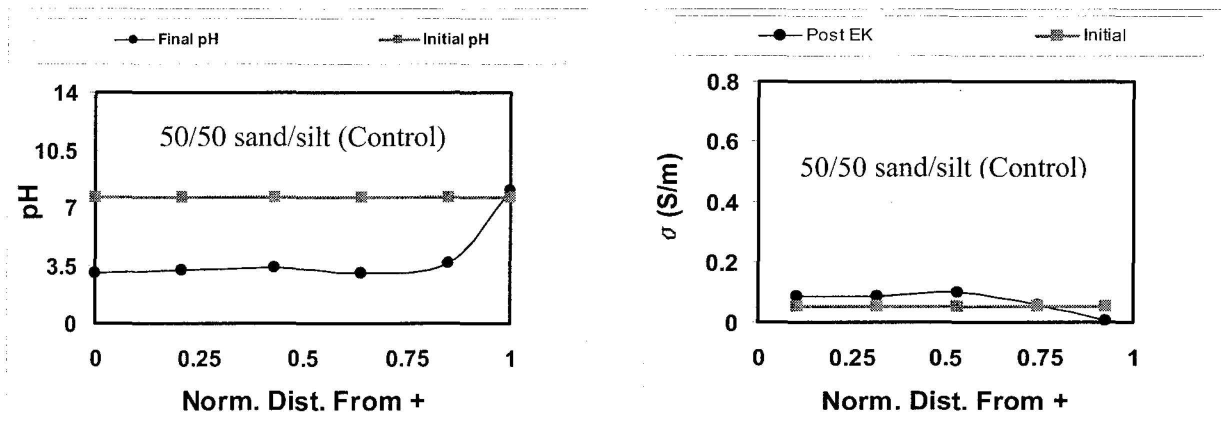


Fig.3 pH and conductivity data – Control Test TEK-20

Silicate Intrusion Rate: Fig.4 shows the electrokinetic permeability (k_e) of the silty soils in tests TEK-19 and 20. In both cases, the fluid intrusion rate is similar and is of the order of 10^{-5} cm/s/v/cm. This would indicate that a 100V/cm voltage gradient would lead to an effective transport velocity of the order of 10^{-3} cm/s, a rate comparable to sands at a hydraulic gradient of 1.0.

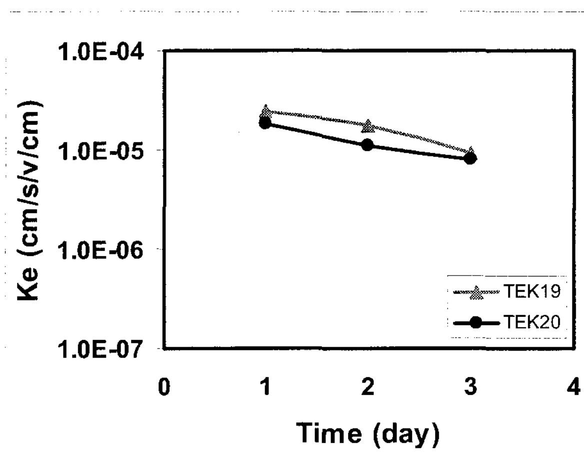


Fig.4 Electrokinetic permeability – TEK19 and TEK-20

Unconfined Compressive Strength: Fig.5 shows the unconfined compression stress-strain data for the test TEK-19. Fig.6 shows the same data for the control test. During the tests, it was also observed that the specimens from the control test crumbled at little or no compressive stress while handling whereas the silicate-E-K treated specimens (TEK-19) showed significant strength.

When all of the above observations are combined, it indicates that silicate intruded into the soil in TEK-19 due to electric gradient.

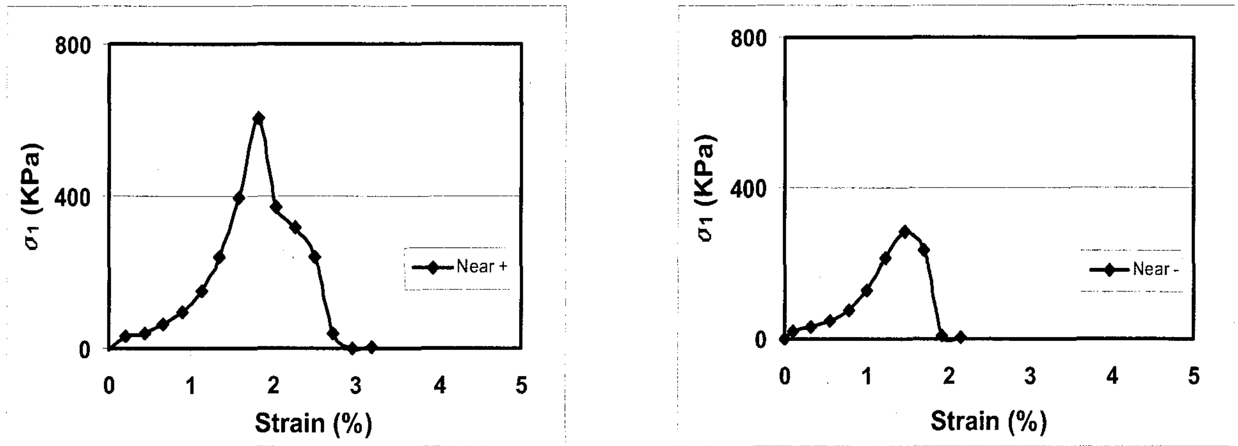


Fig.5 Unconfined compressive strength - TEK19

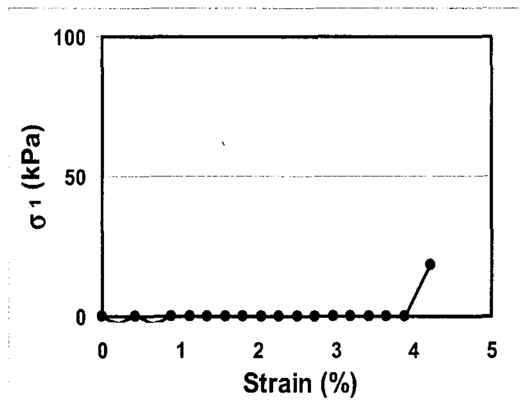


Fig.6 Unconfined compressive strength – Control Test – TEK20

Concluding Remarks

The preliminary test data indicate that it is feasible to inject silicate into silty soils by means of a dc current. The results show that a significant amount of grout can be injected at a rate of about 10^{-5} cm/s per 1V/cm voltage gradient. Further research is ongoing addressing various issues relating to further development and feasibility of this technique. When sufficiently developed, this technique can be used to improve silty soils to prevent liquefaction as well as strengthen/retrofit of foundations beneath critical facilities in a non-intrusive manner. Furthermore this technique can also be combined with other passive grouting techniques and foundation retrofit techniques.

Acknowledgments

The authors wish to thank H. Ahmad and T. Shenthan for their assistance in the laboratory tests. Financial support for this study was provided by MCEER.

Due to brevity of this paper references are not cited. Details of the experiments can be found in "Electrokinetic grouting for soil stabilization – An exploratory study", MS Thesis by H. Ahmad (2001), Univ. at Buffalo, SUNY, NY.

Passive Site Remediation for Mitigation of Liquefaction Risk

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Abstract

Passive site remediation is a new concept proposed for non-disruptive mitigation of liquefaction risk at developed sites susceptible to liquefaction. It is based on the concept of slow injection of stabilizing materials at the edge of a site and delivery of the stabilizer to the target location using the natural groundwater flow. Stabilizer candidates need to have long controllable gel times and low viscosities so they can flow into a liquefiable formation slowly over a fairly long period of time. Colloidal silica is a potential stabilizer for passive site remediation because at low concentrations it has a low viscosity and a wide range of controllable gel times of up to about 200 days. Loose sands treated with colloidal silica grout had significantly higher deformation resistance to cyclic loading than untreated sands. Groundwater and stabilizer transport modeling was done to determine the range of conditions where passive site remediation might be feasible. For a 200-foot by 200-foot treatment area with a single line of injection wells, it was found that passive site remediation could be feasible in formations with hydraulic conductivity values of 0.05 cm/s or more and hydraulic gradients of 0.005 and above.

Introduction

At many sites susceptible to liquefaction, the simplest way to mitigate the liquefaction risk is to densify the soil. For large, open and undeveloped sites, the easiest and cheapest methods for densification are by "traditional" procedures such as deep dynamic compaction, explosive compaction, or vibrocompaction. However, at constrained or developed sites, ground improvement by densification may not be possible due to the presence of structures sensitive to deformation or vibration. Additionally, access to the site could be limited and normal site use activities could interfere with mitigation activities. At these sites, the most common methods for remediation are grouting or underpinning. Passive site remediation is a new concept proposed for non-disruptive improvement of developed sites susceptible to liquefaction. Passive site remediation is based on the concept of the slow injection of stabilizing materials at the up gradient edge of a site and delivery of the stabilizer to the target location using the natural or augmented groundwater flow. The concept is illustrated in Figure 1.

The set time of the stabilizer would be controlled so there would be adequate time for it to reach the desired location beneath the site prior to gelling or setting. If the natural groundwater flow were inadequate to deliver the stabilizer to the right place at the right time, it could be augmented by use of low-head injection wells or downgradient extraction wells. Once the stabilizer reached the desired location beneath the site, it would gel or set to stabilize the formation.

Passive site remediation techniques could have broad application for developed sites where more traditional methods of ground improvement are difficult or impossible to implement. It would be less disruptive to existing infrastructure and facilities than existing ground improvement methods. Additionally, access to the entire site would be unnecessary using this technology, and normal site use activities would probably not need to be disrupted. Finally, excessive deformation and disturbance of the ground around and beneath existing structures could be avoided.

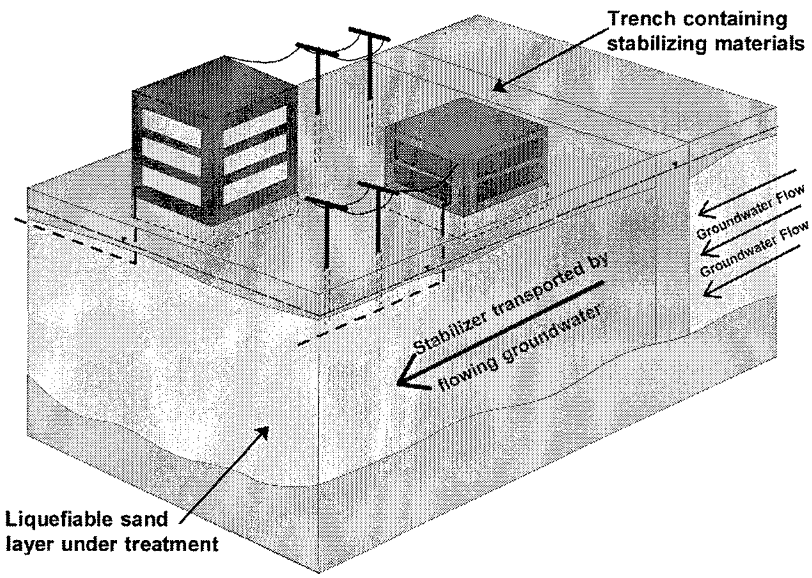

Figure 1. Passive treatment for mitigation of liquefaction risk.

The objective of this study was to establish the feasibility of passive site remediation. The work included identification of stabilizing materials, a study of how to adapt or design groundwater flow patterns to deliver the stabilizers to the right place at the right time, and an evaluation of potential time requirements and costs.

Performance Criteria and Identification of Potential Stabilizers

For a stabilizer to work in this application, it should have a low viscosity and a long induction period between mixing and the onset of gelation. Once gelation starts, it should proceed rapidly. The stabilizer should also be permanent, nontoxic and cost-effective. Materials evaluated as potential stabilizers included colloidal silica, microfine cement grouts, chemical grouts, zero-valent iron, and ultramicrobacteria. Colloidal silica was selected as a potentially suitable stabilizing material because it has a wide range of gel times and a low viscosity. Colloidal silica is an aqueous suspension of tiny silica particles that can be made to gel by adjusting the pH or the salt concentration of the solution. Gel times of more than 200 days have been measured in laboratory tests. Additionally, the initial viscosities of dilute solutions of colloidal silica are about 2 centipoise (water=1 cP) and the viscosities remain very low for most of the induction period.

Microfine cement grout was eliminated because its viscosity is too high to meet the necessary requirements for passive site remediation. Additionally, since cement grouts are particulate suspensions, the particles tend to settle in the suspension and further increase the viscosity. Numerous chemical grouts were considered. All were eliminated as potentially suitable stabilizers, but for different reasons. Sodium silicate was eliminated because gel time is not well controlled at long gel times. Additionally, the chemical durability of sodium silicate formulations with long gel times is questionable. Acrylamide is a neurotoxin in powdered form, so it was eliminated due to environmental, safety, and handling concerns. Additionally, it is very expensive. Acrylate was eliminated due to durability concerns. Epoxy and polysiloxane were rejected because they are very expensive. Zero-valent iron is extremely sensitive to oxidation and reduction, so it would be difficult to treat a large area and the minerals precipitated would probably not be chemically durable. Ultramicrobacteria might be able to clog the pores of a formation with a biofilm, but biofilms can be dissolved by strong oxidants such as bleach, so there are durability concerns.

Feasibility

The feasibility of passive site remediation depends on the answers to the following questions:

1. Will the colloidal silica grout adequately stabilize the soil?
2. Can the stabilizer be delivered to the liquefiable formation and achieve adequate coverage within the induction period of the grout?
3. How much will it cost?

Strength testing of stabilized sands was done to address the first issue. Groundwater and stabilizer transport modeling were done to determine if the stabilizer could be delivered to the formation within the induction period of the grout. Finally, a preliminary cost analysis was done to address the final issue.

Strength Testing of Stabilized Sands

Cyclic triaxial tests were done on Monterey No. 0/30 sand samples treated with colloidal silica grout to investigate the influence of colloidal silica grout on the deformation properties of loose sand (D_r = 22%). The grain size distribution of Monterey No. 0/30 sand is shown in Figure 2. Distinctly different deformation properties were observed between grouted and ungrouted samples. Untreated samples developed very little axial strain after a few cycles of loading and prior to the onset of liquefaction. However, once liquefaction was triggered, large strains occurred rapidly and the samples collapsed within a few additional cycles. In contrast, grouted sand samples experienced very little strain during cyclic loading. What strain accumulated did so uniformly throughout loading and the samples remained intact after cyclic loading.

An example is shown in Figure 3 for two samples at a relative density of 22 percent that were tested at a cyclic stress ratio of 0.27. The cyclic stress ratio is defined as the ratio of the maximum cyclic shear stress to the initial effective confining stress. The untreated sample strained 1 percent in 11 cycles and collapsed in 13 cycles. The sample treated with 10 weight percent colloidal silica was tested for 400 cycles. It strained less than about half a percent in 11 cycles, about 8 percent in 400 cycles, and never collapsed. These results are typical for samples treated with 10 percent colloidal silica by weight. For comparison, a magnitude 7.5 earthquake would be expected to generate about 15 uniform stress cycles.

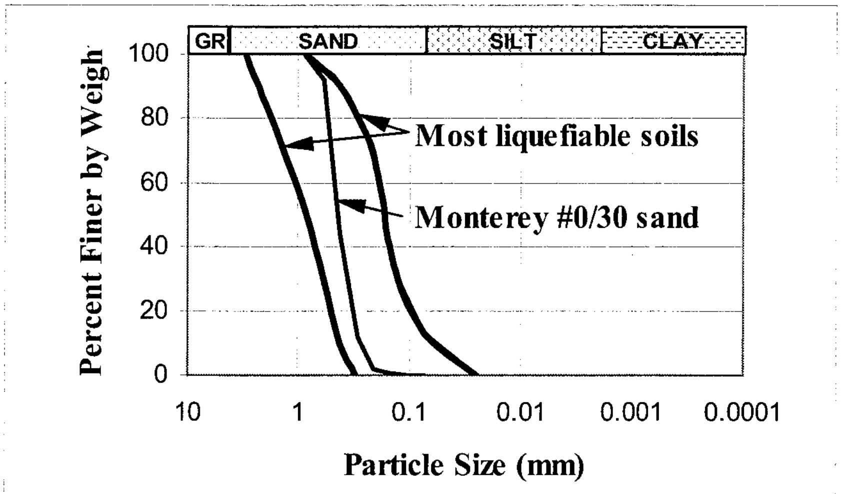

Figure 2 Gradation curve for Monterey No. 0/30 sand

Samples stabilized with concentrations of 15 and 20 weight percent colloidal silica experienced very little (less than two percent) strain during cyclic loading. Sands stabilized with 10 weight percent colloidal silica resisted cyclic loading well, but experienced slightly more (up to eight percent) strain. Overall, treatment with colloidal silica grout significantly increased the deformation resistance of loose sand to cyclic loading.

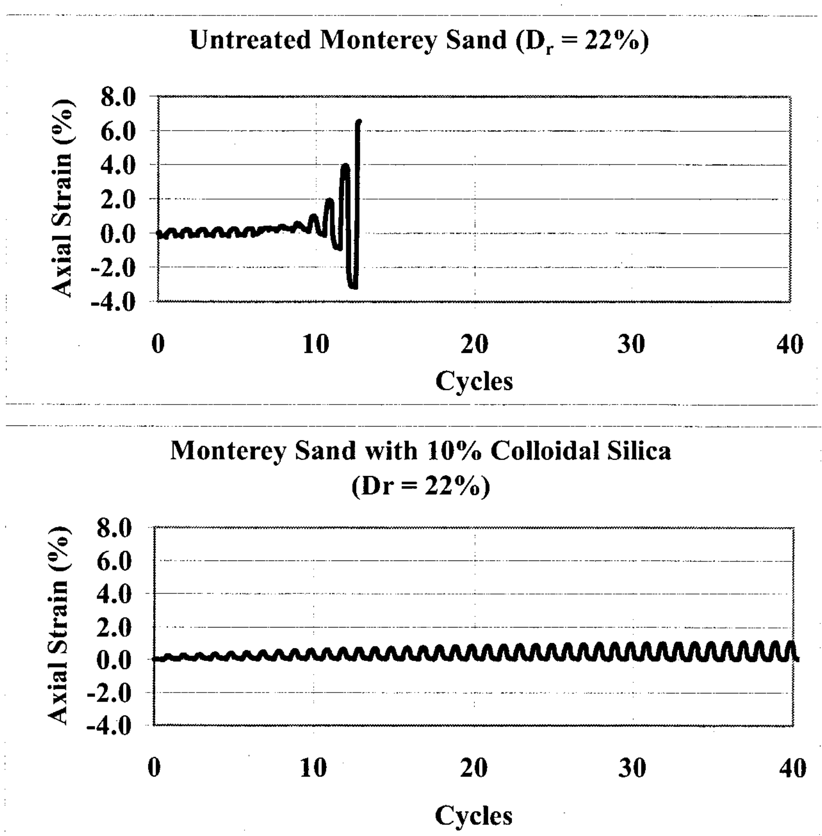

Figure 3. Axial deformation during cyclic loading (CSR=0.27) for treated and untreated sand.

Groundwater and Stabilizer Transport Modeling

Stabilizer delivery is the main feasibility issue with respect to passive site remediation. Preliminary groundwater and solute transport modeling were done using the codes MODFLOW, MODPATH, and MT3DMS for a generic liquefiable formation. A "numerical experiment" was done to determine the ranges of hydraulic conductivity and hydraulic gradient where passive site remediation might be feasible. For a 200-foot by 200-foot treatment area, with single lines of injection and extraction wells, travel times through the treatment area will be about 100 days or less if a formation has a hydraulic conductivity greater than about 0.05 cm/s and a hydraulic gradient higher than about 0.005. Based on the possible gel times, this time frame is considered feasible. Extraction wells will increase the speed of delivery and help control the down gradient extent of stabilizer movement.

The results of solute transport modeling indicate that stabilizer delivery will vary throughout the treatment area. A typical stabilizer contour plot for a hypothetical formation with a uniform hydraulic conductivity of 0.05 cm/s and a hydraulic gradient of 0.005 is shown in Figure 4. A stabilizer concentration of 100 g/l would be delivered through an infiltration trench for 100 days. The best coverage would be achieved close to the source of the stabilizer. Concentrations would decrease laterally away from the source and down gradient of the source. If the minimum amount of stabilizer required for adequate stabilization could be delivered to the majority of the treatment area, it is likely that the formation would be stable enough to withstand seismic loading. However, there could be some differential or variable response across the site. It may be necessary to deliver a higher concentration at the up gradient edge of the treatment area in order to get an adequate concentration at the down gradient edge.

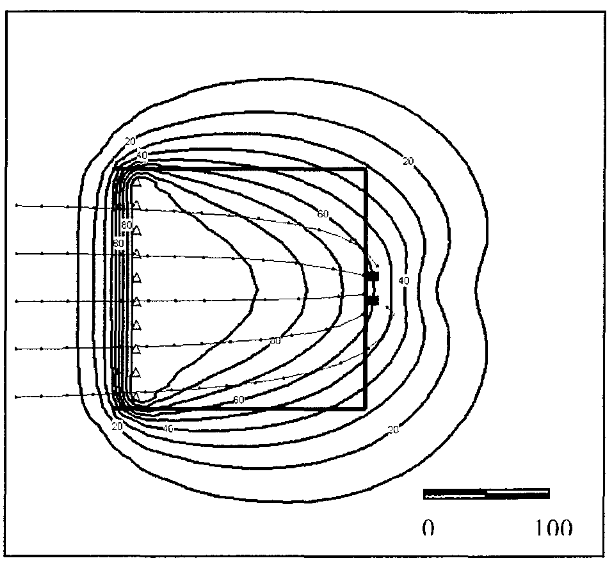

Figure 4 Stabilizer contours for 200' by 200' treatment area (outlined in black) after 100 days of treatment. Stabilizer delivered through infiltration trench at concentration of 100 g/l. Two extraction wells at the down gradient edge withdraw a total of 7500 cfd. Contour intervals are 10 g/l. Concentration at extraction wells is 60 g/l. Travel paths for individual water particles are superimposed over the treatment area in 10-day increments. Particle travel times are about 75 to 80 days.

Heterogeneity in the formation will actually control how well the stabilizer can be delivered. If the formation is highly variable, then the stabilizer concentration will vary from point to point within the formation. An example stabilizer contour profile through a treatment area with a variable hydraulic conductivity is shown in Figure 5. In this case, the hydraulic conductivity was varied slightly in each layer as shown for a total variation throughout the layer of about one order of magnitude. The remainder of the simulation is the same as previous case. The layers with higher hydraulic conductivity have a higher concentration at the down gradient edge. These layers would probably be more stable than layers with lower hydraulic conductivity that receive a lower concentration of grout during the treatment period. However, even if the regions of lower hydraulic conductivity liquefy, the presence of very stable seams will likely lessen the severity of the overall deformation. Accurate characterization of the hydraulic conductivity throughout the treatment area will be essential for successful treatment by passive site remediation.

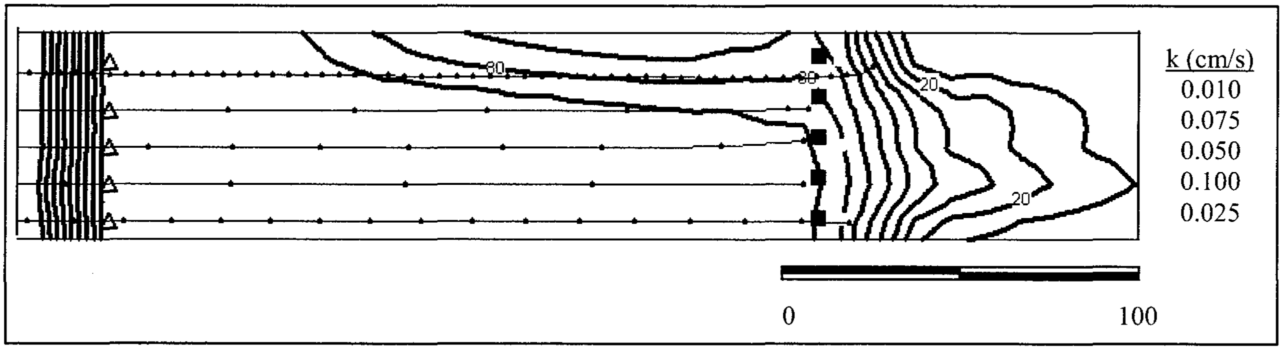

Figure 5 Stabilizer profile through centerline of 200' by 200' treatment area after 100 days of treatment. Stabilizer delivered through infiltration trench at concentration of 100 g/l. Extraction wells at the down gradient edge withdraw a total of 7500 cfd. Contour intervals are 10 g/l. Concentration at extraction wells is about 70 g/l in lower 30'. Travel paths for individual water particles are superimposed over the treatment area in 10-day increments. Particle travel times range from about 40 to 420 days.

Cost

The cost of passive site remediation is expected to be comparable to other methods of chemical grouting. It is likely that a 10 weight percent concentration of colloidal silica will be adequate to stabilize a liquefiable formation. It is possible that lower concentrations could be used. Based on a 10 percent concentration, it is expected that materials costs would be in the range of $120 to $180 per cubic meter of treated soil. These costs are competitive with other methods of chemical grouting.

Conclusion

Based on the feasibility analysis, passive site remediation appears to be a promising new concept for mitigation of liquefaction risk. At this time, a minimum concentration of 10 percent colloidal silica appears to be suitable for stabilizing liquefiable sands. Additional testing is being done with concentrations of 5 weight percent to determine if the level of strain during cyclic loading would be acceptable.

Delivery of the stabilizer is the central feasibility issue with respect to passive site remediation. For a 200-foot by 200-foot treatment area with a single line of injection wells, it was found that passive site remediation could be feasible in formations with hydraulic conductivity values of 0.05 cm/s or more and hydraulic gradients of 0.005 and above. However, the actual concentration profile across the site will depend on the variation in hydraulic conductivity throughout the formation. It is very difficult to accurately characterize the hydraulic conductivity in an aquifer. Although liquefaction tends to occur in fairly uniform formations, it is expected that there would be a fair amount of heterogeneity in any formation that might be a candidate for passive site remediation. Adequate characterization of the hydraulic conductivity would be essential for successful treatment by passive site remediation.

The anticipated final outcome of this work is a new technology for mitigation of liquefaction and ground failure risk. Passive site remediation technology will be less disruptive to existing infrastructure and facilities than existing methods. It is expected that passive site remediation will be cost-competitive with other methods of chemical grouting. Model testing of both the injection method and the performance of grouted ground is planned as the next step in the evaluation of this new technology. It will be done using a geotechnical centrifuge equipped with a shake table.

Mitigation of Liquefaction Hazards

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Hayward Baker, Inc.

Acknowledgements

With the implementation of the Seismic Hazards Mapping Act in California, general guidelines for evaluating and mitigating seismic hazards in California were published by the California Department of Conservation, Division of Mines and Geology (CDMG) in 1997 as Special Publication 117. At the request of Building Officials in the Department of Building and Safety of the City and County of Los Angeles, and under the auspices of the Southern California Earthquake Center (SCEC), a committee of practicing geotechnical engineers and engineering geologists, convened over a 1 ½ year period to develop implementation procedures to meet the requirements of CDMG SP 117 (Martin and Lew, 1999). The text of this paper is largely extracted from the resulting publication of that Committee's work. Appreciation is given to all the members of the committee who contributed in the deliberations and elaboration of the document.

Introduction

In the presence of strong ground motion, liquefaction hazards are likely to occur in saturated cohesionless soils, densification methods, modifications leading to improving the cohesive properties of the soil (hardening or mixing), removal and replacement, or permanent dewatering can reduce or eliminate liquefaction potential. Other methods such as reinforcement of the soil or the use of shallow or deep foundations designed to accommodate the occurrence of liquefaction and associated vertical and horizontal deformations may also achieve an acceptable level of risk.

Often a mitigation measure may involve the implementation of a combination of techniques or concepts such as densification, reinforcement, and mixing. Shallow or deep foundations may also be designed to work with partial ground improvement techniques in order to reduce cost while achieving an acceptable level of risk.

Mitigation should provide suitable levels of protection with regard to potential large lateral spread or flow failures, and more localized problems including bearing failure, settlements, and limited lateral displacements.

The choice of mitigation methods will depend on the extent of liquefaction and the related consequences. Also, the cost of mitigation must be considered in light of an acceptable level of risk. Youd (1998) has suggested that structural mitigation for liquefaction hazards may be acceptable where small lateral displacements and vertical settlement are predicted. Youd cites evidence that houses and small buildings with reinforced perimeter footings and connected grade beams have performed well in Japan, and similar performance should be expected in the United States.

Performance Criteria

Liquefaction mitigation and performance criteria vary according to the acceptable level of risk for each structure type and human occupation considerations. It is not the task of this paper to determine the level of acceptable risk, but to suggest minimum requirements of acceptable liquefaction mitigation.

Implementation of mitigation measures should be designed to either eliminate all liquefaction potential or to allow partial improvement of the soils, provided the structure in question is designed to accommodate the resulting liquefaction-induced vertical and horizontal deformations. In some cases, engineers may decide to design mitigation measures to prevent liquefaction of certain soil types and allow limited deformations in others (i.e., allow some liquefaction).

During the initial site investigation and liquefaction evaluation, the engineer will determine the extent of liquefaction and potential consequences such as bearing failure, and vertical and/or horizontal deformations. Similarly, the engineer will determine the liquefaction hazard in terms of depth and lateral extent affecting the structure in question. The depth of analysis has already been addressed in an earlier section of this report. The lateral extent affecting the structure will depend on whether there is potential for large lateral spreads toward or away from the structure and the influence of liquefied ground surrounding mitigated soils within the perimeter of the structure. Large lateral spread or flow failure hazards may be mitigated by the implementation of containment structures, removal or treatment of liquefiable soils, modification of site geometry, or drainage to lower the groundwater table.

Provided the potential for lateral spreads is addressed and level ground conditions exist, the extent of lateral mitigation beyond the structure footprint is related to bearing capacity and seepage conditions during and after the earthquake event (PHRI, 1997). Because liquefaction mitigation is likely to treat the ground underneath the structure to a sufficient depth, in most cases the bearing capacity reduction due to liquefiable ground outside the structure is not likely to govern the design. Instead, the propagation of excess pore pressures from liquefied to improved ground tends to determine the lateral extent of improvement required. Studies by Iai (1988) indicate that in the presence of liquefiable clean sands an area of softening due to seepage flow occurs to a distance beyond the improved ground on the order of two-thirds of the liquefiable thickness layer. To calculate the liquefiable thickness, similar criteria should be used as that employed to evaluate the issue of surface manifestation by the Ishihara 1985 method addressed in this report (Section 6.6). For level ground conditions where lateral spread is not a concern or the site is not a water front, this buffer zone should not be less than 15 feet and it is likely not to exceed 35 feet when the depth of liquefaction is considered as 50 feet and the entire soil profile consists of liquefiable sand.

The performance criteria for liquefaction mitigation, established during the initial investigation, may be in the form of a minimum, or average, penetration resistance value associated with a soil type (fines content, clay fraction, USCS classification, CPT soil behavior type index I_c, normalized CPT friction ratio), or a tolerable liquefaction settlement as calculated by procedures discussed in Section 6.5 of this report. Soils meeting the discussed Chinese criteria can be excluded from vertical deformation calculations, but they should be carefully considered for loss of strength and potential bearing failure or lateral deformations.

Soil Improvement Options

Soil liquefaction improvement options can be characterized as densification, drainage, reinforcement, mixing, or replacement. As noted before, the implementation of these techniques may be designed to fully, or partially, eliminate the liquefaction potential, depending on input forces and the amount of deformation that the structure in question can tolerate. With regards to drainage techniques for liquefaction mitigation, only permanent dewatering works satisfactorily. The use of gravel or prefabricated drains, installed without soil densification, is not likely to provide pore pressure relief during strong earthquakes and may not prevent excessive settlement. Their use should be evaluated with extreme caution. The following soil improvement methods have demonstrated successful performance in past earthquakes.

Densification Techniques

The most widely used techniques for in-situ densification of liquefiable soils are vibro-compaction, vibro-replacement (also known as vibro-stone columns), deep dynamic compaction, and compaction (pressure) grouting (Hayden and Baez, 1994).

Vibro-compaction and vibro-replacement techniques use similar equipment, but use different backfill material to achieve densification of soils at depth. In vibro-compaction a sand backfill is generally used, whereas in vibro-replacement stone is used as backfill material. Vibro-compaction is generally effective if the soils to be densified are sands containing less than approximately 10 percent fine-grained material passing the No. 200 sieve. Vibro-replacement is generally effective in soils containing less than 15 to 20% fines. However, recent experience (Luehring, et.al., 1998) has verified that even non-plastic sandy silts can be densified by a combination of vibro-replacement and vertical band (wick) drains. In such a case, the vertical band drains are installed at the midpoint of stone column locations prior to installation of vibro-replacement. Due to the usual variation of liquefiable soil types in a given profile and economy of the system, vibro-replacement is typically the most widely used liquefaction countermeasure used in North America (Hayden and Baez, 1994). Detailed design information and equipment characteristics can be found in many publications including Barksdale and Bachus (1983), Mitchell and Huber (1985), Dobson (1987), Baez (1995 and 1997).

Deep dynamic compaction involves the use of impact energy on the ground surface to densify and compact subsurface soils. Weights typically ranging from 10 to 30 tons are lifted with standard, modified, or specialty machines and dropped from about 50 to 120 feet heights. Free-fall impact energy is controlled by selecting the weight, drop height, number of drops per point and the spacings of the grid. Empirical relationships are available to design deep dynamic compaction programs to treat specific site requirements and reconstitute liquefiable soils to a denser condition (Lukas, 1986). In general, treatment depths of up to 35 feet may be achievable in granular soils. If surficial saturated cohesive soils are present or the groundwater table is within 3 to 5 feet of the surface, a granular layer is often needed to limit the loss of impact energy and transfer the forces to greater depths. The major limitations of the method are vibrations, flying matter, and noise. For these reasons, work often requires 100 to 200 feet clearance from adjacent occupied buildings or sensitive structures.

Displacement or compaction grouting involves the use of low slump, mortar-type grout pumped under pressure to densify loose soils by displacement. Compaction grouting pipes are typically installed by drilling or driving steel pipes of 2-inch internal diameter or greater. Injection of the stiff, 3-inch or less slump, cement grout is accomplished with pressures generally ranging from 100 to 300 psi. Refusal pressures of 400 to 500 psi are common in most granular soil projects where liquefaction is the problem. Grout pipes are installed in a grid pattern that usually ranges from 5 to 9 feet. The use of primary spacing patterns with secondary or tertiary intermediate patterns infilled later is effective to achieve difficult densification criteria. Grouting volumes can typically range from 3 to 12 percent of the treated soil volume in granular soils, although volumes up to 20 percent have been reported for extremely loose sands or silty soils. Inadequate compaction is likely to occur when sufficient vertical confinement (less than 8 to 10 feet of overburden) is not present. Theory and case histories on this technique can be found in Graf (1992), Baez and Henry (1993), and Boulanger and Hayden (1995), among others.

Hardening (Mixing) Techniques

Hardening and/or mixing techniques seek to reduce the void space in the liquefiable soil by introducing grout materials either through permeation, mixing mechanically, or jetting. These techniques are known as permeation grouting, soil mixing, or jet grouting.

Permeation grouting involves the injection of low viscosity liquid grout into the pore spaces of granular soils. The base material is typically sodium silicate or microfine cements where the D_{15} of the soil should be greater than 25 D_{85} of the grout for permeation. With successful penetration and setting of the grout, a liquefiable soil with less that approximately 12 to 15 percent fine-grained fraction becomes a hardened mass. Use of this method in North America has been limited to a few projects such as the bridge pier in Santa Cruz, California (Mitchell and Wentz, 1991), and a tunnel horizon in downtown San Francisco. Design methodology and implementation of this technique are described in detail by Baker (1982) and Moseley (1993).

Jet grouting forms cylindrical or panel shapes of hardened soils to replace liquefiable, settlement sensitive, or permeable soils with soil-cement having strengths up to 2,500 psi. The method relies on up to 7,000 psi water pressure at the nozzle to cut soils, mix in place cement slurry and lift spoils to the surface. Control of the drill rotation and pull rates allows treatment of variable soils as described by Moseley (1993). Lightweight drill systems can be used in confined spaces such as inside existing buildings that are found to be at risk to liquefaction after construction.

Deep soil-mixing is a technique involving mixing of cementitious materials using a hollow-stem- auger and paddle arrangement. Gangs of 1 to 5 shafts with augers up to 3 feet or more in diameter are used to mix to depths of 100 feet or more. As the augers are advanced into the soil, the hollow stems are used as conduits to pump grout and inject into the soil at the tip. A trencher device has also been used successfully in Japan. Confining cells are created with the process as the augers are worked in overlapping configurations to form walls. Liquefaction is controlled by limiting the earthquake induced shear strains, and re-distributing shear stresses from soils within the confining cells to the walls. As with jet grouting, treatment of the full range of liquefiable soils is possible and shear strengths of 25 to 100 psi or more can be achieved even in silty soils. The method has been used for liquefaction remediation in only a few cases in North America, including Jackson Lake dam in Wyoming (Ryan and Jasperse, 1989). However, the method has found more extensive use in Japan (Schaefer, 1997).

Structural Options

In some cases, structural mitigation for liquefaction effects may be more economical than soil improvement mitigation methods. However, structural mitigation may have little or no effect on the soil itself and may not reduce the potential for liquefaction. With structural mitigation, liquefaction and related ground deformations will still occur. A competent licensed structural engineer that is familiar with seismic design principles and has an understanding about liquefaction effects should design the structural mitigation. The structural mitigation should be designed to protect the structure from liquefaction-induced deformations, recognizing that the structural solution may have little or no improvement on the soil conditions that cause liquefaction. The appropriate means of structural mitigation may depend on the magnitude and type of soil deformation expected because of liquefaction. If liquefaction-induced flow slides or significant lateral spreading is expected, structural mitigation may not be practical or feasible in many cases. However, if the soil deformation is expected to be primarily vertical settlement, structural mitigation may be economically and technically feasible.

Where the structure is small (in building footprint) and light in weight, such as in single family residential houses, a post-tensioned slab foundation system may be beneficial. A post-tensioned slab should have sufficient rigidity to span over voids that may develop under the slab due to differential soil settlement. Light buildings also may be supported on continuous spread footings having isolated footings interconnected with grade beams. For heavier buildings with a low profile and relatively uniform mass distribution, a mat foundation may be feasible. The mat should be designed to bridge over local areas of settlement.

Piles or caissons extending to soil or bedrock below the potentially liquefiable soils may be feasible. Such designs should take into account the possible downdrag forces on the foundation elements due to settlement within the liquefiable and upper soils. Design must also accommodate seismic lateral forces that must be transmitted from the structure to the supporting soils and displacement demand, due to lateral ground deformations. As there may be a considerable loss of lateral soil stiffness and capacity, the piles or caissons will have to transmit the lateral loads to the deeper supporting soils. Experience from recent earthquakes (EERI, 1990) have shown that battered piles are not effective in seismic conditions and should not be used in general. Floor slabs on grade should be expected to undergo settlements in sympathy with the liquefaction-induced settlements of the ground. If such floor settlements are not acceptable, the floor slabs could be structurally supported on the pile or caisson system.

Subterranean wall structures retaining potentially liquefiable soils may be subjected to substantially greater than normal active or at-rest lateral soil pressures. An evaluation should be made to determine the appropriate lateral earth pressures and structural design for this condition.

It should be recognized that structural mitigation may not reduce the potential of the soils to liquefy during an earthquake. There will remain some risk that the structure could still suffer damage and may not be useable if liquefaction occurs. Utilities and lifeline services provided from outside the structure could still suffer disruption unless mitigation measures are employed that would account for the soil deformations that could occur between the structure and the supporting soils. Repair and remedial work should be anticipated after a liquefaction event if structural mitigation is used.

Quality Assurance

Soil improvement techniques generally use specialized equipment and require experienced personnel. As such, they should be implemented by specialty construction companies with a minimum of 5 years experience in similar soils and job conditions as those considered for the project in question. Minimum quality assurance requirements will vary significantly depending on the technique being implemented.

For dynamic compaction, measurement of energy being delivered to the ground, sequence and timing of drops, as well as ground response in the form of crater depth and heave of the surrounding ground are important quality control parameters. Similarly, the location of the water table and presence of surface "hard pans" could greatly affect the quality and outcome of the densification process. Pore water pressures of an area recently treated should be allowed to dissipate before secondary treatments are implemented.

Vibro compaction and vibro replacement are generally performed with electric or hydraulic powered depth vibrators. When electric vibrators are used, the "free hanging" amperage as well as the amperage developed during construction are strong indicators of the likely success of the densification effort. The equipment should be capable to deliver the appropriate centrifugal force to cause densification. Stone backfill materials should be generally clean and hard with minimum durability index of about 40 (Caltest method 229). When the engineer relies on the stone backfill material to provide reinforcement for vertical or horizontal deformations, the stone should be crushed and have a suitable angle of internal friction. In some cases, computer data acquisition systems may be desired to monitor the depth of the vibrator, stone usage, and amperage developed.

Compaction grouting requires the verification of slump and consistency of the mix, as well as careful monitoring of grout volumes, injection pressures, and ground movement at the surface or next to sensitive structures. Critical projects also monitor pore water pressure and deep ground heave (borros points) development during the compaction grouting procedures. Because grout is typically injected in stages from the bottom up, at each stage a stopping criteria of grout volume, pressure, or heave is followed before proceeding with the next stage. Usage of grout casing with less than 2 inches in internal diameter should be avoided as it could cause detection of high back pressures before sufficient grout is injected. Over injection of grout in a primary phase may lead to early ground heave and may diminish densification effectiveness. Spacing and sequence of the grout points may also affect the quality of densification or ground movement achieved.

In general, the engineer of record or his/her representatives conducts on-site inspection of all the procedures mentioned above. Testing locations are selected at random and tend to be located in the middle of a grid pattern formed by the densification locations. This is somewhat conservative and more realistic average results can be obtained by testing closer to the densification points. To permit pore pressure relaxation, a minimum of 48 to 72 hours after soil improvement is implemented should be allowed for prior to testing.

Soil mixing and jet grouting are also constructed with specialized equipment capable of rate of rotation and lifting rate of the injection ports. The grout or binder may include cement, fly ash, quicklime, or other components and additives designed to obtain the desired strength properties of the mixed soil. The binders are controlled for quality by checking consistency as measured by specific gravity. This is generally checked with mud balance or hydrometer devices. Pumping pressures and rates are designed to achieve production and strength requirements of the product. Installed columns are usually tested by wet sampling, coring with a minimum 3-inch core, CPT, pressuremeter, or seismic devices. Variation in quality and strength should be expected in the final product.

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Performance of Soil Reinforcement Systems in Earthquake Zones

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Abstract

Soil reinforcement systems have been increasingly used over the past decades representing cost-effective construction and retrofitting solutions in earthquake zones with significant performance advantages as compared with traditional systems. The use of soil reinforcement technologies can provide a high strength but more ductile and flexible structural elements. Furthermore, installation techniques can effectively be used with the advantageous of efficient load transfer with minimal displacement for retrofitting and underpinning in seismic zones. This paper illustrates and summarizes the benefits and current developments of using such systems including soil nailing, and micropiles.

1 INTRODUCTION

The catastrophic impacts of earthquakes on urban civil infrastructure systems and networks instigated yet the need for innovative approaches to retrofit, rehab, and mitigate earthquake hazards (*NSF 9836*). Selection of retrofit and ground improvement technologies is generally based upon the economic impacts, environmental consequences, and counter measurements of damaging events. During the past decades post-earthquake observations on in-situ ground reinforcement systems, such as soil nailing and micropiles, consistently demonstrated that due to their composite behavior and energy absorption capacity these systems present high resistance to earthquake loading. The inherent advantages of ground reinforcement systems include high strength but more ductile and flexible structural elements, efficient load transfer with minimal displacement, and cost effective construction and installation techniques specially under areas of difficult access.

Soil reinforcement practice has traditionally preceded theoretical consideration and research. The increasing amount of full scale experiments and performance monitoring of actual structures has provided a significant database for the development and evaluation of design methods, and engineering guidelines. The 1989 Loma Prieta earthquake in the San Francisco bay, the 1994 Northridge earthquake, and the 1995 Kobe earthquake in Japan provided many opportunities for documenting the behavior of ground reinforcement systems in waterfront areas, and along transportation facilities, and through various public and private developments. However, several issues must be considered in order to evaluate the available experiences. First, seismic response is related to a number of factors, such as inertial forces, dynamic pressures, and soil-reinforcement interaction. Unfortunately, there is no specific data that identify directly the relative contribution of these factors (Tatsuoka *et al*, 1996). Second, analysis and design procedures as well as performance of these systems are evident to be uncertain (Bardet *et al*, 1996), suggesting that current design procedures are somewhat conservative. Third, lack of physical data such as centrifuge and shaking table test results, against which the analysis and design procedures can be evaluated. This paper summarizes post earthquake observations, ongoing research, and available numerical studies of soil nailing and micropiles in earthquake zones.

2 SOIL NAILING

2.1. Post earthquake observations

Soil nailed structures are systems that are coherent and flexible, offering inherent advantages in withstanding large deformations and, as illustrated by post earthquake observations (Bara, 1990; Felio et al, 1990; Tatsuoka et al, 1996) they present high resistance to earthquake loading.

Site observations by Felio et al. (1990), after the *October 17, 1989, 7.1 M* Loma Prieta earthquake in California on eight soil nail walls, raised significant interest in the potential use of the technology for construction in earthquake zones. The walls, varying in height from 2.7 meters and 9.8 meters, were the subjects of detailed post earthquake visual inspection, and in some cases, nails were re-tested after the earthquake. None of the walls showed signs of distress even through locations that experienced significant seismic related damage. For example, a 4.6 m high wall located on the University of California Santa Cruz campus approximately 18 km from the earthquake epicenter experienced a horizontal ground acceleration estimated as 0.47 g. Soil conditions at the site consist of a hard clayey sandy silt. Construction of this wall was completed less that three weeks before the earthquake. Prior to the earthquake, some wall footings have also been poured at the bottom of the excavation immediately in front of the wall. The post earthquake inspection revealed significant cracking of the concrete footings. Subsequent pullout testing of nine nails to 150 percent of their design load also indicated no loss of pullout capacity due to seismic activity.

Furthermore, Felio et al (1990) reported that the safety factors of the above eight nailed structures based on the current limit equilibrium analysis are considered lower than unity. They considered that the main reason is the hidden conservation by neglecting effects of the bending and shear resistance of the reinforcement and the rigidity of the facing.

The January 17, 1995 Kobe earthquake, 7.2 M provided many opportunities for documenting the behavior of soil nailed structures. Tatsuoka *et al (1996)* observed that a number of conventional walls were seriously damaged, while seven nailed soil structure performed well (Fujii *et al, 1996*). Observed nailed structures varied in depth between 4.0 and 7.0 m, magnitude of peak horizontal ground acceleration ranged from 0.2 g to 0.4 g.

Of particular interest is the excellent performance of a carefully monitored nail wall (Fujii *et al, 1996*), which support a railway road in an area that severely shaken by the earthquake. Before the earthquake, the slope was excavated to about 5.0 m and exhibited 10cm lateral deformation (*Figure 1*) suggesting that the slope was already near the critical state. At the time of the earthquake, the wall displaced outward about 5 cm showing an overturning mode as observed before and after the earthquake. Although the earthquake did not damage the reinforced slope, a safety factor equal to 0.75 was obtained when applying estimated horizontal ground acceleration.

Tatsuoka *et al (1997)* reported that the effect of flexibility and ductility of the reinforced soil are considered yet the significant performance reasons. The more the structure is flexible, the seismic earth pressure acting on its back becomes smaller, and as the structure becomes more ductile, the design procedure based on maximum ground acceleration and limit equilibrium is too conservative. On the other hand, as the current seismic design procedure of nailed structures are mostly limit-equilibrium based stability analysis, they cannot evaluate the deformation and displacement of nailed structures caused by seismic loads. Therefore, these methods cannot evaluate the effects of flexibility and ductility on the seismic stability of nailed structures.

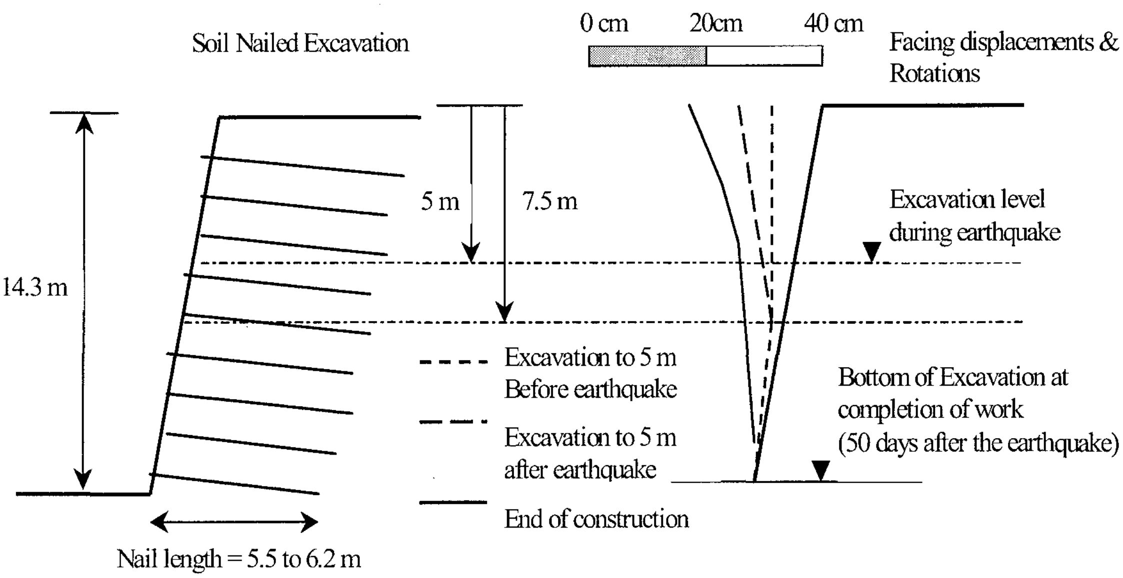

Figure 1. Displacement at the face of a nailed sloped during and after Kobe earthquake.

2.2. Seismic Analysis, Numerical and Experimental Studies

The design methods most currently used for seismic stability analysis of soil nailed systems are derived from the pseudo-static Mononobe-Okabe analysis. Two fundamentally different pseudo-static design approaches have been developed: (i) limit equilibrium analysis (Schlosser, 1983; Koga et al, 1988; Calterance; 1990) which yields only a global safety factor with respect to a rotational or transitional failure of the reinforced soil mass and/or the surrounding ground along the potential sliding surface; and (ii) the working stress analysis using empirical correlations (Richardson and Lee, 1975), or numerically derived design assumptions (Seed and Mitchell, 1981; Dhouib, 1987; Bastick and Segrestin, 1989) to evaluate the seismically induced forces in the reinforcements. Displacement methods have also been incorporated in global limit equilibrium analysis (Bathrust and Cai, 1995), extending the sliding-block theory proposed by Newmark (1965) to predict the permanent horizontal displacements that may accumulate at the base of the structure during seismic events.

The pseudo-static limit equilibrium methods extend available limit force equilibrium analysis in order to assess the seismic loading effect on the global safety factor. Both the Caltrans (1990) SNAIL and French (Schlosser, 1983) TALREN programs take into consideration different design assumptions. Assumptions are related to, (i) the type and magnitude of the applied seismic forces, including the dynamic (FD – dynamic force applied to the reinforced soil by the retained embankment), the inertia force (Fi – dynamic force due to the acceleration of the potential sliding mass limited by the locus of maximum tension forces in the reinforcement) or a combination of both; (ii) the geometry of the active zone under seismic loading, which controls the inertial force.

Table (1) presents the various design assumptions presently used in the different pseudo-static methods. Here S/H is the normalized width of the active zone, H is the height of the wall, S is the horizontal distance from the wall facing to the point of maximum tension force in the reinforcement during a dynamic event. γ, is the unit weight of soil and A, is the maximum wall acceleration coefficient at the centroid.

Table 1. Various pseudo-static design assumptions

Design method	S/H	Inertia Force, F_i	Dynamic Force, F_D
Seed and Mitchell (1981) *AASHTO, 1996*	0.50	$0.50A\gamma H^2$	$\lambda_1(3/8)A\,\gamma H^2$
Dhouib (1987)	0.30+A/2	$\frac{(0.30+A/2)\,A\,\gamma H^2}{4(2K+3)}$	Not applicable
Bastick and Segrestin (1989)	0.30	$0.20A\,\gamma H^2$	$\lambda_2(3/8)A\,\gamma H^2$

Where, $K = 2.5A$; λ_1 =assumed 0.50; , λ_2 =assumed 0.60

The limit equilibrium methods provide only a global safety factor with respect to the shear strength characteristics of the soil and/or the pullout capacity of the reinforcements. They do not allow for an estimate of the seismic loading effect on the maximum tension and shear forces generated in the nails, and therefore cannot be used to evaluate the local seismic stability of the nailed soil at each reinforcement level.

The KADRENSS working stress analysis code developed by Juran and Elias (1991) was extended (Choukeir et al, 1997) to allow for seismic pseudo static stability analysis of soil nailed retaining systems. The basic assumptions considered in this analysis imply that the seismic loading effect can be represented by a pseudo static self-weight inertia force due to the horizontal acceleration of the potentially sliding active zone limited by the locus of maximum tension forces in the nails. This pseudo static inertia force is equivalent to a uniform horizontal earth pressure acting along the potential sliding surface in the soil nailed mass. For a given earthquake acceleration, its magnitude is therefore directly related to the geometry of the active zone determined from the kinematical working stress analysis.

For dynamic loading conditions, corresponding to those during an earthquake, only limited studies have to date been conducted to gain insight into the behavior of soil-nailed excavations. Of particular interest with this regard is the centrifugal model studies conducted by Vucetic et al. (*1993, & 1996*). Four centrifuge models were conducted and analyzed, the models corresponds to 7.6 m deep prototype soil-nailed excavation with three horizontal rows of nails constructed in partially saturated soil (*Figure 2*). The length of nails and their rigidity varied between the models. All four models were subjected to a series of consecutive dynamic loading test with different magnitude of accelerations, namely, 0.1g, 0.28g and 0.43g. *Figure 2* sketches the failure pattern, which includes three main zones (zone I, II, &, III) and two failure surfaces. Zone I is a coherent reinforced mass " relatively rigid", while zone II is relatively sheared zone due to the high base shear stresses resulting in a curvature failure surface, and zone III is the common active wedge developed behind the coherent mass (zone I).

The pseudo static working stress analysis approach was evaluated (Choukeir *et al, 1997*) through the comparison of pullout failure simulations with experimental observations on centrifugal soil nailed model walls conducted by Vucetic *et al., (1993)* and numerical model simulations conducted by Choukeir *(1996). Figure (3)* illustrates the comparison of predicted and measured failure geometry in the centrifugal soil-nailed model walls for different acceleration levels of a_m/g = 0. 1, 0.28, and 0.43. This comparison illustrates that, the method predictions agree fairly well with the experimental values of L/H and with the numerical test simulations. The low L/H values obtained for a/g=0.1 can be probably related to the effect of the experimental technique (Choukeir *et al, 1997*).

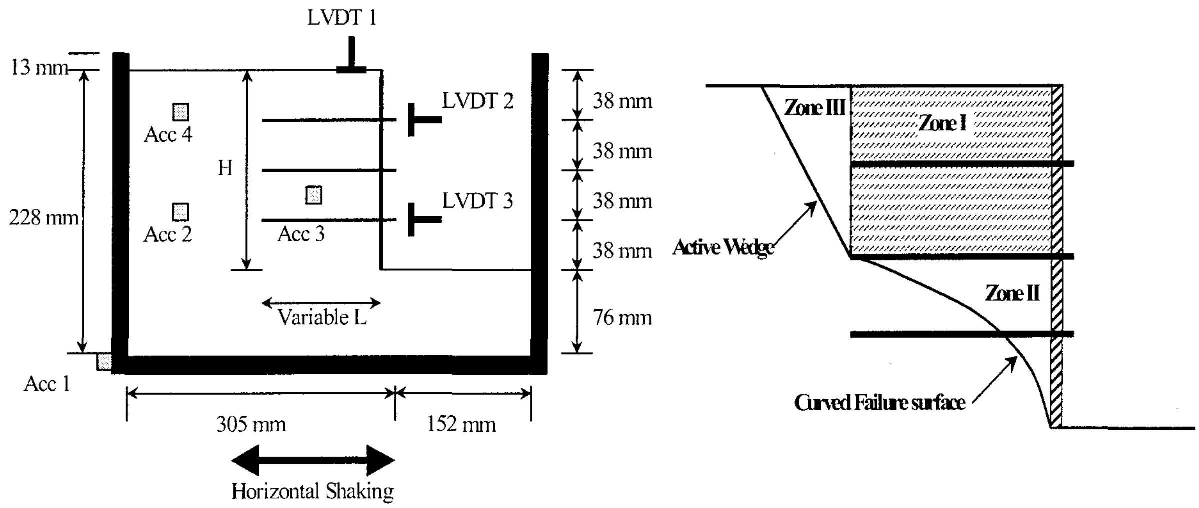

Figure 2. Configuration and failure mechanism of Centrifuge Model Wall (Vucetic et al., 1993)

Figure (4) shows comparison between available pseudo-static design methods with the centrifugal test results conducted by Vucetic *et al, (1993)* with regard to the geometry of the pullout failure. The comparison raised a number of issues with regard to design methods. First, pullout failure envelope increases linearly with maximum acceleration coefficient. Second, current design methods over predict the experimental pullout failure. Third, among the available design methods, Bastick and Segrestin *(1989)* and Kinematical methods tend to be less conservative then *AASHTO, 1996* guidelines.

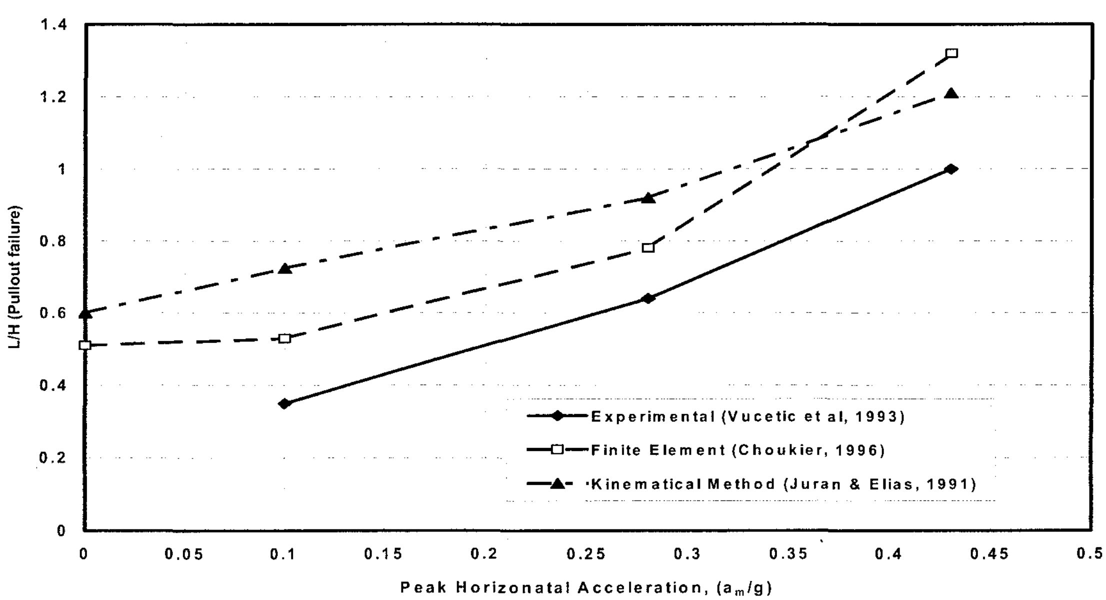

Figure 3. Experimental and Numerical Results of Pullout Failure (L/H)

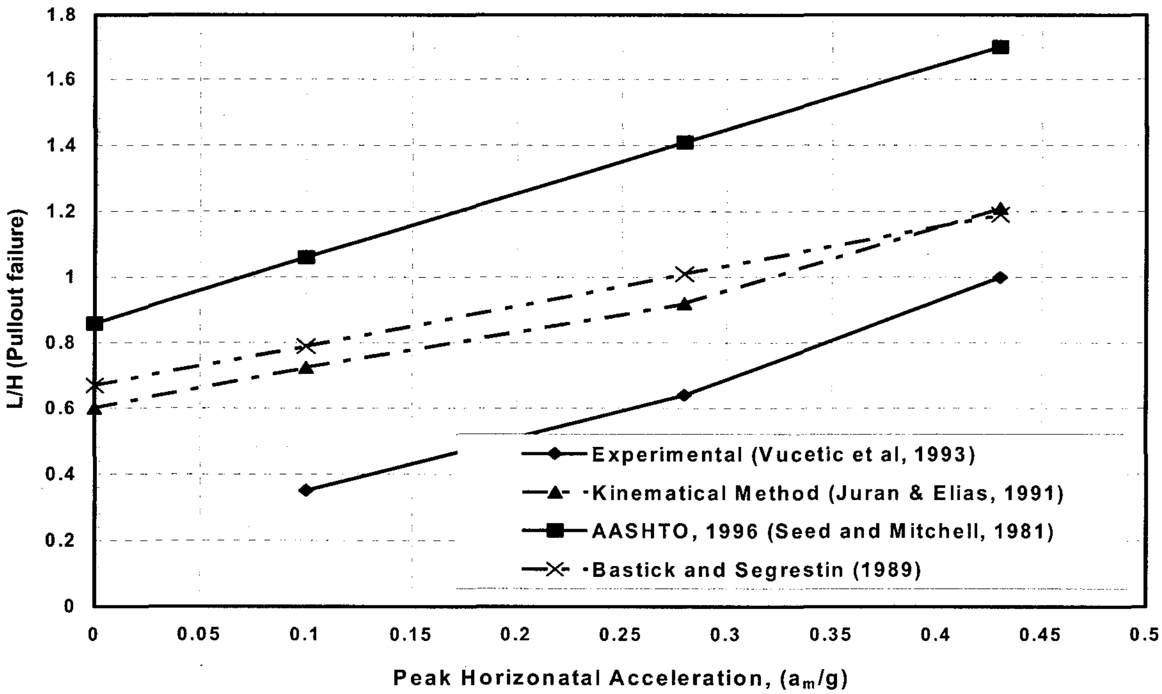

Figure 4. Comparison between experimental results and design methods for pullout failure

At this time, it should be emphasized that while the centrifuge offers effective tools for modeling actual working load conditions it raises R&D challenges with regard to the difficulties involved in complying with scaling requirements, controlling the effect of boundary conditions for seismic loading, and simulating in flight construction processes and installation of reinforcements. However, at this stage further centrifugal model studies on instrumented soil nailed wall as well as performance monitoring of full scale structure are required in order to establish a relevant database for the development and evaluation of reliable seismic design methods for soil nailed structures.

In particular the seismic loading effect on nail pull out resistance needs to be investigated. The potential use of innovative nail installation technologies in earthquake zone raises pertinent questions with this regard. Recent examples for such innovative construction technologies include Jet Nailing (Louis 1986) which combines vibro-percussion driving with high pressure jet grouting, and Nail Launching (Ingold & Miles 1996). The use of such innovative technologies raises the need to investigate the effect of the installation process on soil nail interaction under both static and seismic loading in different types of soils.

3. MICROPILES

Micropiles are defined as small diameter drilled and grouted piles. High capacity steel elements is used as the principle load bearing element, with the surrounding high grout/ground bound serving as load transfer, primarily by friction, to the surrounding soil. Micropiles has been sub classified according to the diameter, construction process or the nature of the reinforcement. However, in the course of the US Federal Highway Administration (FHWA) studies (Bruce and Juran, 1997). It has been concluded that a new, rigorous classification system for micropile design should be adopted based on two criteria: i) the method of grouting, ii) the philosophy of behavior, which dictates the basis of the overall design concept.

Based on the philosophy of behavior, two basically different design concepts which are illustrated in figure (5) have been developed (Bruce et al, 1997) for engineering practice of micropiles, namely:

Case 1: referring to micropiles, which are designed to transfer structural loads through soft or weak soils to more competent strata. *Case 2*, referring to Lizzi's (1978) original "root piles" design concept, relies primarily on using three dimensional networks of reticulated friction piles to create in-situ coherent, composite, reinforced soil systems. According to this concept, the piles are not designed to individually and directly support the load, but rather to circumscribe and internally reinforce the in-situ soil, forming a composite gravity structure to support the applied load with minimal displacement.

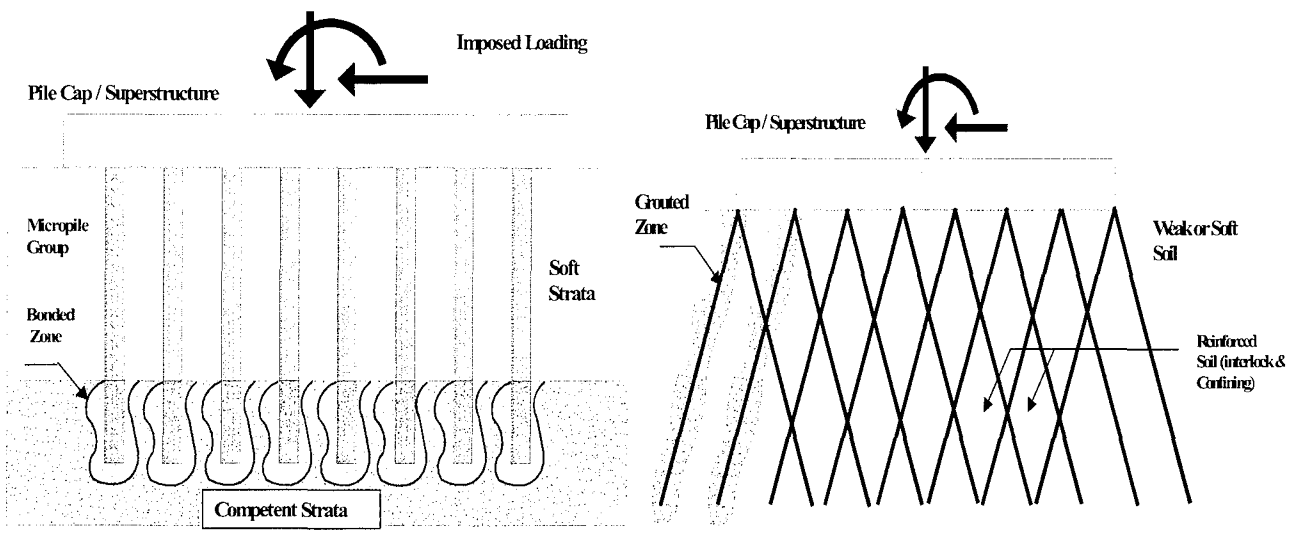

Figure 5. a) Case 1- Micropile Group as direct support *b) Case 2 – Micropile Networks*

Various authors (Herbs, 1994; Maison, 1993; Pearlman et al, 1993; Lizzi and Carnavle, 1981) have indicated that micropile systems appear to provide an innovative and reliable engineering solutions to earthquake disaster mitigation either for seismic retrofitting of existing infrastructure facilities or as a ground improvement technique for the construction of new facilities. Furthermore, post earthquake observations from Loma Prieta earthquake (Bardet et al, 1996) and Kobe earthquake (Gazetas and Mylonakis, 1998) illustrated that significantly lower damage rates occurred with structures supported with battered piles, suggesting the efficient use of reticulated micropile networks for seismic construction and retrofitting. Observations on small and large diameter piles after Kobe earthquake (Tokimatsu et al, 1996) suggest that the steel pile pipes perform better than reinforced concrete piles because of their better ductility. Micropile systems present, therefore, specific advantages, as the micropile is a very flexible structural element due to its slenderness and its ductile steel core.

In the Untied States the 1989 Loma Preita earthquake urged CALTRANS to undertake the investigation of the uplift capacity of micropiles to assess their seismic performance for engineering use in new and retrofitted bridge foundation systems (Mason, 1993). An extensive testing of micropiles was conducted by FHWA/CALTRANS in 1992 and 1993 in San Francisco, California, and as a result micropiles were accepted as an approved CALTRANS foundation option (CALTRANS, 1993) on several sole-source contracts.

Tatsuoka et al, (1997) reported that ten sloped had been stabilized by micropiles in Kobe area where the peak horizontal acceleration varied from about 0.1g to 0.4g. All of the stabilized slopes performed well with minimum damages. Among the above, a 7 m root piled slope located near Suma-Koen railroad station was subjected to 0.4g ground acceleration. Adjacent to the slope, a 3m slope retained by masonry wall completely collapsed in addition to several nearby wooden house were seriously damaged, while the root piled slope showed only hair cracks in the shotcrete facing.

However, the engineering use of micropile systems in earthquake zones requires better understanding of the seismic response of micropiles groups and networks as well as the development of relevant methods to predict the effect of seismic loading on structural displacements. A national research project "FOREVER" is presently being conducted in France in cooperation with the Federal Highway Administration to investigate the behavior of the micropile groups and networks, and establish engineering guidelines for their use in civil engineering applications. Within the framework of this project a cooperative study is conducted to evaluate the seismic response of micropile systems involving cyclic and dynamic calibration chamber tests by the ENPC in Paris, 1 -g shaking table model tests conducted by the University of Canterbury , New Zealand, and centrifugal model tests conducted by polytechnic University in the USA. The prime objectives are to evaluate the seismic response of micropile groups and networks to earthquake loading.

3.1. Experimental Studies

Figure 6 shows the micropile model system tested by Polytechnic University in the centrifuge. The instrumentation involved accelerometers to measure the pile head and free field accelerations in order to characterize the structural response of soil-micropile system, transducers (LVDT's) to monitor soil surface settlements, vertical and lateral pile cap displacements, and strain gauges to monitor both axial forces and bending moments and to assess seismic loading and vibration effect on the soil-micropile interaction.

The experimental program consisted of horizontally shaking the models in flight at 20g, using the Rensselaer Polytechnic Institute (RPI) geotechnical centrifuge facility. The horizontal shaking included sequences of 100 uniform cycles of sinusoidal accelerations at 2 Hz (prototype frequency).

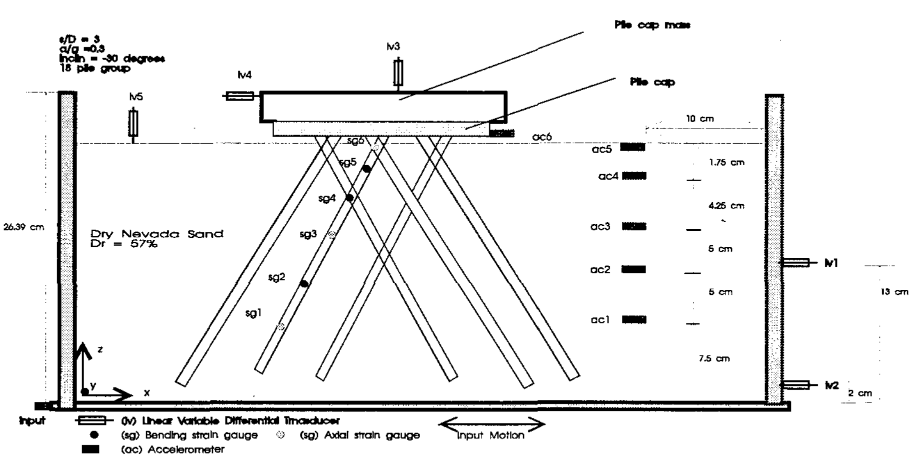

Figure 6. Typical tested micropile network configuration (Juran et al, 1997)

Prototype micropiles were 0.15 m diameter and 5 m length with bending stiffness of 30 $MN.m^2$. The models were first subjected to a prototype acceleration time history with amplitude of 0.3g with cap only and then under 50% and 90% of the estimated static failure load (Weltmann, 1980). Details of testing program and limitations were described by Benslimane et al, 1998 and Juran et al, 2000.

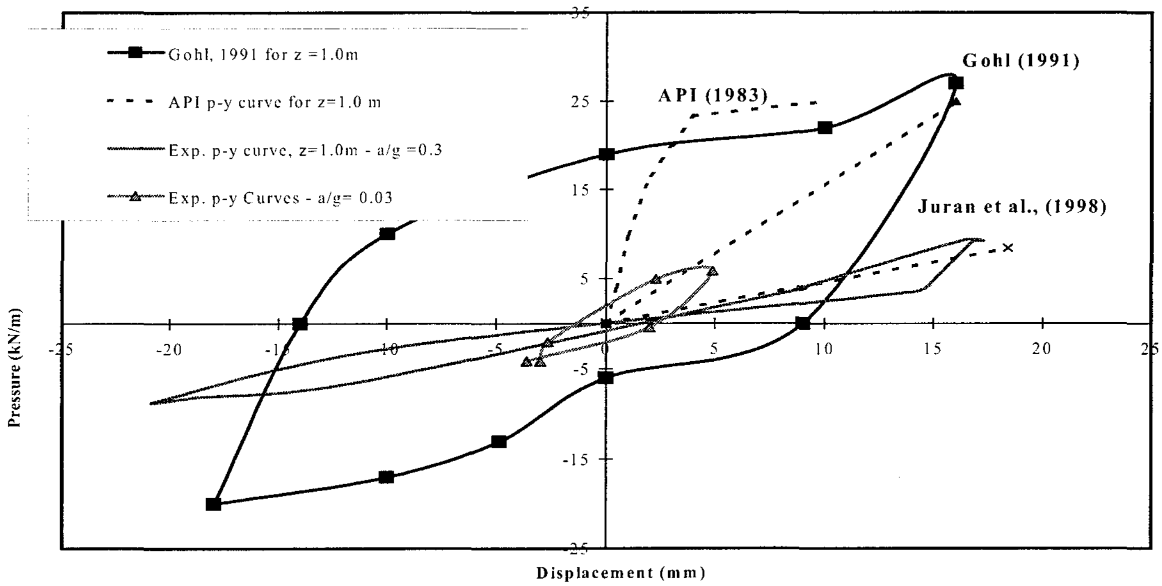

Figure 7. Soil – Micropile Interaction p-y curves under different vibration levels

Soil – Micropile Interaction: Interaction parameters during base motion excitation at different vibration levels were determined using cyclic p-y curves derived from the single pile data using the procedure suggested by Ting (1987). *Figure (7)* compares the computed p-y curves under strong and low level of shaking with the cyclic p-y curves recommended by the existing API (1983) guidelines (using $\phi = 32$ $n_h = 6750$ kN/m^3) and the p-y curves reported by Gohl (1991) based on centrifuge tests on model piles with prototype bending stiffness of EI = 172 $MN.m^2$. It can be seen that for low level of shaking the secant lateral stiffness corresponds fairly well to the results obtained by Gohl (1991). For strong shaking, the API and Gohl (1991) p-y curves are considered stiffer compared to the experimental p-y curves.

Network Effect: The experimental data of 2x1 and 3x2x1 networks with 10 and 30 degrees batter piles, displayed on *figure (8a & 8b)*, illustrate the bi-dimensional network effect results in a decrease of the bending moment as compared with vertical pile as well as in further reduction in lateral pile cap displacement as compared with 2x1 and 3x2x1 vertical pile groups. Furthermore, a substantial improvement is observed in the superstructure response with acceleration reduction to 40% of the values obtained in the case of vertical piles. Nevertheless, a considerable increase in shear and bending moments were observed at pile cap connections as a result of changing the load transfer mechanism from bending moment to axial force. These results strongly suggest that further research is needed on the construct-ability of flexible connections for cap micropile systems.

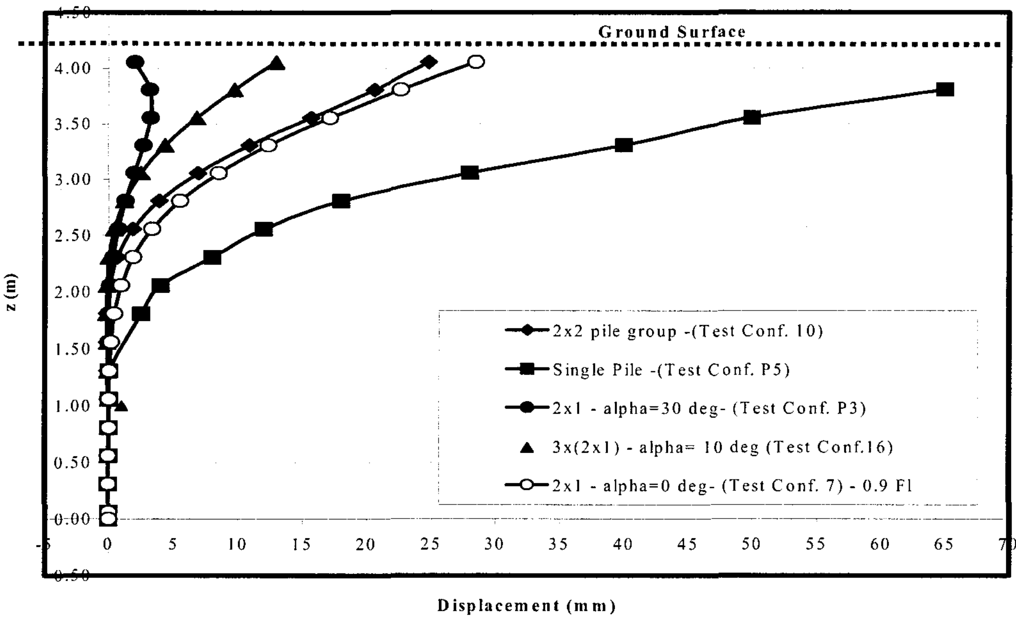

Figure 8a. Experimental results for the recorded maximum micropile displacement at different pile inclination (a/g = 0.3; dia = 0.13m, s/D = 3, and piles networks are subjected to 50% failure load)

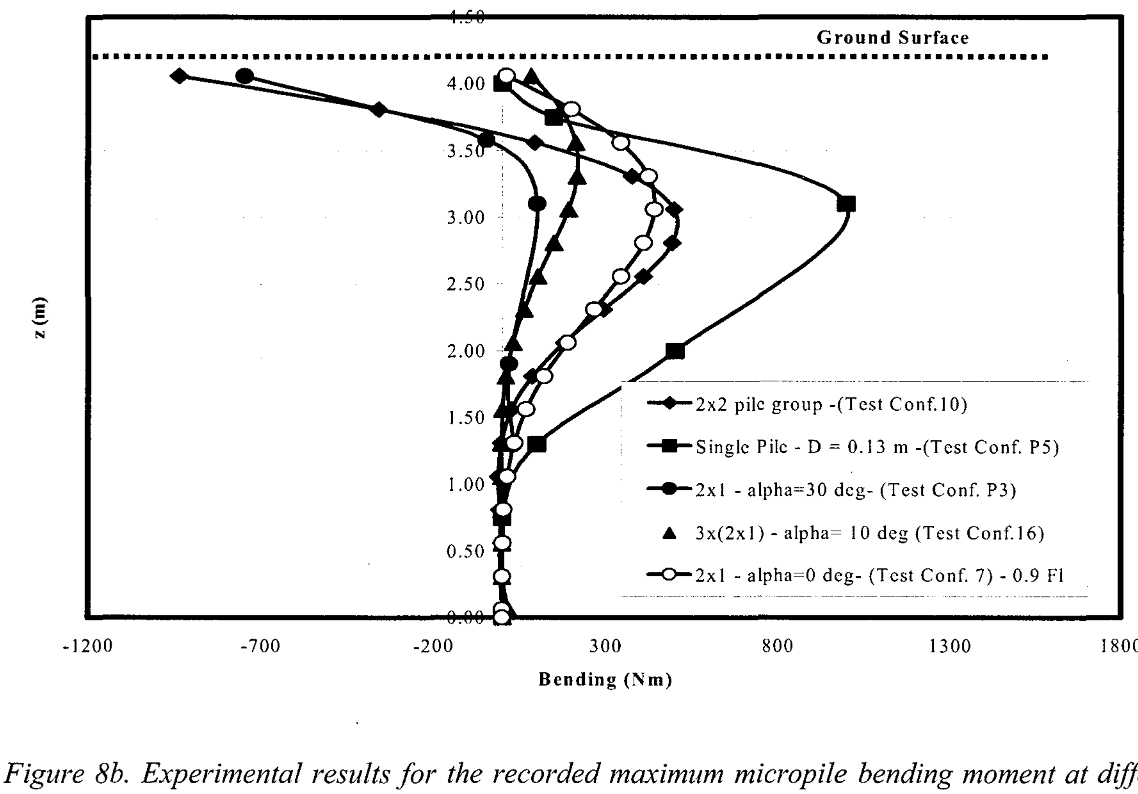

Figure 8b. Experimental results for the recorded maximum micropile bending moment at different pile inclination (a/g = 0.3; dia. = 0.13m, s/D = 3, and piles networks are subjected to 50% failure load)

3.2. Numerical and Analytical Studies

The current state of design and analysis is still primarily based upon the experience and research performed on large diameter drilled shaft piles and ground anchors (e.g. AASHTO, 1996; CALTRANS, 1995). However, the above guidelines appear to underestimate micropiles capacity and performance under static and dynamic loading. One of the main objectives of the on going research is to establish an experimental database for investigating the effect of the main system parameters on the observed seismic performance. For this purpose, pseudo-static analysis methods were used to simulate the experimental results in a quantitative manner. The computer code GROUP developed by Reese and Wang (1994), and well accepted as a useful tool for analyzing the behavior of piles in a group, was adopted.

For this analysis, the experimentally derived p-y curves were used in order to take into account the dynamic soil-pile interaction effect. The maximum unit skin friction derived from the axial load transfer was used to characterize the vertical load transfer. The recorded amplified cap acceleration was used to define the pseudo-static loading. The analysis was carried out using average soil stiffness, k modulus, derived experimentally. K values ranged from 1170 to 5128 kN/m^3

Figure (9) compares the experimentally derived bending moment profile obtained for test configuration P3 (2x1 network, α = 30) under cap loading only and 0.3g base acceleration with the GROUP simulations. The experimental results agree fairly well with the theoretical predictions using the GROUP program.

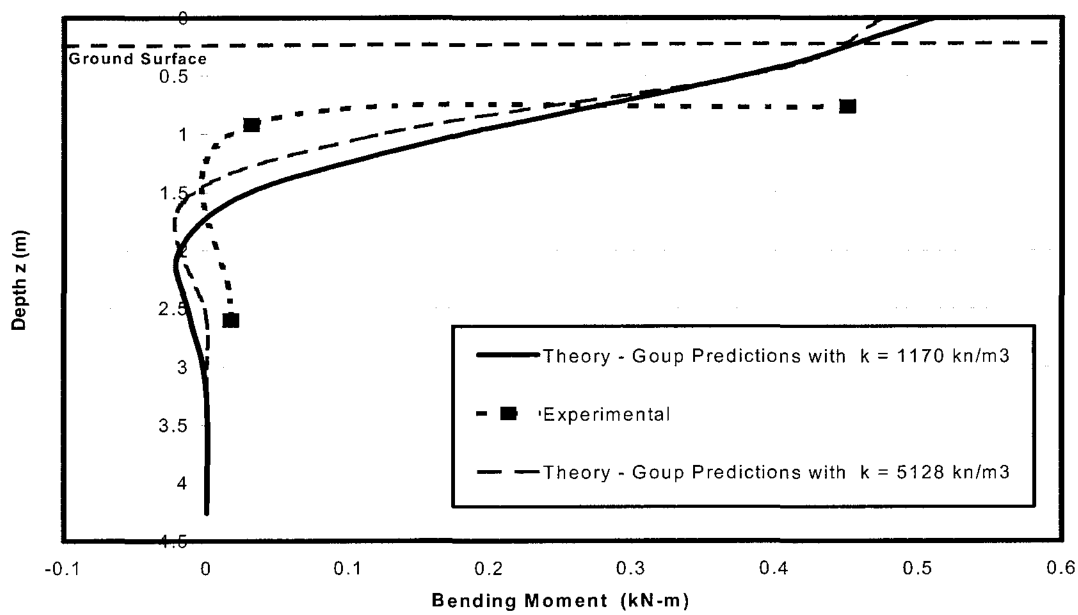

Figure 9. Comparison of the experimental results with the GROUP predictions of pile bending profile of 2x1 pile group system (a/g = 0.3; s/D = 3; cap only)

Comparison between experimental results and GROUP predictions indicate that simplified pseudo-static analysis approach could be used for parametric performance assessment and preliminary seismic analysis provided that soil/pile interface properties defined from the experimentally derived p-y curves. However, figure (10) indicates that for vertical micropiles systems while the experimental bending moment profiles illustrates a positive group effect, GROUP predicts a negative group effect, this is mainly related to a number of reasons. First, the current practice idealizes the surrounding soil using Winkler model in a continuum elastic media. Second, soil stiffness and pile/soil interaction is a function of the exciting frequency and different modes of vibrations. Third, as micropiles are flexible elements, they follow closely the free field displacement profiles, except at shallow depths, resulting in less dynamic forces.

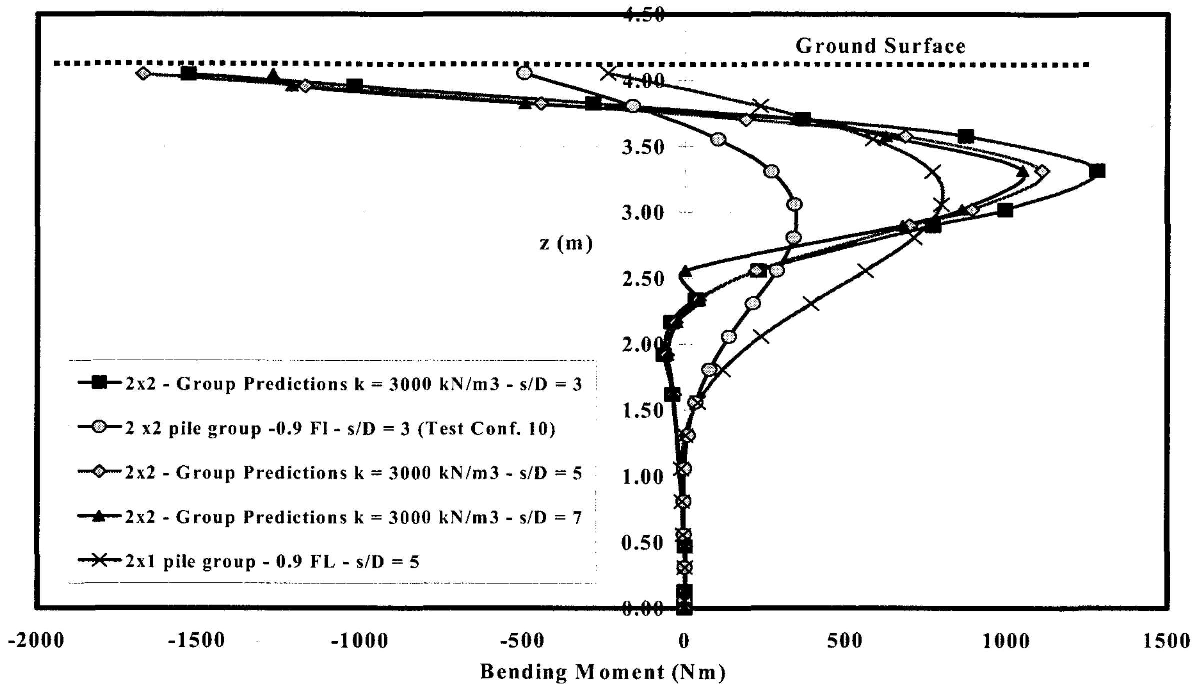

Figure 10. GROUP prediction of the parametric effect of s/D on pile bending moment profile for 2x2 vertical pile group and comparison with experimental data for 0.9Failure Load (FL)

The present study has resulted in the creation of significant database relating to the response of single, groups, and one-dimensional strain plan networks of micropiles to simulated earthquake excitation. Testing methods and interpretation procedures have been presented. However, these preliminary results need to be further investigated in order to develop reliable seismic design guidelines for micropile systems.

ACKNOWLEDGMENTS

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Foundation Liquefaction Retrofit Work - Abstracts

Ignatius Po Lam
Earth Mechanics, Inc.

This presentation summarizes experience from several projects involving retrofit and design analyses conducted to address site liquefaction issues. These projects include:

- Design of pile supported wharfs at port facilities and
- Design and retrofit projects involving typical highway bridges and major water crossing long span bridges.

There are three basic steps involved in the design analysis process:

(1) Liquefaction potential evaluation.
(2) Site stability and ground deformation evaluation.
(3) Soil-structure interaction evaluation, including evaluating the effects of the structure and its foundation on the amplitude of site deformation and its effect on the performance of structural components.

Liquefaction Potential. The-state-of-practice liquefaction potential evaluation generally follows simplified empirical blow count procedures. More complicated effective stress site response analysis methods often would be difficult to verify and generally are not preferred in most situations. They are often adopted only when they give similar results as the more simplified methods. In more complicated soil conditions, involving highly stratified soil layers where the sequence in liquefaction at different soil layers are important to a design problem, effective stress analyses do have the potential for providing more refined solutions and the needed insights to a design problem.

Ground Deformation. Evaluation of ground deformation and their potential implications on constructed facilities is probably the most critical part of the design process. Most often, the conclusion on the ground deformation issue would determine whether site improvement be necessary. Several examples will be presented illustrating analyses conducted for pile wharf facilities where dike deformation analyses were conducted with collaboration with the structural designers. In addition to modeling soil behavior, structural detailing characteristics are included in the model to yield information regarding structural component performance. In some cases, the extent of site improvement is determined to provide the degree in desirable structural performance. Past case histories suggest that constructed foundation systems will reduce the magnitude of ground deformation as compared to the virgin freefield site condition. Some examples incorporating reinforcing effects of the foundation and their reduction in freefield deformation will be presented. A variety of analysis approach, including more complex finite element analysis techniques (involving total and effective stress methods) and more simple Newmark sliding block analyses will be presented and their relative merit will be discussed.

Soil-Structure Interaction. The role of geotechnical engineers most often was to provide geotechnical recommendations in support of structural engineers in design analyses. Two load cases need to be considered in design at a liquefiable site. The first load case is designing against the structural inertial load case (the load case that most structural engineer would be familiar with), while the second load case involves kinematic loading on the foundation/structural system from earth pressure induced from ground deformation. Discussions are presented on the various design issues that often surface in the course of a design process.

Retrofit Innovations. In California, the state has just undergone a very aggressive program to retrofit several thousands highway bridges and seven long-span water crossing bridges. This has resulted in a number of innovations in foundation systems. Some of these advances are discussed.

Technical Block 2: Advanced Technologies for Structural Retrofit

General Overview of Earthquake Engineering Issues for MCEER Hospital Project

Michel Bruneau

General Overview of Earthquake Engineering Issues for MCEER Hospital Project

by Michel Bruneau
Deputy Director, Multidisciplinary Center for Earthquake Engineering Research and Professor, Dept. of Civil, Structural, and Environmental Engineering, University at Buffalo

Abstract

MCEER's research objectives are twofold: to develop seismic evaluation and rehabilitation strategies for critical facilities (hospitals) and systems (electrical and water lifelines) that society expects to be operational following an earthquake; and to develop improved emergency management capabilities to ensure an effective response and recovery following an earthquake.

To ensure that new technologies are continually reviewed for possible addition to the MCEER research agenda, MCEER is conducting a series of workshops on the theme of "Mitigating Earthquake Disasters Through Advanced Technologies' (MEDAT), which are designed to identify new and emerging technologies that can enhance levels of seismic safety. This paper is part of MEDAT-2, focusing on technologies that could be used for the evaluation and retrofit of hospital facilities.

In that perspective, this paper provides a general overview of earthquake engineering issues that must be considered for the seismic evaluation and retrofit of hospitals. It is intended to be informative to engineers from disciplines other than civil engineering.

Introduction

Earthquakes are short duration ground movements that set structures in a vibration motion. As a result of this motion, masses throughout a given structure are displaced and accelerated. Newton's Law (i.e. F=ma) is sufficient to explain, in simple terms, why forces are generated in structures during earthquakes. In that equation, the term "a" is the acceleration felt by the mass. Rigid structures have short natural periods of vibration; their oscillation during earthquakes is of small amplitude, but accelerations are large, which translates to large forces. Obviously, having flexible structures is advantageous in some ways, given that the longer natural period of vibration can translate into smaller accelerations, but the drawback is large displacements, which can result in significant damage to non-structural components and finishes. This is schematically illustrated in Figure 1.

If the inertia forces are small, such as would be the case during a small earthquake, they can easily be resisted by the same structural systems that provide strength against wind-generated loads. Unfortunately, earthquakes frequently generate forces that are significantly more severe than wind. In fact, in most cases, it is recognized that designing structures to remain fully elastic can be prohibitive (unless special technologies are used as indicated later). As a result, for nearly a century, the engineering profession has taken the position that because earthquakes are relatively rare events, structural damage is acceptable in engineered buildings, provided that collapse is prevented. This has been achieved by ensuring that structural components in the systems designed to resist the seismic loads are able to sustain their strength while undergoing large inelastic deformations. This ability to resist large inelastic displacements in a stable manner has been termed "ductility". As a result, the focus throughout the early years of earthquake resistant design has been to establish design rules that make it possible to achieve such ductile behavior. The drawback of this design philosophy, however, is that while making egress of the occupants possible, large inelastic deformations cause major structural and non-structural damage.

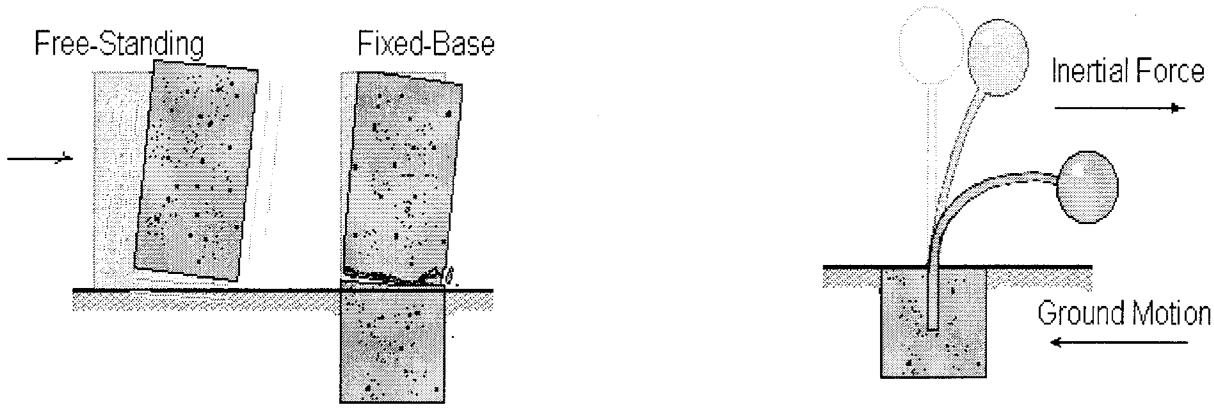


Figure 1: Simple concept of earthquake resistant design (from MCEER's web site)

It has become obvious following recent earthquakes that society has become less tolerant toward the occurrence of this damage (although the willingness to invest in earthquake retrofit measures has not been expressed as firmly yet). Public dissatisfaction with the seismic performance of infrastructure in past earthquakes is on the rise.

This is particularly true for critical post-disaster facilities. In fact, in 1999, a survey[1] was conducted with a random sample of residents in Alameda County, California, where many people experienced the impacts of the Loma Prieta earthquake. Respondents were given a list of 20 types of structures (e.g., public schools, office buildings, apartment buildings, etc.) and systems (highways, water systems, communication systems, etc.), and were asked to identify the five most important structures or systems that must remain functional and operational during and following an earthquake. The three most frequently given responses were: water pipelines and facilities (mentioned by 76% of the respondents); hospitals (76%); and electrical power lines and facilities (73%). The next most frequently given response was the importance of functionality for public safety buildings (48%). It is noteworthy that these four types of structures and systems — identified by the Alameda County residents as the most critical systems that should remain functional — overlap with the three coordinated program areas that MCEER has identified as thrust areas for its research.

[1] "Perceptions of Earthquake Impacts and Loss-Reduction Policy Preferences Among Community Residents and Opinion Leaders" (CMS-9812556), Joanne M. Nigg and Kathleen J. Tierney, Co-Principal Investigators.

Seismic retrofit of existing structures, and in particular of hospitals, has therefore become an important matter, and a number of strategies have been developed over the years for that purpose.

Seismic Retrofit Approaches

In parallel to the discussion above, the traditional approach to seismic retrofitting in earthquake engineering has been to provide new structural members, or strengthen existing ones, to either increase strength or ductility. As part of this trade-off, increasing ductility has often been the preferred approach. It is more cost effective when only the cost of the retrofit measure has been considered. However, when considering the total cost picture, many different types of advanced technologies have found a niche.

The approach using advanced technologies has been not to strengthen the building, but rather to find ways to reduce the earthquake-generated forces acting upon it. This can be achieved in a number of different ways. Base isolation, passive energy dissipation, and semi-active energy dissipation are among the methods that have found various degrees of acceptance and implementation.

Different types of base isolation bearings have been developed and implemented in structural systems in the last decade. Beyond the extensive literature on this topic, a primer outlining the basic concepts, as well as a small tutorial prepared by Professor Constantinou of the University at Buffalo, are available on MCEER's web site at http://mceer.buffalo.edu/infoService/faqs/asdesign.html and at http://mceer.buffalo.edu/infoService/faqs/rsa5_ssi.html, respectively.

Conceptually, flexible bearings are introduced in the structure to increase its period. As a result, nearly all of the structural response develops in these special bearings, which are able to accommodate the very large resulting deformations while dissipating energy to damp-out the seismic response. This is schematically illustrated in Figure 2.

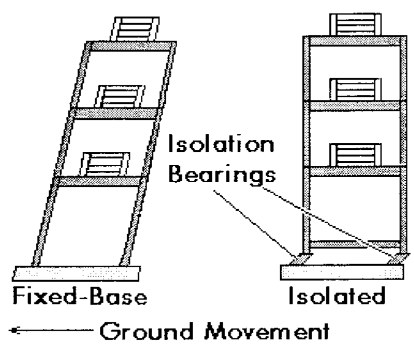


Figure 2: Base Isolation Concept
(from http://mceer.buffalo.edu/infoService/faqs/asdesign.html)

Passive and semi-active energy dissipation systems have been achieved by introducing mechanical devices in buildings. Although buildings possess an inherent ability to damp-out seismically-induced vibrations, this damping is typically small. Mechanical devices are introduced to magnify this damping. In a manner intuitively compatible with everyday experience where dampers are used in many types of applications, the retrofitted buildings undergo significantly lower accelerations and displacements.

Various types of dampers have been considered in the past, namely Friction Dampers (which use frictional forces to dissipate energy), Metallic Dampers (that rely on deformation of metal elements within the damper), Viscoelastic Dampers (that develop controlled shearing of special solids), and Viscous Dampers (that depend on the forced movement (orificing) of fluids within the damper). Structural engineering applications of dampers are illustrated in Figure 3.

Semi-active and active energy dissipation systems are systems that either combine computers and special algorithms that can dynamically vary the amount of damping during response to optimize structural behavior during earthquakes, or use computer-controlled actuators to displace the structure to counteract the seismically induced inertia forces.

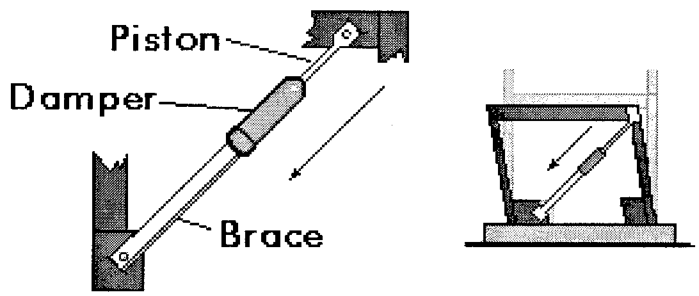


Figure 3: Schematic of Damper Introduced in Structure
(from http://mceer.buffalo.edu/infoService/faqs/asdesign.html)

Building on this philosophy, other approaches using advanced technologies have been proposed and are described in the following section.

MCEER Program 2: Seismic Retrofit of Hospitals

The objective of MCEER's Program 2 is to identify, explore and develop advanced technologies for the rehabilitation of hospitals, to meet or exceed the high level of performance expected of these facilities. The research effort addresses both the seismic retrofit of these facilities, as well as cost or social impediments to their implementation. Engineering and social science researchers work together to identify these impediments and to develop strategies to enable implementation.

The main thrust areas of the program are structural, geotechnical and nonstructural components. The following paragraphs describe current research using terminology introduced in a companion paper in these proceedings, "Overview of MCEER's Vision, Mission and Strategic Plan."

Technology Portfolio: Structural

The technology portfolio, as any other financial portfolio for example, must contain a balanced mix of distinct low-risk, moderate risk and high-risk technologies. Low-risk technologies are defined as those being near implementation and having a high probability of acceptance. Moderate risk pertains to those technologies with a mid-term range for implementation and having a moderate to high probability of acceptance. High-risk technologies, as expected, are those defined as having long-term implementation expectations and moderate probability of being accepted. Together, these technologies provide the necessary diversity to tackle the complex loss reduction problem, and the flexibility required to rapidly adjust the research directions. For instance, when a new earthquake occurs, perceptions and long held beliefs about what previously constituted an acceptable solution may change, and the research program is structured to shift directions, if need be, based on these new discoveries.

The Structural Technology Portfolio is currently invested in the following technologies: passive control technology (dampers and base isolation systems, metallic passive energy dissipation approaches); and variable passive control technology (passive energy dissipation using new composite materials and hybrid systems).

The approach taken is consistent with MCEER's philosophy that advanced technologies may be the most effective way to achieve the stringent performance levels mandated in the retrofit of critical facilities. Although all focus on the concept of passive energy dissipation (which seems to be the most promising at this time), the above technologies can be divided in two distinct approaches: use of specific manufacturer-produced devices, and use of infill materials. In each case, technologies at the various risk levels are being considered.

The first approach, using manufacturer-produced devices, strives to provide satisfactory seismic performance by introducing discrete control devices. The intent is to provide the highest level of control without the need for repairs or replacements following a major earthquake. Because there is no consensus on how this can best be achieved, other new technologies are being developed to modify the existing solutions, enhance their effectiveness, provide better overall control, and ensure fail-safe mechanisms. Such new technologies must be aggressively investigated as they may provide the best solution, in some instances the only solution, to achieve the high level of seismic performance sought for critical buildings in regions of high seismicity.

The second approach, focusing on the use of infill panels, may be particularly suitable to regions where the implementation of advanced devices is less probable, due to lack of earthquake awareness, or to engineer preference for material-based solutions over device-based approaches (for various reasons). All the infill systems studied within this task could be implemented (among many possible solutions) without reinforcing the beams and columns in an existing building. This makes them particularly interesting solutions in the perspective of a hospital retrofit for two reasons. First, they greatly reduce the cost of retrofits. Second, and perhaps more importantly, recognizing that hospitals undergo frequent re-organization of floor space usage, these infills could be moved just like the non-structural partitions they would replace (although not as easily and likely not without some input from the engineer). Additional studies must be completed before this full-portability concept can be established and validated.

Technology Portfolio: Evaluation and Retrofit of Liquefiable Soils

A significant problem is that many hospitals have been constructed on soils that are likely to liquefy during an earthquake. While it is relatively easy to consolidate such liquefiable soils in the freefield, there currently exists no simple retrofit procedure that permits geotechnical remediation at minimal or no disturbance to the occupants. The problem is further compounded by shortcomings of the existing analytical models when attempting to model the behavior of pile-foundations on liquefiable soils. The geotechnical research within MCEER Program 2 therefore aims at the experimental investigation of advanced technologies to fill these gaps in knowledge and to concentrate on the development of analytical tools necessary to permit their reliable consideration in seismic rehabilitation work.

In the MCEER program, there is a strong focus on the rehabilitation of deep foundations subjected to liquefaction-induced lateral spread. Experience from earthquakes and centrifuge models have shown the importance of shallow nonliquefiable soil in increasing the forces and moments imposed on pile caps and individual piles. A promising rehabilitation approach is to replace the shallow soil around pile caps by a frangible material that will yield under constant lateral soil forces but remain resilient under transient motion. This strategy is being explored through a series of centrifuge tests, analyses, and comparisons with case histories.

Passive site remediation techniques promise broad application for developed sites where the more traditional ground improvement methods are difficult or impossible to implement (due to problems such as access, disruption of normal site use, and disturbance to existing structures). Research is needed to establish the feasibility of this approach through identification of stabilizing materials, study of how to adapt or design groundwater flow patterns to get stabilizers to the right place at the right time, and evaluation of potential time requirements and cost. The research builds on background knowledge of soil stabilization and grouting, recent developments in contaminated site remediation using reactive barriers and electrokinetics, and studies of pore pressure plume migrations associated with soil liquefaction.

Technology Portfolio: Non-structural Evaluation and Retrofit

The seismic performance of non-structural components is an important issue in hospitals. The survival of the buildings is of no benefit if they must be evacuated because of water damage, or if key emergency care equipment is rendered inoperational due to damage. At a fundamental level, the various equipments must be sufficiently resilient to resist high levels of accelerations and abuse. This problem, however, depends on the inner working of the medical device itself, and is an issue that must be handled by the equipment manufacturers through certification testing and specification development beyond the scope of MCEER's research. Furthermore, a FEMA-funded project is looking at equipment issues and results will become available by 2001 (these findings obviously will be considered in MCEER's research thereafter).

MCEER's research on the seismic performance and rehabilitation of non-structural components focuses on ensuring that the hospital's expensive equipment is not toppled or dropped, and that lifeline non-structural systems do not become dysfunctional as a result of excessive structural response. In particular, emphasis is placed on piping systems as these have been identified as critical components in a hospital, and have exhibited numerous failures in nearly all recent earthquakes.

A major challenge is to determine the conditions that lead to undesirable performances. The path towards resolution of this complex problem has many barriers. First, the key input parameters that impact seismic performance are not well known for the various types of failure and non-structural systems considered, whether these are peak or relative floor accelerations, velocities, displacements, energies, or others. This requires fundamental research work on calibration and sensitivity of fragility curves, as well as experimental testing to establish the performance of various systems (such as equipment rocking, equipment sliding, light and heavy piping systems, large tanks and reservoirs, and other special critical components such as elevators). Second, the most promising advanced technologies to effectively retrofit these systems must be identified and their effectiveness must be analytically and experimentally established. Third, the complex interaction between structural and non-structural retrofit must be understood and optimized. Indeed, some structural retrofits may abate or eliminate the need for many non-structural retrofits, and whether one should focus on expensive technologies to control the structural behavior or invest to comprehensively retrofit the non-structural equipment is an unanswered question. This issue is particularly important given that the available resources and incentives for seismic retrofit vary greatly across the various seismic regions of the country, and that the acceptable levels of seismic performance differ accordingly. The best strategies for trade-offs in seismic and non-seismic retrofit is likewise tied to these constraints. Complex linkages between fragility analyses for both structural and non-structural components must be established.

Facilitating Technologies: Structural Evaluation and Retrofit

Facilitating technologies within the context of MCEER Program 2 are the principles and approaches that must be developed to implement advanced technologies by the architectural and engineering professional community. It is the heart of the system integrated approach concerning the performance of a system (structure with added device and material components), so that cost-effective strategies can be realized by the designers for a given structure and prescribed earthquake risk. The spectrum of research includes some fundamental dynamic responses of structures that may only be studied by multiple DOF models including nonlinear effects on one hand, and some practical issues such as the formulation of simple design guidelines and procedures on the other. In between, a major challenge is the development of user-friendly computer programs that are tools for engineers to realize the conclusion of a cost-effective retrofit strategy.

Hospitals found in the eastern United States generally have a steel structural frame system, the older ones with frames assembled using flexible semi-rigid connections, and the newer ones with more conventional steel frames with rigid connections. Many, when located in dense urban centers, are mid-rise buildings, with 20 story buildings being common in dense urban centers. As a result, these buildings generally have a longer period which attracts lower seismic forces, but typically undergo large drifts during earthquakes, and may therefore suffer from both structural and non-structural damage, and in some instances risk collapse from global instability. This later damage state is particularly difficult to quantify and may require special consideration. The low-rise hospital buildings typically found across the country also rely on flexible frames to resist earthquakes, the greater architectural flexibility afforded by frames having a considerable appeal. There are also, however, a number of hospital buildings that rely on shear walls or braced frames to resist lateral forces, and new technologies must also be developed to effectively address the retrofit needs germane to these more rigid structures.

Conclusions

This paper has provided a brief overview of earthquake engineering issues that must be considered in seismic retrofit, followed by a summary description of selected MCEER Program 2 activities that are relevant to the MEDAT-2 workshop. Note that although most of the issues discussed have focused on structural engineering examples (author's bias), advanced technologies to solve geotechnical seismic deficiencies are also an integral part of MCEER's research agenda.

Technical Block 2A: Materials and Damage Monitoring

Chairs: Michel Bruneau and Jayanth Kudva

Engineered Cementitious Composites for the Retrofit of Critical Facilities
Keith Kesner and Sarah L. Billington

Polymer Matrix Composite (PMC) Infill Walls for Seismic Retrofit
Amjad J. Aref and W.Y. Jung

Pattern Recognition for Structural Health Monitoring
Charles R. Farrar and Hoon Sohn

The Development of a Wireless Modular Health Monitoring System for Civil Structures
Jerome P. Lynch, Kincho H. Law, Erik G. Straser, Anne S. Kiremidjian and Tom W. Kenny

Damping in Structural Applications
Darel E. Hodgson and Robert C. Krumme

Seismic Control Devices Using Low-Yield-Point Steel
Yasushi Maeda, Tanemi Yamaguchi, Toru Takeuchi and Takumi Ikebe

Use of FRP Composite Materials in the Renewal of Civil Infrastructure in Seismic Regions
Vistasp M. Karbhari

Structural Composites with ECC
Gregor Fischer and Victor C. Li

Engineered Cementitious Composites for the Retrofit of Critical Facilities

Keith Kesner, Sarah L. Billington
Department of Civil and Environmental Engineering, Cornell University

ABSTRACT

Engineered Cementitious Composites (ECC), which exhibit pseudo-strain hardening behavior and steady-state cracking are being explored for use in retrofitting critical facilities in seismic regions. The applications will involve protection of both primary structural and non-structural (secondary systems) components. ECC materials offer several advantages for seismic retrofitting such as energy absorption capacity, excellent tensile and compressive strength, and the ability to be cast into various shapes. Structural components made from precast ECC materials have the ability to be connected by welds, bolts or with grout.

The current investigation involves evaluation and development of ECC materials to create an infill wall system for seismic retrofit applications. To develop the infill wall system, a combination of laboratory and analytical research has been initiated. Laboratory tests are used to develop an appropriate numerical modeling approach for the cyclic behavior of the ECC materials. Preliminary results of the investigation are presented at the MEDAT-2 workshop in Las Vegas and are further described in this paper.

Keywords: cementitious composites, fiber reinforced, ductility, seismic retrofit, steady-state cracking

INTRODUCTION

Engineered cementitious composite (ECC) materials represent an innovative fiber reinforced cement-based composite. ECC materials are made up of a Portland cement matrix reinforced with a low volume fraction of fibers such as ultra high molecular weight polyethylene (UHMWPE) fibers. The constituent proportions are based upon micro-mechanical analyses of the interaction of the fibers and matrix [1]. Previous research on ECC materials has resulted in significant theoretical development of the materials from micro-mechanical principles [1,2,3]. Some researchers have begun to examine the use of these materials for structural applications [1,4,5].

This paper presents preliminary results of a larger investigation, wherein the applicability of ECC materials to seismic retrofit and strengthening applications is investigated. The current paper focuses on the initial development of an infill wall system utilizing ECC materials. The initial analysis highlights a need for laboratory testing to evaluate the cyclic behavior of ECC materials. These laboratory results will be used in the development of material models for ECC with the ultimate goal of identifying and experimentally testing infill panel systems for steel and concrete frame retrofits.

BACKGROUND ON ECC MATERIALS

ECC materials are a class of high performance fiber reinforced cement-based composite. They exhibit a pseudo-strain hardening response and steady-state cracking in tension. This behavior is in contrast to the brittle or quasi-brittle nature of traditional concrete and fiber reinforced concrete materials. The development of ECC materials is based on evaluating the pullout behavior of the fibers from the Portland cement matrix. The majority of the initial development of the ECC materials has been by Li et al. [1,2,6].

Fibers are used as a traction force bridging cracks, with the load carried by the fibers increasing with crack extension. Due to the lack of chemical bond between the polyethylene fibers and the Portland cement matrix (the fibers are inert), the stress-crack opening relationship is based solely upon the frictional debonding behavior of the fibers. The toughening effect due to fiber bridging leads to an increase in the composite's first crack strength. Steady-state cracking arises as a result of the balance between the composite toughness increase and the increase in the stress-intensity factor at crack tips during loading and crack extension. An example of the tensile response of prismatic ECC specimens with different fiber types is shown in Figures 1 and 2.

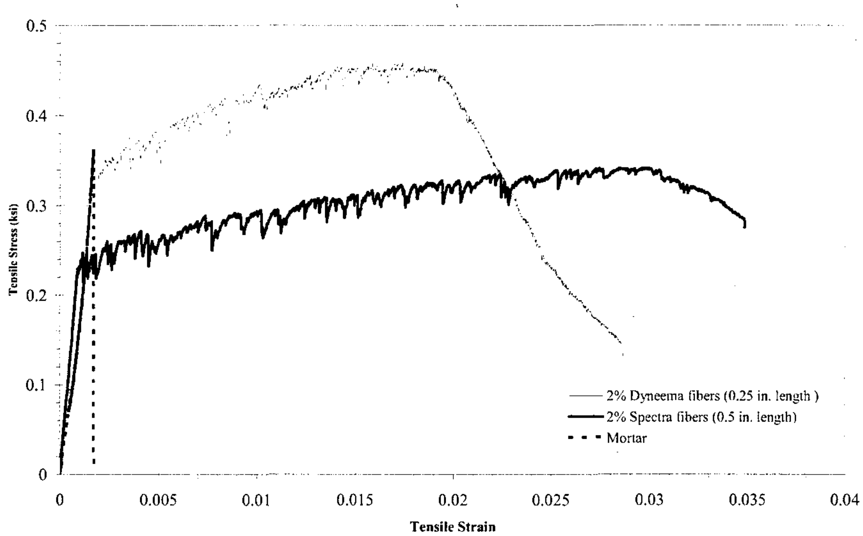
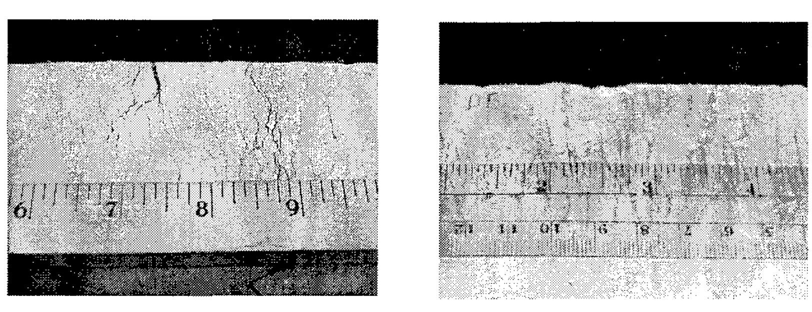


Figure 1 – Comparison of response of ECC made with Dyneema (0.31 mil diam.) and Spectra (1.5 mil diam.) fibers.



Spectra (l/d = 335) Dyneema (l/d = 800)

Figure 2 – Comparison of crack widths with different fiber types (l = fiber length, d = fiber diameter).

APPLICATIONS IN CURRENT RESEARCH

Motivation

The primary goal of the current research is to develop strategies for the seismic strengthening and retrofit of structures using ECC materials. It is believed that the tensile ductility and energy dissipation capabilities of ECC materials will result in more durable and possibly safer seismic performance compared to conventional and typically brittle concrete. Currently several retrofit strategies using infill panels are being investigated, as shown in Figure 3.

The ability of the ductile ECC materials to be connected using simple bolted connections, or as a grout between precast panels represents a significant advantage in terms of construction time and cost compared to traditional materials such as concrete masonry unit (CMU) infills. The scheme of using infill panels of ECC with bolted connections is similar to the preliminary work of Kanda et al. [4]. This scheme offers advantages of being portable and allowing individual panels to be replaced or removed as needed. The scheme of ECC as a grout for precast concrete infill panels builds on the work of Frosch et al. [7]. In the research by Frosch precast concrete infill panels are used in conjunction with supplemental vertical post-tensioning to increase the lateral load capacity of a reinforced concrete frame.

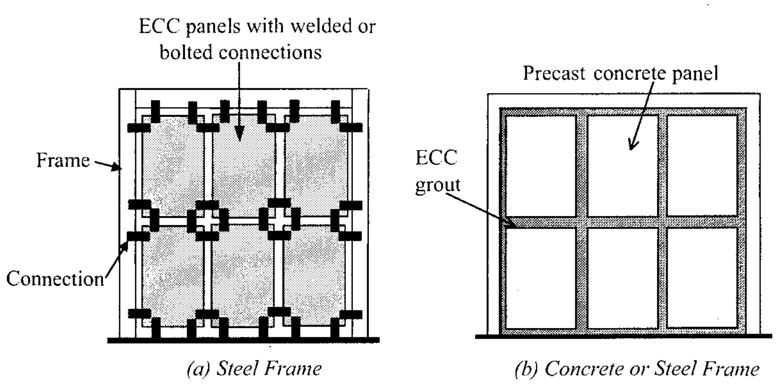
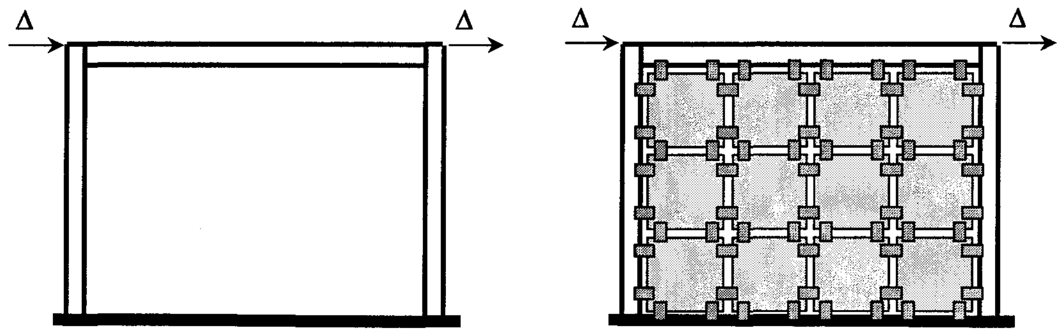


Figure 3 – Schematic representation of frame retrofits using ECC.

Preliminary Analysis

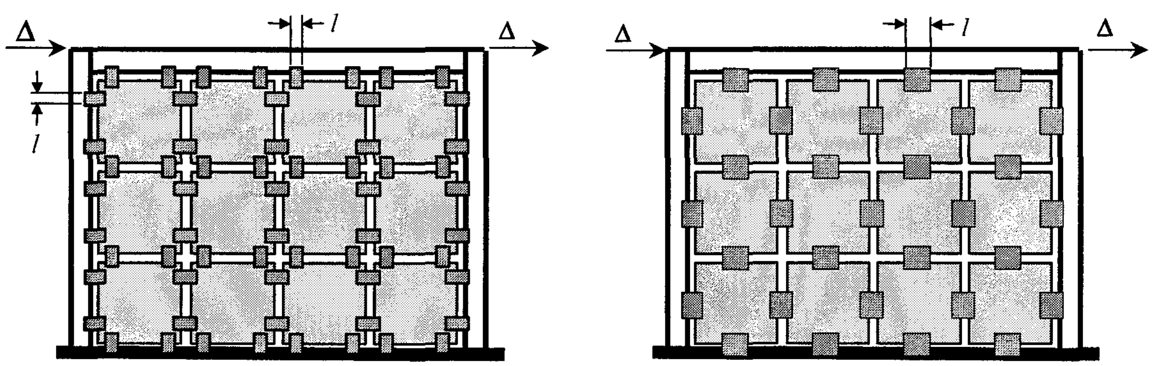
The current investigation focuses on developing a strengthening scheme for the steel framed structures, as shown in Figure 3(a). To strengthen/stiffen the frame, ECC panels are added as an infill wall system as shown in Figure 4. The geometry of the infill wall panels is selected to minimize the number of panels in the frame, while maintaining the portability and flexible use of the system. A preliminary selection of 48" by 48" by 4" thick panels is chosen. This size yields an approximate panel weight of 600 lbs. The concept of the connections between panels is shown in Figure 5. To examine the effect of varying connection tab widths, two different center tab widths (6" and 12") are evaluated. Only one corner tab width (6") is evaluated. Alternatively, a single connector between two thinner panels could be used similar to the work of Kanda et al. [4]. The thickness of the steel connection tabs is ½".

The frame geometry analyzed in the current study is selected from a steel framed hospital located in the northeastern United States. Table 1 shows the geometry of the frame members. In the analysis all beam-column connections are considered to be rigid. Additional studies are underway to develop optimal strategies for panel size and connection geometries.



(a) Bare frame *(b) Frame with infill panels*

Figure 4 – Bare frame (a) and frame with infill panels (b).



(a) Corner tab connections *(b) Center tab connections*

Figure 5 – Infill panel connections geometries (l = tab width).

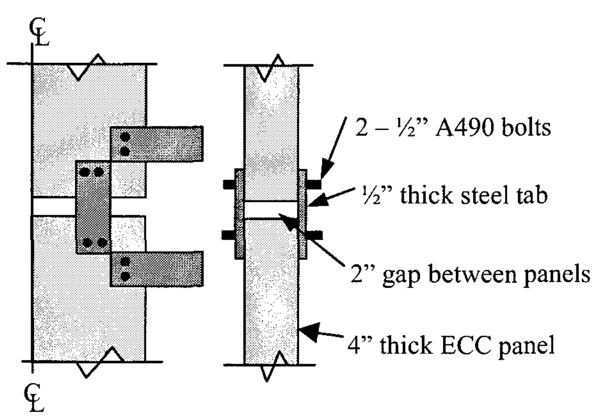


Figure 6 – Infill panel connection option.

Table 1 – Frame member properties

Member	Moment of Inertia (in^4)	Area (in^2)	Depth (in)
Top Beam	1221	39.11	12.88
Columns	446	10.59	15.85

To evaluate the effect of the infill panel addition, a finite element model is created using the Diana finite element program. The frame members are modeled using 2-noded beam elements. The infill panels and steel connections are modeled with 4-noded plane stress elements.

The material models used in the analysis are shown in Figure 7, and the properties are summarized in Table 2. For the steel frame and connection members an elastoplastic material model with isotropic hardening is used. For ECC, an equivalent uniaxial strain model is used for tension and compression. In tension, a multilinear stress-strain relationship is used wherein; the transition points on the stress-strain diagram are obtained from uniaxial tension tests. The tension model used in the current analysis is based upon a total strain fixed-crack model.

For cyclic behavior, secant unloading and reloading is used in both tension and compression. The accuracy of the secant-unloading scheme is a significant area of research in the current study. Material testing (presented later) is being used to develop an improved model for the unloading and reloading of the ECC material.

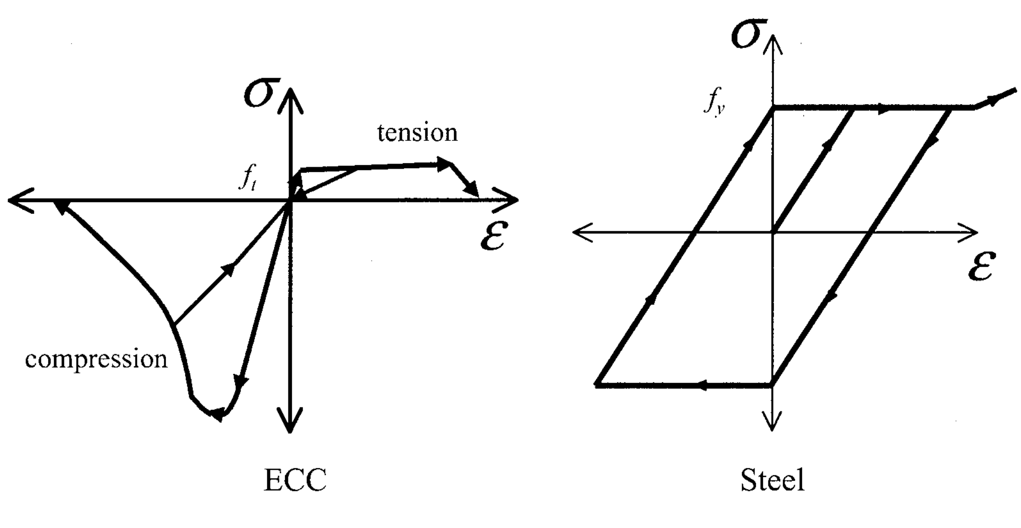


Figure 7 – Material models used in preliminary analysis.

Table 2 – Elastic material properties

Material	Modulus of Elasticity (ksi)	Yield/Cracking Strain	Yield/Cracking Stress (ksi)	Ultimate Strain
Steel	29,000	0.0012	36	0.1
ECC	2,000	0.00015	0.30	0.03

To evaluate the effect of the infill wall addition, a cyclic displacement of the frame to two different drift levels is simulated. The frames are cycled to +/- 0.25%, and +/-0.50% drift. Figure 8 shows the load-displacement response. The results show a significant increase in load capacity and energy dissipation in the frame with the infill panels. The type and length of connection between panels affected the magnitude of lateral load capacity increase. Examination of the results indicated that the load in the frame columns is below the plastic moment capacity of the columns at the peak drifts. The results indicated that some yielding occurred in the tab connections. With further investigation of panel and connection details, a variety of infill wall systems should be able to prevent yielding of the frame, increase the lateral load capacity and stiffness of the frame, and increase the energy dissipation in the structural system.

The largest increase in lateral load capacity, compared to the bare frame, occurred with the 12" center tab connections. The capacity, at +0.50% drift, with the 12" center tabs is approximately 110 kips higher than the frame with the 6" corner tab connections which had the same total steel area (2 - 6" wide connections), and 250 kips higher than the frame with the 6" center tab connection. This difference in capacity between connection types indicates that both the connection location and length will need to be evaluated in the ongoing research program to develop optimal strengthening and stiffening solutions.

The strains in the majority of the panels are above the cracking strain (150 μstrain) in the ECC material. However, the load-displacement response, as seen in Figure 6, does not indicate a decrease in load carrying capacity at this drift level. Due to the pseudo-strain hardening behavior of the ECC materials, the capacity of the system does not appear to have decreased at this drift level.

Preliminary analysis indicates that the infill panel system can be used as a seismic retrofit strategy to strengthen a steel frame. Connection details (length and location) between the panels, and to the frame are an important variable in the determining the system capacity in terms of strength and stiffness. In the current study the size of the infill panels is kept constant. In the ongoing research, the effect of different panel sizes and varying connection details are being evaluated.

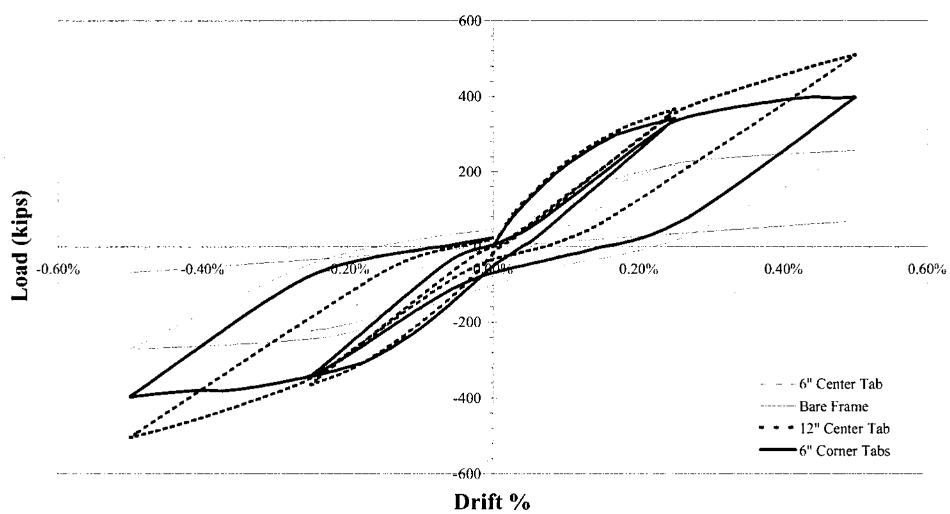


Figure 8 – Load displacement curves from preliminary analysis.

CYCLIC TESTING OF ECC MATERIALS

Motivation

One of the primary goals of the current research is to develop an improved constitutive model for the cyclic behavior of ECC. In the preliminary analysis of the infill wall systems a secant-unloading scheme is used to model the unloading behavior of the ECC, as shown in Figure 7. To develop an improved model for the unloading and reloading behavior of ECC, laboratory tests have been conducted. The development of an improved model will allow for finite element based simulations of ECC panels in strengthening and retrofit applications.

Experimental Program

Laboratory tests are being conducted on cylindrical and prismatic specimens. The different specimen geometries are selected to allow for direct comparisons with results from uniaxial tension and uniaxial compression tests. Figure 9 shows a 2" by 4" cylindrical test specimen in the test frame. Two LVDTs are used to monitor the displacement of the specimen during testing. Several different loading schemes with combinations of tensile and compressive strain limits are used to evaluate the cyclic behavior of the ECC. All tests are conducted under displacement control in a 50-kip testing frame. A strain rate of 0.1% per minute is used. As seen in Figure 10, a swivel joint is located at the top of the specimen to minimize bending stresses in the specimen during testing.

Testing Results

Figure 10 shows one of the results from the cyclic testing program. Here the loading scheme consisted of alternating tension and compression loads. The scheme is selected to approximate previous cyclic tension/compression tests used in the development of a cyclic tension model for concrete materials [8]. The specimen is loaded to predetermined tensile strain levels. After the predetermined strain level is reached the specimen is unloaded, and then loaded in compression. To insure complete closure of cracks in the specimen it is necessary to increase the peak compressive stress in each test cycle. The upper tensile strain limit is reached when the ECC material begins to soften in tension. After this tensile strain limit is reached the specimen is unloaded, and then loaded in compression to failure.

Examination of the test results indicate that the unloading and reloading behavior of ECC is more complicated than the simple secant unloading used in the initial analyses. In Figure 10, three distinct regions can be seen in the unloading portion of the curve. These regions (shown in italics) are the initial elastic unloading, the crack closing with low compressive stiffness (similar to a slip region), and the increasing compressive stiffness as the cracks in the specimen close and bear compressive stress. The tensile reloading curve, similar to the unloading curve, has three distinct regions (shown in bold in Figure 10). These regions include the initial elastic unloading, crack opening with low stiffness, and the increasing stiffness with increasing levels of crack opening. These, and other similar test results, with different loading regimes, are currently being used to develop a material model for ECC for use in nonlinear finite element analyses.

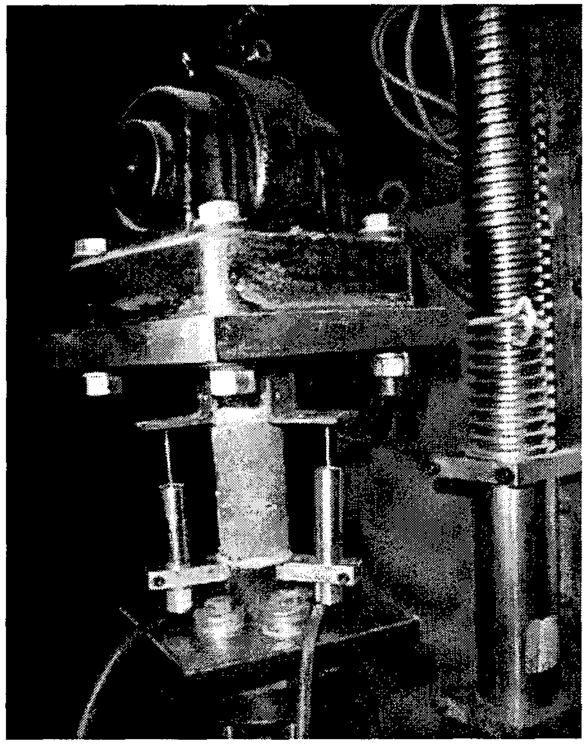


Figure 9 – Experimental set up for cyclic testing of ECC materials.

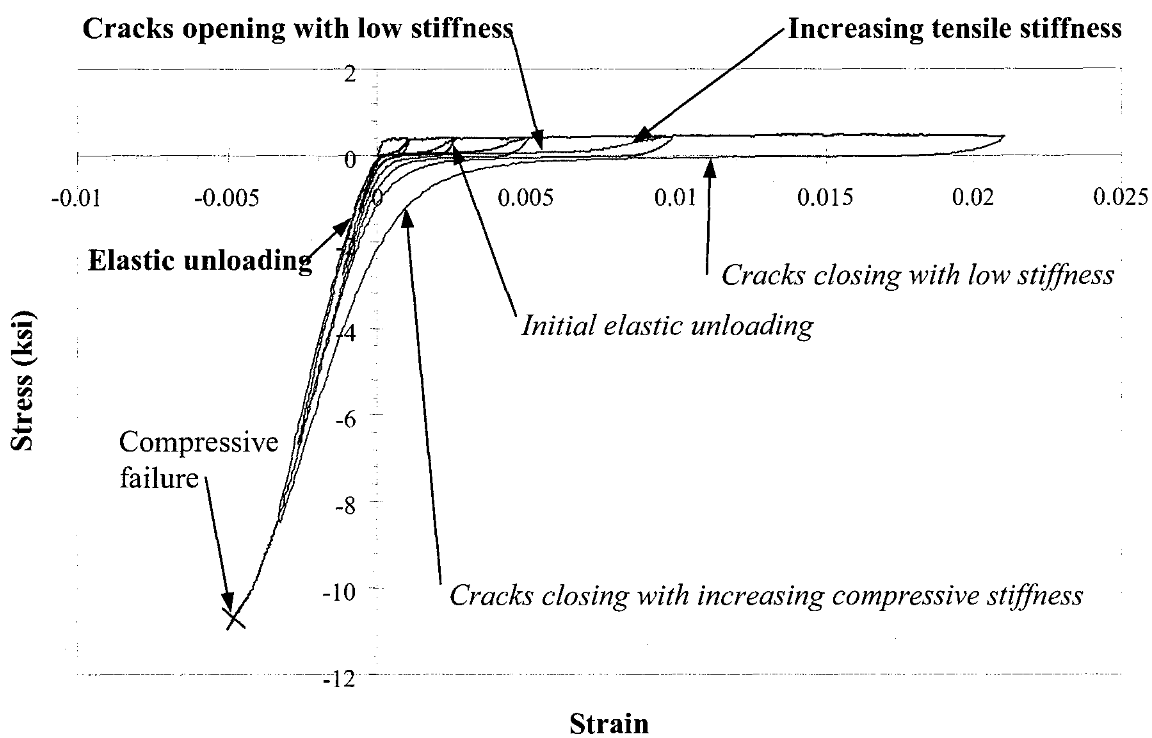


Figure 10 – Results from cyclic tension – compression test, cylindrical specimen.

SUMMARY, CONCLUSIONS AND FUTURE WORK

The results of preliminary studies into the behavior of ECC materials in seismic strengthening applications have been presented. Preliminary analysis indicates that infill panels made from ECC materials have potential for use as a seismic retrofit strategy for steel frames. The ECC infill panels are found to potentially increase the lateral load capacity, stiffness and energy dissipation in a steel frame subjected to cyclic lateral loads.

In ongoing research, connection details, and different panel geometries are being considered for the retrofit of frames with flexible beam-column connections. These numerical studies will be used to further evaluate and develop ECC infill panels for seismic strengthening, stiffening and energy dissipation applications. Results from the studies will be used to identify promising systems for experimental studies.

The need to create a material model to simulate the behavior of ECC in retrofit applications necessitated cyclic testing of ECC materials. These cyclic material tests are used to develop an understanding of the crack opening/closing behavior of the materials. It is found that the unloading and reloading behavior of the materials is more complicated that the simple-secant method used in the preliminary analyses. In the ongoing research, a material model building on previous work [8,9] is being developed. Completion of the model will allow for efficient identification of appropriate panel-frame experiments, as well as more accurate simulation of the performance of ECC in seismic retrofit applications.

ACKNOWLEDGEMENTS

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Polymer Matrix Composite (PMC) Infill Walls for Seismic Retrofit

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Abstract

This paper presents the ongoing research dealing with the design, fabrication and testing of polymer matrix composite (PMC) infill walls as a seismic energy dissipation strategy. The composite infill wall structure is comprised of three fiber-reinforced polymer laminates with an infill of a vinyl sheet foam. At the interface between the laminates, visco-elastic material, which is expected to exhibit maximum shear strain, is used to dissipate energy and to improve the damping characteristics of the structure. Analytical and experimental studies were performed to explore the effectiveness of the energy absorbing strategy and the behavior of a PMC infill wall when subjected to monotonic and cyclic loading. A steel frame retrofitted by a polymer matrix composite (PMC) infill wall will be monitored to assess the enhancements to the seismic resistant capacity. A large scale steel frame and PMC infill panels are considered in this research to avoid typical uncertainties associated with scaling the dimensions.

The design of an optimum infill panel was determined based on performance and material cost using finite element analysis. And, material tests were used to obtain the characteristics of each material used in the infill wall. Finally, the observed behavior of the PMC infilled frame will be assessed on the basis of stiffness, strength, ductility, modes of failure and energy dissipation output.

Introduction

In recent years, civil engineers have recognized the potential of advanced polymer composites as an alternative material for construction. Driven in particular by the recognition that composites can offer improved advantages over traditional materials such as steel and concrete, attempts to use composites for buildings and bridges are on the rise. While it is apparent that fiber-reinforced polymer (FRP) composites play an increasingly important role in civil engineering applications, there is even a greater promise for the new concept of joining composites with traditional materials to form hybrid structures. Our goal in this research is to study the effective application of composite material when combined with steel frames and to generate optimum seismic retrofit strategies using PMC infill walls.

Typically, some low and middle rise building frames have infill wall systems. The infill walls have been built after the frame is constructed as partitions and in some cases infill walls are parts of the structural system. As a matter of fact, there is no resemblance between the response of the infilled frame and the bare one, as the former is substantially stronger and stiffer than the latter. Around 1950's, the behavior of infilled frames had been investigated by Benjamin and Williams (1957, 1958). Since that time, Mainstone (1971), Liauw et al. (1985) and White et al. (1997) studied the role of infill walls in strengthening and stiffening the structure as a whole. As a result, the effects of neglecting the infill walls are accentuated in high seismicity regions where the frame/wall interaction may cause substantial increase of stiffness resulting in possible changes in the seismic demand, and the infilled frame structure exhibits changes in the magnitude and distribution of stresses in the frame members.

In this paper, a conceptual design of an infill wall is presented. The conceptual design is based on using a multi-layer system and allowing for in plane shear deformations to be concentrated in specific layers. Thereby, damage in such layers will provide the energy dissipation in the system.

This paper presents : (1) the optimum design of the PMC infill walls using finite element analysis (2) studying properties of FRP and visco-elastic materials. (3) Manufacturing the structural PMC walls (4) Testing the wall systems when incorporated in a steel frame having semi-rigid bolted connections.

Material Testing

This section presents composite material testing to evaluate the mechanical properties that are needed for analysis and design. The evaluation of the mechanical properties of composites include their strength and stiffness characteristics. In this research, we performed the composite material testing to evaluate several materials for possible use in the composite panel. We present two of the potential choices that may be used for the infill panel. One system incorporated a multi-layer plain weave glass fabrics in a matrix of polyester resin (DERAKANE). The second system incorporated a multi-layer woven roving glass fabric in a similar matrix.

Based on ASTM testing specifications, material tests were done to measure basic composite mechanical properties that are needed for analysis and design. For tension and compression test, each ultimate tensile (σ_{LU} ,σ_{LU}'), compression strengths (σ_{TU} ,σ_{TU}'), Young's moduli (E_1 , E_2) and Poison's ratios (ν_{12} , ν_{21}) were obtained by testing longitudinal (0°) and transverse (90°) specimens according to the ASTM D3039 standard test method for tension and ASTM standard D3410 for compression. For compression test, an ASTM standard D3410 has been applied and the fixture, which was originally known as the Illinois Institute of Technology Research Institute (IITRI) fixture, was used to produce compression in the specimen through side-loading. The side-loading of the specimen was accomplished by pyramidal wedges inside a heavy-housing. And, to avoid local buckling and the corresponding reduction of in-plane compressive strength after delamination due to transverse impact, we redesigned compression test coupon. For evaluating in-plane shear properties, a test method which will generate pure shear loading is needed. There are four most widely used test methods for measuring in-plane shear properties of a unidirectional composite lamina such as the $[\pm\ 45]_s$ laminate tensile test method, the off-axis tensile test method, the 2-rail shear test method, and the torsion test method (Whitney et al., 1982). In this research, we used the 2-rail shear test method, as described in ASTM D4255.

A photograph of the FRP material test is shown in Fig. 1 and the results for each test are shown in table 1.

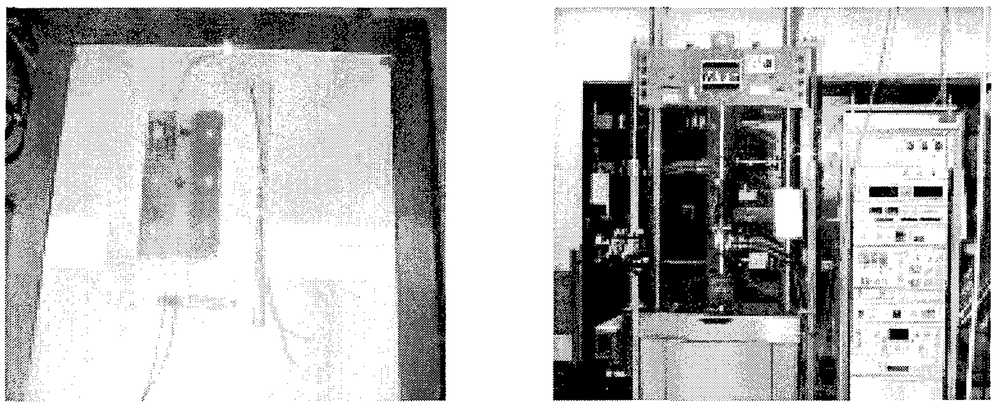
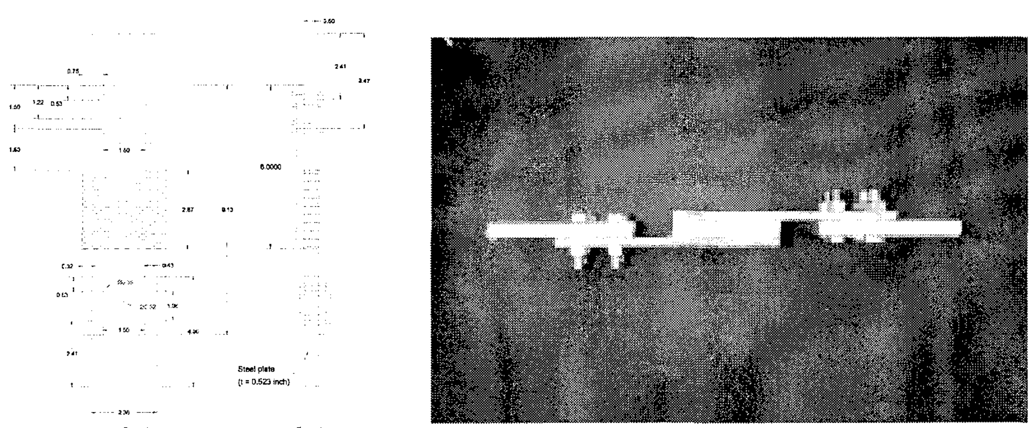


Figure 1: The Configuration of Each Test Coupon and Machine

Table 1: The Results of Material Tests (Ksi)

	Longitudinal direction			Transverse direction		
Method	Tension	Compression	Shear	Tension	Compression	Shear
Width (in)	0.498	0.256	1.0	0.4945	0.25	1.09
Thickness (in)	0.0545	0.197	0.064	0.053	0.2	0.0615
Elastic modulus	8423.12	2172.9		6895.5	1091.8	
Shear modulus			757.8			589.6
Poison's ratio	0.43	0.15		0.33	0.12	
Ultimate stress	32.7	37	8.65	25	18.1	9.09

The visco-elastic material is composed of H8-PP Polypropylene Honeycomb produced by Nida-Core Corp, FL, combined with a resin-rich layer on each surface of the honeycomb. This is a hexagonal cell honeycomb extruded from polypropylene. In our research of the honeycomb and resin-rich layers, the energy dissipation was expected to be through in-plane shear deformation. To investigate the damping characteristic of honeycomb materials, pure shear test was considered. Test coupon and adaptor are depicted in Fig. 2. The test results for shear stress-strain relationship are shown in Fig. 3 and table 2.



a) coupon with adaptor

Figure 2: The Configuration of Visco-elastic Coupon

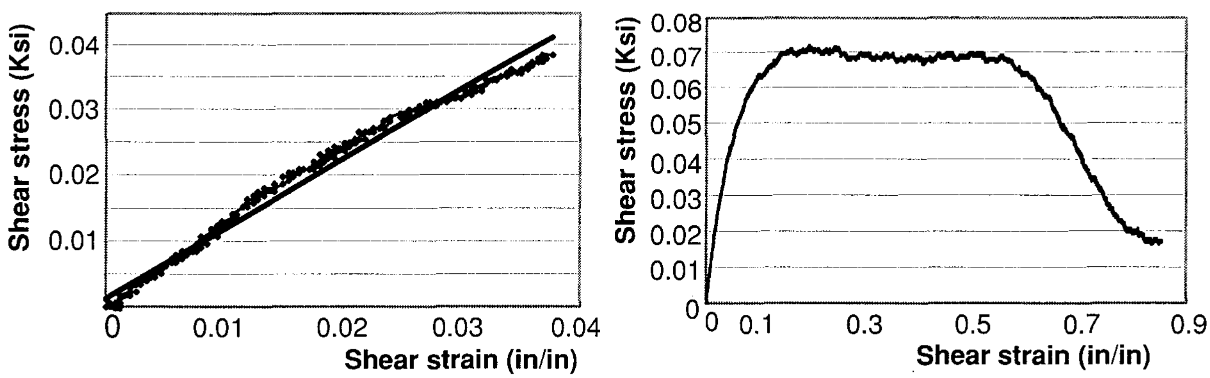


Figure 3: The Shear Deformation of Visco-elastic Materials

Table 2: The Properties of Visco-elastic Material (Nida-Core Corp.)

Property	Value
Shear modulus (G)	1.160 Ksi
Ultimate shear stress	0.0725 Ksi
Ultimate shear strain	0.625 in/in
Modulus of Elasticity	2.175 Ksi
Tensile Strength	0.0725 Ksi
Compressive Strength	0.188 Ksi

Design and Construction of Composite Panel

There are several technical and economical challenges associated with the design of the PMC wall in structural applications such as building. In this section, we present one conceptual application of the PMC wall system.

First, a sandwich construction was considered as a main concept to reduce the weight, sound and vibration as well as to improve structural rigidity. A multi-layer system may allow in-plane shear and therefore sliding along specific layers to take place upon loading the frame. And, the damage of composite panel should be concentrated in such layers. Based on this concept, the wall system was designed with three panels forming the entire wall thickness as shown in Fig. 4. At the interface between the panels, visco-elastic layers were used. The mechanism by which the wall is designed to dissipate energy is through in plane shear deformation as shown in Fig. 5. For optimum design based on the cost and performance, we have relied on detailed 3-D finite element models using ABAQUS. Numerous finite element simulations representing various material lay-up and geometric combinations were evaluated to produce the optimum wall design.

Second, we approached the optimum design of inner and outer layers composed of laminates based on structural performance. Design of each laminate layer includes (1) selecting a material system or a group of material systems, (2) determining the stacking sequence for the laminate based on applied loads, (3) some of the constraints include cost, weight, and stiffness. The choice of the fiber and matrix, processing technique and fiber volume fraction determines the stiffness, strength of a single lamina. In the design process, E-glass fiber was chosen as a proper reinforcement considering economic and practical factors such as reasonable properties and stiffness. After selecting reinforcement, the optimum design of stacking sequence of inner and outer laminates in the PMC wall was performed from the analysis based on structural performance.

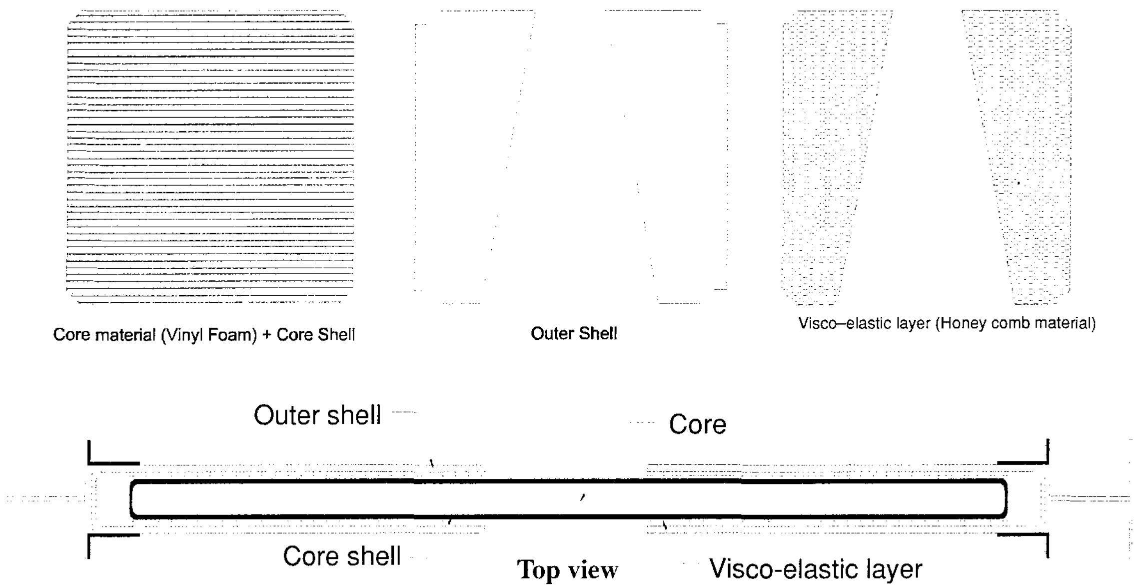


Figure 4: The Shape of Each Components of Composite Panel

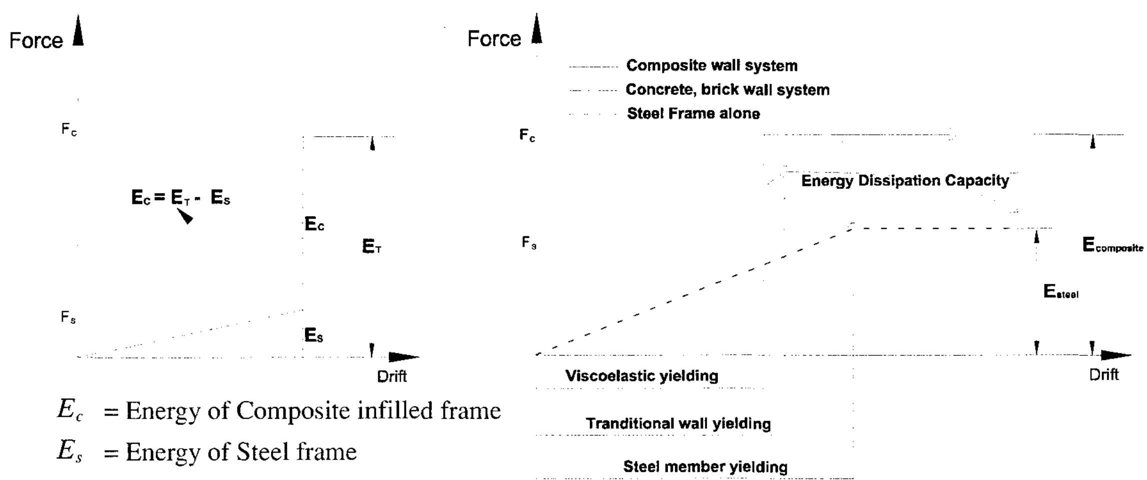


Figure 5: The Energy Mechanism of Composite Infilled Frame

Detailed design drawings as shown in Fig. 4 were delivered to a local PMC manufacturer (AN-COR Industrial Plastics, Inc., Tonawanda, NY) to construct the PMC infill wall. The PMC wall as shown in Fig.6 was constructed considering the most practical and commercial conditions. Plies stacking sequence are shown in table. 3. After manufacturing, the wall was installed in a frame (2500x2400 mm, W8x24 column and W8x21 beams) having semi-rigid bolted connections to be tested.

Table 3: The Stacking Sequence of Laminates (Unit = inch)

	Inner Laminate			Outer laminate		
No.	Orientation	No. of layers	Thickness	Orientation	No. of layers	Thickness
1	0	2	0.03	0	2	0.03
2	30	2	0.03	30	3	0.045
3	45	3	0.045	45	1	0.015
4	60	1	0.015	90	1	0.015
5	90	1	0.015	–45	1	0.015
6	–60	1	0.015	–30	3	0.045
7	–45	2	0.03	0	3	0.045
8	–30	3	0.045	–30	3	0.045
9	0	1	0.015	–45	1	0.015
10	–45	1	0.015	90	1	0.015
11	90	2	0.03	45	1	0.015
12				30	3	0.03
13				0	3	0.03
Total			0.285			0.36

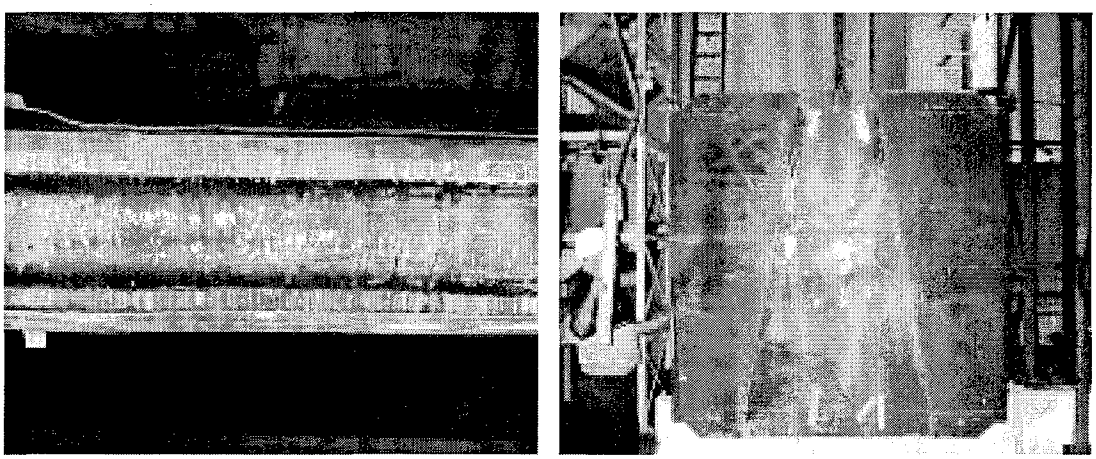


Figure 6: The Configuration of Composite Wall

Finite Element Analysis

The lateral load response and other structural factors of the infilled frame with a PMC composite wall have been studied. The commercial finite element package (ABAQUS) was used to perform detailed analysis. Eight node linear brick elements (C3D8) were used to model both the steel frame and honeycomb materials. Four node shell elements (S4R5) were used to model the composite laminate wall components. The interface between infill and frame members was modeled with gap-friction elements which provided gap between the nodes of frame and the wall along the perimeter. The finite element analysis is used to (1) design the optimum composite panel (2) develop proper simplified analytical model for composite infill wall frame system (3) predict the type of anticipated failure mode for subsequent experiments having various visco-elastic layers and new conceptual wall designs. The finite element model of the infilled frame is shown in Fig. 7.

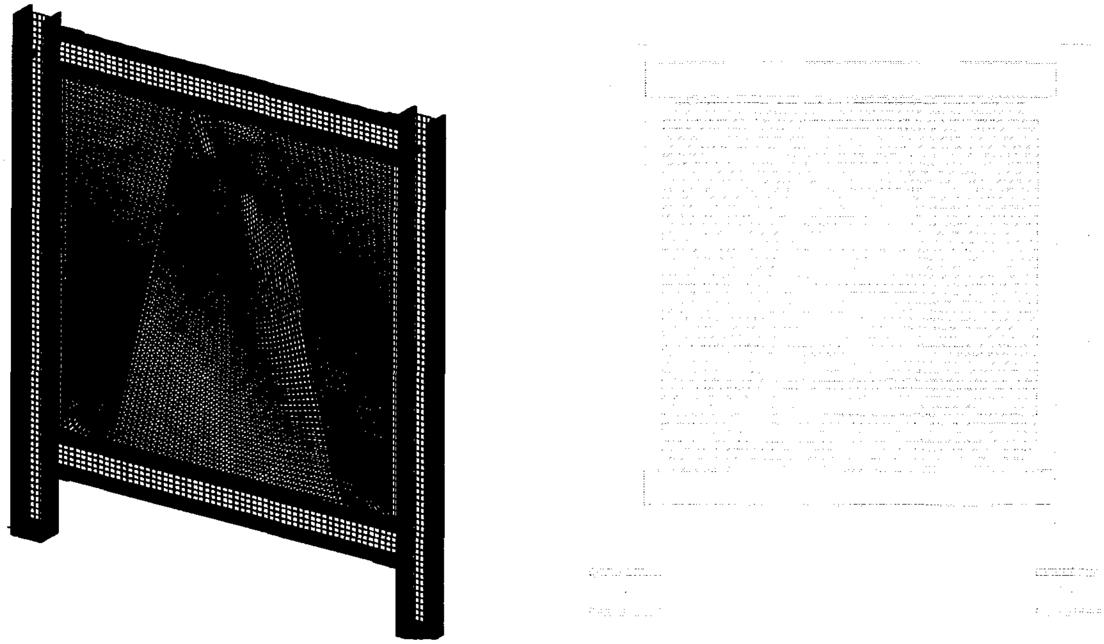
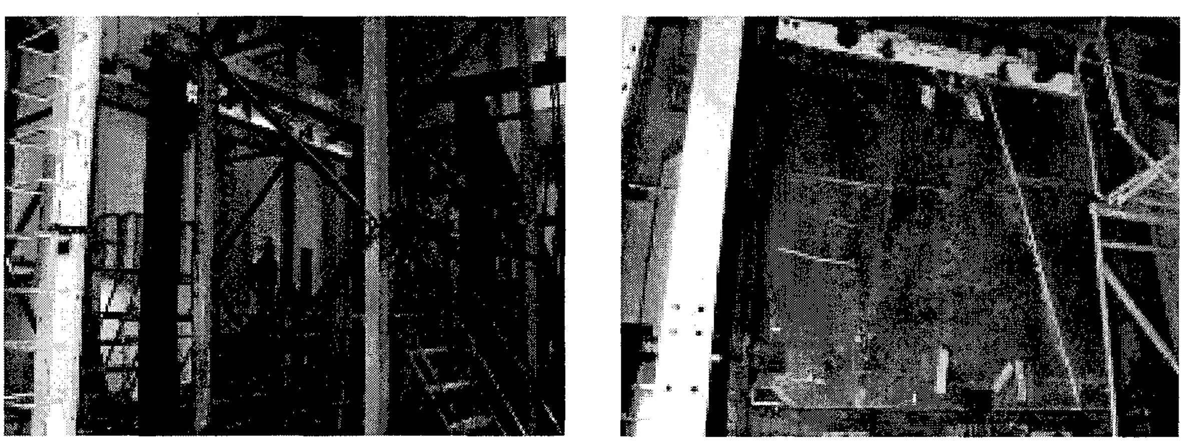


Figure 7: The FE Model and Design of Composite Infill Wall Frame

Configuration of Test Specimens

The test specimens consist of a steel frame with or without the composite infill wall are shown in Fig. 8. The frame members were designed and constructed according to the specifications of the American Institute of Steel Construction(AISC). Both beam and column members were assembled by semi-rigid (top and bottom angle seat) connections. The steel frame test setup is depicted in Fig. 8(a). After erection of the steel frame, the composite infill wall was constructed and installed within steel frame. The test setup of steel frame with the infill wall is shown in Fig. 8(b).



(a) Steel Frame Test Specimen (b) Composite Infill Wall Frame Test Specimen

Figure 8: The Experimental Setup

The out-of-plane instability was prevented by steel plate supports perpendicular to the plane of loading. Also, four stabilizing frames were designed and constructed to resist any accidental out-of plane forces, transverse to the plane of the test specimens.

Description of the Experiments

In the experimental phase of this research, testing of steel frame with and without composite infill wall is planned. Steel frame and composite infilled frame specimens will be tested under monotonic and cyclic in-plane load. The base of test specimens, a heavy concrete beam, was designed to resist the maximum expected load exerted by the frame during testing. The base beam was anchored securely to the strong reaction floor. To apply lateral force, a 250-kips MTS hydraulic actuator with a stroke of $\pm$ 4 *inch* will be used.

Various instruments were attached to the specimen to capture key quantities to characterize the structural response of the composite infill wall and steel frame. These key quantities included the following: (1) Longitudinal and transverse strain at critical point on the composite infill panel (2) The shear deformation of the visco-elastic material using linear potentiometers (3) The hysteresis behavior and the corresponding strength deterioration and stiffness degradation of steel frame and composite infilled frame using displacement transducers.

Testing of Steel Frame

In this research, results from the steel frame experiments subjected to monotonic and quasi-static cyclic loading are necessary to obtain an understanding of the behavior of the steel frame prior to testing composite infill wall system. The results will provide clear evident for the effectiveness of composite wall and validate the analytical modeling of steel frame based on experimental result. ABAQUS is used for the inelastic analysis and damage evaluation under combined static and quasi-static cyclic loading. The steel frame specimen is tested by applying lateral load at the top beam as shown in Fig. 9.

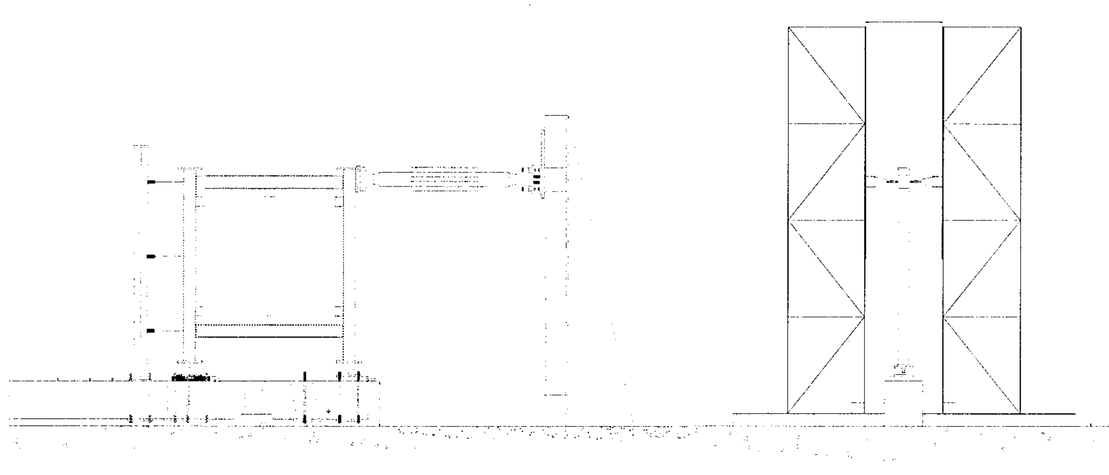


Figure 9: Setup of Steel Frame Specimen

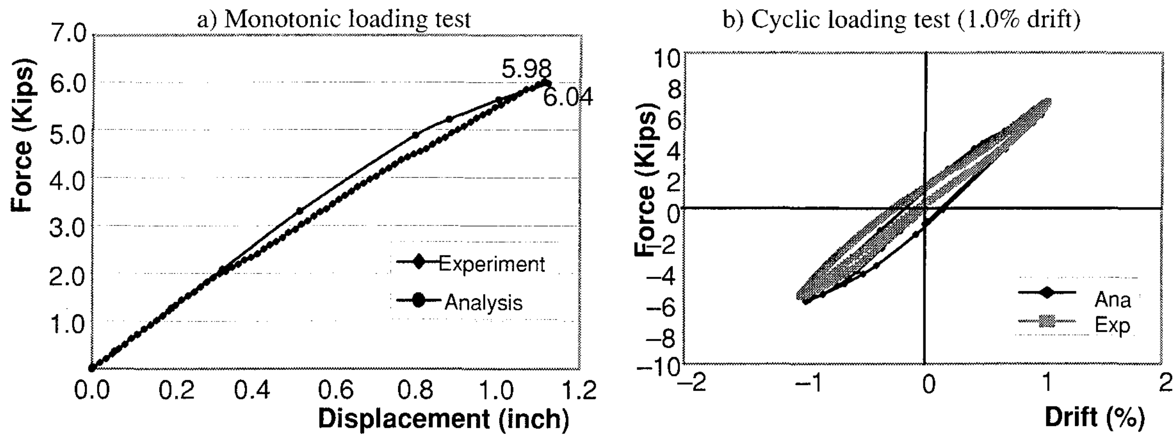


Figure 10: The Results of Steel Frame

The force-drift response for the bare frame is shown in Fig. 10. First, a push-over of the frame was performed up to 1% drift. Second, quasi-static cyclic loading was applied to the bare frame. The quasi-static cyclic experiment was carried out in displacement-control for lateral drifts 0.5%, 1% and 1.5%. For each drift, two cycles of loading were applied. As can be seen, numerical analysis and experimental results match quite well for the push-over and quasi-static cyclic loading test.

Testing of PMC Infilled Frame

Based on the numerical analysis of composite infilled frame, force-drift relationship indicated that the stiffness of the composite infilled frame is extremely higher than that of bare frame structure as shown in Fig. 11. In addition, we expect composite infilled frame should have significant energy dissipation due to visco-elastic material at the interface between laminates. The composite infilled frame will be tested to investigate the strength, stiffness degradation and the mode of failure of PMC infill wall. Also, the effect of net infill contribution to the frame resistance and the behavior of honeycomb materials between outer and inner layers will be studied. The contour of the predicted shear strain for visco-elastic material and the contour of predicted failure mode for outer laminates are shown in Fig. 12.

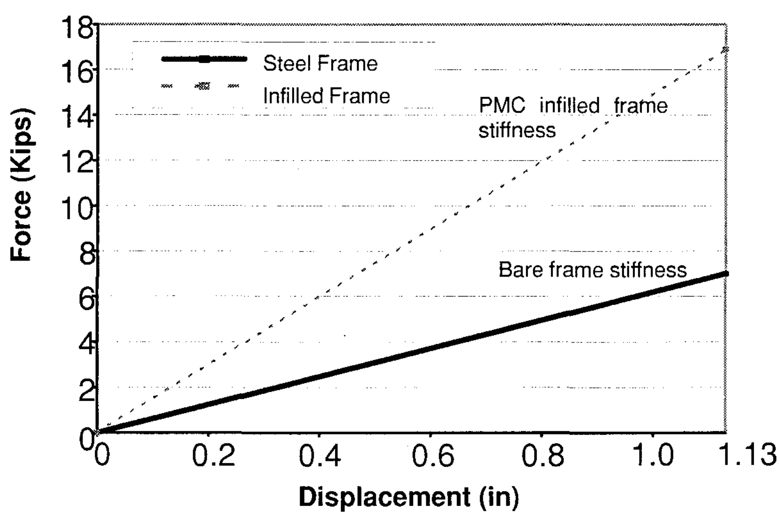
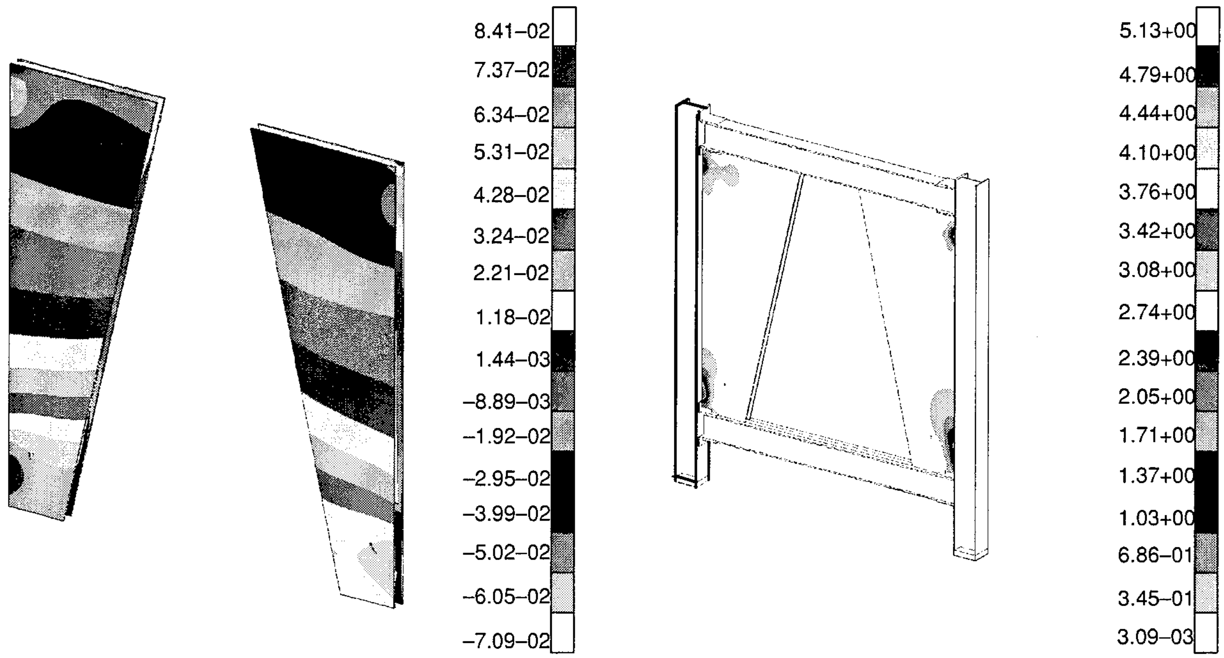


Figure 11: The Results of Push-over Numerical Analysis For The PMC Infilled Frame



a) The in plane shear strain contour of Visco-elastic layer

b) The Failure Contour contour of Outer laminate

Figure 12: The Results of Numerical Analysis For Critical Components

Conclusions

A conceptual design of a PMC infill wall for seismic retrofit is presented. Finite element simulations clearly show that the infill wall will provide significant strength and stiffness to the steel frame. However, testing of the frame with the infill wall is not complete yet and further verification of the structural system is warranted before drawing any conclusions.

Acknowledgments

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Pattern Recognition for Structural Health Monitoring

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Abstract

The process of implementing a damage detection strategy for engineering systems is often referred to as *structural health monitoring*. Vibration-based damage detection is a tool that is receiving considerable attention from the research community for such monitoring. Recent research has recognized that the process of vibration-based structural health monitoring is fundamentally one of statistical pattern recognition and this paradigm is described in detail. This process is composed of four portions: (1) Operational evaluation; (2) Data acquisition and cleansing; (3) Feature selection and data compression, and (4) Statistical model development for feature discrimination. A general discussion of each portion of the process is presented.

Introduction

The process of implementing a damage detection strategy for aerospace, civil and mechanical engineering infrastructure is referred to as *structural health monitoring (SHM)*. Here *damage* is defined as changes to the material and/or geometric properties of these systems, including changes to the boundary conditions and system connectivity, which adversely affect the system's performance. The SHM process involves the observation of a system over time using periodically sampled dynamic response measurements from an array of sensors, the extraction of damage-sensitive features from these measurements, and the statistical analysis of these features to determine the current state of system health. For long term SHM, the output of this process is periodically updated information regarding the ability of the structure to perform its intended function in light of the inevitable aging and degradation resulting from operational environments. After extreme events, such as earthquakes or blast loading, SHM is used for rapid condition screening and aims to provide, in near real time, reliable information regarding the integrity of the structure.

The basic premise of vibration-based damage detection is that damage will significantly alter the stiffness, mass or energy dissipation properties of a system, which, in turn, alter the measured dynamic response of that system. Although the basis for vibration-based damage detection appears intuitive, its actual application poses many significant technical challenges. The most fundamental challenge is the fact that damage is typically a local phenomenon and may not significantly influence the lower-frequency global response of structures that is normally measured during vibration tests. Another fundamental challenge is that in many situations vibration-based damage detection must be performed in an *unsupervised learning* mode. Here, the term *unsupervised learning* implies that data from damaged systems are not available. These challenges are supplemented by many practical issues associated with making accurate and repeatable vibration measurements at a limited number of locations on complex structures often operating in adverse environments. Recent research has begun to recognize that the vibration-based damage detection problem is fundamentally one of statistical pattern recognition and this paradigm is described in detail.

Vibration-Based Damage Detection and Structural Health Monitoring

This statistical pattern recognition paradigm for structural health monitoring is composed of four portions: (1) Operational evaluation; (2) Data acquisition, cleansing and fusion; (3) Feature selection and data compression, and (4) Statistical model development for feature discrimination.

Operational Evaluation

Operational evaluation answers four questions in the implementation of a structural health monitoring system:

1. What are the economic or life-safety justifications for performing the monitoring?
2. How is damage defined for the system being investigated and, for multiple damage possibilities, which are of the most concern?
3. What are the conditions, both operational and environmental, under which the system to be monitored functions?
4. What are the limitations on acquiring data in the operational environment?

Operational evaluation begins to set the limitations on what will be monitored and how the monitoring will be accomplished. This evaluation starts to tailor the health monitoring process to features that are unique to the system being monitored and tries to take advantage of unique features of the postulated damage that is to be detected.

Data Acquisition and Cleansing

The data acquisition portion of the structural health monitoring process involves selecting the types of sensors to be used, selecting the location where the sensors should be placed, determining the number of sensors to be used, and defining the data acquisition /storage/transmittal hardware. This process is application specific. Economic considerations play a major role in these decisions. Another consideration is how often the data should be collected. In some cases it is adequate to collect data immediately before and at periodic intervals after a severe event. However, if fatigue crack growth is the failure mode of concern, it is necessary to collect data almost continuously at relatively short time intervals.

Because data can be measured under varying conditions, the ability to normalize the data becomes very important to the damage detection process. One of the most common procedures is to normalize the measured responses by the measured inputs. When environmental or operating condition variability is an issue, the need can arise to normalize the data in some temporal fashion to facilitate the comparison of data measured at similar times of an environmental or operational cycle. Figures 1 and 2 conceptually illustrate scenarios where measures of the environmental or operational parameter will and will not be need to be incorporated into the normalization procedure.

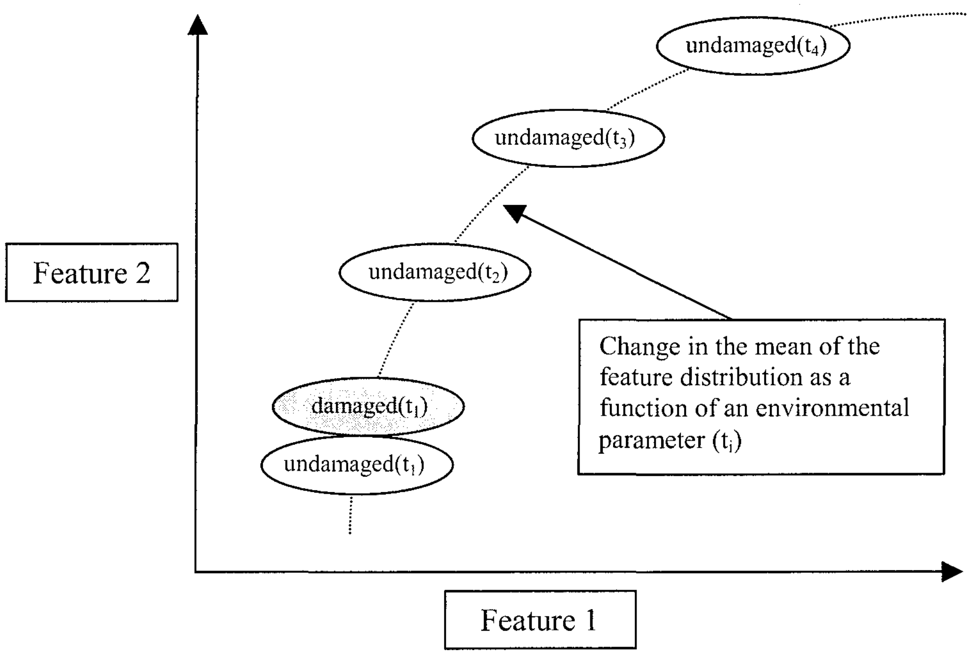

Figure 1. Damage produces changes in the feature distribution similar to those produced by environmental variability. This case will most likely require some measure of the environmental parameter to be included in the normalization process.

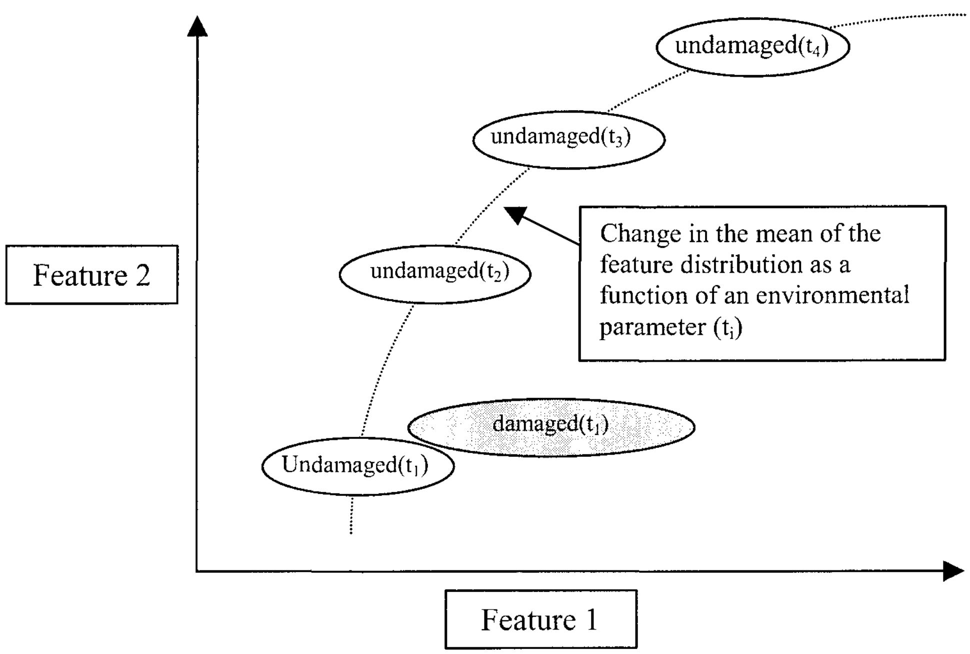

Figure 2. Damage produces a change in the feature distribution that is in some way orthogonal to changes caused by the environmental effects. For this case a measure of the environmental parameter may not be necessary.

Data cleansing is the process of selectively choosing data to accept for, or reject from, the feature selection process. The data cleansing process is usually based on knowledge gained by individuals directly involved with the data acquisition. Data fusion is concerned with integrating information from an array of heterogeneous sensors for better understanding of the system response. Finally, it is noted that the data acquisition, cleansing and fusion portion of a structural health-monitoring process should not be static. Insight gained from the feature selection process and the statistical model development process provides information regarding changes that can improve this process.

Feature Selection

The study of data features used to distinguish the damaged structures from undamaged ones receives considerable attention in the technical literature[1]. Inherent in the feature selection process is the condensation of the data. The operational implementation and diagnostic measurement technologies needed to perform structural health monitoring typically produce a large amount of data. Condensation of the data is advantageous and necessary, particularly if comparisons of many data sets over the lifetime of the structure are envisioned. Also, because data may be acquired from a structure over an extended period of time and in an operational environment, robust data reduction techniques must retain sensitivity of the chosen features to the structural changes of interest in the presence of environmental noise.

The best features for damage detection are typically application specific. Numerous features are often identified for a structure and assembled into a feature vector. In general, a low dimensional feature vector is desirable. It is also desirable to obtain many samples of the feature vectors for the statistical model building portion of the study. There are no restrictions on the types or combinations of data that are assembled into a feature vector.

Statistical Model Development

The portion of the structural health monitoring process that has received the least attention in the technical literature is the development of statistical models to enhance the damage detection. Almost none of the hundreds of studies summarized by Doebling, et al.[1] make use of any statistical methods to assess if the changes in the selected features used to identify damaged systems are statistically significant. However, there are many reported studies for rotating machinery damage detection applications where statistical models have been used to enhance the damage detection process[2].

Statistical model development is concerned with the implementation of the algorithms that operate on the extracted features to quantify the damage state of the structure. The algorithms used in statistical model development usually fall into three categories. When data are available from both the undamaged and damaged structure, the statistical pattern recognition algorithms fall into the general classification referred to as *supervised learning*. *Group classification* and *regression analysis* are general classes of algorithms for supervised learning. *Unsupervised learning* refers to algorithms that are applied to data not containing examples from the damaged

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structure. Some form of outlier detection is typically employed for the unsupervised learning problem.

The damage state of a system can be described as a five-step process along the lines of the process discussed in Rytter[3] to answers the following questions: (1) Is there damage in the system (existence)?; (2) Where is the damage in the system (location)?; (3) What kind of damage is present (type)?; (4) How severe is the damage (extent)?; and (5) How much useful life remains (prediction)? Answers to these questions in the order presented represents increasing knowledge of the damage state. The statistical models are used to answer these questions in a quantifiable manner. Experimental structural dynamics techniques can be used to address the first two questions. To identify the type of damage, data from structures with the specific types of damage must be available for correlation with the measured features. Analytical models are usually needed to answer the fourth and fifth questions unless examples of data are available from the system (or a similar system) when it exhibits varying damage levels.

Finally, an important part of the statistical model development process is the testing of these models on actual data to establish the sensitivity of the selected features to damage and to study the possibility of false indications of damage. False indications of damage fall into two categories: (1) False-positive damage indication (indication of damage when none is present), and (2) False-negative damage indications (no indication of damage when damage is present).

CONCLUDING COMMENTS

Current SHM methods are either visual or localized experimental methods such as acoustic or ultrasonic methods, magnetic field methods, radiograph, eddy-current methods and thermal field methods[4]. All of these experimental techniques require that the vicinity of the damage is known *a priori* and that the portion of the structure being inspected is readily accessible. The need for quantitative *global* damage detection methods that can be applied to complex structures has led to research into SHM methods that examine changes in the vibration characteristics of the structure. Summaries of this research can be found in recent review articles.[5,6] In addition, there are several annual and biannual conferences dedicated to this topic.[7,8,9] To date, most global SHM techniques proposed in these references examine changes in modal properties (resonant frequencies, mode shapes), or changes in quantities derived from modal properties. Drawbacks of these investigations include:

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[6] Housner, G.W., et al., (1997) "Structural Control: Past, Present and Future," (Section 7, Health Monitoring) *Journal of Engineering Mechanics*, ASCE, **123** (9), pp. 897-971.

[7] The 2nd International Structural Health Monitoring Workshop, Palo Alto, CA, 1999.

[8] The 5th International Symposium on Nondestructive Evaluation of Aging Infrastructure, Newport Beach, CA. 2000.

[9] The 3rd International Conference on Damage Assessment of Structures, Dublin, Ireland, 1999.

1. The use of relatively expensive off-the-shelf, wired instrumentation and data processing hardware not designed specifically for SHM.
2. Excitation has, in general, been from ambient sources inherent to the operating environment.
3. Ambient vibrations excite lower frequency global modes that are insensitive to local damage.
4. The data reduction is usually based on classical linear modal analysis.
5. Most studies assume that the structure can be modeled as a linear system before and after damage.
6. Statistical methods have not been used to quantify when changes in the dynamic response are significant and caused by damage. Varying environmental and operational conditions produce changes in the system's dynamic response that can be easily mistaken for damage.

Taken as a whole, the aforementioned characteristics place serious limitations on the practical use of existing methodologies. Indeed, with the exception of applications to rotating machinery, there are no examples of reliable strategies for SHM that are robust enough to be of practical use.

In an effort to address some of the deficiencies listed above a statistical pattern recognition paradigm for vibration-based structural health monitoring has been proposed. To date, all vibration based-damage detection methods that the authors have reviewed in the technical literature can be described by this paradigm with the vast majority of this literature focused on the identification of damage sensitive features. However, few of these studies apply statistical pattern recognition procedures to the damage-sensitive features. This lack of statistical analysis presents some potential problems for the development of vibration-based damage detection technology. As an example, the difficulties associated with accurately quantifying the statistical distribution of large order feature vectors are well documented in the statistics literature. However, most vibration-based damage detection methods discussed in the technical literature do not address this issue and many do not hesitate to suggest the use of relatively large feature vectors. A multi-disciplinary approach to the vibration-based damage detection problem is required to alleviate problems such as the "curse of dimensionality." Such approaches offer the potential to overcome other difficulties associated with this technology such as widely varying length scales of the damage relative to that of the structure and the fact that damage can accumulate vary gradually over multi-year time scales.

The Development of a Wireless Modular Health Monitoring System for Civil Structures

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Abstract
Current structural monitoring systems employ conventional cables to allow sensors to communicate their measurements to a central processing unit. Cabled based sensing systems for structures have high installation costs and leave wires vulnerable to ambient signal noise corruption. To address these disadvantages, a research effort has been initiated towards the development of a wireless modular monitoring system. The developed wireless modular monitoring system (WiMMS) would have lower capital and installation costs as well as ensure more reliability in the communication of sensor measurements. Some key areas of innovations emphasized are the use of a wireless communication system for inter-sensor communication, the utilization of micro-electro mechanical sensing elements, and the use of a microprocessor for advanced damage detection methods.

Introduction
There exists a need for a rational and economical method of monitoring the performance of civil structures over their lifespans. Monitoring systems are currently playing a dominant role in applications such as nonlinear model validation, structural health monitoring and structural control. Current design practice analytically determines a structure's nonlinear response based upon nonlinear models of the key load carrying elements. A monitoring system can provide invaluable insight into the accuracy of these nonlinear models and can assist engineers in refining them. For example, short and long span bridges in California are being instrumented by the California Department of Transportation (Caltran) to monitor the nonlinear response of the bridges during extreme seismic events [1]. Just as important is the need of a rapid assessment of the performance and safety of civil structures. Using a monitoring system to measure structural responses, a damage detection strategy is then employed to diagnose possible short and long-term damage in a structure. Last but not least, in the structural control field, one key component of a control system is an integrated monitoring system that can provide feedback of real time measurements of structural response.

With the rapid advancement of sensing, microprocessor, wireless and other technologies, one of the research challenges is to assess the benefits gained from the application of such technologies in the structural engineering field. Our research efforts have identified wireless communication technology, micro-electro mechanical (MEM) devices, microprocessors and digital signal processors, as key areas of innovation that can be used to develop a novel wireless monitoring system for civil structures.

Traditional Structural Monitoring Systems

The origin of commercially available structural measurement systems is from those regularly used in enclosed, laboratory settings. As a result, the systems are characterized as being of the hub-spoke architecture with accelerometers remotely placed throughout the structure but wired back to a single centralized data acquisition unit.

Among the key problems inherent in these systems are the installation time and cost. From experience, the installation time of a complete measurement system for bridges and buildings, can potentially consume over 75% of the total testing time. Installation labor costs can approach well over 25% of the total system cost. Caltran reported that it costs over $300,000 per toll bridge to install a measurement system comprised of 60 to 90 accelerometers. To isolate the wires from the bridge's harsh environment, a wire conduit is installed at a cost of $10 per linear foot [1]. Within buildings, wires are susceptible to tearing, rodent nibbling and measurement corruption through signal noise.

Wireless Structural Monitoring System

Our primary goal is to change the practice of using extensive cabling and high cost labor as is typical of the traditional monitoring systems to a system of inexpensive wireless embedded systems that can be installed, maintained and operated with ease (see Figure 1).

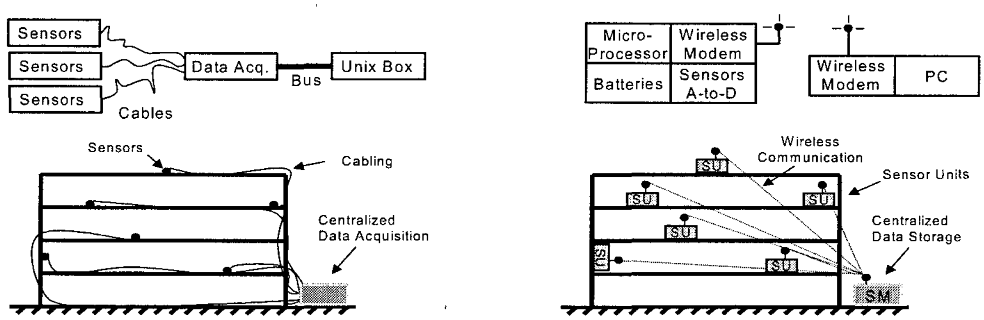


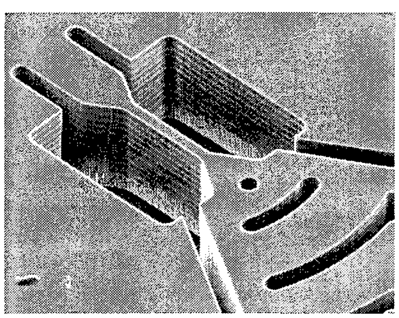
Figure 1 – From Conventional Cabled to Wireless-Embedded Structural Monitoring System

In a previous study, a proof-of-concept system has been fabricated from commercially available components. The system has been successfully used in a full-scale experiment of the Alamosa Canyon Bridge in New Mexico [2]. With the collaboration of researchers at Los Alamos National Laboratory, the units were validated by comparing the wireless sensor's measurements to those of a conventional cabled monitoring system. Installation took well over 2 hours for the cabled monitoring system while the five wireless sensing units took less than a half hour to install. This study has clearly demonstrated the time and cost effectiveness of a wireless monitoring system.

A wireless communication system provides a "free infrastructure" in that the need for the installation of wires is eradicated and the accommodation of direct communication between sensing units is provided. While a centralized data acquisition system can still be designed using the wireless network, the flexibility of the system allows for a decentralized data acquisition system with sensors transmitting measurements directly to the other system sensors.

A primary innovation is the migration of computational power from the centralized data acquisition system to the sensor units. The computational power that is provided by an advanced microprocessor is harnessed when the wireless sensors are implemented in a structural monitoring system used for such applications as structural health monitoring and structural control. The microprocessor is also utilized to coordinate the functionality of the sensor units such as sampling the sensor's output, packaging the measurements for transmission, and operating an integrated radio modem for communication.

While accelerometers are used in most structural sensing applications, the sensor units are compatible with any type of analog sensor. In the Stanford prototype, a micro-electro mechanical (MEM) based accelerometer is used. By fabricating micrometer sized mechanical elements upon silicon, revolutionary sensors can be fabricated along with CMOS based circuits all on one chip. The result is accurate and sensitive sensors in form factors and unit costs not previously possible. One example is the high performance planar accelerometers designed and fabricated by Professor Thomas Kenny's group at Stanford; the accelerometer uses piezoresistive elements along the cantilevering arm of a proof mass for direct acceleration measurements (See Figure 2). By modifying the dimensions of the cantilevering element, desirable sensor characteristics can be attained for structural sensing [3].



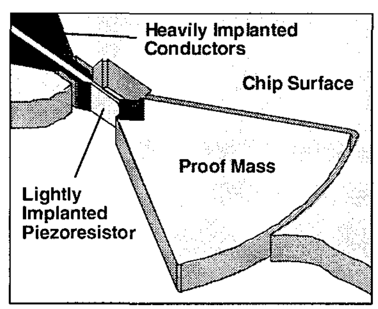


Figure 2 – MEMs Based Piezoresistive Accelerometer

WiMMS Based Structural Monitoring Strategies

Current research efforts are focused on integrating the prototype wireless sensors into a complete structural monitoring system for damage detection. Termed a wireless modular monitoring system (WiMMS), the system will provide rapid and global damage diagnosis. While visual structural inspections would still play a major role in evaluating the safety of structures, the proposed system would serve to assist the inspection teams in prioritizing which structures to consider first and where in them to look.

Structural health monitoring systems utilize a structural sensing system to observe the response of a structure over time at equally spaced intervals. Time and frequency based properties are extracted from these measurements to see their changes over time. Many approaches to health monitoring have been proposed, ranging from those conducted deterministically to those conducted statistically in both the time and frequency domain. Our current research focuses on statistical-based approaches, which can take full advantages of the embedded wireless sensor monitoring system, for rapid damage assessment and global damage diagnosis [4,5].

Conclusion

An embedded wireless sensor monitoring system has been developed. In comparison to its cabled counterparts, the system enjoys the benefit of cheaper and quicker installations as well as superior performance. With computational power pushed forward from the central data acquisition system to the sensor itself, these novel sensors can be integrated with methods for structural health monitoring and damage detection. Rapid and global damage assessment using the sensors is one step towards a modular sensing system for the protection of vital civil structures.

Acknowledgments

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DAMPING IN STRUCTURAL APPLICATIONS

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ABSTRACT

Alloys which display a thermoelastic martensitic transformation can serve as damping elements in a variety of structures due to the unique energy absorption effects which occur in them. Depending on the type of structure and the amplitude and frequency of the loading, different aspects of the transformation and different forms of the alloys would be used. For small amplitude high frequency vibrations, the extreme internal friction peak which occurs near the transformation can be utilized. In high amplitude low frequency loading, one may use deformation of the martensitic structure, and while this has the widest temperature range of application there are some shape limitations. By using creation and annihilation of stress induced martensite, one can maximize the versatility of the damper design in the force/deflection/energy absorption regime, but with some limitation on the temperature range of application. Specific damper design configurations are presented with the application features of each discussed.

INTRODUCTION

In the alloys which exhibit the Shape Memory and Superelastic Effect, there are microstructural processes which occur that convert mechanical energy into thermal energy in the material. These processes do not damage or work harden the alloy as happens with plastic deformation in ordinary alloys, and therefore the Shape Memory Alloys can be used as highly fatigue resistant damping components in structures. To choose which type of damping one must have in any given structure or situation, it is essential that one determine the frequency and amplitude of the vibrations to be damped, the force to be exerted by the damper and the temperature range over which damping must be effective. Proper design of dampers can satisfy frequencies from a few cycles per day to thousands of cycles per second, amplitudes from microns to meters, forces from grams to thousands of tons and wide temperature ranges.

INTERNAL FRICTION

If the type of damping needed is high frequency and low amplitude, such as sound damping or machinery vibration damping, then using the internal friction property may be most effective. At temperatures close to the transformation temperature, it is well known that there is an extreme internal friction peak in alloys which have a thermoelastic martensitic transformation(1). The mechanistic explanation of how small amplitude strain pulses are absorbed and converted into heat energy in the structure has been considered by a number of authors(2,3) and includes theories involving creation and annihilation of transformation nuclei, atomistic movements of martensite platelet boundaries, and other 'lattice softening' phenomena at the transformation temperature.

An example of the type of internal friction peak which occurs near the transformation temperature was reported by Morin, et al,(2) and is shown in Figure 1. The relatively narrow temperature range over which the high internal friction occurs is seen, as well as the extreme damping provided in that range by the material. At the high damping point, the alloy gives a dead, non-metallic sound when struck . Sheets of the alloy can act as a sound deadening barrier as well as a structural element, or an element such as a washer in a bolted connection can block machinery noise and vibration transmission.

MARTENSITE DEFORMATION

In the martensitic condition, shape memory alloys can absorb shear strains of several percent totally by reorientation of the martensite twin variants. The stress required to reorient the twins is shown in the stress strain curve shown in Figure 2. The four quadrant view emphasizes that application of sufficient reversing stress is able to move the structure back and forth from one set of variants which give a certain strain to another set of variants which give a reverse strain. Moving the martensite from one variant to another involves a shear resistance, or 'friction', which determines the plateau stress shown in Figure 2. The resistance is encountered regardless of which directions the variants are moved, and therefore energy must be expended to shear the structure in either direction. The area inside the loop in Figure 2 is a measure of the energy needed to deform the structure back and forth and return it to the starting position. This energy is absorbed in the structure as heat since movement of the variants does not induce a large number of crystal defects and dislocations as occurs in ordinary metals during work hardening.

There are several aspects of energy absorption, or damping, due to deformation of the martensite that merit notice. The range of strain that can be accommodated in NiTi alloys is at least $\pm$8%, while the copper based alloys can absorb roughly half that much. As with other properties of these alloys, the thermomechanical processing has a large impact on the damping they can provide. In NiTi alloys, for example, the plateau stress can be varied from below 10ksi (70 MPa) to over 50ksi (350 MPa) depending on small alloy variations and processing history. One of the desirable aspects of damping with NiTi alloys is what occurs when the strain range extends much beyond the plateaus shown in Figure 2. In that case, the stress rises quickly to several times the plateau stress before any permanent yielding of the material occurs. Thus, the damping element acts as its own strain limiter or protection device in case the applied force rises above the level to be damped. The martensite in NiTi alloys exists from cryogenic temperatures to as high as 100C, so dampers which function over a wide temperature range are possible, fatigue during strain cycling is nearly non-existent, so long cyclic life is possible, and the effective strain range for damping is from a fraction of one percent to as high as 8%. Two limitations which must be considered with martensitic dampers is that the cyclic strain rate must be slow enough that the energy can be removed from the alloy before causing transformation, and the element must be mechanically held in a manner to force the cyclic strain back and forth because the martensite does not return itself to the beginning position. Typical devices which would utilize martensite deformation in damping include hanging brackets for vibrating pipes, shear elements in base isolation devices for

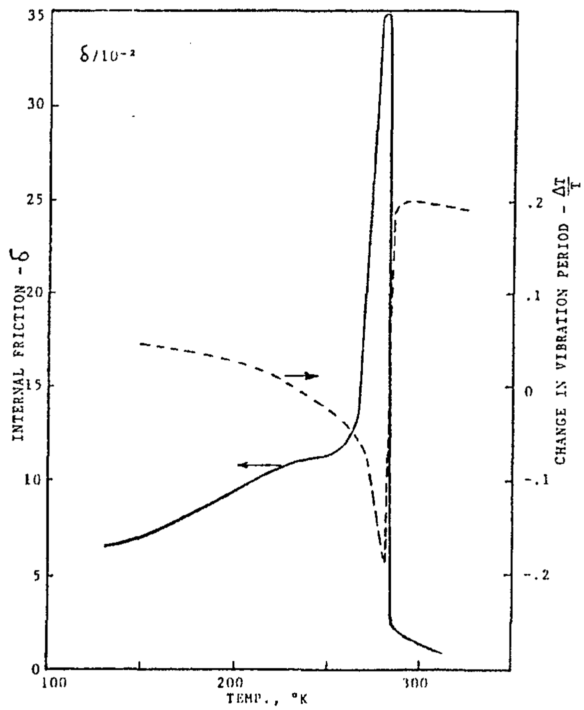

Figure 1. Internal friction, δ, and $\frac{\Delta T}{T}$, the change in vibration period, versus temperature for a CuZnAl Shape Memory Alloy at TTR.

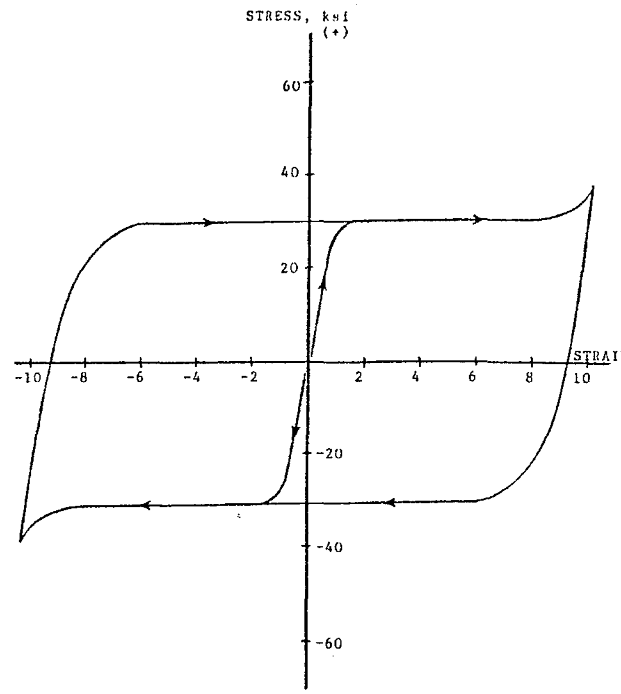

Figure 2. Four Quadrant Stress-Strain behavior of the martensite phase in NiTi Shape Memory Alloy.

supporting buildings, and dampers for wind excitation of structures where movements are small but fatigue life must be very long.

DAMPING WITH STRESS INDUCED MARTENSITE

The third mechanism of energy absorption in these alloys is the creation and annihilation of stress induced martensite. Examination of the well known superelasticity stress-strain curve shown in Figure 3 shows that the force needed to deform an element by creation of stress induced martensite (the upper plateau stress) is significantly greater than the force exerted by the element as the stress is lowered and the martensite is annihilated. The area inside the superelasticity loop is a measure of the energy removed from the stress application system and absorbed as heat in the superelastic element on each deformation cycle. For the loop shown in Figure 3, for example, the energy absorbed in an NiTi element would be approximately 1,000 ft. lbs of energy per cycle for each pound of alloy in the damper (3 joules/gr./cycle). In a properly prepared element, such as NiTi wire, the material can withstand a large number of strain cycles with relatively little change in the stress-strain curve.

Compared to dampers using martensite deformation, one finds that superlastic dampers have a similar requirement that cycle rate be slow enough to allow heat removal so the material is not heated above its superelastic range. This usually requires cycle rates on the order of 1 Hz or slower. In NiTi alloys, the range of useful superelasticity is at least 50°C and in some cases nearly 100°C. This is enough temperature range to allow superelastic dampers in many structural applications, though obviously many outdoor structures in harsher climates would exceed the allowed range. Similar to martensitic dampers, the range of allowed strain in the damper is large. Perhaps the largest difference between the martensitic and the superelastic damper is that the superelastic material will exert a force to return to its original shape. Thus, the damper can help restore the overall structure to its original position after any imposed deformation.

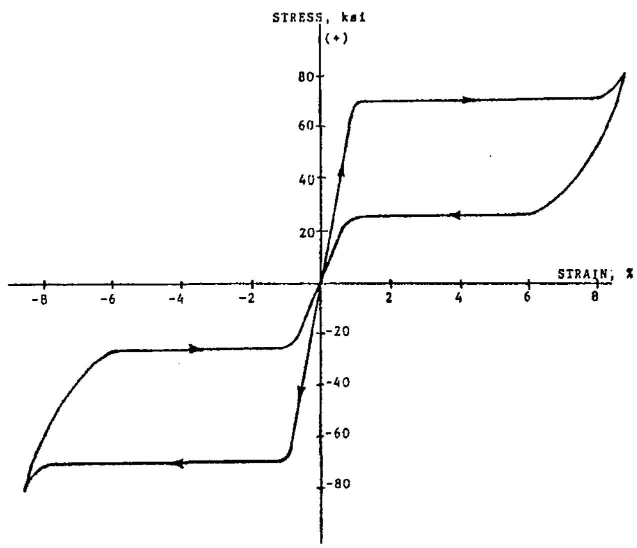

Figure 3. Idealized four quadrant Stress-Strain behavior of the Stress Induced Martensite in NiTi Superelastic Alloy.

DEVICE DESIGN

Use of damping devices which are based on alloys with a thermoelastic martensitic transformation has several distinct advantages over other damping technologies in common use. They will not degrade over time like polymeric materials, they cannot leak like hydraulic dampers, they do not strain harden in a few cycles like mild steel dampers and they are not subject to sticking or change with atmospheric conditions like friction dampers. Perhaps the single over-riding advantage of these dampers, though, is their extreme versatility in design and implementation. As an example, a schematic design of a possible damper is shown in Figure 4 in which the damping material is merely multiple loops of wire in a block-and-tackle type device. This damper can have a completely adjustable stroke by varying the length of the loops, can change the supportable force by changing the number of loops, and can even change the shape of the damping force vs stroke behavior by having some loops tighter than others. All of this is done with the same structural components and can be determined at the point of installation as required for each situation.

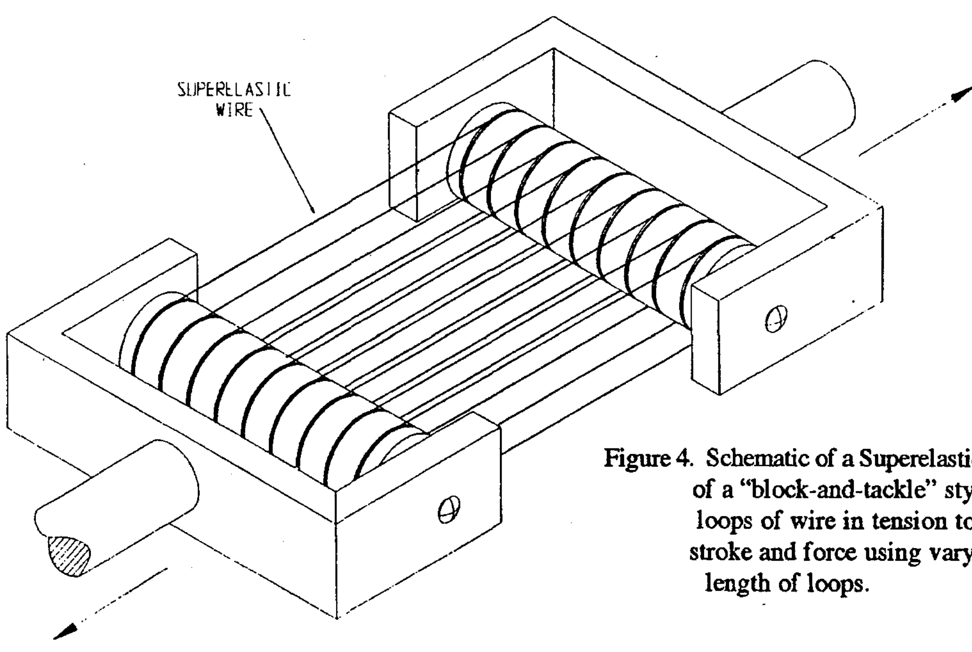

Figure 4. Schematic of a Superelastic damping device of a "block-and-tackle" style using multiple loops of wire in tension to give variable stroke and force using varying number and length of loops.

The device shown in Figure 4 would be used in constant tension mode, but a device of the type shown schematically in Figure 5 would allow push-pull movement. The superelastic element in this device is loaded to the middle of its superelastic strain limit when the device is constructed, and then pushing or pulling on the device will further load half of the element while unloading the other half of the element. This 'center-tapped' configuration will thus absorb energy if deformed in either direction. Another versatility of the Shape Memory Alloy based devices is that one can create almost any shape of force vs stroke behavior one wishes by using devices in which the element is transverse to the force stroke, by having various portions of the element installed at different tensions or strain levels, and using elements with a non-constant cross section along their length. Examples of many of these device variations could be shown, including those which use the elements in torsion or compression as well as tension, but space limitations prevent that in this paper.

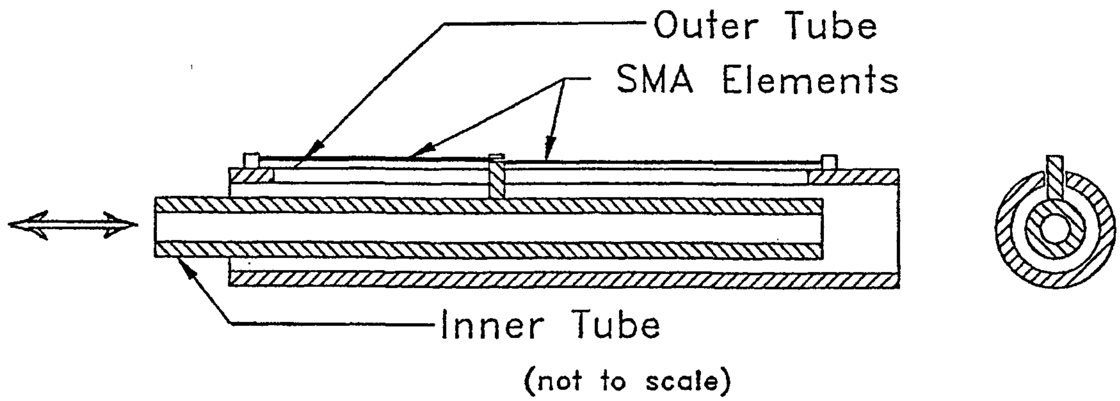

Figure 5. Schematic design of a "center-tapped" Superelastic damping device with a single SMA element to give a rectangular hysteresis curve.

CONCLUSIONS

In alloys which exhibit a thermoelastic martensitic transformation, there are three mechanisms whereby the structure can convert imposed mechanical work to heat in the alloy. For low amplitude higher frequency vibrations, the alloys have an extreme internal friction peak at the transformation temperature and can act effectively as sound or machinery vibration dampers. If the alloys are deformed in their martensitic phase, movement of the martensite platelet boundaries requires significant imposed force yet does not damage or work harden the alloy. Finally, if sufficient force is applied to create stress induced martensite in one of these alloys while it is in its austenitic state, the martensite will revert to austenite at a reduced stress but will leave a large portion of the imposed work as heat in the alloy.

Damping devices for structural applications which utilize the Shape Memory Alloys (SMA's) have a number of distinct advantages over other damping mechanisms. Since the critical components of such dampers are stable metals, they are not subject to atmospheric degradation, are not unduly temperature sensitive, are corrosion resistant and have good fatigue properties. The versatility of design which one can employ with such dampers is noteworthy. Using simple wire constructions allows one to vary the force, stroke and linearity of a damper using modular components. The shape of the damping force vs. deformation curve can be changed by reorientation of the wire elements, and the alloy can be used in tension, torsion or compression mode.

Testing of these alloys in structural dampers to confirm cyclic lifetimes, energy absorption, temperature applicability and overall reliability is in its infancy. The wide range of potential uses for such dampers, though, and the large amount of material needed for them, indicates that in the future such dampers could become one of the largest applications of these alloys.

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Seismic Control Devices Using Low-Yield-Point Steel

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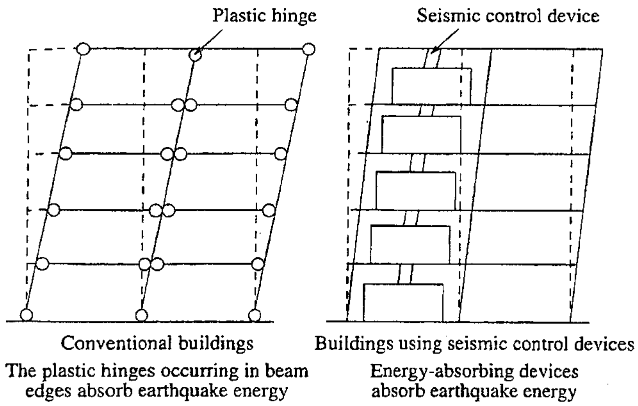
Abstract

With the purpose of developing seismic control structures that utilize the hysteresis energy absorption of steel products, this paper explains the development of new steel products for seismic control devices (dampers) and shows the performances and typical applications where such devices have been put to actual use. The authors investigated the properties required of steels for seismic control devices, developed two kinds of steel, LYP 100 (YP = 100 N/mm^2 class) and LYP 235 (YP = 235 N/mm^2 class), which have lower and narrower-range yield points (YP) and better elongation than conventional steels, and discuss their material properties. The authors also show the test results of the material properties of the new steels that relate to seismic control devices such as hysteresis characteristics, strain rate dependency, low-cycle fatigue characteristics. They then outline the development of unbonded braces and seismic control walls as examples of seismic control devices using the new steels, as well as the structural characteristics of each device. Finally, they introduce typical examples of actual designs which use these devices, demonstrating the seismic control effects of the devices using LYP steels.

Introduction

Conventional structural design of buildings in Japan achieves seismic performance by absorbing seismic energy, with the plastic deformation of the columns or beams of buildings. The damage in the Kobe Earthquake made it clear that conventional structural design (cf. Fig. 1(a)) which deforms and plasticizes such major structural members made it very difficult for repairs after the disaster. Also controlling seismic performance which corresponds to the importance level of buildings is difficult with the current design technique. As a solution to this problem, seismic control structures (cf. Fig. 1(b)) with seismic control devices (dampers) into buildings have been getting attention. The seismic control structures are designed to achieve seismic performance by making the devices absorb the seismic energy. This makes it possible to specify part materials for energy absorption, which has been ambiguous with conventional techniques, and to control seismic damage by identifying the performance. Also, specifying damaged positions facilitates repairs.

Seismic control structures are thus expected to be a key to future seismic design. Nippon Steel Corporation has been the first to conduct studies on seismic control structures that absorbing the hysteresis energy of steel products, and has developed a new steel product for this application. As a result of a 10-year-study, Nippon Steel has developed a new steel product for seis-



(a) **Conventional seismic design** (b) **Seismic control structure**

Fig. 1 Comparison of seismic design

Table 1 Chemical composition of LYP steel (Nippon Steel Corp. standard)

(mass%)

Steel type		C	Si	Mn	P	S
BT-LYP100	Nippon Steel standard	≤0.02	≤0.02	≤0.20	≤0.030	≤0.015
	Actual example	0.001	0.01	0.08	0.008	0.005
BT-LYP235	Nippon Steel standard	≤0.10	≤0.35	≤1.40	≤0.030	≤0.015
	Actual example	0.017	0.008	0.38	0.017	0.006

Table 2 Mechanical composition of LYP steel (Nippon Steel Corp. standard)

Steel type	Yield point (N/mm^2)	0.2%offset proof stress (N/mm^2)	Tensile strength (N/mm^2)	Elongation at fracture (%)
BT-LYP100	—	80-120	200-300	≥50%
BT-LYP235	215-245	—	300-400	≥40%

JIS Z 2201 No. 5 test piece should be used.

mic control devices. Furthermore, Nippon Steel has evolved the seismic control devices using the new steel products into joint studies with major users.

This report describes the results of the low-yield-point (LYP) steel developed for seismic control devices and seismic control technical development using these products.

Development of LYP Steel

Required performance of steel products for seismic control devices

The performance required of steel products for seismic control devices which will absorb hysteresis energy is, from the aspect of structural design, as follows: First of all, because the seismic control devices are passive to seismic input, preceding other structural part materials such as columns or beams and plasticizing at the designed stress level are feasible even with normal steel hysteretic dampers. But the use of a steel product which possesses a clearly lower yield strength and tensile strength than other structural part materials can easily achieve the above.

For this reason, it is necessary that yield strength should be low and scattering of yield point (YP) should be limited as narrowly as possible (narrow yield point). For a large earthquake, the device is going to undergo great repeated deformations in the plastic region, thus needs excellent elongation and low cycle fatigue characteristics. Workability and weldability necessary for construction are also required. Based on manufacturing technical data on works, new materials which possess the above required performances, two specifications for steel products, BT-LYP 100 and BT-LYP 235 in Tables 1 and 2 (hereinafter BT- will be omitted, described as LYP 100 and LYP 235 and both abbreviated as LYP steel) have been established as development objectives for Nippon Steel's internal use[1].

Characteristic of base materials for LYP steel

Fig. 2 shows LYP stress-strain curves compared to other steel products. Because LYP 235 has

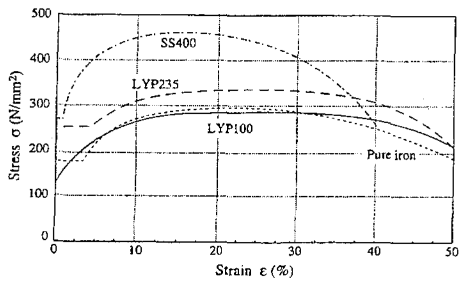
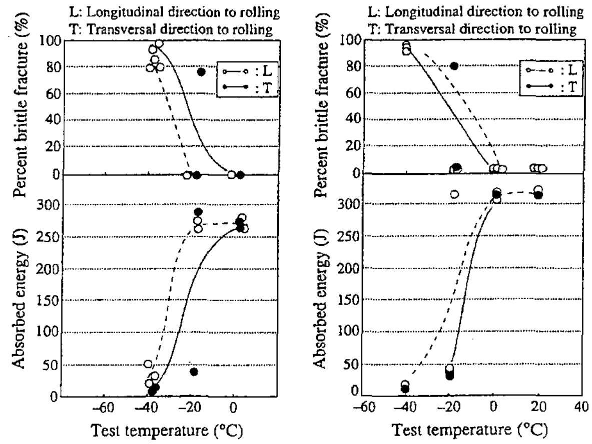


Fig. 2 Stress-strain curve of LYP steel



(a)LYP100

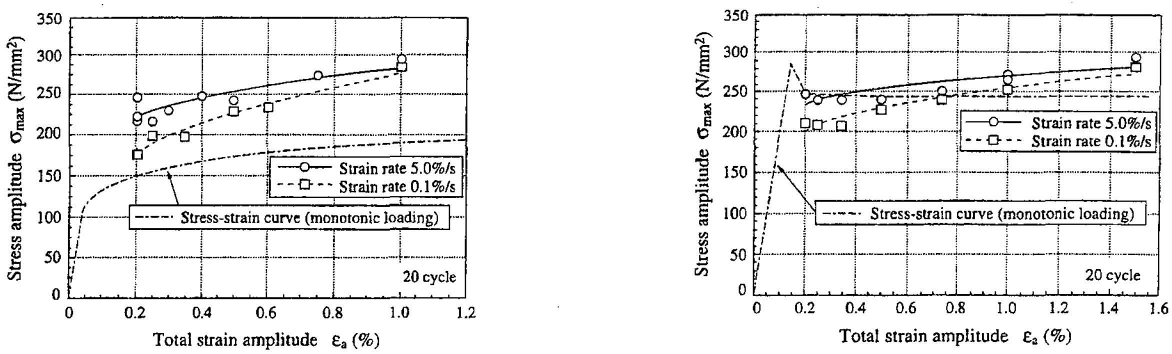
(b)LYP235

Fig. 3 Transition curve of absorbed energy in Charpy impact test of LYP steel(plate thickness: 25 mm)

yield points, but LYP 100 is a round-house with unclear yield points, the yield strength is taken at the time of 0.2% offset strain. Compared with other steel products, both have a bigger stress increase as a result of work-hardening after yield strength, this characteristic is apparent in device hysteresis characteristic. Fig. 3 shows the transition curves of the absorbed energy in the Charpy impact test. Both LYP 100 and 235 possess very good material impact characteristics. Yet such seismic control devices as have fast strain in thick materials and have a fatigue notch like weld joint must sometimes consider brittle failure and the ductility of base materials will be required.

The characteristics of the base materials that affect the performance of seismic control devices using LYP steel include hysteresis characteristic, strain-rate effect and the characteristic of low-cycle fatigue. The relationship between strain and stress in the fixed amplitude low-cycle fatigue test, with the amplitude parameter, is shown in Fig. 4[2], and compared with the monotonic loading test results. In the same way as normal steel products, repetition after a virgin loop causes hardening and the regular loop is made as in Fig. 5[3] after 4 to 5 cycles. The values in Fig. 5 plot the biggest stress and strain on the 20th cycle. The characteristic of hardening has a tendency similar to actual devices, but it is a little higher than the conventional steel products. In design, the structural performance test is conducted corresponding to the device type, e.g. axial or shear to set up hysteresis performance data for each device.

Fig. 6[4] is the research results on the strain-rate effect of LYP steel. The increase of yield strength and tensile strength in response to a is comparatively bigger than such conventional steels as SM490, whose effect must be considered according to the conceivable strain-rate of devices[5]. But it has been confirmed that percentage elongation after failure or uniform elongation have a small effect from strain-rate. Fig. 7[2] shows the low-cycle fatigue test results for base materials and the regression curve of repeated numbers from strain-amplitude to failure. Both LYP 100 and 235 have low-cycle fatigue characteristics similar to SS400. The actual design conducts the fatigue test for each device type to set up the design life.



(a) LYP 100

(b) LYP 235

Fig. 4 Relationship between stress amplitude and total strain amplitude

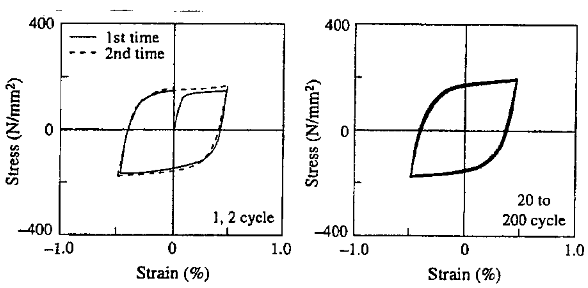


Fig. 5 Hysteresis curve (Example of LYP 100)

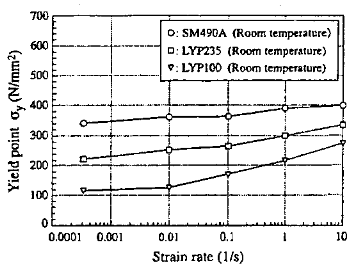


Fig. 6 Strain-rate effect of LYP steel

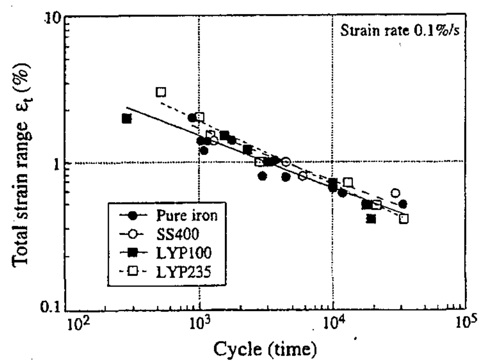


Fig.7 Low cycle fatigue test of LYP steel

Developing Seismic Control Devices

Development of seismic control devices using LYP steel

Fig. 8 shows the types of seismic control devices developed using LYP steel up to now, by Nippon Steel, various construction companies and design offices[7]. Types of part materials can be classified into steel products that have axis yield ((a) in the figure) and products that have shear yield ((b) to (e) in the figure). A typical device of the axis strength type is the unbonded brace which is described later. The shear type device has panels with shear board reinforced from outside with ribs or surrounding plates.

The part materials of each type are assembled in a processing plant and on site they are attached to surrounding frames with gusset plates and friction bolts. The devices of the shear type, which are often used, as is shown in (c) to (e) in the figure, have shear panels intensively set up between part materials to make deformation concentrated and facilitate replacement.

The two types of part materials developed at Nippon Steel are detailed below:

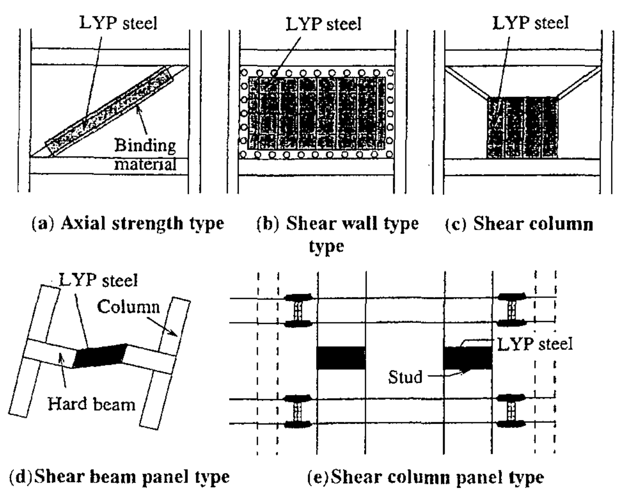


Fig. 8 Types of seismic control devices using LYP steel

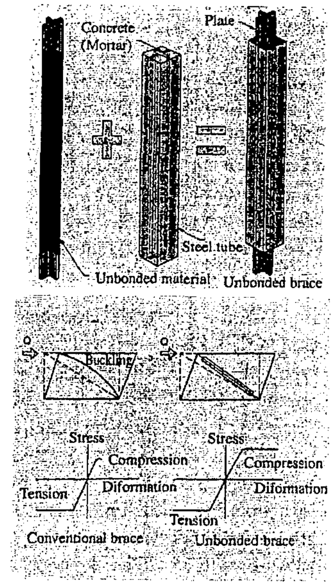


Fig. 9 Unbonded braces

Developing unbonded braces using LYP steel

Unbonded braces are axial hysteretic dampers which are made of flat plates or cross core materials, bound with steel tube concrete through unbonded materials to avoid buckling (cf. Fig. 9). As seismic resistant members and seismic control devices with the precise equivalent hysteresis characteristic in both tension and compression, unbonded braces were developed by Nippon Steel in 1986 and have been applied to over one hundred buildings since then[8]. SS400 or SM490 are often used for core materials and the general authorization by Japanese Ministry of Construction for structural members is acquired in this capacity. Recently, use of LYP 100 and LYP 235 for core materials has been increasing in order to decrease the seismic response by plasticizing at a tremor with less than level 1 and to assure the yield at the intended earthquake level in design.

Fig. 10 shows a practical-scale experiment of the unbonded braces using LYP 100 as the core materials and the load-deformation relationship at the time of static repeated loading[7]. The effect from the work-hardening is a little more apparent than with normal steel but the extremely stable spindle-shaped loop can be obtained. Fatigue and other experiments on part materials have confirmed that the repeated deformation capability in relation to strain amplitude ± (equivalent to story deformation angle 1/100) totals over 200 times or so (cumulative plastic rate is about 7200)[9].

Fig. 11 shows the detailed example of unbonded brace edges using LYP 100 as the core materials (cross type). To prevent bolt slips as a result of developing initial strain, normal steel (SN490B) is used for the edges, which are connected to the core materials by full-penetrated welding. And the core materials inside the bound materials are made to have preceding yield and in order to keep the stress in the welded areas under a certain level after the work-hardening of the core materials, two steps of haunch are set in core material edges and the welded cross section 1.5 times as large as the core materials is secured.

Developing response control walls device using LYP steel

As to the shear type panels, in addition to making the wall type in Fig. 10(b) a standard item for general use, the stud type in Fig. 10(c) is also being jointly developed with design offices and construction companies. Fig. 12 shows the detailed example of attaching the wall type using LYP 100, the three-story shear load experiment of response control walls and the obtained load-deformation relationship. The shear panels have rib plates attached on the front vertically and on the back horizontally. The ratio of the width and thickness of the bound panels should be 80 and the range is set up so as not to cause local buckling, which precedes the shear yield of the panels. The ribs are designed not to cause buckling of the overall walls, including the ribs themselves. As for the obtained restoration strength characteristic, in relation to the biggest deformation angle 1/75, the stable loop, a cumulative plastic deformation rate up to around 250 was confirmed.

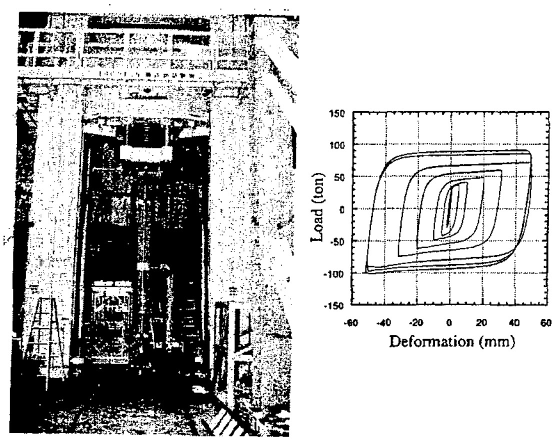


Fig. 10 Experiment of practical-scale unbonded braces using LYP

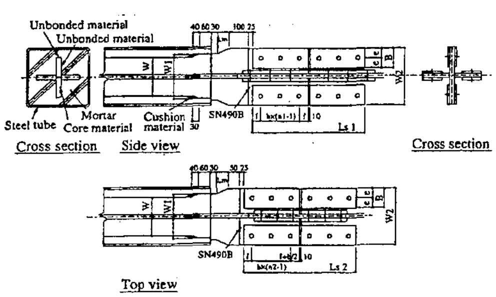


Fig. 11 Detailed example of unbonded brace edges using LYP 100

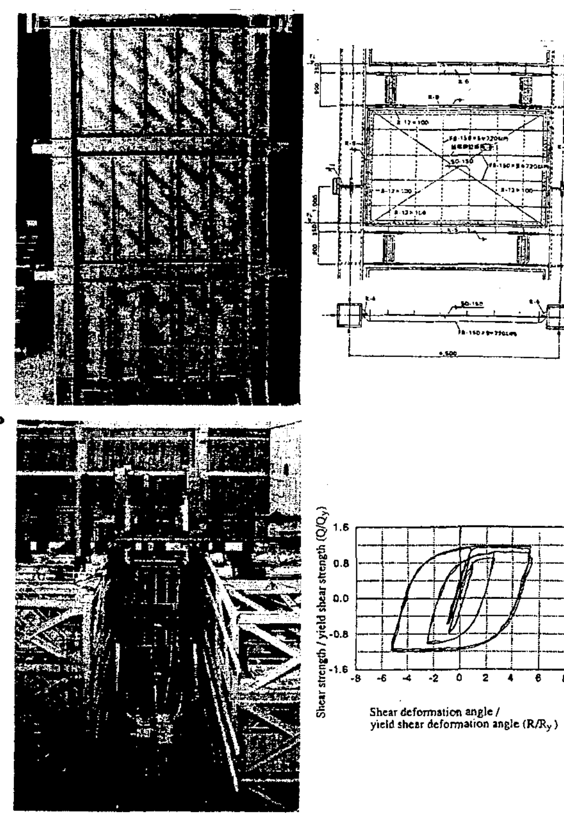


Fig. 12 Load test of shear wall using LYP 100

Application to Actual Buildings

Passage Garden North 1st Section

Building design: Plantech

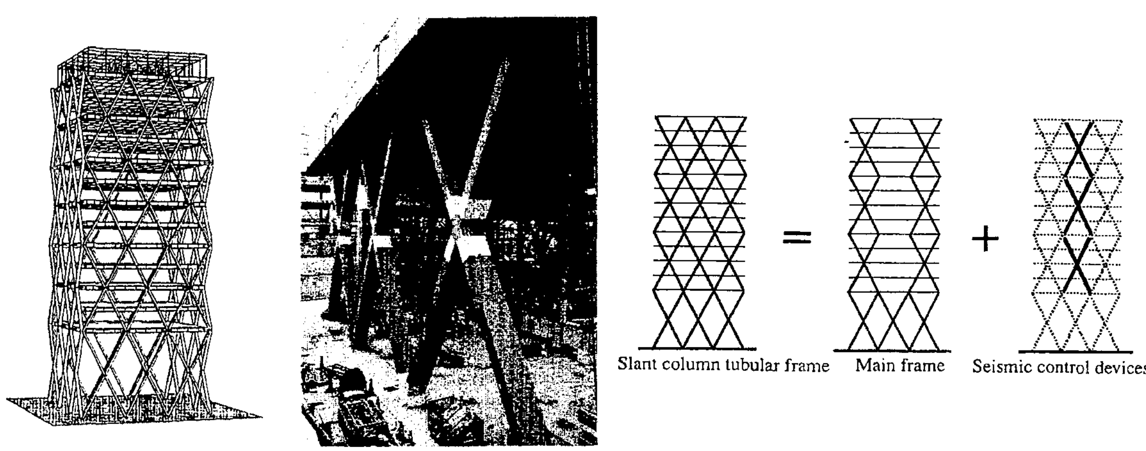
Structural design: Alpha Structural Design

Collaboration: Nippon Steel Corporation

This is an office building with 14 stories above ground and 2 floors underground, 63 m in height and about 10,000 m^2 in total floor area, in front of Shibuya Station. The entire building is supported by the outside slanted lattice frames. This outside frame is designed to be expressed as outer design (cf. Fig. 13(a)).

The outside frame must not only support the building's own weight, but must also resist the lateral strength of an earthquake, and all part materials must be axial strength part material. The buckling of the lattice material in an earthquake loses the independence of the building and leads to the overall collapse, which must be avoided. On the other hand, since it is not economical to have all the part materials be elastically designed against level 2 stress, the damage tolerant design with unbonded braces which use LYP 100 was applied to realize an economical frame.

Fig. 13(c) shows the structural planning outline. Into part of the lattice unbonded braces are inserted as seismic control devices and the preceding plasticizing of this part ensures the absorption of earthquake energy and avoids the buckling of the other part materials that support the building. As a result, the effects of hysteresis damping and longer period of the device decreased the base shear of level 1 to about half of that of an elastic design without a seismic control device. The LYP 100 used for the unbonded brace core materials are flat plates, 500 mm wide, 40 mm thick, and about 9 m long. These part materials begin plasticizing at an earthquake less than level 1 and reach the biggest plastic rate of 3.4 at a level 2 tremor. On the other hand, other part materials supporting vertical load materials stay at an axial strength of less than around half



(a) Frame outline (b) Actual site (c) Damage level control design

Fig. 13 Application cases of LYP steel unbonded braces

of buckling load, even with a level 2 tremor. And it is estimated that the biggest strain rate occurring in seismic control part materials is about 0.7%/sec at level 2 earthquake.

This building has cumulative axial transition measuring equipment attached, separately developed by Nippon Steel, to control the cumulative plastic rate of the seismic control device against wind load or minor earthquakes[10].

Conclusion

This paper has described the material characteristic of LYP steel, the actual situation of the R&D and practical application of seismic control devices that use this steel product. Such seismic control structures are likely to spread widely in application from high-rise buildings to general buildings, as a seismic design technique that responds to the requirement to specify building performance following the amendment of the Construction Standard Law. The seismic control devices using LYP steel in this report are cheaper and more dependable than others, and are expected to become major devices.

The future research agenda involves clarifying the required performance of steel products, unifying the standards of steel products, grasping the dynamic characteristics and generalizing the design methods.

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Use of FRP Composite Materials in the Renewal of Civil Infrastructure in Seismic Regions

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Abstract

Fiber reinforced polymer (FRP) composites have immense potential to effectively assist in the renewal of deteriorating civil infrastructure. Their intrinsic properties of light weight, ease of conformance, high stiffness-to-weight and strength-to-weight ratios and potentially high durability make them ideal choices for purposes of repair, retrofit and strengthening in addition to their use in new structural systems. This paper provides an overview of their use in seismic retrofit and strengthening with specific emphasis on salient aspects of the technology that make it attractive for use in confined spaces and where minimal disruption to occupants and facilities is desired. Opportunities and challenges for the future are also briefly discussed.

Introduction

Fiber reinforced polymer (FRP) matrix composites, consisting of stiff and strong reinforcing fibers (primarily carbon and glass), held together by cost-effective, tough and environmentally durable resin systems show immense potential to add to the current palette of materials being used in civil infrastructure for reasons ranging from light weight (which would result in a lower self weight, ease of use in repairing weak structures unable to carry additional weights of conventional materials during possible strengthening, or the efficient rehabilitation of internal walls and columns without decreasing clear internal space), and increased durability (resulting in lower overall life cycle cost, and the attendant relaxation of the crippling need for large maintenance budgets), to controllable thermal properties (which may be advantageous in combating material cracking due to through thickness temperature gradients) and tailored performance (which can be advantageous in cases such as the seismic retrofit of columns wherein there exists the need for hoop confinement without substantial increase in axial stiffness of the structure) [1]. Table 1 shows the range of typical mechanical properties, which would be attained, from composites used in civil infrastructure as compared to structural steel. It is noted that the use of higher strength or higher modulus fibers (such as carbon fibers with a modulus in excess of 750 GPa) can result in substantially higher levels of performance, but these fibers are currently too expensive to use in routine civil infrastructure applications.

Table 1: Comparison of typical range of composite characteristics with those of steel

Property	Range	Comparison with steel
Modulus	20-138 GPa	1/10 to 2/3 that of steel
Strength	340-1700 MPa	1-5 times the yield strength of mild steel
Strain Limit	1-3%	1/10 to 1/5 that of mild steel
Weight	12-19 kN/m^3	4 to 6 times lighter than steel

Enhanced understanding of structural response and natural threats such as earthquakes and storms has lead to the establishment of new design codes and the consequent need to rehabilitate existing structures to ensure their continued safety. Conventional materials such as timber, steel and concrete have a number of advantages, not the least of which is the relatively low cost of raw materials. However, it is clear that conventional materials and technologies, although suitable in some cases, and with a fairly successful history of past usage, lack in longevity in some cases, and in others are susceptible to rapid deterioration, emphasizing the need for better grades of these materials or newer technologies to supplement the conventional ones used. In some cases retrofit and rehabilitation of existing structures with conventional materials is in itself not possible with the traditional recourse being demolition and reconstruction if budgets permit. In all such (and other) cases, there is a critical need for the use of new and emerging materials and technologies, with the end goal of facilitating functionality and efficiency while increasing the overall durability and life span of the structures. In addition in cases related to the retrofit and strengthening of facilities such as hospitals, schools and other buildings, there is a critical need for the completion of the rehabilitation measures with minimal disturbance to the facilities and occupants. FRP composites present a number of attractive features in all these cases.

In this paper, a brief overview of application to seismic retrofit, repair and strengthening is provided. The reader is cautioned that this area is still in its infancy and hence there are significant areas where comprehensive data is lacking or where there is disagreement over the efficacy of specific techniques.

Seismic Retrofit Of Columns

Recent earthquakes such as Whittier '87, Loma Prieta '89, Northridge '94 and Kobe '95, have repeatedly shown the vulnerability of existing bridge columns built before the 1971 San Fernando earthquake. For reinforced concrete columns conventional retrofit measures range from the external confinement of the core by heavily reinforced external concrete sections, to the use of steel cables wound helically around the existing column at close spacing which are then covered by concrete, and the use of steel shells or casings that are welded together in the field confining the existing columns. Although some of these methods are very effective, (a) they are time consuming needing days for installation, (b) can cause significant traffic disruption due to access and space requirements for heavy equipment, (c) rely on field welding, the quality and uniformity of which is often suspect, and are (d) susceptible to degradation due to corrosion. In addition, with steel casings or jackets, due to the isotropic nature of the material, the jacket not only provides the needed confinement, but also causes an increase in stiffness and strength capacity of the retrofitted column, both of which are not desirable since typically higher seismic force levels are transmitted to adjacent structural elements. The use of fiber reinforced composites not only provides a means for confinement without the attendant increase in stiffness (through the use of hoop reinforcement only, i.e. no axial reinforcement), but also enables the rapid fabrication of cost-effective and durable jackets, with little to no traffic disruption in a large number of cases.

A number of different methods (based on form of jacketing material or fabrication process) have been tested at large or full-scale, many of which are now used commercially in

Japan and the U.S. Generically, fiber reinforced composite wraps (or jackets) can be classified into six basic types as shown in Figure 1. In the wet lay-up process fabric is impregnated on site and wrapped around a column with cure taking place under ambient conditions. In the case of wet winding, the process of fabrication is automated but essentially follows the same schema, with the difference that the ensuing jacket has a nominal prestress due to the use of winding tension.

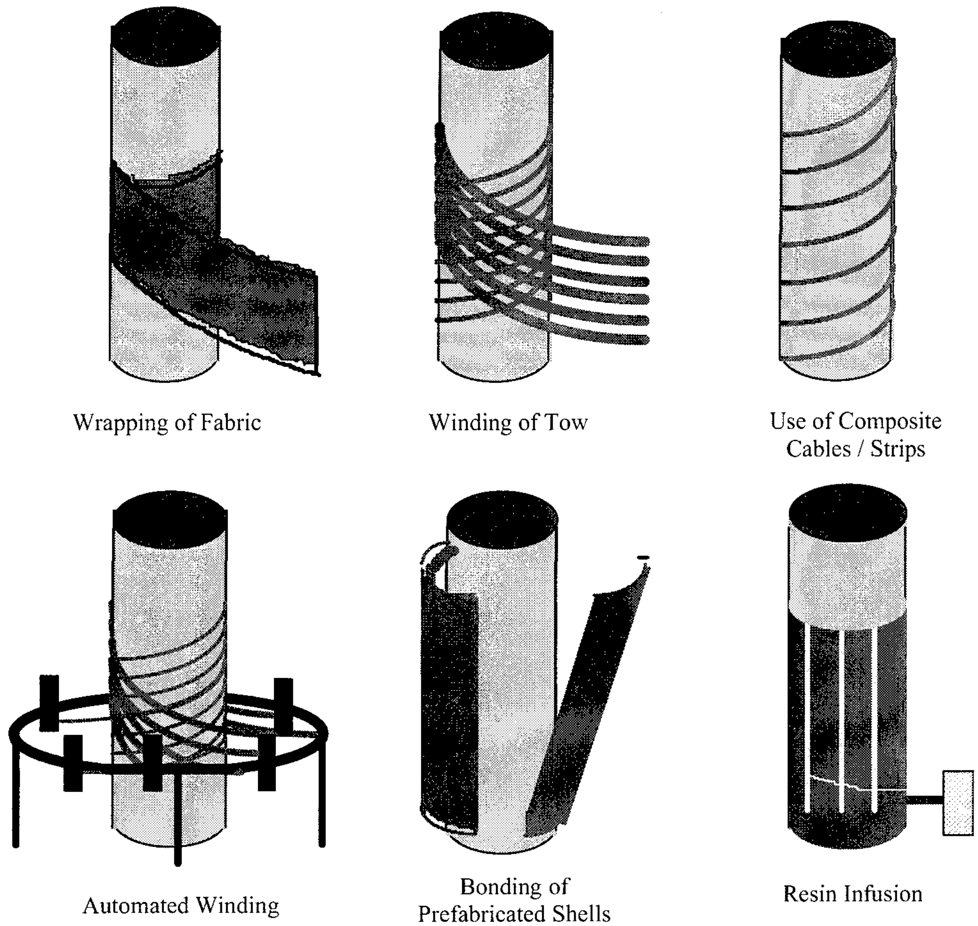
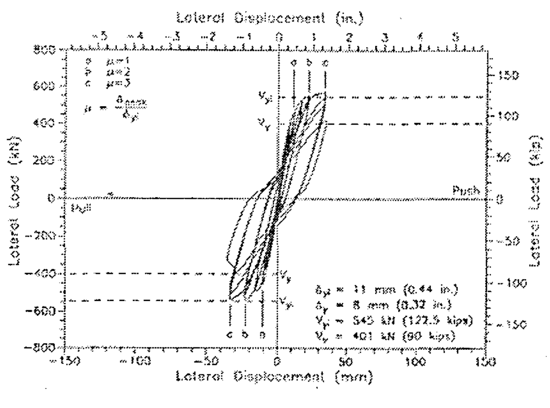


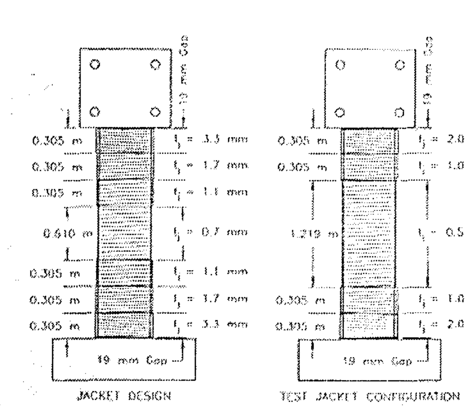
Figure 1: Schematic Showing Methods of Processing/Fabrication of Composites for the Seismic Retrofit of Columns

The use of prepreg tow presents the opportunity not only for elevated temperature cure (with the consequent advantages of higher degree of cure, higher glass transition temperature, and greater overall durability associated with moisture and humidity effect), but also the use of standardized and uniform materials that are easy for the civil designer to specify. In the case of adhesively bonded shells the concept of uniformity and standardization is carried even further through the use of prefabricated single- or dual-section jackets which can be assembled in the field through bonding and layering. This process affords a high level of materials quality control due to prefabrication of the elements under factory conditions, but as in the case of external strengthening, relies on the integrity of the adhesive bond. Aspects related to materials selection and details of manufacturing processes and/or installation schemes are described in [2] and hence will not be repeated herein.

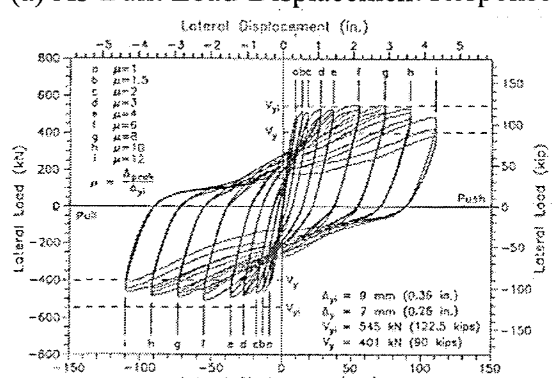
The structural effectiveness of composite jackets has been demonstrated through a large number of large- and full-scale tests [3, 4], and through demonstration projects in the United States and Japan [5]. Design examples and methodology are detailed in [3, 6]. In most cases the use of composites results in a cost-effective and structurally efficient retrofit scheme that significantly increases structural ductility, and thereby seismic resistance. Figure 2 shows retrofit design details for a test shear column emphasizing the extremely small thickness of carbon fiber/epoxy jacket required to attain completely stable load-deflection hysteresis loops up to a displacement ductility level of μ_Δ = 12 (Figure 2c). Comparative load-deflection curve envelopes for the unretrofitted or "as-built" shear column, a steel jacketed system with 5 mm jacket thickness, and the carbon fiber/epoxy jacket as detailed in Figure 2b, are shown in Figure 2d, depicting a clear improvement of deformation capacity in both the steel and the composite retrofit cases over the "as-built" case which failed in brittle shear at a displacement ductility of $\mu_\Delta = 2.0$.



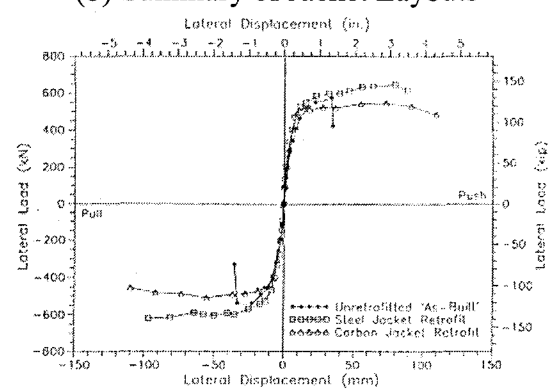
(a) As-Built Load-Displacement Response



(b) Summary of Jacket Layouts



(c) Hysteresis Curve for Carbon/Epoxy Jacket



(d) Comparison of Load-Dispacement Envelopes

Figure 2: Specimen Details for Carbon/Epoxy Jacket for Shear Retrofit of Rectangular Column in Double Bending

The determination of thickness of the composite retrofit depends on the requirements to correct the failure modes expected under seismic load/deformation which can be differentiated into four general classes of shear strengthening, plastic hinge confinement, bar buckling restraint, and lap-splice clamping. A comparison of jacket thicknesses for three typical systems is shown in Table 2, wherein system A is representative of a tow-preg based graphite/epoxy composite similar to that used in automated winding, system B is representative of an Aramid/epoxy system similar to that used in Japan using wet lay-up, and system C is representative of an E-glass/Vinylester similar to that used in prefabricated, adhesively bonded shells. All values in Table 2 are normalized to the thickness values determined for System A. It can be seen that jacket thicknesses for shear, bar-buckling restraint, and lap splice clamping are driven by the modulus of the jacket in the hoop direction which favors the selection of higher modulus materials (carbon, aramid), whereas the requirements for flexural hinge confinement can be efficiently achieved with a lower modulus but high strength and higher strain capacity material (e.g. E-glass).

Table 2: Comparison of hypothetical jacket thicknesses

System	Mechanical characteristics	Normalized jacket thickness			
		Shear strength	Plastic hinge confinement	Bar buckling restraint	Lap splice clamping
Proportionality relationship		$t_j^v \sim \frac{1}{E_j D} \bullet C_v$	$t_j^c \sim \frac{D}{f_{ju}\varepsilon_{ju}} \bullet C_c$	$t_j^b \sim \frac{D}{E_j} \bullet C_b$	$t_j^s \sim \frac{D}{E_j} \bullet C_s$
System A	$E_j = 124$ GPa $f_{ju} = 1.380$ MPa $\varepsilon_{ju} = 1\%$	1	1	1	1
System B	$E_j = 76$ GPa $f_{ju} = 1.380$ MPa $\varepsilon_{ju} = 1.5\%$	1.6	0.7	1.6	1.6
System C	$E_j = 21$ GPa $f_{ju} = 655$ MPa $\varepsilon_{ju} = 2.5\%$	6.0	0.9	6.0	6.0

1 MPa = 0.145 ksi, 1 GPa = 145 ksi

Structural Overlays For Walls

The seismic retrofit and upgrade of walls can be conducted in rapid and effective fashion through the appropriate placement of composites on the surface with fibers oriented horizontally so as to intersect diagonal/shear cracks and thereby constraining them while still enabling the horizontal/flexural cracks to open and thereby ensure overall ductility. It is emphasized that the force transfer into the composite is limited by interlaminar shear strength developed between the masonry and composite, and the tensile strength of the existing material at the wall surface. Retrofit can generally be enabled through the use of a few layers of appropriate basis weight reinforcing fabric impregnated with an appropriate resin system. To improve shear capacities of structural walls of length d with advanced composite overlays of thickness t_o and a conservative diagonal tension crack angle assumption of 45°, the resulting shear capacity increase can be determined based on an allowable overlay stress level derived for a maximum horizontal wall strain of 0.004 above which aggregated interlock is assumed to be lost. For typical structural wall aspect ratios, i.e. height and length of approximately the same dimensions, the above strain criteria inherently assumes large shear deformations, namely 0.4% drift due to shear alone.

Thus, additional limitations on the total allowable shear deformations can be imposed by reducing the allowable overlay stress level. Alternatively, stiffness criteria can be employed in the wall overlay design, limiting shear deformations to deformation levels which can be expected in concrete walls with conventional horizontal reinforcement A_{sh}^{req} (determined based on conventional design requirements), by scaling the amount of horizontal overlay fabric A_{oh} from the equivalent required horizontal steel reinforcement and the ratio of steel to overlay stiffness, as

$$A_{oh} = A_{sh}^{req} \frac{E_s}{E_o} \qquad \ldots\ldots 1$$

Assuming that the composite-substrate bond is at least as good as the bond capacity between conventional steel reinforcement and unconfined concrete an upper limit to the overall enhancement of shear capacity can be estimated through use of the conventional design code regulations, such as in ACI-318 where

$$(V_o)_{\max} = (V_s)_{\max} = 0.66\sqrt{f_c^{'}}\ b_w d \qquad \ldots\ldots 2$$

where $f_c^{'}$ is the nominal concrete strength in compression in MPa, b_w is the wall width, and d is its effective length.

In a large number of cases the seismic deformation limits are controlled by crushing of the toe in compression or by limits on lateral stability of the compression toe region. Nominal levels of confinement can be provided by wrapping additional layers of composite around the toe region and further stability is ensured through connection between the wall and the floor using Simpson ties. Further details on experimental validation can be found in [4, 7].

Slab Strengthening And Modification

Degradation due to corrosion of steel reinforcement, spalling of concrete cover, extensive cracking of concrete due to excessive carbonation or other actions, effects of alkali-silica reaction, and rapidly changing occupancy needs have created a critical need for methods of repair and strengthening of slabs. Conventional methods for this range from the use of external post-tensioning to the addition of epoxy bonded steel plates to the soffit of the deck superstructure. Although the latter technique is simple and has been used in Europe in the past it suffers from a number of disadvantages ranging from difficulty in placement, to concerns related to overall durability and corrosion resistance. Steel plates are heavy and unwieldy and hence difficult to handle during erection. At the minimum, jacks, extensive scaffolding and winches or cranes are needed. The length of individual plates is restricted to a maximum of 6-10 m so as to facilitate handling, and even at these lengths, it may be difficult to erect them due to pre-existing service facilities below the deck slab. Composites fabricated either through wet processes at site or prefabricated in strips and then adhesively bonded to the concrete surface provide an efficient and easy means of strengthening that can be carried out with no interruptions in traffic flow and as-such will not be discussed in depth herein. The strengthening or repair of slabs is increasingly being considered of late, but it should be kept in mind that due to differences in conventional steel reinforcement detailing and structural response between beams and slabs, results derived from the application of composites to beams cannot be directly extrapolated to application of slabs especially as related to the selection of form and positioning of the external reinforcement [8].

In general, composites can be applied in three ways as described in Table 3, of which the first two are the most widely used. It should be noted that although the wet lay-up process afford significant flexibility for work on site, and is still by far the most commonly used process in the field, there may be significant advantages, technical and psychological, in the use of prefabricated and hence presumably standardized strips and plates, which are adhesively bonded to the concrete substrate. Figures 3 and 4 depict the application of unidirectional carbon fabric placed using the wet lay-up process, and the adhesive bonding of prefabricated carbon fiber pultruded strips, respectively, to the underside of a slab for purposes of external strengthening.

Figure 3: In-Place Wet Layup of Fabric

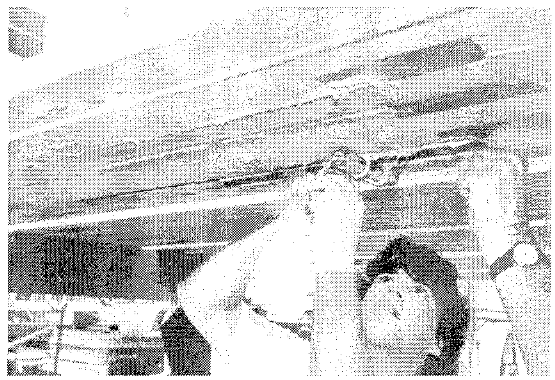


Figure 4: Adhesive Bonding of Prefabricated Composite Strips

Table 3: Methods of Application of External Composite Reinforcement for Strengthening

Procedure	Description	Time/Issues
Adhesive bonding	Composite strip/panel/plate is prefabricated and then bonded onto the concrete substrate using an adhesive under pressure	• Very quick application • Good quality control
Wet lay-up	Resin is applied to the concrete substrate and layers of fabric are then impregnated in place using rollers and or squeegees (or a preimpregnated wet layer of fabric is squeezed on). The composite and bond are formed at the same time	• Slower and needs more setup. • Ambient cure effects • Waviness/wrinkling of fiber • Non-uniform wetout and/or compaction
Resin infusion	Reinforcing fabric is placed over the area under consideration and the entire area is encapsulated in a vacuum bag. Resin is infused under vacuum. In a variant the outer layer of fabric in contact with the bag is partially cured prior to placement in order to get a good surface	• Far slower with need for significant setup • Ambient cure effects • Dry spots

It is important to note that the efficacy of the method depends primarily on the appropriate selection of the composite material based on stiffness and strength requirements, and the efficiency and integrity of the bond between the concrete surface and the composite. The bond between the composite and concrete, whether it be established through the use of an adhesive or through the use of the same resin system as is used in the wet lay-up of the composite itself, must not only be capable of performing under ambient conditions, but must also be capable of providing the required response under extremes of temperature (including temperature gradients between the top and bottom surfaces of concrete) and the resulting stress and strain conditions, and in the presence of moisture (which can not only be absorbed from the atmosphere, but could also collect at the adhesive-concrete interface due to moisture collection and ingress through concrete which itself is a porous material). Results of durability testing of the bond are given in [9, 10] and are hence not repeated herein.

Irrespective of the method used, the external application of composites to concrete beams and slabs, if conducted in an appropriate manner, can result in the significant enhancement of load carrying capacity, flexural and shear strength of the original structural element. The results from a series of scale slab tests conducted on specimens of size 2.29 m length x 0.48 width x 0.1 m depth reinforced with 3 # 3 grade 60 bars at 0.20 m spacing in the longitudinal direction, and # 2 bars at 0.1 m spacing in the transverse direction are shown in Figure 5. It can be seen that with the use of external composite reinforcement, both through the use of fabric placed using wet lay-up and pultruded strips which were adhesively bonded to the surface, the load carrying capacity of the slabs is dramatically increased, whereas the ductility (or deformation capacity) at initial failure (albeit at a significantly higher load than possible with a typical reinforced concrete slab) is drastically reduced. Although the procedure provides an efficient means of deck strengthening, due care has to be taken to ensure that the rehabilitation design addresses the possibility of elastic failure of the system with a sudden drop in strength when the composite fails through catastrophic fracture or through failure of the composite-concrete bond interphase, through the use of limits on capacity increase related to yielding of steel reinforcement, or through the use of an appropriately factored equivalent energy based design approach. Further details of slab strengthening can be found in [11, 12, 13].

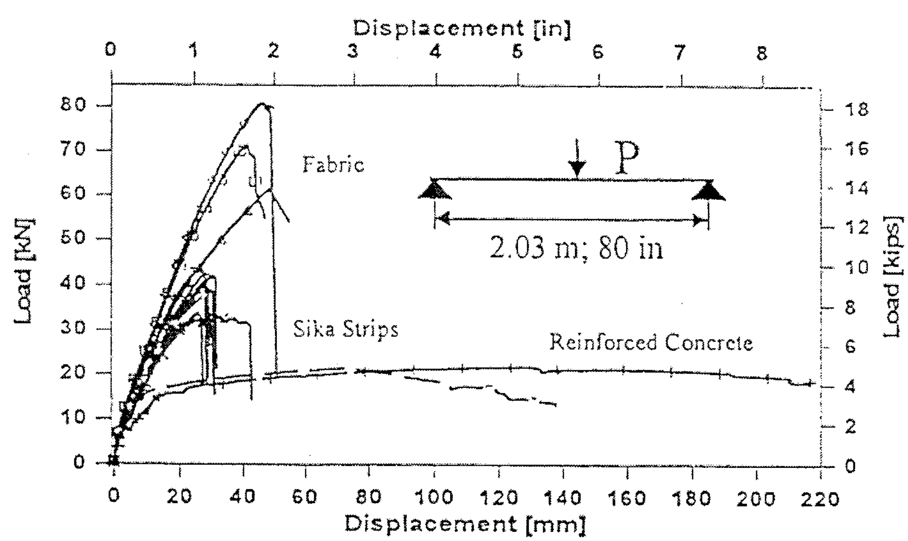


Figure 5: Effect of External Application of Composites for Slab Strengthening

The use of externally bonded composite strips as a means of strengthening of existing concrete structural elements such as slabs is of special interest in enabling the modification of existing structures through the addition of elevators, escalators, service facilities through the cutting of holes in existing slabs. In order to provide these facilities, areas of existing slabs, including the steel reinforcement, are cut necessitating the use of additional columns and walls to support the weakened slab. FRP composite strips can, however, be used around the cut-out to externally reinforce and slab and carry load locally, redistributing it back into the remaining structure, without the need for additional construction. This not only saves space that would otherwise have been needed for construction of additional supporting members around the cutout but also saves cost. In a recently completed series of tests, the response of unstrengthened and strengthened full-scale slabs, of size 6 m x 3.2 m with a central cut-out of size 1 m x 1.6 m, was investigated. The objective of the test was to externally strengthen the slab, after the cutout was made, using prefabricated carbon/epoxy pultruded strips that were adhesively bonded to the tension face of the slab. Equivalent point loads were applied in the longitudinal and transverse directions on either side of the cut-out in two separate sets of tests (unstrengthened and strengthened) for transverse and longitudinal loading, to observe biaxial bending behavior as well as shear behavior in the slab. Strengthening schemes were based on determination of external reinforcement area required for rehabilitation through finite element analysis. Prefabricated (pultruded) FRP composite strips of 1-mm thickness and 100 mm and 50 mm width were used. For loading along the transverse direction a total of four 50 mm width strips of length 250 cm and two 100 mm width strips of length 350 cm were placed in the longitudinal direction centered on either side of the cutout, with one 50 mm width strip placed on either side of the cutout in the transverse direction over the width of the slab at 25.4 cm from the edge of the cutout. Figures 6a and b show a comparison of results between the unstrengthened and strengthened slabs, tested in the transverse and longitudinal directions, respectively.

As can be seen, the use of the FRP strips results in the slab regaining its initial capacity, albeit with less deformation capability. It is noted that in addition to enabling load distribution through the strips resulting in a recovered load capacity, the FRP composite also restrains crack growth especially in areas of local stress concentration formed through the construction of the cutout. Figures 7(a) and (b) depict configurations of the final failure mode of strip debonding with the horizontal crack being either within the cover concrete or within the FRP composite itself.

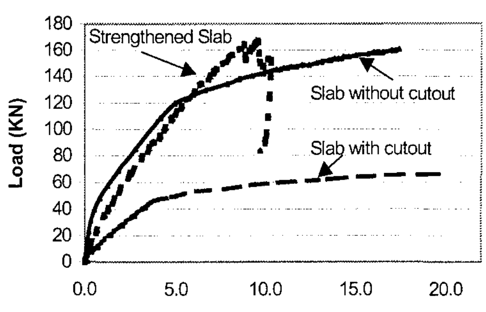


Figure 6(a): Load-Displacement Response Under Transverse Loading

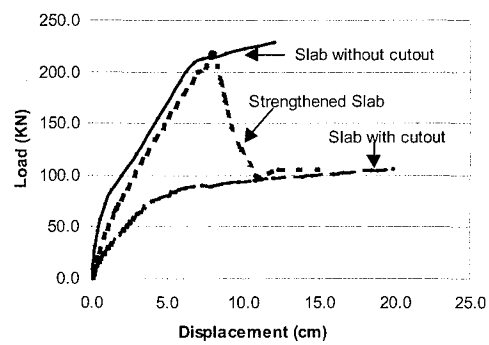


Figure 6(b): Load-Displacement Response Under Longitudinal Loading

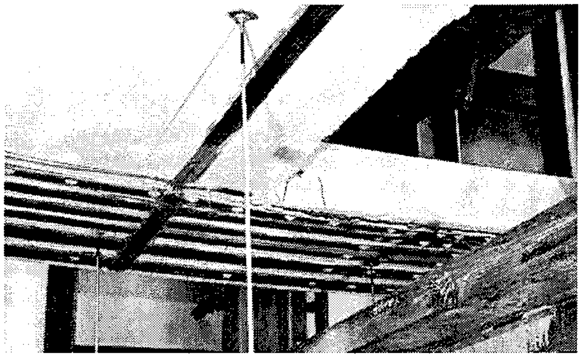


Figure 7(a): Initial Debonding of Strips at a Corner

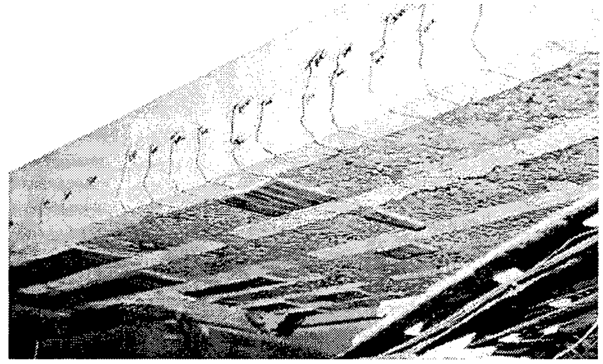


Figure 7(b): Debonding of Strips

Although the initiation of failure is approached in fairly linear fashion, there is significant prior warning through local debonding and concrete cracking. It is also noted that after debonding of the FRP composite the slab response follows that of the original slab with the cutout, thereby emphasizing that if appropriately designed, failure would not be catastrophic.

Challenges And Conclusions

Although the use of FRP composites for the renewal of civil infrastructure is increasing rapidly, and there is no doubt that the intrinsic tailorability and performance attributes of both glass- and carbon-fiber reinforced composites make these materials very attractive for use in civil infrastructure applications and a welcome addition to the current palette of construction materials there are a significant number of challenges that must be overcome before FRP materials can truly be considered at the level of other construction materials such as steel, timber, masonry and concrete.

There is a critical need for further development of new composite material systems (primarily in the area of hybrids and textile structural composites), and new or modified processes capable of fabricating large structural components in a cost-efficient manner. These developments are likely to range from the production of cheaper carbon fibers, and E-glass fibers with greater alkaline resistance, to the greater use of hybrid composites with hybridization taking place at the level of the tow as well as in textile preform structures. Needs for greater resistance to fires has already spurred the development of phenolic resins, while requirements for the fabrication of large structural components using low cost processes such as pultrusion, resin-infusion, and RTM, in an environmentally conscious industry has lead to the development of low viscosity, lower-styrene- and volatile-content vinylesters and phase-transforming resin systems. Field conditions and the intrinsic need for composites that must be able to perform under wide ranges of temperature and humidity levels is leading to intense research in areas related to alternate cure mechanisms capable of achieving rapid cure and higher than ambient glass transition temperatures without the use of external heat.

By their very nature, most composite components used in civil infrastructure have to be cost-competitive with incumbents made of conventional materials, on an acquisition cost basis, if not at the component level, then at least at the systems level. Further, the very nature of the applications set almost assures that significant fabrication will be conducted in the field, or in partially controlled, but changing environments. The exigencies of size also dictate that by-and-large traditional autoclave based fabrication will not be used widely. The emphasis is increasingly being placed on processes such as wet lay-up, pultrusion, winding and resin transfer molding, and in the development of newer processes such as injection pultrusion and resin infusion. It should be noted that these and other developments, brought about due to the needs of civil infrastructure, will not only provide solutions specific to civil infrastructure requirements, but will also have a positive influence on other applications of composites.

Notwithstanding the high level of current interest in this area, the extent and nature of the future use of composites in civil infrastructure will depend on a number of factors, including, (a) resolution of outstanding issues related to durability, fire-resistance and repairability, insofar as to having a good determination of the level of knowledge and assurance of each of these factors, (b) development of manufacturing processes and schemes that are amenable to the high quality, repeatable and uniform production of primary structural elements in a cost-effective (as related to civil infrastructure economics) manner both in controlled factory conditions (for prefabricated elements) and in the field (for insitu fabrication), (c) development of validated codes, standards and guidelines for the use of these materials by the civil engineering community, (d) development of low cost insitu health-monitoring devices and schemes, especially to provide a level of comfort about the safety of use of structures/components fabricated from composites till a time when an appropriate history of in-field use has been attained, and (e) the synergistic education of both the civil engineering/construction and the composites communities about the needs and methods of development in both areas.

As compared to conventional civil construction materials such as steel, concrete, and even timber, fiber reinforced composite materials are both a designer's dream and a nightmare. The myriad combinations of fiber and resin systems possible and the capability for infinite tailoring of performance are aspects that are simultaneously both attractive, and a barrier, in an industry where uniformity and standardization are the norm. Although the creation of standardized laminates has been advocated by a number of people in the past, it must be recognized that, applied generically, this would unnecessarily curtail innovation and functional efficiency and will, more often than not, result in cost- and materials- inefficiency. The use of composites in aerospace applications has been predicated on extensive materials testing for the purposes of qualification, followed by strict adherence to prescribed specifications for autoclave based fabrication in highly controlled factory environments. Civil applications are more likely to (a) use processes such as wet lay-up, pultrusion and resin infusion than autoclave molding, (b) fiber and resin as separate constituents rather than in the form of preimpregnated material, and (c) resin systems such as polyesters, vinylesters, phenolics and lower temperature cure epoxies rather than the higher temperature curable epoxies and thermoplastics. Further, there is likely to be extensive use of processes under ambient conditions in the field, rather than fabrication in factory controlled environments. Thus, the civil engineering environment not only brings with it new challenges for the control of quality and uniformity of composites, but also makes it difficult (if not impossible) to use the well established databases generated by DoD sponsored research (such as those for AS4/3501-6 or T300/5208 based systems). Initial materials characterization and the assessment of durability and damage tolerance, in light of the service life periods required of civil structures, become of critical importance. A significant amount of attention is being paid to this aspect for the development of data bases and for the development and validation of accelerated test methods for the assessment of long-term durability. In the interim, structures are being designed using conservative principles. This often results in the use of factors of safety of 4-6 for glass fiber reinforced components, and factors as high as 2-3 for carbon fiber reinforced composite components. It should, however, be emphasized that a significant portion of civil design is predicated on stiffness criticality rather than strength criticality which alleviates the concern related to strength degradation as a function of environmental exposure and aging to a certain extent. Notwithstanding this, it is critical that durability of composite material forms used in civil infrastructure environments be resolved rapidly. A recent gap analysis shows critical areas of concern [14] related either to lack of data or inaccessibility of existing data.

For the most part civil structures are designed on the basis of well-established design guidelines and standards using standardized values for materials allowables. Steel, for example, is specified by grade, making it easy for the designer, fabricator, and inspector to note the level of properties that must be achieved. The plethora of combinations (constituent materials, fabric forms, and processing options) makes it difficult for the civil engineer, who, at present, has almost no knowledge of composite materials, to make such a determination with a relatively high degree of comfort. It is thus important that the composites industry provide the civil engineering community with a methodology through which uncertainties related to materials form, processing options, and field conditions can be evaluated vis-à-vis the resulting levels of composite performance [15]. Currently efforts are underway to establish load and resistance factor design (LRFD) methodologies for use with composites for civil infrastructure design. In addition, standards and guidelines for individual application categories (seismic retrofit of columns, use of composites for external strengthening, use of composites for bridge decks etc.) are slowly being developed through interactions between professional societies, state departments of transportation and the Federal Highway Administration in the U.S., materials and systems suppliers, and academia. The American Concrete Association, for example, through ACI-440 is spearheading the development of guidelines for the use of composites for external strengthening. Standards and qualification criteria for the use of composite jackets for the seismic retrofit of columns have already been established by the California Department of Transportation, and are now being evaluated for national acceptance. Further evaluation programs are being conducted as a collaborative effort through the Civil Engineering Research Foundation (CERF) arm of the American Society of Civil Engineers (ASCE). Further attempts at standardization are also being made through the International Conference of Building Officials (ICBO).

FRP composites provide an exciting and challenging future to civil infrastructure renewal both as related to material choices and choices in design and installation. A truly multi-disciplinary effort between the civil and composites communities is needed if the full potential of these materials is to be achieved.

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STRUCTURAL COMPOSITES WITH ECC

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Abstract

The concept of performance-based design of structures has received significant attention in recent years in the Structural Engineering community. The concept of performance driven design of materials has been around for some time in the materials engineering community, and has been implemented in recent years in an Engineered Cementitious Composite (ECC). These performance concepts for structures and materials are not only parallel, but also complementary. In this paper, we illustrate the Performance Driven Design concept with a study on high deformation capacity flexural elements with ECC and FRP reinforcements. These elements provide a basis for highly seismic resistant structural systems with controlled failure mode.

Introduction

Engineering materials are often used in combinations in structural applications with the intent of exploiting the attractive properties of the individual constituents. Reinforced concrete, for example, combines the high compressive strength of the concrete matrix with the tensile strength and ductility of the reinforcing steel. In earthquake resistant structural applications, however, negligible inelastic deformations in concrete can also impose strong limitations to such composite systems. These deficiencies can be overcome by engineering the composite systems on the structural and material level.

Depending on the scale of the combination of engineering materials, composites can be divided into composite materials and composite structures. Engineering and designing these composites requires a thorough understanding of their constituent materials characteristics and the optimization of their interaction to achieve the targeted performance of potential structural applications. This performance driven design procedure must not only focus on the optimized design of structural systems and members but also incorporate the engineering process of the constituent materials and composites themselves.

The development of engineered composites and their applications in civil engineering will lead to advanced structural systems with superior performance and reliability.

Performance requirements for HPFRCC

The combination of concrete and fibers in Fiber Reinforced Concrete (FRC) can overcome the inherent brittleness of concrete by fibers bridging across its crack planes, providing FRC with enhanced toughness as compared to plain concrete. However, the tensile strength of FRC is typically similar to that of concrete. At tensile failure, FRC shows quasi-brittle load-deformation behavior, characterized by localized deformation at a single crack under decreasing applied load (Fig.1).

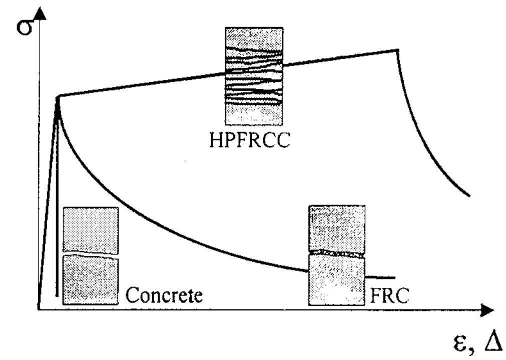


Fig.1 Tensile load deformation behavior of cementitious matrices

High Performance Fiber Reinforced Composites (HPFRCC) are required to show strain hardening load-deformation behavior while undergoing multiple cracking. These two fundamental characteristics of HPFRCC lead to significant improvements in composite strength, toughness, ductility, energy absorption, durability, and stiffness. After reaching the first cracking strength, the fibers bridging across the crack must be able to transfer additional load back into the cementitious matrix in order to initiate multiple cracks. This particular load-deformation behavior is commonly achieved by introducing large volume fraction of fibers into the cementitious matrix, leading to composites such as SIFCON or SIMCON, which require special processing and installation techniques due to fiber contents V_f between 5% and 20%.

In another approach, the University of Michigan has been developing a fiber reinforced cementitious material with relatively low fiber volume fraction ($V_f < 3\%$), known as Engineered Cementitious Composites (ECC), which are very different from commonly known FRC (Li, 1998). For an ECC with polyethylene (PE) fibers, its mechanical properties in terms of compressive strength are comparable to those of high strength concrete (80 MPa) but its ultimate tensile strength (7 MPa) is significantly higher. This tensile strength is achieved by undergoing strain hardening behavior at strain values between 4% and 6% (Fig.2), which leads to tremendous improvement in ductility and fracture toughness of ECC material up to magnitudes usually attributed to metals. The strain hardening behavior of ECC is accompanied by the formation of multiple, closely spaced cracks with a crack width in the sub-millimeter range (Fig.2). Macroscopically, the strain hardening behavior of ECC is akin to that associated with plastic yielding in steel.

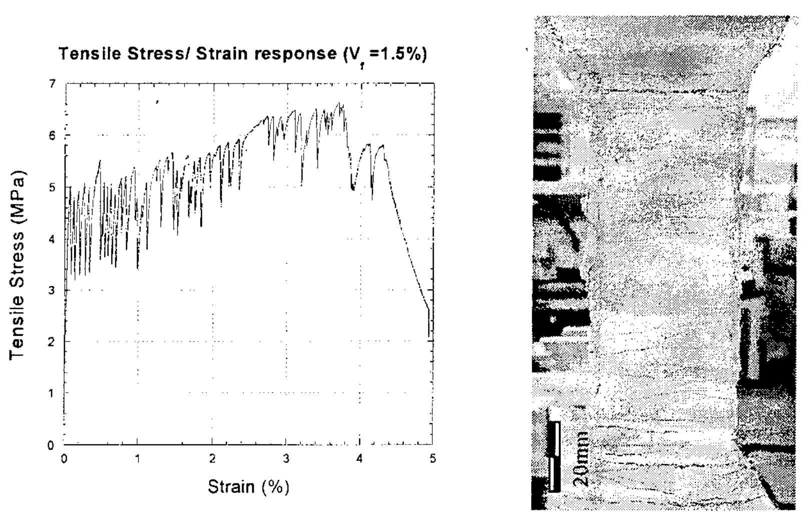


Fig. 2 ECC tensile stress-strain behavior (V_f= 1.5% PE)

Performance driven design for ECC

Engineered Cementitious Composite (ECC) is a special kind of HPFRCC developed at the University of Michigan. Their design is based on micromechanical design principles taking into account the material properties of the cementitious matrix (fracture toughness, elastic modulus, initial flaw size), fiber properties (elastic modulus, tensile strength, length, diameter, volume fraction) and the properties of the interface between matrix and fiber (bond properties, snubbing coefficient). These parameters are incorporated into a micromechanical model, which states the necessary conditions for obtaining the desired composite properties. A brief synopsis is given below. Details can be found in Li (1998).

One requirement, also known as strength requirement, is that fibers bridging the cracked sections must be sufficiently strong to carry the applied composite load at first cracking strength across the cracked section. The second requirement states that the complimentary energy of the fiber bridging vs. crack opening process must be larger than the fracture toughness of the cementitious matrix. This energy requirement is often ignored in composite design, leading to quasi-brittle tension-softening behavior of typical FRC composites, or HPFRCC requiring high fiber content. ECC utilizes similar ingredients as those in FRC such as water, cement, sand, fiber and some common chemical additives but the combination is based on micromechanical principles in order to achieve ECC's unique mechanical properties with a minimum amount of fibers.

Resulting from this approach, ECC shows ultra-ductile, strain hardening deformation behavior accompanied by the formation of multiple very fine, closely spaced cracks (Fig.2). These material properties are achieved at moderate volume fractions (1%-3%) of discontinuous polymeric fibers, depending on the type of fiber used and targeted strength and workability. ECC can be processed on-site and off-site, with various processing methods, such as conventional casting, flowable and self-leveling (Li et al, 1998), and extruding (Stang and Li, 1999).

Requirements for structural elements in seismic resistant design

In earthquake resistant design, it is economically reasonable not to design structures within the elastic limits but to accept large inelastic deformations, resulting preferably in controlled damage to the structural system under large seismic excitation. Therefore, the ductility of the structural system is the primary design criterion in seismic resistant design. Performance, however, is also defined by the need for repair of the damaged structure, considering the replacement of non-structural and structural elements, restoring the initial structural capacity and particularly in reinforced concrete structures preventing the corrosion of the reinforcing steel.

Performance criteria on the structural composite element level are strength, stability, deformability and ductility during the seismic event; residual displacement and capacity, degree of inelastic deformation, crack widths, composite integrity, and repair needs after the event. Judicious tailoring of composites on the material and structural level incorporated in the structural system can significantly enhance its load-deformation behavior during the seismic event as well as minimize structural damage and the need for repair.

Advanced materials in structural composites

The most prominent deficiency in reinforced concrete composites is the inability of the concrete matrix to undergo inelastic deformation in tension. This deficiency also results in limited ductility of reinforced concrete structural elements undergoing flexural deformations. The replacement of brittle concrete with ultra ductile ECC has shown to significantly improve the ductility of steel reinforced ECC structural composite elements.

The deformation characteristics of reinforced ECC flexural members differ fundamentally from those of reinforced concrete. This difference stems from the tensile strength of ECC at large strain levels and enables the total deflection of a flexural member be achieved by a distribution of curvature along its length as opposed to localized hinge formation in reinforced concrete members. Consequently, the maximum local strength and deformation demands on reinforcement in tension and matrix material and compression are reduced, due to reduced local curvature demands in the maximum moment section of the flexural member. The ultra ductile load-deformation behavior of ECC results in deformation compatibility of reinforcing steel and ECC matrix and prevents local yielding of the reinforcement concentrated in the plastic hinge.

This behavior is unique to steel reinforced ECC elements and results in the formation of an extensive region of yielded reinforcement rather than a localized plastic hinge, i.e. the height h_p of the plastic hinge is significantly increased and spread well inside the flexural member. Consequently, the ductility of reinforced ECC structural composites is significantly larger than in conventional reinforced concrete and the energy dissipated in such an extensive plastic deformation region is increased accordingly, as can be observed from the shape of the load deflection curve (Fig.4).

In order to prevent deterioration of these mechanisms under reverse cyclic loading conditions, it is necessary to preserve the integrity of the structural composite system during load reversals. In addition to enhancing the deformation and energy dissipation capabilities, reinforced ECC composites do not have the inherent tendency to disintegrate by crushing and spalling of matrix cover. The deformation of matrix and reinforcement is compatible in the usable and acceptable range of deformation of such structural elements and is limited by either the ultimate strain of the reinforcement material or of the ECC matrix itself (5%). Compatible deformation of ECC matrix and reinforcement material prevents the development of bond splitting forces, which in reinforced concrete structures cause the spalling of concrete and subsequent deterioration of composite action. Furthermore, ECC prevents the hazard of falling debris and scatter because of its ductile deformation behavior, where the reinforcing fibers embedded in the cementitious matrix fully preserve the integrity of the matrix material.

Another major advantage particularly in seismic resistant structural applications is the shear strength of ECC. Generally, flexural members are equipped with large amounts of transverse reinforcement to prevent shear failure and to fully utilize the flexural capacity of the member. In reinforced ECC composite elements, these very labor-intensive detailing requirements can be reduced or even eliminated, because ECC matrix compensates the shear resistance and confinement effect of transverse reinforcement to a high degree. The distribution of curvature along the flexural element, unique to reinforced ECC structural composites, no longer requires the use of a ductile reinforcement material, such as steel, in order to achieve a targeted flexural displacement. It is now possible to use high strength/low strain, elastic reinforcement materials, such as fiber reinforced plastics (FRP), which are known for their superior strength (800 MPa to 2000 MPa) and low strain capacity (1.5% to 4%). The combination of these materials with the ECC matrix provides high strength flexural elements, which can still achieve flexural drift values of several percent.

The most important advantage of using a FRP as reinforcement material in flexural members is, besides high flexural strength, the negligible permanent deformation of FRP reinforced ECC composites. These elements deform quasi-elastically and return to their initial shape upon unloading.

Experimental investigations

In order to verify the described structural composite mechanisms, reduced-scale flexural elements are tested under fully reverse lateral loading conditions. Axial loading is not applied. The specimens have a cross section of 100 mm x 100 mm and are 500mm in height. The lateral load is applied through a specifically designed testing frame. So far 15 specimens with various material and detailing configurations have been tested, of which a few characteristic examples will be presented in the following.

The materials used in this study are for the reinforcement: structural steel and Fiber Reinforced Plastics (FRP) with different material properties and rebar geometries (Fig.3), and Engineered Cementitious Composites (ECC) as matrix material with material properties given in the above sections (Fig.2).

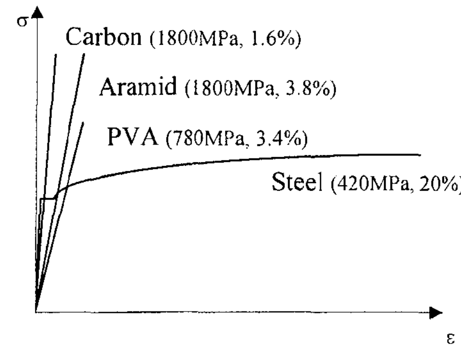
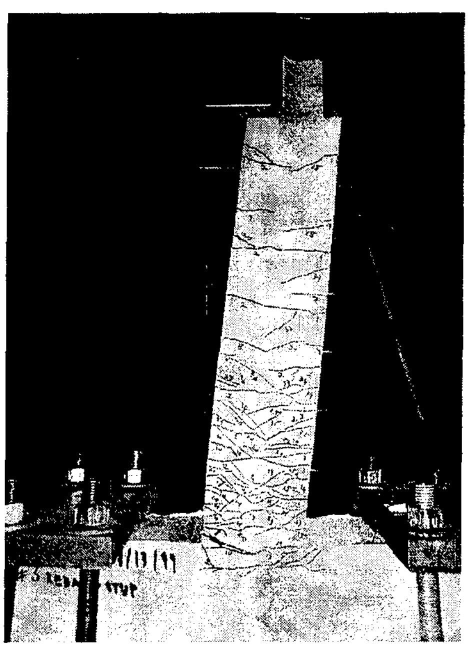


Fig. 3 Tensile stress-strain behavior of reinforcement materials

The improved ductility of steel reinforced ECC composites is shown in the load-deformation behavior of specimen #1 (Fig.4). This specimen is reinforced with four #3 steel rebars (ρ=3.1%) in the longitudinal direction and transverse reinforcement (3mm steel wire) at 30 mm (h < 200mm) and 60 mm spacing (h > 200mm).



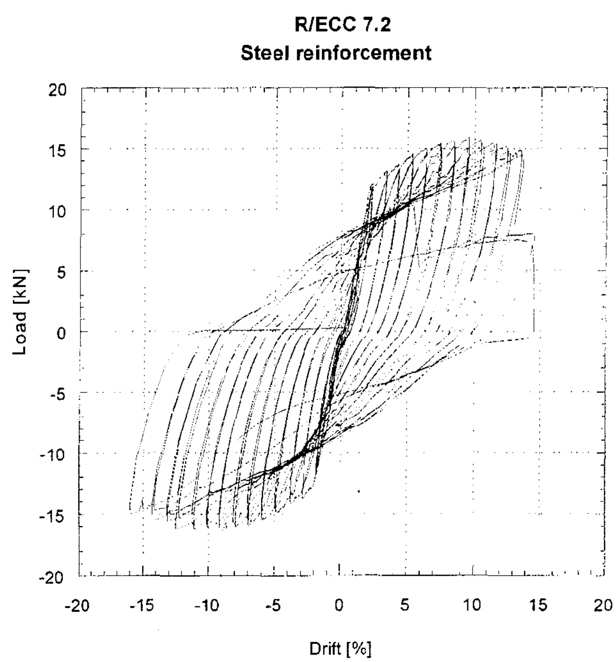
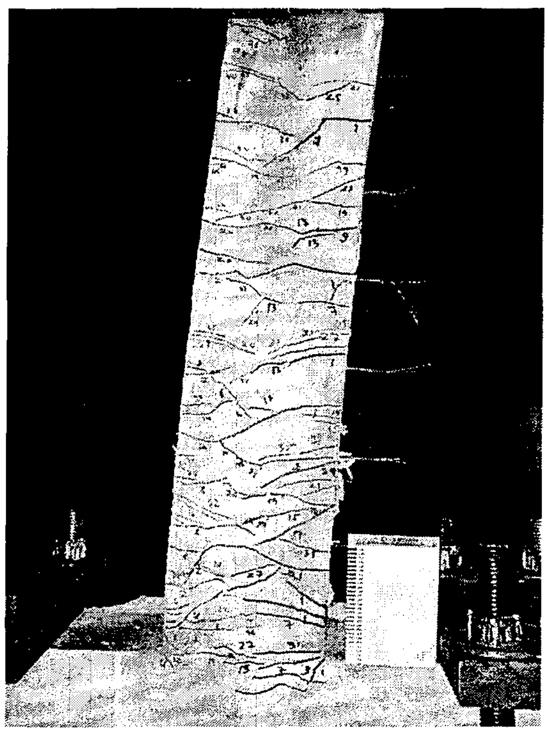


Fig. 4 Specimen #1 at 10% drift and load deformation behavior

The load-deformation behavior shows excellent ductility up to 13% lateral drift without significant decrease in flexural strength. The specimen fails by rupture of longitudinal steel reinforcement at a drift of 15%. Pinching due to shear sliding could not be observed; instead the reinforcement on the tension side goes into compression at positive drift values. First cracking of the ECC matrix material occurs at very small flexural deformation (0.5%). However, these cracks do not increase in width but instead other cracks form with increasing lateral drift along the height of the flexural member (Fig.4); the crack widths at this stage of deformation remains below 200μm. At deformation levels beyond 5%, cracking begins to localize in the maximum moment region and depending on the magnitude of member deflection the maximum crack width exceeds several mm. However, this does not cause spalling of the matrix material or a reduction of rebar confinement.

The combination of ECC matrix with FRP reinforcement shows very different load deformation characteristics (Fig.5). Specimen #2 is reinforced with four Aramid rebars of 5 mm diameter and a nominal rupture load of 35 kN, which is comparable to that of a #3 steel rebar. Transverse



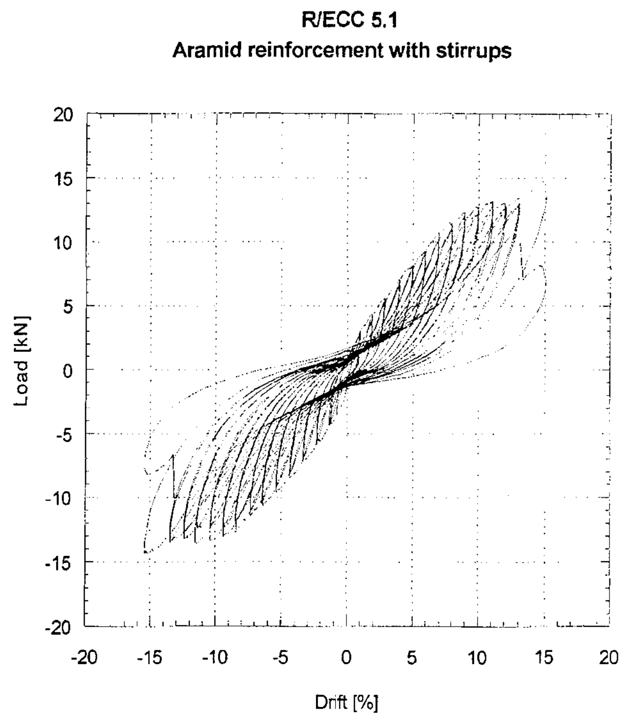


Fig. 5 Specimen #2 at 10% drift and load deformation behavior

reinforcement is provided similar to #1. The flexural load-deformation behavior shows a quasi-linear elastic behavior up to 10% drift. In this deformation range a continuous increase in the number of flexural cracks in the ECC matrix can be observed, which leads to a distribution of curvature along the height of the specimen (Fig.5). Below drift values of 5%, the elastic behavior of the reinforcement results in very small residual displacements after unloading at each cycle. The change in flexural stiffness beyond 10% drift might be caused by a change from flexural to shear deformation mode. This detail is still the subject of an ongoing investigation.

Although the reinforcement itself behaves elastic up to failure and does not dissipate energy by inelastic deformation, the formation of flexural cracks as well as shear friction between reinforcement and ECC matrix, fiber pull out and relative sliding between cracked sections provide the structural composite with energy dissipation capabilities The crack width development of specimen #2 follows the same pattern as in specimen #1. The ultimate failure of the specimen is caused by rupture of the FRP reinforcement at 15% drift.

The shear capacity of ECC is demonstrated in specimen #3, which has the same longitudinal reinforcement as specimen #2, however, without transverse reinforcement in the form of stirrups. This specimen essentially shows the same load-deformation behavior as in the case with transverse reinforcement (Fig.6). However, due to the lack of stirrup reinforcement and resulting lower

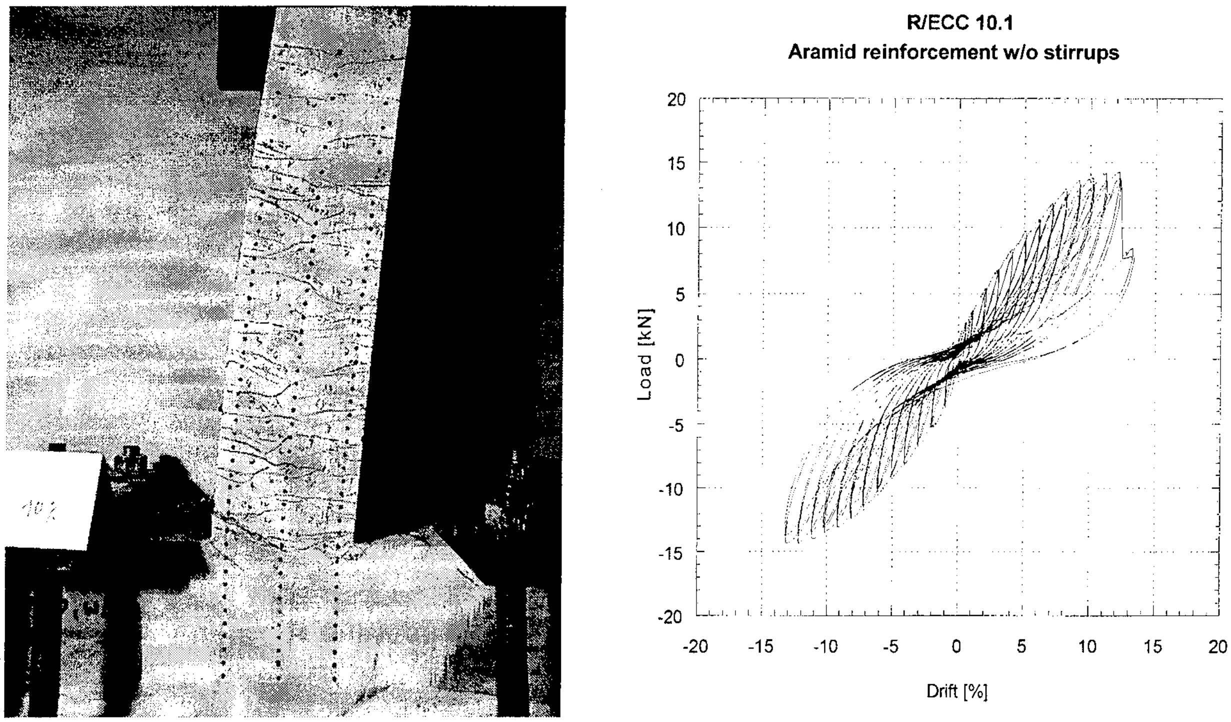


Fig. 6 Specimen #3 at 10% drift and load deformation behavior

stiffness in the maximum moment region of specimen #3, the FRP reinforcement carries a larger portion of the shear load via dowel action. This causes a damaging effect, which results in a deterioration of tensile strain capacity and the specimen fails at a drift of 12%. The comparison of the crack patterns of specimen #2 and #3 shows only a slight difference in the number of inclined shear cracks in the maximum moment region. This underlines a possible redundancy of conventional transverse reinforcement in reinforced ECC structural composites. Experimental investigations with other types of FRP reinforcement, however, have shown a decrease in flexural stiffness in specimens without transverse shear reinforcement. This aspect of composite behavior is currently under investigation.

Conclusions

The replacement of concrete with ductile ECC matrix material in reinforced concrete composites can significantly improve the structural performance of composite elements.

The most important advantage of steel reinforced ECC composites is the increase in ductility. This improvement stems from the composite integrity, which can be maintained up to very large displacement levels. Compatible deformation between reinforcement and ECC in the inelastic deformation regime is the most important feature of the combination of reinforcement material and ECC matrix. The beneficial interaction between reinforcement and matrix is based on the synergistic effect both constituent materials impose on each other. The ultra ductile, strain hardening deformation behavior of ECC prevents stress concentrations and strain localization in the reinforcement. In return, the reinforcement provides additional load transfer between cracked sections of the ECC matrix and supports the distribution of deformation along the entire composite member.

The unique deformation behavior of reinforced ECC structural composites also reduces repair demands by preventing large crack widths, spalling of matrix material and composite disintegration. In case of FRP reinforcement, structural members behave quasi-elastically and have very small residual displacements after unloading. However, energy dissipating mechanisms cannot be sufficiently provided by FRP reinforced ECC structural composites. For seismic resistant structural systems, combination with steel reinforced structural members or installation of energy dissipating devices is necessary.

The proper design and combination of these innovative structural composites will lead towards achieving the targeted performance levels in terms of load deformation behavior, repair needs and serviceability of the structure. Especially the interaction of steel and FRP reinforced ECC elements in the structural system offers promising possibilities. The dependency of strength and stiffness/deformability of steel reinforced concrete members in the structural frame can be overcome and utilized for controlled behavior of structures under seismic excitation.

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Technical Block 2B: Damping and Semi-Active Systems

Chairs: T.T. Soong and A.S. Whittaker

Damping and Semi-Active Systems: A State-of-the-Art Report
T.T. Soong

Passive Seismic Control of Building Structures
Andrew Whittaker

Enabling Technology Testbed Project: Design of a Semi-Active Protective System for a LA Building
G.C. Lee, M. Tong, Yihui We and Shubin Ruan

Computational Aseismic Design and Retrofit with Application to Passively Damped Structures
Gary F. Dargush and Ramesh Sant

The Seismic Safety Program for Hospital Buildings in California - Part 1: Seismic Performance Requirements for New Hospital Buildings and Part 2: The Seismic Retrofit Program for Existing California Hospitals
C.V. Tokas and K. Schaefer

Recent Trends of Damage Controlled Structures in Japan
Akira Wada, Yihua Huang and Mamoru Iwata

Critical Facilities in New York State: General Comments
Thomas M. Jung

Damping and Semi-active Systems - A State-of-the-Art Report

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Abstract

Considerable attention has been paid to passive and active structural control research in recent years, with particular emphasis on alleviation of wind and seismic response. Passive control systems encompass a range of materials and devices for enhancing damping stiffness and strength, and can be used for both natural hazard mitigation and for rehabilitation of aging or deficient structures. In recent years, serious efforts have been undertaken to develop the concept of energy dissipation, or supplemental damping, into a workable technology. Active systems and some combinations of passive and active systems, so-called hybrid or semi-active systems, are force delivery devices integrated with real time processing evaluators/controllers and sensors within the structure. Remarkable progress has been made in both areas of research and implementation. This paper provides an assessment of the state-of the-art of passive damping and active, semi-active in particular, systems for earthquake engineering applications.

Introduction

In recent years, innovative means of enhancing structural functionality and safety against earthquakes have been in various stages of research and development. By and large, they can be grouped into three broad areas: (i) base isolation, (ii) passive energy dissipation and (iii) active control. Of the three, base isolation can now be considered a more mature technology with wider applications as compared with the other two (ATC-17-1, 1993).

Passive energy dissipation systems encompass a range of materials and devices for enhancing damping, stiffness and strength, and can be used both for seismic hazard mitigation and for rehabilitation of aging or deficient structures (Soong and Dargush, 1997, Constantinou et al., 1998, Hanson and Soong, 2000). In general, such systems are characterized by their capability to enhance energy dissipation in the structural systems in which they are installed. These devices generally operate on principles such as frictional sliding, yielding of metals, phase transformation in metals, deformation of viscoelastic solids or fluids and fluid orificing.

Active, semi-active and hybrid structural control systems are a natural evolution of passive control technologies. The possible use of active control systems and some combinations of passive and active systems as a means of structural protection against seismic loads has received considerable attention in recent years. Active/hybrid/semi-active control systems are force delivery devices integrated with real-time processing evaluators/controllers and sensors within the structure. They act simultaneously with the hazardous excitation to provide enhanced structural behavior for improved service and safety. Research to date has also reached the stage where active systems have been installed in full-scale structures for seismic hazard mitigation.

The purpose of this paper is to provide an assessment of the state-of-the-art of these exciting, and still evolving, technologies. In the active control area, particular attention is paid to semi-active systems.

Passive Energy Dissipation (PED)

A large number of PED devices have been developed and installed in structures for performance enhancement under earthquake loads. In North America, passive energy dissipation devices have been implemented in approximately 85 buildings and many bridges, either for retrofit or for new construction. Figure 1 gives a distribution of these buildings as a function of the year in which passive energy dissipation systems were installed. Discussions presented below are centered around some of the more common devices which have found applications in these areas.

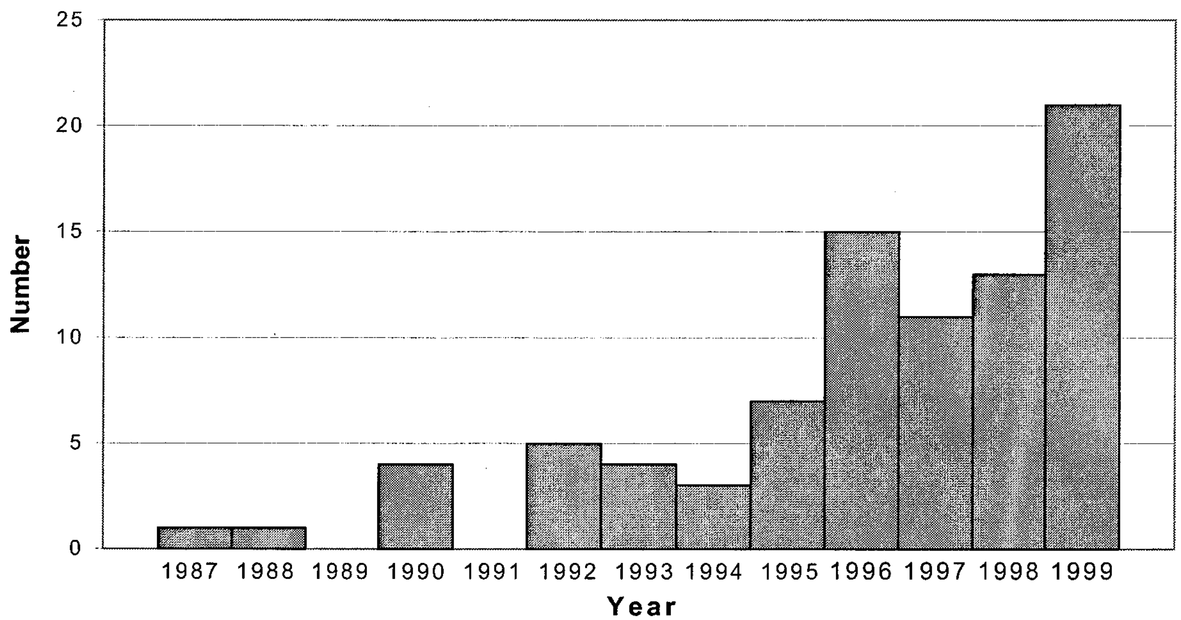


Figure 1. Implementation of PED in North America for Seismic Applications

Metallic Yield Dampers

One of the effective mechanisms available for the dissipation of energy input to a structure from an earthquake is through inelastic deformation of metals. Many of these devices use mild steel plates with triangular or *X* shapes so that yielding is spread almost uniformly throughout the material. Other configurations of steel yielding devices, used most in Japan, include bending type of honeycomb and slit dampers and shear panel type. Other materials, such as lead and shape-memory alloys, have also been evaluated (Aiken and Kelly, 1992). Some particularly desirable features of these devices are their stable hysteretic behavior, low-cycle fatigue property, long term reliability, and relative insensitivity to environmental temperature.

A variation of the devices described above but operating on the same metallic yielding principle is the tension/compression yielding brace, also called the unbonded brace (Wada et al, 1999; Clark et al, 1999), which has found applications in the U.S. and Japan. An unbonded brace is a bracing member consisting of a core steel plate encased in a concrete-filled steel tube. A special coating is provided between the core plate and concrete in order to reduce friction. The core steel plate provides stable energy dissipation by yielding under reversed axial loading, while the surrounding concrete-filled steel tube resists compression buckling.

Friction Dampers

Based primarily upon an analogy to the automotive brakes, Pall et al (1980) began the development of friction devices to improve seismic response of structures. In the intervening years, a number of friction devices have been developed, e.g., *X*-braced friction device (Pall and Marsh, 1982), Sumitomo friction damper (Aiken and Kelly, 1990), energy dissipating restraint (Nims et al, 1993), and slotted bolted connection (Grigorian et al, 1993). The devices differ in their mechanical complexity and the materials used for the sliding surfaces. Generally, friction devices generate rectangular hysteretic loops similar to the characteristics of Coulomb friction. After a hysteretic restoring force model has been validated for a particular device, it can be readily incorporated into an overall structural analysis.

Viscoelastic Dampers

Viscoelastic materials used in structural applications are usually copolymers or glassy substances that dissipate energy through shear deformation. A typical viscoelastic (VE) damper consists of viscoelastic layers bonded with steel plates. When mounted in a structure, shear deformation and hence energy dissipation takes place when structural vibration induces relative motion between the outer steel flanges and the center plates. Significant advances in research and development of VE dampers, particularly for seismic applications, have been made in recent years through analyses and experimental tests.

In Japan, Hazama Corp. developed similar devices by using similar materials, and Shimizu Corp. developed viscoelastic walls, in which solid thermoplastic rubber sheets are sandwiched between steel plates.

Viscous Fluid Devices

The viscous fluid devices developed recently include viscous walls and viscous fluid dampers. The viscous wall, developed by Sumitomo Construction Company, consists of a plate moving in a thin steel case filled with highly viscous fluid. The viscous fluid damper, widely used in the military and aerospace industry for many years, has recently been adapted for structural applications in civil engineering. A viscous fluid damper generally consists of a piston in the damper housing filled with a compound of silicone or similar type of oil, and the piston may contain a number of small orifices through which the fluid may pass from one side of the piston to the other (Constantinou and Symans, 1992). Thus, viscous fluid dampers dissipate energy through the movement of a piston in a highly viscous fluid based on the concept of fluid orificing.

Active, Hybrid and Semi-active Control Systems

An active structural control system has the basic configuration as shown schematically in Fig. 2. It consists of (a) sensors located about the structure to measure either external excitations, or structural response variables, or both; (b) devices to process the measured information and to compute necessary control force needed based on a given control algorithm; and (c) actuators, usually powered by external sources, to produce the required forces.

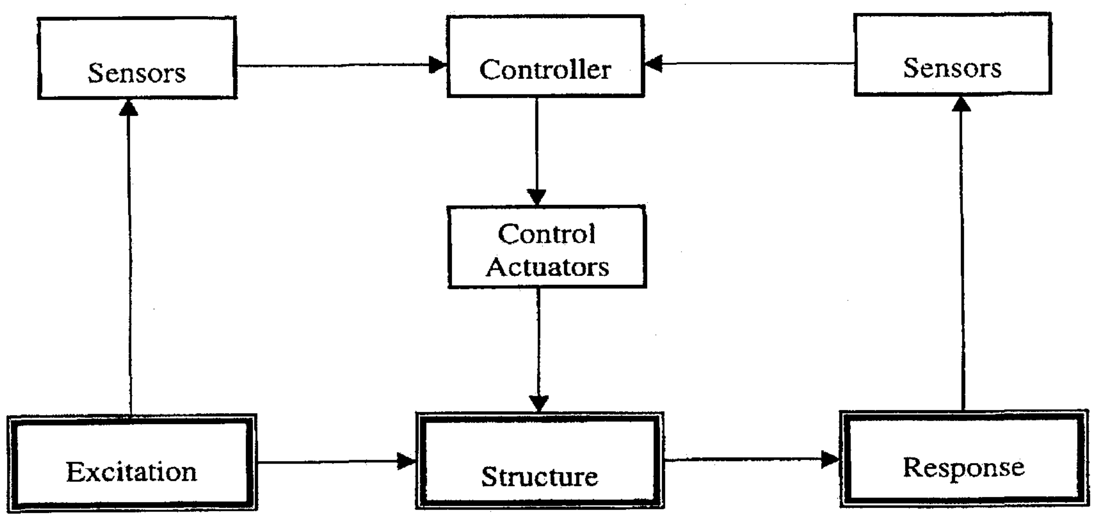


Figure 2. Structure with Active Control

Full-Scale Applications

As alluded to earlier, the development of active, hybrid, and semi-active control systems has reached the stage of full-scale applications to actual structures. There are approximately 42 such installations in building structures and towers, most of which are in Japan. In addition, 15 bridge towers have employed active systems during erection (Soong and Spencer, 2000). Most of these full scale systems have been subjected to actual wind forces and ground motions and their observed performances provide invaluable information in terms of (a) validating analytical and simulation procedures used to predict actual system performance, (b) verifying complex electronic-digital-servohydraulic systems under actual loading conditions, and (c) verifying capability of these systems to operate or shutdown under prescribed conditions.

Semi-active Damper Systems

The focus of this section will be on semi-active systems. Control strategies based on semi-active devices appear to combine the best features of both passive and active control systems. The close attention received in this area in recent years can be attributed to the fact that semi-active control devices offer the adaptability of active control devices without requiring the associated large power sources. In fact, many can operate on battery power, which is critical during seismic events when the main power source to the structure may fail. In addition, semi-active control devices do not have the potential to destabilize (in the bounded input/bounded output sense) the structural system. Extensive studies have indicated that appropriately implemented semi-active systems perform significantly better than passive devices and have the potential to achieve the majority of the performance of fully active systems, thus allowing for the possibility of effective response reduction during a wide array of dynamic loading conditions.

One means of achieving a semi-active damping device is to use a controllable, electromechanical, variable-orifice valve to alter the resistance to flow of a conventional hydraulic fluid damper. A schematic of such a device is given in Fig. 3. Sack and Patten (1993) conducted experiments in which a hydraulic actuator with a controllable orifice was implemented in a single-lane model bridge to dissipate the energy induced by vehicle traffic, followed by a full-scale experiment conducted on a bridge on interstate highway I-35 to demonstrate this technology (Patten, 1998). This experiment constitutes the first full-scale implementation of structural control in the US.

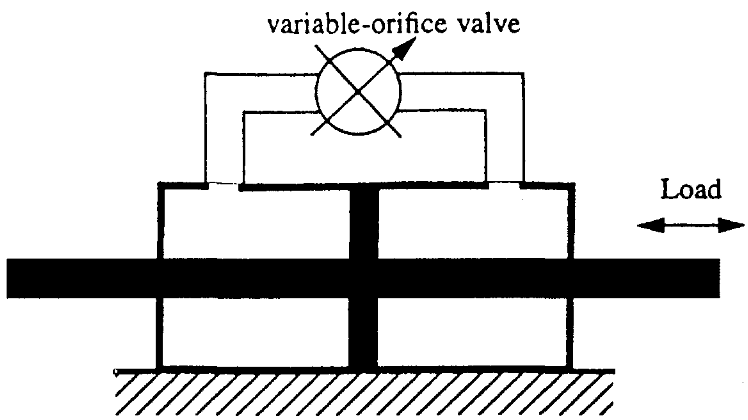
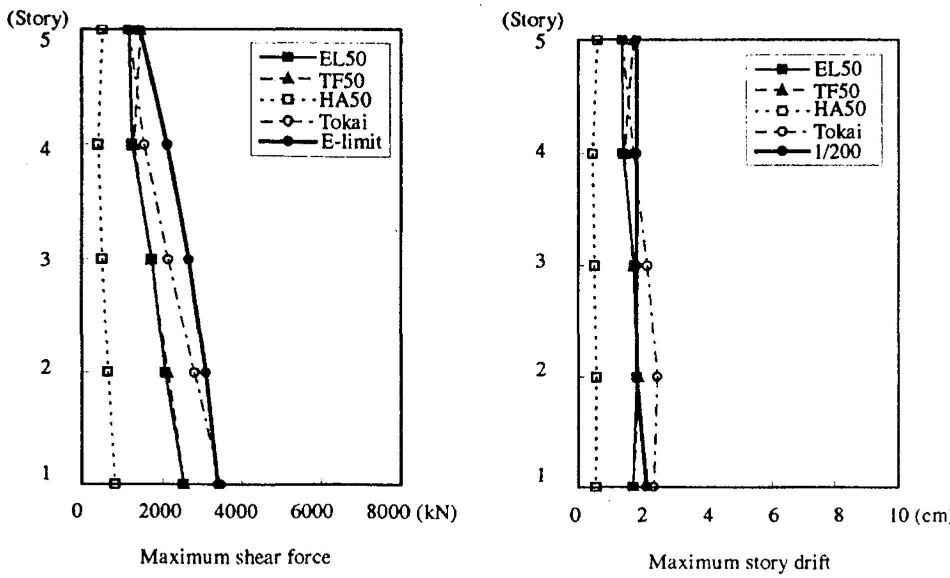
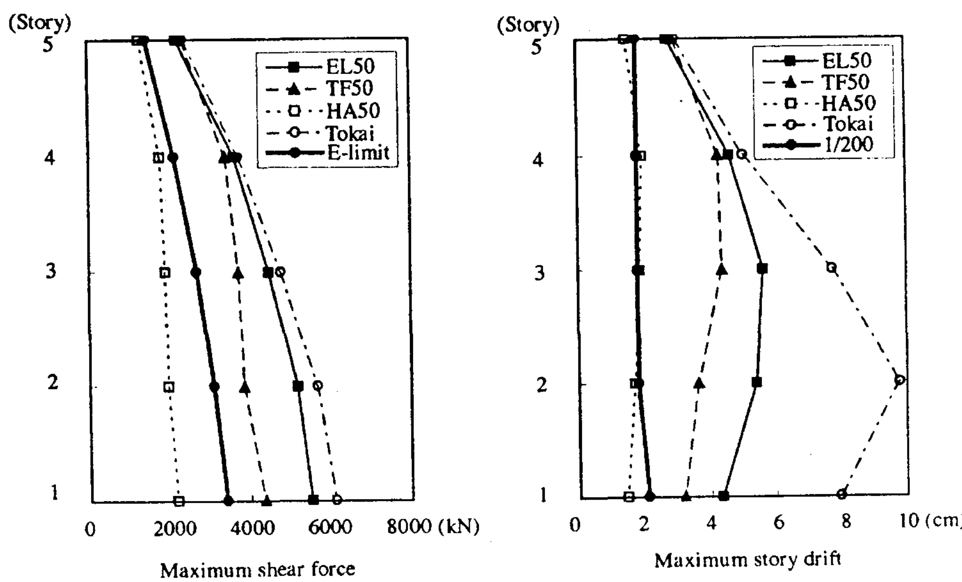


Figure 3. Schematic of Variable-Orifice Damper

More recently, a semi-active damper system was installed in the Kajima Shizuoka Building in Shizuoka, Japan. In this case, semi-active hydraulic dampers are installed inside the walls on both sides of the building to enable it to be used as a disaster relief base in post-earthquake situations [Kobori, 1998; Kurata et al, 1999]. Each damper contains a flow control valve, a check valve and an accumulator, and can develop a maximum damping force of 1000 kN. Figure 4 shows a sample of the response analysis results based on one of the selected control schemes and several earthquake input motions with the scaled maximum velocity of 50 cm/sec, together with a simulated Tokai wave. It is seen that both story shear forces and story drifts are greatly reduced with control activated. In the case of the shear forces, they are confined within their elastic-limit values (indicated by E-limit) while, without control, they would enter the plastic range.



(a) With SAHD Control



(b) Without Control

Figure 4. Maximum Responses (El Centro, Taft and Hachinohe Waves with 50 cm/sec and Assumed Tokai Waves)

A semi-active control scheme using functional switches was proposed by Liang et al (1995). This approach was referred to as Real-time Structural Parameter Modification (RSPM). A structure with RSPM capabilities consists of three integrated components: a sensory unit, a decision-making unit, and an action unit consisting of certain functional switches and/or actuators. The basic functions of these components are self-monitoring, self-decision-making and self-tuning.

Another class of semi-active devices uses controllable fluids, schematically shown in Fig. 5. In comparison with semi-active damper systems described above, an advantage of controllable fluid devices is that they contain no moving parts other than the piston, which makes them simple and potentially very reliable.

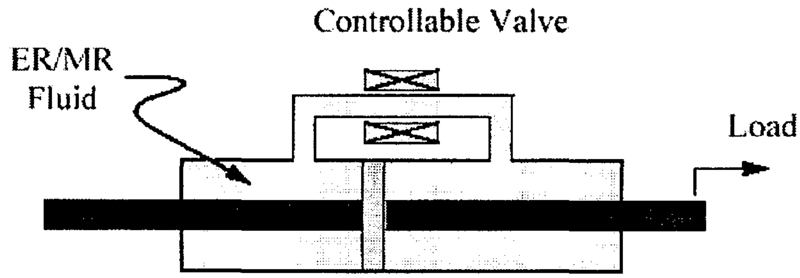


Figure 5. Schematic of Controllable Fluid Damper

The essential characteristics of controllable fluids is their ability to reversibly change from a free-flowing, linear viscous fluid to a semi-solid with a controllable yield strength in milliseconds when exposed to an electric (for electrorheological (ER) fluids) or magnetic (for magnetorheological (MR) fluids) field.

While no full-scale structural applications of these devices have taken place to date, their future for civil engineering applications appears to be bright. Dyke et al (1996) have shown through simulations and laboratory experiments that the magnetorheological damper significantly out-performs comparable passive configurations of the damper for seismic response reduction.

Concluding Remarks

An attempt has been made in this paper to introduce the basic concepts and to bring up-to-date current development and structural applications of some of the passive and semi-active structural control systems. While significant strides have been made in terms of implementation of these concepts to structural design and retrofit, it should be emphasized that this entire technology is still evolving. Significant improvements in both hardware and design procedures will certainly continue for a number of years to come.

The acceptance of innovative systems in structural engineering is based on a combination of performance enhancement versus construction costs and long-term effects. Continuing efforts are needed in order to facilitate wider and speedier implementation. These include effective system integration and further development of analytical and experimental techniques by which performance of these systems can be realistically assessed. Structural systems are complex combinations of individual structural components. New innovative devices need to be integrated into these complex systems, with realistic evaluation of their performance and impact on the structural system, as well as verification of their ability for long-term operation.

Acknowledgement

The author is grateful to the National Science Foundation and the Multidisciplinary Center for Earthquake Engineering Research for their support of his research and device development in this area.

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Passive Seismic Control of Building Structures

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Abstract

Supplemental damping (energy dissipation) hardware is being employed in the United States to provide enhanced protection for new and retrofit building construction. Such hardware includes displacement- and velocity-dependent dampers. The types of dampers being implemented in the United States at this time are presented in the paper. Guidelines and commentary to aid in the implementation of passive supplemental dampers in existing construction are included in the resource documents FEMA 273 and 274: *Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 1997). This paper introduces the FEMA 273 analysis procedures and outlines the modeling and analysis procedures developed for implementing supplemental dampers using the nonlinear static procedure.

Introduction

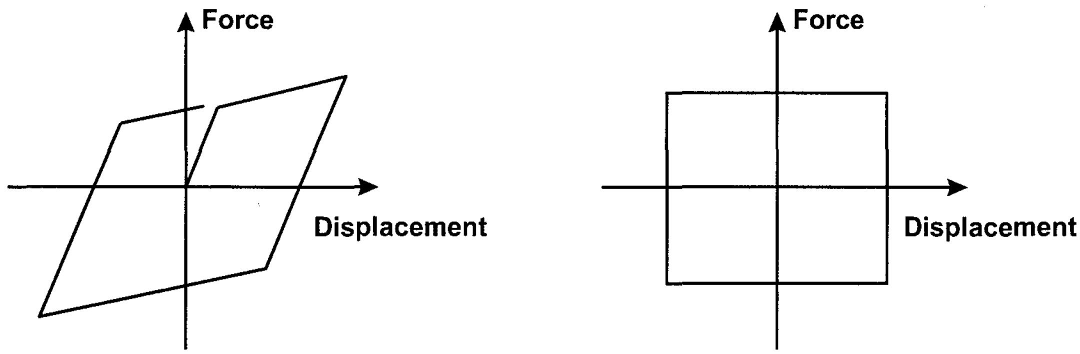
The objective of adding damping hardware to new and existing construction is to dissipate much of the earthquake-induced energy in *disposable* elements not forming part of the gravity framing system. Key to this philosophy is limiting or eliminating damage to the gravity-load-resisting system. Although testing and perhaps replacement of all supplemental damping devices in a building should be anticipated after a design earthquake, evacuation of the building for repair might not be necessary and the total repair cost will likely be minor compared with the costs associated with repair and business interruption in a conventional building.

This paper introduces the different types of supplemental damping hardware being used or considered for use in the United States at this time and introduces the analysis procedures for supplemental dampers in FEMA 273 that were developed by the author together with Professor Constantinou of the University at Buffalo and Dr. Charles Kircher of Kircher & Associates in Northern California.

Supplemental Damping Hardware

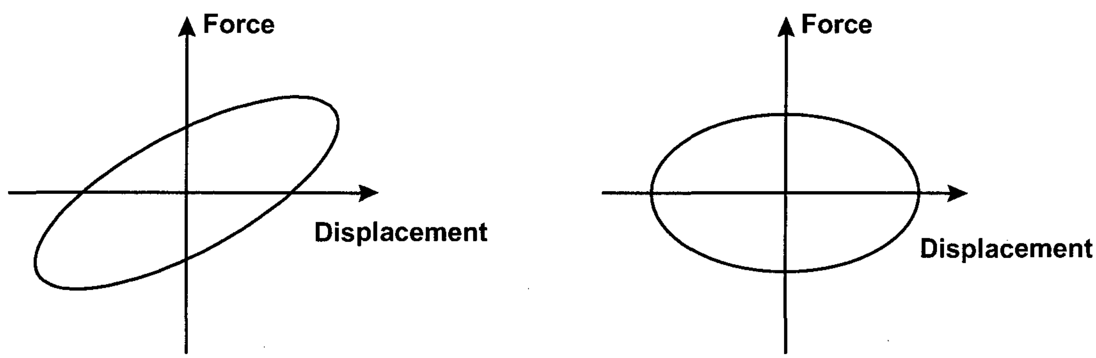
General

Supplemental damping hardware is divided into three categories: hysteretic, velocity-dependent, and other. Examples of hysteretic systems include devices based on yielding of metal and friction. Figure 1 presents sample force-displacement loops of hysteretic dampers. Examples of velocity-dependent systems include dampers consisting of viscoelastic solid materials, dampers operating by deformation of viscoelastic fluids (e.g., viscous shear walls), and dampers operating by forcing fluid through an orifice (e.g., viscous fluid dampers). Figure 2 illustrates the behavior of these velocity-dependent systems. Only hysteretic and velocity-dependent dampers are discussed in this paper because only these types of dampers are being implemented in buildings at this time.



a. Metallic-yielding damper   b. Friction-damper

Figure 1. Force-displacement relations for hysteretic dampers



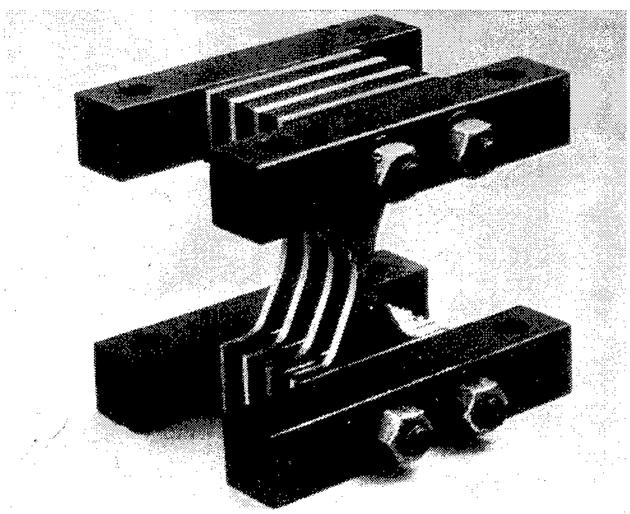
a. Viscoelastic damper   b. Viscous damper

Figure 2. Force-displacement relations for velocity-dependent dampers

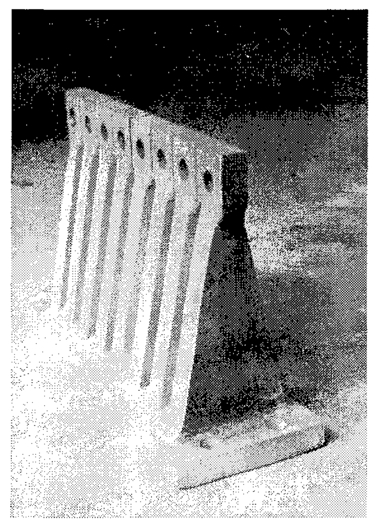
Hysteretic Dampers

Hysteretic dampers exhibit bilinear or trilinear hysteretic, elasto-plastic or rigid-plastic (frictional) behavior, which can be easily captured with structural analysis software currently in the marketplace. Details on the modeling of metallic-yielding dampers may be found in Whittaker et al. (1989); the steel dampers described by Whittaker exhibit stable force-displacement response and no temperature dependence. Two metallic-yielding dampers, ADAS and TADAS, are shown in Figures 3a and 3b, respectively. Added Damping and Stiffness (ADAS) elements have been implemented in the United States and Mexico. Triangular Added Damping and Stiffness (TADAS) elements have been implemented in Taiwan.

An alternate metallic yielding damper, the unbonded brace, is shown in Figure 4. This damper was developed in Japan in the mid-1980s (Watanabe et al. 1988) and is proposed for use on a number of projects in California. The schematic of Figure 4a illustrates the key components of the Nippon Steel brace, namely, a cruciform cross section of welded steel plate (often low-yield steel) that is designed to yield in tension and compression, and an exterior steel tube of circular or rectangular cross section that is selected such that the buckling capacity of the tube exceeds the squash load of the cruciform cross section. The space between the cruciform cross section and the steel tube is filled with a concrete-like material to delay local buckling of the cruciform cross section outstands. Proprietary butyl rubber materials are used to de-bond the cruciform cross section from the concrete-like material. Figure 4b is a photograph of a cruciform cross section. The unbonded brace is designed to have approximately equal strength in tension and compression, and is conceptually superior to the concentrically braced frame because the beam at the intersection point of the chevron braces does not have to be designed for large out-of-balance vertical forces (Bruneau et al. 1998).

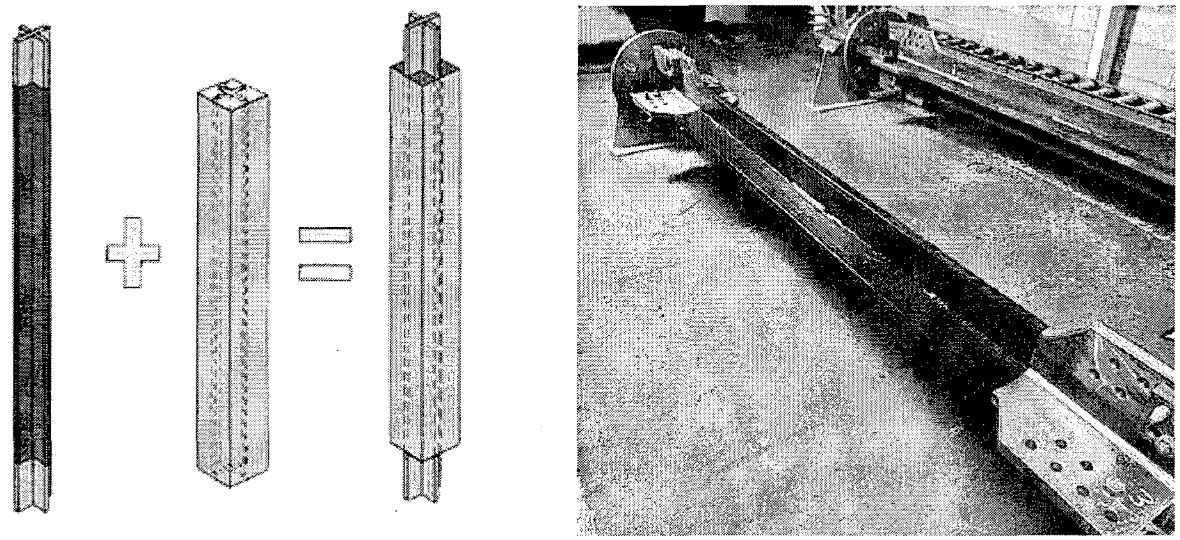


a. Added Damping and Stiffness (ADAS) Element



b. Triangular ADAS Element (TADAS)

Figure 3. Examples of metallic yielding dampers



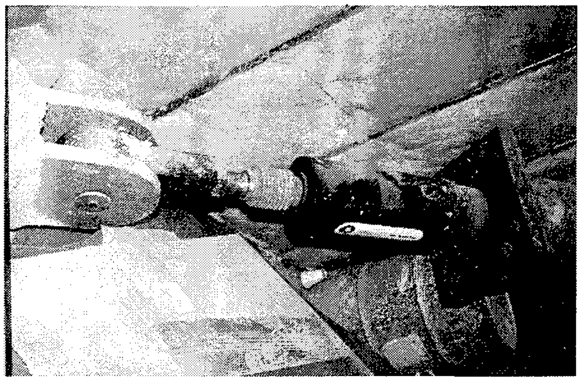
a. Conceptual details b. Cruciform configuration of brace

Figure 4. Nippon Steel unbonded brace

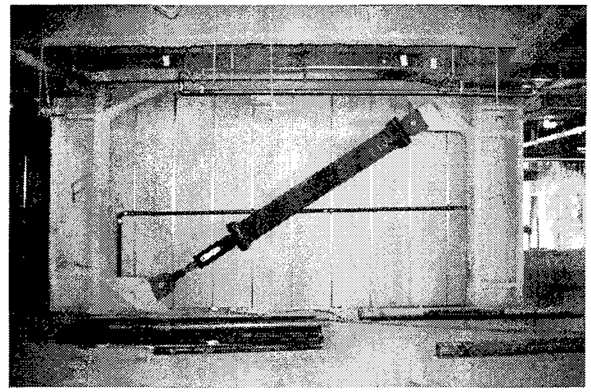
Velocity-Dependent Dampers

Solid viscoelastic dampers typically consist of constrained layers of viscoelastic polymers. They exhibit viscoelastic solid behavior with mechanical properties dependent on frequency, temperature, and amplitude of motion. Fluid viscoelastic devices, which operate on the principle of deformation (shearing) of viscoelastic fluids, have behavior that resembles a solid viscoelastic device. However, fluid viscoelastic devices have zero effective stiffness under static loading conditions. Fluid and solid viscoelastic devices are distinguished by the ratio of the loss stiffness to the effective or storage stiffness. This ratio approaches infinity for fluid devices and zero for solid viscoelastic devices as the loading frequency approaches zero. Solid and fluid viscoelastic dampers are not marketed in the United States at this time and are not discussed further in this paper.

Pure viscous behavior may be produced by forcing fluid through an orifice (Constantinou and Symans, 1993; Soong and Constantinou, 1994). The force output of a viscous damper is the product of a damping constant and the velocity raised to a power in the range of 0.1 to 2.0. Fluid viscous dampers are widely used in the United States at this time. Much of the technology used in this type of damper was developed for military, aerospace, and energy applications. Figure 5 shows photographs of double acting, nonlinear fluid viscous dampers used in a new 14-story building in San Francisco. Such dampers are often compact because the pressure drop across the damper piston head generally ranges between 5000 and 1000 psi (35 to 70 MPa).



a. Fluid viscous damper



b. Diagonal damper configuration

Figure 5. Fluid viscous dampers

Analysis Procedures For Supplemental Dampers

Introduction

The lack of analysis methods, guidelines and commentary was a key impediment to the widespread application of supplemental dampers in buildings in the United States. Prior to 1997, seismic design codes and guidelines in the United States focused on designing structures for strength alone, where the design forces were set equal to the elastic forces divided by a response reduction factor (for buildings). Component deformations, which are indicators of damage and performance, were not checked.

FEMA 273, entitled *Guidelines for the Seismic Rehabilitation of Buildings*, was published in 1997 after more than five years of development. FEMA 273 represented a paradigm shift is the practice of earthquake engineering in the United States because deformations and not forces were used as the basis for the design of ductile components. Performance and damage were characterized in terms of component deformation capacity. Four new methods of seismic analysis are presented in FEMA 273: Linear Static Procedure (LSP), Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP), and Nonlinear Dynamic Procedure (NDP). All four procedures can be used to implement supplemental dampers in buildings although the limitations on the use of the linear procedures likely will limit their widespread use. Of the four, only the NDP can explicitly capture nonlinear deformations and strain- and load-history effects. The other three procedures are considered to be less precise than the NDP, although given the additional uncertainties associated with nonlinear dynamic analysis, the loss of accuracy might be small. The two nonlinear procedures lend themselves to component checking using deformations and displacements; component deformation limits are given in FEMA 273, but most are based on engineering judgment and evaluation of test data. The nonlinear static procedure is described below. Much additional information on this procedure and the other three procedures are available in FEMA 273 and 274.

Nonlinear Static Procedure

The Nonlinear Static Procedure of FEMA 273 is a displacement-based method of analysis. Structural components are modeled using nonlinear force-deformation relations and the stiffness of the supplemental dampers is included in the model. Lateral loads are applied in a predetermined pattern to the model, which is incrementally *pushed* to a target displacement thereby establishing a force versus displacement relation for the building. Component deformations are calculated at the target displacement. Component evaluation involves checking the maximum deformation versus the deformation capacity; force-based checking is not used for deformation-controlled components. Deformation capacities are given in FEMA 273 for different components, materials, and performance levels. Because higher-mode loading patterns are not considered, FEMA 273 limits the use of the NSP unless an LDP evaluation is also performed.

The target displacement is established in FEMA 273 by either the *coefficient method* or the *capacity-spectrum method.* Both methods are equally accurate if the yield strength of the building exceeds 20 percent of the required elastic strength (Whittaker et al. 1998). Only the coefficient method is described below. For information on the capacity-spectrum method, refer to FEMA 274 (FEMA 1997). Regardless of which method is used to calculate the target displacement and the associated component deformations, the nonlinear mathematical model of the building frame must include the nonlinear force-velocity-displacement relations for the dampers and the mechanical characteristics of the framing supporting the dampers. If the stiffness of a damper is dependent upon amplitude, frequency, or velocity, the stiffness value used for analysis should be consistent with deformations corresponding to the target displacement and frequencies corresponding to the inverse of the effective period at the maximum displacement.

Calculation of the target displacement by the coefficient method is based on the assumption that, for periods greater than approximately 0.5 second (for a rock site), displacements are preserved in a mean sense, that is, the mean elastic displacements are approximately equal to the mean inelastic displacements. (Note that the degree of scatter in the ratio of elastic and inelastic displacements may be substantial, and that this assumption is not conservative for buildings with low strength.) The general form of the target displacement equation is:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} \tag{1}$$

where C_0 is a coefficient relating roof displacement and spectral displacement, C_1 is a modification factor to relate maximum inelastic displacements to displacements calculated assuming linear elastic response, C_2 is a modification factor to represent the effect of stiffness and strength degradation on the maximum displacement response, C_3 is a modification factor to account for dynamic second-order effects, S_a is the response-spectrum acceleration at the fundamental period and damping ratio of the building frame, and T_e is the effective fundamental period of the building at the maximum displacement.

The benefit of adding displacement-dependent dampers to a building frame is recognized in FEMA 273 by the increase in building stiffness afforded by the dampers. The increase in stiffness will reduce the effective period T_e in equation 1 thereby reducing the maximum displacement. The spectral acceleration in this equation should be calculated using the effective period of the mathematical model that includes the stiffness of the dampers and the value of the damping factor B (FEMA 1997) assigned to the building frame exclusive of the dampers.

The benefits of adding velocity-dependent dampers to a building frame are recognized in FEMA 273 by (a) the increase in viscous damping and (b) the increase in building stiffness, afforded by the dampers. The increase in damping will reduce the spectral acceleration. The increase in stiffness will reduce the effective period and the spectral displacement as noted in the second-to-last paragraph. The effective damping in the building frame at the point of maximum displacement is calculated iteratively. An estimate of target (maximum) displacement is needed to calculate the effective damping, the secant stiffness at maximum displacement, and the effective period, which in turn are used to calculate a revised estimate of the target displacement. When the assumed and calculated values of the target displacement are sufficiently close, the solution has converged. The component actions in a framing system incorporating velocity-dependent dampers must be checked at the stages of maximum drift, maximum velocity, and maximum acceleration. The procedures for such checking are presented in FEMA 273 and 274. Higher-mode damping forces must be considered if velocity-dependent dampers are being implemented using the NSP. The magnitude of these forces may be similar to those damping forces calculated using the procedure described above.

Summary and Conclusions

Two types of supplemental damping hardware were described: displacement-dependent and velocity-dependent dampers. Examples of each type of hardware, including metallic yielding ADAS, TADAS, unbonded-brace, and fluid viscous dampers were presented.

New procedures for the analysis and design of buildings incorporating displacement- and velocity-dependent dampers were discussed. The nonlinear static method of analysis was described. This analysis method is displacement oriented and as such represents a paradigm shift in seismic analysis and evaluation. The FEMA 273 nonlinear static analysis procedure is displacement-based; component checking focuses on deformations rather than forces. The force-displacement relations for displacement-dependent dampers are modeled explicitly and the key benefit of such dampers is the stiffness they add to the building frame. Velocity-dependent dampers are modeled using their secant stiffness at the stage of maximum displacement, and the primary benefit of such dampers is added viscous damping.

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Enabling Technology Testbed Project: Design of a Semi-active Seismic Protective System for a LA Building

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ABSTRACT

This paper describes an on-going pilot project to design and implement a semi-active seismic protective system in a LA building for seismic protection. This semi-active system is also known as the Real-Time Structural Parameter Modification (RSPM) technology developed at the University at Buffalo with funding initially provided by the University at buffalo and NSF. Continued development was carried out by a DARPA project on technology reinvestment with funding provided by the Office of Naval Research. This paper describes the design and implementation plan for using "smart viscous fluid dampers" to modify the seismic responses of an existing building. Various structural performance issues and practical considerations for using the RSPM system as well as future research challenges for using semi-active control systems in seismic protection of buildings are briefly discussed.

1. Introduction

RSPM is a semi-active control system. The basic principle of RSPM is to dynamically modify the physical parameters (typically damping and stiffness) of a vibrating structure in real time through "switching actions". The theoretical modeling and analysis results and extensive laboratory tests carried out during last several years are given in references [1] [2] [3] [4], and [5].

The report is concerned with the progress during 1999-2000 on the design and implementation plan of the RSPM system for the seismic response reduction of a 7 –story building in Los Angeles. This study is regarded as an Enabling Technology Development (Testbed) Project. It involves two industrial partners (Enidine Inc. for contribution in manufacturing the system and Hart Consulting Group for contributions in design as the professional engineers of record). Cooperation is also received from the building owner.

The key issues related to the analysis and design of a seismic protection system, passive or semi-active alike, are often strongly related to the characteristics and configurations of the structure itself and the performance objectives. In this testbed project of a 7-story building, we have examined the RSPM technology in three specific issues related to the building performances:

- Dependency on system configuration and structural performance objectives
- Building specific characteristics
- Fail safe mode provided by RSPM

2. The Building and the FEM model

The building is a 7 story moment resisting steel frame structure. Finite element model is used to simulate the building dynamic behavior. In figures 1 and 2, the meshes of the model are illustrated. The building is separated by a 6 inch seismic joint from adjacent buildings in both the south and north sides. Due to the small separations among these buildings, there are concerns that the buildings may collide under strong earthquake. The building is also heavily instrumented with accelerometers from the 3rd floor up to the 7th floor. In the 1994 Northridge earthquake, 33 time history responses were recorded. According to the measured records, the maximum lateral acceleration at the top floor was 0.76 (g) and the maximum relative displacement on the roof was 4.97 (in).

The building is non-symmetric in the North-South direction. There is a large stiffness eccentricity in the north-south direction. Table 1 provides the natural frequencies and mass ratios for the first five modes in the north-south direction.

Table 1. Natural frequencies and mass ratios

Mode	1	2	3	4	5
Frequency	1.1	1.5	1.6	3.6	4.5
Mass ratio	0.00	80.6	1.6	0.00	10.5

The seismic responses of the building under ground motions of different exceedance probabilities in 50 years have been studied. For the 10% exceedance probability in 50 years, the maximum roof displacement in the north-south direction could reach 11.6

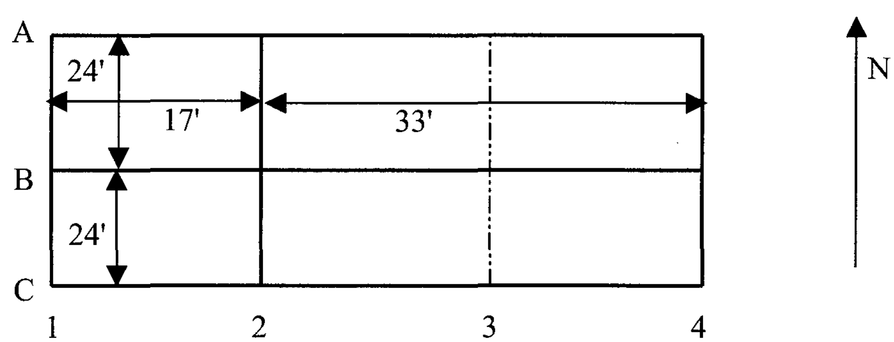

Figure 1. Plan of the Building

inches at the east side and 13.1 inches at the west side. These values are obviously higher than the separation distance among the adjacent buildings.

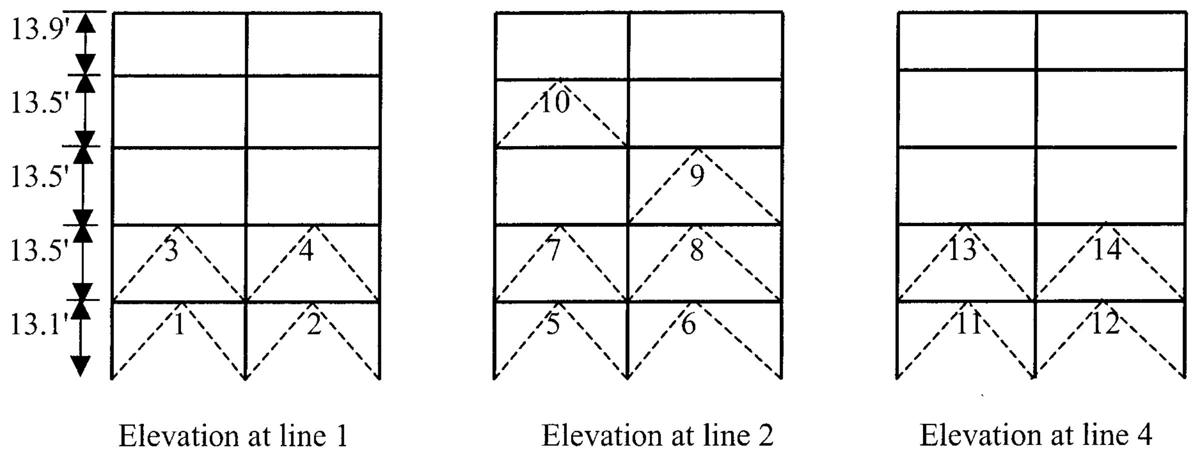

Figure 2. Elevation of the building in the north-south direction and possible damper locations

3. Design methodology, Performance and Configuration

Passive energy dissipation devices have been accepted as alternative method to reduce seismic responses. However, there is no formal design procedure in current major building codes. In IBC-2000 and NEHRP 97, the provisions on passive energy dissipation device design have been removed. Currently, only FEMA 273/274 has included procedures for supplemental damping device design for building retrofit. In brief, there is a major difference in the design methodology for using energy dissipation devices. Many researchers have studied that damping effect is not significant in terms of response modification factor R. When structural responses are in the elastic range, supplemental damping can increase damping ratio and reduce the responses, in particular for structures with low damping. However, in the post yield domain, ductility ratio becomes a major factor to influence the responses.

In a limited scope, the use of passive damping devices are typically designed for reducing structural elastic responses. For low damping structures, viscous damping devices will result in both acceleration and displacement reduction. VE damping or stiffness devices show better displacement control effect, but not so effective on acceleration. The semi-active system, RSPM, is capable of dynamically modifying both damping and stiffness through the control devices, which can significant modify both displacement and acceleration. For protection of buildings in their inelastic response range, the semi-active system will provide additional lateral resistance, which is more effective than the passive damping devices.

As described in [1], [2] and [3], the basic action of RSPM control device is to dynamically modify the stiffness and damping parameters controlled by the device. This modification is realized through the orifice change in the hydraulic device. In this pilot project, we intend to use default damping and variable stiffness control device, such device will behave as a passive damper in compression and spring in tension if the control unit is turned off. Thus, by default, the device is a passive damper or spring. This design reduces the risk of total system failure. It restricts the impact of control electronic failure to only the semi-active control effect of the system.

According to the theoretical analysis, at the resonance frequency, the transfer function of the semi-active device can be approximated as the minimum of the transfer functions of the corresponding damping and stiffness elements under control. It follows that the selection of the range of damping and stiffness parameters can be carried out first by using FE model to seek for the proper parameters for pure passive devices, then using special models (often a reduced DOF model) to identify the optimal parameters for the semi-active device.

In this pilot project analysis, we used SAP 2000 finite element models for structural analysis. The first step to design RSPM system is to evaluate some possible passive damper configurations, then to use reduced DOF model for RSPM system evaluation. In fact, from the performance point of view, one of the major restrictions of a control system performance is the locations for the devices. In this regard, passive or active alike, the control effect often increases in a decreasing rate as the device capacity increases. Eventually, the control effect will be saturated at a level such that any further increase of the capacity of the control device will result in same or decreased system performance. Such performance saturation is more related to the hosting structure rather than the devices themselves.

Considering the building space utilization and occupancy requirements, the candidate locations for K-brace supported device installation are given in figure 2. There are totally 14 locations available for the K-brace type of installation, all of them are in the north-south direction. Because of asymmetry in that direction, the device distribution has to be asymmetric as well, which may significantly excite dynamic torsion response.

Analysis is carried out on the location and passive damper coefficient optimization. With the primary objective to reduce the north-south displacement response of the roof of the building to avoid the adjacent building colliding, the 6 inch gap between the buildings is used as the physical limitation. Assuming the design earthquakes have 10% probability of exceedance in 50 years, 20 records are selected and used as the time history analysis inputs. All the records are modified from actual earthquake ground motions occurred in California. The modification is carried out in accordance with the site specific design spectrum. Among the 20 records, the highest maximum ground acceleration is 1.02g and the lowest one is 0.23 g.

The optimization of braced type configuration is restricted for two cases: 2 damper (one pair) configuration and 4 damper (two pairs) configuration. It is found that for 2 damper configuration, the maximum roof displacement reduction is obtained when the damper pair is placed on the location 13 or 14. Locations 3 and 14 are the best choice for the 4 damper configuration.

For optimization of damper parameters, three time histories generated from site specific spectrum are used. The optimization is based upon consideration of the best displacement reduction and an allowable damping force, the optimized parameters are α=0.7, C_0=80 kips-second/inch. To understand the effect of moderate to small size dampers and, in particular, considering the availability of existing dampers and RSPM devices, we evaluated several more options.

As summarized in table 2, five more braced type damper distributions and a stiffness brace configuration are analyzed to compare with the suggested optimized distribution (the first row in the table). Different damper parameters and locations are used for the new distributions. Four time histories of ground motion are used here which are based on the El Centro and Northridge records. Table 3 illustrates the maximum device forces for different configurations and inputs. Table 4 illustrates the maximum roof displacement of north-south direction in the east and west sides of the building for different damper and brace configurations. The first row data is the response with no damper added. Figures 3-5 display the displacement time response and comparisons for different configurations.

From the tables and the figures, it is seen that moderate size dampers may provide better performance than the large size dampers (C=80, configuration I) and extremely small dampers (configuration V and VI). The stiffness brace configuration seems outperform the damper configurations at some cases. The values in table 4 show that the displacement reduction in east and west sides of the building will not be consistent because of the stiffness eccentricity and damper asymmetric distribution. The added dampers could induce torsion motion, which may have significant effect on the responses.

Table 2. Device distributions
(units are in kips-inch/second)

	C	1	2	3	4	5	6	7	8	9	10	11	12	13	14
I	80			X											X
II	40			X	X									X	X
III	20			X	X				X					X	X
IV	10			X	X			X	X	X				X	X
	0.48	X	X	X	X	X	X	X	X	X	X	X	X	X	X
VI	0.48			X	X			X	X	X	X			X	X
Spring	K= 866		X	X						X	X	X		X	X

Table 3. Maximum device force for different configuration and earthquake record (kips)

	EL1	EL2	NT1	NT2
I	478.1	414.5	187.1	314.3
II	274.3	216.3	111.6	163.0
III	169.3	141.3	77.79	101.7
IV	97.7	84.0	64.1	71.5
V	6.5	4.75	5.00	5.36
VI	6.4	4.87	5.46	5.43
Spring	368	318	219	285

Table 4. Maximum roof displacement of north-south direction on the east and west sides (inches)

	EL1		EL2		NT1		NT2	
	East	West	East	West	East	West	East	West
O	8.05	7.06	5.38	5.03	3.06	3.00	6.82	5.46
I	6.17	5.46	4.67	4.18	1.70	1.63	3.19	2.90
II	5.70	5.06	4.38	3.88	1.77	1.64	3.09	2.82
III	5.59	5.45	4.00	3.83	1.52	1.39	2.79	2.64
IV	5.05	5.81	3.54	3.98	1.62	1.63	2.37	2.52
V	7.56	7.42	5.14	5.09	3.09	2.65	5.78	4.78
VI	7.64	7.48	5.11	5.11	3.28	2.76	6.05	4.99
Spring	3.14	4.20	2.96	3.44	1.60	1.92	2.82	2.36

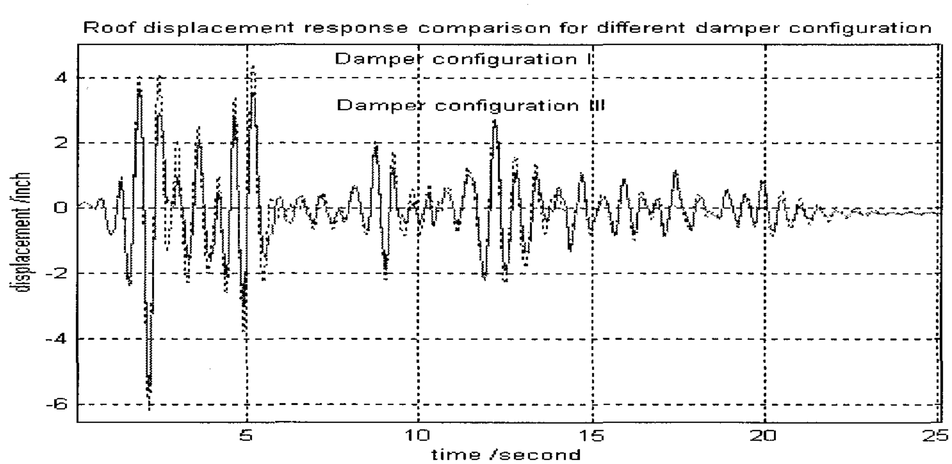

Figure 3. Displacement response comparison

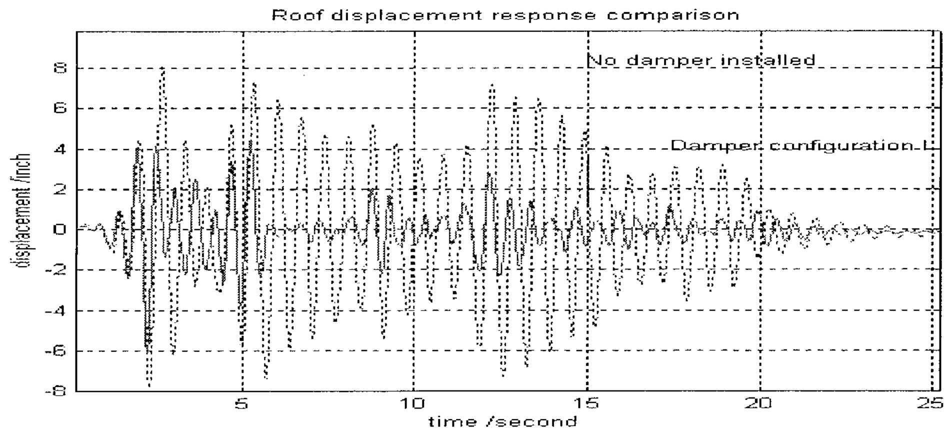

Figure 4. Displacement response comparison

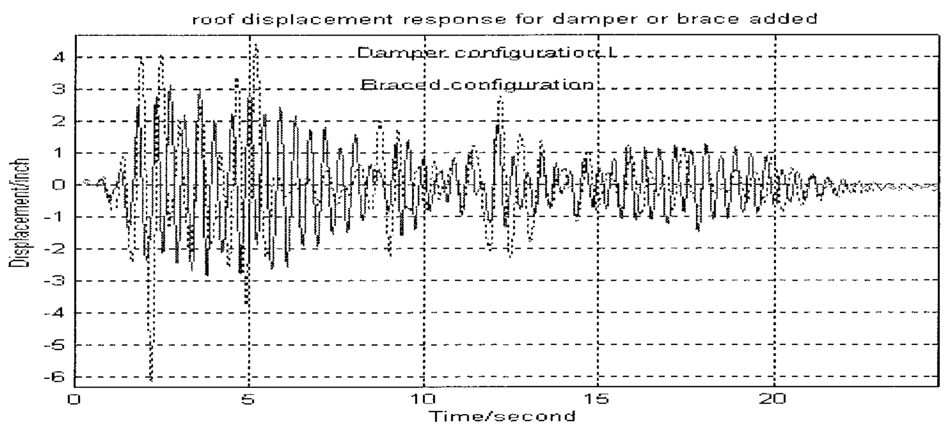

Figure 5. Displacement response comparison

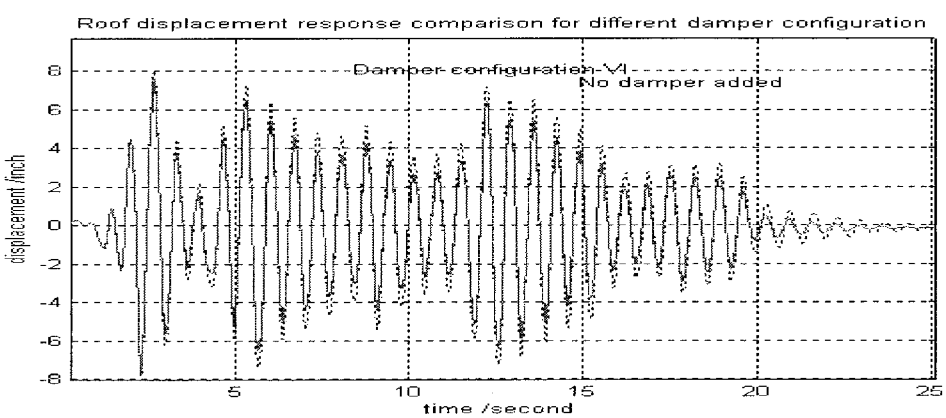

Figure 6. Displacement response comparison

4. Fail Safe Mode of the Semi-active System

One of the advantages of semi-active control over the passive devices is the flexibility to deal with different response range. Because the control devices consist of mechanical and electronic components, which may subject to higher risk of malfunction or failure than structural elements, fail safe mode is often a desired or even required feature.

In the pilot project, we intend to implement a fail safe control level in the RSPM system such that in case of one device malfunction or failure, other devices will be set to passive damping or spring mode or completely disconnect so that the consequence device failures will be prevented. The control strategy is first to check for device overload to ensure that currently working devices are not subject to excessive load. If excessive load is found in a device, its protective mode is activated. A more advanced feature is to check, under the condition of one device failure, whether the remaining working devices can still maintain the required minimum performance level. Such control is in parallel to the mechanical protective measure built in the device such as pressure release valve.

In comparison to the mechanical problems in the control devices, electronic component may subject to even higher chances of malfunction or failure. This risk is dealt with in the designed default mode of each device as described before.

5. Challenges and Future Effort

Throughout the RSPM development and the building testbed implementation project, we have encountered the following challenges:

1. Integration of established structure design approach with new response reduction technologies.

Using new technologies can improve the seismic performance of a structural system; however, various new technologies may have different special features and limitations. Although it is often not a problem to design a new technology based protective system to realize the specific performance requirements, the integration of the devices into traditional structure design approach is a challenging task.

2. Structural system based approach vs. device based approach in system design

For device manufacturers, the interest and focus are always on the improvement of the energy dissipation capacity of the device itself. In reality, we have seen that performance of control device is often limited by the structure characteristics such as ductility, and practical constrains such as the locations and orientations, these characteristics and constrains are more important to the system performance than the device capacities. Thus, a good protective system design often results from systematic evaluation of the structure and creative use of available control technologies. Control devices today are promoted by the various manufacturers without much input from the structural designers. As a result, the structural engineers are faced with the problems of fitting the device specifications to the structure rather than considering the performance of the structure-device integrated system.

3. Lack of real earthquake evaluation of semi-active control system

Extensive research on passive, semi-active control devices have been carried out. Recently smart material, ER/MR fluid based devices have also been developed and evaluated in laboratory. However, there are still too few cases of real building or structure applications. In particular, the lack of real earthquake evaluation of new technology controlled structures becomes a bottleneck for further development of new seismic protection technologies. Because of various components involved in the devices, there could be additional problems and issues to be discovered.

4. Guidance or common ground for basic system design requirement

Currently in the US, there are only a few limited guidance for the use of energy dissipation technologies. FEMA 273/274 is one example. However, as more and more new technologies to be introduced in the seismic protection business, the guidance may need to be further broadened. Some typical scenarios may need to be addressed such as integrated structure-device systems, design safety margins and device malfunction or sudden failure.

5. It is important to utilize building testbeds to evaluate the devices through structural performances and to develop design guidelines for building with added passive and semi-active seismic responses reduction technologies.

Acknowledgement

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Computational Aseismic Design and Retrofit with Application to Passively Damped Structures

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Abstract

A new computational framework is developed for the design and retrofit of building structures by considering aseismic design as a complex adaptive system. For the initial phase of the development within this framework, genetic algorithms are employed for the discrete optimization of passively damped structural systems. The primary objective is to determine robust designs, including both the uncertainty of the seismic environment and the reliability of the passive elements. Several model problems are examined in order to assess the potential merits of the approach.

Introduction

Considerable effort has been directed over the past two decades toward the development and enhancement of protective systems for the control of structures under seismic excitation. In general, these protective systems can be classified into three major categories, namely, seismic isolation, passive energy dissipation, and active/semi-active control. Within each category, a number of different technologies have been introduced. For example, in the area of passive energy dissipation systems, applications typically involve metallic yielding dampers, friction dampers, viscous fluid dampers or viscoelastic dampers (e.g., Soong and Dargush, 1997; Constantinou et al., 1998).

Although the introduction of these new concepts and systems presents the structural engineer with additional freedom in the design process, many questions also naturally arise. In the case of passive energy dissipation systems, these questions range from performance and durability issues to concerns related to the sizing and placement of damping elements. For example, are there advantages at a given site to one particular type of passive device? Is it beneficial to include both rate-dependent and rate-independent devices in a single structure? Should devices be distributed uniformly throughout the height of a uniform structure? What damper distribution should be employed for irregular structures?

Over the past several years, a significant body of work has focused on the development of design guidelines and procedures for passive dissipation systems that address many of the important issues. Of particular note are the NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA-273, 1997; FEMA-274, 1997) and the on-going effort to convert those guidelines into a

Prestandard (FEMA-356, 2000). Section 9.3 in each of those documents pertains specifically to passive energy dissipation systems. The guidelines detail several different analysis procedures with varying levels of complexity. Included are linear static, linear dynamic, nonlinear static and nonlinear dynamic procedures. In some cases, detailed computational work is required. However, it is quite clear, particularly from the Commentary (FEMA-274, 1997), that the design process is still traditional. Within this process, the role of computational analysis is limited to the calculation of intermediate results and to the confirmation of pre-established designs. While in many ways the concept of passive energy dissipation in itself is quite revolutionary, the classical design process remains largely intact.

One promising direction for future research involves the further development of these design guidelines based upon additional numerical simulations and practical experience. Alternatively, we may envision a dramatically different design process for passively damped structures by adopting a computational approach. Such an approach should incorporate the dynamics of the problem, the uncertainty of the seismic environment, the reliability of the passive elements and perhaps also some key socioeconomic factors. With these requirements in mind, we conceptualize aseismic design as a complex adaptive system and begin to develop a general computational framework that promotes the evolution of robust, and possibly innovative, designs.

In the following section, we briefly review the characteristics of complex adaptive systems and the genetic algorithms that currently provide a basis for our approach. Then the proposed computational framework for aseismic design is presented. Included is a description of the overall approach, along with some details concerning the models used for the primary structure and passive elements. In order to provide an indication of the potential performance of this approach, several model problems are then considered. Afterwards, some concluding remarks are given.

Complex Adaptive Systems

There is a broad class of systems in nature and in human affairs that involve the complicated interaction of many components or agents. These may be classified as complex systems, particularly when the interactions are predominantly nonlinear. Within this class are systems whose agents tend to aggregate in a hierarchical manner in response to an uncertain or changing environment. These systems have the ability to evolve over time and to self-organize. In some cases, the system may acquire collective properties through adaptation that cannot be exhibited by individual agents acting alone. Key characteristics of these *complex adaptive systems* are nonlinearity, aggregation, flows and diversity (Holland, 1995). Examples include the human central nervous system, the local economy, a rain forest or a multidisciplinary research center.

Tsypkin (1971) presented perhaps the first major work on adaptation in automated systems. His approach was based primarily on contemporary methods of optimal control theory. Holland (1962, 1992), on the other hand, developed a unified theory of adaptation in both natural and artificial systems. His consideration of natural systems was significant. Not only did this provide more generality, it also allowed Holland to bring the ideas from biological evolution to bear on the problem. Besides providing a general formalism for studying adaptive systems, this led to the development of *genetic algorithms*.

Within the Holland formalism, let α be the set of attainable structures, ε symbolize the class of possible environments, μ indicate the performance measure, and τ represent the adaptive plan. Then by making selections from a set of operators Ω, the adaptive plan τ produces a sequence of structures $A \in \alpha$ based upon the performance measure μ_E associated with environment $E \in \varepsilon$. In a genetic algorithm, the individual structures A are identified by a genetic code, which is often represented as a binary string. The typical genetic operators contained in Ω include crossover, mutation and inversion. At each generation, the best performing structures are selected for reproduction. The genetic operators then work to increase the frequency of good qualities contained in the population, while continually exploring the space of possible structures in α. Further details can be found in Holland (1992) and Goldberg (1989). It is interesting to note that although in the original work by Holland the environment may be uncertain, many implementations and applications of genetic algorithms are limited to fixed environments.

Computational Framework for Aseismic Design

We now return to the problem of aseismic design of passively damped structures. The structural system itself is certainly complicated. The associated design process, based upon the existing guidelines, is also complicated. However, neither would be classified as a complex adaptive system. In the present section, we propose a new aseismic design approach based upon the creation of an artificial complex adaptive system. The primary objective is to develop an automated system that can evolve robust designs under uncertain seismic environments. With continued development, the system may also be able to provide some novel solutions to a range of complex aseismic design problems.

Figure 1 depicts the overall approach for computational aseismic design and retrofit, borrowing terminology from biological evolution. Design involves a sequence of generations within a sequence of eras. In each generation, a population of individual structures A is defined and evaluated in response to ground motions that are realized in association with an environment E. Cost and performance are used to evaluate the fitness, which in turn determines the makeup of the next generation of structures. Performance is judged by performing nonlinear transient dynamic analysis. Presently, this analysis utilizes either ABAQUS (1998) or an explicit state-space transient dynamics code (tda). The implementation of the genetic algorithm controlling the design evolution is accomplished within the public-domain code Sugal (Hunter, 1995).

In order to fix ideas, we will consider an example of a five-story steel moment frame retrofit with passive energy dissipators as shown in Fig. 2. Three different types of dampers are available: metallic plate dampers, linear viscous dampers, and viscoelastic dampers. For each type, five different sizes are possible. Consequently, a 20-bit genetic code is employed to completely specify the dampers used in each story of any particular structure $A \in \alpha$. Thus, for this problem, the set α contains 2^{20} (i.e., more than one million) possible structures. Figure 2 also defines a hierarchical approach in which different structural models with varying levels of complexity are utilized in each era. The idea here is to first use simple models to widely explore the design space and then to employ more complicated and expensive models later in the design process. Currently, a two-surface cyclic plasticity model is applied for the primary structural system and metallic plate dampers, while a coupled thermoviscoelastic model with inelastic heat generation is used for the viscoelastic dampers. Figure 3 provides additional details concerning the relative cost, performance and fitness definition. Both interstory drift and story acceleration limits are set in order to establish acceptable performance. As indicated, moment magnitude M_W and epicentral distance r are required for the synthetic ground motions, which utilize a model for east coast earthquakes (Papageorgiou, 1999). In the following section, we consider a few specific simple examples based upon Era 1 (i.e., lumped parameter) simulations.

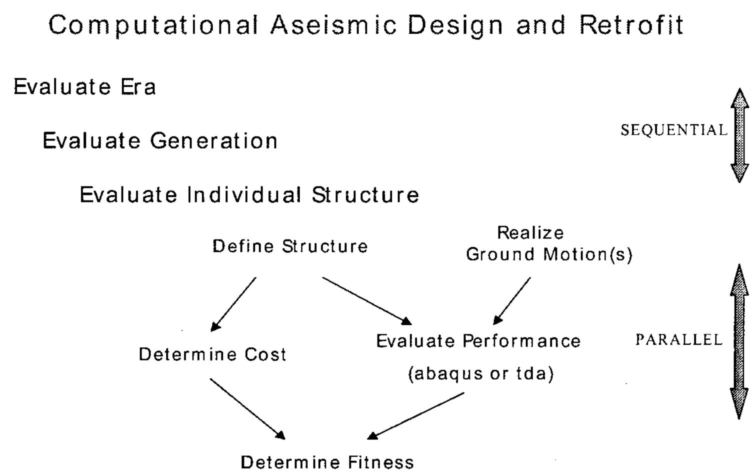

Figure 1: Overall Framework for Computational Aseismic Design and Retrofit

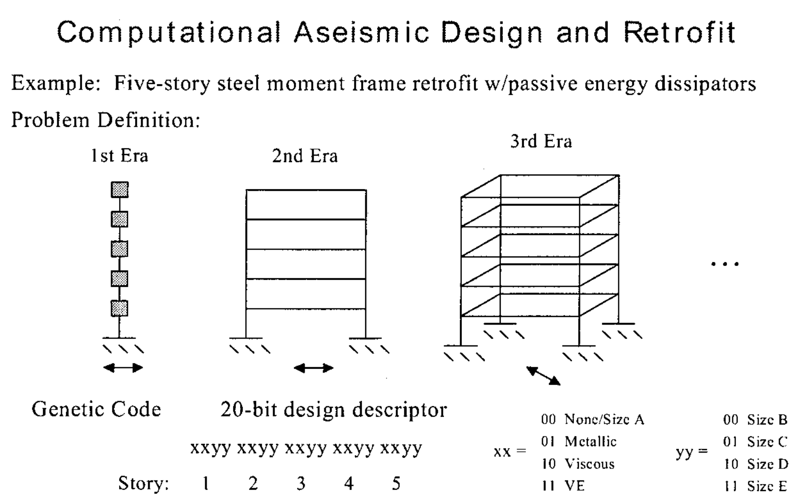

Figure 2: Problem Definition for Five-story Steel Moment Frame

Computational Aseismic Design and Retrofit

Example: Five-story steel moment frame retrofit w/passive energy dissipators

Damper Cost (Relative)

Size A	2
Size B	4
Size C	6
Size D	8
Size E	10

Ground Motion

M_W Moment magnitude

r Epicentral distance

$a_g(t)$ Ground acceleration

Performance Evaluation with abaqus or tda

If ($|\Delta_i(t)| > \alpha H$ or $|a_i(t)| > \beta g$ for any t)

Fitness = Benefit – Damper Cost – Penalty

Terminate structural analysis

Else

Fitness = Benefit – Damper Cost

Endif

Figure 3: Cost and Performance Definition for Five-story Steel Moment Frame

Model Problems

Five-Story Steel Moment Frame

As a first example, we continue the application of the computational aseismic design and retrofit strategy to a five-story steel moment frame. Let k_i and W_i represent the ith story elastic stiffness and story weight, respectively. The baseline frame model has uniform story weights $W_i = W = 125$ kips for $i = 1, 2, ..., 5$ and story stiffness $k_1 = k_2 = k_3 = 193$ kip/in, $k_4 = 147$ kip/in, $k_5 = 87$ kip/in. Thus, the first two natural frequencies are 1.07Hz and 2.72Hz. The two-surface cyclic plasticity model defined in Dargush and Soong (1995) is employed to represent the hysteretic behavior of the primary structure. Within that model, let f_i^y represent the yield force on the inner loading surface for the ith story. Then, $f_1^y = f_2^y = f_3^y = 0.23W$, $f_4^y = 0.17W$ and $f_5^y = 0.10W$.

We attempt to develop a retrofit strategy to protect this structure situated on firm soil in a simplified hypothetical seismic environment that can be represented by a uniform distribution of earthquakes with magnitude $7.2 \le M_W \le 7.8$ and epicentral distance $20\text{km} \le r \le 30\text{km}$. Each ground motion realization is generated according to the model of Papageorgiou (1999). The structure is assumed to perform in a satisfactory manner, if all interstory drifts are less than 1.5in and all story accelerations remain below 0.5g. For the retrofit, it is assumed that linear viscous (visc) dampers, metallic yielding (tpea) dampers and viscoelastic (ve) dampers are available and the 20-bit genetic code defined in Fig. 2 is applied. Hypothetical device cost data for various size dampers were set as indicated in Table 1. Each increment in damper size corresponds roughly to a doubling of the damping capacity. Some preliminary dynamic analyses were done to determine an appropriate range of damper sizes for this structure. There is, of course, considerable subjectivity introduced in setting the relative cost-performance relations for the different damper types. This is only a model problem intended to illustrate the methodology.

The genetic algorithm code developed by Hunter (1995) was employed to identify some potentially robust aseismic designs. In the baseline analysis, a population of $N_p = 40$ individual structures evolved for a total of $N_g = 40$ generations. Fitness was determined, as indicated in Fig. 3, by subtracting the damper cost from an overall benefit of 1000 relative units assigned to the structure. Within each generation, each structure is subjected to a total of $N_s = 10$ seismic events. Whenever the drift or acceleration performance criteria are not met for a given artificial earthquake, a penalty of 100 relative units is deducted from the overall fitness. Crossover and mutation operators were used to evolve new structures from an initially random pool. At the end of each generation, one-half of the structures were replaced with potentially new individuals.

The average fitness obtained at each generation for the baseline Case 1 design evolution is presented in Fig. 4, while the diversity of the population is shown in Fig. 5. We see that initially the random pool of structures has a bit-wise normalized diversity of nearly 0.5 and an average fitness of less than 450. As generations pass, generally speaking, the diversity decreases and the average fitness increases, indicating that the population becomes enriched with more robust structures. Notice, however, that the evolution of average fitness is not monotonic, and that even after 40 generations, some diversity remains. The genetic algorithm continues to explore the design space for better structures. Table 1 also presents the five structures that have appeared most frequently in the population. These are high fitness designs that have survived over many generations of the era. The table data includes the total number of earthquakes that each of the five structures has experienced and the success (or survival) rate. A structure is considered to

Table 1: Five-Story Steel Moment Frame – Baseline (Case 1)

Allowable Drift = 1.500
Allowable Accel = 193.200

Device Cost:

	A	B	C	D	E
visc	2.00	4.00	6.00	8.00	10.00
tpea	2.00	4.00	6.00	8.00	10.00
ve	2.00	4.00	6.00	8.00	10.00

High Fitness Designs:

No. Trials :	2350	1900	570	410	320
Damper Cost :	26.00	26.00	26.00	28.00	26.00
Success Rate:	0.9655	0.9611	0.9614	0.9561	0.9625
Story 5 :	visc A	visc A	visc A	visc A	visc A
Story 4 :	ve C	ve C	visc C	ve C	tpea B
Story 3 :	visc B	ve B	ve B	ve C	tpea C
Story 2 :	visc C	visc C	visc C	visc C	visc C
Story 1 :	visc D	visc D	visc D	visc D	visc D

have survived an earthquake only if the interstory drift and story acceleration criteria are satisfied for all stories. Notice, according to Table 1, that the high fitness designs most often utilize viscous dampers and that the largest dampers are placed on the first story. In four of the high fitness designs, size C dampers appear in the fourth story, suggesting perhaps that the second mode response also requires damping.

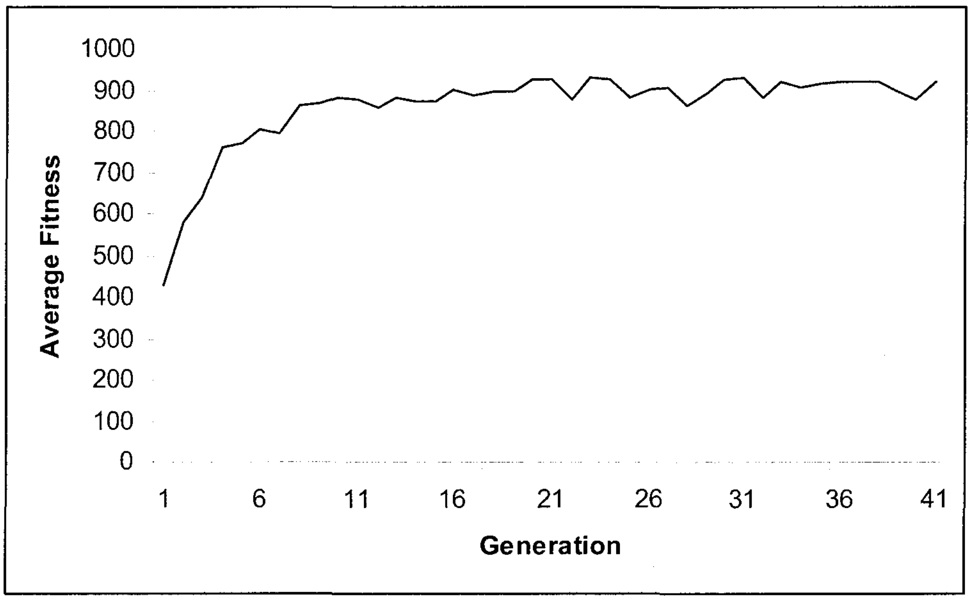

Figure 4: Five-Story Steel Moment Frame – Average Fitness (Case 1)

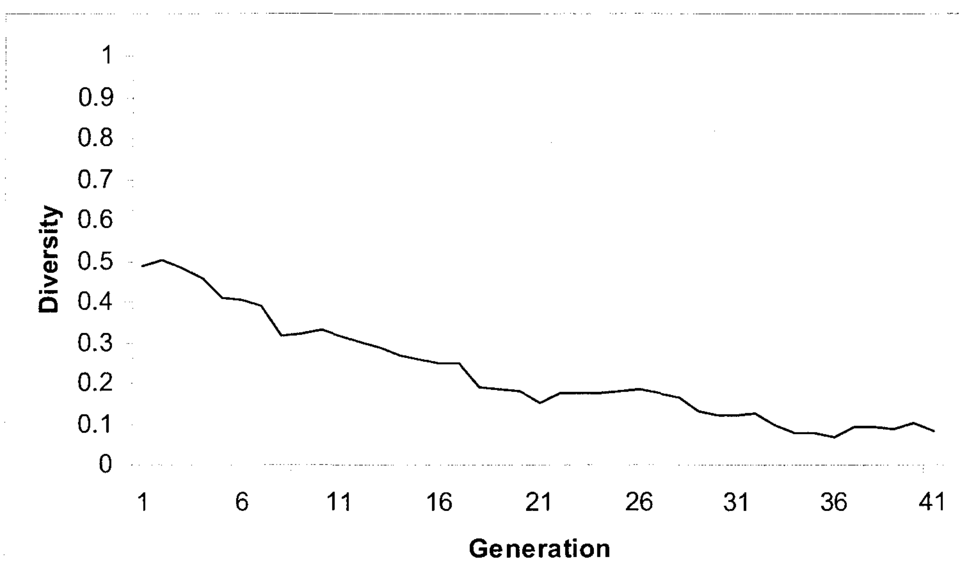

Figure 5: Five-Story Steel Moment Frame – Diversity (Case 1)

The evolutionary process is dependent upon the costs associated with each damper type. In Case 1b, the cost for viscous (visc) and metallic yielding (tpea) dampers was arbitrarily doubled. Results are presented in Table 2. Now we find the highest fitness designs are dominated by viscoelastic (ve) dampers.

Table 2: Five-Story Steel Moment Frame – Modified Costs (Case 1b)

Allowable Drift = 1.500
Allowable Accel = 193.200

Device Cost:

	A	B	C	D	E
visc	4.00	8.00	12.00	16.00	20.00
tpea	4.00	8.00	12.00	16.00	20.00
ve	2.00	4.00	6.00	8.00	10.00

High Fitness Designs:

No. Trials :	3230	1000	340	330	250
Damper Cost :	36.00	32.00	40.00	44.00	64.00
Success Rate:	0.9808	0.9610	0.9824	0.9848	0.9960
Story 5 :	ve B	ve B	ve B	ve B	tpea D
Story 4 :	visc B	ve B	ve B	visc B	visc B
Story 3 :	ve D	ve D	ve D	ve D	ve D
Story 2 :	ve D	ve D	visc D	visc D	visc D
Story 1 :	ve D	ve D	ve D	ve D	visc D

Next, we consider an example with a soft first story. For this Case 2 problem definition, the baseline primary structure was modified such that $k_1 = 96.5\,\text{kip/in}$ and $f_1^y = 0.12W$. All other data is identical to that used for Case 1. The first two natural frequencies for the bare structure are reduced to 0.93Hz and 2.49Hz. Results obtained after 40 generations are shown in Table 3. Notice that the largest size E dampers are now required in that soft first story.

Table 3: Five-Story Steel Moment Frame – Soft First Story (Case 2)

Allowable Drift = 1.500
Allowable Accel = 193.200

Device Cost:

	A	B	C	D	E
visc	2.00	4.00	6.00	8.00	10.00
tpea	2.00	4.00	6.00	8.00	10.00
ve	2.00	4.00	6.00	8.00	10.00

High Fitness Designs:

No. Trials :	560	330	310	310	260
Damper Cost :	30.00	38.00	28.00	34.00	34.00
Success Rate:	0.9500	0.9818	0.9613	0.9290	0.9462
Story 5 :	visc B	visc B	ve B	visc B	ve B
Story 4 :	visc C	visc C	visc B	ve C	ve C
Story 3 :	visc B	ve D	visc B	visc B	visc B
Story 2 :	visc C	visc E	visc C	visc E	visc E
Story 1 :	ve E	visc E	ve E	visc E	ve E

Twelve-Story Steel Moment Frame with Seventh Story Discontinuity

As a final example, we consider a twelve-story steel structure with a severe discontinuity appearing at the seventh story. For this case the primary structure has story weights $W_1 = ... = W_6 = 250$ kips, $W_7 = W_8 = 187.5$ kips, $W_9 = ... = W_{12} = 125$ kips and story stiffness $k_1 = ... = k_6 = 193$ kip/in, $k_7 = ... = k_{12} = 48.25$ kip/in. The first two natural frequencies are 0.35Hz and 0.77Hz. Initial story yield is set at $f_1^y = ... = f_6^y = 0.23W$, $f_7^y = ... = f_{12}^y = 0.057W$ with $W = 125$ kips. For this case, only viscoelastic dampers are available in four different sizes. Results are shown in Table 4 after running $N_g = 40$ generations with a population of $N_p = 20$ structures and $N_s = 5$ seismic events per structure. Notice that the high fitness designs identify the discontinuity by adding stiffness and damping to stories in that neighborhood. Also notice, however, that the success rates of these high fitness designs are somewhat lower than the rates obtained above for the five-story structure retrofit.

Table 4: Twelve-Story Frame with Seventh Story Discontinuity (Case 21)

Allowable Drift = 1.500
Allowable Accel = 193.200

Device Cost:

	A	B	C	D
ve	2.00	4.00	6.00	8.00

High Fitness Designs:

No. Trials :	420	265	170	160	150
Damper Cost :	44.00	38.00	34.00	44.00	48.00
Success Rate:	0.9071	0.9321	0.9294	0.9500	0.8933
Story 12 :	ve C	none	none	ve C	ve C
Story 11 :	none	none	none	none	none
Story 10 :	none	none	none	none	none
Story 9 :	ve B	ve B	ve B	ve B	ve B
Story 8 :	ve D	ve C	ve D	ve C	ve C
Story 7 :	ve D	ve D	ve D	ve D	ve D
Story 6 :	none	ve C	none	ve C	none
Story 5 :	none	none	none	none	ve C
Story 4 :	none	none	none	none	none
Story 3 :	ve D	ve B	ve B	ve B	ve D
Story 2 :	ve B	ve B	ve B	ve B	ve B
Story 1 :	ve C	ve C	ve C	ve C	ve C

Concluding Remarks

Passive energy dissipation has become an attractive technology for the seismic retrofit of structural systems. Although several different design approaches are currently under development, here we argue that a computational design approach has significant potential. This is particularly true as massively parallel hardware continues to advance.

The new approach centers around the development of an artificial complex adaptive system within which robust aseismic designs may evolve. As a first phase of this research program, a genetic algorithm is applied for the discrete optimization of a passively damped structural system, subjected to an uncertain seismic environment.

The results of several preliminary applications, involving the seismic retrofit of multi-story steel moment frames, suggest that continued development of the approach may prove beneficial to the engineering community. However, a number of technical challenges must be resolved in order to convert this research concept into a practical methodology. For example, there is a need to incorporate better models of the seismic environment, to effectively utilize massively parallel computing facilities, to understand the relationship between fitness definition and success rate, and to ultimately create an artificial system capable of innovation.

Acknowledgement

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The Seismic Safety Program for Hospital Buildings in California

Part 1: Seismic Performance Requirements for New Hospital Buildings

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ABSTRACT

The Sylmar Earthquake of 1971 caused the collapse of several hospitals, endangering the lives of patients in those hospitals at the time and rendering the hospitals incapable of providing emergency care to people injured in the earthquake. The poor performance of the hospitals in this earthquake was traced to deficiencies in structural design codes, errors in the design calculations, poor construction quality control. As a result, the California Legislature passed the Alfred E. Alquist Hospital Seismic Safety Act (HSSA) and, since 1973, all hospital construction has been governed by the provisions of that legislation. In essence, the State preempted local building departments in order to ensure statewide uniformity in health facility construction standards. The standards are intended to ensure that:

1. Vulnerable patients are safe in an earthquake; and,
2. The facilities remain functional after such a disaster in order to care for injured persons in the community.

California hospitals since 1973, have been designed and built with four special requirements. First, a geologic hazard study is required for every site that will identify special seismic hazards such as strong ground shaking, liquifaction, or surface fault rupture. Secondly, the normal design base shear is increased by a factor of 1.5 to reduce ductility demands and drift levels. Third, nonstructural components and systems are completely anchored and braced. Lastly, and most importantly, a thorough plan checking and field quality control program is carried out to assure compliance. Subsequent earthquakes to the 1971 San Fernando Earthquake have shown that these provisions, although not thoroughly tested by the strongest ground motions theoretically possible, appear to significantly improve performance.

This paper will discuss the essential elements required to achieve the superior seismic performance level expected from hospital buildings, as implemented by the California Office of Statewide Health Planning and Development (OSHPD).

1. INTRODUCTION

Every earthquake has shown repeatedly that the buildings with the greatest natural resistance to damage are regularly shaped, one- or two-story, shear wall structures with their structural elements fully interconnected, and a minimum of special equipment and utility systems. It would be simple to say that all facilities that need to remain functioning after major earthquakes, such as hospitals, should be housed in these types of buildings. Unfortunately, this style of construction is completely incompatible with the delivery of health care. A balance needs to be purposely struck, therefore, between the functional and performance needs of hospitals and the available structural systems.

The multi-functional characteristics and changing needs of modern hospitals often demand multistory buildings with highly irregular configurations, no interior structural walls, complex networks of utility and mechanical systems, as well as a wide variety of medical equipment and supplies. In order to provide for a functional level of performance following major earthquakes, there is a need for special design and construction procedures. The resulting facilities, while much more capable function following strong shaking, are somewhat more expensive to construct, compared to commercial structures designed to lower standards. Special consideration must be given to the building configuration, the structural system, the anchorage and bracing of architectural components, as well as all mechanical, utility, and medical systems. This can only be accomplished through a high level of interaction and coordination between the architects and engineers throughout the design.

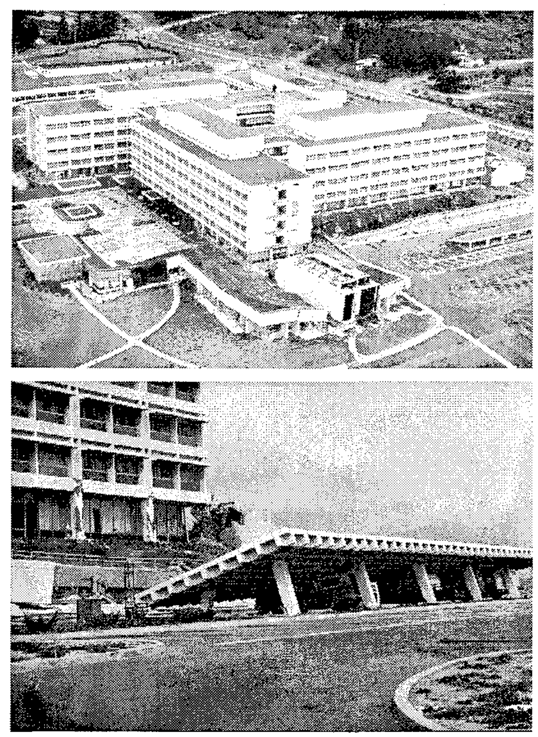

Fig. 1: Olive View Medical Center damaged in the 1971 San Fernando Earthquake.

2. ESSENTIAL STEPS TO IMPROVE THE SEISMIC PERFORMANCE OF HOSPITAL BUILDINGS

Unlike the seismic design process required to attain life-safety performance, achieving functionality requires persistent attention to detail. Every aspect of the structure, its systems and contents need to have proper earthquake resistance. For new hospitals buildings, the current California Building Code (CBC) requires the following essential attributes and steps to achieve this goal:

1. The structural systems of the building must be proportioned in such a manner that the amount of damage to the structure in a strong earthquake is minimized. Modern seismic codes emphasize the need for ductility, and this is an important attribute of any structure. However, in most structures, ductility (and the inelastic behavior that correlates with ductile action) equate to damage. In order to achieve the Immediate Occupancy performance objective, damage, and therefore ductility demand, must be limited. This is achieved through the selection, analysis, and design of a structural system that is complete, fully interconnected, redundant, ductile, and 50% stronger than required for conventional building construction. Special analysis procedures are required as well as additional strength if the structural system is irregular.

2. All nonstructural components, equipment and systems must be designed to resist seismic loading, as well as accommodate the displacements the building is expected to experience during a strong earthquake. To achieve this objective, the design and detailing of appropriate seismic anchorage and bracing for all architectural elements, as well as for all mechanical, utility and medical systems and equipment. An added measure of safety can be obtained by providing back-up systems for essential elements.

3. Analysis and design of hospital structures is complex, and an independent review of the design and analysis procedures assures that the provisions of the building standards have been correctly implement. The independent review includes conducting a complete and comprehensive independent plan review of the entire design. The review may result in recommendations to modify the design as needed to meet code compliance and achieve the expected performance objectives.

4. Quality control of the construction process must be maintained. To achieve this, conduct full-time inspection throughout the construction process. Require the design professionals to visit the project during construction on a regular basis and verify the adequacy of the design and construction. Fully document all construction activities and develop a set of as-built drawings.

5. Monitor all remodel projects to assure that they do not reduce the seismic resistance of the existing building and that they are constructed to the same seismic resistant standards.

3. PURPOSE AND PHILOSOPHY OF THE ENFORCEMENT AGENCY

Pursuant to the Hospital Seismic Safety Act, OSHPD is responsible for overseeing all aspects of general acute care hospital, psychiatric hospital, and multi-story skilled nursing home and intermediate care facility construction in California. This responsibility includes:

1. Establishing building standards which govern construction of these types of facilities;

2. Reviewing the plans and specifications for new construction, alteration, renovation, or additions to health facilities; and,

3. Observing construction in progress to ensure compliance with the approved plans and specifications.

OSHPD's responsibilities under the Hospital Seismic Safety Act are carried out by the Facilities Development Division (FDD). FDD serves as a "one-stop shop" for all aspects of health facility construction. All geo-technical, structural, mechanical, electrical and fire/life safety considerations for inpatient healthcare facility physical plant are handled by FDD. To accomplish its mission, FDD has divided the state into six geographic regions. Each region is supported by an office staff, which provides in depth plan review services and a field staff, which observes construction. These regions are managed from a central office located in Sacramento, California. The FDD oversight process entails the following:

1. Design drawings and specifications are submitted to FDD and reviewed for code compliance by division architects; structural, electrical, and mechanical engineers; and, fire and life safety personnel. Upon plan approval, a building permit is issued and construction begins.

2. Once construction begins, the FDD field staff assumes responsibility for construction oversight. During this process, a District Structural Engineer observes progress on the structural aspects of the project, all Fire and Life Safety issues are observed by a Fire/Life Safety Officer and an Area Compliance Officer monitors progress on mechanical, electrical and architectural aspects. These field personnel can only make periodic visits to the construction site. Therefore, each hospital owner is required to hire an FDD certified Inspector. This inspector is required to provide continuous inspection of all parts of the work. The inspector works in close coordination with the design professional, the owner, and the FDD staff to ensure that the project is constructed in conformance with the approved plans and specifications.

FDD staff also play an important role in the aftermath of an earthquake. Staff are dispatched to assess the extent of damage to health facilities in the affected communities. Based on these assessments, the facilities are cleared to continue providing care without interruption or, access to some areas of the facility may be restricted due to local damage, or, if the damage is severe enough, the facility may be closed. The results of these assessments are communicated to state and local emergency response personnel, so they can route patients to safe facilities. As well, FDD staff review and approve on-site construction required for mitigation of earthquake damage to the facility.

Post-earthquake repairs are reviewed by OSHPD. The level of required repair depends on the severity of the damage the building sustained. Buildings that have suffered substantial damage may require reconstruction to current code standards.

4. CODES & STANDARDS

4.1. General

OSHPD/FDD enforces building standards published in the California Building Standards Code relating to the regulation of health facilities construction projects. The Office adopts these building standards which are published in the model code (i.e. Uniform Building Code, National Electrical Code, Uniform Mechanical Code, Uniform Plumbing Code and the Uniform Fire Code) and which are modified extensively with California amendments to meet the performance requirements established by the HSSA. The California Building Code is the Uniform Building Code with the California amendments. The latest edition of California Building Code (CBC) was published by the International Conference of Building Officials in 1998.

4.2. Building Code Adoption

The State Building Standards Law requires that state agencies proposing changes to building standards (new, amendments to existing, or repeal of existing standards) submit proposals to the California Building Standards Commission (CBSC) for adoption consideration during the annual code adoption cycle. The California amended version of the model code is adopted by the CBSC through an administrative law adoption process which requires public notice and allows public participation.

The required steps in the code adoption process are as follows: When the annual code adoption cycle begins the state agencies submit code changes to the CBSC for review and acceptance. The code change submittals are reviewed by the CBSC's selected Code Advisory Committees for technical review and recommendations to the CBSC. The code changes are then noticed to the interested public to allow review and comments. The last phase of the annual code adoption cycle is the CBSC's approval and adoption of the proposed code changes. Proposed changes are generated in a number of different manners. Representatives of professional organizations, industry groups, and other interested parties may suggest changes or enhancements to model code. Recommendations or procedures from other technical sources or research may be incorporated into the California Amendments. The result is a set of state-of-the-art design codes.

4.3. Ground Motion Requirements

Chapter 16 of the CBC presents methods for determining earthquake shaking demands and considering other seismic hazards such as liquefaction and landsliding. Earthquake shaking demands are expressed in terms of ground motion response spectra, discrete parameters that define these spectra, or pairs of ground motion time histories, depending on the analysis procedure selected. These parameters are presented in a site-specific geotechnical report prepared for each construction project.

4.4. Design Earthquake

The seismic hazards levels of Title 24 CBC are defined on a probabilistic basis. The basic design is based on ground motion hazard levels generated by the *"maximum probable earthquake"* (design basis earthquake). The *"maximum probable earthquake"* ground motion is defined as the motion having a 10 percent probability of being exceeded in a 50-year period (474 years mean return period). This is the same ground motion as that defined in the UBC. Where more detailed analysis methods are required (i.e. dynamic analysis) the structure(s) under consideration must be designed to resist two levels of earthquake hazards. In this situation the structure must be designed to resist the maximum probable earthquake ground motion and in addition it must be demonstrated that the structure can also sustain the upper bound earthquake motion, including P-Δ effects without forming a story collapse mechanism. The *"upper bound earthquake"* ground motion is defined as the motion having a 10 percent probability of being exceeded in a 100-year period (950 years mean return period).

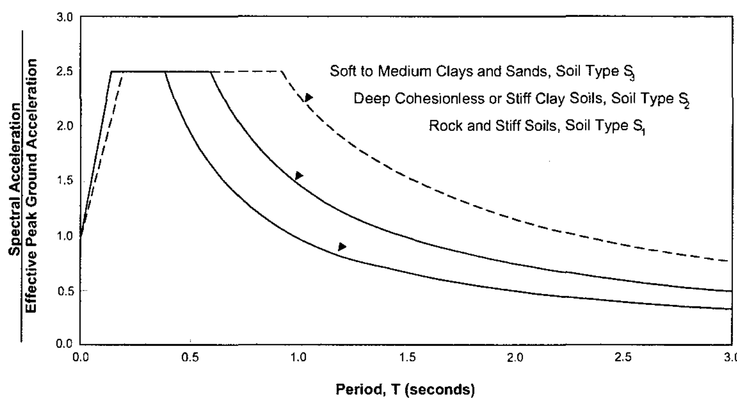

Fig. 2: CBC Normalized Response Spectra, 5% - damping

4.5. Criteria for New Construction

The Title 24 CBC permits two methods to be used in determining the seismic loading: Static Force (SF) Procedure and Dynamic Force (DF) Procedure. The Title 24 CBC is very specific about when the static force procedure can be used. In general, any structures may be designed using the dynamic force procedure at the option of the structural engineer, and some structures must use the dynamic force procedure. Although a static force-based design approach is acceptable for small regular buildings, larger buildings, particularly those with irregularities need more advanced analysis. Performance based design techniques, featuring nonlinear analysis procedures and limits on individual component damage are being developed. Although not in wide use, such techniques are currently being used for retrofit of existing buildings, and will probably be introduced into codes soon.

The majority of the structures under OSHPD jurisdiction are designed by the SF Procedure and therefore are designed to resist ground motion levels generated only by the maximum probable earthquake. In the SF Procedure, the effects of the ground motion are represented by the quantity ZC along with an importance factor of I=1.5. The seismic zone factor, Z, accounts for the amount of seismic risk present in the seismic zone where the subject building is located. The CBC defines the seismic zone factors and the boundaries for each of the zones. The value Z is intended to represent the Effective Peak Ground Acceleration (EPA) that will be generated by the maximum probable earthquake. The quantity C represents the dynamic amplification factor. The curve given by the factor C is a simplified multi-mode acceleration response spectrum normalized to an EPA=1.0g. The effects of the soil characteristics on the ground motion characteristics at a building site are considered through the site coefficient S. The value S is determined from the soil profile underlying the building site. The CBC defines four soil profiles. Generally the value of S varies from 1.0 to 1.5. In special cases (soil profile S_4 - soft sites, i.e. bay mud), the value of S may equal to 2.0. The site coefficient, S, is included in the calculation of C to adjust the curve shape to the appropriate frequency response content for the site soil characteristics.

For complex or irregular structures the seismic loading must be determined by the dynamic force procedure. The ground motion for the dynamic analysis may be in the form of response spectra or time histories. When response spectrum dynamic analysis is used to determine the seismic load for regular structures, the standard site dependent spectra (smoothed average normalized 5% - damped response spectra) furnished by the CBC in Figure 16A-3 may be used. For irregular structures and all structures located on soil profile S_4 the ground motions used in the dynamic analysis must be site specific response spectra or appropriate pairs of time histories scaled to match the site specific response spectra. The CBC requires that the minimum design base shear resulting from the dynamic analysis must be:

1. Regular buildings....... 100% of the base shear value determined from the static method.
2. Irregular buildings...... 125% of the base shear value determined from the static method.

When the base shear from the dynamic analysis is less than these values it must be scaled up to these values. In addition, the CBC allows where site specific ground motions are used for dynamic analysis and the seismic hazard level exceeds that of the code, the maximum resulting base shear for design need not exceed the base shear value determined from the static method scaled by the Spectral Ratio quantity SR.

4.6. Seismic - Isolated Structures

The CBC requires in general seismic-isolated structures to be designed using the Dynamic Lateral-Response (DLR) Procedure. The Static Lateral-Response (SLR) Procedure must be used to establish minimum criteria only, and not be used for design purposes unless these minimum requirements exceed calculated values from the DLR procedure.

In the SLR Procedure the ground motion for design is reflected by the EPA coefficient Z (as a measure of ground shaking level), the site coefficient S (to account for the effects of the local soil profile) and the near source factor N.

For the DLF Procedure either response spectrum or time-history analysis methods are employed utilizing ground motions that reflect the effects of the local seismicity and soil site conditions.

Where the response spectrum analysis method is utilized, the CBC requires the site-specific design response spectra for the DBE and the MCE not be less than 80% of the normalized response spectrum given in Figure 2 for the appropriate soil type at the building site, scaled by the product ZN and M_MZN respectively.

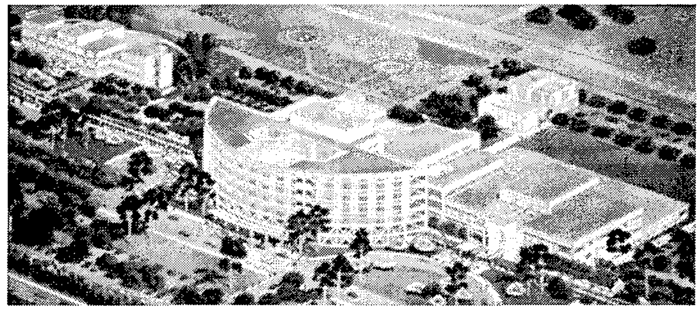

Fig. 3: Arrowhead Medical Center, Hospital Complex of 5 seismic isolated buildings

Where time history analysis is utilized, the CBC requires that the design shall be based on either, the maximum of the results of not less than three separate analyses each using a different pair of horizontal time histories, or the average of seven separate time history analyses. Each pair of time histories shall:

1. Be of a duration consistent with the magnitude and source characteristics of the DBE or the MCE;

2. Incorporate near field phenomena as appropriate;

3. Have response spectra whose SRSS combination of the two horizontal components equals or exceeds 1.3 times the "target" spectrum at each spectral ordinate, and;

4. Have the SRSS of the time history components equal to or greater than the 5% damped response spectra at the period of the isolated structure, T_I.

4.7. Nonstructural Components

The FDD has always placed a high priority on the performance of nonstructural components. Failure of these systems can result in closure or evacuation of a hospital building even though the lateral-force-resisting system has received little or no damage during the seismic event. During the Northridge Earthquake the Los Angeles County Hospital in Sylmar, that replaced the structure that collapsed during the 1971 event, had insignificant damage to the structural system, but flooding due to a failure in a chilled water return line, fire sprinkler head damage, and a loss of "lifeline" water forced the evacuation and transfer of patients to other facilities. Damage such as this made it difficult for facilities to meet the intent of the HSSA that hospitals, "... must be reasonably capable of providing services to the public after a disaster. This performance goal is a significant step above the stated objective of the Uniform Building Code.

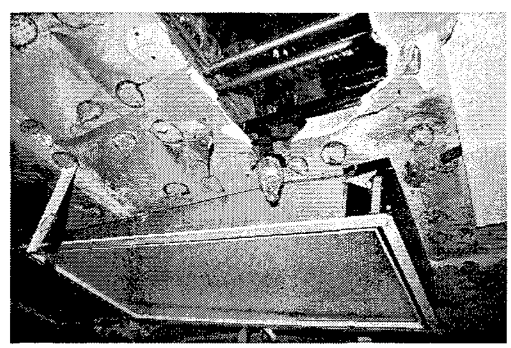

Fig. 4: Non structural damage

The basic formula for determining the horizontal force F_p is:

$$F_p = ZI_pC_pW_p \tag{1.1}$$

where Z, I_p, and W_p are the seismic zone factor, component importance factor, and weight of the component, respectively. The value of C_p varies depending upon the component behavior. In general, this formula generates design lateral forces of between 0.5g and 1.0g in regions of high seismic risk. The variations in design force are the result of consideration of element flexibility, the strength and ductility of the component anchorage.

In addition to being able to resist the inertial forces generated by the earthquake, nonstructural components must withstand the story drifts the building will experience during strong ground shaking. Components and systems that run from floor to floor or building to building must be designed to accommodate the differential movements expected. These movements are computed at the actual shaking levels, not reduce levels used for proportioning components of the lateral force resisting system.

OSHPD has supported programs to standardize the anchorage and bracing of nonstructural components found in typical hospital buildings. This process helps to control construction costs and speed the review process. Manufacturers of equipment, components, and bracing systems may participate in the Anchorage Pre-Approval Program. In this program, the vendor provides standard anchorage and bracing drawings, and supporting calculations for review by OSHPD. The details must cover typical installations of the system or component. Pre-Approvals are valid for a period of three years. When a component or system is installed in accordance with the pre-approval, office plan review of the anchorage is waived.

5. PLAN REVIEW

All hospital and skilled nursing facilities undergo a detailed plan review process. The purpose of the plan review is to validate the design and analysis methodology chosen by the building designer, and to confirm that the provisions of California Building Code are properly applied.

New projects are often given a preliminary review, while the project is still under design. This provides the design professional the opportunity to review fundamental design and analysis procedures with OSHPD before the major design decisions are finalized. Potential areas of concern, such as unusual configuration of the structural systems, special site conditions, or alternative methods of construction or analysis may be reviewed at this time.

Projects received for plan review are initially triaged. Incomplete submittals are returned to the designer for further work. Complete project are accepted for review. Plan review comments are made directly on the drawings. Standardize comments, which cover the typical areas of concern, are used as often as possible. The marked-up set of drawings and specifications are returned to the design professionals for correction. The design professional submits a corrected design package for back-check. Additional cycles of review and back-check are performed as needed. Upon completion of the review, an approval letter is issued, and the hospital owner may apply for a building permit.

OSHPD review focuses only on building design issues. Environmental reports, zoning, and planning issues are resolved at the local level.

6. CONSTRUCTION OBSERVATION AND QUALITY CONTROL

Once a building permit has been issued, the project proceeds to construction. The facility owner retains an Inspector-of-Record (IOR). The IOR acts as the owners agent, assuring that the building is constructed in accordance with the design drawings and applicable codes and standards. The IOR often oversees the work of other inspectors, each trained to review construction specialties, such as concrete placement and welding. The design professionals, IOR, and inspectors must keep written records of their observations, and submit regular reports to OSHPD.

OSHPD provides construction advisory services during the building process. Both structural engineers and construction advisors make regular visits to the jobsite, reviewing progress on the work, and processing change-orders and construction buildings that arise during the course of the job. These individuals can handle most field conditions arising during the project. Extensive or complex changes to the work are forwarded to an OSHPD office for review and approval.

7. CONCLUSION

For the past 28 years, California's hospital seismic safety program has striven to improve the safety of hospital patients, and help insure that hospitals will be able to care for the injured following strong earthquakes. The program has evolved over the years, in response to the changing needs of the people of California, and to adopt lessons learned in earthquakes. Recently, the SB 1953 program was established to reduce the risks posed by buildings constructed prior to the Hospital Seismic Safety Act. By focusing on all aspects of hospital design and construction, the program has produced buildings that have proven far more survivable than structures built to typical building standards. The program will continue to evolve as new information and technologies become available.

REFERENCES

1. Statutes, State of California, Health and Safety Code, Sections 129675-130070 seq.

2. California Building Standards Code, 1997, "Administrative Regulations for the Office of Statewide Health Planning and Development (OSHPD) Hospital Building Safety, Chapter 7. Safety Standards for Health Facilities".

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The Seismic Safety Program for Hospital Buildings in California

Part 2: The Seismic Retrofit Program for Existing California Hospitals

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ABSTRACT

The design and construction of new California hospital buildings has been regulated by special seismic design standards since 1973. Pre-existing hospital buildings have remained largely non-compliant primarily due to the passive requirements of the California Building Code relative to seismic strengthening and secondarily due to the prohibitive costs of upgrading. During the Northridge Earthquake of 1994, several of these older hospitals sustained significant damage. The legislative response was Senate Bill (SB) 1953, which required that all hospitals meet statewide seismic safety standards. The special seismic design considerations that are needed for existing hospital buildings, a program for assessing the projected performance of these facilities in terms of life-safety and ability to remain operational under major earthquakes, and development of effective phased retrofit strategies to improve performance is discussed in this paper.

1. INTRODUCTION

The need for functioning hospitals after a major earthquake is obvious and rarely disputed. While emergency field hospitals, medical tents, and air-lifts to available facilities are often used to supplement for damaged hospitals, they will never provide a sufficient substitute. Only modern health care facilities, located within the damaged region and capable of functioning at full capacity can adequately provide the needed medical assistance.

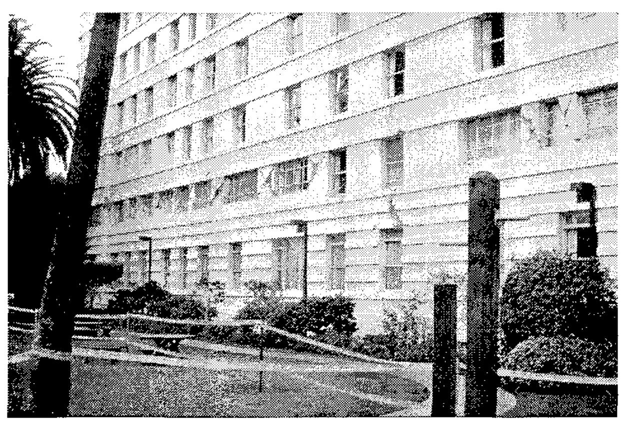

Fig. 1: St. John's Hospital damaged in the 1994 Northridge earthquake.

The 1971 San Fernando earthquake brought government officials, design professionals and health care providers in California to this recognition. As a result, the Legislature passed the first Hospital Seismic Safety Act in 1972. Since March 7, 1973, the design, construction and maintenance of California's hospitals has been governed by special statutes, regulations and design standards aimed at assuring hospital functionality following a major earthquake. The standards are intended to ensure that vulnerable patients are safe in an earthquake, and the facilities remain functional after such a disaster in order to care for injured persons in the community. These standards are implemented by California's Office of Statewide Health Planning and Development (OSHPD) and include stringent seismic design requirements, extensive plan review, approval of all designs, continuous construction inspection, thorough materials testing, and strict monitoring of all remodel projects.

All hospitals either new or remodel or additions in California designed after 1973 have been built under these stringent requirements. To date, the performance of these facilities has been excellent though it must be recognized that none have experienced their maximum credible earthquake.

The 1972 Seismic Safety Act as originally proposed called for the immediate strengthening or replacement of all hospital buildings that did not meet the modern standards. However, it was quickly realized that this was an economic impossibility. The proposed law was changed to apply only to new hospital buildings and existing hospital buildings undergoing substantial structural remodel or expansion and, therefore, all hospitals licensed at the time were "grandfathered" in – that is, they were not required to meet the new statewide standards. The intent was to bring any building whose useful life was being extended by a modernization program up to the modern seismic standards. However, the rate of retrofitting or replacing pre-73 hospital buildings was much too slow. The unexpected result has been to maintain the existing facilities as they are and build new facilities as needed.

In the January of 1994 Northridge Earthquake, several of these older hospitals sustained significant damage. Hospitals built in accordance with the standards of the Seismic Safety Act resisted the Northridge earthquake with minimal structural damage, while several facilities built prior to the act experienced major structural damage and had to be evacuated. It must be noted that certain nonstructural components of the hospitals did incur damage, even in facilities built in accordance with the structural provisions of the Seismic Safety Act (Table 1).

Table 1: Performance of all hospital buildings in the Northridge Earthquake at 23 hospital sites with one or more yellow or red tagged buildings.

Type of Damage	Number (%) of Buildings	
	Pre Act	**Post Act**
Structural Damage		
Red tagged	12 (24%)	0 (0%)
Yellow tagged	17 (33%)	1 (3%)
Green tagged	22 (43%)	30 (97%)
Nonstructural Damage		
Major	31 (61%)	7(23%)
Minor	20 (39%)	24(77%)
Total Buildings	51	31

The lessons from the Northridge Earthquake clearly showed that the majority of California's hospitals located in regions of highest seismicity do not comply with the new "functionality" standards and their expected performance during a major earthquake varies from moderate damage to complete collapse. The California Legislature clearly understood that a program was needed to require hospitals to improve the seismic resistance of their existing buildings in a phased and prioritized manner with the ultimate goal of full strengthening or replacement. The legislative response was SB 1953, which required that all hospitals meet statewide seismic safety standards.

2. POLICIES TO IMPROVE THE SEISMIC PERFORMANCE OF EXISTING HOSPITAL BUILDINGS

Seismic hazards mitigation/reduction programs may be quantified in two distinct categories - those that are *required* (mandatory) and those that are *voluntary* (non-mandatory). A *"voluntary"* program is guided by the building owner's non-mandatory actions to reduce seismic hazards. The *"required"* program has a building owner meeting specific government standards (e.g. building code). Such requirements may be either active or passive within existing statutes, regulations or building standards.

Passive requirements are those which mandate seismic hazard reduction only when a "trigger" (predefined change in existing condition) is activated by the building owner. *Passive* requirements to improve existing buildings incorporate limits on remodeling or alterations beyond which seismic upgrade is required. These limits can be tied to structural alteration, nonstructural remodeling, or the cost of a given improvement project.

An *Active* requirement is one that requires either reduction of seismic hazards for specific buildings, categories of buildings or elements. *Active* requirements use government mandated deadlines to force seismic strengthening by given dates in order to improve the seismic performance of existing buildings.

Since early 1973 the California Building Code (CBC) has specified triggers tied to improvements that are related to structural conditions. The CBC also incorporated provisions for voluntary seismic upgrade of hospital buildings. The structural triggers specified in the CBC are extensive and detailed. They have been based on reductions in lateral force capacity or increases in story mass. They are divided into three categories:

1. Incidental structural alterations, repairs or additions;
2. Minor structural alterations, repairs or additions; and,
3. Major structural alterations, repairs or additions

The incidental category requires only consideration of the alteration and local elements, the minor requires overall structural conformance with force levels of 50% of new buildings, and the major requires full conformance with the code.

Passive triggers in the California Building Code have been ineffective because the triggers can be avoided, and a limit was not placed on nonstuctural alterations allowing hospitals to install costly medical upgrades in existing buildings without overall seismic performance considerations. The voluntary seismic upgrade provisions were taken advantage of only a few times.

In the aftermath of the Northridge earthquake it was recognized that the process in place was inadequate to improve or replace the aging hospital building stock. It was time for active requirements to be utilized in order to fulfill the mandates of the seismic safety act. On September 22, 1994 California passed SB 1953 which requires all hospital buildings provide life-safety to their occupants by the year 2008 and by 2030 to be able to provide continuous operations and acute care medical services after a major earthquake.

3. A RATIONAL AND REALISTIC SOLUTION TO THE SEISMIC HAZARD MITIGATION PROBLEM FOR HOSPITAL BUILDINGS: PRIORITIZED MITIGATION AND REPLACEMENT

A rational and realistic program to improve the seismic resistance of existing hospital buildings based on the success of partial mitigation efforts in previous earthquakes should be phased and implemented over an extended period of time (be cost effective). It should take into account modernization needs of existing hospitals and be prioritized in a manner that yields the maximum protection possible with each step. The principal steps are to develop:

1. An earthquake preparedness plan;
2. Determine the seismic deficiencies of each hospital building;
3. Mitigate those nonstructural items that are required for a safe and orderly evacuation of the building as well as those required for maintaining the *critical* functions of the hospital for patient care;
4. Determine a level of structural strengthening based on life-safety concerns and the economic benefits, schedule the structural strengthening at a time that other collateral deficiencies can be corrected; and
5. Correct the deficiencies in the architectural systems and finishes to be upgraded within the normal remodel process.

A complete earthquake preparedness program should include a variety of disaster plans, an emergency communications network, and training programs for administrative and medical staff. To be successful, the program needs to be developed in conjunction with the users and implemented regularly. The training component should include specific information related to the expected seismic performance of the facilities and include appropriate criteria for immediate evacuation.

A complete seismic assessment of all hospital buildings should be conducted to determine the deficiencies of the various components in terms of life-safety and functionality. Life-safety should be the minimum seismic performance standard that concentrates solely on the safety of the patients and staff during a major earthquake and their ability to exit afterwards. The assessment for life-safety should concentrate on the structural system and any non-structural elements that could be considered as falling hazards. Functionality needs to be judged based on the modern design standards with the expectation that all buildings, systems, and equipment will be operational after a major earthquake. Each of these assessments will result in a list of deficiencies related to life safety concerns, and additional items related to functional concerns. These lists will form the basis of the long-term mitigation program.

An independent retrofit program should be developed and implemented to anchor and brace all mechanical, electrical and medical equipment, major piping systems and all building contents. The program should include procedures for properly anchoring and bracing all new systems and equipment, and include annual inspections to verify that the program is being effectively maintained. This activity will substantially improve the ability of an existing non-conforming hospital building to remain functional after a moderate or greater earthquake.

The evaluation and strengthening of the structural systems should be addressed apart from the other mitigation activities. Buildings should be considered in order of their life-safety concerns with the most dangerous building evaluated and strengthened first. All buildings should be at least strengthened to the point that they are considered to be "life-safe" given the largest expected earthquake. It has been shown in every earthquake that loss of life from building failures is not acceptable. The decision to strengthen a building beyond life safety, in an effort to achieve operational performance (functionality), should be a cost-based decision. Consideration needs to be given to the present and future costs related to various strengthening options beyond life-safety. These include:

1. Strengthen to a life-safety level and plan to repair after each major earthquake.
2. Strengthen to a full functional level to avoid the cost of substantial repairs after a major earthquake.
3. Replace the entire facility.

There are a number of methodologies currently available that direct the evaluation of existing facilities for life-safety concerns. One of these is FEMA 178: NEHRP Handbook for the Seismic Evaluation of Existing Buildings (FEMA 1992). This methodology is based on the actual behavior of buildings in major earthquakes and results in a list of potential "weak-links" that could lead to life-safety concerns.

A life-safety concern is a condition in which the failure of a building or building component could lead to loss of life, injury to the point of immobilization, or entrapment. Buildings judged to meet the FEMA 178 criteria are expected to provide adequate protection for their occupants and allow their egress after the earthquake. No consideration is given to the use or repair of the building after the earthquake. In most cases, buildings that barely meet this life-safety standard are expected to be unusable after a major earthquake.

All hospital buildings should be evaluated for life-safety concerns. Buildings failing to meet this standard should be given first priority for additional evaluation and strengthening. This group of buildings should be followed by those that meet the basic life-safety requirements but not the continuous operation requirements of the modern design standards.

At least two strengthening schemes should be developed conceptually for buildings with life-safety concerns. The first, a minimal strengthening scheme that corrects the deficiencies just to the point of meeting the life-safety standard. The second level should be a solution that brings the building into substantial conformance with the modern standards for continuous operation.

4. SB 1953 - THE SEISMIC RETROFIT PROGRAM FOR CALIFORNIA HOSPITALS

SB 1953 was introduced on February 25, 1994. It was signed into law on September 21, 1994 and became effective on September 22, 1994. The bill was an amendment of the Hospital Seismic Safety Act (HSSA) of 1983. There are approximately 470 general acute care hospital facilities in the State of California comprised of 2,673 hospital buildings that will be impacted by the provisions of SB 1953.
The specific provisions of the SB 1953 statutory language requires the OSHPD to develop definitions of earthquake performance categories in conjunction with seismic evaluation and retrofit procedures for general acute care hospital facilities within California. The regulations developed as a result of this legislation became effective on March 18, 1998.

The implementation of the bill is phased:

1. By January 1, 2001, all hospitals must complete and submit to OSHPD a seismic assessment of each building in which acute inpatient care is provided and if the buildings do not meet current standards a plan for achieving

compliance must also be presented (e.g., taking the building out of inpatient service, seismic retrofitting, or demolition and reconstruction) (Figure 2).

2. By January 1, 2002 all acute inpatient hospitals must meet minimum equipment anchorage standards (affecting, for example, communications systems, emergency power, medical gas systems, and fire suppression systems) (Figure 2).

3. By January 1, 2008, all acute inpatient hospital buildings which, according to the assessment, pose a significant risk of collapse in an earthquake must be taken out of service (Figure 2).

4. By January 1, 2030, all acute inpatient hospital buildings must meet standards designed to assure that they will remain operational after an earthquake (Figure 2).

As stated earlier, hospitals built in accordance with the standards of the HSSA resisted the January 1994 Northridge earthquake with minimal structural damage while several facilities built prior to the act experienced major structural damage and had to be evacuated. However, certain nonstructural components of the hospitals did incur damage, even in facilities built in accordance with the structural provisions of the HSSA. The provisions of SB 1953 and subsequent regulations were developed to address the issues of survivability of both nonstructural and structural components of hospital buildings after a seismic event.

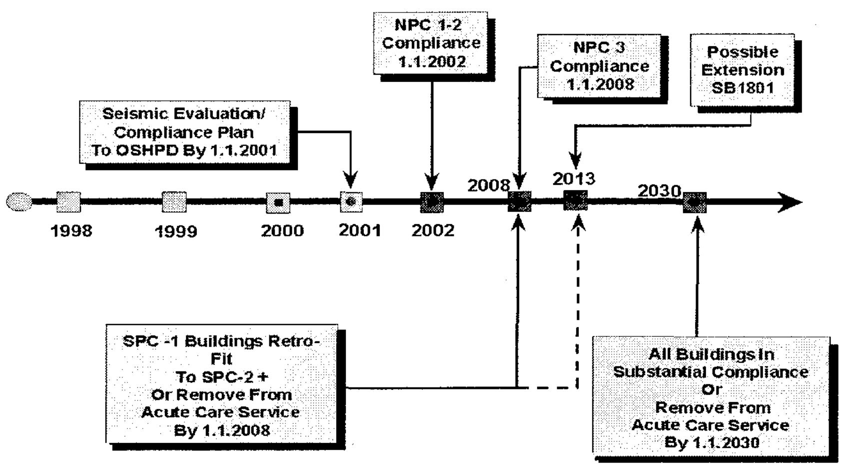

Fig. 2: SB 1953 Major Milestones

SB 1953 required the OSHPD to consult with the Hospital Building Safety Board (HBSB) to identify the most critical non-structural systems and prioritize the timeframes for upgrading these systems. The HBSB is an advisory board, appointed by the Director of OSHPD, and made up of Architects, Structural Engineers, hospital representatives and public members. They are experts in the design, construction and operation of hospital buildings. The Board's collective expertise was vital to the completion of the regulations by the deadlines established in SB 1953.

The regulations were developed in two stages. The first step in the retrofit program is the seismic evaluation of individual buildings. The evaluation places each building in a Structural Performance Category (SPC), and a Nonstructural Performance Category (NPC). There are five levels of each. The combined SPC and NPC rating of a building formulates it's overall seismic performance category, SPC/NPC.

There are two methods which can be used to determine the SPC and NPC of a building. The first is a rapid evaluation process described in regulation. The second is an alternative analysis allowing the design professional to utilize all of the factors associated with the buildings lateral force resisting system to the hospital's advantage. Because hospital buildings are found in areas of different seismic risk, the alternative analysis provides for the use of site specific ground motion data. This will generally result in lower seismic forces and less need for retrofitting.

Once the appropriate category is established, the hospitals must develop a plan for compliance which is presented to the office in the form of a bar graph. The bar graph will depict the time allowed for each step necessary to comply with the dates set in both law and regulation.

The second stage of regulations development was to specify how to accomplish a retrofit. As with the evaluation, both a prescriptive method and an alternative method are established. The prescriptive method often referred to as method A defines each step to be followed as designing a retrofit solution. Under Method B, the alternative method, the design professional is allowed to be creative and use all of the state of the art tools of structural engineering to obtain compliance with the desired structural and non-structural performance category. This promises to be innovative, difficult and challenging.

The timelines are established in the law. OSHPD and the HBSB created the SPC and NPC based on a philosophy expressed in the law. Clearly, buildings which represent a "potential risk of collapse or pose a significant loss of life" are to be closed, retrofitted, or removed from acute care use by January 1, 2008. It is expected that during the years 2008 to 2030 a number of hospital buildings may be heavily damaged and non-functional as a result of earthquakes, but will not collapse. A category for non-structural compliance, (NPC-2) which preserves the systems deemed necessary for evacuation, was established with a January 1, 2002 deadline. In this case, the building may have partially collapsed and those systems need to be reasonably available to keep people alive and aid evacuation. The systems are communications, bulk medical gas, emergency power, and fire alarms.

In addition to preventing building collapse by January 1, 2008, there are additional non-structural system requirements which also must be met by 2008. These items are found in the California Building Code. No new items were created. In keeping with the philosophy of preventing loss of life, it was felt that persons undergoing invasive procedures should have some confidence in the reliability of the physical plant. Therefore, the systems serving defined Critical Care Areas within the hospital must be retrofitted by 2008. Preliminary estimates indicate that this requirement effects between 15% - 30% of the square footage of the hospital.

There is a provision in the law which allows delays in compliance with the 2008 deadline. The provision says, "A delay in this deadline may be granted by the office upon a demonstration by the owner that compliance will result in a loss of health care capacity that may not be provided by other general acute care hospitals within a reasonable proximity". This has been further defined in regulations to be a maximum of five years, in one-year increments.

The final steps occur between years 2008 and 2030. The law requires the buildings to be in substantial compliance with the Act by January 1, 2030. During this 22 year period, retrofitting and new construction will occur to reach substantial compliance while the buildings housing patients will at least not collapse and the systems serving critical care will function.

Since the inception of SB 1953, it has been understood that this is a huge financial as well as physical undertaking.

While working with the HBSB in developing the regulations for the seismic retrofit program considered the financial impact of each provision very carefully . As a group of experts keenly aware of the cost of retrofitting OSHPD attempted to require only the absolute minimum and give as much flexibility as possible for compliance.

5. THE SEISMIC EVALUATION PROCEDURE

OSHPD utilized as a source document *FEMA 178: NEHRP Handbook for the Seismic Evaluation of Existing Buildings* (FEMA 1992) to develop the seismic evaluation methodology for existing hospital buildings currently specified in the regulations of the California Building Code.

The seismic evaluation procedure regulations consist of eleven articles. The primary purpose of these regulations is to evaluate the potential earthquake performance of a building or building components and to place the building into specified seismic performance categories. The evaluation procedures were developed from experience gained in evaluating and seismically retrofitting deficient buildings in areas of high seismicity. These evaluation statements should be used with engineering judgment and the current building codes as a guide. The methodology provided in the seismic evaluation identifies potential "weak links" as stated earlier that could lead to life safety concerns in the event of a major earthquake. This life safety performance level defined in FEMA 178 is stated as follows:

A building does not meet the life-safety objective of this handbook if in an earthquake the entire building collapses, portions of the building collapse, components of the building fail or fall, or exit and entry routes are blocked, preventing the evacuation and rescue of occupants.

The seismic evaluation in the California Building Code, as in FEMA 178, does not predict the level of damage that a building might experience nor is it satisfactory for determining if a building will remain operational. The evaluation methodology recognizes that successful structural performance is based on having a complete lateral force resisting system that has sufficient strength and ductility. The ductility is manifested in the manner in which the building has been detailed. Because it applies to existing buildings, the evaluation methodology provides a significant amount of instruction on how to deal with buildings that are not detailed in a manner consistent with the current code. Fundamentally, the evaluation methodology permits the code requirements for ductility to be superseded by extra strength.

The seismic evaluation procedure utilizes a series of true/ false statements to identify potential weak links in buildings. For each of the statements, procedures are suggested for doing detailed evaluations to determine if these potential-weak links are, in fact, sources of life-safety concern.

The building evaluation process begins with a site visit and the gathering of all information that is needed to do a seismic evaluation. This is followed by selecting the appropriate model building type and set of evaluation statements for the initial screening. Based on the available information, each evaluation statement is considered and answered either true of false. A true answer implies that the potential weak link is not a concern. A false statement implies that the potential weak link needs further evaluation. Following the first level evaluation, the engineer is encouraged to return to the site and gather additional information necessary to carry out the detailed evaluation of potential weak links. The seismic evaluation allows a number of different types of analysis for evaluating potential weak links. The engineer is given complete latitude to select an appropriate evaluation technique and advised to consider the result of the final evaluation in terms of the overall performance of the building.

5.1 Ground Motion Requirements

The ground motion criteria for the seismic evaluation of existing hospital buildings are defined by acceleration coefficients (as a measure of ground shaking level), the site coefficient (to account for the effects of local soil profile) and an elastic response spectrum to represent the change in acceleration with the predominant period of the ground motion.

As in FEMA 178, two parameters are used to characterize the intensity of the ground shaking. These parameters are known as the effective peak acceleration (EPA), A_a, and the effective peak velocity-related acceleration (EPV), A_v. The lower limit for both acceleration coefficients is set at 0.2g.

The methodology used for evaluation of existing buildings in FEMA 178 establishes seismic forces that are lower than those prescribed by the seismic design criteria for new buildings for the following reasons:

1. Buildings should be substantially below the current standards before triggering the requirement for a seismic upgrade; and,
2. A higher level of earthquake damage is acceptable in an existing building

The same concept has been maintained also in the evaluation criteria for existing hospital buildings. This is accomplished by modifying the spectral amplification factors for the ground acceleration and velocity to 85 percent and 67 percent respectively to represent mean values.

Where advanced analysis procedures are utilized for seismic evaluation of existing hospital buildings the ground motion representation shall be elastic response spectra or time histories developed for mean values for the specific site, in accordance with the procedures specified by Title 24.

6. CONCLUSION

Past earthquakes have clearly demonstrated the vulnerability of existing hospitals in major earthquakes and their unquestioned need thereafter. The repair and retrofit of hospital buildings requires determination of structural and non-structural deficiencies that exist, and effective methodologies for mitigating these deficiencies to preserve the usability of the hospital.

A hospital building that does not meet modern standards does not constitute an undependable structure. Even the most deficient buildings can be life-safe for lower intensity events. Partial mitigation and strengthening programs have been shown to be quite effective and can provide the basis for a realistic strengthening program.

OSHPD has developed a rational and realistic program to improve the seismic resistance of existing hospital buildings tailored after the success of mitigation efforts in past earthquakes. The principal steps are as follows:

1) Short term:
   a) Determine the seismic deficiencies of the each hospital building in terms of the structure, all systems, equipment and contents.
   b) Mitigate all non-structural deficiencies that can assure the safe and orderly evacuation of the hospital building as soon as possible.
   c) Determine a level of structural strengthening based on life-safety concerns and the economic benefits.
2) Intermediate term:
   a) Replace the hospital building or strengthen the structural system of the existing hospital building at least to the life safety performance level for an intermediate period in time. Schedule corrections to other collateral deficiencies to take place at the time of the structural strengthening in order to minimize construction impact.
   b) Anchor and brace the nonstructural components and systems (mechanical, electrical, medical, equipment, major piping and building contents) required for maintaining critical functions of the hospital for patient care (critical care areas).
3) Long term:
   a) Correct deficiencies in the architectural systems and finishes during the normal remodel process.
   b) Anchor and brace all mechanical, electrical, medical, equipment, major piping and all building contents to the requirements of modern building standards for the operational performance level.
   c) Replace the hospital building or Strengthen the structural system of the existing hospital building to meet substantial compliance with modern building standards (operational performance level).

In summary, it is postulated that SB1953 and the regulatory framework established to accomplish compliance is a reasonable and rational approach to necessary seismic retrofit. The seismic retrofit program as designed preserves as much flexibility to the process as can be possible.

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Recent Trends of Damage Controlled Structures in Japan

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Introduction

The concept of a "damage tolerant" structure was proposed in Japan about 10 years ago (Wada et al. 1992). Damage tolerant means that the acceptable damage due to an earthquake occurs in specific structural components such as braces, shear walls, or supplemental dampers. These damaged components are called the "sacrifice members" and function somewhat like a fuse to protect the primary structure from severe damage. Since the Northridge and the Hyogoken-Nanbu earthquakes, this kind of structure has received increasing attention by researchers and structural engineers in both the U.S. and Japan.

Since these earthquake-resistant buildings installed with a damping system have also widely increased in both countries as evidenced by the literature (Soong, et al. 1994, 1997; Constantinou et al. 1998; Housner et al. 1997, Whittaker, etc. 1997), (Wada et al. 1992, 1997). On the cover of the *Engineering News-Record* (ENR 1997), the word "sacrifice" was used in the short explanatory notes that appeared together with a conceptual picture of a damped structure. These notes explained that the energy absorption that occurred from the axial yielding of the damping brace became the sacrifice through which high-rise building structures were saved even during a large earthquake. For the framed steel structures without braces, the sacrifice becomes the flange welded part of the beam ends. Little energy absorption can be expected from the plastic deformation of the beam ends during an earthquake, as was clearly demonstrated during the Northridge and the Hyogoken-Nanbu earthquakes, because the plastic deformation of the beam ends is equivalent to the method of mounting elasto-plastic dampers in series shown in Fig. 1 in a part of an elastic frame, leading to large deformation of the whole frame after it becomes plastic.

The main purpose of this paper is to review the concept of the damage tolerant structure so called damage controlled structure that was proposed before the Northridge Earthquake and Hyogoken-Nanbu earthquakes. A couple of actual example projects are introduced which exemplifies the current seismic design trend in Japan.

Philosophy of damage-Controlled structures

Japanese seismic design standards define two levels of earthquake ground motions and allowable damage for each. For level 1, medium earthquake ground motions, only minor damage such as cracks in walls and concrete beams are allowed, while the building structure are secured. For level 2, the largest earthquake ground motions, a building structure is allowed to be damaged as far as human life is guaranteed. The current seismic design and research in Japan are based on this consensus. However, since the buildings have recently increased in scale and value, due to the need to accommodate expensive computer and communication equipment the conventional design consensus should be changed. The lessons learned from the Northridge and Hyogoken-Nanbu earthquakes taught us that the damage to the primary structure would cause both great loss of human life and economic activities. It is obvious that overlarge plastic deformation of a building should not be allowed for large earthquakes; in addition, construction activities requiring the production of cement and steel raise new concerns about environmental problems, such as ruining rain forests and increasing CO_2. These problems will be reduced relatively by lengthening a building's life span, but to be meaningful such solutions must also provide that large buildings remain functional after an extreme event.

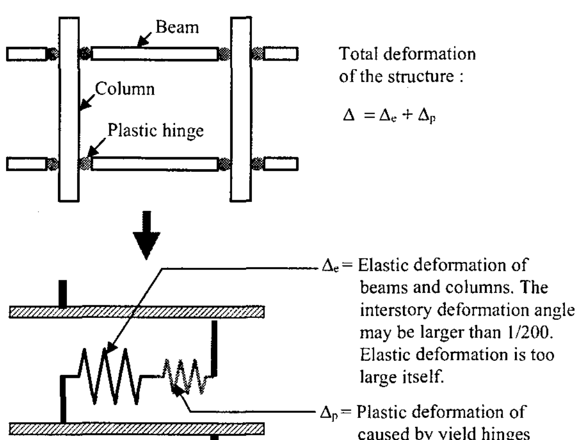

Fig. 1. Strong-column weak-beam model

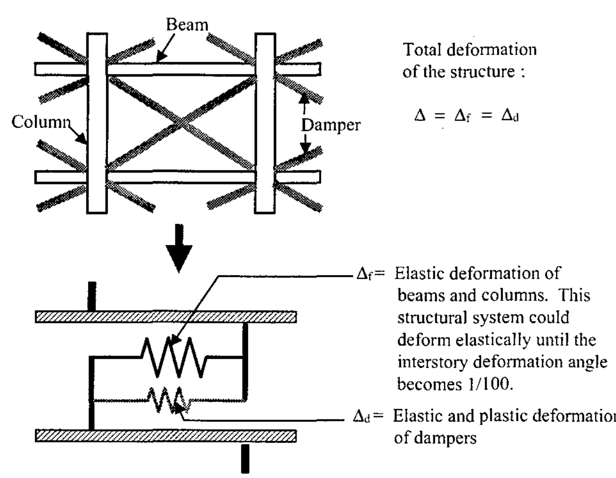

Fig. 2. Structure with damper system

Building structures must be designed to be able to withstand extremely large loading that can occur randomly over the whole span of their useful lives. The earthquake disasters mentioned above highlight the need to build buildings strong enough to tolerate big earthquakes, and the necessary functions of buildings should be able to be restored as soon as possible after an earthquake.

In earthquake-resistant design of building structures, priority is of course given to protection of human lives against extremely infrequent large earthquakes. But there is also a great of economic loss associated with the stoppage of economic activities. This is especially so where the damage occurs in large cities. This problem has been considered very seriously by many people not only in the United States and Japan but also in other countries.

The conventional seismic structure allows the beam-end parts to yield and experience large plastic deformation until plastic hinges are produced under large earthquake excitation. The target of the conventional structure is to use the plastic deformation at the beam-ends to dissipate the energy input from the earthquake excitation. The conventional structures are called "strong-column weak-beam" structures and have been accepted by structural engineers so far. Fig. 1 shows the concept behind strong-column weak-beam structures. Such structures can be treated as a system of two springs that are connected in series. The total deformation of such structure after the beam-end parts yield is the summation of the elastic deformation $\square_e$ and the plastic deformation of the plastic hinges $\square_p$. Obviously, the plastic hinges at the beam-ends increase the total deformation of the entire structure and reduce the lateral stiffness of the whole building structure.

In the Northridge and Hyogoken-Nanbu earthquakes, however, a great deal of collapse and damage was observed in the steel structures that have been considered the structures most resistant to earthquake. The main cause of steel structure damage has been considered the overlarge plastic deformation at the beam-ends where the weld connection between the beam and column is located. The concept of relying on the primary structure's ductility to absorb the energy input of the earthquake will eventually result in the collapse of or severe damage to the primary structures. For reinforced concrete structures, it is obviously impossible to rely on the plastic deformation at beam-ends to dissipate the energy input from the earthquake excitation.

The basic concept of damage-controlled structures (Wada et al., 1992 1997) is stated as follows. The entire building structure consists of two independent parts. One is the primary structure composed of beams and columns, which aims to resist the vertical service load. The primary structure is designed to behave elastically and to keep its building service functions even after an extremely large earthquake. The second is the damper system that aims to resist the lateral forces resulting from the earthquake ground motion. The damage produced from the earthquake is controlled within the damper system, which is easily checked, repaired or replaced after the earthquake.

Fig. 2 illustrates the structural model of damage-controlled structures. The damage is controlled within the brace-type damper system. The primary structural frame and the damper system can be considered a system of two springs connected in parallel. The total deformation of the entire structure $\square$ is equal to the frame deformation $\square_f$ and also equal to the deformation $\square_d$ of damper system. The advantage of this structural system is to increase both energy dissipation capacity and the strength of the structure without increasing the total deformation of the entire structure.

Story Deflection Angle At Material Starting To Yield

Let us consider the deformation of a frame consisting of a beam and column with a brace placed at 45 degrees, as shown in Fig. 3. Compared with the shear deformation □ occurring in the frame, the expansion and contraction of the brace becomes $\Delta/\sqrt{2}$. Because the brace length is $\sqrt{2}$ times the column length L, the axial strain in the brace becomes 1/2 of the shear deformation angle of the frame. Taking into account the fact that the joints at the brace ends are of high rigidity and strength and are thus not made plastic, the findings are as follows. After converting the yield strain of the steel member to be a little higher than 0.1%, it is found that the story deformation angle is as small as 1/500 when the brace yields.

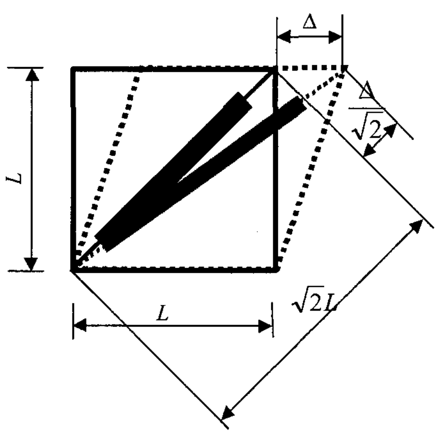

Fig. 3. Axial deformations of brace and shear deformation of frame

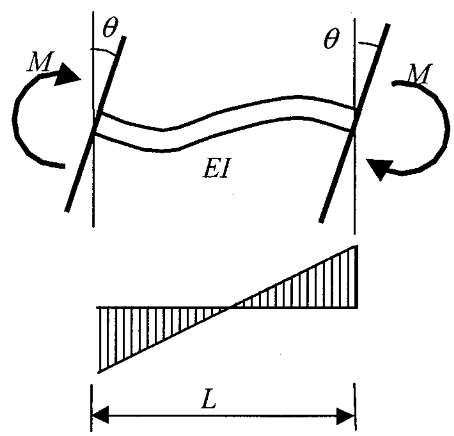

Fig. 4. Deformation angle at beam ends of a rigid frame

For a steel plate shear wall, since the yield shear stress is $1/\sqrt{3}$ times the yield normal stress and the shear elastic modulus is 1/2.6 times Young's modulus, the shear yield strain is then about 1.5 (=2.6/1.732) times the axial yield strain. It means that the story deformation angle becomes about 1/667 when the steel plate shear wall begins to yield and it is made plastic at almost the same level as the small story deformation angle for the brace. It is possible to make the story deformation angle at the beginning of plastic deformation smaller for the brace and steel plate wall by using ultra-low yield steel to locally concentrate the plastic-deforming part.

Let us consider the yield deformation angle of a rigid joint frame comprised of columns and beams which receive an asymmetric bending moment (Fig. 4). For the steel structure frame, discussion is focused on the rotational distortion occurring at the beam ends, because the bending deflection in the beam comprises nearly half of the structure deformation. Let the span and the depth of the beam be L and D, respectively. Then, the deformation angle for the stress of the flange at the beam ends to reach the yield point $\square_y$ becomes $(\square_y/3E)(L/D)$. The span L of the frame is predetermined and Young's modulus E is a constant. Therefore, it is found that the deformation at the yield point of the frame can be increased by using steel having a high yield point $\square_y$ and members having a smaller depth D than conventional ones. In other words, the elastic deformation capacity of a frame can be increased using a slender flexible frame manufactured by high-strength steels. As a result, the yield deformation of the moment-resistant frame can be easily determined by selecting the materials and the cross sections of the structural members. On the contrary, the yield deformations of the damping components such as braces and shear walls are determined from the overall configuration and the material selection. Thus, yield deformation cannot be changed by adjusting the plate thickness and the local configuration.

Typical projects of damage-controlled structures

Since the Hyogoken-Nanbu earthquake, many building projects that were designed based on the concept of damage-controlled structures have been appraised by the Japan Building Center. Some of them have already been constructed. Table 1 gives a list of such typical projects that were designed between 1995 and 1998. In this table, S_F stands for steel frame structure; RC_F stands for reinforced concrete structure; HD_B stands for brace type hysteretic damper made of steels like unbonded braces; HD_S stands for shear panel type hysteretic damper made of steel; HD_BD stands for bending type hysteretic damper like slit damper and honeycomb damper; VD_S stands for shear wall type viscous damper like oil filled shear walls; VD_B stands for brace type viscous damper like oil piston damper.

Table 1. Tall steel buildings designed based on the same concept of damage-controlled structures between 1995 and 1998

Year	Project name	Location	Usage	Height (m)	Structure type	Dampers	Ductility ratio of primary frame
995.6	International Congress	Osaka	Congress	104	S_F	HD_B	0.95
1995.7	Todai Hospital	Tokyo	Hospital	82	S_F	VD_S	0.93
1995.7	Tohokudai Hospital	Sendai	Hospital	80	S_F	VD_S	0.97
1995.8	Central Government	Tokyo	Office	100	S_F	HD_B + VD_S	0.78
1995.10	Harumi 1 Chome	Tokyo	Office, Shop	175	S_F	HD_B	0.88
1996.2	Toranomon 2 Chome	Tokyo	Office, Shop	94	S_F	VD_S	0.94
1996.3	Sankyo	Tokyo	Office	61	S_F	HD_B	0.88
1996.4	Shiba 3 Chome	Tokyo	Office	152	S_F	HD_B	0.97
1996.6	Art Hotel	Sapporo	Hotel	96	S_F	HD_BD	0.85
1996.8	Kanto Post Office	Saitama	Office	130	S_F	VD_S	0.87
1996.10	Nakano Urban	Tokyo	Office, Shop	96	S_F	VD_S	0.68
1997.7	DoCoMo Tokyo	Tokyo	Communication ,etc.	240	S_F	VD_S	0.79
1997.10	Minato Future	Yokohama	Hotel, Shop, Office	99	S_F	HD_BD	0.98
1997.11	Nishiguchi Shintoshin	Yamagata	Office, Hotel, etc.	110	S_F	HD_B	1.00
1998.2	DoCoMo Nagano	Nagano	Communication	75	S_F	VD_S	0.89
1998.4	East Osaka City	East Osaka	Office	120	S_F	HD_S	1.00
1998.5	Kouraku Mori	Tokyo	Office, Shop	82	S_F	HD_B	1.00
1998.7	Harumi 1 Chome	Tokyo	Office, Shop, etc.	88	RC_F	HD_B	1.00
1998.11	Adago 2 Chome	Tokyo	Office, Shop	187	S_F	VD_B	0.71
1998.11	Gunyama Station	Fukushima	Shop, School, etc.	128	S_F	HD_B + VD_S	0.98

Central Government Building

The Central Government Building (Fig. 5) located in Chiyoda-ku, Tokyo, is a typical damage-controlled structure combining hysteretic and viscous fluid dampers. This building was designed by the Architecture Department of the Ministry of Construction and Kume Sekkei Co., Ltd. The total height is 144.5 m, including a 55 m antenna tower on the roof. The superstructure above the ground level is a moment-resistant steel frame installed with various damper systems, while the underground structure is a steel reinforced concrete frame with reinforced concrete shear walls. The columns and beams of the primary structure used SN490B steel (maximum strength is 490MPa). The primary structure is designed to behave elastically even under a large intensity earthquake whose maximum ground velocity is 50 cm/s. Most of the earthquake energy is absorbed by the damping system. The hysteretic dampers (HDs) are steel walls made of extra-low yield point steel (yield point is 100 MPa). The yield shear force level of HDs at the 1st floor location is assumed to be 5% of the total building weight. The distribution of yield shear force throughout the height of the building is assumed to be proportional to the distribution of yield shear force of the primary structure. On the other hand, the viscous dampers (VD) consist of two movable steel plates and three fixed steel plates. The space between the movable steel plates and the fixed steel plates is filled with viscous liquid like silicone oil.

Fig. 5. Central government building (with HD+VD)
(Courtesy of Kume Sekkei Co. Ltd.)

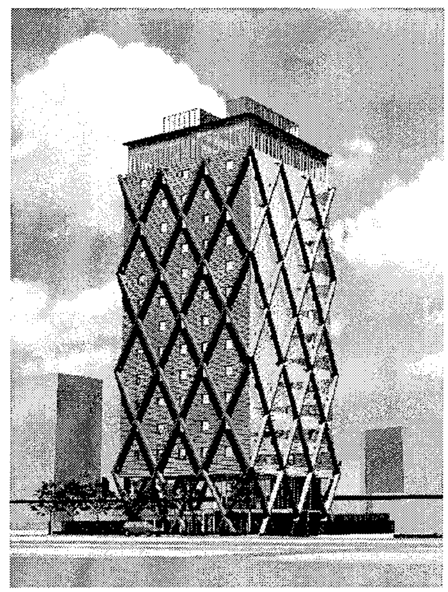

Fig. 6. Sankyo in Shibuya (with HD)
(Courtesy of Nippon Steel Corporation)

Sankyo Building in Shibuya, Tokyo

The Sankyo Building, located in Shibuya, Tokyo (Fig. 6) was designed by Plantec Design Office (structural design was by Alpha Structural Design Office and Nippon Steel Corporation). The structural system of this office building has no vertical columns. The vertical and lateral loads are supported by the inclined column system. This building is 61.4 m high and has 14 stories. The entire structural system is composed of two independent structural systems; elastic column system at each corner and an unbonded member system in central part of each frame. The unbonded member system has an equivalent damping coefficient of about 8%. Since the entire structural system is divided into two independent systems, damage to the building in the case of an extremely large earthquake would be confined to the unbonded members which are designed to be easily replaceable.

Conclusions

In Japan, the current trend for seismically designed building structures is based on the concept of a damage-controlled structure instead of the conventional strong-column weak-beam system. The philosophy behind a damage-controlled structure views the global structure as divided into two independent parts: the elastic primary structure that is designed to support the vertical service load, the other is the damping system that is designed to resist the lateral earthquake load. The damage caused by the earthquake is artificially controlled by the damping system. The primary structure remains in elastic region even during an extremely large earthquake.

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Critical Facilities in New York State: General Comments

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I. **Overview regarding NYS Critical Facilities** (specifically Hospitals/Health Care Facilities) and seismic design

A. AWARENESS & ACCEPTANCE of seismic risk are a major challenge in healthcare facilities in NY State (due to complexity and cost of retrofitting), particularly from a government/policy/regulatory perspective. New York State Healthcare Environment continues to be tightly regulated, resulting in concerns over capital investment and operational costs vs. reimbursement.

1. Rather than "voluntary", need for action/planning must be in policy to ensure that proper regulations and financial recognition exist
2. Seismic risk may be viewed as "possible" but unlikely
3. Again: historic assumption of low probability = high risk (per Dr. Lee)

I. **Current situation in New York State**

A. No STATEWIDE mandate for seismic design specifically (via NYSUFPBC) aside from allowance for local requirements where more strict = NYC seismic requirement update circa 1998.

B. NYS now considering/planning for adoption of Model Code

1. International Building Code (International Code Council/**IBC**) has seismic requirements. Concept of Spectral Response Acceleration & Seismic Use Group/Importance Factor.
2. Adoption on schedule for Early 2002
3. Therefore: seismic design for new construction could be reality Statewide by 2002.

C. There has been a growing (slowly) awareness of the NEED for more comprehensive disaster planning in the HEALTH/HOSPITAL arena

1. Recent incidents involving ICE STORMS and FLOODING have had extended impact on healthcare facilities (mostly NH)
2. Global warming/El Nino/other changes have resulted in anecdotal information regarding increased risk/probability of major category storms/storm surge on the east coast
3. Increased awareness of need for Health care Facilities to be able to accommodate Haz Mat and Bio-Terrorism
4. There is overlap/are similarities in planning for major disaster, no matter what the specific disaster.

II. Opportunities/Requirements

A. Policy

1. Need to increase awareness/recognition and acceptance of RISK
2. Need to consider POLICY, enabling REGULATIONS as appropriate and plan for FINANCIAL support/recognition
3. Need to develop overall policy on how to proceed specific to REGION, PROBABILITY/potential, and a PLAN FOR RETROFITTING that considers all variables.
4. NATIONALLY: **AIA Guidelines for Healthcare Facilities** reference ASCE 7-93, 1988 NEHRP Provisions, 1991 ICBO and 1992 supplements/amendments to both BOCA and SBCC Standard Building Codes. Although language is general, opportunity may exist:

a) Increasing presence of State AHJ's on revision committee (28 this cycle) for the AIA Guidelines for Healthcare Facilities.
b) Approximately 40-states now reference Guidelines all or in part
c) Participation in next cycle may provide another avenue of promulgating seismic requirements/consideration/planning/awareness in Health Care on a national basis.

5. Though NYS is among the first to consider IBC, other states are considering as well.

B. Planning and Operations

1. Disaster planning in-house per state of the art: should build on experience in dealing with other disasters and "customize" for seismic considerations
2. Coordination with local/municipal (State and Federal?) for regional perspective

III. Strategy

No need to start at "square one"...OSHPD clearly has much experience in planning for seismic response, and would be a good starting point to develop strategies for analysis, planning and evaluation of structures.

A. Global/overall

1. Regional Disaster Plan

a) Prioritize Facilities in Region

b) Coordinate Lifeline Networks involving utilities/water/emergency power, traffic arteries, food service, emergency vehicles and communication

B. Facility Specific

1. Prioritize departments
2. Prioritize systems
3. Prioritize equipment (infrastructure and medical)
4. Ie: ER, Blood Supplies, Surgery Facilities, ICU, NICU,
5. Communication advances/computerized medical records

C. Implementation

1. Evaluate each facility for weaknesses and strengths/Develop Plan
2. Opportunities via New Construction
3. Opportunities for renovation
4. Opportunities for mitigation

IV. Miscellaneous

A. Must close gap between academics/research >>>> designers ....contractors.

B. Larger Construction Companies/CM's have engineering capability and for due diligence purposes should be interested in participating right from the start to consider and accommodate seismic design/retrofit strategies up front.

C. Retrofit strategies will be a major area all by itself: cross-over to other technologies is an excellent means of developing new retrofits. Ultimately **applicability** and **acceptance** will be key in New York State (and other low probability areas): If cost is high, applicability and acceptance will be limited to areas of high probability. If cost is lower/more reasonable, applicability and acceptance will be more prominent/likely in areas of lower probability.

Technical Block 3: Advanced Technologies for Nonstructural Retrofit

Chairs: Mircea Grigoriu and Daniel Inman

General Overview of Earthquake Engineering Issues for Nonstructural Systems
Micea Grigoriu, Christopher Roth and Ehab Mostafa

Static Axial Behavior of Some Typical Restrained and Unrestrained Pipe Joints
M. Maragakis, R. Siddharthan and Ronald Meis

Seismic Vulnerability and Retrofit of Nonstructural Components
T.T. Soong

Seismic Protection of Some Nonstructural Components in Hospitals
M.P. Singh and Rildova

Part I - Seismic Retrofit of Critical Piping Systems and Part II - Seismic Design of Critical Piping Systems (Above Ground Piping)
George Antaki

Recycled Plastics: Characteristics and Seismic Application
M. Ala Saadeghvaziri and K. MacBain

Smart Materials Technologies for Bolted-joints in Civil Systems
Gyuhae Park, Daniel E. Muntges and Daniel J. Inman

General Overview of Earthquake Engineering Issues for Non-structural Systems

Mircea Grigoriu, Christopher Roth, Ehab Mostafa
Cornell University

Abstract

Research at MCEER focuses on different methods of developing fragility information for both structural and non-structural components and systems, based on filed data, experimentation and numerical analysis. Fragility information has been developed for buried pipe joints and for block-type equipment. Current work focuses on the modeling, dynamic analysis, testing and retrofit of piping systems in hospitals. Fragility, sensitivity, and cost-benefit analysis are used to assess the seismic performance of alternative designs.

Introduction

Current research at MCEER focuses on different methods of developing fragility information for both structural and non-structural components and systems. Fragility curves, giving the probability of exceeding a specific limit state as a function of ground motion intensity, are commonly used to present the fragility information.

There are two main approaches for generating fragility curves. The first approach is based on damage data obtained from field observations after an earthquake and/or experimental results. The second approach is based on numerical analysis of the structure, either through detailed time-history analysis or through simplified methods. Both methods are discussed in this paper.

Structural system fragility

As structural systems will be covered in Technical Block 2 of the MEDAT-2 conference, a few short comments will be sufficient here. Computer models of the structural systems of three hospitals have been prepared at the University at Buffalo. These models can be used as input to non-structural system analyses and are available at *http://mceer.buffalo.edu/hospitals*.

In other MCEER work, fragility curves have been developed for Caltrans' expressway bridges in Los Angeles County, California. The curves were based on field data collected after the Northridge earthquake (Shinozuka et al, 2000). Barron and Reinhorn (2000) used the simplified Spectral Capacity Method to generate fragility curves for a building in Memphis.

A systems analysis approach was used to evaluate the seismic risk of an expressway network in the Los Angeles area under a postulated magnitude 7.1 Elysian Park earthquake (Shinozuka et al, 2000). The families of fragility curves developed from field data were used to determine the relationship between the peak ground acceleration at a bridge site and the probability of damage to the bridge. States of damage for all 2,225 Caltrans' bridges in Los Angeles and Orange County were simulated, and translated into traffic flow capacity on each expressway link. This information can be used to support decision-making for post-earthquake response activities in near real-time.

Non-structural system fragility

Two assumptions are commonly made in the seismic analysis of secondary non-structural systems supported by primary structural systems in the eastern United States: (1) the supporting structural system remains linear; and (2) cascade analysis applies, that is, there is no feedback from the primary to the secondary system. The first assumption may not hold in the western United States.

Component fragility analysis

Damage assessment curves for buried pipe joints

Pipelines transporting water, gas, or volatile fuels are part of the hospital non-structural system and are critical to the functioning of the hospital following an earthquake. Risk assessment charts for pipe joints based on experimental data have been developed. Twenty-two separate specimens of various pipe materials and various joint types were tested under cyclic axial loading. Several different failure modes such as buckling and fracture of the barrel and spigot end were observed.

Risk assessment charts that compare the pipe force capacity with a force level that may be imposed on the pipe due to seismic motion have been developed. Failure will occur if both the level of force from seismic motion and the level of force that can be transferred from the soil to the pipe surface by friction are greater than the pipe force capacity. Given the pipe capacity, the failure condition can be expressed in terms of earthquake and soil parameters such as predominant period of the earthquake (T_p), peak particle velocity (V_p) and the wave propagation velocity (c). Figure 1 is a typical risk assessment plot for one of the joints. This information can be used along with ground motion data to develop fragility curves.

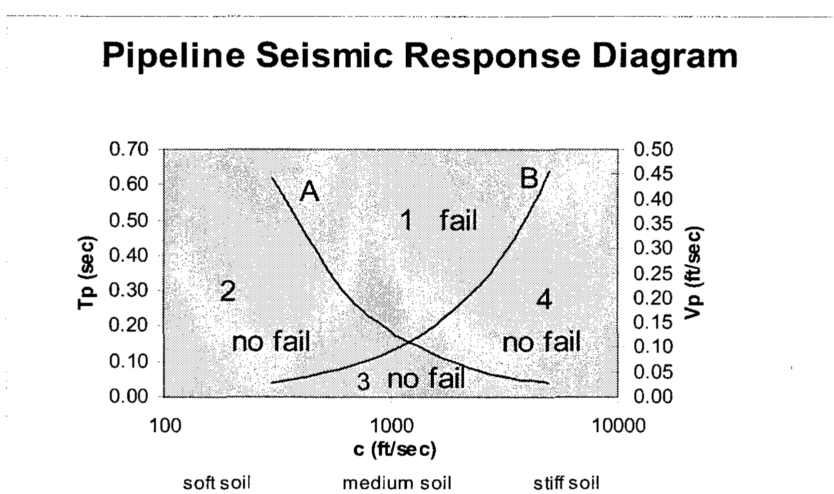

Figure 1. Pipe joint capacity risk plots

Sliding fragility curves of unrestrained equipment

Fragility curves for free-standing rigid equipment were developed based on experimental data and numerical analysis (Chong et al, 2000). The sliding motion of a rigid block against a floor surface was tested on a shaking table using five randomly chosen earthquake time histories. Both horizontal and vertical accelerations were considered in these experiments. Five horizontal peak ground accelerations (HPGA) were considered, namely 0.3g, 0.4g, 0.5g, 0.6g, and 0.7g; and four different scale factors were used to represent the vertical peak ground accelerations (VPGA) in terms of HPGA: 0, 1/4, 1/3, and 1/2. Eight different relative displacement failure thresholds between 0.1 inch and 3 inches were considered, and fragility curves developed for each of these thresholds. Figure 2 shows the experimental fragility curves for failure threshold of 1 inch for the four different ratios between vertical and horizontal peak ground accelerations.

Fragility curves were also constructed analytically for the same eight failure thresholds. Ninety acceleration time histories were generated based on a response spectrum from the 1997 NEHRP recommended provisions for seismic regulations for new buildings and other structures. Each of the ninety acceleration time history inputs was scaled to have eight different horizontal peak ground accelerations between 0.3g and 1.0g. Each of these eight horizontal time histories was combined with four different vertical acceleration inputs, again scaled to give ratios between VPGA and HPGA of 0, 1/4, 1/3, and 1/2. The total number of time history combinations was 2,880. All of the time history combinations were repeated for five different coefficients of dynamic friction.

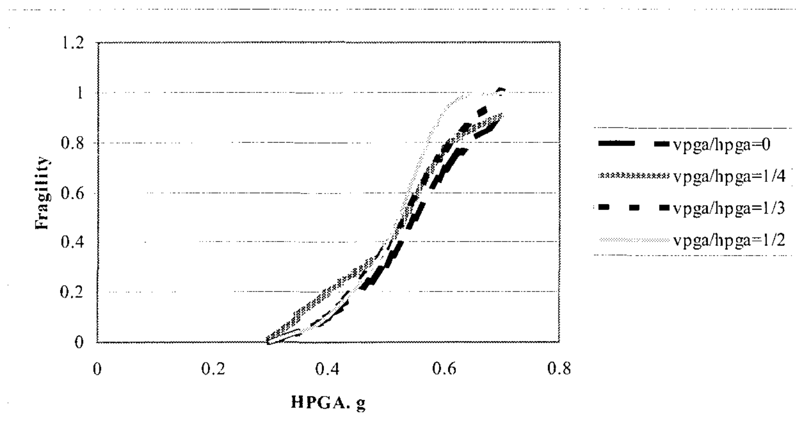

Figure 2. Experimental fragility curves for failure threshold = 1 inch

System fragility analysis

Logic tree analysis of piping systems

Logic trees are frequently used to represent and analyze nonstructural systems. An example of such a tree is shown in Figure 3 for a San Francisco high-rise fire suppression system (Grigoriu and Waisman, 1998). The tree consists of the components along with "gates" describing the logical connectivity of the components. For example, in Figure 3 the water supply will function if one or both of the city water and the 20,000 gallon tank function; while the entire system will function only if all three of the water supply, water pumps and piping function. Equations have been derived for calculating the probability of failure of the system, P_{SF}, from the probability of failure of the individual components, p_{cw}, p_{gt}, etc. These equations can be used to generate a fragility curve for the system from the fragility curves of the components. The logic tree method can also be used for sensitivity analysis to identify the critical components of the system, and to determine confidence intervals for the system fragility (Roth, 1999). For example, Figure 4 shows the sensitivities of the system failure probability to the individual component failure probabilities as functions of the peak ground acceleration.

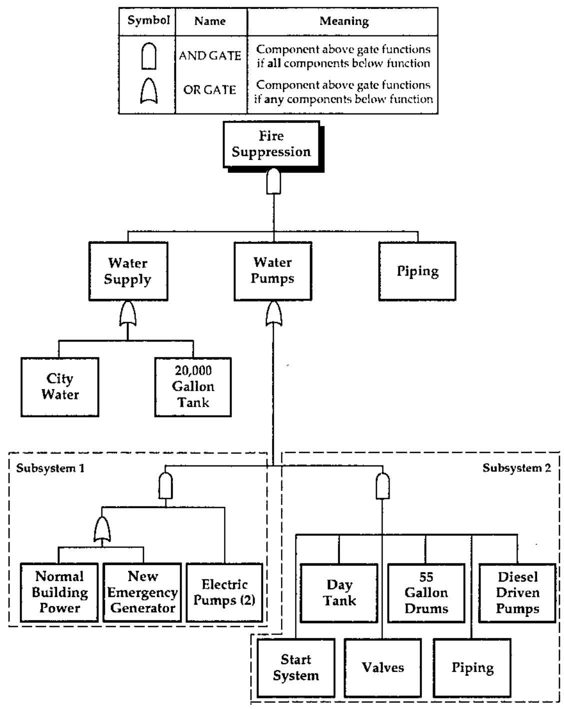

Figure 3. Logic tree for San Francisco high-rise fire suppression system

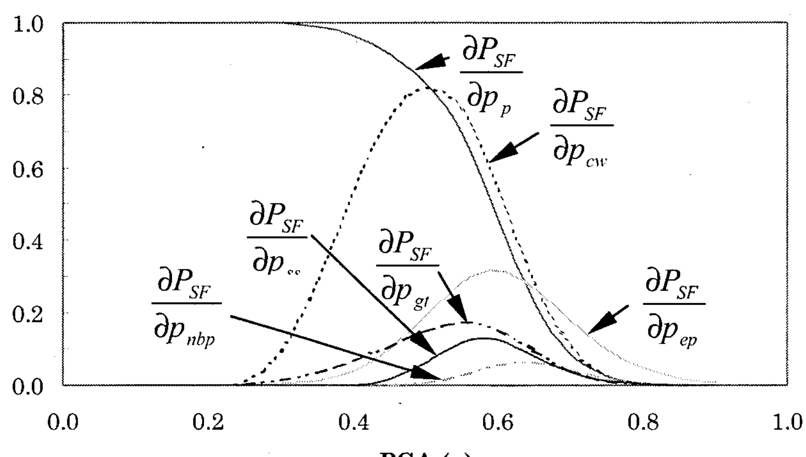

Figure 4. Sensitivity of failure probability of San Francisco fire suppression system

Current work on dynamic analysis of piping systems

Current MCEER work focuses on the modeling, dynamic analysis, testing and retrofit of piping systems in hospitals. The flowchart in Figure 5 shows the approach to a typical problem, and the division of tasks among MCEER researchers. Suitable models for the supporting structure and ground motion have been provided by others. The preliminary work at Cornell has concentrated on the dynamic analyses of piping systems using elementary failure criteria. Input is needed from other researchers on the identification of critical components, and their realistic local fragility curves or failure criteria. Experiments may be needed to determine these failure criteria. Input from others is also needed to identify suitable upgrading and retrofit strategies. Future work at Cornell will demonstrate the use of fragility, sensitivity and cost-benefit analysis in the selection of optimum designs and retrofit strategies.

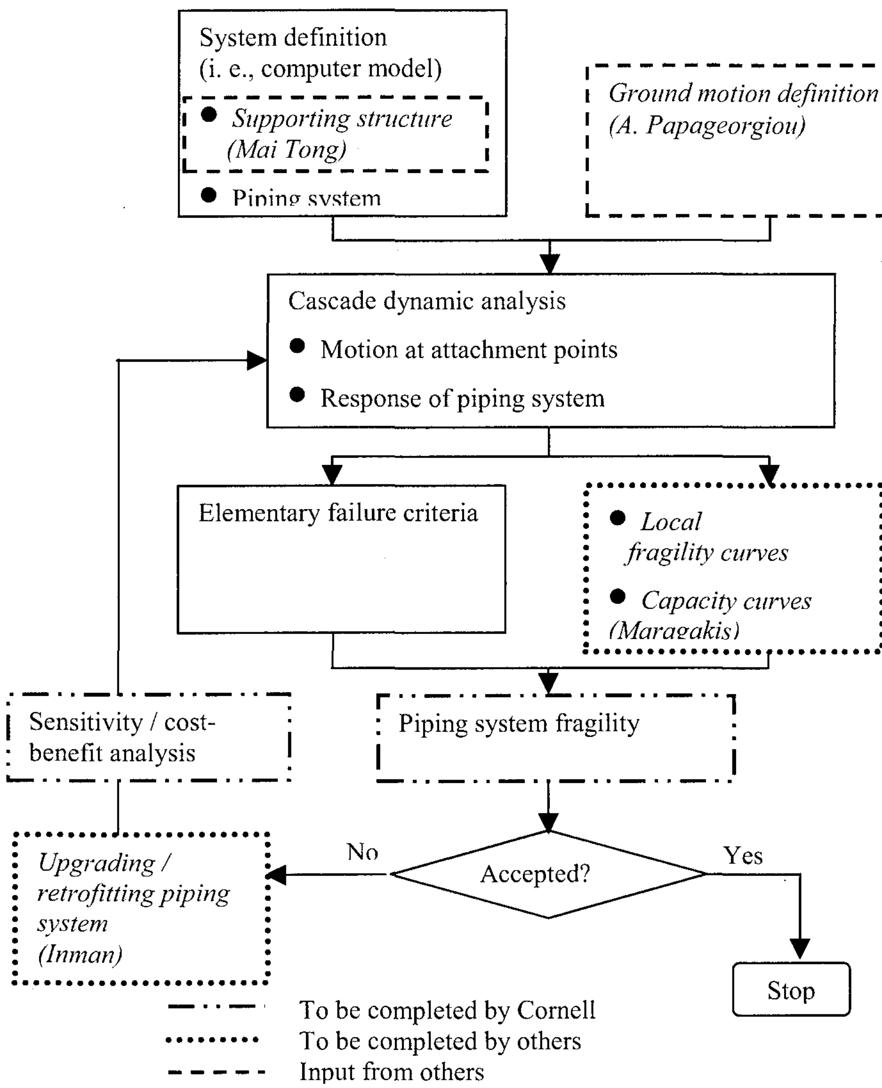

Figure 5. Approach to dynamic analysis of piping systems

Two programs are proposed for the dynamic analyses: (1) DIANA, a finite element analysis program which can be used for shell, beam or hybrid model analyses; and (2) a MATLAB program which is used for less detailed analyses using only beam and spring elements.

Existing finite element preprocessors do not allow the convenient generation of finite element mesh for analysis of complex piping systems. A Piping System Mesh Generator (PSMG) program has been prepared for this purpose. The user inputs the coordinates of the pipe centerlines and details of the mesh geometry, and the PSMG program generates a data file for use with DIANA. PSMG can handle tees and elbows, and automatically generates additional elements and tyings to ensure continuity at beam to shell connections. An example of mesh generated using the program's graphical user interface is shown in Figure 6.

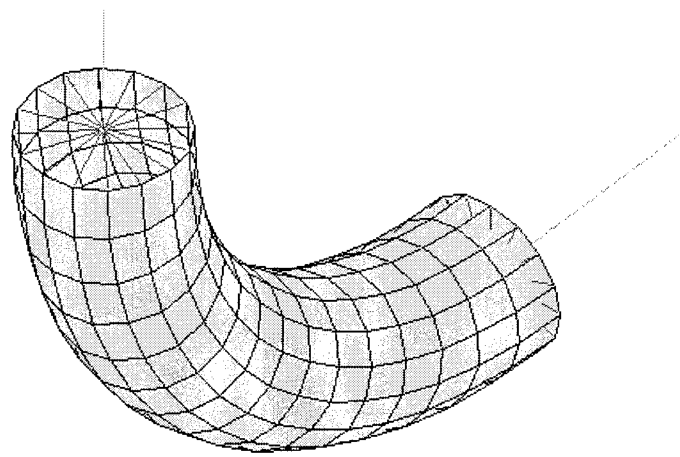

Figure 6. Finite element mesh from PSMG

The MATLAB program is a flexible program that can be used for dynamic displacement and sensitivity analysis of piping systems using beam and hinge elements. A brief numerical example illustrates the use of the program. Figure 7 shows the MATLAB model of the fire suppression system of one of the hospitals analyzed at the University at Buffalo. Motions of the attachment points between the primary structure and the secondary piping system were determined from a dynamic analysis of the structure subjected to an artificial ground motion. The peak displacement of the piping system was found to occur at the highest floor, and the displacement of the critical node is shown in Figure 8. The program also gives the sensitivity of the displacement to the system parameters. The sensitivities with respect to the thicknesses of the various pipes in the system are plotted in Figure 9. This information can be used in identifying critical parameters and selecting retrofit strategies. Monte Carlo analysis was used to generate a fragility curve for the system (Figure 10). The curve shows the probability that the displacement at the critical node will exceed 1 inch as a function of the PGA applied to the primary structure. Random damping parameters and Young's modulus of the pipes were considered in the analysis.

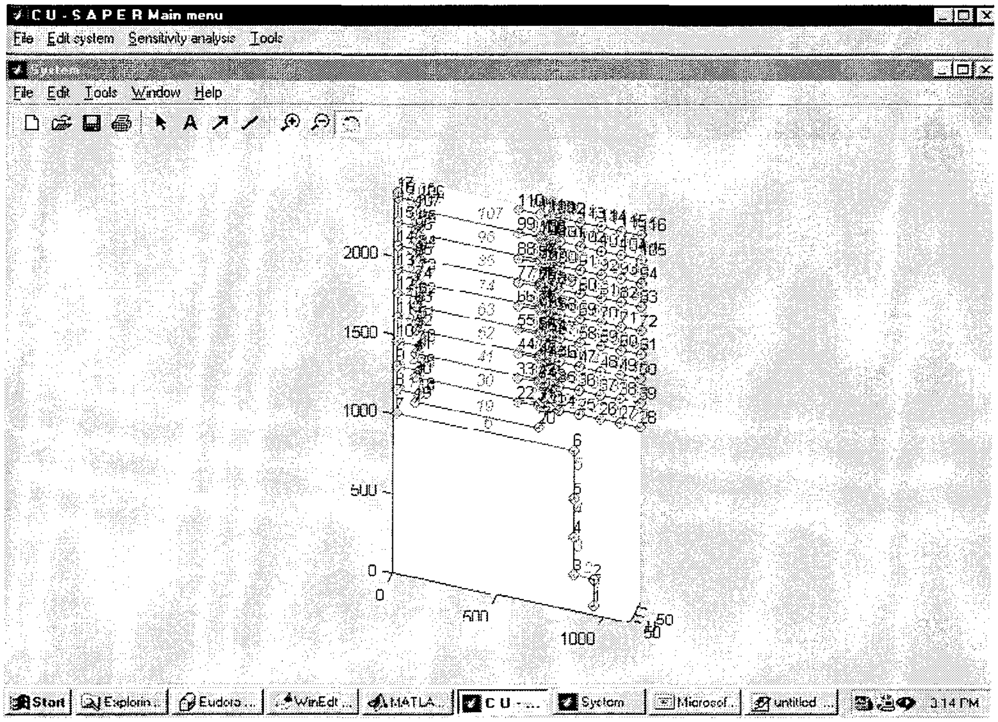

Figure 7. Fire suppression system modeled using MATLAB

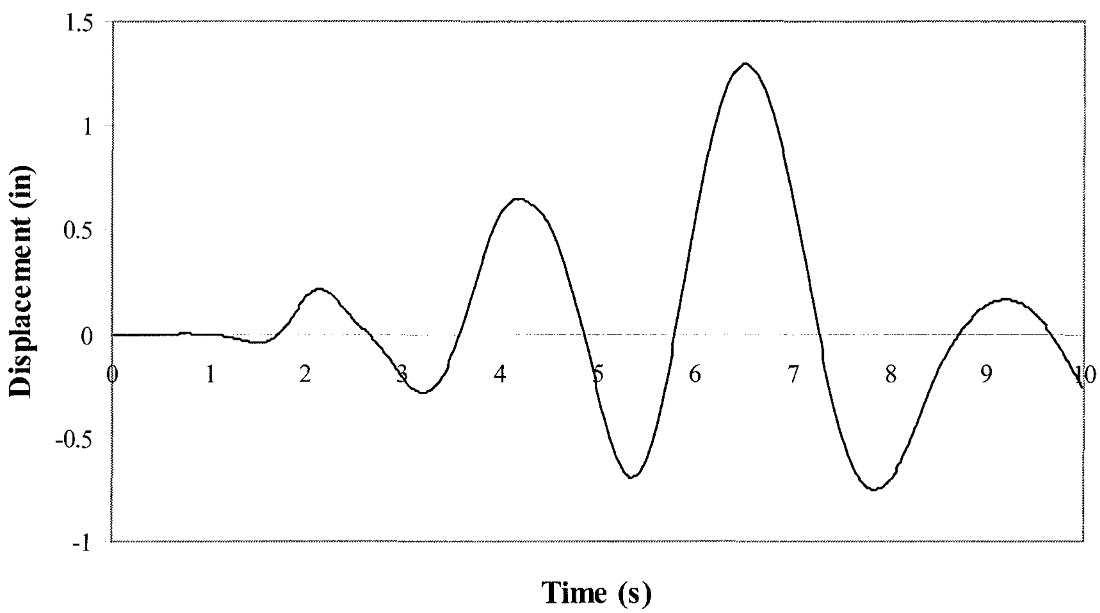

Figure 8. Displacement of critical node

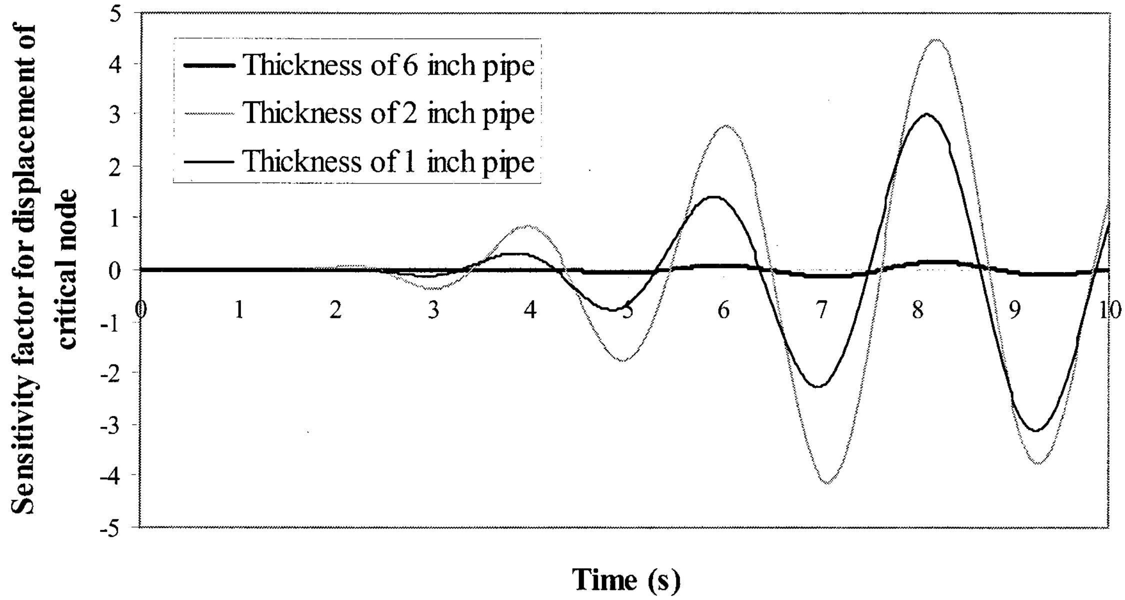

Figure 9. Sensitivity of displacement to pipe thickness

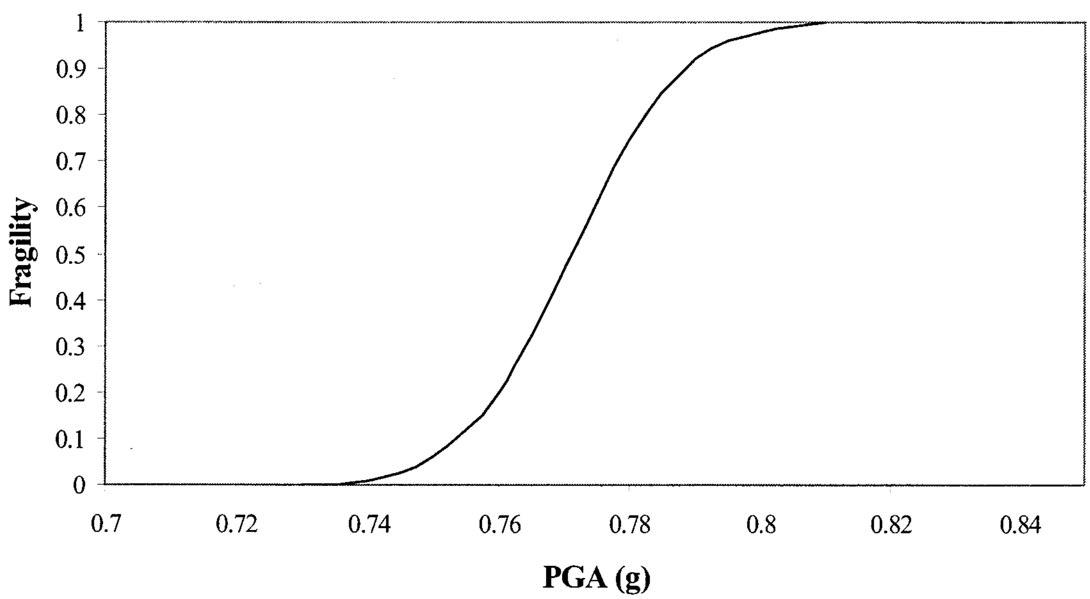

Figure 10. Fragility curve

These numerical results are only preliminary and serve to illustrate what can be done. Issues that need to be examined are local failure criteria for individual components, modeling of boundary conditions, and retrofit strategies. Input is needed from other MCEER researchers on these issues.

Conclusions

Research at MCEER has focused on different methods of developing fragility information for both structural and non-structural components and systems. The current effort for non-structural systems is focused on hospital piping systems. Preliminary dynamic analysis results have been calculated, but input is needed on local failure criteria and retrofit strategies.

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Static Axial Behavior of Some Typical Restrained and Unrestrained Pipe Joints

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Abstract

This paper presents the results of a static testing program designed to determine the axial stiffness and strength characteristics of some typical restrained and unrestrained pipe joints. Pipelines have suffered damage and failure from past earthquakes and are vulnerable to seismic motions. A vast majority of the pipeline failures have occurred at unrestrained pipe joints due to pull-out, and therefore, the effectiveness of restrained pipe joints needs to be examined and their response to earthquake motion needs to be assessed in order to mitigate potential damage.

Introduction

Pipelines transporting water, gas, or volatile fuels are classified as part of the infrastructure "lifeline" system and are critical to the viability and safety of communities. Disruption to these lifelines can have disastrous results, either in the threat they pose in the release of natural gas and flammable fuels, or in the restriction of needed water required for domestic use or to fight fires. M. O'Rourke [1], Kitaura [2], and T. O'Rourke [3], among others have documented that pipelines have been vulnerable to damage and failure when subjected to seismic motions. Table 1 lists some of the damage that has occurred to pipelines in recent notable earthquakes. Singhal [4] experimentally showed that the resistance to pull-out of unrestrained push-on rubber gasket joints is quite low, about 2 kN in magnitude, which readily suggests that the cyclic nature of the forces induced by earthquakes is an important design concern for pipelines with unrestrained joints. The use of commercially available joint restraining devices, such as retaining snap rings, gripper gaskets, and bolted collars can greatly increase their capacity to resist pull-out and decrease the probability of joint failure.

This paper discusses a research project designed to determine the static axial force capacity and stiffness characteristics of some typical pipe joints.

Table 1: Pipeline Damage Summary (Ref: T. O'Rourke [3])

1989 Loma Prieta Earthquake	
San Francisco, Oakland, Berkeley	350 repairs to water lines
Santa Cruz	240 repairs to water lines
Overall area	>1000 repairs to gas lines
1994 Northridge	
Los Angeles area	1400 repairs to water lines 107 repairs to gas lines
1995 Kobe Earthquake	
Kobe City	1610 repairs to water lines 5190 repairs to gas lines

Types of Joints and Joint Restraints

Several different types of joints, both restrained and unrestrained, were tested. The restraining devices that were tested are commercially available and can be used on unrestrained push-on type pipe joints of various sizes. Other joints, such as welded steel joints and polyethylene (PE) butt-fused joints, have pull-out restraint capabilities inherent in the joint fabrication. The types and description of the pipe joints tested as a part of this testing program are listed below.

1) *cast iron pipe with bell and spigot joint*
2) *ductile iron pipe (DIP) with push-on rubber gasket joints*
3) *ductile iron pipe (DIP) with gripper gasket joints:* metal wedge teeth are embedded in the rubber gasket during fabrication of the gasket, engaging and preventing the spigot end from withdrawing from the bell end.
4) *ductile iron pipe (DIP) with retaining snap-ring joints*: a retaining snap ring is inserted in a groove in the bell end and bears against a ring welded on the spigot end, preventing pull-out.
5) *ductile iron pipe (DIP) with bolted collar joints:* a collar with wedge screws that dig into the pipe surface is bolted to a similar collar on the opposite side of the joint, preventing the joint from pulling apart.
6) *lap-welded steel pipe*: a spigot end is inserted and lap-welded to an enlarged bell end.
7) *PVC pipe with push-on rubber gasket joints*
8) *butt-fused polyethylene (PE) pipe:* a joint connection is made by "fusing" the ends of two pipe ends.

Testing Configuration and Procedure

Initial testing was conducted on unrestrained ductile iron pipe with push-on joints using a 500k SATEC testing machine. Load and displacement data was monitored and recorded, and was used to develop load-displacement plots and ultimate force capacities..

The remaining specimens were tested using a MTS 450k hydraulic actuator. A self-contained steel loading frame (Fig. 1) was designed and fabricated that allows a hydraulic actuator to apply axial compression and/or tension load to a test specimen without the use of external reaction walls. The loading and the anchoring setup were designed to readily accept various diameters of pipe specimens and to assemble them within a reasonable amount of time. The instrumentation of this test set-up and specimen consisted of circumferentially placed strain gages attached to the pipe barrel at each end and on the bell, a load cell to monitor the applied force level and a LVDT to monitor displacements both of which are internal to the hydraulic actuator, a Novatechnic LVDT placed between the end mounting plates of the specimen, and a water pressure transducer to monitor the internal water pressure. Internal water pressure of approximately 20 to 28 KPa (3-4 psi) was applied to the specimen and monitored to determine if and when a significant loss of water pressure (leakage) occurred during the loading process.

The MTS actuator applied an axial load under displacement control to the test specimens. For joints that have no restraint against tensile pull-out, the tests were in compression only. For joints where tension restraint devices were used, only tensile loads were applied by incrementally increasing the tensile load. For joints that have both tension and compression restraint capacity inherent in their fabrication, i.e. steel lap-welded joints and PE butt-fused joints, incrementally increasing cyclic tension and compression loading was applied.

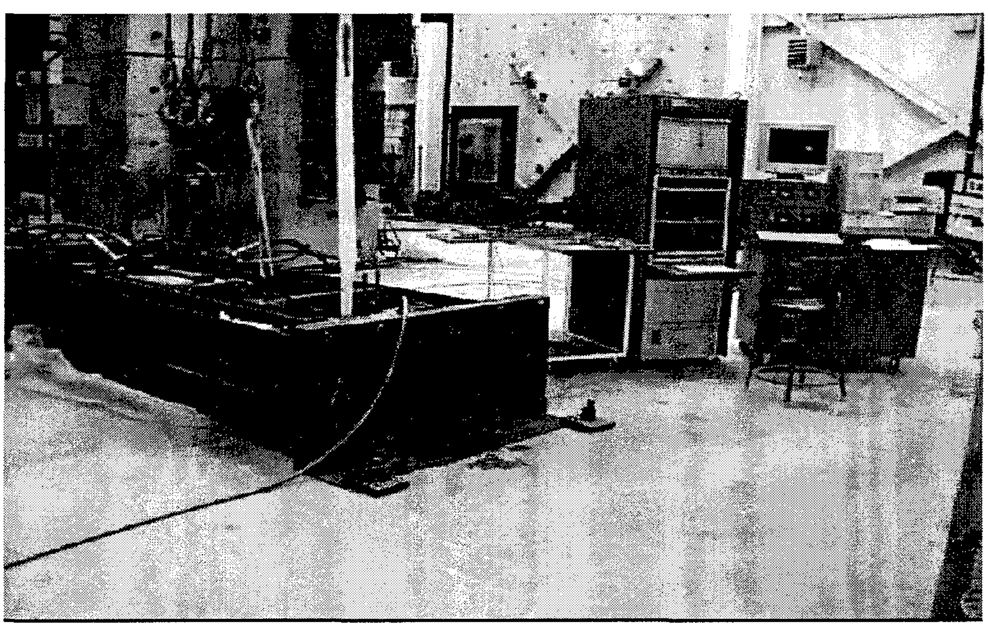

Figure 1: Load Frame and Actuator Configuration

The data recorded during testing using the MTS actuator consisted of:

1) The load-displacement behavior of the joint assembly.
2) The strains on the pipe barrel and bell surface.
3) The internal water pressure monitored during the test.

Typically, at some level of loading, noticeable fracture or buckling occurred, indicating pipe damage and probable failure. Failure was determined to be at a point when the joint strength dropped-off dramatically, and/or when significant water leakage was observed.

Test Results

In all, twenty-two tests on pipe joints were conducted in this testing program with diameters varying between 100mm to 300mm. Axial loads, both tension and/or compression, depending on the joint type, were applied under incremental displacement control. The description of the tests and failure mechanisms are summarized below:

1) A cast iron pipe that was salvaged from past service, 200mm in diameter, was tested in compression only. The compression strength was extremely high and failure did not occur during the test.
2) Ductile iron pipe with push-on rubber gasket joints with diameters of 100mm, 150mm, 200mm, and 250mm were tested in compression only using the SATEC testing machine. Failure occurred due to buckling and fracture of the spigot as it was extruded into the bell end.
3) Ductile iron pipe with gripper gasket joints were tested for diameters of 150mm, 200mm, and 300mm. The specimens were loaded in tension only. The failure mechanism observed consisted of the failure and dislodgement of the metal teeth that were embedded in the rubber gasket.
4) Ductile iron pipe with retaining snap-ring joints were tested for diameters of 150mm, 200mm, and 300mm. These specimens were also loaded in tension only. The failure mechanism observed was the cracking of the bell and failure of the end flange.
5) Ductile iron pipe with bolted collar joints and pipe diameters of 150mm and 200mm were tested for tension only. The predominant failure mechanism was the fracture of the collar at the wedge screw hole.
6) Steel pipe with lap-welded bell and spigot joints for diameters of 100mm, 150mm, 200mm, and 250mm were tested in bi-directional loading. The failure mechanism was the severe buckling at the bell followed by eventual fracture near the joint weld.
7) PVC with push-on rubber gasket joints and diameters of 150mm, 200mm, and 300mm were tested in compression only. The spigot end was extruded into the bell end, but failure did not occur.
8) Polyethylene pipe (PE) with butt-fused joints for diameters of 150mm and 200mm were tested. The applied load was bi-directional. The specimens remained ductile throughout the test even though severe buckling and distortion occurred. Ultimate failure occurred due to fracture of the specimen end flange.

Table 2 lists the material, diameter, joint type, and the maximum force capacity of the tested specimens. The maximum force capacity, F_{max}, is the maximum force level that was achieved during the test and is the maximum of either the force level at yield or at failure. For bi-directional loading, the maximum force capacity listed in Table 2 is the lesser of the F_{max} values from the tension and compression directions.

Table 2: Test Matrix and Test Results Summary

Material	Diameter	Joint Type	F_{max} (kN)
cast iron	200mm (8")	bell-spigot push-on gasket	2046
ductile iron	100mm (4") 150mm (6") 200mm (8") 250mm (10")	bell-spigot push-on gasket	792 1054 1112 1557
ductile iron	150mm (6") 200mm (8") 300mm (12")	bell-spigot, gripper gasket	253 539 488
ductile iron	150mm (6") 200mm (8") 300mm (12")	bell-spigot, retaining ring joint	200 795 750
ductile iron	150mm (6") 200mm (8")	bell-spigot, bolted collar	195 280
steel	100mm (4") 150mm (6") 200mm (8") 250mm (10")	bell-spigot, lap welded	522 491 401 546
PVC	150mm (6") 200mm (8") 300mm (12")	bell-spigot push-on gasket	13 13 13
PE polyethylene	150mm (6") 200mm (8")	butt-fused joint	157 232

Figures 2 and 3 show the plots of load-displacement data for two different types of joints as examples of typical load-displacement data collected in the tests. Plots for the other pipe joints were similarly developed. The "raw" load-displacement data recorded from testing for the specimens in these figures resulted in a hysteretic type curve in the case of the steel pipe, and a load-partial unload type curve in the case of the ductile iron pipe with a bolted collar restraint. The near peak values of the loops were connected to create a "backbone" curves as shown in Figs. 4 and 5. The "backbone" plots of the other pipe joint specimens were also similarly developed. Most specimens typically exhibited a load-displacement curve with a distinguishable elastic curve, a yield point, a post-yield curve, and a failure point. However, some specimens, for example, the 300mm ductile iron pipe with a gripper gasket and the 200mm and 300mm ductile iron pipe with a retaining ring joint, failed at their yield load level without any post-yield behavior.

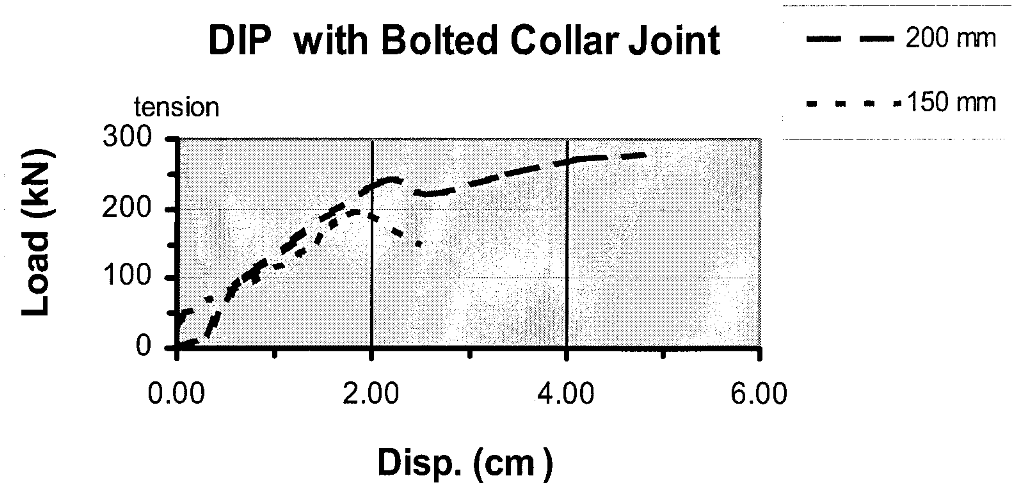

Figure 3: Load-Displacement for Ductile Iron Pipe with Bolted Collar Joints for 150mm (6") and 200mm (8") diameters

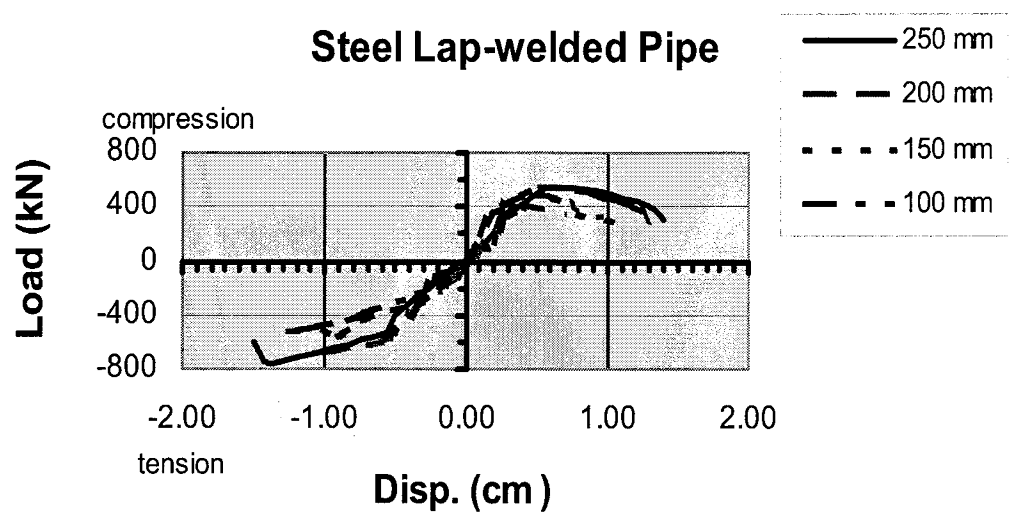

Figure 4: Load-Displacement for Steel Pipe with Lap-Welded Joints for 100mm (4"), 150mm (6"), 200mm (8") and 250mm (10") diameters

Table 3 lists the results of the testing in terms of yield force level and corresponding displacement, elastic stiffness, failure force level and corresponding displacement, and the post-yield stiffness values for each specimen. The pipe joint elastic stiffness and post-yield stiffness were determined by approximating the "backbone" load-displacement curve with a bi-linear curve. In some cases, the post-yield stiffness is a negative value indicating a degradation prior to failure. The symbol in the table, "---" , indicates that the specimen failed at its yield point, without any post-yield behavior. The two stiffnesses, elastic and post-yield, were then calculated directly by determining the slope of the elastic and post-yield portions of the assumed bi-linear curve.

Table 3: Joint Stiffness Values and Failure Force Capacity for Restrained Joints

1	2	3	4	5	6	7	8	9
Pipe Material Joint type	**Pipe Diameter (mm)**	**Test Load Direction**	**Yield Force (kN)**	**Yield Disp. (mm)**	**Elastic Stiffness (kN/mm)**	**Failure Force (kN)**	**Failure Disp. (mm)**	**Post-yield Stiffness (kN/mm)**
DIP bell-spigot gripper gasket	150 (6")	tension	253	16.4	15.4	78	59.0	-4.1
	200 (8")	tension	539	23.1	23.3	300	46.0	-10.4
	300 (12")	tension	488	18.7	26.1	---	---	---
DIP bell-spigot retaining ring	150 (6")	tension	200	1.8	111.1	50	8.0	-24.2
	200 (8")	tension	795	27.0	29.4	---	---	---
	300 (12")	tension	750	15.5	48.4	---	---	---
DIP bell-spigot bolted collar	150 (6")	tension	195	19.0	10.3	150	25.0	-7.5
	200 (8")	tension	220	20.1	10.9	280	49.4	2.0
Steel bell-spigot lap-welded	100 (4")	tension	342	5.3	64.7	522	12.7	24.3
		compression	535	5.2	102.9	309	12.9	-29.4
	150 (6")	tension	400	5.0	80.0	554	9.4	35.0
		compression	491	5.7	86.1	243	11.8	-40.7
	200 (8")	tension	316	2.4	131.7	711	10.0	52.0
		compression	401	4.0	100.3	350	6.8	-18.2
	250 (10")	tension	343	5.2	66.0	761	13.8	48.6
		compression	546	6.0	91.3	400	12.0	-24.3
PE butt-fused	150 (6")	tension	133	15.0	8.9	157	62.0	0.5
		compression	186	39.0	4.8	125	55.0	-3.8
	200 (8")	tension	125	8.7	14.4	232	43.0	3.1
		compression	307	39.0	7.9	250	60.0	-2.7

Experimental Observations and Conclusions

This testing program established the axial stiffness characteristics and maximum force capacity levels due to tensile and/or compression static loading for several different types of pipe joints of different sizes. By examining the results shown in Table 3 and the load-displacement curves for different types of restraint devices, the following general observations can be made:

- It has been established that unrestrained joints have a very low capacity to resist tension pull-out and are, therefore, vulnerable to pull-out failure from seismic motions. Restraining devices can significantly increase the joints capacity to withstand pull-out failure and therefore, decrease the probably of joint failure.

- The maximum force capacity of steel pipe with lap-welded joints, ductile iron pipe with retaining ring, and ductile iron pipe with gripper gasket joints is significantly influenced by the pipe diameter.

- The maximum force capacity of ductile iron pipe with bolted collar joints and polyethylene pipe with butt-fused joints is not greatly affected by the pipe diameter.

- Polyethylene (PE) pipe with butt-fused joints will remain extremely ductile and can withstand severe distortions without failure.

Acknowledgments

The work presented in this paper has been funded by the Multidisciplinary Center for Earthquake Engineering Research (MCEER). This financial support is gratefully acknowledged.

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Seismic Vulnerability and Retrofit of Nonstructural Components

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Abstract

Reported in this short presentation are some completed and current work in the nonstructural component area under the partial sponsorship of MCEER. It addresses two important issues associated with seismic performance of nonstructural components: seismic vulnerability and rehabilitation strategies.

Seismic Vulnerability

In order to improve seismic performance of nonstructural components and to develop effective retrofit strategies, seismic vulnerability of these components must be first established. While many approaches can be followed, the development of fragility information for these components can be very useful. Fragility information not only quantify seismic risk for these components under specified site conditions, but also provide information on the effects of key parameters on the fragility results, leading to the development of effective retrofit strategies.

One approach in developing fragility information for nonstructural components in a systematic way is to first group nonstructural components into the following three main categories:

- Unrestrained nonstructural components
- Restrained nonstructural components
- Nonstructural systems which consist of systems of nonstructural components

Fragility information for the first two groups can be developed analytically and experimentally under a variety of failure modes (e.g., Zhu and Soong, 1998; Chong and Soong, 2000). For example, sliding fragility of unrestrained equipment is studied in Chong and Soong (2000). Using floor response spectra to characterize excitation inputs, fragility curves such as those shown in Fig. 1 can be determined to allow a quantitative assessment of seismic risks for this class of components under sliding. Specifically, fragility curves in Fig. 1 are obtained when the coefficient of dynamic friction (μ_d) is 0.4, the threshold displacement (x_o) is 2 in, and when k varies from 0 to ½, where k is the ratio of the vertical to horizontal component of the peak floor acceleration. Figure 1 shows that a key parameter of interest in this case is k. As k increases, the sliding-related risks increase significantly, which is consistent with damage patterns of nonstructural components that have been observed in recent earthquakes. It underscores the need to consider vertical base motion in seismic vulnerability assessment of unrestrained equipment.

Another significant finding in this study is that the absolution acceleration, thus the inertia force, induced by base excitation at a threshold displacement of the component is relatively insensitive to the base motion but only sensitive to μ_d. Surface properties of the component and the base thus become important when impact with neighboring objects become an issue in seismic design.

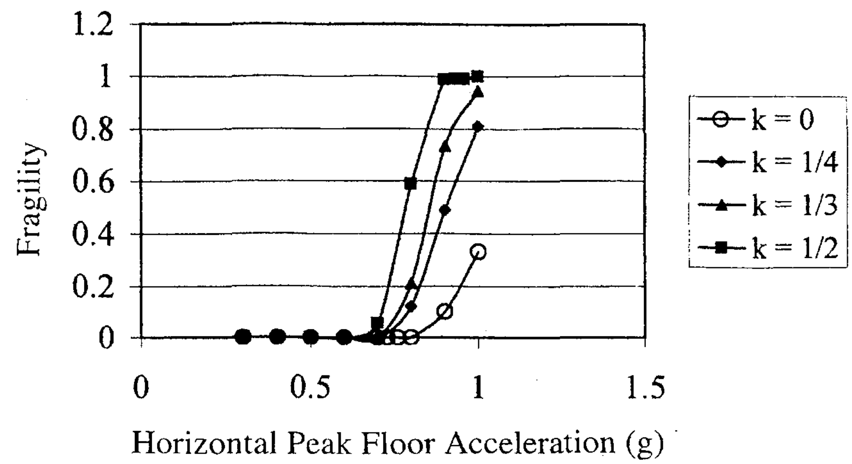

Figure 1. Fragility Curves for $\mu_d = 0.4$ and $x_o = 2$ in

Analytical studies of fragility of some of the restrained nonstructural components have been completed. Many stand-alone nonstructural components and medical equipment are anchored or restrained. Some typical restraint schemes are shown in Fig. 2. Two possible failure modes are of primary interest in this case. They are restraint breakage, which generally renders the equipment damaged and nonfunctional, and excessive acceleration levels experienced by the equipment, which causes internal damage to sensitive equipment such as computers, communication and monitoring systems, and medical instruments and equipment.

Figure 3 gives an example of fragility curves obtained when the failure mode is restraint breakage. These results are functions of four parameters: μ = coefficient of kinetic friction, T_{eq} = natural frequency of the component in the absence of friction, β = normalized initial restraint tension, and k = ratio of vertical to horizontal peak base accelerations. It can be deduced from these results that (a) fragility is significantly influenced by μ, β, and T_{eq}, (b) influence of vertical acceleration coefficient k becomes increasingly important with increasing μ regardless of β and T_{eq}, and (c) assuming that fragility curves for $k = 0.75$ are more realistic, fragility curves obtained without considering vertical ground accelerations (i.e., $k = 0$) are always unconservative.

For the third category in which a nonstructural system consists of many connected individual components, fragility information can be obtained from those of individual components through the construction of a logic tree as illustrated by Fig. 4 for a medical gas supply system (Yao, 1999). Depending on the way in which the components are connected, fragility equations can be established which relate system fragility to the component fragilities. The logic tree approach can also be used for sensitivity analysis to identify critical components of the system, and to determine confidence intervals for the system fragility.

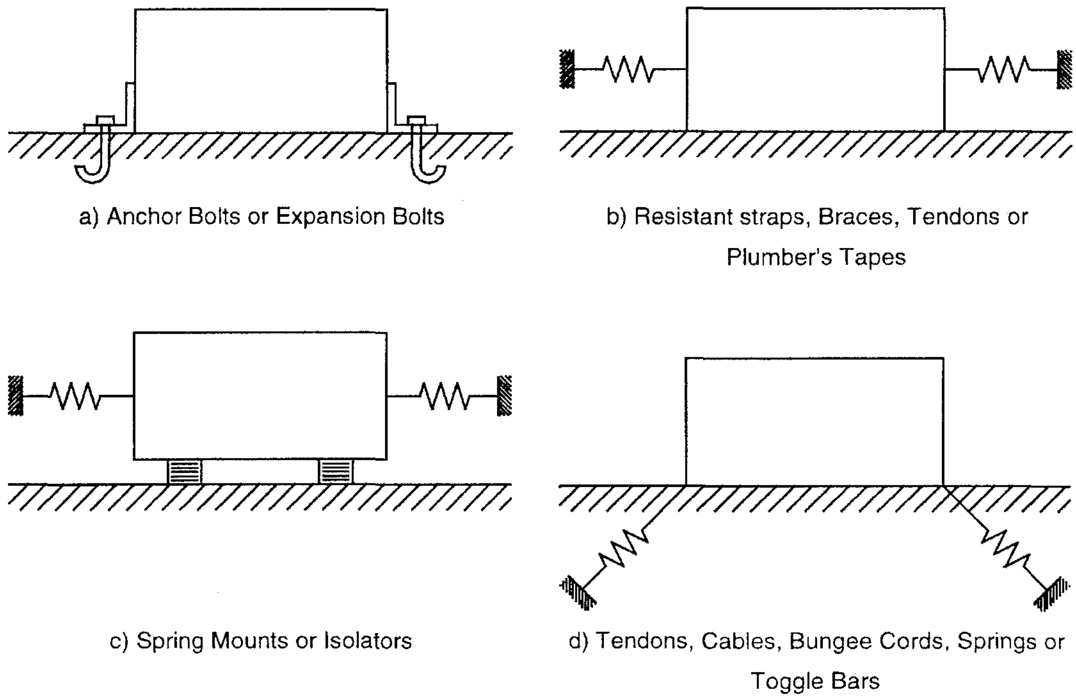

Figure 2. Equipment with Restraints

Retrofit Strategies

Results on seismic vulnerability can guide the development of improved design and installation guidelines for nonstructural components in critical facilities. In a majority of cases, easy and inexpensive solutions can be found which can significantly reduce the risk of seismic damage to nonstructural components (FEMA, 1994). For example, restraint design for computers and data processing equipment at a data center was recently carried out (Meyer *et al.*, 1998). In this work, an attempt was made to provide a sound basis for designing tethers or cables using site-specific response spectra. The important design parameters were initial angle of cable orientation, initial tension in the cable, and stiffness coefficients of the cables. A preliminary design guide was developed and, in general, the following were noted as important considerations:

- The relative displacement is dramatically increased by steep cable angles, by increases in pre-tension, by reducing the stiffness, and by increasing the equipment weight.
- The optimum angle between the floor slab and the cable can be determined. Increasing the angle causes the equipment acceleration and cable tension to increase significantly compared to decreasing the angle. Accordingly, flattening the cable angle to avoid an obstacle is preferable to steepening the angle.
- Increasing initial tension results in a near equal increase in peak cable tension.

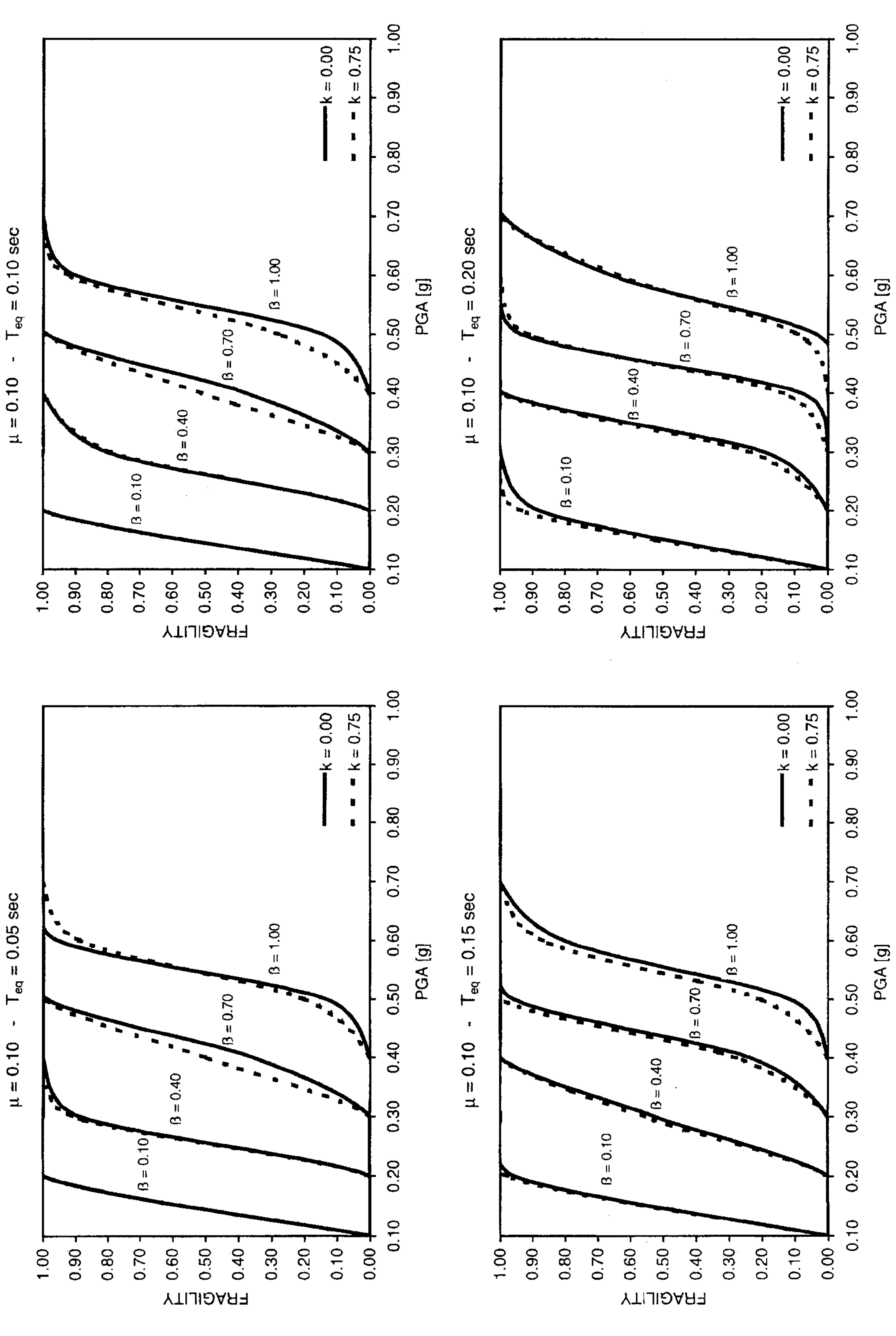

Figure 3. Fragility Curves for Restraint Breakage Limit State

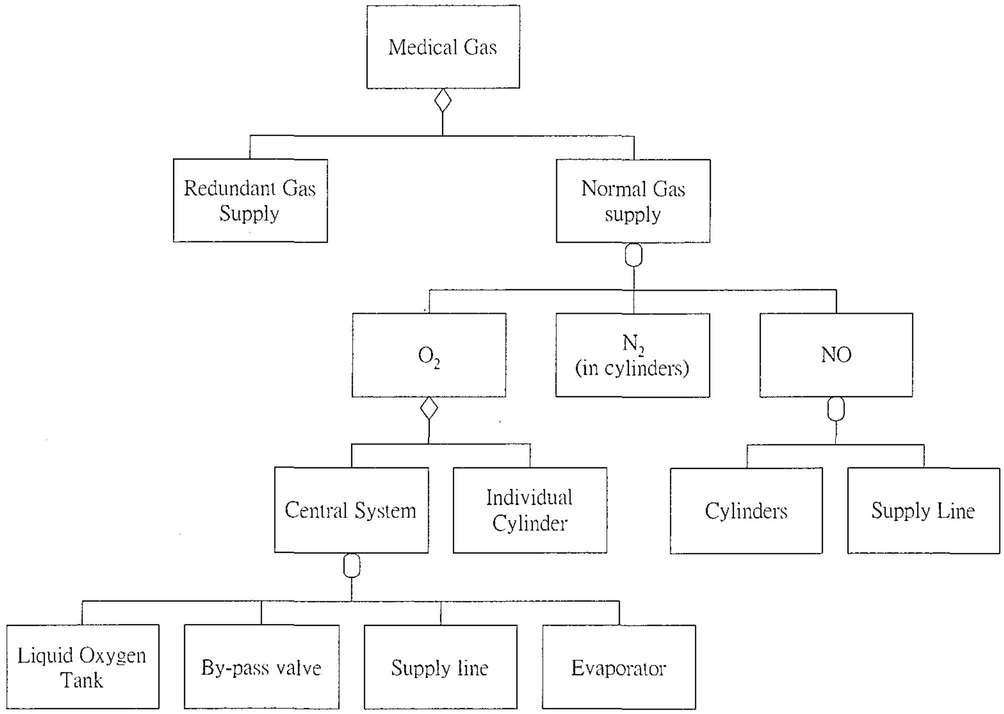

Figure 4. Logic-tree Diagram for a Medical Gas Supply Equipment

- Doubling friction at the equipment-floor interface causes a 60% increase in acceleration, but has minor effect on displacement and cable tension. Low friction is advantageous for protection of the equipment. This implies that unlocked casters are sometimes helpful.
- Reducing cable stiffness significantly increases displacements, but not accelerations or cable tension.
- Increasing the tributary weight to a cable significantly increases displacements. Accelerations decrease slightly, and cable tension increases modestly.

More complicated nonstructural components may require more advanced retrofit techniques. For example, in the case of rotating machines, it presents a dual isolation problem consisting of isolation of housing structures from the machine vibrations and protection of machines during an earthquake to maintain their functionality. The desirable characteristics of machine mounts for the above two purposes can differ significantly due to the difference in the nature of the excitation and in the performance criteria in the two situations. For example, work is continuing on the development of a semi-active mount which can accommodate different seismic and operational requirements. A functional diagram with a variable damping element for this scheme is shown in Fig. 5. This scheme includes a sensor which can detect the start of a seismic event and send ON/OFF signal to a switch in a variable damper and/or spring element which can change the property of the element.

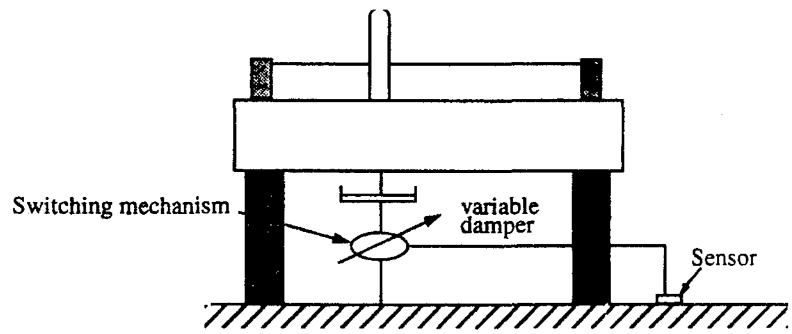

Figure 5. A Semi-active Mount Design

Concluding Remarks

Nonstructural damage to critical facilities caused by past earthquakes underscores the importance of addressing the nonstructural issue in seismic design and installation. There is an urgent need to develop stringent seismic design and installation guidelines to insure not only structural integrity, but also functionality of critical facilities, which require protecting nonstructural components, as well as structures, from seismic damage under strong ground shaking. A systematic development of these guidelines involves the following:

- Review and improve current design and installation practices in nonstructural components.
- Develop effective retrofit strategies for nonstructural components in existing critical facilities.
- Develop effective implementational procedures for existing facilities and new construction.

For nonstructural components in critical facilities, higher performance levels demand a more rigorous approach to assessing their seismic vulnerability and to developing appropriate retrofit strategies. This paper has outlined some of these approaches that can be followed in a systematic development of seismic vulnerability methodologies and retrofit strategies for nonstructural components.

Acknowledgments

This work was supported in part by the Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York. This support is gratefully acknowledged.

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Seismic Protection of Some Nonstructural Components in Hospitals

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Abstract

The paper describes the approaches that can be used to protect of the elevators and some critical pieces of equipment in a hospital. For equipment, a bi-directional isolators can be used. Based on the design criterion used, the isolators period can be defined in terms of the periods of the supporting structure and equipment. During a strong seismic event, elevators are usually damaged by the vibrations of the counterweights and cabs. The supplemental damping, tuned mass dampers, friction interfaces, and semi-active devices have strong potential for application for mitigating seismic induced vibrations in the counterweights and cabs.

Introduction

The hospitals are among the most essential facilities that must function properly during and after an earthquake event, as their services may be needed during this period. The hospitals like any other important service facility contain several nonstructural components that are susceptible to damage by earthquake induced ground motions. The components that are most vulnerable to earthquake induced ground motions are the: (1) emergency medical equipment and data storage components, (2) vertical transportation systems such as elevators, (3) fire protection and water supply systems, (4) power supply and electrical systems, (5) heating, air conditioning, and ventilation systems, and (6) telecommunication systems. If any of these systems are damaged, the hospital services can be seriously affected or even completely stopped. The failure of Olive View Hospital to provide service during and immediately after the 1994 Northridge earthquake, caused by the damage of some of these nonstructural components, is a classical example of the importance of these components. The main structural systems of this facility remained intact during this event; however, it was the failure of the nonstructural components that closed the hospital services. It is necessary that these components be protected or designed properly to avoid such damages. The authors have recently examined the seismic response characteristics of equipment pieces and elevator systems. The focus of this presentation is on seismic protection of these nonstructural components.

Isolated Equipment

The items that fall in this category are the pieces of medical equipment and the computers and other data storage devices that perform a very important function in a hospital. One way to secure equipment is to properly anchor them to the floor on which they are placed. In such a case, it is then necessary to design the attachment such that it can withstand the dynamic forces. Also, the equipment itself should be able to withstand the inertial effect that it is likely to experience at its location in the hospital, including the amplification caused by the building or caused by its own flexibility. Another option is to isolate the equipment from the motion at its base. The isolation scheme should be designed to reduce the equipment acceleration to a level it can withstand.

The equipment can be of different mass and may be placed at different locations in a structure. The isolation system should be customizable to accommodate these different characteristics. The primary goal of an isolation system is to reduce the acceleration of the equipment. For the purposes of reducing acceleration, the best isolation is provided by roller supports on a flat surface. Since such systems do not have any restoring mechanism, they have an obvious disadvantage of being left with an uncontrolled residual displacement of the object after an earthquake occurrence. In isolation systems, the restoring mechanism can be provided by a concave rolling surface or by a simple spring on a flat rolling surface. These restoring devices, however, introduce a period of oscillation in the system. This period of oscillation must not coincide with the predominant period of the motion at the base of the equipment. The device should also be such that the relative motion on the curved rolling surface or the deformation of the restoring spring is within the acceptable range. That is, these relative motions can be accommodated by the device for it to be effective for isolation.

There could be four different situation for which one may need to determine the characteristics of the isolation systems: (1) rigid equipment on ground, (2) flexible equipment on ground, (2) rigid equipment on a flexible structure, (3) rigid equipment in a flexible structure, and (4) flexible equipment in a flexible structure. The isolation surface could be a curved surface, or a flat isolation surface with a linear spring. The isolation system with curved surface is a nonlinear system, especially for a surface with a small radius of curvature. However, for the isolation desired to reduce the acceleration the required radius of curvature is usually large enough to render the system linear. Thus, for such cases, the two systems – one with a curved rolling surface and another with a linear restoring spring are equivalent.

An analytical study was performed to learn about the response characteristic of these system when exposed to different earthquake induced ground motions. A parametric study was carried out to examine the influence on the isolation effectiveness of the isolation system period (determined by the radius of curvature of the curved rolling surface, or the coefficient of the spring on the flat rolling surface), natural period of flexible structure, period of the internal component of the equipment if any. Fifteen recorded earthquake motion time histories were used in the numerical study. Such a study can be used to develop design guidelines for the isolation system. For example, for the design criterion that the maximum acceleration of the equipment be less than the maximum acceleration of the ground motion, the period of the isolation system should be as follows for the four cases of equipment support.

Case 1: Rigid equipment on the ground

$T_e > 1.3$ sec.

Case 2: Rigid equipment on a flexible structure

$T_e < 1.1T_s - 1.35$, or $T_e > 0.91T_s - 1.23$

Case 3: Flexible equipment on the ground

$T_e > 0.75T_m + 1.13$ for $T_m < 1.36$
$T_e > 2.15$ for $1.36 < T_m < 2.35$
$T_e > 1.35$ otherwise

Case 4: Flexible equipment on a flexible structure

$T_e > 2T_s + 2$ for $T_s - 0.3 < T_m < 1.3T_s$
$T_e > 0.86T_s + 1.75$ for $1.3 < T_m < 0.9T_s + 2$
$T_e > 0.88T_s + 1.2$ otherwise

Similar guidelines can be developed for other equipment acceleration limits. There is a design trade-off. Larger acceleration reductions will larger relative displacement accommodations at the base of the equipment, and vice versa. Such isolation devices should be bi-directional to isolate in any two orthogonal directions. That is, the isolation mechanism in the two orthogonal directions should be able to function independently.

Elevator Systems

The elevators have suffered damages in all past earthquakes (Suarez and Singh, 2000). In the 1994 Northridge Earthquake, the elevators in several hospitals were damaged (Finley, et al.,1996). The damaged components were the guide rails, bracket supports, roller guides, equipment anchorage, counterweight frame, cab stabilizers, emergency power, rope guards, stabilizer bents, controller boards, hoistway entrance and walls, snagged ropes and travelling cables, and hydraulic cylinders. In the elevators, the counterweight being the heaviest component is most likely to cause damage to its guiding system. The damage usually occurs in the guide rails, bracket supports, and roller guides or guide shoes. The damage is due to the inertial forces caused by the dynamics of the systems.

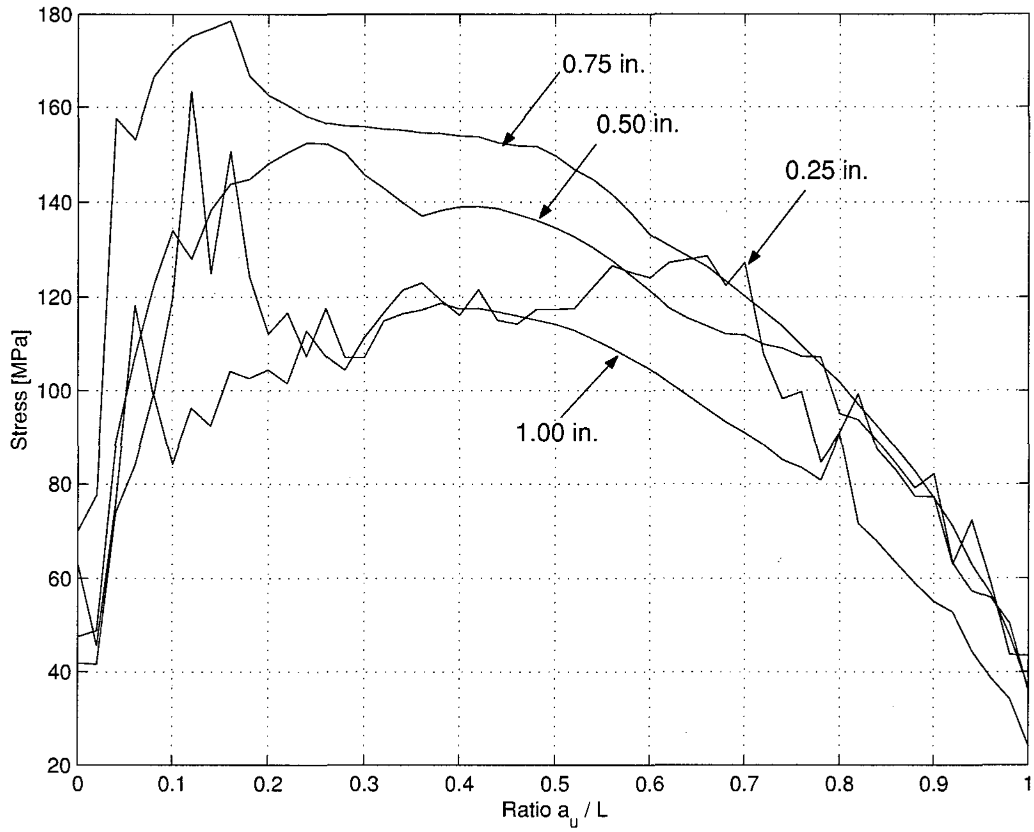

Figure 1: Maximum Stress in Rails for Different Counterweight Positions and Four different Saturation Limits Showing the Effect of Roller Guide Stiffness Nonlinearity

The dynamics of the counterweight-rail system is strongly affected by the flexibilities of the guide rails and the roller guide system. Especially, the nonlinear stiffness characteristics of the roller guide polymeric tires is one of the very important factors that affect the dynamic response of the rail-counterweight and cab-guide rail systems. The roller guides have bilinear stiffness characteristics. The stiffness of the system changes at the saturation limit, which is the maximum deformation of the system before it becomes very rigid. Figure 2, taken from Singh et al. (2000) shows the effect of changing the saturation limit on the maximum stress in the guide rail of an elevator counterweight. The maximum stress in a 18.5 lb guide rail are plotted against different positions of the counter weight in the upper two stories of a ten story building. The zero value for the location parameter a_u / L implies that the top roller guides are on the top floor and the value of 1 means that it is on the lower floor. The variation of the maximum stress is shown for the four different values of the saturation limit. The saturation limit of 1.00 and larger corresponds to the elastic behavior of the assembly. It is noted that stress is increased by the nonlinearity in the roller guide springs. Also, the stress is affected is a complex manner by the saturation limit as there is no trend in the variation of the stress for the increasing or decreasing values of the saturation limit. Similar effects are also observed for the stresses in the brackets. The stress also depends upon the location of the counterweight along the building height. Although the floor acceleration may be highest near the top of a building, the stresses in the rails and the bracket may not be high there. These stress are affected in a complex way by the dynamics of the building structure vis-a-vis the dynamic characteristics of the counterweight and rail system. They, of course, also depend upon the intensity and the frequency characteristics of the input motion. The analytical study (Singh et al., 2000) clear shows that for the level of ground motion intensity experienced in the Northridge and Loma Prieta earthquakes, even the largest rail size could be overstressed.

Protective Systems for Elevator Counterweights

Since the rails, brackets, and roller guides of the counterweight and the cab are among the most vulnerable components, the protective system must focus on reducing the dynamic forces on these components. Other components such as anchors for the traction motor, governors, control panel, etc. can be more easily designed for the inertial effects imposed on them. The following protective systems, currently being examined for their effectiveness, seem quite feasible for the elevator counterweights.

(1) Viscous Dampers in Roller Guides

A counter weight or the cab is guided up and down on two parallel rails by two pairs of the roller guide assemblies. A roller guide assembly consists of at least three rollers with polymeric tires, which are kept in constant contact with the guide rail by preloaded springs. Roller guides with six rollers are also available in the market. Figure 2 shows the schematics of a roller guide assembly with three rollers. The geometry of such an assembly around the spring can be reconfigured to accommodate a small damper in parallel with each spring. This supplemental damping can be used to reduce the response significantly. For the out-of-plane response of the counterweight, this approach is the only convenient way of passively controlling the response. A study is being conducted to quantitatively assess the effectiveness such a passive protective scheme. The optimal size of dampers required to achieve a given level of response reduction can also be calculated. (Singh and Moreschi, 2001)

(2) Tuned Mass Damper

The counterweight-rail system has five degrees of freedom. Two of them are associated with the in-plane response and three with the out-of-plane response. Since the guide rails have only one flange, the in-plane response generally causes higher stresses in the web leading to overstress. The fundamental frequency of the system for the in-plane response does not change much as the counterweight traverses the height of the building. Therefore, for such a system whose response is dominantly affected by a single mode, the use of the tuned mass dampers seems attractive. Such a damper can be placed on the top of the counterweight frame, or on the top deadweight block of the counterweight, as shown in Figure 3. The damper mass can move in the plane of the counterweight perpendicular to the rails. A study is being conducted to obtain the optimum parameters of such a tuned mass damper. Unfortunately, such a system can not be used for the out-of-plane vibration of the counterweight because of the space limitation on the counterweight.

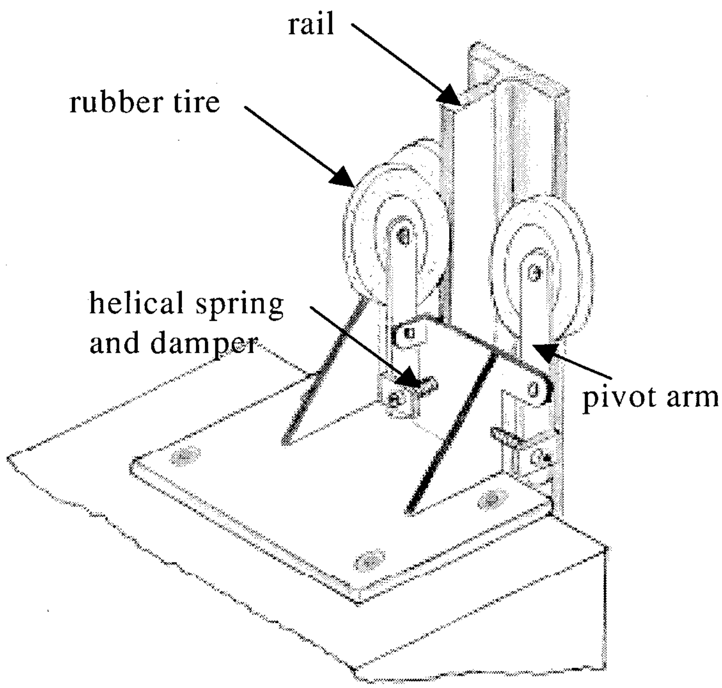

Figure 2: Roller Guide Assembly with Helical Spring and Proposed Dampers

(3) Friction Damping

The dead weight of a counterweight consists of iron blocks that are stacked over each other in the side channels of the counterweight frame. These blocks are usually tied by a rod that prevents its motion sideways. If such motion is unchecked, the mass can collide and bend the frame sides. However, with a proper design of sliding interfaces between the blocks and a flexible elastomer padding between the frame and the sliding blocks, a significant amount of vibration energy can be dissipated to reduce the system response without causing any impact. These sliding interfaces are shown in the schematics in Figure 3. An analytical study is being conducted to examine the effectiveness of this idea for the passive control of the counterweight response. The optimum value of the friction coefficient between the blocks will also be determined to achieve a desired performance objective (Singh and Moreschi, 2000).

(4) Semi-active and Active Control

The effectiveness of damping can be enhanced by using semi-active schemes with devices such as magnetorheological dampers (Spencer, et al., 1997). The properties of the magnetorheological dampers can be easily changed according to an appropriate algorithm to enhance the system performance. Like the viscous damper, these dampers will also be placed in parallel with the springs in the roller guides. The power required for such active control of the magnetorheological devices is, of course, very small and can be provided with simple batteries. The effectiveness of this system will also be examined in an analytical study.

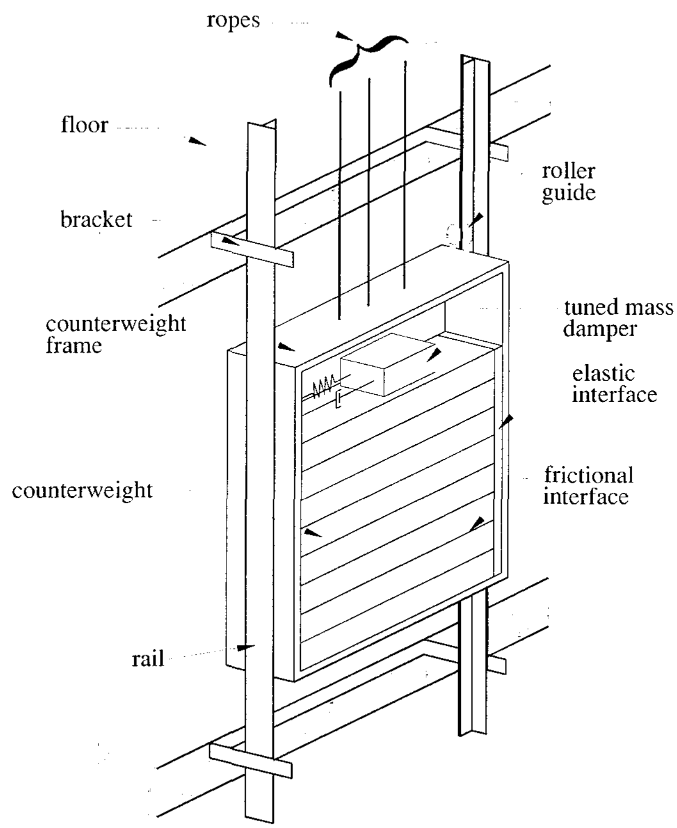

Figure 3: A Counterweight Schematics Showing a Tuned Mass Damper and Frictional Interfaces

One could also activate the tuned mass damper mentioned above according to an algorithm to enhance the performance of the tuned mass dampers. It is not quite clear at this stage about the power requirement for such an active control scheme. However, if the power requirement is not large, this active control also offers an alternative for reducing the system response.

Summary and Concluding Remarks

A hospital has several nonstructural systems and components that can get damaged during an earthquake and thus effect its performance. The paper deals with the seismic protection of medical and other equipment pieces and elevator systems. To protect equipment pieces, one can either anchor them and design the anchor to withstand the forces, or isolate them from the motion at their base and design the stiffness characteristics of the isolation systems. A bi-directional rolling isolation system with linear springs is considered. The design involves determining the stiffness of the isolation spring for a given design criterion. As an example of a design, the paper defines the period of the isolation system to limit the equipment acceleration to the maximum earthquake acceleration. Similar designs can be established for other design criteria.

The paper also describes the seismic vulnerability of the elevator systems. The roller guides in the counterweight and cab are known to affect the dynamic characteristics of the system significantly. Several protective schemes that have potential to improve the performance of the counterweights are being currently examined. They include, supplemental damping in the roller guides, tuned mass dampers, use of frictional interfaces, and active regulation of damping devices.

Acknowledgements

The research on the topics described in this paper is being supported by Multidisciplinary Center for Earthquake Engineering Research (MCEER) at Buffalo through a grant form National Science Foundation. This support is gratefully acknowledged.

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Part I – Seismic Retrofit of Critical Piping Systems (Above Ground Piping)

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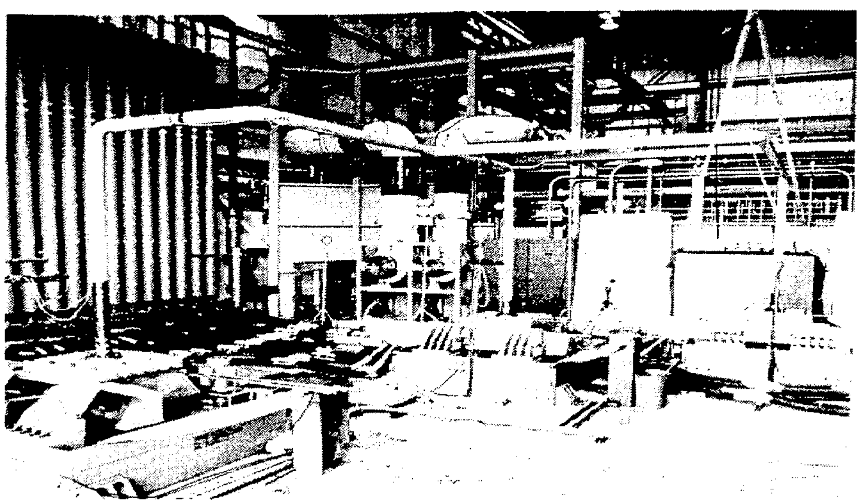
Can we predict the behavior of piping systems in earthquakes?

Yes, to a great extent. Our understanding of the seismic behavior of piping systems comes from four sources:

(1) What we have learned from actual earthquakes.
(2) What we have learned from static, fatigue and shake table tests.
(3) What we have learned by analysis.
(4) What we know of the behavior of piping systems in normal service.



Shake Table Testing of Piping Systems (NRC-EPRI-GE tests)

Do earthquakes fail piping systems?

Yes, piping systems do fail in earthquakes, and these failures are predictable. But we must clarify what we call "failure". In earthquakes, failure of piping systems can occur in three ways:

(1) Loss of Operability: The system fails to operate, in other words, it fails to deliver flow, even if the pipe does not leak. This happens for example if a pump fails to start-up, an operating compressor breaks down, or a valve fails close.
(2) Loss of Pressure Boundary: The pipe cracks or ruptures, and leaks.
(3) Loss of Support: The pipe falls off its supports, or the supports come off the wall and the pipe collapses to the floor or ground.

All three types of failures have occurred.

Post-Earthquake Investigation of the Behavior of Piping Systems

What governs the seismic performance of piping systems?

The seismic performance of piping systems depends on the soundness and quality of five key factors:

(1) Materials: Type of materials used in the system (soundness and ductility).
(2) Design: Mechanical design (wall thickness, layout and supports).
(3) Fabrication: fabrication entails three activities: joining (welding, brazing, solder, bonding), non-destructive examinations (NDE) and leak testing (hydotest, pneumatic test, sensitive leak test, in-service leak test).
(4) Maintenance: Degradation in service and maintenance (corrosion, in-service inspections and repairs).
(5) Structures: The integrity of the soil and building structures.

How can an existing piping system, that was not seismically designed originally, be made seismically rugged?

There are many ways by which a piping system can be seismically retrofitted. But there is one cost-effective approach. This approach addresses the five key attributes listed earlier: materials, design, fabrication, maintenance, and structures.

So, how would the seismic retrofit process start?

First, the owner should take the time to think through, carefully and logically, the "post-earthquake scenario" and define the critical piping systems and their required function: What are the critical systems that should function? What should be that function? What leak or rupture should be avoided? Is there a risk of flooding? Will we rely on an operator to take certain actions? Does it matter if overhead pipes fall? Will we have a normal power supply or will we rely on emergency power, and for how long? Will there be spurious signals starting or stopping pumps or compressors? etc.

The objective of this first effort will be to produce two documents:

(1) First, the owner will prepare an "initial piping and instrumentation diagrams" (P&ID) clearly showing the mains, branch lines and boundaries of piping, instruments and equipment to be seismically retrofitted. Boundaries should be isolation points, beyond which a pipe rupture is of no consequence. It is not uncommon, at this stage, to have to add isolation or check valves to protect the critical piping from failures in non-essential piping systems.
(2) Second, the owner will prepare an "initial piping and equipment list" (P&EL). This list will identify each critical piping system, and the required function: operability (if required to operate during and after the earthquake), pressure boundary (if required to remain leak tight, but not necessarily to function) and

support integrity (to prevent the pipe from falling). If operability is required, then the emergency power and distribution system may also need to be included.

This first step is essential. It defines the scope and therefore the cost of the seismic retrofit effort. It is also the foundation of the seismic safety logic for the facility.

With the scope clearly defined, what follows?

First comes a scoping walk-down. The facility/maintenance engineer and the piping/seismic engineer will inspect the equipment and piping in scope, and will determine the logistics (permits, ladders, scaffolds, lights, etc.) required to access and inspect the system. They also confirm the scope and finalize the "P&ID" (piping and instrumentation diagram) and the "P&EL" (piping and equipment list).

The scoping walk-down is followed by data gathering. The piping/seismic engineer will assemble any data, drawings, vendor reports related to the piping and equipment materials, original fabrication and inspections, leak testing, supports and anchorage details.

At this stage, the seismic input is defined. It could be as simple as a peak seismic acceleration from the governing building code, or as complex as site-specific, in-structure, three-dimensional response spectra.

At the same time, the facility/maintenance engineer will compile maintenance histories, with emphasis on pipe leaks, repairs and equipment performance.

With the information gathered, the piping/seismic engineer is ready for the seismic walk-down, which is followed by an evaluation – and sometimes an analysis – and conclusions and recommendations for upgrades.

In summary, the process is as follows:

☑ Initial P&ID (owner)
☑ Initial P&EL (owner)
☑ Scoping walk-down (facility/maintenance engineer and piping/seismic engineer)
☑ Data Gathering (facility/maintenance engineer and piping/seismic engineer)
☑ Seismic walk-down (piping/seismic engineer)
☑ Evaluation (piping/seismic engineer)
☑ Recommendations (facility/maintenance engineer and piping/seismic engineer)

What is a seismic walk-down?

A seismic walk-down is a detailed inspection of the piping system, looking for and recording a series of attributes important to the seismic adequacy of the system. The piping/seismic engineer uses a checklist, makes notes, records dimensions and takes photographs to document this effort.

If the data gathered earlier is incomplete because design, construction or maintenance records are not retrievable, the piping/seismic engineer may need to take ultrasonic readings of pipe wall thickness (to confirm pipe size and look for corrosion) or anchor bolt lengths (to check embedment depth).

The piping/seismic engineer also checks bracing and equipment anchor bolt patterns and tightness if necessary.

The piping/seismic engineer checks the type and quality of welds, brazed or soldered joints, mechanical joints, unusual fittings or components, valve characteristics, material condition, signs of corrosion or leakage, missing or broken pipe support components, etc. Non-destructive examination may be needed for highly stressed welds of unknown quality. These consist of liquid penetrant (PT) or magnetic particles (MT) for surface examinations or ultrasonic testing (UT) or radiography (RT) for volumetric inspections.

The piping/seismic engineer evaluates overhead and adjacent commodities to determine whether they constitute a credible and significant seismic interaction. In other words, could they fall on the piping and fail it.

During the same walk-down, the piping/seismic engineer will predict where hardware fixes are likely, and will prepare conceptual sketches for these fixes.

When is the decision made regarding the adequacy of the system or the need for seismic upgrades?

Following the seismic walk-down, the piping/seismic engineer will evaluate the integrity of the piping system. This evaluation is based on rules of good practice supplemented by calculations of demands and capacities of piping and support system. In some cases a more detailed stress analysis may be required, as described in the companion paper "Seismic Design of Critical Piping Systems".

If equipment operability is needed, then the seismic attributes of equipment (such as pumps, compressors, electrical distribution, etc.) must be evaluated, and in some cases compared to published operability test data on similar equipment.

The piping/seismic engineer then identifies the required upgrades, as necessary, with the corresponding design sketches and bill of materials, to procure and install these upgrades.

What can the owner expect to receive?

The owner should receive the following documents:

(1) The finalized P&ID and the P&EL.
(2) The design, fabrication and maintenance data gathered.
(3) The seismic walk-down report, with photographs and notes.

(4) The calculations of demand, capacity and conclusions of adequacy.
(5) Recommendations for upgrades and sketches of each required upgrade, with bill of materials.
(6) A clear and concise summary report, written in plain English, explaining the objective, the scope, the conclusions and recommendations.

In practice, because the hardware will have been looked at so closely, possibly for the first time in years, maintenance fixes not related to seismic upgrades will be also pointed out.

What should the owner do?

The owner should:

(1) Plan, budget and implement the upgrades.
(2) If operator actions are required to start-up, check or reset emergency equipment in case of earthquake, the owner should make sure that these responsibilities are clearly assigned.
(3) The critical systems that have been seismically upgraded should be tagged or painted so that future maintenance modifications do not jeopardize their seismic adequacy. The facility/maintenance engineer should be responsible for the maintaining the system's condition.

Part II – Seismic Design of Critical Piping Systems (Above Ground Piping)

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Starting with a clean sheet of paper, where should the seismic design process start?

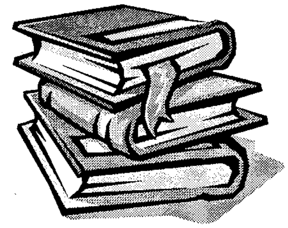
First, start the design process paying no attention to seismic loads, which means:

(1) Materials: Select the pipe or tubing material compatible with the service (fluid, environment, pressure, temperature).
(2) System Design: Size, route and slope the piping to perform the required system function. Select equipment, instruments and components as necessary.
(3) Mechanical Design: Consider the operating loads (pressure, temperature, weight, flow transients, wind if outdoor) and apply the governing code to determine wall thickness, final layout and hanger and support spacing. Specify overpressure protection (pressure relief device size and location).
(4) Fabrication: Specify the types of pipe joints, weld quality and fabrication details.
(5) Inspection: Specify the governing code for type and extent of shop and field examinations (visual, surface or volumetric).
(6) Leak Testing: Specify leak test requirements (hydrostatic, sensitive leak, soap bubble leak detection, etc.).

What is meant by "governing code"?

Governing code refers to the applicable pipe design and fabrication code. These are:

ASME B31.1 for power plant piping.
ASME B31.3 for chemical process plant piping.
ASME B31.4 for oil pipelines.
ASME B31.5 for refrigeration piping.
ASME B31.8 for gas pipelines.
ASME B31.9 for gas, steam and water building services piping.
ASME B31.11 for slurry transportation piping.
AWWA for water works piping.
NFPA-13 and -24 for fire protection mains and sprinkler systems.



Follow Codes and Standards for Design and Fabrication of Critical Systems

Federal laws require oil and gas pipelines to comply with ASME B31.4 and B31.8 respectively. Some states or local jurisdictions require compliance to other ASME B31 codes. Whether required by regulation or not, a critical piping or tubing system, one that will be called upon to operate in case of earthquake, should be designed and fabricated to a piping code.

We do not mention nuclear power plant piping (ASME Boiler and Pressure Vessel Code Section III) for which the seismic design rules, methods of analysis and acceptance criteria have been strictly specified through the plant licensing process.

At what point is seismic introduced into the design?

Once the designer has developed a "competent" design for operating loads, and specified a national standard for design and fabrication, the piping system is analyzed for seismic loads.

For that purpose, the piping designer will need the input response spectra in each direction, at 5% damping. The spectra should envelope the elevation of the highest support attachment point to the structure. The seismic spectra are applied to the elastic, linear model of the piping system, in three orthogonal directions: North-South, East-West, and vertical.

The analysis could also be based on static peak acceleration in each direction.

In either case, by dynamic or static analysis, it will become quickly evident that lateral bracing is required to resist seismic loads. Experience indicates that, having competently designed the pipe for normal loads, placing a lateral brace every four weight supports is a good starting point for seismic design and analysis.

What is the objective of a seismic analysis of the piping system?

After layout and bracing the piping based on the designer's experience, the system is seismically analyzed to verify the following:

(1) The longitudinal stresses in the pipe do not exceed the following values:

$$PD/(4t) + 0.75i\, M_W/Z < S$$
$$PD/(4t) + 0.75i\, (M_W + M_S)/Z < 2S_y$$
$$iM_T/Z < f(1.25S_C + 0.25S_h)$$

where

P = internal pressure, psi
D = outer pipe diameter, in
t = pipe wall thickness, in
i = stress intensification factor
M_W = resultant moment due to weight, in-lb
Z = pipe section modulus, in^3
S = applicable code stress allowable (typically the smaller of (2/3)yield or (1/3) ultimate), psi
M_S=elastically calculated seismic moment, resultant of three directions, seismic anchor motion are to be included here, unless they are added to the resultant moment range M_T, in-lb
S_Y = minimum specified material yield stress, psi
M_T = resultant moment range due to thermal expansion between a cold or ambient temperature T_C and a hot operating temperature T_h, in-lb
S_C = allowable stress at cold or ambient temperature, psi
S_h = allowable stress at hot operating temperature, psi
f = fatigue factor equals 1 for 7000 or fewer cycles, less than 1 for over 7000 cycles

(2) The support loads are within the capacities of support members, which include catalog items (such as struts or clamps), steel structures, welds, and anchor bolts.

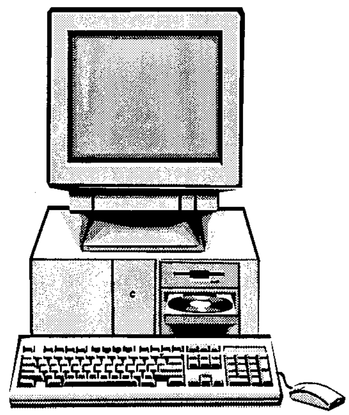
(3) The reactions on equipment and component nozzles (vessels, pumps, heat exchangers, valves, etc.) are within the manufacturer's limits.

(4) The loads or movements at mechanical joints are within the manufacturer's limits.

(5) The pipe sway will not result in impacts or interferences with adjacent equipment or structures.

So the piping system is designed by computer analysis?

It is actually designed by experience and confirmed by analysis. With today's software this can be done very quickly. But there are some real pitfalls if one relies solely on a "design by analysis". It can even happen that analysts chose a design solution because it is easier to analyze, what we may call "design for analysis". It is essential that analyses be used to support experienced judgment, not to replace it.



Stress Analysis is an Efficient Method for Seismic Design of Piping Systems ... when Tempered with Experience.

What are some of the pitfalls of design by analysis?

A few words of advice when seismically designing piping systems by analysis

(1) First, keep the model simple. For design, it should be linear and elastic.
(2) The analysis should be by the response spectra method or by static load factors.
(3) Seismic supports should be modeled simply, as linear restraints with nominal, rounded, stiffness.
(4) The piping response to the three directional input should be combined by the square root of the sum of the squares (SRSS).
(5) In dynamic response spectra analysis, the modes should also be combined by SRSS.
(6) The rigid range mass, the mass of those modes that did not participate in the dynamic range, should also be combined to the dynamic response by SRSS.
(7) Only analyze credible failure modes. For example, formulas do exist for the analysis of stresses in lugs and at pipe-support contact. But an experienced engineer knows that these stresses are, in most cases, not credible failure modes in earthquakes.
(8) Techniques such as multi-input spectra should be avoided in favor of the simpler classical approach of using a spectral envelope.

But the greatest practical difficulty with analysis is to figure out what to do with it once the design is completed and the piping installed. Will someone save the documentation and rerun the analysis every time a change is made to the piping? If a valve is replaced with a new, heavier valve, will the analysis be rerun? Will the analysis keep up with maintenance modifications?

Is there a need for more testing and research, to improve our methods of piping seismic design?

Look at the large variety of piping materials and joints:

(a) Above ground and underground.
(b) Metallic and non-metallic.
(c) Metallic include: ferrous (cast or ductile iron, carbon or alloy steels) and non-ferrous (aluminum and nickel alloys, copper, etc.).
(d) Non-metallic include: plastics (PVC, CPVC, polyethylene, etc.), concrete, fiber reinforced plastics, glass, etc.
(e) Pressure service (above 15 psig), vacuum service or gravity flow (for example water supply or drainage).
(f) Welded joints (arc welded, soldered, brazed and bonded) or mechanically joined pipe (threaded, specialty expanded or flared joints, specialty bolted couplings, ANSI flanges, etc.).

Seismic design of above ground metallic piping systems is a mature engineering discipline. Our knowledge in this field and the analytical tools are quite good.

Some more fatigue or shake table data for plastic and fiber reinforced pipes and joints would be useful, and should be conducted.

Generally, manufacturers of specialty pipe joints focus on their pressure rating, with little attention to bending strength. If these joints are used in critical, seismically designed systems, the fitting manufacturers should develop stress intensification factors for their joints, and some did.

Overall, today we have the knowledge, the experience and the analytical tools to efficiently design above ground piping systems to safely withstand the effects of earthquakes.

FIELD GUIDE

SEISMIC RETROFIT OF CRITICAL PIPING AND TUBING SYSTEMS

(with examples, figures and photographs)

PROJECT SPECIFICATION
Specification for retrofit project.
Owner input.
Contractor deliverables.
Field inspection and engineering evaluation plan.

SCOPE
Scope definition and system diagram.
System boundaries, isolation and interfaces.
System functions: operability, integrity, interactions.

CODES
Codes and standards: ASME, CGA, NFPA, AWWA, ASHRAE, etc.

MATERIALS
Material types and their recognition in the field.
Material properties for seismic retrofit.
Unknown materials.
Recognizing standard fittings.
Pipe, fittings and component markings.
Construction quality.
Pipe and tubing sizes, weights and properties.
Standard components sizes and weights.

CONDITION
Material condition.
Visual inspection checklist.
Non-destructive examination methods and field application.
Assessment of corrosion severity.
Assessing maintenance history.
Inspection for missing parts on pipe components.
Inspection for damage, scratches, gouges, distortion, etc.
Judging weld quality.
Assessment of flanges and gaskets.

JOINTS
Welded and brazed joints.
Soldered joints.
Threaded joints.
Flange joints.
Tubing joints: flared, expanded, mechanical.
Mechanical joints.
Bonded joints.
Nozzles and terminations.

BRACING
Types of pipe supports and pipe braces.
Pipe and tubing span tables.
Pipe support checklist.
Pipe bracing checklist.
Calculation of seismic demand: simplified method.
Calculation of seismic demand: advanced method.
Capacity of standard support components.
Capacity of steel support members.
Measuring weld size and calculating capacity.
Inspection of anchor bolts: recognizing types and installation.
Capacity of anchor bolts.

EQUIPMENT
Equipment integrity and operability.
Power supply.
Pipe – equipment nozzle.
Pump inspection checklist.
Compressor inspection checklist.
Compressed gas bottles inspection checklist.
Manual, motor and pneumatic valves checklist.
Pressure vessels evaluation.
Storage tanks evaluation.

ANCHOR MOTIONS
Checklist for adverse anchor motions.
Quantify adverse anchor motions.
Equipment anchorage.
Differential motion of support attachments.
Assess large motion of header against a stiff branch.
Assess differential soil settlement.

STRESS
Assessment of high stress points.
Detailed stress analysis.
Eccentric weights.

INTERACTIONS
Seismic interactions.
Zone of influence.
Recognizing credible sources of interactions.
Recognizing significant sources of interactions.

DOCUMENTATION
Documentation and submittals.
Configuration management.

RETROFIT
Alternatives for retrofit.
Design of retrofit.

REFERENCES

Recycled Plastics: Characteristics and Seismic Applications

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Abstract

As a result of research efforts over the past decade by academia, government and industry, recycled plastics have evolved as a viable construction material. In addition to the environmental benefit, the mechanical properties of the material make it more suitable for certain applications than traditional materials. Flexibility, high strength, and excellent durability are among its characteristics that can be employed to improve existing designs or develop new ones. Recently, there have been many structural and non-structural applications of recycled plastics. This paper discusses the possible use of the material for seismic rehabilitation and upgrading of existing facilities. Creep is a major design issue with thermoplastics such as recycled plastics. However, seismic loads act only for a short period of time, thus, it appears that recycled plastics are even more suitable for seismic applications. As an example, the use of recycled plastic lumber to increase ductility/stability of unreinforced masonry (URM) walls is discussed and the results of a pilot experimental investigation are presented. It is concluded that the material has merit and deserves further investigation for use in mitigation of earthquake hazards.

Introduction

Solid waste is overloading the landfills and is a major contributor to the environmental problems. Plastics are among the fastest growing segment of solid waste. Recycling of plastics is an environmentally acceptable mean to remedy this problem and it has gained widespread support over the past decade. Consequently, there have been significant developments in the recycling technology of commingled plastic waste. Parallel to technological development in manufacturing of the products, there have been many sustained research efforts to better understand the mechanical characteristics of recycled plastics, to generate performance data, and to develop ASTM standards and test methods. The latter effort is conducted through Section D20.20.01 on Plastic Lumber and Shapes as a part of ASTM's Committee D-20 on Plastics.

This paper discusses recycled plastics and their possible use for mitigation of earthquake hazards primarily because of its inherent properties, which are suitable for design and rehab of seismic resistance elements. Over the past several years the material has gained widespread applications in other segments of the construction industry such as its use as a replacement to treated wood lumber (e.g., residential decking, board walk, marine structures). Thus, development of high-value end uses for the material as an effort to close the recycling loop would be only a beneficial byproduct and not the primary objective of this paper.

In the following sections first the mechanical behavior of the material along with its application in development of a noise wall system are discussed. These are followed by presentation of possible seismic applications, including its use for rehabilitation of unreinforced masonry (URM) walls to increase their stability as a life safety measure. Results of an internal pilot test program to qualitatively assess the proposed rehab approach are also described.

Recycled Plastics Characteristics

Mixed or commingled plastics (thermoplastics such as PET and HDPE), once destined for the landfills, are granulated, melted, and processed in an extruder. The molten plastic is then forced through a die in the shape and size of the final product. Recycled plastic products can be cut and shaped with the same tools and fastening devices as used for wood lumber. These thermoplastics are resistance to attacks from gas, oil, salt, sunlight, chemicals, and insects. They will withstand human and mechanical abuse. Stress-strain behavior for the material is highly nonlinear and it behaves differently in tension and compression [1]. In general, the material possesses good strength in both tension and compression, which is comparable to or exceeds that of wood. Due to inherent characteristics of plastics the material is very ductile, making it suitable for applications that involve load reversals and impact. A typical stress-strain diagram in tension and compression is shown in Figure 1. It can be seen that the material is highly nonlinear and that there are significant differences in tensile and compressive behaviors. The material is stronger in compression than in tension. However, it is initially stiffer in tension. The initial modulus of elasticity for the material in tension can be as high as 600 ksi, and it is only 125 ksi in compression. Note that in compression beyond 20-30% strain the materials exhibit hyperelastic behavior. That is, there were no points of maximum stress or rupture in compression. But rather the stress continued to increase after the material visually failed. This was also characterized by a softening (lower modulus of elasticity) followed by a stiffening of the material, thus, depicting an inflection point in compression. Such a behavioral characteristic can have great seismic application since it can allow for development of rehabilitation schemes that can increase the resilience of a hazardous system, thus, providing life safety. The ultimate tensile strength of recycled plastics can be as high as 3,000 psi. The compressive strength at inflection point can be more than 6,000 psi.

Freeze/thaw test results have shown that recycled plastics with no additives are not affected by exposure to freeze/ thaw cycles [1]. On the other hand, creep deflection can be significant and the material should not be used for applications that involve sustained loads. Thus, recycled plastics appear to be a viable construction material that may have certain structural applications. Successful structural applications will take advantage of the favorable characteristics of the material such as ductility, durability, resistance to moisture and other environmental elements, and lightweight.

Among in-field applications of the material are residential decking, marine structures, walkways, fences, tables, park benches, boardwalks, and a 25-ft long bridge that can support light vehicles. In two studies supported by the New Jersey Department of Transportation the use of recycled plastics for highway appurtenances was investigated at NJIT. The results of these studies are reported elsewhere [2-4]. In the following section the use of the material in development of a noise wall system is briefly discussed.

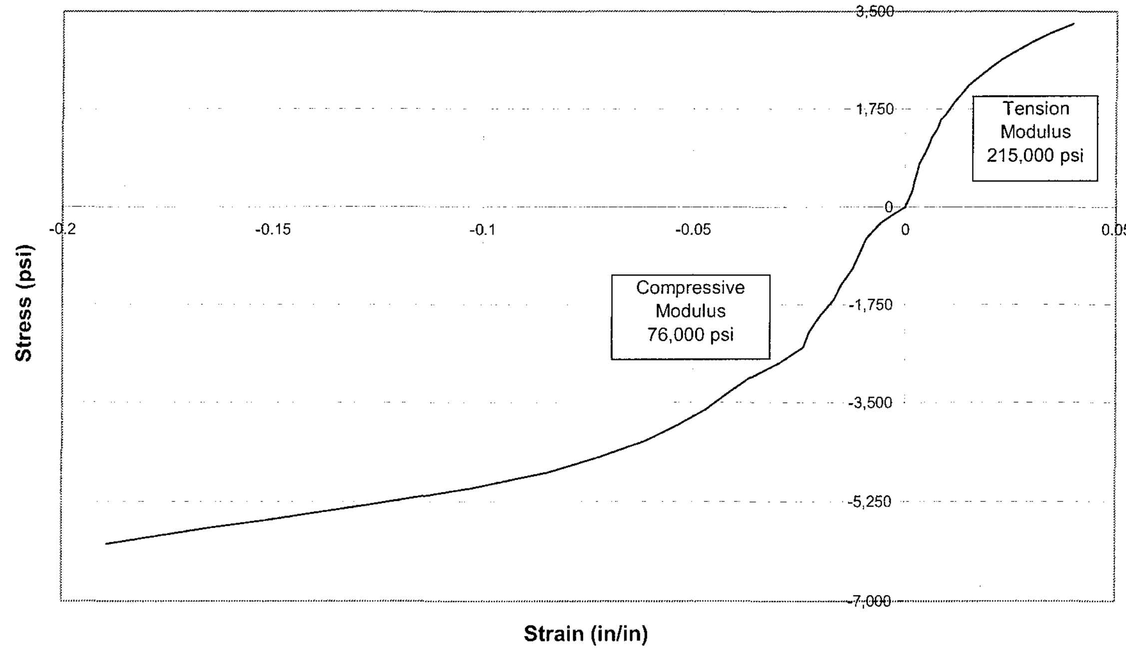

Figure 1 Stress-strain diagram for a recycled plastic material

Noise Wall Application

As it was mentioned before the stiffness of recycled plastics is generally low – much smaller than concrete or even wood. However, an advantage of recycled plastics is that it can easily be manufactured into various shapes and the cross section does not have to be solid. Therefore, a noise barrier design, which uses a modular approach, was proposed. The panels are hollow rectangular in cross section and they can consist of single or multiple cells. Dimensions and geometry, including location of webs, were established using finite element method [2], and it was determined that the panels have adequate stiffness and strength to withstand design wind load. Due to lightweight of the material and its flexibility to increase construction tolerances even longer spans than typical ones are possible. For precast reinforced concrete panels the span is limited by fabrication, transportation and construction requirements. To validate the analytical findings prototype panels were assembled from ½" thickness recycled plastics sheets by fastening them together with screws. The sheets were entirely of recycled plastics with material properties similar to that shown in Figure 1. Figure 2 shows three actual 8-ft long panels stacked on the top of each other as it is envisioned in actual design similar to existing precast concrete panels.

The prototype panels were tested for sound absorption and transmission loss. The sound transmission class of a specimen is a single number that gives an indication of the sound transmitted by fitting the test data to an ASTM defined curve. With an STC of 37 the proposed design satisfies the minimum STC of 23 for noise barrier, and it is superior to some of the exiting designs (such as 1 ¾" wood with STC of 34). The proposed design with total thickness of only 1" is more effective acoustically because of multilayered nature of the design. Layering is the only way to overcome the weight requirement for sound effectiveness. Another advantage of the hollow design is higher stiffness and stability of the system, which was demonstrated analytically and through actual wind tests. The objective of the wind tests was to determine the maximum capacity, flexural stiffness, and failure mode of the recycled plastics panels under wind load. The tests were performed on modules similar to the ones shown in Figure 2. They were made up of ½" thick sheets with length, width, and overall depth equal to 12-ft, 2-ft, and 8-inch, respectively. The wind tests can be summarized as successful. The panels were able to withstand high-pressure load, in excess of 90 psf, without any sign of damage. The mid-span deflections never exceeded 2". The tests had to be stopped due to failure of the pressure chamber seal. Note that based on AASHTO's Guide Specification, the pressure corresponding to 90 mph wind velocity is 42 psf. This is the highest wind velocity for the State of New Jersey.

In summary, the noise wall panels built using recycled plastics had structural and acoustical performance comparable to traditional designs [2, 3]. Due to the success of this project in the use of recycled plastics, it was subsequently used for development and laboratory installation of a combination glare screen pedestrian fence system [4]. That is, the system to be developed using recycled plastics had to have the strength and stiffness to satisfy the structural and geometric requirements of a dual system. Among other objectives of the design were ease of installation and maintenance. It had to be economically competitive or even superior to current designs in terms of both initial and life-cycle costs. This application was also successful so that an in-field demonstration project is under consideration.

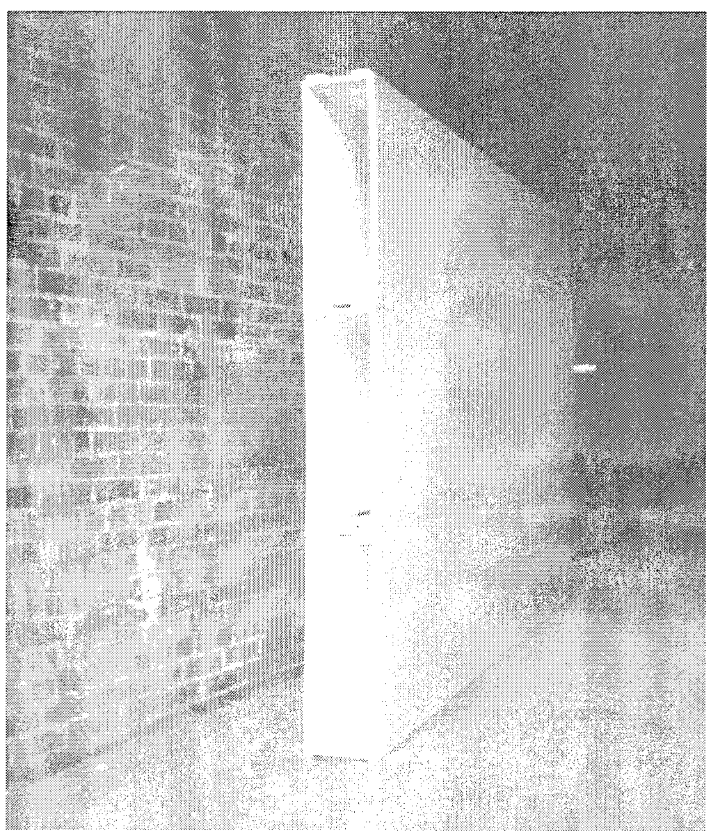

Figure 2 Prototype panels constructed of ½" thick recycled plastics sheets

Possible Seismic Applications

In light of the above discussions, applications of recycled plastics to mitigate the earthquake hazards appear to be a natural extension. Possible applications could include its use for i) rehabilitation of unreinforced masonry (URM) and adobe walls and systems, ii) anchorage of mechanical, electrical and other secondary and non-structural systems, iii) design of partition walls and infill panels, and iv) retrofitting of non-ductile reinforced concrete frames. As discussed before, an advantage of recycled plastics is that it can easily be manufactured into any shape and the cross section does not have to be solid. Another major advantage of extruded recycled plastics is that the color and texture of the finished surface can be controlled, thus, insuring aesthetically attractive rehab scheme. A pilot study was conducted to qualitatively assess the effectiveness of a rehab scheme for URM structures that uses recycled plastics. This section describes the concept and the results of the pilot tests.

It is well known that URM construction is one of the most hazardous constructions in a seismic environment. The stiff but brittle behavior of URM structures makes them susceptible to disastrous collapse even during low intensity earthquakes. Many buildings in the US, especially hospitals and schools, use this form of construction as primary or secondary elements. Failure of URM structures or other variations of this form of construction is the culprit for tens of thousands of casualties during ground motion earthquakes in developing countries. Therefore, it is a necessity to retrofit these structures to at least life safety level in order to reduce the hazard posed by URM construction. It should be mentioned that there are several retrofit methods to increase in-plane shear capacity and out-of-plane flexural capacity of URM walls [5]. It is not within the scope of this paper to discuss these methods. However, there are several problems with current rehab schemes, which rely to a large extend on increasing the strength. First, in almost all rehab techniques an increase in strength is accompanied by an increase in stiffness. The distribution of seismic forces is typically proportional to the stiffness of the structural elements, and by increasing the stiffness more seismic demand is put on the URM elements within the system. Second, most often the foundation must also be retrofitted to accommodate the high increase in the capacity of the structure. Retrofitting the foundation, due to inadequacy of the soil or practical considerations, can be very difficult (if not impossible) and costly. High cost in general and marginal or no increase in the stability/ductility of the retrofitted system (despite higher strength) associated with current approached are additional factors that contribute to the great need for alternative methods of rehabilitation and retrofitting of URM structures.

To increase the ductility/stability of URM walls without adverse effect on other dynamics response parameters, it is proposed that the wall be reinforced with two layers of recycled plastics. That is, the URM wall is sandwiched between two layers of recycled plastics profiles. Under lateral seismic load the layer under tensile stresses will reduce the rate of crack openings, thus, increasing the ductility of the system significantly. The recycled plastic layer on the compression side will prevent excessive sliding of the top portion of the wall due to opening of cracks. That is, the recycled plastic layer will prevent (or delay) push out of the wall by the lateral forces or by the action of the diaphragm. This will add to the stability of the wall preventing the collapse of the wall even when the wall sustains a high degree of damage. It is expected that the recycled plastic layers will also enhance the in-plane response of the wall. As discussed before, recycled plastics have good strength and ductility, and adequate stiffness to provide the needed stability. On the other hand, its flexibility is such that it will not adversely affect the overall system behavior to attract more seismic loads (due to higher stiffness of the retrofitted system) or to require upgrading of the foundation to accommodate higher demand.

It is envisioned that the recycled plastics layers will be connected to the wall and the diaphragm using mechanical connections such as brackets and through bolts. Manufacturing of the material can allow for a recess at the location of the bolts in order to improve aesthetics. To this end, the texture of the finished surfaced can also be controlled during extrusion to further enhance aesthetics.

Pilot Tests

To assess the effectiveness of the proposed rehab scheme an internal pilot study was conducted. The results point to the promising potential of the approach to preserve the ability of a retrofitted URM wall to sustain the gravity load after the earthquake and ensure life safety.

A URM wall retrofitted with recycled plastics planks, as proposed, is shown in Figure 3. The URM wall was built with standard bricks using type N mortar. It is 51" high, 51" wide, and 8" thick. Thus, the masonry wall was two wythes thick, and it was laid in common bond that consisted of three header course seven course apart. The masonry wall was set on the test floor in a layer of mortar and was clamped between two 6"X6" timber beams, which were compressed using two hydraulic pumps and were clamped to the test floor using two 1" thick plates bolted to the floor.

The wall was first tested with no recycled plastics reinforcement. A lateral out-of-plane load was applied at the top of the specimen. The distance between the top of the 6X6 and the location of the actuator was 46" (i.e., a cantilever wall with a length of 46"). It should be mentioned that investigation of actual out-of-plane mode of failure for a URM wall requires consideration to gravity loads to represent the membrane couple, which will develop upon cracking and balances the out-of-plane forces [6]. This will require a more elaborate experimental set up, which was not possible during this pilot study. The results here show more the interaction between the two elements and provide a qualitative assessment.

The load deflection relationship for the URM wall alone is shown in Figure 4 with dashed line. Load controlled loading was applied. As the load reached about 950 lb the wall cracked right at the top of the 6X6 beam clamping the wall and the load suddenly dropped to around 550 lb. The crack was about one foot long (1.5 brick length) and was on one end of the wall indicating a not quite uniform support condition. Since the crack was not full-width, the wall was able to carry more load, however, it was not possible at this point to operate the system in load control. Further load was applied in a displacement-controlled mode and the wall was able to carry further load until it failed at a displacement of 2.2" where the load was 1,082 lb. At this point a long crack formed one brick layer above the base beam. In a stairway fashion this long crack joined the previous crack forming a crack that included the entire width of the wall.

The damaged wall was repaired using epoxy, and it was reinforced on both sides using recycled plastics as shown in Figure 3. The recycled plastic layers were fixed at the base between the wall and the 6X6 wooden beam. At the top, the plastics lumbers were clamped using another pair of timber beams that were bolted together. The recycled plastics planks have a rectangular cross-section with nominal thickness of 1.5". Only a fraction of the thickness is shell material [1]. Later material tests indicated that the tensile modulus of the shell for the recycled plastics is about 600 ksi and the compressive modulus of the material is around 125 ksi. This manufacturer mixed fiberglass with the recycled plastics resulting in a product with one of the highest mechanical properties. This was one of the first recycled plastic products and since then the industry has advanced to a point where there are many products to choose from.

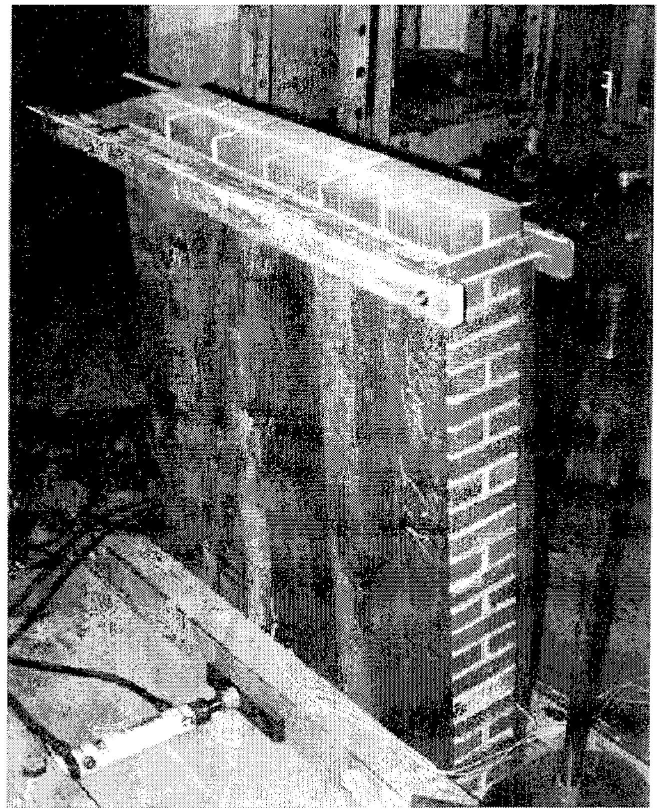

Figure 3 A URM Wall Retrofitted with Recycled Plastics Lumber

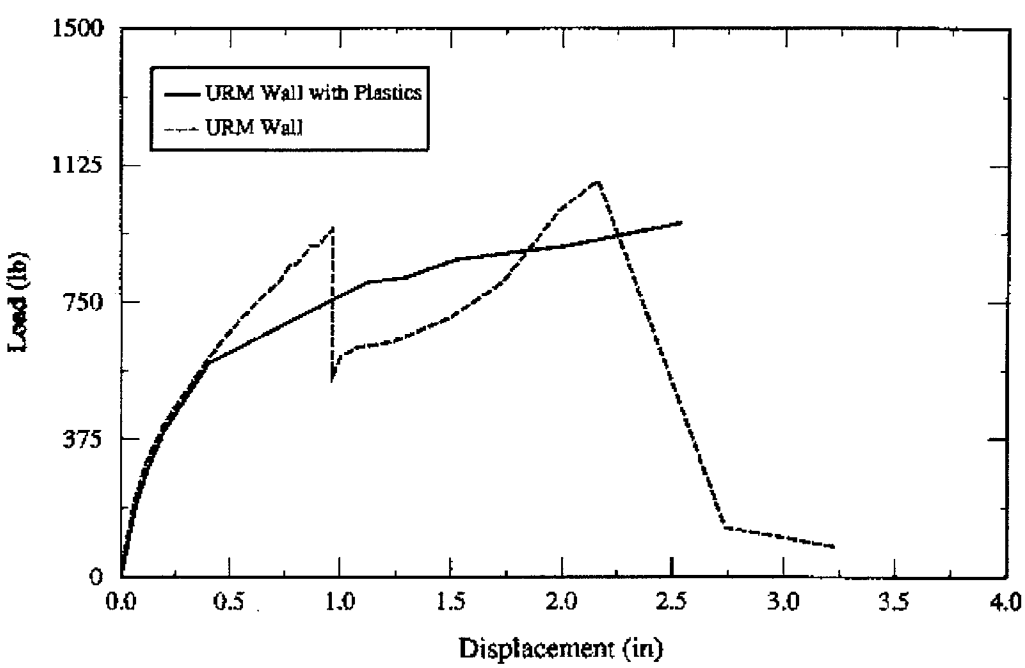

Figure 4 Lateral Load-deflection Curves

The load-deformation curve for the retrofitted URM wall is also shown in Figure 4 with solid line. As it can be seen from the comparison of the two curves, attaching the recycled plastic layers did not increase the initial stiffness of the wall. Normally, one would expect a slight increase in the initial stiffness because, although very flexible compared to the URM wall, the recycled elements do have some stiffness. However, in this case the retrofitted wall has even slightly lower initial stiffness. This is probably due to microcracks formed in the URM wall during the initial test. Repair of the URM wall using epoxy was very effective, however, there were quite likely microcracks that could not have been seen and repaired. The load-deformation curve for the retrofitted specimen is smoother and more ductile. Because of stroke limitation of the actuator, it was not possible to apply higher displacements. However, the undamaged condition of the recycled plastic layers and the shape of the load deformation curve indicate that the retrofitted wall can sustain much larger deformations.

As mentioned before, due to lack of gravity load, definite conclusions about the ultimate ductility and performance of the wall cannot be made. What is clear is that the envisioned rehab scheme certainly has merit and deserves further investigation. The favorable characteristics of recycled plastics for resistance of ground motion forces are such that many other applications can be developed.

Conclusions

Over the past decade, there has been significant advancement in the manufacturing technology of commingled plastics to produce recycled plastics products of good quality. This combined with availability of testing standards and performance data have advanced the market share of the products in the construction industry with many in field applications. Seismic applications of the products appear to be a natural extension. Inherent characteristics of recycled plastics bode well to the design of structural and non-structural elements to resist seismic loads as well as for rehabilitation of exiting structures. The latter has been shown through an example. Thus, the material appears to have good potential for mitigation of earthquake hazards.

Acknowledgment

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Smart Materials Technologies for Bolted-joints in Civil Systems

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Abstract

This paper presents the use of smart material technologies for real-time condition monitoring and active control of bolted-joints in civil structures and components. The impedance-based health monitoring technique, which utilizes the electromechanical coupling property of piezoelectric materials, has been used to detect and identify bolt-connection damage. Real-time damage detection in pipes connected by bolted joints was investigated, and the capability of the impedance method in tracking and monitoring the integrity of bolted-joint civil structures has been demonstrated. When damage occurs, shape memory alloy actuators have been used to adjust bolt tension, in order to restore lost torque and allow continued operation. Experimental investigations are presented in order to demonstrate the performance of smart structure technology to civil bolted-joint problems.

Introduction

Bolted connections are prevalent in civil structures and systems. The importance of these joints in maintaining structural integrity is imperative. It has been estimated that approximately 70% of all mechanical failure occurs due to fastener failure (Simmons, 1986). These connections invariably promote damage growth and are often difficult to inspect due to the nature of geometry and/or the loading in structures. Various types of bolt failure that occur include self-loosening, tensile overload, shear overload, hydrogen embrittlement, and fatigue failure.

This paper summarizes the use of smart material technology in monitoring and control of civil bolted-joints, which have been under investigation at the Center for Intelligent Material Systems and Structures (CIMSS). Smart materials contain sensors, actuators, and control systems that allow structures to respond or adaptively change as the result of external conditions. Such materials form transducers that are able to convert electrical energy into mechanical motion or force (and vice versa). Crawley and deLuis (1992) provides a review of modeling and of the principles of using piezoelectric materials, and Tzou (1998) provides a current literature review on smart materials of use in vibration related technologies.

The objective of this study is to significantly reduce resources that are dedicated to inspection routines of joint connections and allow systems to function after mild earthquakes. The impedance-based health monitoring technique, which utilizes electromechanical coupling property of piezoelectric (PZT) materials, has been used to detect and identify bolt-connection damage. When damage occurs, temporary adjustments of the bolt tension can be achieved actively and remotely using shape memory alloy (SMA) actuators in order to restore lost torque for continued operation. The theory behind these techniques and experimental investigations are presented in the following sections.

Impedance-based Structural Health Monitoring Technique

The impedance-based health monitoring method utilizes the direct as well as the converse piezoelectric effect simultaneously, hence, one PZT patch can be used for both actuation and sensing of the structural response.

Principle of the technique

By analyzing the interaction of the PZT with a host structure, it has been shown that the electrical impedance of PZT is directly related to the mechanical point impedance of the external structure (Sun *et al.*, 1995). If a structure is damaged, the structural parameters, such as mass, stiffness or damping would be changed. In other words, the mechanical impedance would be modified. Since all other PZT properties remain constant, any changes in the electrical impedance signature of piezoelectric materials are attributed to damage or change in the structure A complete description of this technique is found in the literature (Sun *et al.*, 1995). The variation in the electrical impedance of a PZT bonded to the structure, over a frequency range, is analogous to the frequency response but has much higher resolution and is more easily obtained. Such systems can be easily retrofit to existing piping systems

Experimental implementation of the impedance-based structural health monitoring technique has been successfully conducted on several complex structures; a four bay space truss (Sun *et al.*, 1995), an aircraft structure (Chaudhry *et al.*, 1995a), composite patch repair (Chaudhry *et al.*, 1995b), complex precision parts (Lalande *et al.*, 1996), composite-concrete combinations (Raju *et al.*, 1998), applications under significant varying temperature conditions (Park *et al.*, 1999).

Health Assessments of Civil Pipelines Connected by Bolted Joints

This section describes the performance of the impedance-based technique in detecting real-time damage on a sample pipeline structures. Several conditions were imposed to simulate real-time damage, and the capability of the impedance method in tracking and monitoring the integrity of the typical civil facility has been demonstrated.

Bolted joints are commonly used to connect segmented piping lines, and need to be monitored to ensure the integrity of entire pipelines. When an earthquake occurs, this interface can be the most critical source of failure of the pipelines, since significant ground movements can stress the joint beyond its yield or buckling capacity, while the main body of the pipe remains elastic (Eiginger, 1999).

A model of a pipeline with bolted joints is shown in figure 1. This model consists of segmented pipes (d-40 mm), flanges, elbows, and joints connected by more than 100 bolts. The size of this structure is 2 m wide and 1.3 m tall. One PZT sensor/actuator (15 x 15 x 0.2 mm) is bonded on each joint to monitor the conditions of this structure. The HP4194 electrical impedance analyzer was used for the measurement of PZT's electrical impedance in the frequency range of 80-100 kHz.

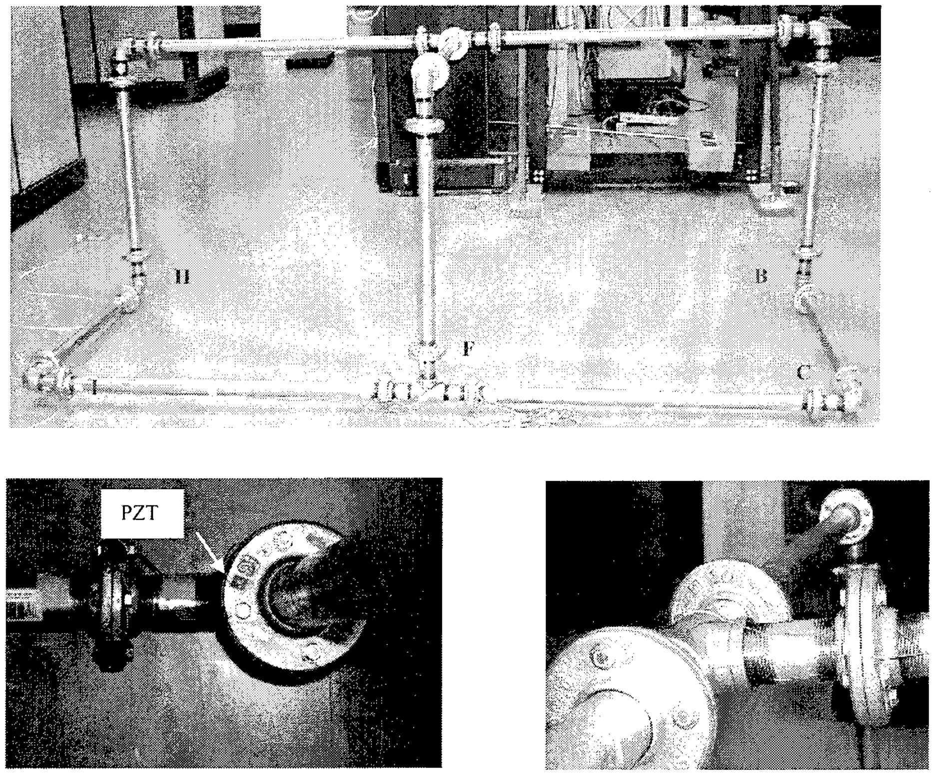


Figure 1. A pipeline used in the experiment

The total impedance of each junction (2 or 3 joints), labeled A to I in figure 2, was utilized to track the damage. The total impedance refers to 'a single impedance signal acquiring from distributed PZTs'. The leads from the several distributed PZTs were physically connected together and this single lead was then connected to the terminal on the impedance analyzer. This procedure may reduce the sensitivity of measured electrical impedance due to the multiplexing nature of measurements; however it drastically reduces the interrogation time as compared to that of analyzing each PZT separately as performed in previous experimental investigations. After measuring the baseline impedance signature, damage was introduced by loosening the bolts over several joints on this structure.

Three conditions were imposed on this structure in sequence, as shown below.

- Damage 1 : loosening 3 bolts at Junctions A and B, respectively
- Damage 2 : loosening 2 bolts at Junctions E and G, respectively
- Damage 3 : loosening 4 bolts at Junctions F, G, and H, respectively

The impedance measurements (real part) of PZTs located at junction A are shown in figure 2. For junction A, when Damage 1 was introduced, the measurement was significantly different from the baseline measurement. This is because loosening the bolts alters local dynamics. However, when the other two damage conditions were imposed, the remaining curves followed the same pattern as that of the second reading, since those were well out of sensing range of PZTs. Other impedance measurements of junction G are shown in figure 3. The location of Damage 1 was out of the sensing range of PZTs, hence almost no change in impedance curve was observed. However, when Damage 2 was introduced, the impedance measurement was significantly different from the previous readings and was affected by the presence of damage. When damage 3 was introduced, the measurements indicated another complete change in the signature pattern.

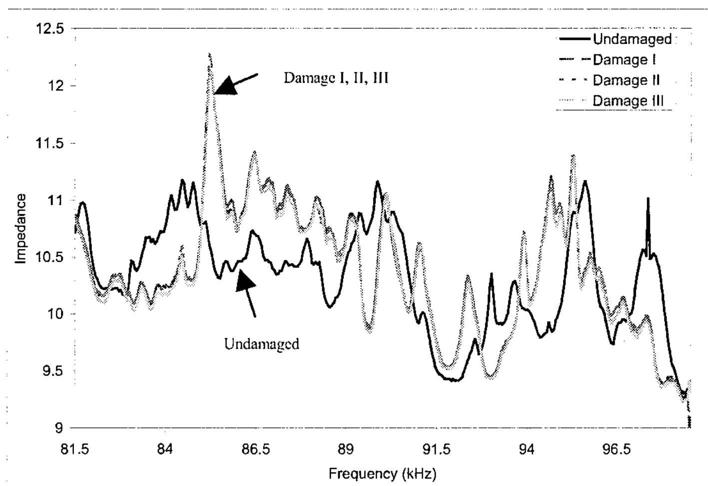


Figure 2. The electrical impedance measurements of PZTs at Junction A.

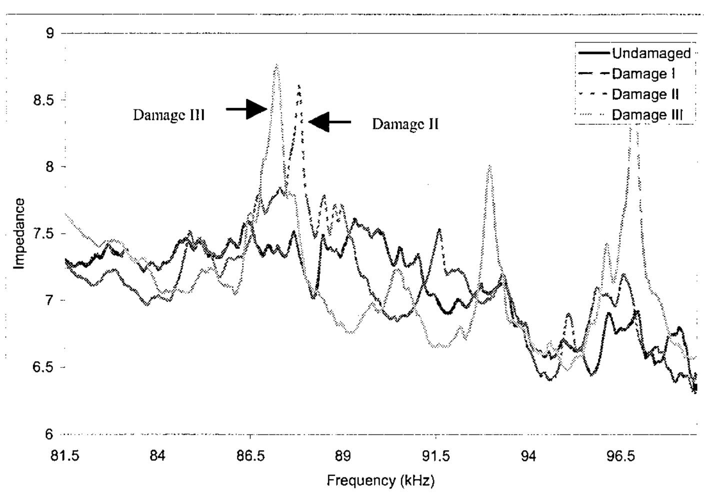


Figure 3. The electrical impedance measurements of PZTs at Junction G.

The damage metric chart demonstrates the results more clearly, as can be seen in figure 4. Damage metric, defined as the sum of the squared differences of the real impedance changes at each frequency step, is used to simplify the interpretation of the impedance variations and provides a summary of the information obtained from each impedance response curve. It can be seen that at Damage 1, there is a large increase in the damage metric value for PZTs at A and B. The other PZTs show a very small change in the damage metric, because they are distant from the damage. Similar results were obtained when damage 2 and 3 were induced. Each PZT showed an increase in the damage metric value, if damage was induced close to the sensors. By looking for variations in the impedance measurement and in the damage metric value, damage in the joints can be detected and the integrity of the structure can be monitored (throughout its service life). It should be noted that the time necessary to take the impedance measurements and to construct the damage metric chart is less than 5 minutes, which is quick enough for an on-line implementation of this technique. It in turn demonstrates the capability of the impedance method to detect imminent damage under normal operating conditions and after a natural disaster, when a quick assessment of a structure is urgently needed.

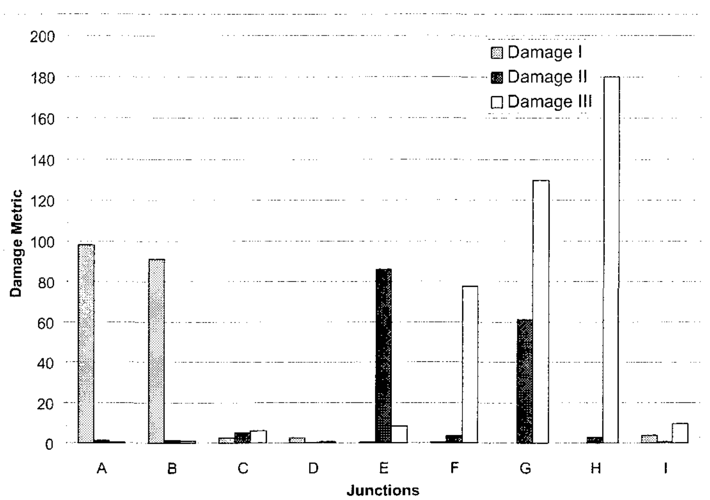


Figure 4. Damage metric chart over the different locations.

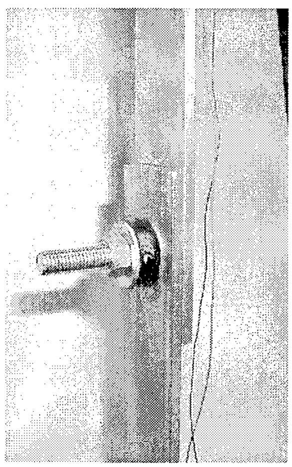
As shown in this example, the impedance method utilizing smart materials has several advantages over traditional damage identification approaches. The sensor/actuators used in the impedance method are very small and unobtrusive, and come in a variety of sizes, allowing them to be placed even in remote and inaccessible locations. Since the method is not based on any analytical model of a structure, the impedance method can be easily applied to very complex structures. The impedance method also shows extreme sensitivities to the minor defects in the structures, because of high frequency employed in the technique. The most important aspect of the impedance method however is its potential to develop into a completely autonomous monitoring system.

Self-Healing Bolts

SMA actuators provide a variety of solutions to engineering problems that require actuators to deliver high force, high stroke, and high force-to-volume (or weight) ratios. Much attention has been devoted to the study of these actuators, as spring design is already an extensively-developed field. The shape memory effect, in brief, is the ability of one of several metal alloys to change between two crystal structures, one at a high temperature (austenite) and the other at a lower temperature (martensite). The crystal change occurs as a result of twinning and de-twinning crystal planes. The macroscopic result is that the alloy can deform without the movement of crystalline dislocations. Rather, material deformation occurs due to the movement of twin planes. Therefore, material strains can be readily recovered, and the material appears to remember its original state when the shape memory transformation occurs.

The most common failure in bolted joints is its loosening modes. As the torque loosens, the joint cannot unite the structural members. A new technique capable of actively resisting the loosening modes of the civil bolted-joints and rendering joint safe for continued operation has been under investigation at CIMSS. The basic principle of this technique is that the temporary adjustment of the decreased torque can be achieved using a shape memory alloy actuator around the axis of the bolt shaft. Specifically, the actuator is a cylindrical Nitinol washer that expands axially when heated, according to the shape memory effect. Upon actuation, the stress generated by its axial strain compresses the bolted members and creates a frictional force that has the effect of generating a preload and restoring lost torque. The following example illustrates how this method can be applied to a real structure.

Proof-of-Concept Application



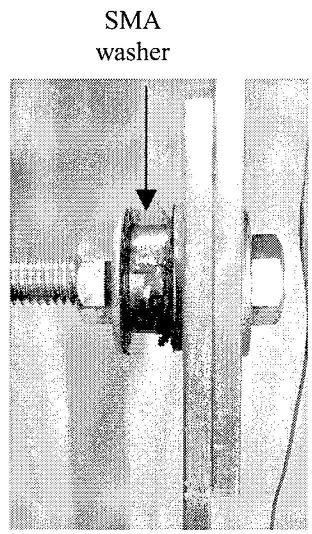


Figure 5. Experimental setup

A test specimen consisting of two aluminum beams was constructed with a bolted joint. The bolted joint structure was hung vertically by a string. One PZT patch bonded to one of the members was used to measure the electrical impedance. An SMA washer was inserted between the bolt and the nut, as illustrated in Figure 5. Initially, the bolt was tightened to 30 ft-lb, and the torque was reduced to 10 ft-lb to introduce a loosening mode of a bolt failure. This damage however can be considered in its incipient stage, which still maintains the integrity of the joint.

The SMA actuator was then electrically activated to create a force to restore the lost torque. Due to complexity of constitutive modeling, no analytical modeling effort has been attempted at this stage. The electrical impedance was measured at each step of torque, and qualitative analysis by the measured impedance was performed to track the changes in mechanical characteristics of the joint.

To identify the joint property of the structure, the real portion of measured electrical impedance was used. The sharp peaks in the real part of electrical impedance correspond to the structural resonances (Sun et al., 1995). Figure 6 shows one of the structural major resonance frequencies. The damage in the bolt causes a regular downward shifting (up to 2%) of the resonance peak, which suggests a stiffness reduction. In the other resonance peaks shown in Figure 6, similar drops in resonance behavior occur as the stiffness of the bolt drops. In addition, the peak amplitudes are reduced, which indicates changes in structural damping.

After the actuation of the SMA washer, one can clearly observe that the actuator causes an upward shifting of the resonance peaks, which in turn suggests restoring the degraded bolt preload. By focusing on frequency ranges around major resonances, the locations of peaks that actuated are even slightly higher than those in baseline measurements. The SMA actuator generates the force to recover the lost torque and allows the structure to continue in operation until a convenient time for a more permanent repair is possible. Note that this method is used in conjunction with the impedance-based health monitoring technique described in the previous section. The impedance method would detect and inspect whether the damage threshold value has been reached or not, and provides a signal to activate the SMA actuator in order to restore the loss of preload, that would otherwise lead to catastrophic failure of a structure.

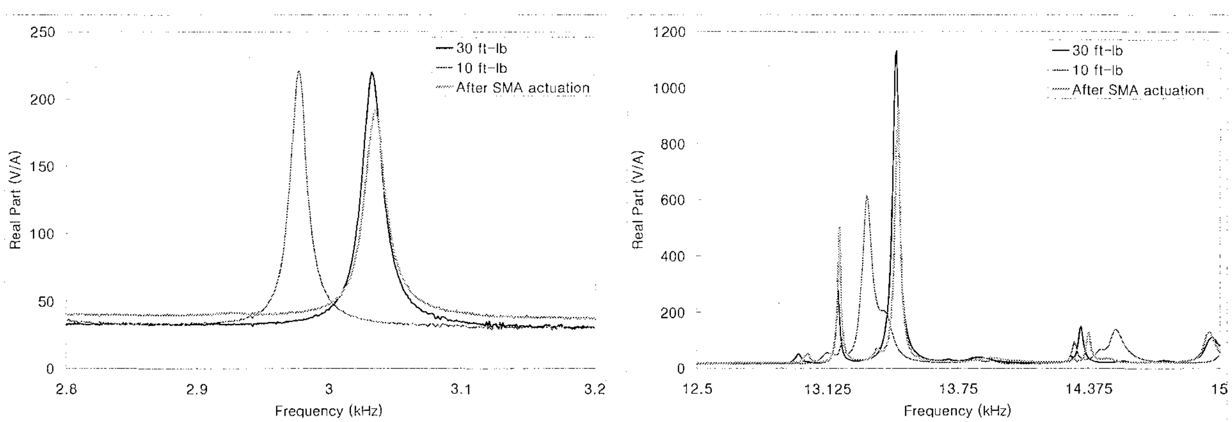


Figure 6. Impedance (real part) signature showing regions of resonance peaks.

CONCLUSION

We have illustrated that the integrity of a bolted piping system can be successfully monitored using an impedance-based method implemented using low voltage piezoceramic patches. Furthermore we have illustrated that shape memory alloy washers can be used to regain lost torque in bolts determined to be out of torque in an automated way. Thus, we have set forth preliminary results, which would allow the construction of a retrofit system that could be added to an existing piping system, and provide both condition monitoring and self repair. This could allow piping systems in critical facilities to be examined remotely after an earthquake. If found to be lose, the system could re-tighten itself and be immediately ready for service, until such a time that a more permanent repair could be made. Beyond this application, the self-healing concept proposed here may be extended to pipe hangers and other non-structural fixtures in critical facilities.

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Appendix A: Workshop Discussions

Technical Block 1: Geotechnical Issues
James K. Mitchell and Ricardo Dobry

Technical Block 2A: Materials and Damage Monitoring
Michel Bruneau

Technical Block 2B: Passive and Active Control Systems in Health Care Facilities
T.T. Soong and Andrew S. Whittaker

Technical Block 3: Nonstructural Retrofit
Daniel J. Inman

Geotechnical Issues
Summary and Discussion

James K. Mitchell and Ricardo Dobry

The presentations and discussions by MCEER researchers and invited practitioners and researchers - as well as discussions from other workshop participants including the OSHPD representatives - were very helpful in defining needs, issues and research opportunities on the subject of application of advanced technologies to mitigate the risk of ground and foundation failure in hospitals.

Many existing hospitals in California and other areas of the US are on liquefiable ground, including a significant fraction of the 470 hospital sites in California containing general acute care facilities. This creates a potential liquefaction problem which may prevent them from remaining operational after an earthquake. While no significant problem has yet occurred due to liquefaction and ground failure in recent US earthquakes, the possible consequences are illustrated by a hospital in Port Island, Kobe, Japan, which ceased operation for several weeks after the 1995 earthquake due to liquefaction. We heard examples during the session of a number of new hospital projects or extensions of existing hospitals in California and Utah, which have addressed the problem through site remediation using existing techniques. This solution - as well as the alternative solution of foundation retrofitting - becomes much more difficult and costly for existing hospitals. Therefore, a need clearly exists for new economical and less disruptive solutions using new and advanced technologies to address the liquefaction problem at existing hospital sites. On that basis, there was consensus that MCEER's geotechnical research is on the right track, especially in its emphasis on rehabilitation and nondisruptive remediation against liquefaction.

In addition to reviewing existing solutions for pile foundation retrofitting and site remediation, a number of advanced technologies being considered for liquefaction/lateral spreading risk mitigation were discussed in the session, including those constituting the focus of MCEER's research. Some of these advanced technologies have been implemented in actual projects. The technologies discussed included: foundation isolation using frangible materials; passive site remediation by grouting; electro-kinetically assisted injection for silts and silty sands; energy-based methods for deep densification design; micro-pile and soil reinforcement (including soil nailing) for seismic retrofitting; compaction grouting applications; and deep soil mixing and in-ground walls. An important related issue is the need for evaluation using deformation-based design. Other issues which are significant to pile foundations and/or micro-piles are capacity, moment resistance, pile structural capacity, high moment at connections for inclined piles, and the need for pile load tests. Performance during actual earthquakes is and will continue to be the ultimate evaluation tool, and encouraging evidence is already available for some of these technologies.

It is important to identify as clearly as possible the main issues relevant to the development and implementation of advanced technologies for mitigation of ground and foundation failure risk in hospitals due to earthquakes. Some of these issues are:

- How does the method work?
- What is the current state of development?
- How effective is it?
- How is the effectiveness evaluated?
- Can it be used in and around existing facilities with minimum disruption?
- How much does it cost at both new and existing facilities?
- How can it be integrated into a comprehensive program for seismic safety?
- Has it been used before?
- Has its effectiveness been tested, ideally by an actual earthquake, or at least through reasonably realistic centrifuge and other model tests?
- What are the desirable next steps in development?

Finally, the session was very fruitful in exploring geotechnical opportunities opened by the emergence of new technologies. Research opportunities toward mitigation of ground failure in hospital and other facilities are being explored by the MCEER team; other related research opportunities exist in more basic liquefaction phenomena which are today poorly understood, such as the nature and determination of residual strength including pore water pressure redistribution during and after shaking. The use of geotechnical centrifuges as a focal point of both applied and basic research opportunities was emphasized. Other geotechnical opportunities arise in field applications, such as modernization of in situ tests and further development and automation of imaging techniques for site characterization; and site/system characterization, damage assessment, quality control, and ground motion and structural response using new-generation sensors which take advantage of MEMS and/or wireless technologies. Some of these new sensor technologies are also being developed toward their use in centrifuge model tests.

Materials and Damage Monitoring

Michel Bruneau

Overall Observations

There was a general agreement among the MEDAT-2 participants that the advanced technologies being investigated by MCEER as part of Program 2 (Hospital Project) are appropriate. More specifically, the focus on energy dissipation materials (composite, engineered cementitious concrete, and metallic infills) is state-of-the-art. Shape Memory Alloys and low yield steels also have excellent potential and could be considered as part of the research activities on metallic infills.

Advanced Materials

Advanced materials have an important advantage over mechanical devices, as they would not require testing prior to implementation in a structural retrofit, provided adequate documentation already exists on that material and component behavior. Advanced materials approaches are also able to provide high reliability and durability of the retrofit scheme, two of the most important major issues in the retrofit of hospitals. For example, use of low-yield steel in a retrofit project was approved by OSHPD in only one year (following a small array of additional required tests). In a more general sense, implementation will accelerate when production software will include materials or element models for the new advanced material-based retrofit systems.

It is apriori no technical barrier against the use of advanced composite materials in hospitals, as some have already used in the wrapping of concrete columns to provide confinement. Fire-resistance of the new materials is not an issue provided the materials are used for seismic resistance only. The only foreseen barriers are economical issues (which evolve with time) and resistance to innovation by practicing engineers. This latter point is not insignificant, and is broader in scope as it applies to any new technology proposed to enhance seismic performance.

Health Monitoring

Past experience has shown that new technologies are unlikely to be implemented unless a clear benefit is perceived. This was clearly the case with implementation of base isolation and damping mechanical devices for seismic retrofit. In that perspective, identification of the benefits is the first requirement toward the implementation of health monitoring systems for hospital structures. Currently, hospitals in California are being instrumented with simple strong-motion recorders. Unfortunately, agencies are under pressure to require that less instrumentation be installed in retrofitted hospitals. The problem arises because owners must pay for these monitoring systems; they do not see the benefits of such instrumentation, particularly since it will not be used until an earthquake strikes in the distant future. This is a major impediment against implementation, even though, conceptually, there was general agreement by the experts at the workshop that the availability of health monitoring is desirable.

On a technical level, it is also understood that for health monitoring to become truly successful, the desired performance limits must be better defined, understood and quantified. Until that is achieved, health monitoring systems cannot be effectively designed because the tools to evaluate the data generated by these systems are missing (and not conceptually defined). As a result, significant research efforts are required to focus on improving the engineering definitions of the performance goals, and the methods to achieve them.

Passive and Active Control Systems in Health Care Facilities

T. T. Soong and A. S. Whittaker

Protection of Health Care Facilities

Presentations by experts in the design and construction of health-care facilities identified a number of important issues related to new and retrofit health-care construction, with and without passive, semi-active, and active control systems. Design and construction must address non-structural and structural components and substantial emphasis must be placed on the protection of critical non-structural systems (communication systems, generators, life-support (gas) systems, fire-alarm and fire-suppression, emergency lighting) as such unprotected components are likely to fail at lower levels of earthquake shaking than structural components. Linkages between building performance levels (such as Immediate Occupancy or IO and Collapse Prevention or CP) and performance levels for structural and non-structural components must be developed. Critical non-structural components in health-care facilities must be identified for each non-structural performance level. Quantitative design parameters (e.g., deformation, acceleration, and velocity) for structural components and critical non-structural components for each structural and nonstructural performance level must be prepared.

Earthquake Protective Systems for Health Care Buildings

The discussion on the use of protective systems in health-care facilities was categorized into two parts: passive control devices and systems, and semi-active and active control devices and systems.

Passive Control Devices And Systems

Passive control of the earthquake response of buildings is a relatively mature technology, with more than 60 applications to date in the United States. Some of these applications have been to health-care facilities in the United States. Codes of practice and guidelines for the implementation and testing of passive control hardware in the United States (IBC 2000, NEHRP 2000) have been prepared and promulgated. Studies by Wada (2000) have shown that the construction cost for a building constructed with passive control devices can be substantially less than a conventionally framed building for the same level of performance. The key concern with the use of mechanical passive control devices is related to their long-term (30 yr) reliability and stability, but this concern can be mitigated or allayed by periodic inspection and testing, which are mandated by current codes and guidelines. The discussions identified a number of opportunities for future research both within and outside the MCEER program, including,

- hybrid protective solutions for structural and nonstructural components
- life-cycle cost-benefit analysis for new and retrofit construction for *minimum* and *enhanced* performance objectives
- new materials (SMA, VE), hardware, and configurations

- procedures to size and distribute dampers for a variety of motions (e.g., near-field, far-field)
- appropriate damper types (e.g., metallic yielding versus fluid viscous) for new and retrofit construction
- innovative and automated strategies for performance-oriented construction of controlled and conventional health-care facilities
- reinforced composites for foundation construction
- control device reliability and redundancy, and influence on building response

Semi-Active And Active Control Devices And Systems

Semi-active and active control devices and systems have not been used to control the earthquake response of buildings, but have been widely used in the United States to mitigate the dynamic response of mechanical, power, aerospace, and defense-related components and systems. Such devices and systems have been implemented in building structures in Japan and China to control response to earthquake shaking. The two key impediments to the widespread implementation of semi-active and active control devices and systems are the lack of widely accepted implementation strategies and control algorithms, and significant skepticism in the design-professional community. The discussions identified a number of opportunities for future research on semi-active and active control devices and systems both within and outside the MCEER program, including,

- benefits of semi-active and active systems for mitigating the effects of near-field earthquake shaking
- hybrid passive and active systems
- new and adaptive control algorithms
- new controllable materials (SMA, ER and MR fluids, thermoplastics, elastomers)
- new actuators (piezo-electric, electro-dynamic) for control of response of non-structural components
- construction and testing of full-scale devices and model structural and non-structural systems
- control device and electronics reliability and redundancy, and influence on building response

Panel Discussion: Non Structural Retrofit

MEDAT 2 @
Las Vegas
November 2000

Virginia Tech

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MEDAT NOV 2000 1

MCEER MEDAT 2 Nov 30 - Dec 1, 2000

Comments and Issues

- PGA given by seismic maps are useless for cost benefit analysis: Same PGA can correspond to many low or a few large earthquakes(Grigoriu)
- Is dynamic (mass) analysis important for buried pipe components? (Maragakis)
- Rehabilitation is a major issue because of limited information from equipment suppliers (Soong)

Virginia Tech
CIMSS
MEDAT NOV 2000 2 MCEER MEDAT 2 Nov 30 - Dec 1, 2000

Maragakis: Static tests are sufficient for single components. Dynamic tests may be needed for systems. We should review standard practice for gas and steam lines.

Soong: need for performance criteria and need information about strength of restraints

SQURTS has data nuclear power plant equipment and may be useful to understand the effects of earthquakes on secondary equipment and may be useful in hospitals.

- Modeling of the rollers and methods of protection (Singh)
- How to define what is essential for inspection, -going beyond anchorage
  - disconnect between engineering disciplines
  - performance of coupled systems under a decoupled design
  - seismic isolation of MRI
  - can a realistic Rp factor be developed? (Staehlin)

MEDAT NOV 2000 3

MCEER MEDAT 2 Nov 30 - Dec 1, 2000

Comments on Staehlin: Need for performance criteria, limited understanding of piping behavior(seismic behavior of pipes, most failures take place at joints. Differentiate between the behavior of component and systems.

Singh: Many failures have been observed in elevators in most earthquakes. New standards (from ASME) include seismic provisions, but need to be tested and studied.

- Need a document explaining how to retrofit piping systems for seismic loads (Antaki)
- Available design information for SMA (Ding)
- Testing in seismic loads, application (Saadeghvaziri)
- “Run flat” modes for components in the context of self-healing systems (Inman)

Virginia Tech

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Some final comments: A large discussion on the need to monitor non structural systems revealed a controversy. It has been suggested to consider monitoring the flow in pipe systems to determine the degree of damage.

What is need in recycled materials are: full scale testing, and an earthquake specific application.

Appendix B
List of Workshop Presentations

Workshop Presentations

Presentations made at the workshop are available on the MCEER web site, under "Publications," "Special Publications." They are in either PowerPoint or PDF format, and complement the 30 papers in this volume. The address is *http://mceer.buffalo.edu/publications/sp_pubs/medat2/default.asp*.

Presentation Title	Author/Presenter
Welcome and Introduction	
Mitigation of Earthquake Disasters using Advanced Technologies (MEDAT-2) Introduction	Michel Bruneau, MCEER and the University at Buffalo
Technical Block 1: General Overview of Earthquake Engineering Issues	
MCEER Geotechnical Research	Ricardo Dobry, Rensselaer Polytechnic Institute
Ground Improvement for Hospitals/Medical Facilities	Juan Baez, Hayward Baker, Inc
MCEER Task 2.3 Geotechnical Rehabilitation of Sites and Foundation Retrofitting	Ricardo Dobry, Rensselaer Polytechnic Institute
Liquefaction & Mitigation in Silty Soils	Sabanayagam Thevanayagam, University at Buffalo
Mitigation of Liquefaction Hazards	Juan Baez, Hayward Baker, Inc.
MEDAT-2: Some Geotechnical Opportunities	Ross Boulanger, University of California, Davis
Retrofitting Highway Bridges and Wharf Facilities: Geotechnical Perspective	Ignatius Po Lam, Earth Mechanics, Inc.
Technical Block 2A: Advanced Technologies for Structural Retrofit	
General Overview of Advanced Technologies	Daniel Inman, Virginia Polytechnic Institute
General Overview of Earthquake Engineering Issues for MCEER Hospital Project	Michel Bruneau, University at Buffalo
Engineered Cementitious Composites for the Retrofit of Critical Facilities	Sarah Billington, Cornell University

Polymer Matrix Composite In-Fill Walls for Seismic Retrofit	Amjad Aref, University at Buffalo
Pattern Recognition for Structural Health Monitoring	Charles Farrar, Los Alamos National Laboratory
Seismic Control Devices Using Low-Yield-Point Steel	Yasushi Maeda, Nippon Steel
Engineered Cementitious Composites (ECC) for Seismic Applications	Victor Li, University of Michigan
Technical Block 2B: Damping and Semi-Active Systems	
Current Practice of Passive Control of Earthquake Response	Andrew Whittaker, University at Buffalo
Enabling Technology Testbed Project: Design of a Semi-active protective system for a LA Building	George Lee, MCEER and University at Buffalo
Computational Aseismic Design and Retrofit	Gary Dargush, University at Buffalo
California's Experience in Seismic Retrofit of Hospital Buildings	Chris Tokas, California OSHPD
Recent Trends of Damage Controlled Structures in Japan	Akira Wada, Tokyo Institute of Technology
Passive and Active Systems in Hospitals	Andrew Whittaker, University at Buffalo
Technical Block 3: Advanced Technologies for Nonstructural Retrofit	
Advanced Technologies for Non-structural Retrofit	Mircea Grigoriu, Cornell University
Performance Criteria for Non-Structural Components	William Staehlin, OSHPD
Seismic Retrofit of Critical Piping Systems (Above Ground Piping)	George Antaki, WSRC
Recycled Plastics: Characteristics and Seismic Applications	M. Ala Saadeghvaziri, New Jersey Institute of Technology
Smart Materials Technologies for Bolted Joints in Civil Systems	Daniel Inman, Virginia Polytechnic Institute
Panel Discussion: Nonstructural Retrofit	Daniel Inman, Virginia Polytechnic Institute

Appendix C: Workshop Information

Agenda

Participants

Agenda

MEDAT-2 Workshop
November 30 - December 1, 2000

Thursday, November 30, 2000

8:00 - 8:10 am	Welcome/Introduction, Presentation of Workshop Objectives, and General Comments (Co-chairs of Workshop)
8:10 - 8:30 am	(1) General Overview of Earthquake Engineering Issues - Dr. Ricardo Dobry and Juan Baez
8:30 - 9:15 am	(2) Examples: (a) Dr. Ricardo Dobry, Rensselaer Polytechnic Institute (on: Centrifuge Study of Nonretrofitted and Retrofitted Pile Foundation Subjected to Lateral Spreading) (b) Dr. Sabanayagam Thevanayagam, State University of New York at Buffalo (on: Silt Liquefaction (c) Dr. James Mitchell, Virginia Polytechnic Institute and State University (on: Passive Site Remediation for Liquefaction Risk Mitigation)
9:15 - 10:15 am	(3) Examples (a) Dr. Juan Baez, Hayward Baker Inc. (on: Ground Improvement for Liquefaction Hazards) (b) Dr. Ilan Juran, Polytechnic University (on: Micropile and Soil Reinforcement for Seismic Retrofitting) (c) Dr. Ross Boulanger, University of California at Davis (on: Ground Improvements) (d) Mr. Ignatius Po Lam, Earth Mechanics Inc. (on: Foundation Liquefaction Retrofit Work)
10:15 - 10:35 am	Break
10:35 - 11:00 am	(4) General overview of Advanced Technologies Issues - Dr. James Mitchell, Virginia Polytechnic Institute and State University
11:00 - 12:00 pm	(5) & (6) Panel discussion, findings and recommendations on Technical Block 1
12:00 - 1:30 pm	Lunch
	Technical Block 2: Advanced Technologies for Structural Retrofit
1:30 - 1:50 pm	(1) General Overview of Earthquake Engineering Issues for MCEER Hospital Project - Dr. Michel Bruneau, Multidisciplinary Center for Earthquake Engineering Research at the State University of New York at Buffalo
	Block 2A - Materials and Damage Monitoring
1:50 - 2:20 pm	(2) Examples: (a) Dr. Sarah Billington, Cornell University (on: ECC Energy Dissipating Panels) (b) Dr. Amjad Aref, State University of New York at Buffalo (on: Advanced Composite Energy Dissipating Panels)
2:20 - 2:50 pm	(3) Examples: (a) Dr. Charles Farrar, Los Alamos National Lab (on: Pattern Recognition in Health Monitoring) (b) Dr. Kincho Law, Stanford University (on: Wireless Sensing)
2:50 - 3:10 pm	Break

3:10 - 4:25 pm	(3) Examples Continued: (c) Dr. Daryl Hodgson, Share Memory Applications (on: Shape Memory Alloys) (d) Mr. Yasushi Maeda, Nippon Steel (on: Low Yield Steel Applications) (e) Dr. Vistasp Karbhari, University of California, San Diego (on: Use of FRP Composite Materials in the Renewal of Civil Infrastructure in Seismic Regions) (f) Dr. Victor Li, University of Michigan (on: Structural Composites with ECC) (g) Fred Isley, Hexcel Corporation (on: Composite Fabrics)
4:25 - 4:45 pm	(4) General Overview of Advanced Technologies Issues - Dr. Jayanth Kudva, Northrop Grumman Corporation
4:45 - 5:45 pm	(5) & (6) Panel discussion, findings and recommendations on Technical Block 2A

Friday, December 1, 2000

	Block 2B - Damping and Semi-active Systems
8:00 - 8:20 am	Damping and Semi-active Systems - A State-of-the-Art Report - Dr. Larry Soong, State University of New York at Buffalo
8:20 - 8:40 am	State-of-Practice Report - Dr. Andrew Whittaker, State University of New York at Buffalo (on: Passive Seismic Control of Building Structures)
8:40 - 9:25 am	(2) Examples: (a) Dr. Andrei Reinhorn, State University of New York at Buffalo (on: Structural Control) (b) Dr. George Lee, Multidisciplinary Center for Earthquake Engineering Research at the State University of New York at Buffalo (on: Demonstration Project of Using Semi-active Control for Seismic Protection of Buildings) (c) Dr. Gary Dargush, State University of New York at Buffalo (on: Automated Design Software for Advanced Technologies)
9:25 - 10:10 am	(3) Examples: (a) Dr. Chris Tokas, California OSHPD (on: California's Experience on Hospital Retrofit) (b) Dr. Edward V. White, Boeing, Inc. (on: Progress in Structural Health Management for Aerospace Vehicles) (c) Dr. Dave Carlson, Lord Corporation (on: Implementation of Semi-Active Control Using Magneto-Rheological Fluids)
10:10 - 10:30 am	Break
10:30 - 11:15 am	(3) Examples - Continued (d) Professor Akira Wada, Tokyo Institute of Technology (on: Recent Trends of Passive Damping Systems in Japan) (e) Dr. Keith Belvin, NASA (on: Structural Vibration Control Systems for Space Station) (f) Mr. Thomas Jung, New York State Department of Health (on: Issues in Seismic Retrofit of Hospitals)
11:15 - 11:35 am	(4) General Overview of Advanced Technologies Issues - TBA
11:35 - 12:30 pm	(5) & (6) Panel discussion, findings and recommendations on Technical Block 2B
12:30 - 1:30 pm	Lunch

	Technical Block 3: Advanced Technologies for Non-structural Retrofit
1:30 - 1:50 pm	(1) General Overview of Earthquake Engineering Issues - Dr. Mircea Grigoriu, Cornell University
1:50 - 2:35 pm	(2) Examples: (a) Dr. Manos Maragakis, University of Nevada, Reno (on: Static Axial Behavior of Some Typical Restrained and Unrestrained Pipe Joints) (b) Dr. T.T. Soong, State University of New York at Buffalo (on: Report on Vulnerability and Retrofit of Nonstructural Components) (c) Dr. M.P. Singh, Virginia Polytechnic Institute and State University (on: Seismic Protection of Some Nonstructural Components in Hospitals)
2:35 – 3:50 pm	(3) Examples (a) Dr. William Staehlin, OSHPD (on: Issues on Seismic Retrofit for Hospitals) (c) Dr. George Antaki, WSRC (on: Seismic Retrofit of Critical Piping Systems (Above Ground Piping)) (c) Dr. Zhonghai Ding, University of Nevada at Las Vegas (on: Shape Memory Alloys) (d) Dr. M. Ala Saadeghvaziri, New Jersey Institute of Technology (on: Use of Plastics and Recycled Plastics for Non-structural Retrofit) (e) Dr. Daniel Inman, Virginia Polytechnical Institute & State University (on: Smart Materials Technologies for Bolted-Joints in Civil Systems)
3:50 - 4:10 pm	Break
4:10 - 4:30 pm	(4) General Overview of Advanced Technologies Issues - Dr. Daniel Inman, Virginia Polytechnic Institute and State University
4:30 - 5:30 pm	(5) & (6) Panel discussion, findings and recommendations on Technical Block 3
5:30 - 5:45 pm	Closure: Summary of Outcomes (by: Co-chairs of the Workshop):

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Multidisciplinary Center for Earthquake Engineering Research
List of Technical Reports

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) publishes technical reports on a variety of subjects related to earthquake engineering written by authors funded through MCEER. These reports are available from both MCEER Publications and the National Technical Information Service (NTIS). Requests for reports should be directed to MCEER Publications, Multidisciplinary Center for Earthquake Engineering Research, State University of New York at Buffalo, Red Jacket Quadrangle, Buffalo, New York 14261. Reports can also be requested through NTIS, 5285 Port Royal Road, Springfield, Virginia 22161. NTIS accession numbers are shown in parenthesis, if available.

NCEER-87-0001 "First-Year Program in Research, Education and Technology Transfer," 3/5/87, (PB88-134275, A04, MF-A01).

NCEER-87-0002 "Experimental Evaluation of Instantaneous Optimal Algorithms for Structural Control," by R.C. Lin, T.T. Soong and A.M. Reinhorn, 4/20/87, (PB88-134341, A04, MF-A01).

NCEER-87-0003 "Experimentation Using the Earthquake Simulation Facilities at University at Buffalo," by A.M. Reinhorn and R.L. Ketter, to be published.

NCEER-87-0004 "The System Characteristics and Performance of a Shaking Table," by J.S. Hwang, K.C. Chang and G.C. Lee, 6/1/87, (PB88-134259, A03, MF-A01). This report is available only through NTIS (see address given above).

NCEER-87-0005 "A Finite Element Formulation for Nonlinear Viscoplastic Material Using a Q Model," by O. Gyebi and G. Dasgupta, 11/2/87, (PB88-213764, A08, MF-A01).

NCEER-87-0006 "Symbolic Manipulation Program (SMP) - Algebraic Codes for Two and Three Dimensional Finite Element Formulations," by X. Lee and G. Dasgupta, 11/9/87, (PB88-218522, A05, MF-A01).

NCEER-87-0007 "Instantaneous Optimal Control Laws for Tall Buildings Under Seismic Excitations," by J.N. Yang, A. Akbarpour and P. Ghaemmaghami, 6/10/87, (PB88-134333, A06, MF-A01). This report is only available through NTIS (see address given above).

NCEER-87-0008 "IDARC: Inelastic Damage Analysis of Reinforced Concrete Frame - Shear-Wall Structures," by Y.J. Park, A.M. Reinhorn and S.K. Kunnath, 7/20/87, (PB88-134325, A09, MF-A01). This report is only available through NTIS (see address given above).

NCEER-87-0009 "Liquefaction Potential for New York State: A Preliminary Report on Sites in Manhattan and Buffalo," by M. Budhu, V. Vijayakumar, R.F. Giese and L. Baumgras, 8/31/87, (PB88-163704, A03, MF-A01). This report is available only through NTIS (see address given above).

NCEER-87-0010 "Vertical and Torsional Vibration of Foundations in Inhomogeneous Media," by A.S. Veletsos and K.W. Dotson, 6/1/87, (PB88-134291, A03, MF-A01). This report is only available through NTIS (see address given above).

NCEER-87-0011 "Seismic Probabilistic Risk Assessment and Seismic Margins Studies for Nuclear Power Plants," by Howard H.M. Hwang, 6/15/87, (PB88-134267, A03, MF-A01). This report is only available through NTIS (see address given above).

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