MCEER

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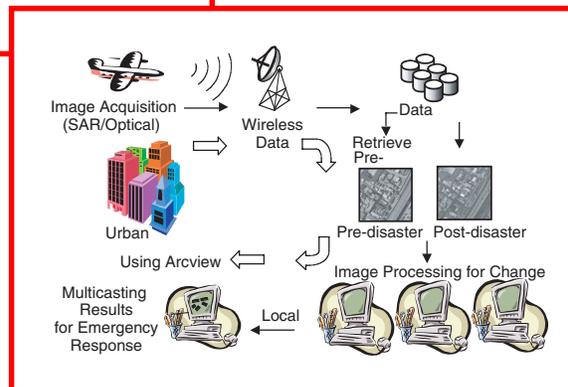
A National Center of Excellence in Advanced Technology Applications

STUDENT RESEARCH ACCOMPLISHMENTS
2000 – 2001

a) Elevation

b) Plan view

Digital Control System

Analog Control System

Custom-Designed Signal Interface System

Active Control Force Generation System

Excitation

Structure

Sensor

Sensor

Image Acquisition (SAR/Optical)

Wireless Data

Data

Retrieve Pre-

Urban

Pre-disaster   Post-disaster

Using Arcview

Image Processing for Change

Multicasting Results for Emergency Response

Local

MAY 2001

▲ *The Multidisciplinary Center for Earthquake Engineering Research*

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) is a national center of excellence in advanced technology applications that is dedicated to the reduction of earthquake losses nationwide. Headquartered at the University at Buffalo, State University of New York, the Center was originally established by the National Science Foundation (NSF) in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission is to reduce earthquake losses through research and the application of advanced technologies that improve engineering, pre-earthquake planning and post-earthquake recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

Funded principally by NSF, the State of New York and the Federal Highway Administration (FHWA), the Center derives additional support from the Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

Student Research Accomplishments

2000-2001

Multidisciplinary Center for Earthquake Engineering Research
University at Buffalo, State University of New York

Edited by Gauri Guha

May 2001

FOREWORD

The Student Leadership Council (SLC) is a formal incarnation of the students who are involved in performing MCEER research under a faculty advisor. Since its inception, MCEER has included and encouraged student efforts throughout its research program and in all of the disciplinary specialties concerned with earthquake engineering.

Throughout the years, students have been an integral component in advancing research in earthquake hazard mitigation. Many former students are now in academia, professional practice or government agencies applying knowledge gained during their exposure to MCEER research. While associated with MCEER, students participate in annual Center Investigator meetings, attend conferences, workshops and seminars, and have the opportunity to make presentations at these events. The SLC was formed last year to formalize these programs and to afford students from many different institutions the opportunity to meet with each other and develop/improve interaction.

The idea for this first issue of the *Student Research Accomplishments* was conceived by the SLC and features the work of some of MCEER's current students. Topics range from traditional civil and lifeline engineering to applications of advanced technologies to social impacts and economic modeling. Papers are presented in two formats: full length, which provide details on the background/motivation for the research, a brief view of prominent work in the area of research, proposed technique/methodology employed, current status of work, how it advanced the state of the art, results from experiments/simulations and how it will be expended in the future. The abstracts either summarize the research, or expand on one of the points listed above.

This issue was coordinated and edited by Gauri Guha, a Ph.D. candidate in the department of energy, environmental & mineral economics at the Pennsylvania State University. MCEER acknowledges Mr. Guha's efforts as well as those of the individual contributors. MCEER also wishes to extend its thanks to the Student Leadership Council for its endeavors, and in particular to Ms. Ani Natali Sigaher for her able guidance of the SLC.

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Addendum

1

ACTIVE STRUCTURAL CONTROL VERIFICATION BY USING REAL-TIME STRUCTURAL SIMULATOR

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Abstract

An active control system for vibration control of structures under environmental loads is a dynamic system that comprises sensors, controller, control algorithm and active control force generator, which acts as an integral system. A number of practical issues arise when a control strategy is implemented in practice. These include discrete-time modeling and control, time delay, model simulation and experimental verification. Verification of active control algorithms prior to their implementation to real structures has always been a time-consuming and costly process. In this research, an alternative real-time verification procedure via a structural simulator is proposed. In this procedure, the control strategy is implemented inside a dedicated PC controller with an independent Digital Signal Processor (DSP) to calculate the required control force. The DSP performs the function of data acquisition through communication with an Analog to Digital Converter (ADC) and a Digital to Analog Converter (DAC). Another PC structural simulator with a similar dedicated hardware is constructed to emulate the real-time response of a theoretic model in order to verify the real-time control effect. The integrated system is dedicated to verifying the effectiveness of any proposed active control algorithm and to providing a pre-implementation testing base to test the functions required for practical utilization.

Introduction

An active structural control system has the basic configuration as shown in Figure 1. It consists of (a) sensors located about the structure to measure either external excitations, or structural response variables, (b) devices to process the measured information and to compute necessary control forces needed based on a given control algorithm, and (c) actuators, usually powered by external energy sources, to produce the required forces. The objective of active structural control is to reduce structural motion during severe external excitations, such as strong wind or earthquake. When an active control system comes to the practical implementation stage, more specified details should be considered before a designed control algorithm is employed.

Specifically, the block diagram in Figure 2 can represent the detail hardware function of an active control system.

The response of a structure under external excitation, the active device status, the remote control status, and the fail-safe monitoring status of Active Control Force Generation System (usually continuous analog voltage signals) are measured by sensors and then sent to a Custom-Designed Signal Interface System. The purpose of this device is to condition and filter signals produced by the sensors. Also some customized fail-safe functions, the remote control function of analog control system, and specified signal communication functions are designed and implemented in this interface system. The required information is filtered and collected and sent to a Digital Control System. The Digital Control System used in the laboratory or in practical applications is usually implemented using a Data Acquisition/Conversion System (DACS) located in one of the expansion slots in the personal computer (PC). To perform the real-time control, the analog signals of the controlled system at each time step are measured by sensors and sent through analog-to-digital converter (ADC) as corresponding digital numbers with a resolution determined by the ADC. The required control forces are calculated by Control Command Calculator and send to the force generator ACFGS, through the interface CDSIS.
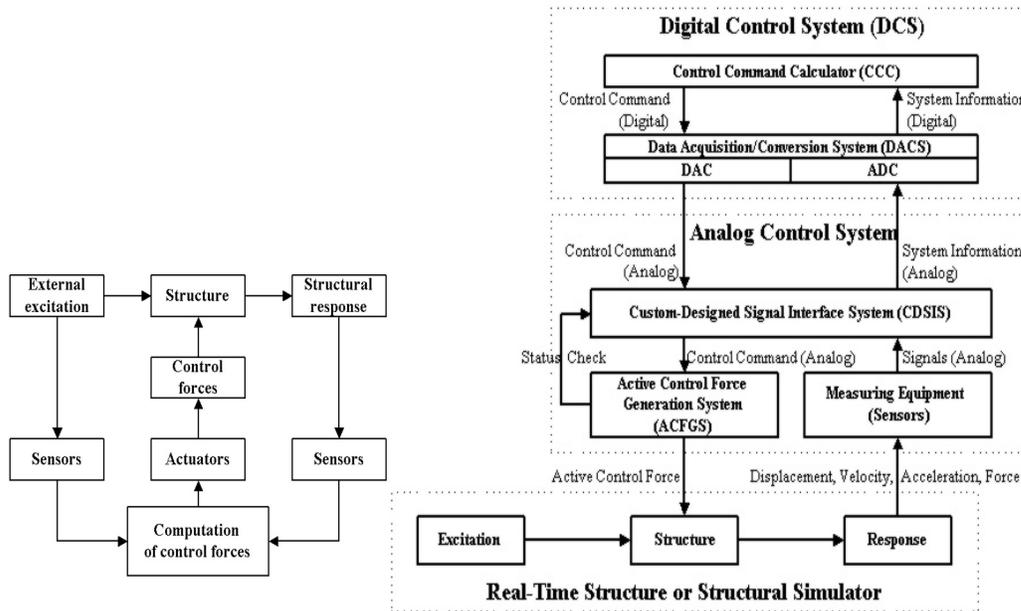
Figure 1. Schematic diagram of active control   **Figure 2.** General hardware function

An example of an active control system is an active mass damper (AMD), which was recently installed in a tower for vibration control under wind and earthquake loads. The required control hardware had been implemented as shown in Figure 3. The active control force is calculated by a dedicated personal computer, PC, controller named here DSP controller (Figure 4). This computer is equipped with a Digital Signal Processor (DSP) control board contained one TMS320C40 processor, a 12-bit 32 channels A/D converter board, and a 12-bit D/A converter board with 16 output channels. This DSP Controller not only performs the control force calculation, but also conducts large amounts of signal manipulation processes, such as monitoring all the control hardware status, trigging the fail-safe protection function, recording the required measurements information, and further more, integrating acceleration measurements to their velocities and displacements. The real-time integration feature overcomes the limitation on sensors or measurement. Although this AMD control system is designed to conduct a full-scale implementational testing platform, the preliminary hardware function and the proposed control algorithms should be verified and tested before the real implementation stage.
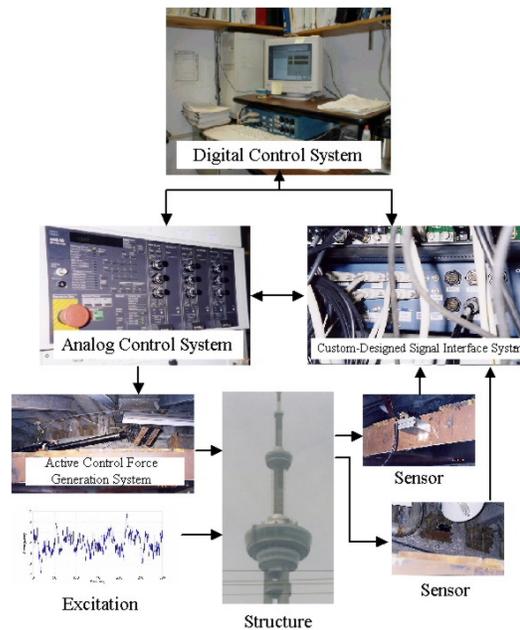

Figure 3. Implemented hardware on the Tower

Real-Time Structural Simulator

In order to verify all the required hardware and software functions and the control effectiveness, the real measurement signals are necessary. However, it is impossible or impractical to perform these experiments on site. Thus a real-time structural simulator is constructed in the laboratory to fulfill this requirement. The real-time simulator is actually a custom-designed function generator that can interact with the DSP Controller in real-time. As shown in Figure 4, it is dedicated to producing the response of a test structure under the external excitation and the action of active control force. The digital real-time structural simulator employed in this experiment utilizes a personal computer equipped with a DT-2801 data acquisition board. A customer-written program downloaded to the microprocessor on the DT-2801 board generates the actual real-time response. This simulator software is written in C language in order to access the hardware and communicate with A/D and D/A converters.
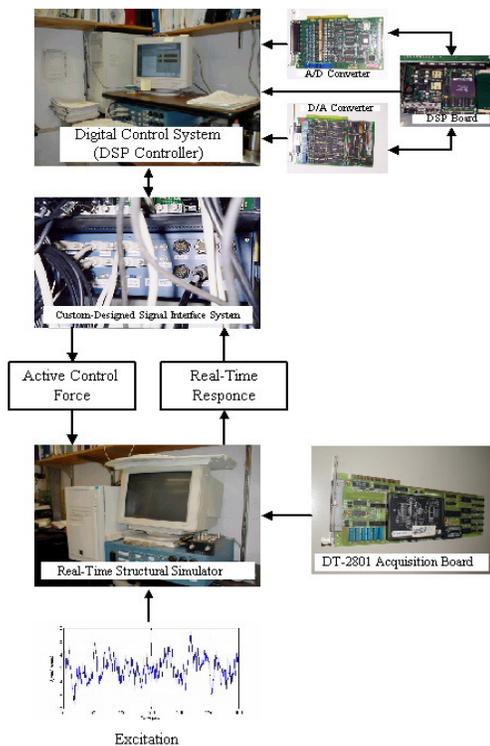

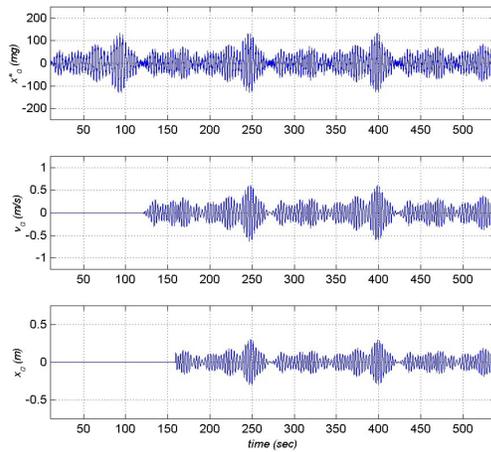**Figure 4.** The setup of real-time integrated verification process

Figure 5.

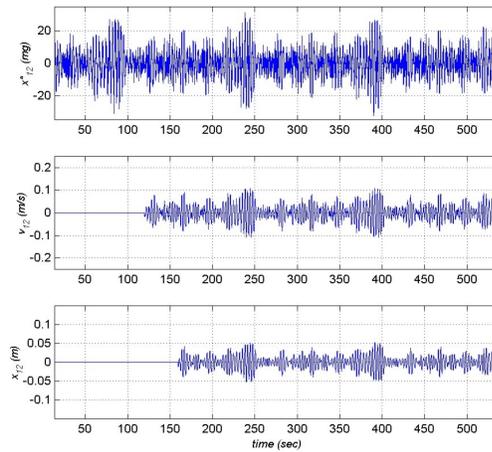The experimental measurements of mass damper

Figure 6.

The experimental measurements of Tower

To perform the real-time control, the output of the system (structural simulator) at each time step is measured by the DSP controller and the required control force is then calculated by the DSP board and sent out the analog voltage signal through D/A converter. In this experiment, the analog voltage signal of the control force is routed directly into the real-time structural simulator to perform the interactive real-time control test. The simulator generates the acceleration measurements of the structure and the mass damper then sends them to the DSP Controller through interface system CDSIS. The corresponding velocities and displacements are then integrated by this DSP Controller. Both sets of measurements are shown in Figure 5 and 6. All these structural responses are the sources to calculate the required active control force.

Concluding Remarks

This real-time simulation technique not only alleviates the complexities of traditional verification procedures for active structural control but also raises the possibility to perform parameter uncertainty tests by changing the parameters inside of the simulator. It also provides a platform to verify any control algorithm implemented in digital control system by the integrated verification procedures.

Acknowledgements

2

SLIDING FRAGILITY OF RESTRAINED BLOCK-TYPE NONSTRUCTURAL COMPONENTS UNDER SEISMIC BASE EXCITATIONS
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Introduction

Effects of recent earthquakes have clearly shown that the overall seismic hazard to structures cannot be efficiently reduced unless the design of nonstructural components receive the same degree of consideration as primary structural members. Many buildings that remained structurally sound after the earthquake lost their functionality due to damage to their nonstructural components. While this is always inconvenient and undesirable for any kind of construction, it is certainly unacceptable for critical facilities, such as hospitals, providing life saving functions during and after an earthquake. Besides, the cost of damage to nonstructural components greatly exceeded in many cases the cost of damage to structural members, particularly in the case of moderate earthquakes causing little or no structural damage.

The former observations indicate in a straightforward way that there is a need to investigate the seismic behavior of nonstructural components in order to assess their vulnerability under seismic events. Because the level of expected damage is in most cases higher than that usually considered acceptable, there is also a need to identify and evaluate potential retrofit measures that could be implemented in order to improve the seismic performance of this type of components.

Scope of this Study

Nonstructural components are all components of a building other than those considered to perform primary structural functions. They include mechanical and electrical equipment, architectural elements and building contents. This research aimed to evaluate the seismic response of block-type nonstructural components in direct contact with a horizontal supporting surface. Block-type components are all nonstructural equipments whose behavior is essentially that of a rigid body, and as such can then be appropriately modeled.

Nonstructural components are subjected to damage during earthquakes either directly due to ground shaking or indirectly due to movement of the supporting structure. In either case, damage is caused by any of three primary effects. These are inertial or shaking effects on the components

themselves, distortions imposed on components when the building structure vibrates, and separation or pounding at the interface between adjacent structures. This research focused on evaluation of the seismic performance of block-type nonstructural components subjected to damage caused by inertial or shaking effects only. Furthermore, there are a number of possible kinds of response of this type of equipments when subjected to inertial or shaking effects. In this study, only the sliding response was of interest.

Block-type nonstructural components resting on a horizontal supporting surface might be either restrained or unrestrained. The sliding fragility for the unrestrained case was already investigated by Chong and Soong (2000). Therefore, this study concentrated on the sliding response of restrained devices. In an attempt to account for all of the many kinds of restraint schemes actually found in real case scenarios, the general representation of restrained equipments shown in Figure 1 was considered. It consists of a rigid block restrained by four symmetrically arranged cables. It was assumed that these cables are post-tensioned, as it is usually the case in practice. Both horizontal and vertical accelerations were considered as base excitations.

Any analytical assessment of the dynamic response of any kind of component under seismic accelerations is affected by uncertainties in material properties, modeling assumptions and characteristics of the excitations. An increasingly popular approach to account for these and other uncertainties consists of developing the so-called *fragility curves*. A fragility curve is a plot indicating the probability of occurrence of a given limit state for different levels of seismic base excitation. In this study, fragility curves were developed for the sliding response of the block-cable system shown in Figure 1. Two limit states were considered: breakage of the restraining cables and excessive absolute acceleration. These limit states represent two common failure modes of restrained nonstructural components.
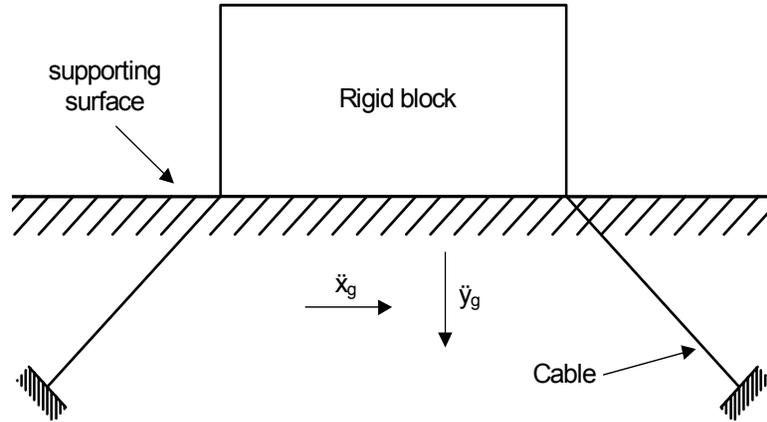
a) Elevation
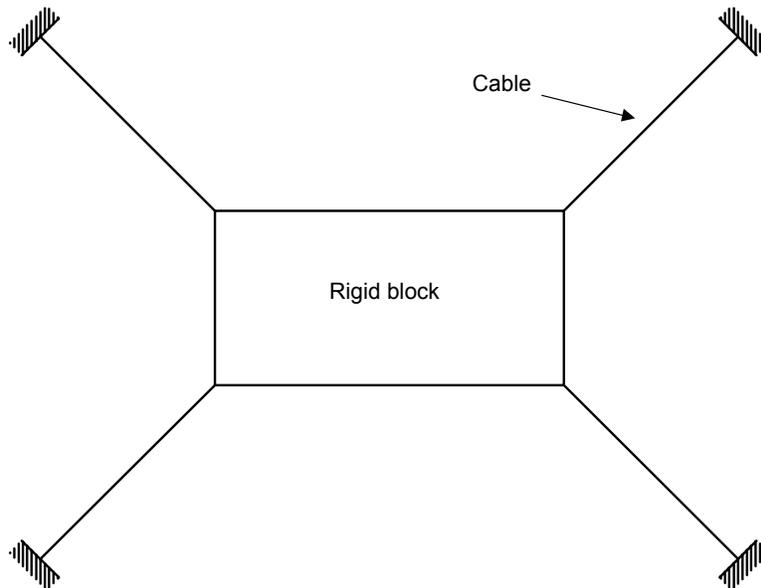
b) Plain view

FIGURE 1: Rigid block attached by cables

Approach of Research

Conditions under which the response is only of sliding nature were investigated. It was found that sliding response occurs only for a certain range of values of the width-to-height ratio of the block. Otherwise, other types of response occur. The equation of motion of the restrained block under sliding response was then developed. Because magnitude of displacements of restrained blocks under the failure modes considered is very small, it was found that forces imposed on the block

by the cables could be assumed as linearly dependent on displacements with negligible error. Thus the only non-linear term in the equation of motion is the term representing the friction force at the block-surface interface. Coulomb-type friction was assumed.

Fragility curves were developed through Monte-Carlo simulations. Ninety synthetic base acceleration time histories were generated using the program SIMQKE (Vanmarcke et al., 1976). They were generated in such a way that their corresponding response spectra match the general spectral shape defined in the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (BSSC, 1998). The acceleration histories were scaled to horizontal peak base accelerations (HPBAs) ranging from 0.10 g to 1.00 g with 0.10 g increments. Due to the many difficulties in assessing the relationship between vertical and horizontal accelerations, vertical accelerations were assumed as proportional to horizontal accelerations.

It was found that the cable-breakage limit state could be defined in terms of a displacement threshold indicating the displacement level at which cables break. The displacement threshold is uniquely defined by some of the parameters involved. The excessive absolute acceleration limit state is obviously defined by an absolute acceleration threshold. However, this threshold depends on the intrinsic nature of the device under consideration and has nothing to do with its dynamic properties (i.e. it does not depend on the parameters of the equation of motion).

Summary of Results

The equation of motion developed as explained in the former section indicates that, for a given base acceleration history, the response of the restrained block depends on four parameters: the dynamic coefficient of friction (μ_d), the vertical-to-horizontal-acceleration ratio (k), the would-be natural period of the system in absence of friction (T_{eq}) and the ratio of the vertical component of the cable forces to the weight of the block (β). In order to get more insight into the influence of these parameters, the response of the restrained block was obtained for different values of the parameters involved. It was found that displacements are of very small magnitude and that maximum absolute accelerations are almost always equal to or greater than the HPBA. Examples of average peak responses can be seen in Figure 2.

$\mu_d = 0.10 \quad - \quad T_{eq} = 0.20$ sec

$\mu_d = 0.40 \quad - \quad T_{eq} = 0.05$ sec
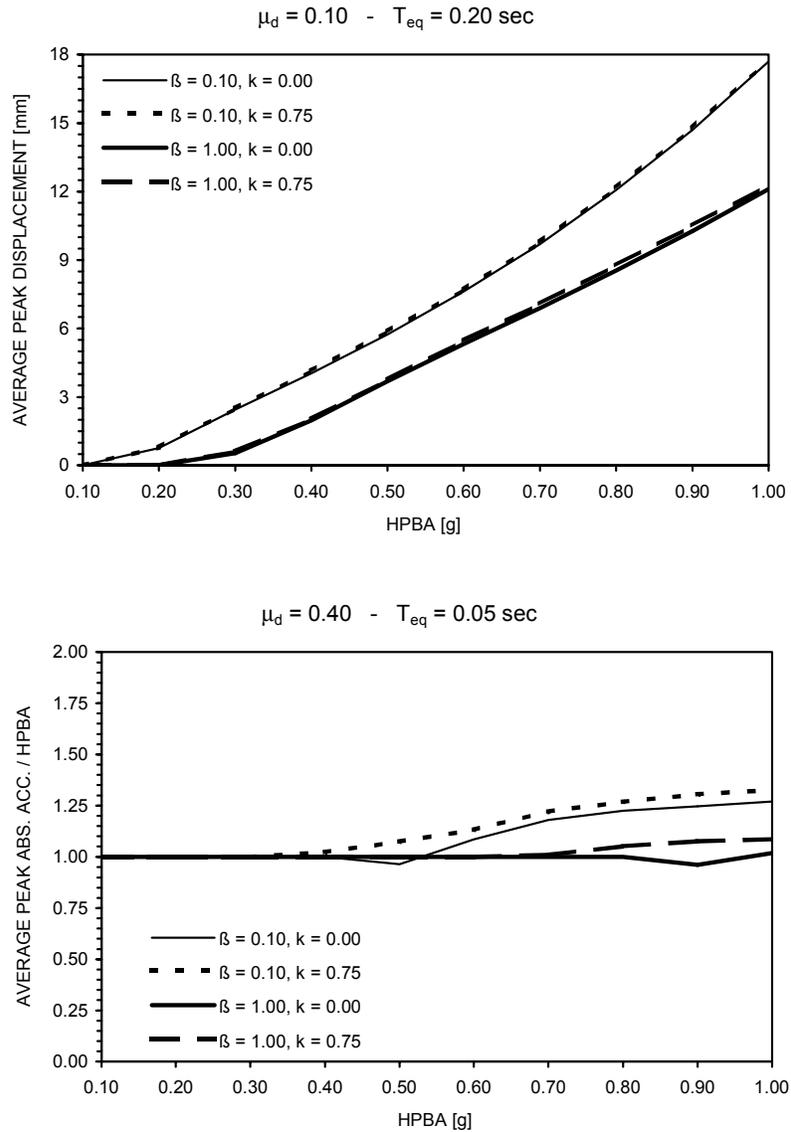
FIGURE 2: Average peak displacements (top) and average peak absolute accelerations (bottom) response for selected values of the parameters of the equation of motion.

Fragility curves were also obtained for different values of the parameters involved. In the case of the cable-breakage limit state, it was found that the probability of failure depends strongly on μ_d and β, and that influence of k and T_{eq} becomes important only for relatively large values of μ_d. In this latter case, fragility curves for $k = 0$ are always below those for $k \neq 0$. In other words, fragility assessments ignoring vertical accelerations are unconservative. In the case of the excessive

absolute acceleration limit state, it was found that the corresponding fragility curves depend strongly on μ_d and only marginally on the other parameters. The interval within $0 < \text{fragility} < 1$ is very narrow and close to the vertical line passing through the HPBA equal to the absolute acceleration threshold. Fragility curves shown in Figure 3 were obtained assuming that the magnitude of the initial tensile forces in the cables are equal to 2/3 the magnitude of the ultimate tensile strength. Absolute acceleration threshold is equal to 0.50 g.

FIGURE 3: Examples of fragility curves for the cable-breakage (top) and excessive absolute acceleration (bottom) limit states.

Conclusions

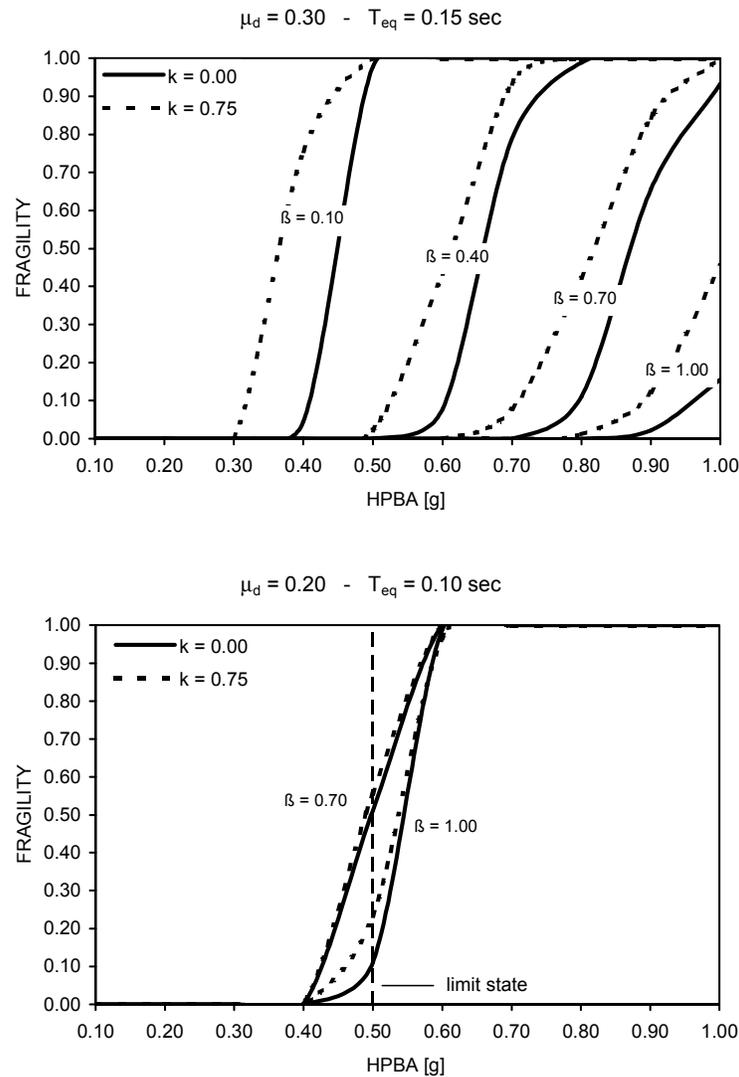Results of this study indicate that restraints are very effective in reducing horizontal displacements, but at the expense of absolute accelerations that are almost always equal to or greater than the peak base acceleration. Therefore, restrained schemes are the obvious choice for block-type nonstructural components whose failure mode is given by excessive horizontal displacements. In the case of components whose failures modes are given by both excessive displacements and excessive absolute accelerations, restraints can also be used as long as the absolute acceleration threshold is larger than the expected peak base acceleration. In these cases, the controlling failure mode is given by breakage of the restraints. Post-tension forces, which significantly influence the corresponding fragility curves, can then be conveniently selected so that the resulting probability of failure is low enough. In order to avoid fragility estimations that might become significantly unconservative, vertical accelerations must be taken into account.

Acknowledgements

References

BSSC (1998), NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Report No. FEMA 302, Washington D.C.

Chong, W. H. and Soong, T. T. (2000), Sliding fragility of unrestrained equipment in critical facilities, Technical Report MCEER-00-0005, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY.

Vanmarcke, E. H., Cornell, C. A., Gasparini, D. A. and Hou, S. N. (1976), SIMQKE: A Program for Artificial Motion Generation – User's Manual and Documentation, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, MA.

3

ESTIMATING INDIRECT ECONOMIC LOSSES FROM ELECTRICITY LIFELINE DISRUPTION FOLLOWING A CATASTROPHIC EARTHQUAKE IN MEMPHIS, TN (USING A CGE MODEL, SURVEY & SIMULATIONS)

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Abstract

The research objective was to develop an economic model that simulates the total economic impact of lifeline losses in the event of large magnitude earthquakes in Memphis, Tennessee. We obtained economic estimates of potential losses to sectoral output arising from a 1 week outage of electricity, considering adaptive responses, based on survey / simulation results of actual business loss expectations. The new methodology isolates electricity lifeline losses, incorporates resiliency of economy and estimates indirect impacts as the difference between recalibrated total (general equilibrium) impacts and the direct impacts shown by the survey. Simulations have been run for various socio-economic scenarios that are plausible in the wake of an earthquake, like a price hike, priority supply to households, etc.

The Problem

In a purely anthropogenic sense, an earthquake may be considered as the joint interaction of an extreme natural event and a human settlement. They differ in terms of duration, intensities, and the nature of devastation they leave behind. Media reports of a natural disaster usually reflect only the directly visible damage. Losses, however, extend beyond what meets the eye, and extend beyond the actual duration of ground-shaking. *Indirect economic losses* from business disruption, due both to the loss of housing and lifeline services, may often be several *multiples* of the initial estimates of direct damage (see for e.g., Boisvert, 1992; Brookshire et al., 1997; Cochrane, 1997).

Interest in modeling these issues arise out of the increasing human susceptibility and scale of damages, and also the potential for impacting a wider area due to strategic linkages existing beyond the regional economy. There are several modeling techniques that may be employed to capture the direct, indirect and institutional effects of an earthquake, ranging from Input-Output Analysis, Simulation Modeling to Computable General Equilibrium (CGE) Modeling.

This research is motivated by the twin objectives of examining a region with a history of a major earthquake event in recent times, and that of employing a sophisticated modeling technique that can best capture the economic impacts of an external stimulus.

The Setting

The *New Madrid Seismic Zone* located near Memphis, Tennessee served as the venue in 1811-12 for the largest earthquake events, of around 8.5 scale, in the U.S. At that stage, the estimated losses were quite modest. However, such an event today could be very expensive in terms of human and economic losses, even after discounting the differences in estimation technologies, because the area is now densely populated and industrialized. Memphis is now a city of 295 Sq. miles and houses close to a million people and sits on the intersection of several lifelines like oil and gas pipelines and electric gridlines, as well as an important center for transportation and distribution (Shinozuka et al., 1998). This fact has prompted previous research in the Memphis area, and is also the basis for the current research. The major difference in this essay is the use of a CGE model for the Memphis economy as opposed to the earlier treatments using Input-Output methodologies.

The CGE model

The computable general equilibrium [CGE] approach is an excellent framework for regional impact assessment since it allows behavioral response simulation. Using this approach, it is possible to analyze disjointed changes and incorporate engineering information. CGE models may beconstructed for any desired level of sectoral disaggregation, since it allows the use of multisector, multi-tier production functions. The model is driven by markets & price signals, which makes it a meaningful policy instrument. Our adaptation of the implementation mechanism of the CGE makes it possible to model resiliency features of the economy, thus reflecting the real world economic conditions (see, for e.g., Scarf, 1973; Partridge et al., 1997).

The CGE framework is a suitable device for tackling the issue of responses to both economic and non-economic but quantifiable exogenous stimuli, since it can suitably map the changes in sectoral equilibria within a region. Since it allows for *input selection*, it is a fairly reasonable simulation of the real economy, and hence it appears to be a good choice for analyzing natural hazard issues. The other favorable features of the CGE model are that it allows for flexible specification of production technology and consumer preferences, and the fact that it uses prices to drive the system's responses to external stimuli. In essence this allows for great choice in terms of functional forms to represent technology and utility. Thus a CGE model is able to reflect the

market's behavioral adaptations, which are, expected responses to natural hazards. The CGE model also allows for a user defined sectoral scheme, thus representing a choice in the levels of aggregation. We have opted for a 20-sector aggregation scheme, following an earlier analysis (Rose et al. 1997): "Agriculture, Mining, Construction, Food Products, Manufacturing, Petroleum Refining, Transportation, Communication, Electric Services, Gas Distribution, Water & Sanitary Services, Wholesale Trade, Retail Trade, Finance, Insurance & Real Estate (FIRE), Personal Services, Business & Professional Services, Entertainment, Health, Education, Government".

We employ nested CES production (KLEM) functions and Cobb-Douglas utility functions. Trade is reflected in 2 layers (Rest of the World & Rest of US) and is defined by CES (Armington) functions for Imports and CET functions for Exports. The Keynesian closure rule, which adopts a fixed wage-rate and allows for unemployment has been used to define the labor market. Benchmark data for the model includes the Social Accounting Matrix (SAM) for 1996, available from the IMPLAN System, substitution elasticities received from other studies, and primary survey / simulation results (Chang, 2000).

Innovation in Methodology

The methodology used herein has been motivated by 2 developments. First is the availability of simulated information on direct impacts from Chang based on an earlier survey data from Tierney. Second, is the development of the idea of combining CGE models and primary data to extract information on indirect effects (Chang et al., 2000). Broadly, the steps involved in incorporating resiliency & estimating indirect effects are:

∗   generating *direct* losses resulting from water outage based on given technology (CES production functions) for each sector, in Partial Equilibrium (PE) framework,

∗   making parametric adjustments (following a logic for associating various resiliency measures with different CES parameters) such that PE estimates equal Chang's,

∗   incorporating the re-evaluated CES parameters into MPSGE and solving for GE effects

∗   the difference (iii – ii) being the *indirect* impacts.

The New Approach

The overall structure of the CGE model may be viewed as having major *blocks* of equations representing the *actors* in an economy, namely, the producers, the consumers, the owners of factors and the government. There are equations that ensure consistency in the activity *levels* of these actors, for e.g., demand-supply parity, investment-savings parity etc. There are also

balancing equations that *close out* the model, for e.g., the government's budget, and the labor market clearance (Rose et al., 2000)

The standard solution procedure is to calibrate the parameters in the model based on a *snapshot* of the actual economy at a point of time, which is given by a *social accounting matrix*, which is a summary record of all transactions in the economy in value terms (see for e.g., Rutherford, 1995).

Having done that, a *counterfactual simulation* is run by changing the value of an economic variable which forces all other variables to take on new values. The difference is known as the regional economc impact (Rose & Guha, 2000). Our advance to the methodology is to use survey data to recalibrate the model, thereby generating a set of parameters whose values include the resiliency features within an economy. The actual procedure involves the following steps:

1. extract the production block (sectoral production functions)
2. compute preliminary impact case with given water outages
3. compare with empirical data on direct effects
4. recalibrate model parameters making sectoral results consistent with data
5. reinsert calibrated production function into CGE model
6. run CGE model with water outages & new parameters
7. CGE results = total effects (direct + indirect)
8. estimate "indirect" effects using steps 7 & 3

Alternative Conditions

Several cases and sub-cases were formulated based on a combination of elapsed time, and restrictions that may be imposed on the economy. The restrictive cases are:

∗ No restrictions
∗ Fixed Electricity Prices
∗ Minimum Supply to Households
∗ Fixed Price plus Minimum Household Supply

Each of these were evaluated for sub-cases representing time elapsed since the earthquake.

i)   Base Case: reduce capital stock of electricity sector to reflect supply loss.

ii)   Very Short Run (VSR < 7days): additionally, reduce substitution elasticities to reflect "shock" (the system is "paralyzed").

iii)   Short Run (SR < 6 months): increase elasticities to 50% of their normal values to show some recovery (systems have been partially restored), and to reflect resilience.

References:

Boisvert R. 1992. "Indirect Losses from a Catastrophic Earthquake and the Local, Regional, and National Interest." In *Indirect Economic Consequences of a Catastrophic Earthquake*. Washington, DC: FEMA.

Brookshire DS, Chang SE, Cochrane H, Olson RA, Rose A and Steenson J. 1997. "Direct and Indirect Losses from Earthquake Damage," *Earthquake Spectra*, 13(4).

Cochrane H, et al. 1997. "Indirect Economic Losses," in *Development of Standardized Earthquake Loss Estimation Methodology Vol. II*, Menlo Park, CA: RMS, Inc.

Partridge MD and Rickman DS.1997. "Regional CGE Modeling: a Survey and Critical Appraisal," *Southern Regional Science Meetings, Memphis, TN* Apr.97).

Chang S, Rose A, Shinozuka M, and Tierney K. "Modeling Earthquake Impacts on Urban Lifeline Systems: Advances in Integration," presented at the *US-China Millenium Symposium on Earthquake Engineering*, Beijing, China, November 2000.

Rose A and Guha GS. "Computable General Equilibrium Modeling of Electric Utility Lifeline Losses from Earthquakes," presented at the *Southern Economic Association Conference*, Washington DC, November 2000.

Rose A, Liao S, and Guha GS. "Estimating the Economic Impact of Climate Change on the Mid-Atlantic Region: A CGE Analysis of Forest Sector Damages," presented at the *International Symposium on Economic Development* in the 21st Century, Mumbai, India, July 2000.

Rose A, Benavides J, Chang S, Szczesniak P and Lim D. 1997. "The Regional Economic Impact of an Earthquake: Direct and Indirect Effects of Electricity Lifeline Disruptions," *Journal of Regional Science* 37: 437-58.

Rose A, Carmichael J, and Oladosu G. 1999. "Modeling the Economics of Natural Hazard Impacts and Policy Responses Using Computable General Equilibrium Analysis," The Pennsylvania State University, University Park, PA.

Rutherford TF. 1995. "Linking IMPLAN Social Accounts to MPSGE: an Overview of the Code," (*mimeo) Dept. of Economics, University of Colorado.*

Scarf H. 1973. *Computation of General Equilibrium*. Yale University Press.

Shinozuka M, Rose A, and Eguchi R. (eds.). 1998. *Engineering and Socioeconomic Analysis of a New Madrid Earthquake*, Buffalo, NY: MCEER.

4

DEVELOPMENT OF ENGINEERED CEMENTITIOUS COMPOSITE MATERIALS FOR SEISMIC STRENGTHENING AND RETROFIT APPLICATIONS

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Introduction

One of the primary missions of the MCEER project is the development of new and emerging materials that have potential for use in seismic rehabilitation of structures. These newer materials may possess significant advantages over conventional civil engineering materials such as reinforced concrete and steel. Ductile cement-based composites such as ECC (Li, 1998) are an example of a new generation engineered material that offers advantages for retrofitting such as energy absorption capacity, excellent tensile as well as compressive strength, the ability to be cast in various shapes, and the ability to be connected by bolts, welds or with grout. These materials are unique in that they exhibit a "pseudo strain hardening" response in tension. Figure 1 shows a comparison of the tensile response of ECC compared to a traditional cementitious mortar. It is believed that these materials have tremendous potential in seismic strengthening and rehabilitation applications. Development of these materials has involved laboratory testing to establish fundamental behavior characteristics, and additional testing to verify the performance of components made with the composite materials. Concurrent with the laboratory testing, material models are being developed to allow for finite element based simulations of the material performance in structural behavior simulations.
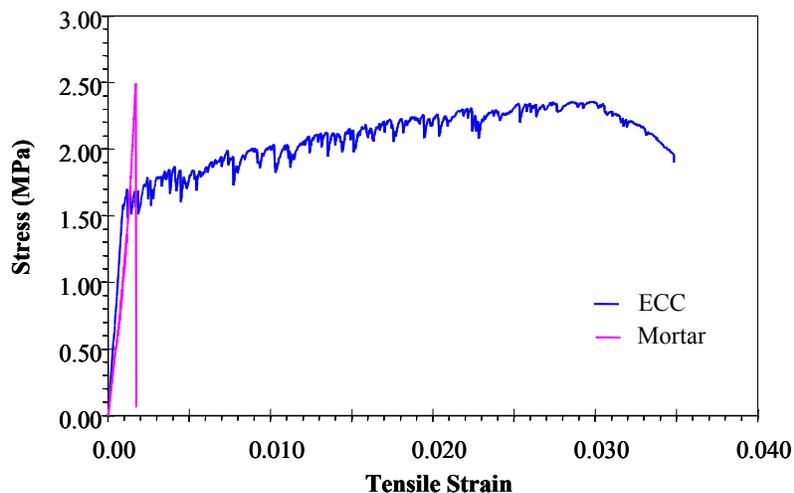

Figure 1 – Comparison of tensile response of ECC and mortar.

This research will explore the applicability of engineered ductile cement-based composites to the retrofit of both steel and concrete structures. Assessment of the material will be made with a combination of experimental and numerical studies to verify both structural performance and constructability.

Experimental studies will focus on characterization of the material and its ability to improve strength, stiffness and ductility of an existing structure. Applications to be explored include using the material as infill panels connected using simple steel tab connections (Figure 2a) and as a ductile grout between concrete panels (Figure 2b). These proposed applications build upon previous retrofit concepts (Frosch et al., 1996 and Kanda et al., 1998). Numerical studies using properties gained from material characterization and joint connection experiments will be used to analyze and evaluate the performance of each retrofit strategy in the demonstration hospital projects.

a – Infill panel system   *b – ECC grout*
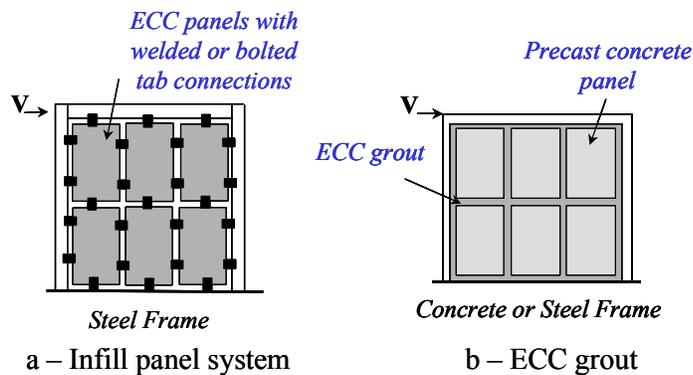
Figure 2 – Schematic representation of structural retrofit concepts with ECC material.

Laboratory Testing

One of the primary goals of the current research is to develop an improved constitutive model for the cyclic behavior of ECC. A laboratory-testing program is currently being used to evaluate the effects of different fiber types, and constitutive materials on the properties of ECC, and to develop a material model for ECC from the results of small-scale tests. The development of an improved model will allow for finite element based simulations of ECC panels in strengthening and retrofit applications. In the following sections some of typical test results from a testing program designed to evaluate the cyclic response of ECC will be discussed.

Experimental Results

Laboratory tests are being conducted on small-scale specimens to evaluate different aspects of ECC material behavior. Figure 3 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 in testing.

Figure 3 – Experimental set up for cyclic testing of ECC materials.
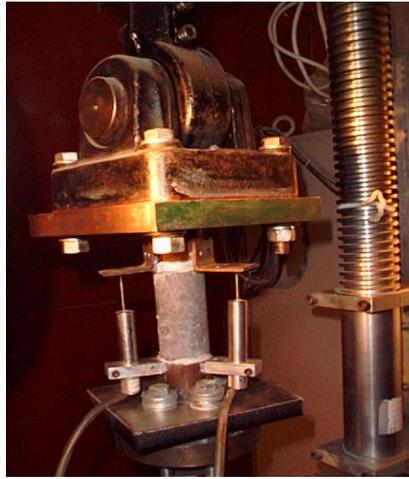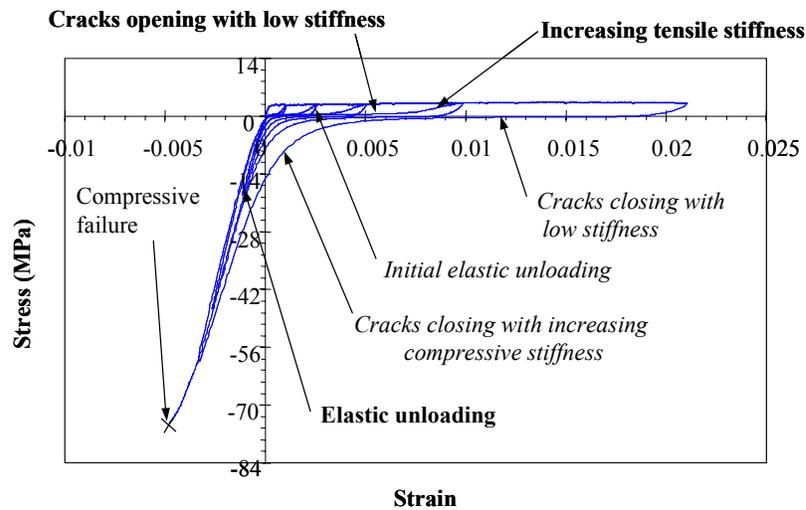

Figure 4 – Results from cyclic tension – compression test, cylindrical specimen.

Figure 4 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 (Yankelevsky and Reinhardt, 1989). 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 commonly used in structural analysis. In Figure 4, 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 4). 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.

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.

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. (1998). This scheme offers advantages of being portable and allowing individual panels to be replaced or removed as needed. Figure 5 shows a schematic representation of the proposed connections.
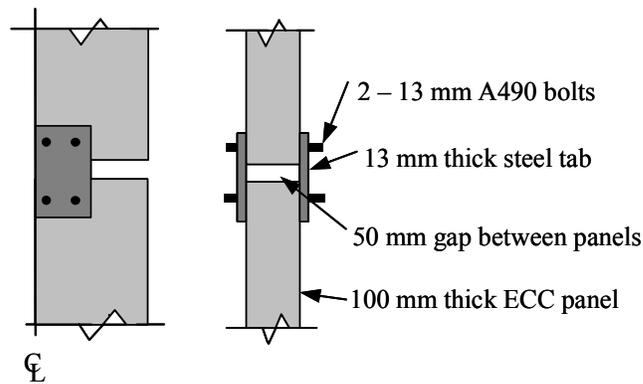
Figure 5 – Schematic representation of panel connections.

Preliminary Analysis

The current investigation focuses on developing a strengthening scheme for the steel framed structures, as shown in Figure 1(a). To strengthen/stiffen the frame, ECC panels are added as an infill wall system as shown in Figure 5. 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. Preliminary analysis indicates that the infill panel system can be used as a seismic retrofit strategy to strengthen and increase the energy dissipation in 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.

Significance of Research

The ongoing development of ECC materials for seismic strengthening and retrofit applications has several unique aspects both in terms of the material and the applications under development. As discussed in the preceding sections, ECC materials offer several advanges over conventional materials including superior energy absorbtion, and high tensile strain capacity while retaining the traditional advantages of cementitious materials. The precast panel system under development utilizes the unique features of the ECC material. The connections between panels consist of steel tabs and high strength bolts. This type of connection allows for easy of assembly and flexibility. If needed, the panel walls can be easily removed or relocated. The completion of the research will provide an effective, flexible strategy for the seismic retrofit of critical facilities.

References

Li, V.C., (1998) "Engineered Cementitious Composites – Tailored Composites Through Micromechanical Modeling," in *Fiber Reinforced Concrete: Present and the Future*, ed. N. Banthia, A. Bentur, and A. Mufti, Canadian Society for Civil Engineering, Montreal, pp. 64-97.

Frosch, R., et al. (1996) "Retrofit of Non-Ductile Moment-Resisting Frames Using Precast Infill Wall Panels," *Earthquake Spectra*, Vol. 12, Number 4.

Kanda, T., Watanabe, W., Li, V.C., (1998) "Application of Pseudo Strain hardening Cementitious Composites to Shear Resistant Structural Elements," *Fracture Mechanics of Concrete Structures*, Proceedings of FRAMCOS-3, AEDIFICATIO Publishers, Freiburg, Germany, pp 1477-1490.

Yankelevsky, D.Z., and Reinhardt, H.W., (1989), "Uniaxial Behavior of Concrete in Cyclic Tension," Journal of Structural Engineering, Vol. 115, No. 1, Jan.-Feb.

5

OPTIMAL DESIGN OF ADDED ENERGY DISSIPATION DEVICES BASED ON BUILDING PERFORMANCE INDICES

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Background

Using supplemental energy dissipation devices (EDD) for seismic retrofit of buildings has become more and more popular. In particular, after the Northridge and Kobe earthquakes, EDD has been accepted as alternative for seismic retrofits in the U.S. and Japan. It has been implemented in building and bridges to reduce the seismic response and to enhance the performance of the system during earthquakes.

Since the EDD are introduced to the market by the individual manufacturers, and EDD implementations are relatively new in the civil engineering field, there is not a comprehensive design procedures for structures using such supplemental devices. And the only preliminary guidelines in FEMA 273/274 are based on characteristics of devices. They provide the device-based approach with the structure system simplified to a single degree of freedom system. The damping of the system is simplified into the damping ratio of the first system dynamic mode. Also, the effect of non-proportional damping is neglected. These simplifications speed up the design procedures, but this approach may not always obtain the right response.

A structure-based approach with MDOF system model analysis is suggested for structural design when using added EDD as a strategy to reduce the earthquake responses. The investigation presented in this paper is about design approach of using bracing type EDD on buildings for optimal response under different performance requirement, which could be represented by a certain value on one of the performance indices.

Literature Review

FEMA273/274 provides a preliminary guideline for application of seismic protective systems to building rehabilitation based on rehabilitation objectives, building performance levels and seismic ground shaking hazard.

The guideline specifies four analysis methods for building with added EDD, the linear static, linear dynamic, nonlinear static and nonlinear dynamic procedures. The linear procedures are the

most common used methods. They require modifying the earthquake load on the building with the consideration of the added damping, in the form of the estimate effective damping. The effective damping is usually formulated by applying device on a single degree of freedom (SDOF) system, and taken as the damping ratio of the first mode. This often oversimplifies the complex structure-device interaction.

To address the effect of configuration of energy dissipation device on the structural dynamic behavior under earthquake excitation, there are some limited existing studies. Chang et al., (1993) investigated the seismic behavior of steel frame structure with added VE damper and gave some design guidelines based on his parametric study. Hahn and Sathiavageeswarm (1992) showed that dampers should be placed to the lower half floors of the building if it is of uniform story stiffness. Many studies use complicated mathematical formulation to optimize the performance index base on optimal control theory. Constantinou and Tadjbakhsh (1983), Gurgoze and Muller (1992), N. Gluch, A. M. Reinhorn (et al., 1996), and De Silva (1981) proposed different methods to derive the optimum damping coefficient for one particular damper placed on the building under random earthquake ground motion. Spencer, Suhardjo (et al., 1994) proposed frequency-domain approach to design the optimal active control force. There are also sequential approaches to obtain the optimal placement and capacity of the devices. Zhang and Soong (1992), and Wu, Ou and Soong (1997) proposed and extended a seismic design method to find the optimal configuration of viscous dampers for a building with specified story stiffness. This method has the intuitive criterion that additional damper should be placed sequentially on the story with the maximum inter-story drift. Shukla and Datta (1999) also use the controllability index to find the optimal locations of passive damper sequentially. Tsuji and Nakamura (1996) proposed algorithms to reduce objective function step by step.

Among all the above studies, none of them address the performance requirements sufficiently so that engineers can design a best configuration based on particular performance index of a particular building.

Proposed Methodology

In order to investigate the effect of bracing type EDD on buildings, both theoretical formulation and numerical simulation will be used. The general effect of additional damping from the added device will be studied by applying the damping matrix into the original equation of motion. The dynamic property of the original structural system will be changed, including the natural frequencies, mode shapes and modal damping ratios. Since the added damping device will often bring non-proportional damping into the system, the dynamic characteristics can not be solved by

using orthogonal property of the mode shapes so that the MDOF system can be transformed into a set of SDOF modal systems. A mathematically equivalent state space method will be used to obtain the structural characteristics and seismic response.

Several typical buildings will be modeled in commercially available finite element software. The mathematic model should include enough detail of the structure, and be checked with the design documentation and design standard to make sure it is an accurate model of the building structure. After the important characteristics of the structure are obtained, this detailed model is simplified into a smaller model, with mass lumped at important positions on the building. The new lumped mass will have less degree of freedom and it will have the same dynamic properties. Then the device configuration is applied on the simple model, which represents the original structure closely and enables more efficient computation. The effect of different device is then analyzed.

It is expected that the seismic response reduction is depending on the structure and its performance requirement, which could be story drift, displacement, acceleration or element force. For the device configuration with optimal seismic response reduction, the structure has to be analyzed carefully first. Then, based on its characteristics and the defined performance index, an optimal device distribution will be generated. The optimization algorithm will search the best device configuration from all the possible device location and combinations based criteria, such as minimum response under one particular earthquake ground motion, under design spectra compatible earthquakes, or under a varied range of earthquakes.

Current status

Several simple MDOF systems have been studied to understand the effect of added non-proportional damping to the system. There are also complicated buildings analyzed to show that the cost effective device distribution is very much depending on the structure configuration and the building performance indices.

To develop the search algorithm for the optimal device configuration, a lot of efforts have been spent on the computation efficiency issues. Because the structure model is always complicated, although we can reduce it degree of freedom for less computation, the dynamic numerical method will still take a period of computation time to get the response for each device distribution. Considering a lot of device possible locations and combinations, it is important to have an algorithm which can compute the response fast and another algorithm to find a global or local optimal in a huge solution space.

Contribution

Currently, the most common retrofit design procedure using EDD is to specify a damping ratio first before the design. Next, the devices are added to the structural system to increase the damping ratio to the specified value. The structure response is then checked for reduction. By this procedure, it is possible to design a device configuration with acceptable building seismic response, but the design may not be optimal. Sometimes, due to the oversimplification of the original system, the neglected higher mode might play an important role in the structural response. The result might be undesirable, for example, some of the response quantities will be larger after adding the damping device, or some local response will not be reduced.

With the proposed guideline in this research, the structure dynamic properties will be analyzed carefully. With the consideration of different mode and different mass points, and their contributions to the particular building performance indices, the simplification will keep the important information about the structure, and the response calculated will closely reflect the accurate response after the added devices.

Further more, with a search algorithm which generates an optimal device configuration based on the building model and specified performance objectives, the practicing engineers will be able to design the building retrofit strategy more efficiently. A lot of trial and error process can be saved.

Results

It is found that retrofitting design of using EDD is not a straightforward matter. There is no rule which can be applied to any type and configuration of structure. The structure dynamic characteristics need to be analyzed carefully. For the particular response quantities which need to satisfy the performance objective of the design, the modes contributing most to this response need to be analyzed as well, so that the most effective device location is found and the devices are placed at those positions.

The largest added damping will not guarantee the best response reduction. The common practice of EDD design procedure, which trys to maximum effective damping, will not always lead to cost effective design. A structure-based design procedure is developed as follows:

1. Structure analysis and simplification

2. Select performance index (displacement, story drift, velocity, acceleration or combind response)

3. Define the constraint of adding passive system

4. Search for optimal configuration of the added device. The search is based on the following rules:

   a. Increase model damping for a particular mode

   b. Modify a particular mode shape

   c. Modify the total modal contribution to a particular mass point

5. Validation

Future work

After the theoretical basis and the optimization algorithm are available, a software package can be developed so that the design engineers can easily use it to obtain optimal design.

The current computation codes are written in the conventional procedure-oriented approach, it is very hard to understand, and it is hard to maintain. Whenever there are requirement changes, there will be trouble of recoding. If we implement object-oriented (OO) approach in this software design, we will have better reusability and extensibility. With the variety of performance objectives and the variety of optimization criteria, OO approach and design patterns will provide a good structure of a reusable system.

Reference

Chang, K. C., Soong, T. T., Oh, S.-T. and Lai, M. L., 1995 Seismic behavior of steel frame with added viscoelastic dampers, *Journal of Structural Engineering*, 1418-1426, October 1995.

Constantinou, M. C. and Tadjbakhsh, I. G., 1983, Optimum design of first story damping systems, *Computer and Structures*, Vol. 17, No. 2.

FEMA-273, October 1997, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*.

FEMA-274, October 1997, *NEHRP commentary on the guidelines for the Seismic Rehabilitation of Buildings*.

Gluck, N., Reinhorn, A. M., Gluck, J., Levy, R., 1996, Design of supplemental dampers for control of structures, *Journal of Structural Engineering*, 1394-1399.

Gurgoze, M. and Muller, O. C., 1992, Optimal positioning of dampers in multi-body systems, *Journal of Sound and Vibration*, **158(3),** 517-530.

G. D. Hahn and K. R. Sathiavageeswarm, Effects of Added-damper Distribution on the Seismic Response of Buildings, Computer and Structures, Vol. 43, No. 5, 1992

Lai, M. L., Chang, K. C., Soong, T. T., Hao, D. S. and Yeh, Y. C., 1995. Full-scale viscoelastically damped steel frame, *Journal of Structural Engineering*, 1443-1447, October 1995.

Lee, G. C., Tong, M. and Liang, Z. 1996, A damping criterion for distributed parameter systems and its application in damping indentification, *The Chinese Journal of Mechanics*, 12(1), 15-24.

Shukla, A.K. and Datta, T. K., 1999, Optimal use of viscoelastic dampers in building frames for seismic force, *Journal of Structural Engineering*, 401-409.

De Silva, C. W., 1981, An algorithm for the optimal design of passive vibration controllers for flexible systems, *Journal of sound and Vibration*, **74(4),** 495-502

Spencer Jr., B. F., Suhardjo, J. and Sain, M. K., 1994, Frequency Domain Optimal Control Strategies for Aseismic Protection, Journal of Engineering Mechanics, **120(1).**

Suhardjo, J., Spencer Jr., B. F. and Kareem, A., 1992, Frequency Domain Optimal Control of Wind-excited Buildings, *Journal of Engineering Mechanics*, **118(12)**.

Tong, M., Liang, Z. and Lee, G. C., 1994, An Index of Damping Non-proportionality for Discrete Vibration Systems, *Journal of Sound and Vibration,* **174(1**), 37-55.

Xu, Y. L., He, Q. and Ko, J. M., 1999, Dynamic response of damper-connected adjacent buildings under earthquake excitation, *Engineering Structures*, **21**, 35-148.

Bo Wu and Jin-Ping Ou, T. T. Soong, 1997, Optimal Placement of Energy Dissipation Device for Three Dimensional Structures, *Engineering Structures*, **19**, 113-125.

Ri-Hui Zhang and T. T. Soong, 1992, Seismic design of viscoelastic dampers for structural applications, Jou*rnal of Structural Engineering*, **118(5)**.

6

FIBER REINFORCED COMPOSITES FOR ADVANCED SEISMIC PERFORMANCE OF WATER SUPPLIES

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Background

Failure of water supply systems due to earthquake loading adversely impacts the locally affected community at a time of greatest need. The water supply system is the first line of attack in arresting earthquake induced structural fires. A case in point, the City of San Francisco had to utilize fire boats, positioned along the city perimeter in the San Francisco Bay and outfitted with special pumps, to provide water to combat the Marina District fires after the 1989 Loma Prieta Earthquake. The need of potable water for the people affected by major earthquakes becomes the next heavy demand on this form of infrastructure. If a major pipeline is damaged due to excessive ground movement, the failure of a component could mean the depletion of the system. Some municipalities have strategically placed shut-off valves to isolate localized failures in the system, many do not have these safety measures. The overall system performance can be substantially improved if the system vulnerabilities can be identified and addressed.

This research addresses the major component of structural weakness of steel water supply systems: the pipe joint. The connecting joint between adjacent segments of steel pipe is composed of two basic parts: 1) the cold-formed bell housing, and 2) the spigot end of the pipe. It is known as a welded slip joint. The bell housing is fabricated by the insertion of a steel forming mandrel into the prismatic steel pipe tube. Pressing the flared pipe back onto the mandrel, during fabrication, forms a coupler. During the field installation, the straight spigot end of one pipe is inserted into the bell housing of the adjacent segment of pipe. The attachment is created via a fillet weld along the complete circumference of the connection. When seismic waves travel through the ground, they can engage pipelines. The ground displacement can manifest in the water supply pipeline as a displacement pulse that ultimately loads a bell housing connection. This loading will be non-periodic, and most likely will be full cycle with both compression and tension components. One method of improving the inherently weak bell housing connection is to retrofit the joint with fiber reinforced composites to inhibit radial dilation and local buckling due to the earthquake induced axial strain. This research program investigates the performance of full-scale pipe specimens retrofit in this manner.

Research Objectives

The primary goal of this research is the strengthening of the welded slip joint to inhibit local buckling. This is being achieved with the external application of fiber-reinforced composites (FRC) that are wrapped around the joint region. Epoxy saturated unidirectional carbon fiber cloth is wrapped in both circumferential and longitudinal directions, with the circumferential layers restraining the outward dilation and buckling, in a similar fashion as barrel bands restraining barrel staves. The complete research program includes both full-scale testing and analytical modeling in the 3-D finite element environment. At present, the ultimate end user is the Los Angeles Department of Water and Power (LADWP). The research program includes the direct involvement of two FRC contractors: 1) Master Builders, of Cleveland, OH, and 2) Fyfe Engineering, San Diego, CA with R.J. Watson, Inc., East Amherst, NY.

Research Methodology

The first phase of testing of full-scale 12 inch diameter steel pipe specimens has been in compression only. All specimens in Phase 1 have a length to diameter aspect ratio of 2. The diameter to pipe wall thickness ratio (D/t = 12.75" / 0.25") was 50. The specimens have included both external and internal welded slip joints. The baseline for overall section performance has been prismatic sections of the pipe, i.e., specimens without a welded slip joint. The load history utilized for testing prismatic and jointed pipes were cycled 3 times at loads below the proportion-

12 in. Diameter Steel Pipe Test
INTERIOR WELDED SLIP JOINT
"KOREAN STEEL" TENSILE COUPON TESTS

Proportional Limit = 38 ksi
Yield Stress = 44 ksi
Elastic Modulus = 29,335 ksi
Ultimate Stress (Strain = 14.8%) = 59.5 ksi
Ultimate Strain = 36%
Poisson Ratio = 0.27

Proportional Limit

Ave. Long. Strain

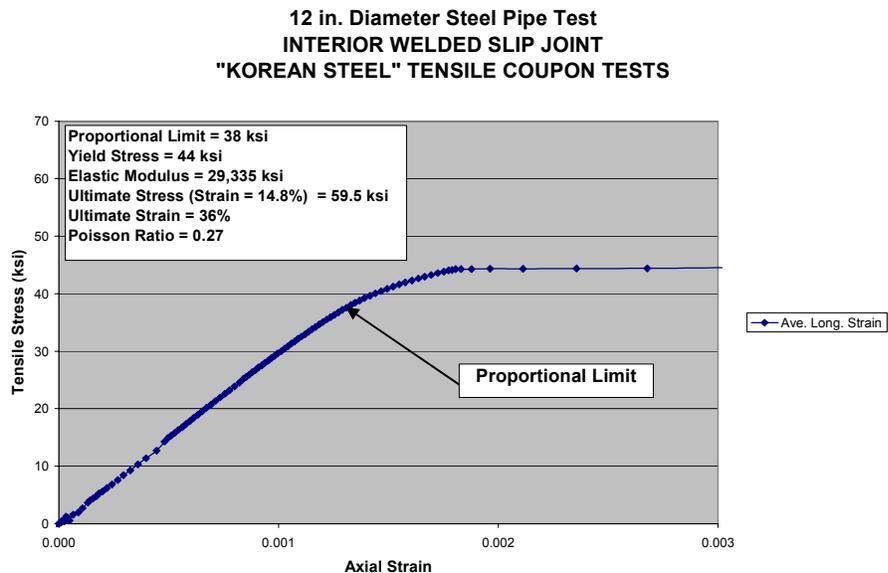Tensile Stress (ksi)

Axial Strain

Figure 1. Tensile Coupon Test of LADWP 12in. Diameter Pipe

al limit of the steel. Then the section was loaded beyond yielding and into large deformations. This load history was used for the as-built specimens, and for the retrofitted specimens. Because of the wide range of acceptable steel type (from 32 to 56 ksi strength steel), as specified by LADWP, tensile coupon tests were conducted for most specimens.

The next series of tests, Phase 2, will be conducted at Taylor Devices, Inc. North Tonawanda, NY, a corporate member of the MCEER consortium. These specimens will all be 6 ft long. The D/t ratio is still 50. The reason for conducting tests at their fabrication facility is that Taylor Devices has constructed a large test frame to proof load the large seismic shock absorbers that they manufacture. This test frame can deliver 1,500,000 lbs force at real earthquake velocities (approximately 70 inches per second) in both compression and tension. Also, real-time earthquake records will be input as the loading regime. The capacity of the load frame at Cornell is 600,000 lbs force, with compression loading only. The second phase will include tensile loading of the full pipe section. These full-scale tension tests to failure will be the first of their kind ever.

The third phase of testing will be of 30in diameter pipe. There are both exterior and interior welded slip joint specimens. The average D/t ratio is approximately 250. These thin walled pipes have a similar D/t aspect ratio of the large diameter pipe that are a possibility for use of this technology. The diameters of some of these pipes are: 48in, 60in, 72in, and possibly 96in, and 102in diameter pipes. The loading will be similar to the Phase 2 load history.

Experimental Results

Figure 2 is representative of the Phase 1 test results. This figure shows the as-built response of the prismatic section (used as the baseline for comparison) compared with the interior and exterior welded slip joints. The initial elastic response of the as-built specimens shows similar elastic stiffness for all three specimens. With the onset of local buckling in the welded slip joint, the two jointed specimens reach a plastic plateau of response with continual degradation of load response.
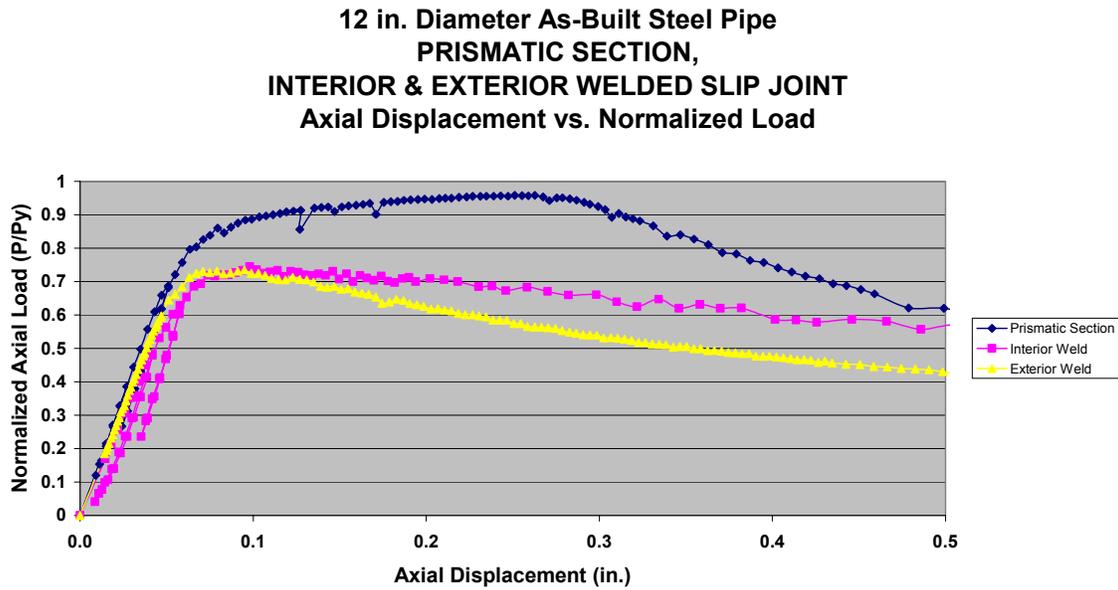
12 in. Diameter As-Built Steel Pipe
PRISMATIC SECTION,
INTERIOR & EXTERIOR WELDED SLIP JOINT
Axial Displacement vs. Normalized Load

Figure 2. As-Built and Prismatic Section Test Results

It has been found that the reduction of capacity of performance due to the welded slip joint was approximately 30% of the prismatic section capacity for the 12" diameter pipe.

Figure 3 shows the improvement of the welded slip joint when retrofitted with FRC wrap. The prismatic section capacity is shown in red. The two vendor retrofitted specimens are shown in Figure 3 as FW_12_W_1, the Fyfe / Watson specimen, and the SPS-MB_12_W_1, the Master Builders / Structural Preservation Inc specimen. The baseline performance of the prismatic section is shown for comparison. The retrofitted specimens achieved approximately 98% of the prismatic section capacity, with similar stiffness throughout the load history.

12 in. Diameter Steel Pipe Test
COMBINED TEST RESULTS
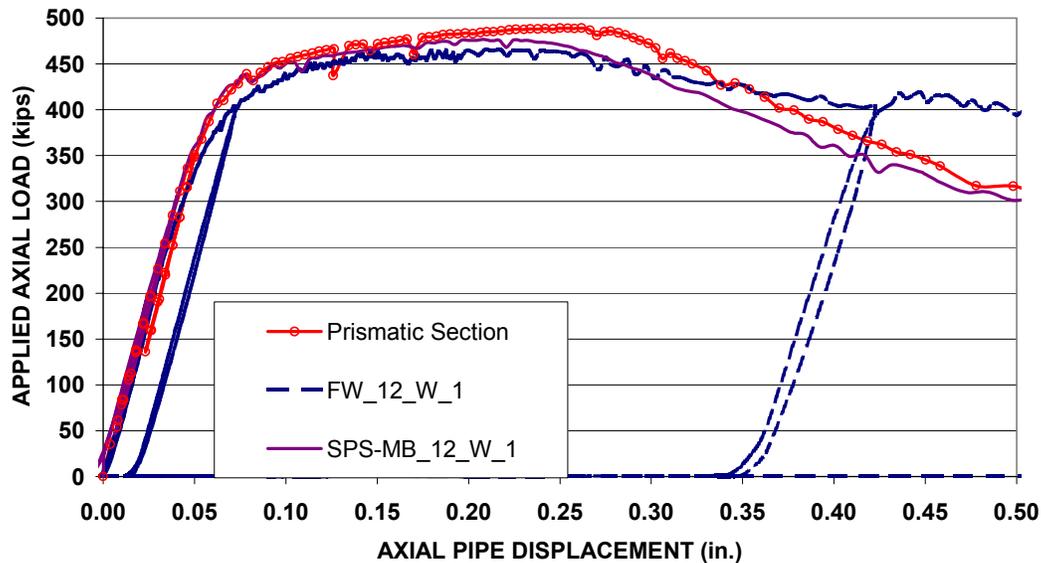**APPLIED AXIAL LOAD vs. DISPLACEMENT**

Figure 3 Fiber Reinforced Retrofitted Welded Slip Joint

The Phase 2 tests are set to begin in April 2001. The Phase 3 tests will be conducted in June 2001. These tests will simulate "real world" loading conditions in which corrected earthquake records from the Northridge Earthquake will be applied at the ends of the pipe.

Transfer of Technology

This research project has had, from the inception, an end user at the core of motivation and focus. The particular end user is the Los Angeles Department of Water and Power. The Cornell research team has directed the concept, the methods of testing, and the materials used for retrofitting. In this close communication and team environment between industry and academia, the transfer of technology from the experimental lab to the field has been smooth and directed. There has been a parallel research project to this testing component; it is the numeric simulation by Ilker Tuntuku, (also under the direction of Prof. T.D.O'Rourke). This work has shown the accuracy and predictive power of finite element analysis when the program model is supplied with physical material test data, even in excursions into the plastic region of deformation. Thus, a direct transfer of technology, from concept inception and development at Cornell University, to end user, LADWP, is being produced in this research program.

Conclusions

The experimental investigation of the use of fiber reinforced composites for the strengthening of welded slip joints against seismic attack has been shown to be an effective and economic solution to this ever-present problem. The experimental results show the improvement of performance, with FRC retrofit, for the 12 inch diameter pipe with ¼" wall thickness to be approximately 30% under compressive loading. The next series of tests, Phase 2 and Phase 3, will investigate the real-time cyclic tension-compression loading of the retrofitted pipes at D/t ratios of 50 and 250. Thus, the research program is moving into the typical large diameter pipe geometries that are used by large municipalities, such as the Los Angeles Department of Water and Power. It is expected that as the D/t ratio increases, the as-built capacity of the welded slip joint will decrease with respect to the prismatic section capacity. It is also expected that as the D/t ratios increase, the net enhancement of capacity of the FRC retrofitted welded slip joint will increase to nearly 100%, to near prismatic section capacity.

7

FRAGILITY CURVES FOR NON-STRUCTURAL SYSTEMS

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Introduction

The main objective of this study is to demonstrate a methodology for generating fragility curves for non-structural systems based on Monte Carlo simulation and specified limit states. The methodology is based on several simplifying assumptions at this stage to keep calculations simple. Most of these assumptions will be removed in the next phase of the study. An example of a 22,000 gallons water tank located in the 20th floor of the VA hospital in NYC is employed to show the proposed methodology.

This study represents the first stage of a general methodology for choosing and optimizing retrofitting techniques for non-structural systems based on cost-benefit analysis approach. The main stages of the methodology are:

1. Generating fragility curves for the system before and after different retrofitting techniques,

2. Estimating the losses associated with different levels of damage in the system,

3. Developing algorithm for generating random scenarios for seismic activities in the site during the lifetime of the system,

4. Using the results of stages 1 to 3 to calculate the potential losses during the lifetime of the system with and without different retrofitting techniques, and finally

5. Choosing the most economical retrofitting technique and optimizing its parameters based on the information obtained in stage 4.

Problem Definition

A simplified model is used to represent the primary-secondary system consisting of the VA hospital and a steel water tank located in its 20th floor. The goal of the analysis is to generate the fragility curves of the water tank for different limit states.

Primary structure: VA hospital consists of three sections, the plaza, the middle tower (core) and the main tower. The plaza section is seven stories high while the two towers are twenty stories high. Figure 1 shows a photo for the plaza and the middle tower of the hospital. Figure 2 shows the location of the water tank on a schematic plan of the hospital. The hospital is a steel frame building with full and partial moment resistance connections. Most floors in the building are made of two way concrete slabs. The building is supported by pile foundations.

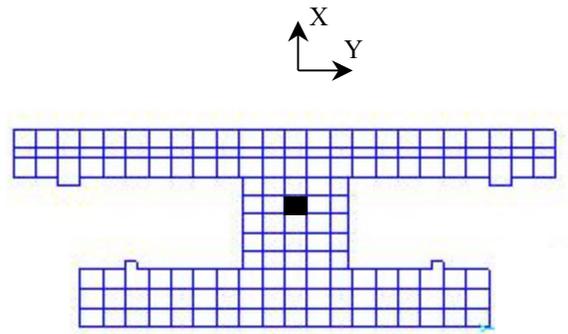**Fig. 1**   Photo of the plaza and the middle tower   **Fig. 2**   Schematic plan of the hospital

Secondary system: The water tank has dimensions of 20.3x16.8x4.8 ft. It is supported at its corners by four vertical legs each of height 6.83 ft and cross section W8*28. This representation is postulated because of lack of detailed information on the secondary systems of the hospital.

Seismic input: The ground excitation is assumed to be a zero-mean band limited Gaussian white

$$G_A(w) = g_o \qquad\qquad w \in [0, \overline{w}],$$
$$G_A(w) = 0 \qquad\qquad elsewhere.$$

noise (BLWN) with the power spectral density function:

It is also assumed that the ground acceleration has duration of 10 seconds and has only one component in the x direction (Fig. 2). The ground excitation depends on a single parameter, g_o, and the fragility curves are plotted against this parameter.

Limit states: Fragility curves are generated for the water tank and four limit states. The limit states considered are $d > d_{cr}$, where d is drift of the water tank legs in the x direction and d_{cr} takes the values of 6, 12, 24 and 36%.

Assumptions

Primary structure

1.   The primary structure remains linear elastic during ground motion. Neither material nor geometrical nonlinearity is considered.

2.   The first and the fourth modes of the primary structure are dominant and are translation modes in the x direction. Therefore the four points supporting the water tank can be assumed to have equal displacements in x direction and zero rotations and translation in the y and z directions.

Secondary system

3.   The body of the tank is modeled as a rigid body. All the deformations are assumed to be concentrated in its four legs.

4.   The tank is assumed to be full of water and closed so that the problem of water slushing is not considered.

Seismic input

5.   The ground excitation is a zero-mean band limited Gaussian white noise (BLWN) with one component only in the x direction applied for 10 seconds.

Analysis

6.   The interaction between the secondary system and the primary structure is ignored. Only the effects of the primary system are considered while the possible feedback from the secondary system on the primary structure is ignored.

7.   The input to the water tank is modeled by the corresponding floor acceleration spectral density defined by analogy with pseudo acceleration floor response spectrum.

8.   The uncertainty in the primary-secondary system parameters is not considered. The only source of randomness is the seismic input.

9.   The first 20 modes of the primary structure are considered in the modal analysis.

Solution Technique

Figure 3 shows the solution sequence. The primary structure is analyzed separately neglecting the interaction with the secondary system. The responses at the attachment points are used as the input to the secondary system. Then the output of the secondary system is processed to generate its fragility curves. The steps of analysis are described below and then summarized in the flow chart of Fig. 4.
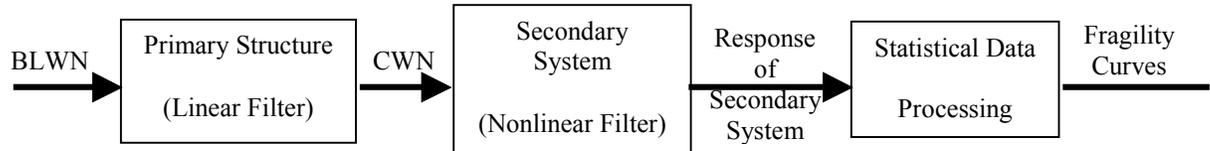
BLWN → | Primary Structure (Linear Filter) | → CWN → | Secondary System (Nonlinear Filter) | → Response of Secondary System → | Statistical Data Processing | → Fragility Curves

Fig. 3 The solution technique

Modal analysis of the primary structure

The mode shapes and natural frequencies of the primary structure were calculated at the University at Buffalo using SAP2000. The stiffness effect of in-fill walls were considered in the Finite Element model using X type brace elements. Rigid diaphragms were assumed in all the floors. The modal participation factors due to ground excitation in x direction have been calculated as well.

Power spectral density function at the attachment points

The theory of random vibration is employed to get the power spectral density function for the response at the attachment points as function of the primary structure parameters. Equation 5.146 in Soong and Grigoriu (1993) is used for this purpose.

Generating realizations of the input excitation to the secondary system

The power spectral density function at the attachment points is used to generate floor accelerations for the secondary system. 240 realizations of the input have been generated for twelve values of g_o covering the range from 0.01 to 0.6.

Nonlinear analysis using DIANA

The secondary system was studied using DIANA under each of the excitations that were generated in the previous step. The body of the tank was assumed to be rigid so that deformations are concentrated in the four legs. The frequencies of the first three vibration modes of the tank are 9.34, 15.19, and 17.95 rad./sec., respectively. Both geometrical and material
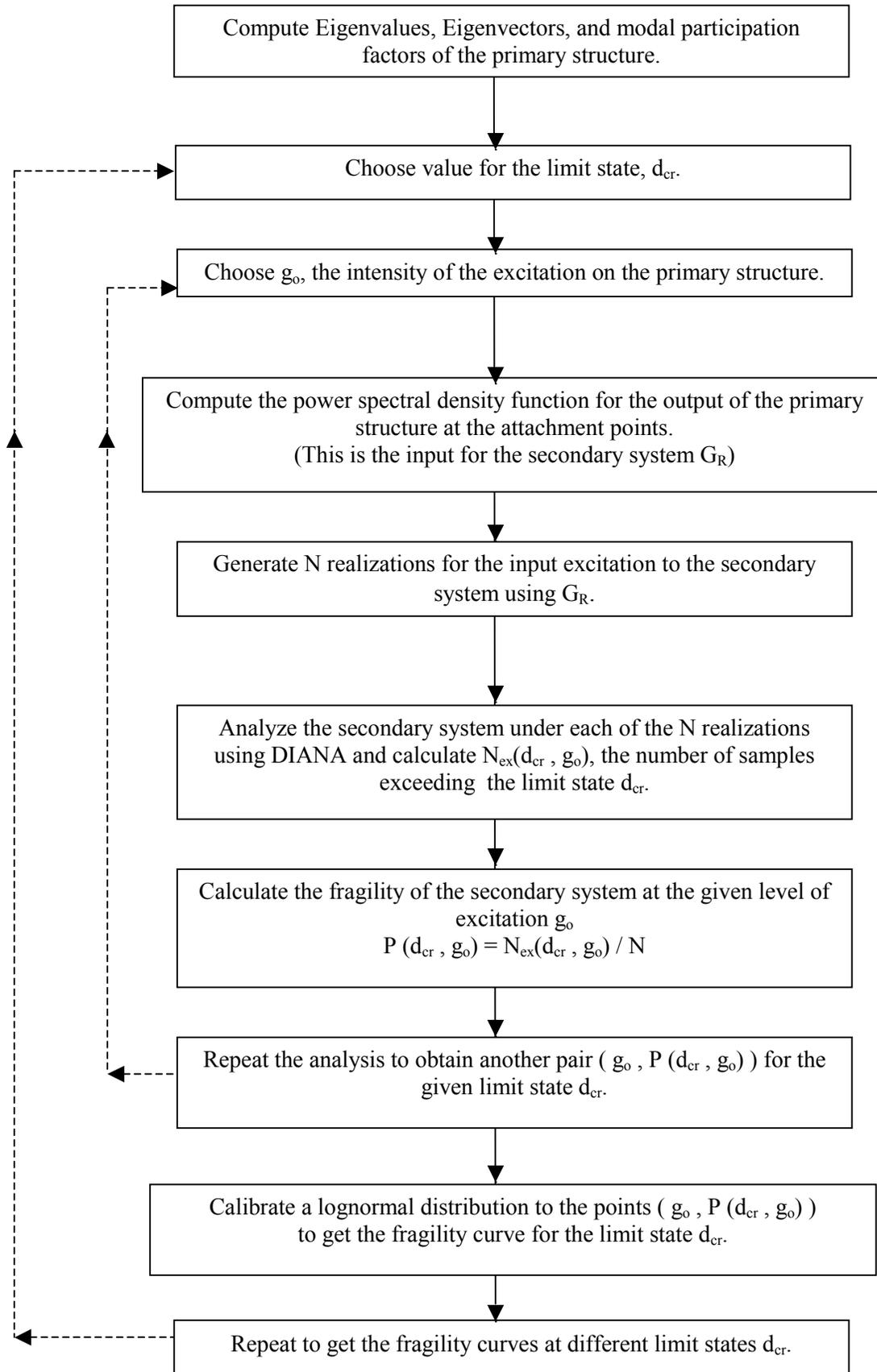
Compute Eigenvalues, Eigenvectors, and modal participation factors of the primary structure.

Choose value for the limit state, d_{cr}.

Choose g_o, the intensity of the excitation on the primary structure.

Compute the power spectral density function for the output of the primary structure at the attachment points.
(This is the input for the secondary system G_R)

Generate N realizations for the input excitation to the secondary system using G_R.

Analyze the secondary system under each of the N realizations using DIANA and calculate $N_{ex}(d_{cr}, g_o)$, the number of samples exceeding the limit state d_{cr}.

Calculate the fragility of the secondary system at the given level of excitation g_o
$P(d_{cr}, g_o) = N_{ex}(d_{cr}, g_o) / N$

Repeat the analysis to obtain another pair (g_o, $P(d_{cr}, g_o)$) for the given limit state d_{cr}.

Calibrate a lognormal distribution to the points (g_o, $P(d_{cr}, g_o)$) to get the fragility curve for the limit state d_{cr}.

Repeat to get the fragility curves at different limit states d_{cr}.

Fig. 4 Flow chart of the solution technique

nonlinearities were considered in the analysis. The material model used for the four legs was von Mises plasticity with strain hardening as shown in Fig. 5. For each sample, the maximum drift of the legs, d, was calculated.
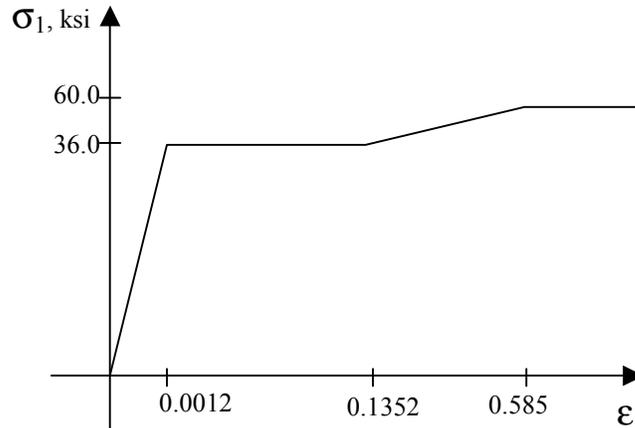

Fig. 5   Material model for the tank legs

Developing of the fragility curves

The fragility curves were generated for four limit states of the secondary system corresponding to drifts of 6, 12, 24 and 36%.

For a given limit state, d_{cr}, and a specific value of g_o, the fragility is defined as the ratio of the samples with maximum drift, d, exceeding d_{cr}.

$P (d_{cr} , g_o) = N_{ex.} (d_{cr} , g_o) / N (g_o),$

where N is the total number of samples generated for the given g_o.

A lognormal distribution was then fitted to the points to give the fragility curves as shown in Figure 6.
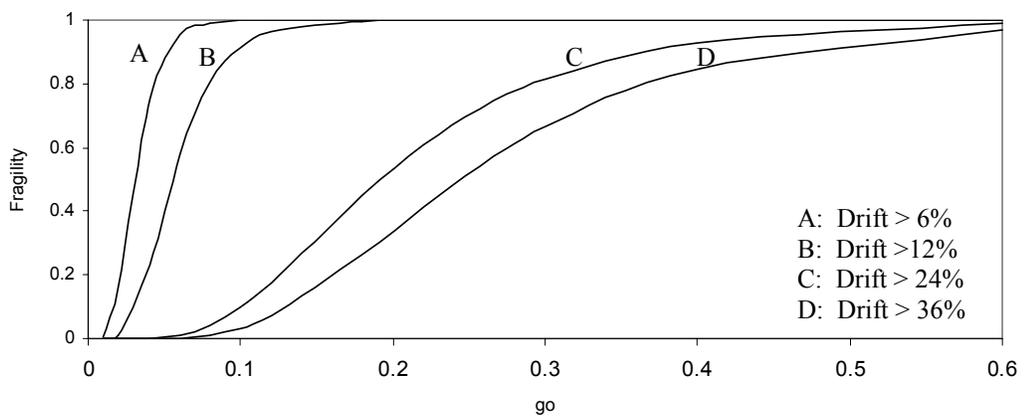

A: Drift > 6%
B: Drift >12%
C: Drift > 24%
D: Drift > 36%

Fig. 6   Fragility curves of the secondary system

Conclusions

A methodology for generating fragility curves of non-structural systems was presented.   The method is based on Monte Carlo simulation, dynamic analysis and statistical concepts.

References

Yongpi Chen and T. T. Soong, 'State-of-the-art review, seismic response of secondary systems', Eng. Struct., Vol. 10, 1988.

M. Grigoriu and S. Balopoulou, 'Random vibration and reliability analysis of primary-secondary structural systems', Tech. Rep. NCEER-89-0031, Buffalo, New York, 1989.

T. T. Soong and M. Grigoriu, 'Random vibration of mechanical and structural systems', PTR Prentice-Hall, New Jersey, 1993.

8

URBAN DAMAGE ASSESSMENT FROM REMOTELY SENSED IMAGES
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Abstract

This study proposes a method to utilize remotely sensed pre- and post-disaster (bi-temporal) imagery data in order to detect the change specifically associated with structural and major regional damage caused by natural disasters such as a strong earthquake. The input is a pair of coregistered remotely sensed images of the same scene acquired at different times and the output is a binary image in which 'changed' pixels are separated from 'not-changed' ones. Correlation analysis generally fails to detect structural change, especially if images are acquired under different illumination conditions. In fact, automated detection in such a case becomes problematic since making distinction of change due to structural damage from that associated with the difference in the illumination condition is difficult. To overcome this problem, a method of principal component analysis (PCA) is employed. The approach produced promising results on the model images and currently under further study to be extended for near real-time damage assessment purposes.

Background

During the last two decades, vulnerability of urban areas to natural disasters such as severe earthquakes has attracted significant attention among researchers and practitioners from disaster management point of view. Many researchers in the field of structural mechanics and earthquake engineering have dedicated extensive efforts and contributed to the reduction of earthquake-induced human and property loss by improving design codes for buildings, bridges and other structures. Yet, it appears that a satisfactory level of structural reliability has not been achieved from the real-time disaster management point of view. Therefore, the further attention towards the post-disaster emergency response is well justified in order to make the urban areas more disaster-safe. In this respect, securing real-time or near real-time information of seriously damaged urban area immediately after a natural disaster is extremely useful for executing efficient search and rescue missions. At the same time, the information is very valuable for developing and implementing strategies for recovery and restoration of the affected area. Hence, this study endeavors to explore ways in which damage information can be obtained with the aid

of advanced technologies. More specifically, this study exploits remotely sensed images of the same scene before and after a disaster event such as a strong earthquake in order to detect structural change by comparing them. In fact, it is the intention of this study to help further enhance real-time damage detection schemes for post-earthquake disaster mitigation.

Remote Sensing and Applications

Remarkable advances in remote sensing technology and availability of high-resolution images in recent years has motivated many researchers to exploit this technology for change detection purposes. Detection techniques can indeed be applied for a variety of purposes such as deforestation, archeology, urban planning and development, damage assessment, defense intelligence, and environmental monitoring. This study investigates the applicability of the detection techniques for the purpose of emergency response as illustrated in Figure 1. In this scheme, images are remotely captured from urban areas that usually include a relatively large number of buildings and other structures that are vulnerable to natural disasters.

The remotely sensed images can be acquired from various platforms including spacecrafts or aircrafts by using both synthetic aperture radar (SAR) and optical imaging devices (see NASA). Each acquired image will be transferred to an image database from which the image of the same area that is captured previously, can be retrieved by matching their attributes such as longitude/latitude of the area. This image pair will be processed for change detection and outputs a binary image in which 'changed' pixels are altered from 'not-changed' ones. Finally, this binary image will be virtually overlaid on a specific kind of already existing database, e.g., a GIS map, which contains only the major structures. Any overlap between the binary image and this database, identifies the location of a single or a group of changed/damaged structures. Each of the above steps in the scheme requires a sufficient study to implement appropriate and practical algorithms. However, the focus of this paper is placed specifically on processing of the remotely sensed coregistered image pairs and identifying structural change in the scene of interest.

Related Work

Singh (1989), who compared a variety of proposed approaches during the 70s and 80s, briefly reviews the history of change detection from remotely sensed digital images. Change detection in imagery data is widely studied in the field of computer vision, remote sensing and image processing. Some of these studies (Carlotto, 1997; Fily and Rothrock, 1987; Lin and Nevatia, 1995; McGlone and Shufelt, 1994; and Noronha and Nevatia, 1996) have developed interesting techniques for a variety of purposes such as identifying buildings from aerial images, monitoring

regional changes, land use, and wide area surveillance over a long period of time. Due to the challenging nature of the change detection problem, usually a certain amount of uncertainty is allowed in their outputs that can only be eliminated using ground-truth data.

Although some of the existing methods provide a very useful basis for 'change' detection studies, they have limited capabilities for structural 'damage' assessment. These methods do not focus on structural damage assessment of the scene in the sense that they do not distinguish any change caused by structural damage from other kinds of change. It is crucial to recognize that any major structural damage induces change in the captured images but not all changes are due to structural damage. Some of the recent works (Singh, 1989) have employed more advanced analytical procedures to characterize change based on the relative distance of the pixels from the regression line in the bi-temporal feature space. These methods achieved reasonable successes to evaluate the change observed in the scene, although they did not suggest how to perform specific structural change detection. This paper proposes a method of advanced structural change detection by enhancing and uniquely tailoring some of the existing methods for seismic hazard mitigation purposes.
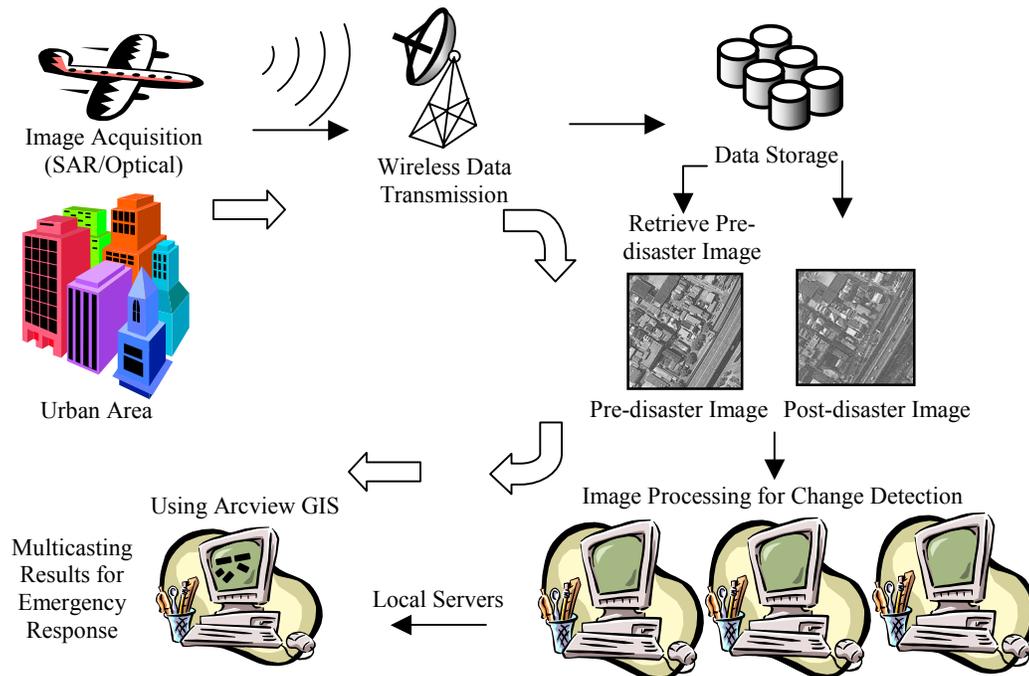

Figure 1: Employing Remote Sensing Imagery Data for Disaster Management

Proposed Approach

Assumptions and Limitations

In order to focus on the image processing techniques and provide a practical framework for the present methodology, it is required to make a few assumptions. The most important assumption is that the input images are coresigtered, namely certain key points that represent the pattern are at the same location (same pixel) in both pre- and post-disaster image pairs, without any relative displacement. Employing the conventional methods of rectification, geocoding and warping (Pratt, 1991) one can perform image registration manually. Acquiring images from an identical viewpoint (i.e., identical camera location or orientation) using calibrated image acquisition devices satisfies the requirements for image registration but may not be easily achievable in practice. It is also assumed that input images and databases that might be utilized for data fusion are coregistered and updated at the desired frequency.

In practice, multispectral high-resolution images are desirable since the level of accuracy in damage detection is controlled by the resolution of the input images. Similarly, images with a high signal to noise ratio and fine spectral resolution are desirable. A larger number of spectral bands implies a finer color separation and distinction. An orthographic projection is also assumed in which the parallel lines in real world coordinates remain nearly parallel in image coordinates.

This study proposes a probabilistic approach (Shinozuka and Rejaie, 2001) for change detection in images that can be considered as unsupervised segmentation or clustering problem. The goal is to classify image pixels into two categories only, i.e., "changed" and "not-changed." The approach includes three major steps; 1) Depending on the local variance of each neighborhood in the image and the intensity value of the pixels in that neighborhood, intensity values are weighted. Employing "Principal Component Analysis" (PCA), one is able to compute the direction cosines of the regression line, i.e., the first PC, and also the direction cosines of the second PC that is perpendicular to the first one. In a bi-temporal feature space, the second PC shows the direction on which non-matching (changed) pixels are located. This is a method by which one may quantify and characterize the contribution of each neighborhood to the observed change in the input image pair. This procedure converts the original pixel intensities to a pair of uncorrelated variables in the corresponding neighborhoods; 2) This value can be used to compute conditional probability for appropriateness of a class ("change" or "not-changed") to explain the classification of a pixel. Assuming normal distribution hypothesis for this variable and using maximum-likelihood method, its mean value and variance are computed; 3) Using Bayesian statistics and initiating the values of conditional probability for pixel to belong to a certain class

in an iterative fashion, one is able to compute this probability after a few iterations and observing a reasonable convergence. In order to interpret the output, it can be virtually overlaid on a set of spatio-temporal database such as GIS. In order to further verify change detection results, human supervision can be employed particularly when either the input or output data are not sufficiently descriptive. In this case, the detection procedure becomes semi-automatic but reliable algorithms can be developed so that human assistance is optimally minimized and eventually it can be improved towards a fully automatic procedure that is the ultimate goal of this study.

Results

In this section, the performance of the straightforward correlation analysis for change detection is demonstrated on a pair of pre- and post-disaster sample images under identical illumination conditions. In order to compare the capability of the correlation analysis with PCA method, both of them are performed on same sample images and their results are indicated and discussed.

Capabilities of correlation analysis in change detection

The straightforward correlation algorithm is used to compare the 8-bit gray-level images in Figures 2a and 2b, which show the Hanshin area before and after the 1995 Kobe earthquake (Hyogoken-Nanbu earthquake), respectively, and the change detection result is illustrated in Figure 3c. All of the computations in this study are performed in a MATLAB environment on a Unix platform. The size of the input images is 607×277 pixels. In this particular experiment, Figure 2a is artificially created from Figure 2b by removing that part of the image depicting the collapsed segment of the elevated Hanshin Expressway viaduct and replacing it with the unchanged segment of the expressway image by pasting and also manipulating the cars on the bridge deck. Therefore, it was expected to produce a high correlation coefficient over the entire image except along the damaged length of the bridge and on a few cars on the deck in the before image. The result indicates that the overturned segment of the bridge and also the cars on the deck are clearly detected which is consistent with human perception. Although this simulated numerical experiment indicates the capability of correlation analysis, unfortunately, it applies only to the cases where the illumination conditions of the corresponding images are identical. These cases are rare in reality due to the random nature of a disaster event in terms of time of occurrence, associated weather conditions and the location and look angle of imaging devices. Local normalization may slightly improve the performance of correlation analysis, but yet the desired accuracy is far from being achieved for the purpose of this study.
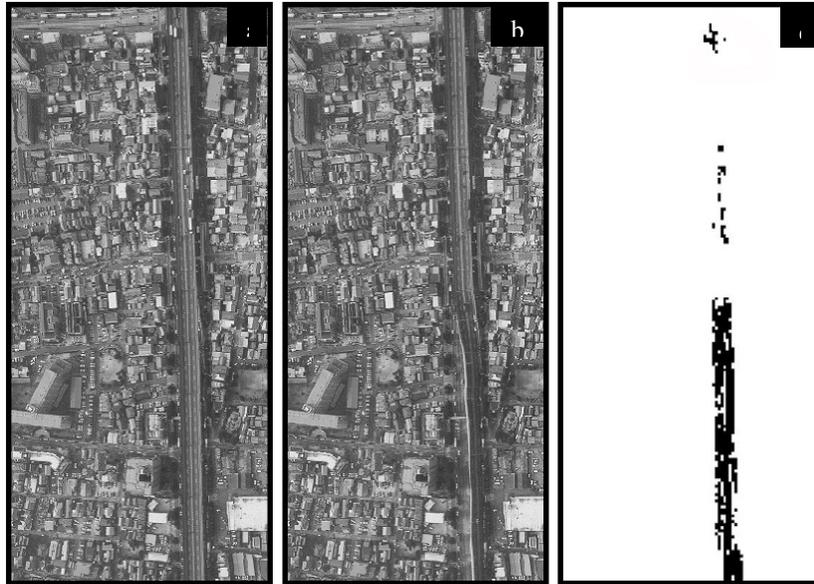
Figure 2: a) Pre-disaster image created from the original image, b) Post-disaster image; courtesy of Asia Kosoku Co., c) Result of correlation analysis

Image Registration

To conduct a more realistic experiment, two multispectral, high-resolution, and coregistered images are required. For this purpose, two aerial photos from Kobe (Asia Kosoku Co., 1995) taken in 1996 (Figure 2a) and 1998 (Figure 2b) were initially considered. These images are good candidates for the proposed application in this study since; 1) they are acquired under different illumination conditions; and 2) substantial structural changes are observed in the scene. However, major problems with the images are: 1) not being coregistered; 2) taken from different viewpoints and also different altitude; and 3) taken using different types of cameras. The problem of not being coregistered is shown in Figure 3 by different coordinate positions of two points in the image pair that represents the same corner of a building in the lower right portion of the images. By post-processing the images, e.g., manual warping techniques (Pratt, 1991) and image rectification, the effects of some of these problems can be slightly to significantly reduced depending on the image characteristics. Impacts of different viewpoints cannot be easily compensated since the 3D information of object corners and also lens calibration parameters are required. Therefore, the indicated Kobe images were recognized as not fully qualified for further experimentation. Thus, a reduced scale three-dimensional model of the scene was physically reconstructed and a few images were captured using a static digital camera. This implies explicitly that input images qualify for the experiment as long as the coregistration is concerned.

By changing the location of the light source and the number of existing buildings while keeping the camera static, the desired synthetic scenarios of change were created and a few pairs of

coregistered multispectral images were obtained. These include three different sets of characteristics: case 1: identical illumination and structures; case 2: identical illumination and different number of structures; and case 3: different illumination and different number of structures. The proposed method performed successfully for all three cases as discussed in the following section. However, the complete results are presented in this paper only for case (3), which presented the most significant challenge.
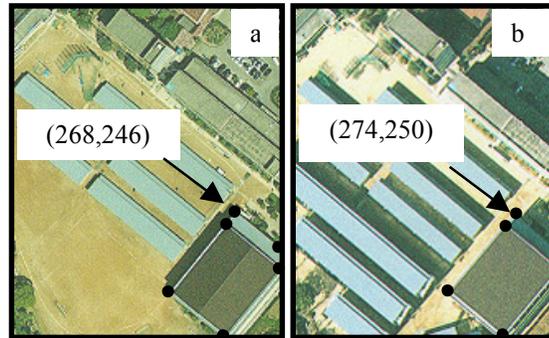

Figure 3: Arial photos from Kobe, a) Acquired in 96, b) Acquired in 98

Principal Component Analysis

A code based on the PCA algorithm was implemented in MATLAB image processing toolbox and was executed on a Unix platform. Change detection was performed on images model of Figure 2, using PCA method. As shown in Figure 4, PCA method exhibits a major improvement over the correlation method and is highly consistent with human perception. However, there still exist false detections. In the specific set of images used here, two types of false detections are
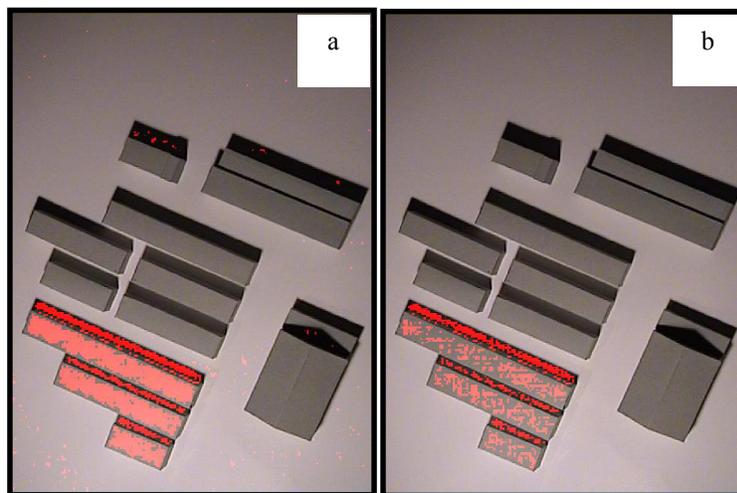

Figure 4: The effect of threshold value; a) PCA result with relatively low threshold; b) PCA result with threshold increased by %5

observed: 1) very small clusters distributed over the images and 2) relatively large clusters that have major overlaps with building shadow areas. The first type of false detection can be removed by applying an appropriate mask and biasing the threshold. In order to study the effects of threshold value on the output, the model images were used with different assignments for the threshold value. Figure 4 illustrates a comparison between two of these experiments.

In Figure 4a, a relatively small threshold is used due to which several falsely detected points are marked. After increasing the threshold value by 5%, false detection is extremely reduced but at the same time, the distinctive detection of the three missing buildings is also highly degraded. Therefore, there exists a tradeoff between reducing the false detection rate and addressing the entire changed objects in the scene distinctively. Dealing with the second type of false detection is not as simple but probably using elevation data might be a good solution for this problem, which is currently under investigation by the authors.

Conclusion

This study addressed the problem of change detection for the purpose of natural hazard mitigation in general and seismic hazard mitigation in particular, and also demonstrated the feasibility of employing remotely sensed images for this purpose. Due to the limited access to actual registered imagery data, scaled structural models are constructed and their images are used for experiments. Some of the major technical challenges involved in the problem of structural change/damage were also identified and successfully dealt with. Due to the instability and inconsistency of correlation analysis, it was replaced by the PCA.

In this paper, the assumption of employing registered images is made which in practice might be difficult to achieve. Therefore, in the future, the authors intend to use images with slight misregistration in order to see the impact of minor registration errors. Employing ground control points (GCP) and global positioning system (GPS) sensors should be helpful for improving registration accuracy. These might provide some means for minimizing the impact of misregistration in the procedure without sacrificing the accuracy of the change detection.

This study does not address the processing time for the algorithm since real-time execution of the entire damage assessment procedure is not intended at this time. The authors believe that processing time has a lower priority compared to the precision of the results as long as the near real-time nature of the algorithm is maintained.

Acknowledgment

References

Asia Kosoku Co., *Collection of Aerial Photos of Great Hanshin-Awaji Earthquake Disaster*, 1995.

Carlotto, M. J., "Detection and Analysis of Change in Remotely Sensed Imagery with Application to Wide Area Surveillance"*, IEEE Transactions on Image Processing*, *Vol. 6*, No. 1, January 1997.

Fily, M., Rothrock, D.A. , "Sea Ice Tracking by Nested Correlations", *IEEE Transactions on Geoscience and Remote Sensing*, **Vol. GE-25**, No.5, Sep.1987.

Kreyszig, E., *Advanced Engineering Mathematics*, pp. 1145-1153, John Wiley & Sons, 1999.

Lin, C., Nevatia, R., "3D Descriptions of Buildings from an Oblique View Aerial Image", *IEEE International Symposium of Computer Vision*, pp. 337-382, 1995.

McGlone, J., Shufelt, J., "Projective and Object Space Geometry for Monocular Building Extraction", *IEEE Proceedings of Computer Vision and Pattern Recognition*, pp. 56-61, 1994.

NASA JPL Imaging Radar, *Imaging Radar Homepage*, http://southport.jpl.nasa.gov.

Noronha, S., Nevatia, R., "Detection and Description of Buildings from Multiple Aerial Images", *Proceedings of the 1996 DARPA Image Understanding Workshop*, pp 469-478, Palm Springs, 1996.

Pratt, W. K., *Digital Image Processing*, 2nd edition, pp.430-440, John Wiley & Sons, 1991.

Shinozuka, M., Ali Rejaie, "Damage Assessment from Remotely Sensed Images Using PCA", *Proceedings of SPIE 8th Annual International Symposium on Smart Structures and Materials*, Newport Beach, CA., March 2001.

Singh, A., "Review Article: Digital Change Detection Techniques Using Remotely Sensed Data", *Int. Journal of Remote Sensing*, **Vol. 10**, No. 6, pp. 989-1003, 1989.

Wiemker, R., "An Iterative Spectral-Spatial Bayesian Labeling Approach for Unsupervised Robust Change Detection on Remotely Sensed Multispectral Imagery", *Proceedings of the 7th International Conference on Computer Analysis of Images and Patterns*, pp.263-270, Springer LNCS 1296 (ISBN 3-540-63460-6), 1997.

9

**COMPRESSIVE LOAD AND BUCKLING RESPONSE OF STEEL PIPELINES
DURING EARTHQUAKES**

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Introduction

The 1994 Northridge earthquake resulted in the most extensive damage to a U.S. water distribution system since the 1906 San Francisco earthquake. Los Angeles Department of Water and Power (LADWP) trunk lines (pipes with nominal diameter greater than 600 mm) were damaged at 74 locations and distribution lines required repairs at 1,013 locations. A comprehensive study of the damage patterns carried out at Cornell University (Toprak, 1998) indicated that approximately 60% of the critical trunk line damage concentrated at welded slip joints in the form of compressive buckling.

A welded slip joint is fabricated by inserting the straight end of a pipe into the bell end of an adjoining pipe and circumferential fillet welding. The bell end is created by the pipe manufacturer by inserting a mandrel in one end of a straight pipe section, and expanding the steel into a bell casing. Larger diameter pipelines can be constructed in the field easily by joining the bell and straight ends of pipe segments.

As illustrated in Figure 1, failure of welded slip joints can be initiated by compressive forces that induce buckling and outward deformation at the location of maximum curvature in the bell casing. Compressive forces sufficient to fail welded slip joints can be generated by near source pulses of high particle velocity as well as permanent ground deformation generated by surface faulting, liquefaction, and landslides. Lateral ground movement triggered by liquefaction near the intersection of Balboa Boulevard and Rinaldi Street in Los Angeles during the Northridge earthquake caused compressive failure of 1,245-mm diameter Granada Trunk Line. Similar compressive failures were observed in the adjacent 1,727-mm diameter Rinaldi Trunk Line. Loss of both Granada and Rinaldi Trunk Lines cut off water to tens of thousands of customers in the San Fernando Valley for several days and put post-earthquake fire fighting efforts into jeopardy.

Research Objectives

The overall goal of the research, part of which is summarized in this paper, is to achieve substantial improvement in seismic reliability of water supply systems through advanced technologies, notably fiber reinforced composites (FRCs). The research objectives can be listed

as: 1) full-scale testing of straight pipe sections with various global and local geometric imperfections, 2) implementation and validation of finite element (FE) models for straight pipe sections, and 3) development of simplified shell and FE models for performance assessment of
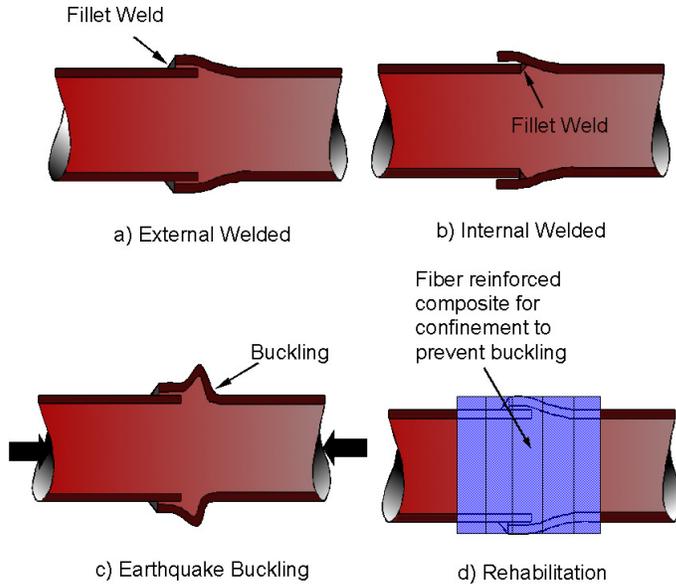
Figure 1. Compressive Response of Welded Slip Joint and FRC Reinforcement

welded slip joints under compressive loads, 4) full-scale testing of welded slip joints with and without FRCs, and 5) development and validation of FE models for compressive load performance of FRC-reinforced welded slip joints.

Straight Pipe Sections

The buckling limit of a straight pipe section establishes the maximum compressive load capacity of a pipeline. The upper bound of the performance with FRC wrapping is the buckling limit of a straight pipe. That is why assessment of buckling limit of straight pipe sections was emphasized in the research. If FRC strengthening of a welded slip joint increases the compressive capacity to the buckling limit of a straight pipe section, then the FRC technology has been successful in achieving the maximum possible improvement.

ABAQUS™ was adopted as the FE package of choice for evaluating welded slip joint and pipeline performance. Benchmark studies were performed. The most appropriate element type was determined to be a linear, four node, reduced integration shell element. Sensitivity of numerical prediction to global geometric imperfections was investigated. Many researchers (e.g., Donnell and Wan, 1950) have shown that initial imperfections due to manufacturing and handling have a strong influence on the buckling limit of pipe sections. Imperfections were measured systematically across a 300-mm diameter straight pipe specimen with wall thickness of 6.2-mm by means of a digital dial gage. The periodicity of the imperfections was analyzed with fast Fourier transform technique. The resulting imperfection spectrum was used to implement global geometric imperfections in the FE models. The maximum imperfection amplitude was found to be approximately 3% of the pipe wall thickness. Global geometric imperfection pattern was numerically generated by using bifurcation-buckling analysis technique of ABAQUS™. The generated imperfection pattern was sinusoidal in axial and circumferential directions, and matched the measured pattern closely at certain locations.
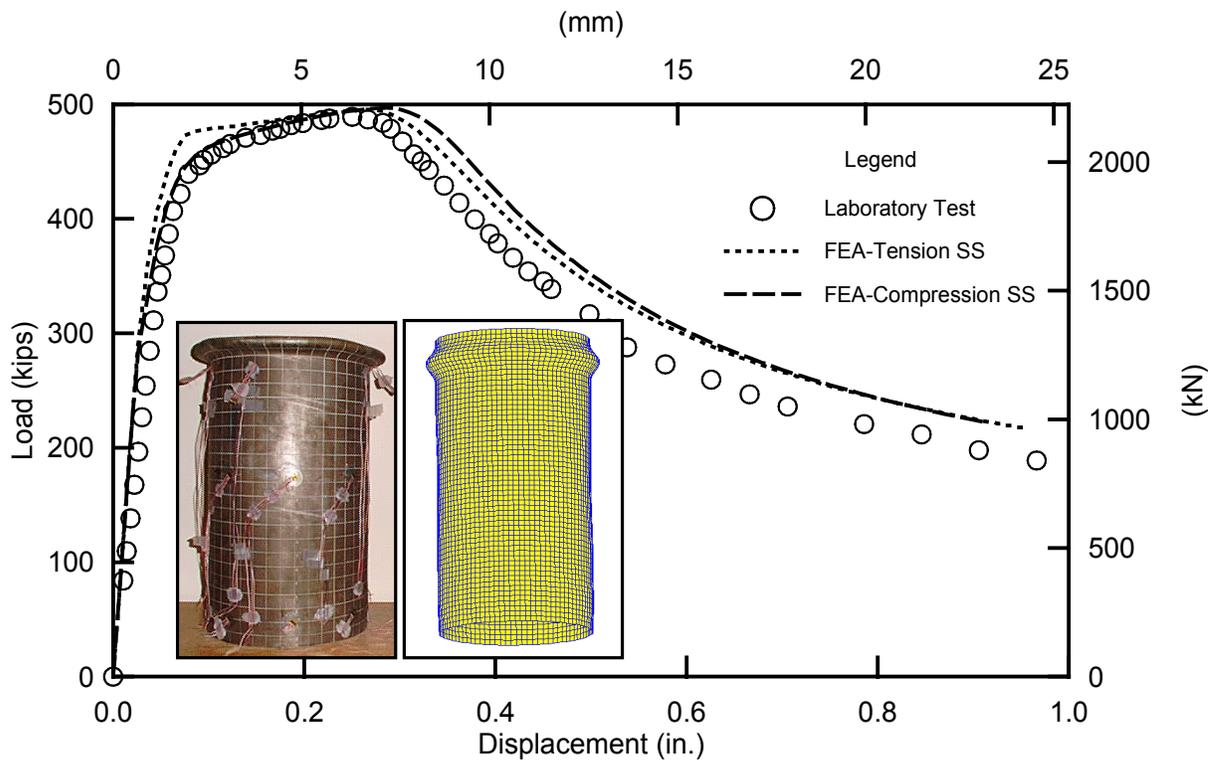

Figure 2. Comparison of Test and FE Results for 300-mm Diameter Straight Pipe

Two 300-mm diameter, 635-mm-long straight pipe sections were tested under axial compression in the George Winter Laboratory of Cornell University. FE simulations of these tests were performed by including numerically generated global imperfection patterns. Very close agreement between the experimentally observed and numerically predicted buckling patterns was achieved. Figure 2 shows the axial load vs. displacement plots for the first test specimen. Both uniaxial tension and compression stress vs. strain data were considered in the FE model and compared in Figure 2. There is a remarkably close agreement between the experimental and analytical results. The peak predicted and measured loads are 2,200 kN and 2,175 kN, respectively. As shown in the inset images, there is also very close agreement with respect to the location of buckling in the experimental and analytical results.

In regard to local geometric imperfections, a 300-mm diameter straight pipe section was indented in the laboratory with 19-mm ball bearing, and then compressed under axial loading. The depth of the indentation was approximately 200% of the wall thickness. FE simulation of this test was performed in a three-step analysis procedure. In the first step, the indentation process was simulated by radially displacing FE nodes. In the second step, the model was relaxed. As a result of the plastic deformation, there was a residual stress distribution around the indented area. In the third and final step, axial compression of the specimen was simulated. Load vs. displacement plots from the FE analysis and the test are compared in Figure 3. Both the peak load and the deformation pattern were closely predicted by the FE model. As compared to the straight pipe section that was described earlier, the peak load did not significantly drop.
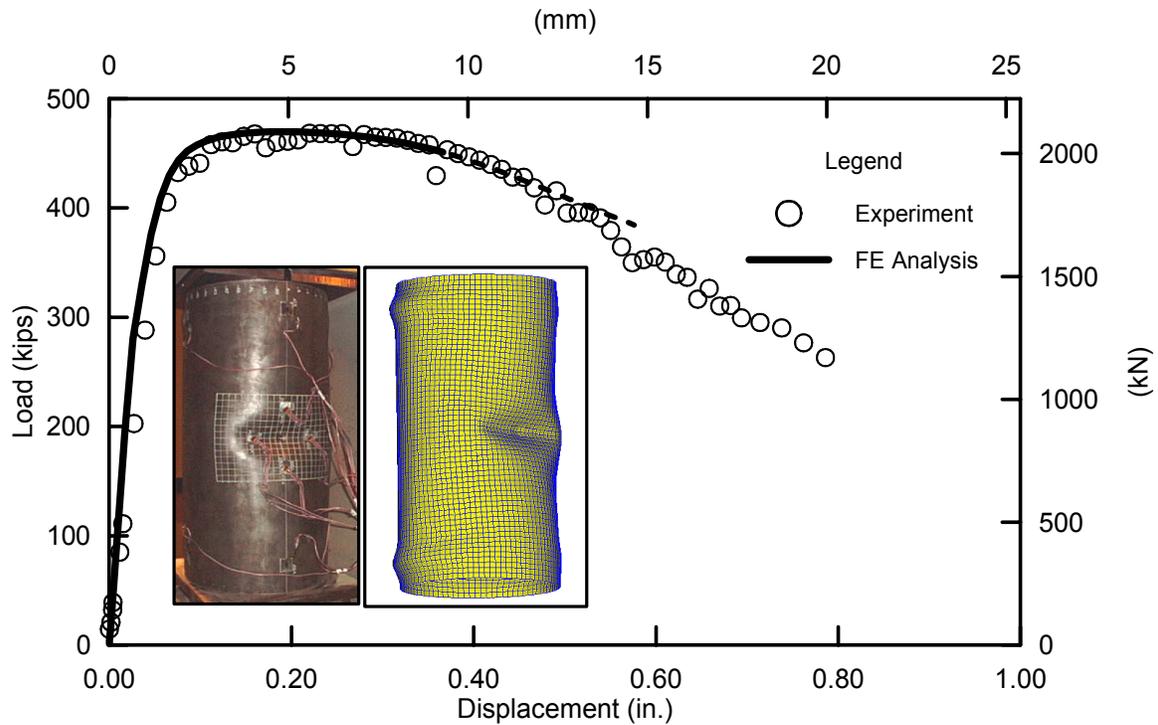
Figure 3. Comparison of Test and FE Results for 300-mm Diameter Straight Pipe with Local Imperfection

Welded Slip Joints

Simulation of the circumferential fillet weld in the FE model was found to have a significant influence on the numerical predictions. An appropriate weld representation technique involving multi-point constraints (MPCs) was adopted. The MPC technique connects nodes around the circumference of the bell to their counterparts on the inserted straight pipe. A rigid weld is then simulated by enforcing identical degrees of freedom in each pair of connecting nodes.

Figure 4 shows the axial load vs. displacement plots for a 300-mm diameter welded slip joint test specimen and the FE simulation of this test. There is a remarkably close agreement between the load vs. displacement relationship and the patterns of buckling, as the inset images indicate.
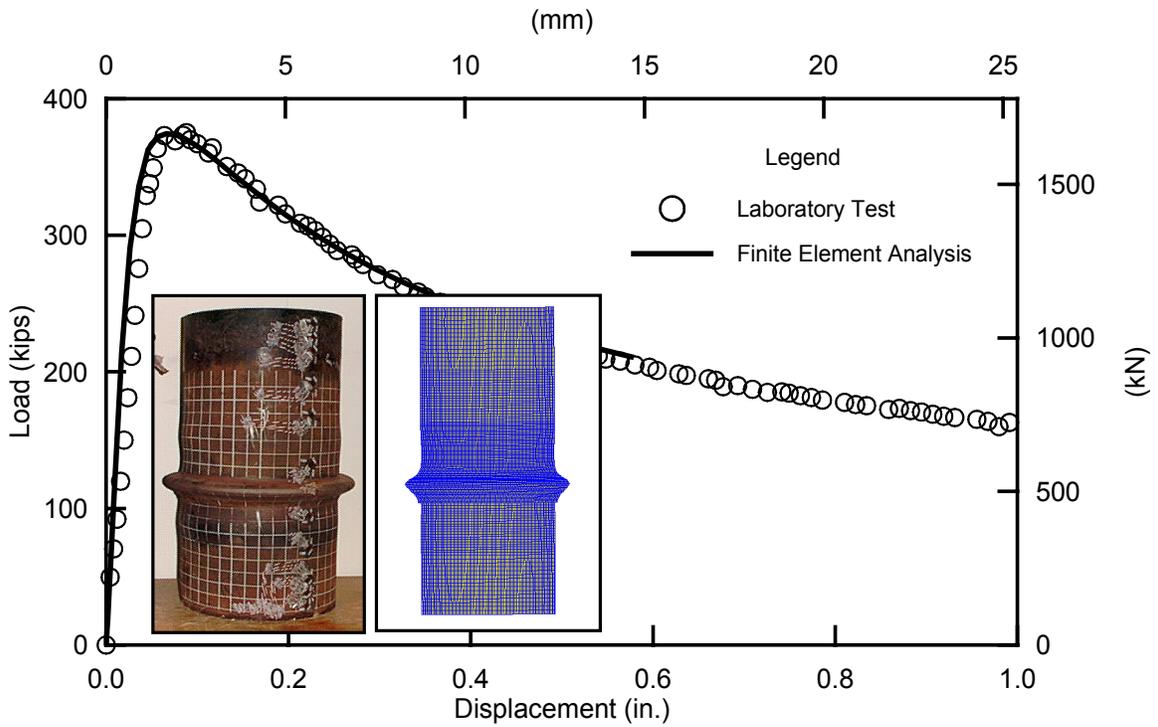
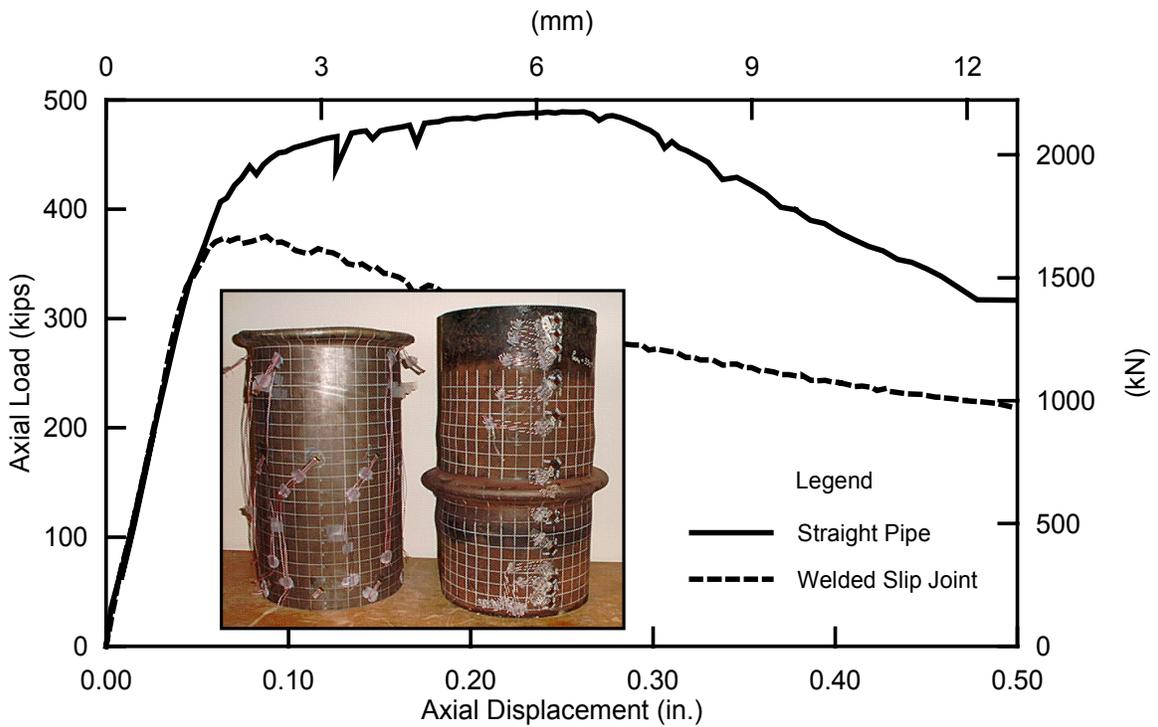**Figure 4**. Comparison of Test and FE Results for 300-mm Diameter Welded Slip Joint

Figure 5. Comparison of Load vs. Displacement Behavior of Straight Pipe & Welded Slip Joint Specimens
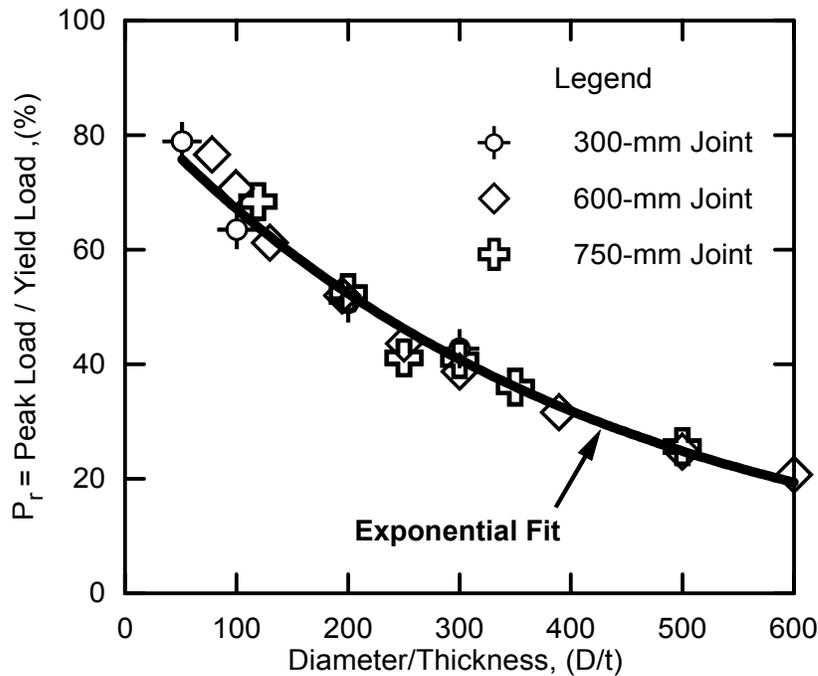
Figure 6. Welded Slip Joint Capacity as a Function of Diameter-to-Thickness Ratio

Figure 5 compares the axial load vs. displacement plots of the straight pipe section and that of the welded slip joint specimen. The maximum load carried by the welded slip joint was 1,670 kN, which was 77% of the straight pipe section. In other words, presence of a welded slip joint resulted in a 23% reduction in the axial compressive load capacity.

The close agreement between the analytical and experimental results indicates that the FE model developed in this research work is robust and sufficiently reliable for evaluating the response of welded slip joints with different geometries. Figure 6 shows the analytical results of welded slip joint axial load capacity, expressed as P_r, the ratio of maximum to theoretical yield load, plotted relative to pipe diameter-to-thickness ratio, D/t. The yield load is the product of the yield stress of the steel and the cross-sectional area of the straight portion of the welded slip joints. For the cases considered in this paper, the yield stress can be taken as 338 MPa at 0.2% offset.

The analytical results in Figure 6 allow one to scale the current findings to larger D/t ratios, representative of larger diameter pipes. For example, the Granada and Rinaldi Trunk Lines described previously have D/t ratios 160 and 180, respectively. Welded slip joints in this D/t range would be expected to mobilize only 50% of the maximum axial load capacity. Hence, FRC strengthening theoretically can result in nearly a 100% increase in compressive load capacity of a pipe with these geometric characteristics.
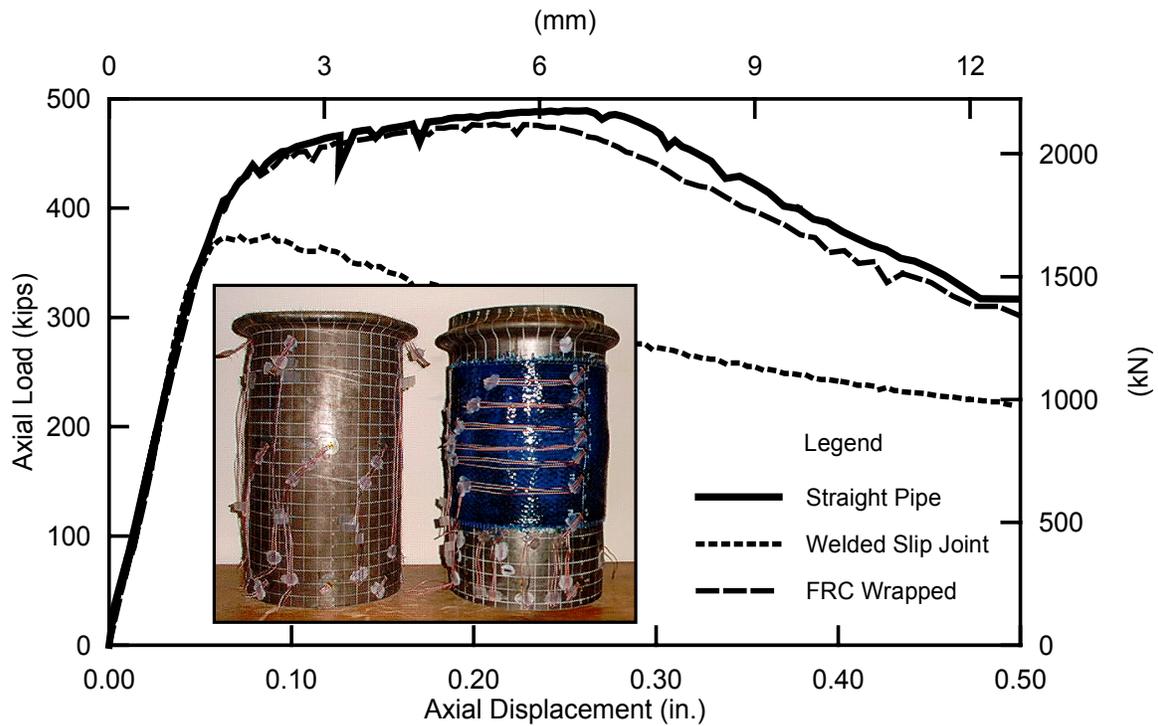
Figure 7. Comparison of Load vs. Displacement Behavior of Straight Pipe, Welded Slip Joint and Retrofitted Welded Slip Joint Specimens

Retrofitted Welded Slip Joints

As schematically shown in Figure 1, the retrofitting technique involves FRC wraps to restrain the radial expansion of a welded slip joint on the bell casing region under axial compressive loads. Figure 7 shows the axial load vs. displacement plots for the 300-mm welded slip joint specimen, straight pipe specimen and the FRC wrapped welded slip joint specimen. As can be seen, the FRC wrap resulted in an increased capacity of approximately 25% as compared to welded slip joint. It can also be seen that FRC strengthening achieves a compressive capacity virtually equal to the buckling limit of the straight pipe section. The inset images show that FRC wrapped specimen failed by buckling approximately at the same location and manner as the straight pipe specimen.

Conclusions

The research summarized in this paper has focused on understanding the compressive and buckling behavior of straight pipes and pipes with welded slip joints, and proposing an effective retrofit technique involving FRC wraps to increase axial compressive load capacity of welded slip joints. FE models were developed and full-scale laboratory tests were performed to substantiate

these models. The degree of agreement between the test results and the FE predictions was generally remarkably good. The FRC wrapping technique restrained radial expansion of the welded slip joints and provided an additional 25% capacity for 300-mm diameter welded slip joint specimens. FRC strengthening may result in a much larger increase in the compressive load capacity for joints with larger D/t ratios. Experimental and numerical simulation work are already in progress at Cornell University to validate the effectiveness of the FRC wrap technique for larger size joints.

References

Toprak, S. (1998), "Earthquake Effects on Buried Lifeline Systems", Ph.D. Thesis, Cornell University, Ithaca, NY.

Donnell, L. H., and C. C. Wan (1950), "Effect of Imperfections on Buckling of Thin Cylinders and Columns Under Axial Compression", Journal of Applied Mechanics, ASME, Vol.72, pp.73-83.

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MOVEABLE INFILLS FOR SEISMIC ENERGY DISSIPATION

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The purpose of moveable steel infills is to provide owners, such as hospitals, the freedom to redesign the interior of the building without the limitations imposed by non-moveable lateral load resisting systems. The scope of the research under way is to investigate two different types of moveable infill panels. The first involves using sheets of cold-formed corrugated steel to create a steel plate shear wall. Conceptually this idea is similar to using thicker hot-rolled flat plates as steel plate shear walls. That latter case was investigated by a number of researchers (e.g. Caccese et al. 1993, and Timler et al. 1998, to name a few). However, the thicker plates may have limited use in retrofit situations because of the high demands they place on the existing columns, requiring column reinforcement to prevent columns failure before the plates have a chance to dissipate energy through metallic yielding. To avoid this, light gauge corrugated steel plate infill could be used for energy dissipation. When properly designed, energy could be dissipated through plate yielding while the demand on the columns is kept below the critical value. The second type of infill that will be tested uses partition walls made of cold-formed steel studs and flat-strap cross bracing to dissipate energy in a seismic event. These walls may not be very strong individually, but several of them used in a typical building could be effective.

The challenge in making this idea work is to achieve satisfactory ways of connecting the panels to the steel frame in an existing building. Hence, to validate some of the developed strategies, full-scale partition walls will be tested inside an actual heavy steel frame. There have been tests performed on light gauge steel stud walls (Adham et al. 1990) but none of these incorporated a heavier steel frame. Both of these tests will be performed at the Structural Engineering Earthquake Simulation Laboratory at the University at Buffalo. The tests will be quasi-static, displacement controlled, and carried out until failure. A 250 kip static hydraulic actuator with a stroke of plus or minus four inches will supply the force. The actuator will be mounted on a large stiff steel reaction frame available in the lab.

Aside from the aforementioned benefits for seismic retrofit, these systems may prove to be competitive with concrete shear walls, which currently dominate the market for mid-rise structures. The author believes that these infill systems will provide adequate stiffness and

superior energy dissipation possibilities when applied to mid-rise structures. In these mid-rise situations steel plate shear walls made of hot-rolled flat plates may not be as economical as their concrete counterparts because of the increase in column sizes needed to ensure that energy dissipation is possible. Corrugated steel plate shear walls, however, could be competitive economically because the column sizes could be kept down. This new design philosophy would provide engineers with another option to resist lateral loads in mid-rise structures.

References

Caccese, V., Elgaaly, M., and Chen, R. (1993), "Experimental study of thin steel-plate shear walls under cyclic load", *Journal of Structural Engineering*, ASCE, Vol. 119, No. 2, pp. 573-587

Timler, P., Ventura, C., Prion, H., and Anjam, R. (1998), "Experimental and analytical studies of steel plate shear walls as applied to the design of tall buildings", *Structural Design of Tall Buildings*, Vol. 7, No. 3, pp. 233-249

Adham, S., Avanessian, V., Hart, G., Anderson, R., Elmlinger, J., and Gregory, J. (1990), "Shear wall resistance of lightgage steel stud wall systems", *Earthquake Spectra*, EERI, Vol. 6, No. 1, pp. 1-14.

11

USING COST-BENEFIT ANALYSIS TO EVALUATE MITIGATION FOR LIFELINE SYSTEMS

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The purpose of this research is to examine how cost-benefit analysis (CBA) can be utilized to evaluate the attractiveness of mitigation for lifeline systems subject to earthquake ground motion. CBA of lifeline systems is a more complicated process than for a single structure. Lifeline systems have unique characteristics that make the calculation of damage to the system difficult. First, the loss of function of the lifeline is dependent on many parts of the system spread across a large geographic region. Second, the damage to the system, measured in outage or service disruption, needs to be translated into dollar loss to the interested parties in the cost-benefit analysis. We propose a framework for the complicated CBA that can be used in conjunction with work being completed by other researchers at MCEER (Shinozuka et al., 2000; Chang et al., 2000).

Currently, we have considered two case studies to show the effects of the disruption of utility lifeline service on two stakeholders in a CBA analysis. First, the indirect economic loss to business owners in Shelby County, Tennessee, was studied (by Wendy Tao). Then, the cost to public agencies to shelter displaced residents in Alameda and Contra Costa Counties, California, was considered (by Chris Cyr). The study in Tennessee analyzed the interruption of electric power service from the Memphis Light, Gas, and Water Division (MLGW) of the area. The study in California analyzed the loss of water service from the East Bay Municipal Utility District (EBMUD). Ultimately, our intention is to undertake a sufficiently rich analysis of how mitigation can be utilized for particular lifeline systems, such as the EBMUD water distribution system (coordinated by Patricia Grossi). At this point, some unanswered questions and integration with other research still remain.

The task of making CBA a useful methodology is a challenging one. It requires bringing together scientists and engineers with social scientists to analyze a problem. It requires one to articulate the nature of the uncertainties associated with the recurrence interval of earthquakes of different magnitudes, as well as the confidence intervals surrounding the expected benefits and costs of

different alternative strategies. In a nutshell, it requires the integration of science with policy. The data and techniques are now available to undertake this type of integration with respect to earthquake mitigation (i.e., IT and GIS). Our challenge is to present the analyses to decision makers so they are willing to defend the proposed recommendation because it makes economic sense to them while satisfying their political concerns. Cost-benefit analysis provides a framework for accomplishing this important task.

References

Chang, S., Rose, A., Shinozuka, M., Svekla, W.D., and Tierney, K., 2000. "Modeling Earthquake Impact on Urban Lifelines Systems: Advances and Integration," *Research Progress and Accomplishments: 1999-2000*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York.

Shinouzuka, M., Grigoriu, M., Ingraffea, A.R., Billington, S.L., Feenstra, P., Soong, T., Reinhorn, A., and Maragakis, E., 2000. Development of Fragility Information for Structures and Nonstructural Components, *Research Progress and Accomplishments: 1999-2000*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York.

12

STRONG GROUND MOTION SIMULATION FOR EASTERN NORTH AMERICA

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Summary

Of great importance to earthquake engineering is the ability to predict strong ground motions, thus providing relevant input motions to research tasks e.g., modeling structural response, inventory analysis, fragility curves, rehabilitation strategies, demonstration projects etc.

The goal of this work is the synthesis of strong ground motion over the entire frequency range of engineering interest. For this task there have traditionally been adopted two separate approaches:

1.   The Stochastic (Engineering) Modeling Approach

2.   The Kinematic Modeling Approach

For both approaches one can adopt the "specific barrier model" (Pagageorgiou and Aki, 1983a, 1983b; Papageorgiou, 1988) to represent the earthquake source. Although various other source models have been proposed in the literature (such as the "ω^2-model", the Atkinson-Boore empirical model, Haddon´s model), the "specific barrier model" is the only one which defines in a clear an unambiguous way how the seismic moment should be distributed on the fault plane. Such a characteristic of the source model is necessary for preserving physical consistency when combining results obtained by the stochastic modeling approach with results from the kinematic modeling spproach, thus generating synthetic ground motions that comprise of all frequencies of engineering interest.

The stochastic modeling approach, with source model specified by the "specific barrier model", has been developed into a computer code that allows the predicion of various measures of strong ground motion (e.g., spectral response, peake response, etc.) as well as synthesis of time histories. The approach has furthermore been documented.

Although the stochastic modeling approach is expedient and therefore cost effective, it cannot be used to predict near source and low-frequency ground motions. For this reason, it is necessary to develop further the implementation of the kinematic modeling approach. It is proposed to investigate the problems described below.

Scattering effects of the lithosphere

Various models have been proposed in the field of Stochastic Seismology that may be valid for different ranges of epicentral distance. These need to sorted out and validated agains actual recorded data.

Variable size subevents

In its present form, the "specific barrier model" is composed of subevents that are identical in size. It is proposed to vary the size of the subevents about a mean value and investigate the consequences on the synthetic motions.

Subevent models

In the current version of the "specific barrier model", the subevent is represented by a circular crack, the rupture of which starts at the center and spreads out radially with a constant rupture velocity (symmetric model). It is proposed to investigate and implement models for which the rupture initates with a rupture of a free asperity, located at the center of a crack and spreads radially outward with constant velocity (asperity model). Such variety of subevent models allows for the more realistic modeling of the complexity of fault rupture.

Any simulation method should be validated by comparing the synthetic seismograms against the recorded ones for as many earthquake events as possible. Such comparisons will also provide a measure of the scatter/uncertainty (modeling and parametric uncertainty) of the predictions as well as any biases of the model. Such assessments of unertainty and bias are necessary if the simulation technique is to be used with confidence in earthquake engineering applications.

References

Papageorgiou, A. S. and K. Aki (1983a), *A Specific Barrier Model for the Quantitative Description of Inhomogeneous Faulting and the Prediction of Strong Ground Motion. I. Description of the Model,* Bull. Seism. Soc. Am. **73**, 693-722.

Papageorgiou, A. S. and K. Aki (1983b), *A Specific Barrier Model for the Quantitative Description of Inhomogeneous Faulting and the Prediction of Strong Ground Motion. Part II. Applications of the Model,* Bull. Seism. Soc. Am. **73**, 953-978.

Papageorgiou, A. S. (1988), *On Two Characteristic Frequencies of Acceleration Spectra: Patch Corner Frequency and f_{max},* Bull. Seism. Soc. Am **78**, 509-529.

13

SIMULATION STRATEGIES FOR RC BUILDINGS UNDER SEISMIC LOADING
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Research has been conducted to facilitate efficient and accurate seismic time history analysis of reinforced concrete frame structures. To simulate reinforced concrete frame structures accurately from basic material properties, a detailed mathematical and computational model is needed. However, since such a detailed model requires substantial calculation time for large-scale seismic analyses, an efficient technique representing material degradation, which do not sacrifice considerable accuracy, is suggested.

In order to improve the solution efficiency, a new modeling technique using non-linear interface elements is investigated. Investigation of failure modes of concrete frame structures due to earthquakes reveals that most of the failure occurs at the connection around joints and the base of columns. Based on these observations, the proposed new approach is to use interface elements at the face of joint connections (Interface Model). The idea is to concentrate the non-linearity of the structure into the sections at the joints (similar to the work of Rots et al. (1999) for a masonry structure). The interface elements are non-linear, and the rest of the structure is modeled as elastic material. The proposed approach can save computational time because the non-linear region of the model is significantly reduced, thus efficient large-scale seismic analyses of reinforced concrete frame structures can be performed efficiently. The non-linear constitutive model for Interface Model is implemented using a user-supplied subroutine in DIANA (1998).

The proposed Interface Model is verified by comparing simulation results with two different 3-story building shake table experiments conducted for NCEER (El-Attar et al., 1991; Bracci et al., 1992). Since non-linearity at hinge location is concentrated into the interface element, the amount of energy dissipation from the hinge region is assigned to the fracture energy in the interface element allowing for energy equivalency. Global behavior (story displacements) prediction of the simulations using the interface element approach was similar to both the experimental observations. Although local resutls (section moments) predicted in simulation was on the same order of magnitude as those in the experiment, local results in the simulation were less accurate

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[1] Research Assistant; with S.L. Billington, Assistant Professor, and A.R. Ingraffea, Dwight C. Baum Professor of Engineering; all from School of Civil and Environmental Engineering, Cornell University

than the global results. However, considering the simple geometric and material representation of the failure process, it is asserted that the overall simulation results using the interface element approach are reasonably accurate. The efficiency of the Interface Model is improved drastically compared to that of a simulation model using distributed non-linearity approach with a similar number of degrees of freedom. The interface model approach could perform the same seismic analysis twice as fast as the distributed non-linearity model.

It is concluded that the proposed Interface Model provides a new computationally efficient method of representing material degradation in large-scale analysis compared to the distributed non-linearity approach. With further calibration of the constitutive model, the Interface Model will increase the computational efficiency of large-scale reinforced concrete frame analyses while capturing (predicting) the local joint behavior.

References

Bracci, J.M., Reinhorn, A.M., Mander, J.B., (1992) "Seismic Resistance of Reinforced Concrete Frame Structures Designed Only for Gravity Loads: Part III – Experimental Performance and Analytical Study of a Structural Model," *Technical Report NCEER-92-0029*.

DIANA Finite Element Analysis User's Manuals rel. 7, (1998) eds. de Witte, F.C., TNO Building and Construction Research, Rijswijk, The Netherlands.

El-Attar, A. G., White, R. N., and Gergely, P., (1991) "Shake Table Test of a 1/8 Scale Three-Story Lightly Reinforced Concrete Building," *Technical Report NCEER-91-0018*.

Han, T.-S., (2001) "Interface Modeling of Composite Material Degradation," *Ph.D. Dissertation*, Cornell University.

Rots, J.G., Frissen, C.M., Feenstra, P.H., (1999) "Towards Design Rules Based on Snap-through Analysis of Composite Concrete/Masonry Structure," Proceedings of the ASCE 1999 Structures Congress, eds. Avent and Alawady, pp. 158-161.

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REHABILITATION IMPEDIMENTS AND INCENTIVES

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This study focuses on impediments and incentives for the adoption and implementation of innovative techniques, materials, and design for the rehabilitation of existing hospital buildings. During the first phase of the project, a review of existing literatures was done to develop models of innovation adoption by individuals, organizations, and communities. The second phase of the project involves interviews with administrators and other key individuals within hospitals who make decisions about disaster preparedness and planning and risk reduction. These interviews address respondents' risk perception, assessments of the criticality of various structural elements and non-structural systems, measures taken to maintain functionality in the event of a community-wide disaster, and the factors that influence the decisions to take those measures.

Three regions of the United States that face different levels of seismic risk were selected as research locations. These regions were: Southern California, Western Tennessee, and Eastern New York. Within each of these regions a large metropolitan area and a smaller municipality were targeted for data collection. During phase two of the study, interviews were conducted with hospital personnel in large metropolitan cities in Southern California and Eastern New York. In addition, personnel working in a hospital located in a smaller independent municipality within the same seismic risk area were also interviewed. Interviews in Western Tennessee will be completed this spring.

The targeted hospitals within each of these areas were selected based on several criteria. Among these was that the facility had to have an emergency room. The facilities were also of varying sizes (based upon number of licensed beds) and had different types of ownership. These characteristics will be taken into consideration during data analysis to determine if they are significant in disaster planning and mitigation strategies.

The interviews took the form of group interviews similar in structure to a focus group. Each group consisted of from three to nine participants. The participating individuals all play some role in the disaster-related decision-making process within their organization. Many were members of the Joint Commission mandated Safety Committee, though this was not always the case. The participants represent many areas within the hospitals such as administration, nursing,

physicians, and engineering.   The interview included open-ended, qualitative questions that addressed: the previous disaster experience of the facility and the individual participants; the types of external and internal disasters that the facility prepared for; guidelines and regulations that dictate levels of disaster preparedness and planning; the facility's role in a community-wide disaster; and any mitigation steps that have been taken to reduce the amount of damage and loss of functionality should a major disaster struck the area around the facility.

In addition to the qualitative aspect of the interview, the participants were also asked to complete a series of quantitative ratings for operational units, internal physical systems and external lifelines. The rating was done on a scale of 1 to 7, with one being essential and 7 being not at all essential. These ratings were to be based on the importance of the units or systems during the first 72 hours after a major community-wide disaster. Each respondent was asked to put their job title or position on each of the rating forms.

Analysis of these data will consist of qualitative analysis of the interview transcripts to identify major themes and other important information and quantitative analysis of the system ratings, including regression analysis. Elements such as facility size, ownership, level of seismic risk to geographic area, previous disaster experience, and position within the facility will be investigated to ascertain their significant to the decision-making process with in the facility as it relates to disaster planning and preparedness and the effect these have on the implementation of rehabilitation techniques or how they impede rehabilitation.

EXPERIMENTAL STUDY ON NONLINEAR INELASTIC SEISMIC RESPONSE

OF IRREGULAR STRUCTURES

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Irregular structures are commonly found in practice. Experience shows that buildings with irregularities are prone to earthquake loading, as demonstrated in many earthquake occurrences. Such buildings can undergo coupled lateral and torsional movement producing non-uniform displacement demands among their elements, and also stress and force concentrations on structural members. Since most buildings are designed to behave inelastically under earthquake loading, the yielding process also has significant effect in the structural response.

The current codes fall short to provide simplified analytical tools for irregular structures. Moreover, seismic response analysis for irregular structures requires complex analysis due to nonlinear and inelastic response, thus making it more difficult than for regular structures. Thus, there is an apparent need of developing a simple procedure for analysis based on more rigorous inelastic computations and experiments of the seismic response of irregular structure.

There are very few experiments in this subject. Therefore an experiment is carried out with the objective of to obtain a better understanding of the seismic response of nonlinear inelastic. The study will provide data for calibration and benchmark for the analytical studies. Moreover, the model will be used as testbed for new protective systems or energy dissipation devices. The experiment will be used also to understand the effect of irregularity on seismic response structures in the near collapse.

The model for the above experiment is a three-story steel structure with vertical irregularity in the form of two unequal towers. The structural model is built at 1:3 scale and is designed within the limitations of the shaking table. The undamageable gravity load system is separated from the damageable lateral load resisting frames. The model is built to have a versatile configuration with removable elements and access to additional devices. Preliminary analysis shows that the model has irregular distribution of properties of members in terms of curvatures and forces. Initial testing shows that permanent deformations and asymmetric mode shapes are visible. The

experiment (in progress) and further studies will be used to develop the simplified analytical tool for irregular structures, based on the "spectral capacity" concept.

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MODELING GENERAL EQUILIBRIUM ECONOMIC IMPACTS OF EARTHQUAKE-INDUCED LIFELINE DISRUPTIONS USING MPSGE APPROACH

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There are several different economic techniques that can be used to assess the economic impacts of "shocks" to a regional economy, including partial equilibrium analyses, econometric models, and general equilibrium analysis. However, a major shortcoming of the first two methods is that they fail to capture the interacting relationships between different economic agents and may thus provide biased economic loss estimates. In contrast, general equilibrium models, which embody the interactions between different economic agents, have been widely applied in evaluating the economic impacts of policy issues such as tax and trading policy reforms. Therefore, a general equilibrium model should be a better economic technique to capture the indirect and indirect economic losses of earthquake-induced lifeline disruptions.

Two different programming languages are commonly used in modeling general equilibrium problems. One is the "Generalized Algebraic Modeling System (GAMS)" developed by Alex Meeraus. The other one is " Mathematical Programming System for General Equilibrium (MPSGE)" developed by Thomas Rutherford. GAMS has been widely used in programming large-scale general equilibrium models since late 1980s, however, it becomes a cumbersome programming environment for the specification of complex nonlinear equations, such as nested CES functions. It needs a complex and tedious procedure to correctly translate a nested CES function into algebraic relations. In contrast, the MPSGE has a built-in translation procedure, which is not difficult to specify. Overall, the setup cost of producing a large-scale general equilibrium model and the cost of testing alternative specifications is much lower in MPSGE compared to GAMS.

The major objectives of this study include: (1) construct a well-defined computable general equilibrium (CGE) model to evaluate the general economic impacts of earthquake-induced lifeline disruptions; (2) develop appropriate procedures to test alternative scenarios, mainly focus on finding what assumptions and constraints should be included in the model to reflect possible

economic situations associated with earthquake-induced lifeline disruptions and how to translate them into algebraic relations.

In this study, the original programming approach used to model general equilibrium economic impacts of earthquake-induced lifeline disruptions was GAMS. We have found that it was hard to correctly translate our complex nested CES functions into algebraic relations and conduct counterfactual simulations after setting up the model, especially for the very-short-run and short-run general economic impacts of lifeline disruptions. This was highlighted in cases where we needed to specify very low substitution elasticities, reflecting the very short-term reality. The GAMS system runs into infeasibilities at very low elasticity levels, since the system tends to evaluate some divisors as arbitrarily close to zero. The MPSGE approach avoids this by automatically dropping infeasible terms and providing local solutions. So after switching to MPSGE, we are able to do a wide variety of counterfactual simulations, including fixed prices of electricity supply, minimum electricity supply to households and sectoral differential electricity supply in both very short run and short run scenarios.

BEHAVIOR OF BURIED PIPE JOINTS SUBJECTED TO DYNAMIC MOTIONS

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The objective of this project is to experimentally test various types of pipe joints and pipe material to determine their dynamic axial strength capacities and axial stiffness characteristics. 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 to fight fires and for domestic purposes. Data on pipeline damage resulting from various earthquakes have shown a serious vulnerability to earthquakes. Therefore, pipelines need to be examined closely and their response to seismic motions needs to be researched in order to develop criteria that will mitigate potential damage and the consequences of failures that can potentially occur.

This research project has three major goals: 1) develop procedures and methodologies for the effective dynamic testing of piping joints, 2) experimentally determine the dynamic axial force resistance capacities, and the axial stiffness characteristics of pipeline joints, and 3) develop potential damage assessment procedures and other analytical methods to be used in seismic resistant design and mitigation.

The dynamic pipe joint testing assembly is based on the use of the University of Nevada, Reno Structures Laboratory shake-table. The assembly consists of reaction blocks to provide end anchorage for the pipe joint specimens, a steel restraint frame to provide a safety mechanism to contain any large lateral movement during loading, and a steel pipe loading arm that provides an extension from the shake-table to the test specimen. Seismic motion records were selected from a database of motions recorded during the 1994 Northridge earthquake and were used to provide time dependent displacement control to the shake table. The displacement motion was transferred to the joint specimen, simulating strains that would be transferred to the pipe from the surrounding soil during an earthquake. The displacement motion magnitudes were applied in increasing levels of amplitude until failure occurred or other limits. The resulting load-displacement values were recorded and used to produce maximum force capacities and stiffness characteristics of the joint which can be used in analytical investigations.

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SENSIVITY ANALYSIS OF DYNAMIC SYSTEMS SUBJECT TO SEISMIC LOADS

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A typical problem in earthquake engineering consists of a dynamic system, such as a building, subjected to a seismic ground motion input. In practical problems, the parameters of both the input and the system are not well known. For example, parameters of the damping, structural stiffness and applied earthquake load are almost never known accurately. Our work investigated a quantitative measure of the seismicity of the response of the system to an input or system parameter. The measure, termed the sensitivity factor, is defined as the derivative of the response with respect to the parameter value. It can be used to identify critical parameters, estimate the effect of a small change in the parameter value, and select optimum values for the parameter.

Methods that have been used to calculate the sensitivity factor include the finite difference method, the adjoint system method, and the direct differentiation method. We selected the direct differentiation method as it is faster and more accurate than the other methods. To apply the direct differentiation method, the governing equation of the system was differentiated to obtain the governing equation of the sensitivity factor. This equation could be solved numerically by a similar method to that used to calculate the system response. The errors in the numerical method were investigated and found to be not significant.

The method was implemented in two computer programs: (1) an entire program to calculate both displacement and sensitivity factors was written using the MATLAB code, for a relatively simple, beam-element level of analysis; and (2) new subroutines were added to an existing finite element analysis program, DIANA, for more detailed analysis. An example was given illustrating applications of the sensitivity factors to a hospital in Buffalo. Both the primary structural and a secondary piping system were considered. The sensitivity factors were used in identification of critical parameters, selection of retrofit strategies, and calculation of fragility curves.

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DEVELOPMENT, TESTING AND MODELING OF HIGHLY EFFECTIVE ENERGY DISSIPATION SYSTEM CONFIGURATIONS

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The use of supplemental devices as energy dissipation systems for improved seismic performance of building and bridge structures has gained increasing interest in the past few decades. The hardware utilized includes yielding steel devices, friction devices, viscoelastic solid devices and mostly viscous fluid devices. A comprehensive review of the characteristics of various devices and their implementation to building structures can be found in Constantinou et al., 1998, Soong and Dargush, 1997.

Traditionally, supplemental devices – also known as 'dampers' – are being implemented in-line with diagonal bracing or as horizontal elements atop chevron bracing. However, these configurations may not always be desirable or the most efficient placement. For example, stiff structural systems under seismic load are characterized by small interstory drifts and velocities. Damping devices installed diagonally or horizontally lead to device displacements that are less than or equal to the interstory drift. For fluid viscous devices, small device displacements imply large forces, which in turn necessitate costly damper designs. Therefore, stiff structural systems may, in most cases, not be suitable for the application of energy dissipation systems.

The recently developed 'Toggle-Brace-Damper' system (Constantinou et al., 2001) provides an attractive alternative to overcome the problems associated with stiff structural systems. As the name implies, this innovative configuration is based on the toggle mechanism that magnifies the damper displacement for a given structural drift so that sufficient energy is dissipated with a reduced requirement for damper force (reduced damper volume, thus reduced cost). Alternatively, the system can be interpreted as a mechanism that magnifies the damper force through the toggles and delivers it to the structural frame.

The toggle-brace-damper theory has been successfully verified by experimental and analytical studies. The system is suitable for stiff as well as for flexible structural systems subjected to ground motions. It is also suitable for the reduction of wind-induced vibrations. Three buildings

in the US, so far, have been designed with toggle-brace-damper systems and are currently under construction.

An additional consideration associated with the application of energy dissipation systems is that in some cases they may conflict with architectural requirements such as open space since they occupy an entire bay in a frame. Taking this factor into consideration, the 'Scissor-Brace-Damper' system was developed as a variant of the toggle-brace-damper system. While the toggles (scissor-braces) achieve the desired displacement magnification, the compactness and near-vertical installation of the system provide unique 'minimal obtrusion' feature.

The validity of the scissor-jack-damper concept was confirmed via strong-floor and shake-table experiments on a ½-scale steel frame model. The description of the system along with samples of experimental results on its behavior can be found in Constantinou and Sigaher, 2000. The effectiveness of the configuration is evident by the significant increase in the damping ratio, and the reduction in drift and acceleration compared to those of the bare frame.

References

Constantinou, M.C., T.T. Soong, and G.F. Dargush (1998), *Passive Energy Dissipation Systems for Structural Design and Retrofit,* MCEER Monograph, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY.

Soong, T.T. and G.F. Dargush (1997), *Passive Energy Dissipation Systems in Structural Engineering,* J. Wiley, England.

Constantinou, M.C., P. Tsopelas, W. Hammel, and Ani N. Sigaher (2001), *Toggle-Brace-Damper Seismic Energy Dissipation Systems,* Journal of Structural Engineering, ASCE, 127(2), 105-112.

Constantinou, M.C. and Sigaher, A. N. (2000), *Energy Dissipation System Configurations for Improved Performance,* Proceedings of the 2000 Structures Congress and Exposition, ASCE, May 8-10, 2000, Philadelphia, PA.

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EXPERIMENTAL INVESTIGATION OF P-Δ EFFECT TO COLLPASE DURING EARTHQUKES

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Statement of Problem and Motivation for Research

The arbitrary lateral drift limits prescribed by modern design codes to limit non-structural damage also indirectly ensure that structural performance is minimally affected by the effect of gravity on the lateral force resistance of structures (a.k.a. P-Δ effect). However, these conventional drift limits are progressively being eliminated and replaced by other performance-based limits. As inelastic behavior is relied upon to a greater extent in the dissipation of seismic input energy, accurate quantification of the destabilizing effect of gravity is becoming more significant in structural design. This is particularly true given that advances in construction technology are allowing the fabrication of more slender structures than in the past. As a result, it is desirable to investigate the behavior of structures to enhance understanding of the condition ultimately leading to their collapse, and to ensure public safety during extreme events. While much experimental and theoretical research has been conducted to determine the strength and deformations capacities of elements and structures, few experimental studies have pushed the shake table tests up to collapse.

Proposed Solution

Experimental results were generated through a program of shake-table testing of simple frames through collapse. Fifteen specimens having various properties were tested in an attempt to identify some of the general parameters responsible for trends in behavior due to P-Δ effects on SDOF structures. Every effort was made to ensure that the experimental data is thoroughly documented (geometry, material properties, initial imperfections, detailed test results, etc.) such that the tests can be used at a later time as a benchmark to which analytical models can be compared. The intent is to make results available to other researchers to assist in the development or validation of analytical tools to model the inelastic dynamic behavior of structures up to collapse.

Results and Applications

In a forthcoming MCEER research report, an example is presented of how to use the specimen test data for the purpose of comparison with a basic analytical model. The effects of inherent damping estimates on analysis results are discussed, along with a preliminary investigation of behavioral trends of the shake table results. Peak responses are compared with limits proposed to minimize P-Δ effects in bridge piers.

This research demonstrates that the stability coefficient, θ, has the most significant effect on the behavior of the structure. As θ increases, the maximum attainable ductility, sustainable drift, and spectral acceleration, which can be resisted before collapse, all decrease. When this factor is larger than 0.1, the ultimate values of the maximum spectral acceleration, displacement ductility, and drift reached before collapse were all grouped below values of 0.75 g, 5, and 20%, respectively. Stability coefficient values less than 0.1 tend to increase each of those response values significantly. Furthermore, it is noteworthy that all specimens exceeded the strength predicted by inelastic moment amplification factors, and some specimens even exceeded the calculated strength based on first order effects.

In this year's Research Progress and Accomplishments report (Vian et. al.), some basic results from this testing program are presented. In addition, some formulations and preliminary results in the development of analytical tools capable of handling the large inelastic excursions exhibited by the specimens in the testing program are presented.

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POLYMER MATRIX COMPOSITE (PMC) INFILL WALLS FOR SEISMIC RETROFIT

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Background/Motivation

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. In recent years, PMC materials have been selectively used to retrofit structural elements such as wrapping of concrete columns to enhance strength and ductility. However, the use of such material in a large-scale structural system to resist seismic excitations has been very scarce. The limited use is mainly due to economical factors, lack of standards for PMC wall design and questions related to longevity. 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.

Previous Research

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 seismic 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 typically exhibits changes in the magnitude and distribution of stresses in the frame members. For the infill wall system with composite material, Gasparini, Curry and Debchaudhury (1981) presented the damping of frame with visco-elastic infill panels for increasing damping and

minimizing vibration. Recently, Haroun, Ghoneam, and Essam (1997) studied the effects of strengthening and repairing masonry infilled reinforced concrete frames by fiberglass composite.

Justification & Objective

This research addresses the design, fabrication and testing of polymer matrix composite (PMC) infill walls for use as a seismic energy dissipation strategy. For the proposed technique, we have introduced a multi-layer system allowing in plane shear and therefore sliding along specific layers to take place upon loading the frame. Finally, we anticipate the damage to be controlled in such layers, thereby, allowing us to reuse the wall components. In order to meet the best result, we plan to consider several design cases to increase the capacity of the damaged layers by utilizing a combination of (1) visco-elastic and Honeycombs, and (2) visco-elastic and thermoplastic polymer materials.

Practically, the new infill PMC wall system may lend itself to the application in seismic areas for three reasons. First, the material has a high stiffness-to-weight and strength-to-weight ratio. The addition of infill walls into an existing building will not significantly alter the weight of the structure; therefore, existing foundations will remain sufficient to carry the gravity loads. Secondly, due to the lightweight of the walls combined with the idea of prefabrication, rapid construction will be achieved and interruption to the occupancy of the structure will be minimal. Third, polymers have good damping characteristics. In designing the walls, polymers that exhibit significant visco-elastic behavior can be utilized in specific layers of the wall.

Actually, there are several technical and economical challenges associated with the use of PMC in structural applications such as buildings. Some of the most serious questions surrounding the viability of PMC structures relate to the availability of tested PMC structural systems to resist seismic loads, the cost of their construction, availability of standards for their design, practical connection details, and the feasibility of their construction. In the design process, there were many design variables that affected both the performance and cost of the infill wall. Having a large number of design variables and a wide range of materials that can be used presented a great challenge. In this research, a conceptual optimum design of an infill wall is presented using the finite element analysis. As mentioned above, 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. Also, the other aspects of the PMC infilled frame system are considered: (1) studying of properties of FRP and visco–elastic materials, (2) manufacturing the structural PMC walls, and (3) testing the wall systems when incorporated in a steel frame having semi–rigid bolted connections.

Progress of Research

1. Material Testing

Fiber reinforcing polymer (FRP) materials were tested to evaluate the mechanical properties that are needed for analysis and design. The evaluation of the mechanical properties of composites includes their strength and stiffness characteristics. For tension and compression test, each ultimate tensile, compression strengths, Young's modulus and Poison's ratios 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 evaluating in–plane shear properties, the 2–rail shear test method, as described in ASTM D4255, was used in this research. Also, the visco-elastic material was tested in this research. It 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 behavior of honeycomb materials, pure shear test was considered as depicted in Figure 1.

(a) FRP Material Test   (b) Visco-elastic Material Test

Figure 1: The Configuration of the Material Tests

2. Design and Construction of Composite Panel

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. Based on this concept, the wall system was designed with three panels forming the entire wall thickness as shown in Figure 2(a). 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. For optimum design based on the cost and performance, we have relied on detailed 3–D finite element models using ABAQUS. 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.

Detailed design drawings 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 Figure 2(b) was constructed considering the most practical and commercial conditions. After manufacturing, the wall was installed in a frame (2500x2400 mm, W8x24 column and W8x21 beams) having semi–rigid bolted connections to be tested.

     (a) Multi-layer System   (b) Constructed Wall Shape

Figure 2: The Configuration of the PMC Infill Wall

3. Description of the Experiments

In the experimental phase, testing of steel frame with and without composite infill wall is planned as shown in Figure 3. Steel frame and composite infilled frame specimens were tested under monotonic and cyclic in–plane loading. To apply lateral force, a 250–kips MTS hydraulic actuator with a stroke of ±4 inch was used. All cyclic tests were performed under displacement control.

(a) Steel Frame Test Setup   (b) The PMC infilled Frame Test Setup

Figure 3: The Configuration of the Bare Frame and PMC Infilled Frame

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 include 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 hysteretic behavior and the corresponding strength deterioration and stiffness degradation of steel frame and composite infilled frame using displacement transducers, and (4) buckling of the PMC inner panel.

Conclusions

Several important results concerning composite energy dissipation of infill panel may be stated. Based on numerical and experimental results so far, the main conclusion may be drawn as follows:

- Initial stiffness of the PMC infilled frame is 3 times that of the steel frame. And, the load-carrying capacity of the PMC infilled frame is 4 times that of the steel frame.
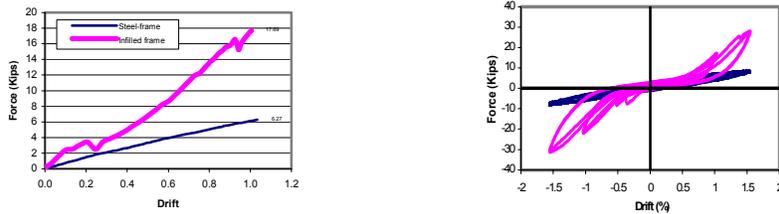
Figure 4: The Results of the PMC Infilled Frame Test (Monotonic & Cyclic loading)

Table 1: The Comparison of Each Component

	Infilled Frame	Steel Frame	PMC Infill Wall
Stiffness (Kips/in)	15.65	5.55	10.1

- The total energy dissipation capacity of the PMC infilled frame subjected to small deformation of visco-elastic layers is 3 times that of the steel frame. The contribution of the PMC infill wall is 65% of overall energy dissipation performance without significant degradation for the stiffness and strength. It is evident that energy dissipation capacity of the PMC infilled system may be larger where the visco-elastic layers contribute a relatively large deformation.
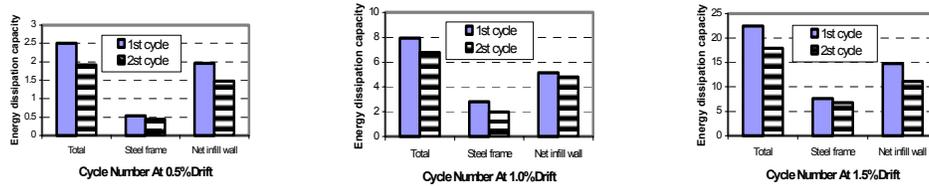

Figure 5: The Energy Dissipation Capacity for the PMC Infilled Frame (Kips-in)

- Main damage of the PMC infill wall is elastic buckling of the inner part at high drift values (3%). It is recommends that a stiffer strong core material for the design and construction should improve the performance of the PMC infill wall.

- Initial gaps affect the behavior of the PMC infilled frame. Like infilled frame with traditional material, the stiffness of the PMC infilled frame is reduced through the introduction of initial gap. However, to approach an ideal construction with no initial gaps, it is preferred to reduce the size of the initial gap as much as possible in the PMC infill wall.

Table 2: The Results of Initial Gap Effect for the PMC Infill Wall

	No side Gap	Initial Gap (0.05 in)
Stiffness (Kips/in)	18.3	16.2

- Based on analytical modeling of the PMC infill wall, considerable stiffness as well as acceptable strain in the energy dissipation material have been considered. However, from the experiments, it is evident that visco-elastic layers provide a large fraction of the total lateral stiffness in the PMC infill wall system. Thus, more studies for the visco-elastic layers are still needed.

- At high drift value, local crushing failure took place at wall corner, a better detail will be needed to overcome this kind of failure.

(a) Behavior of the PMC Infill Wall   (b) Failure Modes of the PMC Infill Wall
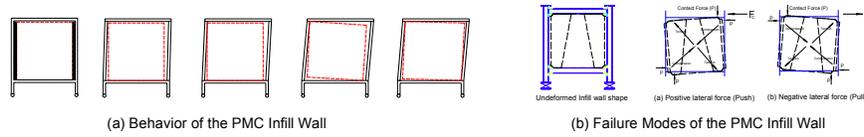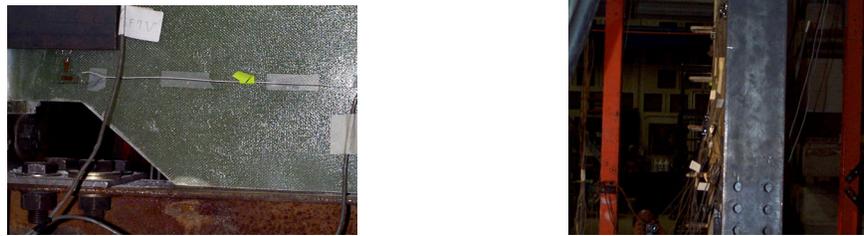
Figure 6: The Behavior and Failure Modes of the PMC Infill Wall

(a) Corner Crushing Damage   (b) Elastic Buckling of the PMC Infill Wall

Figure 7: The Damages of the PMC Infill Wall

Future Study

Additional research is required based on the first test. Especially, advanced modeling related to the structural performance and economic application will be needed. Practically, light and flexible building systems often require specific design features for limiting structural damage, structural control for the vibration, and maximizing occupant comfort and safety. Consequently, our research plans include the following:

- Study the behavior of several composite infill wall systems from the experiment.
- Develop simplified analytical modeling from the experimental results.
- Study and test advanced interfacing materials under seismic excitation.
- Redesign an advanced composite infill panel considering performance and cost.
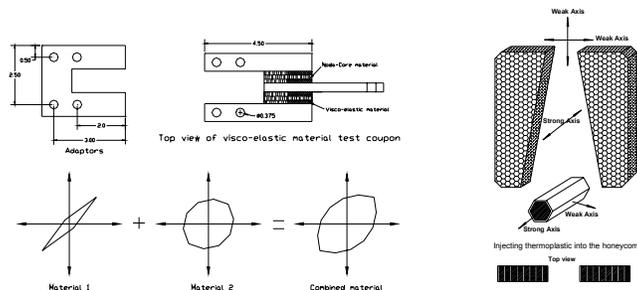
Figure 8: The Examples of the Advanced Interfacing System

- Dynamic analysis and testing of the composite infill wall system

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▲ Cover Images

Cover Images (From left, top row): Keith Kesner, Cornell University, works on part of an exhibit on earthquake engineering displayed at the Ithaca Science Center; group photograph from an SLC meeting at Cornell University, August 7-8, 2000; (middle row): block cable system used in research by Diego Lopez Garcia, University at Buffalo; Yingjuan Wang, Rensselaer Polytechnic Institute, explains her research to Clifford Astill, National Science Foundation (NSF) during an NSF site visit to MCEER in June 2000; active control system implemented in a tower, described by Shih-Yu Chu, University at Buffalo; students involved in EERI, SLC and/or REU programs held a barbecue at the Universtiy at Buffalo in July 2000; (bottom row): James Mason, Cornell University, discusses fibers used in his research during an SLC meeting; and Ali Rejaie, University of Southern California, investigates how remote sensing imagery data can be used for disaster management.

▲ Acknowledgements