UNIVERSITY
OF NEVADA
RENO

National Science Foundation

Natural Hazard Mitigation Grantees Workshop

Lake Tahoe, Nevada
April 27-28, 1995

Sponsored by the National Science Foundation and the University of Nevada, Reno

Workshop Directors:

M. Saiid Saiidi, Professor
E. Manos Maragakis, Professor & Chair
Civil Engineering Department
University of Nevada, Reno

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Abstract: To facilitate timely exchange of information among researchers whose projects are funded by the Natural Hazard Mitigation programs at the National Science Foundation, a workshop was held at Crystal Bay (Lake Tahoe), Nevada, on April 27 and 28, 1995. Five NSF programs, four within the Earthquake Hazard Mitigation (EHM) program, and the fifth and Natural and Technological Hazard Mitigation were represented at the Workshop. The EHM programs were: (1) Architectural and Mechanical Systems, (2) Earthquake Systems Integration, (3) Siting and Geotechnical Systems, and (4) Structural Systems. Representative researchers from these programs were invited to give a brief presentation about their research.

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NATURAL HAZARD MITIGATION
NSF GRANTEES WORKSHOP

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8:30 a.m.	**OPENING SESSION**	
	Moderators:	L. Wang, *Old Dominion University, Norfolk* M. Saiidi, *University of Nevada, Reno*
8:30 a.m.	**Introduction:**	M. Saiidi, *University of Nevada, Reno*
8:35 a.m.	**Welcome Remarks:**	B. Cathy, Associate Vice President for Academic Affairs, University of Nevada, Reno.
8:45 a.m.	**Workshop Details:**	E. Maragakis, University of Nevada, Reno
8:55 a.m.	***"An Overview of NSF Program in Natural Hazard Mitigation"*** W. Anderson, National Science Foundation, Arlington	
9:15 a.m.	***"Directions for Research in the Next Decade"*** J. Jirsa, University of Texas, Austin	
9:35 a.m.	**Discussions**	
9:45 a.m.	**BREAK**	
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	"A Note on Shear Strength of Reinforced Concrete Bridge Piers" M. Sozen, Purdue University, West Lafayette; C. Konwinski, A. Pan, and J. Ramirez	
10:27 a.m.	***"Seismic Evaluation of Concrete Dams Using Nonlinear Analysis"*** G.	

Fenves, University of California, Berkeley

10:39 a.m. ***"Earthquake Engineering and Structural Control Research at Virginia Polytechnical Institute and State University- An Overview"*** M.P. Singh, Virginia Polytechnic Institute

10:51 a.m. ***"Collapse Studies of Mission-Gothic Undercrossing Bridge for Northridge Earthquake"*** F. Cheng and K. Lou, University of Missouri-Rolla, Rolla

11:03 a.m. ***"Preliminary Evaluation of the Response of Base-Isolated Buildings in the Northridge Earthquake"*** J. Inaudi, J. De la Llera, F. Tajirian, and I. Aiken, Seismic Isolation Engineering, Inc., Berkeley

11:15 a.m. ***"Ultimate Limit States for Storefront Architectural Glass under Simulated Earthquake Loadings"*** R. Behr, University of Missouri-Rolla, Rolla

11:27 a.m. ***"Seismic Behavior of Steel Shear Panels and Frame-Wall Systems"*** M. Xue and L. Lu, Lehigh University, Bethlehem

11:39 a.m. **Discussions**

12:00 p.m. **LUNCH**

1:30 p.m. **SESSION II - SYSTEMS INTEGRATION**

Moderators: C. Astill, *National Science Foundation, Arlington*
J. Jirsa, *University of Texas, Austin*

"Earthquake Hazard Response in the United States and Japan: A Cross-Cultural Survey" R. Palm, University of Oregon, Eugene

1:42 p.m. ***"Eqhazmat: A Data of Hazardous Material Incident Occurrence during Earthquakes"*** G. Selvaduray, San Jose State University, San Jose

1:54 p.m. ***"Disaster Warning Responses by Tourists and Other Transient Populations"*** T. Drabek, University of Denver, Denver

2:06 p.m. ***"A Decision Support System for Predicting Casualties in Multi-Hazardous Environments"*** N. Stubbs, Texas A&M University, College Station

2:18 p.m. ***"Catastrophe Risk Management- Earthquake Insurance"*** W. Dong, Risk Management Solution Inc., Menlo Park

2:30 p.m. **Discussions**

2:45 p.m. **BREAK**

3:00 p.m. **SESSION III - SITING AND GEOTECHNICAL SYSTEMS**

Moderators: N. Sabadel, *National Science Foundation, Arlington*
J. Anderson, *University of Nevada, Reno*

"Implications of the Northridge Earthquake for Strong Ground Motion from Thrust Faults" P. Somerville and C. Saikia, Woodward-Clyde Federal Services, Pasadena; D. Wald and R. Graves, United States Geological Survey, Pasadena

3:12 p.m. ***"Geotechnical Earthquake Hazards Using a Spatial Analysis System"*** J. Frost and R. Luna, The Georgia Institute of Technology, Atlanta

3:24 p.m. ***"Damage to Landfills from the Northridge Earthquake"*** E. Kavazanjian and N. Matasovic, GeoSyntec Consultants, Huntington Beach; J. Bray, A. Augello, and R. Seed, University of California, Berkeley

3:36 p.m. ***"National Geotechnical Experimentation Sites Program"*** J. Benoit and P. de Alba,University of New Hampshire, Durham

3:48 p.m ***"Earthquake Ground Motion Modeling in Large Basins"*** J. Bielak, Carnegie Mellon University, Pittsburgh

4:00 p.m. ***"Three-Dimensional Slope Stability"*** T. Stark, University of Illinois, Urbana

4:12 p.m. ***"Project VELACS: RPI Contribution"*** A. Elgamal and R. Dobry, Rensselaer Polytechnic Institute, Troy

4:24 p.m. **Discussions**

4:30 p.m. **BREAK**

5:00 p.m. **DEPART FOR RENO** (Vans will meet in front of the hotel)

6:00 p.m. **HOSTED COCKTAIL AT THE BRIDGE ENGINEERING LAB, UNIVERSITY OF NEVADA, RENO**

7:00 p.m. **DEPART FOR GALENA FOREST INN**

7:30 p.m. **DINNER AT GALENA FOREST INN**

9:00 p.m. **DEPART FOR THE HOTEL**

FRIDAY, APRIL 28, 1995

8:00 a.m. **CONTINENTAL BREAKFAST**

8:30 a.m. **SESSION IV - STRUCTURAL SYSTEMS II**

Moderators: W. Anderson, *National Science Foundation, Arlington*
S. Wood, *University of Illinois, Urbana*

Keynote Presentation - System Integrated Approach in Earthquake Hazard Mitigation: G. Lee, State University of New York, Buffalo

8:52 a.m. ***"Evaluation of Two Precast Parking Garages Damaged during the 1994Northridge Earthquake"*** S. Wood, University of Illinois, Urbana; J. Stanton, University of Washington, Seattle; N. Hawkins, University of Illinois, Urbana

9:06 a.m. ***"Prioritization for Rehabilitation of Buried Lifelines"*** L. Wang, Old Dominion University, Norfolk

9:18 a.m. ***"On the Scale Analysis of Electrorheological Fluid Dampers"*** F. Gordaninejad and R. Bindu, University of Nevada, Reno

9:30 a.m. ***"Computational Issues Associated with the Analysis of Actively Controlled Structures"*** H. Smith and A. Schemmann, Stanford University, Stanford

9:42 a.m. ***"Intelligent Control of Structures"*** R. Shoureshi, Colorado School of Mines; M. Bell

9:54 a.m. ***"Consequences of Damage to Bridges in the Northridge Earthquake"*** A. Kiremidjian, N. Basoz, K. Law, and S. King, Stanford University, Stanford

10:06 a.m. ***"Seismic Damage Assessment of Structures using Cumulative Damage Model"*** Y. Chai, University of California, Davis

10:18 a.m. **Discussions**

10:30 a.m. **BREAK**

10:45 a.m. **SESSION V - NON-EARTHQUAKE HAZARD MITIGATION**

Moderators: S.C. Liu, *National Science Foundation, Arlington*
M.P. Singh, *Virginia Polytechnic Institute*

"An Expert System for Building Vulnerability Assessment in Windstorms" D. Reed, University of Washington, Seattle; T. English and K. Mehta, Texas Tech University, Lubbock

11:00 a.m. ***"Mid Continental Floods"*** K. Georgakakos, Hydrologic Research Center, San Diego

11:12 a.m. ***"Impact of Design Philosophies on Disruption to Critical Environmental Infrastructure from the Great Flood of 1993"*** D. Sanders, University of Missouri-Rolla, Rolla

11:24 a.m. ***"A Distributed Model for Real-Time Rainfall-Runoff Forecasting"*** R. Bras, Massachusetts Institute of Technology, Cambridge

11:36 a.m. ***"Identification and Characterization of Collapsible Soil"*** S. Houston and W. Houston, Arizona State University, Tempe

11:48 a.m. ***"Cooperative Programs in Wind Engineering"*** K. Mehta, Texas Tech University, Lubbock

12:00 p.m. **Discussions**

12:15 p.m. **SESSION VI - CLOSING SESSION**

Moderator: E. Maragakis, *University of Nevada, Reno*

Closing Remarks: S.C. Liu, *National Science Foundation, Arlington*

12:25 p.m. **Closing Remarks:** M. Saiidi, *University of Nevada, Reno*

12:30 p.m. **LUNCH**

NATURAL HAZARD MITIGATION
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OPENING SESSION

Introduction
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E. Maragakis, University of Nevada, Reno

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W. Anderson, National Science Foundation, Arlington

"Directions for Research in the Next Decade"
J. Jirsa, University of Texas, Austin

DIRECTIONS FOR RESEARCH IN THE NEXT DECADE

James O. Jirsa[1]

Abstract

In June 1993, A workshop was held in Washington, D. C. The purpose of the Workshop was to review accomplishments resulting from National Science Foundation funding in Earthquake Engineering and Earthquake-Related Earth Sciences, to assess directions and needs in the next decade, and to examine policies and administrative procedures which may impact on the role of NSF in the National Earthquake Hazards Reduction Program. A summary Report (1) and the Proceedings (2) of the workshop have been prepared. The intent of this brief overview is to summarize discussions from the Workshop and to outline briefly the key accomplishments of past funding and critical areas for research in the next decade.

Background

The Earthquake Hazards Reduction Act of 1977 stated that "It is the purpose of the Congress to reduce the risks of life and property from future earthquakes in the United States through the establishment and maintenance of an effective earthquake hazards reduction program." The National Science Foundation's support of research in earthquake engineering in the areas of siting, design, and societal response and of earthquake-related earth science in the areas of geophysics, seismology, and geology have been key elements in the National Earthquake Hazards Reduction Program. The Workshop was organized to review the role of NSF in NEHRP and to suggest areas and approaches that would be appropriate for the next decade considering the current state-of-the-art and the needs as manifested by the losses resulting from recent events.

Organization of Workshop

Forty participants representing research, practice, and public policy interests were invited to attend the Workshop. Brief presentations by representatives of the NEHRP agencies and by the NSF program managers provided the participants with an overview of NEHRP activities. To provide broad input from the research community on the issues identified above, questionnaires were sent to current NSF Principal Investigators and were available to the participants prior to the workshop. Discussion groups were charged with identifying areas where

[1]Janet S. Cockrell Centennial Professor of Engineering, Ferguson Structural Engineering Laboratory, 10100 Burnet Road, University of Texas, Austin, Texas 78758

major accomplishments have resulted from NSF support and to define areas where future work is needed. The discussions are summarized below.

Significant Achievements Resulting From NSF Support

Improvements in building codes resulting from a better understanding of the behavior of structural systems.

Development of analytical capabilities for non-linear and complex analysis of engineering, soil-structure, and seismological problems.

Improvements in the characterization of the effects of earthquake sources, propagation paths, and local site conditions on strong ground motion.

Better reliability and risk assessment techniques for determining damage and casualty rates for a variety of lifeline and structural systems have lead to improved planning capabilities.

Introduction of structural control capabilities (base isolation, protective systems) for modifying response of buildings and bridges.

Establishment of a base of engineering knowledge which has broad and immediate applicability to newly developing activities and demands for rebuilding the nation's civil infrastructure.

Transfer of knowledge to the earthquake community through support of activities such as "Learning from Earthquakes" to provide feedback on important lessons learned from the field and new developments from research.

Educating and training several generations of experts in earthquake engineering and earth sciences who have provided technical leadership recognized worldwide.

Research Needs for the Next Decade

Develop techniques for evaluation of existing structures and procedures for cost-effective repair and/or strengthening.

Promote collaborative research between structural engineers, geotechnical engineers, and earth scientists to improve capabilities for accurately and consistently determining response of engineered systems subjected to strong ground motion.

Develop improved analytical techniques for dynamic, non-linear response of complex, unconventional materials, structures, lifelines, or earthquake processes.

Improve understanding of plate tectonics and fault mechanics for advancement of the science and technology of earthquake prediction.

Develop site specific strong ground motion and soil data through focussed regional efforts involving seismologists and geotechnical specialists.

Improve data collection and analysis of economic losses and casualties for developing emergency planning and hazard reduction policy.

Replace aging and obsolete instrumentation and laboratory equipment in all areas of research.

Encourage development of innovative control systems and implementation of new materials and processes.

Emphasize the implementation of research findings through workshops, annual technical research summaries, post earthquake studies, and cooperative international activities.

Role of NSF

The participants discussed the relationship of NSF activities to other NEHRP activities and the perceived role of NSF. Several broad issues that emerged from the discussions are summarized below:

1. NSF's unique mission within NEHRP has been and should continue to be fundamental research and development of new technology. NSF is the only NEHRP agency oriented toward the training of experts in earthquake engineering and earth science. The long term benefits of this activity to the nation are immeasurable.

2. Better coordination between the NEHRP agencies is needed and each should have a well-defined role in NEHRP. Coordination is needed at the working levels of the agencies. An advisory panel within NSF comprised of practicing engineers and earth scientists as well as their counterparts in research would help to set directions and goals. Such a panel could also improve integration of NSF research with other NEHRP activities. A clear understanding of each agency's role would minimize the perceived shortcomings of the current program.

3. NSF must make more of an effort to publicize it's contribution to the NEHRP effort. The initiation and development of new technology and new approaches in earthquake hazard mitigation have resulted from NSF-supported research but are not associated with NSF by the public or by policy makers.

4. A substantial portion of NSF funds must be retained for unsolicited, curiosity-driven proposals. Researchers must be given sufficient time and funding to accomplish their goals. Grants of longer duration with funding appropriate to the objective must be provided if meaningful results are to obtained even if such grants reduce the number of awards made each year.

The report includes prepared comments by selected participants to examine accomplishments and future research needs from the view of the researcher, the user, and the policy maker. Brief accounts of working group discussions are included in the report. An outline of the workshop agenda and working group assignments is included. A summary of the responses to the questionnaires circulated to Principal Investigators on NSF projects is presented.

Acknowledgements

The author acknowledges the support of the other members of the Steering Group (K. Aki, M. S. Agbabian, and G. W. Clough) and of the National Science Foundation in making it possible for this Workshop to be held. Division Directors William Anderson and James Whitcomb and Program Managers S. C. Liu and M. P. Singh provided valuable guidance and support for the Workshop and for the publication of the Report and Proceedings.

References

1. Report on a Workshop, "Directions for Research in the Next Decade: National Science Foundation Programs in Earthquake Engineering and Earthquake-Related Earth Science" June 17-18, 1993, Washington, D. C.

2. Proceeedings of a Workshop, "Directions for Research in the Next Decade: National Science Foundation Programs in Earthquake Engineering and Earthquake-Related Earth Science" June 17-18, 1993, Washington, D. C.

NATURAL HAZARD MITIGATION
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SESSION I - STRUCTURAL SYSTEMS I

"A Note on Shear Strength of Reinforced Concrete Bridge Piers"
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R. Behr

"Seismic Behavior of Steel Shear Panels and Frame-Wall Systems"
M. Xue and L. Lu

A NOTE ON SHEAR STRENGTH OF REINFORCED CONCRETE BRIDGE PIERS

Colleen Konwinski, Austin Pan, Julio Ramirez and Mete Sozen

This note summarizes perspectives on shear failure of bridge piers in the course of an investigation of the bridge damage caused by the 1994 Northridge Earthquake. The case history of a single reinforced concrete bridge will be discussed to provide the framework for inferences about the possibility of determining the circumstances leading to failure.

Fairfax-Washington Undercrossing

The Fairfax-Washington Undercrossing (to be called the Fairfax Bridge) carries Interstate 10 over Fairfax and Washington in Los Angeles, CA (1, 2). It is a reinforced concrete bridge supported by two walls and five sets of piers (Fig. 1). The deck comprised four segments divided by longitudinal and transverse expansion joints as shown in Fig. 1a. This note will focus on the four piers of bent 3 supporting the north-west segment of the deck. The 4-ft round piers were designed to be identical. Each was reinforced with 30 #11 bars longitudinally (ρ = 2.6%) and by No. 4 lapped ties at 12 in. (ρ_W = 0.07%) transversely. Yield stress of the reinforcement and the concrete strength were assumed to be 44 ksi and 5000 psi (2). The pier foundations were supported by piles.

The unit selfweight of the north-west segment of the bridge was determined to be 200 psf. A tributary gravity load of 800 kips was assigned to one of the columns. The total was 3000 kips for the four-pier bent.

Ground Motion

The bridge site was approximately 20 km distant from the epicentral region (moment magnitude, M_W = 6.7). Three strong-motion records were obtained within 5 km of the site. Two of the records (Century City, A_{MAX} = 0.26G; and Hollywood Storage Grounds, 0.4G) were at approximately the same radial distance from the epicenter. The third (Baldwin Hills, A_{MAX} = 0.24G) was closer to the epicenter.

Linear response displacement spectra for the period range of interest are shown in Fig. 2 for both horizontal components of the three strong-motion records.

Discussion

All piers of bent 3 failed. Failures occurred near the tops of the columns. Because the upper portions of the piers were totally demolished by the weight of the deck, there was no evidence left to help determine the initial cause of the failure decisively.

The ground motion at the site was not measured. It may have been less or more demanding than those measured at the three sites. The restraint conditions at the pier ends cannot be stated definitely. Because of the different supports (bent 2 and the transverse expansion joint) away from bent 3, the mass tributary to bent 3 can be defined only approximately. Those three attributes, without the uncertainties about material properties, would suffice to discourage astute analysts from attempts at estimating the response of bent 3. The writers would like to hazard a guess.

The N-S initial period of bent 3 was calculated to be 0.3 sec. According to Shimazaki (3), the nonlinear response of a reinforced concrete structure will be bounded by the linear response calculated for a damping factor of 2% and a period of $T_i \sqrt{2}$ (T_i: initial period). Inspecting Fig. 2 ordinates at a period of approximately 0.4 sec., it is seen that Hollywood 3 has a peculiar amplification characteristic. It is reasonable to base the projection of the strong-motion measurements to the Fairfax site using only the remaining five response spectra. On that basis, the displacement response of bent 3 is estimated to be between 3 and 4 cm. To check that estimate, a series of nonlinear response analyses were made of bent 3 using the Takeda (4) hysteresis with lateral forces of 55 tons at cracking and 230 tons at yield. The corresponding displacements were 0.33 and 3.8 cm (including reinforcement slip at connections). The calculated maximum displacements ranged from 2 to 3.6 cm. (excluding Hollywood 3 for which it was 5.5 cm).

Assuming that displacement demand at the Fairfax Bridge site did not exceed that indicated by the five motions selected, it can be concluded that the credible response corresponds to initiation of yield of bent-3 piers. There is no strong evidence of substantial nonlinear response.

If it is assumed that one of the piers with the 800-kip tributary weight yielded and the steel strength and the axial load are correct, the maximum nominal unit shear stress would be 280 psi. There is no reason for suspecting a collapse of the pier caused by combined axial load and bending if the displacement demand is low and the mean gravity stress is low (440 psi). The longitudinal bar splices are short (21 diameters), but there were no signs of splice failure at the foundation connection. It is likely that the failure was initiated by shear.

Viewing the conditions from the viewpoint of a design process (5), it may be reasoned that the nominal unit shear strength is 140 psi ($2\sqrt{f'_c}$). This may be increased by 30 psi to reflect axial-load influence (Eq. 11-4, Ref. 5). Transverse reinforcement at $\rho = 0.07\%$ with lapped splices should be ignored, but their presence may rationalize increasing the unit strength by another 30 psi (Eq. 11-17, Ref. 5). The total nominal strength would amount to approximately 200 psi. From that result, one would infer a shear failure even without invoking the understrength factor. But this is a weak conclusion because reference 5 is a design document and not a tool to be used in analyzing failures.

Two procedures which may be used properly as predictors of shear strength, because they are based on relevant experiments, are described in Reference 6. Recognizing that very little if any response beyond yielding occurred, the unit shear strengths based on these two procedures are 400 (proposed by Ang et al.) and 310 (proposed by Priestley et al.) psi.

Concluding Remarks

Three conclusions may be drawn from the study of the Fairfax Bridge failure described above:

(1) If the shear force corresponding to flexural failure is used in proportioning of a bridge pier, shear failure is likely to be prevented by use of a simple design procedures such as the one in reference 5.

(2) The available evidence does not justify the assumption that the failed piers of the Fairfax Bridge experienced lateral displacement exceeding the yield displacement.

(3) If the piers did indeed fail in shear, they are likely to have failed at nominal shears stresses below those observed in the laboratory.

Acknowledgment

This paper is based on studies made under NSF Grant CMS-9416759 at the School of Civil Engineering, Purdue University.

References

1. Buckle, I.G., "The Northridge California Earthquake of January 17, 1994: Performance of Highway Bridges," National Center for Earthquake Engineering Research Technical Report NCEER-94-0008. Buffalo, NY: March, 1994.

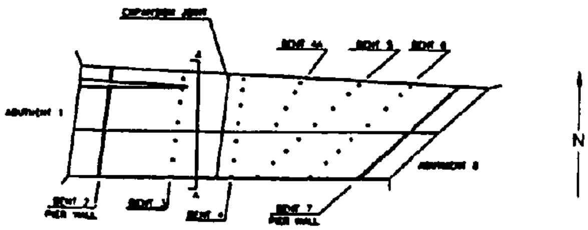
2. Priestley, M.J.N., Seible, F., and Uang, C.M., "The Northridge Earthquake of January 17, 1994: Damage Analysis of Selected Highway Bridges," University of California, San Diego Structural Systems Research Project No. SSRP-94/06, Feb. 1994.

3. K. Shimazaki and Sozen, M.A., "Seismic Drift of Reinforced Concrete Structures," Research Reports, Hozama-Gumi, Tokyo, 1984, pp. 145-166.

4. Takeda, T., Sozen, M.A., and Nielsen, N.M., "Reinforced Concrete Response to Simulated Earthquakes," Journal of the Structural Division, ASCE, Vol. 96, No. ST12, Dec. 1970, pp. 2557-2573.

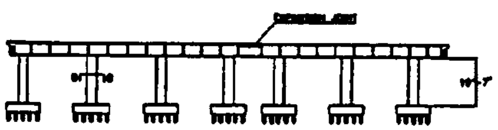
5. ACI Committee 318, "Building Code Requirements for Reinforced Concrete," American Concrete Institute, Detroit, MI, 1989.

6. Priestley, M.J.N., Verma, R., and Xiao, Y., "Seismic Shear Strength of Reinforced Concrete Columns," Journal of Structural Engineering, Vol. 120, No. 8, Aug. 1994, pp. 2310-2329.



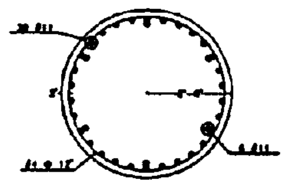
FAIRFAX-WASHINGTON UNDERCROSSING PLAN VIEW

(a)



SECTION A-A

(b)



SECTION B-B

(c)

Figure 1

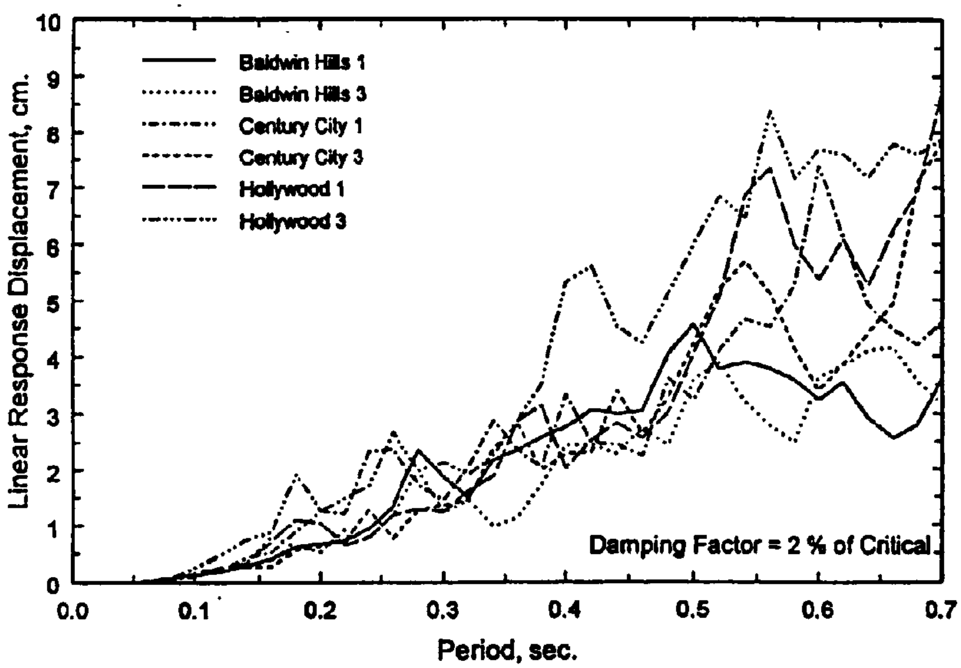


Figure 2

SEISMIC EVALUATION OF CONCRETE DAMS USING NONLINEAR ANALYSIS

Gregory L. Fenves[1]

Abstract

A recently completed research program sponsored by NSF has resulted in the development of improved models and analysis procedures for the earthquake evaluation of concrete dams. The objective of the research has been to incorporate specific aspects of nonlinear behavior while still including interaction effects with the impounded water and foundation rock. The analysis procedures are now being used for the safety evaluation of actual dams.

Introduction

The earthquake evaluation of concrete dams is an important aspect of assuring dam safety. As regulations become more stringent, it is necessary to employ analysis methods that can accurately assess seismic performance of these critical components in the Nation's infrastructure. For linear models of concrete dams, there are now methods of analysis for computing the earthquake response of gravity and arch dams. The linear models rigorously account for important factors that affect the dynamic response of dams to earthquakes: interaction between the dam and water impounded in the reservoir, interaction between the dam and foundation rock, interaction between the water and the reservoir bottom, and spatial variability of the ground motion, particularly for arch dams in canyons.

In an earthquake, however, a dam is expected to exhibit nonlinear response characteristics, such as concrete cracking and damage, and joint opening and deformation. Two important aspects of the nonlinear earthquake behavior of concrete dams have been addressed in a recently completed NSF research project. This paper summarizes the results of the research, describes the implementation, and outlines future research required in this area. The specifics of the models, procedures, results, and conclusions may be found in the cited references.

Base Sliding of Gravity Dams

In the safety evaluation of gravity dams, the traditional check of sliding stability involves computing a factor of safety against sliding using equivalent earthquake loads. Since a statics computation is not appropriate for the transient and oscillatory response of dams to earthquakes, a dynamic analysis should be performed to obtain estimates of base sliding displacements.

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[1]Professor, Department of Civil Engineering, University of California at Berkeley, Berkeley, CA 94720.

An analysis procedure based on the hybrid frequency–time domain procedure has been developed for computing the earthquake response of gravity dams including the nonlinear sliding behavior at the base. The sliding of the dam is represented by a Mohr–Coulomb friction model for the interface zone. Since the equations of motion are solved in the frequency domain, it is possible to account for interaction effects rigorously: dam–water interaction with compressibility, dam–foundation interaction, and reservoir bottom absorption (Chavez and Fenves, 1993). This is one of the first approaches to include nonlinear response characteristics of a structure with frequency dependent behavior.

Extensive parameter studies have shown several trends about base sliding. In all cases base sliding accumulates in the downstream direction and rocking of the dam is not important. The number of large acceleration peaks in the ground motion has the greatest effect on the number and amplitude of sliding displacements. Since dam–foundation rock interaction generally reduces the response of gravity dams, primarily because of energy dissipation, it also reduces base sliding. The assumption of rigid foundation rock can significantly overestimate the base sliding displacement. The isolation effect of sliding is negligible, so sliding may not be considered an isolation mechanism protecting the dam. Sliding displacements may reach 0.20% of the dam height, which should not indicate unstable response, although it may cause damage to shear keys, drains, and grout curtains in the interface zone.

The analysis procedure has been implemented in a computer program EAGD–SLIDE (Chavez and Fenves, 1994), and at this time it is being evaluated for use by several organizations.

Effect of Contraction Joint Opening on Arch Dams

Arch dams are constructed as cantilever monoliths separated by contraction joints. During an earthquake the joints may open and close, affecting the distribution of internal forces between the arches and cantilevers. Despite the possibility of contraction joint opening, standard seismic analysis procedures assume that a dam is a monolithic structure and the behavior is linear elastic. An earthquake analysis of a monolithic model typically shows large arch tensile stresses which cannot be transferred across the contraction joints. It is difficult to anticipate the actual seismic behavior of a dam from the stress distribution computed by a linear analysis.

An analysis procedure for computing the earthquake response of arch dams including the nonlinear effects of contraction joint opening (Fenves, et al., 1989) has been implemented in the computer program ADAP–88 (Mojtahedi, et al., 1992). The nonlinear analysis procedure can be used to evaluate the redistribution of forces that occur when the joints open and close during an earthquake.

The finite element model recognizes that contraction joints separate the cantilever monoliths of the dam. Several nonlinear joint elements through the thickness of the dam capture the opening and closing of a contraction joint. The regions of the model between the contraction joints are substructures. The behavior of the substructures is assumed to be linear, so cracking of concrete in the substructures is not represented. The equations of motion for a structure with a small number of nonlinear elements may be solved efficiently using the substructure procedure. This model of arch dams exemplifies such a structure; the cantilevers and the foundation rock region are linear substructures and the nonlinear joint elements connect the linear substructures.

Water compressibility affects the earthquake response of arch dams. The analysis with compressibility, however, requires either a frequency domain solution or a time domain solution using a radiation boundary condition for the reservoir. If water compressibility is neglected, dam–water interaction is represented by a frequency-independent added mass matrix and a time domain solution can be used efficiently. The full added mass matrix, however, couples all the substructures through the impounded water. The dam–water interaction is simplified by diagonalizing the added mass matrix.

The modeling of the foundation region is complicated by the three–dimensional geometry of canyon sites and highly variable properties of the rock. The common practice is to model a foundation rock region of dimensions approximately equal to the size of the dam. The mass of the foundation rock is neglected to prevent wave propagation in the foundation region.

The ADAP–88 program has been applied to an extensive parameter study of Morrow Point dam (Fenves, et al., 1992). As shown by analysis of several cases, the joints open as the dam displaces in the upstream direction, causing loss of arch action and the transfer of internal forces to the cantilevers. Compared with the linear analysis of a monolithic model, the reduction of arch tensile stresses is accompanied by an increase in cantilever tensile stresses at the downstream face. Since the contraction joints will prevent arch stresses from developing, the maximum cantilever stresses are the most important stress quantities. Typically, contraction joints were found to open one to two inches.

Because of interest in the use of ADAP–88, a shaking table study of arch dams has been completed at the Institute of Water Conservancy and Hydroelectric Research, Beijing, China, to verify the analytical model and the program (Chen, et al, 1994). The conclusions of that stud is that ADAP–88 adequately represents the displacements and stresses caused by contraction joint opening. Currently, ADAP–88 is being used in the evaluation of Pacoima dam in 1994 Northridge earthquake. The dam exhibited joint opening was slightly damaged in the earthquake.

Future Research

The research has concentrated on rigorous treatment of interaction effects and specific nonlinear mechanisms that affect performance of a dam. However, the issue of tensile stresses that exceed the low tensile strength of concrete must be pursued. Two theoretical issues are pertinent: i) an appropriate model for cracking and damage to concrete appropriate for seismic analysis, and ii) formulation and solution of equations of motion to account for frequency dependent interaction effects and general nonlinear force–deformation relationships.

There has been much recent research in models for concrete cracking. In current research, a plastic–damage model for concrete is being developed for cyclic loads. The objective is to have a model that shows the distribution of concrete damage and its propagation in a robust time–stepping procedure. The approach used to solve the base sliding problem of gravity dams involves a hybrid of frequency and time domain analyses. While this approach is appropriate for localized nonlinear behavior, such as base sliding, it is unlikely to be successful for general nonlinear behavior. The solution procedure must be in the time domain, so it will be necessary to evaluate convolution integrals in an approximate manner as part of an iterative time–stepping procedure.

The result of a coordinated research program will be models and analysis procedures for rational evaluation of concrete dam performance in strong earthquakes. Such analyses can better assure owners, regulators, and the public, that concrete gravity dams can safely resist the effects of earthquake without release of the reservoir.

Acknowledgments

This research program has been sponsored by NSF under grant BCS–8896121. The support of Program Director Dr. S.C. Liu is greatly appreciated. The development of ADAP–88 was also sponsored by a consortium of organizations interested in dam safety: U.S. Bureau of Reclamation, U.S. Corps of Engineers, County of Los Angeles Department of Public Works, Harza Engineering Company, Electric Power Research Institute, and Pacific Gas and Electric Company. The two analysis programs, EAGD–SLIDE and ADAP–88, are available from the National Information Service for Earthquake Engineering (NISEE).

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EARTHQUAKE ENGINEERING AND STRUCTURAL CONTROL RESEARCH AT VIRGINIA POLYTECHNIC INSTITUTE AND STATE UNIVERSITY: AN OVERVIEW

M. P. Singh

Abstract

Several principal investigators supported by the National Science Foundation are involved in research on earthquake engineering and structural control. The topics being studied are: seismic response of high voltage transmission towers, simplified methods for seismic design of nonstructural components, end-plate and all bolted moment connections for seismic loading, sliding mode control with regenerative actuators, control of occupant induced floor motion, and control of structures subjected to seismic excitations. PIs are from the departments of civil engineering, electrical engineering, and engineering science and mechanics. Some of these projects are collaborative efforts of PIs from different departments.

Introduction

Interest in the areas of structural dynamics, earthquake engineering and control of engineering systems spans through many departments at Virginia Polytechnic Institute and State University (VPI&SU). A large number of faculty members from the departments of aerospace and ocean engineering, electrical engineering, engineering science and mechanics, mechanical engineering and mathematics are engaged in research in the area the controls with funding from Air Force, Army, NASA, Navy, etc. The faculty members involved in research on civil structures and earthquake engineering have had the benefit of interacting with the faculty members working in controls; they are now involved in the development of new control methods which can be applied to civil structures. The structural control research initiative of National Science Foundation has provided further impetus for accelerated research on the control of civil structures. In addition to research on structural control, the writer and several of his colleagues have also been actively involved in research in the area of earthquake engineering. In the following section, the progress and research accomplishments of the projects on earthquake engineering and structural controls in which the writer has been technically involved are described. Other projects in which the writer is not involved but that are supported by the

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National Science Foundation are also included. The information provided in this paper on these latter projects was graciously supplied by the principal investigators of the projects themselves to the writer.

Research Projects

The titles of the NSF supported research projects at VPI&SU, and their respective principal investigators are as follows:

1. " Seismic Design of High-Voltage Tower". PI: Prof. Luis E. Suarez, Virginia Tech/ University of Puerto Rico.

2. "Improvements to simplified analysis of nonstructural components", An NCEER project. PI: Prof. L. E. Suarez.

3. "Sliding Mode Control of Structures with Re-generative Electric Actuators". PI: Prof. K. Ramu and Prof. L.E. Suarez.

4. "Control of Occupant Induced Floor Motion". PI: Prof. Thomas M. Murray.

5. "End-plate and All-bolted Moment Connections for Seismic Loading". PI: Prof. Thomas M. Murray

6. "A Comprehensive Program for the Control of Structures Subjected to Seismic Excitations". PI: Prof. Leonard Meirovitch

A brief description of the objectives and achievements of each of these projects is provided as follows.

Seismic Design of Transmission Tower-Cable Systems (PI - Prof. Luis E. Suarez): In the design of high-voltage transmission towers, the seismic effects are considered unimportant, although no comprehensive study has been carried out to evaluate these effects. In this project, methods are developed to include the effect of cable dynamics on the response of transmission towers. The connecting towers are assumed to vibrate in phase. Although the out-of-phase motions are also possible, and will be examined later, they are not expected to provide higher responses perhaps because of weak linking of the towers through cables. For the in-phase motions of the tower, the formulations for the in-plane and out-of-plane vibrations are developed since the dynamic models of vibration in the two planes are quite different. In the out-of-plane vibration, the cable acts as a compound pendulum with its own stiffness and damping matrix which can be combined with that of the supporting tower in a quite straightforward manner. In the in-plane vibration case, however, the inclusion of the inertial and stiffness effects in the model is somewhat more involved.

These two models have been used to evaluate the effect of seismic motions on the response of towers with and without cables. Shear forces in the tower are calculated at various levels with a newly developed response spectrum method which can account for acceleration dependent terms coming from the cable interaction. Since transmission towers are usually tall structures, they are likely to be affected by rotational components of the base input. A method to obtain the rotational components from the translational components has been developed. This method was employed to obtain the rotational component consistent with the recorded translational components of the base motions which were used in the study. The numerical results indicate that the effect of cables can not be ignored. For the example problem considered, the cable contributed as much as 50% to the transverse shear force response at the higher level and about 30% of the response at the base. The rotational component only increased the response by about 5% in the example problem. We are extending this formulation to include the case of towers at different elevations. This formulation is expected to be of help in the analysis of equipment in a substation, such as insulation towers connected to incoming cables.

Simplified Methods for Seismic Design of Nonstructural Components (PI- Prof. Luis E. Suarez): In this project we are critically evaluating the NEHRP provisions recommended for nonstructural components. Some simple but rational methods have been proposed which consider the dynamic characteristics of the supporting structure and equipment. Methods are being developed to account for the nonlinearity of supporting structures in the calculation of forces on nonstructural components. Special equipment such as rotating machines and elevators are also being studied to rationally define design forces for them.

Sliding Mode Control with Regenerative Electric Actuators (PI - Profs. K. Ramu and Luis E. Suarez): Sliding mode control approach is a newer method of designing controller. A very desirable characteristics of this method is its robustness. That is, even under the variation of system parameters the method ensures system stability. This makes it specially suitable for application to nonlinear systems. The method is being actively considered for mechanical and electrical engineering applications. Its use in civil engineering is very recent. The method seems to be ideally suited for civil applications. In future, we expect that different facets of this method will be investigated by different researchers to popularize it among civil engineers.

We have used this method to design controller for a multi story building with active tuned mass dampers and active tendon systems. The study indicates that for a reasonable level of reduction in the response, the force and power requirements can be quite large. It was observed that for a realistic 10-story structure, the maximum level of power can be as large as 3 MW, which is not practically feasible. We are exploring the use of parametric control where the power requirement appear to be only nominal. The concept of re-generative actuators is also being explored. These actuators function like regenerative motors used in metro trains. In place of dissipating the kinetic energy of a moving train into heat in the braking pads, regenerative motors are used as generators to convert a train's kinetic energy to electric power which is fed back to the power grid.

Control of Occupant Induced Floor Motion (PI - Prof. Thomas M. Murray): To reduce the problem of occupant induced floor motions, both active and passive damping methods are being examined. In the active method the control forces are imparted to the floor system by an electromagnetic shaker. The control law utilizes collocated rate feedback approach. The effectiveness of the approach was tested and verified by experiments. The passive device consisted of a multi-celled liquid filled damping device used with a tuned mass damper. The effectiveness of this device was also verified by experiments.

End-plate and All-bolted Moment Connections for Seismic Loading (PI - Prof. Thomas M. Murray): Motivation for this research is to improve the design of beam to column connection to avoid failures observed in steel buildings after the Northridge earthquake. End-plate and all-bolted connection designs will be investigated as alternatives to field weld connections. Because both of these connection types require tension bolts and the associated problems with prying forces, it is proposed that a shim plate be placed opposite the beam tension flange or the tee-stub tension web. Using the shim ensures that the bolt tension will not increase until separation of the end-plate or tee-stub flange and the column flange occurs. If the design is such that the pretension force is sufficient to develop the strength of the connected beam, in effect the connection is tested at the time the bolts are tightened. Four tests will be conducted to verify the concept.

A Comprehensive Program for the Control of Structures Subjected to Seismic Excitations (PI - Prof. Leonard Meirovitch): The research is concerned with the mitigation of damaging effects of earthquakes on structures. In particular, the objectives are to protect the integrity of structures and to prevent damage to its content and injury to its occupants. These objectives are to be achieved through the use of active and passive controls, referred to as hybrid control, where the first involves feedback control and the second base isolation, as well as damping materials. The research raises some fundamental questions concerning the control of structures and proposes ways of answering these questions. Some of the issues are as follows: (a) definition of base isolation and its implications, (b) modeling accuracy and its implications, (c) linear versus nonlinear open-loop stochastic systems, (d) linear versus nonlinear closed-loop stochastic systems, (e) methods of control for nonlinear stochastic structures and (f) adequacy of sensors and actuators.

Concluding Remarks

A brief description of the projects supported by the National Science Foundation at Virginia Polytechnic Institute and State University on earthquake engineering and structural controls is provided. Several investigators form different engineering departments of the university are involved. Some projects also involve collaborative research effort.

Acknowledgments

The financial support from the National Science Foundation for these projects is gratefully acknowledged by the respective principal investigators.

COLLAPSE STUDIES OF MISSION-GOTHIC UNDERCROSSING BRIDGE FOR NORTHRIDGE EARTHQUAKE

F.Y. Cheng[1] and K.Y. Lou[2]

Abstract

Collapse behavior of Mission-Gothic Undercrossing is investigated as a three-dimensional structural system with consideration of superstructure, column, and abutment conditions. Seismic input of vertical and horizontal ground motion for the study is modified from the record station at Sylmar County Hospital parking lot, about 18 kilometers from the epicenter, and Mission-Gothic Undercrossing, about 7.5 kilometers from the epicenter. Failure criteria of the bridge columns are developed by using nonlinear finite element analysis as well as biaxial interaction yield surface. Analytical results show bridge collapse behavior to be similar to that observed after the earthquake. The results indicate that, due to the orientation of columns connected to the bridge, force transmitted from bridge deck to columns is acting by a 45° angle from the strong axis of a cross-section which then causes a 13% reduction in column capacity. A circular flare for columns is recommended to avoid unequal column capacity under the lateral load. This study also indicates a need for additional stirrups to prevent spalling of concrete in the lower part of the flare. Unfavorable conditions in load combination for bridge columns will increase if the bridge structure is subjected to three-dimensional ground motion.

Introduction

Mission-Gothic Undercrossing is composed of two parallel bridge structures which carry the I-118 Simi Valley - San Fernando Freeway over the intersection of San Fernando - Mission Boulevard and Gothic Avenue in Los Angeles. It was designed in 1973 and built in 1976. Two bridge structures, left and right, are separated by a longitudinal joint seal (Fig. 1). The left bridge is approximately 130 m long and 29.5 m wide with three spans. The right one is a four-span bridge approximately 190 m long and 30 m wide. Both are supported by two-column bents and abutments.

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A significant difference in length of the two bridges results from skews of the abutments, for they were designed to be parallel to San Fernando-Mission Boulevard and Gothic Avenue. Thus abutments 1 and 5 skew approximately 45° from the longitudinal direction of the bridges. On the left bridge are skews of 45° clockwise for bent 2, and 11° counterclockwise for bent 3. On the right bridge are skews of 45° clockwise for bent 2 and 3, and 11° counterclockwise for bent 4. Angles are measured from the east-west direction as shown in Fig. 1.

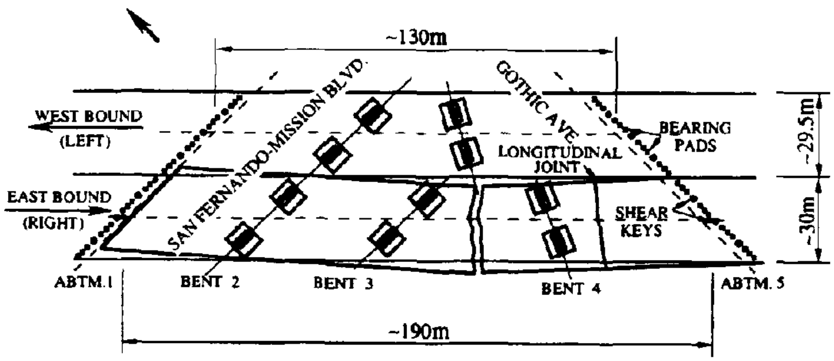

Fig. 1 Mission-Gothic Undercrossing

Superstructure of both bridges is a cast-in-place post-tensioned box girder with a structural depth of 2.3 m, and sits on twelve elastomeric bearing pads (457 mm x 457 mm x 76 mm) at abutments. Each bridge end consists of three square shear keys (762 mm in length and width) which are embedded 380 mm into the abutment seat, allowing the superstructure to move 150 mm in the bridge's longitudinal direction.

During the January 17, 1994 Northridge earthquake, the Mission-Gothic Undercrossing bridge collapsed after severe damage. This structure features an unusual layout of bents, directional dependence of flared column stiffness, and an arrangement of stirrups in the lower part of the column's flare. These unique characteristics of the structure led to its damage and collapse under strong ground motion.

Description of Loading

Seismic loading used for bridge collapse analysis was recorded at Sylmar County Hospital parking lot, which is approximately 18 km from the epicenter. This recording station provided acceleration, velocity and displacement in three directions with equal space at 0.02 second. Peak ground motion was 0.604 g at 4.08 seconds in east-west direction, 0.843 g at 4.20 seconds in north-south direction, and 0.535 g at 3.96 seconds in vertical direction.

Since there was no instrumentation installed in Mission-Gothic Undercrossing to record and collect data during Northridge earthquake, ground motion data recorded at Sylmar County Hospital parking lot need to be modified before application of bridge analysis. The following peak-horizontal-acceleration attenuation expression is used to compute acceleration a_1 at Mission-Gothic Undercrossing

$$a_1 = a \left\{ \frac{R_2 + 0.0606 e^{(0.7M)}}{R_1 + 0.606 e^{(0.7M)}} \right\}^{1.09} \tag{1}$$

where a = peak acceleration in g at Sylmar County Hospital parking lot; M = magnitude of earthquake; R_1 = distance from the epicenter to Mission-Gothic Undercrossing, in kilometers; and R_2 = distance from the epicenter to Sylmar County Hospital parking lot in kilometers. Distance from the epicenter to Mission-Gothic Undercrossing is approximately 7.5 km. Magnitude of the earthquake M is 6.7. Therefore the modifying coefficient is a_1/a = 1.16 for all componants.

Accelerations in east-west direction and north-south direction are also modified to be consistent with longitudinal and transverse directions of Mission-Gothic Undercrossing.

Collapse Behavior of Right Bridge Structure

Four cases under different combinations of ground motion are analyzed for right bridge structure (Cheng and Lou, 1995b): 1) subjected to longitudinal ground motion; 2) subjected to transverse ground motion; 3) subjected to a combination of longitudinal and transverse ground motions; and 4) subjected to a combination of longitudinal, transverse, and vertical ground motions.

Figure 2 is an identification of elements for which details may be found in Cheng and Lou, 1994 as well as Cheng and Lou, 1995a. This structure yields at columns R3R and R3L in the lower part of flare at 3.38 seconds. After 0.06 second, column R4R then yields at 3.44 seconds. Column R2R also reaches yielding at 3.48 seconds. All the columns first yield in the lower part of flare.

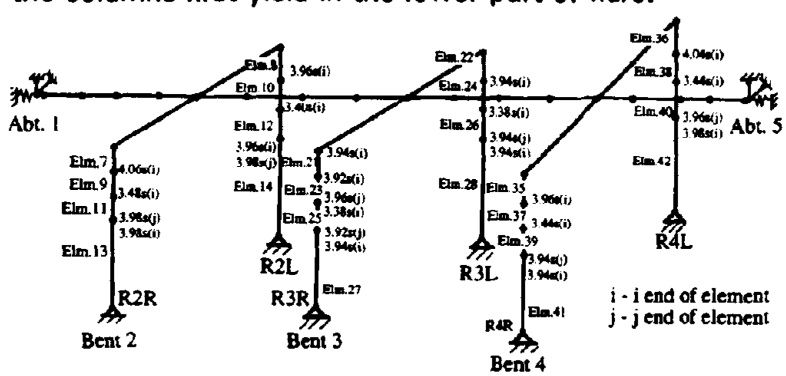

Fig. 2 Identification of Elements

Columns at bent 3 yield severely, while columns at bents 2 and 4 yield in the middle or lower part of the flares. Column failure results in the collapse of the

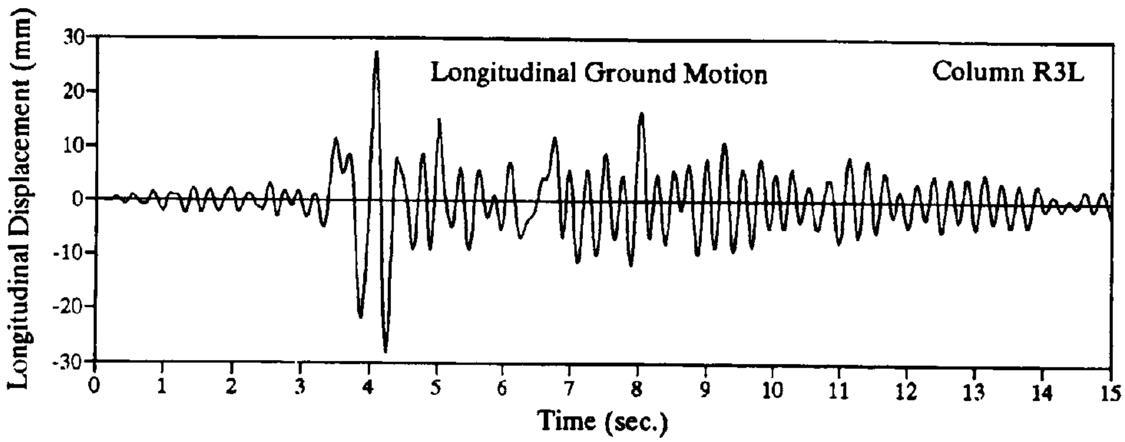

Fig. 3 Longitudinal Displacements of Column R3R Due to 1-D, 2-D, and 3-D Ground Motions

bridge's superstructure. The yield pattern gives a view of collapse behavior which is similar to field observations after the earthquake. All the columns yield within 0.7 second after 3.38 seconds. That means the bridge collapsed rather suddenly. Additional spirals are recommended for the lower part of the flare to enclose reinforcement and prevent propagation of concrete spalling.

Longitudinal displacement of column R3R is made in Fig. 3 for transverse, horizontal and 3-D ground movements. If longitudinal and vertical ground motions are considered in analysis, the longitudinal displacement of the structure greatly increases. Due to the skew of bents 2 and 3, columns in these bents are subjected to two-dimensional loads. An increase in longitudinal displacement leads to an increase of internal forces in columns.

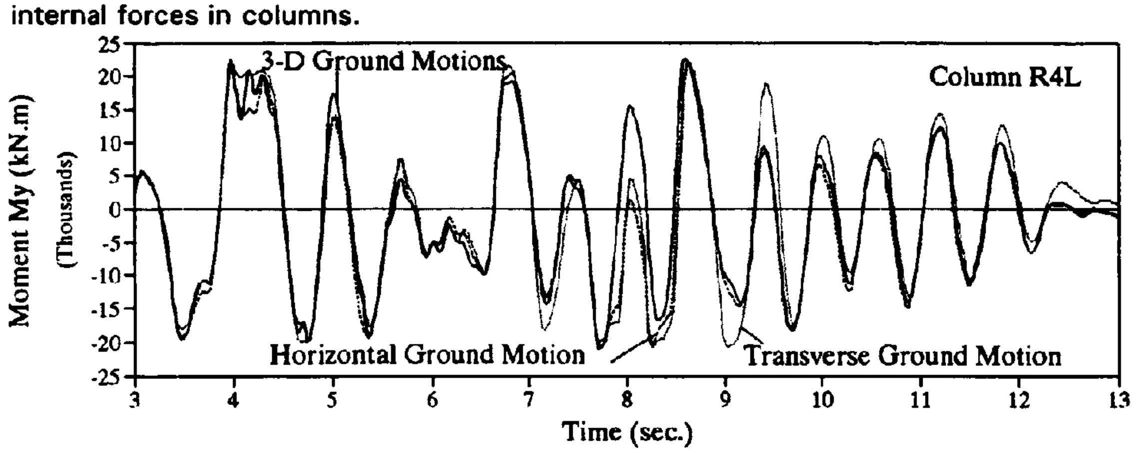

Fig. 4 Moment M_y Due to 1-D, 2-D, and 3-D Ground Motions

Figure 4 represents a comparison of moment M_y (in strong axis) at the bottom of the flare of column R4L (i-end of Element 42). Difference in moments at the same time is observed due to different ground motion. Moment M_y caused by three-dimensional ground motion (12,496 kN-m) is greater than that (11,760 kN-m) caused by transverse ground motion. Thus the increase of moments at two major axes of column cross-section leads to unfavorable conditions.

Conclusions

A three-dimensional structure model is developed for analysis of the right bridge of Mission-Gothic Undercrossing. Computer results reflect that column failure led to collapse of the bridge's superstructure. Collapse behavior is similar to that observed in the field after the earthquake. Nonlinear finite element method is employed to investigate failure behavior of the column subjected to lateral load with an angle to strong axis. For the column studied with lateral load at a 45° angle from strong axis of cross-section, column capacity is reduced about 13%. A circular flare is recommended to improve column capacity under lateral loads. If an unusual layout of skew bents is unavoidable, then circular flare for columns should be used since its capacity is independent of angled lateral load. Additional stirrups should be placed in the lower part of the flare to enclose reinforcement so that spalling of concrete in this region can be prevented.

Acknowledgment

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PRELIMINARY EVALUATION OF THE RESPONSE OF BASE-ISOLATED BUILDINGS IN THE NORTHRIDGE EARTHQUAKE

J. A. Inaudi[1], J. C. De la Llera[1], F. F. Tajirian[2], I. D. Aiken[2]

Abstract

This project investigates the response of base-isolated structures during the Northridge earthquake. Detailed three-dimensional nonlinear analyses as well as simplified code type analyses are being carried out for the University of Southern California Teaching Hospital structure. The approach and goals of this research program are summarized below. A preliminary evaluation of the response of the structure based on recorded data is presented. Finally the results obtained from simplified analyses completed to date are compared with recorded data.

Introduction

The research project will study the response of base-isolated buildings during the Northridge earthquake . The approach to be used in this program will consist of four main phases. The first phase consists of acquisition of digitized data from the four buildings listed in Table 1. All facilities listed in the table are instrumented by the California Strong Motion Instrumentation Program (CSMIP). Additionally, structural plans, bearing test reports, and soil data will be acquired as needed. The second phase consists of processing the digitized data. Important aspects will include an estimation of the isolator and inter-story deformations, an assessment of torsional, and rocking effects, impact, diaphragm flexibility, participation of the superstructure modes, soil-structure interaction (SSI) effects, computing floor response spectra and comparing them with spectra recorded in fixed-base structures, and an estimation of apparent frequencies of response by counting number of zero crossings and the use of spectograms. The third phase consists of dynamic analyses. The analyses will be performed in increasing order of complexity. First, the building will be assumed rigid, and the isolators will be modeled as 1-D elements with equivalent linear then bi-linear elements, and finally bi-directional plasticity elements. The analyses will be performed using INADEL [Inaudi and De la Llera, 1992] and 3D-BASIS [Nagarajaiah, 1989] computer programs. Additionally for the first two buildings listed in Table 1 the isolator models will be represented in 3-D to capture torsion and rocking effects. One dimensional site response analyses will be performed to examine differences in the time histories measured at the surface of the free-field with records obtained at the base of the building. SSI effects will be approximated by computing frequency independent soil springs representative of the footings which will be added to the model. A three-dimensional model of the superstructure using six degrees-of-freedom per floor will be incorporated. In the final phase, the results of the analyses will be evaluated to determine whether changes or addi-

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tions to current base isolation codes are necessary. Specifically an assessment will be made to determine whether codes should implicitly include provisions for the treatment of moderate seismic events.

Preliminary Analysis of Recorded Data

Preliminary results obtained by studying the response of the USC Teaching hospital are presented in the remaining pages. The building which is an 8-story braced frame structure was constructed in 1991. It is supported on 68 lead-rubber bearings (LRB) and 81 low damped elastomeric bearings [KPFF, 1988]. A plan layout of the isolators is shown in Figure 1. The design base shear is 0.15 g and the design corner bearing displacement including torsion is 26 cm. The design isolation period is 2.3 seconds and the fixed-base superstructure frequency is approximately 0.7 seconds. The bearings are square in plan. Two sizes are used, 56 cm and 66 cm width. Both types are configured with and without a 14 cm lead core.

The recorded maximum accelerations and displacements in the building are summarized in Table 2. The maximum relative deformations in the isolators and the superstructure are also included in this table. Deformations in the superstructure are in the same order as the bearing deformations which indicate that superstructure flexibility should be included in the analytical model. Displacements due to torsion are negligible at the base of the structure and increase with height, indicating that the isolation system did not cause any torsion. It is also observed that the predominant frequency of the torsional response is about twice the apparent lateral frequency. The effects of diaphragm flexibility are small. The contribution of diaphragm flexibility on increased deformation was 0.18 cm on the upper foundation, 0.29 cm on the fourth floor, and 0.44 cm on the roof. Thus it is appropriate to assume rigid diaphragms in development of the superstructure model.

An examination of the recorded time histories indicates that there were two distinctive phases. The first fifteen seconds when the response was dominated by the superstructure prior to yielding of the isolators, and the response after fifteen seconds which reflects yielding of the lead plugs and an effective period shift. Figure 2 plots the cumulative sum of the upcrossings of the bearing deformations versus time. Two different predominant slopes are clear in this figure. One for times less than 15 seconds, and the other for times beyond 15 seconds. An approximate estimation of the slope shows that the first frequency (corresponding to fixed base) is about 2 Hz (0.5 sec.) and the second about 0.8 Hz (1.25 sec). These values coincide with apparent frequencies computed from spectograms (time-frequency representations) of the recorded motions. Acceleration response spectra computed from the foundation and superstructure records are shown in Figures 3 and 4. Although the spectral values above 3 Hz are reduced, there is considerable amplification around 2 Hz, the superstructure frequency. During a larger event this amplification should be less as the isolator period approaches the design value.

Results of Preliminary Dynamic Analyses

The results obtained from two simple models of the USC building are presented next. The first model uses a rigid superstructure supported on seismic isolators with effective linear properties. The total effective stiffness of the isolators at the observed deformations is computed using data obtained from the KPFF report. Torsional effects are neglected. The effective system damping for the lead rubber bearings estimated from the design values was 0.19. In the analysis the damping was increased to 0.22 to account for the damping in the bearings without lead plugs. The response of the model subjected to the recorded acceleration time histories at the center of the lower foundation was computed. The displacement time histories in the x and y directions are compared with the recorded response in Figures 3 and 4. In general there is good agreement in the results. The peak displacements and accelerations are compared with the recorded values in Table 3.

The second model accounts for the nonlinearity of the isolation system. A simple model consisting of linear elements in parallel with elastoplastic elements representing the lead plugs is used. Five percent equivalent viscous damping is added for the contribution of the rubber bearings. Geometric distribution of the bearings and torsional effects are not modeled. The superstructure is assumed to be rigid. The peak displacements and accelerations computed from this model are shown in Table 3. The correlation with the recorded data is reasonable, although it is not as good as that obtained in the linear analysis. Currently the model is being refined to get a better correlation. Refinements prior to solving the full 3-D model will include the use of a smoother force deflection curve as well as coupling the lead plug models in the x and y directions using a circular yield surface.

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Table 1 List of Buildings Considered in this Project

Building Name	Epicent. Distance (km)	Isol. Sys.	PGA (g)	Roof Accel. (g)
USC Teaching Hospital	36	LRB	0.37	0.21
LA Fire Command and Control Center	38	HDR	0.22	0.32
Rockwell Computer Center	66	LRB	0.08	0.15
Foothill Communities Law and Justice Center	90	HDR	0.05	0.10

Table 2 Recorded Peak Response in USC Building

Location	Max. Acceleration (g)		Max. Displacement (cm)	
	x (EW)	y (NS)	x (EW)	y (NS)
Free field	0.21	0.48	2.5	2.3
Lower foundation	0.16	0.36	2.2	1.7
Upper foundation	0.07	0.13	2.7	2.8
4th floor	0.08	0.10	3.0	3.1
6th floor	0.14	0.10	3.9	3.3
Roof	0.16	0.20	4.7	3.9
Max. isolator deformation			2.2	3.5
Superstructure drift			2.7	2.1

Table 3 Comparison of Recorded Versus Computed Peak Response at Base of Superstructure

Location	Max. Accel. (g)		Max. Displ. (cm)	
	x (EW)	y (NS)	x (EW)	y (NS)
Recorded	0.07	0.13	2.2	3.5
Linear Model	0.05	0.10	1.9	3.7
Nonlinear Model	0.06	0.06	3.2	3.2

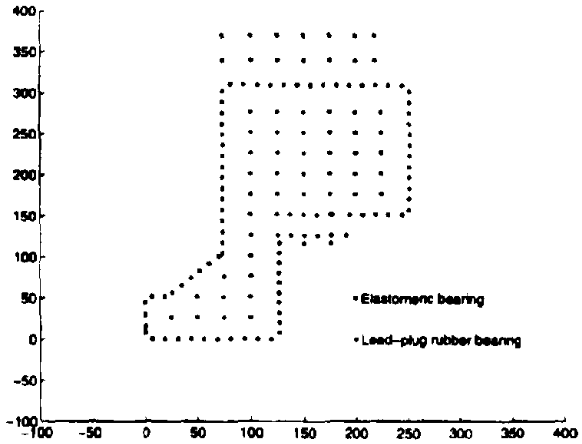

Fig.1 Layout of Isolators for USC Building

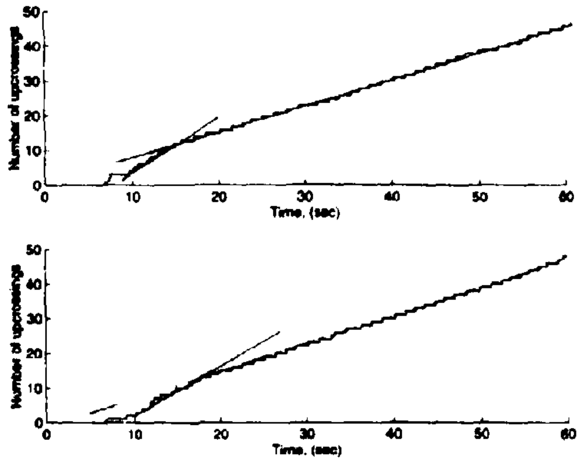

Fig. 2 Cumulative Number of Bearing Upcrossings

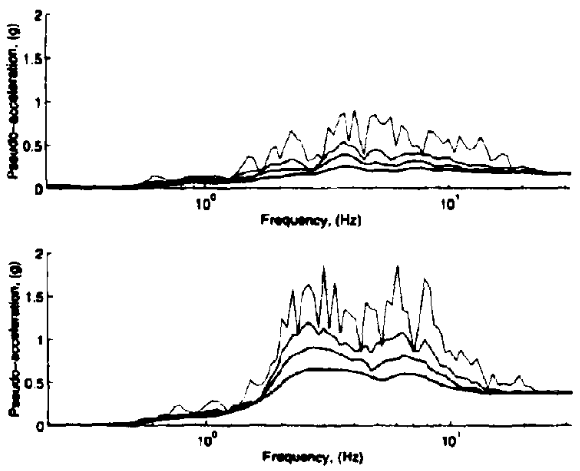

Fig. 3 Response Spectra Below Isolators (Damping = 0.01, 0.05, 0.10, 0.30)

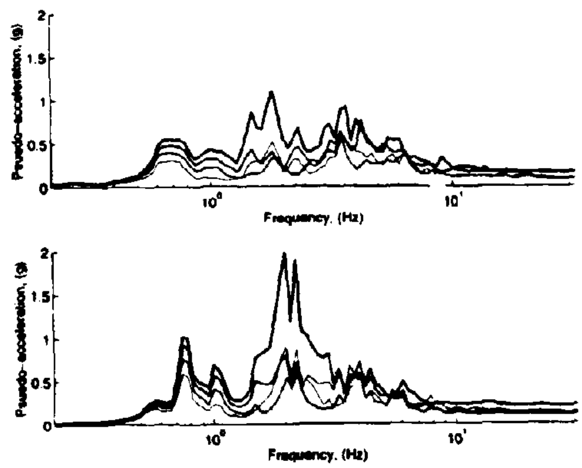

Fig. 4 Response Spectra in Superstructure, Damp=0.01 (EW and NS Directions)

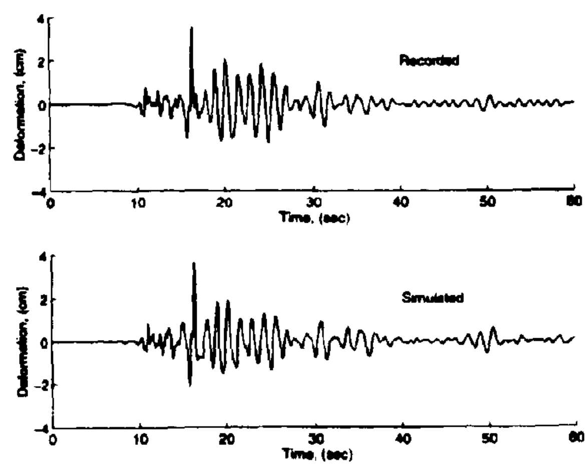

Fig. 5 Comparison of Recorded and Simulated Isolator Deformations, EW Direction

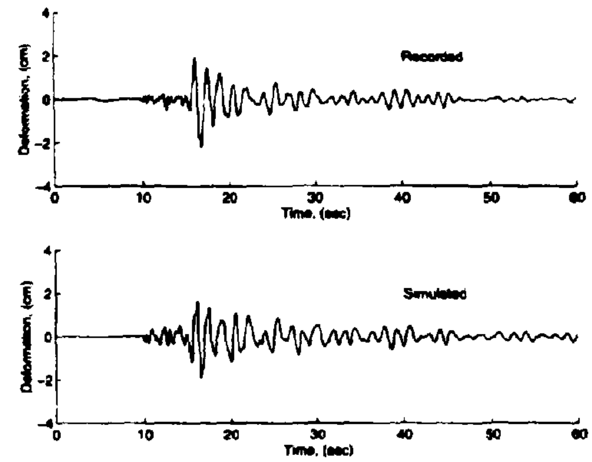

Fig. 6 Comparison of Recorded and Simulated Isolator Deformations, NS Direction

ULTIMATE LIMIT STATES FOR STOREFRONT ARCHITECTURAL GLASS UNDER SIMULATED EARTHQUAKE LOADINGS

Richard A. Behr[1]

Abstract

An ongoing effort is being made at the University of Missouri-Rolla to develop standard laboratory test methods and codified design procedures for architectural glass under seismic loadings. Recent laboratory work has yielded some promising results regarding the development of an appropriate seismic test method for architectural glass, as well as identifying ultimate limit states that quantify the seismic performance of various glass types. Specifically, a straightforward "crescendo-like" in-plane dynamic racking test performed at a constant frequency has been employed successfully. Two ultimate limit states have been defined: (1) a lower ultimate limit state corresponding to major glass crack pattern formation; and (2) an upper limit state corresponding to significant glass fallout. These crescendo tests have yielded fairly distinct and repeatable ultimate limit states for various storefront glass types tested under dynamic racking motions.

Introduction

A deep research void currently exists with regard to the seismic performance of architectural glazing systems that are employed so widely in contemporary curtain wall systems for low- , mid- , and high-rise buildings. Current model building codes contain no direct design provisions for the seismic design of architectural glass elements, nor do standard laboratory test methods presently exist for evaluating the seismic performance of architectural glazing systems. To address this void, a multi-year project was initiated in 1991 at the University of Missouri-Rolla (UMR) to investigate the seismic performance of architectural glazing systems.

In the process of conducting this research, a variety of test procedures were employed on full-size curtain wall test assemblies in the UMR laboratory. Dynamic racking tests were performed at various frequencies and included in-plane, out-of-plane, and torsional motions (Pantelides and Behr, 1994; Behr et al., 1995). Recently, a new test method has been employed at UMR to investigate serviceability limit states and ultimate limit states for architectural glass under seismic loadings. These new tests, called "crescendo tests" because of their progressively increasing racking amplitudes at a constant frequency, have produced some rather distinct and repeatable results in terms of identifying in-plane drift magnitudes associated with pre-defined ultimate limit states for common architectural glass types.

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Crescendo Test Format

A graphical representation of the crescendo test is shown in Figure 1. The crescendo test is a step-shaped swept sine function that has a total duration of approximately four minutes. It consists of a continuous series of alternating "ramp-up" and "constant-amplitude" intervals, each comprised of four sinusoidal cycles at a frequency of 0.8 Hz. Each drift amplitude step is +/- 0.25 in. (+/- 6 mm). The number of cycles at each step and the test frequency were selected to be reasonable representations of drift time histories that could occur in building envelope wall systems under seismic loadings. The crescendo test in Figure 1 is similar in configuration to the "multiple step test" described in ATC-24, 1992.

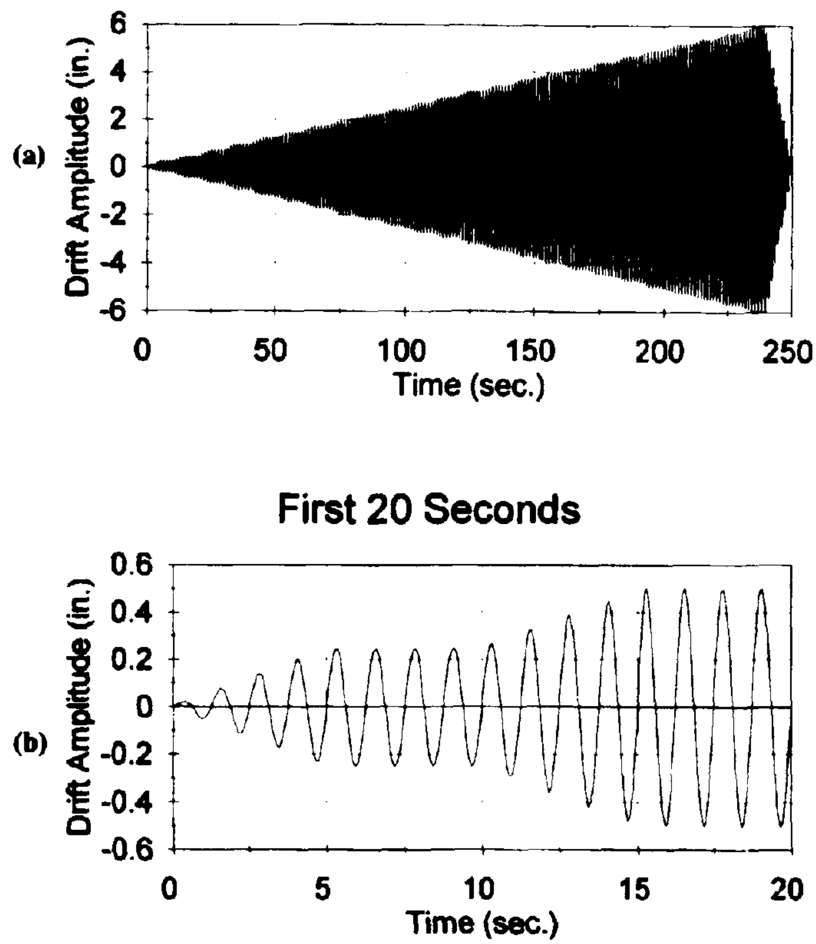
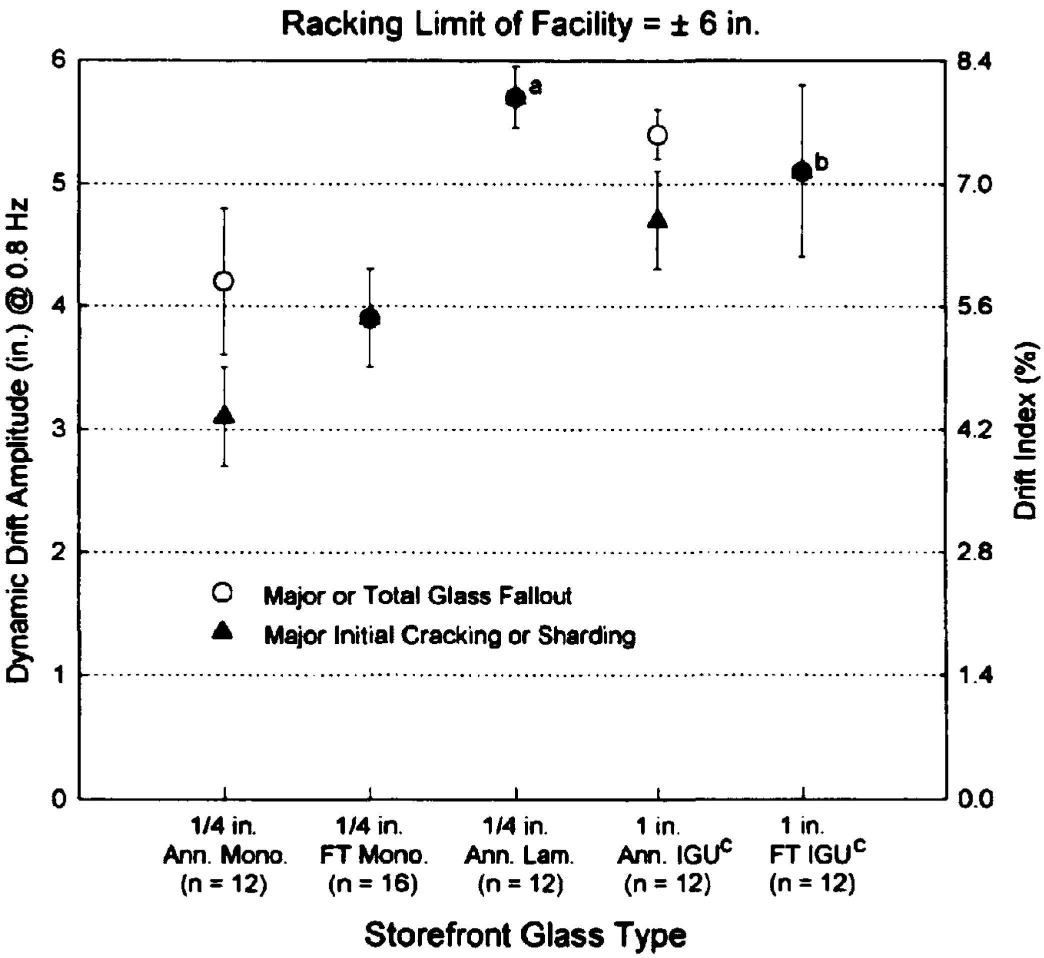


Fig. 1- Drift Time History:
(a) Entire Crescendo Test;
(b) First 20 Seconds of Crescendo Test.

Crescendo Test Results for Storefront Architectural Glass

Recent crescendo tests were performed on storefront architectural glass to identify in-plane dynamic racking amplitudes associated with pre-defined ultimate limit states for architectural glass elements. A "lower ultimate limit state" was defined as formation of a major crack pattern in a glass element that could lead directly to major glass fallout. An "upper ultimate limit state" was defined as the first occurrence of major glass fallout from the wall system frame. A video recording synchronized with the crescendo test was used to identify



Glass Size: 5 ft. wide x 6 ft. high

Storefront Wall System Types:

- "System A" for 1/4 in. glass types
  (left + right edge clearance = 0.65 in.; top + bottom edge clearance = 1.03 in.)
- "System B" for 1 in. IGUs
  (left + right edge clearance = 0.90 in.; top + bottom edge clearance = 1.42 in.)

[a] 8 of 12 annealed laminated specimens exhibited no glass fallout at a final drift of ± 6 in. @ 0.8 Hz.

[b] 1 of 12 fully tempered IGUs exhibited no glass fallout at a final drift of ± 6 in. @ 0.8 Hz.

[c] 1/2 in. air space in insulating glass units (IGUs).

Fig. 2- Ultimate Limit States for Storefront Glass Types From Crescendo Tests.

lower and upper limit states for various types of architectural glass used commonly in storefront wall systems. Relevant UMR experimental facilities are described in Brown et al., 1995.

As shown in Figure 2, the crescendo tests yielded some fairly distinct and repeatable ultimate limit states for storefront architectural glass. Glass types included 1/4 in. annealed monolithic, 1/4 in. fully tempered monolithic, 1/4 in. annealed laminated, 1 in. annealed insulating glass units (IGUs), and 1 in. fully tempered IGUs. Considering the nature of the test, some of the +/- one standard deviation error bars in Figure 2 are surprisingly small. It is clear from Figure 2 that annealed laminated glass exhibited superior resistance to dynamic racking motions, although the annealed and fully tempered IGUs also performed well. It is also clear from Figure 2 that glass tempering did not improve the racking resistance of either the 1/4 in. monolithic or the 1 in. IGU glass types. The type of information contained in Figure 2 could be quite useful in future efforts to develop seismic design procedures for architectural glass.

Conclusion

Crescendo tests offer a promising means of establishing ultimate limit states for architectural glass under dynamic racking motions. Such limit state information will support the development of rational design procedures for architectural glass and glazing systems under earthquake loadings.

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Acknowledgments

The author is grateful to the National Science Foundation (Grant No. CMS-9213172) for major funding of this project. Matching funds from the Monsanto Chemical Company are also gratefully acknowledged.

Seismic Behavior of Steel Shear Panels and Frame-Wall Systems

Ming Xue[1] and Le-Wu Lu[2]

Abstract

Selected results of a comprehensive study of the seismic behavior of steel shear wall panels and of frames infilled with such panels are presented and discussed. The interaction between the frame and the panels in a frame-wall system has been examined and an "optimum" connection arrangement between them has been found. The behavior of the wall panels subjected to monotonic and cyclic shear loading has been studied in detail. Simple formulas and procedures are proposed to provide empirical predictions which can be used in design applications.

Introduction

The behavior of the frame-wall system -- a frame infilled with steel plate shear walls -- is a subject of intensive international research because of its favorable performance under static and dynamic loads. The frame-wall system generally has high lateral stiffness, large deformation range within which it maintains stable strength and considerable energy dissipation capacity. During the Northridge earthquake of January 17, 1994, the Olive View Hospital, one of the seismic-resistant buildings utilizing this system, experienced a ground acceleration as high as 0.89g and a floor acceleration of 2.31g (g = the acceleration of gravity). The building showed no sign of structural damage.

In an NSF funded research project, a systematic study has been carried out on the frame-wall system. In order to develop a full understanding of the behavior of the system, the study was conducted in four phases. In phase one, several analytical approaches were evaluated and verified against the results of a few available tests. In phase two, four connection arrangements suitable for the frame-wall system and their effects on the behavior of the system were investigated. On the basis of the results obtained, a specific connection arrangement was selected for the subsequent phases of the study. In phase three, a parametric analysis was conducted on steel shear wall panels. The parameters included are thickness-to-width ratio and aspect ratio of the panels. The analytical data led to the development of a set of simple empirical formulas for predicting the behavior of the steel shear wall panels. Other factors examined in this phase are

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the effects of girder rigidity, stiffeners, and connection changes on the behavior of the panels. Finally, in phase four, the hysteresis characteristics of shear panels were studied with a finite element program developed at Lehigh during the course of this study. A continuous and tractable hysteretic model has been proposed for the panels used to form the shear walls in the frame-wall system. The principal results obtained from the study are briefly presented below.

Behavior of Frame-Wall Systems and Connection Arrangements

Four types of the frame-wall systems formed with four connection arrangements were analyzed under statically equivalent seismic loading. In addition, two limit systems were also analyzed for comparison purpose. Figure 1 shows the six systems. The symbols used are defined as follows: **F** for **F**ull moment resistant beam-to-column connections, **P** for **P**artial, **G** for the case where the wall panels are connected to the **G**irders, and **C** to the **C**olumns. For the two limit systems, **O** is for the case of an **O**pen frame (no panel) and **NB** stands for "**N**o **B**uckling" (that is the panels are assumed not to buckle).

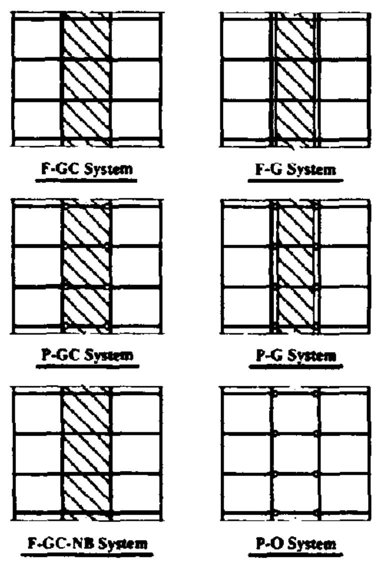

Fig. 1 Structural systems Studied

The following observations and conclusions have been made from the analyses of the systems:

1) All of the four frame-wall systems have high stiffness in comparison with the P-O system, see Fig.2.
2) The shear panels of the P-G system participate in carrying lateral load earlier than those of the F-GC system. The early participation makes the P-G system more desirable because it ensures more effective utilization of the panels.
3) The elimination of flexural deformation from shear panels in the P-G system leads to improved load-carrying capacity of the panels. The distribution of the internal forces in the girders and columns of the P-G system makes a simple analysis and design procedure possible for the system.
4) The dual characteristics of a frame-wall system can be utilized to develop structures with damage control capability. The combination of elasto-plastic, post-buckling behavior of the panels and flexible elastic frame behavior results in a structure that would have small permanent displacement after a moderate earthquake. This could reduce the cost of post earthquake repair.

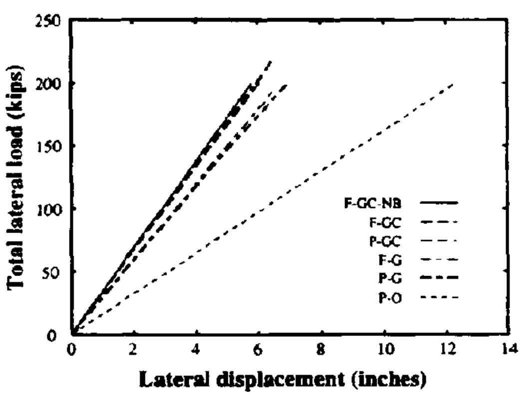

Fig. 2 Behavior of Frame-Wall & Frame Systems with Different Connection Arrangements

Based on an examination of all the data, the connection arrangement used in the P-G system appears to be the "optimal" arrangement for the frame-wall systems. This connection arrangement and the P-G system are utilized in the studies of shear panels under monotonic and cyclic loading and recommended for practical applications.

Monotonic behavior of steel shear panels

It was found from analytical results that the monotonic behavior of steel shear wall panels can be described with a modified Ramberg-Osgood curve. The following formulas have been proposed to define the parameters of the curve, based on the parametric analyses of 20 panels.

$$k = 0.68Et\frac{s}{h}[1-0.3(\frac{a}{b})^{-1.7\frac{a}{b}}] \quad (1)$$

$$k_t = 0.026Et\frac{s}{h}[1-0.3(\frac{a}{b})^{-1.7\frac{a}{b}}]\cdot\sqrt{\frac{t}{a}} \quad (2)$$

$$H_{t0} = 0.68\tau_y st[1-0.3(\frac{a}{b})^{-1.7\frac{a}{b}}] \quad (3)$$

$$C = \sqrt{\frac{1500t}{s}} \quad (4)$$

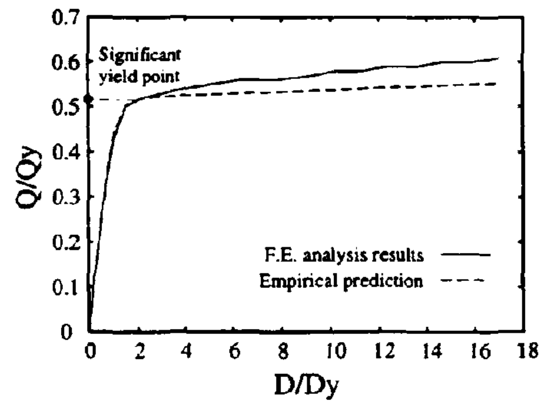

Fig. 3 Comparison between Finite Element Results and Empirical Prediction

in which t is the thickness, s the span, and h the height of a panel. a and b are, respectively, the maximum and the minimum of s and h. Figure 3 shows good agreement between the analytical results and the prediction made with Eqs. (1), (2), (3) and (4).

The use of very flexible girders with P-G shear panels affects both the post-buckling stiffness and the significant yield strength of the panels. However, the case studies also show that the behaviors of the P-G shear panels are rather insensitive to the rigidity change of the girders if the changes are in a practical range. Therefore, the effects of girder rigidity on the behavior of shear panels in the P-G system may be ignored with wide flange shapes commonly used as frame girders.

Stiffening a P-G shear panel with flexible stiffeners can effectively and efficiently increase both the post-buckling stiffness and the significant yield strength of the panel. It is found that flexible stiffeners can dramatically reduce the magnitude of the out-of-plane deformation of a shear panel. For this reason, use of flexible stiffeners is recommended as an effective approach to improve the behavior of the P-G shear panels under cyclic loading.

Hysteretic Behavior and Hysteresis Models of Shear Panels

From the analysis data with cyclic loading, the following can be observed about the hysteretic behavior of the shear panels of the P-G system:

1) The shear panels behave linearly under cyclic loading until the loading is close to the significant yield strength of the panels. The buckling and the out-of-plane post-buckling deformation have negligible effects in this loading range on the hysteretic behavior of the shear panels studied.
2) The shear panels have severe pinching behavior in the moderate loading range. The out-of-plane residual deformations and the yielding dominated by in-plane stresses are mainly responsible for the severe pinching. As loading is increased, the pinching behavior was alleviated, and consequently, the panels have rather wide hysteresis loops in the large loading range.
3) The shear panels did not show any significant stiffness degradation, however, the strength deterioration is quite noticeable. It is found that the previous yielding zones caused by in-plane stresses along the current tension diagonal is primarily responsible for the strength deterioration.
4) The ***a/t*** ratio affects both the pinching and the strength decrease. However, the effects are insignificant on panels even their ***a/t*** ratios varies in a large range. The ***s/h*** ratio is the most influential one of the parameters on the hysteretic characteristics of the panels analyzed. The ***s/h*** ratio can cause severe pinching, stiffness deterioration, and strength decrease if its value is lowered less than 1.0. The hysteresis of the panels seems independent of cyclic loading increments and the sequences of loading amplitudes.

The Masing modeling approach was adopted in the development of empirical hysteresis models for the steel shear panels of the P-G type. With pinching behavior as a major concern, a hysteresis model was proposed and identified with the analytical data. The comparisons made between the predicted results and the analytical date showed that the proposed empirical model can well represent the hysteresis of the shear panels (Fig.4). This model can be easily applied to structural analyses of large frame-wall systems under dynamic loads.

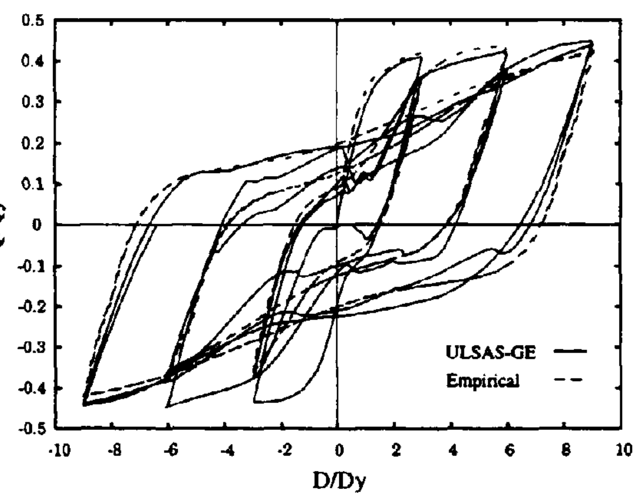

Fig. 4 Comparison between Analytical and Predicted Hysteresis of a Panel

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NATURAL HAZARD MITIGATION
NSF GRANTEES WORKSHOP

SESSION II - SYSTEMS INTEGRATION

"Earthquake Hazard Response in the United States and Japan: A Cross-Cultural Survey"
R. Palm

"Eqhazmat: A Data of Hazardous Material Incident Occurrence during Earthquakes"
G. Selvaduray

"Disaster Warning Responses by Tourists and Other Transient Populations"
T. Drabek

"A Decision Support System for Predicting Casualties in Multi-Hazardous Environments"
N. Stubbs

"Catastrophe Risk Management- Earthquake Insurance"
W. Dong

EARTHQUAKE HAZARD RESPONSE IN THE UNITED STATES AND JAPAN: A CROSS-CULTURAL SURVEY

Risa Palm[1]

Introduction

One of the recurring questions that social/behavioral science researchers on the topic of human response to natural hazards ask is: why don't people make the right decision? Why don't they avoid living in areas susceptible to surface faulting or flooding? Why don't they buy the proper insurance or store emergency supplies? In the search for answers to these questions, researchers have identified some systematic "errors" that people make. For example, researchers have identified the "gambler's fallacy", "editing", and "anchoring and adjustment". The gambler's fallacy is the belief that if a low-probability event has recently occurred that it is unlikely to occur again soon and therefore can be treated as a zero-probability event (Slovic, Kunreuther and White, 1974) . A second error is known as "editing", which occurs when individuals dismiss the risk of loss when the probability falls below a given level (Kahneman and Tversky, 1979; Slovic, Fischoff, Lichtenstein, Corrigan, and Combs, 1977). A third error has been termed "anchoring and adjustment", which refers to a tendency to estimate the loss or gain at a particular level, and then intuitively adjust their estimations over time around this first approximation (Tversky and Kahneman, 1974; Einhorn and Hogarth, 1985). Other hazards researchers have stressed the importance of risk communication in trying to get people to change their behaviors - for example, how credible is the information source, what kinds of media are used, how frequently is the message transmitted and so forth. Previous research has also searched for consistent relationships - in this case between perceived vulnerability and the adoption of hazard mitigation measures such as the purchase of earthquake insurance (Palm, 1995). This research has shown that the more likely one believes an earthquake will damage their home, the more likely they are to purchase earthquake insurance *ceteris paribus*.

This kind of investigation is the search for *general theory* linking human behavior with environmental risk. But the question is whether or not these "errors" or generalizations really apply to "all people". In truth, we do not know - primarily because virtually all of our testing has largely taken place with empirical studies of a single population - Western, middle class, and largely white.

A thought-provoking example of the potential effects of culture in the study of environmental risk perception is research on "illusory optimism". A number of studies have suggested that Americans estimate that they live longer than other people, they are younger for their age than others, and that they are less likely to

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die from cardiovascular diseases or accidents of many different kinds. Research has found an absence of this optimistic bias in self-perception among Japanese college students (Kitayama and Markus, 1994). This cultural difference in degree of optimism about personal well-being could also affect perceived vulnerability and the propensity to take risk-mitigation actions.

The purpose of this research is to investigate some of the cultural factors that impede or enhance the adoption of individual and societal earthquake mitigation measures. We are exploring the ways in which a propensity to emphasize the independence and autonomy of the self (in the United States) or the interdependence and mutual connections between the self and others (in Japan) has led to variability in risk perception, and mitigation behavior.

Survey Plan

The survey involves three stages. The first stage, administered in the spring of 1994 to focus groups to 30 California residents (Contra Costa and Orange Counties) and 115 Shizuoka prefecture residents (Shimizu and Yaizu), used open-ended questions. Respondents were asked to list the primary worries facing themselves and their families, the images they have of earthquake risks, and the behaviors they have or would adopt to mitigate risks. This first stage of the research was used to ensure that questions asked to the full survey population would include potential responses that were salient to both target populations.

Results from the first-stage survey were used to develop a cross-cultural questionnaire invoking responses of the Americans-only, the Japanese-only, and those presented by both groups of respondents. It was administered to approximately 1800 California owner-occupiers and 2000 Japanese home owners in September-November of 1994. The California sample was two-fold: a random sample of about 500 owner-occupiers (drawn from the county tax roles) in each of Redlands, western San Fernando Valley and Cupertino, along with a non-random sample of approximately 65 individuals with Japanese-surnames from the same areas drawn from the local telephone directories. The reason for picking these particular sites were (1) a desire to reach a relatively "homogeneous" population - in order to compare "Americans" with "Japanese"; (2) a need to select areas with a very high percentage of owner-occupiers so that this population would represent that of the study area; (3) a desire to select areas with high seismic risk, comparable to the seismic risk in the Japanese study areas. The Japanese sample was a random sample of 500 tax payers in each of two wards of Yokohama (Tsurumi and Kanazawa) and two cities within the Shizuoka prefecture (Yaizu and Shimizu). Although none of these areas have had major damaging earthquakes in recent years, they have had frequent moderate earthquakes, and both have been areas with predicted "great" earthquakes. The response rate to the 1994 survey was approximately 70 percent in California and 75 percent in Japan.

Examples of findings from 1994 Survey

We have only begun to analyze the results of the mail survey. Three examples are summarized here.

Hypothesis 1. Based on previous observations of "unwarranted general optimism" among Americans (Weinstein, 1987), we expected that Americans would be more optimistic about their chances of surviving a major earthquake.
Findings: This hypothesis was confirmed. US respondents were less l.kely to be less "worried" about earthquakes, felt that their own home was less likely to be seriously damaged, and were less worried in general about an earthquake affecting their community. Japanese respondents tended to say that they were extremely worried about earthquakes (it ranked second of all worries, after pollution), while California respondents tended to rank earthquakes much lower. Similarly, Japanese tended to feel that it was more likely that their own home would be seriously damaged by an earthquake in the next ten years, and were more likely to say they were worried in general about an earthquake affecting their community. All four of the Japanese study cities had higher mean rankings of earthquakes as a source of worry than the three American study sites. Of the three American sites, greatest worry was expressed in the western San Fernando Valley.
Hypothesis 2. Americans are less likely to be fatalistic about earthquakes, to be more skeptical about the potential of scientists to predict earthquakes, and to value uniqueness rather than interdependence with the group.
Findings: This hypothesis also was confirmed. Americans were less likely to agree with the following statements: "If an earthquake is going to harm me it will, and there isn't much that I can do about it - what will be, will be" , "If an earthquake is going to occur, there is not much my city/community can do to lessen its effects" , and "Decisions about how to reduce the damaging effects of earthquakes should be left to the experts". They were less likely than Japanese to agree with the statement that "I believe scientists are able to predict precisely where and when earthquakes will occur." Americans are more likely than Japanese to say that the following statements "describes me very well" : "I enjoy being unique and different from others in many respects", "I prefer to be direct and forthright when dealing with people I've just met", and "My personal identity, independent of others, is very important to me."
Hypothesis 3. Those who are more fatalistic in both countries are also more likely NOT to adopt hazard mitigation measures. This hypothesis is based on the notion that those who feel more "in control" of their destinies are also more likely to be pro-active in affecting their lives.
Findings: This hypothesis was NOT confirmed. Measures of fatalism were related to the adoption of mitigation measures, but not in the expected way. In the United States, those who were less fatalistic were also more likely to adopt mitigation measures (as indicated by the sum of mitigation measures adopted) and purchase insurance). In Japan, the relationship was in the opposite direction. Here, those who were more fatalistic were also MORE likely both to purchase earthquake insurance and also to adopt mitigation measures.

Plans for a Third Survey: spring, 1995

The Great Hanshin earthquake of January 17 near Kobe has important implications for earthquake preparedness in areas even more susceptible to large earthquakes (the Tokyo-Yokohama metropolitan area as well as the Los Angeles region). Both the Kanagawa prefecture (locale of the city of Yokohama) and the Shizuoka prefecture have not only suffered major earthquakes, but also are the settings of predictions for major damaging earthquakes in the not-too-distant future. We now have data about perceived vulnerability, the propensity to purchase earthquake insurance, and mitigation strategies adopted in four study sites within these two prefectures as well as three study sites in California. However, what we do not understand is the impact of experience with a destructive earthquake in Japan on these attitudes and levels of preparedness. In earlier California work, Palm found that experience with the Loma Prieta earthquake seemed to induce increases in perceived vulnerability to future earthquakes and the purchase of earthquake insurance - although this relationship was far from universal (Palm and Hodgson, 1992).

In the spring of 1995, we will re-survey those respondents who participated in the late 1994 survey in both California and Japan. This survey will repeat some of the questions posed in the 1994 survey about perceived vulnerability and expected outcomes of a major earthquake, as well as posing a few new questions about responses to the 1995 earthquake. We expect this survey to show both shifts in perceived vulnerability, level of preparedness and attitudes towards the balance between individual and governmental responsibility for disaster relief.

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EQHAZMAT: A DATA BASE OF HAZAROUS MATERIAL INCIDENT OCCURRENCE DURING EARTHQUAKES

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Abstract

The occurrence of hazardous materials incident during earthquakes was investigated and compiled into two data bases. One data base, consisting of hazardous materials incidents that have occurred during Japanese earthquakes, contains 177 records. Another data base of the incidents that occurred during the Northridge Earthquake contains 175 records. Each record consists of 26 fields for data entry.

Introduction

There are a number of reasons why the occurrence of hazardous materials incidents during earthquakes is important. Response to hazardous materials incidents requires not just specialized equipment, but also personnel with specialized training. The agencies that are trained to respond to hazardous materials incidents do not usually prepare for a large number of occurrences within a relatively small geographical region, and within a very narrow time frame, which is what typically happens during an earthquake. A reduction in the number of hazardous materials incidents during an earthquake can therefore be expected to reduce the strain on response agencies which will already be taxed heavily. Significant damage to the environment can also result from hazardous materials incidents, especially if the incident in question results in contamination of the soil, bodies of water, or subterranean water tables. This form of contamination can also result in severe economic consequences since cleanup of contaminated soil or bodies of water is extremely expensive.

The occurrence of hazardous materials incidents has tended to be under-reported until recently. The incidents that were reported tended to be "major" incidents that had resulted in conflagrations or large spills. Past reports on this subject have included those by Reitherman,[1] Tierney[2] and Selvaduray.[3] The Loma Prieta Earthquake was the first earthquake that was specifically investigated for the occurrence of hazardous materials incidents, the results of which have been reported by Selvaduray, Perkins and Wyatt.[4] With

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funding provided by the National Science Foundation, the Northridge Earthquake was also researched for occurrence of hazardous materials incidents. The National Science Foundation has also funded research of hazardous materials incidents that occurred during Japanese earthquakes.

EQHAZMAT: A Data Base

The initial effort focused on construction of a data base of earthquake caused hazardous materials incidents that have occurred during Japanese earthquakes. The motivation for this effort was the recognition that Japanese officials and researchers have been more throrough in documenting and investigating hazardous materials incidents that had occurred during earthquakes. During the course of this investigation the Northridge Earthquake occurred, and additional funds were obtained to research the Northridge earthquake as well, and include the data in the data base.

A public domain software, Clipper, was used to construct the data base so that users of the data base will not have to purchase commercial software. The data base runs on IBM compatible computers. A total of 26 fields, shown in Table 1, were identified for data collection for each record. While it was recognized that information pertaining to all of the fields would not always be available, these fields were still identified based upon the perception of the information that would be ideally required. Field Number 25, the memo field, was added to permit entry of information that could not be contained within the restricted character sizes of some of the other fields, particularly those pertaining to the consequence of the incident or cause of the incident.

Data Collection Methodology

Hazardous materials releases are required to be reported to the local Fire Department only if threshold quantities are exceeded. If the quantity released is below the threshold quantities, then the reporting of such an incident is voluntary. However, once reported, the information becomes a part of the Fire Department's records.

For purposes of research projects such as this, it would be ideal if all incidents were reported to the local Fire Department, and researchers can obtain the information directly from the Fire Department. However, such is not the case. The experience gained during the research following the Loma Prieta Earthquake became invaluable in the data collection process. In the case of the Northridge Earthquake, the Fire Departments of the Los Angeles County and Ventura County were initially contacted, and information obtained from them. In addition, a detailed survey of facilities/organizations that were considered potential sites for hazardous materials incidents were contacted directly by telephone, and in some cases followed up with site visits. The organizations contacted included all universities, colleges, high school districts, and hospitals, in addition to those identified by the Fire Departments.

In the case of Japanese earthquakes, a significant amount of information was directly available from publications of the Japanese government's Fire Defense Agency and other published sources. For more recent earthquakes, such as the Kushiro-oki Earthquake of

January 15, 1993, trips were made to Kushiro for data collection.

The challenge in data collection has been more in the area of identifying the damage mechanism that led to the incident, rather than just identifying the incident. Other data that has been identified as being relevant, but has not been readily available include the following: soil conditions, peak ground acceleration, cost of damage, hazard reduction measures that may have been implemented before the incident, and hazard reduction measures implemented after the incident. However, the data base has been constructed such that the data contained in it can be updated as new information becomes available.

Current Status of the Data Base

At the present time the data base exists as two separate entities. The data base pertaining to Japanese earthquakes contains 177 records, covering incidents through the Kushiro-oki Earthquake of January 15, 1993. The hazardous materials incidents that occurred during the Northridge Earthquake of January 17, 1994 have been compiled separately at the present time. This data base contains 175 records.

Future Work

The data base in its present form is "editable", i.e., anyone having the data base will be able to change its contents. In the very near future a "non-editable" version of the data base will be prepared and distributed to all those interested.

In addition to distribution of the inform, analysis of the data will also be undertaken. The purpose of the analysis will be to identify problem areas for future efforts. Issues such as occupancies particularly prone to hazardous material incident occurrence, mechanism of damage, relationship to intensity, relationship to urbanization, etc., will be investigated.

Acknowledgements

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Table 1: Fields contained within each record of EQHAZMAT Data Base

Field Number	Field Contents	Field Number	Field Contents
1	Serial/Reference Number	14	Container/vessel
2	Name of Earthquake	15	Consequences of spill
3	Date and Time	16	Direct cause of damage
4	Magnitude	17	Damage mechanism
5	Period of Ground Motion	18	Cost of damage
6	Duration of Ground Shaking	19	Clean-up organization
7	Company name/location	20	Clean-up method
8	Industry type	21	Cost of clean-up
9	Soil/geologic conditions	22	Hazard reduction: "pre"
10	Distance from epicenter	23	Hazard reduction: "post"
11	Peak ground acceleration: horizontal and vertical	24	Information source
12	Material spilled	25	Memo field
13	Quantity spilled	26	Data entry date

Disaster Warning Responses by Tourists and Other Transient Populations

Thomas E. Drabek[1]

Abstract

Following five major disasters, including Hurricanes Andrew and Iniki and the Northridge earthquake, data were collected from 682 transients, 69 lodging firm executives, and 76 community officials so as to determine: 1) modal warning responses; 2) pattern variations, and 3) policy implications. Differential behavior patterns and low customer satisfaction levels underscore the need for improved private-public sector partnerships required to enhance community disaster response capabilities.

Introduction

The tourist industry represents a vulnerability of catastrophic potential (Drabek 1994). Minimal levels of disaster evacuation planning were documented through in-depth interviews conducted with managers of 185 tourist businesses located in nine communities. Data collected produced a multivariate model comprised of six variables (e.g., managerial risk perception, contacts with local emergency manager) that accounted for over one-half of the variance in the extent of disaster planning among the firms (Adj. R^2 = .552). Prior research (e.g., Mileti and O'Brien 1992; Lindell and Perry 1992; Sorensen et al. 1987) highlights the absence of information regarding warning responses by tourists and other types of transients.

Research Objectives

This study had five objectives: 1) describe the sequence of behavior that culminates in evacuations from disaster sites by persons who are away from their residents, e.g., tourists, business travelers, migratory workers, or homeless persons; 2) assess the range of variation among these behavioral sequences for different types of evacuees, events, and locations; 3) identify factors related to variation in these behavioral sequences; 4) document perceptions of disaster victims regarding evacuation policies and procedures implemented by private firm executives and government agency representatives; and 5) formulate relevant policy recommendations for local emergency managers and business executives.

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Methods

Ten experts in tourism, emergency management, and disaster research (project advisory committee) critiqued all data collection instruments, provided liaison assistance, and performed other functions. Data were collected at 18 field sites following five disasters: 1) Hurricane Bob (August, 1991); 2) Big Bear Lake Earthquakes (June, 1992); 3) Hurricane Andrew (August, 1992; 4) Hurricane Iniki (September 1992); and 5) Northridge Earthquake (January, 1994). With the assistance of lodging firm owners and businesses which maintained tourist registries, I conducted 30-40 minute telephone interviews with 520 tourists and 83 business travelers. After each interview a questionnaire was mailed (87% return rate). Following Hurricane Andrew we conducted face-to-face interviews with 34 migrant worker families whose dwellings in the Everglades were destroyed. And after that event, Hurricane Iniki and the Northridge earthquake, I completed face-to-face interviews with 45 homeless persons. All data were interpreted within a "stress-strain perspective" which has guided numerous group and organizational analyses.

Findings

Significant pattern variations were documented regarding: 1) *disaster phase;* 2) location type; and 3) transient type. Multivariate models are being tested to identify the best network of variables that constrain some victims to take adaptive action in a timely manner

Disaster Phase.

In contrast to behavioral studies of residential populations responses following the Whittier Narrows (Goltz et al. 1992) and Loma Prieta (Bourque et al. 1993) earthquakes, most of these transient earthquake victims took actions, like rushing to another room or immediately running outside, that placed them at greater risk of injury. In hindsight they emphasized that their lack of familiarity with their location and absence of post-impact instructions had produced such responses. Once these transients decided on a course of action, or sometimes while they were in the process of doing so, 51 percent contacted relatives or friends. This was less true for hurricane victims than those impacted by the earthquakes (61% vs. 50%; $X^2 = 8.24$; $p < .05$). More earthquake victims went a short distance with the intent of returning when the threat was gone (30% vs. 1%). Typically, they went to a hotel parking lot and waited to learn about the building safety. Others did nothing (13% vs. 7%) or took some specific protective action (24% vs. 17%). In contrast, those threatened by one of the hurricanes were much more likely to leave with the intent of going to another county (16% vs. 1%) or to head for a public shelter (17% vs. 0%) ($X^2 = 195.81$; $p < .001$). These initial response actions constrained the choices made regarding a place of refuge for the night after the event ($X^2 = 64.95$; $p < .001$). Earthquake victims more frequently started for home (59% vs. 23%) or just went a short distance for the day and then returned to their original location (21% vs. 1%). In contrast, far more of the hurricane victims went to public shelters (33% vs. 0%) or to a relative's home (8% vs. 0%). Since prior research (Drabek 1986) has documented that most disaster evacuees, typically 80%, select the home of a relative or friend for a place of refuge, it is clear that transient populations present unique challenges to disaster planners. Earthquake victims registered higher impact levels than hurricane victims on three of four self-report measures: 1) group member physically injured (13% vs. 2%; $X^2 = 22.22$; $p < .001$); 2) property loss (33% vs. 20%; X^2 = 8.15; $p < .01$); 3) lasting psychological injury (17% vs. 5%; X^2 = 20.02; $p < .001$) and 4) filing an insurance claim (9% vs. 4%; X^2 = 1.00; p = n.s.).

Location Type.

Hurricane victims who were in rural areas, like the Florida Keys where escape routes were limited, departed much more quickly than did those in urban areas (X^2 = 86.92, $p < .001$). Location type also patterned the place of refuge selected (X^2 = 17.85; $p < .01$). For example, more of those visiting rural areas quickly headed for home (29% vs. 17%) or went to another county where they found a private lodging firm (20% vs. 12%). In contrast, much higher proportions of those visiting urban communities went to public shelters (40% vs. 27%). Despite these pattern differences, there were no variations on the four impact measures when the urban-rural settings were compared.

Transient Type.

Initial warning sources differed significantly among the four types of transients (X^2 = 83.60; $p < .001$). For example, nearly all (91%) migrant families received initial warnings through the media as did some (36%) homeless persons. They, however, frequently reported that they first were warned by others on the street (24%) or by local emergency officials (20%) who combed areas where the homeless congregated. Upon receiving an initial warning, most transients, like residential populations, sought confirmation. But this too varied significantly among the four transient types (X^2 = 124.49; $p < .001$). Homeless people more often did nothing (27%) whereas most migrants prepared to leave immediately (71%). Business travelers evidenced the highest levels of activism regarding confirmation actions and the types of sources to which they turned. For example, 23% immediately contacted staff of the lodging firm, higher than even tourists (19%) whereas migrants (88%) continued to watch television. Homeless persons said they turned to others on the street (20%) or went to look at the waves, structures, or whatever (37%), as they tried to decide what to do. Places of refuge selected also varied by transient type (X^2 = 232.94; $p < .001$). Migrants rushed to a public shelter (22%) or to the homes of relatives or friends (49%). Some of the homeless nested into makeshift protected areas such as getting under a bridge (29%) although nearly one-half (48%) ended up in some type of a public shelter. In contrast, most (45%) of the business travelers and a goodly proportion of the tourists (21%) simply relocated rooms within the lodging establishment so as to be safer. Many, of course, headed home (24% tourists; 11% business travelers) although some did end up in public shelters (23% tourists; 11% business travelers). And, of course, many went to another private firm (20% tourists; 26% business travelers). Disaster impacts varied by transient type on two measures, i.e., physical injury (X^2 = 11.50; $p < .01$) and property loss (X^2 = 138.88; $p < .001$). As usual, those with the least were impacted the most adversely.

Policy Implications.

While many (40%) of the tourists and business travelers were satisfied with staff responses most were not. Only one-fourth (27%) disagreed with this questionnaire item: "Despite some public relations efforts, I suspect that managers of most business firms have little or no commitment to disaster evacuation planning." Customer enthusiasm was documented for such actions as hazard brochures (88% favored having one in the room) and modified refund policies (86%). Many tourists and business travelers were surprised and disappointed to encounter policies that precluded refunds when they chose to evacuate. A perceptual gap was documented when these responses were compared to those we documented (Drabek 1994) among business executives. For example, nearly all (91%) customers favored government mandates requiring lodging firms to have written disaster plans; only 50 percent of the executives shared this view. While 77 percent of the customers favored annual disaster exercises by such firms, only 52 percent of the executives agreed with this idea in principle, and only five percent said they actually did it.

Conclusions

This first comparative study of transient responses to disaster warnings suggests four conclusions. First, tourists and other types of transients have disaster performance expectations for business executives and local government officials that are not being met. Second, disaster preparedness is a cost-effective investment that yields returns through satisfied customers. Third, there are important social barriers that must be confronted through executive leadership or else the constraints of threat denial and inertia will prevail. Fourth, business executives must invest in the enhancement of local emergency management programs. Many of these executives discovered that even with top quality internal preparedness, if the community level response fails, so too will they.

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A DECISION SUPPORT SYSTEM TO PREDICT CASUALTIES IN MULTI-HAZARDS

by

Norris Stubbs[1] and Karen Butler[2]

Abstract

A decision support system to aid in the prediction and understanding of casualties sustained by occupants in single hazard or multi-hazard environments is developed and described. Using the experience gained in the investigation of eleven types of disasters over the past five years, the requirements for such a decision support system is proposed. Next problems that must be solved in order to meet the stated requirements are identified. In addition, solution strategies that match these problems are developed and executed. Finally, the resulting decision support system is described.

Introduction

This paper deals with the problem of the estimation of casualties in hazards. Of all the losses to be estimated in disasters, the estimation of casualties is perhaps the most devastating loss to society at large. Early studies (see, e.g., FEMA, 1989) essentially extrapolated the limited casualty data available and adjusted the results to account for changes in building types and construction practices. Some of the main limitations of the previous approaches include the following: 1) the predictions are based entirely on historical data, 2) casualties are tied only to structural damage, and 3) estimates do not take into consideration other important factors such as occupant behavior and time of day of the initiating event.

The objective of this paper is to describe the elements of a decision support system that may aid in the prediction and understanding of casualties sustained by occupants in multi-hazardous environments. This objective is achieved in the following four steps: 1) the requirements for the decision support system are developed, 2) problems that must be solve in order to achieve the stated requirements are identified, 3) solution strategies to these problems are developed and executed, and 4) the resulting system is described.

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Minimum Requirements for The decision support system

The set of minimum requirements listed below were generated from the experience gained in the research project entitled: "Search and Rescue: An Integrated Multi-disciplinary Study."[3] The objectives of the study included: 1) an integration of previous interdisciplinary research findings that combine the disciplines of sociology, engineering, and epidemiology; and 2) an integration of the various understandings of the relationship of the built environment to victim behavior, and to informal and formal Search and Rescue efforts. During the course of the project, we studied at least eleven major disasters which included two Amtrack derailments, an aircraft accident, a major pipeline explosion, Hurricane Andrew, the World Trade Center Bombing, the Northridge Earthquake, and the Guadalahara Explosion. At each site, data were gathered by the multidisciplinary team to meet the project objectives. From an analysis of the data provided by this spectrum of disasters, we propose that any decision support system to predict, or provide a comprehensive understanding of, casualties should be designed to meet the following minimum requirements. The decision support system should: estimate casualties, consider casualties during and after the initiating event, consider extent of damage sustained by the structure, consider the extent of damage to nonstructural components, consider damage to the contents of the structure, incorporate the level of exposure of occupants before the event, incorporate vulnerability of occupants, incorporate hazards present after the initiating event, incorporate direct exposure of occupant after the initiating event, incorporate indirect exposure of the occupant after the event, and incorporate the vulnerability of the occupant after the event.

We make the following comments regarding the requirements listed above. First, the requirements are fairly independent of each other. For example, although structural damage, content damage, and nonstructural damage may be correlated, from the standpoint of injury to the occupant, the three modes of potential injury can be considered separately. Certainly, items such as the exposure and vulnerability of the occupant are independent. By "exposure" we mean the condition of being vulnerable to some degree if a hazard occurs; by "vulnerability" we mean the capacity of an object to resist a given hazard. Second,each and every one of the requirements above are functions of more basic variables. For example, the structural damage is a function of the magnitude(s) and duration(s) of the hazard(s) impacting the structure, the exposure of the structure to the hazard(s), and the vulnerability of the structure to the hazard(s). As another example, the exposure of the occupant before the initiating event will depend upon such items as the time of day, the type of building, the location in the building, the mobility of the occupant, the amount of warning before the event, the age of the occupant, etc... As yet another example, the vulnerability of the occupant during the initiating event will depend upon such items as the ability of the occupant to resist injury resulting from contact with parts of the structure as a result of structural damage, or to resist injury or death resulting from contact with nonstructural components as a result of nonstructural damage.

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[3]NSF BCS-9014830

Functions to Satisfy Requirements

The requirements listed above can be satisfied if the following relations can be developed: 1) relate casualties during and after the event; 2) relate casualties during the event to structural, nonstructural, and content damage; exposure of occupants before the event; and the vulnerability of occupants before the event; and 3) relate casualties after the event to existing hazards; the new direct exposure, the indirect exposure of the occupants after the event, and the vulnerability of the occupant after the event.

Selection of Requirements to Satisfy Requirements

One method that guarantees an internally consistent system whose output will imply the design requirements is the so-called "backward chaining method". A backward chaining system tries to prove a given goal by checking known facts in the context of the goal. If these conditions cannot be verified, they become sub-goals and the conditions for the sub-goals are in turn examined. The system continues chaining backwards until conditions are reached that can be either verified directly, extracted from known facts, determined from a user query, or obtained by invoking some algorithm.

Description of the Decision Support System

The application of the backward chaining method to the set of requirements results in Figure 1 to Figure 4 which collectively define a hierarchical tree structure of the decision support system to aid in the estimation of casualties. In Figure 1, the hierarchal relationship between the casualties, initiating hazards, structural and nonstructural damage, exposure and vulnerability of the occupant, are presented for causalities during and casualties after the initiating event. Note that in the figure, the nodes indicated by the upper case "A" represent various algorithms the combine elements at a lower level to a higher level. The occupant exposure during the initiating event is shown in Figure 2. Note that this figure is an extension of Figure 1. From the figure, we see that the occupants predisposition to harm depends upon such items as the time of day, age, preparedness, prior warning, etc. The occupant exposure after the initiating event is broken into two parts: the direct exposure and the indirect exposure. By direct exposure we mean conditions at the site that may modify the existing hazard to the occupant. For example, if the hazard is crushing, the exposure of the occupant may depend upon such factors as the amount of structural damage, the amount of nonstructural damage, the level of injury sustained during the initiating event, etc. A grouping of such direct exposure elements is shown in Figure 3. By indirect exposure we mean those characteristics of the entrapped individual or the site that make a safe extraction difficult. Such items include the relative isolation of the structure, the location of rescuers, the prevailing weather conditions, the type of collapse, the density of the debris, etc... These items are depicted in Figure 4.

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Estimating Losses From Future Earthquakes, FEMA, Earthquake Hazard Reduction Series 5-1, FEMA, 1989, Washington, D.C.

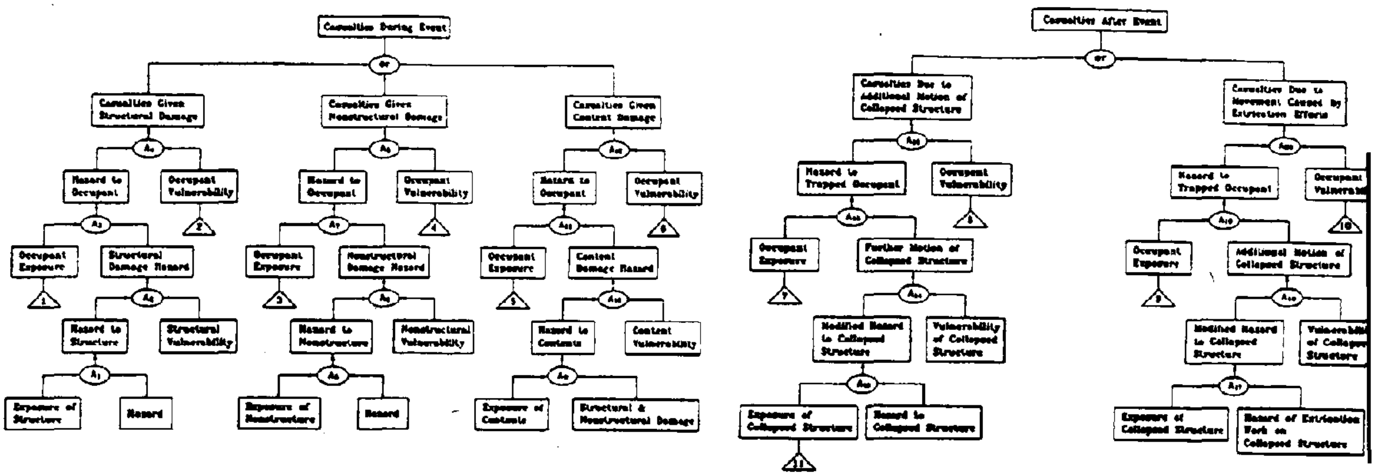

Fig. 1- Hierarchical Network for Estimating Casualties

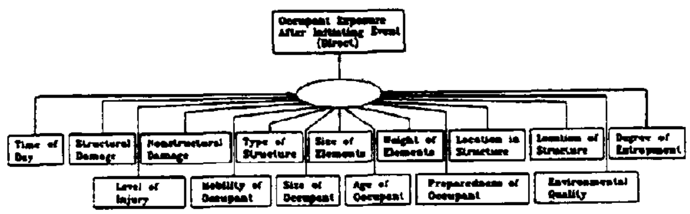

Fig. 2- Subnetwork for Evaluating Initial Occupant Exposure

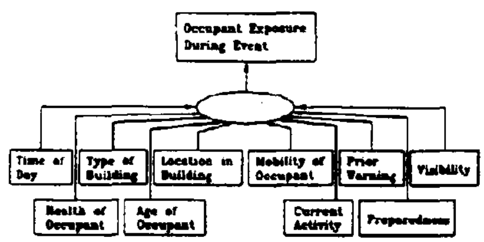

Fig. 3- Subnetwork for Evaluating Direct Post-Event Occupant Exposure

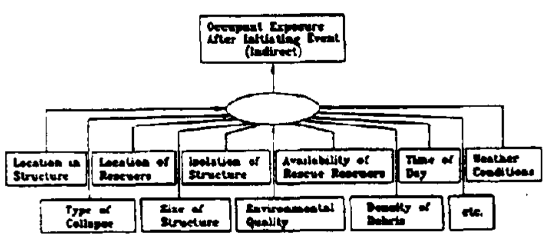

Fig. 4- Subnetwork for Evaluating Indirect Post-Event Occupant Exposure

CATASTROPHE RISK MANAGEMENT
A Rational Approach To Earthquake Insurance

Weimin Dong[1]

Abstract

Insurance is an effective means of spreading catastrophic risks, as evidenced by the $12 billion of insured losses in Northridge. This paper reviews the evolution of catastrophe risk management (CRM) with emphasis on earthquake insurance. Past and present practices in earthquake risk management are described, and the impact of earthquake engineering research on risk management effectiveness is delineated. Future trends in catastrophe insurance and research needs are also discussed.

Introduction

Catastrophes have several unique characteristics. They occur infrequently, but when they do their large footprints can inflict significant losses ($30 billion for Hurricane Andrews and $300 billion for the Hanshin-Kobe earthquake). Catastrophes also tend to be region specific: earthquakes in the Western U.S., hurricanes in the Southeast, snowstorms in the North and tornadoes in the Midwest. Exceptions to the rule exist[2], but unpredictable occurrence and devastating human-socio-economic loss will most likely constitute a catastrophe.

Whereas most insurance contracts are directed primarily at protecting the individual risk, *the catastrophe contract is designed primarily to reinsure the insurer against an accumulation of losses, which generally are a result of a single large occurrence, such as a hurricane, tornado, flood or earthquake*. For smaller or financially marginal insurance companies, there are two basic reasons for catastrophe reinsurance: to protect its (policyholder) surplus and to lessen large fluctuations in year-to-year operating results. For large companies in sound financial conditions with a good spread of business, catastrophe reinsurance is purchased to level results, thereby protecting earnings per share and avoiding large fluctuations in loss ratio which would have an adverse effect on the market price of the company's stock.

Hence, whether a catastrophic event becomes one or not to an insurance company depends on the strength and capacity of the company and how the risk is managed. Minor fluctuations can

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[2] For example, the Oakland Hills fire of 1990 can be considered a minor catastrophe even though fires in general are not because they occur with regularity and everywhere. Same remarks apply to the Mississippi flood of 1993 and the California flood of 1995.

become fatal swings if they are disproportional to the company's capacity, a tendency that becomes more accentuated the rarer the event and more severe the impact. This is understood by all risk managers. As insurance companies are profit-making organizations, the holy grail of catastrophe risk management (CRM) is how to adjust pricing and leverage capacity for the best return within a competitive market environment and yet assuring solvency and robustness of the company in the face of catastrophes.

Catastrophe Risk Management (CRM), Past and Present

Considering the predominance of casualty and realty insurance in the industry, it is not surprising that the initial approach to earthquake CRM was sought in actuarial foundations that worked well for life and fire. However, it soon became obvious that the database for major earthquakes was too minuscule and a simple carryover of the methodology would be flawed: the events have recurrence periods of the order of several hundred years; damage records are fragmentary; assets and technology can evolve through several generations in that time; and valuations can be drastically different from those at last occurrence. Hence, judgment and experience were combined with an approach based on the Probable Maximum Loss or PML (Steinbrugge, 1982). The underwriting territory was divided into zones, each of which was assigned a maximum event and subjugated to its influence. For example, for Zone A in California that includes the San Francisco Bay area, the PML event is an M8.3 event on the North San Andreas as that is the most severe event on record for the region.

Despite its namesake the PML method is essentially a deterministic method that fit in well with the state of knowledge and technology 20 years ago. Since that time, much progress has been made in our understanding of building response and damage, but, most important, computer technology has made leaps and bounds that the underwriting industry needs no longer be constrained by hand-calculations and table lookups. Consequently, very sophisticated engineering models can be tightly integrated with detailed, expert knowledge from various disciplines such as seismology, geology, geophysics and structural engineering, to generate in-depth damage estimates for the increasingly-demanding risk management process (e.g., see IRAS by Dong et al., 1988). Important enhancements include the following: (1) Recognizing that the risk exists but has only a probability of occurrence within a given time window, and taking advantage of this fact; (2) Recognizing that damage varies even for nominally identical buildings depending on structural details, soil conditions, distance to source, etc., and incorporating the effects of such variations[3]; (3) Complete integration of engineering and financial models to allow evaluations of all risk management options. One example is layering, i.e., vertical division of the potential loss with the intention of assigning the layers to other insurers or reinsurers; it is a powerful diversification technique which heretofore cannot be exercised with PMLs.

[3] In terms of losses, the PML estimates are "expected" values which, though useful in some sense, can be very misleading. For example, an average loss of 7% will incur no loss on the part of the insurer when the deductible is 10%. In point of fact, the loss distribution will vary over a wide range, with some over 10% and some below; those over will incur a loss for the insurer.

Catastrophe Risk Management (CRM), Future Trends

From an earthquake engineer's viewpoint, the confluence of several trends will make CRM in this and next decade even more challenging. One is increasing valuation of building stock and infrastructure assets. The table below lists the most costly earthquakes in the U.S. Even after adjustment for inflation is made, the spiraling cost with time is quite obvious. The price for a major event, based on the Hanshin-Kobe figure, is even more staggering. For San Francisco, the total loss is estimated at $115-$150 billions, and that for Los Angeles is $120-$180 billions.

Year	Locality	Estimated Property Damage (in millions)
1994	Northridge, Calif.	30,000
1989	San Francisco, Calif.	7,000
1971	San Fernando, Calif.	553
1964	Alaska	500
1987	Whittier, Calif.	358
1992	Joshua Tree, Calif.	92
1992	Eureka, Calif.	66
1952	Kern County, Calif.	60
1933	Long Beach, Calif.	40
1979	Coalinga, Calif.	31

(from National Oceanic and Atmospheric Administration; cost not adjusted for inflation)

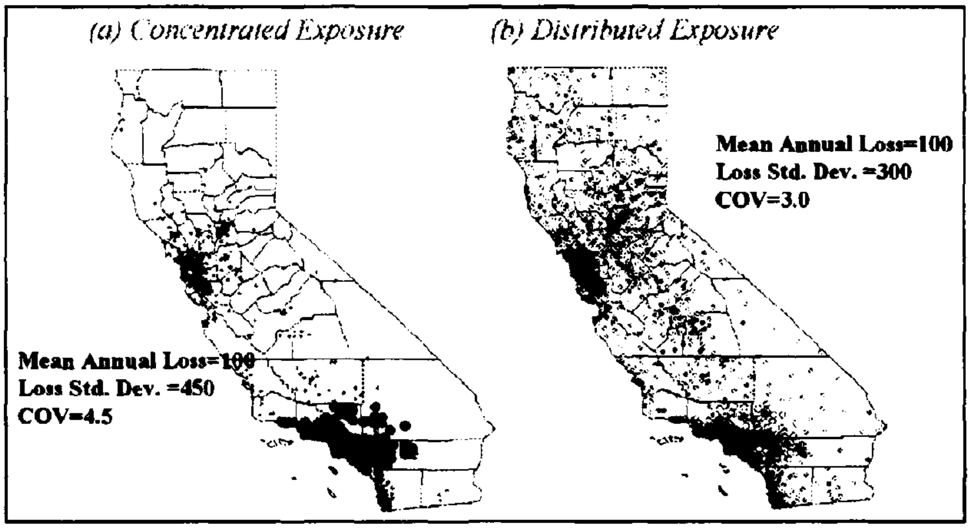
The second is demographic. Population growth in California increased by 90% from 1960-1990, by 78% in Texas, and by 163% in Florida. Although the rate fluctuates with economic conditions and state, it is estimated that by the year 2000, 75% of the nation's population will live within 10 miles of the coast. The implication is that there will be significantly increased concentration of population, with the accompanying increase in manufactured houseing, in urban centers susceptible to catastrophes. The insurance business in the meantime becomes ever-more global and competitive. There is need for increased sharing of information on global events, between insurers and reinsurers, and with clients. Finally, back-end financial tools are getting very sophisticated, and demand to be integrated with the engineering tools.

CRM can channel these pressures into advantages by leveraging advances in technology and engineering, and by extending interdisciplinary integration to the global scale. We shall illustrate the ideas involved using the context of diversification, or spread.

CRM Through Diversification

Diversification reduces the fluctuation and volatility in loss by spreading the risk. The "simple" spread uses portfolio leveraging, i.e., picking the proper mix of constituents in a portfolio. Spatial spread or diversification corresponds to spreading the coverage area. The figure below compares a spatially spread portfolio in the State of California with a concentrated one, and shows the reduction in volatility achieved; the reduction in exposure ratio, $ER = \sigma / E$ or standard deviation over mean, is about 1/3.

The same technique is also effective between states in the U.S., and internationally. When the perils covered are different, this technique is called peril diversification or hazard spread. Layering or attachment spread amounts to passing pieces with higher (lower) risk to others for a price, and is considered by most as the main role of reinsurance.



Incorporating these and other CRM options in earthquake insurance studies requires quantitative models of capacity and stability. A comprehensive approach to measure volatility is possible by extending the IRAS framework to include finer measures of uncertainty, e.g., higher probability moments of the estimated loss such as the exposure ratio mentioned above.

Summary

Great demands lie ahead for the earthquake engineering community not only to advance the state of knowledge but also to package and transfer that knowledge so that it can be used in CRM. It is no longer sufficient to limit ourselves to our own specialty. Multiple and interdisciplinary integration will be the norm of the day. Effective CRM requires investigating all options, which means that models for all perils such as earthquakes, hurricanes and floods must be brought under one roof, and that catastrophe impacts be considered in the global scale.

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NATURAL HAZARD MITIGATION
NSF GRANTEES WORKSHOP

SESSION III - SITING AND GEOTECHNICAL SYSTEMS

"Implications of the Northridge Earthquake for Strong Ground Motion from Thrust Faults"
P. Somerville, C. Saikia, D. Wald, and R. Graves

"Geotechnical Earthquake Hazards Using a Spatial Analysis System"
J. Frost and R. Luna

"Damage to Landfills from the Northridge Earthquake"
E. Kavazanjian, N. Matasovic, J. Bray, A. Augello, and R. Seed

"National Geotechnical Experimentation Sites Program"
J. Benoit and P. de Alba

"Earthquake Ground Motion Modeling in Large Basins"
J. Bielak

"Three-Dimensional Slope Stability"
T. Stark

"Project VELACS: RPI Contribution"
A. Elgamal and R. Dobry

Implications of the Northridge Earthquake for Strong Ground Motions fromThrust Faults

Paul Somerville[1] and Norman Abrahamson[2]

Abstract

The peak accelerations from the Northridge earthquake were about 40% larger than the median value predicted by current empirical attenuation relations at distances less than about 30 km. This paper addressed the question of whether the ground motions from the Northridge earthquake are anomalous for thrust events, or are representative of ground motions expected in future thrust earthquakes. We find that the high frequency ground motions can be predicted using a rupture model based on the low frequency ground motions. We also show that the high frequency ground motions that we simulated prior to the Northridge earthquake for a hypothesized magnitude 7 earthquake on the Elysian Park blind thrust are very similar to the motions recorded during the Northridge earthquake. This suggests that the Northridge strong motion records are not anomalous, and are representative of ground motions close to thrust faults.

Introduction

The Northridge earthquake produced the largest set of strong motions ever recorded from a large thrust earthquake, including the largest peak velocities ever recorded. The peak accelerations from the Northridge earthquake were significantly larger (about one standard deviation, or about 40% larger) than the median value predicted by current empirical attenuation relations at distances less than about 30 km, as shown in Figure 1a. Since the Northridge earthquake was widely recorded on strong motion instruments, and had a very large societal impact because of its occurrence in a densely populated urban community, it may have a strong influence on the development of empirical ground motion attenuation relations, and on the revision of seismic building codes. It is important to determine whether the ground motions from the Northridge earthquake are anomalous for thrust events, or if they are representative of ground motions expected in future thrust earthquakes, in order to provide a rational basis for evaluating the need for modifications in attenuation relations and design coefficients in the seismic provisions of building codes.

The empirical data base contains few strong motion records close to large thrust earthquakes, making it difficult to assess whether the Northridge ground motions are anomalous based on recorded data alone. To augment the existing data base, we have used the results from our

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broadband strong motion simulation procedure to help assess whether the ground motions from this event were anomalous. We have also used simulations together with recorded data to examine differences in strong motion characteristics on the foot wall and the hanging wall of thefault (Abrahamson and Somerville, 1995), and have found peak acceleration values that are up to 67% larger on the hanging wall than on the foot wall of the fault over the closest distance range of 10 to 30 km.

Comparison Between Broadband Simulations and Recorded Data

A rupture model of the Northridge earthquake that explains the strong ground motions at low frequencies (less than 1 Hz) was was developed by Wald and Heaton (1994). We tested a broadband strong motion simulation procedure, described by Somerville et al. (1995) and Somerville (1995), against the strong motion data using this rupture model. In general, the simulated ground motion time histories show a fairly close resemblance to the waveforms of the recorded motions at low frequencies. This is as expected since the rupture model was derived from the lowpass filtered strong motion waveforms. In Figure 2, we compare the recorded three component time histories at Arleta (top row) with those simulated using empirical source functions derived from the Whittier Narrows aftershock (center row) and the Imperial Valley aftershock (bottom row of each panel). There is considerable resemblance between the recorded and simulated velocity waveforms, especially in the lower frequency features. For sites locatated in the Los Angeles basin, 2-D and 3-D basin effects are required to explain the recorded motions (Graves, 1995; Somerville and Graves; 1993).

The ground motions from the Northridge earthquake and our simulations of these ground motions have a similar pattern of departure from empirical attenuation relations for thust earthquakes: the peak accelerations are at about the 84th percentile level for distances within 20 to 30 km, and follow the median level for larger distances, as shown in Figure 1a and 1c. The same pattern of departure from empirical attenuation relations was obtained in our pre-Northridge simulations of the peak accelerations of an Elysian Park blind thrust event (Saikia, 1993; Figure 1b). The fact that we are able to model this pattern with broadband simulations, and had done so before the Northridge earthquake occurred, suggests that the recorded Northridge motions are not anomalous.

The uncertainty in ground motions predicted by the model is characterized by the procedure described by Abrahamson et al. (1990) using comparison of recorded and simulated motions. For recent well recorded earthquakes, the model predicts the recorded ground motions with no significant bias and with a standard error of a factor of about 1.4 in the frequency range of 0.2 Hz to 35 Hz. A similar result was found for the 1994 Northridge earthquake by Somerville et al. (1995) and Somerville (1995).

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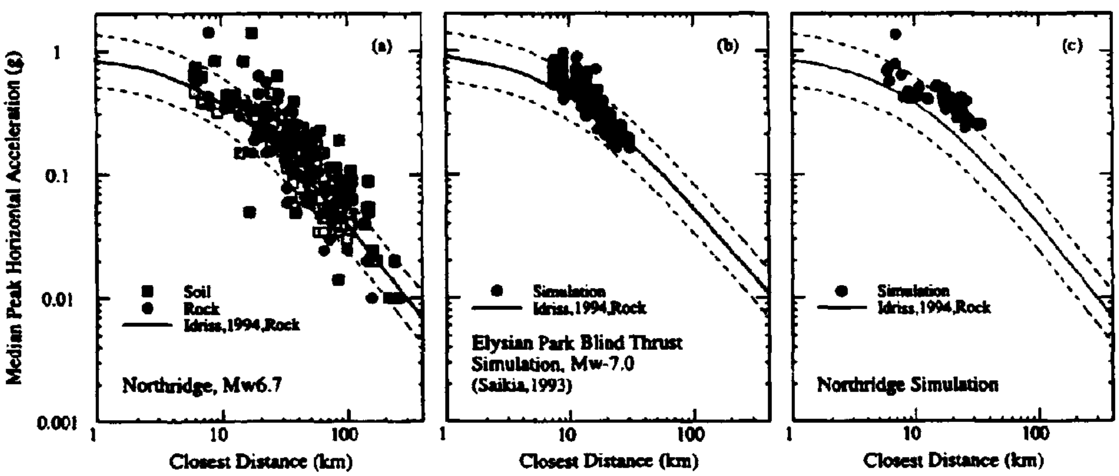

Figure 1. (a). Comparison of recorded peak accelerations from the Northridge earthquake with the attenuation relation of Idriss (1991) for thrust earthquakes recorded on rock and stiff soil. (b). Comparison of simulated peak accelerations for a M_w 7 earthquake on the Elysian Park blind thrust (Saikia, 1993) with the relation of Idriss (1991). (c). Simulations using the Northridge rupture model of Wald and Heaton (1994) compared with the Idriss (1991) relation.

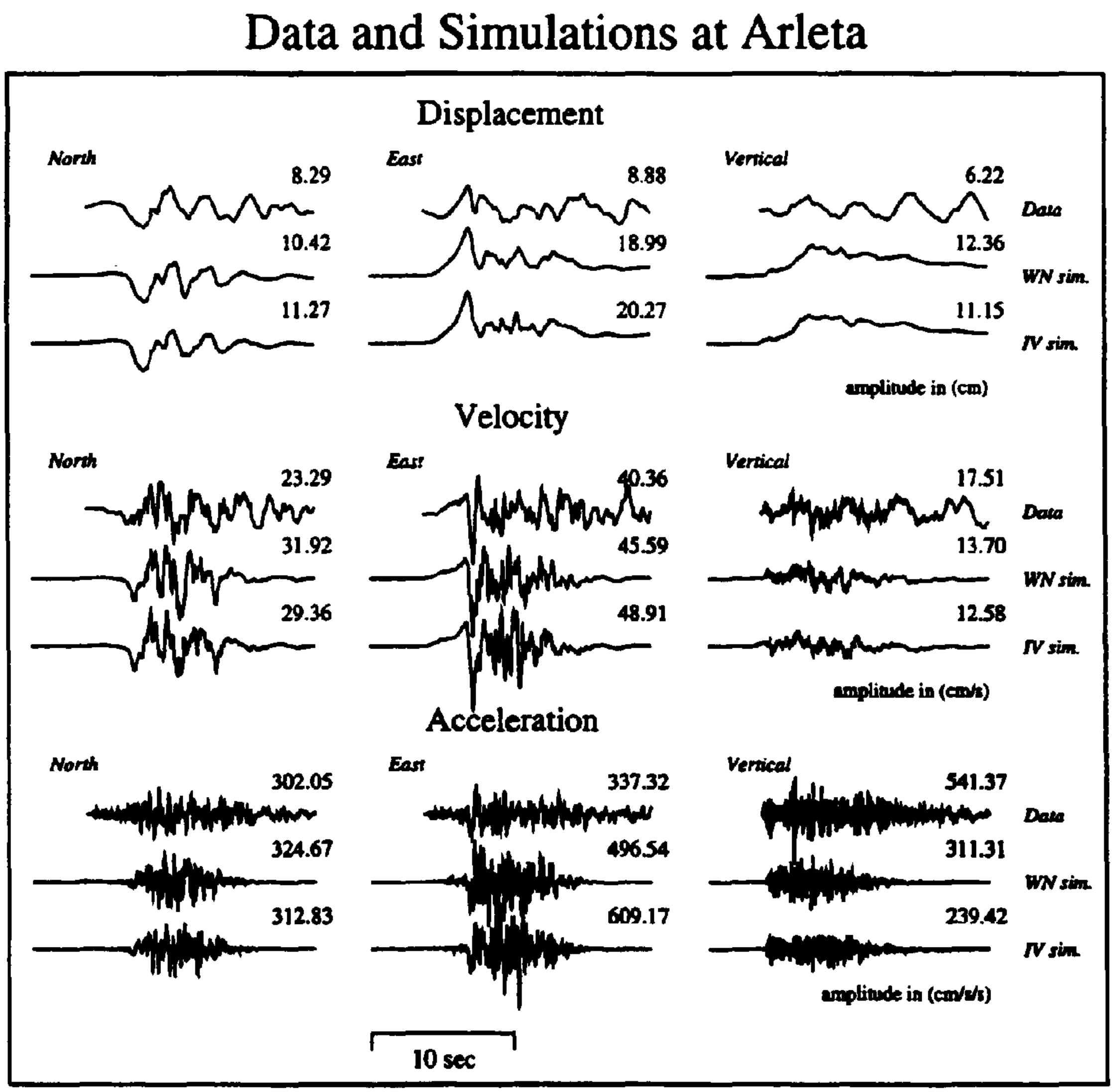

Figure 2. Comparison of recorded (top row) and simulated (middle and bottom rows) displacement, velocity and acceleration time histories at Arleta from the 1994 Northridge earthquake, plotted on a common scale, with peak value given in the top left corner.

Geotechnical Earthquake Hazards Using a Spatial Analysis System

J. David Frost[1] and Ronaldo Luna[1]

Abstract

Geotechnical hazards play a key role in identifying and mitigating against the potential consequences of an earthquake. Geographic Information Systems (GIS) provide an environment which is ideal for conducting earthquake hazard and risk analyses where the different geotechnical and geological hazards associated with seismic events can be combined and the large number of factors required to describe these hazards can be taken into account. Common geotechnical hazards that occur during earthquakes include soil liquefaction, ground motion amplification, and landslides. This paper describes the development of a system that incorporates a methodology for evaluating geotechnical earthquake hazards within a GIS. The methodology combines the spatial distribution and uncertainty associated with geotechnical parameters used in the analytical operations. The methodology has been used in a pilot study of a site in the San Francisco Bay Area.

Introduction

The feasibility of gathering, storing, and processing data has increased exponentially in the last ten years with the proliferation of information systems. Perhaps nowhere is this more evident than in the manner that geographically referenced information is now processed. In many instances in the past, the lack of viable methods to manipulate spatially referenced data was simply bypassed by ignoring their spatial component. For example, in the geotechnical field, the specific location of borings were discounted and an average profile was adopted to develop design criteria for the site in question. This is no longer necessary because of the availability of spatial information systems and computer technology. Given that the occurrence of earthquakes cannot be predicted with the desired certainty, it is therefore important to develop an understanding of the resultant spatial distribution of hazards that can threaten the safety of infrastructure and people.

Significant advances based on a thorough understanding of earthquakes, soil dynamics, and soil mechanics have taken place in the past 3 decades in explaining ground failures and deformation observed during earthquakes and in quantifying the potential for hazard in future events. These types of hazards are responsible for much of the catastrophic damage recorded in recent seismic events (e.g., 1985 Mexico City, 1989 Loma Prieta, 1994 Northridge, 1995 Hyogo). The above noted events were similar in that geotechnical earthquake hazards were manifested in densely populated areas. The damage was spatially distributed within relatively small areal extents due to the local subsurface conditions.

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Geotechnical Earthquake Hazards

Geotechnical earthquake hazards occurs at the end of a series of related steps which start at the seismic source and end at the site where the damage occurs. If one looks at the factors involved in the assessment of ground motion, they can be divided into three topics: (1) regional seismicity, (2) attenuation of ground motion intensity, and (3) local site effects on ground motion.

The Technical Committee for Earthquake Engineering (TC4, 1993) has identified procedures for zonation of seismic geotechnical hazards, which proposes to evaluate the hazards in three levels of detail or complexity (Grade 1, Grade 2 and Grade 3). Zonation grades 1 and 2 use diversified forms of information for qualitative and regional estimates of the hazard. Grade 3 is limited to information obtained from geotechnical investigations within a much smaller areal extent. The spatial evaluation of earthquake hazards from a geotechnical point of view should typically include Grade 3 level data. The more common geotechnical earthquake hazards include:

- Ground motion amplification
- Liquefaction
- Landslides
- Ground rupture
- Earth and retaining structure failure

There is an abundance of literature in the subject of GIS and earthquakes focusing either on a specific site (case study) or for a specific purpose (lifelines). The methodologies and issues surrounding the spatial analyses and geotechnical data were discussed at a NSF sponsored workshop on GIS (Frost and Chameau, 1993). The workshop participants identified areas that needed more attention in the research and industry communities as listed below:

- good links to external analytical models
- the representation of the 3D subsurface in a GIS
- integration of vector and raster data types
- standards for electronic transfer of geotechnical data
- metrics for geotechnical data quality

With these issues in mind, a research program was initiated at Georgia Tech in 1993 with support from NSF and USGS. The focus of these activities has been the development of a spatial geotechnical earthquake hazard analysis system which integrates a number of commercially available and custom engineering programs to permit quantification of hazard at a site.

The Spatial Hazard Analysis System

The system described herein was designed independently of the scale (site specific vs. regional) even though a site specific pilot study was used to validate its functionality. Any site at any scale can be used with the system if the relevant data is introduced to the GIS environment. The development of the system was described by Frost et al., (1992) and Luna, (1995).

The system architecture required decisions about the organization of the system into subsystems, the allocation of subsystems to hardware and software components and major conceptual and policy decisions that formed the framework for detailed design (Rumbaugh, et al., 1991). Each major component of a system is called a subsystem and encompasses aspects of the system that share some common property, similar functionality, the same physical location, or are executed on the same kind of hardware. A subsystem is usually identified by a particular service that it provides to the system. Examples of subsystems are third party software packages as well as original programs developed in "C", shell scripts and higher level macro languages. There are several prototypical architectural frameworks that are common in existing systems and the one adopted for this system was an *Interactive Interface.* With this framework, the system is dominated by the interactions between external agents such that the control of the analysis remains with the user who is expected to have thorough knowledge of geotechnical engineering and spatial analysis.

A tree structure diagram (Figure 1), illustrates the organization of the system with respect to functionality and grouping of subsystems. Under geotechnical earthquake hazards only the three most common hazards are shown and were intended to be custom programs or well-established existing programs [e.g., SHAKE90 for ground motion amplification (GMA)]. While Figure 1 illustrates the components integrated in the system, the data flow and operation of the system is more clearly shown in Figure 2. In order to reduce the dependencies among subsystems, most interactions should be within subsystems. The information flow among the major subsystems implemented at this time are shown as solid lines in the data flow diagram and the anticipated data flow paths are shown shaded. Part of the data flow diagram has a triad arrangement between the GIS, geostatistics and visualization components.

One of the major issues regarding spatially distributed point data is the process of estimating a value at locations where there is no measured or calculated data. The use of geostatistics and the techniques to solve the regional variable problem of estimation are well-established and coded into programs. For this purpose the need for highly interactive tools with the access to graphic displays are required. This motivated the integration of an external program available as public domain code (GeoEAS) with a commercial GIS package (Arc/Info). This allowed for the manipulation of the newly generated spatial data structure from grid raster to vector topology.

Within the geotechnical analytical models subsystem (Figure 2), the liquefaction hazard module has been evaluated with data from Treasure Island in San Francisco Bay (200 borings and 80 cone penetrometer soundings). The representation of the 3D subsurface data in the GIS space was accomplished with a geo-relational database designed using object-oriented concepts that allows for the handling of tables as classes and records as objects. Previous researchers have performed comprehensive risk analyses by identifying seismic sources to evaluate damage, however, the geotechnical hazards have been addressed in a qualitative manner. That is, none of the physical soil behavior during an earthquake was modeled to represent the geotechnical earthquake hazard. The research described herein marks, to the authors' knowledge, the first GIS based implementation that includes quantitative geotechnical hazard analysis. Continuing research is focusing on adding functionality to permit evaluation of other hazards using custom software modules and integrating commercial software for visualizing 3D subsurface data into the system. Hazard analysis studies will be performed for several sites in the United States.

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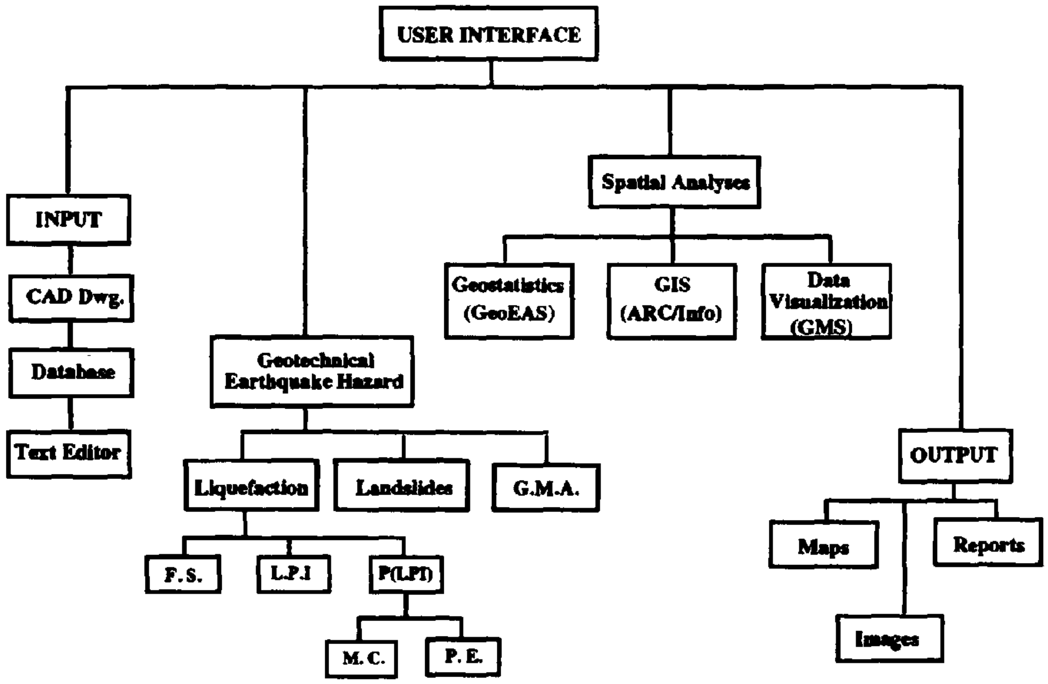

Figure 1 - System architecture (after Luna, 1995)

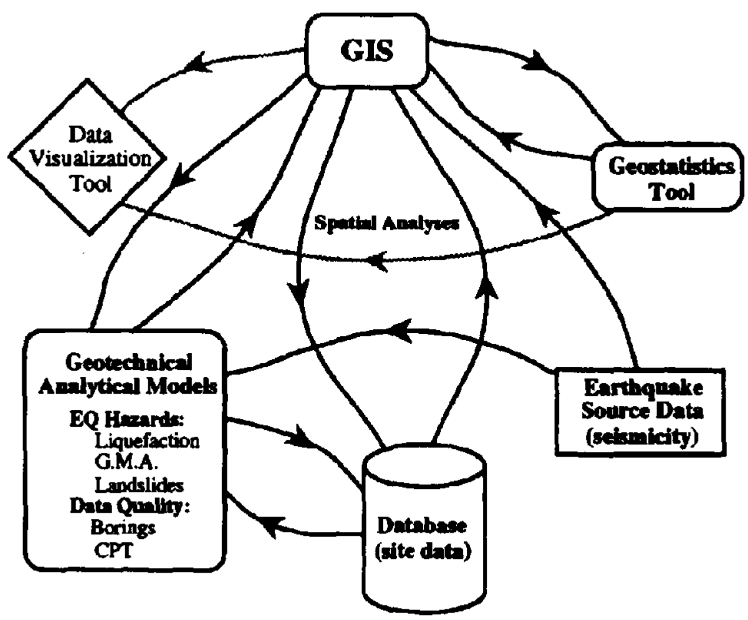

Figure 2 - System Data Flow Diagram (after Luna, 1995)

DAMAGE TO LANDFILLS FROM THE NORTHRIDGE EARTHQUAKE

Edward Kavazanjian, Jr.[1], Neven Matasović[1],
Jonathan D. Bray[2], Anthony Augello[2], and Raymond B. Seed[2]

Abstract

Observations indicate that the general performance of solid waste landfills during the Northridge earthquake was from good to excellent. No landfill showed signs of major damage. One landfill close to the epicenter experienced tears in the geomembrane liner. Other landfills in the epicentral region suffered cracking in cover soils at waste/natural ground transitions, breaking of gas extraction system header lines, and loss of power to the gas collection system.

Introduction

The 17 January 1994 Northridge earthquake provides important observational data on the response of solid waste landfills to strong ground shaking. There are numerous active, inactive, and closed solid waste landfills within 100 km of the epicenter. In a cooperative study funded by the National Science Foundation, GeoSyntec Consultants and the University of California at Berkeley have compiled information on damage at 22 major landfills that experienced estimated free field peak horizontal ground acceleration in excess of 0.05 g.

The location of each of the 22 major landfills is shown on Figure 1. Sixteen of the landfills are classified as California Class III municipal solid waste landfill facilities (MSWLF). OII, BKK, Calabasas, Palos Verdes, Spadra and Simi Valley landfills have hazardous waste disposal units (California Classes I and II), but also received municipal solid waste (MSW). MSW in the greater Los Angeles area has the following typical composition (by volume):

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demolition and construction waste (29%), residential waste (39%), commercial waste (21%), industrial waste (5%), miscellaneous waste (3%) and non-hazardous liquid waste (3%). Sewage sludge, occasionally disposed of at MSWLF, forms less than one percent of the waste accepted at MSWLF. Disposal of non-hazardous liquid waste in solid waste landfills was banned in California in 1985.

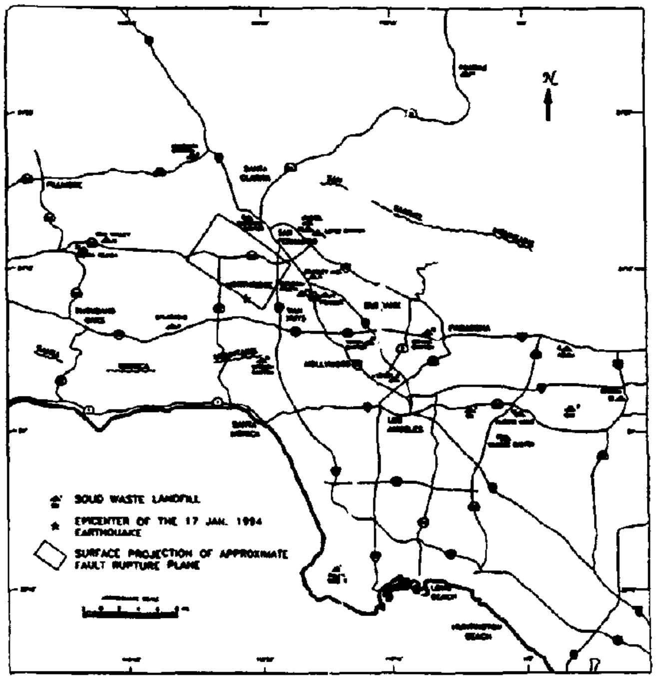

Fig. 1- 22 Major Solid Waste Landfills Within 100 km of Earthquake Epicenter

New waste units and lateral expansions of existing waste units that received waste after 9 October 1993 had to have composite liners (a geosynthetic liner overlying a low permeability soil layer) and leachate collection and removal systems (LCRS) on their base and side slopes. Base and side slopes on which waste was discharged prior to 9 October 1993 may have only low permeability soil liners or may have no liner at all. All of the active and inactive waste cells surveyed after the earthquake were covered by at least 150 mm of daily or interim soil cover. Closed landfills had compacted soil covers. No geosynthetic covers were in place at any of the major landfills subject to moderate or strong ground shaking in the earthquake.

The inclination of active and interim waste slopes at the 22 landfills was typically 1.75H:1V - 2H:1V (horizontal to vertical). At the BKK and OII landfills, interim waste slopes were as steep as 1.3H:1V and 1.4H:1V respectively. At closed landfills, waste face slopes were typically 2H:1V or flatter. Side slopes that underlay waste in the canyon-fill and gravel pit type landfills were typically graded at 1.5H:1V (average grade). However, at the Chiquita Canyon and Bradley Avenue landfills, some side slopes underlying the waste fill approach 1H:1V. Gas collection systems were in place at approximately 50 percent of the surveyed landfills at the time of the earthquake and LCRS were in place at approximately 30 percent of the surveyed landfills.

Observed Damage

The damage to each landfill in Figure 1 was based on post-earthquake inspections by the California Integrated Waste Management Board (CIWMB, 1994), damage surveys conducted by the authors and their co-workers immediately after the earthquake (Stewart, et al., 1994; GeoSyntec, 1994; Kavazanjian, 1994), and earthquake impact reports filed by landfill owners and/or operators with the Regional Water Quality Control Board. A simple five-category damage classification system was developed based upon impairment to the waste containment system and requirements for post-earthquake repair. Categories of damage include Major, Significant, Moderate, Minor, and Insignificant.

The most notable damage to landfills observed following the earthquake, the only damage classified as Significant, were two localized tears in the geomembrane component of the composite liner, one of which was approximately 23 m in length, at the Chiquita Canyon landfill. Both tears were above the level of the waste and were repairable, though specialty contractors were required to complete repairs. Neither disruption of the low permeability soil liner beneath the geomembrane nor disruption to the containment system below the top of the waste was reported by either CIWMB or the landfill operator. At both the Lopez Canyon and Bradley Avenue landfills, two other geosynthetic-lined landfills within 12 km of the zone of energy release were damage was classified as Moderate, CIWMB (1994) reported a small tear of uncertain origin in the exposed geotextile for the side slope liner. No damage was reported at either landfill to the underlying liner and subsequent investigation at both sites attributed the tear in the geotextile to landfill operations, not the earthquake.

The most commonly observed damage to landfills in the Northridge earthquake was surficial cracking in cover soil near transitions between waste fill and natural ground areas. At several sites, cracking in cover soils due to limited amounts of downslope movement (typically less than 150 mm) was observed. At most landfills where cracking of cover soil was observed, the cracks were typically 12 to 75 mm wide with 12 to 75 mm of vertical relief. At Chiquita

Canyon, the waste moved downwards a distance of over 300 mm along the entire geosynthetic-lined backslope of the active waste cell.

Temporary shutdown of the landfill gas extraction system occurred at a number of landfills due to power loss. At several landfills, breaks in the landfill gas extraction system headers and condensate lines were reported. In all cases, operation of the gas extraction system was restored within 48 hours. At the Russell Moe landfill, a closed landfill without an active gas extraction system, reports of gas odors, later attributed to ruptured lines from propane tanks, caused the trailer park on top of the landfill to be evacuated.

Conclusion

Observations of damage indicate that the general performance of solid waste landfills during the 17 January 1994 Northridge earthquake was from good to excellent. These observations are in general agreement with the observations on landfill performance in past earthquakes. However, none of the landfills subjected to strong ground motions in past earthquakes were lined with a geosynthetic liner. Furthermore, no landfill with a geosynthetic cover has ever been subjected to strong ground shaking from an earthquake. These observations, combined with observation of damage to the geomembrane liner at one landfill in the Northridge event and the fact that no landfill with a geosynthetic cover has ever been subjected to strong ground motions, indicate that, despite the general observation of the good to excellent performance of solid waste landfills in past earthquakes, caution is warranted in the design of modern, geosynthetic-lined and/or covered landfills subject to seismic loading.

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NATIONAL GEOTECHNICAL EXPERIMENTATION SITES PROGRAM

J. Benoît and P.A. de Alba[1]

Abstract

National Geotechnical Experimentation Sites (NGES) have been recently implemented in the United States. These reference sites are documented into a convenient database which allows site characteristics and soil properties to be examined by potential users. Currently a data dictionary and a DOS version of the database, with information on forty sites have been completed. A Windows® version with 2-D and 3-D graphics is under development and will be accessible through internet.

National Geotechnical Experimentation Sites Program

A system of test sites is now available in the U.S. through the National Geotechnical Experimentation Sites (NGES) Program. The NGES system of multiple user test sites provides easy access to well-documented field sites, thus greatly facilitating the development of new techniques of soil characterization and earthwork construction and allowing geotechnical researchers to select the most appropriate site for their needs on the basis of soil type, site location and available geotechnical data. Associated with this NGES program is a central data repository which provides a database designed to promote exchange of information, resulting in a more cost effective use of available research funds.

To date, information regarding forty sites has been stored in a central database and in an NSF-funded catalog (Benoît and de Alba, 1993). These sites are listed in Table 1. The database is designed as a user-friendly system shell, with on-line computer search and data retrieval capabilities for essential information about multiple-user test sites, such as generalized soil conditions and representative soil properties, list of available test data, site logistics, conditions and services, published references, and other pertinent site information. The database is continually updated as new information becomes available. An initial DOS version has currently been completed using funds from the Federal Highway Administration (FHWA). As part of a new initiative, a Windows® version will allow graphical capabilities in 2-D and 3-D. The database will eventually reside at the FHWA research center and will be accessible via internet.

At an NSF/FHWA Workshop on Selection and Management of National Geotechnical Experimentation Sites (Benoît and de Alba, 1991), five of the forty sites previously described were selected, classified as Level I or Level II sites, and recommended for immediate funding

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by NSF and FHWA. The remaining sites are classified as Level III. Level I sites are those sites which most closely fit the combined criteria of research areas identified through several workshops as of significant national importance, and of favorable site characteristics. Theme research areas are: geotechnical earthquake engineering (liquefaction, site amplification, and permanent deformations), calibration of new equipment, proof testing of site improvement techniques, geo-environmental problems, expansive clay problems and foundation prototype testing. Sites qualifying in the theme areas were also screened based on a short list of site characteristics consisting of: soil types and stratification, site size, interest and energy of site proponents, security, and long-term accessibility. Level II sites fit most of the requirements, but have size limitations in their current configurations. These sites may be expanded under the initiative of their proponents so that they might eventually be upgraded to Level I. Level III sites currently do not adequately meet the requirements. These sites are not recommended for immediate financial support, but may be considered at a later date should their condition improve to fit the requirements. For both Level I and Level II sites, detailed individual field and laboratory test results are an integral part of the database, and are made accessible to potential users and researchers, allowing them to review the quality and numerical details of the results.

NSF/FHWA funding is currently available over a three-year period (1992-1995) to enhance the database at these five sites. A system management board has been created to oversee the development of these sites and, most importantly, to insure the continued maintenance and enhancement of the NGES system in the long term, by encouraging the use of these sites, and identifying sources of private and public funding for further development.

In an attempt to standardize all the geotechnical information to be stored in the NGES database, a manual was written to provide site managers with a standard format for entering data into the database. The manual is called the NGES Data Dictionary and is available to all site managers and other interested geotechnical engineers. The Data Dictionary was modeled after existing standards and databases, and is similar to that generated by the Association of Geotechnical Specialists (AGS) in the United Kingdom for their electronic transfer of geotechnical data in ground investigations (AGS, 1992). At the present time, the Data Dictionary has standards for site information, borehole information, stratum descriptions, specimen information, laboratory tests and in situ tests. An example of the Data Dictionary is shown in Table 2. Spreadsheet templates are currently being created to allow the site managers to easily generate complete and accurate data files.

Acknowledgements

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Table 1. National Geotechnical Experimentation Sites

Level	Site Name	Location
I	Treasure Island Naval Station, (Fire Station #1)	San Francisco, California
	Texas A&M University, Riverside Campus - Clay Site	College Station, Texas
	Texas A&M University, Riverside Campus - Clay Site	College Station, Texas
II	Northwestern University, Lake Fill Site	Evanston, Illinois
	University of Massachusetts - Amherst	Amherst, Massachusetts
	University of Houston Foundation Test Facility	Houston, Texas
III	EPRI/USGS Earthquake Soil Liquefaction Site	Cholame Valley, California
	Hamilton Air Force Base	Novato, California
	Minor Creek Landslide, Redwood Creek Drainage Basin	Northwestern California
	EPRI Seismic Array	Parkfield, California
	San Francisco Waterfront	San Francisco, California
	Wildlife Site	Calipatria, California
	Expansive Clay Shale Test Site	Fort Collins, Colorado
	CSU Explosives Test Site (AFOSR)	Fort Collins, Colorado
	Platte River Test Site	Kersey, Colorado
	Anacostia Naval Air Station	Washington, DC
	University of Florida - Kanapa	Gainesville, Florida
	"American Bottoms" Mississippi River Floodplain	Collinsville, Illinois
	Old Route 95, Test Embankment	Saugus, Massachusetts
	University of Minnesota Underground Space Center, Foundation Test Facility	Rosemount, Minnesota
	Minnesota Cold Regions Pavement Test Facility	I-94, Between the Towns of Albertville & Monticello, MN
	Eaton Dam	Leadwood, Missouri
	Frost Effects Research Facility at U.S. Army Cold Regions Lab	Hanover, New Hampshire
	I-87/I-90 Interchange	Albany, New York
	Lockport Expressway	Erie County, New York
	Massena High School	Massena, New York
	Route 37 over OBPA Railroad	Ogdensburg, New York
	State Fair Boulevard/Oswego Boulevard	Syracuse, New York
	6 miles west of Wagoner, OK on SH52	Wagoner, Oklahoma
	Chamberlain, South Dakota	Chamberlain, South Dakota
	Family Hospital Center Site	Amarillo, Texas
	Texas A&M Univ., College of Agriculture Equip. Compound	College Station, Texas
	State Highway 146 at Houston Ship Channel	Baytown, Texas
	Arthur V. Watkins Dam	Willard, Utah
	Continuous Wave Electron Beam Accelerator Facility (CEBAF)	Newport News, Virginia
	Kipp's Farm	Blacksburg, Virginia
	Parking Lot of Schnabel Engineering Associates	Richmond, Virginia
	Schnabel Engineering Site	Richmond, Virginia
	Manchester - Dorset U.S. Route 7	Manchester, Vermont
	Brandon, Vermont - Route 73	Brandon, Vermont
	Teays Valley	Near Charleston, West Virginia

Table 2: CPT Data Dictionary

Group Name: CPT			Cone Penetration Test	
Status	**Heading**	**Unit**	**Description**	**Example**
*	SITE_ID		Site identification	CATIFS
*	HOLE_ID		Exploratory hole identification number	CPTU.3
*	CPT_TYPE		Type of cone	CPTU
	CPT_BRND		Cone manufacturer	Wissa
	CPT_TIPA	cm^2	Tip area	10
	CPT_TIPX	degrees	Tip apex angle	60
	CPT_SLVA	cm^2	Friction sleeve area	150
	CPT_SLVP	mm	Distance from center of sleeve to tip	103
	CPT_FLTP		Position of filter element(s)	Tip
	CPT_FLTT	mm	Thickness of filter element	
	CPT_FLTM		Filter element(s) material	Carborundum
	CPT_FLTD	micron	Filter material pore diameter	1
	CPT_SATF		Saturation fluid	Water
	CPT_SATT		Saturation technique	Vacuum 24 hours
	CPT_ARCT		Area ratio correction for tip	0.9
	CPT_ARCS		Area ratio correction for sleeve	0.015
	CPT_CAPS	KN	Capacity of surface load cell	
	CPT_CAPT	MN	Capacity of tip load cell	0.04448
	CPT_CAPF	MN	Capacity of friction sleeve load cell	0.04448
	CPT_CAPU	kPa	Capacity of pore pressure transducer	1380
	CPT_CALD	mm/dd/yy	Last calibration date	07/26/91
	CPT_REDD	mm	Friction reducer diameter	51
	CPT_REDL	mm	Friction reducer location from tip	470
	CPT_DYNH		Details of hammer system for dynamic cone test	
	CPT_ADVR	mm/sec	Rate of penetration	20
#1	CPT_DPTH	m	Depth of tip measurement	
#1	CPT_QC	kPa	Static or dynamic cone tip resistance (q_c)	
#1	CPT_FS	kPa	Friction sleeve resistance at tip depth (f_s)	
#1	CPT_U	kPa	Penetration pore pressure	
#1	CPT_SHRW	m/sec	Shear wave velocity	
#1	CPT_INCL	degree	Cone inclination	
#1	CPT_OTH1		Additional measurements: pore pressure sensor, conductivity, lateral stress, temperature (specify units)	
#1	CPT_OTH2		Additional measurement: pore pressure sensor, conductivity, lateral stress, temperature (specify units)	
#2	CPT_DPTD	m	Depth of dissipation test	
#2	CPT_DIST	min	Elapsed time of reading for dissipation test	
#2	CPT_DISU	kPa	Pore pressure during dissipation	
	CPT_REM		Remarks	

EARTHQUAKE GROUND MOTION MODELING IN LARGE BASINS

J Bielak[1]

Abstract

The main objective of this research is to develop the capability for predicting, by computer simulation, the ground motion of large basins during strong earthquakes and, ultimately, to use this capability to study the seismic response of the Los Angeles Basin. A brief description of the overall approach is provided herein, together with some results for an idealized three-dimensional basin, and for a two-dimensional model of a zone of Kirovakan, where structures experienced extensive damage during the 1988 Armenia Earthquake due to soil and valley effects.

Approach

A set of tools is being developed for simulating earthquake ground motion in large basins on parallel computers. Such simulations represent a grand challenge both because of their importance to hazard mitigation, and due to their modeling and computational complexity. The purpose of the tool set is to shield the application specialist from the details of high-performance computing. The central part of our tool set is "Archimedes," a special purpose compiler being developed to support the mapping of unstructured meshes computations arising from the solution of PDEs on parallel systems. The user provides two things to Archimedes: a description of the surface geometry of the problem domain, and a high-level algorithm for performing finite element calculations which solve the physical problem. Archimedes includes a mesh generator, a mesh partitioner, placement and routing heuristics, and a code generator. With the exception of the mesh generator, which is still under development for three-dimensions, all of these components can be used for two- or three-dimensional problems. For the high level algorithm we have implemented both a conditionally stable explicit method and an unconditionally stable implicit method, in which the equations at each time step are solved iteratively.

Numerical Examples

As a first example we examine the ground motion of the section of Kirovakan shown on Fig. 1a. What is of great interest here is that in zone 2 a large percentage of the 4-5 story buildings and a smaller fraction of 1-3 story buildings suffered total collapse during the 1988 Armenia Earthquake. By contrast, no structures collapsed and only a small number of structures experienced heavy damage in zone 3. In zone 2 the soil consists of very stiff clay deposits up to 150 m deep, with shear wave velocity varying between 200 m/s and 900 m/s. The soil in zone 3 consists of much shallower deposits of silty gravel with sands, with shear wave velocities varying between 500 m/s and 900 m/s; the surrounding bedrock has a shear wave velocity of 1200 m/s. In an effort to explain the damage distribution we represented the valley as a two-dimensional damped elastic model with two layers in zone 3 and four layers in zone 2 (see Fig. 1b for corresponding finite element mesh), and subjected it to an incoming vertical SH-wave. Figure 1c shows the amplitude of the generalized transfer function of the ground displacement at all surface points within the valley, for a wide range of frequencies of excitation. The dark spots for which the amplification is largest correspond to the resonant frequencies of the valley within this range. In zone 3 the maximum dynamic amplification due to site conditions and valley

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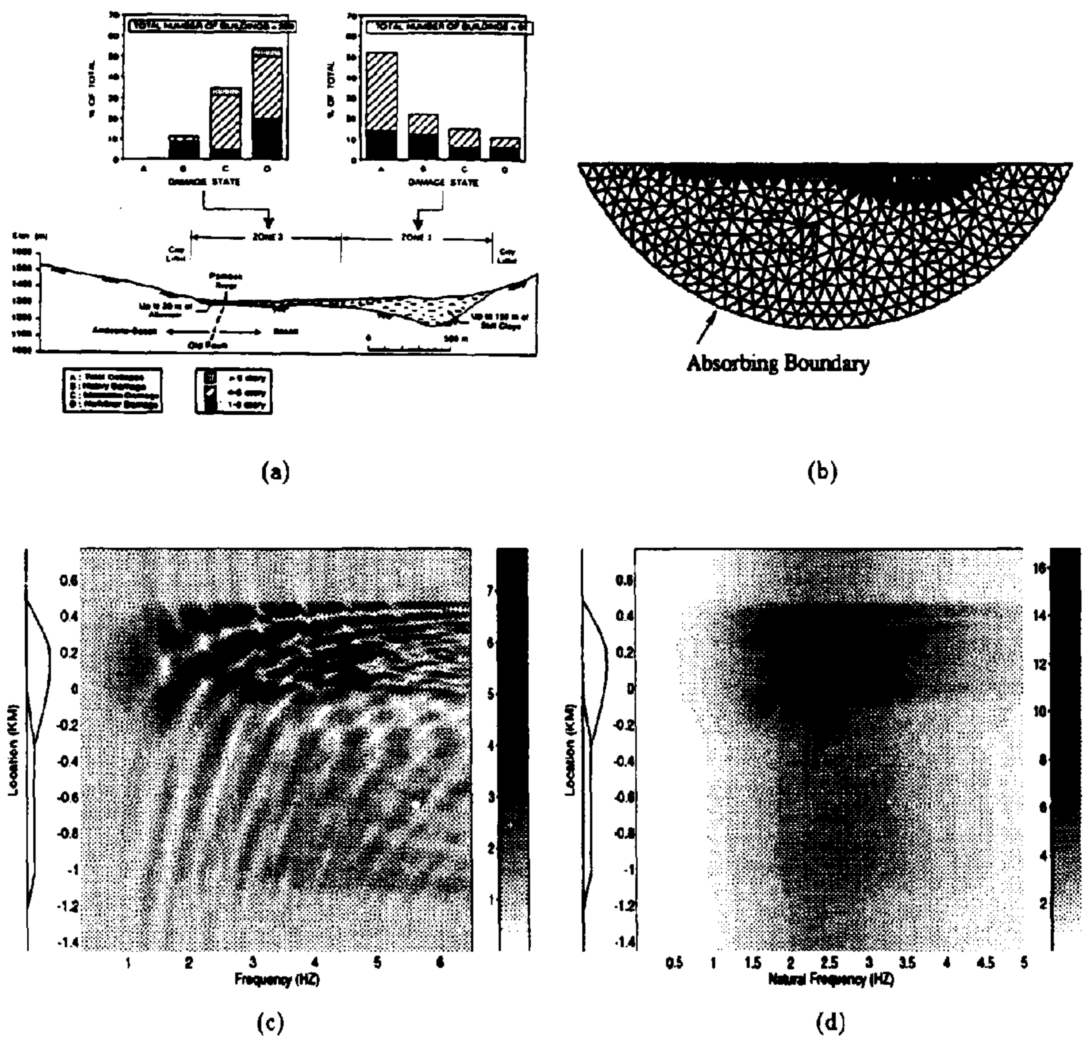

Figure 1. Simulation of ground motion and structural response in Kirovakan during the 1988 Armenia Earthquake. (a) Geotechnical profile through zones 2 and 3 in Kirovakan and their corresponding damage statistics (after Yegian et al, 1994); (b) Finite element mesh for model of valley and surrounding bedrock in zones 2 and 3; (c) Amplitude of transfer functions for valley displacement at all surface points for vertical incident SH-wave; (d) Generalized response spectra due to incident transient Ricker SH-wave for simple oscillator at all points within the valley (2% structural damping).

effects, with respect to the free-field amplitude of surface ground displacement, is about 2.5. On the other hand, within the softer zone 2 this amplification reaches values up to 7.8 for many frequencies. To examine the effect that this ground amplification has on structural response, Fig. 1d shows the generalized response spectra for a simple damped oscillator for all locations within the valley, when the incident wave representation in time is defined by a Ricker pulse with a central frequency of 1.7 Hz. This value was chosen so that the peak ordinate of the corresponding response spectrum for the structure subjected directly at its base to a Ricker pulse excitation would occur at the same frequency as that produced by the back-calculated rock outcrop motion reported by Yegian et al (1994) for the 1994 Armenia earthquake. From Fig. 1d it is seen that large structural response, with normalized displacements up to 16 can occur within zone 2, especially for frequencies between 2.2 and 3.5 Hz, typical of 4-5 story buildings. By contrast, the normalized response is only about 4.5 in zone 3 and 2.5 outside the valley. These results demonstrate that the severe damage in zone 2 in Kirovakan was due primarily to local site conditions. A double resonance phenomenon seems to have occurred in which the seismic excitation caused resonance in the valley at frequencies close to the natural frequencies of the 4-5 story buildings which suffered the most extensive damage.

To illustrate our methodology for three-dimensional situations we investigate the response of the idealized basin depicted in Fig. 2a. This basin is approximately 4km x 3km with a maximum depth of 1km. The shear wave velocity of the material within the valley is 1 km/s and that of the surrounding medium is 2 km/s. A finite element mesh consisting of piecewise linear tetrahedra has been constructed, with a cylindrical absorbing boundary of 6km radius and a base 1.5km deep, made up of dashpots and springs. The seismic excitation consists of an incoming transient SV-wave, for which the time variation is again defined by a Ricker pulse. The central frequency is chosen as 0.42 Hz, with a corresponding wavelength of the SV-waves of 2.4km , and a pulse duration of approximately 3s. The three components of the resulting surface displacement for points lying along the x axis are shown on Fig. 2b. Notice that surface Rayleigh and Love waves are generated within the valley, giving rise to a seismic motion of long duration. Figure 2c depicts the amplitude of the transfer function of the horizontal displacement u for points on the x axis. As in Fig. 1b, this figure clearly identifies resonant frequencies of the systems for which the amplifications tend to be large. To illustrate the spatial distribution of motion over the entire valley, Fig. 2d shows the normalized horizontal displacement parallel to the y-axis for a particular frequency. The variation of ground motion intensity over short distances is quite dramatic. This means that within a closed basin it might be possible for two structures with very similar dynamic characteristics to respond quite differently to a seismic excitation, even if they are located within the same vicinity.

Even though the examples considered herein are quite limited in scope, they help demonstrate that by allowing one to examine the spatial distribution of ground motion during earthquakes, simulations of the seismic response of realistic basins with the aid of high performance computing can be a useful tool for seismic hazard assessment, and for seismic zonation and microzonation.

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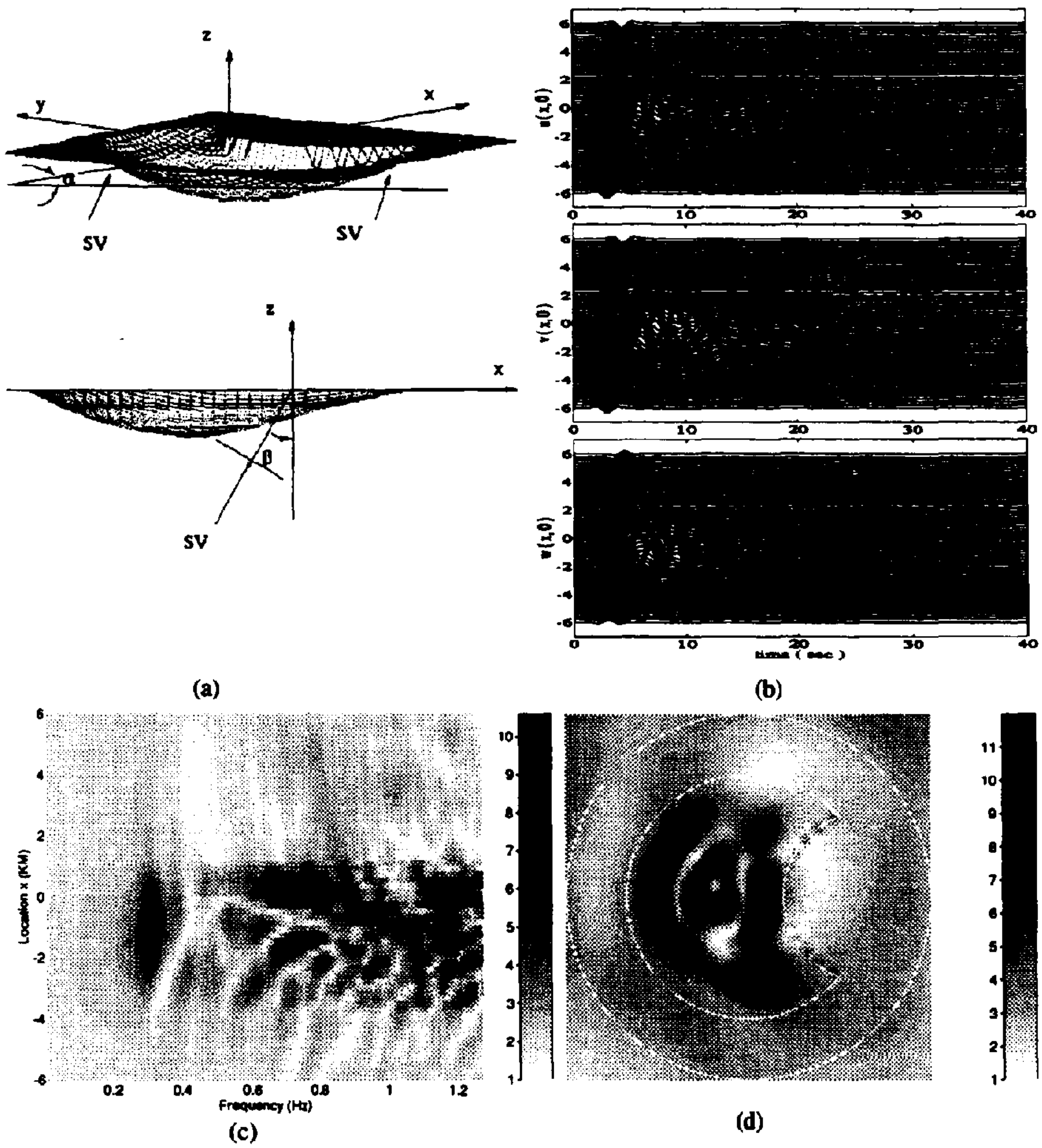

Figure 2. Simulation of seismic ground motion in a three-dimensional basin due to an inclined incident SV-wave. (a) Model of basin, angle of line of strike with x axis, $\alpha = 45^\circ$, $\beta = 20^\circ$; (b) Synthetics of displacements, u and v, in two horizontal directions and w in vertical direction, at points on the x axis; transient excitation is Ricker pulse with $f_r = 0.42$ Hz; (c) Amplitude of transfer functions for displacement u on x-axis for steady-state harmonic incident wave; (d) Normalized horizontal displacement v parallel to the y axis for a steady-state frequency of excitation f = 0.55Hz.

THREE-DIMENSIONAL SLOPE STABILITY

Timothy D. Stark[1] and William R. Monson[2]

Abstract

The 1988 slope failure at the Kettleman Hills Waste Repository has forced government agencies to consider and sometimes require three-dimensional (3-D) slope stability analyses. However, 3-D slope stability analysis is in its infancy, and thus 3-D stability methods are not readily available to government agencies or practicing engineers. More importantly, the accuracy of existing 3-D stability methods has not been verified using field case histories. The main objectives of this ongoing research is to determine: (1) the accuracy and applicability of existing three-dimensional slope stability methods using field case histories, (2) the parameters or assumptions that significantly affect the three-dimensional factor of safety, (3) the field situations, if any, where a three-dimensional factor of safety is less than an appropriate two-dimensional factor of safety, (4) the three-dimensional effects on two-dimensional back-calculated shear strength parameters, and (5) to develop a 3-D slope stability method that incorporates beneficial features of existing 3-D methods, predicts field slide surface and slide mass geometries, provides excellent agreement with field case histories, and is suitable for practice.

Introduction

At present, most slope stability analyses are performed using a two-dimensional (2-D) limit equilibrium method. These methods calculate a factor of safety against failure for a slope assuming plane-strain conditions. Therefore, it is implicitly assumed that the slip surface is infinitely wide, and thus the three-dimensional (3-D) effects are negligible. Clearly, slopes are not infinitely wide and 3-D effects influence the stability of all slopes. There are a number of situations where three-dimensional effects are significant and definitely should be considered. These situations include: (1) slopes that are curved in plan (Baligh and Azzouz, 1975) or form ridges or corners (Giger and Krizek, 1975 and 1976; Hungr et al., 1989), (2) slopes that have asymmetry caused by inclusions such as soil-geosynthetic liner systems, drainage blankets, faults, or rock joints, (3) slopes that have shear strength or piezometric conditions that vary in the transverse direction, (4) dams in narrow or curved valleys, and (5) slopes that are surcharged by loads or cut by excavations (Giger and Krizek, 1976).

Previously, it was anticipated that consideration of three-dimensional effects will increase the factor of safety, and thus conventional two-dimensional methods were assumed to be conservative. Therefore, three-dimensional stability analyses were rarely performed in

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practice. However, Seed et al. (1990) reported that three-dimensional effects can result in a factor of safety that is lower than the two-dimensional factor of safety. This conclusion was based on their investigation of the slope failure in Landfill Unit B-19 at the Kettleman Hills Hazardous Waste Repository. The waste repository consists of a large oval-shaped bowl that was excavated in the ground to a depth of approximately 100 feet. After the failure, Byrne et al. (1992) revealed that sliding occurred along the secondary clay/geomembrane interface in the landfill liner system. They concluded that the slopes surrounding the center of the "bowl" produced three-dimensional driving forces that could not be incorporated into a two-dimensional analysis. Seed et al. (1990) computed a 3-D factor of safety that is less than the 2-D safety factor. As a result, Seed et al. (1990) stated that "possible three-dimensional effects may have a significant impact on the overall stability during placement of waste fill." It should be noted that this conclusion is based on a 3-D factor of safety calculated using a "multiple-block analysis" that was developed by Seed et al. (1990). In this analysis, a number of simplifying assumptions are made and only force equilibrium is satisfied.

Based on the research by Seed et al. (1990), government agencies and private firms have been forced to consider three-dimensional effects in slope stability analyses of natural and man-made slopes. However, three-dimensional slope stability analysis is in its infancy. As a result, existing 3-D slope stability analyses are not readily available to practicing engineers nor have these methods been verified using field case histories. In addition, recommendations for 3-D design factors of safety and the applicability of the methods to field conditions have not been developed. As a result, there is considerable confusion over the importance of 3-D slope stability effects and how to conduct 3-D slope stability analyses.

Objectives of Ongoing Research

The main objectives of this ongoing research program are to (1) verify the performance of existing 3-D slope stability methods using field case histories, (2) develop guidelines/recommendations for selecting and using the most appropriate 3-D stability method for problems typically encountered in practice, (3) determine field situations, if any, where a 3-D factor of safety is less than 2-D factors of safety, (4) develop a back-analysis procedure that accounts for 3-D effects, and thus yields back-calculated shear strength parameters that are in better agreement with field strengths, (5) investigate the effects of seismic forces on the 3-D factor of safety, (6) investigate the 3-D geometry of the slide surface and slide mass using field case histories to verify assumptions concerning slide surface and/or slide mass geometry, and (7) develop a 3-D slope stability method that incorporates the important features of existing methods, predicts field geometry of the slide surface and slide mass, and provides excellent agreement with field factors of safety. This research will provide a much needed advance to the State-of-the-Art and the State-of-the-Practice of 3-D slope stability analyses.

Preliminary Results

Limitations of existing 3-D slope stability methods and the urgency to conduct 3-D stability analyses in practice have resulted in the initial phase of this research focusing on techniques for incorporating 3-D effects in a 2-D stability analysis. The main uncertainty in a 2-D analysis is the selection of the critical 2-D cross section. Preliminary results indicate that the critical 2-D cross section will be parallel to the direction of the resultant of the 3-D driving and resisting forces. Therefore, a simplified method was sought to determine the orientation of the resultant 3-D force, which would facilitate selection of the critical 2-D cross section and reduce the need for a 3-D analysis.

Initial research results indicate that the resultant 3-D force can be evaluated using a vector analysis similar to that presented by Hendron et al. (1980) for the analysis of slopes in jointed rock masses. In this analysis, the geometry of the failure mass and the orientation of individual planes are taken into account to determine the direction of the resultant 3-D force acting on the failure mass. Two-dimensional cross-sections parallel to the resultant 3-D force can be analyzed to determine the critical 2-D cross section and factor of safety. The vector analysis is better suited for practice than a comprehensive three-dimensional analysis, and should facilitate the consideration of three-dimensional effects in two-dimensional stability analyses. Additional research is being conducted to determine the applicability of a vector analysis to a wide range of natural and man-made slope geometries.

Evaluation of field case histories using existing three-dimensional slope stability analyses has shown that the 3-D slope stability software CLARA 2.31, developed by Hungr et al. (1989), provides the best agreement with field factors of safety. However, improvements could be made in the stability program by incorporating a non-linear Mohr-Coulomb failure envelope, a back-calculation routine, and using a stability method that satisfies all conditions of equilibrium (e.g., Spencer's method, 1967). The evaluation of case histories also revealed that the three-dimensional factor of safety is dependent on the direction of sliding of the sliding mass. This is especially true in the case of waste containment facilities because of the complex geometries that are involved in sliding. Sliding along a three-dimensional failure surface will occur in the direction of the resultant three-dimensional force acting on the failure mass, and the critical two-dimensional cross-section will be parallel to the resultant three-dimensional force direction. This fact was used to evaluate the potential for the 3-D factor of safety to be less than the 2-D factor of safety. As a result, another initial phase of the research focused on clarifying whether a 3-D factor of safety can be less than representative 2-D values. Re-evaluation of the Kettleman Hills landfill failure using Janbu's simplified stability method indicates that the three-dimensional factor of safety is greater than the two-dimensional factor of safety when the appropriate direction of sliding is used to determine the two-dimensional cross-section. Therefore, if a 2-D cross section is selected parallel to the direction of sliding, the 2-D factor of safety will be less than the 3-D value, and will be representative of the field stability.

Conclusions

The following conclusions are based on the comparison of existing three-dimensional slope stability methods and the following field case histories: (1) Kettleman Hills Waste Repository, (2) Oceanside Manor landslide, (3) San Luis Dam slide, and (4) Ririe Dam.

1. The microcomputer program CLARA 2.31 provides an accurate estimate of the three-dimensional factor of safety for the types of slope stability case histories considered to date. The program utilizes an extension of Janbu's simplified or Bishop's modified stability method to three-dimensions. However, improvements could be made in the program by incorporating a non-linear Mohr-Coulomb failure envelope, a back-calculation routine, and by satisfying all conditions of equilibrium.

2. The three-dimensional factor of safety is dependent on the direction of sliding of the failure mass.

3. The three-dimensional factor of safety increases as the concavity of the slope curvature increases.

4. The three-dimensional factor of safety is greater than the two-dimensional factor of safety for the field case histories considered to date.

5. A vector analysis can be used to identify the 3-D direction of sliding, and thus the orientation of the critical two-dimensional cross section. This allows three-dimensional effects to be incorporated into 2-D stability analyses. The proposed vector analysis is better suited for practice than a comprehensive 3-D analysis.

6. The most important aspect of any slope stability analysis is the determination of the mobilized shear strength parameters.

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PROJECT VELACS: RPI CONTRIBUTION

Ahmed-W. Elgamal[1] and Ricardo Dobry[2]

Abstract

The NSF funded VELACS (VErification of Liquefaction Analysis by Centrifuge Studies) project involved cooperative efforts of seven universities. Centrifuge dynamic shaking tests were conducted in an effort to quantitatively study the destructive consequences of soil liquefaction; and verify the state-of-the-art in associated predictive computational-simulation capabilities. An overview and assessment of this project is presented herein, along with the studies conducted at Rensselaer Polytechnic Institute (RPI).

Introduction

Soil liquefaction has been shown to be a major source of damage and costly repairs during many earthquakes, including the recent Hyogo-Ken Nanbu event (Bardet et al. 1995, Comartin et al. 1995). Until recently, most soil liquefaction research had been concerned with the estimation of liquefaction potential. VELACS (Arulanandan and Scott 1993, 1994) was focused on studying the destructive consequences of liquefaction and calibrating predictive computational techniques, through conducting a broad-band series of centrifuge shaking tests (Fig. 1). The centrifuge applies a high gravity field (N) on small-scale soil models in order to simulate the behavior of an N-times larger soil stratum (in linear dimensions). Each employed centrifuge machine, was equipped with an in-flight base shaking capability (shake table). Tests were conducted through the cooperative efforts of seven universities: University of California at Davis, California Institute of Technology, Cambridge University in the UK, University of Colorado at Boulder, Massachusetts Institute of Technology, Princeton University, and Rensselaer Polytechnic Institute. Ten of the twelve models (Fig. 1) were tested by different groups of researchers at two or three of the above mentioned research institutions; in order to check accuracy and repeatability of the conducted experiments and testing procedure.

Within VELACS, an extensive a-priori (blind or Class A) predictive computational effort was also undertaken. The numerical predictions (presented by a total of 23 research groups world-wide) provided an overall picture regarding reliability and limitations of currently available numerical tools. Overall, VELACS constitutes a prime example of problem focused research (Whitman 1994).

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Centrifuge Modeling At RPI

Models 1, 2, 4b, 7, and 12 (Fig. 1) were tested at RPI (Adalier and Elgamal 1993; Carnevale and Elgamal 1993a, 1993b; Taboada and Dobry 1993a, 1993b). An overview and lessons learned concerning models 2, 4a, 4b, and 6 were also presented (Dobry and Taboada 1994; Elgamal, Adalier, and Zeghal 1994) in an International Conference held at Davis to discuss VELACS (Arulanadan and Scott 1994). The results provided valuable insights into: i) post-liquefaction behavior of loose sand deposits (model 1), ii) lateral ground deformation (Fig. 2) of mildly sloping ground (model 2), iii) effect of overlying low permeability strata on liquefaction (models 4a, 4b, and 6, Fig. 3), and iv) response of shallow foundations supported on saturated granular soils (model 12)

Conclusions

The centrifuge-model VELACS tests and associated numerical predictions provided a wealth of data concerning: i) the engineering consequences of liquefaction on soil and soil-structure systems in terms of associated permanent deformation patterns and magnitudes, and ii) the capabilities and limitations of current state-of-the-art computational tools. The results constitute an extensive and valuable resource of insights and knowledge.

Acknowledgments

VELACS was supported by the National Science Foundation grant No. BCS- 9016880 with Dr. Clifford Astill as Program Manager.

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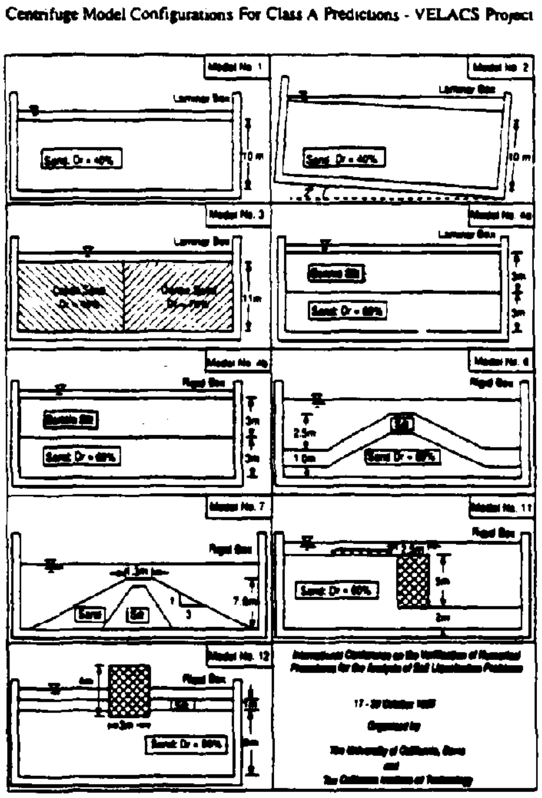
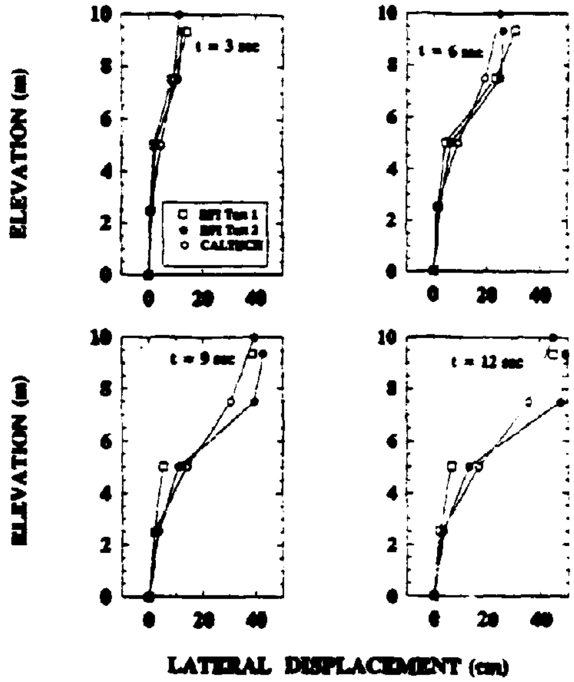


Fig. 1 (From Arulanandan and Scott 1993).



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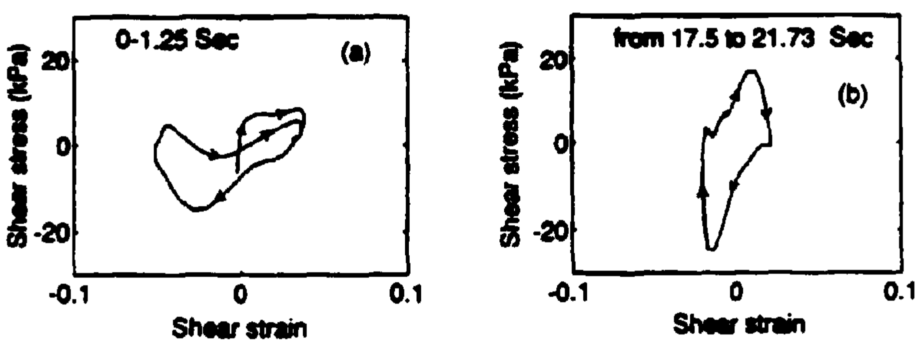

Fig. 3 Sample calculated stress-strain response. (From Elgamal et al. 1994).

NATURAL HAZARD MITIGATION
NSF GRANTEES WORKSHOP

SESSION IV - STRUCTURAL SYSTEMS II

"System Integrated Approach in Earthquake Hazard Mitigation"
G. Lee

"Evaluation of Two Precast Parking Garages Damaged during the 1994Northridge Earthquake"
S. Wood, J. Stanton, and N. Hawkins

"Prioritization for Rehabilitation of Buried Lifelines"
L. Wang

"On the Scale Analysis of Electrorheological Fluid Dampers"
F. Gordaninejad and R. Bindu

"Computational Issues Associated with the Analysis of Actively Controlled Structures"
H. Smith and A. Schemmann

"Intelligent Control of Structures"
R. Shoureshi

"Consequences of Damage to Bridges in the Northridge Earthquake"
A. Kiremidjian, N. Basoz, K. Law, and S. King

"Seismic Damage Assessment of Structures using Cumulative Damage Model"
Y. Cha

SYSTEMS INTEGRATED APPROACH IN EARTHQUAKE HAZARD MITIGATION

G.C. Lee[1]

Abstract

Experiences of developing a systems integrated approach in earthquake hazard mitigation by the National Center for Earthquake Engineering Research will be presented, and its utilization and significance in natural hazards reduction will be discussed.

Introduction

The National Earthquake Hazards Reduction Program (NEHRP) was established in the United States in 1977, the product of increasing national concerns about seismic safety. NEHRP was introduced as US Congressional legislation to provide a comprehensive, integrated national program to reduce losses of life and property resulting from earthquakes. NEHRP serves as the central focus for leading, coordinating, and conducting earthquake research, hazard mitigation, and disaster preparedness. The Federal Emergency Management Agency (FEMA), the United States Geological Survey (USGS), the National Institute of Standards and Technology (NIST) and the National Science Foundation (NSF) have been charged with carrying out the principal Federal responsibilities outlined within NEHRP. Research conducted under NEHRP has advanced knowledge and characterization of earthquake source mechanisms, estimation of recurrence intervals for historic earthquakes, more thorough understanding of ground motion and ground failure, and the effects of such phenomena on man-made facilities.

The National Center for Earthquake Engineering Research (NCEER) was established in 1986, by NSF through its Earthquake Hazard Mitigation Program (EHM) in the Division of Civil and Mechanical Systems. The EHM program is one of NSF's contributors to NEHRP. NSF uses a number of different mechanisms to conduct earthquake research (Figure 1), including individual solicited and unsolicited proposals, and through coordinated sponsored programs such as TCCMAR (Technical Coordinating Committee for Masonry Research), the Repair and Rehabilitation Research Program for the Seismic Resistance of Structures (RRReP), and PRESSS (Precast Seismic Structural Systems). NCEER offers NSF an additional, unique mechanism to address earthquake engineering problems by using a systematically-integrated and interdisciplinary approach. Teams of as many as 80 researchers from across the country are annually charged with advancing knowledge to minimize earthquake disruption, with particular attention to the lesser-understood Central and Eastern US earthquake risk.

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ORGANIZATION	MECHANISM	GENERAL NEHRP OBJECTIVES
FEMA NIST USGS	• In-house Projects • Sponsored Projects	• Enhance knowledge and understanding of earth-quakes and their effects
NSF	• Individual Sponsored Projects	• Increase availability of information
	• Coordinated Sponsored programs (TCCMAR, RRRep, etc.)	• Target outreach and implementation efforts • Transfer technology
	• NCEER: Systematically integrated interdisciplinary approach	• Conduct problem-focused research
State and Local Government Other Mission Agencies	• In-house Projects • Sponsored Projects	• Encourage community adoption of codes

Fig. 1. NCEER's role in NEHRP

NCEER: A Systematically Integrated Approach to Earthquake Hazards Research

NCEER Mission and Management

Since its inception, NCEER has focused on conducting mission-oriented research to advance the state-of-the-art in earthquake engineering through both curiosity-driven and problem-focused research, with a parallel effort to promote knowledge utilization. The NCEER research program reflects the interdisciplinary nature of disaster research and management, involving multidisciplinary, cross-professional teams to identify knowledge gaps, develop potential solutions, evaluate applications and recommend improvements in seismic methodologies and policies. Experts are drawn from fields of seismology, geotechnical and structural engineering, economics and the social sciences and represent academia, industry and the public sector. Teams contribute to the seismic performance, retrofit and enhanced design of four primary research areas: buildings, nonstructural systems, lifelines, and highways and bridges. These problem focused projects incorporate integral fundamental expert knowledge on ground motion and the seismic hazard, geotechnical influences, risk assessment, social systems and economics. Research on smart materials and intelligent systems for structural vibration control is conducted to develop innovative approaches to seismic design. Interrelationships between fundamental and applied studies link discipline-specific research findings with user need, allow integration of multidisciplinary issues, and broaden the potential applicability of findings (Figure 2).

NSF requires an equal amount of matching funds from external non-federal sources. NCEER's leveraging of NSF research dollars has allowed the development of a cost-effective program supporting fundamental research, with additional funding for knowledge utilization and implementation activities.

In addition to technical expertise, the involvement of multiple institutions provides access to diverse experimental and testing facilities.

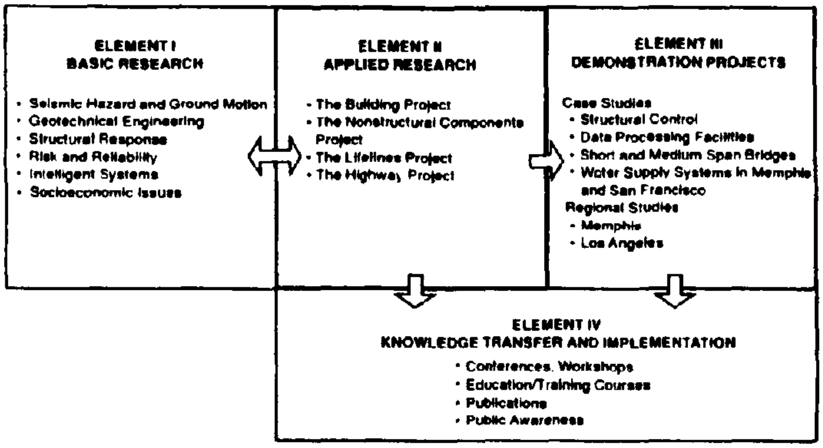

Fig. 2. NCEER Research and Implementation Plan

The Systems Approach to Earthquake Mitigation Research

Earthquake damage to buildings and their nonstructural systems and components can lead to substantial loss of life and economic disruption. The Center's **Building Project** examines two common classes of buildings - unreinforced masonry and lightly reinforced concrete buildings, which are vulnerable to moderate and low shaking. Objectives are to develop rational methods for evaluating these structures and to devise cost-effective methods for adequate seismic resistance. The **Nonstructural Components Project** focuses on minimizing damage to building equipment and contents by improving analysis and design methods and developing innovative support and restraint systems for specialized items (e.g., computer mainframes and hospital equipment).

NCEER's **Lifelines Project** examined the water supply systems in San Francisco and Memphis, and expanded to oil and gas transmission, electric power and telecommunications systems. Geotechnical studies focus on liquefaction and other sources of large ground deformation which seriously impact buried lifelines. Systems modeling, fragility assessments, and socio-economic impacts of service disruption have also been examined.

Transportation systems form the focus of the **Highway Project** where all components of both new and existing highway systems are being studied, including short and long span bridges. Complementary research defines seismic and geotechnical hazards, develops conventional and innovative retrofit methods using smart materials and protective systems, applies risk assessment methodologies to highway networks, defines importance and performance criteria, improves prioritization algorithms, and inventories highway stock. The development of manuals for evaluation and retrofit of existing lifelines and the preparation of performance-based standards for all new lifelines are long term goals.

Case studies and demonstration projects are valuable inventories to the NCEER research program as opportunities to validate assumptions and models and to evaluate practical applications of technical solutions. NCEER also places a strong emphasis on post-earthquake reconnaissance investigations and international cooperative research.

NCEER's program in knowledge and technology transfer consists of traditional activities such as publications, computational tools, workshops, conferences, graduate student education, and professional development courses. Less traditional mechanisms such as the NCEER Information Service, provide relevant reference materials to interested users through a computerized, query-driven bibliographic database. NCEER promotes the involvement of end users on research teams, participation of academic researchers in codes and standards development, and establishing active research partnerships with industry.

The matrix approach to NCEER research facilitates a range of accomplishments from the discovery of basic knowledge to increased public awareness. Its general achievements include research management by interdisciplinary coordinated teams, optimized spending of available research funds and the raising of seismic awareness. More specific technical accomplishments include: enhanced understanding of potential seismic hazards under varying site conditions across the US, the development of design and guideline recommendations for buildings, bridges, lifelines and nonstructural components at State and Federal levels; research, development and implementation of innovative approaches to structural vibration control; and assessments of risk and impact of earthquakes on socioeconomic systems.

As a particular example of the Center's systems approach to research, various tasks within the NCEER Building Project are being integrated into a coordinated research program that will focus on the stock of non-ductile reinforced concrete and unreinforced masonry buildings in the Memphis area. The immediate goal of the project is to assess probable losses and risks for specific building types in this locale. The overall purpose is to develop a unified methodology that can be applied to other building types in various geographical regions.

As noted in Figure 3, the LAMB project builds on existing strengths of the NCEER Building Project and the related Programs of Seismic Hazard, Risk and Socioeconomic Issues. The project will utilize Memphis information obtained from the Socioeconomic Issues Program on basic population-building relationships and building inventories. A cursory identification of typical concrete and masonry buildings will be made to establish which particular building types will be studied in the first phase of the project. Information obtained from past NCEER laboratory experiments on nonlinear behavior and response of lightly reinforced concrete frame members and systems, unreinforced masonry infill panels confined in-frame systems, and unreinforced masonry bearing wall systems will be used to develop and verify nonlinear dynamic response models. The Seismic Hazards Program will develop synthetic ground motions for the Memphis region, which will be necessary input for the nonlinear dynamic analyses. Models will be used by other investigators to determine fragility curves for these three structural types. Loss estimation will then be deduced using these curves, and compared with acceptable values to determine whether rehabilitation measures may be necessary.

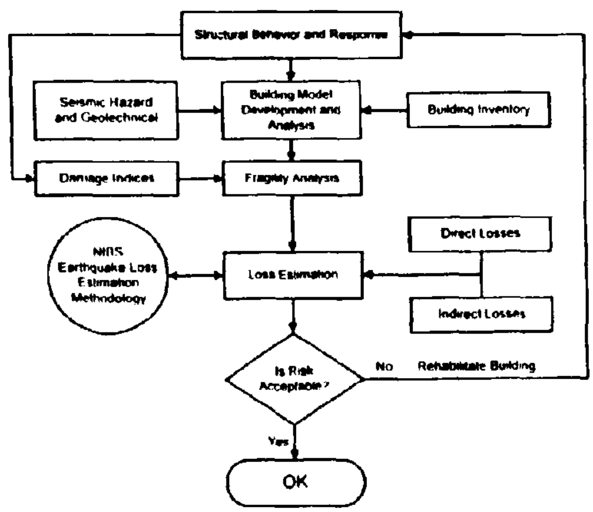

Fig 3. NCEER Integrated Project on Loss Assessment of Memphis Buildings (LAMB)

Conclusions

The preceding pages summarize accomplishments realized by NCEER toward earthquake hazard mitigation through effective use of systematically-integrated approach.

Evaluation of Two Precast Parking Garages Damaged during the 1994 Northridge Earthquake

Sharon L. Wood[1], John F. Stanton[2], and Neil M. Hawkins[3]

Abstract

Structural analyses of two parking garages located less than two miles from the epicenter of the 1994 Northridge earthquake are summarized. The buildings have common structural features: structural walls comprise the lateral-load resisting systems, and precast concrete column, beam, and double tee sections form the gravity-load resisting systems in all three buildings. Portions of both garages collapsed during the earthquake. One possible failure scenario is presented.

Introduction

Structural damage to buildings and bridges in the epicentral region of the 1994 Northridge earthquake was severe. Parking garages were identified as being particularly susceptible to damage during the earthquake (Iverson and Hawkins, 1994). An investigation is underway to evaluate the seismic performance of a number of parking garages and to develop procedures to prevent collapse of garages in future earthquakes. Two precast parking garages, located less than 2 miles from the epicenter of the Northridge earthquake, are discussed in this paper.

Description of the Parking Garages

The parking garages under study have a number of common features. Both structures were less than five years old at the time of the earthquake. Cast-in-place reinforced concrete walls formed the lateral-load resisting system. Precast concrete columns, beams, and double tees comprised the gravity-load framing system. The double tees were overlain by a topping slab which served as a diaphragm and transferred inertial forces generated in the floor system into the structural walls. The garages were supported on pile foundations.

Parking garage A was approximately 400 ft by 220 ft in plan (Fig. 1). The garage consisted of two adjacent and independent structures, physically separated by an expansion joint. Parking was provided on three levels: one on grade and two supported levels. The ramps that connected the parking levels were located near the center of the southern structure. A pedestrian ramp and bridge to the adjacent department store were located at the north end of the garage. Both sections of the garage contained two structural walls in each direction.

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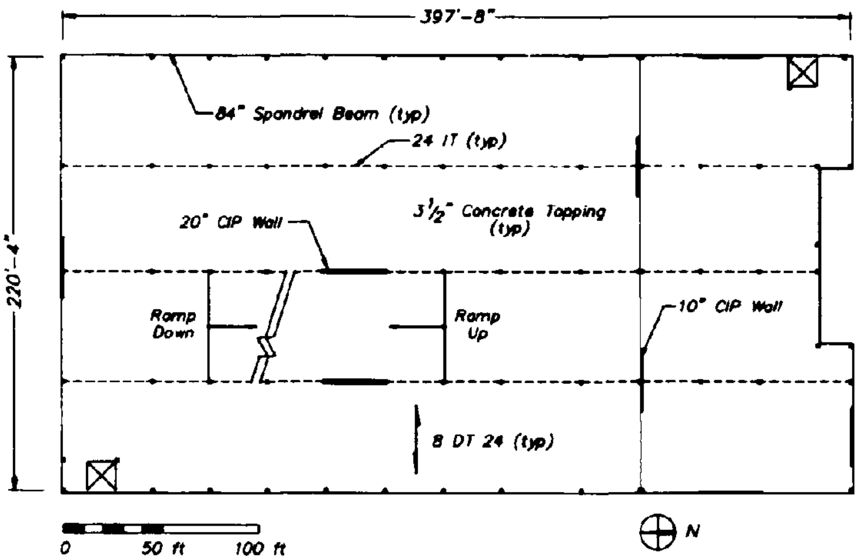

Fig. 1 Second–Floor Plan of Parking Garage A

The cast–in–place topping was at least 3½ in. thick, and was reinforced with welded wire fabric. Additional reinforcement was provided around the walls to permit force transfer and along the edges of the diaphragms to act as a tension chord in the diaphragm. The topping thickness was increased to approximately 5½ in. to accommodate the added reinforcement

The columns were precast in a single two–story section and connected to the foundation with four, 1–in. diameter bolts. The columns supported the inverted tee beams and exterior spandrel beams on corbels. These beams carried the double tees that formed the floor system. Elastomeric pads were placed on the corbels and beneath the stems of the double tees.

Parking garage A sustained extensive damage during the earthquake. Large sections of the garage collapsed. Only those portions of the structure immediately adjacent to the structural walls remained standing.

Garage B was approximately 300 ft by 270 ft in plan (Fig. 2), and also contained three levels of parking. The structural system used in parking garage B was essentially the same as that in garage A. The automobile ramp was located along the east side of the structure, and a pedestrian ramp was located at the south end of the building. The north–east corner of parking garage B collapsed during the earthquake. Both parking garages were demolished shortly after the Northridge event.

Evaluation of Building Response

Analyses completed to date have focused on the transfer of the seismic forces through the structure. The induced inertial forces are intended to be carried by the diaphragm into the walls and to the foundation. Cast–in–place drag struts served as collector elements within the diaphragm for load transfer into the walls. Based on this assumed load path, the lateral strength of the structure may be limited by the shear

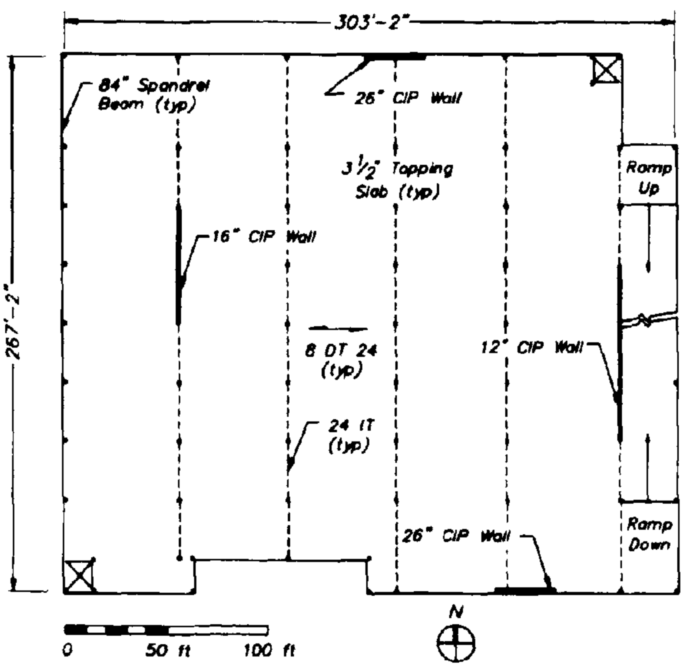

Fig. 2 Second–Floor Plan of Parking Garage B

strength of three components: the strength of the walls, the strength of the connection between the collector elements in the diaphragm and the walls, and the strength of the diaphragms. The strength of each component was evaluated using the applicable provisions of the ACI Building Code (ACI 318–89). Potential failure planes through the diaphragm were assumed to form along column lines and did not cross through any precast or cast–in–place elements.

The forces carried by each structural element were evaluated using a simple representation of each garage in which the inertial force in a given direction was distributed equally to the walls parallel to that direction. All torsional response was ignored. As a basis of comparison, the magnitude of the forces was calculated using the equivalent lateral force procedure defined in the Uniform Building Code (1994) assuming elastic response ($R_w = 1$).

Ratios of the elastic shear demand to the nominal capacity of the structural elements are plotted in Fig. 3. The two sections of Garage A are shown separately in Fig. 3. In all three structures, the elastic shear demand at the base of the walls was approximately twice the nominal capacity. The elastic demand on the connection between the walls and the diaphragm was slightly more than twice the nominal capacity for both sections of Garage A and was approximately four times the nominal capacity for Garage B. The diaphragm appeared to be the critical element in the south section of Garage A and in Garage B. The demand to capacity ratios in the diaphragm are approximately 6 in these structures. Demand to capacity ratios are not calculated for the north section of Garage A because all possible load paths intersected cast–in–place members. The data shown in Fig. 3 indicate that the shear capacity of the diaphragms is the weak link in the precast

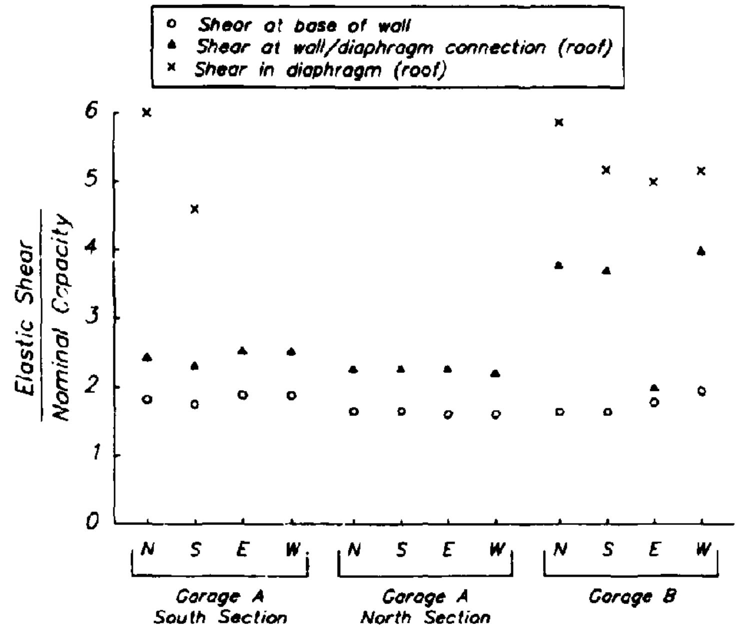

Fig. 3 Elastic Demand to Capacity Ratios

parking garages considered. Seismic forces were not transferred into the structural walls, as intended, because the diaphragm failed could not carry the loads.

Summary

The likely cause of collapse of two precast parking garages during the 1994 Northridge earthquake is shear failure of the diaphragm. Potential failure paths within the diaphragm were crossed only by welded wire fabric, the area of which was determined based on shrinkage and temperature requirements, and by chord reinforcement near the perimeter of the structures.

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Acknowledgment

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Prioritization for Rehabilitation of Buried Lifelines

Leon R.L. Wang[1]

Abstract

Seismic rehabilitation or retrofit of existing lifelines is a cost-effective way to prevent damage caused by future earthquakes. In general, it is very difficult, if not impossible, to rehabilitate all buried pipelines at the same time because of limited funds and time available. The purpose of this study is to establish a priority strategy for rehabilitation of buried pipelines by taking several important factors such as pipeline damage probability, rehabilitation cost, rehabilitation time (e.g. km/day), pipeline importance and total funds available, etc. into account.

Introduction

Buried water, sewer and gas pipelines have been damaged heavily by recent earthquakes, including the Northridge Earthquake of January 17, 1994 in the U. S. and the Hyogo-ken Nambu earthquake of January 17, 1995 in Japan. Because of the importance of buried lifelines to the health, supply and safety of the populace, mitigation of earthquake damage of buried lifelines becomes a worldwide concern. Many existing water, sewer and gas pipelines have been built several decades ago without seismic considerations. They are unsafe even under moderate earthquakes. The improvement of earthquake resistance of buried pipelines is urgently needed in earthquake prone areas. With the proper rehabilitation, earthquake damage can be mitigated.

Pipeline rehabilitation is costly. It is impossible to rehabilitate all pipelines in a system in a short time for insufficient manpower, equipment, materials and funds. The paper provides a scheme/guideline showing the priority for rehabilitation of pipelines of any existing system.

Rehabilitation Scheme

A water, sewer or gas pipeline system in general can be divided into mains and branches. Pipelines in the main system are much more important than that in the branch system. Before starting the rehabilitation of the branch system, pipelines in the main system must be rehabilitated completely. The rehabilitation of the main system is performed on pipeline by pipeline basis, while branch system on area by area. Once a pipeline or an area has been rehabilitated, the performance of the retrofitted pipeline or area should be monitored to evaluate the effectiveness of the actions taken.

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Database

To accomplish the rehabilitation task effectively, a comprehensive database with information on various aspects of the pipeline system should be available. The following are some important parameters for a typical pipeline segment: 1) pipe diameter; 2) pipe material; 3) joint type; 4) segment length; 5) soil conditions; 6) buried depth; 7) buried age; 8) number of customers served; 9) rehabilitation cost; 10) rehabilitation time (km/day); 11) rehabilitation methods; 12) pipeline usage (emergency, hospital, school, factory or resident); and 13) expected earthquake intensity (MMI-Modified Mercalli Intensity).

Pipeline Damage Probability

Previous studies[1 to 3] revealed that pipeline damages are strongly correlated with the following factors: 1) earthquake intensity; 2) soil conditions; 3) pipe diameter; 4) pipe type (combination of pipe material and joint type); 5) buried age; 6) buried depth, etc..

It is assumed that these factors are statistically independent[4] and pipeline damage follows **Poisson's Law**[5] along the pipe length. According to the first assumption, the damage probability model can be developed as:

$$P_j = 1 - [1 - P_j^d(i)] \cdot [1 - P_j^s(k)] \cdot [1 - P_j^m(y)] \cdot [1 - P_j^e(x)] \cdot [1 - P_j^a(l)] \cdot [1 - P_j^b(e)] \quad (1)$$

where P_j is the damage probability of pipeline j. P_j^d, P_j^s, P_j^m, P_j^e, P_j^a, and P_j^b are damage probabilities with respect to pipe diameter, soil condition, pipe type, earthquake intensity, buried age and buried depth, respectively. i, k, y, x, l, and e are indices for parameter domains of pipe diameter, soil condition, pipe type, earthquake intensity, buried age and buried depth, etc..

According to the second assumption that pipeline damage follows Poisson's Law along the pipe length, the damage probability with respect to each variable can be expressed as:

$$P_j^c(z) = \frac{[R_j(z)\ L_j]^c}{c!} e^{-R_j(z)\ L_j} \quad (2)$$

where z is an index correlated to i, k, x, y, i or e. c is a random number. When c equals zero, P_j^0 means reliability. L_j is the pipeline length (km). R_j is the damage rate which is determined by the analysis of damaged data collected from past earthquakes worldwide. Eq. 2 can also be expressed as:

$$P_j(z) = 1.0 - e^{-R_j(z)\ L_j} \quad (3)$$

Prioritization for Rehabilitation of Pipelines

Node Weight: Node weight, which is denoted by NW_i, represents the importance and the reliability of the node i. It is defined as

$$NW_i = \frac{1}{2} \sum_{j=1}^{N_i} \alpha_j \, P_j \tag{4}$$

where N_i is the number of pipelines directly connected to node i. α_j is the importance factor of pipe j. The node which has the largest node weight in the system is the most important node.

Determination of Importance Factor: The importance factor α_j of pipe j is determined by

$$\alpha_j = \alpha_j^1 \, \alpha_j^2 \, \alpha_j^3 \tag{5}$$

where α_j^1 is the pipeline usage factor; α_j^2, the system factor; and α_j^3, the population factor.

Link Weight: Link weight which is denoted by LW_j represents the importance of pipe j. It is defined as:

$$LW_j = NW_i + NW_k \tag{8}$$

where i and k are two nodes of pipe j. The more link weight the pipeline has, the more important the pipeline is.

Rehabilitation Time of each Pipeline: Rehabilitation time T_j of a pipeline j is defined as

$$T_j = \frac{L_j}{\gamma_j} \tag{9}$$

where γ_j is the rehabilitation time factor (km/day) of pipe j.

Link Weight Efficiency: Link weight efficiency denoted by LWE_j represents the link weight per unit rehabilitation time, which is defined as:

$$LWE_j = \frac{LW_j}{T_j} \tag{10}$$

Prioritization Scheme for Main System

To decide the priority for a pipeline rehabilitation in a system with the objective of minimizing the rehabilitation time, there are four variables to be considered: 1) total funds available; 2) damage probability of each pipeline; 3) rehabilitation time of each pipeline; and 4) rehabilitation cost of each pipeline. The procedures of prioritization for pipeline rehabilitation include the following steps:

Step one is to make a list of all pipelines in the main system according to damage probabilities from the largest one to the smallest one.

Step two is to sum the rehabilitation costs required according to the list in step one to be equal or within the total funds available.

Step three is to separate the main system into M sub-systems by pipelines which have damage probability larger than or equal to P^c.

Step four is to establish the optimization objective. The objective is to minimize the total rehabilitation time T of the system.

Step five is to search for a **Generalized Source Expansion Tree** (GSET). The details of GSET can be found in Ref. 4 and will not be repeated herein.

Step six is to calculate sub-system weight.

Step seven is to calculate generalized link weight. The generalized link weight GLW_j of pipeline j is defined as the summation of all sub-system weights associated with pipelines served by the pipeline j

Step eight is to calculate link weight efficiency of other pipelines in the system which have damage probability larger than or equal to P^c.

Step nine is to establish the priorities for pipeline rehabilitation. No.1 in the list is the one on which the rehabilitation work will be carried out first. No.2 is the second and so on.

Prioritization Scheme for Branch System

According to the rehabilitation scheme, the rehabilitation work of branch system is performed on area by area basis as mentioned earlier. The procedures of pipeline prioritization for rehabilitation in branch system is as follows:

Step one and step two are the same above by changing main system to branch system.

Step three is to calculate average link weight efficiency.

Step four is to make a list of all areas according to the average link weight efficiencies from the largest one to the smallest one. Number one is the area that will be rehabilitated first.

Examples

Examples to test the developed prioritization scheme using 1971 San Fernando earthquake data were studied and discussed in Ref. 4. Due to the limited space, readers are referred to Ref. 4 for details.

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ON THE SCALE ANALYSIS OF ELECTRORHEOLOGICAL FLUID DAMPERS

F. Gordaninejad[1] and R. Bindu[2]

Abstract

A scale analysis for the damping coefficient of electrorheological fluid (ERF) dampers is presented. Theoretical as well as experimental studies are conducted to investigate the effects of length and the applied voltage on the damping coefficient of ERF dampers. It is demonstrated that by employing the results obtained from smaller size prototypes, the energy absorbing capacity of larger dampers can be accurately estimated.

Introduction

Large civil structures such as bridges undergo predictable (e.g., wind excitations and traffic loads) as well as unpredictable (e.g., severe storms and earthquakes) vibrations. Predictable motions can be suppressed by using passive damping devices such as friction or viscoelastic dampers and elastomeric bearings. Control of unpredictable structural vibration is receiving increasing attention. One type of device that can potentially be used for unpredictable vibration control of structures is an ERF damper.

Performance of cylindrical ERF dampers to suppress vibration was addressed by (Ehrgott and Masri, 1994). Bang-bang, and linear vibration control of cylindrical ERF dampers, and the effects of number of electrodes, length, electrode gaps were investigated in (Gordaninejad, *et al.* 1993, Gordaninejad, *et al.* 1994a and Bindu, 1995). Performance of a hybrid ER/viscous oil dampers in controlling vibration, and comparisons with regular ERF dampers was studied by (Gordaninejad, *et al.* 1994b). Recently, the behavior of a small and a large ERF damper was addressed by (Gavin, 1995). In this paper selected results from a scale factor study on cylindrical ERF dampers are presented.

Experimental Study

The behavior of single-electrode dampers with different lengths was experimentally studied under a forced vibration generated. A spring-mass-damper system was designed, built and set up as illustrated in Figure 1. A schematic of the cross-section of a single-electrode damper is shown in Figure 2. An aluminum rod was placed at the center of the damper to transmit the force directly to the moving cylin-

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drical inner electrode inside the Plexiglas casing of the damper. The aluminum rod, which was rigidly fixed to the cylindrical inner electrode was connected to the ground. The rod was constrained by a bearing at the top and a pair of nylon guides located below the inner, moving electrode and can move only in the axial direction. The voltage was applied to the stationary outer electrode which was fixed tightly inside the cylindrical Plexiglas casing of the damper. The damper is filled with Versa Flo ER-100. The fluid between the electrodes is electrically stressed by the voltage applied to the damper using a 408B Fluke High Voltage Power Supply. The vibration was measured by using two accelerometers. The signals obtained from the accelerometers were amplified and passed through a band pass filter. The filtered signals were fed into a Keithly Metrabyte DAS 1601 data acquisition board.

The variable damping coefficient of ERF dampers is a function of the following variables:

$$C = C(d,D,\omega,L,X,E) = k\pi DLdXE^{n} \tag{1}$$

where C is the damping coefficient, d is the gap between the electrodes, ω is the frequency, L is the damper length, X is the displacement amplitude of the piston, E is the applied voltage, k is the scale factor coefficient and n ranges from 1.75 to 2. In this study the gap between the electrodes, the diameter of the cylinder and the frequency of the motion were fixed at, 0.10 in, 1.17 in and 10Hz, respectively. This was done to focus on the effect of the length of the damper on the damping coefficient.

Logarithmic Decrement method was employed to obtain the damping coefficient of the dampers with different lengths. The ratio of the length of the largest to the smallest damper was 5.

Theoretical Analysis

For steady-state forced vibration, the loss of energy is balanced by the energy due to excitation. In the case of cyclic oscillation the energy loss per cycle, W_d, due to damping force, F_d (which is linearly proportional to the velocity), is:

$$W_d = \pi C \omega X^2 \tag{2}$$

Considering both Eqs. (1) and (2), one has

$$C = k\frac{\pi DLdE^{n}}{\omega X} \tag{3}$$

Since the gap between the electrodes, the diameter of the cylinder and the frequency of the motion are fixed, k can be of the following form:

$$k = \bar{k}L^{\alpha}E^{\beta}C^{\gamma} \tag{4}$$

where $\bar{k}, \alpha, \beta$, and γ are constants that will be determined experimentally.

Results and Discussion

Experiments were conducted on six single-electrode dampers ranging from 0.6in to 3.0in. The experiments were performed in three different days to evaluate the consistency of the results. In each day five experiments were conducted on each damper. A maximum relative error of 3% was observed.

The damping coefficient of each prototype was determined experimentally by using the Logarithmic Decrement method. As shown in Figure 3, the damping coefficient increases with the length of the piston for different electric field strength.

The experimental results of the two smaller dampers (0.6 in and 0.8 in) at 4Kv and 5Kv were used to determine $\bar{k}, \alpha, \beta$ and γ. The following results were obtained:

$$\bar{k} = 6901; \alpha = -0.79; \beta = -1.75; \gamma = 1.42$$

These values were substituted in Eqs. (4) and (3) and the results are presented in Figure 4. As can be seen, the damping coefficient increases by increasing both the length and the electric field strength. Comparison of Figures 3 and 4 demonstrates 1%-9% error between the experimental and predicted results for different size of the dampers.

Summary and Conclusion

Theoretical and experimental scale studies was performed to determine the damping coefficient of large-scale ERF dampers. The focus of the research was on single-electrode cylindrical dampers.

It was demonstrated that the damping coefficient of the larger dampers can be predicted by using the data obtained from small prototypes with a relatively low percentage of error.

Acknowledgements

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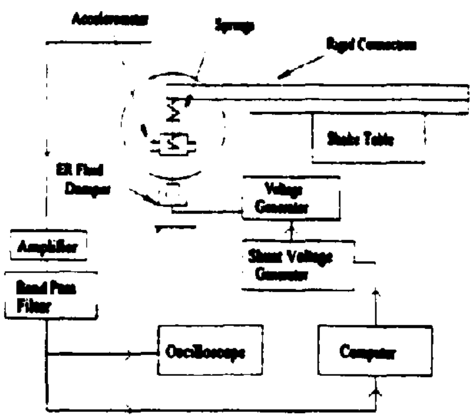

Figure 1. Schematic of the experimental set up.

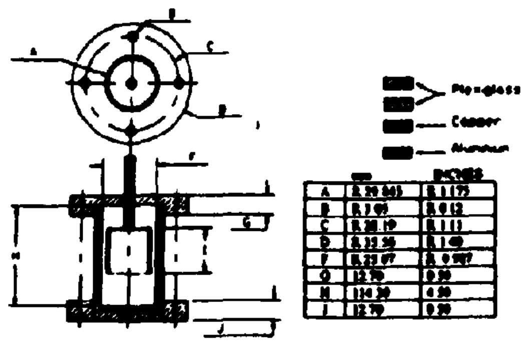

Figure 2. Schematic of the single-electrode ERF damper.

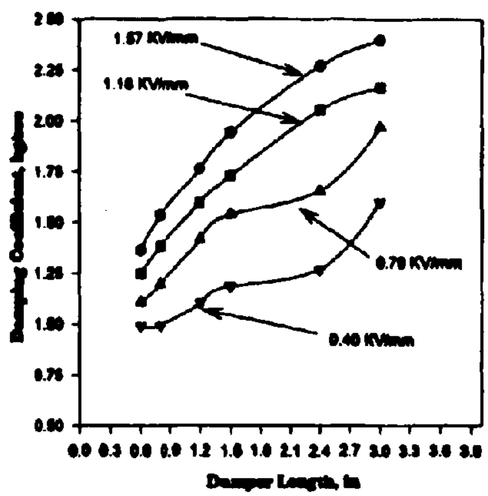

Figure 3. The experimental results for damping coefficient of a ERF damper under a forced vibration.

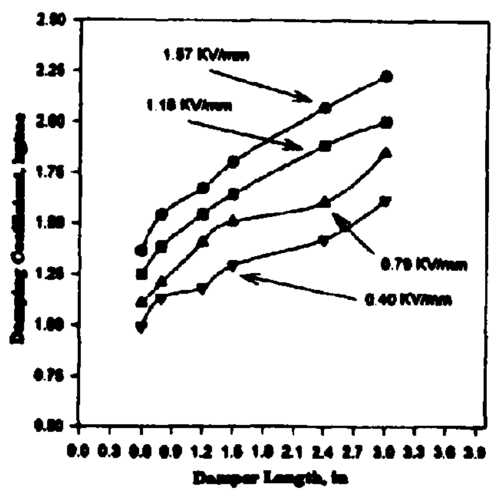

Figure 4. Theoretical prediction for damping coefficient of a ERF damper under a forced vibration.

COMPUTATIONAL ISSUES ASSOCIATED WITH THE ANALYSIS OF ACTIVELY CONTROLLED STRUCTURES

H. Allison Smith[1] and Armin G. Schemmann[2]

Abstract

Despite the extensive research focused on active structural control in the past few years, the majority of structural models utilized in this research are highly simplified representations of the vibrating system. Three research investigations, which focus on the re-formulation of control algorithms to consider some of the complexities arising in seismically excited civil structures, are summarized here. The objectives of these investigations are to assess the effectiveness of vibration control in the presence of: (1) soil-structure interaction, (2) time-varying changes in structural stiffness, and (3) structural nonlinearities encountered in cable-stayed bridges.

Soil-Structure Interaction

It has been generally recognized that soil-structure interaction (SSI) can drastically affect the responses of structures subjected to earthquake excitations. Therefore, the inclusion of SSI effects is particularly important in the analysis of structures located in active seismic zones. However, in the majority of the research devoted to active structural control, the structural models utilized are highly simplified fixed-base systems. Only recently has soil-structure interaction been included in the control problem.

To consider SSI effects in structural control, the analysis is extended from the structural model to include the total structure-foundation-soil system, which involves an infinite half-space. The substructure method is commonly adopted in the SSI analysis where a mixed boundary-value problem in elastodynamics is first solved to obtain the foundation impedances of soils and then utilized to analyze the structure-foundation model. Using this method to include SSI effects in a control algorithm, the problem formulation is similar to that of the control/structure interaction (CSI) problem.

The major difficulty in including soil-structure interaction in optimal structural control comes from the fact that the accurate analysis of an SSI system usually is formulated in the frequency domain due to the frequency-dependent foundation impedances, whereas conventional optimal control problem is performed in the time domain. Using an equivalent fixed-base structural model to represent the whole SSI system, an effective approach has been developed to include SSI effects in the optimal control algorithms. Smith, et al. [1994] studied the formulation of the externally controlled system where the actuator exerts control forces only to the structure and is not connected to the foundation. The SSI/control equations were formulated with the approximation via replacement oscillator to determine the optimal control gains. A second study, Wu and Smith [1995], presented a formulation for the internally controlled system where the exerted control forces are internally reacted between the superstructure and foundation. An iterative

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algorithm was employed to determine the optimal control gains, which are interrelated to the SSI factors in this case. With this proposed method, the control scheme can be implemented in the framework of a conventional optimal control algorithm used for a fixed-base structure such that computational complexity is significantly reduced.

Smith and Wu [1995] extended the above research to consider general multi-degree-of-freedom (MDOF) structural models. The SSI transfer functions for ground motion and control force in the physical space are derived [Smith and Wu, 1994] followed by a methodology for using system identification techniques to find an equivalent fixed-base model of an MDOF SSI system. An iterative technique is applied to connect all the pieces together for the determination of optimal control gains. Numerical examples are presented to show that the control algorithm considering SSI effects is more effective than the corresponding control algorithm assuming a fixed-base system model. In addition, the advantage of applying this methodology is observed to be more prominent in the cases where the SSI effects are more significant.

Time-Varying Parametric Uncertainties and Actuator Saturation

The primary goals when applying optimal control theory to civil structures is the maintenance of stability and the achievement of specific performance criteria, including control efficiency, in the presence of random disturbances such as wind and earthquake loadings. Two important issues in achieving these goals is the maintenance of stability in the presence of non-linear actuator saturation effects and achievement of the performance criteria considering the unknown, time varying, structural parametric uncertainties. Most importantly both of these issues must be addressed concomitantly within the same control design.

The majority of civil engineering control applications employ linear quadratic regulator (LQR) methods to obtain an optimal controller. Because environmental loads are random in both time and space, analytical solutions to the LQR optimal control equations are not available. Problems typically are reduced to obtaining the analytical solution of an algebraic Riccati equation (ARE), obtained by ignoring the external excitation terms and assuming that the structure can be modeled as a linear time invariant (LTI) system. This LTI model implicitly assumes that the structural properties are not only constant, but, exactly known.

Actuator saturation occurs when the actuator is given a demand requiring an output greater than the designed peak output. The result is a clipped control input that is hard-limited by the magnitude of the actuators' peak output. Failure to account for this nonlinear effect can render the closed loop structure unstable as the optimal control input is changed arbitrarily to a value which guarantees neither performance or stability.

In considering the complications associated with structural parametric uncertainty and actuator saturation, this investigation re-formulates the H-infinity control algorithm specifically for application to civil structures in seismic zones. H-infinity control is a relatively new area of research in control theory. This approach minimizes the worst case response of a system over an entire family of inputs. This task is accomplished by minimizing the infinity norm of the transfer function from the disturbance inputs to the regulated outputs. Similar to the LQR algorithms, the H-infinity approach performs this minimization on a LTI state space model.

Chase and Smith [1995a,b] present a two-part study devoted to the theory and application of H-infinity state feedback optimal control to civil structures in the presence of actuator limitations and time varying parametric uncertainties. Robust H-infinity state feedback controllers are developed which achieve the desired H-infinity norm bound while accounting for pre-specified bounds on the time varying parametric uncertainties [Smith and Chase, 1994]. Stability of these controllers in the presence of nonlinear actuator saturation can be proven through the construction of a Lyapunov function for the saturated control system using a nonlinear state space model and new mathematical programming techniques. Lack of a feasible solution to these optimization problems serves as proof that stability cannot be guaranteed for these controllers with this type of Lyapunov function.

Part 1 [Chase and Smith, 1995a] represents the theoretical developments associated with re-formulation of the H-infinity control algorithm for civil structures in the presence of time varying parametric uncertainties and actuator saturation. Application of this algorithm to a full scale actively controlled structure is discussed in Part 2 [Chase and Smith, 1995b]. Specifically, a comparison is made between the LQR and H-infinity algorithms based on application of each algorithm to an actively controlled 33-story structure, the Riverside Sumida Central Tower. This structure, located in Tokyo, is equipped with two rooftop active mass dampers (AMD) capable of increasing damping and controlling vibratory behavior in four vibration modes. Comparisons of the control algorithms are performed on shear building representations (five degrees-of-freedom) of the structure. Robust H-infinity controllers, which account for model uncertainty and actuator saturation, are designed for these AMD's. The effectiveness of these H-infinity controllers is assessed in comparison to a LQR controller employed in the actual structure. Simulations are performed using records from the Tabas, Iran earthquake of 1978, Imperial Valley earthquake of 1979 (Holtville record), and the Northridge earthquake of 1994 (Sylmar record).

Cable-Stayed Bridges Subject to Multi-Support Excitations

The purpose of this research project is to develop and assess the effectiveness of low-power actuators for vibration control of cable-stayed bridges subjected to support excitations. As part of a joint, inter-institutional, and interdisciplinary project with Professor Rahmat Shoureshi of the Department of Mechanical Engineering, Colorado School of Mines, a goal of this work is to develop and calibrate an analytical model of the actively controlled experimental bridge model constructed by Professor Shoureshi and his students. Although a substantial amount of research on control of cable-stayed bridges has been done in Japan, the majority of the research in the U.S. has focused on more conventional civil structures. Thus, work is needed to better understand how the complexities associated with cable-stayed bridges, such as multi-support excitation and geometric nonlinearities, affect the overall effectiveness of active control schemes.

Due to their complex geometry, cable-stayed bridges behave nonlinearly. Specifically, this nonlinearity is due to: (1) the axial force deformation relationship of the inclined cables caused by the deadweight induced sag; (2) the axial force-bending moment interaction of the towers and longitudinal girders (i.e., beam-column interaction); and (3) the geometry change due to large deformations. Material nonlinearities are not considered. A nonlinear static analysis is necessary to obtain the stiffness matrix associated with the deadload deformed shape. However, prior research has shown that a linear time history dynamic analysis is adequate once the stiffness matrix is obtained from the nonlinear static analysis. Other complexities associated with the cable-stayed bridge include: (1) multiple support excitation, caused by the exposure of the bridge towers (which are spaced far apart) to the spatially random seismic excitation; and (2) the participation of highly coupled, high order, three-dimensional vibration modes in the overall dynamic response. In dynamic analysis of cable-stayed bridges, it is not unusual for more than forty modes to participate in the maximum response, unlike more conventional civil structures where only the lowest few modes are of concern. Cable-stayed bridges are considered to be lightly damped with damping ratios for the bridge estimated to be 5% of critical and 0.5 to 4% of critical for the cables.

Since cable-stayed bridges have very low structural damping, it is necessary to provide the structure with economical means to dissipate seismic energy. The addition of hybrid actuation systems, located at the tower foundation and at the connections between the bridge deck and tower, would economically increase the resistance of these bridges to large earthquakes. Specifically, the bridge deck-tower connection plays an important role in the dynamic behavior of the bridge. If the deck is isolated completely from the towers, the seismically induced forces will be kept to a minimum; however, the bridge may be too flexible under service loads. In contrast, if the deck is rigidly connected to the towers, the deck movements will be kept at a minimum under service loads, but the bridge will attract much higher seismic forces during an earthquake. To solve this problem, the adaptive fluid mount is proposed for use as an passive/adaptive isolation device [Shoureshi, et al., 1995]. Accurate representation of these devices in the analytical model of the bridge is a topic of research during this second year of the investigation. In addition, the use of smart elements (i.e., structural members constructed of smart materials) to control the coupling of the higher-order vibration modes will be investigated. Reduction of these coupling effects should decrease overall structural response.

Acknowledgments

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Intelligent Control of Structures

Rahmat A. Shoureshi,[1] Mark Bell

Abstract

Advances in the areas of materials, electronics, artificial intelligence, and automatic control have introduced new horizons for the design of smart structures. Smart structures would be capable of handling unexpected natural events, e.g. earthquakes and typhoons, and can change, in real time, their mechanical characteristics to compensate for major load variations. this paper presents some ideas on intelligent control for structures by using neural networks, a hybrid actuation mechanism, and a predictive control system.

Introduction

Infrastructure systems consist of the constructed physical facilities which support the activities of the whole society. These structures and systems have evolved in a piecemeal fashion with more emphasis on lower construction cost than the resulting quality. Structures undergo a variety of loading and disturbances with both periodic and broad band frequencies. From a control point of view, structures are plants that require control systems with a high degree of disturbance rejection and robustness. However, due to insufficient damping, the control system has to provide an optimal energy to the system without causing instability. In general, the following key elements have to be considered in the design of a structural control system:

i - Control architecture
ii - Actuation mechanism
iii - Optimal location of sensors and actuators

In this paper the above three issues are considered for the case of a cable-stayed bridge.

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Control Architecture

Prior studies in the general area of intelligent control systems (ICS) have indicated that a combined feedforward/feedback control with an optimal predictor provides an effective mechanism for robustness and uncertainty compensation (Shoureshi et.al 1995). A neural network is used to provide estimates of disturbances and thus formulate the feedforward action. A minimum variance (MV) feedback controller is designed to reject harmonic disturbances, while the feedforward signal covers broad band excitations. In order to reduce the on-line computational complexity, an iterative off-line optimization is employed to derive the optimal settings for the minimum variance controller. This optimization utilizes the 3D nonlinear finite element model that Professor A. Smith, Co-PI of this research, is developing at Stanford. The neural-based estimation provides an on-line adaptation. The overall intelligent controller is applied to a prototype stayed bridge, as shown in Figure 1.

Neural-Based Estimation

Neural networks are used for disturbance estimation, and structural model update. An analytical model of the vertical vibration of the cable-stayed bridge with fixed end-piers is developed for estimation of the bridge response when the first three natural frequencies are considered. This model is then used to generate training and testing data for a neural network that operates as a multiple time step-ahead predictor. It was determined a priori that the network would be trained using time series data with frequencies below 80 Hz. This is because no modes in the model or the experimental prototype contribute significantly to the response above 80 Hz threshold. The training data is developed using Gaussian white noise filtered by a tenth order Butterworth low-pass filter with a cutoff frequency of 80 Hz as the input signal. The output signal is determined by simulation of the analytical model. The total number of layers as well as the number of neurons per layer was determined by simulation based on total error in the training set and generalizability to testing sets of data. One structure that worked extremely well is a two-layer neural network that has seven input neurons, four hidden layer neurons and two output neurons, as shown in Figure 2. The inputs to this network are the five previous outputs (y_n - y_{n-4}) and the two previous inputs (u_n - u_{n-4}) and the outputs of this network are the estimates of the next two outputs (y_{n+1} - y_{n+2}). This network approximates the training vectors very well and generalizes to the cases of impulse response, random vibration response, and the signal frequency sine wave test with additive noise. Results of an earthquake simulation and the networks response are shown in Figures 3, 4. It is important to note that the two curves are almost identical.

Actuator Placement

Placement of the actuators at the base of the bridge structure is based upon recent work by Al-Sulaiman and Zaman. In their paper a criterion for optimal actuator placement is developed for -Sulaiman and Zaman. They have developed a criterion for optimal actuator placement for suspension models. This criterion states that the optimal actuator location is the location closest to the source of the disturbance, i.e. co-location of the actuator with the disturbance for maximum disturbance rejection capabilities. This criterion was developed based on the following feedback control application. Constant system eigenvalues for each set of possible actuator locations were maintained using state variable feedback as a basis for comparison. A priori knowledge of the disturbance location is also assumed. This assumption is non-restrictive in suspension models as the disturbance is generated by road roughness. Additionally, in base isolation systems used for earthquake engineering applications, the disturbance location is also known.

In the case of a bridge structure or a building subjected to ground motion, co-location of actuators and disturbance can be argued from energy standpoint, although no formal proof is given. Consider the goal of minimizing structural energy subject to strong ground motion. If the base of the structure is permitted to move with the ground motion, energy increase will occur due to total structure motion (kinetic energy) and relative structure motion (strain energy). If an actuator device is used that removes energy from a vibrating structure, like a tuned mass damper, the total energy content in the structure must initially increase before it can be removed. Thus, if an actuator is used to remove the energy before it reaches the structure, then total energy is reduced if the actuator has a low power consumption.

Results

In order to verify performance of the proposed control system, simulation results have been developed for the prototype bridge. Figure 5 shows the comparison of an open loop and a closed response of the structure. It is interesting to note that during the initial stage, due to estimation and identification, performance of the controller is not impressive, but after the adaptation, significant reduction of vibration is achieved.

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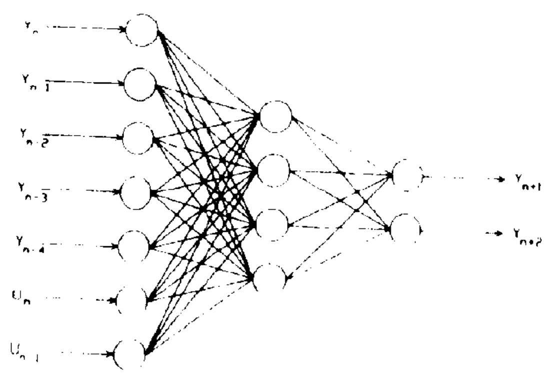
Figure 1: Experimental Cable-Stayed Bridge



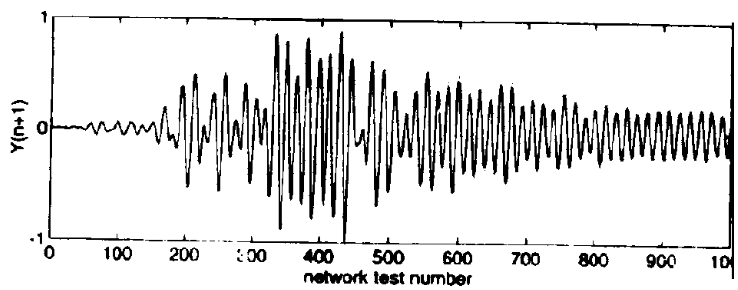


Figure 2: Applied Neural-Network Configuration

Figure 3: Neural Network Response Based on Earthquake Excitation

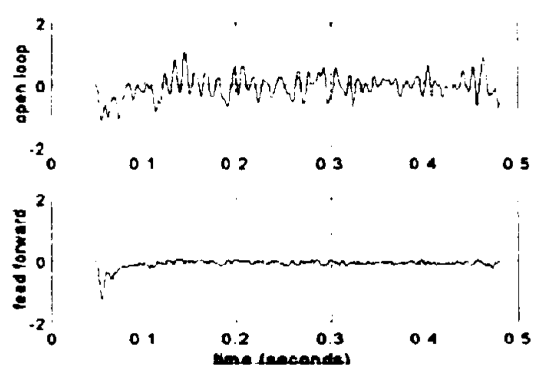


Figure 4: Simulation Results of Controller Performance

CONSEQUENCES OF DAMAGE TO BRIDGES IN THE NORTHRIDGE EARTHQUAKE

Anne Kiremidjian[1], Nesrin Basoz[2], Kincho Law[3] and Stephanie King[4]

Abstract

The January 17, 1994 earthquake in Northridge, CA caused significant damage to the transportation system within the greater Los Angeles area. Several bridges and roadways were damaged and are still under reconstruction. In this project the effect of bridge failures is investigated with the objective of evaluating the performance of the transportation system under two conditions (a) emergency response and long-term economic recovery. The results of the study will provide valuable information for emergency preparedness, retrofit decision strategies and disaster mitigation planning.

Introduction

Emergency response and economic recovery of a region following a major earthquake is greatly dependent of the performance of several systems of our infrastructure. Transportation systems appear to be particularly vulnerable with bridges often failing and causing major disruption of the systems operation. For emergency response purposes, identification of the shortest path for certain origins and destinations are of critical importance immediately following the earthquake. In the recent earthquake in Kobe, Japan firefighting efforts were seriously limited due to the failed transportation system. The January 17, 1995 earthquake in Northridge, CA has provided important data to evaluate the performance of the system immediately following the earthquake.

The objective of the current project is to study the impact of the failures of bridges and roadways within the Northridge area on the transportation system. In particular, the effect on emergency response and long term economic recovery is being investigated. The study has several tasks that will be accomplished over a period of one year. In this paper, we describe the approach and the progress to date on the project.

Methodology for Transportation System Evaluation

The methodology for evaluating the consequences of failure of transportation systems following an earthquake is considered from two points of view. The first relates to the period immediately following the earthquake when the transportation system is required to be functional for emergency response purposes. The second is the long term economic recovery time frame when traffic delays in addition to unavailability of particular links needs to be considered. The initial focus is on the emergency response of the system.

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Transportation system evaluation for emergency purposes can be performed, in general, with the objectives (a) to identify critical links within the system that are particularly vulnerable and should be considered as high priority in retrofit decisions, and (b) to determine the optimal alternate routes when critical routes become unavailable following an earthquake. In order to achieve these goals, a general framework has been developed. The main components of the methodology are

- the evaluation of the site hazards throughout the network system,
- the evaluation of the performance of each component of the transportation system when subjected to the various hazards,
- modeling of the transportation system as a network flow system,
- evaluation of critical paths using various algorithms (e.g., minimum path maximum flow),
- estimation of time delays, direct and indirect economic losses, and casualties due to direct damage and time delays in emergency response from the failure of particular components of the system, and
- development of a decision support system for bridge retrofit prioritization and alternate routing of emergency vehicles from origin to destination.

Several components of this methodology were developed under and earlier study by the authors where the primary objective is to prioritize bridges for retrofit purposes (Basoz and Kiremidjian, 1995). In the current study, the objective is not only to identify critical paths and utilize that information for bridge prioritization purposes, but also to evaluate potential losses and casualties from the failure of particular bridges.

Various site hazard estimation methods have been developed over the past twenty years. These, in general can be divided into probabilistic and scenario hazard analyses. Both approaches are being considered in the current study. A considerable improvement in the computational capabilities of hazard models has been achieved most recently through the implementation of geographic information systems (GIS). These systems enable efficient evaluation and manipulation of spatially distributed information such as the data generated through seismic hazard analysis over a large geographic area that defines the domain of the transportation system. Information on the physical and engineering characteristics of bridges and roadways comprising the transportation system can also be stored within the GIS. The performance of individual bridges and roadways can then be evaluated by implementing a damage model. Critical paths within the system can be identified through network analysis for all origin destination pairs. Most frequently the network analysis is performed external to the GIS, however, results from this analysis can be easily imported within the system for display and further analysis. Figure 1 shows schematically the steps in the model.

Several critical issues remain unresolved within this model. In particular, damage evaluation methods are either nonexistent or too rudimentary to enable reliable forecasts of potential losses. In addition, definition of the transportation system poses particular difficulty since several systems may be part of the overall transportation network of a region comprised of the highway and roadway network, underground subway system, a railway system and on occasion a waterway system. For example, in the 1989 Loma Prieta earthquake, the ferry system was used when a section of the San Francisco-Oakland Bay Bridge collapsed breaking a critical link between two major areas of the region. BART increased the number of trains to further aid in the additional volume of people that needed to travel from the two origins and destination. Although both of these examples, pertain to the long-term economic recovery problem, they provide important insight to the emergency response model considered in this project.

The Northridge earthquake provides a unique opportunity to study the implications of the model for transportation systems developed in this study. The approach considered in this project is as follows:

• All bridges within the greater Northridge area have been identified and a database has been created within the dBase database management system containing information on the physical characteristics of each bridge. The original bridge data were provided by CALTRANS.

•Bridge locations have been matched to the major highway transportation system in the greater Northridge area within the geographic information system ARC/INFO™.

•Ground motion data and information on local soil conditions at all bridge sites and the greater Northridge area have been accumulated. Ground motions from the main shock of the January 17, 1994 earthquake are not available for all the bridge sites since these were not instrumented, however, at several sites, USGS placed GEOS instruments to collect aftershock motions. These aftershock motions will be used to estimate the ground motions of the main shock at all bridge sites.

•Soil parameter data have been obtained at bore holes locations throughout the Los Angeles Basin containing information on shear wave velocities, soil densities and standard penetration test blowcount with depth. These will be used to generate shear wave contours of the region and will be used for the generation of regional ground motion maps.

•Bridge damage data are being accumulated from the Northridge and Loma Prieta earthquake. These data will be used to develop damage states. Difficulties exist with the interpretation of these data and their correlation to specific ground motion levels. Various definition of damage have been developed in previous studies and these have been extensively reviewed in the current project. New damage states will be defined based on the information obtained from the observed data. One possible approach to describing the different interpretations is through utilization of fuzzy sets and is being considered in the study. An expert system approach is also being reviewed for possible use in the project. A statistical method based on classification regression tree is being considered to categorize the damage data into a manageable data classes.

•Ground motions will be evaluated from scenario earthquakes and from probabilistic hazard analysis of the study area. Existing computational tools are being used for this purpose. The scenario ground motions will be compared to recorded ground motions from the Northridge earthquake of January 17, 1995. In addition, a spatial averaging method is being used to estimate the wave field over the area based on information from the recorded acceleration time histories and soil parameters from the bore hole data.

• In order to evaluate the direct economic losses, time delays and potential casualties, damage estimates of bridges, a network analysis method is being developed. One possible approach that is considered is based on the maximum flow minimum cut algorithm.

• Economic loss data from the Northridge earthquake are also being gathered in order to make appropriate comparisons with the estimated losses. Several scenarios will be considered in addition to duplication of the Northridge observations. These, for example, will include estimation of casualties at different times of the day to capture the various traffic patterns and the implications of these traffic patterns. Other scenarios may include evaluation of the system under more severe earthquakes and earthquakes originating on different faults. Probabilistic hazard estimates for time periods of 30, 50 and 100 years will be used to estimate the direct dollar losses from the failure of the system.

Summary

Considerable insight can be gained from the analyses being performed in this study. The methodology for transportation system evaluation can be used prior to any major future event to develop disaster mitigation policies through bridge and roadway rehabilitation and alternate route construction, emergency response planning and economic planning.

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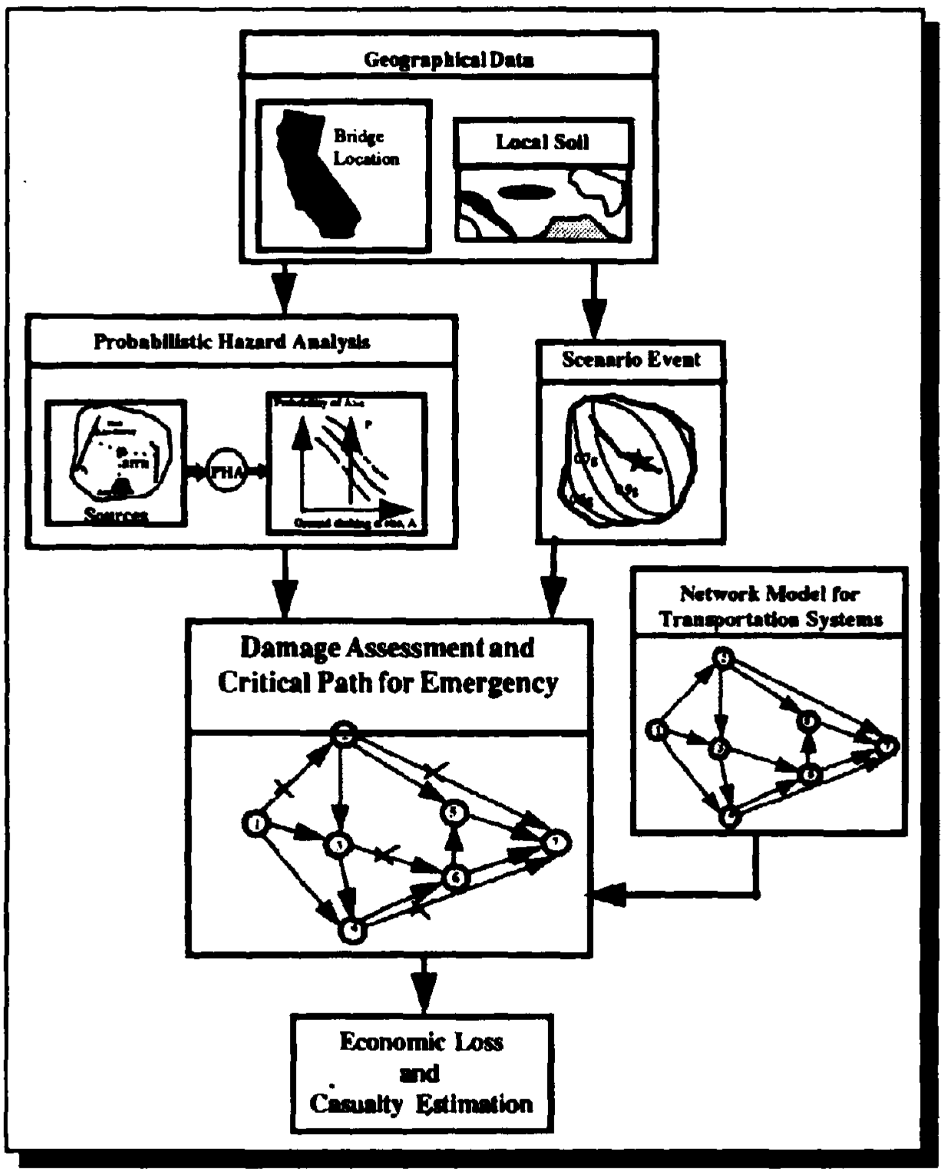

Figure 1. Methodology for Seismic Consequence Analysis of Transportation System

SEISMIC DAMAGE ASSESSMENT OF STRUCTURES USING CUMULATIVE DAMAGE MODEL

Y. H. Chai[1]

Abstract

Damage assessment of structures under high-intensity earthquake ground motion in terms of displacement ductility factor is inadequate since it is implicitly assumed that damage occurs only due to the maximum deformation and is independent of the number of non-peak inelastic cycles or the plastic strain energy dissipated by the structure. In this paper, structural damage is considered as a linear combination of normalized maximum displacement and normalized plastic strain energy. At ultimate limit state, the plastic strain energy decreases linearly with increased displacement amplitude, and is reached by the structure independent of the path. Experimental data provided by testing of small-scale steel cantilevers under large inelastic displacement cycles supports the linear assumption between plastic strain energy and imposed displacement, and the assumption of path-independence of response to the ultimate limit state.

Introduction

Most seismic codes today specify design force levels which are considerably lower than the maximum lateral inertial force expected in an equivalent elastic system during the design level earthquake. Consequently a certain degree of structural damage can be expected in the structure during such an event. To ensure adequate safety in the structure, dependable ductile mechanism capable of dissipating large amount of energy must be provided for the structure. Collapse of the structure which may lead to loss of lives must be avoided. Careful considerations of the structural response at the ultimate limit state are of paramount importance.

The structural damage implicit in current seismic design codes has traditionally been assessed using a displacement ductility factor which is defined as:

$$\mu = \frac{\Delta_m}{\Delta_y} \qquad (1)$$

where Δ_m = peak response displacement experienced by the structure during the design level ground motion; and Δ_y= characteristic yield displacement beyond which the structure is assumed to experience a large increase in displacement without an increase in the lateral force. The displacement ductility factor defined by Eqn. 1 must be interpreted as a cyclic displacement ductility factor (denoted as μ_c herein) since a response displacement is used in the definition. Damage characterization in terms of this parameter is

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rather inadequate since it is implicitly assumed that the structural damage occurs only due to the maximum deformation, and is independent of the number of load cycles imposed on the structure or the plastic strain energy dissipated by the structure.

Energy-Based Damage Model

A damage model based on a linear combination of damage due to excessive displacement and cumulative plastic strain energy has been proposed [Park and Ang, 1985]. A damage index D_I is used to characterize structural damage and is defined as:

$$D_I = \frac{\Delta_m}{\Delta_{um}} + \frac{\beta E_h}{V_y \Delta_{um}} \qquad (2)$$

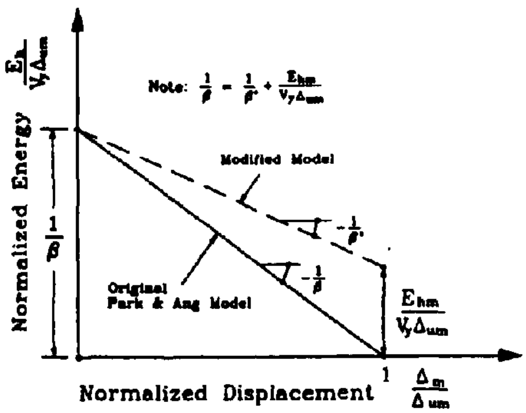

Figure 1: Ultimate Limte State Represented by Park and Ang Model

where Δ_m = maximum response displacement; Δ_{um} = maximum displacement under a monotonic loading; E_h = plastic strain energy dissipated by the member; V_y = yield strength of the member; and β = strength deterioration parameter which can be obtained by calibrating the model against experimental data such that a damage index of $D_I = 1$ corresponds to the ultimate limit state of the structure. By equating the damage index to unity to signify structural failure, Eqn. 2 can

$$\frac{E_h}{V_y \Delta_{um}} = \frac{1}{\beta} - \frac{1}{\beta}\frac{\Delta_m}{\Delta_{um}} \qquad (3)$$

be rearranged into:
The ultimate limit state expressed by Eqn. 3 indicates that the plastic strain energy capacity of the structure is a linear function of the peak response displacement, and can be plotted as a straight line in the normalized energy $E_h / V_y \Delta_{um}$ versus the normalized displacement Δ_m / Δ_{um} space. This limit state is shown as a solid line with y-intercept of $1/\beta$ and a slope of $-1/\beta$ in Figure 1. Note that, under monotonic loading, the plastic strain energy dissipated by the structure as implied by the model at $\Delta_m / \Delta_{um} = 1$ is zero. The actual structure, however, is expected to exhibit a non-linear behavior and to dissipate a finite amount of plastic strain energy under monotonic loading. In this paper, the lack of agreement between the

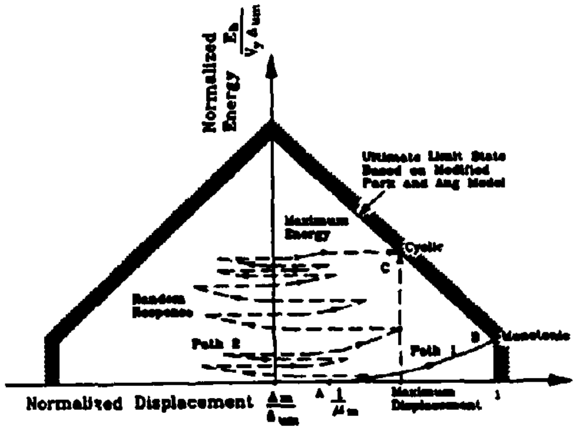

Figure 2: Non-Linear Response in Normalized Energy-Normalized Displacement Space

model and expected behavior is overcome by incorporating only the portion of dissipated energy above that of monotonic loading as contribution to damage i.e. modifying Eqn. 2 to the form:

$$D_t = \frac{\Delta_m}{\Delta_{um}} + \frac{\beta^*(E_h - E_{hm})}{V_y \Delta_{um}} \qquad (4)$$

where E_{hm} = plastic strain energy dissipated by the structure under a monotonic loading; and β^*= modified strength deterioration parameter. The ultimate limit state for the modified model can still be represented as a straight line in the normalized energy versus the normalized displacement space, and is shown as a dashed line in Figure 1. The slope of the modified model at the ultimate limit state is given by $1/\beta^*$. The modified model is assumed to have the same y-intercept as the original model.

The non-linear response of a structure under a high-intensity seismic loading can be represented by a two-dimensional path in the normalized energy versus normalized displacement space as indicated in Figure 2. The ultimate limit state in this case by two straight lines assumed to be symmetrical about the zero displacement axis. Under monotonic loading, the response of the structure can be represented by a path along the x-axis (normalized displacement) until yielding of the structure, as signified by point A in Figure 2. Further displacement beyond yield is represented by a path with a non-zero positive slope along path 1 until failure is reached at point B. Under a random excitation, the response of the structure can be characterized by an irregular path in the normalized energy versus normalized displacement space, as exemplified by path 2 in Figure 2. Similar to the damage characterization by displacement ductility factor, the Park and Ang model implicitly assumes a path-independent response up to the ultimate limit state of the structure.

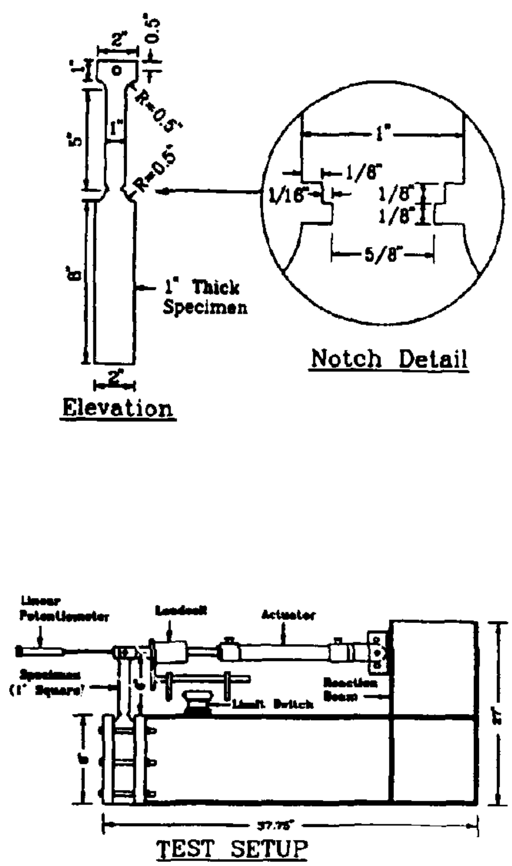

Figure 3: Specimen Dimensions and Test Setup

Experimental Testing

Experimental verification of the damage model is provided by two series of tests on small-scale steel cantilevers under reversed cyclic loading. Figure 3 shows the test setup where inelastic cyclic displacements in the range of 0.10 to 0.58 Δ_{um} were imposed on the steel cantilevers using a double-acting actuator.

Figure 4(a) shows the plot of normalized plastic strain energy versus normalized peak displacement for the first series of tests conducted under constant displacement amplitudes. It can be seen from Figure 4(a) that the plastic strain energy dissipated by the specimen decreases almost linearly with increased displacement amplitude over a large inelastic displacement range. The slope of the best-fit line gives a strength deterioration parameter of $\beta^* = 0.229$ for the modified model. Figure 4(b) shows the plot of normalized hysteretic energy versus normalized displacement for the second series of tests. The circle symbol in Figure 4(b) corresponds to the monotonic and constant displacement amplitude tests, while the square symbol corresponds to the irregular displacement history tests. It can be seen from Figure 4(b) that the data from constant amplitude tests also support the linear assumption of the damage model. A different slope is obtained for the best-fit line in the second test series due to different steel being used for the specimens. The slope of the best-fit line gives a strength deterioration parameter of $\beta^* = 0.38$ for the modified model. In spite of the irregular displacement history, the data from the variable displacement amplitude tests lie fairly close to the same straight line obtained from the constant displacement amplitude tests.

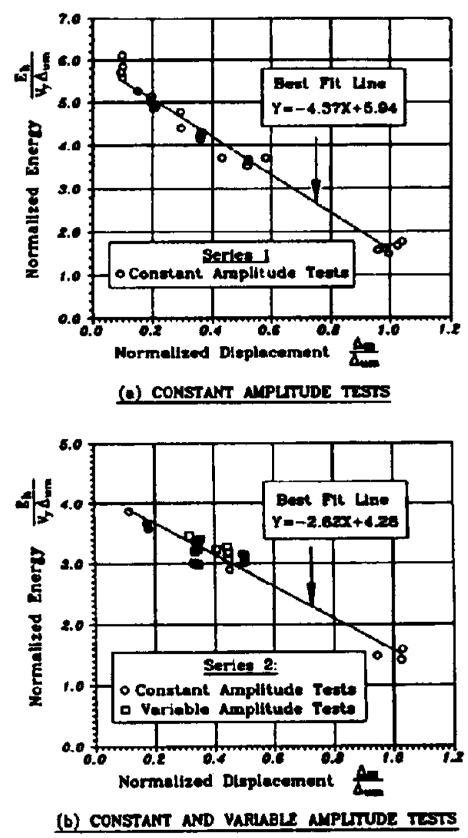

Figure 4: Normalized Energy Versus Normalized Displacement

Conclusions

An existing energy-based linear damage model is modified to take into account of the plastic strain energy dissipated by the structure under monotonic loading. Only the portion of plastic strain energy above that of monotonic loading is considered as contributing to structural damage. An inelastic response under high-intensity ground shaking can be characterized by a path in the normalized plastic strain energy versus normalized peak displacement space with ultimate limit state represented by a linear reduction of normalized plastic strain energy with increased normalized peak displacement. Test data from small-scaled notched steel cantilever beams cycled at constant displacement amplitudes supports the assumption of linear reduction of plastic strain energy with increased displacement amplitudes. Also test data from specimens subjected to variable displacement histories support the path-independent characterization of ultimate limit state for notched steel cantilever beams.

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NATURAL HAZARD MITIGATION
NSF GRANTEES WORKSHOP

SESSION V - NON-EARTHQUAKE HAZARD MITIGATION

"An Expert System for Building Vulnerability Assessment in Windstorms"
D. Reed, T. English, and K. Mehta

"Mid Continental Floods"
K. Georgakakos

"Impact of Design Philosophies on Disruption to Critical Environmental Infrastructure from the Great Flood of 1993"
D. Sanders

"A Distributed Model for Real-Time Rainfall-Runoff Forecasting"
R. Bras

"Identification and Characterization of Collapsible Soil"
S. Houston and W. Houston

"Cooperative Programs in Wind Engineering"
K. Mehta

AN EXPERT SYSTEM FOR BUILDING VULNERABILITY ASSESSMENT IN WINDSTORMS

D.A. Reed[1], T. English[2] and K. Mehta[3]

Abstract

The development of a knowledge-based system called *WIND* is examined in this paper. Background material on expert system use in for structural vulnerability assessment is presented. The object-oriented data modeling is briefly discussed.

Introduction

Building vulnerability to wind is by nature an uncertain, subjective and ill-defined process. Because the subjective diagnostic process is imprecise, and there are many types of uncertainties associated with damage assessment and vulnerability analysis, the process is not straightforward. Knowledge-base systems are one approach to the modeling of this decision-making process. This paper will describe a prototype knowledge-based system for wind vulnerability assessment of industrial buildings. In vulnerability analyses, knowledge of structural behavior, as well as environmental factors, is essential.

A review of the literature of the various systems available for building safety assessment reveals emphases on earthquakes and blast loadings, Casciati, et al., (1991); Krauthammer,et al., (1992); Rojahn, et al., (1993); Ross,et al., (1990); Thurston, (1990); Yao, (1985). These are summarized in Table 1. Obviously, a gap in the existing assessment methodologies for wind engineering presently exists. The program WIND described here addresses this need in the wind community. Guidelines for the prediction of the wind damage to buildings were outlined by Mehta, et al., (1981), and their subjective and analytical approach provided the basis for the present program shown in the table as WIND. WIND was developed using the M4 shell; WIND's primary task is vulnerability assessment due to wind loadings; its structural analysis capability is in the elastic range via an interface with SAP90, and the wind loadings are based upon ASCE-7.

Structural Vulnerability Assessment for Extreme Wind Conditions

Under extreme wind conditions, framing systems generally do not collapse; however, envelope breach occurs, with costly interior damage caused by accompanying rain, wind and debris. Therefore, for wind effects, the critical components are the wall or cladding panels, the fasteners and local boundary conditions for the cladding, and the roofing. In addition, the breach of the envelope for a low-rise building may significantly alter the loading on the framing system, or on other portions of the envelope. Wind loadings of the program conform to the ASCE-7 (1988) document; however, heuristics involving the immediate surroundings or local environment were developed for the consideration of wind effects. The dominant wind engineering failure mode may be stated as "breach of any envelope member or component." In the computer program described here, the modes of failure or damage states are defined in Table 2.

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Table 1. Summary of Expert Systems for Structural Damage Assessment

Name	Task	Implementation Environment	Analysis Capability	Loading Emphasis
SPERIL e.g., Yao, (1985)	Seismic vulnerability of existing structures	Rules	None	Earthquakes
EVAL e.g., Ibarra-Anaya and Fenves, (1991)	Seismic vulnerability of existing structures	Production Rules Algorithms	None	Earthquakes
QUIKTILT Thurston, (1990)	Expected Seismic Performance of Tilt-Up Buildings	PC-based Expert System Shell Deciding Factor	None	Earthquakes
EXQUAKE2 e.g., Shwe and Adeli, (1993)	Earthquake Damage Evaluation and Knowledge of Tilt-Up Buildings	EXSYS Professional: Rule-Based	None	Earthquakes
Casciati and Faravelli, (1991)	Seismic Vulnerability Assessment of Masonry Buildings	NEXPERT	None	Earthquakes, with specific emphasis on Italian construction
California Rapid Seismic Evaluation Expert; Rojahn and Kiremidjian, (1993)	Rapid visual screening of buildings for potential seismic hazards	VP-Expert	None	Earthquakes, with particular emphasis on ATC-21
DAPS Ross, et al., (1990)	Damage Assessment of shallow-buried reinforced concrete protective structures	EXSYS	None	Blast Loadings
Krauthammer, et al., (1992)	Damage Assessment of shallow-buried reinforced concrete protective structures	Personal Consultant Plus	None	Blast Loadings
WIND [discussed here]	Wind vulnerability of existing buildings	M4 Expert System Shell: Productions Rules	Elastic analysis interfaced via SAP90	Wind loadings based upon ASCE-7

Table 2. Definition of Failure Modes and Damage States

Failure Mode	Definition
Minor Bieach	Failure Mode: Removal of Covering or Damage State: Partial Removal
Major Breach	Fastener Pull-Out or Fastener Shear Failure or Panel Failure
Frame Failure	Failure of any Primary Building Framing Member

Object-Oriented Modeling

An object oriented data model has been used to characterize the building. The representation of the building envelope presents several challenges in the modeling of the structural components and their interactions. Naeher, et al., (1993) employed an object-oriented (OO) data model for wind safety analysis; representations of the framing system and wind loading were developed. Numerous investigations into the use of the OO data model for framing systems and connections exist, such as G.R. Miller, (1988), Powell, et al.,(1989), and G.L.Fenves, (1989). Recent investigations into the OO representation of buildings have focused on structural design; e.g., Howard, et al., (1992). Implementation of an OO data model for adequate representation of the building envelope in addition to the primary framing system has been challenging. Connections and other spatial as well as functional relationships are difficult to model, although they may be easy to visualize. Table 3 provides an example of the OO model employed in WIND. Here the various subclasses of the class *Cladding* are described, with their associated methods. *Fasteners* are modeled as classes separate from their components (class *cladding*) they connect to the framing system. Data modeling will continue to evolve as the program is further developed and calibrated.

Table 3. Examples of OO modeling in WIND.

Class	Subclass of	Methods
Cladding	Has no parent or superclass	Evaluate panel (Compare panel strength to wind pressure capacity) Print failed components (Show results)
Opening	Cladding	Evaluate impact (Compare missile hazard to window/ door strength) Evaluate panel (Inherited) Print failed components (Inherited)
Masonry	Cladding	Evaluate construction (Checking for loadbearing or reinforcement) Evaluate panel (Inherited) Print failed components (Inherited)
Roofing	Cladding	Evaluate membrane (Check for membrane failure) Evaluate panel (Inherited) Print failed components (Inherited)
Fastener	Has no parent or superclass	Evaluate fasteners (Propagate pressure to fasteners)

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MID CONTINENTAL FLOODS

Konstantine P. Georgakakos[1]

Abstract

Reviewed are fundamental diagnostic studies pertaining to hydrologic processes active during extreme mid continental flood events. The important role of soil water in the production of high runoff, the type of spatial scaling that floods exhibit and the presence of a strong association from soil water to subsequent surface air temperature are quantified for several drainage basins in the central United States.

Introduction

Floods are the result of natural processes that operate on many temporal and spatial scales and, in some cases, over large distances. A good understanding of such processes is a necessary prerequisite for the design of a robust flood prediction and control system that provides useful lead time for disaster mitigation. Traditional hydrologic research in operational forecasting (e.g., Georgakakos and Smith, 1990, Georgakakos, 1994) and operational implementation of forecast systems (Bae and Georgakakos, 1992, and Bae et al. 1995) has focused on forecast lead times comparable to the response time of drainage basins, and on spatial scales of order 10^3 km^2. Well calibrated hydrologic models complemented with robust state estimators for probabilistic predictions and real-time state updating provide accurate and reliable real time predictions of flood flows. These predictions may be used to improve the operational management of flood control reservoirs (Georgakakos, A., and Yao, 1994). Current research efforts focus on: *(a)* the role of soil water in the forcing and feedback active over continental drainage basins (Georgakakos et al. 1995, and Cayan and Georgakakos, 1995); *(b)* hydrologic processes over the small scales pertinent to flash flooding (AMS, 1993 and Georgakakos et al. 1993); and *(c)* the links of precursor oceanic and atmospheric processes to streamflow for extending the useful forecast lead time of reliable predictions over large scales (e.g., Guetter and Georgakakos, 1993, Roads et al. 1994, and Guetter and Georgakakos, 1995).

Significant results of the author's current research pertaining to soil water features are highlighted in the next section. Implications for prediction and research issues that remain unresolved are presented in a concluding section.

Selected Results

Soil water stores past land-surface forcing and it provides a buffer for the land surface system. As such, it exerts control over the rainfall forcing and its temporal and spatial variability is closely linked to the initiation and the ultimate magnitude of surface runoff and floods. Lack of extensive observations of soil moisture (e.g., Georgakakos and Baumer, 1995) necessitate inference of soil water (depth-integrated soil moisture) from hydrologic models, using the observed record of precipitation, temperature and flow (about 4 decades). Such inference is

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expected to be reliable if the models reproduce streamflow well on a daily scale (Bae and Georgakakos, 1994). An adaptation of the operational National Weather Service rainfall-runoff model was used in this study with 40 years of daily data. The model explains about 70 percent of the variance of the observed daily flows.

Characteristic of model-reconstructed soil water in the midwestern U.S. is an annual cycle with an early peak in March-April for the soil water in the upper 20-30 cm of soil and a later peak in June for the soil water in deeper soil layers. These two peaks correspond to two flow peaks in the annual flow cycle. The first peak results from the saturation of the upper soil when precipitation exceeds evapotranspiration substantially in spring, while the second peak is due to the gradual saturation of the total soil column down to a meter (or so) of depth in summer. This climatological behavior of soil water makes heavy persistent summer rainfall an almost sure precursor to flooding in the midwest (e.g., Great Flood of 1993).

The reconstructed series of soil water for several basins in the mid continental United States (e.g., Georgakakos and Bae, 1994, Georgakakos et al. 1995, and Cayan and Georgakakos, 1995) show rich temporal and spatial scales of variability. Upper soil water estimates possess serial auto-correlation coefficients near 0.5 for a 1-month lag. Lower soil estimates possess much stronger temporal coherence with the analogous auto-correlation coefficient values being near 0.9. This significant serial dependence of soil water estimates is stronger in the higher latitudes. The spatial coherence of monthly soil water anomalies (residuals obtained from soil water values by subtracting the long-term monthly means) depends on the strength and the sign of the anomalies. Positive anomalies (wetter soils) are likely to be of a smaller spatial extent than negative anomalies (drier soils) of the same strength. Soil water estimates from three Iowa catchments, which collectively covered an area of $2.5^o x 2.5^o$, revealed that the frequency of occurrence of positive anomalies of strength $1.5\sigma^+$ and of areal extent as large as the $2.5^o x 2.5^o$ area is about 10 % (Figure 1). The standard deviation σ^+ signifies long-term monthly standard deviation of soil water. The analogous frequency for corresponding negative anomalies is five times larger. For the south-central U.S., analysis of reconstructed soil water time series shows that no individual catchment within a $3^o x 3^o$ region is representative of the range of soil water variability in the region, both for positive and for negative anomalies. It was also found that positive soil water anomalies of extreme strength over individual basins tend to inhibit the occurrence of such anomalies in nearby basins. This was not observed for negative anomalies.

The analysis of the cross-correlation of soil water with surface air temperature (daily maximum, T_{max}), with soil water leading T_{max} by a week or two (especially in the south-central U.S.), showed that there is a strong association between those two indicators for extreme soil water anomalies. The drier the soil becomes, the warmer the surface air is expected to be in the week or so that follows. Conversely, the wetter the soil becomes, the cooler the surface air will be. These results, an example of which is shown in Figure 2, are a consequence of the evapotranspiration and cloud development processes and have direct implications for temperature prediction. No substantial feedback from soil water to rainfall over areas as large as $3^o x 3^o$ was detected.

The Great Flood of 1993 provides a unique opportunity to study the behavior of the land surface hydrology under extreme conditions of large areal extent. A comparison was made of the behavior of soil water during the Flood with its behavior during other past extreme events for selected midwestern basins with long historical records. It showed that the Flood was unique for the unusually high value of lower soil water in July and August. While the upper soils remained near saturation, the lower soil water gradually reached a saturation level in July and remained near saturation in August, thus producing prodigious amounts of surface runoff under persistent rains. The soil water in midwestern basins increased steadily toward saturation levels during the period from the 1988 Drought to the 1993 Flood (Figure 3). Thus important seasonal and interannual trends were present during this period. It is noted that the operational hydrologic model used to reconstruct the historical series of soil water is capable of simulating successfully the flow during both periods of droughts and periods of floods (residuals in Figure 3).

Concluding Remarks

Successful medium and long-range prediction of floods in basins of the mid continental United States will require meteorological and hydrological models that are capable of simulating the scaling properties of soil water and the forcing and feedback mechanisms between soil water and the atmosphere. The diagnostic analysis of the historical record and, especially, of the extreme events such as the Great Flood of 1993 provides valuable guidance for the parametrization of these processes in predictive models.

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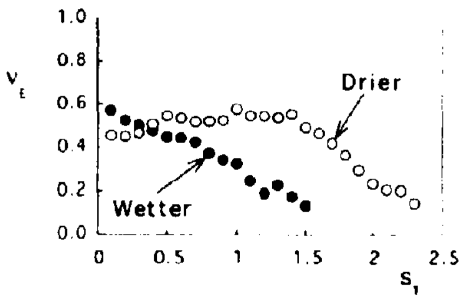

Figure 1. Relative frequency v_E of occurrence of soil water anomalies of strength s_T which cover all three basins: Boone, S. Raccoon, Thompson. Anomaly magnitude is $s_T\sigma+$, with $\sigma+$ being the long-term monthly standard deviation.

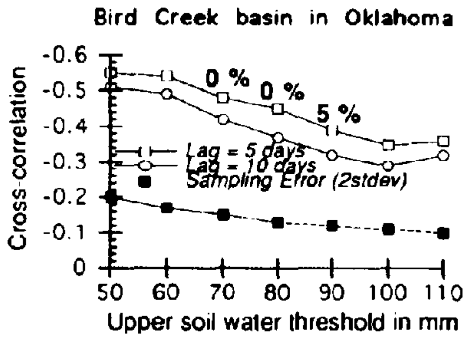

Figure 2. Cross-correlation coefficient between 5-day averaged soil water and Tmax, lagged by 5 or 10 days, for soil water less than a set threshold, as a function of the threshold. The percentages shown indicate the frequency of having the values of cross-correlation occur in a chance experiment that preserves serial autocorrelations and concurrent cross-correlations of the two variables.

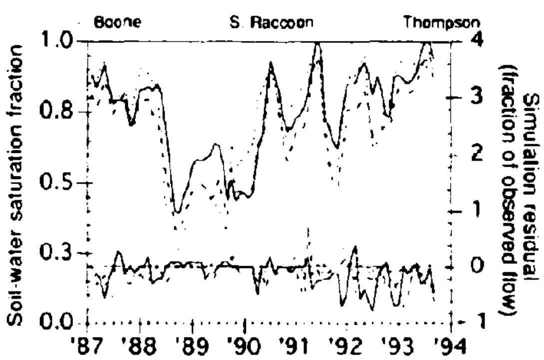

Figure 3. Saturation fraction (left ordinate) and simulation flow residuals for three Iowa basins. Residuals are fractions of observed flow for flows greater than 0.5 mm/day.

IMPACT OF DESIGN PHILOSOPHIES ON DISRUPTION TO CRITICAL ENVIRONMENTAL INFRASTRUCTURE FROM THE GREAT FLOOD OF 1993

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Abstract

The Great Flood of 1993 tested the ability of conventionally-designed environmental infrastructure facilities to remain operational. This study examined ten such facilities (four drinking water treatment plants and six wastewater treatment plants) impacted by the 1993 flood to determine: 1) How facilities operated during the flood; 2) Which facilities had flood plans, how well flood plans were followed, and how effective the flood plans were; 3) What assumptions were implicit in the design of each facility with respect to flood survivability, and 4) How future facilities can be designed to withstand the worst impacts of a catastrophic flood.

Introduction

Major U.S. cities obtain their drinking water from rivers, and virtually all inland cities discharge their wastewater to rivers; hence, water and wastewater treatment facilities are usually located in river valleys. A "river valley" is, in reality, an historical flood plain. All land within a river valley has been subjected to water action sufficient to erode a valley over geological time. Location within a river valley therefore implies that a risk of catastrophic flooding exists.

Regulations have little impact on the design of environmental infrastructure facilities to withstand flooding. The only federal requirement is that facilities built or upgraded using federal funds must be located outside the 100-year floodplain. State laws vary, but in the state of Missouri (the location of this research study), water treatment plants must be located above the historical flood of record, while wastewater regulations require protection from the 100-year flood for major facilities (treatment plants), and the 25-year flood for minor facilities (lift stations). A 100-year floodplain is an area having a one percent probability of being inundated in any given year; a 100-year flood is one causing inundation of the 100-year floodplain. Land-use changes, particularly urbanization, can result in increased flood frequency and severity. Floodplain maps are not, and cannot be, updated frequently enough to take all these changes into consideration. A 100-year floodplain map is, therefore, likely to give a false sense of security to the operator of a facility that is located outside the indicated area of flooding.

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Procedures

The ten facilities studied were selected from a list of impacted facilities provided by the Missouri Department of Natural Resources (MDNR). The facilities were located along the Missouri and Mississippi Rivers in Missouri (see Table 1). The selected facilities were subject to the full range of flood impacts, from minor operational problems to total destruction of the facility. Prior to performing a site visit, a questionnaire was developed to ensure the information obtained during the visits was complete, and that data would be consistent from site to site. During the site visits, we met with managers and operational staff, obtained copies of flood plans (where they existed), inspected flood damage prior to repairs (where possible), gathered photographs of current and flooded conditions, inspected records taken during the flood, and obtained all other information available on the impact of the flood.

Findings

The major finding during the study is that even the most rigorous conventional design procedures do not completely protect facilities. Several of the facilities were either protected by a levee, built to an elevation above flood level, or built to structurally withstand a flood. The facilities were damaged or disrupted, nonetheless.

However, most facilities were not designed with any integrated approach to hydraulic management in a flood. Many facilities were designed with extensive piping at elevations considerably below flood level (sometimes below normal river elevation); these facilities frequently lack adequate valving, pumping, and pipe sizing. As a consequence, the facilities flooded from the inside as water backed up through the systems.

Few facilities have accurate *as-builts* (hand-annotated copies of plan drawings that incorporate changes made during construction); as-builts are critical items during emergency operations. Most of the older facilities have been modified many times during the past, often without any records being kept at all. Some plants flooded through pipelines or manholes that were totally unknown to current operational staff.

Computer or remote control of facilities during the flood was problematic at some facilities, but saved others. One facility, a major wastewater treatment plant, operated unattended for ten days when the facility had to be evacuated because of an explosion threat at an adjacent propane gas tank farm. Unattended operation was only possible through excellent planning and a reliable computer control system.

Few facilities have flood plans. Flood plans that exist have been prepared by the operating staff, not by the facility's design engineer, and plans are not comprehensive. Plans do not contain even basic emergency contact information, which is critical during a flood. The only flood planning typically consists of a brief list of actions triggered by water elevation (i.e., at a specified water elevation, the operators are to close critical valves, move chemicals from a lower storeroom, and start sandbagging cited low areas of the plant).

TABLE 1. IMPACTS OF THE FLOOD OF 1993 ON TEN CRITICAL ENVIRONMENTAL INFRASTRUCTURE FACILITIES

FACILITY	DAMAGE	DESIGNED FLOOD PROTECTION	FLOOD PLAN?
Water Treatment			
St. Louis County	Inundated	Levee	Yes
N. Jefferson City	Inundated	None	No
Howard Bend (St. Louis)	Impacted	Levee	Yes
Columbia	Impacted	Levee	No
Wastewater Treatment			
Missouri River (St. Louis)	Inundated	Levee	No
Jefferson City	Inundated	Elevation	No
Watkins Creek (Lift Station)	Inundated	Floodproofed	N/A
Columbia Wetlands	Inundated	Levee	No
Bissell Point (St. Louis)	Impacted	Levee	No
LeMay (St. Louis)	Impacted	Elevation	No

Conclusions and Recommendations

The design assumptions used in preparation of plans and specifications for environmental infrastructure clearly do not give adequate weight to the possibility of an extreme natural event, such as a flood. Design engineers rely on federal and state guidelines, which do not demand the design rigor required of buildings constructed within the same areas. The "limit states" principles used for building design throughout the world could and should be adapted to the design of critical infrastructure. Briefly, "limit states" principles require that structural engineers consider two scenarios in the design of any structure: protection of human life and limb through prevention of the collapse of structures (called "extreme limit states"), and preservation of structural integrity during normal operation (called "serviceability limit states"). Critical environmental infrastructure could be designed to protect the public health (i.e., provide safe drinking water and convey sewage safely away from the public) through any extreme event, while non-critical portions would be designed to survive frequent flood events with minor damage.

Complete protection of the public health will require that critical processes of new environmental infrastructure be constructed outside and above the flood plain--not just above an arbitrary flood level. The Presidential Task Force on Floodplain Management concluded, "Risk exists in all areas within a floodplain--both areas protected by channel modifications, dams or levees and areas outside the 100-year floodplain.... Even though areas protected by levees are considered safe, the potential for catastrophic loss still exists."(1) Other researchers are beginning to realize the consequences of building in historical flood plains and are recommending against critical structures in river valleys (2).

Existing facilities should be evaluated to develop a complete vulnerability analysis, using procedures as recommended by the Water Environment Federation in their Manual of Practice No. 11 (3). Prior to receiving any federal (or state) funding for upgrades or flood mitigation, facilities should be required to flood-proof the most vulnerable portions of each facility. No modifications should be allowed to a facility unless the modifications are designed by a competent design professional and are in accordance with the overall vulnerability analysis of the facility.

Computer control systems which can allow for unattended operation should be considered for critical environmental infrastructure facilities which must remain in the floodplain, but are due for major upgrades. Many systems are implementing computerized control; control systems should be designed for flood conditions.

Every critical environmental infrastructure facility should possess a current, comprehensive flood plan. The flood plan should contain a section on emergency communications, which is updated on a routine basis (at least annually) to include emergency ingress and egress routes, emergency phone numbers, formats for public releases, and other items deemed critical by local officials. The vulnerability analysis discussed above should be used to develop an action plan that will protect personnel, facilities, and supplies as flood waters rise.

Acknowledgment

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A Distributed Model for Real-Time Rainfall-Runoff Forecasting

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Abstract

A distributed model for real-time rainfall-runoff simulation during floods is presented. The model is largely based on the detailed topographical information provided by digital elevation models (DEMs) and the availability of radar rainfall measurements. The model applies a kinematic model of infiltration to evaluate local runoff generation in grid elements as defined by the DEM. Additionally the model accounts for lateral moisture flow between elements and routes surface flow consistent with the local gradient in the terrain.

Introduction

The model and research presented here is motivated by the operational needs of hydrologists to deterministically forecast river discharges in basins of moderate to large scales. In the past, deterministic forecasts at this scale were infeasible due to data and computational constraints. This is no longer the case with the advent of powerful computational resources and two recent developments in data availability: (1) detailed topographical descriptions of river basins at low cost, in the form of Digital Elevation Models, and (2) instantaneous measurement of the spatial development of precipitation, in the form of radar-generated rainfall maps.

Model Formulation

The objective of the distributed model is to transform spatially distributed information of elevations, precipitation, and soil type into a predicted discharge hydrograph. The model determines the discharge hydrograph through a series of intermediate computations at each pixel in the basin. Calculated quantities include local infiltration capacity, inter-pixel moisture transfers, local runoff generation, and surface flow routing.

Infiltration is computed first using a simple model developed by Cabral *et al.* (1993) to determine the local infiltration capacity at each location. This infiltration capacity is based on the moisture dependent hydraulic conductivity given by the Brooks-Corey parameterization scheme (Brooks and Corey, 1964) coupled with an exponentially decreasing hydraulic conductivity with depth (Beven, 1984). Anisotropy of hydraulic conductivity in the directions parallel and normal to the land surface is also accounted for. Figure 1 shows shows the coordinate system used for the infiltration segment of the model.

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Moisture transfers between pixels occur wherever a pressure gradient is encountered between adjacent pixels. This may be caused by local differences in the terrain elevations. Additionally, even neighboring pixels at the same elevation my exchange moisture if there exists a difference in the moisture distributions between adjacent pixels.

The infiltration capacity is compared to the local rainfall rate with two possible runoff generation scenarios: infiltration-excess runoff and return flow. Infiltration excess runoff occurs when the infiltration capacity of the soil column is less than the rainfall rate. In contrast, return flow occurs when the unsaturated volume of the soil column is smaller than flux into the column due to precipitation and moisture exchanges between pixels. Figure 2 shows a hypothetical vertical moisture profile which may generate infiltration excess runoff. Return flow would occur if the volume represented by the lightly shaded areas were to become saturated.

By assuming that all flows travel along the path of steepest descent within the rectangular grid of DEM, it is possible to determine the flow paths from any pixel to the outlet of the basin. Additionally, pixels can be differentiated into two categories: hillslope and channel pixels. Each type of pixel has a characteristic travel velocity which is assumed uniform in space, but which varies with time according to the discharge at the basin outlet. By coupling lengths of travel to the outlet and travel velocities, it is possible to determine the distribution of arrival times of incremental runoff produced by each pixel. Summing over all pixels gives the total discharge at the outlet.

One of the greatest strengths of the model is its modular structure and graphical interface. This allows for the user to graphically analyze a snapshot of any of the state variables or basin properties. Figure 3 shows four such snapshots of (clockwise from top-left): current infiltration capacity, maximum infiltration capacity, moisture content above wetting front, and runoff generation.

Calibration of Model

The process of calibration is illustrated here for the Sieve basin, a sub-basin of the Arno river, Italy. The parameters involved in the calibration exercise are the decay rate of hydraulic conductivity normal to the land-surface, the hydraulic conductivity anisotropy ratio, the channel velocity coefficient, and the ratio of channel to hillslope velocity. All other information is fixed from the DEM, radar-rainfall map, and soil survey map. Figure 4 shows three simulated hydrographs at different initial saturation levels for the basin, along with asterisks "*" indicating the observed hydrograph for a storm in January 1985. Although we illustrate only one example, we were generally able to calibrate the model to an acceptable level of performance with only a limited amount of data.

Summary

This paper presents our distributed rainfall-runoff model. The model incorporates spatially varied topography, precipitation, and soil characteristics and couples this data with a simple series of infiltration, runoff generation, and streamflow routing procedures to produce a predicted discharge hydrograph from the basin being studied. Our efforts have shown that this model can be calibrated and verified with a limited amount of data and ultimately be applied to the task of real-time rainfall runoff forecasting in large scale river basins.

Appendix

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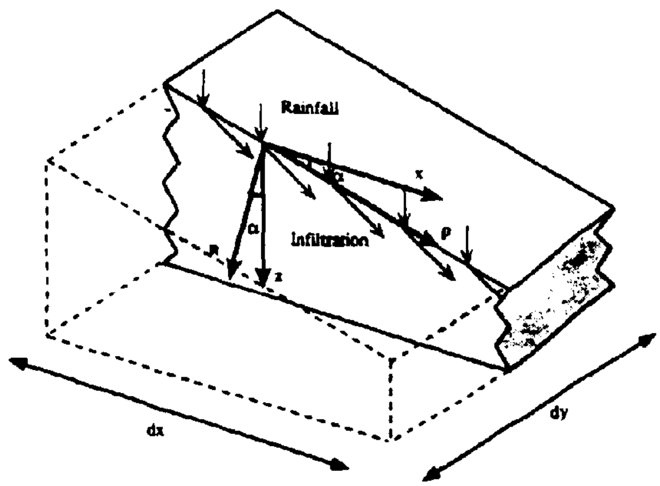

Figure 1: Coordinate system used for infiltration analysis on a grid element.

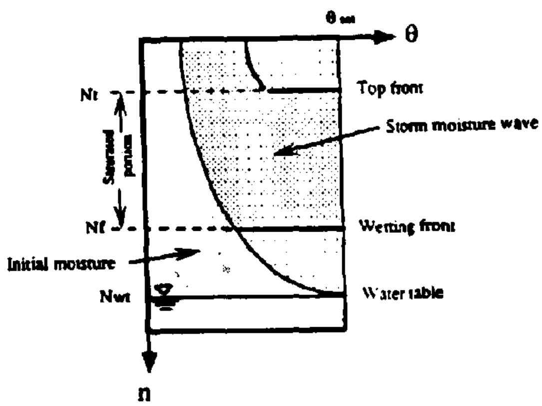

Figure 2: A typical vertical moisture profile for a soil column.

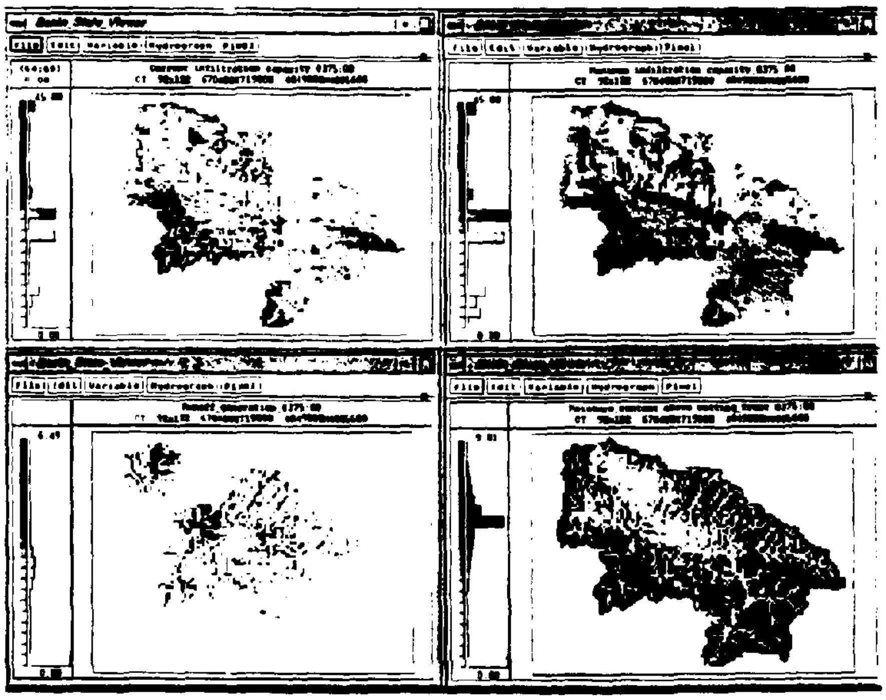

Figure 3: Four variables at the same stage of the storm.

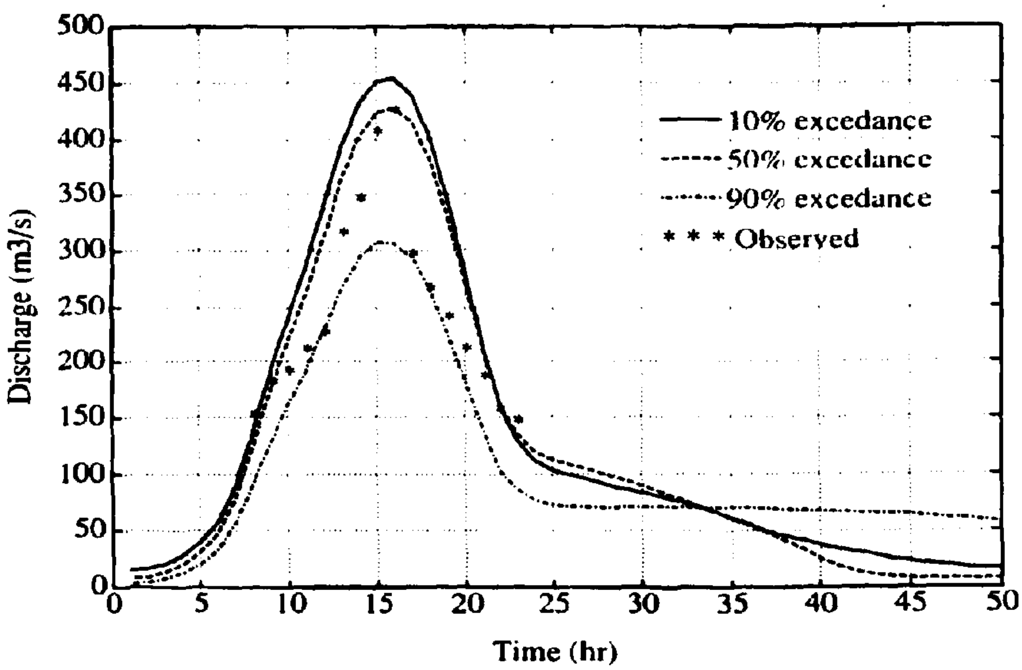

Figure 4: Observed and simulated hydrographs for the storm of January 1985. simulated results correspond to initial moisture states with 10%, 50%, and 90% probability of exceedance.

IDENTIFICATION AND CHARACTERIZATION OF COLLAPSIBLE SOILS

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William N. Houston[2]

Abstract

Arid-lands soils can pose many challenges to engineers. One major problem associated with man-induced changes in the surface and groundwater flow patterns is that of collapsible soils. Collapsible soils are typically stable in their dry natural state, but may exhibit large settlements when wetted, often under overburden pressures alone. In many cases, collapsible soils can be identified using relatively inexpensive methods. In-situ techniques for testing the more difficult-to-sample collapsible soils have been developed and demonstrated. It is possible to predict collapse settlements from the results of these laboratory or field tests. The importance of direct measurement of the wetting-induced volume change potential of these soils is evident from numerous studies. Research findings also demonstrate the importance of degree and extent of wetting in making collapse settlement predictions.

Introduction

Primary geotechnical problems in arid regions are soil expansion, soil collapse, and salt damage. Other very common problems in arid soils include fines migration, dispersion, leaching, erosion, and dust. The two most-often-cited concerns in arid regions are soil expansion and soil collapse. While expansive soils can lead to chronic structural distress due to cyclic shrink/swell behavior, soil collapse often results in sudden and rather dramatic damage of existing structures. Soil expansion and collapse are both moisture-sensitivity issues. However, the mechanism of soil collapse is quite different from the mechanism of shrink/swell. The focus of the discussion in this paper is collapsible soils.

Soil collapse is almost always associated with man-induced changes in the groundwater and surface water regime. The changes in the water content (suction) of the soil are frequently a result of engineering modifications such as irrigation, pumping and recharging of groundwater, construction of dams, and alterations of natural flow patterns by cut/fill operations. Uncertainties associated with the degree and extent of wetting over the lifetime of the structure, and the typical high degree of nonhomogeneity of arid region soil profiles add to the complexity of moisture-sensitive soil analyses. The greatest problems with collapse arise when the moisture-sensitive

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soils are not identified prior to construction or when the depth and extent of wetting has not been conservatively estimated.

Problem foundation and subgrade materials consisting of collapsible soils are common in arid environments, which have climatic conditions and depositional and weathering processes favorable to their formation. Deposits of collapsible soils provide unique challenges to foundation designers. These soils are typically open-structured, unsaturated ,and lightly to moderately cemented materials that exhibit low compressibility and high strength in their natural dry condition. However, collapsible soils are metastable in structure, and the addition of water can trigger large settlements that result in damage to structures and roadways. The cause of soil collapse is primarily related to a loss in cementation resulting from water-induced reduction in soil matric suction (negative pore water pressure) and weakening of cementing agents. Often the collapsible soil problem goes undetected until damage occurs, resulting in the need for more challenging and expensive engineering solutions.

The key findings and recommendations from National Science Foundation supported research on collapsible soils are reported in the following sections of this paper. The focus of the studies has been to develop relatively low cost and simple methods that will be readily adopted by practicing engineers.

Collapse Settlement Predictions

Response-to-wetting tests can be used to obtain estimates of collapse settlements of structures founded on collapsible soils. Numerous studies are available comparing actual field collapse settlements to settlements predicted using results of laboratory tests. In general, predicted settlements range from 70% underestimation to 300% overestimation of actual field settlements. Several possibilities for the differences between estimated and observed collapse settlement have been identified. These include, differences in the estimated extent and degree of wetting, sample disturbance, and soil nonhomogeneities.

Sample Disturbance of Collapsible Soils

Collapsible soils usually exhibit a significant degree of cementation. Because of the cemented and contractive nature, collapsible soils are not particularly susceptible to disturbances caused by the use of large-area-ratio samplers or vibration resulting from hammering. In general, sampling is likely to cause loss in shear strength due to breaking bonds between particles. However, when the remaining cementation is adequate to maintain the initial void ratio, settlement responses in terms of the position of the wetted stress/strain curve are unlikely to be significantly affected. Results of studies on sampling disturbance of cemented collapsible soils indicated that differences between block and tube specimens are too small to warrant continued use of the more expensive and difficult-to-obtain block samples for collapse testing of cemented soils. Although block samples are probably the best samples that can be obtained, they are not generally perfect because of selective sampling of the most-cemented sections that occurs during trimming of specimens. It is recommended that tube samples be used for obtaining laboratory specimens for engineering practice. It was also concluded that the effects of sample tube area ratio were negligible up to 56% for the cemented soils studied; and effects of sample length up to 305 mm were negligible, for 70 mm diameter specimens. Likewise, the effects of hammering versus pushing were found to be insignificant, unless the cementation is very light or nonexistent. It is recommended that when sampling cemented soils for collapse testing that tube samples may be driven, if convenient, and that the wall thickness of the tube be as small as practical, but large enough to successfully sample soils without highly frequent bending of the sample tubes.

Partial-Wetting Considerations

Laboratory and field infiltration studies show that the degree of saturation a collapsible soil attains during infiltration of ponded water may be much less than 100%. Partial wetting results in partial collapse which is linked to the reduction in soil suction caused by wetting. The effect of partial wetting on collapse potential has been investigated in the laboratory for several collapsible soils. The percentage of full collapse resulting from the anticipated maximum degree of wetting for the field situation may be expected to range from about 40 to 85% of full collapse, depending on the degree and type of cementation of the soil. For many field conditions, the degree of saturation may actually be much less than that which would be achieved by soils near a pond of water, and thus the actual strain potential may be lower than 40% of full collapse strain, even negligible in some cases. Partial wetting considerations are extremely useful in interpretation of field performance of collapsible soils and in selection of mitigation alternatives and methods. Figure 1 depicts typical partial-wetting response of collapsible silty soils. It is evident that collapse settlements increase as the soil matric suction decreases. This is because a major contributor of the soils "cementation" is negative pore water pressure (suction).

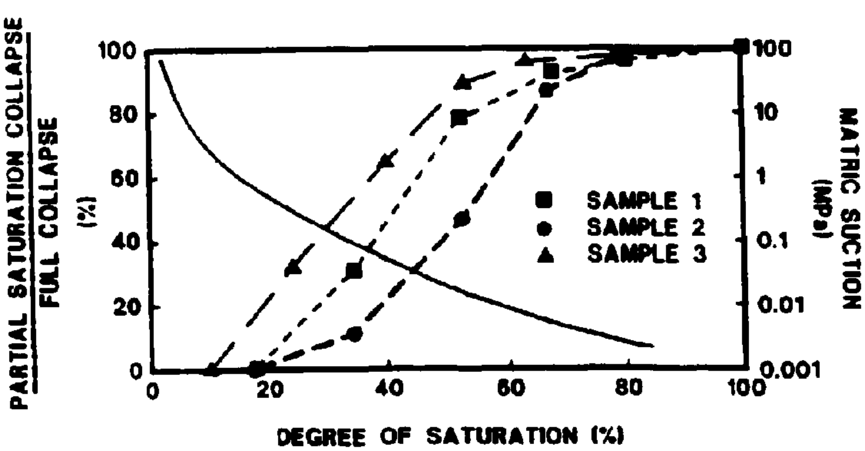

Figure 1. Partial Collapse Due to Partial Wetting for Three Collapsible Silts

Field Testing of Collapsible Soils

Laboratory and *in situ* tests for response-to-wetting each have their advantages and disadvantages. Laboratory tests can be performed with conventional apparatus, and the state of stress and strain for the laboratory specimen are relatively uniform and easy to evaluate. However, laboratory testing has the disadvantage of sample disturbance (particularly for lightly cemented soils), difficulty in sampling gravelly and uncemented soil, and typically higher degree of wetting compared to feild conditions. *In situ* collapse tests normally consist of some type of plate load test, wherein water is introduced to the loaded soil. Advantages of the *in situ* test include minimization of sampling disturbance, attainment of a degree of wetting that is likely to be similar to the prototype, immediate results in the field, and an ability to test difficult-to-sample materials. The disadvantage of *in situ* testing is primarily associated with the relatively nonuniform state of stress under the plate and difficulty in translating the results to the test to the prototype foundation. The usefulness of an in-situ collapse test depends primarily on our ability to develop reasonable stress-strain relationships for the soil tested. Techniques for conducting and interpreting down-hole plate load tests have been developed. These techniques allow translation of load/settlement data from the *in situ* plate load test to stress/strain curves for predicting field collapse settlement.

Analysis of the down-hole collapse test requires understanding the state of stress and strain in the region of wetting beneath the load plate. At the beginning of a test, the region beneath the load plate is subjected to stress resulting from two sources. The first source is overburden stress. The second is the stress resulting from the loaded plate at the base of the borehole. When water is introduced, the soil becomes softer as suction decreases and cementing agents weaken. The stress state beneath the load plate changes. The wetting affects the contribution of stress from overburden. Also, wetting affects the stress distribution caused by the loaded plate within the region of interest.

Simplified methods for conducting the in-situ test and performing the analysis have been developed. The time required to develop stress-strain relationships from an *in situ* test is actually less that that required to develop curves from laboratory response-to-wetting data. The in-situ collapse test allows for immediate detection of collapsible soils in the field, to permit adjustments to the exploration and testing plan in the field during site investigation.

Summary and Conclusions

Because the response-to-wetting under load is often dramatic and normally corresponds to deterioration of the soil's load-carrying capacity, it is common that the wetted condition governs foundation design for unsaturated soil. The degree of wetting which occurs after construction may be essentially complete, as in the case of a rising groundwater table or embankment dam, or only partial, as in the case of typical infiltration of water from a temporary pond or water pipe leak. It is therefore good engineering practice to measure the response-to-wetting under load when conducting site investigation of unsaturated soil sites. Both laboratory and field tests can be used for this purpose.

COOPERATIVE PROGRAMS IN WIND ENGINEERING

Kishor C. Mehta[1]

Abstract

Texas Tech University (TTU) is pursuing multidisciplinary research in wind engineering, which involves internal cooperation between scientists and engineers and external cooperation between institutions. Specifically, two NSF-sponsored cooperative projects are with Colorado State University and the University of Washington; a third project involves faculty from the Colleges of Engineering and Arts and Sciences at TTU. This paper outlines the overall wind engineering program at TTU and describes NSF-sponsored projects.

Introduction

The wind engineering research program at Texas Tech University (TTU) started in May 1970, when a severe tornado struck the city of Lubbock, including the university campus and surrounding area. The tornado killed 26 persons and caused damage exceeding $100 million. Faculty and students spent more than a year documenting and analyzing the structural and building damage inflicted by this tornado. Since then, over the past twenty-five years, personnel of the Institute for Disaster Research (IDR) and the Wind Engineering Research Center (WERC) have documented damage in more than seventy windstorm incidents throughout the country and overseas. Results of significant documentation efforts are published in the open literature and in-house reports; these are listed in the publications list (WERC/IDR, 1994).

In addition to building damage and associated analyses, the wind engineering research program at TTU has expanded to include a wide range of subject areas, including soil erosion, wind dispersion, wind energy, automotive stability and others. A variety of sponsors support the program, including the National Science Foundation, Lawrence Livermore National Laboratory, Insurance Institute for Property Loss Reduction, Florida Housing Finance Agency, Texas Department of Transportation, Texas Department of Insurance, GAF Materials Corporation, and Monsanto Chemical Company. NSF-sponsored projects are described below; a listing of other projects is included.

CSU/TTU CPWE

The NSF-sponsored project, *Colorado State University (CSU)/Texas Tech University (TTU) Cooperative Program in Wind Engineering*, was initiated in 1989. The first five-year program was completed in 1994. A second five-year program, which focuses on preserving the integrity of low-rise buildings in windstorms, has started in 1995.

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The first CPWE project involved eight tasks and twelve Co-PIs from the two institutions, five from CSU and seven from TTU. The eight tasks were: (1) wind loads on low buildings, (2) building ventilation, (3) building roofing, (4) nonboundary layer wind, (5) structural glazing, (6) extreme wind climate, (7) numerical modeling, and (8) wind erosion. The Co-PIs represented the disciplines of civil, mechanical and chemical engineering, as well as atmospheric science. The program effectively used expertise of physical modeling in wind tunnels at CSU and field experiments and experience of TTU. The project has made many contributions to the research infrastructure, including over 100 publications, 30 graduate degrees and participation by 21 U.S. visitors, 35 international visitors, and 40 Research Experience for Undergraduate (REU) students. A list of publications and thesis topics are given in the final report to the NSF (CSU/TTU, 1995). The project was guided by a nine-member Technology Assessment and Advisory Council (TAAC); the membership comprised wind engineers, a meteorologist, practicing professionals and individuals from the roofing, building and insurance industries. Dr. J. Eleonora Sabadell of the Natural and Man-Made Hazard Program was the program director for NSF. Some of the tangible practice-oriented contributions of this project include development of a new 3-second gust design wind speed map for ASCE 7-95, upgrading of internal pressure coefficients in the national standard, development of a computer model to assess uplift on roof pavers, establishment of a field wind research laboratory, and development of software for the insurance industry.

The second CPWE project, 1995-2000, is a focused and integrated research effort toward securing the integrity of low buildings in high winds. Cooperative research focuses on three areas: (1) wind loads, (2) wind engineering meteorology, and (3) wind flow around buildings. These elements will merge to maintain the mixture of full-scale field experiments and wind tunnel physical modeling that enhance the application of the results to the wind effects on buildings. Each area activity will complement the others, both within the contexts of full-scale measurements and wind tunnel modeling. Joint measurements of wind loads, background flow conditions (wind micrometeorology), and local wind fields (wind flows) are necessary to assure use and integration of all measurements. Co-PIs for this project are Robert N. Meroney , Jack Cermak and Bogusz Bienkiewicz from CSU and Kishor C. Mehta, Richard E. Peterson and Partha P. Sarkar from TTU. A complementary research area of economic analysis of wind damage will be pursued by Hal Cochrane at CSU, and the promotion of technology transfer effort will be guided by Dr. James R. McDonald at TTU. These eight Co-PIs, representing the disciplines of engineering, atmospheric science and economics, will follow an integrated approach to research. A new TAAC is established to provide guidance to the project. Dr. J. Eleonora Sabadell continues as NSF program director.

A five-year research plan for the second CPWE is established. The first TAAC meeting will be held in April 1995 at Texas Tech University.

Hurricane Damage to Constructed Facilities

A cooperative project sponsored by the NSF between Texas Tech University (TTU) and the University of Washington (UW) is continuing to develop a predictive model for wind-induced damage to buildings. This project is a part of the Engineering Research for Coastal Zone initiative. Dorothy Reed, Professor of Civil Engineering at UW is experienced in expert systems, object-oriented programming, and decision support systems. Kishor C. Mehta, Professor of Civil Engineering at TTU, provides knowledge on wind-induced damage, while Thomas English, Assistant Professor of Computer Science at TTU, provides the knowledge of intelligence system architecture, functional models, and uncertain inference. The Co-PIs make this project cross-disciplinary, cross-departmental, and cross-institutional in nature.

Building vulnerability to wind is, by nature, an uncertain, subjective, and ill-defined process. A WIND system architecture is developed as shown in Figure 1. The main system retrieves the information on frame members and envelope (cladding) from a database. In addition, it drives wind load computation, frame analysis and envelope analysis. The system processes input, performs analysis, and predicts gradation of damage in terms of removal of covering, breach of envelope, and collapse of frame. The components of frame analysis and wind loads computation are reasonably well established. A paramount problem is the database and analysis models for the building envelope. A building envelope (roof and wall cladding) has large variations in materials, configurations and fastenings. It will be necessary to use a subjective knowledge base for envelope analysis. At the same time, the main system should be capable to accept replacement of the knowledge base with an analytical solution, when available. The results of the project will be a WIND system that can permit development of upgrade in the future.

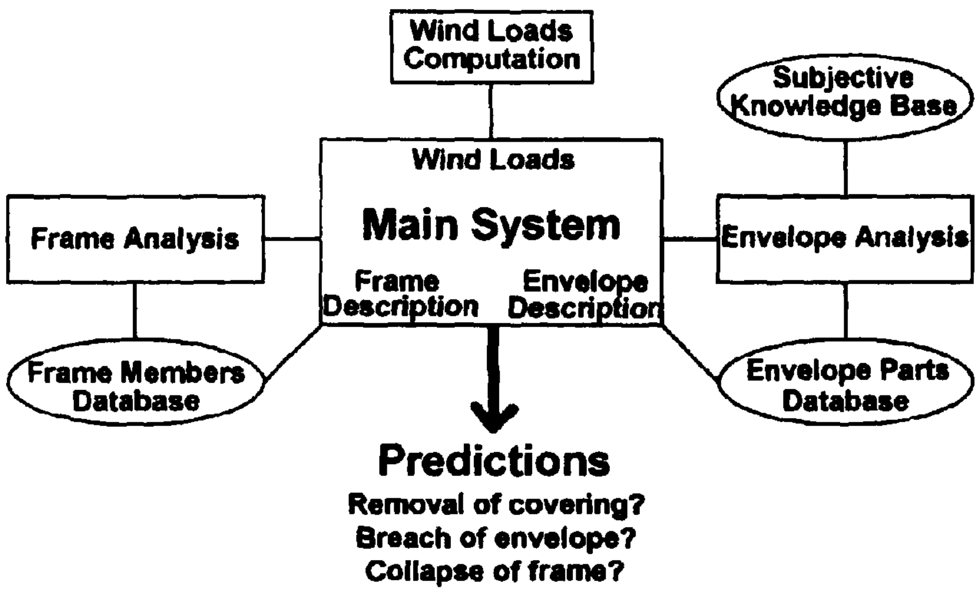

Figure 1. WIND System Architecture

Transfer of Wind Engineering Research to Curriculum

The third NSF-sponsored project involves development of multi-media modules that can help in the educational process. The TTU wind engineering program has a twenty-five year history. A large amount of wind-induced damage investigation data exists, as well as establishment of a good understanding of windstorms, wind-structure interaction and building performance. Multi-media modules are produced using the faculty expertise from the disciplines of civil and mechanical engineering and atmospheric science at TTU. Specific modules planned during the eighteen month project are: (1) Tornadoes and Hurricanes, (2) Damages Caused by

Hurricanes and Tornadoes, (3) Impact of Windborne Debris, and (4) Wind Loading on Low-Rise Buildings.

The videos, along with the color slides and text, will be assembled in CD-ROM modules that can be used as instructional aids in different courses of the curriculum of civil engineering, mechanical engineering, and atmospheric science. These instructional aids will be at first used at Texas Tech University and will be later sent out to other universities offering similar courses like Texas Tech. The primary thrust is to upgrade the undergraduate curriculum. However, modules will be designed for use in a few introductory graduate courses. These instructional aids will also be useful in short courses for professionals and practicing engineers as part of the continuing education program. Further, these aids will be sent out to the insurance, consulting and construction type industries. It is noted that these instructional aids will be used as supplementary lecture materials in the existing courses. The project is scheduled for completion in 1995.

Wind Engineering Program at TTU

The multidisciplinary wind engineering program at TTU involves 12 faculty members, 3 research associates, 3 staff members, 20 graduate students, and as many undergraduate students. In addition to the NSF-sponsored projects, current projects are as follows:

- *Dynamic Response of Tied Arch Bridges;* Texas Department of Transportation
- *Mitigation of Galloping of Signal Light Structures;* Texas Department of Transportation
- *WINDRITE™: Building Categorization for Wind Damage;* Insurance Institute for Property Loss Reduction
- *Wind Load Standards Compliance Methodologies;* Florida Housing Finance Agency
- *Wind Testing of Air Permeable Mat;* GAF Materials Corporation
- *Metal Edge Flashing Wind Pressures;* Roofing Industry
- *Wind Engineering/Fluid Mechanics Research;* State of Texas and TTU
- *Alternative Energy Source;* Central and Southwest Utilities Corporation
- *VORTEX: Field Investigation of Tornado Wind Fields;* NSSL/NOAA/NSF

Specialized Facilities

Specialized facilities to conduct wind engineering research include the Wind Engineering Field Research Laboratory (WERFL), tornado missile impact facility, wind library, wind tunnel, and tow tank.

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