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Impact Assessment of Selected MCEER Highway Project Research on the Seismic Design of Highway Structures

by

C. Rojahn, R. Mayes, D.G. Anderson, J.H. Clark,
D'Appolonia Engineering, S. Gloyd and R.V. Nutt

Applied Technology Council
555 Twin Dolphin Drive, Suite 550
Redwood City, California 94065-2102

Technical Report MCEER-99-0009

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C. Rojahn¹, R. Mayes², D.G. Anderson³, J.H. Clark⁴,
D'Appolonia Engineering⁵, S. Gloyd⁶ and R.V. Nutt⁷

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- 1 Principal Investigator, Applied Technology Council
- 2 Co-principal Investigator and Project Director, Dynamic Isolation Systems, Inc.
- 3 Project Team Member, CH2M Hill
- 4 Project Team Member, Consulting Engineer
- 5 Project Team Member
- 6 Project Team Member, Parsons Brinckerhoff
- 7 Project Team Member, Consulting Structural Engineer

MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
University at Buffalo, State University of New York
Red Jacket Quadrangle, Buffalo, NY 14261

Preface

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

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

MCEER's research is conducted under the sponsorship of two major federal agencies, the National Science Foundation (NSF) and the Federal Highway Administration (FHWA), and the State of New York. Significant support is also derived from the Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

The Center's FHWA-sponsored Highway Project develops retrofit and evaluation methodologies for existing bridges and other highway structures (including tunnels, retaining structures, slopes, culverts, and pavements), and improved seismic design criteria and procedures for bridges and other highway structures. Specifically, tasks are being conducted to:

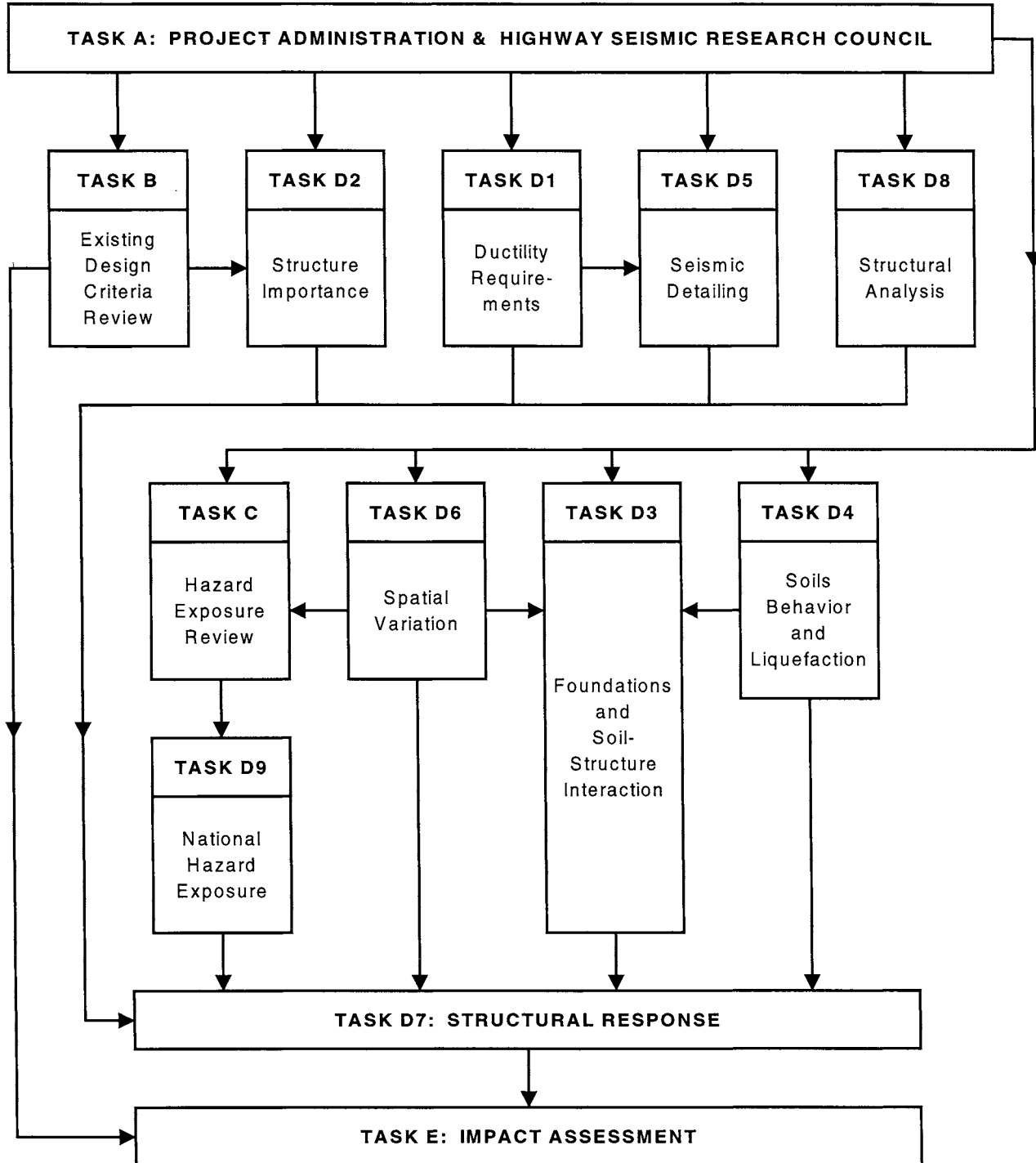
- assess the vulnerability of highway systems, structures and components;
- develop concepts for retrofitting vulnerable highway structures and components;
- develop improved design and analysis methodologies for bridges, tunnels, and retaining structures, which include consideration of soil-structure interaction mechanisms and their influence on structural response;
- review and recommend improved seismic design and performance criteria for new highway structures.

Highway Project research focuses on two distinct areas: the development of improved design criteria and philosophies for new or future highway construction, and the development of improved analysis and retrofitting methodologies for existing highway systems and structures. The research discussed in this report is a result of work conducted under the new highway structures project, and was performed within Task 112-E, "Impact Assessment and Strawman Guidelines for the Seismic Design of Highway Bridges" of that project as shown in the flowchart on the following page.

The overall objective of this task was to perform an independent review of the results obtained from each of the studies conducted under the four-year MCEER research project on the "Seismic Vulnerability of New Highway Construction" (Project 112) and to assess the potential impact of

these results on future seismic design specifications for highway structures. This effort was conducted over an 18-month period as the final task of the project. The Applied Technology Council reviewed 32 MCEER research reports and developed recommendations regarding future bridge seismic design guidelines. The results of the research impact assessment and specification recommendations are documented in this report. A previously published report (NCEER-97-0002) reviewed and assessed current domestic and foreign seismic design criteria for new highway construction.

SEISMIC VULNERABILITY OF NEW HIGHWAY CONSTRUCTION
FHWA Contract DTFH61-92-C-00112



FOREWORD

In 1992, the Multidisciplinary Center for Earthquake Engineering Research (MCEER), formerly the National Center for Earthquake Engineering Research (NCEER), in conjunction with the Applied Technology Council (ATC) and several other institutions under contract to MCEER, commenced the MCEER Highway Project. The MCEER Highway Project consisted of two projects sponsored by the U.S. Federal Highway Administration (FHWA). Both projects included research studies on the seismic vulnerability of U.S. highway construction, including bridges, tunnels, retaining structures, slopes, and embankments. The goal of the first project was to develop methods and manuals for seismic risk analysis of highway systems and for the screening, evaluation, and retrofit of highway bridges, tunnels, retaining structures, embankments, culverts, and pavements. The goal of the second project was to conduct a series of special research studies, which will provide a basis for the future development of improved seismic design specifications for new highway construction.

ATC, as one of the subcontractors on the second project (FHWA contract DTFH61-92-C-00112), had responsibility for two tasks. The first was to review and assess current domestic and foreign seismic design criteria for new highway construction (Project ATC-18/MCEER Task 112-B). This work was documented in previously published MCEER and ATC reports (Rojahn et al., 1997; ATC, 1997). The second task was to review the results of studies conducted under the four-year-long MCEER Project 112 research project to assess the potential impact of these results on future seismic design specifications for highway structures, primarily bridges. During this effort, which was conducted over an 18-month period at the end of the MCEER Highway Project, ATC reviewed 32 MCEER research reports and developed recommendations for future bridge seismic design guidelines ("strawman" guidelines). The results of the research impact assessment and the strawman guidelines are documented in this ATC-18-1 report.

The research impact assessments cover a wide variety of seismic design topics: ground motion issues, such as site factors, site categories, spatial variation, vertical motions, and development of inelastic design spectra; determining structural importance; foundations and soils; liquefaction mitigation methodologies; modeling of pile footings and drilled shafts; damage-avoidance design of bridge piers, column design, modeling, and analysis; structural steel and steel-concrete interface details; abutment design, modeling, and analysis; and detailing for structural movements in tunnels. Each research impact assessment includes a research summary, research findings and review comments, a list of related reports and research, limitations of the research, and an assessment of the potential impacts on future seismic design specifications.

The recommendations for improved seismic design specifications for bridges (strawman guidelines) address a broad range of issues: performance criteria, design approach, seismic loading, design forces, analysis, design displacements, concrete and steel details, foundation design, and several miscellaneous items.

The research impact assessments were prepared by the ATC-18-1 Project Team, which consisted of Ronald L. Mayes (Project Director), Donald Anderson, John H. Clark, Stewart Gloyd, and Richard V. Nutt, with assistance provided by a geotechnical consultant, D'Appolonia Engineering. Review and comment were provided by the research investigators and a Project Engineering Panel, consisting of Ian Buckle, Roy Imbsen, John Mander, Newland Malmquist (ATC Board Representative), Geoffrey Martin, Maurice Power, M. Shinozuka, and W. Phillip Yen. A. Gerald Brady coordinated the impact assessments and compiled this report. Technical editing and report production services were provided, respectively, by Nancy Sauer and Peter Mork. The affiliations of these individuals are provided in the list of project participants.

Applied Technology Council gratefully acknowledges the funding provided by FHWA and the support and cooperation provided by Ian M. Friedland, MCEER Assistant Director for Transportation Research.

Christopher Rojahn (Principal Investigator)
ATC Executive Director

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SECTION 1 INTRODUCTION

1.1 Background

In 1992, with funding from the Federal Highway Administration (FHWA), the Multidisciplinary Center for Earthquake Engineering Research (MCEER), formerly the National Center for Earthquake Engineering Research (NCEER), commenced a research project (FHWA contract DTFH61-92-C-00112) to identify ways to reduce the seismic vulnerability of new highway construction. The objective of this 4-year project (MCEER Project 112) was to perform a comprehensive study of the seismic vulnerability of highways, bridges, tunnels, and retaining structures, in order to develop technical information on which new seismic design specifications could be based in the future. It is anticipated that current specifications for the seismic design of bridges will be revised and that new seismic design guidelines will be prepared for other highway system components, in part on the basis of this work.

This project was one of two initiated around the same time under the title of the MCEER Highway Project. The second project under this effort (MCEER Project 106) is also sponsored by the FHWA (FHWA contract DTFH61-92-C-00106), and consists of studies focussed on reducing the seismic vulnerability of the existing national highway system.

The MCEER project on new highway construction was prompted in part because significant progress had been made over the last two decades in several key areas: (1) knowledge on seismic risk data throughout the United States, (2) geotechnical engineering, and (3) seismically resistant design. At the time MCEER Project 112 was initiated, however, there were still many gaps in basic knowledge and some of the recently-developed information and data required additional study before they could be applied directly to highway engineering applications. Consequently, MCEER Project 112 has included both analytical and experimental studies related to the seismic design and performance of bridges, tunnels, and foundations. The project has focused on the following elements:

- Seismic hazard exposure of the U.S. highway system
- Foundation design and soil behavior
- Determining the relative importance of specific bridges
- Structural issues of response and analysis
- Structural design issues
- Assessment of the impact of recent research
- Review of design criteria.

The research conducted under MCEER Project 112 had a national focus and was intended in part to address differences in seismicity, bridge types, and typical design details between eastern or central United States bridges and those that have been studied in California and the western United States. In particular, unlike the western United States, design strategies used in the eastern and central United States need to reflect the statistical probability that an earthquake significantly larger, that is, up to four times larger, than the "design" earthquake can occur. Critically important bridges on vital transportation routes and those providing access to emergency care facilities have been studied to evaluate their performance following both design earthquakes and large, infrequent earthquakes (popularly termed "maximum considered earthquakes"). In many cases, California design practice requires modification before being implemented in the eastern and central United States.

A range of studies was carried out that encompasses research on: seismic hazard; foundation properties, soil properties, and soil response; and the response of structures and systems. These studies were

conducted by a consortium of researchers, coordinated by MCEER. The consortium included a variety of academic institutions and consulting engineering firms, bringing together more than 25 earthquake and bridge engineers and scientists. This consortium provided a balance between researchers and practicing professionals from the eastern, central, and western United States.

Technical issues in MCEER Project 112 have been grouped into several primary subject areas, as shown in table 1-1. This table also lists the research task numbers within each primary subject area. Each of these research tasks is related to new highway construction and the relationships between tasks are identified in figure 1-1.

MCEER Project 112 research is fairly broad and covers a number of important technical areas. Table 1-2 contains a listing of all tasks that were initiated under the project. The results of research from this

Table 1-1 Subject Areas and Research Tasks in the MCEER Highway Project for New Construction (MCEER Project 112)

Subject Areas	Research Tasks
Seismic Hazard and Ground Motion	C, D6, D9
Design Issues	
• foundations and soils	D3, D4
• structures and special issues	D2, D7, D8
• design issues and details	D1, D5
Design and Performance Criteria	B, E
Project Management and Administration	A, F

project are intended to provide the basis for developing new design criteria and specifications, particularly for bridge structures. Secondary products include task reports and synthesis reports describing the advances made in design for other highway transportation systems and components.

Approximately five years have elapsed since Project 112 began. At the time of this writing, the results of many project tasks have been documented in a series of MCEER technical reports, while others are in various stages of the review and publication process. Project reports have been grouped into the following categories:

- Seismic hazard exposure and bridge performance
- Structural analysis, design, and response
- Foundations and soil/structure interaction
- Liquefaction and soil behavior.

1.2 Purpose and Scope of the ATC-18-1 Project

The Applied Technology Council commenced MCEER Task 112-E (ATC-18-1 project) to assess the effect of research, completed under MCEER Project 112, on the future seismic design of highway structures. During the ATC-18-1 project, a team of bridge design and geotechnical engineers with extensive knowledge of current bridge and highway structure seismic design practice and specifications reviewed 32 MCEER research reports.

SEISMIC VULNERABILITY OF NEW HIGHWAY CONSTRUCTION
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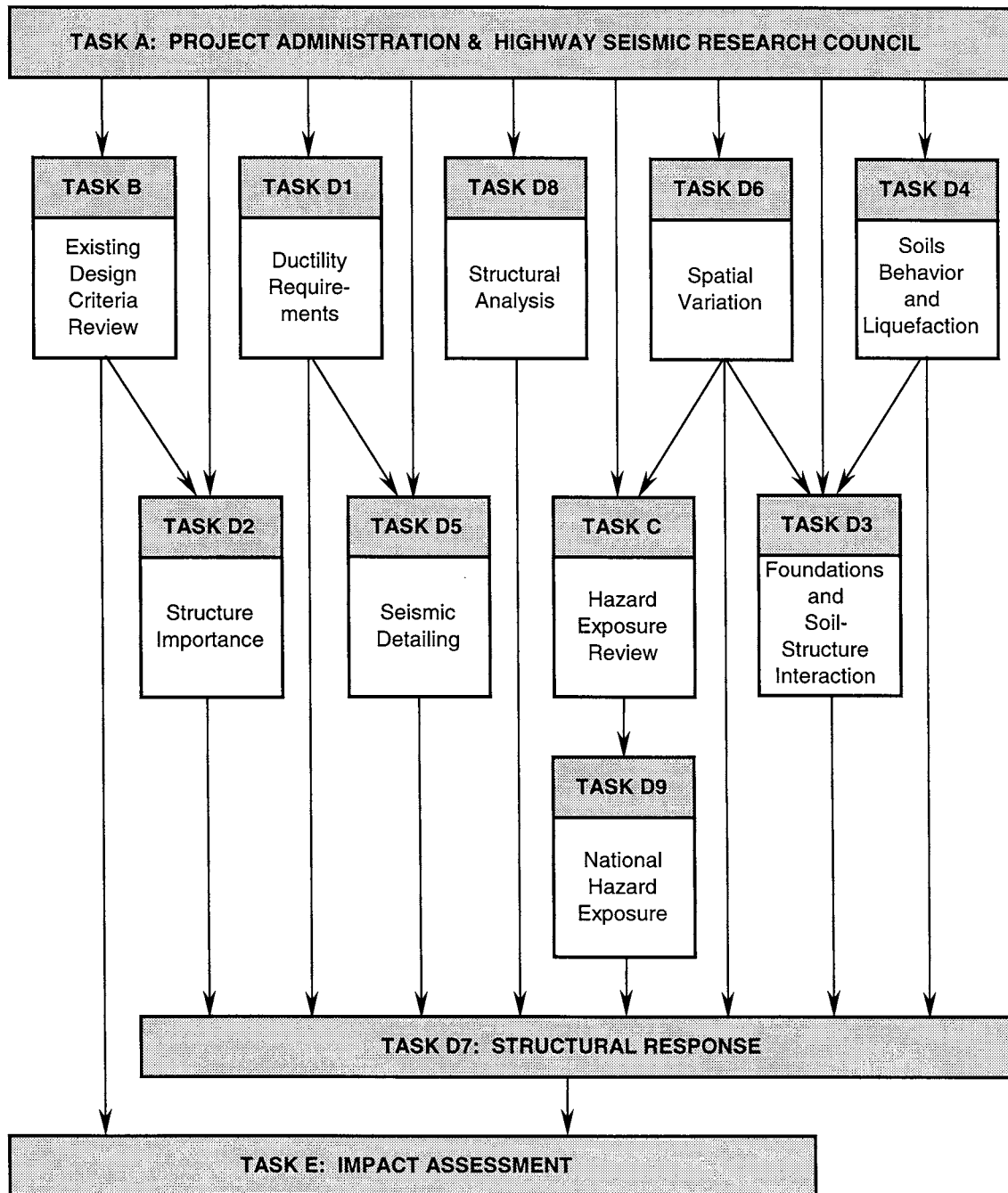


Figure 1-1 Relationships Between the Tasks of MCEER Project 112 — New Highway Construction

Table 1-2 Research Tasks Initiated Under MCEER Project 112

Area/Task No.	Task Description	Principal Investigator(s)
Project Administration and Reporting		
112-A	Project Administration & Support for the Highway Seismic Research Council	I. Buckle/I. Friedland
112-F	Project Reporting	I. Friedland
Design and Performance Criteria		
112-B	Review Existing Design Criteria and Philosophies	C. Rojahn/R. Mayes/ I. Buckle
112-E	Impact Assessment and Strawman Guidelines for the Seismic Design of Highway Bridges	C. Rojahn/R. Mayes/ I. Buckle
Seismic Hazard and Exposure		
112-C	Compile and Evaluate Maps and Other Representations, and Summarize Alternative Strategies for Portraying the National Hazard Exposure of the Highway System	M. Power
Ductility Requirements		
112-D1.1(a)	Establish Representative Pier Types for Comprehensive Study – Eastern U.S.	J. Kulicki
112-D1.1(b)	Establish Representative Pier Types for Comprehensive Study – Western U.S.	R. Imbsen
112-D1.2	Physical and Analytical Modeling to Derive Overall Inelastic Response of Bridge Piers	J. Mander
112-D1.3	Derive Inelastic Design Spectra	R. Imbsen
Structure Importance		
112-D2	Evaluation of Structure Importance	J. Kulicki
Foundations and Soil-Structure Interaction		
112-D3.1	Compile Data and Identify Key Issues	I.P. Lam
112-D3.2	Abutment and Pile Footing Studies by Centrifuge Testing	R. Dobry
112-D3.3	Develop Analysis and Design Procedures for Abutments	I.P. Lam/G. Martin
112-D3.4	Develop Analysis and Design Procedures for Retaining Structures	R. Richards/ K. Fishman
112-D3.5	Develop Analysis and Design Procedures for Pile Footings	I.P. Lam/G. Martin
112-D3.6	Develop Analysis and Design Procedures for Drilled Shafts	I.P. Lam/G. Martin
112-D3.7	Develop Analysis and Design Procedures for Spread Footings	G. Gazetas

Area/Task No.	Task Description	Principal Investigator(s)
112-D3.8	Performance and Sensitivity Evaluation, and Guideline Development	P. Lam/R. Dobry/ G. Martin
Soil Behavior and Liquefaction		
112-D4.1	Site Response Effects	R. Dobry
112-D4.2	Identification of Liquefaction Potential	T.L. Youd
112-D4.3	Development of Liquefaction Mitigation Methodologies	G. Martin
112-D4.4	Design Recommendations for Site Response and Liquefaction Mitigation	G. Martin
Special Seismic Detailing		
112-D5.1(a)	Capacity Detailing of Columns, Walls, and Piers for Ductility and Shear – Analytical Studies	J. Mander
112-D5.1(b & c)	Capacity Detailing of Columns, Walls, and Piers for Ductility and Shear - Review Proposed Design Details	R. Imbsen/J. Kulicki
112-D5.1(d)	Capacity Detailing of Columns, Walls, and Piers for Ductility and Shear - Experimental Studies	M. Saiidi
112-D5.2(a)	Capacity Detailing of Members to Ensure Elastic Behavior	J. Mander
112-D5.2(b)	As above	R. Imbsen
112-D5.2(c)	As above	J. Kulicki
112-D5.3(a)	Detailing for Structural Movements - Bridges	R. Imbsen
112-D5.3(b)	As above	J. Kulicki
112-D5.3(c)	Detailing for Structural Movements - Tunnels	M. Power
112-D5.4	Structural Steel and Steel/Concrete Interface Details	J. Kulicki
Spatial Variation of Ground Motion		
112-D6	Spatial Variation of Ground Motion	M. Shinozuka/ G. Deodatis
Structural Response		
112-D7	Effects of Vertical Acceleration on Structural Response	M. Button
Structural Analysis		
112-D8	Review Existing Analytical Methods, and Identify and Recommend Analytical Procedures Appropriate for Each Structure Category and Hazard Exposure	I. Buckle/J. Mander
Seismic Hazard Exposure		
112-D9	Recommended Approach and Further Development for Portraying the National Hazard Exposure	M. Power

Table 1-3 lists the Project 112 research reports reviewed. As indicated in the table, some of the reports also pertain to tasks in Project 106 on existing construction. The purpose of the reviews was to identify research findings that could provide a basis for improving seismic design criteria and other specifications for highway structures, particularly bridges.

Issues addressed in the reviews and impact assessment included:

- Appropriate seismic hazard definition for use in a nationwide design code
- Different performance levels for bridges of different importance
- Foundation behavior under seismic loading and the interaction between soil and structure
- Mitigating the effects of soil liquefaction
- Attaining required ductility for sufficient capacity under various seismic demands
- Special seismic detailing for reinforced concrete
- Structural analysis methods using the latest available software
- Special technical issues for bridges and tunnels
- General content and format of future seismic design specifications
- Application of cost/benefit considerations in future seismic design specifications

Results from the impact assessments were also used by the ATC-18-1 Project Team to develop recommended improvements for future seismic design specifications for highway bridges. It is anticipated that these recommendations (informally referred to as “strawman” guidelines) will serve as a starting point for developing the next generation of seismic provisions in the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications*. They are also intended to be beneficial in the development of seismic design guidelines for other types of highway transportation structures.

1.3 Report Organization

This report is the primary product of the ATC-18-1 Project. Following this introductory section, Section 2 summarizes the reviews of each of the MCEER Project 112 Technical Reports. These summaries address primarily those research results that have sufficient impact to be considered for inclusion in the next generation of nationally applicable AASHTO seismic design specifications for highway bridges.

Section 3 contains the complete text of the reviews of the MCEER Project 112 Technical Reports, resulting from the tasks performed under that project. Each research impact assessment includes a research summary, research findings and review comments, a list of related reports and research, limitations of the research, and an assessment of the potential impacts on future seismic design specifications. The introduction to Section 3 describes the process followed in conducting the reviews and impact assessments.

Recommendations regarding future AASHTO seismic design specifications are provided in Appendix A.

Table 1-3 Summary of the MCEER Project 112 Research Reports Reviewed under the ATC-18-1 Project

Author(s)	Title	Technical Report No. MCEER-	Project-Task No.
Seismic Hazard, Exposure, and Bridge Performance			
Edited by: I.M. Friedland/ M.S. Power/ R.L. Mayes	Proceedings of the FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities	97-0010	112-C 112-D9 106-F5.4.1
R. Dobry/ R. Ramos/ M. Power	Site Factors and Site Categories in Seismic Codes	99-0010	112-D4.1 106-E2.9
M. Shinozuka/ G. Deodatis	Effect of Spatial Variation of Ground Motion on Highway Structures	*	112-D6 106-E2.2/2.5
M.R. Button/ C.J. Cronin/ R.L. Mayes	Effect of Vertical Ground Motions on the Structural Response of Highway Bridges	99-0007	112-D7
A. Thomas/ S. Eshenaur/ J. Kulicki	Methodologies for Evaluating the Importance of Highway Bridges	98-0002	112-D2
Structural Analysis, Design and Response			
J.H. Kim/ M.R. Button/ J.B. Mander/ I.G. Buckle	Application of Simplified Methods of Analysis to the Seismic Design of Bridges	*	112-D8
J. Kulicki/ Z. Prucz	Establish Representative Pier Types for Comprehensive Study: Eastern U.S.	96-0005	112-D1.1(a)
R.A. Imbsen/ R.A. Schamber/ T.A. Osterkamp	Establish Representative Pier Types for Comprehensive Study: Western U.S.	96-0006	112-D1.1(b)
J.B. Mander/ C.T. Cheng	Seismic Resistance of Bridge Piers Based on Damage Avoidance Design	97-0014	112-D5.1(a) 112-D5.2(a)
C.T. Cheng/ J.B. Mander	Seismic Design of Bridge Columns Based on Control and Repairability of Damage.	97-0013	112-D1.2 112-D5.1(a) 112-D5.2(a)
A. Dutta/ J.B. Mander	Capacity Design and Fatigue Analysis of Confined Concrete Columns	98-0007	112-D5.1(a)
J.B. Mander/ A. Dutta/P. Goel	Capacity Design of Bridge Piers and the Analysis of Overstrength	98-0003	112-D5.2(a)
G.A. Chang/ J.B. Mander	Seismic Energy Based Fatigue Damage Analysis of Bridge Columns: Part I – Evaluation of Seismic Capacity	94-0006	112-D1.2 106-E5.2
G.A. Chang/ J.B. Mander	Seismic Energy Based Fatigue Damage Analysis of Bridge Columns: Part II – Evaluation of Seismic Demand	94-0013	112-D1.2 106-E5.2

Author(s)	Title	Technical Report No. MCEER-	Project-Task No.
N. Webbe/ M. Saiidi/ D. Sanders/ B. Douglas	Ductility of Rectangular Reinforced Concrete Bridge Columns with Moderate Confinement	96-0003	112-D5.1(d)
R.A. Imbsen/ R.A. Schamber/ M. Quest	Capacity Detailing of Members to Ensure Elastic Behavior	*	112-D5.2(b)
P. Ritchie J.M. Kulicki	Capacity Detailing of Members to Ensure Elastic Behavior – Steel Pile-to-Cap Connections	*	112-D5.2(c)
P. Ritchie/ N. Kauh/ J. Kulicki	Structural Steel and Steel/Concrete Interface Details for Bridges	98-0006	112-D5.4
R.A. Imbsen/ R.A. Schamber/ E. Thorkildsen/ A. Kartoum/ B.T. Martin/ T.N. Rosser/ J.M. Kulicki	Structural Details to Accommodate Seismic Movements of Highway Bridges and Retaining Walls	97-0007	112-D5.3(a & b)
W.D. Liu/ R. Imbsen/ X.D. Chen/ A. Neuenhofer	Derivation of Inelastic Design Spectrum	*	112-D1.3
M. Power	Summary and Evaluation of Procedures for the Seismic Design of Tunnels	*	112-D5.3(c)
Foundations and Soil/Structure Interaction			
I.P. Lam	Foundations and Soils – Compile Data and Identify Key Issues	*	112-D3.1
A. Gadre/ R. Dobry	Centrifuge Modeling of Cyclic Lateral Response of Pile-Cap Systems and Seat-Type Abutments in Dry Sand	98-0010	112-D3.2
I.P. Lam/ G. Martin	Modeling of Abutments in Seismic Response Analysis of Highway Bridges	*	112-D3.3
K.L. Fishman/ R. Richards	Seismic Analysis and Design of Bridge Abutments Considering Sliding and Rotation	97-0009	112-D3.4
I.P. Lam/ M. Kapuskar/ D. Chaudhuri	Modeling of Pile Footings and Drilled Shafts for Seismic Design	98-0018	112-D3.5 112-D3.6
G. Gazetas/ G. Milonakis/ A. Nikolaou	Development of Analysis and Design Procedures for Spread Footings	*	112-D3.7
I.P. Lam/ G. R. Martin/ M. Kapuskar	Synthesis Report on Foundation Stiffness and Sensitivity Evaluation on Bridge Response	*	112-D3.8

Author(s)	Title	Technical Report No. MCEER-	Project-Task No.
Liquefaction and Soil Behavior			
Edited by: T.L. Youd/ I.M. Idriss	Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils	97-0022	112-D4.2 106-E3.2(b)
G. Martin	Development of Liquefaction Mitigation Methodologies / Ground Densification Methods	*	112-D4.3
G. Martin	Design Recommendations for Site Response and Liquefaction Mitigation	*	112-D4.4

*Unpublished report

SECTION 2 SUMMARY OF RESEARCH AND IMPACT ASSESSMENTS

The reports developed as part of the MCEER/FHWA research program on new highway construction covered a wide range of topics. Some were based on the Proceedings of workshops on major areas of interest while others covered specific issues. Many of the reports address important issues that need to be considered in future seismic design codes, whereas others provide design procedures and computer programs that will be useful as design aids to the profession. Still others provide information that will be useful as background information to the design profession. For the purposes of this document the reviewed research reports have been grouped into four main categories.

2.1 Seismic Hazard, Exposure, Bridge Performance, and Structural Importance

Proceedings of the FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities, edited by I.M. Friedland, M.S. Power and R.L. Mayes (1997). These proceedings document the findings of a forum of more than 50 researchers and design professionals who met to present papers and discuss issues relating to national representation of seismic ground motion for new and existing highway facilities. Consensus recommendations were developed on six issues. The *Proceedings* is a valuable resource for design code developers because it covers several issues in depth and presents consensus recommendations derived after discussion by the entire group. Specific recommendations are given regarding seismic mapping and site factors, and clear positions are stated supporting the implementation of provisions that cover vertical motion, near-source effects, and spatial effects to highway facility seismic codes.

Site Factors and Site Categories in Seismic Codes, by R Dobry, R Ramos and M Power (1999). This study addresses the amplification of earthquake ground motion by local soil conditions and provides new definitions of site categories and site coefficients. These recommendations were incorporated in the 1994 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995) and, more recently, in the 1997 *Uniform Building Code* (ICBO, 1997). The site categories and site coefficients are compared with those in previous codes, including the current AASHTO specifications. Ground-motion recordings from the 1994 Northridge and the 1995 Kobe earthquakes generally validate the recommended values. The recommended site factors given in this report should be incorporated into the AASHTO seismic design specifications simultaneously with a change in the shape of the long-period segment of the response spectra from a $1/T^{2/3}$ to a $1/T$ relationship.

Effect of Spatial Variation of Ground Motion on Highway Structures, by M. Shinozuka and G. Deodatis (*unpublished report*). A method for generating spectrum-compatible time histories is introduced, which accounts for traveling seismic waves, loss of coherency with distance, and differing soil conditions. The method reproduces complicated motion such as the 1964 Niigata earthquake ground motion recording, which showed an abrupt change in frequency content at the onset of liquefaction. The method is used to examine the response of several realistic bridges with total lengths varying from 34 m (112 feet) to 483 m (1584 feet). The number of spans varied from three to ten, and the longest span ranged from 13 m (44 feet) to 63 m (208 feet). Both linear and nonlinear time-history analyses were conducted. The results are expressed as the ratio of response quantities considering spatial variation to the same quantities with no spatial variation, i.e., identical support motions. The ratio of maximum moment to yield moment of the columns, displacement values, and joint openings and closings were considered. The angle of incidence of the seismic waves and vertical acceleration were less important parameters. Identical support motion is recommended for representing unskewed bridges under 305 m (1000 ft) in length with at least two spans and uniform soil conditions; more conservatively, the seismic

response coefficient is increased by 10% or the R factor decreased by 20%. It is recommended that time-history analyses be performed using spatial variation, with several scenario earthquakes, and several different values of wave velocity when any of the following are true: the bridge is over 305 m (1000 ft) in length, the bridge has supports on different local soil conditions, or the bridge is severely skewed.

Effects of Vertical Ground Motions on the Structural Response of Highway Bridges, by *M. Button, C. Cronin and R. Mayes (1999)*. This study addresses the question: How critical is the vertical component of seismic ground motion in determining the demands placed on key components of highway structures? Current bridge design codes do not require consideration of vertical motion effects, and few bridges have been designed considering this effect. Available ground motion records show that vertical acceleration can exceed horizontal acceleration. This study considers all of the important issues affecting the design, thus providing new insight into design considerations that have up to now been handled by assumption or approximation, or overlooked due to lack of information. The major impact of the report is that vertical acceleration can be a critically significant factor in the seismic response of highway bridges, and should be appropriately covered by design codes and practices. This report contains data and guidance that are practical and can be readily put into practice. It will help designers of structures close to major faults by showing what structural and foundation factors are of most significance with respect to vertical motion, aiding the optimization of structure configuration. The report also offers simple design procedures for most cases. It will aid the development of codes for bridge design, as it offers many recommendations that can be used in a design-specification format. It will aid researchers by pointing out areas that need further study.

Methodologies for Evaluating the Importance of Highway Bridges, by *A. Thomas, S. Eshenaur and J. Kulicki (1998)*. This investigation used a number of existing and proposed numerical methods to evaluate the importance of bridges. The methods were evaluated by taking a representative sample of bridges, having a group of qualified persons independently select the 20 most important bridges from this group, averaging the opinions, and comparing the averaged results with the results generated from the various numerical ranking methods. The best numerical methods were selected and further modified to give better correlation. Based on this, a modified version of the Montana method known as MTN5 was developed. This method uses the National Bridge Inventory (NBI) for data on both the route carried and the route crossed. The method seems best suited for making retrofitting decisions for existing bridges. Calculation of a numerical importance rating may be useful, but it alone should not dictate the decision of whether or not a bridge should be designed for a higher level of seismic performance. Such a subjective decision is influenced by the incremental cost increase of strengthening a bridge. This incremental cost, as a percentage of the base construction cost, may vary among jurisdictions. It is difficult to arrive at a numerical method that fits all situations, and therefore the method is better placed in a commentary.

2.2 Structural Analysis, Design and Response

Application of Simplified Methods of Analysis to the Seismic Design of Bridges, by *J.H. Kim, M.R. Button, J.B. Mander and I.G. Buckle (unpublished report)*. This study addresses simplified methods that may be used to develop limitations and recommendations for design. The uniform-load method, the single-mode method, and multi-mode spectral analysis are compared. Curvature, span-length ratios, pier height, skew, and span connectivity are discussed as they relate to the use of the simplified methods. A set of 27 continuous bridges with integral abutments defined the characteristics of regular bridges for which the simplified methods of analysis are applicable. These recommendations will modify the minimum analysis requirements.

Establish Representative Pier Types for Comprehensive Study: Eastern U.S., by *J. Kulicki and Z. Prucz (1996)* and **Establish Representative Pier Types for Comprehensive Study: Western U.S.**, by *R.A. Imbsen, R.A. Schamber, and T.A. Osterkamp (1996)*. These studies, which identify representative pier types in the eastern and western United States, are useful for the design specification development process, pointing out current design practices and typical detailing. Provisions can be written to cover the various cases, or steps can be taken to discourage undesirable details and practices.

Seismic Resistance of Bridge Piers Based on Damage Avoidance Design, by *J.B. Mander and C.T. Cheng (1997)*. This is the first of two reports on new design concepts. This design approach ensures that plastic hinges do not develop in the columns, thereby avoiding loss of service after a significant earthquake. Rocking piers are forced to rotate at their ends, but are restrained from toppling through gravity and the optional use of central, unbonded, post-tension reinforcement in the core of the columns. This design procedure could be incorporated in the commentary to a code for designers who wish to explore it. It is not sufficiently developed to become part of a design code. Once some of the existing limitations have been resolved, the design approach will need to be compared with others, such as seismic isolation, to establish its cost-effectiveness. One advantage of the concept is that prefabricated columns can be used, reducing total construction time and costs. From a constructability perspective, the concept may be attractive in areas where prefabricated construction minimizes the traffic disruption that would be associated with conventional construction.

Seismic Design of Bridge Columns Based on Control and Repairability of Damage, by *C.T. Cheng and J.B. Mander (1997)*. This second report (see above) developed and tested construction details in reinforced concrete columns that are intended to provide a replaceable or renewable sacrificial plastic hinge zone or fuse. Hinge zones are deliberately weakened with respect to the adjoining components; all regions outside a hinge zone are detailed to be stronger than the hinge zone and to remain elastic during seismic loading. The special detailing of the hinge zone permits relatively quick repair of the earthquake damage. Repair of the damaged hinge zones permits use of the bridge for at least minimum levels of traffic after a major earthquake. This work will not have a direct impact on code provisions, although it is an important design concept that will reduce the downtime of bridges after an earthquake. The design concept should be in the commentary of any future performance-based code. Compared with conventional construction, the hinge zone construction will have a higher initial cost. Repair costs following a major earthquake will be less. Moreover, in the event of a large damaging earthquake that would render a conventionally built structure irreparable, a structure designed with hinge zones might still be repaired with its service life extended indefinitely.

Capacity Design and Fatigue Analysis of Confined Concrete Columns, by *A. Dutta and J.B. Mander (1998)*. In this study, transverse hoop fracture is explicitly predicted, based on energy balance principles. The objective was to develop a complete design procedure for columns to eliminate all undesirable modes of failure, including buckling of the longitudinal bars. Transverse reinforcement requirements will either increase or decrease relative to current requirements depending on the axial load and the longitudinal steel reinforcement ratio. These recommendations will be useful additions to the current requirements.

Capacity Design of Bridge Piers and the Analysis of Overstrength, by *J.B. Mander, A. Dutta, and P. Goel (1998)*. This report determines the moment overstrength capacity of reinforced bridge columns for use in capacity design. The upper-bound overstrength factors tend to validate some prescriptive overstrength factors such as those in the ATC-32 report (ATC, 1996) and to indicate that such factors in other specifications, (e.g., the California Department of Transportation and AASHTO) may sometimes be too low. These prescriptive factors can be overly conservative for some columns. The moment-

curvature method developed in this study could also be used for design. It would aid in the development of design specifications. The effect of uncertainty in material and geometric properties needs to be evaluated so that the appropriate load and resistance factors can be developed for reliability-based seismic design codes.

Seismic-Energy-Based Fatigue-Damage Analysis of Bridge Columns: Part I – Evaluation of Seismic Capacity, by *G.A. Chang and J.B. Mander (1994)*. Part I of this study resulted in a computer program, UB-COLA, which is capable of accurately predicting the behavior of reinforced concrete columns subjected to inelastic cyclic deformations. The axial, flexural, and shear cyclic behaviors are modeled as well as the low-cycle fatigue properties of reinforcing bars and high-strength, prestressing steel bars. The program was capable of predicting the failure mode of either low axial-load columns (low-cycle fatigue of longitudinal reinforcement) or high axial-load columns (fracture of confining reinforcement and crushing of concrete). For shear-critical columns, the cyclic inelastic behavior was simulated through the cyclic inelastic strut and tie-modeling technique. Although this work will not have a direct impact on codes, it forms a key element in the implementation of the pushover method of analysis. The computer program UB-COLA, developed as part of this research, will become a valuable tool for design offices.

Seismic-Energy-Based Fatigue-Damage Analysis of Bridge Columns: Part II – Evaluation of Seismic Demand, by *G.A. Chang and J.B. Mander (1994)*. A smooth, asymmetric, degrading, hysteretic model (Takeda) is presented that is capable of accurately simulating the behavior of bridge columns. The parameters for the analytical model are determined automatically by using a system-identification routine. The model was integrated into a single-degree-of-freedom (SDOF) inelastic dynamic analysis program and a significant number of nonlinear analyses were performed, which resulted in design recommendations regarding the assessment of fatigue failure in reinforcing steel. The report includes a proposed methodology for the seismic evaluation of bridge structures that incorporates the traditional strength and ductility aspects plus the fatigue demand on reinforcing steel. The current code use of force-reduction factors that are independent of natural period are not conservative for short-period stiff structures and may lead to fatigue failure of the reinforcement. Recommendations are made for minimum values of force-reduction factors that prevent fatigue failure in the reinforcement. The design procedure would, as a minimum, be appropriate for inclusion in a code commentary to indicate how energy, and in particular low-cycle fatigue effects, can be accounted for in the design process.

Ductility of Rectangular Reinforced Concrete Bridge Columns with Moderate Confinement, by *N. Webbe, M. Saiidi, D. Sanders and B. Douglas (1996)*. Detailing guidelines are developed for reinforced concrete bridge columns and walls in areas of moderate seismicity. The report suggests that current confinement requirements may be relaxed in areas of moderate and low seismicity.

Capacity Detailing of Members to Ensure Elastic Behavior, by *R.A. Imbsen, R.A. Schamber, and M. Quest (unpublished report)*. This is primarily a compendium of 1994 California Department of Transportation (Caltrans) practices. Four areas are covered that have been identified by Caltrans as requiring new procedures for design. The four areas are joint shear in the connection of cap beam to column, superstructure flexural capacities that force plastic hinging in the columns, footings, and outrigger and knee joint connections.

Capacity Detailing of Members to Ensure Elastic Behavior – Steel Pile-to-Cap Connection, by *P. Ritchie and J. M. Kulicki (unpublished report)*. This report discusses the connections between steel piles and concrete pile caps that should remain elastic during earthquakes. Both axial-load and moment-resisting connections are explored. Limited, common-sense, design guidelines are presented, but some

issues are not addressed, and further review is required before incorporation in a code. The connections could well be standardized when the work is complete, and therefore not add to design costs. For the achievement of desirable performance, it is important to avoid brittle failures in these underground connections.

Structural Steel and Steel/Concrete Interface Details for Bridges, by *P. Ritchie, N. Kauhle and J. Kulicki (1998)*. This report assesses the seismic performance of details associated with steel bridge towers extending from a massive concrete substructure to the superstructure, as well as the seismic performance of other steel substructure and superstructure details for new construction. Seismic related work on steel bridges has lagged that of concrete bridges. The report summarizes the information available and suggests mechanisms within a steel bridge that can act as energy-dissipating components.

Structural Details to Accommodate Seismic Movements of Highway Bridges and Retaining Walls *R.A. Imbsen, R.A. Schamber, E. Thorkildsen, A. Kartoum, B.T. Martin, T.N. Rosser and J.M. Kulicki (1997)*. This report addresses details for bridges and retaining structures in the eastern and western United States and develops seismic design recommendations for these details based on the need to accommodate structural movements. These recommendations and details are to be used as a basis for developing improved bridge design standards. The report includes many illustrations of the devices used to accommodate structural movements. Advantages and disadvantages of some of the devices are noted. Examples of approaches used in specific states are also given.

Derivation of Inelastic Design Spectrum, by *W. D. Liu, R. Imbsen, X. D. Chen and A. Neuenhofer (unpublished report)*. The research was to have developed inelastic response spectra for nationwide use, allowing engineers to assess inelastic deformations and thereby design for improved seismic performance. Not all of the stated objectives of the research were achieved, and the report does not explain how the methodology would incorporate the large amount of data prepared by the USGS on the nationwide seismic hazard. The use of such data is essential in any new practical, simplified approach to generating inelastic design spectra. There is potential for code impact in future generations of the design code, but significantly more research needs to be done. Such research should be coupled with development of analysis and design procedures that can make proper use of inelastic spectra.

Summary and Evaluation of Procedures for the Seismic Design of Tunnels, by *M. S. Power, D. Rosidi, J. Kaneshiro, S. D. Gilstrap, and S.-J. Chiou (unpublished report)*. This report reviews the seismic evaluation and design of three types of tunnels: bored, cut-and-cover and submerged. The information could be used in preparing code requirements for tunnels. The racking of cut-and-cover tunnels appears to be the seismic response most in need of careful attention.

2.3 Foundations and Soil/Structure Interaction

Foundations and Soils – Compile Data and Identify Key Issues, by *I.P. Lam (unpublished report)*. This report on geotechnical, abutment, and foundation issues provides results of a survey of state transportation agencies regarding typical foundations and abutments in their existing bridge inventories. The primary purpose of the survey was to identify foundation systems commonly used in bridge design within the United States. This information was intended to provide background information for other research studies being conducted as part of the MCEER-sponsored research program. While the primary focus of the survey was to identify typical foundation systems, the survey also had two secondary objectives. The first of these was to provide a preliminary assessment of procedures that might be used for screening the seismic vulnerability of existing bridges. The second was to identify major foundation design issues that warrant consideration during the MCEER-sponsored research program.

Centrifuge Modeling of Cyclic Lateral Response of Pile-Cap Systems and Seat-Type Abutments in Dry Sand, by A. Gadre and R. Dobry (1998). The translational response of pile-cap foundations and seat-type abutment walls during seismic loading was investigated. The project had two primary objectives: (1) to understand the lateral response of pile-cap foundations and seat-type abutments and (2) to verify current design procedures used to estimate stiffness and capacity of these elements. Of specific interest was the contribution of the cap to the lateral-load capacity of a pile-cap foundation system, and whether additional rules can be used to account for lateral-load resistance contributions from the pile and footing. A second area of interest was the effective damping of pile-cap systems and abutment foundations. Results from this test program were interpreted to provide valuable guidance involving the relative contributions of a single pile and the pile cap to lateral-load resistance of the structure. Methods for determining abutment wall capacity and stiffness are also discussed.

Modeling of Abutments in Seismic Response Analysis of Highway Bridges, by I.P. Lam and G. Martin (unpublished report). The objectives of the report were limited to (1) clarifying the process of design for service loads versus design for seismic loading, (2) reviewing abutment modeling alternatives, and (3) providing a simplified approach for design that still incorporates key issues affecting abutment response. The report addresses the task objectives by investigating the relevant abutment design issues that affect seismic performance. The primary focus of the investigation was passive loading. The research effort included numerical modeling using simplified and rigorous methods. The research showed that calculated abutment forces can differ significantly, as a function of the modeling methods. The period of a bridge model is affected by the abutment model, potentially resulting in longer periods of vibration, more displacement, and reduced forces. The current AASHTO seismic design specifications provide guidance for the design of abutment walls under active loading conditions. In some situations, passive loading conditions will be more critical for abutment design. New AASHTO seismic design specifications should be expanded to address passive loading. Considering the complexity of the passive loading case, a detailed commentary covering methods for determining passive capacity and stiffness is needed.

Seismic Analysis and Design of Bridge Abutments Considering Sliding and Rotation, by K.L. Fishman and R. Richards, Jr (1997). This report provides a new procedure for determining the earthquake-induced displacement of retaining walls and bridge abutments founded on spread footings. The new procedure differs from existing displacement-based procedures for determining the sliding response of bridge abutments by addressing mixed-mode behavior, which includes both rotation due to bearing capacity movement and sliding response. The procedure also extends existing methods for estimating sliding and rotation by introducing a pinned-restraint condition at the top of the retaining wall and by accounting for reductions in bearing capacity caused by seismic loading. The procedure for predicting permanent (mixed-mode) displacements was calibrated against test cases that were modeled in the laboratory. The boundary conditions at the top of the abutment were varied during the test program to include sliding, rotation-about-the-top, and mixed sliding and tilting. Procedures for using this new approach in seismic design are described. A computer program for estimating sliding and rotational displacements is included in an appendix. Information developed as a result of this work is in a form that could be easily integrated into new AASHTO seismic design specifications. However, conclusions from independent numerical analyses by Gazetas and others were sufficiently different from those reached in this report that some type of resolution on the appropriate approach should be reached before the method is adopted.

Modeling of Pile Footings and Drilled Shafts for Seismic Design, by I.P. Lam, M. Kapuskar and D. Chaudhuri (1998). Seismic design methods used to represent pile foundations and drilled shafts for bridge structures were evaluated. The project had the following objectives: (1) to evaluate the influence

of modeling methods on response of the pile footings, (2) to establish the influence of modeling methods on the estimated displacement and force demand, (3) to summarize methods for characterizing the stiffness of pile footings and the response of drilled shaft foundations, and to discuss their limitations; and (4) to provide guidelines on seismic design practice. The focus of the first part of the report is on pile-group foundations rather than single-pile extensions. Information in this section gives a practical summary covering the state-of-the-practice for the seismic design of pile foundations. The second part of the report focuses on drilled shafts, and the primary focus is on lateral loading. This section provides a practical summary of the state-of-the-practice. A key contribution is that differences in design procedures relative to those used for driven piles are addressed. These differences are related to the installation procedure, drilled-shaft versus driven-pile diameter, the length-to-diameter ratio, and the structural configuration.

An important feature of the report is that it attempts to provide an interface between the structural and geotechnical design processes. The current AASHTO seismic design specifications do not cover most of the topics included in this report. For this reason, it is recommended that information in this report be incorporated, as appropriate, into the new AASHTO seismic design specifications.

Development of Analysis and Design Procedures for Spread Footings, by *G. Gazetas, G. Milonakis and A. Nikolaou (unpublished report)*. Five issues are addressed: (1) when to incorporate foundation stiffness in the dynamic analysis of bridge piers; (2) the significance of properly modeling the effect of embedment on the dynamic stiffness of the foundation; (3) the importance of radiation damping and kinematic interaction in response; (4) conditions under which uplift becomes significant, including how it is modeled in design and analysis; and (5) the significance of local soil nonlinearities under the edges of a rocking foundation and methods to account for it in the analysis. The report includes interesting observations regarding the response of a spread footing foundation system during seismic loading. These observations deal with the importance of soil-structure interaction, embedment, and radiation damping to the overall system response. Observations are also made regarding seismic bearing capacity and footing uplift. The results, as currently developed and presented, cannot be easily adapted into code provisions. The parametric study of the seismic response of footings without uplift essentially covers a single combination of soil, structural type, and earthquake loading. The validity of these results for other combinations of soils, structures, and earthquake characteristics needs to be assessed. Some results regarding the seismic effects on bearing capacity differ from those of other researchers. The differences should be documented and reconciled. With additional evaluation and further documentation of the method of analysis, the results of this task could be adopted into new codes.

Synthesis Report on Foundation Stiffness and Sensitivity Evaluation on Bridge Response, by *I. P. Lam, G. R. Martin, G. R., and M. Kapuskar (unpublished report)*. This report discusses bridge response for different values of abutment stiffness and bent foundation stiffness. The sensitivity study considered a typical two-span bridge with the center bent supported by four different foundation systems: piles, spread footings, pier wall, and drilled shafts. The examples of foundation stiffnesses developed here will be useful in commentary on future codes, even though some California consultants are already using the design procedures. The procedures call for close coordination between the design and geotechnical engineers, particularly for flexible-base conditions.

2.4 Liquefaction and Soil Behavior

Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, edited by *T.L. Youd and I.M. Idriss (1997)*. The objectives of the workshop were to review recent developments in the simplified procedure for evaluating the liquefaction resistance of soils and to gain consensus on further improvements and additions that should be incorporated in the simplified procedure. This

workshop was the first since a similar workshop was sponsored by the National Research Council (NRC) in 1985. Emphasis was placed on developments that had been published after the 1985 NRC workshop. Workshop participants focused their review and discussions on procedures used to predict the triggering of liquefaction. They further limited their discussions to shallow soil deposits on level or nearly level ground. From these discussions the workshop participants developed consensus recommendations on: (1) use of standard and cone penetration tests, (2) use of shear-wave velocity measurements, (3) use of Becker penetration test for gravelly soils, (4) magnitude scaling factors, and (5) correction factors K_σ and K_α . The workshop participants also addressed issues relating to earthquake magnitude and peak ground acceleration. Liquefaction criteria based on probabilistic and seismic energy methods were also considered.

Development of Liquefaction Mitigation Methodologies / Ground Densification Methods, by *G. Martin (unpublished report)*. Densification procedures currently being used to mitigate the potential for ground liquefaction and the associated hazard were evaluated. The objective was to compile the results of a number of liquefaction remediation-related tasks performed as part of the MCEER Highway Project into a form that can be used for the development of codes and guidelines. The work included in this compilation can assist in the selection of optimum available liquefaction mitigation or ground improvement methods using ground densification. The methods addressed in this compilation can be used at bridge sites where a high potential for liquefaction is identified during site investigations. The report is useful for identifying more efficient design methods, which will lead to lower unit costs for ground improvement. Furthermore, the information on the mechanisms governing the mitigation process should lead to better efficiency and higher reliability in design. Key components of the information presented in this report could either be incorporated in the new AASHTO seismic design specifications or included as a provisional commentary in an appendix.

Design Recommendations for Site Response and Liquefaction Mitigation, by *G. Martin (unpublished report)*. This report synthesizes research work and current practice related to those seismic design problems involving the influence of site soils on earthquake ground response and the identification and mitigation of ground liquefaction. The objective of the task was to compile results of recent MCEER-sponsored research into a form that can be used to develop seismic design guidelines and codes. This synthesis report provides fundamental information about the effects on bridge structures of ground acceleration and displacement-induced loading caused by liquefaction. It separates the presentation into the areas of site response without liquefaction and ground deformation with liquefaction. Because the current AASHTO guidelines for seismic design use response spectra and site-response factors that are outdated, procedures summarized in this synthesis report can be used to update these methods to be consistent with the standards of the profession. With regard to two-dimensional effects, a decision still needs to be made whether future AASHTO codes should address the potential for these effects. It is unclear from the existing information whether simplified procedures can be established for identifying when these effects should be considered, and if so, what methods should be used to determine these effects. Information from this report should provide an excellent basis for developing guidelines and commentaries on liquefaction for the new AASHTO seismic design specifications. The site response and liquefaction design recommendations in this report could also result in some additional requirements for the design engineer. These additional design requirements are believed to be very valuable in that they will provide a common basis for engineers to use when performing their analyses and making their design decisions.

SECTION 3 REVIEWS AND IMPACT ASSESSMENTS

This section contains the complete reviews of 32 reports prepared under MCEER Project 112 in accordance with the review process described below. In some cases, reviewed reports were published final reports; in others, draft final task reports were reviewed. If the report has been assigned an MCEER Technical Report No. (in the form NCEER- or MCEER-yy-nnnn) as of the date this ATC-18-1 report was prepared, this number is listed; otherwise the report is identified as “unpublished report.” It should be noted that not all task reports will be published by MCEER. In the course of the research, some tasks have resulted in more than one report on topics related to the main task topic. The task numbers listed in this document attempt to reflect the correlation between task and report, when applicable.

Each review contains the following sections:

1. A research summary
2. Research findings and review comments. This section includes the principal findings of the research (usually presented in bulleted format) and review team comments/assessments (in italics). The review cycle for the comments included an opportunity for the authors to comment on the reviewers’ assessments.
3. Related research and reports. These are mostly other MCEER tasks and technical reports from the Highway Project, but may include the works of others.
4. Limitations of the research results or conclusions, insofar as the impact on seismic design is concerned.
5. Impact assessment. This section addresses one or more of the following issues:
 - Code requirements: Are the results ready for inclusion in codes or the commentary to codes, or not yet ready?
 - Design procedures: Do the research results lead to new design procedures?
 - Design cost: Would the procedures described or developed result in additional costs in the design office?
 - Structure performance: How would structures perform if designed according to a code that had incorporated the recommendations of this research?
 - Construction cost and constructibility: How would these be affected?
 - Research recommendations of the report authors and the review team.

The ATC-18-1 project team shared the responsibility for the reviews and impact assessments of the technical reports. At least two individuals were assigned primary responsibility for reviewing and assessing each technical report. All members of the project team, however, had the opportunity for input to the review of each report. In most cases the review and impact assessment were sent to MCEER for transmission to the report authors for their comments. When necessary, the reviewers took steps to include the gist of the authors’ comments in the final version of the review. Prior to publication, the project team and the Project Engineering Panel were provided with the opportunity to critique each review and impact assessment.

In some cases, research project tasks were jointly supported and funded by more than one task within this project, or via the two contracts comprising the MCEER Highway Project (new construction contract DTFH61-92-C-00112 and existing construction contract DTFH61-92-C-00106). Assessments note all tasks under which the particular report was produced.

3.1 Proceedings of the FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities
Edited by I.M. Friedland, M.S. Power and R.L. Mayes.
Technical Report NCEER-97-0010
Task Nos. 112-C, 112-D-9 and 106-F-5.4.1

3.1.1 Research Summary

The research leading to these Proceedings explored several important issues regarding representation of seismic hazard and ground motions for the design of highway facilities throughout the United States, and made recommendations for use of the ground-motion representations in nationally applicable design guidelines and specifications such as those published by AASHTO.

3.1.2 Research Findings and Review Comments

The Proceedings document the results of a workshop held in May 1997 in Burlingame, California, that provided a forum for more than 50 experts in this field to present papers, hold discussions, and develop consensus recommendations on six critical issues pertaining to representation of seismic ground motion for new and existing highway facilities nationwide. Following are workshop summary discussion points, conclusions, and recommendations for each issue:

Issue A: Use of new U. S. Geological Survey (USGS) maps as a basis for highway facility design.

- These maps, which are probabilistically based, depict peak ground accelerations and selected spectral accelerations for return periods of approximately 500, 1000, and 2500 years. The development of these maps by a team of experts included complete examination and re-interpretation of historical data. Increased activity is shown in some Pacific Ocean coastal areas, and generally reduced activity in eastern states.
- The maps are readily available from the USGS web site at: <http://geohazards.cr.usgs.gov>
- The workshop participants showed confidence in the process used to develop the new maps. They concluded that the USGS maps are a major step forward and should be used in place of current AASHTO maps.
- The participants also recommended that a return period longer than 500 years be considered for design level ground motions.
- An important new aspect of the USGS work is the deaggregation of the seismic hazard for key U.S. locations.

Issue B: Use of energy and duration in a design procedure

- Many engineers now believe that energy or duration of ground motion should be considered in the procedures for design or evaluation of structures because cumulative damage effects of the cyclic response can be better represented.
- The workshop concluded that some measure of energy is important for modeling bridge response but that there is currently no well-developed and accepted design procedure to account for it. Ground motion input that reflects energy and duration can be readily developed, but a valid structural damage model is key to developing design procedures.
- The workshop felt that energy, rather than duration, is the fundamental parameter that should be considered.

- Research should be considered to develop and validate suitable energy-based methods that can be used to supplement current response-spectrum-based design.

Issue C: Characterization of site effects

- The workshop recommended that the improved site factors developed and accepted into the 1994 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995) and the 1997 *Uniform Building Code* (UBC) (ICBO, 1997) be proposed for highway facilities design.
- The soil profile type is made solely dependent on one parameter, the average shear-wave velocity of the top 30 meters of soil.
- Site coefficients in the current *NEHRP Recommended Provisions for New Buildings* and UBC are now dependent on intensity of rock motions, and are larger than the previous factors for low-seismicity regions.
- A short-period amplification site coefficient is introduced that did not previously exist. Separate factors are therefore recommended for use in the short- and long-period parts of the response spectrum.
- The workshop recommended that these new factors be coupled to $1/T$ rather than $1/T^{2/3}$ (T = building period), to better represent the long-period branch of the spectrum.

Issue D: Consideration of vertical ground motion

- Vertical ground motion, not currently addressed by the AASHTO specifications, has been observed and recorded during past events and may be particularly high in the near-source region.
- The structural response to vertical acceleration can be significant. Peak vertical acceleration is traditionally set at $2/3$ of the peak horizontal acceleration, but the $2/3$ fraction is not technically justified. At short periods the vertical component may significantly exceed $2/3$, and at long periods it may be much less.
- Structural design concerns that may be affected by vertical ground acceleration include superstructure shear and flexure demands, the effect of uplift on column shear capacity, and combined directional effects.
- The workshop recommended that specific design procedures to account for vertical motion be developed for bridge design.

Issue E: Consideration of near-source ground motion

- Near-source ground motion has unique characteristics, including a single large pulse of long-period motion. This effect is most evident within 15 km of the fault at periods longer than 0.5 seconds.
- The directional relationship between fault and structure is also a factor. For toll bridge retrofit design, the California Department of Transportation (Caltrans) has adopted separate design spectra for ground motion components normal and parallel to the fault.
- Forward-rupturing directivity is more severe, and for scenario earthquakes it can be represented by the mean-plus-one-standard-deviation motion, where the mean represents motion averaged over all directions.
- The workshop recommendation is to use site-specific time-history analyses when considering near-source effects until research can provide simpler procedures suitable for design codes.

Issue F: Consideration of spatial effects

- Parametric analyses have been made showing the effects on typical bridges caused by spatial incoherency, wave passage, attenuation due to distance, and variations in geologic conditions. (Results of evaluating differential geologic conditions within the bridge length were not available at the time of the workshop).
- The analytical study showed that significant increases in structural demand may occur in certain structures due to spatially-varying ground motion, but that ordinary bridges are not greatly affected.
- The workshop concluded that, in general, spatial effects cannot be ruled out. However, further research is needed to define the importance of these effects, and to develop simplified design procedures covering a broad range of bridge configurations.

3.1.3 Related Reports and Research

1. Task 106-E-1.1/1.2: Compile and Evaluate Maps and Other Representations of National Hazard Exposure
2. Task 106-E-1.3: Develop Alternative Strategies for Portraying the National Hazard Exposure
3. Task 106-E-1.4: Evaluate Alternatives and Recommend Approach for Portraying the National Hazard Exposure
4. Task 106-E-1.5: National Hazard Exposure Portrayal and Recommendations
5. Task 106-F-5.4.1: Workshop on National Representation of Seismic Ground Motion of New and Existing Highway Facilities

Many of the workshop presentations were based upon research projects or documents recently completed or in progress. These include the USGS National Seismic Hazard Maps and 1997 *NEHRP Recommended Provisions for New Buildings* and its *Commentary*. Numerous research projects were referenced by the workshop participants and are listed in the proceedings.

3.1.4 Limitations

Although the workshop included highly qualified representatives from many technical specialty areas and a variety of agencies, it did not attempt to represent all parties impacted by seismic design practice. The consensus recommendations reached are therefore not an official position of any entity other than the workshop participants. The recommendations, however, are a strong indication of an appropriate direction for others to consider and to explore in the further development of seismic design guidance for the profession.

3.1.5 Impact Assessment

Code Requirements: The workshop proceedings will be a valuable resource for design code developers, not only by covering several issues in depth, but also by presenting consensus recommendations derived after discussion by the entire group. Specific recommendations are given regarding seismic mapping and site factors, and clear positions are stated supporting the addition of provisions to seismic codes that cover vertical motion, near-source effects, and spatial effects, to the extent supported by available research. Consideration of energy-based provisions for design should be added in the future after additional research and substantiation. The potential use of energy-based design is an opportunity for substantial improvement in the seismic design of highway facilities.

Design Procedures: The addition of design procedures for vertical motion, near-source motion, and spatial effects will add complexity to the design process. Guidelines will be needed to identify those situations that warrant the inclusion of these procedures.

Design Costs: Design costs will tend to increase whenever new design procedures are required.

Structure Performance: The use of improved maps and better site factors will improve the accuracy of the seismic loading input used in design, including the probabilistic and spectral characteristics of ground motion. Better ground-motion representations should improve the reliability of bridge designs. In addition, many aspects of ground motion such as vertical motion, near-source effects, and spatial effects are not specifically addressed in current standard design practice. Adoption of design procedures that address these aspects of ground motion could lead to improved performance of certain bridges.

Construction Cost: The impact on construction costs of adopting the new USGS maps will depend a great deal on the return period adopted for design, and on the design limit state linked to that return period. For many regions in the eastern United States, the new motions for the 500-year return period are less than previous maps had indicated, and thus construction costs could decrease. However, many feel that adoption of a 2500-year return period is more appropriate, given the potential for very large, although infrequent, earthquakes in the eastern United States. Even the use of the 2500-year earthquake for design may not have a significant impact on construction cost for large areas of the eastern United States. Costs will be higher for certain locations having previous damaging earthquakes, such as New Madrid and Charleston. On the west coast, use of the 2500-year maps will generally result in higher construction costs. If adopted in California, bridge construction costs at certain sites may become prohibitive. For this reason, many feel that a deterministic approach should be used in California to limit some of the high peak ground acceleration values predicted by the 2500-year USGS maps. Adoption of NEHRP site factors may increase construction costs for low peak ground acceleration (PGA) sites, since the factors for many typical soil profiles are increased. Design procedures that address vertical motion, near-source effects, and spatial effects could also increase construction costs for some bridges, but the number of these bridges is expected to be limited to those with special structural characteristics and those located in certain regions with special seismic characteristics.

Research Recommendations: The workshop proceedings include a number of recommendations for future research in specific subject areas. These include:

- Develop energy-based design methods that can supplement current elastic response-spectrum-based design methods.
- Continue to refine the current site factors in the *NEHRP Recommended Provisions for New Buildings*.
- For specific bridge types, develop design criteria and procedures that consider the effect of vertical ground motion.
- Evaluate the effects of near-source ground motion on bridge response and incorporate the findings in code design procedures.
- Define the importance of spatial variations of ground motions as a function of bridge characteristics and develop simplified procedures for incorporating the effects of these variations in design.

In addition, there is a need to develop a rational method for selecting the amplitude of ground shaking to be used for design. At the current time, this selection is based on the judgement of code writers, and to a large degree, on political considerations. It is difficult to select a single level of hazard, or acceleration or spectral amplitude, that has the same probability of exceedance for any site in the United States. This is because of the variation across the country in the characteristics (and knowledge of) the seismic

environment. Also, the incremental cost of providing earthquake protection does not vary linearly with the amplitude of ground shaking. These difficulties have been addressed in the 1997 *NEHRP Recommended Provisions for New Buildings* by including the 2500-year ground motion, and by allowing both probabilistic and deterministic methods for design value determination.

Research is needed to establish a better understanding of the relationship between incremental construction costs for improved seismic resistance and the amplitude of seismic ground shaking for typical highway bridges and bridge sites. This information should be incorporated into a cost-benefit study that considers the cost to society of bridge failure during an earthquake. The objective of this research would be to establish a new basis for determining ground motion amplitudes for design. It is possible that this new basis would not use the concept of a uniform nationwide seismic hazard. This study could also lead to a more rational basis for establishing bridge “importance” for the purposes of hardening selected structures.

3.2 Site Factors and Site Categories in Seismic Codes

Dobry, R., Ramos, R., and Power, M.

Technical Report MCEER-99-0010

Task No. 112-D-4.1 (and 106-E-2.9)

3.2.1 Research Summary

The report discusses the amplification of earthquake ground motions by local soil conditions and culminates in new recommended definitions of site categories and site coefficients. These recommendations were incorporated in the 1994 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings*, and more recently in the 1997 *Uniform Building Code*. The recommended site categories and site coefficients are described and compared to previous code provisions, including the current AASHTO specifications for bridges. Results of recent studies of recordings of the 1994 Northridge and the 1995 Kobe earthquakes are discussed. These recordings generally validate the new recommended site coefficients.

The report proposes methods for establishing peak ground acceleration and earthquake magnitude for liquefaction studies. These methods are based on the USGS maps for soil profiles B and C, and deaggregated earthquake magnitude and distance information. Recommendations for further research are also given.

3.2.2 Research Findings and Review Comments

Past and Future Code Recommendations

- Site effects associated mainly with the types and spatial distribution of soils, and also to a certain extent with the ground surface topography, play a very significant role in determining the potential for damage to highway facilities during earthquakes.
- While in some cases bridge damage is due to liquefaction and associated ground failure and large ground displacements, in many others the damage is caused by amplification of the strong ground motions on softer soils, as compared to the motions on rock or stiffer soils.
- The simplified approach now used in AASHTO bridge seismic design specifications has several limitations: (a) definitions used to describe four soil categories, from which amplification factors are determined, are subject to interpretation and do not always define one soil profile for a given

location; and (b) this approach results in severe underestimation of spectral accelerations in areas of low seismicity, such as the eastern United States. The report also suggests the AASHTO Specifications indicate that soil acceleration is assumed to be about the same as rock acceleration, with amplification factors applied only at periods in excess of 0.5 sec. *This limitation does not seem to appear in AASHTO. Moreover, it is common practice to apply the amplification factors for all periods, rather than just periods greater than 0.5 sec.*

- The first of three innovations in the recommended soil factor changes is that the site characterization (i.e., Soil Profile Type) be based only on the top 30 m (100 ft) of soil. The soil profile type is made solely and unambiguously dependent on one parameter, the average shear wave velocity, V_s , of the top 30 m of soil. More readily available soil properties such as the Standard Penetration Test (SPT) blow count or undrained shear strength, are also allowed to characterize the top 30 m of soil for the purpose of assigning a more conservative (higher) site coefficient.
- The second of the three innovations is that a short-period amplification site coefficient (F_a) is introduced, which did not exist before. That is, the one-parameter model of local site amplification characterized by the coefficient S is replaced by a two-parameter model characterized by F_a and F_v . Once the response spectrum on rock is specified through the A values in the AASHTO specifications or Z in UBC, the spectrum on soil is now calculated by using both F_a (which amplifies the short-period part of the rock spectrum in the neighborhood of $T = 0.3$ sec) and F_v (which amplifies the long-period part of the rock spectrum, at periods in the neighborhood of $T = 1$ sec and above). In the current AASHTO specifications, F_a is effectively equal to one, so no soil amplification is provided for at short periods.
- The third of the three innovations is that the effect of soil nonlinearity is introduced by making both site coefficients F_a and F_v functions of both the level of intensity of rock motion (A in the AASHTO specifications or Z in the UBC) and the Soil Profile Type. This should be contrasted with the existing AASHTO code in which the site coefficient S depends only on the site category with no contribution from A .
- The recommended site coefficients are compared with the existing provisions. The site factors incorporated in the AASHTO specifications are, in effect, approximately equivalent to those recommended in the report at higher levels of shaking. The AASHTO values are even more conservative than the recommended values at long periods because of the flatter long-period branch of the spectrum. The difference between the recommended values and the AASHTO site factors is at low levels of shaking, where earthquake data clearly show larger ground amplification by soft soils. Figure 3-1 graphically shows these comparisons.

Data Comparisons

- Reviews of Northridge data, which were recorded for the most part on relatively stiff soil, appear to support the recommended site coefficients. Data from Northridge also confirm that amplification of peak ground acceleration and the F_a amplification factors are about the same (within 30%). Some discrepancies were observed for F_a at sites with soil profile type C.
- Recordings from the Kobe earthquake include near-fault measurements with acceleration levels in excess of 0.4g and measurements on soft soil profiles (i.e., Soil Profiles D and E). Results indicate significant variation in acceleration, which is attributed to radiation pattern/directivity effects. These variations were such that the general trends of de-amplification at high levels of ground acceleration ($\approx 0.6g$) at soft ground sites could not easily be determined, leading to the conclusion that additional research is required to resolve any possible trends.

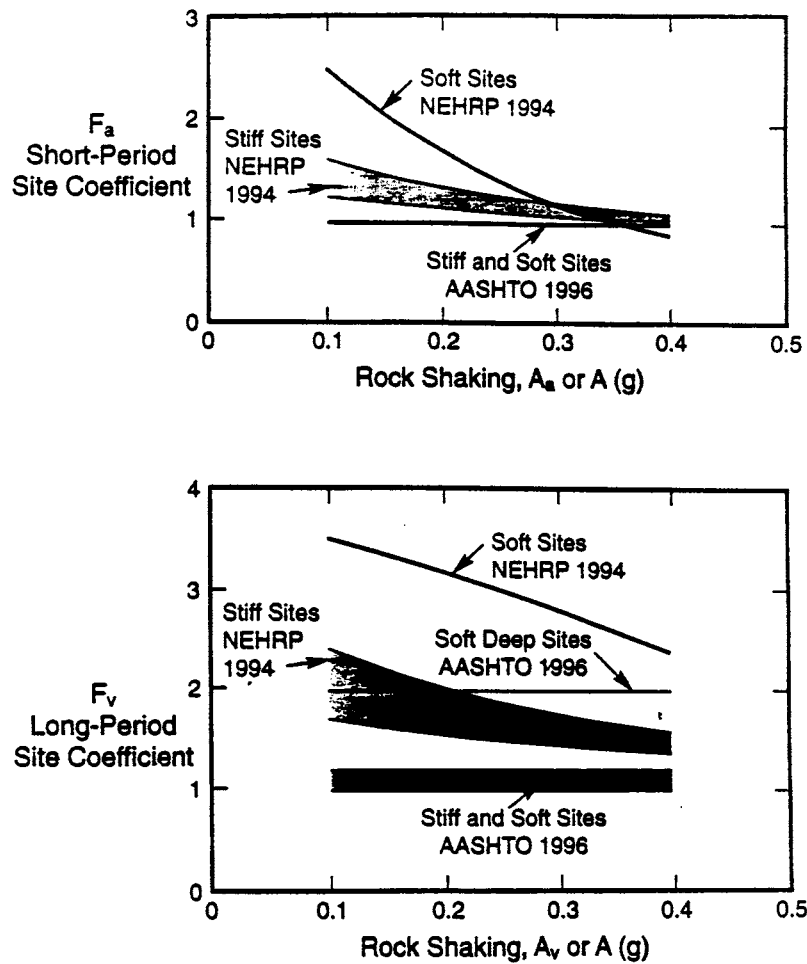


Figure 3-1 Comparisons of Site Coefficients Contained in NEHRP 1994 (BSSC, 1995) and Specifications for Highway Bridges (AASHTO, 1996). From Dobry et al.

Liquefaction

- The report recommends using the F_a values as a basis to convert the mapped acceleration value into the acceleration value on the soil surface for evaluating liquefaction potential. It also recommends selecting the magnitude for liquefaction evaluations by considering only earthquake magnitudes that contribute more than a certain percentage to the ground motion, based on a deaggregation of the probabilistic hazard information.
- For determining liquefaction potential, the report recommends that guidelines recently developed as part of Task 112-D-4.2 be followed and that these guidelines be incorporated into the new AASHTO seismic design specifications.

3.2.3 Related Reports and Research

1. Task 112-D-9: Recommended Approach and Further Development for Portraying the National Hazard Exposure

2. Task 106-E-1.1/1.2: Compile and Evaluate Maps and Other Representations of National Hazard Exposure
3. Task 106-E-1.3: Develop Alternative Strategies for Portraying the National Hazard Exposure
4. Task 106-E-1.4: Evaluate Alternatives and Recommend Approach for Portraying the National Hazard Exposure
5. Task 106-E-1.5: National Hazard Exposure Portrayal and Recommendations
6. Task 106-F-5.4.1: Workshop on National Representation of Seismic Ground Motion of New and Existing Highway Facilities

3.2.4 *Limitations*

- Implicit in the adoption of the 1994 NEHRP/1997 UBC site factors is the acceptance of the acceleration maps. These maps are evolving, and will continue to evolve with time. For example, deterministic limits of ground motion have been included for near-fault locations in high seismic areas (e.g., in California) in the design value maps developed for the 1997 *NEHRP Provisions for New Buildings* (BSSC, 1997). In addition, the maps developed for the 1997 *NEHRP Provisions* reflect longer return periods than previously used. These more recent maps better reflect the state of knowledge and professional consensus.
- Information from Northridge and Kobe events suggests that the 1994 NEHRP/1997 UBC site factors may need to be changed. For example, the recent work of Borchardt (1996) on Northridge data suggests that stiff soils at higher levels of ground motion ($> 0.1g$) are less nonlinear than suggested in the 1994 NEHRP/1997 UBC site factors; his preliminary interpretations for soft soil sites suggest that some modifications could also be required for higher input motions ($> 0.3g$). Another example is a preliminary analysis of peak ground accelerations recorded on rock and soil sites very close to the fault in the 1995 Kobe earthquake that failed to show the expected de-amplification of acceleration on soft ground at levels typically in excess of $0.4g$ at rock and other stiff sites.
- Varying degrees of uncertainty are embodied in the use of the 1994 NEHRP/1997 UBC site factors. While the 1994 NEHRP/1997 UBC site factors may lessen the uncertainties and eliminate some of the unconservative response estimates relative to existing AASHTO specifications, additional consideration of the implications of these uncertainties may be warranted.

3.2.5 *Impact*

Code Requirements: The recommended site factors given in this report should be incorporated into the AASHTO seismic design specifications as soon as practical. The adoption of the site factors must be done simultaneously with a change in the shape of the long-period segment of the response spectra from a $1/T^{2/3}$ to a $1/T$ relationship. Considerations should also be given to incorporating any modifications to the recommended factors that were made in the 1997 *NEHRP Provisions for New Buildings* (BSSC, 1997).

Design Procedures: The recommended improvements in site factors will have no impact on current design procedures, although the response spectra embodied in some computer programs will need to be changed.

Design Cost: There will be no additional design time involved in this change.

Structural Performance: The recommended site factors will improve the performance of bridges constructed on soft soil sites.

Construction Cost/Constructibility: The recommended site factors will result in higher design forces for bridges on soft soil sites. The increase is higher in areas of low seismicity. As a consequence, it is expected there will be an increase in the cost of construction for bridges on soft soil sites, particularly in areas of low seismicity.

Research Recommendations: Areas requiring further research are listed below.

- Determine the influence of soil and rock properties below 30 m.
- Characterize the amplification of long-period motions at deep sites and amplification of motions from nearby earthquakes at shallow stiff sites.
- Refine the understanding of the effects of soil nonlinearity at stiff sites and at soft sites subjected to very strong motions.
- Characterize two-dimensional and three dimensional and basin effects.
- Characterized the effects of site conditions on ground motion duration, near-fault ground motion, and spatial variation.
- Determine the validity of using F_a for amplification of peak ground motions.
- Characterize site effects for long-period motions ($T > 2$ sec).

3.3 Effect of Spatial Variation of Ground Motion on Highway Structures *Shinozuka, M. and Deodatis, G. (unpublished report)* *Task No. 112-D-6*

3.3.1 Research Summary

The research addressed procedures for determining spectrum-compatible time-histories that represent spatial variations in ground motion, including the effect of different soil conditions. These procedures were used to examine the effect of spatial variability on critical response quantities for typical structures. This analysis is intended to show if the effect on bridge response is significant enough to warrant specific code provisions or design guidelines.

The first part of the research describes in detail a method for generating spectrum-compatible time-histories that can account for traveling seismic waves, loss of coherency with distance, and different soil conditions. This procedure is shown to be able to reproduce complicated motions such as the 1964 Niigata earthquake, which showed an abrupt change in frequency content associated with the onset of liquefaction.

The second part of the research uses these procedures to examine the response of several realistic bridges with total lengths varying from 34 meters to 483 meters, number of spans from three to ten, and longest span from 13 meters to 63 meters. Both linear and nonlinear time-history analyses were conducted. The results were expressed as the ratio of the response quantities considering spatial variation to the same response quantities with no spatial variation; that is, with synchronous motion of all bridge supports. The response quantities studied were (1) the ratio of maximum moment to the yield moment of the columns, (2) displacement values, and (3) joint openings and closings. Results of Monte-Carlo-type analyses are presented. Other factors studied to a lesser degree were the effect of the angle of incidence of the seismic waves and vertical acceleration.

3.3.2 *Research Findings and Review Comments*

The methodology uses a spectral representation to simulate stochastic vector processes having components corresponding to different locations on the ground surface. An iterative scheme is used to generate time-histories compatible with prescribed response spectra, coherency, and duration of motion.

Analysis results for eight example bridges are tabulated showing the relative ductility demand ratio for column flexure due to seismic wave-propagation spatial effects. In general there is about a 10% maximum increase when using linear analysis, and a 25% maximum increase when using nonlinear analysis for bridges up to 305 m in length. *This can be considered within the range of error to be expected, or within the range of uncertainty of input.*

Results are also tabulated showing the relative opening and closing at expansion joints, for bridges with hinges in the superstructure, due to seismic wave propagation spatial effects. *In general the relative joint opening movement is up to two times that calculated for the same bridge with coherent ground motion using either linear analysis or nonlinear analysis for bridges up to 305 m in length.*

Larger increases in flexural demand and joint movement are likely for longer bridges. However, the study scope did not include enough examples to draw strong conclusions. Only one structure analyzed was clearly long enough that one would expect significant spatial effects. That bridge was a linear (no curvature) model of the SR14/5 Interchange Structure that collapsed in the Northridge earthquake. The total length is 483 m with significant differences in stiffness among the piers.

Gavin Canyon Bridge, one of the eight example bridges in the study, was analyzed to study the effects of different ground conditions and spatial variation in seismic wave propagation simultaneously. In the analysis it was assumed that some of the supports are on softer local soil than the rest. *The results of this analysis show that the maximum effect, when the support ground differential is between hard and medium ground, or medium and soft ground, is a factor of about 2. When the differential is between hard and soft ground, the ratio has a maximum of about 3. Most of the difference is likely due to the soil irregularities rather than seismic wave effects.*

Two bridges were modeled in three dimensions to study the effect of different angles of incidence of the horizontal seismic waves with respect to the axis of the bridge, and for one of these bridges the effect of vertical acceleration was also included. For these analyses, all of the supports were assumed to be founded in the same soil conditions. *Results of these limited analyses show no significant increase in structural demand due to variance in angle of incidence. However, a potentially significant increase is shown when vertical acceleration is included.*

Three of the eight example bridges were identical except for the number of hinges in the superstructure. The results either imply that such hinges would not be effective in reducing structural seismic demands for this 242-m-long bridge, or that spatial effects may not be significant for this case.

The study shows that variation in seismic wave velocity from 1000 to 2000 m/sec in general does not have a significant effect on the numerical results. Also, the slowest waves do not always generate the greatest differential effect.

3.3.3 *Related Reports and Research*

Task 106-E-2.2/2.5: Development of a unified model of spatial variation.

3.3.4 Limitations

The study included only one very long bridge, so no clear trends can be observed for such cases. However, at least two additional long bridges may be analyzed in the final year of the study.

The analytical model of the hinges in superstructures was very simple. No information was provided on the design of restrainers, nor on the usefulness of energy-dissipation devices in dealing with spatial effects. A more sophisticated analysis may be made in the final year of the study.

The results are presented without discussion of seismic zone, proximity of faults, or structure importance. These factors must be considered when selecting response spectra for design.

3.3.5 Impact

Code Requirements: The study recommends that for unskewed bridges under 305 m in length with at least two spans and uniform soil conditions:

- Use synchronous support ground motion.
- If it is desired to be conservative, increase the seismic design coefficient by 10% or decrease the R factor by 20%.

For this category of bridges, it is not clear why two alternatives are recommended that give different results. It is also noted that while the above cited recommendations are limited to bridges without skew, the study did not use skew angle as a variable. The conservative recommendations could be used for the most important category of structures.

For bridges over 305 m (1000 ft) in length or having supports on different local soil conditions, or severely skewed, the study recommends:

- Perform time-history analyses involving asynchronous support ground motion.
- Use several scenario earthquakes and several different values of wave velocity.

Again, this study provides recommendations pertaining to bridges with severe skew, although the study did not use skew as a variable nor did it define severity. The design recommendation that time-history analysis be used for all long bridges (over 305 m or 1000 ft) and for all bridges having non-uniform ground support conditions is much more conservative than another cited reference by Priestly et al. (1996), which suggests 600 m as the threshold with geologic irregularities, and 2000 m without.

Design Procedures: Design procedures for bridges outside the threshold limits would be required to be analyzed using a time-history analysis with spectrum-compatible, asynchronous support ground motions reflecting different soil conditions, wave propagation, and loss of coherency effects.

Design Cost: Design costs for seismic analysis would be significantly increased if these restrictions were adopted. Time-history analyses require additional time for input preparation as well as significantly increased time for interpretation of the voluminous output. Eventually, computer programs might be developed to reduce this extra analysis time, but the current state of practice does not warrant the development effort.

Structure Performance: Structure performance might be improved by the ability to account for different soil conditions. Current practice is to analyze the structure for both the firmest and the softest soil conditions and to take the envelope of the results. In terms of expansion joint behavior, current practice is

to add displacement values from tension models of the adjacent units at critical expansion joints, assuming 180 degree phase difference.

Construction Cost: Some bridges having differential soil conditions may have additional construction cost due to large increased structural demand or joint displacements resulting from the spatial effect, while others may cost less due to demands being smaller than the envelope design approach; e.g., piers on firm soil would not be unduly penalized by the more stringent input motions associated with softer soils.

Research Recommendations: Due to the limited number of cases studied the report recommends additional bridge studies regarding:

- Vertical component of ground motion.
- Angle of incidence of seismic waves with respect to the axis of the bridge.
- Different local soils conditions.
- Number of spans of the bridge.

Other factors not included in this report that relate to spatial effects and need systematic study are:

- Soil-structure interaction.
- Topographical conditions.
- Stiffness of bridge structure.
- Restrainers at expansion joints.
- Pounding at expansion joints.

3.4 Effect of Vertical Ground Motion on the Structural Response of Highway Bridges

M.R. Button, C.J. Cronin, and R.L. Mayes

Technical Report MCEER-99-0007

Task No. 112-D-7

3.4.1 Research Summary

The research addressed the conditions under which the vertical component of seismic ground motion is critical for determining the demands placed on key elements of typical highway structures. Current bridge design codes do not require consideration of vertical motion effects, and relatively few bridges have been designed considering this issue. However, available ground motion records clearly show that under some conditions, vertical acceleration can exceed horizontal acceleration.

3.4.2 Research Findings and Review Comments

This study investigates several aspects of vertical motion effects by performing a series of parameter studies that analyze the response of representative bridges of various configurations. The parameters include:

- Earthquake magnitude (6.5 or 7.5): six bridges analyzed
- Distance from fault (1, 5, 10, 20, or 40 km): six bridges analyzed
- Site conditions (soil or rock): six bridges analyzed
- Deck stiffness (relative stiffnesses 1/4, 1, or 4): two bridges analyzed
- Foundation restraint (fixed or spring): two bridges analyzed

A comparison of time-history analyses with response-spectrum analyses was made for three of the six bridges, and one bridge was analyzed using both nonlinear modeling and linear analysis. The study also included a comparison of various methods for combining the effects of vertical and horizontal response. The report discusses the effects of vertical ground motion on the following forces and responses in the superstructure and substructure:

- Vertical, longitudinal, and transverse displacement at midspan
- Vertical, longitudinal, and transverse displacement at the top of the pier
- Vertical shear force in the deck at midspan
- Vertical bending moment in the deck at midspan
- Vertical shear force in the deck at the pier
- Vertical bending moment in the deck at the pier
- Axial force at the base of the pier
- Transverse and longitudinal shear force at the base of the pier

After consideration of several possible formats, the report expresses the results as a ratio of (1) the increase in structural response values due to combined vertical and orthogonal horizontal motion minus the values due to orthogonal horizontal only, to (2) the dead load (DL) response values, termed 3-2/DL. This ratio gave the most meaningful comparisons for most of the parameters studied.

The bridges used in the study as representative structures were six of the seven examples developed for FHWA by Berger/Abam Engineers to illustrate the AASHTO seismic design requirements. These provide a range of geometric configurations and materials, as shown in Table 3-1.

Table 3-1 Descriptions of Representative Example Bridges

Bridge	Superstructure or Deck Material	Spans	Max Span	Columns or Piers at each Bent	Alignment
1	Cast-in-Place Concrete (CIP)	2	43 m	3	Straight/Square
2	Steel	3	47 m	1	Straight/Skewed
3	Precast Concrete	1	21 m	n/a	Straight/Skewed
4	CIP Concrete	3	37 m	2	Straight/Skewed
5	Steel	9	56 m	1	Curved/Square
6	CIP Concrete	3	33 m	1	Curved/Square

There are two general approaches to implementing vertical motion effects into the design process. One is to perform an explicit analysis for each bridge within the zone that is significantly affected. This requires that an appropriate vertical response spectrum be obtained or developed. The other approach, which is much simpler for designers, is to use multipliers on the dead load response values that envelope many of the vertical motion effects. The multipliers can vary as a function of the more significant factors such as earthquake magnitude, distance from fault, and the portion of bridge under consideration.

Following are conclusions given in the report.

- Response values computed from vertical response spectra based upon the attenuation relationships of Abrahamson and Silva (1997) are up to 40% greater or less than the traditional value of 2/3 of the horizontal response spectra.

- Use of the 2/3 of the horizontal response spectra generally gives conservative results for vertical response of the deck, but unconservative for the pier axial force.
- Bridges with the greatest percentage of modal mass lying in the range of the peak spectral acceleration of the vertical response spectra sustain the greatest impact from the vertical seismic motions. However, it was not possible to develop a specific recommendation regarding the threshold conditions for significant response.
- The effect of vertical motion increases with proximity to the fault.
- Values of horizontal response are not significantly affected by the vertical component of ground motion.
- Vertical effects at rock sites are more severe than soil for all responses studied, with two exceptions. The first is that vertical effects at soil sites are always more severe for pier axial forces. Additionally, vertical effects at soil sites are more severe for deck shear at the pier and flexure at midspan when the bridge is located more than 10 km from the fault.
- The early arrival of strong vertical motions does not have a significant effect on the structural response of typical highway bridges.
- Bridges within 10 km of a M7.5 fault that are unrestrained in the longitudinal direction, have relatively short columns effectively fixed to the deck, and have uneven span lengths should be checked for adequate shear resistance at midspan.
- Softening of the bridge deck due to cracking during an earthquake reduces the impact of the vertical component of motion.
- The period and modal-mass participation ratio of the fundamental vertical mode increase slightly with an increase in foundation flexibility. However, the resulting effect on structural demands does not show any specific trends.
- In general, the use of the SRSS (square root of sum of squares) method for combining directional results from response-spectrum analyses gives values close to those obtained from a frequency-scaled time-history analysis. Responses calculated by the 30% or 40% rule are all conservative.

The major recommendations given in the report summary are:

1. The traditional use of vertical spectrum ordinates = 2/3 of horizontal spectrum ordinates should be discontinued.
2. Bridges located 60 km or more from a potential 6.5-or-greater magnitude fault are not significantly affected by vertical motion. Vertical motion effects can be ignored for such structures.
3. Design of bridges located within 10 km of a potential 6.5-or-greater magnitude fault should explicitly include vertical motion.
4. For bridges located between 10 and 60 km from faults, dead-load multipliers should be considered in lieu of explicit vertical analysis. (The report provides proposed dead-load multipliers for M6.5 and M7.5 faults.)
5. If linear analysis is appropriate for a particular bridge, response-spectra analysis can accurately represent vertical response.

3.4.3 Related Reports and Research

Following are published research reports and papers pertaining to the effects of vertical ground motion on bridge response:

- Master of Science Thesis by Broekhuizen (1996), *Effects of Vertical Acceleration on Prestressed Concrete Bridges*, University of Texas at Austin;
- A note on the occurrence and effects of vertical earthquake ground motion by Foutch (1997) in the *Proceedings of the FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities*;

- A paper by Gloyd (1997) on the “Design of ordinary highway bridges for vertical seismic acceleration”, *Proceedings of the FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities*;
- A report by Priestly; et al. (1994), *The Northridge Earthquake of January 17, 1994 - Damage Analysis of Selected Freeway Bridges*;
- A report by Saadeghvaziri and Foutch (1988), *Inelastic Response of R/C Highway Bridges under the Combined Effect of Vertical and Horizontal Earthquake Motions*;
- A paper by Saadeghvaziri and Foutch (1991), “Dynamic Behavior of R/C Highway Bridges under the Combined Effect of Vertical and Horizontal Earthquake Motions,” *Earthquake Engineering and Structural Dynamics*.
- Ph.D. Thesis by Yu (1996), *Effect of Vertical Earthquake Components on Bridge Responses*, University of Texas at Austin;
- A report by Yu et al. (1997), *Effect of Vertical Ground Motion on Bridge Deck Response*.

3.4.4 Limitations

The total number of bridges studied, six, was relatively small. Therefore, the dead-load (*DL*) multiplication factors developed for bridges located between 10 and 60 km from the fault may depend more on the characteristics of the bridge than indicated in the report. Some of the results indicate that bridges with unbalanced spans, skewed supports, or curved horizontal alignment may be more susceptible to vertical motion than other bridges.

The $3-2/DL$ parameter may not be indicative of when a specific analysis of vertical motion effects is needed. Because service-load design includes live load and uses load factors, the $3-2$ factor could (depending on the structural characteristics) be in excess of 1.0 before vertical motion would control superstructure design. It would seem that a $3/DL$ factor in excess of a given value could indicate when vertical motions are important, while the $3-2/DL$ factor could be used for simplified design.

The study focused on the use of linear analysis methods, and only one comparison was made using a nonlinear model. Based on this limited effort, it appears that nonlinear behavior reduces the effects of vertical motion. However, since the scope of this aspect of the research was not extensive, no conclusions or recommendations were made.

The SRSS method of combining response-spectrum results proved to be the most accurate method for determining a single design quantity. However, in the case of column axial load, where interaction with bending moments about both axes is important, it is not clear that the SRSS method yields the most realistic results for column design.

The research found that the rule-of-thumb for using $2/3$ of the horizontal spectrum as a vertical spectrum is not valid. Although the report presents some vertical design spectra for various fault distances and some spectral relationships developed by Silva, it is not clear whether a designer should use these spectra and/or relationships or use site-specific vertical spectra.

The report provides recommended factors for use as dead-load multipliers as one method of accounting for vertical motion. These are developed for 6.5- and 7.5-magnitude events at various distances from faults. There are no factors for lower-magnitude events, which may be needed for structures quite close to lesser-magnitude faults.

3.4.5 Impact

This is the first major study of vertical motion effects on bridges that looks at all of the important issues affecting the design, thus it provides new insight into design considerations that have up to now been handled by assumption, gross approximation, or overlooked due to lack of information.

The major impact of the report is that it clearly shows that vertical acceleration can be a critically significant factor in the seismic response of highway bridges, and it should be appropriately covered by design codes and practices. The results are expressed as a ratio of the dead load, to be applied plus or minus, which varies depending on the portion of the structure under consideration, seismic event magnitude, fault distance, and, to a lesser extent, the foundation material. The use of these proposed factors will envelope the effects of vertical motion. Consideration of horizontal motion would still be done in the usual manner.

This report contains data and guidance that is practical and can be readily put into practice. It will be of value to designers of structures close to major faults by showing what structural and foundation factors are of most significance, aiding the optimization of structure configuration. The report also offers simple design procedures for most cases. It will be of great value in the development of bridge design codes, as it offers many recommendations that can be used in a design specification format. It will also be of value to researchers by pointing out areas that need further study.

Construction Cost: Design of the bridge superstructure for vertical motion effects may require that the superstructure be strengthened in areas that it would otherwise not be strengthened. This can add to the cost of the bridge when needed. However, the recommended method may result in less cost compared to use of $2/3$ of the horizontal acceleration, which has been used by some agencies, and it avoids a cost increase where vertical motion effects are small.

Design Cost: The simplified design approach requires minimal additional design effort. Specific consideration of vertical motion in the analysis would require a noticeable increase in design effort.

Bridge Performance: The consideration of vertical motion effects would theoretically improve the performance of a bridge during an earthquake, particularly in near-fault regions. However, there has been very little evidence in past earthquakes that failure to design for the effects of vertical motion has had a significant impact on the seismic performance of typical bridge structures.

Research Recommendations: Because of the potential significance of vertical acceleration indicated by this report, additional studies to expand and verify the findings are needed, as follows:

- An expanded parameter study should include specific bridge structural characteristics such as skew, curvature, and span imbalance. The $3-2/DL$ factors to be used for design should be developed as a function of these bridge structural characteristics, or the calculated factors should envelope all of the effects.
- Further nonlinear analyses on a broad range of bridges are needed to draw conclusions regarding the effect of nonlinear behavior.
- The capacity of prestressed concrete bridge decks to undergo reverse loading when subjected to seismic uplift greater than 1.0g needs to be investigated, as this case can occur in critical situations and current design practice does not cover it.
- The shear capacity of columns subjected to seismic loading cycles that create axial tension may not be adequately covered by current design practice.

- The development of “standard” vertical acceleration response spectra (ARS) for use in design in conjunction with “standard” horizontal ARS would be desirable for design purposes.
- Recommendations for when vertical motion can be ignored should be expanded to include structural characteristics. The recommendations should be based on a $3/DL$ factor.
- Analytical findings should be correlated with field observations of the performance of bridges subjected to strong vertical motions. Analytical models should be further verified by comparison with strong motion data.

3.5 Methodologies for Evaluating the Importance of Highway Bridges

Thomas, A., Eshenaur, S., and Kulicki, J.

Technical Report MCEER-98-0002

Task No. 112-D-2

3.5.1 Research Summary

This study examines a number of existing and proposed numerical methods for determining the importance of a bridge. To evaluate these methods, the report compared the numerical ranking of the importance of a representative sample of bridges to the averaged subjective ranking performed by a group of experts in the field. For the experts’ ranking, every member of the group of experts selected and ranked the 20 most important bridges from the representative sample, and their opinions or rankings were averaged. Based on this result, the best numerical methods were selected and further modified to give better correlation with the expert opinion. A modified version of the Montana method known as MTN5 was developed as the preferred method for determining importance. This method uses data from the National Bridge Inventory (NBI) for both the route carried and the route crossed.

The report recommends language to be used in several existing design codes that would allow individual entities to use the importance-rating methods that they currently employ. To use the new methodology for design, importance ratings would be calculated for the entire bridge inventory of a given jurisdiction. A rating for the proposed bridge would then be made and compared with the ratings of existing bridges. The importance of the bridge to be designed would be determined from its ranking relative to the existing bridge inventory.

3.5.2 Research Findings and Review Comments

The report recommends applying the importance factor regardless of the seismic performance category. (Presently, importance does not affect design unless $A > 0.29$, where A = mapped acceleration coefficient.)

The report recommends that jurisdictions be required to rank their bridges in order of importance. The MTN5 method may be used for this purpose. Consideration of emergency routes is optional. Agencies may use their own importance equation if they prefer.

Three importance categories are defined in the report. The design objective for each category is stated in terms of functionality after the design event:

- Critical bridges are to remain functional for all traffic after an earthquake larger than the design event. *How much larger, or whether there is an upper limit to the earthquake considered, is not stated.*

- Essential bridges are to remain functional for emergency, security, and defense traffic after an event equal to the design earthquake.
- Other bridges are not assigned functionality objectives.

The importance category is determined by percentiles. The top five percent are critical, and the next 20 percent are essential. *No information is given for the basis of these percentile values, however. The percentile method of determining importance means that a particular bridge could have its importance category changed solely due to other changes in the bridge inventory. Using percentiles, there can be significant differences between jurisdictions in the actual importance of critical bridges, for example, due to variation in bridge population. It would be more logical to assign a threshold value of importance, as calculated using the equation, for importance categorization.*

In order to apply this method to the design of new bridges, it is necessary to rank all of the existing bridges within a jurisdiction. *Political and economic factors must also be considered when categorizing bridges by importance, but this was beyond the scope of the report.*

Factors of 1.2 and 1.1 are applied to critical and essential bridges, respectively. The factors are used to increase the acceleration; the factored acceleration is used to determine seismic performance categories (SPC). *No information is given for the basis of these values. The importance factor can elevate the SPC for borderline cases.*

The report indicates an upper limit of 0.29 for the value of A times the importance factor for other (i.e., not essential or critical) bridges. The purpose is to avoid an increase from current Division 1A requirements. *This is not clear. It could be interpreted that regardless of the value of A from the acceleration map, a maximum of 0.29 should be used for other bridges.*

Increasing the design acceleration based on importance is logical, in the sense that it raises the damage threshold. *It may be more rational, however, to relate the increase to accelerations resulting from a longer return period.*

The proposed method was developed through a process that included a survey of state practices regarding importance evaluation, comparison reviews of many equations, correlation of results against experienced bridge engineer judgment, and trial use with actual bridges in several jurisdictions.

Many states indicated that they were satisfied with the system they presently use for importance, and are not interested in changing. Thus, the recommended provisions allow for that option.

3.5.3 Related Reports and Research

Report by Werner et al. (unpublished report), "Seismic Risk Analysis of Highway Systems," prepared under MCEER Task 106-E-7.3.1, Risk Assessment of Highway Systems.

3.5.4 Limitations

This study deliberately avoided addressing the political and economic issues related to a decision about which criteria to use for the seismic design of a bridge. From a practical point of view, these are the very issues most likely to dominate such a decision.

With respect to the engineering criteria that were addressed, many of the decisions and recommendations of this study seem to be subjective. For example, how was the five percent criterion for critical bridges determined? Since the issues discussed in this report address the only potentially objective aspect of determining importance, it is desirable that the report reflect a consensus of the profession. It does not appear that this is the case; it may not be easy, or even possible, to achieve such a consensus within the context of a rigid numerical rating method.

3.5.5 Impact Assessment

Code Requirements: This method seems best suited for making retrofitting decisions for existing bridges. Calculation of a numerical importance rating may provide useful information, but should not dictate the decision on whether or not a bridge should be designed to a higher level of seismic performance. Such a decision is by its nature very subjective and is greatly influenced by the incremental cost increase of hardening a bridge. This incremental cost, as the percentage of the base construction cost, may vary among jurisdictions. Therefore, it is very difficult to come up with a numerical method that fits all situations. This rating method is better left in the commentary with sufficient caveats about its use rather than including it in the specification as a hard requirement.

Design Procedures: Design procedures are likely to be more complicated for critical and essential bridges. Therefore, the number of bridges so classified will determine the overall effect on design procedures.

Design Costs: Design costs will be higher for critical and essential bridges that have more complicated design procedures. The number of bridges so classified will determine the overall impact on design costs of any importance classification scheme.

Structure Performance: Proposed performance criteria dictate that essential and critical bridges will have superior performance during large earthquakes. Therefore, the overall impact of importance classification on structure performance will depend on the number of such bridges.

Construction Costs: In general, improved structure performance will be purchased with increased construction costs. Again, the overall impact of this or any other importance classification method will depend on the number of bridges designated for special treatment.

Research Recommendations: At this point, it is not recommended that any additional research be performed to perfect an importance rating system. Jurisdictions who do not have an importance classification system in place should be allowed to use the method developed in this project to see if it works for them. The decision to use the actual numerical results of this approach to determine importance should be left to individual state transportation departments.

3.6 Application of Simplified Methods of Analysis to the Seismic Design of Bridges

Kim, J.H., Button, M.R., Mander, J.B. and Buckle, I.G. (unpublished report)

Task No. 112-D-8

3.6.1 Research Summary

The research investigated the applicability of simplified methods of analysis to various bridges to characterize the limitations of the methods and to make recommendations for their use in design. The

study compared the results from the uniform-load method, the single-mode method, and multi-mode spectral analyses. Parameters covered in the analyzed suite of bridges were curvature, span length ratios, pier height, skew, and span connectivity. A suite of 27 continuous bridges with integral abutments defined the criteria for regular bridges.

3.6.2 *Research Findings and Review Comments*

Important conclusions of the study are:

- In general, strong doubts were raised about the validity and necessity of the simplified methods of analysis, particularly given the availability of computer programs to perform multi-mode spectral analysis.
- When bridges contain more than 3 spans, multi-mode spectral analysis should be used regardless of uniformity.
- The simplified methods were thought to require more computational effort than a multi-mode spectral analysis.
- Simplified methods were thought to be more applicable to bridges in which the superstructure is continuous across the piers than for simple span bridges.

Criteria for “regular” continuous bridges derived from this study are:

- Three or fewer spans.
- Mass distribution with an average unit weight in adjacent spans varying by less than 50% (based on the smaller weight).
- Maximum ratio of adjacent span length not greater than two except for two-span bridges where the ratio may be up to three.
- Maximum ratio of adjacent bent stiffness, both longitudinal and transverse, not greater than four.
- Subtended angle in plan not greater than 90°.

Recommendations for design of continuous bridges include:

- Multi-mode spectral analysis may be used for any bridge.
- If the bridge is not “regular,” as defined above, multi-mode spectral analysis must be used.
- Either the single-mode or uniform-load method may be used for “regular” bridges. If the uniform-load method is used, the abutment shear values may be over-predicted by factors of 1.5 to 2.0.
- “Unwrapped” models of curved bridges may be analyzed using the multi-mode method, provided that:
 - The bridge meets the requirements for a “regular” bridge.
 - For two-span bridges the ratio of the span lengths is not greater than two.
 - The subtended angle in plan is not greater than 30°.

Recommendations for simple-span bridges include:

- The multi-mode spectral method should be used whenever possible.
- Alternative uniform-load methods and single-mode methods may be used for up to six-span bridges when the span length ratio does not exceed 1.5 (or 3.0 for two-span bridges).
- Modeling methods for the alternative simplified methods are defined in the report. Models with uncoupled spans, which include the transverse separation of columns for multi-column bents should be used. Additionally the models must distinguish between sliding and fixed bearings for regular simple-span bridges in which
 - the maximum subtended angle does not exceed 90° for two-span bridges, nor 30° for three-span bridges,

- the maximum span length ratio does not exceed 1.5 for three-span bridges, nor 3.0 for two-span bridges, and
- the maximum bent-to-pier stiffness ratio does not exceed one for three-span bridges.
- The single-mode method is not recommended for simple-span curved bridges.
- “Unwrapping” of simple-span curved bridges with the simplified methods is not recommended.
- Skewed geometry should be considered.

Simplified methods should not be lightly dismissed because of the availability of multi-mode computer programs. Much useful information and bounding response values can be obtained from such analyses if properly modeled.

The correct modeling of the support conditions (bearings, columns) is critical no matter what form of analysis is used, even multimode methods.

Table 3-2 summarizes the recommendations of the study for the use of simplified methods

Table 3-2 Bridges Recommended for Simplified Methods

<i>No. of spans:</i>	Straight bridges				Curved bridges	
	2	3	4	5 or 6	2	3
<i>Simple span bridges</i>						
Maximum span length ratio	3.0	1.5	1.5	1.5	3.0 ¹	1.5 ¹
Maximum pier stiffness ratio	N/A	8.0	4.0	3.0	N/A ¹	1.0 ¹
Maximum subtended angle	N/A	N/A	N/A	N/A	90 ⁰¹	30 ⁰¹
<i>Continuous span bridges</i>					<i>“Unwrapped” limits</i>	
Maximum span mass ratio	2.0	2.0	Not Allowed		2.0 ²	2.0 ²
Maximum span length ratio	3.0	2.0			2.0 ²	2.0 ²
Maximum pier stiffness ratio	N/A	4.0			N/A ²	4.0 ²
Maximum subtended angle	N/A	N/A			30 ⁰²	30 ⁰²

N/A = Not applicable

(1) Not recommended

(2) Multimode method required

3.6.3 Related Reports and Research

None

3.6.4 Limitations

The recommendation for use of the multi-mode method in preference to the simplified methods is predicated on the proper use of the multi-mode method; that is, a model that can capture all important modes and that accounts for foundation flexibility. The simplified methods are very useful in preliminary studies and sensitivity studies of the various input parameters carried out before constructing a multi-mode model. Correct technical interpretation of the analysis results is required.

The studies were based on comparison of the results of elastic analyses.

3.6.5 *Impacts*

Code Requirements: The study will be helpful for defining the criteria to be met by regular bridges in order that simplified methods of analysis be acceptable.

Design Cost: Restricting the use of the simplified procedures will require more effort for analysis and interpretation of results in many cases.

Structure Performance: The results will help avoid misuse of simplified analysis methods when this practice could result in an unsafe design.

Construction Cost/Constructibility: Not applicable.

Research Recommendations: Similar studies using inelastic analyses should be performed to validate the use of multi-mode methods for predicting maximum displacement values when significant inelastic action is expected.

3.7 **Establish Representative Pier Types for Comprehensive Study: Eastern United States** *Kulicki, J. and Prucz, Z.* *Technical Report NCEER-96-0005* *Task No.112-D-1.1(a)*

3.7.1 *Research Summary*

This report describes, with examples, bridge types and seismic design and detailing procedures typical of the eastern United States since about 1980. The designs follow AASHTO *Specifications* and specific policies that address the unique conditions of each state. Most bridges fall into SPC A and B, but in Pennsylvania and New York many of the more conservative requirements of SPC C are incorporated even though they are not required.

Bridge pier types include pile bents, column bents, and solid wall piers; bent cap types include rectangular, inverted T, and hammerhead. The report highlights the unique nature of the seismic response of various bridge configurations and details, and it identifies common and dissimilar elements between eastern U.S. and western U.S. construction practices.

3.7.2 *Research Findings and Review Comments*

Although examples are presented from only four states (Louisiana, New York, North Carolina, and Pennsylvania), these states are considered representative of the entire region.

Pile Bents

- A pile bent consists of four or more timber, steel or prestressed concrete piles, placed in a single or double row, and embedded at least nine inches into a cast-in-place reinforced concrete cap. In bents with more piles, some piles may be battered, increasing stability but reducing ductility for seismic loading.
- Cap width is sized to allow for pile mislocation or seat width requirements for girders. Continuous reinforcing bars in the bottom corners and transverse stirrups enhance seismic performance by

accommodating load reversals and avoiding cap-beam plastic hinging. *However, it does not appear that this reinforcement is specifically designed for seismic loading.*

- No states require a positive tension connection from pile to pile cap. Some states provide a tension anchorage and ductile connection through use of a steel cage in larger piles that have a center void. *However, it does not appear that this reinforcement is specifically designed for seismic loading.*

Column Bents

- Column bents usually consist of two or three circular reinforced concrete columns with a rectangular cap forming a frame. Column spacings seldom exceed 6.0 m. Intermediate struts are usually used on bents over 7.6 m in height. *This creates a vulnerable location for seismic loading if the reinforcement is spliced just above the strut, a common practice.* Variation in column height within the same bent is allowed. *This practice is undesirable for seismic behavior, as it concentrates loading on shorter columns.*
- One-column bents with a solid rectangular cross section and hammerhead caps are also common.
- Inverted T caps are becoming more common.

Solid-Wall Piers

- Solid-wall piers are used at stream crossings and as crash walls at railroad crossings. Pennsylvania has adopted SPC C design rules for use in SPC B; otherwise the eastern states follow AASHTO.

General

- For all pier types, the superstructure is supported on bearings anchored to the top of the cap.
- Column vertical rebar splicing practice follows AASHTO, except that Pennsylvania and New York limit lap splices to the center half of column height.
- Column transverse reinforcement practice varies: Louisiana uses spirals, but they are not common in Pennsylvania.
- Extension of column reinforcement into caps and footings follows AASHTO, except that Pennsylvania requires $1.25 f_y$ development, and Pennsylvania and New York require that transverse reinforcement extends into the adjoining members.
- Some states have rules regarding cap width relative to column size. *Wider caps help force plastic hinging into the columns.*
- Pile-supported footings are common in the southern states; spread footings are common in the northeast. Exterior piles are often battered.
- Practice varies regarding use of a top mat and footing stirrups. Pennsylvania and New York use both; North Carolina requires a top mat but not stirrups; Louisiana uses neither.
- Some eastern states have adopted seismic design practices that exceed AASHTO requirements, indicating that a higher level of safety is recognized as desirable and feasible.

3.7.3 Related Reports and Research

Report by Imbsen et al., 1996, *Establish Representative Pier Types for Comprehensive Study: Western United States*, prepared under MCEER Task 112-D-1.1b.

3.7.4 *Limitations*

Though not a limitation, the scope of the study does not include abutments. The focus is on common practices; therefore the full range of practice is not described. Since only four states' practices were reviewed in detail, there may be common but unreported variations to these practices in other states.

3.7.5 *Impact*

Code Requirements: This report is useful for the design specification development process in that it points out current design practices and typical details that are being used. Therefore, provisions can be written to cover the various cases, or steps can be taken to discourage undesirable details and practices.

Design Procedures: This report illustrates the types of piers and details for which suitable seismic design procedures need to be available.

Cost Impact: This report identifies pier types that have been found to be practical; thus, it serves as a base reference for evaluating other approaches, either modifications to current practice or different pier types.

Structural Performance: This report identifies current common practices, and identifies desirable and undesirable types with respect to seismic performance.

Research Recommendations: This study is complete and although additional work could be done to broaden its scope, no further research on this subject is critically needed at this time.

3.8 **Establish Representative Pier Types for Comprehensive Study: Western United States** *Imbsen, R., Schamber, R.A., and Osterkamp, T.A.* *Technical Report NCEER-96-0006* *Task No. 112-D-1.1(b)*

3.8.1 *Research Summary*

This study reviews typical seismic design and detailing practice for a number of western states, including Alaska, Arizona, California, Idaho, Montana, Nevada, Oregon, Washington, and Wyoming. Topics covered include:

- Pier types
- Design and detailing issues for columns and piers
- Design and detailing issues for bent caps
- Footing details
- Construction materials

The original intent of this project was to collect information on the different types of piers used by the various western states so that MCEER research could be focused on subjects relevant to current practice.

3.8.2 *Research Findings and Review Comments*

General

- Information and details from the nine listed states were included. *There is no indication of the exact geographic scope intended to be included as the western United States. Possibly other states did not respond to a request for information. It is not clear just how the data were solicited or obtained. However, the columns described seem representative of the area.*
- A table summarizes certain information regarding state practices, apparently based on response to a questionnaire. *However, it appears no attempt was made to follow up and get refined responses in order to make a more uniform and complete database. It appears that the survey was limited to whatever information the states chose to provide. The survey reports common practice, not the full range of design practice.*
- AASHTO standards are used by all reporting states, except California, which has its own standards. The differences are pointed out. Much of the information in the report pertains to Caltrans practice, since Caltrans has focused on column design and detailing in recent years, and has developed more refinements.
- In addition to identifying representative types, the report discusses advantages and disadvantages of various practices and makes recommendations on certain items.

Pier Types

- The pier types emphasized in the report were reinforced concrete column bents and pier walls. Examples of multiple-column and single-column bents and pier walls are provided.
- Some states (Alaska, Montana and Wyoming) indicated that they used steel pile bents, but the only details provided were for a concrete-encased version. Other than this, no states reported the use of steel columns.
- Oregon is the only state that indicates it has introduced base-isolation systems.
- No states report using monolithic connections between steel girder superstructures and concrete piers.
- Both seat- and diaphragm-type abutments are in common use.

Design and Detailing Issues for Columns and Piers

- Because of the greater frequency of earthquakes in the west, most states use seismically-resistant detailing.
- Despite the increased construction costs and time associated with transverse-spiral or welded-hoop reinforcement, this type of reinforcement seems to be more widely used than ties. This is also the case for square column sections or oblong sections, in which overlapping spirals are often used.
- Even though it is not required, several states seem to be using the maximum transverse reinforcement over the full length of the column. Other states limit this heavy reinforcement to the end regions of columns, where plastic hinges are likely to form.
- There are special shear requirements in plastic hinge zones to ensure ductile behavior. Caltrans requirements are more stringent than those of AASHTO.
- Transverse reinforcement generally extends into adjacent members (bent caps and footings) for a significant distance, with California requirements being more stringent than those of AASHTO.
- Longitudinal column steel is lap-spliced only under certain conditions. California allows lap splices only outside the plastic hinge zones, if the column exceeds 10.4 m in height. Only #11 bars and smaller are allowed to be lap-spliced. Mechanical couplers and welding are used for larger bars.

- Caltrans does not allow lap-splicing of transverse spiral reinforcement. Other states use AASHTO splicing provisions, which are less stringent than Caltrans requirements and limit splices to the middle half of the column. Caltrans also has special requirements that require reinforcement to be continuous through knee joints. This requirement grew out of their experience during the 1989 Loma Prieta earthquake.
- The report discusses the use of R or Z factors. AASHTO varies the use of R factors depending on the Seismic Performance Category. Caltrans Z factors combine ductility and risk.
- The use of a pinned condition at the base of multi-column bents and pier walls in continuous structures is common because of the savings that can be realized in the foundation quantities and in the simplification of construction. Reduced seismic forces are often realized at the expense of higher displacement values. Caltrans prefers pinned detailing while other states (Arizona and Washington) prefer fixed-base detailing.
- The use of architectural flares in columns has lost favor as a result of experiences in the 1994 Northridge earthquake. When used, these flares are either reinforced as part of the column, or isolated to ensure that they do not unintentionally stiffen the column. The questionable reliability of isolation detailing and the extra shear and superstructure demands resulting from the use of ductile flares are disadvantages to their use.
- Piers are defined as having an H/L ratio less than or equal to 2.5. These members, which are typically used in river or stream crossings, are generally designed as tied columns in the weak direction. This requires both horizontal reinforcement and considerable lateral cross-ties. Because of the amount of ties, Caltrans generally limits this heavy reinforcement to end regions.
- Partial-height, infill pier walls are discouraged because of experiences during the 1994 Northridge earthquake.
- For pier wall design in the weak direction, Caltrans uses nominal transverse reinforcement in low seismic areas with the condition that the pier be designed with a Z factor of two.

AASHTO design criteria are more stringent than Caltrans in some cases. For example, at some localities the probabilistic design earthquake loading used by AASHTO is larger than the deterministic earthquake loading used by Caltrans. AASHTO response-modification factors are lower and thus result in higher design moments. In addition, AASHTO uses capacity-reduction factors for flexural design, while Caltrans does not. These differences would result in significantly more elastic flexural capacity in bridge columns, in some cases, if AASHTO criteria were used. Caltrans uses ACI reinforcing steel anchorage requirements but does not increase the anchorage length of longitudinal bars in cap members by 25 percent as required by AASHTO.

Design and Detailing Issues for Bent Caps

- In column bents, cap beams that are monolithic with the columns are typically at least 150 mm. wider than the column. Current Caltrans practice is to make them 610 mm wider.
- Caps are designed with continuous top and bottom longitudinal reinforcement (mechanical splicing or welding allowed) placed inside closed transverse stirrups. Sufficient reinforcement is provided so that the caps remain essentially elastic during an earthquake. *Caltrans also has begun to use prescriptive requirements based on research for joint shear reinforcement at column-cap joints*
- The importance of torsion in outrigger bents has been recognized as a result of the 1989 Loma Prieta earthquake experience. Because no AASHTO or Caltrans design provisions exist, designers are using ACI torsion provisions. *Torsion is often mitigated in outrigger bents by pinning the top of the columns or providing "super" beams between bents in the longitudinal bridge direction.*

Footing Details

- Footings have a top mat of reinforcing steel and vertical stirrups (Caltrans only). Footing reinforcement is designed to carry either the column plastic moment or the column elastic moment from the earthquake. *Caltrans currently requires that sufficient horizontal footing reinforcement be placed in a zone that includes the column width plus twice the depth of the footing. In addition, sufficient joint shear reinforcement is currently required in footings.*
- Piles are designed for tension during earthquakes whenever feasible. Pile-to-pile-cap connections must be sufficient to transfer this tensile load. *Caltrans discourages the use of batter piles in column footings.*

Construction Materials

- Caltrans currently specifies A-706 reinforcing steel. This steel is specified to be more ductile and easier to weld than A-615 steel. *Some previously approved mechanical rebar splicing systems were redesigned because of the mechanical properties of A706 reinforcing.*

3.8.3 Related Reports and Research

Following are related MCEER technical reports:

- *Establish Representative Pier Types for Comprehensive Study: Eastern United States*, by Kulicki and Prucz (1996);
- *Capacity Detailing of Members to Ensure Elastic Behavior*, by Imbsen et al.(1996).

3.8.4 Limitations

- The report focuses on Caltrans practice and does not give equivalent treatment to the practices in other western states. It would have been preferable to see a thorough treatment of design and detailing practices in other states. Also, Caltrans practice is changing rapidly and some sections of the report are already out of date.
- Pier walls are not mentioned (in the table on pier types) as a commonly used type of pier.
- Drilled shafts were included in the table (Arizona and Montana), but not discussed in detail in the report.

3.8.5 Impact

The original intent of this project was to collect information on the types of piers used by the various western states so that MCEER research could focus on subjects relevant to current practice. However, there are a number of secondary benefits to this work that are beyond its original intent. These benefits are discussed below.

Code Requirements: This report is useful in the design specification development process in that it points out design practices and typical details that are being used. Therefore, provisions can be written to cover the various cases, or steps can be taken to discourage undesirable details and practices.

Design Procedures: The experience of Caltrans is very useful because of their seismic design experience and ongoing research program, but adoption of their design provisions nationwide will complicate designs outside California. The principal impacts of these provisions are splicing limitations, joint shear reinforcement, footing shear reinforcement, and bent-cap reinforcement. Column detailing

practice in the United States for the higher seismic performance categories is already similar to California practice. If Caltrans experience is to be adopted for nationwide use, some adjustment will be necessary to accommodate bridge types and details that are more commonly used in other states.

Design Cost: In that adoption of Caltrans practices would complicate designs, design costs would increase.

Structure Performance: Structure performance at joints and in members connecting to piers would be improved, to the extent that damage of these elements would be minimized with the adoption of Caltrans practices.

Construction Cost: Adoption of Caltrans practices would increase the amount and congestion of reinforcing steel in certain locations of the structure, and thus construction costs would increase slightly.

Research Recommendations: Surveys such as this will be required from time to time to update the database of current practice. It is essentially a method for researchers and design specification writers to become familiar with current practice so that their work is more meaningful to the practitioner. Communication between researchers and practitioners is a longstanding problem and can always stand improvement. Various tools such as workshops, seminars, the activities of various professional committees, and surveys such as this have been used in the past with some degree of success. There are certain limitations to these methods, however.

Perhaps it is time to address this issue by considering how recent technological developments (for example, the Internet) can facilitate this communication. The subject should be extended to include better communication and sharing of information within the research community and among practitioners. Caltrans, for example, uses the Internet to make standard plans and specifications available in electronic form to anyone who wants them. Maintaining repositories of technical information in a form that is meaningful will require a substantial effort and commitment of human resources. If properly planned, this effort could result in significant benefits.

As a first step, a small research project to review, evaluate, and summarize the experiences of various technical organizations that have used the Internet as a means for disseminating technical information would be useful. The report from this project could help organizations plan for a more efficient use of computer technology. This effort could facilitate better communication of technical information in general, including the lines of communication between researchers and practitioners.

3.9 Seismic Resistance of Bridge Piers Based on Damage Avoidance Design

Mander, J.B. and Cheng, C.T.

Technical Report NCEER-97-0014

Task No. 112-D-5.1(a) and 112-D-5.2(a)

3.9.1 Research Summary

A design concept called Damage Avoidance Design (DAD) is the subject of the report. The concept is intended to avoid the development of plastic hinges in the columns, thereby avoiding loss of service after a significant earthquake. The concept involves the use of rocking piers that are forced to rotate at their ends but are restrained from toppling by gravity loads and the optional use of central, unbonded, post-tensioning reinforcement in the core of the columns. The report covers the theoretical aspects of the concept, a near full-size test, and the development of a design procedure.

3.9.2 *Research Findings and Review Comments*

- The concept of rocking structural members is not new. It was first developed theoretically by Housner in 1963 and subsequently researched. The first application of the concept was to a bridge and an industrial chimney in the mid-1970s in New Zealand.
- The theory of rocking piers was developed in this research and incorporated in a pushover analysis technique. It was compared to earlier theoretical work and good correlation was obtained. The theory indicated that rocking piers and the associated bilinear force/deflection relationships are capable of developing the base shear capacity.
- Energy dissipation occurs in the process of rocking, by means of radiation damping on impact. Contact surfaces must be designed for high stress values to ensure damage-free performance. *This mode of damping is not well understood.*
- Conventional ductility is not relevant to the rocking concept. Displacement capacity is critical.
- The dynamic performance of a rocking column is bilinear-elastic: the initial stiffness is due to the structural flexibility of the columns, while the rigid-body kinematics of the rocking mechanism controls the subsequent stiffness.
- Lateral base-shear resistance is provided by friction due to the gravity load and the prestressing force, supplemented by pintles.
- The authors developed equations to quantify the energy dissipation in each cycle, and this is converted into an equivalent damping ratio. *Caution is advised in using this classical concept of damping as representative of radiation damping.*
- A complete design procedure was developed, and an example was presented.
- The elastic spectral demand can be determined by modifying the usual five percent damping curve for the equivalent damping ratio.
- Lateral force demand and capacity can be determined by modifying the usual five percent damping curve for the equivalent damping ratio.
- Alternative theoretical design approaches are compared, and found to be acceptable, provided that displacement values remain less than $\frac{1}{4}$ of the rocking base width.
- Near-full-size tests were performed on a number of different configurations, including those with and without unbonded tendons.
- Excellent correlation was obtained between the experimental results and the theory that formed the basis of the design procedure.

3.9.3 *Related Reports and Research*

None

3.9.4 *Limitations*

The design concept presented is an important development but it will be controversial because it is so different from current practice. The theory is well developed and correlates well with the experiments performed to date. The limitations of the work performed to date are summarized below:

- The theory was developed for unidirectional ground motions, and it must be extended to incorporate bi-directional and vertical ground motions.
- The torsional resistance of skewed and irregular bridges with rocking columns is not well understood and was not included in the research.
- The limitations of radiation damping could be made less severe with the addition of other energy dissipation devices, as occurred in the two New Zealand applications in the 1970s.

3.9.5 Impact

Code Requirements: Consideration could be given to incorporating the design procedure in the commentary to a code, for designers who wish to explore this design concept. It is not sufficiently developed to become part of the design code.

Design Procedures: The report includes a design procedure for assessing the design concept. There are a number of limitations that should be placed on the applicability of the procedure but it appears to be well founded.

Design Cost: The design time required by the procedure will not be significantly different from that required for a conventional design.

Structural Performance: The design concept provides a means of achieving limited or no damage. It therefore provides a means of achieving higher levels of performance.

Construction Cost/Constructibility: Once some of the existing limitations have been resolved, the concept will need to be compared with other concepts, such as seismic isolation, to establish its cost effectiveness. One advantage of the concept is that prefabricated columns can be used. This could reduce the total construction time and reduce the overall cost. From a constructibility perspective, the concept may be attractive in areas where prefabricated construction minimizes the traffic disruption that would be associated with conventional construction.

3.10 Seismic Design of Bridge Columns Based on Control and Repairability of Damage

Cheng, C.T. and Mander, J.B.

Technical Report NCEER-97-0013

Task No. 112-D-1.2, 112-D-5.1(a) and 112-D-5.2(a)

3.10.1 Research Summary

The research addressed the development and testing of construction details in reinforced concrete columns that provide a replaceable or renewable sacrificial plastic hinge zone. Hinge zones are deliberately weakened with respect to adjoining elements; all regions outside the hinge zones are detailed to remain elastic during seismic loading. The special detailing of the sacrificial hinge zone permits repair of earthquake-induced damage. The basic concept allows for repair of the damaged segments with the intent of permitting use of the bridge, for at least minimum levels of traffic, after a major earthquake.

3.10.2 Research Findings and Review Comments

- The report covers the theoretical aspects of the concept and includes experimental testing on one-third scale and nearly full-scale specimens. Theoretical models for predicting the fatigue life of reinforcing steel are compared with experiments from other researchers. Good correlation was observed. *This is the first time that the rupture of longitudinal reinforcing steel was hypothesized to be a fatigue-related problem.*
- Three one-third-scale models of the columns were constructed, two with the replaceable detail and one conventionally designed for comparison purposes.
- Test results show that damage to all replaceable hinge columns was controlled and constrained within the plastic hinge zone, due to the deliberate weakening of the fuse bar zone. The portion of the

column outside the hinge zone remained elastic and undamaged at all times. This led to the possibility of full repair of the damaged columns.

- The repaired columns performed as well as the original undamaged column.
- The first replaceable detail was repaired five times to examine the effect of transverse confinement and fuse bar length.
- The second replaceable detail was repaired ten times to examine the effects of aspect ratio, axial load and drift level. The design concept was insensitive to changes in aspect ratio and axial load.
- An experiment was performed on one near-full-size specimen in order to investigate the viability of the replaceable hinge concept with prototype construction materials.
- The near-full-size specimen was repaired twice and the repaired column performed as well as the original.
- A specially detailed central core within the hinge zone had sufficient capacity to support the gravity axial load. *The need for shoring during the repair process needs further evaluation.*
- A design procedure and design guidelines are presented, together with a well-developed example.
- The yield penetration of longitudinal reinforcement for conventionally constructed columns spreads into the beam-column joint. This yield-penetration phenomenon effectively prevents repair of conventional construction, as bond and anchorage, as well as the fatigue life, of the longitudinal reinforcement within the beam/column joint zone are impaired. In contrast to this, the joint zone is well-protected against damage if the replaceable hinge detailing is used.
- In the test of the near-full-size replaceable-hinge column, the use of wire rope is justified for rapid repair procedures. Wire rope proved to be effective for confining the fuse-bars in the hinge zone.
- Due to the specific length of the replaceable hinge zone, the fatigue capacity of reinforced concrete bridge columns can be predicted by a strain-to-rotation transformation. A normal strain-life fatigue rule can then be used for predicting member fatigue capacity. For low-drift amplitudes, the low-cycle fatigue theory tends to be marginally conservative.
- The program UB-COLA, developed by Chang and Mander (1994a), was used to model the hysteretic behavior of the replaceable-hinge columns. The program provides a good prediction of the cyclic loading performance in terms of the strength and stiffness and, most importantly, the fatigue life.

3.10.3 Related Reports and Research

Following are related MCEER technical reports:

- *Seismic Energy Based Fatigue Damage Analysis of Bridge Columns, Part I – Evaluation of Capacity*, by Chang and Mander (1994a);
- *Seismic Energy Based Fatigue Damage Analysis of Bridge Columns, Part II – Evaluation of Demand*, by Chang and Mander (1994b).

3.10.4 Limitations

This work appears to be ready for implementation. The following topics were recommended for future investigation, but they are enhancements rather than limitations.

- Instead of the mechanical threaded couplers used in this investigation, alternative force-transfer mechanisms between the fuse-bar and longitudinal reinforcement may be advantageous. This would enable extension of the concept to the retrofit of existing bridge piers.
- This research has demonstrated that the use of wire rope for transverse reinforcement in the plastic hinge zones of columns is effective for providing confinement and anti-buckling support of longitudinal reinforcement. The use of such reinforcement has significant installation advantages. It is therefore suggested that the use of wire rope be investigated for other applications.

3.10.5 Impact

Code Requirements: This work will not have a direct impact on code provisions although it is an important design concept that will reduce the downtime after an earthquake. The design concept should be discussed in the commentary in any future performance-based code. Requirements for the use of wire rope as confinement steel in a repair mode will need to be developed.

Design Procedures: The design procedure developed as part of this research should be helpful to any designers wishing to implement the concept. The procedure should be referenced in any future performance-based code.

Cost Impact: Compared with conventional construction, the new replaceable-hinge type of construction will have a higher initial cost. Repair costs following an earthquake, however, will be less. Moreover, in the event of a large, damaging earthquake that would render a conventional structure irreparable, a structure designed in accordance with the replaceable-hinge philosophy could be repaired with its service life extended indefinitely. An analysis of cost impacts should be investigated in terms of discounted annualized costs (including the cost of repairs following a damaging earthquake) and compared with the discounted annualized cost of conventional construction (including the effect of rebuilding after irreparable damage has been caused by an earthquake).

Structural Performance: It appears that a bridge designed with this concept will perform as well as a conventionally designed bridge, and that a repaired bridge will perform as well as the original.

Research Recommendations: None.

3.11 Capacity Design And Fatigue Analysis of Confined Concrete Columns

Dutta, A., and Mander, J.B.

Technical Report MCEER-98-0007

Task No. 112-D-5.1(a)

3.11.1 Research Summary

The research addressed the development of methods for explicit prediction of transverse hoop fracture based on energy-balance principles. A complete design procedure for columns eliminates all undesirable modes of failure, including buckling of the longitudinal bars.

3.11.2 Research Findings and Review Comments

The report lists six potential failure modes of reinforced concrete columns:

1. Shear or shear/flexure failure outside the plastic hinge zone.
2. Failure of the connections by bond failure of lap splices at the end of the columns, anchorage bond failure within the connections, or joint shear failure adjacent to the column.
3. Premature concrete failure due to lack of confinement.
4. Failure of the confined core concrete due to compression buckling of the longitudinal reinforcing bars.
5. Fracture of the transverse hoop reinforcement, leading to failure modes 1 to 3.
6. Failure due to low-cycle fatigue of the longitudinal reinforcing bars.

All of these modes except mode 6 can be suppressed by providing adequate transverse reinforcement. Modes 1 through 4 can be avoided by using the principles of capacity design. Only mode 6 is unavoidable using these techniques. The structure must be designed to provide sufficient capacity to avoid mode 6 during the design-level earthquake.

Design to avoid mode 6 is covered in Chang and Mander (1994a). A simple equation estimating the cycles to failure at a given nondimensionalized curvature is compared with the curvature demands, shown by inelastic analysis, of a suite of earthquake records. The given expression, based only on structure period, is shown to envelope the results. The duration of the seismic input is thus included in the analysis.

In this report, an energy-based method is developed to determine the fatigue limit, based on the fracture of the transverse hoops (mode 5). This fatigue expression gives the number of cycles at a given nondimensionalized curvature to failure of the transverse hoops. Both an “exact” and a simplified method are developed. Design procedures are proposed, and the results of the analyses are compared to experimental results. A simplified direct approach suitable for design use is illustrated with an example.

Design equations for adequate confinement to avoid failure modes 1 through 5 are proposed. The proposed formulation is compared to current design provisions. The proposed formulation is less stringent than current specifications for sections with low ratios of longitudinal steel ($< 2\%$) and low axial-load ratio (< 0.25), and more stringent for sections with high ratios of longitudinal steel ($> 3\%$) and high axial-load ratio (> 0.35).

The analysis of buckling of the longitudinal bars is based on energy balance, with the transverse hoops in a state of yield. This is in contrast to all previous studies, which have assumed elastic behavior of the transverse hoops. The current study shows that in some cases longitudinal bars can buckle over a length of more than one hoop spacing. In some cases, current recommendations (ATC-32, NZ 3101, for example) are shown to be inadequate for preventing global lateral buckling prior to the attainment of yield stress in the longitudinal reinforcing. Design expressions are proposed that allow for limited global buckling, considered adequate (i.e., greater than mode 6 limits) to preclude buckling.

The report includes design charts, which show the minimum spiral steel volume for circular columns and the minimum lateral steel volume for square sections needed to avoid the undesirable modes of failure for a given axial load ratio. The report suggests a method of reducing reinforcing steel congestion by providing two concentric transverse spirals and layers of longitudinal reinforcing. Failure of the confined section in low-cycle fatigue is prevented by designing the section to limit the ultimate curvature in the plastic hinge zone.

3.11.3 Related Reports/Research

This is one of a series of reports on the fundamental theory of seismic response of reinforced concrete columns. Others in the series include:

- *Fundamental Shear-Flexure Interaction Theory for Strength and Deformation Analysis of Structural Concrete Elements*, by Kim and Mander (unpublished report);
- *Capacity Design of Bridge Piers and the Analysis of Overstrength*, by Mander et al. (1998);
- *Seismic-Energy-Based Fatigue-Damage Analysis of Bridge Columns: Part I - Evaluation of Seismic Capacity*, by Chang and Mander (1994a);
- *Seismic-Energy-Based Fatigue-Damage Analysis of Bridge Columns: Part II- Evaluation of Seismic Demand*, by Chang and Mander (1994b).

3.11.4 Limitations

It is presumed that the shear assessment is based on the analysis presented in *Fundamental Shear/Flexure Interaction Theory for Strength and Deformation Analysis of Structural Concrete Elements* (Kim and Mander, unpublished report). This analysis was stated to be limited in its ability to replicate the degradation of strength associated with cyclic loading.

The experimental validation was taken from published literature in which it was sometimes difficult to isolate the exact mode of failure, in particular to distinguish between failure due to transverse hoop fracture and failure due to longitudinal bar buckling.

3.11.5 Impact Assessment

Code Requirements: Transverse reinforcement requirements may be either reduced or increased depending on the axial load ratio and longitudinal reinforcing steel ratio. This research has the potential for consensus-based new design requirements for transverse reinforcement in columns.

Design Procedures: A five-step design procedure is proposed and illustrated. This procedure is suitable for design office use, although some iteration is required. Design aids can be developed using the tools presented in the report.

Design Cost: The procedure is more complicated than current prescriptive requirements, but an example calculation is provided that can be easily automated using available software.

Structure Performance: The procedure would ensure adequate protection against brittle shear failures and would thus ensure reliable formation of desired energy-dissipation mechanisms.

Construction Cost/Constructibility: Use of the theory will permit more explicit capacity design, thus reducing structure cost through reduction of excess conservatism.

Research Recommendations: Further experimental work is recommended to define the requirements for antibuckling steel. The work must be based on carefully designed and instrumented experiments in systems where this mode of failure can be clearly expected.

3.12 Capacity Design of Bridge Piers and the Analysis of Overstrength

Mander, J.B., Dutta, A., and Goel, P.

Technical Report MCEER-98-0003

Task No. 112-D-5.2(a)

3.12.1 Research Summary

The research addressed moment overstrength capacity of reinforced bridge columns for use in capacity design.

3.12.2 Research Findings and Review Comments

In one phase of the research, a moment/curvature analysis method was used to determine overstrength. Stress-strain relationships for confined and unconfined concrete and for reinforcing steel were assumed.

A Gaussian quadrature numerical integration technique was used to integrate stress values within a concrete section at varying levels of curvature to determine the corresponding moment. The analysis considered several factors including the enhancement of concrete strength and the strain at maximum strength due to confinement, the shape of the descending branch of the concrete stress-strain curve, and the strain-hardening of reinforcing steel. Existing experimental data were used to calibrate the proposed predictions.

A parameter study was also conducted, in which the effect of several variables on the moment overstrength factor was studied. The moment overstrength factor is defined as M_p/M_n . Both confined and unconfined concrete sections were examined. In the case of confined columns, the effect of certain parameters was appreciable, with increased overstrength at both low and high axial loads. Therefore, a simpler method for determining column overstrength was explored.

The simplified approach involves developing a diagram of the interaction between axial-load and moment for the plastic overstrength condition. This is achieved by calculating the balance point as the combination of a reinforcing steel couple and an eccentric concrete stress block. The overstrength axial load at zero moment is given by the maximum tensile strength of the reinforcing steel. A parabolic curve fit between these two points defines the interaction diagram below the balance point. A similar approach is used to calculate the nominal interaction diagram. The overstrength factor is then determined as the ratio of the plastic overstrength moment to the nominal moment for a given axial load. A formula is developed to accomplish this task mathematically and a design example is provided.

The parameters studied included axial load, the percentage of longitudinal steel and the strength of this steel, placement of the steel, concrete strength, and the level of confinement. Based on these studies, it was recommended that an overstrength factor for unconfined concrete columns could be conservatively estimated as 1.2. For confined columns, only the concrete within the confined core of the column is considered effective. Overstrength factors for confined columns are greatest at both low and high axial loads, and least at moderate axial loads. *High axial loads are typically not a representative loading for bridge columns.* Current prescribed overstrength factors of 1.3 are exceeded in some cases, particularly at low axial loads (less than $0.05P_u/f'_c A_g$) or at high longitudinal steel ratios (greater than two percent). Because of the high steel strains that occur at low axial loads, strain-hardening of the steel plays a major role in moment overstrength. Thus, overstrength tends to be higher for higher longitudinal reinforcement ratios.

At high axial loads, overstrength is governed by the contribution of confined concrete and thus the level of confinement has a major effect on overstrength. *However, this is not a typically representative loading for bridge columns.* Increased reinforcing steel strength also results in higher overstrength factors at low axial loads. The increase is not as pronounced for high-strength steel, which strain-hardens at relatively low strains.

Concrete strength has its greatest effect at high axial loads that are not typical of bridge-column loading. As would be expected, the level of confinement increases the effect of concrete strength at high axial loads. At low axial loads and normal levels of confinement, there is little difference in the overstrength factors for various concrete strength values.

Steel placement is measured as the ratio of the concrete cover from the center of the transverse reinforcement divided by the total depth of the section. The amount of overstrength decreases as the amount of cover increases.

The analytical method tends to be conservative in that it closely predicts the upper bound of experimentally-observed column overstrength factors. The simplified method, involving the approximation of diagrams of the interaction between axial force and moment, tends to be slightly conservative with respect to the moment-curvature method developed for this study. This is desirable because the effect of cyclic work-hardening is not considered in the moment-curvature method.

Overall, this is an excellent, usable study defining a method for predicting the overstrength factor for capacity design.

3.12.3 Related Reports and Research

Following are related MCEER technical reports:

- *Seismic Based Fatigue Damage Analysis of Bridge Columns: Part I – Evaluation of Seismic Capacity*, by Chang and Mander (1994a);
- *Seismic Based Fatigue Damage Analysis of Bridge Columns: Part II – Evaluation of Seismic Demand*, by Chang and Mander (1994b);
- *Fundamental Shear-Flexure Interaction Theory for Strength and Deformation Analysis of Structural Concrete Elements*, by Kim and Mander (unpublished report);
- *Design and Fatigue Analysis of Confined Concrete Columns*, by Dutta and Mander (1998).

3.12.4 Limitations

The moment-curvature method is valuable for studies aimed at deriving suitable prescriptive values for codes, but is time-consuming for routine design-office use.

Although the parameter study covered a wide range of parameters, it is still necessary to perform an investigation to explore the probabilistic variation of each parameter in order to establish appropriate overstrength factors for design. The present study developed the analytical tools for conducting such an investigation.

The design example used an uncracked-section moment-of-inertia for the columns because the section was assumed to be partially prestressed. This either needs to be clarified in the report, or the example should be reworked for a non-prestressed concrete column using cracked-section properties, as would typically be done in practice.

3.12.5 Impact

Code Requirements: The upper-bound overstrength factors developed in this study tend to validate some prescriptive overstrength factors (ATC-32) but also indicate that others (Caltrans and AASHTO) may be too low in some cases. The study also demonstrates that these prescriptive factors can be overly conservative for some columns.

The moment-curvature method developed in this study could also be used for design. In addition it would be useful in the development of design specifications. The effect of uncertainty in material and geometric properties needs to be evaluated so that the appropriate load and resistance factors can be developed for reliability-based seismic design codes.

Design Procedures: The moment-curvature method discussed in the report is lengthy for routine design use. Practical use will depend upon development of reliable computer programs, since possibly tedious iterations are required. The simplified method could be easily applied.

Design Costs: The simplified procedure for determining moment overstrength would result in a small increase in design costs that would be more than offset by savings in construction cost.

Structure Performance: Use of either the moment-curvature or simplified procedures would ensure adequate protection against brittle shear failures and thus reliable formation of desired energy-dissipating mechanisms, provided that the probabilistic variation in material properties is adequately addressed in the final design procedures.

Construction Costs: This study developed a simple hand calculation method for accurately calculating the moment overstrength factors for confined reinforced concrete columns. If widely adopted, more designers may calculate overstrength instead of relying on prescriptive values. This in turn could lead to significant construction savings.

Research Recommendations: The analytical tools developed in this project should be used to incorporate uncertainty estimates into the proposed design procedures for determining overstrength factors.

3.13 Seismic-Energy-Based Fatigue-Damage Analysis of Bridge Columns: Part I – Evaluation of Seismic Capacity

Chang, G.A. and Mander, J.B.

Technical Report NCEER-94-0006

Task No. 112-D-1.2

3.13.1 Research Summary

This study resulted in development of a computer program, UB-COLA, that is capable of accurately predicting the behavior of reinforced concrete columns subjected to inelastic cyclic deformations. The program models axial, flexural, and shear behavior of columns under cyclic loading, as well as the low-cycle fatigue properties of reinforcing steel and high-strength prestressing steel. The program was capable of predicting the failure mode of either low-axial-load columns (low-cycle fatigue of longitudinal reinforcement) or high-axial-load columns (fracture of confining reinforcement and crushing of concrete). For shear-critical columns, the cyclic inelastic behavior was simulated through the cyclic inelastic strut-tie modeling technique.

3.13.2 Research Findings and Review Comments

The report covers enhanced modeling of steel and concrete and the development of a computer program to calculate the moment-curvature and force-deflection characteristics of a reinforced concrete column.

A universally applicable model was developed for the hysteretic behavior for all types of steel including high-strength steels, which are being used more frequently. A method for assessing the degradation of steel was implemented in the model. This characteristic is important since it also influences the degrading characteristics of a reinforced concrete member. An algorithm for steel fracture behavior due to low-cycle fatigue was also developed and incorporated in the model. Correlation with experimental

results showed that both the hysteretic characteristics and low-cycle fatigue fracture of steel could be accurately simulated by the models.

The study developed a refined model to simulate the hysteretic behavior of confined and unconfined concrete in both cyclic compression and tension. Particular attention was paid to the nature of degradation within elements with partial reinforcement looping and also to the transition between the opening and closing of cracks. The cyclic model for concrete accurately described the experimentally observed hysteretic behavior, although it should be noted that there is only limited experimental data on cyclic tension behavior. Using the refined models for steel and concrete, a computer program called UB-COLA was developed to obtain the moment-curvature and force-displacement response of structural concrete columns using a fiber-element approach.

The main objective of the program is to perform analytical rather than physical experiments to obtain the relevant input information for nonlinear dynamic analysis programs such as IDARC and DRAIN-2DX.

Excellent correlation was obtained between a wide range of experimental results and those obtained from the UB-COLA computer program.

The computer program is capable of predicting the following modes of failure:

- Low-cycle fatigue of longitudinal reinforcement
- Fracture of transverse hoops
- Buckling of longitudinal compression reinforcement
- Shear failure when the concrete struts crush

3.13.3 Related Reports

Following are related MCEER technical reports:

- *Seismic-Energy-Based Fatigue-Damage Analysis of Bridge Columns: Part II – Evaluation of Seismic Demand*, by Chang and Mander (1994);
- *Seismic Design of Bridge Columns Based on Control and Repairability of Damage*, by Cheng and Mander (1997).

3.13.4 Limitations

This work is ready for implementation. There are items that can be incorporated in future work but these are refinements or enhancements rather than limitations of the current work.

3.13.5 Impact

Code Requirements: Although this work will not have a direct impact on codes, it provides a key element for the implementation of the pushover method of analysis.

Design Procedures: The UB-COLA computer program developed as part of this research will become a valuable design tool for design offices.

Design Cost: The pushover method of analysis, when required, will be more complex than current methods of analysis. The UB-COLA program will reduce design time when the pushover method of analysis is used.

Structure Performance: Structure performance would be more accurately predicted and critical modes of failure avoided.

Construction Cost/Constructibility: It is not expected that this report will have any significant impact on the cost of construction.

Research Recommendations: Additional research topics on this subject are refinements of the work described. These refinements are as follows:

- The nature of the cyclic behavior of concrete, with incursions into tension and compression regimes, needs to be established. Limited experimental information exists regarding the cyclic behavior of concrete, especially in tension.
- The fatigue model needs to be calibrated with additional experimental results to reliably establish the parameters.
- Well-designed experiments to assess shear deformation and crack formation are needed to validate or refine the proposed cyclic inelastic strut-tie model.
- The fiber-element analysis in its present form is curvature-controlled. That is, for a given curvature, the moment, and hence shear, is assessed. The inelastic shear strain is then determined from a force-controlled (shear-controlled) algorithm. This process works well except for columns failing prematurely in shear. It is therefore recommended that an inverse form of the solution be explored for such shear-critical elements, in which the response is perhaps controlled by shear strain.
- A study of the interaction between orthogonal cracking and yielding during biaxial flexure is needed.
- A modified shear model for assessing shear deformation on biaxial shear needs to be developed.

3.14 Seismic-Energy-Based Fatigue-Damage Analysis of Bridge Columns: Part II – Evaluation of Seismic Demand

Chang, G.A. and Mander, J.B.

Technical Report NCEER-94-0013

Task No. 112-D-1.2

3.14.1 Research Summary

This study (the second of two parts) deals with determining energy and fatigue demands on bridge columns. A smooth, asymmetric, degrading, hysteretic model (the Takeda model) is presented, capable of accurately simulating the behavior of bridge columns. The parameters for the analytical model are determined automatically by using a system-identification routine. The model was integrated into a SDOF inelastic dynamic analysis program, and a significant number of nonlinear analyses were performed.

The study resulted in design recommendations regarding the assessment of fatigue failure in reinforcing steel, based on the results of the nonlinear dynamic analyses. A method for the seismic evaluation of bridge structures is proposed, which incorporates the traditional strength and ductility aspects of reinforcing steel, plus the fatigue demand. It is shown that the current use of force-reduction factors that are independent of natural period in design codes is unconservative for short-period stiff structures and may lead to fatigue failure of the reinforcement. Recommendations are made for minimum values of force-reduction factors that prevent fatigue failure in the reinforcement.

3.14.2 Research Findings and Review Comments

The study developed a modified version of the Takeda model with improvements in the modeling of local cycling. The model was calibrated with full-size column experiments performed in earlier work by Mander. The Takeda model was refined by using continuous smooth curves rather than piece-wise linear curves. A system-identification routine was developed for the selection of suitable parameters for this refined Takeda model of a specific reinforced concrete column.

Excellent agreement was achieved between the output results from the refined Takeda SDOF model and the experimental hysteretic behavior of full-sized bridge piers. Good correlation was also achieved between the refined Takeda model and the analytical behavior determined by the more refined fiber-element model (UB-COLA). The refined Takeda model is suitable for inclusion in nonlinear dynamic analysis programs such as IDARC and DRAIN-2DX.

The study evaluated the spectral demands, including low-cycle fatigue and energy, from six recorded ground motions on a limited number of full-size bridge piers. *Although these runs were insufficient to develop design recommendations for spectral demands, they were useful in indicating trends.* It was concluded that the low-cycle fatigue demand is both earthquake-dependent and hysteretic-model-dependent. As a consequence, further sensitivity studies are needed to determine spectral fatigue demands for different types of columns and different ground motions.

A design procedure was developed that incorporates an assessment of the energy demand as it relates to the low-cycle fatigue of longitudinal and transverse steel. *The procedure depends on inelastic spectral demand and the study envelopes values obtained from the six ground motions included. For design purposes, these spectral demands need to be obtained from a greater range of column types and ground motion inputs.* Based on the limited results obtained from the study, a conservative force-reduction factor, R , was recommended to avoid low-cycle fatigue failures in reinforcing steel. They were

$$R_{\mu} = 10T$$
$$\text{and } 1 \leq R_{\mu} \leq 7$$

where T is the fundamental period. The design procedure demonstrated the need for a more rigorous assessment of low-cycle fatigue failure.

The study has shown that low-cycle fatigue failure is possible and may be critical in those cases in which ductility-based stability concerns do not govern.

3.14.3 Related Reports

Following are related MCEER technical reports:

- *Seismic-Energy-Based Fatigue-Damage Analysis of Bridge Columns: Part I – Evaluation of Seismic Capacity*, by Chang and Mander (1994);
- *Seismic Design of Bridge Columns Based on Control and Repairability of Damage*, by Cheng and Mander (1997).

3.14.4 Limitations

The design method developed as part of this work is a very important contribution to the development of an energy-based design approach for reinforced concrete members. Further work to establish spectral demands from a wider range of columns and ground motions is required before it could be adopted as a code design procedure.

3.14.5 Impact

Together, these two reports are important contributions in the search for an energy-based design procedure. The work has been developed for reinforced concrete structures, and it was further refined in a paper at the 1996 Convention of the Structural Engineers Association of California. The critical energy demand was related to the low-cycle fatigue failure of reinforcing steel in concrete columns. The work does need to be extended by additional sensitivity studies.

Code Requirements: The design procedure presented in the report would, as a minimum, be appropriate for inclusion in a code commentary to indicate how energy considerations can be incorporated in design, particularly low-cycle fatigue.

Design Procedures: A procedure is presented for energy-based design of reinforced concrete structures. It needs additional sensitivity studies before being incorporated in a code.

Design Cost: The energy-based design procedure, when adopted, will initially add complexity to the design process. However, once the procedure has been used and understood, it will not have a significant impact on design cost.

Structure Performance: Structure performance will be better understood and a critical mode of failure, not currently addressed in current codes, will be avoided. This will improve structure performance.

Construction Cost/Constructibility: It is not expected that the use of this procedure will have any significant impact on the cost of construction.

Research Recommendations: Additional research required to enhance the work performed to date is listed below.

- Inelastic energy spectra need to be generated for different types of structures and a larger range of ground motions.
- The macro Takeda model needs to be integrated into a general-purpose, nonlinear dynamic analysis program such as IDARC or DRAIN-2DX to study the effect of having realistically-calibrated models in a multiple-degree-of-freedom system.
- The effects of site-dependent earthquake excitation need to be addressed, specifically as they affect fatigue and energy demands.

3.15 Ductility of Rectangular Reinforced Concrete Bridge Columns with Moderate Confinement *Wehbe, N., Saïidi, M., Sanders, D., and Douglas, B.* *Technical Report NCEER-96-0003* *Task No. 112-D-5.1(d)*

3.15.1 Research Summary

The research addressed detailing guidelines for reinforced concrete bridge columns and walls in areas of moderate seismicity.

3.15.2 Research Findings and Review Comments

This study provided experimental and analytical investigations of the response of rectangular bridge columns with less transverse reinforcement than current code requirements. Four half-scale columns (610 mm by 380 mm, model scale) with 2.2 percent longitudinal reinforcement and either 0.33 percent or 0.44 percent transverse reinforcement (approximately one-half that required by current AASHTO guidelines) were tested under vertical load ratios of $0.1 f'_c A_g$ or $0.25 f'_c A_g$. Lateral loads were applied in strain-controlled cycles up to displacement ductility of between four and seven.

Analytical investigations used various concrete stress-strain models and the plastic-hinge-length models of Baker and Amarakone (1964) and of Paulay and Priestley (1992). The model of Baker and Amarakone (1964) appeared to give better results than Paulay and Priestley's model. Response was predicted with reasonable accuracy. Including shear deformation in the predictions enhanced the accuracy of the predicted ultimate displacement values.

The study concluded that displacement ductility of four to seven could be achieved with confinement ratios approximately one-half that required by current AASHTO specifications, provided that shear failure and premature buckling of the longitudinal bars were prevented. Failures were initiated by opening of the hoop bends in the plastic-hinge regions, followed by buckling of the longitudinal bars. The J hooks with a 90° bend were found to be ineffective for restraining the longitudinal bars.

3.15.3 Related Reports and Research

Report by Dutta and Mander (1998), *Capacity Design And Fatigue Analysis Of Confined Concrete Columns*.

3.15.4 Limitations

The study tested a limited number of specimens. Results were as anticipated. It would be interesting to predict the response of these specimens using the procedures of Dutta and Mander (1998).

3.15.5 Impact Assessment

Code Requirements: The confinement requirements may possibly be relaxed for areas of moderate-to-low seismicity.

Design Procedures: Procedures are available to quantify the available ductility when less transverse reinforcement is used than current code permits (See Dutta and Mander, 1998).

Design Costs: No direct impact on design costs is foreseen, although if it is necessary to show that sufficient ductility is provided by the reduced transverse reinforcement. A procedure similar to that proposed by Dutta and Mander (1998) would be required to accomplish this.

Structure Performance: Since the critical performance would be that associated with the upper-level seismic event, it would be necessary to show that the lower ductility levels achieved would be adequate to ensure survival of the columns.

Construction Cost/Constructibility: Adequate structure performance could be achieved with a reduced amount of transverse reinforcement, thus reducing reinforcement congestion and cost.

3.16 Capacity Detailing of Members to Ensure Elastic Behavior
Imbsen, R.A., Schamber, R.A., and Quest, M. (unpublished report)
MCEER Task No. 112-D-5.2a to -5.2c

3.16.1 Research Summary

The research addressed capacity details of members in which plastic hinges must not form. To ensure elastic behavior of these members and to force plastic hinges to occur in the columns, where intended, the capacity detailing is to be applied to superstructure longitudinal members, cap beams and footings

3.16.2 Research Findings and Review Comments

This report is primarily a compendium of current (1994) Caltrans practices.

The report covers four areas that have been identified by Caltrans as requiring new procedures for design. The four areas are:

- Joint shear in the cap-beam-to-column connection
- Superstructure flexural capacity to force plastic hinging in the columns
- Footings
- Outrigger and knee joint connections

Several recommendations are made in the report for design and detailing.

- Superstructure joint shear is limited to $12\sqrt{f'_c}$.
- Cap width is recommended to be 0.61 m greater than column width.
- No cut-offs are permitted for longitudinal cap steel.
- Hair-pin horizontal reinforcing requirements are given.
- Superstructure flexural capacity to ensure plastic hinging in the columns may require additional longitudinal reinforcing in the top and bottom slab and perhaps additional top-slab thickness.
- It is recommended that the mild steel reinforcing develop a minimum capacity equal to 1.2 times the cracking moment of the gross section in addition to the capacity produced by post-tensioning.
- Structures that may be sensitive to vertical seismic response include those with long spans (unspecified length), C-bents, and outriggers.
- Vertical capacity equal to 1.5 times dead load is recommended.
- Columns should be analyzed for overstrength using probable strength values as recommended by ATC-32 ($f_{ye} = 1.1 f_y$ specified, $f_{ce} = 1.3 f'_c$ specified, and $e_{cu} = 0.4 \%$) and $f_o = 1.4$.
- Transverse steel requirements are increased, and a factor accounting for the axial load ratio, longitudinal steel ratio, and number of longitudinal bars is included.

Additional recommendations were made for footing details.

- Limiting ratios for the projection of the footing beyond the column face relative to the depth of the footing ($L/d \leq 2.5$).
- Provide vertical ties (#5 at 30 cm in both directions) between the top and bottom mats of reinforcing.
- Provide hooks (turned outward) on the longitudinal column bars.

3.16.3 Related Reports and Research

Report by Mander, et al. (1998), *Capacity Design of Bridge Piers and the Analysis of Overstrength*

3.16.4 Limitations

Although the principle is good, the details presented may not be suitable for general use. They are pertinent to design of bridges in high-seismicity regions. If the selected energy-dissipation mechanism is plastic hinging in the columns, then the remainder of the structure must be detailed to ensure that this mechanism will be realized before the formation of undesirable modes of failure elsewhere in the structure.

The procedures of Mander et al. (1998) can be used to provide a more refined overstrength factor than the prescriptive values advocated in this report. *The recommendations for $f_o = 1.4$, along with the use of probable strengths, may overstate the requirements.*

3.16.5 Impacts

Code Requirements: Limitations on joint shear are directly applicable to code requirements. Specifications are largely empirical and in accordance with ATC-32.

Design Procedures: The general philosophy is useful in design procedures. For high-seismicity regions, the prescriptive details may be applicable.

Design Cost: Design cost is not thought to be adversely affected by these recommendations. Some additional checking of the ability of the superstructure to force plastic hinging into the columns may be required.

Structure Performance: Structure performance would be enhanced by the adoption of these recommendations.

Construction Cost/Constructibility: Unit reinforcing steel ratios for various elements in the example provided are approximately as follows:

Columns	270 kgm/m ³ (430 lbs/cy)
Footing	120 kgm/m ³ (188 lbs/cy)
Cross beam	190 kgm/m ³ (300 lbs/cy)

3.17 Capacity Detailing of Members to Ensure Elastic Behavior – Steel Pile to Pile Cap Connections

P. Ritchie and J.M. Kulicki (unpublished report)

Task 112-D-5.2(c)

3.17.1 Research Summary

The objective of this research was to identify connections between steel piles and concrete pile caps that would remain elastic during earthquakes. Both axial-load and moment-resisting connections were explored.

3.17.2 Research Findings and Review Comments

- Past design practice includes:
 - a 0.3-m (12") embedment of the piles into the pile cap;

- design of the pile cap as a bending member;
 - pile-cap reinforcing steel located 75 mm (3") above the top of piles;
 - equal spacing of pile-cap reinforcing; and
 - vertical piles assumed to carry axial loads only.
- Current design practice is changing. The LRFD code suggests using the strut and tie model to design most pile caps. Pile cap reinforcement may be located with 75 mm (3") of concrete in contact with soil. Consideration of passive resistance of soil on the sides of piles has led to the use of piles to resist lateral loads.
 - Pile connection forces should be determined using capacity design principles based on the overstrength of the supporting columns. Overstrength factors of 1.25 are used for steel sections. Concrete sections use overstrength factors of 1.3. (ATC-32 recommends 1.4).
 - Pile connections could be designed to resist the maximum capacity of the piles themselves, but it was felt that this may be too conservative.
 - Research has shown that the blunt end of steel H-piles is capable of transmitting compressive forces in the pile to the pile cap. Welded bearing plates are usually not necessary, but can be used to lower bearing stresses.
 - Even though 0.3 m (12") of embedment will provide some resistance to pull-out of the pile from the pile cap, it is usually ignored during design. Some form of mechanical anchorage should be used to resist uplift.
 - Steel bearing plates, shear studs or welded rolled shapes will provide uplift resistance, but the predominant failure mode will be a conical failure cone in the pile cap concrete. Because this is a brittle mode of failure, it is undesirable, unless the designer can be assured that only elastic response will occur.
 - In the past, uplift has been resisted by reinforcing "hairpins" that are run through holes cut in the pile.
 - The use of bent, straight or welded rebars that are welded to the flanges of the steel pile are felt to be preferable due to ease of design and construction. *Given the difficulty and expense associated with quality control of field-welded connections, the assessment that this detail is the easiest to construct is questioned. Some of the details shown in the report appear to require that pile cap reinforcement either be in place before this welding can take place, or be threaded through the welded steel bars, which will make construction even more difficult.*
 - Significant moment capacity can be developed by the 0.3-m (12") embedment that is typically used for steel H-piles.
 - The approximate moment resistance attributed to plain concrete is given by the following equation:

$$M_{concr} = 5\sqrt{f_c} \cdot d^3$$

where d is the depth of embedment. The equation should be used with care for pile groups because of the brittle failure mode and the uncertainty related to group effects.

- Moment capacity and ductility can be increased if the bottom mat of cap reinforcing steel is below the top of the pile or if spiral reinforcement is added. Because more research is needed to assess the increase in moment strength, it may be prudent at this time to use these details to gain an unspecified amount of added ductility and not rely on the detailing for any moment capacity increase.
- Moment resistance can be increased by welding vertical bars to the pile flanges, similar to the detail for increasing axial pull-out capacity, or by using horizontal cap reinforcement in conjunction with increased depth of pile embedment in the cap. *The combination of moment-resisting mechanisms is not clear from the report. For example, can the vertical bars and horizontal bars be used simultaneously and both be considered 100% effective?*

- An example problem is presented. *In the example problem, resistance derived from the horizontal bars is added algebraically to the moment resistance of plain concrete. This may be unconservative in light of the conclusion drawn above that more research is required to determine the effects of bottom mat steel and spiral reinforcement placed around the pile. It is not clear how this design assumption is justified.*

3.17.3 Related Reports And Research

Following are related MCEER technical reports:

- *Capacity Detailing of Members to Ensure Elastic Behavior*, by Imbsen et al. (1996);
- *Capacity Design of Bridge Piers and the Analysis of Overstrength*, by Mander et al. (1997).

3.17.4 Limitations

- The details presented in the report are limited to those used in certain states. For example, 0.3-m (12") embedment, pile bearing plates, and welded vertical bars are not typical California practice. Use of bent reinforcing "hairpins" placed through holes in the webs and flanges of piles is common in California and is generally preferred over welded bars for constructibility reasons. Also, other detailing, besides those shown, may be possible for developing moment capacity at the head of the pile.
- The report did not specifically address corner and edge piles, which may be critical when trying to develop moment fixity at the head of a pile. Also, there was no discussion of minimum distance between the edge of the pile cap and the edge piles; this distance may be a factor in developing a satisfactory pile connection.
- Steel pipe piles, which may or may not be filled with concrete, are becoming increasingly popular for seismic resistance. The report did not address this pile type.
- The applicability of the details to batter piles was not discussed, nor were the connection problems that could result from the use of batter piles in seismic zones.
- A discussion of the wide range of connection forces was not included. These forces could result, for example, from variability in the soil properties and thus the pile stiffnesses, accidental misalignment or batter of individual piles, batter piles whose capacity cannot always be protected, abutment and pier wall piles whose capacity is not always protected, and dynamic response of the pile cap itself.
- Although the example problems deal with the combination of some resisting mechanisms, there was no discussion of why certain design assumptions were made. In the absence of additional research, some reasonable recommendations should be made that err on the side of safety.

3.17.5 Impact Assessment

Code Requirements: The report presents some limited, common-sense, guidelines for designing elastic steel pile-to-pile-cap connections. However, there are some design-related issues that should appear in a design code or commentary that were not addressed. These guidelines should be subjected to a consensus review prior to being incorporated in a design code.

Design Procedures: The report covers some design procedures for steel pile-to-pile-cap connections. These are practical measures that are based on an extension of current code requirements. They do not reflect any extensive research on the subject and thus do not address many unanswered questions about the performance of pile connections.

Design Costs: Proper design and detailing of pile connections can often be standardized and should not add significantly to design costs.

Structure Performance: Use of adequate pile connection details is important to achieving desired performance during an earthquake. If connections fail prematurely and in a brittle fashion, they could lead to excessive displacements of the structure that could have undesirable consequences. Also, detection of a failed pile connection would be difficult and repair would be expensive if not economically infeasible in some cases.

Construction Costs: The details presented in the report will achieve the desired performance if properly designed, but they should be value engineered to identify the most cost-effective construction method. The use of field welding is expensive because of the need for rigorous quality control. Effective details that do not require field welding should be identified.

Most structures do not warrant the use of a fixed-head pile connection, but when they do it is important that details be carefully chosen to minimize construction cost. The reviewer is aware of a case in which a change order was written to modify steel H-pile connections to achieve a fixed-head condition of the piles. The details included the field welding of Nelson studs, and resulted in a 1.7% (~\$200,000.00) increase in the overall cost of the structure. This is a significant amount that warrants careful detailing of pile connections to ensure constructibility and to minimize cost.

3.17.6 Research Recommendations

- Laboratory testing of “pinned-headed” pile connections is required to assess the effectiveness of different details for preventing pull-out of the piles. If simply constructed methods can be shown to be effective, they could result in significant savings. Also, these tests could establish the effect of the connection on pile stiffness by determining the amount of unintended “fixity” that results from the various pile connection details.
- The report cites the need for research to determine the effect of the bottom layer of cap reinforcement and spiral reinforcement around the piles. It is felt that this reinforcement will result in an increased ductility and moment capacity of the pile connection, but the amount needs to be quantified.
- Larger piles may require the use of multiple resistance mechanisms to achieve moment “fixity” of the pile head. This could involve the use of both vertical and horizontal reinforcement in the pile-to-pile-cap connection. The relative effectiveness of this reinforcement needs to be determined through detailed analytical studies and laboratory testing. Because it may be acceptable to allow a limited amount of ductile behavior at the pile head, this research should include behavior in the nonlinear range. It should also include assessment of edge and corner piles and should investigate the need for maintaining a minimum amount of concrete between the pile and the edge of the pile cap.
- Design requirements and details for filled and hollow steel pipe piles should be developed similar to what was done for H-piles in this report.
- Batter piles are felt by some to be undesirable in a seismic event because they are perceived to have limited ductility and will attract large axial loads. The connection of these piles to the pile cap is often the key to their behavior. Some recent analytical studies conducted during the planning of the new segment of the San Francisco-Oakland Bay Bridge suggest that there may be significant economic advantages to using batter piles for seismic resistance under certain circumstances. More analytical and experimental research is required to understand fully the behavior and performance of batter piles during an earthquake and the piles’ sensitivity to various soil parameters. Seismic design guidelines for batter pile systems need to be developed.

- Large-diameter composite steel and concrete piles are being used on several large bridges in seismically active areas. Connection forces can be very large and elaborate detailing is required to achieve moment fixity at the pile head. A research program for these types of pile systems is needed. It should address the development and maintenance of composite action throughout a potential seismic event, the relative effectiveness of currently used and proposed pile-to-pile-cap connection details, design of pile caps, the advantages and disadvantages of batter piles, constructibility of these foundation systems, and the ductile behavior of composite steel and concrete piles.

3.18 Structural Steel and Steel-Concrete Interface Details for Bridges

Ritchie, P., Kuhl, N., and Kulicki, J

Technical Report MCEER-98-0006

Task No. 112-D-5.4

3.18.1 Research Summary

This research effort assessed seismic performance of details associated with steel bridge towers extending from a massive concrete substructure to the superstructure, as well as the seismic performance of other steel substructure and superstructure details for new construction.

Issues addressed include:

- Identifying the most ductile cross-sections
- Applicability of eccentrically-braced frames
- Details for repair of steel sections following an earthquake
- Performance of anchor bolts under lateral uplift loads
- Economical moment-connection details between steel superstructures and concrete substructures.

3.18.2 Research Findings and Review Comments

- Steel possesses desirable characteristics for seismic resistance: ductility and a high strength-mass ratio.
- Tubular sections have been identified as the most efficient members due to their self-stiffening nature and their optimal resistance to both local and general buckling. The width-thickness ratio for plates needs to be reduced from commonly accepted values in order to ensure adequate cyclic performance.
- Moment-resistant frames (MRF), concentrically-braced frames (CBF), and eccentrically-braced frames (EBF) can all supply adequate seismic resistance, provided that proper detailing is employed. There appears to be no reason why eccentrically-braced frames cannot be successfully employed as the lateral bracing systems in bridges. Energy-dissipation mechanisms can be incorporated into bracing systems so that the inelastic action in these mechanisms will not degrade the gravity-load-carrying capacity of the main members. Good design practice both directs the forces to those locations that can accommodate them and limits displacements. Connections and splices are critical locations in bridges, and plastic hinging must be forced into the members as opposed to the connections and splices. Structural fuses that use friction, viscous materials, and material yielding can provide effective energy dissipation when placed in parallel with the bracing system. Material-yielding devices avoid environmental exposure problems and are thus attractive.
- Member repair and replacement schemes are effective when incorporated into the original design.
- Embedment of anchor bolts must receive close attention during design, and properly embedded bolts provide the best means of connecting steel to the concrete substructure. Sometimes embedment of

the member itself can be an effective repair or retrofit method. Concrete filling can be used effectively to prevent local buckling.

- Shear in the panel zone is preferred to flexural yielding as an energy-dissipating mechanism for steel members. This is the opposite of the situation for concrete members.
- The difference in fundamental behavior between steel bridges and concrete bridges should be recognized by the code. Steel bridges can incorporate multiple energy-dissipation mechanisms distributed throughout the structure, whereas concrete bridges generally depend on energy dissipation through plastic hinging at the top and bottom of the columns.
- A table of recommended force-reduction factors for steel bridges is included in the report. *It is difficult to understand how the type of superstructure could influence the force-reduction factor for the substructure element since the force-reduction factor is only used to reduce the forces used to determine the required moment capacity of the plastic-hinge region in the substructure.*

3.18.3 Related Reports And Research

Task 106-E-5.5: Critical Seismic Issues for Steel Bridges

3.18.4 Limitations

The research seems to be primarily based on the work of one researcher, although reference is made to unpublished Japanese research in progress.

Providing low-cycle fatigue values for steel sections, similar to Mander's work in reinforced concrete columns would be helpful. A value for rotational ductility of six to eight for steel plastic hinges is implied in the proposed k_p values.

3.18.5 Impacts

Code Requirements: Width-thickness ratios for steel sections to prevent local buckling can easily be codified. Force-reduction factors could be adopted for sizing of bracing details or other energy-dissipating mechanisms. Adoption of these factors would recognize the distributed energy dissipation available with properly designed steel bridge systems, provided that rules are included for the total available energy dissipation for the steel details. There is no justification, however, for changing the force-reduction factors for the substructure elements based on energy-dissipating elements in the superstructure.

Design Procedures: No specific design procedures were recommended. The impact would be on designing suitable energy-dissipating mechanisms. Procedures for design of eccentrically-braced frames could be adopted from building design practice.

Design Cost: Not affected, except as noted above for design procedures.

Structure Performance: Retrofit of braced members may become more frequently necessary in retrofit situations if ductile performance is required. The total amount of energy dissipation must be adequate to provide for the force-reduction factors assumed.

Construction Cost/Constructibility: Impact should be minimal. Constructibility and repairability should be enhanced by proper design and detailing.

Research Recommendations: Materials-based research is required to quantify low-cycle fatigue behavior for different steel sections and for bracing details intended for use as energy-dissipating mechanisms. Research on high-strength steels ($f_y \geq 345$ kPa (50 ksi)) is necessary. Research is needed to quantify the total energy dissipation required in a structure.

3.19 Structural Details to Accommodate Seismic Movements of Highway Bridges and Retaining Walls

Imbsen, R.A., Schamber, R.A., Thorkildsen, E., Kartoum, A., Martin, B.T., Rosser, T.N. and Kulicki, J.M

Technical Report NCEER-97-0007

Task No. 112-D-5.3(a&b)

3.19.1 Research Summary

This report describes structural details being used on bridges and retaining structures in the eastern and western United States to accommodate structural movements during seismic events. The research addressed the development of seismic design recommendations and details for bridges and retaining structures based on the need to accommodate these structural movements. The resulting recommendations and details are intended to be used as a basis for developing improved bridge design standards.

The report includes many illustrations of the devices used to accommodate movements. Advantages and disadvantages of some of the devices are noted. Examples of approaches used in specific states are also given. An appendix to the report presents two cases showing how substructure flexibility can be accounted for in the modeling of an isolation system.

3.19.2 Research Findings and Review Comments

Specific consideration is given to the following topics: (1) restraining devices, (2) sacrificial elements, (3) passive energy-dissipating devices and isolation bearing systems, (4) minimum support-length requirements, (5) earth-retaining structures, and (6) the effects of column flexibility when using isolation.

Longitudinal Restrainers at Joints

- Longitudinal joint restrainers have been used since 1971 in California, but their use in the eastern and central United States is more recent. Eastern states prefer to provide adequate seat width, and generous movement-capacity bearings rather than using restrainers.
- Modeling joint restrainers is difficult. Simplified design rules have been developed by some states (e.g., California and New York) and by FHWA. The equivalent static analysis method, developed at Caltrans and summarized in the 1995 FHWA report *Seismic Retrofitting Manual for Highway Bridges* (Buckle and Friedland, 1995), is recommended.
- A new hinge restrainer design for multiple-frame bridges is noted as being studied by Caltrans. Linkage slabs are identified as an alternative to longitudinal restrainers.
- Examples of longitudinal restrainers used in California, New York, New Jersey, and Pennsylvania are discussed.

Vertical Motion Restrainers

- AASHTO and Caltrans provide design specifications. Vertical restrainers are usually not economical unless the bridge is Seismic Category D; consequently they are typically not needed in the eastern United States.
- Vertical restrainers are inexpensive but are usually difficult to install. The report recommends not specifying vertical ties as standard if they are not required.

Shear Keys

- AASHTO specifications imply that the shear key must transmit the entire force across a joint. This requires the substructure to withstand the full seismic force. Caltrans deviates from this policy by limiting the transferred force, thus reducing the likelihood of damage to the substructure.
- The two chief failure modes for reinforced concrete shear keys are (1) a direct shear failure at the horizontal interface between the key and the abutment seat, and (2) a separation between the key and the abutment stem. The second mode of shear key failure was common in the 1994 Northridge earthquake. *Drag steel in the stem below the seat is required.*
- Examples of shear keys used in California, Massachusetts, Pennsylvania, Michigan, and New York are shown. Reference is made to the use of “stopper blocks” in California and Japan.

Integral Connection: Substructure-to-Superstructure

- Bridges outside western United States usually do not have a fixed connection between the superstructure and the substructure. If a plastic hinge forms at the column base, instability can occur. A fixed connection would greatly reduce seismic displacements.
- University of California at San Diego is exploring the benefits of integral bent cap connections for precast concrete bridges. Steel bridges have also been built with integral bent caps using transverse post-tension across the bent cap.

Sacrificial Elements for Protecting Bridge Abutments

- Caltrans designs their abutment walls to fail during an extreme event with the intent of protecting the substructure. Failure of the abutment during a major earthquake could close traffic. *Joint damage may occur during small events.*
- Knock-off devices that use a pre-formed joint located at the top of the abutment backwall have been used. These devices can be inspected after an earthquake. Damage may occur during smaller events. Examples of these details are provided.
- Another alternative is to provide a sacrificial joint. Several bridges in California were retrofitted using this approach following the Loma Prieta and Northridge earthquakes. One problem is providing for motion in the transverse direction. Joints can be designed with repairable sacrificial damage features, or they can be designed to handle transverse displacement.

Energy-Dissipating Devices

- Various types of energy-dissipating devices have evolved, including yielding steel dampers, lead-extrusion dampers, friction dampers, hydraulic dampers, viscoelastic dampers, and isolation bearing systems. These systems differ in their mode of dissipating energy.

- Typical applications of energy-dissipating systems on bridges are identified. The locations of the systems include (1) span hinges, (2) superstructure-to-substructure connections, (3) columns, (4) tower-deck connections, (5) superstructure-abutment connections, and (6) steel truss diagonals.
- A discussion of the use of energy-dissipating devices on bridges in New Zealand, as well as a number of states (California, Illinois, Kentucky, Missouri, New Hampshire, and Oregon) is given.

Minimum Support-Length Requirement

- Procedures given in AASHTO for determining minimum seat width are reviewed, including modifications to account for skew angle. The formula for skew is being adopted in several states. The code does not currently provide any guidance for support lengths in the transverse direction.

Earth-Retaining Systems

- There are few guidelines for the design of earth retaining systems. Most states in the central and eastern United States do not consider seismic loading with respect to retaining systems. Rather, it is assumed that the static factor of safety provides sufficient reserve capacity. For those states that do include seismic loads, the pseudo-static Mononobe-Okabe method is used to estimate soil pressure.
- Free-standing walls have generally performed well due to their flexibility. Most wall damage has involved walls that are an integral part of the bridge substructure or in proximity to the bridge. Some bridge wingwalls have been damaged in previous earthquakes. Settlement of earth materials behind the wall has also occurred.
- There have been minimal reports of damage to anchored walls. The main concern for these walls is design of the anchorage. The Mononobe-Okabe active pressure method is used to estimate wall pressures. Soil nail walls and mechanically stabilized earth (MSE) have also performed successfully during past earthquakes. The flexibility of the MSE wall may result in densification of the soil; therefore, Caltrans requires piling support of bridge abutments that use MSE walls to retain earth.
- Prefabricated modular walls have not had significant damage in past earthquakes, but their limited ability to tolerate large differential displacements is of concern.

3.19.3 Related Reports and Research

None

3.19.4 Limitations

- This report is a catch-all for issues and details not covered elsewhere. Yet some items, such as approach slabs, have been omitted. Therefore, it is not an inclusive checklist. Considerable information on many of Caltrans practices is described, some of which could be used in a design specification. Very little information for other jurisdictions is given. The authors responded to this comment by noting that the intent of the details in the report was to show what is typically used in high and low seismic zones. The intent was not to provide a comprehensive study.
- Linkage slabs would not work at Caltrans bridge hinges because they don't handle thermal effects.
- Typical vertical cable restrainer details can tolerate limited horizontal displacement at the joint. Alternative solutions to uplift problems are not discussed in the report, such as revising the span arrangement, use of ballast, or other types of restrainers. Pipe and sleeve restrainers can be designed for greater displacement.
- There is no discussion of approach slabs and their effect on the seismic performance of bridges. These slabs could be used as part of the structural system.

- Modular joints are considered a maintenance problem by Caltrans: They prefer to minimize joint maintenance and plan for repairs after major events.
- Lock-up type energy-dissipating devices are not discussed.
- Consideration of differences between SPCs is not given in the report for all items.

3.19.5 *Impact*

Code Requirements: This report covers a variety of items and many practical aspects of bridge details that need to be considered in the seismic design process. While the report is a valuable summary, few recommendations are provided on how to use this information in developing new AASHTO seismic design provisions. Some of the information in this report could be used in a commentary.

Design Procedures: Some of the information included in this report could eventually result in changes to current design procedures. The effect of these changes is not expected to be significant.

Design Costs: Information included in this report is not expected to result in significant changes in design costs.

Structural Performance: Some of the suggestions made in this report could result in better structural performance. Since the report relies heavily on procedures that are currently being used by Caltrans, areas outside of California are expected to benefit most by these suggestions.

Construction Cost/Constructibility: Information given in the report could result in higher construction costs, particularly if methods currently used in California are incorporated in the new AASHTO seismic design provisions.

Research Recommendations: Many of the topics warrant additional research.

3.20 **Derivation of Inelastic Design Spectrum**

Liu, W.D., Imbsen, R., Chen, X.D., and Neuenhofer, A. (unpublished report)

Task No. 112-D-1.3

3.20.1 *Research Summary*

The research addressed inelastic response spectra that allow designers to assess inelastic deformation demands, which will lead to the improved seismic performance of new bridge construction. The goal was to derive inelastic design spectra for nationwide use; i.e., for different seismic environments and for different site soil conditions and to account for the scattering and variability that exist in real earthquake ground motions and in nonlinear structural responses.

3.20.2 *Research Findings and Review Comments*

A computer program (WES-RASCAL) was used to adjust a family of recorded ground motions for rock and stiff soil sites to be compatible with the median five-percent-damped response spectrum. The variability in nonlinear response was greatly reduced because of the spectrum-matching process that was used. *It is not clear how the target median spectrum was obtained or whether and how it related to response spectra data recently developed and mapped by the USGS (see Section 3.1.2 in this report).*

An Inelastic Response Amplification Factor (IRAF) is proposed that would modify elastic spectral displacements to give inelastic displacement response. The authors propose to simplify the IRAF for design by making it a function of structure period, damping, strength coefficient, and the soil type at the site. *The IRAF seems to be very similar in concept to the displacement amplification factor, R_d , used in ATC-32.*

Four inelastic spectra were produced for each of three classes of earthquake. These include median acceleration, median displacement, median peak displacement ductility demand (for various strength coefficients), and IRAF spectra. For the cases studied, results show that IRAF approaches unity for long period structures, *as is expected*. The inelastic response results were shown to depend on the magnitude and distance of the earthquake.

The report also considers Normalized Hysteretic Energy (NHE) spectra. These spectra, which reflect the cumulative inelastic response, are understandably more severe for large magnitude events that tend to have longer durations and thus can result in a greater cumulative energy demand.

3.20.3 Related Reports and Research

Following are related MCEER technical reports:

- *Seismic-Energy-Based Fatigue-Damage Analysis of Bridge Columns: Part I – Evaluation of Seismic Capacity*, by Chang and Mander (1994);
- *Seismic Design of Bridge Columns Based on Control and Repairability of Damage*, by Cheng and Mander (1997).

3.20.4 Limitations

The report does not fulfill all of the stated objectives of the research, and it does not explain how the methodology would incorporate the large amount of data prepared by the USGS on the nationwide seismic hazard. It would seem that a considerable effort is required to develop simplified inelastic spectra for nationwide use.

The research is purely theoretical, and no evidence is presented to demonstrate that this method will yield accurate results for real bridges or that it is superior to other methods that have already been proposed (e.g., ATC-32). An approach using a normalized hysteretic spectrum approach is suggested, but no guidance is given on how it might be used.

3.20.5 Impact

Code Requirements: The subject matter covered by this research has the potential to make a major impact on the seismic design code requirements for bridges. This work has not progressed to the point that any immediate impact can be expected, however.

Design Procedures: If successfully developed, inelastic spectra will be one of the elements of displacement-based or energy-based design procedures. Much more work needs to be done, however.

Design Cost: Except for the substantial initial cost of learning a new design methodology, adoption of displacement- or energy-based design procedures need not increase design costs significantly.

Structure Performance: Structure performance will be more accurately predicted if suitable inelastic design spectra are part of the design procedure. This could improve structure performance in some cases.

Construction Cost: It is not expected that adoption of displacement- or energy-based procedures will have a significant impact on construction costs.

Research Recommendations : Research to develop suitable inelastic design spectra should be continued. It must be coupled with development of analysis and design procedures that can make proper use of these spectra, however. Guide design specifications should be developed that implement a displacement- and/or energy-based approach to seismic design. These methods should be available to practitioners as an alternative design approach for a period of time until their impact on the design process can be evaluated.

3.21 Summary and Evaluation of Procedures for the Seismic Design of Tunnels
Power, M.S., Rosidi, D., Kaneshiro, J., Gilstrap, S.D., and S-J Chiou (unpublished report)
Task No: 112-D-5.3c

3.21.1 Research Summary

This report reviews procedures used for the seismic evaluation and design of tunnels. The report covers three types of tunnels: bored tunnels, cut-and-cover tunnels, and submerged tunnels. The research addressed the following: (1) compiling and evaluating empirical data and correlating this information with the seismic performance of tunnels, (2) synthesizing methods for seismically analyzing tunnels, and (3) summarizing design strategies and details to improve the seismic performance of tunnels.

3.21.2 Research Findings and Review Comments

The report begins with a description of tunnel classifications and a summary of factors influencing the seismic performance of tunnels. This information is followed by discussions of empirical observations and correlations related to the seismic performance of tunnels, analytical procedures for the seismic analysis of tunnels, and design strategies to consider for the seismic design of tunnels.

Classification System and Factors Influencing Seismic Performance

- Tunnels are categorized as being bored tunnels, cut-and-cover tunnels, or submerged tunnels. The bored tunnels are further identified as either soft-ground tunnels or rock tunnels. The cut-and-cover tunnels are normally excavated from the ground surface and therefore are located at relatively shallow depths (<50 feet). Submerged tunnels are typically prefabricated and then placed in excavated trenches on a river or sea bottom.
- Factors influencing the seismic performance of a tunnel are identified. These factors include (1) intensity of ground shaking, (2) fault rupture, (3) landsliding, and (4) soil liquefaction. Geological conditions and tunnel construction methods also will influence seismic performance. Each of these factors is briefly discussed.

Seismic Performance Observations and Correlations

A number of studies involving correlations between acceleration and level of damage have been completed, beginning in the late 1950's. The report describes a detailed review of these studies to develop a general correlation between seismic ground shaking level and tunnel performance. The

correlation is for bored tunnels only, and does not include the effects of ground failure, such as might result from faulting, liquefaction, or slope instabilities. The level of damage to bored tunnels due to ground shaking is shown to depend on peak ground acceleration (pga) and liner type. Observations following seismic events typically show that damage is minimal if the level of ground acceleration is below 0.2g. Only a few cases of slight-to-heavy damage are found from 0.2 to 0.5g, but when the pga exceeds 0.5g there are a number of instances of slight-to-heavy damage. Tunnels with reinforced concrete and steel liners generally perform better than unlined, or timber- and masonry-lined tunnels.

- The report discusses the performance of cut-and-cover tunnels, which have performed poorly relative to bored tunnels. This is attributed to softer ground, higher accelerations, and vulnerability to racking. Cut-and-cover tunnels in soil tend to be more vulnerable than those excavated in rock because of the larger soil shear deformations, which cause tunnel racking.
- Results of a recent study by O'Rourke and Shiba (1997) are also described. Information from the O'Rourke and Shiba study suggests that the portals were the most often location of damage, and this damage was often due to landslides. Severe damage from fault movements is also mentioned by O'Rourke and Shiba.

Seismic Evaluation Procedures

- Procedures for evaluating the effects of ground shaking are given. Two principal types of deformation are of concern: (1) deformation along the axis of the tunnel which includes both axial and curvature deformations, and (2) deformation perpendicular to the tunnel axis in the plane of the tunnel cross section. Figure 3-2 (copied from Figure 5 of the report) shows tunnel response for the two cases.
- Simplified, closed form and numerical analysis methods for estimating axial and curvature deformations along a tunnel are discussed. During simplified analyses, key considerations include the determination of the apparent wave propagation velocity and the attenuation of ground motion with depth. Guidance is provided in both areas. For soft ground conditions, it is also shown the surface waves (Rayleigh waves) may result in more severe loading than compressional (P-) and shear (S-) waves. *For the surface wave analysis, consideration should be given to the use of effective shear modulus, similar to the approach for ovaling analysis. If so, you might want to mention.* For numerical analyses, reference is made to computer codes such as ADINA and ABAQUS for making these analyses.
- Methods for determining the deformation of the tunnel cross section are also discussed. These methods also involve both simplified, closed form and numerical analyses. Discussions are given for both bored tunnels and cut-and-cover tunnels. For cut-and-cover tunnels, both force and deformation methods of design are summarized.
 - For bored tunnels pseudo-static equilibrium methods are identified as a method for determining the potential for caving into the tunnel. It is noted that this approach is conservative, as the destabilizing forces act for short instances of time. *This also implies that the peak ground acceleration, rather than 1/2 or 2/3rds the peak, should be used in the analysis.* Methods for analyzing the free-field compressive and shear stress, as well as ovaling of flexible tunnels for lined and unlined conditions are also given. For ovaling, diameter change, thrust force, shear force, and bending moment are shown to be a function of the maximum shearing strain in the free field, and the relative stiffness of the geologic medium and liner. Guidance is provided for the determination of maximum shearing strain. This guidance includes the effects of material non-linearities. *It would be of value to the profession if some general rules-of-thumb could be given to indicate the relative amount of ovaling that might be expected. For example, because it appears that the diameter change will be roughly proportional to the shearing strain in the soil, identifying typical values of shearing strain for earth materials would be useful.*

- The report notes that for cut-and-cover (rectangular) tunnels simplified force methods are most often used to estimate wall loads. These methods for estimating lateral pressures are only reasonable for depths of embedment to the top of the tunnel of less than one-half the tunnel height. The simplified equation proposed by Seed and Whitman (1970) for computing the dynamic increment of lateral earth pressure is identified as being reasonable for shallow cut-and-cover tunnels in soil. This equation assumes that the wall yields enough to develop active conditions within the soil. Equations that consider the wall as rigid and non-yielding, based on the formulation of Wood (1973), should be used if the tunnel walls retain soil, but the structure is founded on rock. Issues of vertical and traction forces on the roof of tunnels are also addressed.

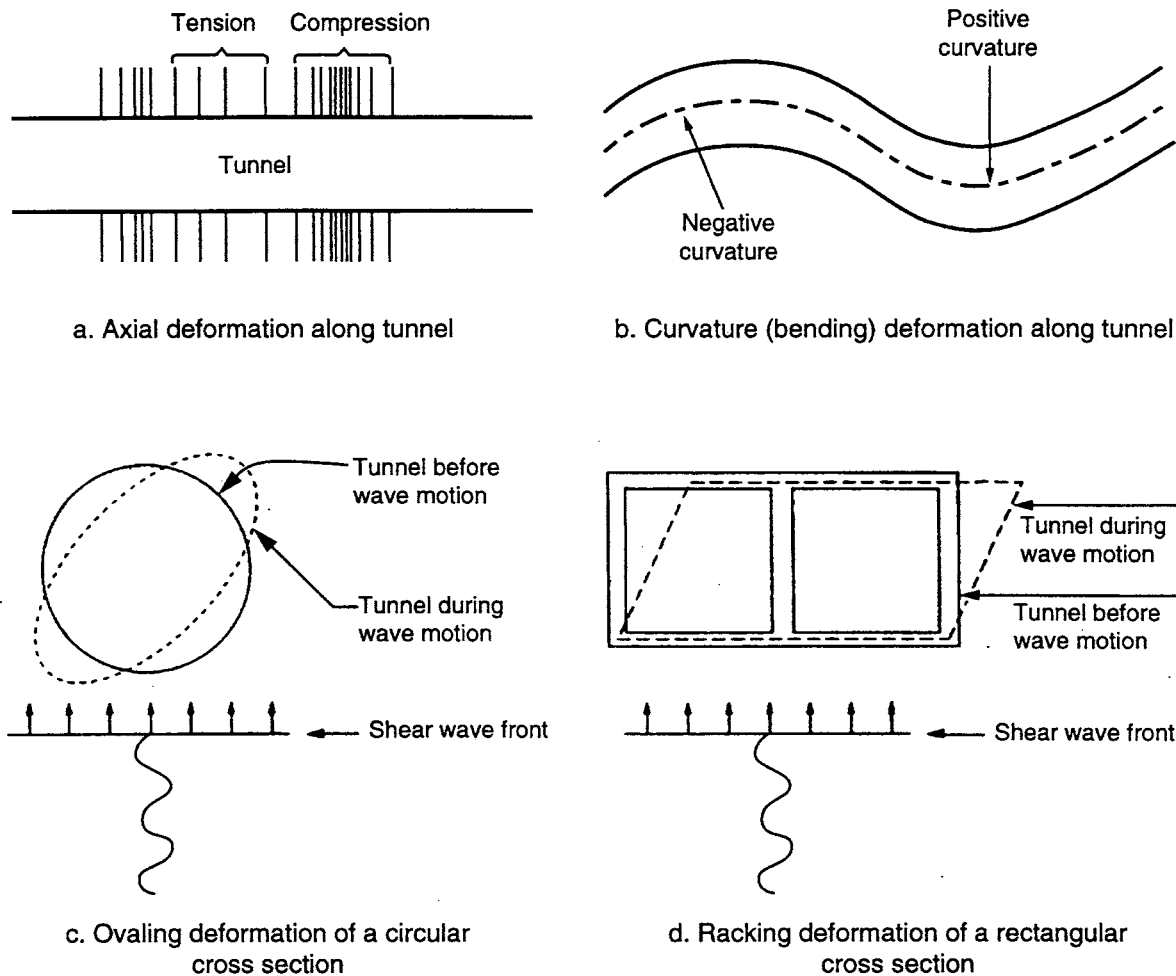


Figure 3-2 Tunnel Response to Seismic Waves (after Wang, 1993)

- For cut-and-cover tunnels simplified deformation approaches have also been developed. These methods depend on the relative flexibility between the soil and structure, and the estimated free-field shear deformation. Results are presented in terms of a racking coefficient, which relates the distortion of the structure to the free-field distortion. Pseudo-static force methods are then used to determine the forces necessary to produce the same racking distortion. *If the racking coefficient was derived for specific conditions, the general applicability of this coefficient may be limited. More explanation also might be appropriate regarding the determination of the flexibility ratio. Other questions of interest to the design professional include: Is the shear*

modulus reduced for strain amplitude effects. How is the force required to cause a unit racking deflection determined, and how does this relate to the seismic-induced ground motion?

- Two-dimensional numerical codes, such as FLUSH, SASSI, and FLAC can also be used to analyze either bored or cut-and-cover tunnels. These methods are often appropriate because of the complexity of the problem. This is especially thought to be the case for cut-and-cover tunnels because of the lack of vulnerability information from past earthquake observations. *Some of the computer programs identified are relatively expensive and application requires specialized expertise in numerical analysis. One such example is the Discrete Element Program, UDEC, referenced in the report and recommended for analysis of tunnel cross sections where the presence of weak planes in the geologic media control the seismic response of the system. While these discussions focused on bored and cut-and-cover tunnels, it was not clear which of the approaches should be used for evaluating submerged tunnels. Guidance on an appropriate approach would be useful.*
- Guidance is also provided for fault evaluations. The effects of faulting depend on the direction of slip, the width over which slip occurs, and the amount of slip. Empirical equations exist for estimating the amount of slip. Procedures for evaluating tunnels subjected to fault displacements typically follow those used for buried pipelines. However, the report concludes that the “Newmark-Hall Method” and the “Kennedy and others procedure”, often used to evaluate buried pipelines, are not applicable or are too conservative to be used for tunnels. The report explains that Finite Element Analysis is recommended for evaluating the induced shear and bending stress within the tunnel lining and accounting for the effects of soil-structure interaction.
- During landsliding and liquefaction, the amount of deformation and the change in earth pressures must be considered. The report notes that the potential for liquefaction to cause uplift of the tunnel should also be checked.

Design Strategies

- Design strategies to accommodate ground shaking are discussed. For tunnels in rock these strategies include selection of construction and support methods that limit the amount of deformation and enhance the ability of the ground to arch. The need to have the lining in good contact with the ground is noted. For soft ground conditions it is desirable to reinforce concrete liners. Bolted segmental liners are also reported to perform well during earthquakes. The key consideration for cut-and-cover tunnels is accommodating racking and vertical response. For submerged tubes guidance is provided to mitigate liquefaction flotation, as well as excessive shear and tensile forces. Mention is also made of detailing requirements at transitions to relatively stiffer structures and at abrupt changes in geologic media.
- A number of strategies are identified for crossing faults. These typically involve an enlarged tunnel section within the shear zone of the fault. In areas where fault movement is expected to be small, articulated liners are suggested as a possible design approach.
- For landsliding and liquefaction various ground stabilization methods might be used. If liquefaction is expected, an alternate design strategy is to strengthen the walls of the tunnel to withstand the increased pressure from the liquefied soil. *Identifying how the increase in pressure might be determined would also be worthwhile.*

3.21.3 Related Reports and Research

1. Task 106-G-3.1: Strawman for Volume III of the Seismic Retrofitting Manual: Tunnels, Slopes, Retaining Structures, Culverts, and Pavements.

3.21.4 *Limitations*

- For bored tunnels, it would seem that some caution must be used in extrapolating results of past performances (e.g., no damage below 0.2g) for new design. Since over one-half of the case studies are from the Kobe earthquake, some bias related to the construction methods might exist. As more efficient design and construction methods are introduced, response during seismic loading can change.
- The assumption that deformation of a cut-and-cover tunnel cross section is sufficient to mobilize active earth pressure needs further investigation. As described in the task report, the assumption of active conditions is based on Whitman's conclusion that this is a reasonable assumption for basement walls. If the wall behaves stiffer than the soil in the free field at the elevation of the tunnel, lateral earth pressures greater than active could be realized.
- Additional guidance would be helpful in the following areas: (1) selection of ground motion parameters for soft to medium stiff clay used in simplified evaluations of axial and curvature deformations, (2) selection of ground motion input for numerical analysis, (3) methods to account for near-fault ground motions, including long-period pulses, (4) methods for determining the width of the fault zone, and (5) procedures for estimating the potential for flotation; e.g., unit weight of liquefied soil to use in the assessment.
- Frequency was noted as being potentially important to design. A discussion of the relevance of frequency would be valuable. The assumption of a perfectly flexible tunnel was noted to lead to overly conservative design, justifying the use of beam-on-elastic foundation procedures when "significant interaction" exists. Some additional guidance on when significant interaction exists and the choice between these alternatives would be helpful.
- For dynamic lateral pressure estimates, it is understood that peak ground acceleration rather than an effective or reduced acceleration is used. It would seem that some mention should be made of the relationship between a single transitory peak versus an effective acceleration, so that structural designers can apply an appropriate load factor during design.
- The report provides only limited discussion of the seismic vulnerability and evaluation of tunnel portals. Tunnel portals probably are the most vulnerable portion of the tunnel during a seismic event, and therefore, require special consideration. Additional guidance for evaluation would be very valuable.

3.21.5 *Impact Assessment*

Code Requirement: This report presents information that could be used in preparing code sections and commentaries for bored, cut-and-cover tunnels, and submerged tunnels. Such guidelines do not presently exist in AASHTO or other similar codes in the United States. The results of the review suggest that cut-and-cover tunnels are the more vulnerable to seismic damage, and therefore warrant the closer attention during development of code sections. If code provisions are developed, they should specifically address racking behavior; special attention should also be given to design requirements for fault rupture, liquefaction, and landslides.

A question was posed in Section 7 of the ATC-18 report, *Seismic Design Criteria for Bridges and Other Highway Structures: Current and Future* (ATC, 1997), whether there is a need for a code provision that identifies seismic design procedures for highway-related tunnels. This question was raised because the need for a code could be limited, as highway-related tunnels are typically designed by specialty firms with the experience and technical expertise required to ensure good seismic design. While this may be an accurate statement for bored tunnels, it is clear from the recent experience during the Kobe earthquake that code provisions would be valuable for cut-and-cover tunnels.

Design Procedures: Implementation of code provisions for tunnels in the new AASHTO seismic design provisions would likely have little effect on design procedures for bored tunnels. However, it could change the approach for cut-and-cover tunnels and submerged tunnels, particularly if deformation analyses are required. This additional design could result in the use of rigorous computer codes, requiring greater expertise and additional time.

Design Costs: Design cost would likely increase, particularly if more rigorous methods of analysis are required. The increase could be significant, unless guidance can be given on when simplified versus more rigorous design methods should be used.

Structural Performance: The code provisions for tunnels would likely result in better performance of cut-and-cover tunnels and possibly for submerged tunnels. Changes in performance of bored tunnels would be limited, except for issues associated with fault crossings and behavior at portals.

Construction Cost/Constructibility: A code provision for tunnels will not have a direct bearing on construction costs. It is generally believed, however, that current design methods result in conservative performance. With implementation of new methods, it is possible that construction costs could be reduced.

Research Recommendations: The area of tunnel performance during seismic loading warrants additional research in a number of areas, including development of (1) simplified deformation-based design methods for cut-and-cover tunnels, (2) guidance on the selection of ground motion parameters for use in design, (3) general-use racking coefficients if this approach to cut-and-cover tunnel design is implemented, (4) methods for estimating fault zone width, and (5) procedures for estimating flotation in liquefied soils.

3.22 Foundations and Soils – Compile Data and Identify Key Issues

Lam, I.P. (unpublished report)

Task No. 112-D-3.1

3.22.1 Research Summary

This report provides results of a survey of state transportation agencies regarding typical foundations and abutments in their existing bridge inventories. The primary purpose of the survey was to identify foundation systems commonly used in bridge design within the United States. This information was intended to provide background information for other research studies being conducted as part of Tasks 106 and 112 of the MCEER-sponsored research program.

While the primary focus of the survey was on identifying typical foundation systems, the survey also had two secondary purposes. The first involved providing a preliminary assessment of procedures that might be used to screen existing bridges for vulnerability to seismic loading. The second involved identifying major foundation design issues that warrant consideration during the MCEER-sponsored research program.

3.22.2 Research Findings and Review Comments

The original motivation for this task was the result of MCEER meetings dealing with bridge retrofit (i.e., Task 106). A conclusion reached from these meetings was that there was a lack of documentation of

existing bridge foundation systems, which led to some questions regarding the appropriate research needs in the area of retrofitting.

Results of Survey

- Substructure and foundation plans were obtained from ten states, representing five FHWA regions and slightly more than 100 bridges. The states responding included both seismically active and inactive areas. This information was compiled to provide general statistics regarding (1) the types of structures (e.g., single-span versus multiple-span and single-column bents versus multiple-column bents), (2) the types of bent substructures (e.g., pier walls, columns on pile-supported footings, columns on spread footings, and columns without footings), and (3) the types of abutment foundations (e.g., diaphragm or frame versus seat-type spread footings versus piles). *South Carolina DOT has performed or is performing an assessment of the seismic vulnerability of its bridges. They should have some valuable information that would be useful in this study. In addition, surveys completed by FHWA and the University of West Virginia in the 1980s resulted in the compilation of considerable information on highway bridges such as bridge type, foundation type, and soil conditions that could provide other useful data (Moulton, 1986, DiMillio, 1982).*
- The report notes that the screening exercise could yield meaningful information for vulnerability screening of bridges, but it would have to be expanded to include information about the site soil conditions. *It appears that the study would have to be much more comprehensive to be useful; e.g., it may have to involve seismic stability evaluations of each structure. This would be a major undertaking and beyond the scope of the survey. Even with the addition of soils data, the study may have limited value.*

Design Issues

A number of major design issues requiring clarification were identified in the survey, including (1) the need to design the foundation for seismic forces, especially base shear from the inertial response of the structure, as currently required by AASHTO, (2) the acceptability of allowing some damage to the abutment foundation during large events, (3) the philosophy on seismic design of bridge abutments (i.e., whether to design for active pressures as indicated in AASHTO, or passive pressure), (4) the use of shear keys to protect abutment foundations (e.g., should foundation damage be allowed, so as to take better advantage of soil capacities), and (5) how to account for unstable ground in the design code.

The report notes that design strategies involving shear loads on piles and passive pressures for abutments make it difficult for designers to provide economical designs. It suggests that new design strategies are necessary. A number of research projects initiated by MCEER under Tasks 106 and 112 will help define these new strategies.

The report suggests that applying ductility design concepts to the foundation system might promote the use of foundation types with better seismic performance details or characteristics.

3.22.3 Related Reports and Research:

Following are related research reports:

- A report by Moulton (1986), *Tolerable Movement Criteria for Highway Bridges*;
- A report by DiMillio (1982), *Performance of Highway Bridge Abutments Supported on Spread Footings on Compacted Fill*.

3.22.4 Limitations

The ages of the bridges considered in the survey ranged from 9 to 71 years, with an average of roughly 30 years. The relatively old average age limits the usefulness of this information for new code design, but it may provide significant input for development of retrofit guidelines.

While this survey appears to have achieved its primary purpose of defining the range of design conditions for existing bridges, information in the survey is from a limited number of FHWA Regions; i.e., Regions 4, 5, 7, 8, and 10. It is possible that other trends in the data or design conditions and constraints would have been identified if the database were more complete. At present, it must be assumed that regions not responding to the survey are adequately covered by information from these regions. *It would be helpful to append the questionnaire sent out for the survey.*

The title of this draft report implies a rather ambitious study of the seismic vulnerability of existing highway bridge construction related to foundations and soil-structure interaction, but the result is a limited database of existing bridge information that provides no information relative to soil conditions or potential seismic input. As it stands, the report appears to provide little tangible information relative to the actual seismic vulnerability of existing highway bridges in the United States.

It is not clear whether the report recommends that this initial survey effort should be implemented on a wider scale, both in terms of FHWA regions included in the survey and in terms of expanding the database to include geotechnical conditions. Also, the survey was performed as an early task within the overall program. Over three years have passed since it was completed. It would be of interest to learn of current views on the vulnerability assessment and the key design issues. With regard to the key design issues, the obvious question is whether recent changes in the code have addressed them.

3.22.5 Impact

Code Requirements: The purpose of this project was to establish a set of baseline conditions rather than to resolve a technical question or to identify a design process. As a result, the direct impact or usefulness of the survey results relative to the development of the new AASHTO seismic design provisions is minimal.

Design Procedures: The report presents several important major design issues that warrant careful consideration. Specifically, the issues of allowable damage and ductility within bents and abutment walls represent important departures from current philosophy. These changes could result in more economical design; they could also result in better overall system performance. However, such changes would also preclude simple inspection of the bridge following a major seismic event, possibly resulting in additional risk to those using the bridge if major damage to the foundation system has occurred but was not detected. While the technical basis for adopting these changes seems ample, the political realities of the change may present significant obstacles.

Design Cost: Some of the procedures or approaches developed to address the major design issues identified during the survey could result in increased design costs. However, the amount of increase is expected to be minimal, relative to overall design costs.

Structural Performance: This task has little direct bearing on structural performance. Clearly, if research for other MCEER-sponsored tasks focuses on realistic bridge conditions and addresses the

major design issues identified during the survey, more relevant conclusions will be reached that will ultimately result in better structural performance.

Construction Cost/Constructibility: Results of this survey will not have a direct effect on construction cost and constructibility.

Research Recommendations: This draft report presented neither research recommendations nor explicit recommendations regarding the usefulness of expanding the survey. While research funds could be allocated for continuing the survey, it was concluded from this impact assessment that there are other better uses of research funds.

3.23 Centrifuge Modeling of Cyclic Lateral Response of Pile-Cap Systems and Seat-Type Abutments in Dry Sand
Gadre, A., and Dobry, R
Technical Report MCEER-98-0010
Task No. 112-D-3.2

3.23.1 Research Summary

This report summarizes results of a centrifuge test program performed to investigate the translational response of pile-cap foundations and seat-type abutment walls during seismic loading. The research has two primary purposes: to understand the lateral response of pile-cap foundations and seat-type abutments; and to verify current design procedures used to estimate stiffness and capacity. Of specific interest was the contribution of the cap to the lateral-load capacity of a pile-cap foundation system, and whether addition rules can be used to account for resistance contributions from the pile and footing. The other area of interest was the damping of the pile-cap and abutment foundations.

Results from this test program were interpreted to provide valuable guidance on the relative contributions of a single pile and pile cap to resistance during horizontal loading. This effort led to the identification of methods for determining the relative contributions of the pile and pile cap. Methods for determining abutment wall capacity and stiffness are also given and discussed.

The tasks undertaken as part of this project included: (1) a review of literature related to the lateral response of foundations and abutments; (2) a series of centrifuge tests in which models of abutments and pile-cap foundations were tested under prototype stress conditions; and (3) nonlinear finite-element analyses to investigate the influence of soil shear modulus and foundation geometry on the response of the pile-cap foundation. Information from the project was used to prepare recommendations on pile-cap foundation and abutment foundation design.

3.23.2 Research Findings and Review Comments

Literature Review

- The lateral response of a single pile has been studied extensively in the past, but the response of a cap, which is equivalent to an embedded footing, is not completely understood, particularly at large deformations when the footing is fully embedded. As a result, the response of a pile-cap system is not well understood.

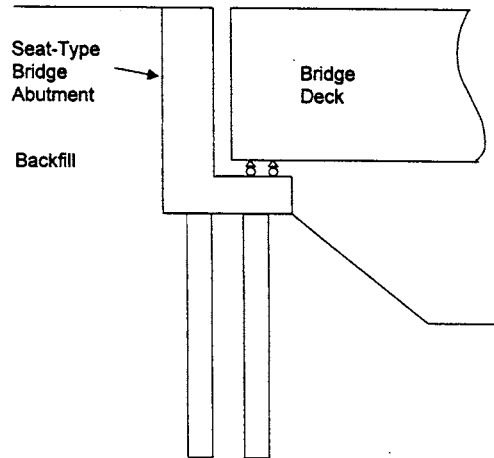
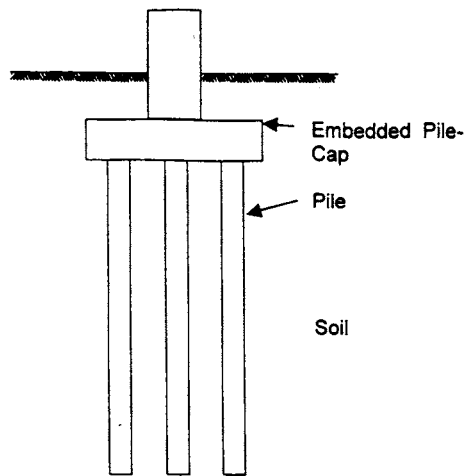


Figure 3-3 Pile Foundation with Embedded Cap Figure 3-4 Bridge Abutment in Dense Dry Sand

- In the case of abutments, most previous studies have involved elastic analyses or small-amplitude field tests, with few studies of the response under intense earthquake motions (i.e., large-displacements that result in nonlinear response).
- These current limitations led to the development of a test program that focused specifically on the horizontal response of pile-cap foundations and abutments, such as shown in Figures 3-3 and 3-4 (Figures 2-1 and 2-2 of the report).

Centrifuge Testing Program

- Centrifuge tests were conducted using dry sand compacted to a single relative density of 75 percent. The equivalent prototype system for the pile-cap foundation was a $1.1 \times 1.1 \times 1.1$ -meter footing with a 38-cm diameter by 5.8-m-long pipe pile (0.6-cm wall thickness). The installation process was most similar to a cast-in-drilled-hole (CIDH) pile.
- For the abutment tests, the prototype had a height of 1.5 to 3 meters; the length-to-height ratio for all tests was 3.75.
- Lateral loads were applied slowly (i.e., 60 to 200 seconds per cycle at prototype scale), precluding study of the radiation-damping component of lateral response.
- Contributions of base shear, side shear, and active and passive pressures were isolated in the testing program.

Results of Pile-Cap System Evaluations

- The measured ultimate capacity of the cap alone agreed reasonably well with theory for an embedded footing. The contribution from the passive side accounted for more than half of the total reaction for the embedded footing. Displacement at failure is about 5 percent of the cap height, reasonably in agreement with the displacement needed to mobilize passive thrust behind retaining walls.
- A passive earth-pressure coefficient, $K_p = 14.3$, was back-figured from the test results. The wall friction used for this calculation, $\phi_w = 39^\circ$, was developed from results of base-shear measurements during one set of tests. The back-calculated value of $K_p = 14.3$ compares very well with that computed based on theory. *Good comparison with theory can also be obtained using a lower value for the wall friction angle. If one assumes that $\phi_w = 22^\circ$ rather than 39° , K_p is back-calculated as 11.98 which still compares very well with theory (11.77 or 10.4, corresponding to the Coulomb, or*

Caquot and Kerisal (1949) methods, respectively). It is assumed in the report that the shear strength at the wall-soil interface is the same as that between the soil and base of the model. Even though both the base and side-walls of the model have similar roughened surfaces, their interface shear strength is not necessarily the same. The shear strength of soils is affected by stress path and the corresponding rotation of principal stress, which is not the same for failure in direct shear at the base compared to passive failure of soil at the side-walls. This suggests that it may not be justifiable to assume that the friction angle derived from base shear ($\phi_w = 39^\circ$) is appropriate for passive pressure.

- Little interaction was found between the stiffness contributions of the base, side shear, and active/passive ends for the embedded cap. The addition rule for secant stiffness was approximately valid (e.g., within 10 percent); the measured areas of loops representing the energy dissipated through material damping also satisfied the addition rule. The secant stiffness was also reasonably predicted from elastic theory when an equivalent linear shear modulus at shallow depth was used. Equations are given for estimating the ultimate lateral capacity for base shear, side shear, and passive resistance of a pile cap.
- Results from a test on a free-head pile were consistent with predictions that model the soil as a system of discrete, nonlinear springs (p - y curves) described by Reese, et al (1974). Measured lateral stiffness and lateral force were also consistent with values recommended by Caltrans for an allowable deformation of approximately 2.5 cm. Material damping values for the pile ranged from 0.15 to 0.20.
- The greatest relative contribution of the cap to the lateral stiffness of the pile-cap system was at small displacements, and it decreased as lateral displacement amplitude increased. For normal working-load levels (lateral displacements less than 2.5 cm), the single pile behaved roughly as a free-head pile, allowing total response of the pile-cap system to be determined by adding the stiffness of the free-head pile to the stiffness of the cap alone. Similar results occurred for material damping at the working level. At larger deformations, the rotation of the pile cap contributed to added reaction, which would not be consistent with an actual pile group, due to the axial stiffness of the pile.
- Three-dimensional, nonlinear, finite-element analyses using the computer program ABAQUS provided a reasonable representation of the load test results obtained during centrifuge tests for an embedded pile cap alone. From the numerical analyses, it was determined that the soil shear modulus is critical at small displacements, but is less important at larger displacements. Material damping computed with the Masing criteria was less than that measured at small displacements.

Results of Abutment Wall Evaluations

- Results of centrifuge tests on models of seat-type bridge abutments show that the lateral response of the abutment is very nonlinear. Passive earth-thrust computations (with wall friction) gave reasonable estimates of ultimate capacity. *Reasonable estimates of ultimate capacity could also be obtained using lower values of wall friction than the value of $\phi_w = 39^\circ$ employed in the passive thrust computations.*
- The measured capacity values are comparable to and somewhat smaller than the values obtained by the usual design procedures recommended by Caltrans, AASHTO, and ATC-6.
- The measured stiffness in the centrifuge is smaller than that computed by frequently used design procedures. Results are more consistent with new design methods recently adopted by Caltrans. Secant stiffness of the abutment appears to be independent of the dimensions of the abutment for displacements greater than about 0.2 inches.

Design Recommendations

From the results of these centrifuge and nonlinear, finite-element analyses, the report recommends use of: (1) simple addition to account for contributions from the base, side, and active/passive ends when estimating the lateral capacity of embedded spread footings in dense sand; (2) elastic solutions with an equivalent linear soil shear modulus at shallow depths to estimate secant stiffness of the footing; (3) the sum from single-pile contribution to the cap contribution when estimating lateral displacements up to 2.5 cm; and (4) lateral stiffness per unit area for abutment displacements greater than approximately 0.5 cm.

3.23.3 Related Reports and Research

1. Task 112-D-3.3: Modeling of Abutments for Seismic Design
2. Task 112-D-3.4: Develop, Analysis, and Design Procedures for Retaining Structures
3. Task 106-E-4.5: Abutments and Retaining Structures

3.23.4 Limitations

- The project was limited to an investigation of the translational response of a foundation system comprised of a single pile and pile cap. Conditions were also for a single soil type. While the results provide important information about the behavior of pile-cap foundations during horizontal loading, additional testing with other dimensions of caps and piles and other modes of deformation, such as rocking, will be required to confirm these trends and to provide encompassing design information.
- The project does not address kinematic interaction between the pile foundation and soil, which may be an important effect as described by Nikolaou, Mylonakis, and Gazetas (1995). Future testing in the centrifuge should include kinematic interaction. Tests also did not account for the effects of inertial loading within the soil on passive resistance. It is believed that inertial loading could be important beyond 0.3g and therefore warrants additional consideration in future test programs.
- The present study models the sides of the cap as a roughened surface. Although a roughened surface may be appropriate for the base of the cap, representative of concrete cast directly against the soil, it may not provide an adequate representation of side-walls that have a formed concrete surface. The inference that very high values of ϕ_w are appropriate when computing passive earth pressures could be unconservative from a design standpoint. Additional evaluation in this area appears to be necessary.
- More guidance is necessary for estimating parameters such as K_s , which is the lateral earth-pressure coefficient during shear of the side-walls, and the wall friction angle, ϕ_w , that contribute to passive resistance of the end-walls. Also, guidance is needed on how to determine the vertical load transferred to the base of the pile cap.
- For the pile-cap foundation studies, the effects of pile-group interaction have not been considered. Issues such as soil gapping (side and bottom), soil dilation, and rate of loading effects are also likely to warrant consideration in future studies.
- More specific guidance is required for the determination of strain-compatible shear modulus values for use in elastic footing equations. If shearing strain is to be used to estimate the reduction in low-strain modulus, then a method for estimating shearing strain needs to be identified.
- Methods of introducing radiation and material damping into design equations are still required for pile-cap foundations. Material damping values for the footing appear to be very high (25 to 30 percent), suggesting that the contribution from this form of energy dissipation should not be neglected in response analyses.
- Abutment tests were conducted for a single soil type and a single height-to-width ratio. Additional abutment tests are required for different abutment height-to-width ratios, different soil types, and

different abutment stiffness conditions. Guidance is still needed for the determination of radiation and material damping in abutments.

3.23.5 Impact Assessment

Code Requirements: Information developed during this research program can be used as background information when developing new AASHTO seismic design provisions. In view of the limited amount of available test results and the special conditions under which the information was obtained, extreme caution will have to be used when developing new provisions to assure that the provisions apply to a broad range of cases encountered in design practice.

Design Procedures: The current AASHTO provisions provide only limited guidance regarding the determination of passive earth pressure for abutment walls and the determination of soil pressure for pile cap system interaction. A more explicit treatment of these areas in the new AASHTO seismic design abutment provisions appears to be warranted.

Design Costs: Modifications to the current AASHTO provisions to address pile-cap system interaction and abutment wall reaction could result in additional design costs for bridge structures requiring this type of modeling. However, it is believed that in many cases these design methods are already being used. Changes to the AASHTO provisions would, therefore, provide a more consistent methodology when accounting for these effects.

Structural Performance: The performance of a bridge could be improved by better modeling of the pile-cap system and by more representative treatment of passive earth pressures developed by abutment walls. The degree of improvement, if any, will depend on a number of factors, including the type of bridge, the type of foundation system, and the level of seismic loading.

Construction Cost/Constructibility: Implementation of new AASHTO provisions for handling the pile-cap system and abutment walls will not necessarily result in any changes in construction costs. In some situations, more rigorous treatment in the design of the pile-cap system or the abutment wall could result in lower demand during seismic loading, which could lead to more efficient, less-costly construction.

Research Recommendations: Additional research in the areas of pile-cap and abutment wall performance are warranted. In particular, research is needed on (1) importance of dynamic (kinematic and inertial response) versus pseudo-static loading; (2) performance of other combinations of soil types and foundation systems; (3) influence of factors such as soil gapping, soil dilation, rate of loading, and group interaction; (4) methods of relating nonlinear properties of soil to stiffness of pile cap; and (5) procedures to account for material and radiation damping of the pile cap and abutment wall. Further research is needed on current design procedures and practices used to specify the stiffness of bridge abutments, so as to arrive at a generally-accepted method.

3.24 Modeling of Bridge Abutments in Seismic Response Analysis of Highway Bridges *Lam, I.P. and M. Kapuskar (unpublished report)* *MCEER Task No: 112-D-3.3*

3.24.1 Research Summary

This research on the seismic design of bridge abutments addresses (1) providing a brief summary of the major design issues associated with the design of abutments, (2) reviewing the basic concepts of classical earth pressure for passive loading under both seismic loading and non-seismic (service) loading conditions, (3) summarizing the results of parametric studies showing the interaction between the backfill and bridge deck, (4) giving recommendations for the process of bridge abutment design, and (5) presenting conclusions regarding current practice in modeling bridge abutment response.

3.24.2 Research Findings and Review Comments

The report contains an informative discussion on bridge abutment design. The abutment can attract a large portion of the inertial loads from the structure during a seismic event, and to model this condition realistically it is usually necessary to treat the superstructure, abutment, and approach fill as a tightly coupled system. Information in this report provides a more realistic basis for designers to model this coupled system. By implementing recommendations given in this report, the level of damage to bridges during future seismic events should be reduced.

The report considers both squared and skewed bridges, and provides commentary on the adequacy of the iterative procedure used during the design of some bridges.

This report covers a wide range of topics related to the seismic design of bridge abutments, from the fundamental issues associated with abutment response, to design recommendations. It is unique among discussions on abutment design in that it considers the response of the abutment and structure as a coupled system when making conclusions regarding appropriate modeling.

Overview of Abutment Response Issues

- There have been a number of documented examples of abutment damage during recent earthquakes. The most common form of damage has been slumping of approach fills, resulting in loss of accessibility to the bridge, but structural damage has also occurred. *While slumping of the approach fills may be more common, they are generally simpler and faster to repair than, for example, a damaged bridge abutment. Improved design methods for the bridge abutment are, therefore, viewed as having more overall benefit to the transportation system.*
- Confusion currently exists within the design profession regarding the design of bridge abutments. This results from (1) the large number of combinations of bridges and abutments in current use, (2) the different design strategies that exist in design codes (e.g., Caltrans, AASHTO), and (3) the fundamental differences that exist between seismic and nonseismic loading conditions.

Earth Pressure Theories and Abutment Design Strategies

- Three general types of earth pressure loads can exist on a bridge abutment: at rest, active, and passive. Key assumptions regarding the development of each of these loading cases are summarized. For nonseismic (service) loads either the active or at-rest loading case usually applies, depending on the amount of movement of the abutment. For seismic loading the passive case can control if the

movement of the bridge deck exceeds the thermal expansion gap (1 to 2 inches) during the seismic event. *This amount of movement is likely to occur for even moderate seismic events, and therefore, must be given serious consideration during design.* The range of values between the active and the passive loading cases is shown to vary by a factor of 10 to 20 for normal backfill conditions, making it critical to understanding the likely response of the bridge during a seismic event. Figure 3-5 (copied from Figure 2.4 of the report) illustrates the loading and resisting forces during static and seismic loading cases.

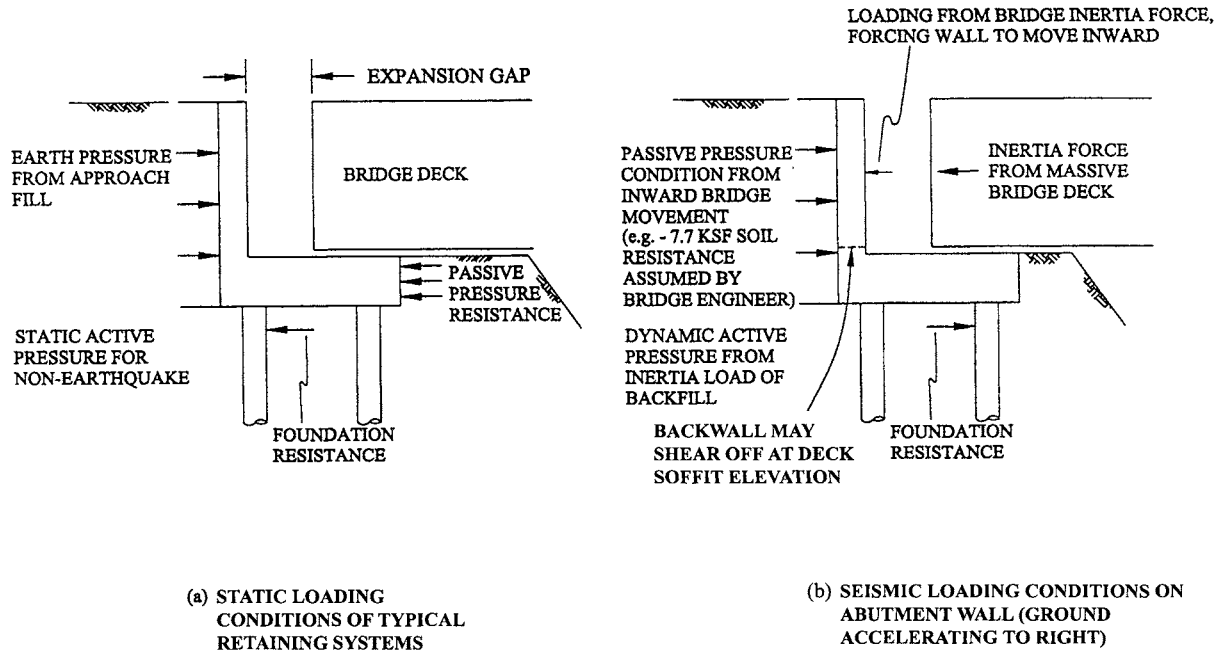


Figure 3-5 Static and Seismic Loading Conditions on Walls

- Friction between the wall and the backfill soil is identified as a critical contribution to the high passive earth pressure coefficients measured during the Caltrans tests. Subsequently, in Section 4.2.3 of the report, additional discussion of the determination of static passive pressure is also given. *It would be valuable if more discussion were provided, either at this point or in Section 4.2.3, on the determination of passive earth pressure, since it is clearly shown as the critical loading case. Specifically, a clear statement on the applicability of the Caquot and Kerisel (1949) procedure would have been useful. Since wall friction is an important factor in the application of the Caquot and Kerisel procedure (or the Coulomb procedure, for that matter), its determination should have been discussed in more detail. For example, should full wall friction be relied on during seismic loading? Should the wall friction be equal to the soil friction angle or should it be reduced to one-half the friction angle? Will the wall construction method affect the selection?*
- Two different design strategies are currently being used for the design of abutments, the Caltrans method and the AASHTO method. The difference relates to the acceptance of some damage at the abutments by Caltrans and the “no-damage” approach given in AASHTO. Caltrans allows a certain amount of passive resistance, whose magnitude is limited by shear keys. The focus of the Caltrans design is on characterizing the load/deflection characteristics of the abutment system and controlling the location of damage. In contrast, under its “no-damage” approach, AASHTO does not allow any passive reaction from the abutment; it uses only active earth pressures for design, with the active pressures increased for seismic loading using the Mononabe-Okabe method of seismic earth pressure

determination. The report states that the AASHTO approach does not account for the possibility that passive pressures will develop, and implies that the AASHTO “no-damage” performance criterion may not be valid.

Abutment Seismic Analysis

- The abutment reaction can have a dominant effect on the seismic response of bridges that are less than 90 m (300 ft) long. The modeling of this abutment-bridge interaction is complex due to asymmetric load/unload characteristics, gapping, and nonlinear soil behavior. A discussion of the Caltrans approach for modeling this interaction is provided. This approach uses elastic springs to represent the abutment; nonlinear effects are handled by using an interactive procedure until stiffness and deformation are compatible. Typical amounts of deformation that can be tolerated range from 76 to 152 mm (3 to 6 in.) for longitudinal loading, and smaller levels for transverse loading. Abutment design also must check to confirm that both allowable capacities and deformations within the soil and structural members have not been exceeded.
- Results of a sensitivity study of different modeling assumptions for the abutment stiffness are presented. The study involved a simple, two-span bridge with zero skew and seat-type abutments. A computer program, SPRING, was used during the study to show the importance of the nonlinear, gapping behavior that occurs during seismic loading. A description of the capabilities of the SPRING program is given in an appendix to the report. Results of this study, which are documented in two unpublished papers, and are included as appendices of the report, showed that the ultimate abutment capacity has an important effect on the spectral accelerations and force levels during overall bridge response. *Table 1 from the paper by Lam, Kapuskar, and Lew (in Appendix B) demonstrates the differences that were found during the parametric study. Information from this table and in Figures 10 and 11, which provide dramatic evidence of the importance of abutment model assumptions, is important and would have been worthwhile additions to the main text. Similarly, key comments on damping made in the same paper should have been included in the main text of the report.*
- Abutment/bridge interaction for skewed bridges is also discussed. For these analyses the contributions of abutment end walls must be considered. A torsional mode of vibration with torsional resistance develops. A simple stick model is reported to result in good accuracy when compared with the response of two bridges. Relatively high modal damping (20 to 30%) was required to given realistic response.
- A number of other factors may influence seismic design of the bridge-abutment system, including (1) differences in input motion between bent foundations and the abutments, (2) wave scattering in the abutment fill, and (3) the response of the approach fill. These factors are described as being very complex and requiring future research.

Abutment Design Procedure

- A two-step abutment design process is identified, with the first step involving the static design and the second involving the dynamic characterization of the bridge system. For service loads either the active or at-rest pressures are appropriate. Guidance is provided on the determination of these pressures, including the need to apply an appropriate factor of safety during global stability assessments. Procedures for determination of passive pressures are also summarized. *As noted previously, more guidance is needed for the determination of wall friction. Also, important information such as Figure 6 in the paper by Martin, Lam, Yan, Kapuskar, and Law (in Appendix C) would have been valuable in the main text. By having this information only in the Appendix, there is a distinct possibility that the reader will miss it. Also, simple guidance is needed on the extent of the*

passive zone behind the wall. Does it extend to five times the wall height? Other issues needing to be answered include: When two materials (a thin granular layer against the wall, with common fill behind) exist within the passive zone, how should wall friction be defined? Is it the lesser of the friction between the wall and the granular backfill and that between the granular and common backfill?

- Procedures for the seismic design of the abutment are summarized. The active earth pressure coefficients are not always reasonable for typical backslopes and high acceleration levels. *Guidance is needed on the questions: Should vertical accelerations be included during the determination of active and passive earth pressures, using the Mononobe-Okabe method? A common procedure of reducing the acceleration by one-half is applicable only if the wall can move sufficiently. Any restraint of the abutment could lead to higher active pressures, for cases where the abutment is not designed for passive loading. Mention is made of the dynamic passive pressure but little guidance is given regarding its use. It would be worthwhile to identify whether the limiting pressure for dynamic passive loading is the static passive pressure or the dynamic pressure determined using the corrected Mononobe-Okabe method.*
- Recommendations for determining the capacity and stiffness of endwalls and wingwalls are given. The method for determining stiffness of an abutment with a thermal expansion gap is included. *A little more explanation on this method would be appropriate. What is being done is not obvious from Figure 3.2.* The recommendations for the wingwalls include the limiting height of the wall, the contributions from each wall, the determination of transverse stiffness, and the effects of pile support systems.

3.24.3 Related Reports and Research

1. Task 112-D-3.2: Abutment and Pile Footing Studies by Centrifuge Testing
2. Task 106-E-4.7: Structural Implications of Foundation Modeling
3. Caltrans Project 59Q121: Development and Implementation of Improved Seismic Design and Retrofit Procedures for Bridge Abutments

3.24.4 Limitations

- The report focuses on procedures used by Caltrans for the seismic design of abutments. While it is generally recognized that Caltrans is a leader in the seismic design of bridges, there is some concern that too much reference is made to Caltrans. It would be valuable to know (1) whether there are any other agencies that are following the same philosophy as Caltrans and (2) the reason that some agencies in seismically active areas, such as Washington, have decided to use the AASHTO approach rather than the Caltrans approach.
- The reported numerical and experimental studies (e.g., Caltrans tests at U.C. Davis) appear to consider only the application of quasi-static loads to the abutments; i.e., the soil behind the abutment wall behaves as a static resisting element. The load condition used in the analysis and the experiments are consistent, and therefore good agreement is achieved between the two sets of results. While it may be reasonable to use static values of passive earth pressure for horizontal ground acceleration less than 0.3g, for higher levels of acceleration the effect of dynamic loading on the soil behind the wall may be important. Additional consideration should be given to this issue.
- Additional treatment of the material and radiation damping mechanisms for abutments appears to be needed. The multiple spring model accommodates material energy dissipation, but dynamic effects are ignored. Response spectra methods usually introduce 5% damping to account for radiation effects; however, this level seems low if the abutments are absorbing a significant amount of load.

- Additional guidance is needed regarding the treatment of wall friction. Realizing that the value of wall friction has a significant effect on the calculation of passive resistance, a more detailed discussion of this topic appears to be warranted. For example, use of the angle of wall friction, $\delta = 0$ is a conservative assumption from the standpoint of developing ultimate resistance but lead to excessive deformations. Similarly, use of $\delta = \phi$, the angle of internal friction, can render a passive thrust that may be as much as three times higher than that computed with $\delta = 0$, resulting in unrealistic load transfer.

3.24.5 *Impact*

Code Requirements: The current AASHTO seismic design provisions provide guidance for the design of abutments walls under active loading conditions. In some situations the passive loading case will be more critical. New AASHTO seismic design provisions should be expanded to address this type of loading. Considering the complexity of the passive loading case, a detailed commentary covering methods for determining passive capacity and stiffness is needed.

Design Procedures: The passive pressure is already being considered during design of bridge abutments in areas of high seismicity. A section within the new AASHTO provisions that documents the approach for treating this type of loading would, therefore, result in little additional design effort. Rather, the section within the provisions would provide a standard basis for approaching design. In areas of low-to-medium seismicity, the design effort would increase in only those cases where the potential contribution from passive loading is required.

Design Costs: The change in design cost from implementation of passive loading provisions in the new AASHTO provisions is expected to be minimal.

Structural Performance: The passive pressure guidelines could result in more efficient design of bridge systems in areas of high seismicity. In areas of low-to-moderate seismicity, these guidelines would only affect structural performance for those bridges that are specifically designed to account for passive loading.

Construction Cost/Constructibility: The recommended approach to passive earth loading could result in lower construction costs by using the passive capacity of the abutment to carry a higher percentage of the longitudinal load during a seismic event.

Research Recommendations: Additional research is required in the area of seismic response of abutments. Areas warranting research include (1) the effects of abutment acceleration on passive earth pressure determination, (2) methods for quantifying the amount of material and radiation damping, (3) appropriate values of wall friction to use during passive pressure computation, (4) the amount of movement to mobilize passive pressures for different loading conditions, and (5) the transverse response of the abutment.

3.25 Seismic Analysis and Design of Bridge Abutments Considering Sliding and Rotation
Fishman, K.L. and Richards Jr., R.
Technical Report NCEER-97-0009
Task No. 112-D-3.4

3.25.1 Research Summary

This report provides a new procedure for determining the magnitude of earthquake-induced displacements of retaining walls and bridge abutments founded on spread footings. The new procedure differs from existing displacement-based procedures for determining the sliding response of bridge abutments in that it addresses mixed-mode behavior, which includes the rotation due to bearing capacity movement as well as the sliding response. The procedure also extends existing methods for estimating sliding and rotation by introducing a pinned-restraint condition at the top of the retaining wall and by accounting for reductions in bearing capacity caused by seismic loading.

The procedure for predicting permanent, mixed-mode displacements was calibrated against test cases that were modeled in the laboratory. Figure 3-6 (Figure 3.1 of the report) shows a schematic diagram of

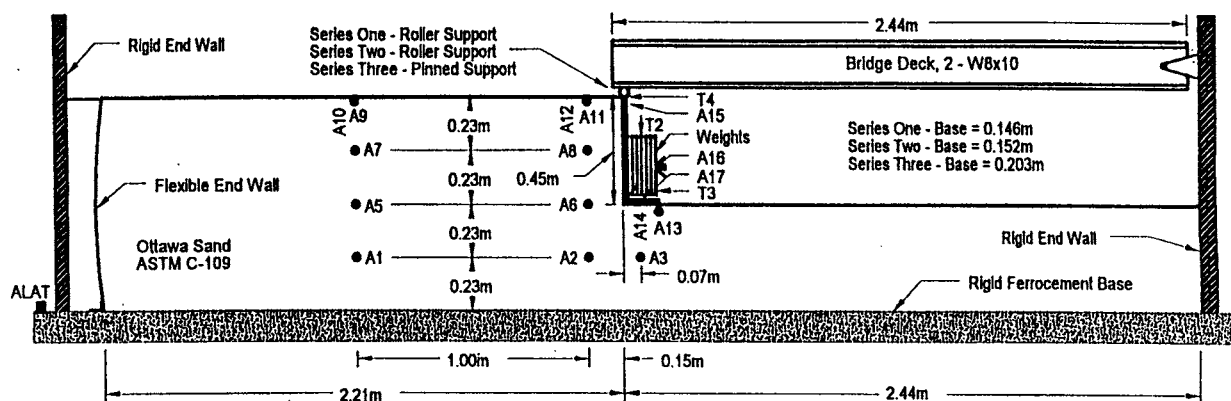


Figure 3-6 Schematic of Bridge Abutment Model

the model test setup. The boundary conditions at the top of the abutment were varied during the test program to include sliding, rotation-about-the-top, and mixed sliding and tilting. The report describes procedures for using this new approach in seismic design. A computer program for estimating sliding and rotational displacements is included in an appendix to the report.

3.25.2 Research Findings and Review Comments

- Wall failure by rotation during seismic events is quite common. Current methods given in AASHTO assume that a bridge abutment fails by excessive translation and do not consider the possibility of rotation.
- Coupled equations of motion account for both translation and rotation. A recently developed theory of seismic bearing capacity by Richards et al. (1990, 1993) is reproduced in the report. Normal forces are limited by a bearing capacity value that is reduced to account for seismic loading. Equations address the active mode of deformation. Inertial response of the system is handled through

application of pseudo-static methods. Seismic bearing-capacity reduction factors apply to cohesionless soils. *A companion report, Technical Report NCEER-97-0011, extends the treatment to cohesive soils.*

- The report provides step-by-step procedures for computing the outward movement of a wall for two boundary conditions at the top of the wall: (1) free movement of the abutment; and (2) restrained movement at the top of the wall. The approach involves an iterative application of the Mononobe-Okabe (M-O) method. For restrained-wall conditions, the active pressure from the M-O method is doubled to account for the additional rigidity of the wall. This procedure was programmed into a FORTRAN computer code presented in Appendix C of the report.
- Model tests were conducted on a shake table. The soil for the tests was sand. The connection at the top of the wall could be fixed, to force failure by rotation about the top. The model was subjected to accelerations that increased from 0.05g to 0.70g.
- Results of comparisons between measured and predicted response for two modes of failure, sliding and rotation about the top, were good. However, the comparisons were less successful for mixed tilting and sliding. *Apparently gravity scaling does not make a difference in the testing. Some may argue that scaling issues associated with shake-table testing as opposed to centrifuge testing introduce non-representative response. This consideration may be more important for rotational behavior than it is for translation. Information presented by Gazetas et al. (unpublished report) seems to show that the reduction in seismic bearing capacity is much less significant than indicated in this report. The differences need to be resolved.*
- The report recommends that gravity walls be proportioned to avoid seismic loss in bearing capacity and subsequent rotational deformations. When this is not feasible, the effect of tilting on the overall deformation must be determined. Because no semi-empirical equations are provided for this determination, a more rigorous seismic analysis is recommended using a representative earthquake record.

3.25.3 Related Reports and Research

Task 112-D-3.7: Develop Analysis and Design Procedures for Spread Footings

Task 106-E-4.5: Abutments and Retaining Structures

3.25.4 Limitations

- The procedures described in this report are for walls and abutments on spread footings supported on a cohesionless soil. This limits the applicability to a somewhat special set of design circumstances. *The authors have indicated, however, that Technical Report NCEER-97-0011, which was prepared as part of MCEER Task 106-E-4.5, treats the response of cohesive soils.*
- The differences between this work and the work of Gazetas et al. (unpublished report) on seismic bearing capacity result in some uncertainties about the accuracy of this approach. Each group has arrived at somewhat different conclusions. In the absence of conforming data or more information, it is difficult in this review to judge which of the two groups is more correct. In response to this observation the authors have provided their views on the reason for the differences with the work of Gazetas et al. (unpublished report). This difference is explained on the basis of a Dynamic Fluidization Theory developed by one of the principal investigators on this task. Experimental results from shake table tests are used as support for the conclusions reached here. Gazetas' team has not provided any comments regarding their views on the difference, however, so a degree of uncertainty exists.
- Contributions from passive resistance at the toe of the embedded portion of footing are not currently considered in this task report. The importance of the passive-pressure contribution at the toe of the

wall will depend on site-specific conditions. Guidance on the approach for the common case of an adverse slope at the toe of the spread footing would be useful. The authors have indicated, however, that Technical Report NCEER-97-0011 addresses this issue.

- The approach for evaluating combined translation and rotation for outward movement of the wall does not apply when the inertial response of the bridge forces the abutment into the soil (passive failure). It may be useful to extend the approach for this type of loading. Additional guidance on the selection of acceleration coefficients and the absolute limitations on deformation is also needed.
- The computer code is limited to the sinusoidal input used in the shaking table tests. Some modifications will be required in order to incorporate a more general acceleration record as input.

3.25.5 Impact

Code Requirements: Information developed as a result of this task and companion task 106-E-4.5 are in a form that could be easily integrated into new AASHTO seismic design provisions. However, conclusions from independent numerical analyses by Gazetas and others were sufficiently different that some type of resolution on the appropriate approach should be reached before the method is adopted.

Design Procedures: Adoption of the methods will result in some additional design effort. This effort will, however, result in more realistic response predictions and, presumably, more reliable system response.

Design Costs: The effects on costs are expected to be minimal.

Structural Performance: The recommended approach could improve structural performance of free-standing retaining structures and abutments supported on spread-footing foundations.

Construction Cost/Constructibility: These procedures could result in higher construction costs, if reductions in seismic bearing capacity are accounted for in design. In this situation, it may be necessary to use larger footings to support the same load to compensate for the lower bearing capacity.

Research Recommendations: Additional research involving the response of abutments and free-standing retaining walls supported on spread footings is warranted. Areas of research should include (1) model testing on cohesive soils to confirm the applicability of the approach for these soil types, (2) development of guidelines for selecting acceleration coefficients and deformation limits applicable to this method of analysis, (3) development of guidelines for selecting and incorporating earthquake time-histories in the analyses, (4) identification of methods for computing the soil moment resistance, and (5) determination of the line of action of the active earth pressure. It is also critical that the difference between conclusions reached in this report and those reached by Gazetas and others be resolved. To this end, it may be desirable to conduct a set of centrifuge tests using the same model to determine if gravity-scaling issues could explain any of the differences in the two methods.

3.26 Modeling of Pile Footings for Seismic Design

Lam, I.P. and Kapuskar, M

Technical Report MCEER-98-0018

Task Nos. 112-D-3.5 and 112-D-3.6

3.26.1 Research Summary

This report summarizes results of a project that evaluated seismic design methods used to represent pile foundations and drilled shafts for bridge structures. The information on pile footings from this task (112-D-3.5) was later combined with similar information obtained for drilled shafts under task number 112-D-3.6 into one MCEER technical report, MCEER-98-0018. At the time this report was written, two draft reports had been submitted for review. Therefore, two reviews appear for one technical report (see Section 3.27).

This task had the following main objectives: (1) to establish the influence of modeling techniques on the estimated displacement and force demand; (2) to summarize methods for characterizing the stiffness of pile footings and to discuss their limitations; and (3) to provide guidelines on seismic design practice. The focus of the report is on pile-group foundations rather than single-pile extensions (see Figure 3-7, copied from Figure 1-1 of the report).

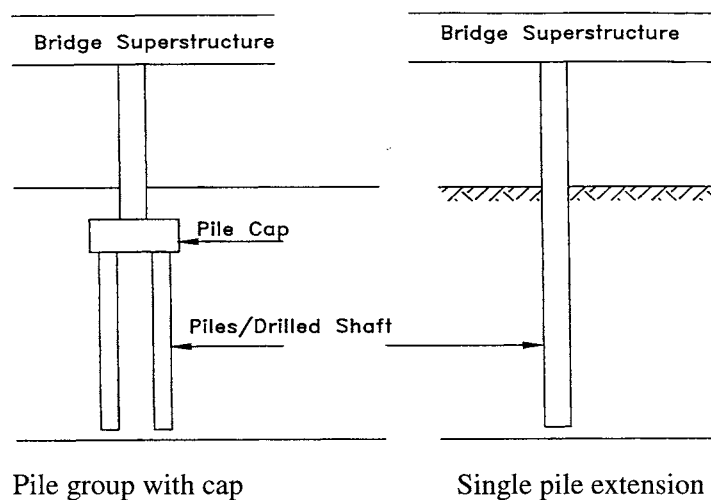


Figure 3-7 Pile Foundation Configurations

Information in this report gives a practical summary covering the state-of-the-practice for seismic design of pile foundations. A primary feature of the report is that it attempts to provide the interface between the structural and geotechnical design processes.

3.26.2 Research Findings and Review Comments

This review covers the two-step design process, single- and group-pile stiffness determination, and consideration of pile-cap stiffness effects. Results of sensitivity studies are used to illustrate these effects.

Two-Step Seismic Design Process

- The two-step design process is summarized. The first step normally involves linear response analyses to determine the displacement demand; the second step involves nonlinear analyses to determine the load demand. Nonlinear behavior of the soil makes selection of a linear stiffness model difficult. Since the equivalent stiffness at peak load would be too soft for the overall duration of the earthquake, a reduced stiffness equal to either $n = (M_w - 1)/10$ or 0.50 to 0.65 of the peak deflection is recommended. *It is not clear if this is done for both time-history and response-spectrum analyses.*
- Static nonlinear pushover analyses are conducted to determine forces corresponding to the displacement demand. Both structural and foundation nonlinearity are *(or can be)* accounted for in this step, with structural nonlinearity being more important. The linear analysis performed to estimate displacements generally overshoots the design base-shear force.
- *In agreement with other modern codes, the 1997 AASHTO specifications do not mandate a pushover analysis for calculating design forces, and allow the use of reductions factors R (equal or higher than 1.0). The forces resulting from the linear analysis are divided by R. However, the specifications include some level of pushover analysis (column plastic hinging) as an option for calculating the design forces for bridges classified as seismic performance category C or D. Therefore, the presentation of concepts related to pushover analysis is very pertinent and could be enhanced by adding illustrative examples.*
- The report mentions that if the stiffness of the foundation is underestimated, the computed shear demand in columns could be too low. This error has resulted in catastrophic failures during earthquakes. Softer foundation stiffness also affects load distribution. *Design specifications should stipulate the use of lower-bound, best-estimate, and upper-bound values for soil parameters relevant to determining soil-structure interaction stiffness. As a rule, it cannot be anticipated which value will lead to higher seismic demands. With the current availability and capabilities of personal computers, repetition of the analysis does not constitute an extraordinary task.*

Pile-Head Stiffness for Single Piles and Pile Groups

- This discussion covers axial loading and lateral loading. The importance of axial response is emphasized as it often controls the rocking response of a pile group. Lateral stiffness is assumed to be uncoupled from vertical stiffness because it is normally developed near the ground surface while axial stiffness is developed at deeper depths.
- Guidance for determining the axial stiffness of a single pile is given. For axial loading, the initial static offset of the load-deformation response from dead loads must be considered. The report notes that the axial stiffness can change drastically during a cycle of loading, making determination of an equivalent stiffness value problematic.
- Procedures for determining the lateral load-deflection characteristics are also summarized. Simplified stiffness charts for fixed- and free-head conditions are provided. The importance of pile length, sectional modulus (for concrete piles), and embedment depth are also addressed.
- The report summarizes methods for determining the stiffness of a pile group, including an equivalent-cantilever approach. The methods assume that the pile cap is infinitely rigid. It is noted that for the equivalent-cantilever approach, the resultant model represents the individual pile head stiffness properly, but the calculated shear and bending moment may not bear any relevance to the actual pile.

Pile-Cap Stiffness

- The contribution of the pile cap to the response of the pile system is also reviewed. The review concludes that the largest component of resistance is from the passive pressure at the face of the pile cap. Small displacements are needed to mobilize this resistance, suggesting that the pile cap be represented by a force capacity rather than a stiffness.
- Methods for calculating the passive earth pressure in sands (cohesion, $c = 0$), cohesive soils (angle of internal friction, $\phi = 0$), and general soils (c, ϕ , non-zero) are also discussed. Other important recommendations include the use of (1) an interface wall friction of one-half the soil friction (in sand), (2) deformations of 0.01 to 0.02 of the cap height to mobilize ultimate resistance, and (3) a factor of 0.50 to 0.65 on the calculated stiffness to develop an equivalent stiffness. *Whereas the recommendations above are consistent with recommendations from Task 112-D-3.2, it is important to be aware that the impact assessment for Task 112-D-3.2 identified potential concerns regarding task conclusions involving wall-interface friction and the amount of deformation necessary to mobilize the ultimate passive resistance. The impact assessment recommended that additional research be carried out to resolve what are believed to be uncertainties. Some additional consideration of these uncertainties in this discussion may be warranted.*

Nonlinear Load-Deflection Sensitivity Study

- Results of nonlinear load-deflection analyses are presented to illustrate the sensitivity of results to uncertainties in p - y stiffness (under lateral loading, p is the soil reaction per unit pile length and y is the deflection—both depend on the distance from the pile caps), gapping, pile-head fixity, bending stiffness parameters, and embedment effects. Results of analyses demonstrate that the load-deformation response is more affected by input variations than is the moment within the pile. Uncertainties in p - y curves are also less important than pile-head fixity, bending stiffness, and embedment issues.
- Effects of pile-group reduction factors are also reviewed. For pile-group effects, a p -multiplier of 0.5 is recommended to cover the potential for gapping, cyclic degradation, and shadowing effects. The difference between normal groups and very large groups (e.g., several hundred piles) is also mentioned.
- The effects of local and free-field pore pressures on cyclic response are discussed. Guidance for accommodating the effect of pore-pressure buildup on lateral response is provided. Effects of lateral ground spreading are also addressed.

3.26.3 Related Reports and Research

1. Task 112-D-3.6: Modeling of Drilled Shafts for Seismic Design
2. Task 106-E-4.7: Structural Implications of Foundation Modeling
3. Task 106-E-4.10: Synthesis Report: Soil-Structure Interaction and Pile Group Effects

3.26.4 Limitations

- The foundation system for a bridge is often assumed to be completely rigid during pushover analyses under the premise that this assumption is conservative. Some additional discussion of the limitations of this approach might be useful.
- The ultimate capacity during uplift is often assumed to be 80 percent of the side friction in compression. Some comment on the appropriateness of this reduction during seismic loading would be helpful.

- Coefficients for determining subgrade stiffness during lateral pile analyses are appropriate for piles up to 61 cm in diameter. Because it is becoming more common to use 76- and 91-cm diameter piles, some discussion on diameter effects would be valuable.
- Very little mention is made of pile installation effects. Additional discussion on differences between displacement and nondisplacement piles would be valuable.
- The general topic of damping warrants some discussion. Work by Dobry and others suggests relatively high material damping during the cyclic response of the pile foundation system. For other types of embedded foundations, radiation damping is also identified as a significant contributor to response.
- Stiffness and damping parameters for soil-structure interaction analyses are reported to be frequency dependent. The basis for using frequency-independent values deserves some discussion.
- A variety of concepts related to determining foundation stiffness for a pushover analysis are presented. The addition of examples that illustrate these concepts with specific numerical values would facilitate their understanding and correct application.

3.26.5 *Impact*

Code Requirements: Information in this report provides important documentation regarding the seismic design of pile foundations. The current AASHTO seismic design provisions do not cover most of the topics included in this report. Rather, AASHTO refers to work completed by Lam and Martin in 1986. While the fundamental approaches may be similar to those provided in the 1986 report, this report updates the earlier work based on conclusions reached and the experience gained over the past 10 years. For this reason, it is recommended that information in this report be incorporated into the new AASHTO seismic design provisions.

Design Procedures: The seismic design procedures discussed in this report will not necessarily result in any significant modification to current design procedures. It is believed that individuals involved in design in high seismicity areas have already adopted these procedures. In areas of low-to-moderate seismicity, it may be appropriate to simplify these procedures. Adoption of these procedures into the new AASHTO seismic design provisions would: (1) provide a better degree of consistency between different designers; (2) potentially eliminate some of the unnecessary and undesirable conservatism in design; and (3) assist in avoiding errors that are commonly made during design.

Design Costs: It is anticipated that the change in design costs to account for recommendations given in this report will be limited.

Structural Performance: Adoption of these design procedures in the new AASHTO seismic design provisions will likely result in more realistic and reliable structural performance predictions.

Construction Cost/Constructibility: The recommended design procedures could result in higher or lower construction costs, depending on the specific conditions of the site and the foundation system. In most cases it would be expected to result in a more efficient foundation system, which generally will lead to lower construction costs.

Research Recommendations: Additional research is warranted in the area of pile design for seismic loading. Some of the topics warranting research consideration include: (1) the effects of pile diameter on lateral pile capacity; (2) the importance of pile installation effects on response; (3) methods of quantifying cyclic degradation and soil-pile gapping; (4) the energy dissipation of single piles and pile groups; and (5) methods for analyzing small versus very large pile groups.

3.27 Modeling of Drilled Shafts for Seismic Design

Lam, I.P. and Chaudhuri, D

Technical Report MCEER-98-0018

Task Nos. 112-D-3.5 and 112-D-3.6

3.27.1 Research Summary

This report summarizes results of a project that evaluated seismic design methods used to represent pile foundations and drilled shafts for bridge structures. The information on drilled shafts from this task (112-D-3.6) was later combined with similar information obtained for pile footings under task number 112-D-3.5 into one MCEER technical report, MCEER-98-0018. At the time this report was written, two draft reports had been submitted for review. Therefore, two reviews appear for one technical report (see Section 3.26).

This task had the following main objectives: (1) to provide information regarding the influence of the modeling procedure on the response of the structure; (2) to evaluate the effects of modeling on the estimated displacement and force demand on the foundation; (3) to summarize methods of characterizing the response of drilled-shaft foundations, including their limitations; and (4) to give guidance on seismic design practice. The primary focus was on lateral loading.

The contents of this report provide a practical summary of the current state of practice. A key contribution is that the report addresses differences in design procedures from those used for driven piles. These differences are related to the differences in installation procedure, the larger diameter of drilled shafts, the smaller length-to-diameter ratio of drilled shafts, and the differences in structural configuration. A unique feature of the report is that it attempts to provide an interface between the structural and geotechnical design processes.

3.27.2 Research Findings and Review Comments

Seismic design procedures for drilled shaft foundations are reviewed. The review describes methods used to model drilled-shaft foundations for seismic design and then examines the applicability of conventional p - y methods for use in the modeling process.

Seismic Design Procedure

- Foundation stiffness is correctly identified as a very important factor in the dynamic response of a structure. Properly estimating this response necessitates realistic estimates of foundation stiffness and proper integration of that stiffness into a structural analysis. The seismic response of a soil-foundation system is nonlinear; for practical purposes, however, an equivalent linear representation is normally used. Limitations associated with this simplification are noted.
- The two-step process used in seismic design is summarized. *See synopsis in report for Task 112-D-3.5 "Modeling of Pile Footings for Seismic Design" for a summary of and comments on this process.* Three procedures for representing foundation stiffness in the modeling process are also reviewed. These procedures include use of: (1) a coupled foundation stiffness matrix; (2) an equivalent-cantilever approach; and (3) an uncoupled base-spring model. Figure 3-8 (Figure 2-1 of the report) shows sketches of each representation. Features of each, including limitations, are noted in the report.
- Regarding a suitable stiffness matrix, the report notes that one way to ensure that such a matrix is "positive-definite" is to invert it and to verify that the diagonal terms of the inverse matrix are all

positive. *It should be pointed out that a necessary and sufficient condition for a matrix to be positive-definite is to extract its eigenvalues and verify that they are all positive.*

The report provides guidance on the development of equivalent linear and nonlinear stiffness values. It summarizes information on the importance of foundation geometry and boundary conditions at the shaft head. An important point in this discussion is the use of an equivalent stiffness value equal to 0.50 to 0.65 of the stiffness at peak deflection to obtain a more representative response estimate. A key conclusion is that the realistic representation of pile-head fixity can lead to more economical designs. The importance of the cracked-section stiffness for representing the drilled shaft is also noted.

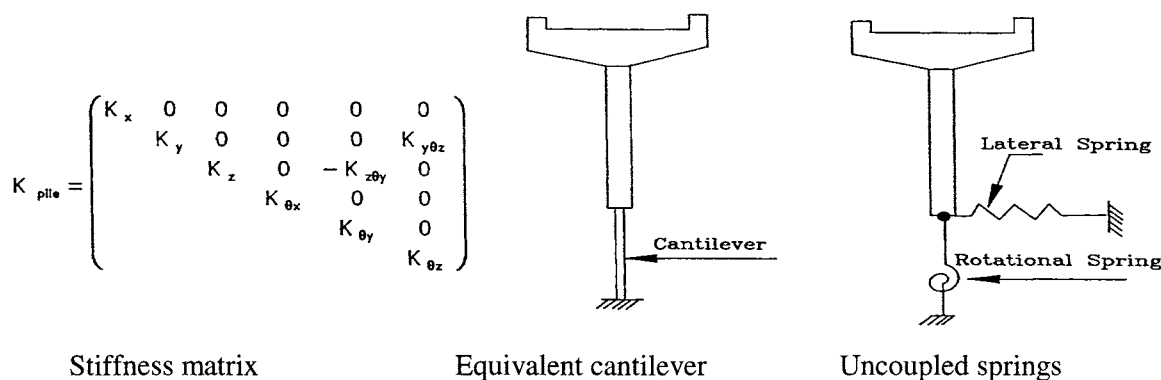


Figure 3-8 Pile Head Stiffness Representations

- A number of other key topics are also covered, including: (1) the consequences of selecting overly soft foundation stiffness values; (2) the location of hinges; (3) the effect of near-ground support (e.g., pavement or jersey barriers) on shear demand; and (4) the minimum length of the shaft needed to ensure stability against excessive rotation. It is pointed out in the discussion that the Caltrans approach to stable-length calculations can lead to unreasonable decisions regarding design. *Regarding foundation stiffness, note that the lower-bound, best-estimate, and upper-bound values for soil parameters relevant to determining soil-structure interaction stiffness should be used for seismic design. By adopting this approach, response variations resulting from uncertainties in material properties will be rationally established.*

Applicability of p-y Model

- The p-y approach is identified as the most common method of analyzing the nonlinear response of the pile to lateral loads, and the applicability of the method to drilled-shaft foundations is examined. Parameters considered in this review include: (1) the effects of soil property variation; (2) degradation effects; and (3) embedment, gapping, and scour effects. The discussion points out that the properties of the pile are more important than soil properties in most cases, and that pile-head displacement values (and equivalent stiffness) are more strongly influenced by soil property variation than pile moment. Gapping and scour are mentioned as having a very important effect on response.
- The load-transfer mechanism for laterally-loaded shafts is reviewed. Much of this review focuses on the cause of and the method of accounting for pile-diameter effects. The report points out that the observed difference between driven-pile and drilled-shaft behavior, which is inferred to be the result of diameter effects, appears to be due at least in part to differences in installation methods. Shaft-head fixity is also mentioned as another important factor, as it influences shaft friction. For typical

conditions under which drilled shafts are installed, a scaling factor directly proportional to the pile diameter greater than 24 inches is recommended for design.

3.27.3 *Related Reports and Research*

1. Task 112-D-3.5: Modeling of Pile Footings for Seismic Design
2. Task 106-E-4.7: Structural Implications of Foundation Modeling
3. Task 106-E-4.10: Synthesis Report: Soil-Structure Interaction and Pile Group Effects

3.27.4 *Limitations*

- A pseudo-static approach is used to model the seismic response of the drilled shaft, under the premise that dynamic effects will be small for the expected period of vibration. Further discussion of the basis for using frequency-independent stiffness and damping coefficients would be helpful. For example, some guidance regarding the period of vibration at which dynamics become a consideration in the modeling process could be provided. Views on radiation- and material-damping effects on drilled-shaft seismic response would also be valuable, particularly as these effects relate to rate-of-loading or loading frequency.
- The soil modulus (E_s) is used to determine the characteristic length of the shaft. A clear explanation of the relationship of this modulus and the Young's modulus derived from, for example, geophysical methods would be helpful.
- This report identifies a number of areas related to the shaft boundary condition for which the current approach to drilled-shaft design requires additional study. These areas include the effects of gapping, liquefaction, and pile-head fixity. These areas should be addressed in a practical manner during future research work.
- A more explicit process for deciding whether or not to include diameter effects appears to be needed. If corrections are required, the method of making the correction for different construction methods and soil conditions should be presented.
- The topic of group effects for drilled shafts is not addressed in this report, although a companion report on driven piles treats this topic in some detail. It would be helpful if some comments on group effects for drilled shafts were added to the discussion. Of specific interest is whether the different installation method used for drilled shafts lessens the potential for group interaction.
- Research on drilled pier foundations subjected to lateral loading has been conducted by the Electric Power Research Institute (EPRI). None of this work was cited in the report. It is unclear whether no useful information is in the EPRI reports or whether EPRI's normal policy of charging high prices to non-members for copies of their research reports precluded the receipt of this information. If the latter is true, it may still be worthwhile referencing journal articles or conference proceedings that summarize the EPRI research.

3.27.5 *Impact*

Code Requirements: Information in this report provides important documentation regarding the seismic design of drilled-shaft foundations. The current AASHTO seismic design provisions do not cover most of the topics included in this report. Rather, AASHTO refers to work completed by Lam and Martin in 1986. While the fundamental approaches may be similar to those provided in the 1986 report, this report updates the earlier work, based on conclusions reached and experience gained over the past 10 years. For this reason it is recommended that information in this report be incorporated into the new AASHTO seismic design provisions.

Design Procedures: The seismic design procedures discussed in this report will not necessarily result in any significant modification to current design procedures. It is believed that individuals involved in design in high seismicity areas have already adopted these procedures. In areas of low-to-moderate seismicity, it may be appropriate to simplify these procedures. Adoption of these procedures into the new AASHTO seismic design provisions would (1) provide a better degree of consistency between different designers, (2) potentially eliminate some of the unnecessary and undesirable conservatism in design, and (3) assist in avoiding errors that are commonly made during design.

Design Costs: It is anticipated that the change in design costs to account for recommendations given in this report will be limited.

Structural Performance: Adoption of these design procedures in the new AASHTO seismic design provisions will likely result in more realistic and reliable structural performance predictions.

Construction Cost/Constructibility: The recommended design procedures could result in higher or lower construction costs, depending on the specific conditions of the site and the foundation system. In most cases, it would be expected to result in a more efficient foundation system, which generally leads to lower construction costs.

Research Recommendations: Additional research is warranted in the area of drilled-shaft design for seismic loading. Some of the topics warranting research consideration include: (1) the effects of scour, gapping, liquefaction, and shaft-head fixity on the lateral capacity of drilled shafts; (2) the influence of shaft diameter on lateral response; (3) the energy dissipation of single shafts and groups of shafts; and (4) the influence of construction method on seismic response.

3.28 Development of Analysis and Design Procedures for Spread Footings *Gazetas, G., Mylonakis, G., and Nikolaou, A. (unpublished report)* *Task No. 112-D-3.7*

3.28.1 Research Summary

This report provides information about the seismic design of bridge piers founded on spread footings. The report addresses five issues: (1) when to incorporate foundation stiffness in the dynamic analysis of bridge piers, (2) the significance of proper modeling of the effect of embedment on the dynamic stiffness of the foundation, (3) the importance of radiation damping and kinematic interaction in response, (4) conditions under which uplift becomes significant and how to model it in design and analysis, and (5) the significance of local soil nonlinearity under the edges of a rocking foundation and how to account for it in the analysis.

3.28.2 Research Findings and Review Comments

The contents of this report include some interesting observations regarding the response of a spread-footing foundation system during seismic loading. These observations deal with the importance of soil-structure interaction, embedment, and radiation damping for system response. Observations are also made regarding seismic bearing capacity and footing uplift.

The report describes a series of numerical analyses that address the objectives of this research project. These analyses were conducted for a single-column bent founded on a footing on the surface of, or embedded in, a layered soil profile.

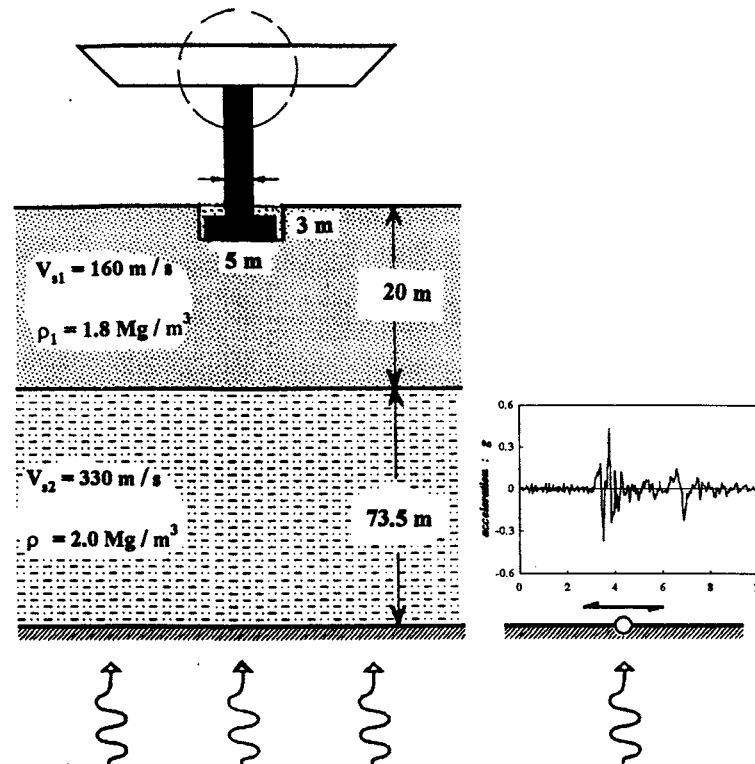


Figure 3-9 Pier on Footing Model Studied in this Report

Problem Formulation

- The report investigates the issues by means of a semi-analytical formulation for the dynamic analysis of pier-foundation systems. This formulation is applied to an idealized bridge pier subjected to harmonic excitation as well as two representative earthquake records scaled to 0.4g. Figure 3-9 (Figure 1.1 of the report) shows the case selected for evaluation.
- Kinematic and inertial interaction effects are incorporated in the analyses. Soil nonlinearity is introduced through a viscoelastic formulation in the expressions for swaying and rocking impedance values for a rigid strip footing using a secant shear modulus and an effective damping ratio.
- Results are presented for “free” support conditions, where the top of the column-deck support connection can rotate.

Results of Parametric Study

- Results show the following for the no-uplift case: (1) ignoring soil-structure interaction (SSI) reduces the fundamental period of the system, which results in higher acceleration values; (2) increasing the effectiveness of embedment increases radiation damping and reduces the fundamental period of the system; and (3) neglecting radiation damping has a minor effect on results. *Some of these*

conclusions (for instance the one regarding the impact of embedment) apply to general cases. Other results are valid for the particular case investigated, but perhaps not for other soil-foundation combinations.

- Results indicate that horizontal inertial forces within the failure zone below the footing have a small effect on the bearing capacity of the soil during seismic loading. This observation led to the conclusion that static methods for including inclination and eccentricity in static bearing-capacity determinations can be used to account adequately for the inertia of the superstructure during a seismic event. *These observations differ from recent work (Richards et al., 1993). The difference between results given by Gazetas et al. (unpublished report) and those given by Richards et al. (1993) should be addressed by each set of researchers in an effort to reconcile current discrepancies.*
- Results show that uplift of the footing results in a softer vibrating system. The fundamental period increases with the amount of uplift, while effective damping for the system decreases. The net effect of these changes on the seismic response will depend on the particular soil-structure system and the characteristics of the excitation.

3.28.3 Related Reports And Research

Task 112-D-3.4: Seismic Analysis and Design of Bridge Abutments Considering Sliding and Rotation.

3.28.4 Limitations

- Documentation of the method used in the study is limited. It seems that two independent computer codes were used: one for analyzing the soil-footing-bridge seismic response and the other for studying seismic bearing capacity. In the absence of information about the code(s), it is very difficult to judge (in the draft report reviewed) the validity of any of the conclusions reached in the report. This situation should be remedied before the report is published or cited elsewhere.
- Only a single case was treated in the analysis of soil-footing-bridge seismic response, rather than the “range of bridge pier types” that was discussed in the original scope of work. The original task description also states that caissons would be included in the study, in addition to spread footings.
- Soil stiffness for the case study was very low; i.e., $V_s = 160$ mps. No discussion is provided regarding the applicability of the results at sites where soils are stiffer and where spread footings would more likely be used. Additional cases also need to be studied to examine the applicability of the conclusions regarding bearing capacity for stiffer, cohesive soils and all cohesionless soils.
- The study considered a rigid spread footing located on a very deformable foundation. In many situations, the ratio of footing rigidity to soil rigidity will not be as great. Some consideration needs to be given to this issue, if for no other reason than to identify when it is an issue and when it is not.
- It is inferred from the report that the Gazetas (1991) formulas for foundation springs and dashpots can be used to represent the seismic response of spread footings. However, no guidance is provided on the selection of strain-compatible shear modulus values to use within the formula. This guidance is required to achieve the overall goal of providing “design procedures.”
- The study concludes that the effect of uplift on the seismic response will depend on the parameters of the system and the characteristics of excitation, and that it cannot be concluded if it is beneficial for the bridge or not. It would be valuable if the study could relate this conclusion to the results of model or full-scale tests to show that the uplift phenomenon is not strictly a function of boundary conditions within the computer code. Also, assuming that the uplift phenomenon is physically correct, some guidance has to be developed for estimating these effects in practice. For instance, is it sufficient to modify spectral ordinates in a response-spectrum analysis on the basis of the increased

period and the reduced damping? This approach might be appropriate to calculate shear forces but not displacements.

3.28.5 *Impact*

Code Requirements: The product of this research provides some interesting results regarding the performance of spread footings during seismic loading. However, the results, as currently developed and presented, cannot be easily incorporated into code provisions. The parametric study of the seismic response of footings without uplift essentially covers a single combination of soil, structural type, and earthquake loading. The validity of these results for other combinations of soils, structures, and earthquake characteristics needs to be assessed. Some results regarding the seismic effects on bearing capacity differ from those of other researchers. The differences should be documented and reconciled. With additional evaluation and further documentation of the analysis method, the results of this task could be adopted into new codes.

There is a clear need within current codes to provide bridge designers with simple-to-use methods for estimating the stiffness and geometric damping of spread footings. One approach for making this estimate is to use existing spring and dashpot formulas such as are given in other publications (Gazetas, 1991; Gazetas et al, unpublished report). However, for these formulas to be useful, a method of introducing a strain-compatible shear modulus must be presented. This method must account for the likely variation in soil response at different levels of earthquake loading and for variations in soil properties.

Design Procedures: Most designers currently use procedures suggested by Lam and Martin (1986) for the seismic design of spread-footing foundations. An update of these procedures based on work completed by Gazetas and his co-workers will not necessarily result in significant changes in the current design process. Adoption of these procedures into the new AASHTO seismic design provisions would (1) provide a better degree of consistency between different designers, (2) potentially eliminate some of the unnecessary and undesirable conservatism in design, and (3) assist in avoiding errors that are commonly made during design.

Design Costs: It is anticipated that the change in design costs to account for developments that have occurred after the publication of Lam and Martin (1986)¹ will be limited.

Structural Performance: Adoption of modification to the Lam and Martin (1986) design procedures in the new AASHTO seismic design provisions will likely result in more realistic and reliable structural performance predictions.

Construction Cost/Constructibility: The modified design procedures could result in higher or lower construction costs, depending on the specific conditions of the site and the foundation system. In most cases it would be expected that a more efficient foundation system would result, which generally leads to lower construction costs.

Research Recommendations: Additional research is warranted in the area of spread footing design for seismic loading. Some of the topics warranting research include:

¹ Information in the Task 112-D-3.7 report cannot currently (May 1998) be used to update the work by Lam and Martin (1986). However, reference is made in the task report to work that Gazetas and his colleagues have published in earlier reports and journal articles. This published information could be used to update the work by Lam and Martin and could form the basis for sections within a new AASHTO seismic design provisions. For this reason, reference is made to "updating work" by Lam and Martin, rather than referring to work presented by the Authors.

- Methods of determining strain-compatible stiffness and damping values
- The validity of conclusions and recommendations provided in the report for other soil conditions and foundation types
- Simplified procedures for incorporating the effects of uplift during design

It is also critical that the difference between conclusions reached in this report and those reached by Fishman and Richards (1997) be resolved. To this end, it is recommended that Gazetas and his co-workers review the research results of Fishman and Richards and then reconcile questions and uncertainties relative to the Fishman and Richards report. In the absence this effort, the use of information developed during this task will be very limited.

3.29 Synthesis Report on Foundation Stiffness and Sensitivity Evaluation on Bridge Response *Lam, I.P., Martin, G. R., and M Kapuskar (unpublished report)* *MCEER Task No: 112-D-3.8*

3.29.1 Research Summary

This report summarizes the results from a limited sensitivity study of bridge response to different values of abutment and bent foundation stiffness. The sensitivity study considered a typical, squared, two-span bridge. Abutments for the bridge consisted of a pile-supported end wall monolithically connected to the deck structure. During the sensitivity study, the central bent was supported by four different foundation systems: pile groups, spread footings, pier walls, and drilled shafts. The first objective of the report was to demonstrate procedures given in three other MCEER Task 112-D-3 Projects (Task 112-D-3.3: Modeling of Bridge Abutments in Seismic Response Analysis of Highway Bridges; Task 112-D-3.5: Modeling of Pile Footings for Seismic Design; and Task 112-D-3.6: Modeling of Drilled Shafts for Seismic Design) for determining the stiffness for each of these foundation systems. The second objective was to show how different values of foundation stiffness, representative of the different foundation types, affect the natural periods of vibration of the bridge. From these results it was possible to identify components of foundation stiffness that are most significant to typical applications.

A companion study involving the sensitivity of liquefaction-induced lateral spreading to the relative density of liquefied sands is also included as an appendix to the report. This is an important load case that must be considered during the design of pile foundations.

3.29.2 Research Findings and Review Comments

This is a valuable reference demonstrating the determination of stiffness values. Appendices to the report show actual calculations for the selected representative foundation types. This information should serve as a useful guide to designers as they determine foundation stiffness values for bridge foundation analyses. Results of the sensitivity study also demonstrate the importance of the abutment stiffness in the overall bridge response analysis.

- The computer program DRAIN-3DX was used to conduct the sensitivity evaluation. The basis of the model was the Painter Street Overcrossing in California. (See Figure 3-10, copied from Figure 2-1 of the report.) *It would have been helpful to include a few drawings showing the Painter Street Overcrossing, to give some perspective to the descriptions provided in the text. Also, information about the soil conditions would have been valuable.*

- The study was limited to a linear bridge/foundation system. *It is assumed that this means that no attempt was made to achieve compatibility between stiffness values for the foundation and predicted displacements. If displacement compatible springs had been used, would the results (dominant natural periods) have changed to any extent? From Table 2 it appears that nothing affected the 1st mode period. This seems a little surprising, considering the factor of 3 difference in abutment stiffness.*

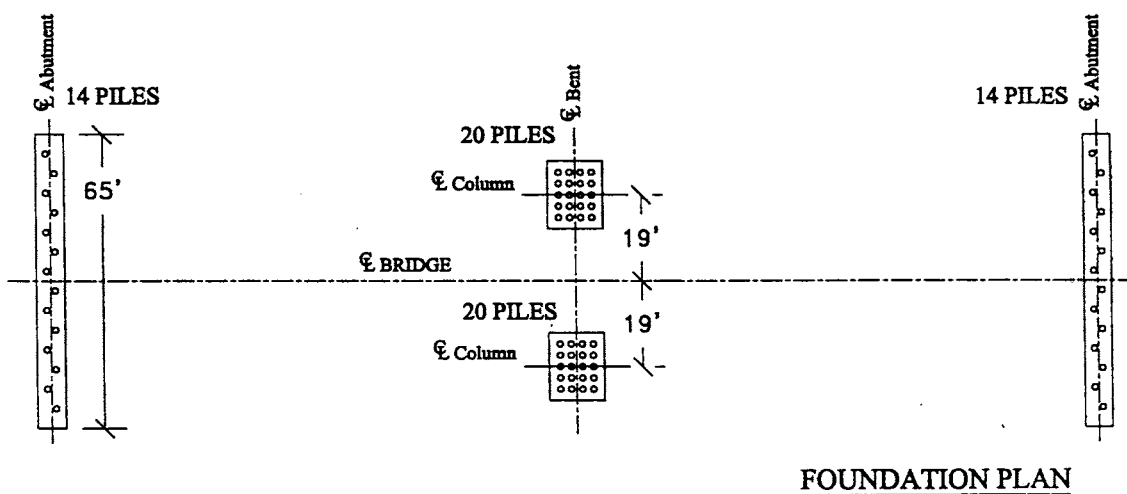
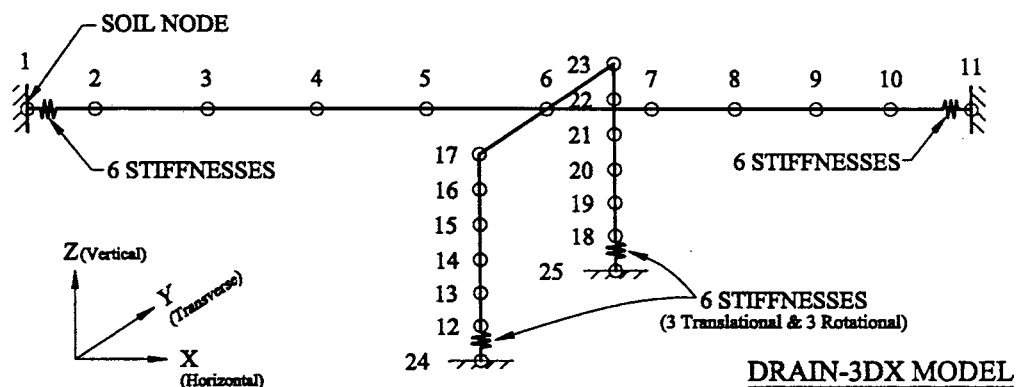
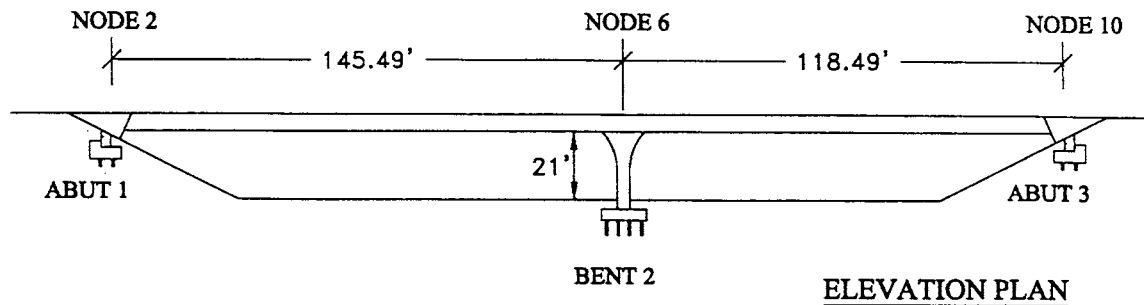


Figure 3-10 Model of Painter Street Bridge used for Sensitivity Studies

- Procedures for determining the foundation stiffness for the abutments in translation (longitudinal and transverse) and rotation are given. *Information in Appendix A showing these computations is relatively easy to follow, although a little more explanation on the rotational stiffness determination for the pile group would have been helpful. Cross-coupling was apparently ignored, which leads to the question: When should cross-coupling terms be considered?*
- Bent stiffness values were calculated for each of the bent foundation types (i.e., pile groups, spread footings, pier-walls, and large diameter shafts). In addition to presenting computations, the discussion identifies when shear keys are normally introduced, resulting in zero stiffness. Methods of performing the stiffness determinations for the pile group are summarized (Appendix B). The determination includes the contribution from the pile cap. In this evaluation the pile cap contributed a relatively small percentage to the overall lateral stiffness. For spread footing foundations, conventional elastic equations for stiffness determination are presented. Appendix C, which summarizes the computational method for spread footings, suggests reducing the low-strain amplitude shear modulus by up to 80 percent to account for large strain and cyclic strength degradation effects. *The appropriate strain reduction factor continues to be an uncertainty. Future research needs to identify a simplified method for making this adjustment.* The pier-wall stiffness determination is summarized in Appendix D. The shaft stiffness determination was made for 9-foot diameter shafts (Appendix E). A table showing the stiffness for each case is given.
- Results of sensitivity studies based on the different stiffness values are presented. *No information was found indicating the response spectrum and peak ground acceleration used for the analyses. Would the selection of the ground motion record influence any of the results in Table 2?* Results of the sensitivity study indicate that the “half/half” approach used for the model was adequate for the linear model. For these analyses the bent stiffness did not have a significant effect on the natural period of the system, and it was evident that abutment stiffness and ultimate capacity of the abutment were very important.

3.29.3 Related Reports and Research

1. Task 112-D-3.3: Modeling of Bridge Abutments in Seismic Response Analysis of Highway Bridges
2. Task 106-D-3.5: Modeling of Pile Footings for Seismic Design
3. Task 106-D-3.6: Modeling of Drilled Shafts for Seismic Design

3.29.4 Limitations

- This is a valuable summary of procedures for determining the stiffness for different foundation types. It would have been helpful if more details of the computation process were provided. In several cases the computations required careful study to see the progression of the work. It might have been worthwhile to demonstrate the computations for several foundation types in each group to illustrate different issues that might be encountered during a design problem.
- The sensitivity study was for a single bridge system subjected to a specific response spectrum or earthquake record. It would be worthwhile extending the sensitivity study to other cases, specifically to determine when the abutment foundation springs are not so important. It might also be worthwhile considering the “East Coast” model, which does not account for abutment stiffness, to show the sensitivity of response to different bents.
- Information in Appendix F should have been tied to the overall report. It currently seems to be without context.

3.29.5 *Impact*

Code Requirements: The current AASHTO seismic design provisions provide little guidance in the determination of stiffness values for abutments and bent foundations. While the form of this report does not lend itself directly to the development of codes, it will be a useful document when preparing commentary. Specifically, the examples in the appendixes to the report show where uncertainties will occur in the determination of stiffness values for different foundation systems, and where extra discussion in code commentary will be warranted.

Design Procedures: The design procedures identified in this report are being used in some form by consultants in California. If the new AASHTO specifications were amended to include these procedures, a more uniform process of design would result. In many cases, the procedures presented in the report are shown to be straightforward. If these methods can be clearly presented in a new version of the seismic design section, there may not be as much resistance by some groups outside California to implement the methods. Nevertheless implementation of these methods will involve increased design effort. It will also require more coordination between the bridge designer and the geotechnical engineer than currently occurs, if the bridge designer is forced to consider a condition other than a fixed base.

Design Costs: The increase in design cost by implementing foundation stiffness, and particularly abutment stiffness, into bridge design methods is not expected to be great.

Structural Performance: Structural designs that use more realistic foundation stiffness conditions are expected to perform better during seismic events. It is also possible that, with better modeling of the system of bridge, abutment, and approach fill, new and better methods of resisting seismic loading will be identified. In addition to improving performance, these developments could lead to lower costs.

Construction Cost/Constructibility: The use of foundation stiffness values in the bridge response modeling could result in lower construction costs by using more efficient designs. Following a seismic event, it is expected that repair costs to damaged bridges would be reduced.

Research Recommendations: Research recommendations for bridge foundation modeling are found in the task reports upon which this sensitivity study was based.

3.30 **Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils** *Edited by Youd, T.L., and Idriss, I.M.* *Technical Report NCEER-97-0022* *Task No. 112-D-4.2 (and 106-E-3.2b)*

3.30.1 *Research Summary*

The report presents results of a workshop convened in January 1996, for the purpose of reviewing developments in the simplified procedure for evaluating the liquefaction resistance of soils and gaining consensus on further improvements and augmentations that should be made to the simplified procedure. This workshop was the first since a similar workshop sponsored by the National Research Council (NRC, 1985). Emphasis was placed on developments that had been published subsequent to the 1985 NRC workshop.

Workshop participants focused their review and discussions on evaluation procedures used to predict the triggering of liquefaction. They limited their discussions to shallow soil deposits on level or nearly level ground. From these discussions the workshop participants developed consensus recommendations on: (1) the use of standard and cone penetration tests; (2) the use of shear-wave velocity measurements; (3) the use of Becker penetration resistance for gravelly soils; (4) magnitude scaling factors; and (5) correction factors K_{σ} and K_{α} . The workshop participants also addressed frequently asked questions regarding the determination of earthquake magnitude and peak ground acceleration that trigger liquefaction. Liquefaction criteria based on probabilistic and seismic energy methods were also considered.

3.30.2 Research Findings and Review Comments

The proceedings consist of a summary report and nine papers that were written by workshop participants. The following review comments are directed at the contents of the summary report; the review of the nine papers is limited to a listing of their topics.

Simplified Method for Predicting Liquefaction Resistance of Soils

- The report notes that the simplified procedure for evaluating liquefaction resistance has become the standard of practice. This simplified method is based on field data from sites on level to gently-sloping terrain underlain by Holocene alluvial or fluvial sediments at shallow depths (less than 15 m).
- The workshop participants recommended minor modifications to the determination of the stress-reduction factor, r_d , used in the calculation of the cyclic stress ratio (CSR). These equations allow the stress-reduction factor to be calculated for depths greater than 30 m. *By presenting these equations, it is implied that the simplified method can be used for depths in excess of 30 m. The limitations, if any, associated with use of this method beyond 15 m should be clearly stated.*

Cone Penetration Test (CPT)

- The workshop summary reviews the latest procedures for determining cyclic resistance ratio (CRR) using CPT methods. One of the primary advantages of CPT methods is the repeatability of the results. The greatest limitation appears to be that CPT does not provide direct soil samples, so soil type must be determined indirectly.
- The summary report presents plots and equations for determining liquefaction resistance directly from CPT data, rather than converting to an equivalent standard penetration test (SPT) N -value. The report includes procedures for correcting CPT data for overburden pressures, grain-size characteristics including fines content, and thin layers.

Standard Penetration Test (SPT)

- The workshop summary reviews the methods for determining CRR using the SPT. The primary benefit appears to be the long history of use of the SPT in developing the CRR. The greatest limitation is the large number of equipment adjustments that are needed.
- The summary report presents a revised plot for determining CRR from SPT data. This plot revises the CRR for $(N_1)_{60}$ less than 10. Procedures for correcting SPT data for fines content, overburden stress, and equipment variables are also given. Criteria based on both CPT and SPT measurements are quantified in equations for ease of use in programming, spreadsheets, and other computational aids.

Shear-Wave Velocity

- The workshop summary reviews the latest methods for determining CRR using shear-wave velocity measurements. The primary benefit appears to be the ability to obtain data without drilling a borehole and without testing material such as gravel that cannot be reliably penetrated with the SPT and CPT. The limitations include the low-strain characteristics of the measurement and the absence of soil samples (unless a limited boring program is included with the velocity measurements).
- The summary report presents plots for determining CRR from shear-wave velocity data. Procedures for correcting velocity data for overburden stress and fines content are given.

Becker Penetration Tests

- The workshop summary reviews the latest method for determining CRR using the Becker penetration test. The primary benefit appears to be the ability to obtain data in gravelly soils. The greatest limitations seem to be deviations in hammer energy and in understanding and incorporating the influence of casing friction.
- The summary report presents plots for determining corrected SPT N -value from the Becker Penetration Test. Guidelines for conducting the test are provided.

Magnitude, Overburden, Static Shear-Stress Corrections, and Seismic Factors

- Available methods for adjusting the CRR for different earthquake magnitudes are reviewed in the summary report, and a consensus recommendation is given. The recommendation involves modifications to the original Seed and Idriss (1982) method. The recommended magnitude scaling factors (MSF) values for less than magnitude 7.5 are larger than the original Seed and Idriss (1982) MSF values, indicating a lower calculated hazard. For magnitudes greater than 7.5, the reverse occurs.
- Corrections for effective confining pressures greater than 108kPa (one ton per square foot) are given. *This seems to imply that it is okay to use the simplified method beyond a stress of 108kPa (one ton per square foot) as long as K_{σ} is introduced; however, the uncertainty associated with the prediction of liquefaction increases.* The status of corrections for sloping ground (i.e., slopes > six percent) were discussed. The consensus was that this case should not be covered in the guidelines because the effect of sloping ground is not well understood at present.

Topic Papers

Special topic papers were also included in the report. The topics are listed below.

- Cyclic liquefaction and its evaluation based on the SPT and CPT
- Liquefaction resistance estimates based on shear-wave velocity
- Application of the Becker penetration test for evaluating the liquefaction resistance of gravelly soils
- Magnitude scaling factors
- Application of K_{σ} and K_{α} correction factors
- Seismic factors
- Liquefaction criteria based on probabilistic analyses
- Liquefaction criteria based on seismic energy analyses
- Cyclic liquefaction predictions based on the cone penetrometer test

3.30.3 *Related Reports and Research*

1. Task 106 State-of-the-Art Review: Soils and Liquefaction

3.30.4 *Limitations*

- The original stress-reduction coefficient, r_d , used to determine the cyclic stress ratio inferred a degree of uncertainty, particularly at greater depths. For non-critical projects, the new consensus document recommends equations for various depth ranges. While this may be of greater convenience for spread-sheet programming, the equations seem to lose some of the uncertainty conveyed by the original plot. Some additional discussion of the limitations in this regard or the basis for the change in approach would have been valuable.
- A clear statement on depth limitations for the simplified method is needed. In the introduction and at other points within the Proceedings, the workshop participants suggest that the method was originally developed for depths of 15 m or less. However, values are given for the stress-reduction coefficient and K_σ for depths of 30 m or more, implying that the method can be extrapolated to this depth. Since low SPT or CPT values exist at depths greater than 15 m, there will be potential for confusion regarding whether or not to extrapolate the procedure to greater depths.
- The SPT discussion identifies a number of corrections that are needed to adjust the N -values to a “standard” N -value. One of the most significant is the energy ratio. A range of approximately two is given for two types of hammer—the safety hammer and the donut hammer. Energy ratios for the automatic hammer are not provided. Because this summary report is a guide that will be used by many within the profession, it would be very valuable to provide a little more direction on the selection of an energy ratio.

3.30.5 *Impact*

Code Requirements: Procedures presented in the workshop should be incorporated within the new AASHTO seismic design provisions.

Design Procedures: The recommended code change could result in small changes in design procedures used by some individuals. However, the workshop report has been referenced extensively and will likely be regarded as the standard of practice at the time the AASHTO seismic design provisions are next modified.

Design Cost: There will be little if any changes in design cost if the recommendations in the workshop report are imposed on designers through code requirements.

Structural Performance: The recommended liquefaction guidelines are not expected to change structural performance for well-designed projects. These guidelines will provide minimum standards for all highway bridge projects, which should result in greater consistency in response predictions. This could lead to better performance in some cases.

Construction Cost/Constructibility: As with structural performance, adopting the workshop guidelines should have only a minimal effect on construction costs.

Research Recommendations: While the report from the liquefaction workshop represents a consensus of experts in 1996/1997, the whole topic of liquefaction still requires a significant amount of additional research. Some of the areas requiring research include: (1) methods of determining liquefaction

resistance below 15 m; (2) the effects of sloping ground on determining liquefaction potential; (3) procedures for estimating the liquefaction resistance in gravelly soils and silts; and (4) procedures for estimating pore-pressure buildup in soils that undergo only partial liquefaction.

3.31 Development of Liquefaction Mitigation Methodologies: Ground Densification Methods
Martin, G.R. (unpublished report)
Task No. 112-D-4.3

3.31.1 Research Summary

This report presents results of investigations that were completed to evaluate liquefaction mitigation methods. The objective of the report is to present information that will add to the understanding and optimization of in situ ground densification. The report focuses on the vibro-replacement method of ground improvement, which is the most widely used method in the United States.

Densification techniques currently being used in the United States and Japan are briefly summarized. The results of a series of studies related to the vibro-replacement ground improvement method follow. The studies involved optimization of the cost-effectiveness of the ground densification method, development of new design models for the method, and application of vibro-replacement methods to silty soils.

3.31.2 Research Findings and Review Comments

State of Practice and Effectiveness

- In an overview of this discussion, five recent publications considered to be of particular relevance to the ground densification topic are listed. *These references provide an excellent basis for further reading on the topic.*
- Each site is unique and this uniqueness will likely influence the selection of a particular method. Considerations in the selection of the optimum method include: (1) the location, area, depth, and volume of soil involved; (2) site conditions; (3) earthquake ground shaking levels; (4) structural type; (5) economic and social factors associated with the site improvement technique; (6) the availability of necessary materials such as sand and gravel; and (7) the availability of equipment and skills.
- Ground densification methods being used in the United States include vibro-compaction, vibro-replacement, dynamic compaction, compaction grouting, and blast-induced densification.
- Methods used for ground improvement in Japan include procedures similar to the vibro-methods and dynamic compaction methods used in the United States, as well as two methods not commonly used in the United States, namely, sand compaction piles (SCP) and rod compaction method (RCP).

Vibro-Replacement Ground Improvement Method

- Results of a field research experiment involving the use of vibro-replacement ground improvement methods are presented.
- In addition to the normal densification measurements, pore pressure and vibration information was collected. Cone penetrometer data were also collected one day, and again one year, following improvements.

- Results include information about the change in pore pressure and vibration level as a single stone column was installed and as a function of distance from the point of improvement. The vibration results address an often-asked question regarding the use of vibro-replacement procedures in areas of existing utilities or developments. *These results suggest that only the area within 3 m or so (10 ft) has levels of vibration that are high enough to cause damage.*
- Penetration measurements were also made at the same site for groups of stone columns installed at 1.8, 2.4 and 3.0 m (6, 8, and 10 ft). The confining effects of multiple columns was shown to cause significantly more ground improvement. The time effects recorded for single columns are also thought to occur for the group of columns. *This might be a topic for future research, given the amount of increase that occurs for single columns.*
- The results of the field experiments are used to identify four mechanisms contributing to ground improvement: (1) vibrator-induced liquefaction; (2) displacement in the absence of liquefaction; (3) changes in lateral confinement; and (4) time effects.
- Results of a field study of the permeability and aggregate stiffness of a stone column is also reported. This study included field measurements of the hydraulic conductivity of the stone column and crosshole shear wave velocity tests.
- Results of field measurements in the columns were compared to hydraulic conductivity values from laboratory tests and empirical estimates. Observations are made on the results from the different methods.
- Shear wave velocity measurements were made to determine the change in velocity in the soil between the columns, as well as the velocity in the columns. Results are used to define the ratio of velocity in the stone column to the improved soil. Procedures for adjusting the modulus ratio (i.e., stone column to improved soil) are also suggested.
- A model of the ground improvement process is described. This model includes the densification, drainage, and column stiffness effects. In contrast to practice in the United States, the Japanese use the stone column itself as a method of dissipating pore pressures. *Based on discussions, it is understood that in the Japan study, the columns were closely spaced.* Adoption of such procedures in the United States could allow increased column spacing.
- As an example the model is used to design a ground improvement program.

Application of Vibro-Replacement Method to Silty Soils

- The vibro-replacement method is normally successful in densifying soils where the fines content is less than 15 percent. The reason for this is the need for rapid dissipation of pore pressures. Higher silt percentages will impede drainage, which could limit the effectiveness of the densification process. However, if the compressibility of the silt is low, then radial consolidation coefficients may be high enough to offset this effect, permitting the use of vibro-replacement in non-plastic silty soils.
- A detailed description of a case study involving densification of non-plastic silty soil is presented. This presentation includes a discussion of results from standard penetration tests, cone penetrometer tests, dilatometer tests, and seismic cone tests completed before and after densification. Silt content and the at-rest earth pressure coefficient are identified as important factors in the comparison of the results. *There is a significant amount of important information in the case history. It would have been helpful to have an interpretation and recommendation on how to treat other silty sites based on this case history.*

3.31.3 Related Reports and Research

Task 106-F-1.1: Liquefaction Remediation Techniques for Bridge Foundations

3.31.4 Limitations

- Several key components of this compilation depend on results from Task 106-F-1.1 (Liquefaction Remediation Techniques for Bridge Foundations), which is still being completed. As a result, the guidelines provided in this compilation report will have to be regarded as interim.
- A number of the procedures for ground improvement used in Japan are proprietary. Some of the key factors in determining their success cannot be determined or evaluated, relative to methods available on a non-proprietary basis.
- Considerable debate exists regarding the occurrence of deep liquefaction, both in terms of the maximum depth that liquefaction can occur and the methods for assessing the liquefaction potential beyond depths of 21 m (70 ft) below the ground surface. Considering the costs of improving ground beyond this depth, the potential for deep liquefaction seems to be an issue that warrants more attention.

3.31.5 Impacts

Code Requirements: Key components of this report could be either incorporated in the new AASHTO seismic design provisions or included as a commentary. The current specifications do not provide guidance on mitigation methods.

Design Procedures: The information in this report is valuable from the standpoint of identifying a more rational and systematic approach for investigating, designing, and implementing ground improvement methods. Limited guidance currently exists in this area. This undoubtedly has led to incorrect or inefficient ground improvement methods on past projects.

Design Cost: Design costs associated with the adoption of these methods could increase the level of engineering work on some projects. However, the amount of the increase is expected to be nominal, except in special circumstances.

Structural Performance: Information given in this report will result in a better understanding of the behavior of improved ground. This should lead to more reliable predictions of structural performance for structures supported on or in the ground.

Construction Cost/Constructibility: Design methods developed as a result of this task could alter construction costs, depending on the particular design and specific site conditions. In most cases it would be expected that more efficient ground improvement would lead to lower construction costs. Implementation of these methods should also result in lower postearthquake repair costs.

Research Recommendations: While the general topic of liquefaction mitigation methods has evolved significantly over the past 10 years, a number of areas associated with mitigation or ground improvement warrant further research. These research areas involve: (1) methods of predicting the most efficient ground improvement method at sites comprised of mixed and layered soils conditions; (2) the validity of simplified methods, such as presented in this report, for modeling the response of the soil/structure system (for example, deformation and porewater pressure buildup); and (3) additional postearthquake evaluations of sites where ground improvement has been carried out. Results of the postearthquake evaluations should be used to confirm the results of research in areas (1) and (2).

Other areas warranting research include the use of wick drains to enhance the likely success of vibro-replacement methods and the effects, over time, of ground improvement for groups of stone columns. In

a related area, additional research appears to be required in the area of deep liquefaction, for example, the maximum depth and methods for assessing potential.

3.32 Design Recommendations for Site Response and Liquefaction Mitigation

Martin, G.R. (unpublished report)

Task No. 112-D-4.4

3.32.1 Research Summary

This report synthesizes research work and current practice related to those seismic design problems involving the influence of site soils on earthquake ground response and the identification and mitigation of ground liquefaction. The objective of the task was to compile results of recent MCEER-sponsored research into a form that can be used to develop seismic design guidelines and codes.

The research focused on integrating results of three tasks within the New Highway Construction Project (Tasks 112-D-4.1, -D-4.2, and -D-4.3) and four tasks within the Existing Highway Construction Project (Tasks 106-E-2.7, -E-3.1, -E-3.2, and E-3.3). These tasks addressed a number of key topics, including one- and two-dimensional site response in the absence of liquefaction, liquefaction assessment procedures, and liquefaction mitigation techniques. Displacement demands created by the lurching of the ground after liquefaction and from the lateral movement of abutments during liquefaction are also considered. The synthesis of information from these tasks focused primarily on issues that are of interest or of concern to engineers specifically involved in the design of bridges and related transportation structures

3.32.2 Research Findings and Review Comments

This synthesis report provides fundamental information about the effects on bridge structures of ground acceleration and displacement-induced loading caused by liquefaction. It separates the presentation into the areas of site response without liquefaction and ground deformation with liquefaction. Information that has been collected to date and presented in interim reports and professional papers is summarized below.

Site Response

- The report reviews procedures for determining response spectra that have recently been introduced in the *Uniform Building Code* (ICBO, 1997) and the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC, 1997). These new procedures account for differences in soil conditions in the upper 30 m of the soil profile and the anticipated peak ground acceleration at the site. Since these procedures provide a better representation of site response than existing AASHTO specifications, the report recommends that the UBC/NEHRP procedure be adopted in the next update of the AASHTO specifications.
- The issue of two-dimensional response of longer-span bridges is also reviewed. By reference to specific examples, this discussion covers conditions under which two-dimensional nonlinear response leads to significant departures from more conventional one-dimensional analyses and the role of two-dimensional response in generating incoherent ground motions. Review of the two-dimensional studies suggests that complex topography and subsurface stratigraphy can significantly influence the coherence of surface ground motions.

Liquefaction

- Results of recent workshops on liquefaction are summarized. These workshops dealt with three issues related to bridge design and retrofit: (1) estimating the potential for liquefaction at a site; (2) the residual strength of liquefied soil; and (3) methods for estimating soil deformations during liquefaction. The report discusses the relevance of the workshop results for bridge foundation design.
- The report identifies special issues related to determining liquefaction potential, including silty soils and layered soils. Procedures for addressing these issues are suggested.
- The effects of liquefaction on free-field ground motions at a level-ground site are also reviewed. This review addresses the common occurrence of a nonliquefied surface soil layer above a liquefied layer with respect to changes in peak ground acceleration, frequency, and ground displacement.
- Methods for estimating deformations of approach-fill slopes located on liquefied soil layers are reviewed. This review covers the use of simplified lateral spreading equations such as those suggested by Youd and Bartlett, sliding-block (Newmark) analysis procedures, and two-dimensional nonlinear response methods. Uncertainties associated with each of these approaches are also identified.
- Procedures for mitigating the occurrence of liquefaction are reviewed. Key considerations in the planning and conduct of the mitigation programs are summarized. These considerations include suggestions on the extent of liquefaction mitigation necessary to protect a structure. The effect of silt content on mitigation methods is described. The report also discusses the minimum factors of safety that should be considered during liquefaction assessments.

3.32.3 Related Reports and Research

1. Task 106-E-2.7: Two-Dimensional, Non-Linear Site Response
2. Task 106-E-3.1: Effects of Liquefaction on Vulnerability Assessment
3. Task 106-E-3.2: Synthesis Report: Liquefaction Vulnerability Assessment
4. Task 106-E-3.3: Centrifuge Studies on Embankment Ground Deformations
5. Task 112-E-4.1: Site Response Effects
6. Task 112-E-4.2: Identification of Liquefaction Potential
7. Task 112-E-4.3: Development of Liquefaction Mitigation Methodologies

3.32.4 Limitations

- Recent information suggests that some of the factors in the UBC/NEHRP procedure for evaluating site effects will change in the near future. These changes are not unexpected, as they represent the normal process of refining procedures as new information becomes available. This situation leads to a question as to whether additional conservatism should be incorporated into the site response factors to accommodate future changes.
- The need for two-dimensional ground response evaluations appears to be a key issue. Such studies will likely require special expertise in ground response modeling. If simplified rules cannot be identified for introducing these effects, then guidance will be necessary on how to identify when special studies are needed and how to carry out the studies.

3.32.5 Impact

Code Requirements: The current AASHTO specifications for seismic design use response spectra and site-response factors that are outdated. Procedures summarized in this synthesis report can be used to

update these methods to be consistent with the standards of the profession. With regard to two-dimensional effects, a decision still needs to be made whether future AASHTO specifications should address the potential for these effects. It is unclear from the existing information whether simplified procedures can be established for identifying when these effects should be considered, and if so, what methods should be used to determine these effects. Information from this report should provide an excellent basis for developing provisions and commentaries on liquefaction for the new AASHTO seismic design specifications.

Design Procedures: The site response and liquefaction design recommendations in this report could result in some additional requirements for the design engineer. These additional design requirements are believed to be very valuable in that they will provide a common basis for engineers to use when performing their analyses and making their design decisions.

Design Costs: Changes in design costs as a result of adopting these design recommendations should be small.

Structural Performance: Implementation of these design recommendations will result in more predictable structural performance, which should lead to more efficient structural designs.

Construction Cost/Constructibility: Design recommendations developed in this task should result in lower construction costs through the use of more efficient designs. Implementation of these design recommendations should also result in lower postearthquake repair costs.

Research Recommendations: Areas requiring additional research include: (1) site effects for periods greater than two seconds; (2) two- and three-dimensional basin effects; (3) near-fault effects; and (4) effects of liquefaction on ground motions. Additional research recommendations are included within individual impact assessments for the tasks that were used as a basis for preparing Task 112-D-4.4.

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**APPENDIX A
RECOMMENDATIONS FOR FUTURE AASHTO BRIDGE SEISMIC DESIGN
SPECIFICATIONS**

A.1 Performance Criteria

There are two sets of recently-adopted performance criteria that were evaluated for their relevance to future AASHTO guidelines. The ATC-32 (ATC, 1996) performance criteria are a two-level design criteria that specify upper-level and lower-level design events and two categories of bridge importance. The ATC-32 criteria follow in Table A-1:

Table A-1: ATC-32 Performance Criteria

Ground Motion At Site	Important Bridges	Ordinary Bridges
60% probability of not being exceeded during life of structure	Service Level — Immediate Minimal Damage	Service Level — Immediate Repairable Damage
1000-2000 year return period	Service Level — Immediate Repairable Damage	Service Level — Limited Significant Damage

Alternative performance criteria are included in the most recent Canada code (CSA, in preparation). This code specifies three earthquake return periods and three importance categories, as indicated in Table A-2:

Table A-2: Ontario Performance Requirements

Bridge Descriptor: Return Period	Lifeline	Emergency Route	Ordinary
150 year: small to moderate earthquake	All traffic Immediate use	All traffic Immediate use	All traffic Immediate use
475 year: design earthquake	All traffic Immediate use	Emergency vehicles Immediate use	Emergency vehicles Repairable damage No collapse
2500 year: large earthquake	Emergency vehicles Immediate use	Emergency vehicles Repairable damage	No traffic No collapse

The most recently adopted AASHTO LRFD (load and reduction factor design) Bridge Design Specifications defines three importance categories: critical bridges, essential bridges, and ordinary bridges.

The AASHTO specifications state that classification should be based on consideration of societal and survival requirements as well as security and defense requirements. In classifying a bridge, consideration should be given to possible future changes in importance and requirements.

Essential bridges are generally those that should, as a minimum, be open to emergency vehicles and for security and defense purposes immediately after the design earthquake, for example a 475-year-return-period event. However, some bridges must remain open to all traffic after the design earthquake and be

usable by emergency vehicles and for security and defense purposes immediately after a large earthquake, for example a 2500-year-return-period event. These bridges should be regarded as critical structures.

A combination of the LRFD definitions of importance and the ATC-32 performance criteria result in the following recommended performance criteria shown in Table A-3.

Table A-3: Recommended Performance Criteria

Return Period of Ground Motion at Site	Bridge Descriptor:	Critical	Essential	Ordinary
150 year	Service Level: Damage:	Immediate None	Immediate Minimal	Immediate Repairable
2500 year	Service Level: Damage:	Immediate Minimal	Immediate Repairable	Limited Significant

Relevant References: Rojahn et al. (1997); Thomas et al. (1998)

A.1.1 Service Levels

Definitions for two service levels are recommended:

- *Service Level—Immediate.* Full access to normal traffic is available almost immediately (that is, within hours) following the earthquake. (It may be necessary to allow 24 hours for inspection of the bridge.)
- *Service Level—Limited.* Limited access for light emergency traffic with reduced lanes is possible within three (+)days of the earthquake. Full service can be restored within months.

A.1.2 Damage Levels

Three damage levels are defined.

- *Minimal Damage.* Although minor inelastic response may occur, earthquake damage is limited to narrow flexural cracking in concrete. Permanent deformations are not apparent. Criteria for abutments, wing walls, and steel members need to be developed.
- *Repairable Damage.* Inelastic response may occur, resulting in concrete cracking, reinforcement yielding, and minor spalling of cover concrete. The extent of damage should be sufficiently limited so that the structure can be essentially restored to its pre-earthquake condition without replacing reinforcement in structural members. Repair should not require closure. Permanent offsets are small.
- *Significant Damage.* Although there is no collapse, permanent offsets may occur and damage consisting of cracking, reinforcement yielding, and major spalling of concrete may require bridge closure for repair. Partial or complete replacement of components may be required. Criteria need to be developed for abutments, wing walls, and steel members.

An attempt at more specific criteria for concrete columns follows in Table A-4. The work of Dutta and Mander (1998) with regard to low-cycle fatigue needs to be incorporated in these more detailed requirements. Similar specific criteria will be required for other earthquake-sensitive elements such as foundations and abutments.

Table A-4: Quantitative Column Performance Requirements

Damage Level	Concrete Strain	Steel Strain
Minimal	0.004	0.01
Repairable	0.007	0.015
Significant ¹	See Note ²	See Note ³

Notes:

¹Assume concrete cover has spalled.

²For confined concrete, the energy model of Chang and Mander (1997a,b) for confined concrete should be used (ultimate concrete strain, $\epsilon_{cu} \cong 0.02$).

³Low-cycle fatigue capacity recommendations of Dutta and Mander (1998) for plastic strain should be followed ($\epsilon_{su} \cong \epsilon_{sy} + 0.021T^{1/6}$).

Allowable section curvatures are to be determined by moment-curvature section analysis. Allowable structural displacement values are to be determined by pushover analysis using an equivalent constant-strain plastic hinge length in the place of the traditional plastic hinge length. A method for determining this length needs to be developed, but the following relationship is recommended:

$$L_{eq} = (L + 6d_b^{1/2}) (1 - M_y/M_{max}).$$

Total curvature over this plastic hinge length may be assumed to be concentrated at the quarter-point closest to the end of the column.

Relevant References: Mander. and Cheng (1997); Cheng and Mander (1997); Mander et al. (1998); Dutta and Mander (1998)

A.1.3 Issues to be Resolved

The following issues need to be resolved before performance criteria can be finalized:

a. Definition of Critical, Essential, and Other Bridges

Unambiguous definitions for these categories are needed since they are not provided by Thomas et al. (1998). A further complication occurs with toll bridges funded by bonds. These may be categorized as critical or essential, not because of their size or role in the transportation system, but because of financing requirements in the bond market.

b. Limited Service Level

The Project Team recommends that the *limited* service level be defined as being able to open a bridge to some traffic within a specific number of days (for example, three days) after it has been inspected. This definition is based on the premise that no shoring would be required to restore service. A firm definition of limited service must be established, since it will impact the design requirements and performance levels.

c. Definition of lower-level and upper-level earthquakes

The specific design earthquakes could be different for the three importance categories of bridges, although for the purpose of this document, it is recommended that 150-year and 2500-year return period events be used. For ordinary bridges, the upper-level ground-motion input could be set at a percentage

(e.g., 75 percent) of the input for the 2500-year return period. The full value of the 2500-year design acceleration would be used for essential and critical bridges.

d. Significant damage

The definition of significant damage needs to include criteria on the level of aftershock the damaged structure needs to resist.

A.2 Design Approach

It is recommended that: the current AASHTO Seismic Performance Category (SPC) concept be continued in future codes, since it is a good method of varying design requirements in different seismic zones.

It is proposed that detailed analysis would not be required for the two lowest-level Seismic Performance Categories. SPC A does not currently require an analysis, and it is proposed that in SPC B, analysis requirements be eliminated for as many bridge types as possible. A pushover analysis could be required for some classes of bridges once the prescriptive design for SPC B is complete. The columns would have specified minimum amounts of confinement reinforcement and a minimum longitudinal reinforcement ratio (not less than one percent). Shear reinforcement would be based on capacity design principles. The foundation-to-column connections would be designed for column overstrength capacities. There is concern that some design concepts (for example, no one pier taking all seismic loads) might not be properly implemented under such code provisions.

With regard to allowable displacement values, it is recommended that a short-period multiplier be incorporated, similar to the approach in ATC-32. In addition, in a limit-state (pushover) analysis, the seismic displacement demand would be increased by a factor of 1.5.

Table A-5 below displays an attempt at setting *R*-factors that are consistent with the recommended performance requirements. This table provides the Project Team’s perspective, but we expect that more refinement will be needed. Note that we have recommended a conservative requirement for the one-level design procedure. This encourages the use of a two-level design procedure.

Table A-5: R Factors for Bridges of Varying Importance and Return Periods

Bridges:	Critical	Essential		Ordinary	
<i>Return Period of Design Event</i>	<i>Two-Level Design</i>	<i>One-Level Design</i>	<i>Two-Level Design</i>	<i>One-Level Design</i>	<i>Two-Level Design</i>
150 year	$R = 1$		$R = 1.5$		$R = 2$
2500 year	$R = 2$ plus Limit-State Analysis	$R = 2$	$R = 3$ to 4 plus Limit-State Analysis	$R = 3$	$R = 6$ plus Limit-State Analysis

Note: For Essential and Ordinary Bridges, if *liquefaction* may occur, then two analytical models will be required.

A.3 Seismic Loading

- It is recommended that the probabilistic USGS maps be used, including the deterministic values developed for parts of California, Oregon and Washington.
- The upper-level event will be based on the 2500-year maps. It may be appropriate to use a percentage (e.g., 75 percent) of these values, at least for ordinary bridges.
- The response spectral acceleration values shall be those mapped, with an attenuation of $1/T$ in the long-period range.
- The soil factors in the 1997 *NEHRP Recommended Provisions for New Buildings* (BSSC, 1997) should be used together with the new mapped response spectral acceleration values.
- Recommendations for developing relevant time-histories will need to be developed.

Relevant References: Friedland et al. (1997); Dobry et al. (in preparation); Button et al. (in preparation) Liu et al. (in preparation); Bazzurmo and Cornell (1998); Shome and Cornell (1998); Carballo and Cornell (1998).

A.4 Design Forces

Comments and recommendations are provided for both a two-level design approach and a one-level design approach.

A.4.1 Two-Level Design Approach

1. *Ductile components.* These should be sized to remain undamaged for the lower-level event and to have adequate ductility to meet the performance criteria for the upper-level event. A definition is required for undamaged; e.g., either (1) elastic response and uncracked, or (2) cracked with limited inelastic response but no spalling. Concrete and steel strains and ductility levels for minimal, repairable, and significant damage need to be developed.
2. *Nonductile components.* These should be sized to remain undamaged for the lower-level event. The design requirement for the upper-level event will depend on whether or not the component is sacrificial. For sacrificial components, the ultimate strength should be close to, but larger than, that required for the lower-level event. The actual capacity of sacrificial components must be low enough to ensure that these components will fail in the upper-level event. If they do not, they may make the consequences of the upper-level event more serious. It will be important to address the reliability of sacrificial components. Nonsacrificial essential components should be designed either to respond elastically during the upper-level event or by the use of capacity design procedures.
3. *Foundations.* A capacity design procedure should be used for all foundations to ensure there is no foundation damage in either design event. Special studies are required to develop design criteria for pile bents, drilled shafts, and caissons. As with the evaluation of the columns and abutments, more attention to foundation deformation capacity and demand is required.
4. *Seismic isolation, energy dissipation and damage avoidance design concepts.* These design concepts, which are capable of producing higher levels of performance, need to be specifically addressed in the new specifications.

A.4.2 One-Level Design Approach

One philosophic approach to the use of both one-level and two-level design procedures is to make the one-level design approach more conservative than the two-level procedure. The intent would be to

provide an incentive in the form of lower construction costs to offset the higher design costs associated with the more thorough assessment of the response of the bridge in the two-level approach.

1. *Ductile components.* These should be sized by the *R*-factor elastic design procedure or by nonlinear static analysis. The *R*-factors or permissible ductility levels in a nonlinear static analysis may depend on the desired performance of the upper-level event.
2. *Nonductile components.* Nonsacrificial components should be designed to respond elastically during the upper-level event or by the use of capacity design procedures. Sacrificial components would need to be designed using a guideline that would correspond to the design level of an unspecified lower-level event. For example, the sacrificial components could be designed to withstand one-half or one-third of the force required for the upper-level event.
3. *Foundations.* A capacity design procedure should be used for all foundation designs to ensure that there is no foundation damage in the design event. Special studies are required to develop criteria for pile bents, drilled shafts, and caissons.

Relevant References: Mander et al. (1998); Chang and Mander (1994a); Chang and Mander (1994b); Imbsen et al. (in preparation); Ritchie and Kulicki (in preparation).

A.5 Analysis

Comments and recommendations are provided for both a two-level design approach and a one-level design approach.

A.5.1 Two-Level Design Approach

Current elastic analysis procedures (equivalent static and multimodal) are appropriate for the lower-level event. It is envisioned that the lower-level design procedures would use component stiffness values consistent with little or no damage – possibly cracked-section properties in concrete columns. Soil stiffness values need to be included as appropriate, potentially using a range of ± 20 percent of recommended values.

The recommended procedure for the upper-level event should include a nonlinear static (pushover) analysis with the use of inelastic response spectra (see ATC, 1997). However, this analysis method requires additional development work before it can be used as a standard procedure in design offices. It is recommended that, as in the current requirements, the level of analytical complexity depend on seismic performance category, with simpler methods permissible in the lower seismic performance categories.

Requiring different analysis procedures for different seismic input levels (for example, multimodal elastic analysis for lower zones and pushover analysis for higher zones) in a two-level approach seems worthy of consideration.

A.5.2 One-Level Approach

There is no consensus on the best method of implementing a one-level procedure. The current proposal (see Section A.2) would use conservative *R* values for a one-level design procedure so that there would be significant advantages to using a two-level design procedure. The Project Team believes that it is desirable for nonlinear static analysis to be a part of the analysis requirements in a two-level approach. Current elastic design and analysis procedures, however, may be sufficient for ordinary bridges.

Relevant References: Kim et al. (in preparation); Fishman and Richards (1997); Lam and Martin (in preparation); Button et al. (in preparation); Shinozuka and Deodatis (in preparation).

A.6 Design Displacements

- Design displacement values should be determined from the upper-level design event, using stiffness properties appropriate for the expected level of displacement in both structural and soil components. These values could be determined directly from a nonlinear static analysis. In a one-level elastic approach, an iterative analysis procedure may be needed to ensure that the appropriate stiffness values are used.
- It is recommended that the current minimum seat-width requirements remain since it is not considered practical to calculate accurately the relative displacement for all applicable parameters (e.g., surface wave effects) in a design office.
- It is also recommended that overall drift limits be incorporated to avoid $P-\Delta$ effects in long-period structures.
- It is recommended that a short-period multiplier be incorporated similar to that used in ATC-32 (ATC, 1996). In a limit-state pushover analysis, the seismic demand would be increased by a factor of 1.5. On this subject, the report by Hudgings, Eberhard, and Stanton (1997) needs to be evaluated for its impact.
- In two-level design criteria, it would be appropriate to specify limiting displacement or rotation criteria for joints.

Relevant References: Imbsen et al. (1997); Bjornsson et al. (1997); Hudgings et al. (1997); Trochalakis et al. (1997).

A.7 Concrete and Steel Details

- It is recommended that capacity-design procedures be used to prevent brittle modes of failure in all critical members. Confinement, shear, joint shear, torsion, anchorage, and splice reinforcement requirements will be improved as research progresses over the next several years.
- The specified design details need to be consistent with the R -factors required in the design.
- There is a significant lack of research information for developing appropriate details for steel bridges.

Relevant References: Kulicki and Prucz (196); Imbsen et al. (1996); Ritchie et al. (1998); Dutta and Mander (1998); Mander et al. (1998); Imbsen et al. (in preparation); Ritchie and Kulicki (in preparation); Webbe et al. (1996).

A.8 Foundation Design

Comments and recommendations are provided for both a two-level design approach and a one-level design approach. As with steel and concrete design, this area will be significantly affected by the work performed as part of the MCEER program.

A.8.1 Two-Level Design Approach

Geotechnical analyses should be conducted for both levels of design acceleration to confirm that the soil response will not adversely affect the performance of structures supported on or within the soil. These analyses should include assessment of the potential for liquefaction, lateral spreading, and slope

instability; dynamic earth pressures on buried walls; soil-structure interaction; and uplift and rocking of the foundation. Since some of the analyses depend on the magnitude of the earthquake causing the design acceleration, care must be used to select an earthquake magnitude that is compatible with the seismotectonics in the area, both in terms of source mechanism and source distance. Where partial or total liquefaction of saturated, cohesionless soil layers is predicted, the effects of the loss in soil strength must be addressed to confirm that structures supported on or within the soil can tolerate these effects. Particular attention should be given to the stability of sloping ground and the vertical and horizontal bearing capacity of the soil. The evaluation should also consider the amount and effects of settlement likely to result from liquefaction or densification of granular soil and the increased soil downdrag forces on pile foundations resulting from liquefaction or densification of the soil.

It is critical that geotechnical analyses be completed for both levels of design, rather than just the higher level. The implications of soil response at both design levels should be considered in light of bridge performance criteria. In all cases, soil behavior that degrades the structural capacity of the foundation must be prevented at the lower-level event; soil behavior that leads to damage in the upper-level event may be permissible as long as it does not lead to catastrophic failure of the foundation. For pile-supported and spread-footing foundations, this generally means that permanent vertical or horizontal foundation movement should not occur during the lower-level event and that movement should be less than a maximum acceptable amount during the upper-level event. For the lower-level event, uncertainties in soil and soil/structure performance should be incorporated in the evaluation by incorporating a factor of safety when estimating soil strength. For the upper-level event, best-estimate soil properties and a factor of safety of 1.0 should be used, given the low probability of this event.

A.8.2 One-Level Design Approach

Procedures identified for the two-level design approach should apply to the one-level design approach. Given that the design event in the single-level approach is generally the long return-period event, it is likely that liquefaction, soil spreading, and slope instabilities would be common. These ground failure problems can often be corrected by avoiding susceptible areas or through ground improvement techniques. In many cases, thorough corrective measures would be economically unrealistic. For bridges that are not considered essential or critical, a lower level of ground motion might be appropriate for the geotechnical analyses. It is suggested that this lower level be set at 50 percent of the design acceleration. Under this lower acceleration level, large ground movement should not occur.

Relevant References: Lam (in preparation); Lam and Kapuskar (in preparation); Lam and Chaudhuri (in preparation); Gazetas et al. (in preparation); Youd and Idriss (1997); Martin (in preparation); Power (in preparation); Gadre and Dobry (in preparation).

A.9 Miscellaneous Issues

There are a number of issues for which there is not yet enough information to make specific recommendations, but that should be considered in future seismic design guidelines.

- Should there be limitations on some configurations (e.g., severe skew, unusual column framing)?
- Current analysis requirements do not incorporate any requirements for accidental eccentricities — should such requirements be included?
- It appears that the issue of spatial variation in ground motion is an unresolved issue. What key issues should be considered — use of nonlinear analysis, varying ground motion input?

PROJECT PARTICIPANTS

APPLIED TECHNOLOGY COUNCIL

Mr. Christopher Rojahn, Principal Investigator (PI)
Applied Technology Council
555 Twin Dolphin Drive, Suite 550
Redwood City, California 94065

Dr. Ronald Mayes, Co PI & Project Director
Dynamic Isolation Systems
3470 Mt. Diablo Blvd., Suite A200
Lafayette, California 94549

MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

Mr. Ian M. Friedland, Project Manager
Multidisciplinary Center for Earthquake
Engineering Research
State University of New York, Red Jacket
Quadrangle
Buffalo, New York 14261

Dr. George Lee, Highway Project Director
Multidisciplinary Center for Earthquake
Engineering Research
State University of New York, Red Jacket
Quadrangle
Buffalo, New York 14261

PROJECT TEAM

Mr. Donald Anderson
CH2M Hill
777 108th Avenue N.E.
Bellevue, Washington 98004

Mr. Stewart Gloyd
Parsons Brinckerhoff
201 E. Sandpointe, Suite 300
Santa Ana, California 92707

Dr. John Clark
Consulting Engineer
4195 April Drive
Wenatchee, Washington 98801

Mr. Richard Nutt
Consulting Structural Engineer
9048 Hazel Oak Court
Orangevale, California 95662

D'Appolonia Engineering, A Division of
Ground Technology Inc
275 Center Road
Monroeville, Pennsylvania 15146

PROJECT ENGINEERING PANEL

Dr. Ian G. Buckle
Deputy Vice Chancellor
University of Auckland
Private Bag 92019
Auckland, New Zealand

Mr. Newland Malmquist*
Larsen & Malmquist, Inc.
1574 West 1700 South
Salt Lake City, Utah 84104

Dr. Roy Imbsen
Imbsen & Associates, Inc.
9912 Business Park Drive, Suite 130
Sacramento, California 95827

Dr. Geoffrey Martin
University of Southern California
Civil Engineering Department
Los Angeles, California 90089

Prof. John Mander
SUNY at Buffalo
Department of Civil Engineering
212 Ketter Hall
Buffalo, New York 14260

Mr. Maurice Power
Geomatrix Consultants, Inc.
100 Pine Street, 10th Floor
San Francisco, California 94111

*ATC Board Representative

Dr. M. Shinozuka
University of Southern California
Department of Civil Engineering
3620 South Vermont Avenue, KAP 254
Los Angeles, California 90089

Dr. W. Phillip Yen
Federal Highway Administration, Turner-Fairbank
Highway Research Center
Structures Division HNR-10
6300 Georgetown Pike
McLean, Virginia 22101

REPORT PREPARATION SERVICES

Mr. Peter Mork
Applied Technology Council
555 Twin Dolphin Drive, Suite 550
Redwood City, California 94065

Ms. Nancy Sauer
RDD Consultants
1163 Franklin Avenue
Louisville, Colorado 80027



MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

A National Center of Excellence in Advanced Technology Applications

University at Buffalo, State University of New York
Red Jacket Quadrangle ■ Buffalo, New York 14261-0025
Phone: 716/645-3391 ■ Fax: 716/645-3399
E-mail: mceer@acsu.buffalo.edu ■ WWW Site: <http://mceer.buffalo.edu>



University at Buffalo *The State University of New York*

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