PROCEEDINGS

MID-AMERICA HIGHWAY
SEISMIC CONFERENCE

Seismic Risks and Solutions for Highways and Bridges in the Central and Eastern United States

February 28 ~ March 3, 1999
St. Louis, Missouri

Sponsored by:
Federal Highway Administration
Missouri Department of Transportation
Mid-America Earthquake Center
Multidisciplinary Center for Earthquake Engineering Research
MID-AMERICA HIGHWAY SEISMIC CONFERENCE
St. Louis, Missouri ~ 1999

Sponsoring Organizations:

Federal Highway Administration
Missouri Department of Transportation
Mid-America Earthquake Center
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Supporting Organizations:

Arkansas Highway and Transportation Department
Association of CUSEC State Geologists
Center for Earthquake Research and Information, University of Memphis
Central United States Earthquake Consortium
Federal Emergency Management Agency
Illinois Department of Transportation
Indiana Department of Transportation
Kentucky Transportation Cabinet
Mississippi Department of Transportation
Missouri State Emergency Management Agency
Missouri Seismic Safety Commission
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- James M. Wilkinson, Jr., Central United States Earthquake Consortium
- W. Phillip Yen, FHWA - Turner Fairbanks Highway Research Center
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PREFACE

The 1999 Mid-America Highway Seismic Conference was organized around sessions on nine different topics. The presenters at each of those sessions and their topics are listed in the attached table of contents.

A letter “A” preceding the title in the Table of Contents denotes that the presenter provided an abstract only of the talk and no formal paper. A letter “V” preceding the title in the Table of Contents denotes that the presenter provided a copy of the visuals they used in their presentation but no formal paper. A letter “P” preceding the title denotes that the presenter provided a formal paper. That paper is reproduced in this document. If there is no letter preceding the title the presenter did not provide any hard copy materials.

The support of the Federal Highway Administration, the Missouri Department of Transportation and the Multidisciplinary Center for Earthquake Engineering Research for this reproduction of the Proceedings of the 1999 Mid-America Highway Seismic Conference is gratefully acknowledged.

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SESSION 1
REGIONAL SEISMICITY
SEISMICITY AND HISTORICAL BACKGROUND OF THE NEW MADRID SEISMIC ZONE

Arch Johnston

ABSTRACT

In the winter of 1811-12, three powerful earthquakes struck the Mississippi River Valley. The two largest were about 50 times more powerful than the recent Kobe, Japan shock (>5000 dead, ~$200 billion damage). According to current scientific understanding, these were 'earthquakes where they shouldn't be.' They are the outstanding examples of the rare major-to-great earthquakes that happen remote from the usual tectonic plate boundary or active intraplate seismic zones. Our understanding of the faulting process and repeat times of New Madrid characteristic earthquakes has greatly improved recently but major questions remain. Advances toward answering these questions have required interdisciplinary studies, integrating seismology, historical research, statistics, archaeology, geophysics, geotechnical engineering and tectonic geomorphology. In the nineteenth century New Madrid released more seismic energy than the western U.S., including the San Andreas Fault. In the twentieth century the fault zone has been relatively quiescent with only a few, minor-damage events exceeding magnitude 5. Understanding which century is more representative of "normal" New Madrid behavior is perhaps our greatest current challenge.
Seismic Hazard Mapping for the Central U.S.

A. Frankel, C. Mueller, E. Leyendecker, S. Harmsen, D. Perkins, N. Dickman, S. Hanson, and M. Hopper

U.S. Geological Survey, MS 966, Box 25046, DFC, Denver, CO 80225
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The U.S. Geological Survey (USGS) completed new probabilistic seismic hazard maps for the contiguous United States in 1996. These hazard maps form the basis of design maps used in the 1997 edition of the NEHRP Recommended Provisions for Seismic Regulations for New Buildings prepared by the Building Seismic Safety Council and published by FEMA. The maps depict peak horizontal ground acceleration (PGA) and spectral response at 0.2, 0.3, and 1.0 sec periods, with 10%, 5%, and 2% probabilities of exceedance in 50 years, corresponding to return times of about 500, 1000, and 2500 years, respectively. The maps are the result of a set of regional workshops of geoscientists and engineers, where the methodology was revised based on feedback from the participants. The construction of the maps involved three basic components of the seismic hazard. First, we used spatially-smoothed historic seismicity as one portion of the hazard calculation. Second, we considered large background source zones based on broad geologic criteria to quantify hazard in areas with little or no historic seismicity, but with the potential for generating large events. Third, we included the hazard from specific fault sources. Recurrence estimates for large earthquakes in New Madrid and Charleston SC were taken from recent paleoliquefaction studies. While the probabilistic accelerations are lower in New Madrid than in California for a return time of 500 years, the probabilistic accelerations for firm-rock sites are similar between New Madrid and areas of California near the San Andreas fault system at the longer return time of 2500 years. The influence of large New Madrid earthquakes (moment magnitude about 8) varies depending on the location, return time, and ground-motion frequency. For St. Louis, the hazard for PGA is dominated at a 500 year return time by magnitude 4.5-6 events within about 75 km of the city. For spectral response at 1 sec period, the relative contribution to the hazard at St. Louis from large New Madrid events increases. At Memphis, the PGA hazard for a 500-year return time is influenced about equally by large New Madrid events and smaller, close-in earthquakes. However, for longer return times of 1000 and 2500 years, the PGA hazard at Memphis is dominated by large New Madrid earthquakes. The USGS hazard maps are for a firm-rock site condition. When using the maps for sites on the Mississippi Embayment, these probabilistic ground motions must be adjusted for propagation through the sedimentary deposits within the Embayment. The national seismic hazard maps, documentation, interactive mapping tool, hazard look-up by zipcode, hazard de-aggregations, and hazard look-up by zipcode are available from our website at http://geohazards.cr.usgs.gov/eq/.
New USGS Seismic Hazard Maps for the United States

- Methodology revised during 6 regional workshops
- Basis for design maps in 1997 NEHRP Provisions
- Maps, gridded values, and documentation available on Internet:
  http://geohazards.cr.usgs.gov/eq/
- PGA and 0.2, 0.3, 1.0 sec response values
- 10%, 5%, 2% probabilities of exceedance in 50 years
  (return times of about 500, 1000, 2500 years)
Alternative Models of Seismic Hazard For Central and Eastern U.S.

1. $M_{mx} = 6.5$ in craton
2. $M_{x} = 7.5$ outboard of craton
3. $M_{x} = 7.5$ Wabash Valley
4. mbig \( \min = 5.0 \) for hazard calculation

- M3+ Since 1924, smoothed spatially
- M4+ Since 1860, smoothed spatially
- M5+ Since 1700, smoothed spatially

New Madrid, Charleston, Meers Fault, Cheraw Fault

- Background Source Zones

$M \approx \text{approx.} \ 7.0$
Saint Louis Seismic Hazard

PGA 0.0942 g

PE = 10% per 50 yr

Binning: DeltaM=0.50; DeltaD=25 km; DeltaSigma=1

Modal M=4.79; Modal Distance=14.6 km.

log(PGA) > μ + 2 sigma
μ + σ < log(PGA) ≤ μ + 2σ
μ < log(PGA) ≤ μ + σ
μ - σ < log(PGA) ≤ μ
Memphis TN Seismic Hazard
Pk. Gr. Accel 0.138 g PE = 10% / 50 yr
Mean M 6.64 Mean Dist 49.7 km
Modal M 8.0 Modal Dist 31.7 km (15.7% contrib)

log(PGA) > mu + 2 sigma
mu + sigma < log(PGA) <= mu + 2 sigma
mu < log(PGA) <= mu + sigma
mu - sigma < log(PGA) <= mu

10% probability of exceedance in 50 yr. Site on rock, NEHRP B-C boundary
USGS National Seismic Hazard Mapping Project Products

- Maps available via Internet, large-format paper, GIS
  Website address: http://geohazards.cr.usgs.gov/eq/
- On-line documentation (USGS OFR 96-532), parameters for 500 Quaternary faults, seismicity grids, catalogs (Web)
- Gridded values for 150,000 sites used in maps (Web)
- De-aggregation of hazard into magnitude and distance bins for design earthquakes (Web)
- User-customized maps; hazard by zip code (Web)
- Hazard curves and uniform hazard spectra for 150,000 sites on CD-ROM; lookup by lat-lon and zipcode
- Hazard curves with uncertainties for selected cities (in prep.)
EARTHQUAKE HAZARD MAPPING

in the

NEW MADRID AND WABASH VALLEY SEISMIC ZONES

Norman C. Hester, Director
Association of CUSEC State Geologists
Indiana University
Bloomington, IN 47405

Robert Bauer, Mapping Coordinator
Association of CUSEC State Geologists
Illinois State Geological Survey
Champaign, IL 61820

Earthquake hazard mapping at a scale of 1:250,000 in the Central United States Earthquake Consortium (CUSEC) region has been undertaken by the Association of CUSEC State Geologists (CUSEC-SGs). This association includes the states of Arkansas, Illinois, Indiana, Kentucky, Mississippi, Missouri, and Tennessee. For this region, mapping directed at assessment of amplification of shaking and potential for liquefaction was given the highest priority. Ground motions generated by earthquakes can be amplified by the nonlithified geologic materials (NGM) resting on the bedrock (lithified geologic materials). The degree of amplification is directly related to variations in the geotechnical properties and thickness of the NGM. Mapping, therefore, requires the documentation of the NGM (soils) and their geotechnical characterization of the NGM (soils) in three dimensions.

The classification of map units follow the procedure introduced by Borcherdt, 1994. He developed a “soils” classification dependent on correlation among measured amplifications, shear-wave velocity characteristics, and the physical properties of the NGM as mapped at the surface in California. With these correlations, he established classification criteria for “Soil Profile Types” which are primarily dependent on the shear-wave velocity values of NGM. Shear-wave velocity values for similar “soil profile types” described in the Midwest and California were found to be essentially the same. However, for NGM's that are dissimilar to Borcherdt's descriptions, estimates or direct shear-wave velocity measurements were required for the CUSEC area.

Our maps show “soil profile types” coded A through F. Not only does each “soil profile type” amplify earthquake ground motions by various amounts, a single type will amplify
earthquake ground motions a different amount depending upon the level of shaking of the underlying bedrock. This dependency is displayed in the legend as a range of amplification per “soil profile type” shown for acceleration on bedrock from 0.1g through 0.5g.

CUSEC-SGs are preparing both paper and electronic versions of our maps. Because we continue to collect borehole, shear-wave velocity, and standard penetration test data throughout the CUSEC region, the electronic version of the mapping program will be upgraded continuously. Any one of our maps can be used as a “stand-alone” product showing relative amounts of shaking among areas, or the map can be used in conjunction with other maps which show the expected acceleration on bedrock for various earthquake source locations and magnitudes. The combination of the acceleration map with the “soil profile type” map can be used to project the expected shaking on the ground surface for various earthquake scenarios.

Our mapping procedure has direct application to both the Federal Emergency Management Agency’s (FEMA) Earthquake Loss Estimation Program (HAZUS) and the 1997 Unified Building Code use. Our maps were produced primarily for use by FEMA, State Emergency Management agencies, and other state and local governments for earthquake preparedness planning and exercises. However, the maps and associated databases also have application for the needs of the Federal and State Departments of Transportation and the Environmental Protection Agencies.
HOW OFTEN DO LARGE NEW MADRID EARTHQUAKES OCCUR?

Eugene S. Schweig and Martitia P. Tuttle

ABSTRACT

Inherent in the new U.S. Geological Survey National Seismic Hazard Maps is that every 1,000 years the New Madrid seismic experiences magnitude 8 earthquakes, similar to those that occurred in 1811 and 1812. There are several lines of thinking that makes this a reasonable estimate to use. Primary among these is evidence from paleoseismology, which is the study of the geological record of prehistoric earthquakes. In the New Madrid seismic zone, the most dramatic record comes from earthquake-induced soil liquefaction. During large earthquakes, saturated sand liquefies and is erupted through fissures onto the ground surface forming sand blows. Sand blows from the 1811-1812 and earlier earthquakes are commonly preserved on the ground surface or buried by river deposits. We can estimate the ages of these sand blows if we can determine the ages of the materials above and below them. We do this mainly through radiocarbon dating of organic materials and with Native American artifacts. We have found that, for the past few thousand years, widespread liquefaction has occurred in the seismic zone every few hundred years, and that within a hundred years of AD 900, an earthquake caused liquefaction as intense and widespread as in 1811-1812. The rate of historical and instrumentally measured seismicity also suggests that 1,000 years between magnitude 8 earthquakes is appropriate.
HOW OFTEN DO LARGE NEW MADRID EARTHQUAKES OCCUR?

Eugene Schweig\textsuperscript{1} and Martitia Tuttle\textsuperscript{2}

\textsuperscript{1} U.S. Geological Survey and CERI, Memphis, Tennessee
\textsuperscript{2} Independent Consultant, Bowie, Maryland
Evidence for the time between large earthquake come from:

- Seismic instruments: Takes us back 25 years
- Historical record: Takes us back 200 years
- Accurate surveying: Takes us back 50 years (only about 10 accurately)
- Prehistoric record of ancient earthquakes preserved in the geology: Takes us back 20,000 years!

If great earthquakes are hundreds or thousands of years apart, only geology allows us to see past an entire earthquake cycle.
Looking at the instrumental and historical record, and projecting to large earthquakes:

<table>
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<tr>
<th>Magnitude</th>
<th>Repeat Time</th>
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<tr>
<td>M ≥ 6.0</td>
<td>70-140 years</td>
</tr>
<tr>
<td>M ≥ 8.0</td>
<td>550-1,200 years</td>
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If magnitude 8 earthquakes happen this often, there should be a record of them in the geology...
Interpreting the location, timing, and magnitude of prehistoric earthquakes

In the New Madrid seismic zone, our most powerful tool has been ancient liquefaction deposits, which we can date with carbon-14 and Native American artifacts.
During strong ground shaking, pore water pressure in saturated, loose sand increases until the sand loses its shear strength and acts like a liquid, finally erupting to the ground surface through fissures, forming sand blows.
An example of a sand blow in a drainage ditch, southeast Missouri.
We can use sand blows to date old earthquakes if:

- they bury old plant remains of archeological artifacts we can date
- the sand blows are themselves buried by materials we can date

We then know the earthquakes occurred between the two time periods
What have we learned?

- Earthquakes large enough to cause widespread liquefaction happen every few hundred years in the New Madrid seismic zone.

- In addition to the 1811-1812 events, there were at least two strong ground shaking earthquakes in the past 2000 years, in A.D. 1530±135 and A.D. 900±100.

- There is evidence at several sites, for significant earthquakes prior to A.D. 900.
Magnitudes?

- Based on published relationships of liquefaction distribution vs. magnitude, the A.D. 1535 and A.D. 900 events are estimated to be of $M > 7.2$ and $> 7.4$, respectively.

- Similarities in number and thickness of sedimentary units within prehistoric and historic sand blows, suggest that A.D. 1535 and A.D. 900 events were earthquake sequences including more than one very large earthquake of $M > 7.6$. 
SESSION 2
CURRENT STATE OF PREPARATION
OVERVIEW OF THE
NATIONAL EARTHQUAKE HAZARD REDUCTION PROGRAM

Joseph A. Rachel
Federal Emergency Management Agency - Region 7
NATIONAL EARTHQUAKE LOSS REDUCTION PROGRAM

Federal Emergency Management Agency

Background

- 1977 - Congress passed the National Earthquake Hazards Reduction Act (NEHRA)
- Established the NEHRP,
- NEHRP designed to reduce the risks to life and property in the US from earthquakes through the establishment and maintenance of an effective earthquake risk reduction program

NEHRP

- Program agencies:
- Federal Emergency Management Agency (FEMA)
- United States Geological Survey (USGS)
- National Science Foundation (NSF)
- National Institute of Standards and Technology (NIST)
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<td>- Coordinating</td>
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<td>» Program lacks strategic plan</td>
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<td>» Insufficient coordination among agencies</td>
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<tr>
<td>» Not enough research application</td>
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<tr>
<td>» Too little research on how to mitigate damage</td>
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<tr>
<td>- March, 1994 - OSTP launched study to review NEHRP, focusing on:</td>
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<td>» Earthquake research and development</td>
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<tr>
<td>» Implementation of the resulting knowledge</td>
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<td>- Workshop - June 6-8, 1994</td>
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<td>- Need for expanded program (NEP)</td>
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<td>- Add to the four NEHRP program agencies all federal agencies involved in EQ hazards reduction activities</td>
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<td>- Establish goals</td>
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OTA Study (cont'd.)

- Congress should consider:
  - Oversight authority for FEMA
  - Using disaster assistance as incentive for mitigation
  - Increased role in disaster insurance
  - Increase financial incentives to promote mitigation

National Earthquake Loss Reduction Program (NEP)

- Implementation
- FEMA to provide coordination and leadership
- Full-time Program Director
- SNDR Overview

NEP

- Budget-neutral
- Does not replace NEHRP, but encompasses a wider range of activities than NEHRP
- An administrative construct of the Executive Branch with no legislative authority
- NEHRP authorities are maintained under the NEHRA and P.L.101-614
## NEP Strategic Objectives

- Reduce the:
  - Loss of life
  - Number and severity of casualties
  - Property loss
  - Social disruption
  - Adverse impact on the natural environment

## NEP Office Objectives

- Represent federal earthquake risk reduction efforts
- Increase awareness of and support for NEP
- Improve planning for earthquake risk reduction
- Improve the transfer of earthquake risk reduction knowledge
- Facilitate implementation of earthquake risk reduction measures
- Evaluate program effectiveness

## NEP Interagency Working Group Objectives

- Improve the integration of earthquake-related activities of the federal agencies
- Provide means of communicating with their stakeholders
- Improve earthquake risk reduction efforts carried out by state and local governments and the private sector
NEP Objectives for Each Agency

- Carry out programmatic responsibilities
- Support NEP
- Carry out earthquake Executive Orders

NEP Activities

- Monthly coordination meetings
- Strategic planning
- Recommend program priorities
- Consolidation of annual budget
- Fulfill biennial reporting requirements

NEP and NEHRP have Common Elements

- Fundamental objectives are nearly identical
- FEMA serves as the lead agency of both
- FEMA, USGS, NIST and NSF have key roles in both
- State and local governments and the private sector are key to achieving objectives of both
### NEHRP is Statutory
- Establishes the national policy objective to "reduce the risks to life and property from future earthquakes in the United States."
- Establishes a program for earthquake risk reduction
- Authorizes activities by FEMA, USGS, NSF and NIST
- Appropriates funds
- Establishes the Interagency Coordinating Committee
- Calls for programmatic plans and reports

### NEP is Administrative
- NEP increases the effectiveness of NEHRP
  - Incorporates a larger number of agencies
  - Recognizes importance of additional federal earthquake activities
  - Addresses integration of earthquake into programs
  - Calls for strategic planning
  - Calls for research priorities
- NEP encourages knowledge implementation

### NEP is Administrative (cont’d.)
- NEP addresses the need for agencies to work together
  - Addresses the need to eliminate redundancy
  - Calls for budget coordination
- NEP calls for policy advocacy and incentives
- NEP calls for increased mitigation
- NEP has an advisory working group
- NEP is broad and encompasses NEHRP
Will NEP replace NEHRP?

- Perhaps
- Will take legislation
- Congress will "wait and see"

How do NEP and NEHRP relate to the National Mitigation Strategy?

- The NMS is a statement of policy: Make mitigation the core of national disaster policy
- NMS is broad, it will influence all FEMA programs
- It includes all natural and technological hazard programs
- It affects all response and recovery programs
- It affects federal insurance programs and policy
- It affects training and planning efforts
- NEP is the embodiment of the NMS for earthquakes

NEP

<table>
<thead>
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<th>Other Federal Agencies</th>
<th>NEHRP</th>
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<tr>
<td>DOD/USACE OSTP</td>
<td>FEMA</td>
</tr>
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<td>DOI/USGS</td>
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<tr>
<td>EPA DOT</td>
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<tr>
<td>NASA HHS</td>
<td>DOC/NIST</td>
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<tr>
<td>DOC/NOAA HUD</td>
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<tr>
<td>NRC VA</td>
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Marketing Challenges

- Acknowledgment of risk
- Acceptance of responsibility
- Appropriate action

Marketing Approaches

- One size fits all
- Smorgasbord
- Context

Distribution of Effort

- To eat an elephant....
- Capitalize on relative strengths
- GPRA
**NEHRA Amended (Public Law 101-614)**

- Defined specific responsibilities for each Program agency
- Required Advisory Committee to be established
- Established post-earthquake investigations program in USGS
- Increased authorized budgetary levels over the 3-year period

**Updated goals of P.L. 101-614 include:**

- Increase earthquake education
- Develop improved design and construction techniques
- Implement system to predict and characterize EQs and their effects
- Develop model building codes and land use practices
- Research our ability to deal with earthquakes
- Apply research results
- Assure availability of earthquake insurance
MISSOURI DEPARTMENT OF TRANSPORTATION

EARTHQUAKE RESPONSE PLAN

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ABSTRACT

This presentation will be a brief overview of the preparedness and actions the personnel of MoDOT will take in the event of a major earthquake.

Discussion will center around activating and mobilizing field and office personnel to respond to the damaged areas. Identification of the three staging areas and why they were chosen along with the initial plan of action to inspect the roadways and structures on the two primary routes which will serve St. Louis and our Southeast (Bootheel) area. Other priority routes will be identified to provide access to these two areas.

Use of other state forces for bridge inspection, road and bridge repair, and traffic diversion will also be presented including traffic control once highways are available. Review of the other three modes of transportation and the potential effects of a major earthquake and their ability to provide assistance.

Because of the probable effect of a major earthquake on employees in the areas most likely affected, discussion of training, mentor program, and incorporation of the individual district action plans will be reviewed. The assumption that has been made is that for some undetermined length of time our employees will not be available in the work force.

Finally, MoDOT's interaction with the Southeastern Regional Emergency Management Assistance Compact (SREMAC) will be discussed.
I. STATEMENT OF PURPOSE

The primary mission of the Missouri Department of Transportation (MoDOT) in the event of a catastrophic earthquake is to provide manpower, equipment, and the necessary material to ensure the operational capabilities of a transportation system into and out of the area affected by the earthquake.

II. SITUATION AND ASSUMPTIONS

A. Situation: Earthquake Disaster

There is an high possibility of widespread damage and loss of life if a major earthquake occurs within the New Madrid seismic zone in Southeast Missouri. It is also recognized that an earthquake may occur with little or no warning.

B. Assumptions: Earthquake Disaster

1. A major earthquake in the New Madrid seismic zone would affect large areas of eastern and southern Missouri, as well as the states of Indiana, Illinois, Tennessee, Kentucky, Arkansas and Mississippi.

2. The scale and scope of a major earthquake similar to the ones that occurred during 1811 and 1812 would be catastrophic and require detailed advance planning.

3. The damage occurring from a catastrophic earthquake would affect the state's transportation system causing damage to roadways, bridges and support equipment.

4. MoDOT's planning is based on the effects of an earthquake measuring 7.6 on the Richter Scale, which is the standard used by the State Emergency Management Agency (SEMA) for a major earthquake. Because of the damage an earthquake of this magnitude would cause, the plan does not include use of employees from districts 6 and 10 in the early stages (these workers would have more immediate family concerns).

   However, this plan is designed to be flexible enough to use even in less catastrophic scenarios. In a smaller earthquake more local resources may be available. Response may be best suited for the district level, or perhaps only portions of this plan will be appropriate. The level of response will be determined by the Assistant Chief Engineer-Operations.
III. ORGANIZATION

A. General

1. The Governor has been given the ultimate responsibility for emergency management activities in the state. When the Governor declares that a state of emergency or earthquake disaster exists within the state, he may delegate authority to the Adjutant General who may provide for the subdelegation of the authority for overall coordination and control of disaster relief operations to the Director, State Emergency Management Agency, who will exercise control of disaster relief operations through the personnel and facilities of the State Emergency Operations Center (SEOC).

2. The Missouri Department of Transportation (MoDOT's) Chief Engineer will be the individual with responsibility and authority to activate the MoDOT Earthquake Emergency Response Plan when called upon by the Governor or his authorized representative.

3. Upon order to activate the MoDOT Earthquake Emergency Response Plan, the Chief Engineer will establish and provide staffing to the State Emergency Operations Center (State EOC), the Missouri Department of Transportation (MoDOT) Support Center EOC, the State Emergency Operations District (SEOD) EOC, the Missouri Department of Transportation (MoDOT) District EOC, and the Missouri Department of Transportation (MoDOT) Assembly Area EOCs.

The following MoDOT personnel will be assigned the responsibility of staffing and operational control of the Emergency Operations Centers (EOCs):

a. State EOC (SEMA headquarters in Jefferson City) - Assistant Chief Engineer-Operations
b. MoDOT Support Center EOC - Deputy Chief Engineer
c. SEOD EOC (SEMA district field office) - MoDOT District Engineers
d. MoDOT District EOC (district office) - District Operations Engineer
e. MoDOT Assembly Areas EOC (see Attachment 5a) - Maintenance Liaison Engineers and District Area Engineers

(See Attachment 1 for organizational block diagram.)
(See Attachment 4 for additional staff assignments from designated MoDOT Earthquake Response Personnel and each Division's Responsibilities.)

4. In the event the District Engineer and the District Management Team are unable to staff the various EOCs within the disaster area, the Deputy Chief Engineer shall assign personnel to staff the EOCs.

B. Earthquake Emergency Operations Interaction

Upon declaration of an earthquake emergency by the Governor, the MoDOT section of the State EOC will coordinate with the MoDOT Support Center EOC to begin assembling personnel, equipment and material for movement to designated assembly areas. The control and movement of the personnel, equipment and materials will be directed by the MoDOT Assembly Area EOC until closure of the personnel, equipment and supplies in the assembly area. Upon closure in the Assembly Area, the manpower, equipment and material command and control will transfer to the MoDOT District EOC for future operations.

C. Coordination with Other Agencies

1. MoDOT has been assigned primary responsibility for the transportation function. The following agencies have been assigned support roles under this function: Missouri Army National Guard, Office of Administration, Department of Conservation, Missouri State Highway (Rev. 03-10-98)
Patrol, Department of Public Safety (Adjutant General), Missouri State Water Patrol, Department of Corrections, Department of Economic Development (Division of Transportation), Department of Natural Resources (Division of Parks, Recreation and Historic Preservation), Missouri Volunteer Organizations Active in Disasters, and Federal Agencies. Reference Missouri (SEOP), pages 28-31.

2. Coordination with other agencies shall occur at all levels of EOCs. When conflicts occur, the conflict will be forwarded to the next higher level EOC.

If the conflict cannot be resolved below the State EOC level, the State EOC (MoDOT Deputy Chief Engineer) shall resolve the conflict for the department.

3. In the event that MoDOT manpower, equipment and material is insufficient for the emergency, the MoDOT District Engineers will have authority to negotiate and enter into contracts with contractors for manpower, equipment and materials as required. (See Attachment 12 for a List of Prime Contractors) Also refer to RSMo 44.100 for emergency powers of Governor. (See Attachment 13 for "Chapter 44, Revised Missouri Statutes".)

4. In the event that the state and local resources are depleted, federal agency assistance may be requested. Congress authorized in Title 23, United States Code, Section 125, a special program from the Highway Trust Fund for the repair or reconstruction of federal-aid highways and federal roads which have suffered serious damage as a result of (1) natural disasters, or (2) catastrophic failures from an external cause. This program that is commonly referred to as the Emergency Relief, or ER, program and is administered by the Federal Highway Administration (FHWA), supplements the commitment of resources by states, their political subdivisions or other federal agencies to help pay for unusually heavy expenses resulting from extraordinary conditions. Thus, to obtain ER program funds for assistance in repairing earthquake damage on federal-aid highways, MoDOT must request assistance from FHWA.

Damage to highway facilities that are neither federal-aid highways nor federal roads may be eligible for other federal funds authorized by the Stafford Act, P.L. 93-288 and administered by the Federal Emergency Management Agency (FEMA). Federal agency assistance that may be available through FEMA may be requested by SEMA from the State Emergency Operations Center (SEOC) to FEMA Region VII, which will contact individual federal agencies from which assistance is requested.

The Assistant Chief Engineer-Operations will coordinate these activities with the assistance of the Maintenance Division.

For natural disasters, coordination among federal agencies is handled through an interagency agreement among the FEMA and the 11 federal agencies involved with hazard mitigation. Hazard mitigation teams are activated by these federal agencies immediately following a disaster.

For more information, guidance and instructions on the FHWA Emergency Relief program and procedures for requesting, obtaining and administering ER funds, see Attachment 8.

5. In the event that MoDOT manpower, equipment or material is required outside the boundaries of the State of Missouri, authority may be granted by the Deputy Chief Engineer to pass operational control to the requesting agency. Also refer to the multi-state, Interstate Earthquake Compact. (RSMo 256.155)

D. MoDOT Coordination

1. When the Governor declares that the earthquake emergency is over, the State EOC will notify the MoDOT Support Center EOC and SEOD EOC; in turn, the MoDOT Assembly Area EOC will be notified of the cessation of the emergency. Upon release of the manpower, (Rev. 03-10-98)
equipment and materials from the assembly areas, command and control will revert to each home MoDOT District EOC.

IV. LOGISTICS

A. Damage Assessment

1. Upon declaration of an earthquake emergency by the governor and subsequent establishment and staffing of the State EOC, SEOD EOC and the MoDOT EOC, the Deputy Chief Engineer will immediately mobilize all district alert forces and available field bridge inspection personnel for the purpose of assessing the condition of the highways and bridges on the State highway system. The first priority of the bridge inspection personnel will be to inspect all major river crossings within the disaster area, followed by other major bridges along the earthquake emergency highway routes. (For a List of Missouri and Mississippi River Bridges see Attachment 14). (For maps showing Pre-selected Priority Routes see Attachment 9). (For lists of bridges on priority routes to the St. Louis Area, information about those bridges, county maps showing the locations of those bridges, Missouri Maps showing 1st, 2nd, 3rd and 4th priority routes to the St. Louis Area and to the Southeast Missouri Area and Area Engineer Locations, Names and Telephone Numbers see Attachment 10).

Each district and division shall maintain a list of all available bridge inspection personnel and provide this list to the Assistant Division Engineer-Bridge Maintenance, at the end of the off-system bridge inspections each year. Bridge inspection personnel are listed by District and Division in Attachment 2. A list of consultants that can provide detailed bridge inspection will be maintained by the Assistant Division Engineer - Bridge Maintenance. This listing shall include a section on inspectors that are certified for underwater inspections.

2. Immediate aerial surveillance will be flown over each preselected route into the affected area by MoDOT and/or other available aircraft. Aerial photographs of each bridge along the route will be taken and forwarded to the MoDOT Support Center EOC for review by the Deputy Chief Engineer and staff. Then, to facilitate the collection and analysis of all damage information, all reports must be forwarded to the SEOC immediately for processing and dissemination. See Attachment 9 for preselected priority routes. Also refer to VI (Operations) A and B.

Note: In the event that Federal Agency assistance is or may be requested for the repair or reconstruction of federal-aid highways from FHWA or for the repair or reconstruction of other highways through FEMA, it is required that the field surveys be made in cooperation with FHWA and FEMA. Also, it is required that the field report and subsequent reports document the damage. Pictures showing the kinds and extent of damage and sketch maps detailing the damage areas should be included in the field report to FHWA. For more information on the FHWA Emergency Relief (ER) Program, see Attachment 8.

3. The Deputy Chief Engineer shall consult MoDOT bridge maps, aerial photographs, and other available information, to determine the routes upon which MoDOT forces will concentrate efforts. MoDOT bridge maps are included in Attachment 3.

4. Transportation for bridge inspection personnel will be provided, as far as possible, by MoDOT vehicles. When MoDOT vehicles are no longer adequate, the Deputy Chief Engineer shall coordinate with support agencies, as per the State Emergency Operation Plan (SEOP) for procurement of support vehicles. Note: The Missouri Army National Guard (MOANG) agreed in a meeting with MoDOT, SEMA and other agencies on September 21, 1997 that they will provide the helicopters to transport MoDOT Bridge Inspectors to inspect routes and bridges; and will revise MOANG's "Emergency Response Operations Plan" to state this. Also, the Civil Air Patrol (CAP), an auxiliary of the U.S. Air Force, has agreed to provide aerial surveillance and communications relay over Southeast Missouri. The CAP has several fixed-wing aircraft, but no helicopters.

(Rev. 03-10-98)
Air traffic separation and coordination is essential. The assigned altitudes will be from 0 to 1000 feet for helicopters and 1500 feet and above for fixed wing aircraft. SEMA will request that the airspace be closed by the FAA immediately following an earthquake. Flight authorization in the damaged area will come from the SEOC. Ingress/Egress routes are set along major highways. Inbound flights will stay on the South/West side of the route; outbound flights will stay on the North/East side of the routes. Communications between aircraft and between aircraft and the ground will be established on predesignated frequencies. This basically covers aircraft operations during the first 12 hours following a catastrophic earthquake. These procedures, however, will continue to provide aircraft safety and eliminate communication problems as follow-on and emergent aircraft missions are assigned. If necessary, the Deputy Chief Engineer may enlist the services of the Governor’s office under RSMo 44.100 for the seizure of appropriate methods of transporting bridge inspection personnel. These methods may include, but will not be limited to; aircraft, boats, four-wheel drive and all-terrain vehicles.

B. Assembly Areas

1. Upon receipt of field reports from the bridge inspection teams, the Deputy Chief Engineer will contact the unaffected MoDOT District EOCs to obtain needed equipment, personnel, and material and dispatch them to appropriate assembly areas. A list of MoDOT personnel is included in Attachment 4. In addition, he will select the Assembly Area EOC Directors and will provide them with information regarding the personnel and equipment under their direction and the routes upon which they will concentrate. A list of possible assembly areas is included in Attachment 5. (See Attachment 5a for maps showing locations of pre-selected Assembly Areas.)

2. The Assembly Area EOC shall report to the SEOD EOC and the SEMA EOC concerning site conditions and progress. Assembly Area EOC Directors shall be responsible for keeping accurate records of personnel and equipment assignments for those items under their direction.

Note: See Attachment 8 for a "Detailed Damage Inspection Report" that is to be completed by MoDOT as part of the requirements for obtaining federal funds from FHWA for the repair and reconstruction of federal-aid highways.

3. Should field conditions warrant a move to a more desirable location, the Assembly Area EOC Director may change the location of the assembly area by coordination with the SEOD EOC.

C. Equipment

1. A comprehensive list of specialized equipment, including, but not limited to, dozers, motor graders, trucks, backhoes, loaders, draglines, excavators and compressors, shall be maintained by the Director of the MoDOT Division of General Services and is included in Attachment 6. Each MoDOT District Engineer will maintain a current listing of contractors that may be called on to furnish heavy equipment through lease or rental, with a copy of the listing made available to the MoDOT Support Center EOC. (See Attachment 12 for a list of prime contractors.)

2. Support for equipment in the affected area, including fuel, lubricants, tools, and spare parts, will be shipped to the assembly areas as requested by the Assembly Area EOC from the unaffected MoDOT Districts. Forward refueling bases need to be established as quickly as possible to maximize flight time in the impacted zones. The Missouri Army National Guard will consolidate and coordinate the aircraft fuel requirements of the various aircraft used. Where this is not a viable alternative, the MoDOT District Engineer shall have authority to negotiate and enter into a contract with local suppliers for the necessary provisions. In addition, the State EOC may request assistance from the Governor’s office under RSMo 44.100 for the seizure of necessary fuel, lubricants, tools, and spare parts.

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3. Repair and servicing of equipment shall be the responsibility of MoDOT district field and shop mechanics assigned to the Assembly Area EOC.

D. System Repair Supplies

1. A comprehensive list of all specialized bridge repair supplies shall be maintained by the Assistant Division Engineer-Bridge Maintenance and is included in Attachment 7. A list of temporary steel and box girder bridges shall be maintained by the Division Engineer-Bridge and is also included as part of Attachment 7.

2. In addition to existing MoDOT surface repair supplies, a list of suppliers of surface repair material, including, but not limited to, aggregate, asphalt, concrete, corrugated metal pipe, etc., shall be maintained by each MoDOT District Engineer. This list is also available through the MoDOT Materials Division in the MoDOT Support Center. In addition, each MoDOT District Engineer will maintain a current listing of metal and concrete pipe that is stored at various MoDOT maintenance facilities.

E. Impediment Removal on Priority Routes

MoDOT will be responsible for the removal of traffic obstacles from all state highways which are designated priority routes. Impediment removal is the responsibility of the nearest Assembly Area EOC. City and county officials may also request MoDOT assistance through SEMA. SEMA will request assistance from the MoDOT representative in the SEOC.

F. Personnel Support Facilities

1. Where personnel support facilities are not available from other agencies but where such facilities exist in the form of hotels and restaurants on the fringe of the affected area, the Assembly Area EOC Directors will be authorized to purchase blocks of rooms and meals for the personnel under their direction with a field purchase order. See V. (Administration) E.

2. If necessary, the Deputy Chief Engineer may enlist the assistance of the Governor’s office under RSMo 44.100 for the seizure of private facilities such as motor homes, campers, privately owned buildings, etc., as required for the billeting of personnel.

G. Medical Support

In the case of minor injuries incurred during the emergency, initial first aid will be administered by MoDOT personnel trained in CPR or first aid. In the case of major illness or injury, contact the nearest MoDOT assembly area to request emergency medical assistance.

V. ADMINISTRATION

A. Financing shall be from MoDOT funds, supplemented by funds as provided by RSMo 44.028 and 44.032.

Note: See Attachment 8 for information, guidance and instructions on the FHWA Emergency Relief (ER) program and procedures for requesting, obtaining and administering ER funds for the repair or reconstruction of federal-aid highways. Damage to other highways that are neither federal-aid highways nor federal roads may be eligible for other federal funds administered by FEMA.

B. All charges for labor, equipment rental and materials will be made to Function 604 and the following appropriate AFE:


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2. Work within the state, but not on the state highway system - AFE Prefix 3 - Roadway Groups M or N.

3. Work outside the state under mutual aid agreements - AFE Prefix 3 - Roadway Group T.

C. Records of accomplishment, employee work hours, equipment usage petroleum usage and material usage shall be kept by field crews and MoDOT Assembly Area EOC Directors in accordance with existing department policies. See Attachment 11 for guidelines for recording information. Attachment 11 should be forwarded to Business And Benefits Support Division weekly marked "Emergency Documents."

D. Temporary maintenance employees may be hired as necessary at wage rates and benefits under existing department policies by District Engineers, Assembly Area Coordinators or Area Engineers.

E. Local purchases may be made by field purchase order (form E-66), limited to $1,000 per project as per RSMo 44.032.10, and shall be subject to price controls as provided by RSMo 44.100.1(4)(d). Purchases up to $25,000 may be authorized by the Assembly Area EOC Director or their supervisor using a district purchase order (form E-100). The $25,000 limit may be waived by the Assistant Chief Engineer-Operations or his backup at SEMA, or by the Deputy Chief Engineer.

F. The requirement of determining local prevailing wage rates for contracted work shall be waived.

G. Personnel Administration

1. Personnel shall report to assigned work areas as directed. Any previously approved vacation or compensatory time off may be canceled.

2. Failure to report to assigned work areas may be cause for disciplinary action. Consideration may be given to cases of extreme personal emergency, such as critical illness in the immediate family, or severe loss of property due to the earthquake.

3. Personnel are subject to normal work policies, including, but not limited to, working hours, overtime, expenses and disciplinary action.

4. Where possible, personnel should be relieved from emergency duty periodically so as to minimize the length of the period of relocation.

5. Employees may designate that their paycheck be forwarded to a location of their choice. If no designation is made, the paycheck shall be handled consistent with current MoDOT policy.

H. Employee Support

1. Employees that have been temporarily reassigned to work in the disaster area may be away from their normal work group and families for an extended period of time. (See Attachment 15, "Guide for Staff Assigned to Disaster Operations") (See Attachment 16 for "EOC Provisions")

2. Each District Engineer and Division head shall appoint a contact person familiar with employee records to serve as an employee mentor. These mentors will be available to provide assistance for families and employees affected by the reassignment, such as providing information to the families about the MoDOT employees and other information that will assist the families with their needs.

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3. Mentors should coordinate their efforts with each division and district and keep the employees informed of where their families can call or go to for the purposes of making contact.

VI. OPERATIONS

A. Primary Routing

Based on the assumption that an earthquake of severe magnitude occurs in the New Madrid fault area, aerial reconnaissance will begin immediately on preselected routes into the damaged area. State highway maps indicating these preselected routes are included in Attachment 9.

B. Aerial Reconnaissance, Damage Assessment and Movement of Forces to Assembly Area EOCs.

General:

Immediately upon declaration of an earthquake emergency, the MoDOT aircraft with photographic equipment and personnel will begin aerial reconnaissance of the preselected routes. This aerial reconnaissance will consist of low-level photographs of each bridge on the preselected route and any other section along the route that appears to have sustained damage. These aerial photographs, along with any other information obtained that is considered pertinent to the investigation, will be immediately returned to the MoDOT Support Center EOC for review and assessment. The MoDOT Support Center EOC will select primary and alternate routes for further detailed investigation by on-site field bridge inspectors. These field bridge inspectors will be dispatched by either radio-equipped MoDOT vehicles or by helicopter support furnished by other state or federal agencies. As bridges along the preselected routes are investigated, the details of damage and availability for use of the bridge shall be immediately reported to the MoDOT Support Center EOC. This reporting will be by county, route, and bridge number. This information will be used by the MoDOT Support Center EOC to establish the best routing available for insertion of field repair crews into the damage area.

Concurrently with the aerial and ground surveillance and detailed field bridge inspection of the possible routing to the damaged area, district field forces will be placed on alert for possible movement to designated MoDOT Assembly Area EOCs. This alert will consist of marshaling the necessary manpower, equipment and supplies in preparation for movement to the damaged area. The MoDOT Support Center EOC will notify each MoDOT District EOC and SEOD EOC of the movement order, routing and MoDOT Assembly Area EOC as the final destination.

Upon receipt of a movement order, district field forces will proceed to their assigned MoDOT Assembly Area EOC and, upon arrival, will report to the director of the MoDOT Assembly Area EOC. This report will include, but not be limited to, personnel by name and job title, equipment by type and MoDOT number, and materials available at the time of arrival.

Phase One (Initial Inspection Phase and Critical Information Gathering of Priority Routes)

In the event of a major earthquake, MoDOT employees with the following job titles must report As Soon As Possible to their normal work station:

- current and former Support Center bridge inspectors
- photolab photographers
- airplane pilots (report to the Jefferson City airport)
- Liaison Engineers (of the Maintenance, Traffic, Construction, Design and Bridge Divisions)
- assistant division engineer-bridge maintenance (to coordinate bridge inspection)
- bridge inspection technicians

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• car dispatcher (of the General Services Division)
• bridge maintenance clerk and secretary
• Division Directors for Maintenance, General Services, Traffic, Construction and Bridge Divisions

Note: See Attachment 4 for MoDOT Earthquake Response Personnel and their Responsibilities.

In addition, each district must immediately staff and activate its emergency operations center and establish communication with the Support Center in Jefferson City.

Districts 1, 4 and 7 shall detour traffic that is eastbound into Missouri from states west and north of Missouri to the north through northern Missouri and/or Iowa to keep them out of the areas damaged by the earthquake, which would be primarily Districts 6 and 10. Districts 1, 4 and 7 shall detour this traffic northward over Routes 71, I-29 and I-35 into Iowa, where they could then go east on Route 1-80, or at least northward to Missouri's Route 36, east on Route 36 to Macon and then north on Route 63 into Iowa if the Mississippi River bridge at Hannibal is closed.

The Assistant Chief Engineer - Operations shall report what planes are available. The Assistant Division Engineer - Bridge Maintenance shall assign inspectors and instruct them to go to the Jefferson City airport for aerial inspections of priority routes. The Photolab coordinator shall do the same for photographers. Photographers may use still cameras and/or video cameras. (See Attachment 9 for pre-selected priority routes.)

The Bridge maintenance clerk and/or secretary shall verify the satisfactory operation of all radio towers immediately and if one or more are not responding, shall investigate possible causes and solutions to make repairs.

The Car dispatcher shall immediately provide three radio-equipped vehicles. One vehicle will be for two bridge inspectors and two liaison engineers. The inspectors will immediately begin inspection of the Route 50-44-100 priority route from Jefferson City to St. Louis, and the liaison engineers shall be dropped off at St. Clair to staff the Assembly Area EOC. They shall go as far as possible on Route 100, then backtrack to determine the best route from Route 100 to the St. Louis-Lambert Airport. Heading north from Route 100 onto Route 340, then north on Lindbergh to the airport appears to have the fewest obstacles. Once the best route has been determined and communicated to the Assembly Area EOC, it becomes the priority route to the airport from Route 100.

A second vehicle shall take two liaison engineers to staff the Wentzville Assembly Area EOC and two inspectors to inspect the Route 54-70 priority route to the St. Louis-Lambert Airport from Jefferson City. If I-70 is blocked, detour to the north in St. Charles and take the 370-270-67 priority route from Jefferson City to Wentzville and then to the airport.

A third vehicle shall take two inspectors south along Routes 63 and 60 to inspect the priority route to Willow Springs and Van Buren Assembly Area EOC from Jefferson City.

The National Guard has agreed to provide two helicopters with pilots waiting at the Jefferson City airport that are designated for MoDOT use. One shall carry two inspectors and one liaison engineer along Routes 63 and 60 to inspect that route to the Van Buren Assembly Area EOC. There, the liaison engineer will help staff the EOC and the inspectors will then drive east as far as possible to inspect Route 60. The Support Center EOC radio operator will confirm with District 9 at Willow Springs that radio-equipped vehicles will be waiting at the Van Buren Assembly Area for bridge inspectors flying there.

The second helicopter shall carry four bridge inspectors along the priority Routes of 50, I-44, and 100 to the St. Louis area. Two inspectors shall be dropped off at Park Hills, where a vehicle shall be waiting to take them north to inspect Route 67. The vehicle shall be made available by the Regional Maintenance Building at Farmington at an agreed upon meeting place in Park Hills. The (Rev. 03-10-98)
other two bridge inspectors will stay with the helicopter for stop-and-go inspections in the St. Louis area as directed by the St. Clair and/or Wentzville EOC's.

District 9 staff shall immediately travel to the Van Buren Maintenance Building to staff an EOC. Combined with the vehicle traveling Routes 63 and 60 from Jefferson City, this will confirm the availability of the Route 63-60 priority route to near the edge of the affected area of Southeast Missouri.

A MoDOT plane will carry one bridge inspector and one photographer to determine and document the status of Route 50-44-100 from Jefferson City to Downtown St. Louis.

The Rolla Area Engineer shall immediately drive east as far as possible on I-44 to inspect that route and its bridges.

All of the above workers must stay in frequent radio contact with the Assembly Area EOCs that they are assigned to as work progresses.

Phase Two  (Opening Major Emergency Routes)

By this time, SEMA will be in continuous contact with the Support Center EOC which will have established radio or telephone contact with each district's EOC.

Each district EOC will have workers and resources standing by and prepared to travel as requests for assistance are received from the SC EOC.

District 6 Field EOC/Assembly Areas will be set up at St. Clair and Wentzville. District 10's Field EOC/Assembly Area will also be activated at Van Buren by District 9 personnel. These locations may be changed if more suitable locations are available.

Requests for MoDOT assistance will be received at SEMA's EOC. The requests will be relayed to the Support Center (SC) EOC, which will contact the appropriate district. The district will deploy the needed resources and make work assignments. If more resources are needed, the Assembly Area EOC will contact the SC EOC, who will determine other sources.

When the work is completed, the Assembly Area EOC will notify the SC EOC. All completed projects and deployed resources must be documented at the SEMA EOC in order to track available resources for their possible future use on other projects.

Since district offices in Districts 6 and 10 may not be available, requests for assistance in these areas will go to the Assembly Area EOCs. These EOCs will utilize the assistance of Area Engineers from Districts 6 and 10, who will be available by this time, as well as work crews and equipment that have arrived from other districts.

As District 6 and 10 employees become available for duty they will report to the nearest maintenance building or project office, then contact the nearest Assembly Area EOC to report their availability for work.

C.  Organization of MoDOT Assembly Area EOC

1.  The MoDOT Assembly Area EOC directors will assume command and control of all personnel, equipment and material upon arrival at the MoDOT Assembly Area EOC and will be responsible to the SEMA district field office for coordination and control of all field crews assigned to that MoDOT Assembly Area EOC. The size and structure of each field crew will be determined by their assigned duties. Specific personnel, by name, will be placed in charge of each

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individual field crew. Each individual field crew will be assigned a specific job duty to include location, priority of work and probable emergency repairs required. Coordination between Assembly Area EOCs and field crew supervisors will normally be by MoDOT radio or other means of communication that may be available. Each field supervisor will be expected to report damage assessment and repairs accomplished at least twice daily, as this information will be necessary to maintain current situation maps in the SEMA district field office (and the State EOC, if possible).

2. Personnel

a. Volunteer - Volunteer personnel may be utilized at the discretion of the MoDOT Assembly Area EOC Director and may either be incorporated individually into MoDOT crews, or assigned specific tasks to be performed as a group. Volunteers will be issued, and required to use, standard MoDOT safety equipment (i.e. vests, hard hats, etc.) where applicable.

b. Temporary - Temporary employees may be obtained by contract per III-(Organization) C.3. of this document, or may be hired per V-(Administration) D. of this document. All temporary employees will be considered "wage only" employees and will be subject to all existing MoDOT rules, regulations and procedures for such employees. The duration of the temporary positions will be at the discretion of the MoDOT Assembly Area EOC Director.

D. Repairs

The MoDOT Assembly Area EOC Directors, in coordination with the Support Center EOC, shall determine the extent of the repairs to be completed under emergency conditions and shall use damage assessment reports to determine the priority and scheduling of repairs.

E. Traffic Control and Security

Traffic control and security within the damage area is the primary responsibility of all law enforcement agencies, but MoDOT personnel may be assigned short-term traffic control responsibilities. Assembly Area EOC Directors shall determine needs for detours, bypasses, etc. to ensure usability of priority routes. Traffic control in MoDOT work areas is the responsibility of the MoDOT field crews. Additional traffic control requirements and security will be requested through the SEMA district field office as needed.

F. Oversize/Overweight Vehicle Route Clearance Program

MoDOT's Motor Carrier Services' Unit staff will coordinate an oversize/overweight vehicle route-clearance program for the movement of large equipment. This action will be focused upon delivering needed machinery or other commodities that exceed normal size and weight to emergency sites.

The Motor Carrier Services Administrator, or his backup, will be stationed at the SEMA EOC to gather overweight/overdimension travel requests. These will be radioed to the Motor Carrier Services office for route clearance. When the best route is determined it will be radioed back to SEMA; this communication will replace the standard paper documents.

VII. COMMUNICATIONS

Since an earthquake of severe magnitude will probably disrupt all means of communication, primary communication will be with the MoDOT radio system. The MoDOT radio system is a high band mobile relay radio system composed of a number of district control stations, field base stations, mobile relays/repeaters and mobile units. Eight frequencies are used in the system. Separate transmit and receive frequencies are assigned to each of three channels- Channel A, Channel B and Channel C. The seventh frequency, along with the mobile radio transmit frequency assigned to Channel A, are used to form the St. Louis metro channel. Likewise, the eighth
frequency, along with the mobile radio transmit frequency assigned to Channel B, are used to form the Kansas City metro channel.

A. The districts will install temporary power poles as needed to maintain temporary communications, until towers are put back up. A radio antenna can be put on top of each one of the temporary power poles for communication.

B. Maximum use of the multi-channel, MoDOT radio system will be made by field repair crews. Since field repair crews may be using equipment from different districts, the multi-channel radio system will allow intercommunication. Each one of MoDOT's ten districts operates within the radio system on one of three channels. Channel A is assigned to Districts 1, 5, 7 and 10; Channel B is assigned to Districts 2, 6 and 8; and Channel C is assigned to Districts 3, 4 and 9. The radio system utilizes mobile repeaters/relays to repeat (re-transmit) all signals transmitted by mobile units, field base stations or district control stations. This process strengthens the signal, thus extending the talking range of the mobile, field base stations and district control stations.

The licenses for these radios, with maximum of 100-watt output, is issued to MoDOT by the Federal Communications Commission (FCC). MoDOT's Support Center keeps a permanent file of these licenses with copies issued to the districts.

There are several tower locations with repeaters/relays and/or district control stations at various places throughout Missouri. To talk to the repeaters or district control stations, the person has to tune his radio to the channel for the repeater at the tower in the area where he wants to talk. In order to talk to another district, such as District 6, from Jefferson City, we would use the District 5 control station to talk to an intermediate repeater/relay, which can talk to the District 6 control station. The district control stations are either located at, or connected to, the district office, sometimes using microwave links and sometimes using direct phone lines. Microwave links are point to point communications; therefore alignment of the antennas at the tower locations is critical and very vulnerable to earthquake damage. Each of the radio towers has a portable power generator that can be used to provide power to the district control stations and/or repeaters/relays at the tower when the regular power supply has failed.

Communication by cell phones is very difficult during an emergency because so many people try to use cell phones at the same time that the cell phone system gets overloaded. MoDOT has no proprietary rights or privileges, in the event of an emergency, to public cellular service. An alternate, almost sure way of communicating, is to use mobile radios in cars spaced at intervals of 10 to 50 miles apart and these mobile car radios will relay messages back and forth between Jefferson City and the disaster area (St. Louis Area or Southeast Missouri Area) from car to car. The spacing of car radios depends on the atmospheric conditions that day as well as the terrain, so the spacing of cars has to be determined by trial.

C. Secondary communication networks may be utilized as they become available for use. Such networks may include the Missouri State Highway Patrol, National Guard, ham amateur radios, Civil Air Patrol, CB's and privately owned networks.

Also, the MoDOT radios in the Support Center and the district offices shall be staffed. Available telephone capability in MoDOT facilities will be used to supplement radio communications.

D. The MoDOT radio shall be staffed in the SEOC when needed. Reference Annex B - COMMUNICATIONS of the SEOP for additional information concerning communication facilities.
VIII. PUBLIC INFORMATION

Press releases and public information reports will be made by the SEMA Joint Public Information Center. All information for news releases will be developed jointly by representatives of the affected emergency organizations. The MoDOT information representative will be the Support Center Public Affairs Director or designated backup.

IX. EXTENDED OPERATIONS (24-HOUR OPERATIONS)

When it is foreseen that the need for extended operations imminent, the individual exercising command and control over each area of operation, such as each Assembly Area and EOC, will declare the need to establish and implement a shift work schedule. Employees, as groups or as individuals, may be placed on eight or twelve hour shift schedules depending on the nature of the emergency staffing requirements or functional assignments.

X. MANUAL UPDATE

It shall be the responsibility of the Assistant Chief Engineer - Operations to keep this manual updated in the future, at least as frequently as on an annual basis, preferably on a continuous basis with some specific person assigned to perform this task as part of his or her regular job duties.

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NEW DEVELOPMENTS IN SEISMIC RISK ANALYSIS 
OF HIGHWAY SYSTEMS

by

Stuart D. Werner1, Craig E. Taylor2, James E. Moore II3, 
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ABSTRACT

This paper summarizes current research to develop a new seismic risk analysis (SRA) procedure for highway and roadway systems. The procedure synthesizes geoseismic, engineering, network, and economic models to assess earthquake effects on system-wide traffic flows and travel times. The SRA results provide an improved basis for prioritizing highway components for seismic retrofit, and for defining seismic performance requirements for these components.

1.0 INTRODUCTION

Past experience has shown that earthquake damage to highway components (e.g., bridges, roadways, tunnels, retaining walls, etc.) can severely disrupt traffic flows and this, in turn, can impact the economy of the region as well as post-earthquake emergency response and recovery. Furthermore, the extent of these impacts will depend not only on the seismic response characteristics of the individual components, but also on the characteristics of the highway system that contains these components. System characteristics that will affect post-earthquake traffic flows include: (a) the highway system network configuration; (b) locations, redundancies, and traffic capacities and volumes of the system's links between key origins and destinations; and (c) component locations within the links (e.g., Moore et al, 1997).

From this, it is evident that earthquake damage to certain components (e.g., those along important and non-redundant links within the system) will have a greater impact on the system performance (e.g., traffic flows) than will other components. Unfortunately, such system issues are typically ignored when specifying seismic performance requirements and design criteria for new and existing components; i.e., each component is usually treated as an individual entity only, without regard to how its damage may impact highway system performance. Furthermore, current criteria for prioritizing bridges for seismic retrofit represent the importance of the bridge as a traffic-carrying entity only by using average daily traffic count, detour length, and route type as parameters in the prioritization process. These criteria do not account for the systemic effects associated with the loss of a given bridge, or for combinatorial effects associated with the loss of other bridges in the highway system. However, consideration of these systemic and combinatorial effects can provide a much more rational basis for establishing seismic retrofit priorities and performance requirements for highway components.

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In recognition of these issues, the National Center for Earthquake Engineering Research (NCEER) has included system seismic risk analysis (SRA) in its current six-year seismic research project entitled "Seismic Vulnerability of Existing Highway Construction." This paper describes the SRA research being conducted under the NCEER project including: (a) a new SRA procedure that has been developed under the project; (b) an initial demonstration application of the procedure to the Memphis Tennessee highway system; (c) current research to further develop the procedure; and (d) the applicability of the procedure for real-time post-earthquake loss estimation.

2.0 SEISMIC RISK ANALYSIS PROCEDURE

2.1 General Description

The highway system SRA procedure is shown in Figure 1. It can be carried out for any number of scenario earthquakes and simulations, in which a "simulation" is defined as a complete set of system SRA results for one particular set of input parameters and model uncertainty parameters. The model and input parameters for one simulation may differ from those for other simulations because of random and systematic uncertainties (Werner et al., 1996).

For each earthquake and simulation, this multi-disciplinary procedure uses geoseismic, geotechnical and structural engineering, transportation network, and economic models to estimate: (a) earthquake effects on system-wide traffic flows (e.g., travel times, paths, and distances); (b) economic impacts of highway system damage (e.g., repair costs and costs of travel time delays); and (c) post-earthquake traffic flows along vital roadways (to facilitate emergency response planning). Key to this process is a modular GIS data base that contains the data and models needed to implement the system SRA.

This SRA procedure has several desirable features. First, it has a GIS framework, to enhance data management, analysis efficiency, and display of analysis results. Second, the GIS data base is modular, to facilitate the incorporation of improved data and models from future research efforts. Third, the procedure can develop aggregate SRA results that are either deterministic (consisting of a single simulation for one or a few scenario earthquakes) or probabilistic (consisting of many simulations and scenario earthquakes). This range of results facilitates the usefulness of SRA for a variety of applications (e.g., seismic retrofit prioritization and criteria, emergency response planning, planning of system expansions or enhancements, etc.). Finally, the procedure uses rapid engineering and network analysis procedures, to enhance its future use as a real-time predictor of system states and traffic impacts shortly after an actual earthquake.

2.2 GIS Data Base

The GIS data base contains four modules with data and models that characterize the system, seismic hazards, component vulnerabilities, and economic impacts of highway system damage. To facilitate analysis efficiency, these modules are pre-processors to the four-step SRA procedure shown in Figure 1.

2.2.1 System Module

The system module contains the following information to characterize the highway system, as provided by transportation and urban planning specialists:
• **System Data** – including: (a) system network configuration linkages, and component types and locations; (b) numbers of lanes, traffic flows, capacities, and congestion functions for each roadway link; (c) origin-destination zone locations and trip tables; and (d) any special system characteristics, such as certain roadways being critical for emergency response or national defense.

• **Traffic Management** – including measures by transportation officials for modifying the system to ease post-earthquake traffic flows (e.g., detour routes, changing roadways from two-way to one-way traffic, etc.)

• **Transportation Network Analysis Procedures** – to estimate post-earthquake traffic flows for each simulation and scenario earthquake.

### 2.2.2 Hazards Module

The hazards module contains input data and models provided by geologists and geotechnical engineers for characterizing system-wide ground motion, liquefaction, landslide, and surface fault rupture hazards. Input data include: (a) the ensemble of scenario earthquake events developed during the initialization phase of the SRA (Sec. 3.1); (b) locations and topographic data for slopes within the system that could be prone to landslide; and (c) local soil conditions throughout the system, as needed to estimate local geologic effects on ground shaking and the potential for liquefaction and landslide. Models contained in the hazards module will estimate: (d) the attenuation of rock motions with increasing distance from the earthquake source, for a range of earthquake magnitudes; (e) the effects of local soil conditions on the motions at the ground surface; and (f) permanent ground displacements due to earthquake-induced landslide, liquefaction, and surface fault rupture. A deterministic representation of hazards models will use mean values of these quantities. A probabilistic representation will use probability distributions to account for uncertainties in the seismologic, geologic, and soil input parameters and in the hazard evaluation models.

### 2.2.3 Component Module

The component module contains input data and models provided by structural and geotechnical engineers to characterize each component using a “loss model” and a “functionality model”. The loss model represents the component’s direct losses (i.e., repair costs), and the functionality model represents its “traffic states” (i.e., whether the component will be partially or completely closed to traffic during the repair of the earthquake damage, the durations of these closures, and speed limits for traffic along the component during repair). Both models are a function of the level of ground shaking at the component’s site, as well as the level of permanent ground displacement due to liquefaction, landslide, or surface fault rupture. The models for each component are developed by evaluating: (a) its seismic response to each designated level of ground shaking and permanent ground displacement; (b) its “damage state”, (i.e., the degree, type, and locations of any earthquake damage to the component); (c) its damage repair procedures; and, from this (d) its traffic states at various times after the earthquake (to reflect the rate of traffic restoration as repairs proceed).

After each component’s traffic states are obtained, they are incorporated into the highway system network model to obtain the “system state”, i.e., the ability of each link in the system to carry traffic at various times after the earthquake (in terms of number of open lanes, speed limits, etc.). These system states will reflect the effect of each component’s damage state on adjacent and underlying roadways. They will depend on the component’s location in the system, as well as system network characteristics.
A deterministic representation of loss and functionality models will use mean values of the component repair costs and traffic states. A probabilistic representation will use probability distributions to account for uncertainties in the evaluation of the component seismic response, and in the estimation of the resulting repair costs and traffic states.

2.2.4 Socio-Economic Module

The socio-economic module contains models and data for evaluating broader social and economic impacts of earthquake-induced traffic flow disruptions. These impacts can include indirect dollar losses (e.g., to commuters and businesses), effects on emergency response (e.g., reduced access to medical, police, fire-fighting, airport, government centers, etc.), and societal effects (e.g., reduced access to residential areas, shopping areas, etc.). This module is developed by transportation specialists, urban planners, and economists.

2.3 Analysis Procedure

2.3.1 Step 1: Initialization of Analysis

The initialization of the SRA (Step 1) contains two parts. First, regional earthquake source models are used to define an ensemble of scenario earthquakes, in which each earthquake is most commonly defined in terms of its magnitude, location, and frequency of occurrence. Uncertainties in defining the values of the various earthquake input parameters may also be modeled at this stage. The second part of Step 1 establishes the total number of simulations for each scenario earthquake, as further described in Werner et al. (1996).

2.3.2 Step 2: Development of Each Simulation for Each Scenario Earthquake

Under Step 2, the following evaluations are carried out to develop each of the simulations for each scenario earthquake:

- **Hazard Evaluation.** First, the data and models contained in the hazards module are used to estimate the earthquake ground motions and geologic hazards throughout the system.

- **Direct Loss and System State Evaluation.** Once the hazards are estimated, the data and models from the component module are used to evaluate direct losses and system states (defined at various times after the earthquake).

- **Traffic Flow Evaluation.** The system data and transportation network analysis procedure from the system module are applied to the pre-earthquake system and post-earthquake system states, to assess earthquake effects on system-wide travel times, travel distances, and travel paths, as well as traffic flows along roadways vital to emergency response.

- **Socio-Economic Impact Evaluation.** Once the earthquake effects on traffic flows within the system are evaluated, the data and models from the socio-economic module are used to evaluate impacts of the impeded traffic flows in terms of: (a) indirect dollar losses; and (b) reduced access to and from emergency response centers.
2.3.3 Step 3: Incrementation of Simulations and Scenario Earthquakes

Under Step 3, the evaluations from Step 2 are repeated, in order to develop multiple simulations for multiple scenario earthquakes (if the SRA is to be probabilistic).

2.3.4 Step 4: Aggregate System Analysis Results

This final step in the SRA process is carried out after the system analyses for all simulations and scenario earthquakes have been completed. In this step, the results from all simulations and earthquakes are aggregated and displayed. Depending on user needs, these aggregations could focus on the seismic risks associated with the total system or with individual components. Furthermore, the system or component results could be provided: (a) for individual simulations, which is termed a seismic vulnerability analysis, and/or (b) for the broader (probabilistic) range of simulations, leading either to loss statistics (e.g., average annualized loss) or to loss distributions that show the severity of earthquake-induced system losses for different probability levels. For research purposes, the impacts of incorporating uncertainties into the SRA will be of considerable interest. For other purposes, such as the planning of seismic strengthening programs for existing highway systems, outputs can be adapted and/or simplified to meet the particular requirements of each user audience.

3.0 DEMONSTRATION ANALYSIS

3.1 Objective and Scope

Early in the NCEER Highway Project, the SRA procedure was used with then-available data and models to carry out a demonstration SRA of the Memphis, Tennessee highway-roadway system (Fig. 2). The objective of the analysis was to: (a) illustrate the applicability of the SRA procedure; and (b) provide a basis for prioritizing research needs to improve the procedure. Because of limitations in many of the then-available data and models, the results from this analysis are preliminary. This analysis is described in more detail by Werner and Taylor (1995 and 1996).

3.2 Assumptions

The Memphis highway-roadway system is shown in Figure 2. This SRA consisted of deterministic analysis of the response of this system to four different earthquakes (Fig. 3a). This paper presents results from one of these earthquakes, termed Earthquake D, which has a moment magnitude of 5.5 and is centered 35 km to the north of the city of Memphis. Assumptions for this SRA are summarized below.

3.2.1 System Input Data

The system’s network configuration was obtained from the University of Memphis. Traffic data and O-D zones within the system were provided by the Memphis and Shelby County Office of Planning and Development (OPD). The traffic flow data were from their 1988 traffic forecasting model.

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7 The "loss" can be defined in several ways, such as direct repair cost, travel time delays due to earthquake damage (between certain key origin-destination zones or aggregated over all zones), indirect losses due to travel time delays, or other adverse consequences.
3.2.2 Network Analysis Procedure

The MINUTP traffic forecasting software (Comsis, 1994) was used to analyze pre- and post-earthquake traffic flows. This software was chosen because it is used at the Memphis-Shelby County OPD, and all regional traffic data were available in the input format for this software. MINUTP is based on the Urban Transportation Planning System (UTPS), which was developed over two decades ago by the U.S. Dept. of Transportation (see Sec. 4.5.1). Also, the then-available version of MINUTP was not GIS-compatible, which increased the effort needed for our system analysis.

3.2.3 Seismic Hazards

The system-wide ground shaking due to Earthquake D was represented in terms of peak ground acceleration (PGA), and was based on soil conditions estimated from prior local geologic mapping by the University of Memphis (Fig. 3b). The PGA at each bridge site was estimated by: (a) using an early version of the Hwang and Huo (1997) attenuation equation to compute site-specific rock accelerations; and (b) applying Martin and Doby (1994) soil amplification factors to these rock accelerations, to obtain corresponding ground surface PGAs that include effects of local soil conditions (Fig. 4).

3.2.4 Bridge Loss Models

Loss models previously developed under the ATC-25 project for conventional highway bridges were used to estimate direct losses for each bridge in the system (ATC, 1991). In these models, the direct losses depend only on whether the bridge has simple spans or is continuous/monolithic; i.e., other bridge structural attributes that could impact seismic performance are not considered.

3.2.5 Bridge Functionality Models

Functionality models for this demonstration SRA represented bridge traffic states as the number of lanes open at discrete times after an earthquake, as a function of PGA and the original number of lanes along the bridge. They were developed by modifying ATC-25 bridge restoration models based on prior observations of the seismic performance and repair and reconstruction processes for California bridges during the Loma Prieta and Northridge Earthquakes (Werner and Taylor, 1995). Two different models were developed in accordance with the ATC-25 conventional highway bridge designations -- one for simple-span bridges and one for continuous bridges. In addition, to illustrate effects of bridge damage repair rates on post-earthquake system performance, functionality models were developed for two discrete times -- three days and six months after the earthquake.

3.2.6 Economic Model

Studies of economic impacts of earthquake-induced highway system damage have shown that indirect dollar losses due to such damage can far exceed the direct losses for repair of the damage (e.g., Gordon and Richardson, 1996). However, methods for estimating such impacts for future earthquakes are not yet well developed. Therefore, for this demonstration SRA, a simplified procedure from BAA (1994) was used to estimate costs due to deterioration in commute time only. These cost estimates are based on vehicle-hours of delay (as obtained from the MINUTP system analyses), corresponding person-hours of delay (based on an assumed average vehicle occupancy rate of 1.4 persons/vehicle), truck-hours of delay (assuming 30 percent of the vehicles are trucks), and excess fuel costs due to travel time delays.
3.3 Results

3.3.1 Direct Losses

In accordance with the ATC-25 model used in this demonstration SRA, direct losses due to damage to the system's bridges are represented as a damage ratio, DMG (%), which is defined as the ratio of the repair cost for each bridge to its total replacement cost. For Earthquake D, the average damage ratio (averaged over all of the 286 bridges in the system) was 37.4%.

3.3.2 Travel Times and Distances

System State Results. Figure 5 shows the pre-earthquake system state and post-earthquake system states at times of three days and six months after Earthquake D. This figure indicates that, although Earthquake D has only a moderate magnitude (MW = 5.5), its proximity to the northern segment of the Memphis highway system causes extensive roadway closures in that segment, with lesser impacts on other segments of the system.

Total System-Wide Travel Times. Table 1 contains the total pre- and post-earthquake travel times and distances for the Memphis highway system. This table shows that the modified system states due to Earthquake D result in a total system-wide travel time three days after the earthquake that is nearly 34 percent longer than the pre-earthquake values. At six months after the earthquake, the bridge repairs within that time have reduced the total travel time; however it is still nearly 20 percent longer than the pre-earthquake value.

Total System-Wide Travel Distances. Table 1 shows that the total system-wide travel distances at times of three days and six months after the occurrence of Earthquake D are not sensitive to the modified system states. This trend may be due to the significant loss of service along the faster but less direct highway segments at the north and northeastern portions of the beltway, because of the many damaged bridges along those segments. As a result, drivers would be forced to use ground surface routes with fewer damaged bridges that are shorter but slower than the beltway routes.

O-D Zone Travel Times. Table 2 shows that, at a time of three days after the earthquake, the travel times between the O-D zones listed in the table are, on the average, nearly 16 percent larger than those for the pre-earthquake system. The travel time increases are largest for northernmost of the highlighted zones, which are at Shelby Farms (Zones 249 and 252), Bartlett (Zone 264), and the Covington Pike (Zone 274). This is because, as previously noted, it is this section of the Memphis area highway and roadway system that is most severely damaged. At a time of 6 months after the earthquake, Table 2 shows that the travel times to and from these zones have been reduced substantially, and are now only 5.3 percent larger than the pre-earthquake values.

O-D Zone Travel Distances. The travel distances to and from the O-D zones listed in Table 2 are insensitive to system damage from Earthquake D (Werner and Taylor, 1995).

Economic Impacts. Estimates of economic impacts for times of both three days and six months after the earthquake are shown in Table 3. They are based on total system-wide travel time delays per 24-hour day of 126,000 vehicle-hours and 73,000 vehicle-hours at times of three days and six months after the earthquake respectively (as previously shown in Table 1). From this, the BAA (1994) cost estimation procedure leads to
a total cost per day of the earthquake-induced time delays of $1.6 million at three days after the earthquake, and $930 thousand at six months after the earthquake. We then estimated the total time delay costs over a one-year time period after Earthquake D, by assuming an average daily time-delay cost for the year of $930 thousand (which corresponds to the above daily cost at a time of six months after the earthquake). From this, the total cost of the system-wide time delays over this one-year time period was computed to be 365 days x $930,000 = $340 \times 10^6.

4.0 NEW DEVELOPMENTS

Since the above demonstration SRA was completed, we have implemented significant improvements to the SRA procedure. This improved procedure has just been used in a probabilistic re-analysis of the seismic risks to the Shelby County, Tennessee highway-roadway system, which includes the city of Memphis. This new analysis is described in Werner et al. (1998).

The improvements to the SRA procedure include upgraded models for: (a) multiple scenario earthquakes; (b) ground shaking and liquefaction hazards; (c) bridge vulnerability modeling; and (d) transportation network analysis. They are summarized in the remainder of this section.

4.1 Scenario Earthquakes

In a SRA of a system with spatially dispersed components, individual scenarios are required to evaluate correlation effects of earthquakes, i.e., the simultaneous effects (including systemic consequences of damages) of individual earthquakes on components located at diverse sites. For our updated SRA of the Memphis highway system, we have adapted scenario earthquake models for the Central and Eastern United States (CEUS) that were developed by Frankel et al. (1996) as part of the United States Geological Survey (USGS) National Hazard Mapping Program. For other regions of the United States, Frankel et al. models for those regions (or other appropriate models that account for regional seismologic and geologic characteristics) should be similarly adapted.

The Frankel et al. work for the CEUS uses four different spatially smoothed models based on historical seismicity data, plus a special model for the New Madrid Seismic Zone (NMSZ). Our adaptation of these models is summarized in Sections 4.1.1 and 4.1.2.

4.1.1 Historical Seismicity Models

For developing scenario earthquakes for our SRA of the Memphis highway-roadway system, we define a large seismicity zone around Memphis that extends from 88.0 to 92.0 degrees longitude and from 34.0 to 38.0 degrees latitude. This zone (denoted as Zone A) has been divided into 1,763 microzones, with dimensions of about 11.1 km in both length and width.

Three different models are weighted to establish the earthquake activity within each microzone, based on historical seismicity data from a USGS catalogue that is an updated and improved version of the Seeber-Armbruster (1991) earthquake catalogue. These models are developed from earthquakes with the following magnitude cutoffs and completeness times: (1) magnitude 3+ earthquakes since 1924; (2) magnitude 4+ earthquakes since 1860; and (3) magnitude 5+ earthquakes since 1700. In addition, a fourth model by Frankel et al. that represents background seismicity over a larger zone was weighted with the above three
models to establish earthquake activities. This larger zone (denoted as Zone B) extends from -80.0 to -112.0 degrees longitude and from 30.0 to 40.0 degrees latitude.

The number of earthquakes shown in the USGS catalog to exceed the respective minimum magnitude of Models 1 through 3 respectively is counted and, based on the starting and end date of the model (e.g., 71 years for Model 1), is converted to a frequency of occurrence. As work progressed, Frankel et al. overrode these estimates within a special aerial zone (denoted as Zone C) that covers about 20 percent of Zone A. For microzones within Zone C, frequencies of earthquake occurrence were established by assuming that the earthquakes from the historic seismicity model for this zone are uniformly distributed throughout the zone.

To account for uncertainties in the locations of earthquakes estimated for each of the 1,763 microzones in Zone A, a relatively flat gaussian model is applied that redistributes and smooths the earthquake locations among the microzones. Given this redistribution of earthquake occurrences for each of Models 1-3, and assuming a threshold magnitude of 5.0 for the onset of earthquake damage, a "b" value of 0.95 in the Richter magnitude-frequency relationship (derived elsewhere) is used to estimate the frequency of occurrence of earthquakes with magnitude ≥ 5.0 in each microzone (for each of these models). For Model 4, a uniform distribution is used to allocate potential earthquakes with magnitudes ≥ 5.0 among all of the microzones.

Based on a method of adaptive weighting, the four above-mentioned models are combined to derive frequencies of occurrence of earthquakes of magnitude ≥ 5.0 in each microzone. Next, these frequencies are summed to determine the corresponding frequency of occurrence within the overall seismicity zone. Using this frequency and the frequencies in each microzone, a conditional cumulative probability matrix is then developed for the overall seismicity zone. This two-column matrix contains microzones (numbered) in one column, and cumulative conditional probabilities (from 0 to 1) in the other column. In addition, a Poisson model is used to convert the frequency of occurrence of earthquakes with magnitudes ≥ 5.0 in the overall seismicity zone to a corresponding probability of occurrence.

At this stage, a natural way to develop these scenarios for purposes of analyzing system performance and for eventually compiling information on loss distributions and their variability over a time dimension is to employ a "walk-through" analysis. (Daykin et al., 1994). The first step in this analysis selects an appropriate time frame over which the analysis would be carried out (e.g., one or more time frames of 10 years, 50 years, 100 years, etc.). Then, for each year in each time frame (starting with Year 1 and then repeating the process for each successive year), successive uniform random number generators are applied with the appropriate cumulative conditional probability distribution to evaluate: (a) whether at least one earthquake of magnitude ≥ 5.0 has occurred somewhere in the large seismicity zone during the year; (b) if so, whether a second earthquake has occurred in the zone during the year; and (c) for each earthquake that has occurred in the zone during the year, the microzone where the earthquake is located. We also use a random generation technique to estimate the earthquake magnitude, with the likelihood of diverse magnitude levels assumed to be represented by a Richter (lognormal) magnitude-recurrence relationship.

4.1.2 New Madrid Fault Zone

The modeling the New Madrid fault zone involves the following steps: (a) modification of the Der Kiureghian et al. (1977) approach to distribute earthquake occurrences within the fault zone; (b) use of estimates of the frequency of occurrence for earthquakes in the zone based on the Frankel et. al. approach and other relevant studies (e.g., Johnston, 1996; Johnston and Schweig, 1996; Crone, 1998); use of a Poisson
model to convert these frequencies to probabilities of occurrence; and (c) postulation that the fault zone is comprised of four parallel linear faults.

Following this, a walk-through analysis is used to develop a random sequence of earthquakes occurring within the zone during the time period of interest. To illustrate, let us assume that the probability of occurrence of an earthquake with a given magnitude (say magnitude 8.0) within the New Madrid fault zone is 0.002. From this, the walk-through process for each year involves the use of successive random number generators to indicate: (a) whether an earthquake of this magnitude has occurred within the fault zone (by checking whether the random number has a value \( \leq 0.002 \)); and (b) if so, which of the four fault traces is the source of the earthquake. Then, subsequent steps involve: (c) estimation of the rupture length along the fault trace, by first using the Wells-Coppersmith (1994) relationship between rupture length and earthquake magnitude to obtain best estimate rupture lengths, and then by accounting for uncertainties in this relationship through use of the polar method in log space (Law and Kelton, 1991) with a standard deviation of 0.22; and (d) estimation of the location of the rupture length within the overall fault trace, by applying a random number generator to the difference between the fault trace and the estimated rupture length.\(^8\)

The results of this walk-through analysis of earthquakes occurring within the New Madrid fault zone are combined with the results of the walk-through analysis of potential earthquakes from the historical seismicity models (Sec. 4.1.1) to estimate the total earthquake activity during each year of the time frame of interest.

4.1.3 Results

The above approach has been used to develop an ensemble of 2,320 earthquakes with moment magnitudes that range from 5.0 to 8.0 and occur within the above-indicated zones that surround Memphis and Shelby County, Tennessee. This is further described in Werner et al. (1998).

4.2 Ground Motion Hazards

The ground motion hazards used in the new SRA of the Shelby County highway-roadway system are represented as peak ground accelerations (PGAs) at the ground surface. The estimation of these PGAs for a particular site involves: (a) use of a rock motion attenuation relationship to estimate bedrock peak acceleration levels; and (b) application of soil amplification factors to these rock accelerations, to develop corresponding PGAs at the ground surface that incorporate effects of local soil conditions. When new bridge vulnerability models are incorporated into the SRA procedure during this year (1999), ground motion hazards will be represented as five-percent damped spectral accelerations, rather than PGAs.

In the new SRA of the Shelby County highway-roadway system, we use: (a) the Hwang and Huo (1997) rock motion attenuation relationships for peak acceleration and for spectral accelerations over a wide range of natural periods; and (b) the Hwang et al. (1997) soil amplification factors for NEHRP site classifications A through E. These procedures have the following benefits: (a) they are internally consistent, i.e., they are intended for use together to compute ground surface peak accelerations and spectral accelerations (as the product of the Hwang and Huo rock motions and the Hwang et al. soil amplification factors); (b) they specifically focus on anticipated CEUS ground shaking characteristics; (c) the Hwang and Huo rock motion

\(^8\)In this, the difference between the rupture length and the total length of the fault is computed, and a uniform random number generator is used to indicate where the fault rupture is initiated relative to one end of the fault trace.
attenuation relationships compare well with other well-established relationships for the CEUS; (d) the Hwang et al. soil amplification factors are developed from state-of-the-practice analytical procedures; and (e) effects of uncertainties in various input parameters are considered.

4.3 Liquefaction Hazards

The treatment of liquefaction hazards within the multi-scenario framework of the SRA procedure involves the following steps: (a) compilation of soils data for the region; (b) for a given scenario earthquake and simulation, evaluation of the potential for liquefaction throughout the highway-roadway system, including estimation of permanent ground displacements; and (c) estimation of traffic states at bridges and along roadways within the system due to these ground displacements. Our plans for carrying out these steps in our SRA procedure are summarized below.

4.3.1 Soils Data

The first step in the characterization of system-wide liquefaction hazards is, of course, to compile appropriate soils data throughout the highway-roadway system. For Shelby County, such data have been compiled by the Center for Earthquake Research and Information (CERI) of the University of Memphis, from 8,500 boring logs throughout the county (Ng et al., 1989). In this, the county is divided into a series of cells with dimensions of about 2,500 ft. by 3,000 ft. Then, the data from the boring logs are used to develop the following information for those cells where boring logs are available: (a) estimated average values SPT blowcounts, natural soil density, and unconfined compressive strength for each soil layer, as well as ground surface elevation, and groundwater level; and (b) development of a representative soil log for the cell. For those cells, where no data are available, soil properties are estimated using data from the nearest cells with soils of the same geologic unit (Hwang and Lin, 1997).

A GIS data base containing these data has been made available by CERI for use in our SRA of the Memphis highway-roadway system. An update of this data base is reportedly underway and nearing completion (for release during early 1999).

4.3.2 Hazard Evaluation Procedure

Evaluation of liquefaction hazards throughout the highway-roadway system will follow the approach by Youd (1998). Our adaptation of this approach (see Werner et. al, 1998) consists of the following steps:

- **Initial Screening.** An initial screening of soils and geologic is carried out (as a pre-processor to the actual SRA) to initially establish which sites in the system have a low potential for liquefaction and therefore can be eliminated from further analysis. For the SRA of the Shelby County highway-roadway system, these initial screening efforts are guided by prior liquefaction evaluations of the Memphis area by Hwang and Lin (1997).

- **Further Screening.** For those sites shown by the initial screening to have a potential for liquefaction, further screening is carried out through simplified and conservative assessment of the range of possible ground shaking hazards at each site along the highway-roadway system due to each given scenario earthquake. Sites that are thereby shown to have a low liquefaction potential are eliminated from further evaluation for that earthquake.
• **Seed-Idriss Procedure.** For those sites that are still shown to have a potential for liquefaction, the Seed-Idriss (1982) procedure is used to compute factors of safety against liquefaction at each site for each scenario earthquake.

• **Permanent Ground Displacement.** For the sites shown from the above step to have a factor of safety against liquefaction that is less than 1.0, permanent ground displacements are estimated as follows: (a) for bridge or roadway sites with gently sloping ground or a free face condition, the Bartlett-Youd (1995) procedure is used to estimate lateral spread displacements; and (b) the Tokimatsu-Seed (1987) procedure is used to estimate vertical settlements.

4.4 **Bridge Modeling**

4.4.1 **Background**

The estimation of bridge damage states and traffic states is an essential step in the SRA process. This section describes bridge models that are currently used in the SRA process for this purpose, or are planned for use in the future.

This SRA procedure will incorporate a default method for rapid analysis of bridge damage states due to ground shaking that is named the rapid pushover method (Dutta and Mander, 1998). This procedure has the following features: (a) it provides rapid estimation of bridge damage states, and is therefore practical for application to the large numbers of bridges that will be involved in SRA of highway-roadway systems; (b) it is a simplified but rational engineering procedure based on capacity-demand spectrum-analysis and pushover-analysis concepts; and (c) it uses readily available information on each bridge that is contained in the NBI data base, in order to infer input parameters needed for the damage state evaluation.

The focus of the rapid pushover method is to carry out rapid analysis of large numbers of bridges in a highway-roadway system. As a result, the method contains certain simplifications for representing the distributions, types, and locations of damage throughout a bridge. Although these simplifications will be of only secondary importance for most bridges, they could be more important for certain special bridges whose performance could have a significant impact on overall highway-roadway system performance (e.g., major bridges located along highway segments with limited or no redundancy).

Because of this, the SRA procedure will provide the user with an option of directly specifying alternative damage state fragility curves that would be independently developed using detailed analysis methods deemed appropriate by the user. However, the application of such methods will invariably be time consuming. Therefore, the methods will be impractical for application to many bridges, and should be applied only to limited numbers of special bridges that may be encountered.

4.4.2 **Rapid Pushover Method**

The rapid pushover method involves the development of a capacity spectrum, a demand spectrum, and the resulting damage state fragility curves. This process is summarized in the following subsections (Dutta and Mander, 1998).

4.4.2.1 **Capacity Spectrum**

The bridge capacity spectrum for the onset of each damage state is computed as the sum of the capacity contributions of the piers and the three-dimensional arching action of the deck.
• **Pier Contribution to Bridge Capacity.** Under longitudinal or transverse excitation, the strength capacity of a bridge pier will usually decay as the earthquake shaking proceeds. The magnitude and rate of this decay will depend on the design details at or near the potential plastic hinge zones -- particularly connection details such as lap splices and anchorage zones -- and on the shear capacity of the columns and the column-to-cap connections. Although sophisticated energy-based evaluation techniques are available for evaluating these sources of strength decay, a more simplified displacement-based method of analysis is instead used, in order to increase the speed and efficiency of the evaluation process. This method uses a simplified strength degradation model for the bridge pier, in which the total pier capacity consists of: (a) diagonal strut (or arch) action which constitutes the concrete resistance; and (b) resistance contributions arising from the longitudinal and transverse reinforcing steel. These contributions to the pier capacity are expressed in terms of geometric factors alone, which can be obtained or inferred from the NBI data base (FHWA, 1995b).

• **Deck Contribution to Bridge Capacity.** The contribution of the deck to the bridge's total base shear capacity is overlooked in most capacity analyses. This contribution is due to the resistance of the deck resulting from plastic moments that are mobilized by the bearings working as a group. This action occurs because, as the deck rotates, lateral displacement also occurs which is resisted by frictional forces in each bearing and by membrane action in the deck when the span gap closes. Dutta and Mander (1998) have evaluated this effect for bridges with multiple simply-supported spans and with continuous spans. For these cases, a plastic mechanism analysis is used to establish the deck capacity as the lowest capacity of all possible postulated failure mechanisms. These failure mechanisms incorporate the geometry of the deck spans, the relative flexibility of the pier bents, and the resistance and capacities of the bearings.

4.4.2.2 Demand Spectrum

A simplified form of the each bridge's demand spectrum for each scenario earthquake and simulation is constructed by: (a) representing the bridge’s demand spectrum by a short-period segment with a constant spectral acceleration, and a long-period segment that is inversely proportional to the natural period; and (b) anchoring the short-period and long-period segments of the spectrum to site-specific spectral accelerations at periods of 0.3 sec. and 1.0 sec. respectively. In this, the site-specific spectral accelerations for a damping ratio of five-percent of critical are obtained from the ground motion model contained in the Hazards Module.

4.4.2.3 Fragility Curves for “Standardized” Bridges

Median values of spectral acceleration (at a period of 1.0 sec.) that lead to the onset of each of five specified damage states are developed for a series of “standardized” bridges, which are defined in Basoz and Mander (1998) as corresponding to “long” bridges with “no appreciable three-dimensional effects”. For each damage state, this process involves: (a) computing an effective damping ratio from specified deformation levels for the onset of the given damage state; (b) modifying the five-percent-damped demand spectrum obtained from the ground motion model to correspond to this effective damping ratio; (c) obtaining a median spectral acceleration for the particular damage state and standardized bridge type, by scaling the modified demand spectrum until it intersects the capacity spectrum at the deformation level corresponding to the onset of the given damage state; and (d) obtaining a corresponding fragility curve by assuming that uncertainties associated with the method of analysis and with randomness of material properties are lognormally distributed with specified standard deviations.
4.4.2.4 Fragility Curves for Specific Bridge in Highway-Roadway System

For a specific bridge within the highway-roadway system, the fragility curve is developed by: (a) selecting the appropriate standardized bridge type for that bridge; and (b) modifying the fragility curve for the standardized bridge to account for effects of three-dimensional arching action in the deck and effects of skew, which are estimated from geometric parameters available in the NBI data base.

4.4.3 Other Damage State Models

Development of the rapid pushover method has been ongoing throughout this past year of the Highway Project. As a result, it is not yet incorporated into the SRA procedure, and alternative procedures have been used to carry out the new SRA of the Shelby County highway system that is described in Werner et al. (1998). These procedures are summarized below.

- **Typical Bridges in Shelby County.** Jernigan (1998) recently used the capacity-demand method described in the FHWA (1995a) seismic retrofit manual in order to develop damage state fragility curves for the 452 bridges within the Shelby County highway-roadway system, Tennessee. This work involved development of a GIS data base of structural attributes for these bridges, grouping of bridges according to superstructure and substructure characteristics, and development of fragility curves for each group that establish the probability of achieving none/minor, repairable, or significant damage as a function of peak ground acceleration (ATC, 1996). Dynamic analysis was used to develop the demands for bridges within each group, whose attributes are selected from random sampling of the range of attributes for that group. This approach was used to develop fragility curves for six bridge groups that typify nearly all of the bridges in Shelby County.

- **Major Bridges in Shelby County.** The Shelby County highway-roadway system contains two large steel bridges that cross the Mississippi River along Interstate Highways 40 and 55. For these bridges, special evaluations were conducted to develop bridge damage states, corresponding traffic states and repair costs, and resulting fragility curves. The fragility curves for the Mississippi River crossing along Interstate 40 was based on prior seismic analyses of each segment of the bridge that were conducted as part of a current seismic retrofit of the bridge (IAI, 1993). For the Mississippi River crossing along Interstate 55, experience and judgement were used to estimate the fragility curves for each damage state. The development of the fragility curves for each of these major bridges is described in Werner et al. (1998).

4.4.4 Traffic States

An important step in the bridge modeling process is the establishment of traffic states along the roadways at each end of the bridge, and also along roadways that pass beneath the bridge. These traffic states represent the ability of the various roadways to carry traffic, in terms of the number of lanes that remain open to traffic and possibly any reduction in speed limit as well. Once the traffic states are established for a given scenario earthquake and simulation, they are incorporated into a system network model to establish overall post-earthquake system states. Transportation network analysis procedures are then applied to these system states to estimate earthquake effects on system-wide traffic flows (see Section 4.5).

These traffic states will vary with time after the earthquake, to reflect the estimated rate and type of post-earthquake repair of the bridge. This rate of repair will, in turn, depend not only on the type and extent of damage to the bridge, but also on construction practices within the region of the country that contains the highway system being analyzed. To account for these variables when establishing improved bridge models for
our updated SRA of the Shelby County roadway system, we proceeded as follows: (a) a series of damage state definitions was established for the various bridges in the system; (b) experienced bridge engineers from the Memphis area and the Tennessee Department of Transportation reviewed these damage states and provided their opinion as to the costs, types, durations, and traffic impacts associated with the repair process for each damage state; (c) similar reviews were conducted by experienced engineers from the California Department of Transportation who were involved in repairs of bridges damaged during the Loma Prieta and Northridge Earthquakes; (d) from this, interim models were established that provide repair costs and traffic states (at various times after the earthquake) for each bridge damage state; and (e) these models were incorporated into the component module for the SRA procedure, as default traffic states for bridges and roadways subjected to ground shaking and ground displacement hazards.

4.5 Transportation Network Analysis Procedure

4.5.1 Background

As previously noted, the network analysis portion of our prior demonstration SRA of the Memphis highway system relied on MINUTP -- a standard program that implements Urban Transportation Planning System (UTPS) algorithms (Werner et al., 1996). Experience from that analysis showed that data preparation and implementation of MINUTP was unacceptably time consuming -- partially due to the fact that MINUTP was not GIS-compatible. Also, although UTPS models and their derivatives are standard planning tools in cities receiving Federal support for local transportation projects, such models have the following deficiencies for SRA applications: (a) consideration of an adequate level of detail for representing the region served by the system (i.e., region boundaries, O-D zones, and the system network structure) is costly; (b) loss-of-service measures developed from UTPS network assignment models are inconsistent with loss-of-service measures from other UTPS models; (c) behavioral shifts due to major disasters such as large earthquakes are difficult to represent; and (d) the UTPS procedure has little capacity for considering time dependence of system performance characteristics.

In view of this, it became clear that an alternative transportation network analysis procedure was needed that: (a) provides a capacity for more rapid estimation of network flows; (b) represents the latest well-developed technology and circumvents technical limitations of the UTPS algorithms; (c) is compatible with the GIS-based framework of our current SRA procedure; and (d) provides a capability for using transportation system input data typically available from Metropolitan Planning Organizations (MPOs).

We have found that a new Associative Memory (AM) procedure for rapid estimation of traffic flows that was developed at the University of Southern California (USC) best meets the above objectives (Moore et al., 1997). The objective of this AM work has been to provide rapid and dependable estimates of flows in congested networks, given changes in link configuration due to earthquake damage, and to attach these changes to the decision-making procedures used to prioritize bridges for seismic retrofit. Such a procedure articulates well with existing efforts in the field, because these flow estimates are input to both total transportation system cost and accessibility measures.

4.5.2 Overview of AM Procedure

The AM procedure is derived from the artificial intelligence field to predict changes in highway system flows. These predictions are based on good approximate solutions to constrained optimization problems that represent the economic determinants of network flows. As such, the AM procedure has the capability to
determine changes in the system's total commuting time due to changes in the highway system network. To illustrate, if one link of a freeway is being considered for retrofit, the change in the total system commuting time due to removal of this link is calculated by the following steps: (a) identification of equilibrium flows and commuting times for each link in the intact (pre-earthquake) network; (b) calculation of total system commuting times by summing the commuting times for all links; (c) removal of the link from the highway system, simulating closure due to earthquake damage; and (d) determination of the change in total system commuting time due to the link's removal.

4.5.3 Development of AM Matrix

The AM procedure focuses on the development of an AM matrix that is used to map given sets of system network configurations (stimulus) to lead to corresponding traffic flows (response). The AM matrix is developed from the following steps:

- **Step 1. Training and Test Cases.** Standard numerical analysis is used to develop an ensemble of user equilibrium traffic flows for various network configurations, all of which represent the same general type of network and traffic flow characteristics. Most of these solutions are designated as training cases, with the remainder designated as test cases. These flows are computed for each link in the system in terms of equivalent passenger-car-units per hour.

- **Step 2. AM Training.** The training cases from Step 1 are used to train the AM; i.e., to determine the elements of the AM matrix that minimize the mean-square difference between the true user equilibrium traffic flows for all training cases and the estimated flows using the AM matrix.

- **Step 3. AM Testing.** The basic premise of this approach is that the AM matrix will provide a good estimate of traffic flows from other network configurations that represent similar conditions to those of the training cases, but have not been included in these cases. To check this, the AM matrix is used to predict traffic flows for each test case from Step 1, and these predicted flows are compared to the actual flows for the test cases as obtained during that step.

- **Step 4. AM Refinement.** From past experience, Step 3 will usually lead to excellent comparisons between predicted and actual traffic flows if an adequate number of training cases has been selected. However, if needed, additional training and test cases can be developed in Step 1 and used to further refine the AM matrix and the accuracy of its traffic flow predictions.

4.5.4 Current Status

We are continuing to refine the programming the AM procedure for rapid estimation of traffic flows, to minimize data storage requirements. This programming considers that the procedure has both training and stimulus-response sub-modules. The training sub-module solves conventional user-equilibrium flow problems given different configurations for the Memphis network. The stimulus-response sub-module constructs an AM matrix that best fits these user equilibrium inputs (network configurations) and outputs (traffic flows). In this way, the exact user-equilibrium solutions "train" the AM, which then can be used to obtain very rapid estimates of traffic flows for other network configurations not included in the training sub-module. These AMs have been shown to provide good approximations of user-equilibrium flows associated with new network configurations, including networks in which capacity has been lost.
Although creation of training data is a pre-processing step, users of the SRA methodology will also have the opportunity to use the training sub-module to solve for exact user-equilibrium flows for selected post-earthquake system states. Such solutions will be particularly beneficial if the user elects to show deterministic SRAs for a limited number of scenario earthquakes. In addition, even for probabilistic SRAs involving multiple scenario earthquakes and simulations, results from such exact solutions can usually be added to the training data provided to the stimulus-response sub-module.

5.0 ECONOMIC IMPACTS

Future development of the SRA procedure will include a major effort to incorporate improved models for estimating how damage to the highway-roadway system will affect the productivity of economic sectors in the surrounding region. This effort will consider that damage to buildings and contents will reduce the demand for transportation services. Damage to the highway-roadway network will reduce transportation supply. How the regional economy responds to these changes requires a detailed model that is disaggregate in terms of economic sectors as well as geographic space. Developing and packaging such a model is challenging. A sophisticated user interface is needed to link user needs with a set of interacting models and a substantial site-specific data bank. Such models should also include assessment of economic losers and gainers due to earthquake damage to the system, and due to policies implemented to mitigate this damage.

6.0 CONCLUDING COMMENTS

This paper has described a new SRA procedure for highway systems, a preliminary demonstration application of the procedure to the Memphis highway system, and new research that has dramatically improve the procedure and its highway-roadway system seismic performance results.

The principal benefit of the SRA procedure is its ability to directly represent seismic performance of highway systems – in terms of post-earthquake traffic flow – and to represent systemic effects associated with the damage to various highway components. This information will provide a much improved basis for decision-making pertaining to such issues as prioritizing various components for seismic strengthening, establishing component seismic performance requirements and design criteria, and justifying funding for seismic retrofit or other seismic risk reduction measures.

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REFERENCES


**TABLE 1. EFFECTS OF EARTHQUAKE D ON TOTAL SYSTEM TRAVEL TIMES AND DISTANCES**

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>PRE-EARTHQUAKE VALUE</th>
<th>TIME AFTER EARTHQUAKE = 3 DAYS</th>
<th>TIME AFTER EARTHQUAKE = 6 MONTHS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Value</td>
<td>Percent Increase over Pre-EQ</td>
<td>Value</td>
</tr>
<tr>
<td>Total vehicle hours traveled over 24-hour period (incl. congestion)</td>
<td>3.73 x 10^5</td>
<td>4.99 x 10^5</td>
<td>33.8</td>
</tr>
<tr>
<td>Total travel distance (mi) over 24-hour period</td>
<td>15.5 x 10^6</td>
<td>15.6 x 10^6</td>
<td>small</td>
</tr>
</tbody>
</table>
### Table 2. Effects of Earthquake D on Travel Times Between Origin-Destination Zones (Over a 24-Hour Time Period)

<table>
<thead>
<tr>
<th>Origin-Destination Zone</th>
<th>Number</th>
<th>Pre-Earthquake Travel Time (Hours)</th>
<th>3 Days After Earthquake</th>
<th>6 Months After Earthquake</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Travel Time (hrs)</td>
<td>Percent Increase over Pre-Earthquake Time</td>
<td>Travel Time (hrs)</td>
</tr>
<tr>
<td>Government Center (downtown Memphis)</td>
<td>7</td>
<td>128</td>
<td>143</td>
<td>11.7</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>122</td>
<td>141</td>
<td>15.6</td>
</tr>
<tr>
<td>Medical Center</td>
<td>25</td>
<td>122</td>
<td>136</td>
<td>11.5</td>
</tr>
<tr>
<td></td>
<td>26</td>
<td>114</td>
<td>129</td>
<td>13.2</td>
</tr>
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<td></td>
<td>27</td>
<td>114</td>
<td>129</td>
<td>13.2</td>
</tr>
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<td></td>
<td>28</td>
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<td>129</td>
<td>12.2</td>
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<tr>
<td></td>
<td>29</td>
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<td>133</td>
<td>11.8</td>
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<td>University of Memphis</td>
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<tr>
<td>President’s Island (Port)</td>
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<td>138</td>
<td>153</td>
<td>10.9</td>
</tr>
<tr>
<td>Memphis Airport</td>
<td>188</td>
<td>136</td>
<td>150</td>
<td>10.3</td>
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<tr>
<td>Federal Express</td>
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<td>Mall of Memphis</td>
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<td>127</td>
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<td></td>
<td>241</td>
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<td>Shelby Farms</td>
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<td></td>
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<tr>
<td>Covington Pike</td>
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<td>137</td>
<td>181</td>
<td>32.1</td>
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<td><strong>TOTALS</strong></td>
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<td><strong>3255</strong></td>
<td><strong>15.7</strong></td>
<td><strong>2963</strong></td>
</tr>
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</table>

### Table 3. Economic Impacts of Travel Time Delays Due to Earthquake D

<table>
<thead>
<tr>
<th>Time After Earthquake</th>
<th>Time Delay (Vehicle-Hours/24-Hour Day)</th>
<th>Cost/Day</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total</td>
<td>Non-Trucks</td>
</tr>
<tr>
<td>3 Days</td>
<td>126,000</td>
<td>88,200</td>
</tr>
<tr>
<td>6 Months</td>
<td>73,000</td>
<td>51,100</td>
</tr>
</tbody>
</table>
a) Overall Four-Step Procedure

- **SYSTEM MODULE**
  - Network Inventory
  - Traffic Data
  - O-D Zones
  - Trip Distributions
  - Traffic Management
  - System Analysis Models
  - Model Uncertainties

- **HAZARDS MODULE**
  - Seismic Zones
  - Topography
  - Local Soils
  - Ground Motion Attenuation
  - Geologic Hazard Models
  - Model Uncertainties

- **COMPONENT MODULE**
  - Data
    - Structural
    - Repair Cost
    - Repair Procedure
  - Models
    - Loss
    - Functionality
    - Uncertainties

- **SOCIOECONOMIC MODULE**
  - Data
  - Models
  - Model Uncertainties

b) GIS Database

**FIGURE 1. SRA PROCEDURE FOR HIGHWAY TRANSPORTATION SYSTEMS**
FIGURE 2. MEMPHIS TENNESSEE HIGHWAY-ROADWAY SYSTEM
a) Scenario Earthquakes

b) Local Geology (Hwang and Lin, 1993)

FIGURE 3. SCENARIO EARTHQUAKES AND LOCAL GEOLOGY
FIGURE 4. PEAK ACCELERATION (G) DUE TO EARTHQUAKE “D”
FIGURE 5. SYSTEM STATES
SESSION 3
PANEL DISCUSSIONS
Session 3 - Panel Discussion

Panel discussion among representatives from the seven State DOTs (Arkansas, Illinois, Indiana, Kentucky, Mississippi, Missouri and Tennessee) most directly effected by the New Madrid Seismic Zone.

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SESSION 4
SEISMIC CODES AND SPECIFICATIONS
Seismic Design of Highway Bridges in Washington State

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ABSTRACT

Washington State is a high seismicity state. The seismic design of highway bridges has gone through significant changes in the past 50 years. It changed from no code requirements to using equivalent static approach to adopting the state-of-the-art AASHTO Seismic Design Specifications. The SEISAB and GTSTRUDL computer programs are used to model bridges for dynamic analysis. Washington State Department of Transportation (WSDOT) supports research and studies on seismic zonation, seismic response spectra, long duration earthquakes, soil-structure interaction, and subduction zone earthquakes. The results of research and studies become part of the quality improvement process in design, analysis and structural detailing.

INTRODUCTION

Washington State is earthquake country. Hundreds of earthquakes are recorded in the State each year. Most of the earthquakes are controlled, directly or indirectly by the tectonic activity in the Pacific Northwest. The tectonic activity is dominated by the interaction of the Juan de Fuca plate with the North American plate. The Juan de Fuca plate is known to be subducting beneath the North American plate along the Cascadia subduction zone (Figure 1) at a rate of about 3-4 cm/year (1.2-1.6 in/year). Fortunately, very few of these earthquakes (Figure 2) produce significant ground shaking or damage.

The two most recent major earthquakes were the 1949 Olympia earthquake with Richter magnitude of 7.0 and the 1965 Seattle-Tacoma earthquake with Richter magnitude of 6.5. Both of these Puget Sound earthquakes occurred within the subducting Juan de Fuca plate at depths of 54-63 kilometers (33-39 miles). Neither earthquake had significant aftershock activity. Almost all large Puget Sound earthquakes have lacked aftershocks. The lack of aftershocks is considered characteristic of deep earthquakes. A study by USGS in 1973 predicted that the largest earthquake to occur in the Puget Sound region in Washington State would occur as deep as 50 kilometers (31 miles) and have a magnitude as large as 7.5.

Based on information on earthquakes in the last 150 years, some researchers suggest that shallow earthquakes in the Puget Sound area may not have magnitudes exceeding 6.5. On the other hand, the largest earthquake estimated for Washington and Oregon is a subduction earthquake with magnitude 8 or greater, located off the coast between the Juan de Fuca plate and the overlying North American plate (1). Large subduction zone
earthquakes have long duration in excess of 60 seconds. There is no written record to indicate that such an earthquake occurred in the last 150 years.

Recent seismologic study shows that the Seattle Fault, which runs in the east-west direction through Seattle, is active and produced a large earthquake about 1,100 years ago. Recent USGS research suggests that earthquakes of magnitude up to 7.4 may be expected from rupture of the Seattle Fault.

Geologic evidence suggests that such large subduction zone earthquakes might have occurred at least eight times off the Washington coast in the past 5,000 years (1).

When and where the next big earthquakes will occur remain to be researched and answered by the seismologists and geologists. Meanwhile, bridge engineers rely on current research information, the AASHTO Seismic Design Specifications, and USGS Seismic Zonation Maps to design seismic resistant structures.

BACKGROUND

Seismic design of highway bridges in Washington State has gone through progressive changes in the past 50 years. In the 1950s and earlier, there were no seismic code requirements. The engineers took a small percentage of the weight of the superstructure and applied it horizontally to the structure to account for earthquake forces.

In the 1960s, engineers used the equivalent static force approach, commonly known as the Lollipop Method, to estimate earthquake forces. A structure was represented by one or more lollipops of single degree of freedom. The period of vibration of the lollipops were calculated to determine the percentage of structure weight to use for earthquake design. The Seismic Code imposed a 3% minimum and a 10% maximum for design.

In the 1970s, after the San Fernando Earthquake, the engineers still used the Lollipop Method, but the minimum was intuitively increased to 10% and the maximum increased to 26.6%.

In the 1980s, the Federal Highway Administration (FHWA) published a report under the title, Seismic Design Guidelines for Highway Bridges. This report was subsequently approved by the AASHTO Subcommittee on Bridges and Structures as Standard Specifications for seismic design of highway bridges. Washington State immediately adopted the new AASHTO Standard Specifications for the design and construction of new bridges to resist earthquakes. Bridges designed, analyzed and detailed using these Specifications are expected to withstand earthquakes up to 7.5 in the Richter scale without collapse or major damage while maintaining function of essential bridges.

Currently, Washington State uses SEISAB and GTSTRUDL to perform dynamic analysis of bridges. Bridge engineers are very good at computer modeling of the structures. They can produce sophisticated mathematical models and analyze simple and complex structures to a high degree of refinement. However, the results are only as good as the
input data. The weakest parts of the data are in the area of earthquake characteristics, soil conditions, and soil-structure interaction. In recent years, significant efforts have been devoted to gaining better understand in these areas.

To address this critical need, WSDOT contracted with J.P. Singh and Mansour Tabatabaie of Geospectra of Pleasanton, California in the spring of 1995 to develop a manual and a training course covering SSI for a range of foundation systems, soil types, and ground shaking levels commonly used and encountered by WSDOT designers. The primary objective of the design manual, “Design Manual for the Foundation Stiffnesses Under Seismic Loading” (2), was to provide bridge designers with a quality SSI design tool that significantly reduce the time required to design and analyze foundations for seismic conditions. The manual, which was completed in April 1996, presents a simple and practical “stiffness versus deflection” design charts for typical WSDOT foundation systems and soil conditions. These charts are used by bridge engineers to speed the process of determining the foundation stiffness matrices needed for the iterative WSDOT coupled foundation to superstructure seismic analysis procedure.

DESIGN GROUND MOTION

The AASHTO Seismic Design Specifications emphasize that the causes of earthquakes are still not well understood and experts do not fully agree as to how available knowledge should be interpreted to specify ground motions for use in design. It is an enormously complex task to combine the uncertainties of the influence of source mechanism of the earthquake; magnitude, distance and duration of the earthquake; geology of the travel path; and nature of the underlying soil to predict the ground motions at a proposed site.

The use of probabilistic concepts has allowed the uncertainties to be considered in the evaluation of ground motion characteristics. Probabilistic seismic hazard analysis provides a means in which the uncertainties can be identified, quantified and combined in a mathematical way. The most probable ground motions for a location can then be determined for use in designing structures to prevent collapse and protect public safety.

Specifying or selecting a design ground motion involves a balancing of risks and costs. The current AASHTO Seismic Design Specifications recommend the use of design ground motions with a 10 percent probability of exceedance for an exposure period of 50 years. These motions are likely to be exceeded about once every 475 years. This ‘10 percent in 50 years’ risk level is the basis for the Washington State seismic zonation map.

SEISMIC ZONATION

New data on seismicity and ground motion in the Puget Sound region and the Pacific Northwest have been researched and published. WSDOT initiated a research study to review and evaluate the applicability of the data with the objective of developing a seismic zonation map based on the unique seismicity and geology of the region (3).
Figure 3 shows the larger seismic activities in Washington State over the past 125 years. The majority of the large earthquakes occurred in the Puget Sound region. Earthquakes originating within the Puget Sound region can be divided into shallow zone earthquakes extending to a depth of approximately 19 - 30 km (12 - 19 mi), and deeper zone earthquakes ranging in depth between 39 - 70 km (24 - 44 mi). The deeper zone is coincident with the subducing Juan de Fuca plate.

The return intervals of major earthquakes in the Puget Sound region have been estimated as follows:

<table>
<thead>
<tr>
<th>Richter Magnitude</th>
<th>Recurring Interval</th>
<th>Last Occurrence</th>
<th>Events</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.0</td>
<td>10 years</td>
<td>15 years</td>
<td>South Cascades, 1981</td>
</tr>
<tr>
<td>6.5</td>
<td>23-35 years</td>
<td>31 years</td>
<td>Puget Sound, 1965</td>
</tr>
<tr>
<td>7.0</td>
<td>73-110 years</td>
<td>47 years</td>
<td>Olympia, 1949</td>
</tr>
<tr>
<td>7.5</td>
<td>300-500 years</td>
<td>Unknown</td>
<td>Unknown</td>
</tr>
<tr>
<td>8.0</td>
<td>300-1000 years</td>
<td>Unknown</td>
<td>Unknown</td>
</tr>
</tbody>
</table>

The seismic zonation map was developed utilizing probabilistic concepts in extrapolating ground motions from past earthquakes and potential earthquake sources. The main steps involved in the development of a probabilistic ground motion map are: delineation of seismic source zones, development of statistical relationships from the historic earthquake record in each zone and calculation and mapping of the extreme cumulative probability. The probabilistic approach is similar to that adopted by AASHTO in developing the zonation map for the Seismic Design Specifications. The risk associated with the map is based on a ground motion with a return period of 475 years having a 10% probability of exceedance during a 50-year exposure period. The probability is based on the relationship

\[ P_o = (1 - e^{-t/T}) \]  

where \( P_o \) = Probability of exceedance 
\( t \) = Exposure time in years 
\( T \) = Return period in year

The above relationship allows an engineer to determine the return periods for different exposure times while maintaining the same probability of exceedance. Seismic zonation maps can then be developed for ground motions corresponding to the desired return periods.

**Temporary or Detour Structures**

The above relationship has been used to determine the appropriate acceleration coefficient for temporary or detour structures with a service life of 3 to 5 years. It results in the recommendation that temporary or detour structures be designed for a seismic acceleration coefficient equal to one half the acceleration coefficient for a permanent structure at the same site. All other requirements of the AASHTO Seismic Design
Specifications shall apply. The Seismic Performance Category shall be based on the magnitude of the reduced acceleration coefficient.

CURRENT DESIGN AND ANALYSIS PROCEDURES

The AASHTO Seismic Design Specifications contain flowcharts outlining the steps involved in seismic design and analysis procedures, and a design example in Supplement A to illustrate the application of the Specifications and the Single Mode Spectral Analysis Method - Procedure 1. The NHI Course No. 13063 Seismic Bridge Design Applications, which was broadcast via satellite from the University of Maryland in April and July 1996, provides a very thorough coverage of the application of the AASHTO Seismic Design Specifications. The NHI course added seven more seismic design examples to illustrate the design, analysis and detailing requirements of the AASHTO Specifications. Additionally, the course covers modeling guidelines, multimodal analysis and column and foundation design.

Washington State follows the AASHTO Seismic Specifications (4), except for practices, procedures and data specifically applicable to Washington State (5). WSDOT currently uses the 1996 USGS Seismic Maps. The Multimode Spectral Method - Procedure 2 is used for all continuous bridges. SEISAB (6) and GTSTRUDL (7) computer programs are used to model the structures for dynamic analysis. The bridge designer has the preference on which program to use, although divergent alignments or some complex models normally incorporate the general purpose structural analysis program like GTSTRUDL. The following are general steps involved in seismic bridge design using the Multimode Spectral Method.

Step 1: Review geotechnical foundation report prepared by the Geotechnical Engineer: The foundation report will provide recommendation for foundation type and soil bearing pressures, seismic performance category if site specific study is conducted, acceleration coefficient, soil type, and other basic soil properties, soil layers, depth, density, water table elevation, liquefaction potential and residual strength.

Step 2: Perform Initial Dynamic Analysis: The structure is modeled using SEISAB or GTSTRUDL with an assumed fixed foundation or an initial foundation stiffnesses may be selected from the “Design Manual for the Foundation Stiffnesses Under Seismic Loading (2). A multimode spectral analysis is performed using Applied Technology Council’s ATC-6 response spectrum curves. These curves are embedded in the SEISAB computer program and are referenced by acceleration coefficient.

Step 3: Verify the Computer Model: It is prudent to verify that the modeling of the structure is correct before proceeding further. This step can save time and headache. For a program with good graphic user interface, such as GTSTRUDL, a graphic display is all that will be needed to verify the correctness of the model. Otherwise, some hand
calculations are needed to verify the output to check for reasonableness.

For a large structure in which the computer model is also large and complex, it is easier to build and verify the structural model in small modules, assuring each module is correct before moving on to the next. This approach will save time and labor in debugging the program if it becomes necessary.

Step 4: Determine Elastic Forces and Displacements:
These are the unfactored results from the initial dynamic analysis.

Step 5: Perform Preliminary Column Design:
The "Yield" computer program (8) is used for preliminary and eventually final design. This program calculates ultimate applied moments and axial forces, moment magnification effects, design column moment capacities and predicts the ultimate plastic hinge moment capacity. The appropriate response modification factor, R, is incorporated into the program in accordance to the AASHTO Seismic Specification. The "R" factor is an allowable reduction in elastic seismic moment if the column member reinforcing details provide superior ductility performance.

Step 6: Calculate the Column Plastic Hinging Moment and Plastic Shear:
The plastic hinging capacity of the column is recognized by AASHTO to be approximately 1.3 x Mn (nominal ultimate moment capacity). The plastic shear of the column is determined by the end fixity conditions of the column and is a function of the column length. An example of this calculation is in the worked examples of the AASHTO Seismic Code (4).

Step 7: Select Forces for Foundation Design:
The moments, shears and axial loads transmitted to the foundation is taken as the smaller of the elastic column loads from the dynamic analysis or the plastic hinging forces from the designed column. In general, the plastic hinging forces will be smaller than the elastic dynamic column loads.

Step 8: Evaluate and Select Foundation Type:
The foundation type selection should be based on recommendation from the Foundation Report to resist the structural loads as determined in Step 7. The designer must also consider foundation efficiency, economy, constructability and environmental impact.

Step 9: Trial Foundation Design:
The footing size, pile or shaft size and arrangement are determined. For a spread footing, compute the elastic springs and model the foundation into a 6x6 stiffness matrix. For pile/shaft foundations, LPILE1 (9) computes stiffnesses to develop an individual pile stiffness matrix. The pile group geometry is defined along with the individual pile stiffness matrices and input into a WSDOT developed program called GPILE (10). This program
builds a global stiffness matrix for the pile group. It also distributes column loads to the individual piles and solves for pile cap deflections and rotations. Alternately, the foundation stiffnesses may be obtained from the SSI Design Manual for the Foundation Stiffnesses Under Seismic Loading (2).

Step 10: Re-Analyze Forces and Displacements:
With the 6x6 stiffness matrix included in the model, the forces and displacements will change. With the new set of forces and displacements, the stiffness matrix is adjusted. Steps 9 and 10 are then repeated until the forces and displacements are compatible with the stiffness matrix used. Generally, convergence is achieved in 2 to 3 cycles. The foundation design is an iterative and tedious process. This iterative process is illustrated by a flowchart shown in Figure 4. In order to facilitate the process, WSDOT developed the SSI Design Manual for the Foundation Stiffnesses Under Seismic Loading (2).

Step 11: Final Design:
With the final forces and displacements from Steps 9 and 10, the final design of the columns and foundation elements is completed. The remainder of the superstructure, specifically the connections, restrainers and seat widths are designed and detailed per the AASHTO Seismic Specifications.

New 1996 USGS Seismic Maps

The new 1996 USGS seismic maps represent an effort by USGS to produce a set of probabilistic national ground motion maps that incorporate up-to-date information on seismicity, seismic sources, and ground motion attenuation. These maps depict contours of peak ground acceleration (PGA) and spectral accelerations (SA) at 0.2, 0.3 and 1.0 second for 5% damping of ground motions on rock for probabilities of exceedance of 10%, 5%, and 2% in 50 years, corresponding to return periods of approximately 500, 1000 and 2500 years respectively. These maps for the contiguous United States and for Alaska and Hawaii are available from and can be viewed or downloaded on the USGS website at http://geohazards.cr.usgs.gov. These maps are considered to be in better accord with current understanding of seismic sources, seismicity and ground motion attenuation than earlier maps. A comparison of ground motions from the New 1996 USGS maps with ground motions from the current AASHTO maps shows that the new maps give PGA values generally lower in the eastern states and higher in the western states. Care must be exercised in using the New 1996 USGS maps, especially when the maps give PGA values lower than those given in the current AASHTO Specifications.

WSDOT has adopted the New 1996 USGS Map with PGAs for 10% probability of exceedance in 50 years for seismic design of highway bridges, because the map reflects the effect of the Cascadia subduction zone earthquakes and the PGA values are higher everywhere within Washington State than those given by the current AASHTO maps. The new map, as shown in Figure 5, results in more conservative designs for Washington State.
RESEARCH

WSDOT supports and participates in seismic research at the state and national levels. WSDOT has accomplished useful and practical seismic research through the University of Washington and Washington State University. The findings of the research are implemented to improve design, analysis and detailing, to provide effective seismic retrofit techniques, to assess structural performance, to prepare earthquake emergency response plan, and to develop post earthquake inspection procedures.

The latest seismic research is on Ground Motions due to Large Magnitude Subduction Zone Earthquakes. The main objective of this research is to use currently developed analytical tools to evaluate the effects of large magnitude subduction zone earthquakes on ground motions at a number of sites in Washington State. Large magnitude megathrust earthquakes have occurred along the Cascadia Subduction Zone off the coasts of Washington, Oregon and Northern California. Such earthquakes have been postulated to have magnitudes of up to 9.5. However, there is no written evidence of such events within the last 150 years in the Pacific Northwest. The characteristics of these earthquakes are not completely understood, resulting in a great amount of debate. It is expected that this research will reduce the considerable uncertainty over the possible effects of subduction zone earthquakes on the design of new structures and on the seismic evaluation of existing structures in Washington State.

WSDOT benefits from research performed by CALTRANS and other national earthquake research centers. Continued seismic research is needed to gain fuller understanding in seismicity, geology and tectonics, characteristics of earthquake source zones, ground motion attenuation, local site conditions, and soil-structure interaction.

Recent finite element soil-structure interaction analysis of shaft, 1.0 m (3 feet) diameter or larger, has shown that the shaft has much higher stiffness than that computed by the conventional methods, such as COM624. Research is needed to adequately analyze the soil resistance to large diameter shafts.

CONCLUSIONS

Bridge engineers have a variety of simple and sophisticated analytical tools and computer programs available for seismic design and analysis of structures. However, considerable uncertainties still exist in seismic ground motions and soil-structure interactions. Continued research is needed to provide better design seismic ground motions and gain better understanding on the behavior of foundations under dynamic loading.

There is general agreement that the stiffness of a group of piles is not the same as the sum of the stiffnesses of the individual piles. However, there are many different opinions in quantifying the differences. Further analytical evaluation and experimental studies are needed to provide better information to the practitioners.
Better understanding of the seismic ground motions and the soil-structure interaction under seismic loadings will assure more reliable and cost effective seismic design.

REFERENCES

Figure 1 Cascadia Subduction Zone
Figure 2  Smaller Earthquakes
Peak Ground Acceleration
10% Probability of Exceedance in 50 Years

Figure 5 1996 USGS Seismic Map
NCHRP PROJECT 12-49
PRELIMINARY RECOMMENDATIONS FOR NEW AASHTO LRFD
SEISMIC DESIGN SPECIFICATION

Ian M. Friedland (MCEER)
Christopher Rojahn (ATC)
Ron Mayes (Dynamic Isolation Systems, Inc.)

ABSTRACT

On August 1, 1998, the ATC/MCEER Joint Venture, a partnership of the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCEER), initiated work on a project sponsored by the National Cooperative Highway Research Program (NCHRP) of the Transportation Research Board. The objective of NCHRP Project 12-49 is to develop new specifications for the seismic design of highway bridges, which can be incorporated into the AASHTO LRFD Bridge Design Specifications. These new specifications will be nationally applicable with provisions for all seismic zones. The results of research currently in progress or recently completed, along with current demonstrated practice, will be the principal resources for this project.

The design criteria to be developed under NCHRP Project 12-49 will address the following: (1) strength-based and displacement-based design philosophies; (2) single- and dual-level performance criteria; (3) acceleration hazard maps and spectral ordinate maps; (4) spatial variation effects; (5) effects of vertical acceleration; (6) site amplification factors; (7) inelastic spectra and use of response modification factors; (8) equivalent static nonlinear analysis methods; (9) modeling of soil-structure interaction and structural discontinuities at expansion joints; (10) duration of the seismic event; and (11) design and detailing requirements for both steel and concrete super- and substructures.
PRELIMINARY RECOMMENDATIONS
FOR A NEW AASHTO SEISMIC DESIGN CODE

IAN M. FRIEDLAND, P.E.
ASSISTANT DIRECTOR FOR TRANSPORTATION RESEARCH
MULTIDISCIPLINARY CENTER FOR
EARTHQUAKE ENGINEERING RESEARCH
NCHRP Project 12-49, FY '98
Comprehensive Specification for the Seismic Design of Bridges

Sponsor:
National Cooperative Highway Research Program
Transportation Research Board, National Academy of Sciences

For:
American Association of State Highway and Transportation Officials

Contract Agency:
ATC/MCEER Joint Venture
NCHRP Project 12-49 – Project Objectives

Development of new specifications and commentary for the seismic design of highway bridges. Project will consider all aspects of the seismic design process, including:

(1) design philosophy and performance criteria;
(2) seismic loads and site effects;
(3) analysis and modeling; and
(4) design requirements.

The new specifications resulting from this effort will be nationally applicable with provisions specific to all seismic zones. They will be prepared in a format compatible with the current AASHTO LRFD Bridge Design Specifications, and will draw upon the results of recently completed and current research, and current state-of-the-art practices in seismic analysis and design.
NCHRP Project 12-49 – Research Team

<table>
<thead>
<tr>
<th>Role</th>
<th>Names/Institutions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Administrative Officer</td>
<td>Chris Rojahn, ATC</td>
</tr>
<tr>
<td>Principal Investigator</td>
<td>Ian Friedland, MCEER</td>
</tr>
<tr>
<td>Project Manager</td>
<td>Ron Mayes, DIS</td>
</tr>
<tr>
<td>Seismic Hazard Foundations</td>
<td>Maury Power, Geomatrix Consultants</td>
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<td></td>
<td>Don Anderson, CH2M Hill</td>
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<td>Geoff Martin, USC</td>
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<tr>
<td>Substructures</td>
<td>Rick Nutt, consultant</td>
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<td>John Mander, SUNY at Buffalo</td>
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<tr>
<td>Superstructures</td>
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<tr>
<td></td>
<td>Michel Bruneau, SUNY at Buffalo/MCEER</td>
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<tr>
<td>Analysis Methods</td>
<td>Greg Fenves, UC Berkeley</td>
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<tr>
<td></td>
<td>Andrei Reinhorn, SUNY at Buffalo</td>
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<tr>
<td>Code Calibration</td>
<td>Andy Nowak, University of Michigan</td>
</tr>
<tr>
<td>Trial Designs</td>
<td>Bob Griebenow, BERGER/ABAM</td>
</tr>
</tbody>
</table>
**NCHRP Project 12-49 – Project Engineering Panel**

| Co-chairs | Ian Buckle, University of Auckland, NZ  
|           | Chris Rojahn, ATC |
| Members   | Serafim Arzoumanidis, Steinman Engineers (NYC)  
|           | Mark Capron, Sverdrup Corporation (St. Louis)  
|           | Po Lam, Earth Mechanics, Inc (Fountain Valley, CA)  
|           | Paul Liles, Georgia DOT (Atlanta)  
|           | Brian Maroney, Caltrans (Sacramento)  
|           | Joe Nicoletti, Consultant (San Francisco)  
|           | Charles Roeder, Univ Washington (ATC Board Rep)  
|           | Frieder Seible, UCSD (San Diego)  
|           | Ted Zoli, HNTB (Fairfield, NJ) |
NCHRP Project 12-49 – Tasks and Schedule

Phase I (8/1/98 – 2/28/99)

Task 1  Literature, research, and practice review

Task 2  Conceptual LRFD design criteria

Task 3  Additional research or development needs

Task 4  Interim report

Phase II (5/1/99 – 11/30/99)

Task 5  (a) draft LRFD specification and commentary
        (b) draft design examples
        (c) analysis of impacts and benefits

Mid-America Highway Seismic Conference, St. Louis, Mo – February 28 - March 3, 1999
NCHRP Project 12-49 – Tasks and Schedule, continued

Phase III (3/1/00 – 6/30/00)

Task 6 Revised (1\textsuperscript{st} revision) specification, commentary, design examples, impacts and benefits analysis

Phase IV (9/1/00 – 1/31/01)

Task 7 Final report with final (2\textsuperscript{nd} revision) specification, commentary, design examples, impacts and benefits analysis
Figure 2-11. Trends for changes in 0.2 second ground motion with return period at selected cities in eastern U.S.
Figure 2-13. Trends for changes in 0.2 second ground motion with return period at selected cities in western U.S.
Figure 2-15. Trends for changes in 0.2 second ground motion with return period at selected cities in California.
Table 4-3.
SITE FACTORS IN 1997 NEHRP PROVISIONS (BSSC, 1997).

(a) Values of $F_A$ as a Function of Site Class and Mapped Short-Period Spectral Response Acceleration, $S_A=S(0.2 \text{ sec})$

<table>
<thead>
<tr>
<th>Mapped Spectral Acceleration at Short Periods, $S_A$</th>
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<tbody>
<tr>
<td>Site Class</td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
<td>A</td>
</tr>
<tr>
<td>B</td>
</tr>
<tr>
<td>C</td>
</tr>
<tr>
<td>D</td>
</tr>
<tr>
<td>E</td>
</tr>
<tr>
<td>F</td>
</tr>
</tbody>
</table>

NOTE: Use straight-line interpolation for intermediate values of $S_A$.
* Site-specific geotechnical investigation and dynamic site response analysis should be performed.

(b) Values of $F_Y$ as a Function of Site Class and Mapped Spectral Response Acceleration at One-Second Period, $S_1$

<table>
<thead>
<tr>
<th>Mapped Spectral Acceleration at One-Second Period, $S_1$</th>
</tr>
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<tbody>
<tr>
<td>Site Class</td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
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</tr>
<tr>
<td>B</td>
</tr>
<tr>
<td>C</td>
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<tr>
<td>D</td>
</tr>
<tr>
<td>E</td>
</tr>
<tr>
<td>F</td>
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</tbody>
</table>

NOTE: Use straight-line interpolation for intermediate values of $S_1$.
* Site-specific geotechnical investigation and dynamic site response analysis should be performed.
Figure 4-2. Proposed soil response spectrum definition in 1997 NEHRP Provisions (BSSC, 1997).
### Proposed Performance Criteria

<table>
<thead>
<tr>
<th>Return Period &amp; Service</th>
<th>Bridge Importance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Other</td>
</tr>
<tr>
<td>150 yr Service Damage</td>
<td>Immediate Minimal</td>
</tr>
<tr>
<td>2500 yr Service Damage</td>
<td>No Collapse Significant</td>
</tr>
</tbody>
</table>
Proposed for discussion only!!!
Not reviewed or approved by any agency or authority!!!

Service Level Definitions

Uninterrupted (R = 0.75) – Full access to normal traffic immediately following earthquake

Immediate (R = 1 to 1.5) – Full access to normal traffic following inspection of bridge

Limited (R = 2 to 4) – Limited access (lane reductions, emergency traffic) possible without shoring/repairs following inspection

No Collapse (R = 6 to 8) – Limited access (lane reductions, light emergency traffic) possible after shoring; bridge may require major repair/replacement
Proposed for discussion only!!!
Not reviewed or approved by any agency or authority!!!

Quantitative Performance Requirements

Specific performance objectives, related to material behavior and limits.

For example, concrete columns:

<table>
<thead>
<tr>
<th>Service Level</th>
<th>Concrete Strain</th>
<th>Steel Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uninterrupted</td>
<td>0.003</td>
<td>0.002</td>
</tr>
<tr>
<td>Immediate</td>
<td>0.004</td>
<td>0.01</td>
</tr>
<tr>
<td>Limited</td>
<td>0.007</td>
<td>0.015</td>
</tr>
<tr>
<td>No Collapse</td>
<td>(a)</td>
<td>(b)</td>
</tr>
</tbody>
</table>

(a) For confined concrete, use Mander energy model ($\varepsilon_{cu} \approx 0.02$)
(b) Low cycle fatigue limits ($\varepsilon_{st} \approx 2(\varepsilon_{sy} + 0.021T^{1/6}) - \varepsilon_{sc}$)
Proposed for discussion only!!!
Not reviewed or approved by any agency or authority!!!

Quantitative Performance Requirements, Cont’d

For example, abutment wall on spread footing, designed for active soil pressure only:

<table>
<thead>
<tr>
<th>Service Level</th>
<th>Perm settlement</th>
<th>Perm lateral movement</th>
<th>Loss in structural strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uninterrupted</td>
<td>No perm settlement</td>
<td>No perm lateral movement</td>
<td>No loss in structural strength</td>
</tr>
<tr>
<td>Immediate</td>
<td>Perm settlement &lt; 25 mm</td>
<td>Perm lateral movement &lt; 25 mm (top &amp; bottom)</td>
<td>No loss in structural strength</td>
</tr>
<tr>
<td>Limited</td>
<td>Perm settlement &lt; 100 mm</td>
<td>Perm lateral movement &lt; 5% wall height</td>
<td>Loss in structural strength &lt; 10%</td>
</tr>
<tr>
<td>No Collapse</td>
<td>Perm settlement &lt; 300 mm</td>
<td>Perm lateral movement &lt; 500 mm (top &amp; bottom)</td>
<td>Loss in structural strength &lt; 30%</td>
</tr>
</tbody>
</table>

Mid-America Highway Seismic Conference, St. Louis, Mo – February 28 - March 3, 1999
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Special Provisions for Zone 2 Bridges

For “regular” bridges within defined limits, no seismic analysis required providing specified minimum detailing requirements have been satisfied.

- bridge must be regular and simply supported or fixed at all transverse locations and one longitudinal location
- full participation of all bents for continuous decks
- columns must be designed for all load cases, with minimum confinement steel required at top and bottom
- shear reinforcement determined by capacity design

Problem: not currently sure how to address abutments and foundations under this scenario
Design Approaches

Damage Avoided
  Seismic isolation
  Conventional ductile design (R<1.5)

Damage Minimized or Repairable
  Energy dissipation
  Damage avoidance detailing
  Control and repairability detailing
  Conventional ductile design (1.5<R<3)

Damage Accepted
  Conventional ductile design (R>3)
Proposed for discussion only!!!
Not reviewed or approved by any agency or authority!!!

Design Approach

3 levels of design/analysis:

Level 1 – No analysis, but capacity design principles and details

Level 2 – Elastic analysis (cracked section properties) for governing design spectra (either 150 year or 2500 year), with conservative R-factors (i.e., R = 5 rather than 8 for 2500 year)

Level 3 – Elastic analysis (cracked section properties) for governing design spectra (150 year/1.5 or 2500 year/8) for preliminary column sizing. Pushover analysis used so design forces in columns can be reduced (but limited to 150 year minimum); must satisfy displacement demands

Level 1 should be applicable to vast majority of bridges in U.S.
The NEW AASHTO
Guide Specification for Seismic Isolation Design

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Engineering Service Center
California Department of Transportation
Sacramento, CA

ABSTRACT

The first AASHTO Guide Specification for Seismic Isolation Design was published in 1991. This guide specification represented a major step in the acceptance of bridge isolation design in the United States. Since adoption of the Guidelines, there has been significant progress in the understanding of isolation technology and the need for updating the document became obvious. A special task group was appointed by the AASHTO T-3 Seismic Design Committee to update this Guide specification. The work was completed by February 1997 and has been balloted by AASHTO. This paper addresses the highlights and issues in the new document.

INTRODUCTION

Since adoption of the 1991 Guide Specification for Seismic Isolation Design by AASHTO, significant progress in the understanding of performance of isolated structures and development of new isolation bearings other than elastomeric based have been made. To fully utilize the potential usage of all isolation systems, the AASHTO T-3 Seismic Design Committee decided to update the existing document. A task group consists of three state engineers, three academic professors, three industry representatives, and one FHWA representative was formed. The task group expands the current 1991 AASHTO Guide Specification by considering the current state-of-practice research results, completed and ongoing activities, rather than develops an entirely new document.

SIGNIFICANT CHANGES

The new Guide Specifications for Seismic Isolation Design contains the following major modifications:
2. The response modification factors (R-factors) have been reduced.
3. Consideration of substructure response.
4. Expand upper and lower bound concept to Lambda factor methodology.
5. Add sliding bearing design and construction.
6. Add a section for other isolation systems that are not covered by either elastomeric or sliding systems.
7. More specific system characteristic, prototype, and proof tests.

**EDITORIAL CHANGES**


A new soil type D was added and a corresponding value was assigned to the table. The analysis procedures are consists of four procedures the same as the Standard Specification now. That is:

- Procedure 1: Uniform Load Method
- Procedure 2: Single Mode Spectral Method
- Procedure 3: Multi Mode Spectral Method
- Procedure 4: Time History Method

Numerous commentaries were also added to give a more detail background information of the codes.

**RESPONSE MODIFICATION FACTOR (R)**

The 1991 Guide Specification allowed designer use R-factors on the substructure as it is used for regular seismic design. The new Guide Specification requires the use of a lower R factor to ensure proper performance of the isolation system. The R-factors are now limited to the range of 1.5 to 2.5 or no larger than one half of the corresponding R factor as conventional design.

To demonstrate the need of lower R factor, let's considered a case of an isolated bridge without redundancies. Considered isolator units with $Q_d = 0.06w$ and $F_{max} = 0.18w$ where $w$ is the gravity load on the isolator units. $F_{max}$ represents the seismic force at
design displacement, which was calculated on the assumption of elastic substructure behavior. Considered that the substructure is single column and an R factor of 3 was used, the substructure is designed to have strength of 0.06w. Assume a 30% overstrength, then the actual strength is about 0.08w. Since concrete column has lower post yielding stiffness than the isolators above, deformation will occur in the column more rapidly once strength demand is over 0.08w. This will result with excessive ductility demand in the column beyond original design. If this is a retrofit design then it is clear that the isolators can not protect the structure from collapse.

SUBSTRUCTURE RESPONSE

Traditional isolation design put isolators at ground level. This setup is most effective to separate ground motion from structure above the isolators. However this kind of layout seldom used on bridges. Most bridges have support bearings at top of columns or bent caps. Because of that, it becomes an ideal location to place isolation bearings instead of regular bearings at these locations.

For new structures designed with the isolation concept, it may be not critical if the substructure response is not included. Whereas it may not be the case if isolators are used for retrofitting exist bridges, especially for bridge built before 1970. The additional consideration of substructure response will reduce the effectiveness of isolation since both effective stiffness and equivalent viscous system damping were reduced.

No matter what material made of substructures, the post yield stiffness of columns or piers are usually low compared to isolators. To ensure proper performance of isolated bridges, substructure performance has to be considered not only to account for its share in overall performance but also to ensure it stay within its deformation and strength limit states.

LAMBDA METHODOLOGY

The most significant change to the Specification is the requirement to consider performance variables. These include both long-term environmental effects and short term performance variation caused by velocity, frequency, temperatures, workmanship, and original conditions. The variations will change the effective stiffness and damping of the system.

A upper and lower bound analysis is required if the performance of the structure varied by more than 30 percent while the maximum and minimum values of the isolator properties are used. The purpose of this upper and lower bound analysis is to determine the maximum force on the substructure elements and the maximum displacement of the isolation system.
Figure 1 Characteristics of Bilinear Isolation Bearings

The hysteresis behavior of an isolator unit is usually represented by a bilinear stiffness system as seen in figure 1. This force deflection relationship has two important variables, \( Q_d \) and \( K_d \), and both of which can be impacted by environmental and temperature effects. The energy dissipation per cycle, \( EDC \), is influenced by \( Q_d \) and the equivalent viscous damping, \( B \), is primarily influenced by \( EDC \) and \( K_{eff} \). The effective stiffness \( K_{eff} \) is influenced by \( Q_d \) and \( K_d \). The variation of \( K_{eff} \) may alter the effective period and which will influence the response. Thus the Guide Specification require that, regardless of the method of analysis used, the analysis shall be conducted once with the maximum probable properties and once with the minimum probable properties.

Figure 2 Upper and Lower Bound Properties

Values of the probable maximum and minimum properties will be based on a statistical analysis of the relevant property variations. The Guide Specification defined the maximum and minimum properties to the nominal one by a series of System Property...
Modification Factor $\lambda$. This provides a rational and systematic tool for dealing with numerous variables. Different variables are associated with different aspects of specific isolation systems, such as wear, temperature, environmental effects, etc. In the absence of such information, provisional values defined in Appendix A will be used. It is expected that individual suppliers will determine if they can gain any benefits from conducting such characteristic tests.

ADD SLIDING AND OTHER TYPE BEARINGS

The 1991 Guide Specification for Seismic Isolation Design by AASHTO has long been criticized for its bias for elastomeric isolation system. Since adoption of the early Guidelines, there has been significant progress in the development and understanding of new isolation bearing technologies that are not elastomeric based. The new specification has two sections that cover sliding bearing design and construction. It also has a section cover all other type isolation systems that were not covered in the previous sections of the specification.

The sliding bearing sections cover bearing that use PTFE and stainless steel as the primary sliding interface. The PTFE sections were modeled after the existing AASHTO LRFD specification and the NCHRP report No. 10-20A. Although service contact stresses for PTFE are the same as those defined by AASHTO LRFD specification, they may be exceeded when demonstrated by test on full size bearings and wear test as for other sliding interfaces requirements.

REQUIRED TESTS

All isolation system shall have their seismic performance verified by testing. In general, the new Guide Specification has defined three types of tests to be performed on isolation systems: system characteristic tests, prototype tests, and production quality control tests.

System Characteristic Tests

Before any isolation systems can be considered as an option for isolation design, it should be perform system characteristic tests to demonstrate the overall stability and performance under different environmental conditions. Therefore, these tests include both component tests of individual isolators and shake table tests of complete isolation system.

These tests are not project specific and are only need to perform once. They are normally conducted as a new isolation system. For system with similar material and performance, there is no need to repeat the shake table tests. For example, a new high damping rubber
bearing or lead rubber bearing does not require to repeat shake table tests since similar bearing systems already demonstrated the system stability performance.

Prototype Tests

The prototype tests have been expanded to a total of six tests with the test 2 and test 6 are identical. These tests are used to verify system performance under local thermal loading, non-seismic live load response, seismic performance, displacement dependent performance, survivability, and non-seismic performance after earthquake. These tests are:

1. Thermal – three fully reversed cycles of loads at a lateral displacement corresponding to the maximum thermal displacement.

2. Wind and Braking – Twenty fully reversed cycles between limits of plus and minus the maximum non-seismic live load caused by wind or braking at a duration not less than 40 seconds. After the cyclic testing, the maximum load will be held for one minute.

3. Seismic – Three fully reversed cycles of loading at each of the following multiples of the Design Displacement: 1.0, 0.25, 0.50, 0.75, 1.0 and 1.25 in the sequence shown at a period equal to the isolation system.

4. Seismic – 15 (Si/B) fully reversed cycles, not to exceed 25 but not less than 10, of loadings at 1.0 times the Design Displacement. The test shall started from an offset displacement equal to the thermal displacement.

5. Wind and Braking – Repeat test 2 to verify service load performance after a seismic event.


7. Supplemental Tests – Under special situation, more tests may be required to meet the project specific requirement. These tests may include stability test, wear and fatigue tests, and low temperature tests.

Production Quality Control Tests

Unlike isolators used in a building project, production quality control tests shall be conducted on every isolator bearings. Bearings used for bridge isolation project are generally large with limited quantity. A large variation between bearings can cause a great deal difference in system performance. To ensure all bearings within certain performance range, quality control tests on every bearing are needed.
CONCLUSION

An updated AASHTO Guide Specification for Seismic Isolation Design has been developed by the T-3 Task Group. Although there are significant changes and improvements reflect the current state of knowledge development, it is far from prefect and will continue to be refined as with any code document. The contribution of all the members of the T-3 task group is gratefully acknowledged.

REFERENCES

SEISMIC ACCELERATION COEFFICIENTS FOR WEST TENNESSEE

Shahram Pezeshk, and Lijun Liu
Department of Civil Engineering
The University of Memphis
Memphis, Tennessee 38152

INTRODUCTION

The objectives of this study are to generate bedrock motions for West Tennessee and to estimate surface ground motion parameters from the bedrock motions, using wave propagation concepts. This approach is appropriate because the geologic features of the area include a deep soil layer owing to the presence of the Mississippi Embayment. The results of the project are a set of maps indicating the spatial variation of peak ground acceleration (PGA) and acceleration response spectrum values at certain frequencies for earthquake events with return periods of 500 and 2500 years. The information generated is suited specifically to the design and hazard assessment of bridges, buildings, and other structures.

This research study is divided into three major parts. The first part is devoted to the estimation of bedrock accelerations. The second part is to perform ten seismic downhole experiments to determine soil dynamic properties of the top 200 feet of various soil formations in West Tennessee. The last part of the project is to use the information obtained in first two parts to convert the bedrock accelerations to surface accelerations.

BEDROCK SEISMIC HAZARD ANALYSIS FOR WEST TENNESSEE

Probabilistic seismic hazard analysis (PSHA) quantifies the ground motion amplitude for a site or an area for a specific probability of exceedence or evaluates the probability given an amplitude due to influences of seismic sources around the site. To quantify the bedrock seismic hazard, we must identify seismic source zones and their seismicity parameters, and determine ground motion attenuation functions.

Seismic Source Zones and Their Seismicity Parameters

The seismic hazard analysis is sensitive to the determination of seismic source zones and their seismicity parameters. In the central United States, there are many uncertainties in the delineation of seismic source zones and the determination of seismicity parameters because of low seismic activities and lack of complete knowledge about causes of earthquakes. To consider uncertainties
of seismic source zones and seismicities of each zone, in this study, we estimate the seismic hazard for West Tennessee on the basis of different experts' opinion about the seismic source zone and seismicity parameters. Consequently, three seismic source zone sets and corresponding parameters are used in this study.

**First Set of Seismic Source Zones**

The first set of seismic source zones (Case One) is modified from the seismic source zone developed for the U.S. national seismic zonation maps (Hanson and Perkins, 1995). Parameters of seismicity for each seismic zone are given in Table 1. In this table, the magnitude is the intensity magnitude. However, the magnitude used in the ground motion attenuation function is $L_g$ wave magnitude $m_{Lg}$. Therefore, we must convert the intensity magnitude to $m_{Lg}$.

In this study, we first calculated the epicentral intensity from intensity magnitudes given in Table 1, then we related the $m_{Lg}$ value to the epicentral intensity using seismic moment relationships of epicentral intensity and $m_{Lg}$. According to Hanson and Perkins (1995), the intensity magnitude in Table 1 is determined by the relation

$$M_I = 1.3 + 0.6 \; I_0$$

(1)

where $M_I$ is the intensity magnitude, and $I_0$ is the epicentral intensity (Modified Mercalli scale).

<table>
<thead>
<tr>
<th>Zone</th>
<th>$M_{I_{max}}$</th>
<th>$a$</th>
<th>$b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>78</td>
<td>6.4</td>
<td>1.763</td>
<td>.902</td>
</tr>
<tr>
<td>79</td>
<td>6.4</td>
<td>2.361</td>
<td>.901</td>
</tr>
<tr>
<td>80</td>
<td>6.4</td>
<td>2.348</td>
<td>.901</td>
</tr>
<tr>
<td>81</td>
<td>6.4</td>
<td>2.181</td>
<td>.901</td>
</tr>
<tr>
<td>82</td>
<td>6.4</td>
<td>2.422</td>
<td>.901</td>
</tr>
<tr>
<td>83</td>
<td>6.4</td>
<td>1.880</td>
<td>.904</td>
</tr>
<tr>
<td>84</td>
<td>6.4</td>
<td>2.486</td>
<td>.901</td>
</tr>
<tr>
<td>85</td>
<td>6.4</td>
<td>2.459</td>
<td>.903</td>
</tr>
<tr>
<td>86</td>
<td>6.4</td>
<td>2.542</td>
<td>.903</td>
</tr>
<tr>
<td>87</td>
<td>8.8</td>
<td>3.349</td>
<td>.902</td>
</tr>
<tr>
<td>88</td>
<td>6.4</td>
<td>2.861</td>
<td>.902</td>
</tr>
<tr>
<td>89</td>
<td>6.4</td>
<td>3.070</td>
<td>.902</td>
</tr>
<tr>
<td>93</td>
<td>6.4</td>
<td>0.919</td>
<td>.901</td>
</tr>
<tr>
<td>98</td>
<td>6.4</td>
<td>1.947</td>
<td>.899</td>
</tr>
<tr>
<td>100</td>
<td>7.6</td>
<td>1.358</td>
<td>.882</td>
</tr>
</tbody>
</table>
To calculate the seismic moment from the epicentral intensity, we used the relationship developed by Johnston (1996b), which is based on the observation of earthquakes of stable continental regions:

\[
\log(M_o) = 19.26 + 0.481 \ I_{\text{Max}} + 0.0244 \ I_{\text{Max}}^2
\]  

(2)

where \( I_{\text{max}} \) is the maximum intensity of an earthquake. In equation (2), we used \( I_{\text{max}} \) substituting the \( I_0 \) in equation (1). There may be some differences between epicentral intensity \( I_0 \) and maximum intensity \( I_{\text{Max}} \); however, we assumed that \( I_0 \) and \( I_{\text{Max}} \) are the same, which is reasonable for most earthquake damage. From \( I_{\text{Max}} \), we can compute the seismic moment and then convert the seismic moment to magnitude \( m_{Lg} \), using the relationship also developed by Johnston (1996a):

\[
\log(M_o) = 17.76 + 0.360 \ m_{Lg} + 0.140 \ m_{Lg}^2
\]  

(3)

Using equations (1), (2), and (3), we can convert the intensity magnitude in Table 1 to \( m_{Lg} \). Table 2 gives results of \( m_{Lg} \).

**Table 2. Conversion of \( m_{Lg} \) from Intensity Magnitude**

<table>
<thead>
<tr>
<th>( M_I )</th>
<th>( 6.4 )</th>
<th>( 7.6 )</th>
<th>( 8.8 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( m_{Lg} )</td>
<td>( 6.2 )</td>
<td>( 7.0 )</td>
<td>( 7.6 )</td>
</tr>
</tbody>
</table>

**Second Set of Seismic Source Zones**

The second set of seismic source zones (Case Two) combine the seismic source zones from Toro et al. (1992) and Bollinger et al. (1989). In Toro's seismic source zones, the influence of eastern Tennessee seismic zone was not considered. However, this zone may affect the seismic hazard analysis for West Tennessee, particularly in the case of sites close to the Tennessee River. Hence, the East Tennessee zone modified from Bollinger et al. (1992) is used to consider the southern Appalachian seismic source zone. Parameters of seismicity for these source zones are given in Equation (4) and Table 3 and are taken from Toro et al. (1992):

\[
N(m_{Lg} > m) = \begin{cases} 
0.05 \left[ \frac{1 - 10^{-0.84(m-5)}}{1 - 10^{0.84(7-5)}} \right] + \frac{1}{600} & 5 < m < 7 \\
\frac{1}{600} \left[ 1 - \frac{m - 7}{m_{Lg,max} - 7} \right] & 7 < m \leq m_{Lg,max}
\end{cases}
\]  

(4)

In Table 3, the probability corresponding to each magnitude and occurrence rate is taken from Toro et al. (1992). The parameter \( \nu \) (m > 5/yr) in Table 3 is the annual occurrence rate of earthquakes with magnitude larger than 5. The annual occurrence rate of earthquakes for the eastern Tennessee seismic source zone is taken from the mean value given by Bollinger et al. (1989):
\[ \log[N(m_{Lg})] = 2.753 - 0.900 \, m_{Lg} \quad (5) \]

**Third Set of Seismic Source Zones**

The third set of seismic source zones (Case Three) is a combination of seismic source zones from Toro et al. (1992) and EPRI (1986). Seismic source zones from EPRI (1986) are used to consider seismicity in the East Tennessee. Seismicity parameters for these seismic source zones are given in Table 4. In this table, different alternatives and their probabilities or weighted values are also given for maximum magnitude and occurrence rate.

**Ground Motion Attenuation**

For strong seismicity areas, the ground motion attenuation can be estimated from the ground motion recordings of strong earthquakes. However, it is impossible to estimate the attenuation function from recording data for the New Madrid area due to low seismicity. To overcome the difficulty, we estimated the attenuation function for the New Madrid area based on the generated ground motion. To simulate ground motion, we used the time domain synthetic program developed by Boore (1996). Input parameters used in this study are summarized in Table 5 for the seismic source and wave transmission path.

<table>
<thead>
<tr>
<th>Zone</th>
<th>( m_{Lg,max} / \text{Prob.} )</th>
<th>( a {v(m &gt; 5/yr)} )</th>
<th>( b / \text{prob.} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>NMSZ</td>
<td>7.2 / 0.2</td>
<td>Equation (4) / 0.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7.5 / 0.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>7.8 / 0.2</td>
<td>0.050</td>
<td>0.84 / 0.5</td>
</tr>
<tr>
<td>ARKANSAS</td>
<td>5.4 / 0.2</td>
<td>0.001</td>
<td>0.96 / 0.5</td>
</tr>
<tr>
<td></td>
<td>6.6 / 0.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>7.2 / 0.2</td>
<td>0.013</td>
<td>0.87 / 0.5</td>
</tr>
<tr>
<td>OZARKS</td>
<td>5.3 / 0.2</td>
<td>0.004</td>
<td>0.98 / 0.5</td>
</tr>
<tr>
<td></td>
<td>6.0 / 0.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>7.2 / 0.2</td>
<td>0.033</td>
<td>0.92 / 0.5</td>
</tr>
<tr>
<td>W. TENN.</td>
<td>5.2 / 0.2</td>
<td>0.001</td>
<td>0.98 / 0.5</td>
</tr>
<tr>
<td></td>
<td>6.0 / 0.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.9 / 0.2</td>
<td>0.013</td>
<td>0.88 / 0.5</td>
</tr>
</tbody>
</table>

Table 3. Parameters of Seismicity for Seismic Source Zones: Case Two
Table 4. Parameters of Seismicity for Seismic Source Zones: Case Three (EPRI, 1986)

<table>
<thead>
<tr>
<th>Zone</th>
<th>$m_{Lg}$ max / Probability (%)</th>
<th>$a$ /Prob. (%)</th>
<th>$b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>6.5 / 15 6.3 / 55 5.2 / 30</td>
<td>4.372</td>
<td>0.960</td>
</tr>
<tr>
<td>6</td>
<td>6.5 / 15 6.3 / 55 5.2 / 30</td>
<td>3.775</td>
<td>1.030</td>
</tr>
<tr>
<td>8</td>
<td>6.8 / 25 6.5 / 60 5.8 / 15</td>
<td>3.940 / 30</td>
<td>0.810</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.883 / 30</td>
<td>0.890</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.035 / 30</td>
<td>1.040</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.343 / 10</td>
<td>0.900</td>
</tr>
<tr>
<td>9</td>
<td>6.8 / 25 6.5 / 60 5.8 / 15</td>
<td>3.570 / 70</td>
<td>0.890</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.628 / 30</td>
<td>0.810</td>
</tr>
<tr>
<td>13</td>
<td>6.5 / 15 6.3 / 55 5.2 / 30</td>
<td>4.295</td>
<td>0.940</td>
</tr>
<tr>
<td>14</td>
<td>6.5 / 15 6.3 / 55 5.2 / 30</td>
<td>3.699</td>
<td>0.970</td>
</tr>
<tr>
<td>24</td>
<td>7.0 / 20 6.8 / 60 6.6 / 20</td>
<td>4.949</td>
<td>1.020</td>
</tr>
<tr>
<td>25</td>
<td>7.0 / 10 6.8 / 60 6.6 / 30</td>
<td>5.615</td>
<td>1.150</td>
</tr>
<tr>
<td>26</td>
<td>6.8 / 25 6.5 / 60 5.8 / 15</td>
<td>5.030</td>
<td>0.970</td>
</tr>
<tr>
<td>27</td>
<td>6.5 / 15 6.3 / 55 5.2 / 30</td>
<td>4.339</td>
<td>0.930</td>
</tr>
<tr>
<td>28</td>
<td>7.0 / 10 6.8 / 60 6.6 / 30</td>
<td>4.010</td>
<td>0.900</td>
</tr>
<tr>
<td>40</td>
<td>7.0 / 10 6.8 / 60 6.6 / 30</td>
<td>4.876</td>
<td>1.170</td>
</tr>
</tbody>
</table>

Functional Form of Attenuation Equation

Past experience shows that ground motion attenuations are different for intermediate and large earthquakes. Seismic wave attenuations — geometric and anelastic for distances less than 100 km are different from those for distances greater than 100 km in the eastern North America (ENA). To consider all these factors, following EPRI (1993), we used the following formulation for attenuation function:

$$
\ln Y = C_0 + C_1 (m_{Lg} - 6) + C_2 (m_{Lg} - 6)^2 - C_3 \ln R_H - \\
- (C_4 - C_3) \max \left[ \ln \left( \frac{R_H}{100} \right), 0 \right] - C_5 R_H + \varepsilon_{\text{error}}
$$

(6)

and

$$
R_H = \sqrt{R^2 + 10^2}
$$

(7)

where $Y$ is the ground motion characteristic (PGA or Spectral values) in units of cm/s², $C_0$ through $C_5$ are constants to be evaluated by fitting the equation with synthetic values for different magnitudes and distances, $R$ is the epicentral distance, and $\varepsilon_{\text{error}}$ is an error term to consider modeling variability. In Equation (7), the constant 10 is the focal depth in the central United States.
Table 5. Parameters of Seismic Source and Propagation Path

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Wave Velocity</td>
<td>3.6 km/s</td>
</tr>
<tr>
<td>Density</td>
<td>2.75 gm/cm³</td>
</tr>
<tr>
<td>Focal Depth, h</td>
<td>10 km</td>
</tr>
<tr>
<td>Stress Drop</td>
<td>100, 150, and 200 bars</td>
</tr>
<tr>
<td>Source Spectra</td>
<td>Brune's omega square model</td>
</tr>
<tr>
<td>Magnitude</td>
<td>( m_{Lg} )</td>
</tr>
<tr>
<td>Seismic Moment</td>
<td>[ \log(M_0) = 17.76 + 0.360 m_{Lg} + 0.140 (m_{Lg})^2 ]</td>
</tr>
<tr>
<td>Geometric Spreading</td>
<td>( G(R) = \begin{cases} \frac{1}{R} &amp; 1 &lt; R \leq 70 \text{ km} \ \frac{1}{70} &amp; 70 &lt; R &lt; 130 \text{ km} \ \frac{1}{70 \sqrt{\frac{130}{R}}} &amp; R \geq 130 \text{ km} \end{cases} ) (Atkinson and Boore, 1995)</td>
</tr>
<tr>
<td>Quality Factors</td>
<td>( Q(f) = \begin{cases} 680 f^{0.36} &amp; \text{Case A} \ 270 f^{0.87} &amp; R &lt; 100 \text{ km} \ 210 f^{0.78} &amp; R \geq 100 \text{ km} \end{cases} ) (Dwyer et al., 1984)</td>
</tr>
<tr>
<td>Source Scaling Parameter</td>
<td>( C = \frac{R_{\phi\phi} \times FS \times PRTITN}{4\pi\rho_0^3} )</td>
</tr>
</tbody>
</table>

where \( R_{\phi\phi} \) is the average radiation pattern (\( R_{\phi\phi} = 0.55 \)), FS is the free-surface amplification (\( FS = 2.0 \)), and \( PRTITN \) is the fraction of energy of the seismic wave partition onto two horizontal components (\( PRTITN = 0.71 \)).

Comparing the generated ground motion, especially spectral values at period of 1.0 second, shows that there are obvious differences between these two quality factors at distances greater than 100 km. Thus, the constants of attenuation equations are determined independently for each quality factor. Tables 6 and 7 give the constants for these two cases.

Table 6. Constants of Attenuation Equations for Case A

<table>
<thead>
<tr>
<th>Period (Sec.)</th>
<th>( C_0 )</th>
<th>( C_1 )</th>
<th>( C_2 )</th>
<th>( C_3 )</th>
<th>( C_4 )</th>
<th>( C_5 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>9.1109</td>
<td>1.5198</td>
<td>-0.1205</td>
<td>1.0505</td>
<td>0.1560</td>
<td>0.0049</td>
</tr>
<tr>
<td>1.0</td>
<td>7.3518</td>
<td>2.2258</td>
<td>-0.3588</td>
<td>0.9804</td>
<td>0.0573</td>
<td>0.0031</td>
</tr>
<tr>
<td>PGA</td>
<td>9.6808</td>
<td>1.4026</td>
<td>-0.0588</td>
<td>1.3357</td>
<td>0.6956</td>
<td>0.003</td>
</tr>
</tbody>
</table>
Table 7. Constants of Attenuation Equations for Case B

<table>
<thead>
<tr>
<th>Period (Sec.)</th>
<th>$C_0$</th>
<th>$C_1$</th>
<th>$C_2$</th>
<th>$C_3$</th>
<th>$C_4$</th>
<th>$C_5$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>9.0972</td>
<td>1.5151</td>
<td>-0.1181</td>
<td>1.0394</td>
<td>0.4868</td>
<td>0.0063</td>
</tr>
<tr>
<td>1.0</td>
<td>7.3584</td>
<td>2.2229</td>
<td>-0.3549</td>
<td>0.9794</td>
<td>0.2416</td>
<td>0.0053</td>
</tr>
<tr>
<td>PGA</td>
<td>9.2752</td>
<td>1.3602</td>
<td>-0.0387</td>
<td>1.1821</td>
<td>0.9534</td>
<td>0.0055</td>
</tr>
</tbody>
</table>

**Variabilities of Attenuation**

Conceptually, variability can be separated into two parts: randomness and uncertainty. "Randomness represents variability that is inherent to the parameter; uncertainty represents variability that is due to our lack of knowledge of the parameter. Therefore, randomness can be refined but not reduced, whereas uncertainty may be reduced in the future with additional information" (p.9-1, EPRI, 1993). Thus, in equation (6), the error term $\varepsilon_{\text{error}}$ stands for the variability of both randomness and uncertainty.

There cannot be unanimous opinion about the estimation of variability. According to Abrahamson et al. (1997), the total standard error is usually from 0.4 to 0.7 natural log units for PGA, and 0.4 to 0.75 natural log unit for spectral acceleration of periods 0.01 to 10 seconds. In the national seismic hazard analysis, Frankel et al. (1996) used values of 0.75, 0.75, 0.75, and 0.80 natural log units as standard deviations of PGA, 0.2-, 0.3-, and 1.0-second spectral acceleration, respectively, for attenuation in the central and eastern United States. In this study, we used a value of 0.75 natural log units as the standard deviation for PGA and spectral acceleration.

**Comparisons with Current East North America Ground Motion Attenuations**

The most recently developed attenuation relationships are those of Atkinson and Boore (1995) and Toro et al. (1997). Atkinson and Boore (1995) used an empirical source model while Toro et al. (1997) used Brune’s point source model. Toro et al. (1997) also considered parametric uncertainties in stress parameters, in near site attenuation factor $\alpha$, and in quality factor $Q(f)$. We will compare our results with these two attenuation relationships because they are widely used in the central and eastern United States.

In this study, we used $L_g$ magnitude $m_{L_g}$ as the magnitude measure. Atkinson and Boore (1995) used moment magnitude. Thus, we convert the magnitude $m_{L_g}$ to the moment magnitude $M$ based on seismic moment using relationships developed by Kanamori (1979) and Johnston (1996a). Figures 1, 2, and 3 show comparisons for the PGA and spectral acceleration at periods of 0.2 and 1.0 seconds and a 5% damping ratio.
Figure 1. *Comparison of PGA Attenuations for the Central and Eastern United States*

It is clear that, for a moderate magnitude, the PGAs are compatible for all relationships at distances less than one hundred kilometers. At distances greater than one hundred km, our case B attenuates faster than others. For large magnitude ($m_{Lg} \approx 7$), PGAs predicted by our relationships are higher than those predicted by Toro et al. (1997) and Atkinson and Boore (1995). For a 5% damped spectral acceleration at one second, our results are very close to those of Toro et al. (1997). However, for the entire distance range, Atkinson and Boore’s (1995) results are much lower than others.

**Seismic Hazard Estimation for West Tennessee**

To quantify the bedrock seismic hazard, we developed a program entitled PSHA (Probabilistic Seismic Hazard Analysis). In the calculation, we first grid the entire West Tennessee area with an interval of 0.1° for longitude and latitude and compute the seismic hazard at the grid points. For each seismic source zone, we first evaluated the seismic hazards for all alternatives. The seismic hazard of each zone is then obtained by weighted summation according to the weight value assigned to each parameter. Then, the total seismic hazard for each set of seismic source zones can be computed by the simple summation of values of all zones in that set. At the same time, we have six
combinations of seismic source zones and attenuation functions—three set of seismic source zones and two attenuation functions. Thus, we also need to compute the seismic hazard for all the combinations.

Seismic Hazard Contour Maps at Bedrock

In this study, we need to estimate the seismic hazard for three ground motion characteristics: PGA, spectral acceleration at periods of 0.2 seconds and 1.0 seconds. We also computed the seismic hazard for two probabilities which correspond to return periods of 500 and 2500 years. Therefore, we generated a total of 36 contour maps for three sets of seismic source zones, two ground motion attenuation functions, three characteristics of ground motion, and two return periods. Figure 4 shows a typical contour map for 500 year return period. From these contour maps, it is clear that the seismic hazard comes mainly from the New Madrid seismic zone (NMSZ) because most earthquakes are concentrate on the NMSZ.
SEISMIC DOWNHOLE SURVEY

The second part of this study was focused on selecting ten borehole sites to conduct downhole seismic tests. The purpose of the testing was to determine dynamic soil properties of the varying geological compositions in West Tennessee from a depth of 200 feet to the surface. How these ten sites were selected and a detailed discussion of the West Tennessee geological condition is summarized in detail in Pezeshk et al. (1998). Pezeshk et al. (1998) also provide information on how seismic downhole experiments were performed on these ten sites in West Tennessee and presents the findings of these tests. In addition, the data collected is used to correlate blow counts (SPT) with the shear wave velocity and the geology of the site and to correlate the field shear-wave velocities with laboratory resonant column shear-wave velocities. In addition to the ten sites mentioned above, approximately 1,600 soil boring logs were collected and to be used for site characterization of West Tennessee. We used the computer program ACCESS to store these 1,600 shallow soil boring logs. In addition, Pezeshk et al. (1998) provide information on shear wave profiles in soil layers below 200 feet to the Paleozoic bedrock.
Figure 4. Contour Map of Peak Ground Acceleration for Return Period = 500 Years for Zone Case Two and Attenuation Case A

SOIL EFFECT

The third part of the work concerns propagating the bedrock motions to the surface through the layered soil deposits. The computer program SHAKE is used to generate these motions. The primary information required by this one-dimensional wave propagation program are the shear wave velocities of the soil layers. Shear-wave velocities for the input are obtained from downhole seismic testings and from the blow counts of the 1,600 soil borings (see Pezeshk et al., 1998). A detailed procedure and verification for converting blow counts to shear-wave velocity can be found in Pezeshk et al. (1996) and Wei et al. (1996). The shear-wave velocities at various depths (between 100 to 200 ft) at each site are tied to 14 borings in the vicinity of the site, which provide a great deal of information on the soil profile. Acceleration coefficients determined at ground level can be improved with the availability of more data. Another important point to note is that the soil profile in West Tennessee changes dramatically from one site to another for locations as close as a couple hundred feet apart. Therefore, contour maps of acceleration coefficients at ground surface provided in this study should be used with care or as a qualitative measure unless the site under consideration
falls directly on a soil boring location. To perform site specific analysis, one needs to use the actual soil boring at that site.

Pezeshk et al. (1998) provide the contour maps of the acceleration coefficients at the surface in the varying soil types common to West Tennessee. These acceleration coefficients may be used in seismic design and damage assessment of bridges. As an example, Figure 5 shows peak ground acceleration contour maps for 500 year return period at bedrock, at a depth of 200 ft, and at the ground surface.

**SUMMARY**

This paper is a summary of a recent research project funded by the Tennessee Department of Transportation to develop maps of acceleration coefficients at 200 ft and at the ground surface in the varying soil types common to West Tennessee. As the result of this project, the seismic hazard in West Tennessee has been quantified in series of contour maps for (1) peak ground acceleration, (2) horizontal spectral acceleration at 1 second, and (3) horizontal acceleration at 0.2 seconds at bedrock, a depth of 200 ft, and ground surface for 500 and 2500 year return periods.

**REFERENCES**


Figure 5. Contour Map of Peak Ground Acceleration Spectrum at (a) Bedrock, (b) at a Depth of 200 ft, and (c) at Ground Surface — 500 Year Return Period (g) for West Tennessee — West of Tennessee River.


FHWA SEISMIC RETROFITTING MANUALS

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Abstract

Under the sponsorship of the U.S. Federal Highway Administration, the Multidisciplinary Center for Earthquake Engineering Research (MCEER) [previous called National Center for Earthquake Engineering Research (NCEER)] is developing a set of guidelines for conducting seismic hazard evaluations and retrofitting of complete highway systems. These guidelines will provide assistance in performing seismic vulnerability screening, evaluation and retrofitting of all major highway system structural components (bridges, retaining structures, slopes, tunnels, culverts, and pavements) and in determining the impacts of scenario earthquakes on highway system performance (traffic flows, economic consequences, etc.). The guidelines will be contained in the 3-volume “Seismic Retrofitting Manuals for Highway Systems,” in which Volume I will contain procedures for conducting a seismic risk assessment of highway systems and regional networks, Volume II will contain procedures for seismic screening, evaluation and retrofitting of highway bridges, and Volume III will contain similar procedures for retaining structures, slopes, tunnels, culverts and pavements.

This paper provides an overview of the coverage for each of the three volumes. In addition, the paper presents an example application of the seismic risk analysis methodology and it discusses advancements being incorporated into the procedures for seismic screening, evaluation, and retrofitting of highway bridges (Volume II).

INTRODUCTION

In the fall of 1992, the Multidisciplinary Center for Earthquake Engineering Research (MCEER) (previous called National Center for Earthquake Engineering Research (NCEER)) initiated work on a comprehensive research program to develop tools to evaluate the seismic vulnerability of the national highway system in the United States. This program is sponsored by the Federal Highway Administration (FHWA) of the U.S. Department of Transportation. The research program includes a series of tasks which will result in an improved understanding of the seismic hazard in the eastern and central U.S., the behavior and response of soils, foundations and highway structures during earthquakes, and the overall impacts on highway systems resulting from earthquake damage. In addition, the program is developing and improving appropriate retrofit technologies for those highway system components deemed vulnerable to earthquake damage.

An important end-product of this program is the development of a seismic evaluation and retrofitting manual for existing highway systems which will have national applicability. This manual is being prepared in three volumes, where:

• Volume I contains the methodologies, procedures, and examples for conducting a seismic risk assessment of highway networks and systems;
• Volume II contains guidance on seismic vulnerability screening, analysis, and retrofitting of highway bridges; and
• Volume III contains guidance on screening, analysis, and retrofitting of other major highway system components, including retaining structures, slopes, tunnels, culverts, and pavements.

As a first step towards the completion of each of these volumes, a "strawman" was prepared in late 1996 for Volumes I (system risk assessment) and III (evaluation and retrofitting of components other than bridges). Each strawman:

• summarized the current state-of-practice in highway system assessment, component evaluation, and retrofitting;
• incorporated current research results; and
• identified important gaps in knowledge which required resolution prior to completion of each volume.

The 1995 FHWA Seismic Retrofitting Manual for Highway Bridges (FHWA, 1995), which was also prepared by NCEER, serves as the "strawman" for Volume II. Work on the final versions of each volume commenced in late 1997; drafts will be completed near the end of 1998 at which time they will be distributed to select researchers and practitioners for detailed technical reviews. Following final revisions and edits, it is anticipated that each of these three volumes will be published and available for distribution by the FHWA in the year 2000.

This paper provides an overview on the intent and coverage of each volume of the retrofitting manual and also presents the results of an example seismic risk assessment which was performed on the highway network around the city of Memphis, Tennessee based on the Volume I strawman. It should be noted that the coverage within Volume II will be greatly expanded over that currently contained in the 1995 FHWA Seismic Retrofitting Manual for Highway Bridges.

SEISMIC RISK ASSESSMENT OF HIGHWAY SYSTEMS

Past experience has shown that the direct impacts of earthquake damage to highway components (e.g., bridges, retaining structures, and tunnels) is both a life-safety issue and a repair and replacement cost issue. Furthermore, the indirect impacts can be almost as severe due to closed routes, disrupted access and delayed post-earthquake emergency response, repair, and reconstruction operations.

The extent of these impacts depends not only on the seismic performance characteristics of the individual components, but also on the characteristics of the highway system that contains these components. For example, studies of highway systems in the San Francisco Bay area after the 1989 Loma Prieta earthquake have shown that post-earthquake traffic flows strongly depend on (Hobetka et. al., 1991; Wakabayashi and Kameda, 1992):

• the configuration of the highway system;

• the locations of the individual components within these links and within the overall system; and

• the locations, redundancy, and traffic capacities and volumes of the links between key origins and destinations within the system.

When one considers these system characteristics, it is evident that earthquake damage to certain components (e.g., those along important and nonredundant links within the system) will have a greater impact on overall system performance than will other components. Currently, such system issues are not considered when specifying seismic performance requirements and design or strengthening criteria for new or existing components, due primarily to the lack of adequate systems-based evaluation tools. Instead, each component type is usually evaluated individually, with screening and ranking criteria applied specifically to the inventory of structures of the same type. For example, formal screening and ranking guidelines for highway bridges have existed since the early 1980s (FHWA, 1983). However, this procedure treats each individual component independently, without regard to how the extent of its damage may affect overall highway performance.
Consideration of each component's importance to system performance can provide a much more rational basis for:

- establishing seismic strengthening priorities;
- defining seismic design and strengthening criteria; and
- estimating economic impacts due to component damage.

It is important to recognize that system performance issues are important to all regions of a country that are at risk due to earthquakes. As a result, system issues are now being incorporated into newly developed methods for prioritizing bridges for seismic retrofit (Basüz and Kiremidjian, 1995; Moore et al., 1995).

Volume I of the NCEER Highway Project "Seismic Retrofitting Manuals for Highway Systems" contains the procedures for conducting a seismic risk assessment (SRA) of highway networks and systems (Werner et al., 1996). The procedures contained in this volume provide a basis for addressing these seismic performance issues and incorporate data and methodologies pertaining to engineering issues (structural, geotechnical, and transportation), repair and reconstruction, system network and risk analysis, and socio-economic considerations for impacts from system damage. They also provide a mechanism to estimate system-wide direct losses (i.e., costs for repair of damaged components) and indirect losses due to reduced traffic flows and/or increased travel times (economic impacts).

Volume I provides:

- a detailed framework for carrying out deterministic and probabilistic evaluations of seismic risks to highway system;
- a discussion on the types of data needed to characterize the system, hazards, and components, together with the form of structural, geotechnical, and transportation engineering analysis results needed for these characterizations;
- a procedure for rapid analysis of post-earthquake traffic flows based on artificial intelligence concepts and the current state of knowledge for traffic flow modeling; and
- a socio-economic module for characterizing economic, emergency response, and societal impacts of earthquake damage to highway systems.

Key to the SRA procedures are four GIS-based modules that act as pre-processors to the procedure in order to model the system, hazards, components, and socio-economics for the system. This modeling comprises the bulk of the effort in the application of the risk analysis procedure.

The volume is organized into eight chapters. It provides the background and an overview of the methodology and procedures, and details the four principal modules comprising the SRA:

- The system module, which contains system and inventory data, traffic management measures, and system analysis procedures;
- The hazards module, which contains the earthquake ground motions, geologic hazard evaluation, liquefaction, landslides, and surface fault rupture information;
- The component module, which contains the overall model development including loss and functionality models, seismic response evaluation, and repair and reconstruction procedures; and
• The socio-economic module, which contains the models and local or regional demographic and economic data needed to estimate the socio-economic impacts due to reductions in traffic flows resulting from earthquake damage.

In addition, the volume contains examples demonstrating the application and interpretation of the results of the SRA procedure.

**SRA Analysis Procedure**

The analysis procedure consists of four primary steps:

1. Initialization of the analysis and modules.
2. Development of simulations for the various scenario earthquakes.
3. Incrementing the simulations and scenario earthquakes.
4. Aggregating the system analysis results.

Step 1 (initialization of the analysis procedure) is carried out in two parts. First, earthquake source models for the region are used to define an ensemble of scenario earthquakes that could affect the highway system being analyzed. Each earthquake is most commonly defined in terms of its magnitude, location, and frequency of occurrence. In some cases, additional seismo-tectonic parameters, such as orientation of the rupture surface or directivity of the rupture propagation, may also be estimated. Uncertainties in defining the values of the various earthquake inputs are also modeled at this stage. The second part of Step 1 involves identification of an adequate number of simulations for each scenario earthquake. In this procedure, "simulation" corresponds to one estimate of direct losses, traffic flows at various times after the earthquake, and any corresponding socio-economic impacts for the scenario earthquake.

Steps 2 and 3 provide the methodology for determining the $n$th simulation for $i$th scenario earthquake and applies them to each scenario earthquake. This methodology is based on evaluations of the hazard, direct loss and system state, traffic flows, and socio-economic impacts.

Step 4 is carried out after the system analyses for all simulations and scenario earthquakes have been completed. In this step, the results from all simulations and earthquakes are aggregated and displayed. Depending on user needs, these aggregations can focus on the seismic risks associated with the total system or with individual components.

The SRA methodology is being coded for use on standard PC-based workstations, and will run under the Microsoft Windows 95™ operating system. It should be noted that, by its nature, the SRA methodology is extremely data-intensive. A series of default values and options will be built into the methodology and software, but the use of the procedures with these defaults will not provide results that are reliable and sufficiently accurate for engineering applications; it is therefore important to obtain and collect sufficient inventory and hazard data, as described in the manual. Another weakness in the current version of the system is in the calibration of the various models to historical earthquake performance, especially those related to damage and restoration of highway system components other than bridges. These issues continue to be addressed under the NCEER Highway Project but may require additional work in the future.

**SCREENING, EVALUATION, AND RETROFITTING OF HIGHWAY BRIDGES**

In the late 1970s, the Applied Technology Council developed a set guidelines for the seismic retrofitting of highway bridges under FHWA sponsorship. These guidelines were published in 1983 by the FHWA as the *Seismic Retrofitting Guidelines for Highway Bridges* (FHWA, 1983). The guidelines represented what was then the state-of-the-art for screening, evaluating, and retrofitting seismically deficient bridges. At the time the guidelines were issued, experience with highway bridge retrofitting in the U.S. was limited and many of the proposed techniques had not actually been implemented in field applications. In the 15 or so years since, there has been significant progress in understanding the seismic response of bridges and the development of new and improved retrofitting technologies for bridge columns and footings, methods to
stabilize soils to prevent liquefaction, and to ensure adequate connectivity between the bridge superstructure and substructure. Many of these advances in the state-of-the-art and the state-of-practice are the result of an aggressive research program which was started by the California Department of Transportation (Caltrans) following the 1989 Loma Prieta earthquake. Since then, seismic screening, evaluation, and retrofitting procedures for highway bridges have been widely implemented in North America, Asia, and Europe.

In order to capture these advances in seismic retrofitting and to make the current state-of-the-art available to bridge owners and engineers across the U.S., the FHWA initiated a project to update the 1983 guidelines as part of the NCEER Highway Project research program. This effort resulted in a report titled Seismic Retrofitting Manual for Highway Bridges, which was published by the FHWA in 1995 (FHWA, 1995).

The 1995 FHWA manual offers procedures for evaluating and upgrading the seismic resistance of existing highway bridges. Specifically, it contains:

- a preliminary screening process to identify and prioritize bridges that need to be evaluated for seismic retrofitting;
- alternative methodologies for quantitatively evaluating the seismic capacity of an existing bridge and determining the overall effectiveness of alternative seismic retrofitting measures by either the component-based capacity/demand (C/D) approach or the lateral strength ("pushover") methodology; and
- suggested retrofit measures and design requirements for increasing the seismic resistance of existing bridges.

The manual does not prescribe requirements dictating when and how bridges are to be retrofitted – the decision to retrofit is left to the engineer and depends on a number of factors, several of which are outside the scope of the manual. These include, but are not limited to, the availability of funding, and political, social, and other economic considerations. The primary focus of the manual is directed towards providing guidance on the engineering factors for seismic retrofitting.

The 1995 FHWA manual is being updated and significantly expanded, and will be reissued as Volume II of the "Seismic Retrofitting Manuals for Highway Systems" on the basis of additional research and development conducted under the FHWA-sponsored research program at NCEER and by others. At this time, it is anticipated that Volume II will be issued in three parts:

- Volume II-A will contain the screening, evaluation, and retrofitting procedures;
- Volume II-B will contain a series of case-studies demonstrating the application of key parts of various procedures; and
- Volume II-C will contain engineering drawings and details of typical retrofits, and vendor details and applications of seismic protective systems.

Among the major changes that are included in the new bridge evaluation and retrofitting manual are:

- Significantly expanded coverage of seismic hazards. This includes methods for characterizing the seismic hazard, selecting appropriate return periods and parameters, consideration of local site factors and effects, procedures and rules for constructing elastic spectral demand, guidance on developing time histories, and consideration of vertical ground motions and near-field effects.

- Expanded coverage of potential geotechnical hazards and evaluation of geotechnical components. This includes characterization of geotechnical hazards, identification of liquefaction potential and quantification of liquefaction effects in the free-field, settlement of approach slopes due to ground shaking, and concerns with active faults and fault rupturing underneath the structure. In addition, the new bridge manual contains guidance on foundation modeling for soil-structure interaction, and stiffness and capacity evaluation of abutments, footings, pile groups, drilled (pier) shafts, and caissons, along with methods to determine liquefaction-based displacement demands.
• The incorporation of additional methods for bridge system evaluation, including the capacity/demand spectral evaluation (linearized elastic and inelastic R-factor) method, lateral strength (pushover) analysis, and detailed computation methods including nonlinear time history analysis.

• Expanded coverage of widely employed and new seismic retrofit strategies. Among these are strategies related to strengthening, displacement enhancements, and ground remediation, and a discussion on when the engineer should consider either the full-replacement or "do-nothing" option. The engineering and economic evaluation considerations for selection of appropriate strategies are fully described in the manual.

A large number of new and modified retrofit techniques are described in the manual, including many intended for low-to-moderate seismic zones where cost-effectiveness and simplicity may be as important as moderate increases in seismic performance. Among these new retrofits are hinge shifting and fusible hinge techniques for substandard columns, and methods that provide improved performance for existing steel roller and rocker bearings. In addition, the new volume includes expanded coverage of the use of protective systems as part of the arsenal of potential retrofit approaches, including the use of energy dissipation systems, seismic isolation, restrainers, and combinations of these technologies.

• Recommendations concerning post-earthquake emergency assessment and repair. A new chapter has been added to the manual which describes the functions, makeup, and pre-event training and planning for both an emergency damage assessment team and a structural performance investigation team. The emergency damage assessment team is responsible for immediate on-site evaluations of damaged structures to determine if they are capable of safely carrying traffic, while the structural performance investigation team performs on-site evaluations to gather information and data to determine or theorize and document the causes of failure or damage, primary and secondary failure modes and sequences, and the validity of current design practice and codes, relative to observed damage. This chapter also provides guidance in identifying the significance and extent of damage to bridge components commonly put at risk during earthquakes, and in providing suggested temporary shoring and repair strategies.

As noted earlier, the bridge manual is being prepared in three parts. Volume II-A contains the basic guidance and recommended procedures for highway bridge screening, evaluation, and retrofitting, with bridge retrofits discussed in functional and conceptual terms. Volume II-B will contain several case studies and detailed examples demonstrating the application of the procedures to typical bridges representative of the eastern, central and western U.S. Meanwhile, Volume II-C will contain engineering drawings of typical retrofitting details that have been employed by various bridge-owning agencies, vendor details for earthquake protective system component designs and applications, and additional materials that will aid in the understanding and implementation of Volumes II-A and II-B.

SCREENING, EVALUATING, AND RETROFITTING OF RETAINING STRUCTURES, SLOPES, TUNNELS, CULVERTS, AND PAVEMENTS

Current national highway seismic standards in the U.S. are primarily limited to design and retrofitting provisions for highway bridges. However, a typical highway system is composed of a number of major structural and geotechnical components, which include retaining structures, engineered slopes (cuts and fills), tunnels, culverts, and pavements. In addition, a typical highway system also contains other functional and peripheral elements such as sound walls, sign and light structures (towers and sign bridges), and motorist service facilities. While these peripheral elements are widespread, the potential impact on traffic flow from their failure during an earthquake is expected to be limited. However, traffic flow impacts from the failure of the other major structural and geotechnical components during an earthquake could be as severe as that historically demonstrated by the failure of highway bridges. This was evident during the 1995 Hanshin-Awaji earthquake in Japan, which resulted in the failure of numerous retaining structures, rail tunnels, and roadway beds.

As a result, Volume III of the "Seismic Retrofitting Manuals for Highway Systems" focuses on the development of screening, evaluation, and retrofitting methods and technologies for these other major highway system structural components. Over time, individual structures have been evaluated and retrofitted on a case-by-case basis. Much of this
experience, however, is fragmented and not well-documented. This volume is the first known effort to capture the important aspects of screening, evaluation, and retrofitting of these non-bridge highway system structural components and to present results and recommendations in a formal, procedural manner.

Unlike highway bridges, there is no precedent for a manual for screening, evaluation, and retrofitting of highway retaining structures, slopes, tunnels, culverts, and pavements. The first step in this development therefore was the preparation of a "strawman" document. As noted earlier, the strawman summarized the current state of knowledge and practice for these highway system components, incorporated current research results, and identified important gaps in knowledge where additional research was necessary to complete the volume (NCEER, 1996).

Volume III is composed of five sections (one for each highway component covered in the document). Topics covered within each section cover:

- classification of the structural component;
- seismic vulnerability screening;
- detailed structural evaluation; and
- recommended retrofitting concepts.

The volume discusses factors related to performance criteria and the expected level of service or acceptable damage for each component, based on the anticipated level of seismic shaking. Due to limited previous work on some of these highway system components, the overall philosophy of this volume is life-safety oriented with collapse-prevention as the primary criterion.

Retaining Structures

Among the retaining structures covered in Volume III are rigid walls which resist significant deformation during the application of inertia forces and lateral earth pressures, gravity and cantilever walls which resist lateral earth pressures due to the mass of the wall and encaptured soil, mechanically-stabilized earth walls which are now being used widely throughout the U.S., and tied-back walls. The screening process for retaining structures requires a minimum amount of information including location, site conditions, wall height, special wall features, and the proximity of nearby structures. Most of this information is readily obtained from existing records, soil and geologic maps, and via site visits; it is important to recognize, however, that there is no national inventory of highway system retaining structures, and it is also unlikely that formal State-maintained inventories exist widely.

Detailed evaluations of retaining structures are accomplished by a combination of theory, equations, and analytical methods. Walls that fail via translation can be evaluated with relatively simple seismic analysis procedures which estimate induced permanent displacements. Evaluations of retaining walls that undergo tilting or mixed modes of failure require more computational effort, and computer programs may be required to perform numerical integration to solve the coupled equations of motion.

Retrofitting strategies discussed in Volume III for retaining walls include:

- tie backs, with grouted or expanding anchors;
- increasing footing widths;
- adding passive restraint by burying the toe of the wall;
- adding mini-piles and soil nails; and
• soil remediation and strengthening.

The volume provides guidance on various evaluation and retrofitting options. It is noted that the evaluation itself often suggests the appropriate retrofitting strategy, since not only is the critical acceleration predicted by the limit state analysis, but also the expected mode of failure for the wall.

Slopes

The section on slopes is concerned with the effects of instability on slopes, rockfall, and landslides, and is applicable to all types of highway cuts and fills. For purposes of screening and evaluation, classification is based on observed performance during past earthquakes by Keefer (1984) and Keefer and Wilson (1989). This provides for a three-level classification scheme, as follows:

• Category I landslides include rock and soil falls, slides, and avalanches, and are generally widespread. These are characterized by highly disrupted masses and generally shallow failure surfaces, and are fast moving and dislodged from steep slopes.

• Category II landslides include rock and soil slumps and block slides, and slow earth flows. These are coherent sliding masses that move on bounding basal shear surfaces. They are slow moving and located on generally deep-seated failure surfaces, occurring mainly on relatively sleep slopes.

• Category III landslides include soil lateral spread and rapid soil flows. These include a significant component of fluid flow, are fast moving, and usually occur on gentle to moderately steep slopes. They are typically initiated as a result of soil liquefaction in saturated cohesionless deposits and man-made fills.

Among the analysis procedures discussed in the volume are pseudo-static procedures and deformation procedures. Pseudo-static analysis can be used in initial evaluations and may suffice for many slope analyses; deformation procedures may be required for slopes with special characteristics or in regions of high seismicity. Retrofitting of slopes is recognized as a potentially costly and time-consuming activity. The manual therefore provides suggestions for reducing the driving forces which can detrimentally affect the performance of slopes during earthquakes, and providing increases in the resisting forces by applying resisting forces at the toe of a landslide, or increasing the internal strength of soils in the failure zone. For rock slopes, potential retrofits include rock reinforcement, rock removal, and the addition of protective measures against rockfall, such as catchment ditches, fences and barriers, and rock sheds.

Tunnels

Until recently, tunnels have performed quite well during earthquakes resulting in limited concern for them. This performance has been called into question, however, as during the 1995 Hanshin-Awaji earthquake several cut-and-cover rail and transit tunnels collapsed or were widely damaged, and the lining in at least one bored tunnel was damaged. Based on this recent performance and available literature, Volume III provides specific guidance for identifying areas of tunnel vulnerability based on their different design features and construction methods.

Bored tunnels are constructed by excavating the opening and constructing the support system below ground, and they may be constructed in a variety of geologic environments, ranging from hard rock to soft soils. Construction techniques involve the use of tunneling machines or other underground excavation methods such as drilling and blasting. The lining or support system is usually constructed as the tunnel is excavated. The cross-section shape of these tunnels is usually circular or semi-circular. Bored tunnels are constructed in soft ground by either a one- or two-pass lining method, while tunnels bored in rock use methods of support that vary from none (in massive hard rock) to the installation of temporary ribs and lagging liners, followed by cast-in-place final concrete liners.
Cut-and-cover tunnels are constructed by excavating the opening from the ground surface, constructing the tunnel structure within the excavated opening, and backfilling above the top of the structure and adjacent to the sides of it, if it has not been constructed immediately against the excavation face. This method of construction is most often used for shallow tunnels. The structure shape in this case is typically rectangular and is constructed of reinforced concrete. Screening, evaluation, and retrofitting of these types of tunnels reflects these different excavation and construction methods, and construction materials. For tunnels identified by the screening process as having such a risk, further evaluation is then required to assess the degree of deficiencies within the tunnel system unless there is an obvious need of retrofitting. The decision as to whether to conduct further evaluations and retrofit design and the priority for such actions will depend on the importance of the tunnel as established by the owning agency.

In terms of screening, the volume identifies a number of factors which could influence seismic performance, including:

- **seismic hazard** which considers the intensity of ground shaking, fault rupture, landsliding, and soil liquefaction;
- **geological conditions** which considers soft soils, rocks with weak planes intersecting a tunnel, failures encountered during tunnel construction that may have weakened geologic formations, and squeezing ground; and
- **tunnel design, construction, and condition** and whether seismic loadings were considered in the design, the nature of the tunnel lining and support system, the history of static tunnel performance (failures, cracking, or distortion), and the current condition of the lining and support system.

Evaluation procedures are provided for vibratory ground shaking, fault rupture displacement, landsliding, and liquefaction. Analytical procedures include closed-form equations and numerical analysis techniques, depending on the type and form of the tunnel. Also, retrofitting strategies are provided for tunnels likely to fail by ground shaking, fault displacement, and landsliding or liquefaction. However, the retrofit measures are dependant on the type of tunnel (bored or cut-and-cover), and on the damage mode identified by the screening and evaluation procedures.

Among the retrofitting strategies discussed are the following:

- **addition of grout behind a concrete lining**, if it is in poor shape or in poor contact with the geologic media behind the lining;
- **strengthening or stiffening** the lining for cases where excessive ovaling and expected failure of the tunnel cross-section may occur;
- **providing additional flexibility and circumferential joints** in the lining for situations where excessive axial or bending stresses are predicted;
- **addition of reinforcement** to existing linings or of steel plate jackets to linings and columns for cut-and-cover tunnels;
- **enlarging the tunnel opening** in situations where large fault displacements are anticipated, or adding articulation of the tunnel liner with ductile joints if the expected displacements are small; and
- **stabilizing the soil** around the tunnel if landslides or liquefaction are expected to affect the tunnel.

### Culverts and Pavements

The performance of culverts and pavements during past earthquakes has generally been good, although some damage has been seen in recent events. Youd (1996) has documented several culvert failures and has concluded that they perform well except in areas affected by foundation failure or that are subject to large lateral or inertial forces. Little information on the performance of pavements has been documented in the literature, but it is known that such failures are usually associated
with sub-base or soil failure, similar to that experienced in slopes and fills which have been affected by liquefaction or that cross faults.

The consequences of failure of these highway structures is not expected to be large, and it is also unlikely that such failures would create long-term restrictions on access to the affected routes. Relatively simple temporary measures have been employed in past earthquakes to make routes accessible to traffic, although speed or weight restrictions may be necessary until permanent repairs can be made to the culvert or pavement. As a result, it is unlikely that a highway agency would spend any significant amount of its limited financial resources on culvert and pavement evaluation and retrofitting. Even so, guidance is provided in Volume III for evaluating culverts and pavements for potential seismic vulnerabilities, and recommendations for reducing such vulnerabilities are provided. It should be remembered, however, that these highway structures are still important in an overall transportation systems-based analysis as discussed under Volume I, and that they could have a direct impact in the short term on system performance and emergency response following an earthquake.

SUMMARY

The development of the "Seismic Retrofitting Manuals for Highway Systems" is a major component of the NCEER research program being conducted for the FHWA. It is expected that drafts of the three volumes will be completed in late 1998 and that the final versions will be available in late 1999 following a detailed period of technical review and revision. These volumes will provide the basis for guidance to agencies embarking on a program to evaluate and reduce the seismic vulnerability of highway systems.

ACKNOWLEDGEMENTS

Volume I of the 3-volume "Seismic Retrofitting Manuals for Highway Systems" is being coordinated and prepared under the direction of Stuart D. Werner, with assistance from Craig E. Taylor, James E. Moore II, Masanobu Shinozuka, and Jon Walton. The primary authors of Volume II are Ian G. Buckle, Ian M. Friedland, John B. Mander, Geoffrey R. Martin, and Richard V. Nutt. Volume III is being coordinated by Maurice S. Power with the assistance of Geoffrey R. Martin, and the primary authors of the volume are Rowland Richards, Kenneth Fishman, Faiz Makdisi, Maurice S. Power, Dario Rosidi, Jon Kaneshiro, Samuel C. Musser, and T. Leslie Youd.

REFERENCES


SESSION 5
BRIDGE MODELING AND ANALYSIS
SEISMICITY, FIELD TESTING, SEISMIC EVALUATION AND RETROFIT OF BRIDGES

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ABSTRACT

This paper presents the background for seismic analysis and evaluation of highway bridges. It discusses the different categories of bridges and the appropriate seismic analysis techniques as outlined in the AASHTO code and in the Seismic Retrofit Manual. Case studies are presented on four long span truss bridges crossing the Ohio River.

INTRODUCTION

The 1989 Loma Prieta earthquake [1] and the 1994 Northridge earthquake [2] brought to the public's attention the seismic risks to bridges and elevated structures. The partial collapse of the San Francisco - Oakland Bay Bridge and the Cypress Viaduct portion of the Interstate 880, not only caused loss of life, but created considerable problems to the transportation infrastructure.

Bridges in need of retrofit must be identified by considering their structural seismic deficiencies and socio-economic aspects in order to prevent bridge failures, and most effectively allocate the limited financial resources. Hence it becomes necessary to identify the appropriate seismic evaluation procedures for different types of bridges. This paper describes the seismic evaluation of bridges, the necessary seismic analysis methods, seismic input, necessity for conducting field testing, etc. A brief discussion is presented on the seismic analyses performed on the following Ohio river bridges: the Brent-Spence bridge on I-75, the US 41 Southbound and Northbound bridges in Henderson County., and the US51 bridge in Ballard County.

SEISMIC EVALUATION OF EXISTING BRIDGES

Seismic design of bridges is governed by AASHTO Standard Specifications [3] and AASHTO - LFRD [4]. Generally, the AASHTO's specifications are intended: (1) to allow the structure to yield during a major earthquake, (2) to produce damage (yielding) only in areas that are accessible (visible) and repairable, and (3) to prevent collapse even during large earthquakes [5].

The seismic rehabilitation of older bridges in regions of high seismicity, which were designed prior to the advent of seismic design codes and have not yet been subjected to a severe earthquake, is a matter of growing concern. Guidelines for the seismic evaluation of existing bridges are presented in the 'Seismic Retrofitting Manual for Highway Bridges' [6].
The retrofitting manual [6] presents two methods, the ‘Capacity/Demand (C/D) Ratio Method’ and the ‘Lateral Strength Method’ for seismic evaluation of bridges requiring a detailed analysis based on their vulnerability rating. The (C/D) ratio method compares the capacities of individual components with the expected demands on the individual components. On the other hand, the Lateral Strength method treats the entire bridge, or its individual segments between expansion joints, as a single structural system. The structural system is then evaluated using an incremental collapse mechanism approach. The C/D ratio method typically results in conservative retrofitting requirements, however it is preferred to other methods because of its simplicity.

LIMITATIONS OF AASHTO SPECIFICATIONS

The specifications given in AASHTO Standard Specifications [3] and AASHTO-LRFD [4] apply to bridges of conventional slab, beam girder, box girder, and truss superstructure construction with span not exceeding 500 ft (150 m). Suspension bridges, cable-stayed bridges, arch type and movable bridges are not covered by these Specifications.

CLASSIFICATION OF BRIDGES

Regular and Irregular Bridges

The approximate seismic methods of analyses presented in AASHTO Standard Specifications [3] are limited to bridges that are categorized as ‘regular bridges’. More rigorous analysis procedures, such as time history analysis, are required for bridges categorized as ‘irregular bridges’. The ‘regular bridges’ are defined by AASHTO Standard Specifications [3] as bridges having less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry, and no large changes in these parameters from span to span or support to support (abutments excluded). Any bridge not satisfying these requirements is to be identified as ‘irregular’. Parameters identifying ‘regular bridges’ are presented in Table 1.

Importance Categories (IC) of Bridges

AASHTO Standard Specifications [3] assigns Importance Classification (IC) based on the requirements of bridges as follows:

1. Essential Bridges - IC = I
2. Other Bridges - IC = II

AASHTO Standard Specifications [3] classifies bridges which perform a critical function, as ‘Critical Structures’ (article 4.2.3 [3]).
AASHTO-LRFD [4] classifies bridges based on social/survival and security/defense requirements as follows:

1. Critical Bridges: These bridges must remain open to all traffic after the ‘design earthquake’ and must be usable by emergency vehicles and for security/defense purposes immediately after a ‘maximum probable earthquake’.

2. Essential bridges: These bridges should be opened to emergency vehicles and for security/defense purposes immediately after a ‘design earthquake’ (50 year event with a 90% probability of not being exceeded in 50 years).

3. Other Bridges: The bridges not categorized as critical or essential are categorized as ‘Other Bridges’

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Spans</td>
<td>2 3 4 5 6</td>
</tr>
<tr>
<td>Maximum Subtended Angle (Curved Bridge)</td>
<td>90⁰ 90⁰ 90⁰ 90⁰ 90⁰</td>
</tr>
<tr>
<td>Maximum Span Length Ratio From Span-to-Span</td>
<td>3 2 2 1.5 1.5</td>
</tr>
<tr>
<td>Maximum Bent/Pier Stiffness Ratio From Span-to-Span (Excluding Abutments)</td>
<td>- 4 4 3 2</td>
</tr>
<tr>
<td>Maximum Span Length</td>
<td>Less than 500 ft (150 m)</td>
</tr>
</tbody>
</table>

Note: Bridges that do not meet the requirements in Table 1 are identified as ‘Irregular Bridges’

Seismic Performance Category

AASHTO Standard Specifications [3] and the Seismic Retrofitting Manual [6] define the ‘Seismic Performance Category’ (SPC) for each bridge based on seismic risk and importance, as shown in Table 2. AASHTO-LRFD [4] defines seismic zones based on the acceleration coefficient (Table 2).
Table 2 - Seismic Performance Category [3,6] and Seismic Zones [4]

<table>
<thead>
<tr>
<th>Acceleration Coefficient</th>
<th>Seismic Performance Category</th>
<th>AASHTO- LRFD [4]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>IC = I</td>
<td>IC = II</td>
</tr>
<tr>
<td>A ≤ 0.09</td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>0.09 &lt; A ≤ 0.19</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>0.19 &lt; A ≤ 0.29</td>
<td>C</td>
<td>C</td>
</tr>
<tr>
<td>0.29 &lt; A</td>
<td>D</td>
<td>C</td>
</tr>
</tbody>
</table>

SEISMIC INPUT

During an earthquake, the ground motions at a site can be dramatically altered by the presence of soils through the mechanisms of amplification, de-amplification, frequency modulation, and duration. Therefore, in developing a seismic risk map for an area, the seismic waves generated by an earthquake at its source, and the manner by which these waves propagate away from the source, are extremely important considerations. Unfortunately, estimates of ground motions based on direct regression of strong-motion data from an area should be used for that area only, unless source characteristics, travel paths, and local site factors can be shown to be similar [7].

Time History

Time histories are derived through the random vibration analyses, and take into account the probability of earthquakes from nearby seismic zones, the attenuation of ground motions with distance in the region under consideration, and the possibility of a random event occurring outside of the generally recognized zones of seismicity in the area. Most of the times, time histories are presented at the top of bedrock level in vertical and two horizontal directions. The resultant of the three will yield the overall response. A typical acceleration time history is presented in Figure 1 for the horizontal component of a 50-year event in Henderson County, Kentucky.

Response Spectra

In general, response spectra are prepared by calculating the response (acceleration, velocity or displacement) to a specified ground acceleration of single degree-of-freedom systems for different ratios of damping. Numerical integration, with short time intervals, is applied to calculate the response of the system. The step-by-step process is continued until the total earthquake record has been completed. The largest value of the function of interest is recorded and becomes the response
of the system to that excitation. Changing the parameters of the system to change the natural frequency, this process can be repeated to obtain a new maximum response. This process is repeated until all frequencies of interest are covered and the results are plotted.

Figure 1 - Acceleration-Time History for the Horizontal Component a 50-Year Event in Henderson County, Kentucky

Peak Acceleration

The 'peak acceleration coefficient' is derived from the recorded/derived acceleration time histories. It is the maximum absolute vector value of accelerations in the two horizontal directions. The coefficient does not account for soil amplification, structural amplification, and damping reduction, as the time histories are recorded at the bedrock level.
Time History and Response Spectra for the Commonwealth of Kentucky

For the Commonwealth of Kentucky, time histories of hypothetical earthquakes, along with their peak particle accelerations, and response spectra, are available for the seismic design of highway structures [8]. The peak acceleration contour maps and smoothed response spectra are available for each County in Kentucky. These maps are prepared for 50 year (Figure 2) and 500 year events[9]. The 50 and 500 events have a 10% probability of being exceeded in 50 and 500 years, respectively. A typical response spectra for the 50 year event for the Counties identified by 15%g (Figure 2) is presented in Figure 3 for 5% damping.

Seismic Analysis Procedures

Table 4.2A of AASHTO Standard Specifications [3], Table 4(a) of Seismic Retrofit Manual [6] and Table 4.7.4.3.1-1 of AASHTO-LRFD [4] provide the minimum requirements for the selection of the analysis method for a particular bridge (Tables 3 and 4). The method of selection is determined by the regularity of a bridge which is a function of the number of spans and the distribution of weight and stiffness.

Finite Element Modeling

For bridges requiring three dimensional time history or push-over analyses, a finite element model of the structure is developed to carry out the analyses. The finite element method is the appropriate tool for ‘irregular’ and ‘critical’ bridges, since it permits the modeling complex boundary conditions from soil-structure interaction, non-linearity, etc..

Field Testing

The development of a finite element model for a bridge structure requires detailed information regarding material properties, boundary conditions, etc.. A number of assumptions and approximations are generally made in order to model the structure. Calibration of the model to match the response of the actual structure is achieved by conducting field testing on the structure. There are several methods of field testing available [10], and one of these methods measures the response of the bridge under ambient conditions [11,12]. This method relies on traffic and wind to excite the bridge structure, and equipment is required to record the vibrations. Using results from the field testing, the analytical model of a bridge is calibrated to experimentally determined mode shapes and frequencies.
**Time History-Response Spectra (TR-50Y-0.xxg-x)** Identification Map for 90 Percent Probability of Not Being Exceeded in **50 Years**.

- 0.30g-1
- 0.15g-1
- 0.15g-2
- 0.15g-3
- 0.15g-4
- 0.09g-1
- 0.09g-2

Figure 2 - Time History and Response Spectra Identification Map for the Commonwealth of Kentucky
Table 3 - Method of Analysis for Multispan Bridges [3,6]
(Note: Seismic design is not required for Single-Span Bridges)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Other Bridges (IC = II)</td>
<td>Essential Bridges (IC = I)</td>
</tr>
<tr>
<td></td>
<td>SPC</td>
<td>Regular</td>
</tr>
<tr>
<td>A ≤ 0.09</td>
<td>A</td>
<td>*</td>
</tr>
<tr>
<td>0.09 &lt; A ≤ 0.19</td>
<td>B</td>
<td>ULM or SM</td>
</tr>
<tr>
<td>0.19 &lt; A ≤ 0.29</td>
<td>C</td>
<td>ULM or SM</td>
</tr>
<tr>
<td>0.29 &lt; A</td>
<td>C</td>
<td>ULM or SM</td>
</tr>
</tbody>
</table>

SPC = Seismic Performance Category;
* = No analysis required;
ULM = Uniform-Load Method (Procedure 1);
SM = Single-Mode Response Spectra method (Procedure 2);
MM = Multi-Mode Response Spectra Method (Procedure 3);
Note: Time History Method (Procedure 4) is required for 'Critical Structures' (article 4.2.3 of AASHTO Standard Specifications [3])
CASE STUDIES: LONG SPAN TRUSS BRIDGES

There are a number of bridges in Kentucky that have been designed prior to the introduction of seismic provisions in the AASHTO Standard Specification [3]. The majority of the bridges, while not yet subjected to moderate or major earthquakes, are within the influence of a number of seismic zones. In western Kentucky, New Madrid is the dominant zone for seismic design.

Recently, four long span truss bridges, over the Ohio River, have been evaluated [13-18]. The Brent-Spence bridge (Figure 4) on Interstate 75 connecting Covington, Kentucky to Cincinnati, Ohio [13-15]; the US 51 bridge (Figure 5) connecting Wickliffe, KY to Cairo, Illinois [16]; the Northbound US 41 [17] and the Southbound US 41 [18] bridges connecting Henderson, Kentucky to Evansville, IN (Figure 6).

Seismic evaluations were conducted on all four bridges to assess their structural integrity when subjected to a seismic event from various seismic zones. The scope of work was divided into the following tasks: field testing of the main bridge, analytical modeling and calibration, determination of site specific ground motion, and seismic response analysis.

Table 4 - Minimum Analysis Requirements for Multi-Span Bridges (AASHTO-LRFD [4])
(Note: Seismic design is not required for Single-Span Bridges)

<table>
<thead>
<tr>
<th>Seismic Zone</th>
<th>Other Bridges</th>
<th>Essential Bridges</th>
<th>Critical Bridges</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Regular</td>
<td>Irregular</td>
<td>Regular</td>
</tr>
<tr>
<td>1</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>2</td>
<td>SM</td>
<td>SM</td>
<td>SM</td>
</tr>
<tr>
<td>3</td>
<td>SM</td>
<td>MM</td>
<td>MM</td>
</tr>
<tr>
<td>4</td>
<td>SM</td>
<td>MM</td>
<td>MM</td>
</tr>
</tbody>
</table>

* - No Seismic Analysis Required;
SM - Single-Mode Response Spectra Method;
MM - Multi-Mode Response Spectral Method;
TH - Time History Method.
The ambient vibration properties of each bridge were determined through field testing using traffic and wind to excite the structure. The purpose of measuring the ambient vibration properties was to determine the mode shapes and the associated natural frequencies. These vibration properties were subsequently used as the basis for calibrating the analytical computer model for seismic response analysis. A three dimensional finite element model of each bridge was used for free vibration and seismic response analysis. The model was calibrated by comparing the free vibration analysis results with ambient vibration properties from field testing. Once calibrated, the model was used for seismic response analysis. The approach spans were modeled using simple single degree of freedom systems. The mass was concentrated over the piers at points of longitudinal fixity (i.e. Lollipop Model).

Site specific ground motion scenarios were developed for each bridge to represent probable earthquakes that may occur in nearby seismic zones. The time histories of these events were then used in the seismic analysis of the main bridge while the response spectra were used to evaluate the approach spans. Generally the time histories were applied at the bedrock level/footing level in all the three orthogonal directions (longitudinal, transverse and vertical) simultaneously. In order to derive conservative stress and displacement results, each bridge was subjected to two combinations of the longitudinal and transverse components with respect to the longitudinal and transverse combinations for each event. For the first combination, the longitudinal component of the event was placed along the longitudinal direction of the bridge, while the transverse one was placed along the transverse direction of the bridge. In the next combination, the longitudinal component of the time history was applied along the transverse direction of the bridge while the transverse one was applied in the longitudinal direction.

The calibrated three-dimensional model of each main bridge was subjected to the time histories to determine maximum displacements, forces and stresses. For the approach spans, the seismic analysis dealt only with the potential for loss of span due to excessive longitudinal displacement along the highway main line. The Single or Multi-Modal Response Spectra Method was applied in the seismic evaluation of the approach spans.

The main spans of all four bridges were found to be capable of resisting the projected seismic events without any damage or loss of span. The approach spans were found to be susceptible to span loss. Retrofit measures, such as cable ties and additional anchor bolts at the supports, were proposed. Details of the seismic evaluations of all the four bridges are presented elsewhere [13-18].

CONCLUSIONS

This paper presented an overview of seismic evaluation and analysis of bridges in accordance with the AASHTO Standard Specifications [3], AASHTO-LRFD [4] and the Seismic Retrofit Manual [6]. Finite element modeling, field testing and model calibrations required for critical bridges were discussed. Case studies were presented for four long span truss bridges over the Ohio river.
Figure 5 - US51 Bridge Across the Ohio River at Wickliffe, KY

Figure 6 - US41 Northbound and Southbound Bridges over the Ohio River at Henderson, KY
REFERENCES

EARTHQUAKE DESIGN CONSIDERATIONS FOR THE CAPE GIRARDEAU CABLE-STAYED BRIDGE OVER THE MISSISSIPPI RIVER

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ABSTRACT

Although often overshadowed by the seismicity of the American west coast, the New Madrid, Missouri region is a very real and significant seismic threat to the midwest. The general public, even if aware that earthquakes often occur in the central United States, does not readily admit the potential destruction that would follow a major event. Fortunately, state departments of transportation, the Federal Highway Administration, and many local building code officials are acutely aware of the risk and the damage - in terms of loss of life as well as economic losses - which would follow even a moderate event in the New Madrid region.

New structures are now designed in accordance with current seismic guidelines developed from observation of structural behavior during earthquakes and millions of dollars worth of research. However, both the research funding and empirical evaluations focus on conventional structures. In this paper, we will look at a bridge structure which is somewhat outside the norm for earthquake design and the methods used to ensure that the structure is as capable of resisting seismic loading as our current state of knowledge will permit.

This bridge, comprised of a 3-span, 1,150-foot main span, symmetrical cable-stayed unit and 1,870 feet of conventional approach structure is currently under construction within 50 miles of New Madrid, Missouri. Because of its proximity to the New Madrid Seismic Zone, it is highly probable that this bridge will experience a significant seismic event within its design life.

THE PROJECT

In 1927, the Missouri Highway Department, constructed a 4,744-foot crossing of the Mississippi River at Cape Girardeau, Missouri. Now this two lane bridge is being replaced with a new four lane cable-stayed structure.

The relocation of Missouri Route 74 - Illinois Route 146 will cross the river approximately 500 feet downstream of the existing bridge at an angle of approximately 15 degrees to the direction of flow. The proposed structure has an overall length of 3,956 feet and is comprised of a three-span, 2,086-foot steel/concrete composite cable-stayed unit and 1,870 feet of conventional steel
plate girder approach structure. The main span unit will be a 4-lane, symmetrical cable-stayed unit supported by two planes of cables 92 feet apart. The cables are attached to the steel edge girders at a uniform spacing of 35 feet. The Illinois approach structure has 11, 170-foot steel plate girder spans supported on concrete piers and founded on deep, large diameter drilled shafts.

The city of Cape Girardeau, Missouri is located within the New Madrid Seismic Zone, the location of the most violent series of seismic events ever recorded and is a candidate to experience a significant earthquake within its design life. Although not actually recorded by seismographs, studies of the available data indicate that the three most significant events of the winter of 1811-1812 had surface wave magnitudes ($M_s$) of about 8.6, 8.4, and 8.7 and it is suggested that the recurrence interval of magnitude 8 quakes in this region is between 550 and 1,200 years.

![Project Location](image)

**FIGURE 1**

In addition to the probability of a significant earthquake, the geology of the site may be characterized as having deep, liquefiable soils which are subject to frequent flooding and the potential for extensive scour. These site conditions, combined with the significance of the design earthquake event, generated some unique design challenges for both the structural and geotechnical engineers.

**THE SITE**

The Mississippi River is one of the world's great rivers. Flowing with commerce; it provides a major transportation corridor for inland barge traffic and generally contributes to the economy of the entire midwest. The Mississippi River flows for some 2,300 miles, draining approximately 40 percent of the continental United States. At St. Louis, Missouri, the Mississippi is joined by the Missouri River, the longest river in the U.S., which adds approximately 64,000 cfs to the average discharge of the river. Near Cape Girardeau, Missouri, the Mississippi River drains
more than 712,000 square miles spread over twelve states and three Canadian provinces. At the site, the channel is 1,900 feet in width with a 3,600-foot wide floodplain. The floodplain is bounded by high bluffs on the west and controlled by a levee on the east.

The river is also a significant route for inland shipping with more than 75 million tons of cargo shipped through the region annually. [1] This cargo is transported in barge tows up to 1,200 feet in length; typically comprised of up to a dozen barges, tied three across, and powered by a single tug. Therefore, the navigation requirements for this location are critical as demonstrated by the U.S. Coast Guard requirement for a channel width of 820 feet normal to the flow of the river. By increasing the navigation span to 1,150 feet it was possible to achieve considerable savings in the foundations by not having to construct a major foundation in the deepest section of the Mississippi River channel. In addition, a privately owned dry-dock facility is located immediately adjacent to the project right-of-way, downstream of the bridge and an added benefit of the longer span was to continue to allow access to the dry-dock.

SCOUR

At Cape Girardeau, the 5-year flood discharge is over 600,000 cfs at a mean velocity of about 6 fps. However, at times the discharge may be as much as 1,140,000 cfs with an average velocity in excess of 8 fps. Several hydraulic models were developed using the U.S. Army Corps of Engineers HEC-2 computer program. The models utilized the velocity distribution and normal bridge routines to generate the water surface elevations, flow depth, and stream velocity. The FHWA publication "Evaluating Scour at Bridges," HEC-18, was used to predict scour for both the 100-year and 500-year flood frequencies.

The scour analysis indicated that the total scour depth, defined for this project as the sum of the effects of long-term scour (aggradation/degradation) and local scour, may be as much as 50 feet near the main channel and up to 24 feet near the Illinois levee. Since the new bridge will span from the east levee, near the existing bridge abutment, to well beyond the west bank, it was determined that the contribution of contraction scour to the total scour value would be negligible.

SITE GEOLOGY

Geologically, the project is located on the eastern edge of the Ozark uplift and the southwestern boundary of the Illinois basin. The bedrock formations at the site are mostly limestone, with minor amounts of shale, upon which the new bridge is to be founded. The limestone is overlain by a granular, liquififiable material to a depth of approximately 70 to 100 feet. Although the area is heavily faulted, the faults are considered to be inactive.

The bridge is located within approximately 50 miles of New Madrid where there is a significant probability for a devastating earthquake within the next few years. During the series of events of 1811 and 1812, there were more than 200 moderate to large earthquakes and some 2,000 total
events with well documented evidence of liquefaction and having effects being felt as far away as Washington, D.C. [2]

DESIGN CRITERIA

The Bill Emerson Memorial Bridge represents a significant investment of public funds, and as such, required that the Missouri Department of Transportation (MoDOT) develop a design criteria to protect the travelling public, both roadway and navigation traffic, and their investment against those external events which could reasonably be expected to occur. Additionally, the criteria recognizes the importance of the structure to the economic well-being of the region as well as the difficulty of certain types of post-seismic repair. Therefore the general criteria for the bridge were established as follows:

- provide for six lanes of AASHTO HS20-44 (modified) live load
- provide minimum navigation clearances of 820 feet normal to the flow of the river and 60 feet above the 2 percent flowline
- protect against barge impact based upon a 1,200-foot tow, travelling at 15 feet per second at the 2 percent flowline elevation
- design for the 100-year scour condition, with only one-half of the anticipated scour during earthquake design
- design for earthquake forces in accordance with the "Geotechnical Seismic Evaluation" report
- provide for operation of the structure following the design event
- resist seismic forces in the tower piers within the elastic range

DESIGN EARTHQUAKE

The New Madrid region has been the most seismically active region of central and eastern North America. The events of the winter of 1811-1812 are well documented and have been the subject of a significant volume of research over the years. Nuttli's study [3] of the damage and felt effects of these events indicates that the surface wave magnitudes were on the order of 8.5. Other studies have reached similar conclusions regarding the magnitude of that series of events.

Between 1813 and 1990 over 23 earthquakes having magnitudes of 4.5 or greater have been documented in the New Madrid area. [4] By using a map of acceleration contours having a 90 percent probability of not being exceeded in 250 years, it can be shown that the peak rock acceleration at the site is approximately 0.36g. Based on input from the project design team, MoDOT selected this as the design event and, considering that M, 8 or larger events are anticipated every 550 to 1,200 years, [5] the design earthquake is essentially a repeat of the 1811 and 1812 events.
Woodward-Clyde Consultants, the project seismic subconsultant, then developed response spectra for the site based on published data for the central and eastern U.S. with the peak rock acceleration of 0.36g. Exploratory borings were made and shear wave and compression wave velocity tests conducted. The results of these investigations were used to develop three separate spectrum compatible site specific acceleration time histories for the seismic analysis of the bridge. These time histories were derived from the 1985 Michoacán, Mexico (Mexico City) earthquake and the Val Pariso and Pichulema records of the 1985 earthquake in Chile. These records were selected for their epicentral distances and the magnitude of the recorded event; however, these events do not necessarily represent a large earthquake on a continental intraplate source.

The time history files provided were in the form of accelerations, given as a fraction of gravity, over a period of about sixty seconds. These time histories were established for two orthogonal directions with consideration given to the directional uncertainty of the design event. These files also included the effect of spatial incoherency and the phased effect of the ground motion due to the piers being located over such a large expanse. Although the vertical component of the design earthquake was not directly considered it was included in the model by applying a percentage of one of the horizontal accelerations in a vertical direction simultaneously with the separate horizontal components.

In addition to the rock accelerations, Woodward-Clyde generated surface spectra which included the soil amplification effects of the soils along the Illinois bank of the Mississippi River.

LIQUEFACTION AND LATERAL SPREADING

As previously noted, the Illinois side of the site consists of some 100 feet of alluvium; primarily loose to medium-dense sands. Both the comprehensive geotechnical investigation and the investigation conducted to evaluate specific geological conditions related to the seismic evaluation for the bridge revealed Standard Penetration Test (SPT) blow counts as low as 4 with only thin seams of material having blow counts above 16 in the upper 70 to 100 feet of alluvium. With these poor soil conditions and the high level of shaking which is expected to occur during the design earthquake, widespread liquefaction is anticipated to a depth of up to 70 feet below grade.

In addition to the liquefaction, lateral spreading is also anticipated. The gently sloping banks, especially between the main channel and the levee on the Illinois shore, could flow as much as ten feet toward the channel while in a liquefied state. Clearly this will produce large horizontal forces on the bridge foundations at a time when there is little lateral support.

PRELIMINARY DESIGN

As with all projects, the design process for a bridge of this magnitude is an iterative one, requiring multiple revisions and redesign of many components along the way. Because of the
location of the bridge, and the potential for a significant earthquake, the design team attempted to minimize backtracking by working with the seismic subconsultant at the earliest stages. After the development of the basic structural concept, input from the seismic engineers was necessary to confirm the preliminary design and to prepare the models for final design analyses.

It was noted that liquefaction presents little problem for the cable-stayed unit since the three supporting piers are founded on huge footings keyed into rock; however, the approach spans are considerably different. As noted earlier, these foundations are located in an area with very deep, highly liquefiable soils. When combined, the liquefaction and the depth of anticipated scour eliminated spread footing type foundations from consideration. After extensive studies of various soil improvement techniques, it was determined that any soil improvement would be ineffective due to repeated degradation and aggradation of the channel. Thus, the early input from the geotechnical engineers permitted the elimination of both spread footings and driven steel piles as viable foundation alternatives and led to the selection of large diameter drilled shafts socketed into rock.

DESIGN METHODS

It was obvious at the earliest stages of design that the governing load case would be a combination which included earthquake forces. Other combinations, those including scour and barge impact, were also considered significant but not viewed as potentially governing the design of the bridge. Due to the large number of seismic related load combinations, those combinations with and without scour and those with and without liquefaction, it was determined that the design would be for one event only, with a final force check with the two secondary events.

The computer program used for the analysis of the structure, T187, was developed by HNTB specifically for the design and analysis of segmental and cable-stayed bridges. Within its “dynamics” module, the program performs a linear time history analysis based upon support accelerations. Using the Wilson-theta method, the program computes and stores velocities and displacements for each degree of freedom at each time step, thereby allowing the user to stop and restart the dynamic analysis and to modify the structure at any predetermined point within a dynamics run. The program also allows the user to accelerate each supported degree of freedom with a different transient load and to begin the acceleration at different times.

Based upon user input of estimated damping percentages, the program calculates the appropriate Rayleigh damping coefficients, assuming damping to be proportional to mass and stiffness, and applies these coefficients to the mass and stiffness matrices during the run. The program will then compute displacements, forces and reactions for each time step and provide the user with his choice of maximum or minimum values for a given list of members. Since the joint displacements are saved by time step, the user may elect to open the dynamic displacement file at a later date in order to compute additional results.

For design, it was determined that this bridge is an essential structure, thereby requiring that the bridge remain serviceable following a moderate earthquake, and sustain only minor damage as a
result of the maximum credible event. By minor damage, we intend that the structure would remain operational although expansion joints, bearings and other easily repaired components could sustain some damage. And since there is very little data regarding the confinement of large, hollow concrete sections, or the performance of such sections beyond the elastic range, the tower piers were designed to remain elastic throughout the design earthquake. Additionally, the approach span piers are sufficiently large that they remain elastic under all load conditions.

**Cable-Stayed Spans**

The initial steps in the analysis were to confirm that the acceleration time history files provided by Woodward-Clyde Consultants were being read correctly by the analysis program. This included a preliminary run which computed maximum relative joint displacements between the accelerated supported joints and generated a plot of the relative displacements throughout the event. These were compared to, and corresponded well with, the 10 centimeter maximum relative displacement and the continuous record of relative displacements predicted by the seismic subconsultant. Computed absolute displacements at the supports were also compared to the predicted values, and again the values correlated well. These investigations provided the confidence that the 10,000 points in each of the acceleration records provided were being correctly read by the analysis program.

Initial earthquake design runs for the cable-stayed unit indicated that without any longitudinal restraint at the tower piers, the design preference, the bridge would experience movements up to 48 inches in each direction at the ends of the unit. Further study indicated other undesirable effects with full fixity and full longitudinal restraint at the tower piers. These conditions caused live load rotations and temperature rise and fall to place higher, often undesirable, demands upon other bridge elements. Erection analyses concluded that construction of the bridge with full fixity would generate forces much higher than those observed with no restraint.

However, the fixity studies also revealed that there were some advantages to fixity as well. The wind induced motion of the bridge could be reduced while the flutter velocity threshold increased and longitudinal displacements under the various live load combinations could be minimized. Reduced movements would then require smaller expansion joint devices and relieve the required movement capacity of the side span tie down devices. These studies led to the conclusion that the cable-stayed spans should be restrained longitudinally, either with some type of bearing or key. Development of preliminary alternative details for this type of restraint indicated that the most effective solution would be one which allowed limited translation, that caused by slowly applied loads such as a uniform temperature change, and free rotation under all loading conditions.

Several types of seismic isolation and damping systems were considered; however, it was determined that force transfer would provide the most efficient solution. Both isolation and additional damping were studied to evaluate the potential impact on the design and cost of the structure. The overall stiffness contribution of the stay cables and the length of the main span reduced the effectiveness of both alternative solutions.
It was determined that force transfer was the most practical solution, and for this bridge it is achieved through the use of an earthquake shock transfer device. This device, comprised of a cylinder filled with silicon putty and a piston, is capable of transferring forces in both tension and compression. Therefore, the "double action" of the unit simplified the design of the connections to the structure and permitted transfer of earthquake forces at an elevation much lower than the stay cable connections.

The tower piers, which support the bulk of the load, are supported by a spread footing and dredged caisson on rock at Piers 2 and 3, respectively. Other foundation types were studied, however, it was determined that another foundation type was not feasible given the magnitude of the earthquake forces and the depth of water and alluvial soils.

**Approach Spans**

Liquefaction presents little problem for the cable-stayed unit since the supporting piers are founded on huge footings keyed into rock; however, the approach spans are considerably different. As noted earlier, these foundations are located in an area with very deep, highly liquefiable soils. When combined, the depth of liquefaction and anticipated scour eliminated spread footing type foundations from consideration. Even soil improvement techniques were considered ineffective due to degradation and aggradation of the channel. After studying several possible alternative solutions including pile and drilled shaft foundations and various soil improvement methods, it was determined that piers supported on drilled shafts to rock would be the most economical solution.

These shafts, which are drilled through water-bearing sand to a depth of up to 100 feet will require casing for installation. Therefore, the decision was made to require that the casing for the shafts be left in place and used as additional confinement steel during extreme condition seismic events. These conditions are generally after loss of lateral support of the shafts due to either scour or liquefaction. The analysis of the structure for the extreme event condition was conducted within the T187 "dynamics" routine. Since the solution method is based upon the values of displacement, velocity and acceleration of the joints at the previous time step, it was possible to simulate liquefaction and loss of lateral support in the foundation by varying the foundation stiffness during the run.

In this way, we were able to analyze the structure with various foundation support conditions, fixity at the piers, and multiple restrainer combinations. Ultimately it was determined that the drilled shaft option, placed in permanent casing was the best overall foundation for the site. We found that no one foundation support condition governed all aspects of the design. Primarily the half-scour and liquefied states governed the design of the piers and foundations while the forces in the structure with full support controlled the design of the bearings and superstructure connections.
CONCLUSIONS

Although the New Madrid fault zone lies within approximately 50 miles of the bridge site, the design seismic event is for a magnitude 8.5 earthquake, and the bridge must withstand the seismic forces within the elastic range, the design of the Cape Girardeau replacement bridge was a success. Each point of the design criteria was met without creating unnecessarily complex details and without significant added cost to the structure. The electronic transfer of files related to the design earthquake and the cooperation of all parties, including MoDOT, Woodward-Clyde Consultants, and HNTB Corporation all helped to identify and solve problems before they became too difficult or costly to handle. In this, the seismic analysis, wind studies and final design and detailing of the bridge remained on schedule and within the client's budget.

ACKNOWLEDGMENTS

The author extends his thanks to the Missouri Department of Transportation for permission to share their experiences and knowledge of the seismic evaluation and design of the bridge and Woodward-Clyde Consultants for much of the data, testing and graphics used in the compilation of this paper and its presentation.

1 Based on U.S. Army Corps of Engineers data for locks at River Mile 274, at St. Louis, Missouri in 1989 and 1990.
2 Nuttli, Otto W., The Effects of Earthquakes in the Central United States, 2nd ed., May 1990, Center for Earthquake Studies, Southeast Missouri State University
4 Nuttli, 1990
Design Strategies Used to Upgrade the Seismic Resistance of the I-40 Mississippi River Crossing

Roy A. Imbsen, Dennis D. Pecchia, Ging-Song Chang, and Gerry Davis

Imbsen & Associates, Inc.
Sacramento, California

ABSTRACT

The I-40 Bridge is located in the New Madrid Seismic Zone one of the most active zones in the Central United States. This crossing is a vital transportation link on the Interstate System. The bridge crossing is comprised of 164 spans which include, for the main channel crossing, five steel box girder spans and two steel through tied arch truss spans and for the approach spans and ramps precast I and steel plate girders. The design strategies and details used for the various types of bridges will be presented.

The most recently developed technologies being used which includes:

- Fully serviceable performance following the maximum probable earthquake (2500 year return period).
- Time history analysis, including spatially varying ground motion, radiation damping and foundation rocking.
- Friction pendulum isolation bearings.
- Additional piles and strengthening of footings, columns, and webwalls.
- Construction utilizing cofferdams is required.
- Strengthening of selected main superstructure members.
- Replacement of some and inelastic performance of other secondary superstructure framing members.
- Construction retrofit details
Parameters of the Seismic Analysis and Design of Medium Span Steel Bridges

A. Itani\textsuperscript{1}, A. Krimotat\textsuperscript{2}, and C. Rubeiz\textsuperscript{3}

ABSTRACT

This paper discusses the seismic modeling and analysis of five Design Examples of Steel Highway Bridges that are published by the American Iron and Steel Institute. The five bridges were modeled as 3-D “stick” models and 3-D finite element models to determine and compare their dynamic characteristics. The paper also discusses the results of nonlinear “push-over” analysis of a steel bridge with inelastic activity allowed to occur in the bridge superstructure. The results of this analysis showed the response of this bridge is comparable to a similar bridge, where the inelasticity is limited to the substructure. Therefore, inelasticity may be allowed to occur in the superstructure of steel bridges. This will limit the damage in the substructure and thus limiting the amount of the required retrofit work in the columns and footings for existing bridges.

INTRODUCTION

The current AASHTO Specifications [1] do not have adequate guidelines for seismic modeling and analysis of steel bridges. Bridge designers in seismic zones have always raised the concern about the adequacy of beam models in performing seismic analysis of steel bridges. Recognizing this fact, the American Iron and Steel Institute (AISI) initiated a study at the University of Nevada, Reno and at SC Solutions to model, analyze, and seismic design the Four LRFD Design Examples of Steel Highway Bridges [2]. These design examples consisted of one span, two spans steel plate girders and a two span twin steel box girder. These bridges did not include any substructure. Several substructures were chosen for these bridges to cover a wide spectrum of current bridge configurations.

The Automated Dynamic Incremental Nonlinear Analysis (ADINA) computer program [3] was chosen to model and to perform the seismic analysis on these bridges. The 3-D “stick” model was constructed by lumping the superstructure moment of inertia into one beam element. Mechanics of materials principle were followed to determine the equivalent moment of interia (Ixx, Iyy) and the torsional moment of inertia (Izz). The 3-D finite element model consisted of modeling the plate girder, concrete deck, and the cross bracing each as a separate element. The plate girder was modeled as three plates with 4-node linear shell element, the concrete deck was modeled as 20-node linear solid element, and the cross frames as nonlinear beam element. The bent cap was modeled as linear beam element, while the column was modeled as nonlinear moment curvature element.

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\textsuperscript{2} Principle Engineer, SC Solutions, Inc., Santa Clara, CA 95054
\textsuperscript{3} Program Manager, American Iron and Steel Institute, Washington, D.C. 20036
Table 2 – Dynamic characteristics of the steel bridges

<table>
<thead>
<tr>
<th>Bridge #</th>
<th>Direction</th>
<th>3-D “Stick” Model</th>
<th>3-D “Finite Element Model”</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Period (sec)</td>
<td>MPF (%)</td>
<td>Period (sec)</td>
</tr>
<tr>
<td>Example 2</td>
<td>Longitudinal</td>
<td>1.39</td>
<td>97.9</td>
</tr>
<tr>
<td></td>
<td>Transverse</td>
<td>0.21</td>
<td>78.1</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>0.25</td>
<td>62.9</td>
</tr>
<tr>
<td>Example 2M</td>
<td>Longitudinal</td>
<td>1.36</td>
<td>89.4</td>
</tr>
<tr>
<td></td>
<td>Transverse</td>
<td>0.23</td>
<td>73.5</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>0.29</td>
<td>62.9</td>
</tr>
<tr>
<td>Example 3</td>
<td>Longitudinal</td>
<td>1.23</td>
<td>97.3</td>
</tr>
<tr>
<td></td>
<td>Transverse</td>
<td>0.43</td>
<td>80.7</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>0.28</td>
<td>63.3</td>
</tr>
<tr>
<td>Example 3M</td>
<td>Longitudinal</td>
<td>1.21</td>
<td>97.8</td>
</tr>
<tr>
<td></td>
<td>Transverse</td>
<td>0.7</td>
<td>81.8</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>0.28</td>
<td>63.1</td>
</tr>
<tr>
<td>Example 4</td>
<td>Longitudinal</td>
<td>1.5</td>
<td>98.1</td>
</tr>
<tr>
<td></td>
<td>Transverse</td>
<td>0.54</td>
<td>81.6</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>0.5</td>
<td>62.8</td>
</tr>
</tbody>
</table>

Figures 1 and 2 show the fundamental mode shapes of bridge Example 2 and 3. It is important to mention here that even though the 3-D “stick” models were able to capture the overall dynamic behavior of the bridge, however, some local modes that exhibited in the 3-D finite element models were not captured.

SEISMIC DESIGN PHILOSOPHY AND PERFORMANCE CRITERIA

The current AASHTO Standard Specifications along with the AASHTO LRFD Design Specifications [4] treat structural steel bridges the same as structural concrete bridges in terms of seismic design and performance. The AASHTO basic seismic design philosophy intends to limit the inelastic activity in the substructure since it is neither feasible nor economical to allow plastic hinging to occur in the superstructure. This concept is mainly derived from structural concrete bridges where the flexural strength of the superstructure exceeds by many times that of the column. However, in steel bridges there is a clear distinction in the various elements of the superstructure such as cross frames, diaphragms, lateral bracing, steel girders, and bridge deck [5]. Therefore, it is possible to take this advantage of this distinction by allowing the inelasticity to occur in the non-gravity carrying members such as cross frames. Provided that these designated elements should be designed as ductile members.

Elastic and inelastic studies conducted on several bridges showed that the cross frames over supports, bents and abutments, lie in the lateral load path [6, 7]. Therefore, it is possible to allow these members to act as a "fuse" during severe earthquakes and reduce the lateral forces on the substructure. This will limit the damage to the cross frames, which can be easily replaced after severe events.
Elastic linear analysis was performed to determine the dynamic characteristics of these bridges and compare between models. Elastic response spectrum analysis using AASHTO Spectrum was performed to size the substructure elements.

BRIDGE DESCRIPTIONS

The five bridges are taken from the AISI Four LRFD Design Examples of Steel Highway Bridges. Table 1 summaries the geometrical properties of these bridges.

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Spans #</th>
<th>Bent Type</th>
<th>Skew</th>
<th>Span Length</th>
<th>Column Size</th>
<th># X-Frames/Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ex. 2</td>
<td>2</td>
<td>Drop-Single</td>
<td>No</td>
<td>27,000 mm</td>
<td>1,219 mm</td>
<td>6</td>
</tr>
<tr>
<td>Ex. 2M</td>
<td>2</td>
<td>Drop-Single</td>
<td>35</td>
<td>27,000 mm</td>
<td>1,219 mm</td>
<td>6</td>
</tr>
<tr>
<td>Ex. 3</td>
<td>3</td>
<td>Drop-Double</td>
<td>No</td>
<td>43,000 mm/53,000 mm</td>
<td>1,219 mm</td>
<td>6</td>
</tr>
<tr>
<td>Ex. 3M</td>
<td>3</td>
<td>Integral-Doubl</td>
<td>No</td>
<td>43,000 mm/53,000 mm</td>
<td>1,219 mm</td>
<td>6</td>
</tr>
<tr>
<td>Ex. 4</td>
<td>3</td>
<td>Drop-Double</td>
<td>No</td>
<td>61,000 mm/66,000 mm/61,000 mm</td>
<td>1,219 mm</td>
<td>6</td>
</tr>
</tbody>
</table>

Example 1 consisted of a two span-single column bent bridge. The bent is a dropped type, where the column is fixed at the base. Example 3 consisted of a three span-two column bent. The bent is a two column bent, the spacing between the columns is equal to 7155 mm, with the columns fixed at the base. Example 3M is identical to Example 3, however the bent integral where the columns are pinned at the base. Example 4 consisted of three spans-two columns bent. The columns are fixed at the base and the bent cap is a dropped cap.

DYNAMIC CHARACTERISTICS

The five bridges were modeled as discussed earlier using 3-D “stick” and 3-D finite element models using ADINA program. The dynamic characteristics which includes the fundamental longitudinal, transverse, and vertical period along with the corresponding percentage mass participation factors are presented in the Table 2. As Table 2 shows, the 3-D “stick” models were able to capture the dynamic characteristics of the five bridges is a reasonable way.

Therefore, it can be concluded that using the 3-D “stick” model can capture the dynamic properties of steel bridges. This conclusion is very important since it is not necessary to model each component of the steel bridge in the dynamic model.
To investigate the effect of allowing cross frames in the superstructure to yield and buckle, an interior segment of Example 2 was subjected to incremental lateral load. Two types of superstructure were used in this investigation to compare between the two concepts. Model 1 consisted of chevron braced cross frames designed to stay elastic while the yielding is limited to the substructure only. Model 2 consisted of X-braced cross frames designed as ductile elements. These members will buckle and yield before any inelasticity will occur in the substructure.

The column was modeled as nonlinear moment-curvature element using Mander’s model [8] for confined concrete. The cross frames where modeled as buckling beam element that is able of capturing the post-buckling phenomenon of axial members. Table 3 presents the geometrical and material properties of the R/C column. Table 4 gives the cross section that was used for the X-braced model and the yield/buckling capacity of the member.

<table>
<thead>
<tr>
<th>Diameter</th>
<th>Height</th>
<th>$f_c$</th>
<th>$F_y$</th>
<th>Longitudinal Bars</th>
<th>Transverse bars</th>
<th>Cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>5'-0&quot;</td>
<td>25'-0&quot;</td>
<td>4 ksi</td>
<td>60 ksi</td>
<td>36#11 (2%)</td>
<td>#8 @ 4&quot; o.c.</td>
<td>3&quot;</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Length</th>
<th>Section</th>
<th>b/t</th>
<th>$P_y$</th>
<th>$P_{cr}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>11'-6&quot;</td>
<td>L2x2x5/16</td>
<td>6.4</td>
<td>41.4 kips</td>
<td>7.8 kips</td>
</tr>
</tbody>
</table>

The analysis was performed using the capabilities of ADINA program under displacement control. The two models were subjected to incremental lateral displacement at the deck level. Figures 3 and 4 show the results of the force-displacement curve of the push-over analyses for Model 1 and Model 2, respectively. As can be seen in Figure 3, at a lateral force equal to 891 KN a plastic formed at the base of the column. Figure 4 shows the response of the Model 2. The compression diagonals of the cross frames buckled while the tensioned members yielded. The response of this system is comparable to the response of the special Truss Moment Frame [9]. Therefore, it can be concluded that inelastic activity can occur in the bridge superstructure and will perform adequately. More details of this study will be discussed in details in the AISI seismic report [10].

**SUMMARY AND CONCLUSIONS**

This paper presented results of the dynamic characteristics of five steel bridges modeled as 3-D “stick” models and 3-D finite element model. It was concluded that the dynamic properties of a 3-D “stick” model were reasonably comparable to that of 3-D finite element model. Also, the paper discussed the results of push-over analysis of two bridge models designed according to AASHTO requirements and to the suggested seismic design procedure by allowing the inelastic activity in the superstructure.
ACKNOWLEDGEMENTS

The authors would to thank the American Iron and Steel Institute of supporting this study. The authors appreciate the input of the AISI Seismic Task Force during various stages of the study in particular the Chair Mr. Edward Wasserman of the Tennessee DOT. The technical input of Dr. Hassan Sedarat is sincerely appreciated.

REFERENCES


Figure 1  Comparison between 3-D "stick" model and the 3-D finite element model for Example 2
Comparison between 3-D "stick" model and 3-D finite element model of Example 3

Figure 2
Figure 3  Force-displacement response of Model 1

Figure 4  Force-displacement response of Model 2
MODELING AND ANALYSIS FOR SEISMIC RETROFIT OF THE US 40/I-64 DOUBLE-DECK COMPLEX IN ST. LOUIS, MISSOURI

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ABSTRACT

This paper presents the modeling and analysis US 40/I-64 double-deck complex in St. Louis, Missouri. A brief description of the structure and project chronology is presented along with discussion of the analytical model, response spectrum and time history analysis, and pre- and post-processors used for computationally intensive portions of the project.

INTRODUCTION

The US 40/I-64 double-deck bridge complex provides east and westbound approaches to the Poplar Street Bridge over the Mississippi River in St. Louis. This complex carries over 130,000 vehicles per day between Missouri and Illinois and has been identified by the Missouri Department of Transportation (MoDOT) as one of the most critical transportation links in the state. As part of MoDOT’s seismic retrofit program, Sverdrup Civil has provided seismic evaluation and retrofit design for this important bridge complex.

The objective of this paper is to present the modeling and analysis of this structure. In keeping with this objective, the following paragraphs present a brief description of the bridge and chronology of the project to provide context for the subsequent discussion of modeling and analysis. Additional discussion of other aspects of this project, such as the initial seismic retrofitting strategy, retrofit details and the competitive procurement strategy for the seismic isolation system is presented in references [1] and [2].

Description of the Bridge

The US 40/I-64 Double-Deck bridge complex was designed in the mid-1960’s and includes more than 15,700 feet side-by-side and double-deck elevated roadway. Figure 1 shows the location and layout of the complex in downtown St. Louis. Following is a summary of the major features of the structure:

- Figure 2 shows examples of the 139 single and multi-column reinforced concrete bents included in the scope of the retrofit. Bent construction includes round, elliptical, and rectangular columns with reinforcement lap splice and confinement details typical of pre-1980’s design standards.

- 118 of the bents are supported on battered H-pile’s driven to rock at depths of 14 to 83 feet. 21 of the bents are supported on 3’-6 to 5’-0 diameter drilled shafts that are founded on rock at depths of 24 feet or less.
• The superstructure is typically continuous steel two-girder and multi-girder framing with composite reinforced concrete deck slabs. The superstructure is made discontinuous every three to five spans by internal hinges and drop-in spans that are supported on 270 steel fixed and rocker type expansion bearings, as shown in figure 3.

• The superstructure is supported on the substructure by 764 steel fixed and expansion bearings of the types shown in figure 4.

Figure 1 – Project location and layout

Figure 2 – Representative bents

Figure 3 – Typical expansion hinge

Figure 4 – Representative bearings
Project Chronology

Due to the size and complexity of this bridge, the seismic evaluation and retrofit design were developed in the following steps:

- **Initial Evaluation** Force and displacement Capacity/Demand (C/D) ratios were computed for all of the bearings based on the empirical equations given in the “Retrofitting Guidelines” [3]. This study found that the support lengths of the expansion hinges, the force capacity of the bearings in restrained directions, and the displacement capacity of the expansion bearings were inadequate. Typical bent column details were also studied and it was found that in critical high moment regions near the base of the columns, the lap splice lengths of main reinforcement, and the confining tie reinforcement, were inadequate when compared to the “Retrofitting Guidelines” [3].

- **Detailed Evaluation** Based on the initial evaluation, a detailed evaluation was performed on the structure with (a) all bearings replaced with conventional elastomeric bearings designed for seismic displacements and forces, (b) all bearings replaced with lead-rubber seismic isolation bearings (which was the predominant isolation technology for bridges at the time of the study), (c) longitudinal restrainers at all expansion hinges, (d) longitudinal restrainers at all bent expansion bearings. Dynamic analysis of the entire complex was conducted using the multi-mode response spectrum method and C/D ratios were computed for 13 representative bents, based on the “Retrofitting Guidelines” [3,4]. Key seismic parameters include Acceleration Coefficient, A=0.10; Soil Type = II; and the AASHTO design spectrum [5]. Based on the detailed evaluation, a retrofit strategy was adopted that included (a) adding force transmitter type restrainers to all expansion hinges, (b) adding steel jackets in the regions of column splices, and (c) replacing all bent bearings with seismic isolation bearings.

- **Seismic Retrofit Design** As retrofit design progressed, C/D ratios were computed for all of the bents in the complex and additional studies were conducted that identified the need to strengthen 74 pile foundations, 9 of the drilled shaft foundations, strengthen the transverse cross-frames at the bents, and replace the rocker type expansion bearings at the expansion hinges. During the timeframe of this effort, a number of alternative seismic isolation technologies began to gain acceptance for bridge seismic retrofit. Therefore, an approach for competitive procurement of the isolation bearings was developed [2], and the construction cost of replacing each of bearings was estimated. These studies found that the estimated cost of bearing replacement (exclusive of the cost of the bearings and jacking towers) ranged from $19,000 to $25,000 per bent for typical level 1 bents with multi-girder framing, to $55,000 per bent for level 2 bearings, to over $500,000 per bent for locations with particularly restricted construction conditions. From these studies it became clear that replacing all of the bearings on the complex represented significant construction cost and risk, so the procurement strategy was designed to encourage developing alternatives to bearing replacement by basing selection on total cost, rather than cost of the isolation hardware alone. Bidding documents that allowed MoDOT to directly purchase the isolation system were developed [2], and at the conclusion of the technical proposal phase, only two proposals were received from manufacturers of isolation hardware, both of which proposed to replace all of the bearings in the complex, and neither of which included integration of multiple isolation
technologies, or significant investigation of alternatives to replacing all of the bearings. Based on the proposals received from the isolation industry, MoDOT elected to terminate the procurement process and tasked Sverdrup with investigating alternatives to bearing replacement.

- **Alternatives to Bearing Replacement** Initial investigation of alternatives to replacing all of the bearings was based on the multi-mode response spectrum method and representative portions of the double deck and two girder roadways. These studies found that there were alternatives to replacing all of the bearings through combinations of dampers or energy dissipaters and isolation bearings; however, it was also found that changing the least number of high cost/risk bearings required strengthening of portions of the bents beyond the levels initially planned for the seismic retrofit. Based on comparison of the estimated cost of the alternatives, MoDOT elected to modify the seismic retrofitting strategy to include retaining as many of the high cost/risk bearings as practical, using combinations of isolation technologies, and strengthening portions of the bents to resist anticipated seismic demands.

- **Revised Seismic Retrofit Design** As of writing of this paper, Sverdrup is developing analysis and designs to implement the revised retrofitting strategy. Analysis for this effort was based on the non-linear time history method.

The following paragraphs present the major modeling and analysis portions of the project. Because of the significant computational effort associated with developing analysis input and reducing analysis results, discussion is also included for major pre- and post-processors used on the project.

**MODELING AND ANALYSIS FOR DETAILED SEISMIC EVALUATION**

**Analytical Models**

The analytical model of the bents used 3D beam elements to represent the bearings, cap beams, columns, and footings, as shown in figure 5. Eccentricities between the center of gravity (CG) of the beams and the bottom of the bearings, and the CG of the superstructure and the top of the bearings were represented with "rigid" mass-less 3D beam elements. Member and material properties for the bents were based on data given on the record drawings and variation of properties and member releases were used to represent the different types of bearings and restrainers included in the study.

Spring elements were used to represent the foundations at the base of the bents. Linear stiffness for the springs representing pile groups were derived using CPGA [6] and pile configurations from record drawings, soil properties from project specific geotechnical investigations and records from the original construction, and vertical and horizontal loads from preliminary
analyses. Similarly, stiffness for the springs representing drilled shafts was derived from analysis with COM624 [7].

The analytical model of the superstructure used 3D beam elements located along the CG of the roadway, as shown in figure 6. Three intermediate nodes were located within each span to allow for distribution of mass, and internal hinges and hinge restrainers were modeled with member releases within the spans. Material and element properties for the beam elements representing the superstructure were calculated from data shown on the drawings and standard strength of materials assumptions for built-up transformed sections. Mass was added to the superstructure joints to account for the difference between the modeled mass of the elements and the computed mass of the slab, girders, cross-frames, curbs and parapets, as shown on the record drawings, plus an additional 640 lb/ft live load, which was applied as static dead load to the structure.

Due to the volume of input data required to describe this complex, a series of in-house pre-processors were used to generate the analytical models. Models of the individual bents were first generated in a local bent coordinate system using parametric data shown on the record drawings. After the bent models were verified using an in-house graphical display program, the joint coordinates were transformed into a global coordinate system based on the survey coordinates and elevations shown on the drawings. A numbering system was established that allowed up to 100 joints and members per bent, and the base numbering for each bent was incremented by 100 to simplify processing of the data. To assist in data generation, the model of each bent included the associated joints and elements for the bearings and links to the superstructure CG(s) in the plane of the bent, as well as those representing the bent components and foundation springs. After generation of the bent models in the global coordinate system, the superstructure segments and internal hinges for each span were generated by linear interpolation between the superstructure CG’s for each related bent, and the results were graphically verified. This multi-step approach to model generation allowed the necessary opportunity to develop special case data that did not easily fit into a repetitive generation scheme using spreadsheets or manual data entry. In addition, each of the pre-processors described to this point generated data in an in-house “neutral” format. This approach has allowed the use of standardized pre-processing programs on multiple projects with a variety of structure types and analytical requirements. The final step in the generation process was a pre-processor that translated “neutral” format data into the final input files for static and dynamic analysis required by the general purpose analysis program, which in the case of this project was the OS-2 version of M-STRUDL.

Based on preliminary analyses it was found that dividing the analysis of the complex into the nine models shown in figure 7 provided a reasonable balance between execution time, output file size, and analysis results in terms of mode shapes and mass participation, within the limitations of M-STRUDL. The models were divided at expansion hinges and were designed to overlap to ensure that all bents were fully loaded by adjacent spans in at least one analysis. With this
approach, each model had approximately 15 spans, 630 joints and 3800 DOF, and 90% or more mass participation was obtained in the first 30 modes that were available from M-STRUDL. In total, the nine models for the entire complex (including overlap) required approximately 5,700 joints and over 34,000 Degrees of Freedom (DOF).

![Analytical models](image)

Figure 7 – Analytical models

**Analysis**

For each of the nine models, static analysis was conducted for dead load (DL), and dynamic analysis was conducted using the multi-mode response spectrum method for the following conditions: (a) elastomeric bearings at the bents with hinge restrainers on and off, (b) lead-rubber isolation bearings at the bents with hinge restrainers on and off, and (c) restrainers on at bent bearings and hinges. In each of the dynamic analyses, mode shapes were reviewed graphically to confirm that modes representing the global vibration of the structure were obtained in the longitudinal and transverse directions, and mass participation was reviewed to confirm that approximately 90% mass participation was achieved in the direction of each horizontal axis. Modes were combined using the SRSS and CQC methods and in each analysis the AASHTO spectra was applied in the bridge longitudinal (EQX) and transverse directions (EQZ) and the load cases given by equations 1 and 2 were formed [3, 4, 5].

\[
\text{Case 1: DL + 1.0 EQX + 0.3 EQZ (1)}
\]

\[
\text{Case 2: DL + 0.3 EQX + 1.0 EQZ (2)}
\]

**Capacity/Demand Ratios**

The results from the dynamic analyses were used initially to evaluate C/D ratios for 13 representative bents using the procedures given in the “Retrofitting Guidelines” [3,4]. This effort involved first computing a set of ratios of nominal ultimate capacity to elastic demand, and then computing a set of ratios for critical structural details. The first set of these ratios is intended to determine if a component can be expected to yield under design level earthquakes, while the second set is intended to determine if the details will fail prior to developing the ultimate capacity of the components.

Ultimate capacities for the bents were computed using the procedures for plastic hinging of columns and bents, as given in Section 7.2.2 of reference [5]. Out-of-plane hinging of single and
multi-column bents and in-plane hinging of single column bents were based on cantilever behavior, as shown in figure 8. In-plane hinging of multi-column bents was based on Level 1, Level 2, and Beam failure modes, as shown in figure 9.

![Figure 8](image1)  ![Figure 9](image2)

Figure 8 – Cantilever hinging modes  Figure 9 – In-plane hinging modes for multi-column bents

In the Level 1 failure mode, yielding was assumed to occur at the top and bottom of the level 1 columns. Similarly, yielding in the Level 2 failure mode of double deck bents was assumed to occur at the top and bottom of the level 2 columns. The Beam failure mode for double deck bents involved yielding at the bottom of the level 1 columns, the beams at the top of level 1, and at the top of the level 2 columns. In cases where the details at the top of the columns did not provide moment resistance, the yielding moment was set equal to zero in the hinging analysis.

A strength reduction factor (φ) of 1.3 was applied to all reinforced concrete members for calculation of the plastic hinging shear for the various failure modes. The ultimate moment capacity of the horizontal beam members was evaluated with the dead load axial load, while column ultimate moment capacity was evaluated for dead load plus and minus overturning, using the specified iterative procedure [5]. The hinging moment at each joint was taken as the minimum capacity of the end of the column or beam under consideration or the summation of the moment capacities of all other members framing to the joint. In addition, the hinging moment at the bottom of the level 1 columns was taken as the minimum of the column or foundation hinging moment.

Ratios of nominal ultimate capacity (φ=1.0) to elastic demand were computed for the top and bottom of the columns, the foundations, and the ends of the beams. These ratios were based on the minimum moment capacity of each component from the failure modes considered in the hinging shear analysis and the elastic moment demands computed for the two load cases given by equations (1) and (2) for each of the analyses. It is noted that the using minimum component capacities from multiple failure modes for these ratios can be overly conservative, because yielding of a single component does not necessarily indicate failure of an assembly of components in a multi-column bent. However, this approach was judged acceptable for comparing bearing replacement alternatives with the design objective of limiting damage to the bents. The ratios for the 13 representative bents indicated that replacement of the bearings with isolation bearings would prevent yielding in the columns and footings.
Detail C/D ratios were evaluated for anchorage of the column reinforcement into the footings, splices in the main column reinforcement, column shear, transverse confinement reinforcement, and footing rotation and yielding [3,4]. These ratios indicated that critical details, such as the splices in reinforcement at the base of the columns with lap lengths of 24 bar diameters and #5 ties spaced at six inches to one foot, were inadequate to develop the ultimate moment capacity of the columns.

As work progressed into design of seismic retrofits, an in-house post-processor was used to compute C/D ratios for all of the bents in the scope of the project. This program was designed to use input such as joint and member information from the analytical models, dimensions and reinforcing parameters derived from the record drawings, and member forces from the M-STRUDL output files. The output from this program included detailed reports of the computations of both sets of C/D ratios for each bent, and summary data that was imported into spreadsheet programs for graphing. Review of this data identified specific foundations that required modification and components such as the beam/column joints that required additional study.

Analysis of Bent Shear Capacity

A major objective of the isolation system procurement strategy [2] was to allow each bidder to propose the location, type, and combination of seismic hardware was best suited to their particular isolation technologies. To allow consideration of different types of isolation systems, including alternatives that used combinations of isolation technologies to avoid changing all of the bearings, performance of the isolation system was specified in terms of bent shear capacity, which was taken as the minimum of the following:

- The shear necessary to produce plastic hinging of the bents, which was calculated with an in-house post-processor that used the procedure of Section 7.2.2 of reference [5] and the same input files as the C/D ratio program. Output from this program included the shear necessary to form enough plastic hinges to cause a collapse mechanism for each failure mode of each bent.

- The shear necessary to cause yielding of the foundations, which was calculated using the CPGA program and the moments and vertical loads from the bent hinging analysis.

- The shear necessary to produce first yielding of the columns and the shear necessary to produce six inches of lateral deflection at the top of the bents, which were computed from a unit load analysis of the bents using M-STRUDL.

In each case, the horizontal shear was assumed to act at the CG of the superstructure(s) at the bent, and interaction between longitudinal and transverse loading was required to satisfy equation 3 for single level bents, and equations 4 and 5 for double deck bents.

\[
t1/T1 + 11/L1 \leq 1.0 \quad (3)
\]

\[
t2/T2 + 12/L2 \leq 1.0 \quad (4)
\]

\[
((t1*A1) + (t2*A2))/(T12*A12) + (((11*A1) + (12*A2))/(L12*A12)) \leq 1.0 \quad (5)
\]
In these equations, \( l_1, l_2, t_1, t_2 \) are the longitudinal and transverse forces at levels 1 and 2 from analysis of the structure with the proposed isolation system; \( L_1, L_2, T_1, T_2 \) are the specified longitudinal and transverse capacities for levels 1 and 2; and \( A_1, A_2, \) and \( A_{12} \) are the specified moment arms from the CG of the superstructure to the bottom of the footings at levels 1 and 2, and the combination of levels 1 and 2.

MODELING AND ANALYSIS OF ALTERNATIVES TO BEARING REPLACEMENT

Analysis of Alternative Concepts

Analysis of alternatives to bearing replacement included consideration of adding dampers or restrainers to hinges and bents with existing bearings, strategic replacement of limited numbers of existing bearings with isolation bearings, and structural modifications of the bearings and bents to increase capacity or seismic performance. SAP2000 Non-linear was selected as the analysis program for this portion of the project to allow modeling of the non-linear characteristics of isolation devices. The modeling effort began with using an in-house pre-processor to translate the final M-STRUDL input files into SAP2000 format. Conversion of the models was verified graphically in SAP and by comparing the results of multi-mode response spectrum analysis using SAP2000 with the previous M-STRUDL results.

With the conversion to SAP2000, the nine models of the main-line roadway, and models of selected ramps that were included in the scope of work, were combined to form a single model of the entire complex, as shown in Figure 10. Multi-mode response spectrum analysis was conducted on the combined model and the results were found to be in good agreement with the results obtained from the overlapping models of portions of the complex. To facilitate analysis of portions of the complex, a pre-processor was developed in-house to extract a complete SAP input file from the combined model for any specified subset of bents and spans.

![Figure 10 – Model of complex](image)

Initial evaluation of alternative concepts was based on multi-mode response spectrum analysis of representative sections of double deck and critical single level portions of the complex. Various configurations of bearings and structural modifications were represented in these analyses as follows:
• Existing steel fixed and expansion bearings were modeled through material properties, section properties, and member releases. Similarly, the effective stiffness of replacement isolation bearings was modeled with material and section properties.

• Damping provided by isolation bearings and dampers was modeled by modifying the acceleration response spectra with a damping coefficient, $B$, as given in table 1 [8]. Preliminary analysis with the modified spectra shown in figure 11 was used to identify combinations of retrofit components that would likely provide acceptable seismic performance.

<table>
<thead>
<tr>
<th>Damping (Percent of Critical)</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B$</td>
<td>0.8</td>
<td>1.0</td>
<td>1.2</td>
<td>1.5</td>
<td>1.7</td>
<td>1.9</td>
</tr>
</tbody>
</table>

Table 1 – Damping coefficient, $B$

![Figure 11 - Modified response spectra](image)

• Structural modifications, such as shear walls or external braces were modeled by adding 3D beam elements with appropriate properties to the basic analytical models. Similarly, structural modifications such as column strengthening were modeled with changes in member properties, while foundation modifications were modeled with changes to the spring stiffness.
• Restrainers such as force transmitters, rods, or cables, at hinges and bents were modeled with member releases and properties.

An in-house post-processor was developed to read the SAP2000 response spectrum output files to compute bent shear from the summation of bearing element forces, and to evaluate equations 3, 4, and 5 using the specified shear capacities. This program also computed relative displacement across the bearings, based on shear force, the length and properties of the bearing elements. Output from the post-processor included detailed computations for each bent, for each load case, and each damping level, and summary data which was imported into a spread program for graphing of results.

These analyses showed that with the nominal reinforcement of splices and foundation modifications on which the specified capacities were based, it was likely that some of the high risk/cost bearings could be retained in the isolated structure through combinations of dampers and replacing existing bearings with isolation bearings. However, with the initially planned level of structural retrofit, many high risk/cost bearings would require replacement, particularly most (or all) of the fixed bearings in the complex, including those on level 2, and in some cases, additional strengthening of the bents would likely be required.

Based on the initial findings, additional response spectrum analyses were conducted to explore alternatives that minimized bearing replacement by increasing capacity of the bents, rather than reducing or distributing demand through an isolation system. These analyses used the same analytical models as before, with modification of member properties and soil springs to reflect various combinations of structural modifications. Capacities for modified bents were recomputed to reflect structural modifications using the hinging shear analysis program and the SAP output files were post-processed to evaluate equations 3, 4, and 5 for each bent, loading, and damping level. The results from these studies, along with conceptual cost estimates for the various alternatives, showed that structural modifications beyond those initially planned, would allow the seismic performance objectives to be achieved, for comparable or less cost than total bearing replacement, while minimizing (or eliminating) the number of high cost/risk bearings that would require replacement. Therefore, this approach was selected as the basis for final analysis and design.

Final Analysis

Final analysis of the modified structure was based on the time history method. Because there are no records of design level earthquakes for the project site or region, five pairs of artificial, design spectra compatible, ground motions were generated for the project [5]. This work was done by specialists under sub-contract, and detailed discussion of this effort is beyond the scope of the current paper.

Following is a summary of the sequence of development of the final analysis of the structure:

• Linear time history analyses were conducted on representative portions of the double deck and single level roadways. As expected from spectra compatible ground motions, good agreement was found between the maximum element forces computed by the time history method, and the previous results found from the response spectrum method.
- Output from SAP2000 time history analysis includes maximum and minimum element forces, but does not include the option of obtaining consistent element forces for each step in time. Although step-by-step output is available for single components of single elements through the graphical interface, no facility is provided for producing this output for large numbers of elements and multiple time histories with minimal user input. Consequently, the ability to compute sets of design forces that are in equilibrium, which is a major advantage of the time history method over the response spectrum method, is not available from the SAP2000 output files. Fortunately, the internal SAP2000 data files contain the information necessary to compute these values and a series of post-processors was developed in-house. The first of these programs was designed to read the SAP2000 internal data files and compute the forces and moments for each step in time for a set of members, such as all of the bearings, specified in a data file by the user. The second of these programs read the step-by-step force data and computed the longitudinal and transverse shear for each bent, for each time step, based on a user specified data file that contained the association between member numbers and bents. The third program read the step-by-step bent shear data and evaluated equations 3, 4, and 5 for each bent, for each time step, based on a user specified data file of bent capacities. The third program also produced a summary of the worst case value of the specified equations for each bent, including the time and consistent forces associated with the worst case. Although the output from these programs was large (10's to 100's of megabytes), the files were will within the capacity of current microcomputer hard drives, and the approach allowed determination of the worst case loading from each time history, rather than designing for combinations of maximum forces that did not occur simultaneously.

- Review of the time history output showed that the transverse bearing shear forces along the level 1 beams, while in equilibrium with the forces acting on the bent, did not always act in the same direction at a given point in time; consequently, the summation of these forces, accounting for direction, was appreciably less than the summation of absolute values, or the summation of maximum values obtained from response spectrum or time history analyses. Unit load analysis of a representative bent showed that this condition could be duplicated with a lateral static load applied at the superstructure CG node(s). Unit load analysis of each of the rigid link configurations shown in figure 12 produced similar results with bearing elements representing the stiffness of the existing bearings, while analysis with elements representing "soft" elastomeric bearings tended to minimize the effect. Review of the equilibrium checks for these analyses showed that the solutions balanced within 1E-08 kips out of 100 kips applied, and deflections and rotations were reasonably small; therefore, the results were caused by small deflections and rotations of the continuous level 1 beams acting through the eccentricities from the CG to the top of the beams (see figure 13), rather than ill-conditioning of the stiffness matrix. Since these internal forces were self-balancing, the computation of bent shear from time history bearing forces was correct. However, in the case of response spectrum analysis, or with combination of maximum absolute values of force, this effect will go undetected with elements representing existing steel bearings, and the shear on the bent will be overestimated.
The 3D Beam elements representing all of the bent bearings were replaced with SAP2000 NLLINK elements [9] using an in-house pre-processor. The NLLINK element is capable of representing non-linear behavior of viscoelastic damping, compression only, tension only, uniaxial plasticity, biaxial plasticity isolation bearings, and friction pendulum isolation bearings. These elements can be used in both linear and non-linear analyses, and were used to represent existing bearings, isolation bearings, and dampers in the model. Results from preliminary linear time history analyses with NLLINK bearings showed good comparison with those obtained with 3D Beam bearing elements. In addition, the three time history post-processors previously discussed were modified to process the NLLINK data.

The single joint representations of expansion hinges were replaced with multi-element models, as shown in figure 14, using an in-house pre-processor. This revised expansion hinge model allowed analysis of combinations of dampers, restrainers, and isolation bearings at each existing hinge bearing location. Test analyses were conducted to confirm that results with these changes were in good agreement with the previous analyses.

Rigid body constraints were generated with a pre-processor for the joints that connected the superstructure CG to the NLLINK’s at the hinges and bents. These constraints were designed to enforce displacement of the joint assemblies while avoiding potential problems caused by large differences in stiffness between the elements representing the bearings and the “rigid” links. Test analyses were conducted to confirm that results with these changes were in good agreement with the previous analyses.

Non-linear time history analysis with SAP2000 is based on the Fast Nonlinear Analysis method. The program documentation “strongly recommends” using the Ritz vector method.
for this type of analysis, and cautions that it is important to use a sufficient number of Ritz vectors to capture the deformation of the non-linear elements completely [9]. A test problem, consisting of Bents 24 to 27 with the associated spans and 418 mass DOF, was analyzed to determine the sensitivity of the results to the number of Ritz modes included in the analysis. A summary of the results from this analysis is shown in Table 2. These results show that for this problem, the maximum bent shear is essentially the same with 25% or more of the total modes, although the non-linear deformations are not well captured until nearly 100% of the modes are considered. Preliminary analyses of models of larger portions of the complex (4000+ mass DOF) showed similar trends; therefore, the number of Ritz modes necessary to produce modal participating ratios near 100% in the direction of both horizontal axes, and approximately equal to 25% of the total mass DOF was selected for use in the final analyses.

<table>
<thead>
<tr>
<th>Number of Ritz modes</th>
<th>10</th>
<th>20</th>
<th>40</th>
<th>100</th>
<th>200</th>
<th>400</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total modal participating mass ratio, UX (%)</td>
<td>85.7</td>
<td>86.8</td>
<td>89.1</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>Total modal participating mass ratio, UY (%)</td>
<td>88.1</td>
<td>91.9</td>
<td>95.8</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>Non-linear dynamic modal load participation ratios (average %)</td>
<td>≈ 0.0</td>
<td>≈ 0.0</td>
<td>≈ 0.0</td>
<td>≈ 0.0</td>
<td>≈ 0.0</td>
<td>100.0</td>
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<tr>
<td>Maximum resultant shear on bent 24 (kips)</td>
<td>138.1</td>
<td>125.7</td>
<td>149.5</td>
<td>168.5</td>
<td>166.3</td>
<td>166.3</td>
</tr>
<tr>
<td>Maximum resultant shear on bent 25 (kips)</td>
<td>229.5</td>
<td>266.5</td>
<td>220.0</td>
<td>261.3</td>
<td>253.8</td>
<td>253.8</td>
</tr>
<tr>
<td>Maximum resultant shear on bent 26 (kips)</td>
<td>147.3</td>
<td>152.0</td>
<td>157.9</td>
<td>164.5</td>
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</table>

- As of the writing of this paper, final non-linear time history analysis of the structure is being conducted. These analyses include dampers, and several types of replacement isolation bearings. Properties for the isolation devices were initially developed based on preliminary designs and from data that was developed on other projects. As in previous efforts, partial models with overlapping end segments are being used for the final analysis. With each analysis, plots of input energy, kinetic and potential energy, and energy dissipated through the isolators, dampers, and modal damping are reviewed to assess the effectiveness of the solution under consideration. In addition, bent shears, displacements, and hysteretic loops for the isolation devices were reviewed, and the modifications were adjusted for subsequent analyses. Properties and capacities were also updated as previously discussed, to account for the structural modifications under consideration. It has been observed in this process that it is often best to change only one parameter in each iteration and to maintain the same number of Ritz modes over a number of analyses, in order to quantify the impact of changes in the design. Based on the work completed to date, it is likely that most, if not all of the high cost/risk bearings can be retained in the structure with the selected approach.

CONCLUSION

This paper has presented the approach to modeling and analysis for seismic retrofit of a significant double-deck bridge structure. As the project was developed, the requirements for analysis progressed from multi-mode response spectrum analysis using M-STRUDL to non-linear time history analysis with SAP2000. With each step in this progression, pre- and post processors were used for critical data manipulation and reduction of the results. In addition, test problems were used at each step to validate changes as compared to previous results.
ACKNOWLEDGMENT

The author wishes to express appreciation to the Missouri Department of Transportation for their efforts on the project, especially those of Lesley Hoffarth, Bill Stroessner, and Jeffrey Ger. The contributions to the project of Carlos Lizana, Sverdrup's project manager, and Francisco Medina of Sverdrup; Bob Herrmann and Norm Abrahamson for development of ground motions; and Glenn Fulkerson of FHWA are also gratefully acknowledged.

REFERENCES


Ranking and Assessment of Seismic Stability of Highway Embankments in Kentucky

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ABSTRACT

Loss or significant deformation of a highway embankment could prevent access to a stable bridge, damage the first span of an otherwise stable bridge, or result in blockage or loss of a critical roadway. A rapid method of classifying and ranking numerous embankments along priority routes is crucial for delineating those embankments most in need of detailed assessment. Over 400 highway embankments and slopes along priority routes in western Kentucky were evaluated and ranked to determine critical embankments requiring further attention. Information evaluated for each embankment included height, slope angle, geologic formation, depositional environment, Soil Conservation Service soil unit, anticipated groundwater depth, liquefaction potential, and expected magnitude and peak ground acceleration for the 50 year and 500 year events. The ranking system model uses geometry, geology, and soil unit to estimate the yield acceleration of each embankment. The yield acceleration is divided by the anticipated peak ground acceleration to develop a yield factor. For yield factors greater than one, the risk is indicated by the magnitude of the yield factor, with larger yield factors implying lower risk. For yield factors less than one, and taking into account event magnitude, a series of equations developed for central U.S. ground motions is used to estimate potential deformation of the embankment. While not suitable for design, the method is useful as a means of ranking critical embankments for further evaluation.

Introduction

In the 1980's, Kentucky undertook a number of initiatives to consider earthquake hazards and their mitigation with respect to the state's infrastructure. Part of this effort included the transportation infrastructure. An assessment of seismic performance and risk to the highway infrastructure resulted in the 1988 report (Allen et al., 1988) entitled "Earthquake Hazard Mitigation of Transportation Facilities." That report recommended assignment of priority routes for movement of goods and services in western Kentucky, where widespread damage could result from a major seismic event in the New Madrid Seismic Zone (NMSZ). After selection in that study, each priority route was visually surveyed to catalog natural and man-made features that could potentially hamper rescue and relief efforts in the event of a major earthquake. The features cataloged included bridges, dams, pipelines (natural gas and petroleum), power lines, high fills of more than 15 to 20 feet in height, cut slopes, signs, buildings, faults, storage tanks, trees, and active or abandoned mines. The method used for ranking of these embankments and slopes, herein titled Kentucky Embankment and Slope Stability Ranking System (KESRS) is summarized below. A preliminary summary of this work was provided in a paper by
Sutterer et al. (1998). The model and study summarized herein is an update and improvement to that previous work.

**Basis of KESR**

This study surveyed all embankments of more than five in height along priority routes in western Kentucky and ranked the embankments with respect to their predicted relative seismic stability. The designated priority routes in western Kentucky are unlimited access highways and secondary roads, as compared to limited access parkways or interstates. KESR bases the priority ranking of the embankment on estimated mechanical stability only. Factors to account for relative importance, uncertainty in loading and stability, and other issues can be easily incorporated, if desired.

KESR assumes one of three types of embankment behavior during a major seismic event: (a) no significant movement, (b) significant movement without loss of the embankment, and (c) loss of the embankment. Priority for embankments exhibiting no significant movement will be ranked using the estimated factor of safety. Embankments predicted to exhibit significant movement are ranked using estimated embankment deformation. Case (c) is only assumed to occur in the event of high liquefaction potential for the specified seismic loading, or in the event that the displacement predicted in case (c) exceeds 1 meter. All embankments under category (c) are considered high priority embankments and are ranked equally critical.

KESR requires estimation of representative ground motion, soil mechanics properties, geometry of the embankment and foundation, and a tool for estimating mechanical performance of the embankments. Following is a summary of the methods used to do this. The results are intended to provide relative ranking of the embankments, not to predict actual expected performance. Since the model used to predict mechanical behavior determines the soil properties, site geometry, and ground motion parameters that are required, the model of mechanical behavior will be addressed first.

**Limit Equilibrium Slope Stability**

The potential for slope movement to occur during an earthquake is assessed using a two dimensional limit equilibrium stability analysis developed specifically for KESR. The stability analysis is summarized by Sutterer et al. (1998). It considers critical circular and wedge shaped failures for each of the slopes using a numerical formulation of the static equilibrium for the conditions shown in Figure 1. As described in Sutterer et al. (1998), 98 pseudo static analyses of homogeneous slopes showed that seismically loaded embankments with a uniform foundation soil, and slope inclinations flatter than 1 horizontal to 1 vertical and steeper than about 4 horizontal to 1 vertical, generally failed as a base failure. Steeper slopes are generally subject to a toe circle failure in the embankment alone, a condition modeled by Figure 1a. Most highway embankments fall within the range dominated by base failures, although occasional steep embankments may be more likely to fail as a sliding wedge, so these two modes of failure were simulated in this analysis.

Following the work of Janbu (1954), but incorporating the possibility of different undrained strengths of the embankment and foundation and a horizontal $K_h$ acceleration
coefficient as shown in Figure 1, Sutterer et al. (1998) found the factor of safety, \( FS \), can be computed as:

\[
FS = \left[ \frac{R_\text{i} - R_\text{z}}{D_\text{i} + K_h \cdot D_\text{z}} \right] \cdot \frac{S_\text{i}}{\gamma \cdot H} \tag{1a}
\]

\[
R_\text{i} = 40 \cdot \sqrt{\frac{d}{r}} \cdot r \cdot (d + 12 \cdot r) \cdot (2 - \lambda) \tag{1b}
\]

\[
R_\text{z} = \sqrt{\frac{1 + d}{r}} \cdot (9 \cdot (1 + d)^2 + 40 \cdot (1 + d) \cdot r + 480 \cdot r^2) \cdot \lambda \tag{1c}
\]

\[
D_\text{i} = 40 \cdot \sqrt{2} \cdot (1 + b^2 + 3d + 3d^2 - 3r - 6dr - 3bx + 3x^2) \tag{1d}
\]

\[
D_\text{z} = 40 \cdot \sqrt{2} \cdot (b + 3bd - 2 \cdot (d \cdot (d - 2r))^{3/2} - 2 \cdot ((-1 - d) \cdot (1 + d - 2r))^{3/2} - 3br - 3x - 6dx + 6rx \tag{1e}
\]

Most of the parameters are defined in Figure 1. \( S_\text{i} \) is the undrained shear strength of the foundation soil beneath the embankment, \( \lambda \) is the ratio \( S_\gamma/S_\text{i} \), where \( S_\text{i} \) is the undrained shear strength in the embankment. The parameter \( \gamma \) is the density of the soil in both layers. For the values of \( x \) and \( r \) that result in the lowest factor of safety, designated \( x_c \) and \( r_c \), the term in brackets in equation 1a is the stability number for the designated slope.

Although a base failure predominates for the slope geometry typically encountered in highway embankments, a wedge failure extending upward from the toe of the embankment may be most critical for steeper slopes. This failure geometry is depicted in Figure 1b. For this condition, the factor of safety is indicated by:

\[
FS = \frac{2 \cdot (1 + a^2)}{(a - b) \cdot (1 + a \cdot K_h)} \cdot \frac{S}{\gamma \cdot H} \tag{2a}
\]

So that for a \( FS = 1 \), the critical \( K_h \) causing failure is found to be

\[
K_h = \frac{1}{a} \left[ \frac{2 \cdot (1 + a^2)}{(a - b)} \cdot \frac{S}{\gamma \cdot H} - 1 \right] \tag{2b}
\]

With \( a \) being the variable that is optimized for the minimum \( K_h \) and \( S \) selected as the estimated shear strength along the base of the failure wedge. Given equations (1) and (2) for estimating the yield acceleration, the minimum yield acceleration of the two is selected as being most likely for each specific embankment.

Selection of \( K_h \) for equations (1a) and (2a) is subject to judgement. The horizontal earthquake acceleration in slope stability analyses often ranges from 1/2 to 1 times the peak ground acceleration (PGA) predicted for the site. Although any
acceleration exceeding a slope's yield acceleration should cause some plastic deformation, the peak acceleration is often a single "spike" of motion of very brief duration and thus often causes little if any significant movement. A reasonable value of $K_h$ would be more on the order of 2/3 of the predicted PGA, which was the value selected for the limit equilibrium component of KESR.

Use of equation (1) in a spreadsheet with an optimization function provided reliable estimates of the above parameters over the designated slope inclinations. Specifically, the Solver® function in Microsoft Excel 97® was used to find $r_c$ and $x_c$ to minimize the factor of safety. Microsoft Excel Solver® uses the Generalized Reduced Gradient (GRG2) nonlinear optimization code developed by Leon Lasdon, University of Texas at Austin, and Allan Waren, Cleveland State University (Lasdon et al., 1978, Lasdon and Waren, 1989, Waren et al., 1987). Application of the optimization function to the approximately 400 slopes in the database described later required about one full working day of computations using a Pentium class desktop computer. To validate the analysis after the optimization, ten specific slopes were randomly selected from the database and subjected to modified Bishop stability analyses using a popular slope stability program for comparing the computed stability with the optimized stability from the ranking model. For all ten sites, the factor of safety using the optimization model matched the modified Bishop factor of safety. Given this validation of the model for limit equilibrium behavior, the optimization model factor of safety was used to rank all slopes with a computed factor of safety greater than one.

**Slope Displacement Estimate**

Using $K_h$ equal to 2/3 of the peak ground acceleration in the above limit equilibrium analysis accounts for embankments in which the seismic acceleration never exceeds the yield acceleration. That $K_h$ value also accounts for those embankments where the seismic acceleration very briefly exceeds the yield acceleration and thus results in little to no movement. Since the selected $K_h$ represents one or several brief loads during the seismic event, rather than a constant load, the question remains for those slopes with a factor of safety less than one as to how far the mass actually moved while the ground motion was taking place. The estimate of slope displacement for those embankments where FS < 1.0 was based on the results of an in-depth study of synthetic central U.S. time histories using a sliding block analysis (Newmark, 1965).

The sliding block analysis has been in use for over 30 years for assessing potential deformation of a slope or embankment due to a specific ground motion. The method assumes a slope can be simulated as a wedge resting on an inclined plane. The acceleration causing the slope to yield, $A_y$, is determined using a pseudo static analysis like that developed above with the FS set equal to 1.0. For a specific location, seismologists can provide an estimate of the peak ground acceleration expected, $A_{max}$, for an event of a specific return period. For slope movement to occur, $A_{max}$ must exceed $A_y$. As the ratio of $A_y/A_{max}$ decreases, deformation increases. The sliding block analysis is simply double integration of all portions of an earthquake time history that exceed $A_y$.

For his work, Dodds (1997) modified the desktop PC based computer program DISP (Chugh, 1980) to examine the relation between $A_y/A_{max}$ and sliding block deformations using 128 different synthetic time histories specifically developed to model
central U.S. bedrock motions (Street et al., 1996 and Wang and Street, 1997). Using the form of equation developed by Ambraseys and Menu (1988), it was assumed

$$\log_{10}(u) = \alpha + \beta_1 \log_{10}\left(1 - \frac{A_y}{A_{max}}\right) + \beta_2 \log_{10}\left(\frac{A_y}{A_{max}}\right)$$  \hspace{1cm} (3)

where \(u\) is the predicted displacement, in centimeters, of the embankment. For the 128 synthetic time histories considered, Dodds found the “bedrock” coefficients from equation (3) provided in Table 1 gave the best fit to the predicted displacements.

Dodds’ results are based on bedrock ground motion, but the soil overburden in portions of western Kentucky is often more than 30 feet thick, and is over 100’s of feet thick near the Mississippi River. Local overburden should change the ground motion and thus the displacement behavior. To investigate this, boring data and shear wave velocity profiles for nine sites throughout western Kentucky were acquired from University of Kentucky records (Street et al., 1997). The sites were selected to represent the most common combinations of alluvium, continental deposits, and bedrock depth in the region. This subsurface data was compiled for use in the computer program for one dimensional wave propagation, SHAKE91 (Idriss and Sun, 1992), to model the propagation of shear waves from the bedrock to the ground surface at each of those sites. Using the several of the referenced synthetic bedrock time histories (Street et al., 1996 and Wang and Street, 1997) for each of the nine sites as the bedrock motions, 38 additional ground surface motions were produced. The resulting ground surface (“soil”) time histories were then used in the modified DISP program prepared by Dodds (1997), with the resulting displacements being fit to equation (3) to predict embankment behavior for sites in western Kentucky with deeper overburden. The resulting coefficients from equation (3) for “soil” sites in western Kentucky are also shown in Table 1.

Since the parameters \(\alpha, \beta_1, \) and \(\beta_2\) vary with magnitude, it is possible to use an equation to predict the variation of these parameters for magnitude 5 to 7 events on both bedrock and soil sites. The following linear relations were recommended to predict the parameters, \(\alpha, \beta_1, \) and \(\beta_2\) for bedrock and soil sites in western Kentucky:

\[
(p\alpha)_{\text{bedrock}} = 0.735 \cdot M_{b,lg} - 4.41 \hspace{1cm} (4a)
\]

\[
(p\alpha)_{\text{soil}} = 1.025 \cdot M_{b,lg} - 6.292 \hspace{1cm} (4b)
\]

\[
(p\beta_1)_{\text{bedrock}} = 0.35 \cdot M_{b,lg} + 1.94 \hspace{1cm} (5a)
\]

\[
(p\beta_1)_{\text{soil}} = 0.35 \cdot M_{b,lg} + 1.94 \hspace{1cm} (5b)
\]

\[
(p\beta_2)_{\text{bedrock}} = 0.21 - 0.15 \cdot M_{b,lg} \hspace{1cm} (6a)
\]

\[
(p\beta_2)_{\text{soil}} = -0.794 - 0.056 \cdot M_{b,lg} \hspace{1cm} (6b)
\]

Equation (3) is plotted in Figure 2 using equations (4) through (6) to predict parameters for a magnitude 7 event for both bedrock and soil sites. Also shown in Figure 2 is the relation developed by Ambraseys and Menu (1988). It is noteworthy that the results for
the “soil” sites for central U.S. events closely approximates the behavior predicted by Ambraseys and Menun for predominantly western U.S. sites. Further examination of the response spectra for the bedrock time histories used by Dodds indicated a significant high frequency (10-30 Hz) component in which the PGA of the time history occurred. This is atypical for most measured bedrock motion, in which the PGA typically occurs at substantially lower frequencies. Since the Newmark method normalizes findings to the PGA, use of the synthetic time histories in which PGA occurs at unusually high frequencies is not likely appropriate. For this reason, this study used the relations shown in equations (4b), (5b), and (6b).

To complete the estimation of potential displacement, it is necessary to determine the acceleration causing yield for each embankment. This is achieved by setting the factor of safety equal to one in equations (1a) or (2a), whichever is critical, and optimizing for \( r_c \) and \( x_c \). For equation (1a), which was found to be critical in most cases:

\[
K_{hf} = \frac{(R_1 - R_2) \cdot S_1}{\gamma_1 \cdot H - D_1} \cdot \frac{S_2}{D_2}
\]

(7)

It is then assumed that \( A_y = K_{hf} \) from equation (7). For values of \( A_y/A_{max} \) less than one, some displacement would be expected to occur during the intervals when the ground acceleration exceeds the yield acceleration. The magnitude of that deformation may be quite small, but it will increase as \( A_y/A_{max} \) decreases.

By combining equations (3), (4b), (5b), (6b) and (7) in a ranking analysis, it is possible to quickly estimate potential displacements for multiple embankments. The ratio \( A_y/A_{max} \) is designated the yield factor, \( Y \), for this analysis. The displacement, \( u \), retains the dimension of centimeters because the critical displacement is generally the same regardless of embankment height, bridge length, or other characteristic dimensions of the embankment and structure.

**Mechanical behavior immediately following the event**

Cohesionless soils in the region may be susceptible to liquefaction. The cohesionless soils present are either alluvium or sandy/gravelly continental deposits. Of these, the alluvium will be the most likely to experience liquefaction. An assessment of the likelihood of liquefaction in cohesionless alluvium in the region was crucial to the study. Kentucky Transportation Cabinet (KTC) is compiling a database of soil boring information throughout the state of Kentucky (Pfalzer, 1995). While still in development, the database already includes thousands of samples, particularly in the western Kentucky region of concern for this study. This database includes sample-specific standard penetration blow counts, grain size data, ground water depth, geologic origin (alluvium) and soil classification. It was thus easy to load the database into a commercial spreadsheet for selection of those samples classified as coarse grained for further analysis of liquefaction susceptibility. A total of 489 samples were selected from the database for evaluation. Of these, 27 were classified as loess, 294 as continental clayey sands and gravels, and 168 as alluvium. The alluvium data consisted of 27 samples classified as fine grained, 77 as coarse grained, and the remainder could not be
identified as either due to lack of data. Based on this information, approximately 3/4 of the alluvial soils in the region and within reach of typical drilling programs are coarse grained.

Determination of liquefaction potential in the referenced soils was possible using the Seed et al. Method (1983) based on standard penetration test N values. Numerical correction of the N values to the equivalent 1 kg/cm² N value was achieved by using an approximation of the C_N correction summarized by Seed et al. (1981) as follows:

\[ C_N = \left( \frac{98.5}{\sigma'_v} \right)^{0.57} \]  

(8)

where \( \sigma'_v \) is expressed in kPa. Note that for \( \sigma'_v \), expressed in tons per square foot, or kg/cm², the constant 98.5 in equation (8) is replaced with 1.0. It was not possible to correct the N values in the database for SPT efficiency since details of the performance of those tests were not available.

The Seed et al Method (1983) includes recommendations for estimating the cyclic shear stress ratio, \( (\tau_{ave}/\sigma'_o) \), induced in the soil for a specific earthquake:

\[ \frac{(\tau_{ave})_ave}{\sigma'_o} = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma'_o} \cdot r_d \]  

(9)

\( a_{max} \) is the maximum earthquake ground surface acceleration and \( r_d \) is a correction factor for stress reduction. For this study, the mean effective and total stresses, \( \sigma'_o \) and \( \sigma_o \) were replaced with the effective and total vertical stresses computed using an assumption of a soil mass density of 1.92 g/cm³ and an assumed ground water level 3 meters below the surface. These are conditions that are not unreasonable for alluvial bridge abutment sites. The stress reduction factor, \( r_d \), was computed using the depth, \( z \), in meters and the following equation to estimate the correction as:

\[ r_d = (1 - \frac{z}{91}) \]  

(10)

This is a reasonable correction for depths up to 15 meters.

Determination of \( a_{max} \) in Equation (9) required interpretation of earthquake accelerations and response spectra provided by Street et al. (1996) for western Kentucky, and site periods also provided by Street et al. (1997). The latter report provides lower bound, mean, and upper bound dynamic site periods for 84 different sites throughout western Kentucky. This report also provides interpreted depths to bedrock at the various locations studied. Comparison of the site period and depths to bedrock at each site provided a good correlation, as shown in Figure 3. The depth to bedrock data from the report by Street et al. (1997) was used to infer bedrock depths for the samples in the KTC drilling samples database, and the resulting site period was subsequently determined using a linear fit to the trend indicated in Figure 3.

Once the site period was obtained, the 5% damped design response spectra recommended for each Kentucky county by Street et al. (1996) was used to estimate the amplification of the design bedrock acceleration for each county, also provided in the same report, to obtain the value of \( a_{max} \). This was done by assuming a linear variation between the log of acceleration response and the log of period at 0.1 and 1 second on the response spectra for each event as follows:
\[
\log \left( \frac{a_{\text{max}}}{a_{\text{peak}}} \right) = m \cdot \log(T_e) + d
\] (11)

The peak acceleration, \(a_{\text{peak}}\), was for each specific time history, while the value of \(a_{\text{max}}\) is that used in Equation 9. The values for the parameters used for the different time histories are provided in Table 2.

None of the sample data from the KTC database included depth to bedrock since that normally exceeds 100 meters in the western Kentucky region. The depths provided by Street et al. (1997) were based primarily on seismic refraction testing along with some data from deep placement of seismic monitoring stations. Further only, some of the KTC database data included sufficient soil test results to complete the above computations. A total of 35 sites were assessed in this way, using both the design 50 year and 500 year event. The results are shown on Figure 4 along with the liquefaction limit line for soils with less than 5% fines subjected to a magnitude 7.5 event.

Inspection of the figure quickly shows that most of the cohesionless sample data analyzed indicated a “no liquefaction” condition. Of the 35 samples assessed, 26 contained between 5% and 15% fines, so the liquefaction susceptibility of those samples was even lower than that indicated in the figure. This provided evidence that liquefaction of cohesionless alluvial soils in western Kentucky is possible, though most samples would not be considered susceptible to liquefaction.

On the basis of the above study, it was concluded that the alluvium throughout the region could be characterized as having low to moderate liquefaction potential. The undrained shear strength of the alluvium at those locations where liquefaction was believed to be a risk, was conservatively assumed to be on the order of 20 kPa as indicated for a clean sand blow count on the order of 12-16 suggested by Seed and Harder (1990). Liquefaction was assumed to not be a problem in the cohesionless continental deposits based on a review of the very high SPT N values observed therein.

**Representative ground motion**

As described previously, the displacement model developed by Dodds (1997) and summarized previously in equations 4 and 5 were used to estimate the displacement from the input Yield Factor, \(Y = A_y/A_{\text{max}}\). These equations also require earthquake magnitude, however. For illustration purposes, this portion of the study examined only the 500 year event. The 500 year event was determined to be of magnitude 7 to 7.5 located in the New Madrid Seismic Zone (NMSZ) in southeast Missouri and northeast Arkansas (Street et al., 1996 and 1997). Estimated peak bedrock accelerations, \(A_{\text{max}}\), for this event ranged from 17 to 63% of gravity, depending on the location of the embankment being considered.

**Embankment and Foundation Geometry**

Embankment geometry includes embankment slope inclination, embankment height, and width and length of the crest of the embankment. Geometry also includes depth and distribution of multiple layers in the embankment. The above analyses assume a simple two dimensional geometry like that shown in Figure 1. It is further assumed that the embankment is constructed of a single material, both the embankment top and base
are level, and that the base of the embankment corresponds to the elevation of the toe of the embankment slope. Embankment geometry was thus defined using two parameters: height, \( H \), and slope inclination, \( b \), which is the ratio of horizontal to vertical slope inclination, as shown in Figure 1.

Embankment geometry was defined by field surveys conducted by undergraduate engineering students from the University of Kentucky. Field measurements included slope distance from toe to crest, and slope inclination angle along the same interval. The measurements were made using a hand held Brunton compass to obtain slope inclination and a surveying tape for slope length. These procedures were initially checked with careful slope measurements using a level and surveying tape. Upon verifying the methods were sufficiently accurate for the typically irregular slopes, the method was adopted as standard practice for this study. Each slope was also carefully inspected for evidence of current impending failure, swampy conditions, or other terrain conditions that might be relevant to embankment stability and later assignment of stability parameters. The methods applied for this study permitted field assessment of between 8 and 12 locations per day.

Foundation stratigraphy and geometry are likely more variable than for the embankment, since the foundation is usually natural soil rather than controlled fill. The contact between softer foundation soils and a harder "bedrock" surface or stiff soils will also typically be irregular. However, definition of these conditions requires a detailed subsurface exploration, which is not possible for KESR. The slope stability component in KESR thus assumes the foundation soils have a uniform undrained shear strength, \( S_u \), usually different from the embankment soils, and that this soil is continuous to contact with a level layer of very high strength at some depth below the embankment.

**Assignment of Soil Mechanics Properties of the Embankment and Foundation**

Rapid seismic loading will cause undrained failure in cohesive soils and saturated cohesionless soils. Dry and partially saturated cohesionless soils will be subject to behavior intermediate between drained and undrained under seismic loading. Shear strengths assigned to each embankment were adjusted to reflect these cyclic loading effects. Assignment of these parameters obviously involved a level of judgement that introduces significant uncertainty. However, the relative uncertainty is likely comparable to the ground motion prediction, lower than the deformation prediction, and greater than the limit equilibrium analysis. For the purpose of ranking relative stability of the embankments, this was considered acceptable.

Soil Conservation Service (SCS) soil unit and United States Geologic Survey (USGS) geologic formation was compiled for each site. The soils predominant in the western Kentucky region are of four types: (1) alluvium, (2) weathered loess, (3) sandy and gravelly old alluvium that is quite dense and geologically referred to as continental deposits, and (4) residuum. Assignment of bedrock depth and shear strength for each embankment and underlying natural soils was based on comparison between the site-specific SCS and USGS information, the KTC drilling sample database, and the information indicating bedrock bedrock depth and elevation in Street et al. (1997). Ground water was assumed to be below the base of the embankment in the foundation.
The local site geometry and terrain was also given careful consideration in judging the likely parameters.

Assignment of shear strength for cohesionless materials was based on standard penetration resistance. As noted previously, those soils judged to have significant liquefaction potential were assigned low shear strength based on work by Seed and Harder (1990). The shear strength of the cohesive soils was selected through examination of unconfined compression data and rough correlations with standard penetration resistance obtained in the relevant deposits and reported in the referenced KTC database. These data are shown in Table 3. The density and shear strength of the embankment soils was conservatively estimated assuming marginal compactive effort may have been applied during construction of older embankments.

Findings and Conclusions

Figure 5 depicts ranking versus both factor of safety based on maximum acceleration and estimated displacement, based on the analyses herein, for approximately 100 of the sites. It is notable that the results of this study predict relatively low deformations for even the most critical embankments. However, it should be borne in mind that the study was intended as a ranking system, not as a direct interpretation of actual displacements. It will be recommended that several of the most critical embankments be selected for detailed assessment to determine the relative accuracy of the ranking model. For this specific study, it is reasonable to select those sites with predicted displacements exceeding 3 to 5 centimeters for closer examination. A more detailed deformation assessment is thus recommended for those sites, including careful site inspections by a geotechnical engineer, review of existing boring logs in that region, and geotechnical exploration to obtain at least index strengths, such as with unconfined compression (cohesive soils) or standard penetration testing (cohesionless soils).

This study showed it is possible to rapidly assess the condition and external geometry of over 400 slopes using limited personnel of minimal qualifications and simple, rugged measurement devices. Careful determination of each site’s location on USGS topographic quadrangle sheets facilitated easy determination in the office of expected SCS soil unit and geologic formation. This was further assisted by review of a database including field and lab data from hundreds of borings completed in the past decade by the Kentucky Transportation Cabinet. The combined data provided useful insights for estimating approximate conservative shear strengths and liquefaction potential of important geologic formations in western Kentucky.

As slope stability decreased, larger displacements were observed, providing a stronger indication of risky embankments than that obtained from a factor of safety analysis. The analysis eliminates the misleading condition of how to assess a factor of safety less than one, and instead forces consideration of the possible displacements that may be observed, a better indication of real behavior. The methodology will be easily adaptable for Geographic Information Systems applications and reliability based analysis for assessing failure potential of many sites rapidly.
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Lasdon, L. and Waren, A., (1989), GRG2 Microsoft Excel Solver®, University of Texas at Austin and Cleveland State University, Optimal Methods, Inc.


Table 1. Coefficients for Central U.S. Displacement Model

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<th>Magnitude Range</th>
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<tr>
<td>$5.5 &lt; M_b, L_g &lt; 6.5$</td>
<td>-0.1</td>
<td>4.09</td>
<td>-0.65</td>
</tr>
<tr>
<td>$6.5 &lt; M_b, L_g &lt; 7.5$</td>
<td>0.78</td>
<td>4.37</td>
<td>-0.86</td>
</tr>
<tr>
<td><strong>Soil Sites</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$4.5 &lt; M_b, L_g &lt; 5.5$</td>
<td>-1.044</td>
<td>2.726</td>
<td>-1.011</td>
</tr>
<tr>
<td>$5.5 &lt; M_b, L_g &lt; 6.5$</td>
<td>-0.388</td>
<td>2.503</td>
<td>-1.248</td>
</tr>
<tr>
<td>$6.5 &lt; M_b, L_g &lt; 7.5$</td>
<td>1.006</td>
<td>2.378</td>
<td>-1.122</td>
</tr>
</tbody>
</table>
Table 2. Parameters used for Estimating Local Amplification at Specific Sites

<table>
<thead>
<tr>
<th>County</th>
<th>Ballard</th>
<th>Carlisle</th>
<th>Fulton</th>
<th>McCracken</th>
<th>Graves</th>
<th>Marshall</th>
<th>Trigg</th>
<th>Lyon</th>
</tr>
</thead>
<tbody>
<tr>
<td>Return Period (years)</td>
<td>50</td>
<td>500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Approx. Magnitude (g)</td>
<td>0.3</td>
<td>0.15</td>
<td>0.09</td>
<td>0.6</td>
<td>0.4</td>
<td>0.3</td>
<td>0.19</td>
<td></td>
</tr>
<tr>
<td>d</td>
<td>-0.106</td>
<td>0.063</td>
<td>0.093</td>
<td>0.045</td>
<td>-0.111</td>
<td>-0.139</td>
<td>-0.244</td>
<td></td>
</tr>
<tr>
<td>m</td>
<td>-0.555</td>
<td>-0.361</td>
<td>-0.376</td>
<td>-0.317</td>
<td>-0.512</td>
<td>-0.553</td>
<td>-0.647</td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Selected Density and Strength Parameters

<table>
<thead>
<tr>
<th>Geologic Formation</th>
<th>Mass density (g/cc)</th>
<th>Shear Strength (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alluvium</td>
<td>1.92</td>
<td>0.20</td>
</tr>
<tr>
<td>Weathered Loess</td>
<td>1.84</td>
<td>0.35</td>
</tr>
<tr>
<td>Continental Deposits</td>
<td>2.00</td>
<td>0.75</td>
</tr>
<tr>
<td>Residuum</td>
<td>2.08</td>
<td>1.00</td>
</tr>
<tr>
<td>Embankment</td>
<td>2.00</td>
<td>0.50</td>
</tr>
</tbody>
</table>
Figure 1. Parameters defining pseudo-static (a) circular base failure and (b) single wedge failure.
Figure 2. Displacement functions for magnitude 7 event in western Kentucky.
Figure 3. Correlation between site period and depth to bedrock for sites in western Kentucky.
Figure 4. Liquefaction susceptibility of sites in western Kentucky.
Figure 5. Comparison between factor safety for ranking versus estimated displacement.
ERRATUM

Ranking and Assessment of Seismic Stability of Highway Embankments in Kentucky

Kevin G. Sutterer, P.E.

Findings and Conclusions

Of the 408 embankments evaluated in this study, 68 could not be ranked due to unusual site conditions or inadequate data. The remaining 340 sites were ranked relative to either their factor of safety or their estimated displacement using the model described herein. Figure 5 depicts estimated displacement versus factor of safety (shear capacity divided by shear demand, C/D) for the sites where displacement could be estimated. This was limited to those sites with C/D values less than or just slightly above 1.0. Each site was classified as either class A, B, C, or Z. Class Z embankments were not ranked, as noted above. Class A embankments were those sites where the estimated displacement exceeded 10 cm. Class B sites featured C/D ratios less than one, but displacements less than 10 cm. Class C embankments had a C/D ratio greater than or equal to 1.0. For the 500 year event, 35% of the embankments were rated class A while another 20% were class B. Only 1% of the embankments was class A for the 50 year event, with an additional 22% being class B. A more detailed assessment is recommended for the class A sites, plus class B sites at critical locations should also be given further attention.

This study showed it is possible to rapidly assess the condition and external geometry of over 400 slopes using limited personnel of minimal qualifications and simple, rugged measurement devices. Careful determination of each site’s location on USGS topographic quadrangle sheets facilitated easy determination in the office of expected SCS soil unit and geologic formation. This was further assisted by review of a database including field and lab data from hundreds of borings completed in the past decade by the Kentucky Transportation Cabinet. The combined data provided useful insights for estimating approximate conservative shear strengths and liquefaction potential of important geologic formations in western Kentucky.

As slope stability decreased, larger displacements were observed, providing a stronger indication of risky embankments than that obtained from a factor of safety analysis. The analysis eliminates the misleading condition of how to assess a factor of safety less than one, and instead forces consideration of the possible displacements that may be observed, a better indication of real behavior. The methodology will be easily adaptable for Geographic Information Systems applications and reliability based analysis for assessing failure potential of many sites rapidly.
ESTIMATING LIQUEFACTION POTENTIAL IN MID-AMERICA

Timothy D. Stark and Scott M. Olson
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University of Illinois @ Urbana-Champaign
205 N. Mathews Avenue
Urbana, IL 61801

ABSTRACT

Liquefaction during a major earthquake originating from the New Madrid seismic zone is likely to cause severe damage to highway structures in Mid-America. Therefore the estimation of liquefaction potential is an important step in the design process for new construction and even more important for retrofit/remedial studies for existing construction because of the potential cost to improve liquefiable soils.

The cone penetration test (CPT) offers many advantages over the standard penetration test for use in a liquefaction potential analysis. Stark and Olson (1995) and Olson and Stark (1998) present a procedure to estimate the liquefaction potential of sandy soils using CPT results and soil index properties. Relationships were presented for clean sands, silty sands, and silty sands to sandy silts based on fines content (percent passing the U.S. standard sieve #200) and/or median grain diameter, D₅₀.

Currently, research is being conducted to clarify the use of these liquefaction potential relationships in Mid-America. These efforts are focused on: (1) the use of paleoliquefaction sites, and (2) on the applicability of the magnitude correction factor (Cₘ), the depth reduction factor (r₉), and the overburden correction factor (Cₒ) to Mid-America.
EVALUATION AND SELECTION OF LIQUEFACTION MITIGATION MEASURES FOR EXISTING BRIDGES

by Harry G. Cooke and James K. Mitchell
Department of Civil and Environmental Engineering
Virginia Polytechnic Institute and State University
Blacksburg, VA 24061-0105

ABSTRACT

Systematic procedures have been developed for (1) assessing the need for ground and/or foundation improvement to mitigate liquefaction risk to bridges, (2) selecting and designing appropriate remediation measures, and (3) evaluating the effectiveness of improvements performed in the field. These procedures are presented and discussed in relation to the seismic retrofit of existing bridges.

Performance criteria used to evaluate the potential for liquefaction-induced damage to bridges are introduced. A review is provided of the suitability of different ground and foundation improvement methods for use at various abutment and pier types. Key issues in the design of ground improvement measures are highlighted including treated zone size, location, and properties. Factors related to improved ground performance are discussed, including pore water pressure migration, ground motion amplification, inertial force phasing, dynamic fluid pressures, soil-structure interaction, and lateral spreading forces. Guidelines and observations relevant to ground improvement design are included.

INTRODUCTION

Many existing bridges, particularly in the central and eastern United States, are potentially susceptible to liquefaction-induced damage during an earthquake due to the presence of loose, saturated, cohesionless soils at the sites. These deposits could lose strength due to increases in pore water pressures associated with liquefaction, which can result in loss of foundation support and large deformations.

An important part of the seismic retrofit of existing bridges includes evaluating the risk of liquefaction-induced damage, and where that risk is judged to be unacceptable, undertaking appropriate remediation measures. These remedial measures can include foundation and/or ground improvement. Characteristics of bridges that must be taken into consideration in the assessment of liquefaction and mitigation of ground failure by improvement methods include:

- Foundations above, in, or through unstable ground;
- Abrupt elevation changes at abutments;
- Isolated supports;
- Need to maintain the continuity of a long, linear structure;
- Presence of thick, unconsolidated subsoil layers;
- Sloping ground surface; and
- Underwater work.

This paper presents an overview of methods developed for (1) assessing the liquefaction risk and need for remediation at existing bridges, (2) selecting potentially feasible remediation methods and designing improvement schemes, and (3) verifying that the anticipated improvements are achieved in the field. Issues and guidelines associated with the design of foundation and ground improvement measures are discussed. The concepts presented in this paper were primarily developed as part of research performed for the Multidisciplinary Center for Earthquake Engineering Research. More details concerning the methods and related issues can be found in the draft report for that research project (Cooke and Mitchell, 1998).

ASSESSMENT OF LIQUEFACTION RISK

Performance Criteria

To establish whether the risk of liquefaction-induced damage to a bridge is unacceptable, appropriate performance criteria for the bridge must be established. These criteria typically include the stability and allowable deformations/displacements of the foundations. Although both the factor of safety for foundation stability and anticipated foundation displacements should be evaluated, it is the displacements that ultimately control the performance and should be used in the final evaluation.

Deformation criteria for bridges are typically expressed in terms of either distortion, \( \delta/l \) (where \( \delta \) is the differential settlement between bridge supports and \( l \) is the distance between supports), and/or gross horizontal and vertical movements of the bridge. AASHTO (1996) adopts the distortion and horizontal movement limits recommended by Moulton et al. (1985) which include:

- \( \delta/l \) of 0.005 for simple span bridges and 0.004 for continuous span bridges, and
- horizontal movements of 38 millimeters (mm) with small vertical movements and 25 mm with larger vertical movements.

Similar deformation criteria have been recommended by Lok and Mitchell (1994) and Duncan and Tan (1991), with the exception that Duncan and Tan suggest increasing the \( \delta/l \) limit of simple span bridges to 0.008. These criteria are for the service limit state and include all post-construction deformations.

Youd (1998) and Youd et al. (1998) provide a range for limits of liquefaction-induced ground deformations that typically are not damaging to bridges. Selection of appropriate limits from the ranges given below is dependent on the age and structural details of the bridge, with larger values more likely being used for well-built, modern bridges.

- Ground settlement of 25 to 100 mm for shallow foundations supported above or within a liquefiable layer and 100 to 200 mm for deep foundations extending through a
liquefiable layer into a non-liquefiable layer capable of supporting the entire foundation load.

- Lateral ground displacement of 100 to 200 mm.

Unlike deformation criteria, specific recommendations for foundation stability safety factors for earthquake and post-earthquake loading cases are not given in many bridge codes. Typically, safety factors for these cases should range somewhere between 1.1 and 1.5 depending on the conservativeness of assumptions and parameters used in analyses. The factor of safety for stability during earthquake loading is only a preliminary indication of performance, because it may not accurately reflect the deformations that can develop.

**Assessment Procedure**

Evaluating the liquefaction risk for existing bridges involves (1) establishing if seismic loading and soil conditions exist that will likely result in liquefaction, (2) predicting the stability safety factor and deformations for the bridge foundations, and (3) assessing whether the safety factor and/or deformations are within allowable limits. Assessment procedures that incorporate these evaluation steps have been developed in flow chart form for shallow foundations (above or partially in liquefiable soils) and deep foundations (extend completely through liquefiable strata) by Cooke and Mitchell (1998). The overall premise of the flow chart procedure is the evaluations should proceed from simple to progressively more complex analyses until the need for liquefaction remediation is established. Analytical methods for evaluating liquefaction potential, stability, and deformations, as well as methods for obtaining relevant parameters and soil properties, are recommended in the charts.

**SELECTION AND DESIGN OF REMEDIAL MEASURES**

At existing bridges where the risk of liquefaction-induced damage is evaluated and judged to be unacceptable, remediation measures will need to be considered to improve performance. These remediation measures may include ground and/or foundation improvement methods.

**Bridge Configuration**

The feasibility of different improvement methods is dependent in part on the configuration of a bridge. Inherent in the bridge configuration are the abutments and piers with their supporting foundations; approach embankments; span lengths and widths; and limited clearance restrictions. Site factors that influence the bridge configuration and also impact the feasibility of remediation methods include the ground surface topography and the presence of a water body, structures, or roadways.

Since it is not possible to discuss the feasibility of using different ground and foundation improvement methods for all bridge configurations, their feasibility are discussed relative to some commonly used abutment and pier types, as listed below.
Abutments (refer to Fig. 1): Pile-supported stub, full-height wingwall, and floating stub. Piers (refer to Fig. 2): Solid-wall or multi-column, hammerhead, and single column circular.

These abutment and pier types are typical for small (6 to 15 meter spans) to medium (15 to 60 meter spans) bridges. Since the majority of bridges in the United States fall in these categories, the discussion of remediation method applicability presented herein is focused on these two sizes.

**Improvement Options**

Several types of ground and foundation improvements can potentially be used to mitigate the effects of liquefaction on existing bridges. The objective of improvement is to provide satisfactory support for key bridge components during and immediately after seismically-induced liquefaction of surrounding soils, so that unacceptable bridge damage is prevented. Ground improvement achieves this objective primarily by providing mechanisms that either increase the soil shear strength or decrease the shear stresses induced in potentially liquefiable soils around the bridge foundations. Foundation improvement, on the other hand, typically involves installing additional foundation elements so that deformations and loads on the bridge system are within acceptable limits, even if the surrounding ground has liquefied.

The applicability and feasibility of ground and foundation improvement methods are dependent on:

- Space and geometry limitations and ground surface conditions affecting accessibility and working space for the construction equipment,
- Potential damage to the bridge due to anticipated movements and/or vibrations during improvement operations,
- Adequacy of the anticipated improvement to mitigate liquefaction-induced damage, and
- Cost.

**Ground Improvement**

Ground improvement methods that are potentially applicable for remediation of liquefiable soils at existing highway bridges include:

- Grouting - compaction, permeation, jet;
- Vibro systems - vibratory probe, vibro-compaction, vibro-replacement (stone columns);
- Reinforcement - Root piles, mix-in-place walls and columns;
- Surcharge and Buttress Fills; and
- Drains.

Other methods, such as deep dynamic compaction, are not generally feasible due to the constraints imposed by the presence of the bridge.
FIGURE 1 - Some Typical Bridge Abutments (from Mitchell and Cooke, 1995)
FIGURE 2 - Some Typical Bridge Piers (from Mitchell and Cooke, 1995)
The characteristics and applicability of the potential ground improvement methods for liquefaction mitigation are summarized in Table 1. Included in the table are a brief description of the principle behind each method, soil types for which it is suitable, properties of the treated material, relative cost, applicability to the different abutment and pier types shown in Figs. 1 and 2, and general comments. The stated applicability of an improvement method is subjective, based only on the limitations imposed by space and geometry factors and the potential for damage to the structure due to movements or vibrations induced by the improvement procedures. The adequacy of any method to achieve the required level of ground improvement and the cost for improvement are dependent on site specific conditions; therefore, these factors are not included in the assessment of general applicability given in Table 1.

Review of Table 1 indicates grouting methods have high applicability to many of the abutment and pier types. A major advantage of grouting techniques, particularly compaction and permeation grouting, is that little ground support loss or settlement occurs beneath the foundation during the process. Grouting operations can generally be performed in small work areas and on slopes. Inclined holes can be drilled and grouted allowing difficult zones to be reached. New developments in directional drilling enable treatment of zones around and beneath footings and piers that previously were inaccessible. Significant increases in the relative density of loose, granular soils can be achieved with compaction grouting. Permeation and jet grouting can usually produce improved soils having high strength.

Significant improvements in the relative densities of loose, granular soils can also be achieved with vibro system methods. However, these methods can cause significant settlement of the ground being treated unless extreme care is taken during the installation process. Therefore, the use of vibro systems are generally restricted to creating improved zones around existing deep foundations, which extend completely through the liquefiable soils, provided (1) the foundation and superstructure will not be damaged by construction vibrations and (2) the foundation can safely withstand the temporary loss of support and subsequent downdrag forces which may occur during densification. Additional benefits of vibro-replacement are the reinforcement of the soil mass by the densified columns, as well as drainage, if a gravel backfill is used.

Surcharge fills have moderate to low applicability for remediation of liquefiable soils when used solely to increase the effective stress in the foundation soils. Buttress fills located to intercept potential earthquake and post-earthquake stability failure surfaces can provide a significant stabilizing effect. The size of the surcharge or buttress fill may be restricted by space limitations, and the potential for foundation damage due to settlement and increased loading must be considered.

Soil reinforcement using Root piles may have only moderate applicability for improvement of the foundation soils around existing bridge foundations. On the other hand, mix-in-place walls or columns may be effective in confining and/or reinforcing liquefiable soils. However, care must be exercised during the installation process to ensure that the low strength of the disturbed zone, prior to set-up of the treated soil, does not impair the stability or cause settlement of the existing foundation.
<table>
<thead>
<tr>
<th>Method</th>
<th>Principle</th>
<th>Most Suitable Soil Types</th>
<th>Properties of Treated Soil</th>
<th>Relative Costs</th>
<th>Applicability to Abutment Stub</th>
<th>Full-Height Wingwall</th>
<th>Applicability to Piers</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compaction Grout</td>
<td>Highly viscous grout acts as spherical hydraulic jack when pumped under high pressure resulting in densification.</td>
<td>Compressible soils with some fines</td>
<td>Increased $D_1$, SPT: $(N_v)<em>{90} = 25$ to $30$ CPT: $q</em>{vo} = 80$ to $150$ tsf (Kg/cm$^2$)</td>
<td>Low material, High injection</td>
<td>1. High. Treat anywhere between abutment and embankment toe; under and around foundation.</td>
<td>1. Generally high. Treat under and around foundation.</td>
<td>High for solid wall, multi-column, and hammerhead piers.</td>
<td>High to moderate for circular column piers. Must control heave and/or hydraulic fracture of soil. Particulate and chemical grouting: verify size and strength of grouted soil mass. Jet grouting: stage work to limit settlements. Evaluate potential damage to piles from jetting pressure.</td>
</tr>
<tr>
<td>Particulate Grouting</td>
<td>Penetration grouting: fill soil pores with cement, soil and/or clay.</td>
<td>Clean, medium to coarse sand and gravel</td>
<td>Cement grouted soil: high strength</td>
<td>Lowest of grouting systems</td>
<td>2. High. Treat around pile groups.</td>
<td>2. High. Treat around pile groups.</td>
<td>1. Treat under and around foundation.</td>
<td>2. Treat around pile groups.</td>
</tr>
<tr>
<td>Chemical Grouting</td>
<td>Solutions of two or more chemicals react in soil pores to form a gel or solid precipitate.</td>
<td>Medium silts and coarser</td>
<td>Low to high strength</td>
<td>High to very high</td>
<td>2. Treat around pile groups.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jet Grouting</td>
<td>High speed jets at depth excavate, inject and mix stabilizer with soil to form column or panels.</td>
<td>Sands, silts and clays</td>
<td>Solidified columns and walls</td>
<td>High</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: 1. Foundations over or in liquefiable soils. 2. Pile (or drilled shaft) foundations extending through liquefiable soils.
The characteristics and applicability of the potential ground improvement methods for liquefaction mitigation are summarized in Table 1. Included in the table are a brief description of the principle behind each method, soil types for which it is suitable, properties of the treated material, relative cost, applicability to the different abutment and pier types shown in Figs. 1 and 2, and general comments. The stated applicability of an improvement method is subjective, based only on the limitations imposed by space and geometry factors and the potential for damage to the structure due to movements or vibrations induced by the improvement procedures. The adequacy of any method to achieve the required level of ground improvement and the cost for improvement are dependent on site specific conditions; therefore, these factors are not included in the assessment of general applicability given in Table 1.

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<th>Applicability to Piers</th>
<th>Comments</th>
</tr>
</thead>
</table>
| Compaction Grout     | Highly viscous grout acts as spherical hydraulic jack when pumped under high pressure resulting in densification. | Compressible soils with some fines           | Increased D<sub>i</sub>  
SPT: (N)<sub>i</sub><sub>soil</sub> = 25 to 30  
CPT: q<sub>p</sub><sub>i</sub> = 80 to 150 tfs (Kg/cm<sup>2</sup>) | Low material. High injection.           | 1. High. Treat anywhere between abutment and embankment toe; under and around foundation. | 1. Generally high. Treat under and around foundation. | High for solid wall, multi-column, and hammerhead piers.  
Particulate and chemical grouting: verify size and strength of treated soil mass. |
| Particulate Grouting | Penetration grouting: fill soil pores with cement, soil and/or clay.       | Clean, medium to coarse sand and gravel      | Cement grouted soil: high strength | Lowest of grouting systems | 2. High. Treat around pile groups          | 2. High. Treat around pile groups. | Jet grouting: stage work to limit settlements. Evaluate potential damage to piles from jetting pressure. |
| Chemical Grouting    | Solutions of two or more chemicals react in soil pores to form a gel or solid precipitate. | Medium silts and coarser                    | Low to high strength      | High to very high    | 2. High. Treat around pile groups          | 2. High. Treat around pile groups. | Jet grouting: stage work to limit settlements. Evaluate potential damage to piles from jetting pressure. |
| Jet Grouting         | High speed jets at depth excavate, inject and mix stabilizer with soil to form column or panels. | Sands, silts and clays                      | Solidified columns and walls | High                |                          |                          | Notes: 1. Foundations over or in liquefiable soils.  
2. Pile (or drilled shaft) foundations extending through liquefiable soils. |
<table>
<thead>
<tr>
<th>Method</th>
<th>Principle</th>
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<th>Full-Height Wingwall</th>
<th>Applicability to Piers</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vibro-Compaction</td>
<td>Densification by vibration and compaction of backfill at depth.</td>
<td>Sand (&lt;20% passing No. 200 sieve)</td>
<td>$D_r$: up to 85 + % SPT: $(N_s)<em>60 = 25$ to $30$ CPT: $q</em>{cl} = 80$ to $150$ tsf $(Kg/cm^2)$</td>
<td>Moderate</td>
<td>1. Low. Potential for excessive settlement and vibrations of bridge. Overhead clearance limitations.</td>
<td>2. Low. Treating around piles difficult due to access problems.</td>
<td>2. Moderate to high. Treat around pile groups.</td>
<td></td>
</tr>
<tr>
<td>Vibro-Replacement</td>
<td>Densely compacted gravel columns provide densification, reinforcement, and drainage.</td>
<td>Soft silty or clayey sands, silts, clayey silts</td>
<td>Increased $D_r$, SPT: $(N_s)<em>60 = 25$ to $30$ CPT: $q</em>{cl} = 80$ to $150$ tsf $(Kg/cm^2)$</td>
<td>Moderate to high</td>
<td>1. Low. Treating around piles difficult due to access problems.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Surcharge/Buttress Fill</td>
<td>Weight of surcharge increases liquefaction resistance by increasing effective stresses in ground. Buttress fill increases post-earthquake stability by reducing overturning moment and increasing resisting moment.</td>
<td>Any soil surface provided it will be stable</td>
<td>Increase in strength</td>
<td>Low</td>
<td>1. High for slope stabilization; low for settlement. Place buttress fill at toe of embankment.</td>
<td>1 &amp; 2. Moderate. Place buttress fill in front of wall.</td>
<td>1 &amp; 2. Moderate to low. Place surcharge around pier.</td>
<td>Need large area. Evaluate loads and settlement imposed on bridge.</td>
</tr>
</tbody>
</table>
### TABLE 1 (cont.)
SUMMARY OF GROUND IMPROVEMENT METHODS FOR LIQUEFACTION REMEDIATION AT EXISTING BRIDGES

<table>
<thead>
<tr>
<th>Method</th>
<th>Principle</th>
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<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Root Piles</td>
<td>Small-diameter inclusions used to carry tension, shear and compression.</td>
<td>All soils</td>
<td>Reinforced zone behaves as a coherent mass</td>
<td>Moderate to high</td>
<td>1. Moderate to low. Zone for installing piles same as that described for for grouting. 2. Moderate to low. Install piles around pile groups.</td>
<td>1. Moderate to low. Install piles beneath and around foundation. 2. Moderate to low. Install piles around pile groups.</td>
<td>1. Moderate to low. Install piles around pier foundation. 2. Moderate to low. Install piles around pier groups.</td>
<td>Extend piles to firm strata. Large number of piles may be required to provide adequate reinforcement. Avoid damage to existing piles.</td>
</tr>
<tr>
<td>Mix-In-Place Walls &amp; Columns</td>
<td>Lime, cement or asphalt introduced through auger or special in-place mixer.</td>
<td>All soft or loose soils</td>
<td>Solidified soil walls or columns of relatively high strength confine and/or reinforce potentially liquefiable soils</td>
<td>High</td>
<td>1. Moderate for lateral spreading; low for settlement. Install along toe of embankment. 2. Low. Difficult to install around pile group supporting abutment.</td>
<td>1. Moderate for lateral spreading, low for settlement. Install at toe of wall. 2. Moderate to low. Install around pile group supporting abutment.</td>
<td>1. Moderate to low. Install completely around pier. 2. Moderate to low. Install completely around pier pile groups.</td>
<td>Extend to firm strata. Stage work to control construction settlements. Space limitations may restrict use. Construction in water requires special procedures.</td>
</tr>
<tr>
<td>Drains: Gravel Sand Wick</td>
<td>Relief of excess porewater pressure to prevent liquefaction. Intercept and dissipate excess porewater pressure plumes from adjacent liquefied soil.</td>
<td>Sand, silt</td>
<td>Improved drainage</td>
<td>Low to moderate</td>
<td>1&amp;2. Moderate. Install drains around zone improved by other method(s).</td>
<td>1&amp;2. Moderate. Install drains around zone improved by other method(s).</td>
<td>1&amp;2. Moderate. Install drains around zone improved by other method(s).</td>
<td>Topography and space limitations may restrict use.</td>
</tr>
</tbody>
</table>
Theoretical analyses and model tests have shown that vertical gravel drains can prevent the build up of excess pore water pressures to levels that cause complete liquefaction. However, the drains must be closely spaced and the soil being protected must be fairly coarse and have a high hydraulic conductivity. Furthermore, the drainage could be accompanied by unacceptable settlements. Thus drains by themselves are likely to be useful for liquefaction prevention beneath an existing bridge foundation only under special conditions. On the other hand, drainage zones can be used to intercept post-earthquake pore water pressure plumes migrating from liquefied zones adjacent to bridge foundations into improved soils beneath the foundations. The drains thus prevent pore water pressure build up and strength loss in a foundation soil that otherwise would be stable (i.e. - not susceptible to large deformations due to the earthquake shaking).

Foundation Improvement

Mini-piles or pin-piles connected to an existing bridge foundation and extending through a liquefiable soil stratum into a non-liquefiable bearing stratum may be a suitable means for liquefaction risk mitigation. These piles, along with any other existing piles/shafts extending through the liquefiable stratum, must be capable of withstanding the loads imposed on them by the structure, as well as those from deformation of the ground. The performance of these foundation underpinning systems will depend strongly on effectively tying them into the existing foundation system.

Design Issues

Design issues to be considered for foundation improvement include: (1) type, number, and size of new foundation elements to be added, (2) location of the new elements, and (3) interaction of the elements with existing foundations. When designing improvements that include additional deep foundation elements, the engineer should account for loss of foundation support due to the strength and stiffness degradation of liquefiable and non-liquefiable soils under cyclic loading. Deep foundations should also be designed for downdrag loads associated with soil densification and resulting settlement.

The design of ground improvement should include consideration of: (1) appropriate size of treated zones, (2) location of treated zones relative to the bridge foundations, and (3) the level of improvement in soil properties required. Factors related to these issues which affect the performance of improved ground and the supported bridge are:

- Pore water pressure migration (Fig. 3a): During and after an earthquake, pore water pressures will be higher in a liquefiable zone than an adjacent improved ground zone. The difference in pressures between the two zones induces flow into the treated zone, causing an increase in pore water pressure and a potential decrease in strength, particularly in densified ground.

- Ground motion amplification (Fig. 3b): The earthquake-induced ground motion at the base of an improved ground block may be amplified as shear waves travel upward through the block, resulting in more severe loading on the supported structure.
Figure 3 - Factors Affecting Improved Ground Performance
(d) Dynamic Fluid Pressure

Static Pressure = \gamma h
Dynamic Pressure = \frac{7}{8} \frac{a}{g} \gamma (y h)^{0.5}

where:  
- \(a\) = maximum acceleration
- \(g\) = gravitational acceleration
- \(\gamma\) = unit weight of liquefied soil

(e) Lateral Spreading Forces

Figure 3 (cont.) - Factors Affecting Improved Ground Performance
• Inertial force phasing (Fig. 3c): During an earthquake, differences may exist between the times when the maximum inertial forces act on the improved ground and supported structure. This difference will influence the severity of loading on the system.

• Dynamic fluid pressure (Fig. 3d): Movement of a relatively stiff body, such as an improved ground zone, within a fluid medium, such as a liquefied soil, may result in a dynamic component of fluid pressure acting on the soil body in addition to the static fluid pressure.

• Influence of structure: Placement of a structure on an improved ground zone alters the stress state within the zone and can thereby affect the dynamic response and stress-strain behavior of the improved soil.

• Lateral spreading forces (Fig. 3e): In areas of sloping ground, additional pressures/forces can be exerted on the improved ground zone as surrounding unimproved soil undergoes lateral spreading and moves relative to the treated zone.

These factors are discussed in greater detail in Mitchell et al. (1998). Unfortunately, generally accepted methods for evaluating all of them are not yet available and agreed upon, and they remain the subject of continued research. Nonetheless, some design guidelines on ground improvement design appear warranted based on information from analytical studies, physical modeling (e.g., centrifuge and shaking table tests), and field experiences.

Design Guidelines

Some relevant observations and rough guidelines for ground improvement design include:

1. Improvement of liquefiable soil beneath most structures should generally extend to the bottom of the liquefiable material.

2. Where possible, the lateral extent of treatment beyond the edge of the structure should be a distance equal to the depth of treatment beneath the structure, although some shaking table test data (Hatanaka et al., 1987) suggest the distance could be as small as one-half the depth.

3. Laboratory model tests indicate there may be an optimum depth and width of treatment around a structure or footing beyond which the reduction of settlements is negligible (Hatanaka et al., 1987; Liu and Dobry, 1997). Centrifuge tests indicate that increasing the depth of treatment can increase the acceleration of a supported footing or structure (Liu and Dobry, 1997).

4. Remediation for reducing the lateral deformations of existing embankments is generally more effective when the foundation soils are treated in a zone that is between the crest and toe of the embankment (Riemer et al., 1996).
5. Improvement schemes which provide continuous, rigid confinement (e.g. - enclosures formed with steel sheetpiling or mix-in-place walls) of soils that liquefy beneath an embankment typically result in better performance with respect to deformations than those which do not (Adalier et al., 1998).

6. Experience in the 1995 Kobe earthquake showed that mix-in-place columns were particularly effective for supporting structures in liquefiable soil. Good performance was observed when the structures were supported directly on the columns or contiguous columns were used to form cells for containment of the liquefiable soil beneath the structures.

7. The perimeter of zones improved by densification generally become unstable during earthquake shaking and liquefaction. The portion of the densified zone enclosed by the vertical boundary of the treated zone and a line drawn at an angle of 30° to 35° to the vertical from the bottom of the boundary (Figure 3(a)) can be treated as liquefied material for design purposes (Iai et al., 1988; Akiyoshi et al., 1994).

8. Positive pore water pressure may develop in a densified soil zone during and after an earthquake. This pressure is generated by both induced shear strains within the zone and migration of pore water pressures from surrounding liquefied soil. The rate of pore water pressure migration depends primarily on the permeability and compressibility properties of the treated soil, with faster migration occurring for higher permeability and lower compressibility. In cases where the rate of pore pressure migration is rapid and the earthquake duration is long, maximum pore pressures can be estimated reasonably using steady-state seepage analyses.

9. Lateral spreading forces exerted on an improved zone by an unliquefied surface layer can be estimated using passive earth pressure theory (Berrill et al., 1997). Where there is crust of significant thickness (i.e. - approximately one meter or more), the lateral spreading force from the crust will likely be substantially greater than that from the underlying liquefied soil.

10. Inertial forces acting on an improved soil mass should be included when evaluating the stability of an improved ground zone supporting a structure. Guidelines are generally not available for estimating pseudo-static coefficients to represent these inertial forces in simplified stability and deformation analyses. Although estimates of the coefficients can be made from dynamic response analyses using programs such as QUAD4M (Hudson et al., 1994), some of these programs do not account for important effects associated with time-dependent changes in frequency response and ground conditions. Therefore, the results from these analyses can potentially be unconservative.

11. Whether densification improvement by itself will be adequate to prevent liquefaction in the design earthquake can be evaluated using the "simplified method" for liquefaction potential assessment (NCEER, 1997) in which the standard penetration test \( (N_1)_{60} \) values,
cone penetration test $q_{60}$ values, and/or shear wave velocities for the treated ground are used. Field performance data suggest that liquefaction effects on structures will be minor when the supporting ground is improved to the "no liquefaction" side of liquefaction potential curves (Mitchell et al., 1995).

These guidelines and observations provide the engineer with some simple "rules of thumb" that can be used in the preliminary design process. However, for major projects more detailed analyses may be required for refinement of the design and verification of expected performance.

**Design Procedure**

A suggested design procedure for selecting a remedial measure scheme for mitigating liquefaction effects at an existing bridge is outlined in Figure 4. The steps in the procedure include a sequence of evaluations that proceed from simple to more detailed, allowing a preferred method of improvement to emerge by the end of the design process through comparison and elimination of alternatives. At any point during the process, the designer may decide to return to an earlier step and perform another iteration of previously performed steps, perhaps based on new information or the need for further refinement in the analyses. Brief descriptions of the steps are given below; more detailed descriptions can be found in Cooke and Mitchell (1998).

![Flowchart](image)

**Figure 4 Remedial Measure Design Procedure**
Performance Requirements

Criteria are established for the expected performance of the bridge being evaluated. These criteria generally consist of deformation limits for the bridge foundation, as well as some minimum factor of safety for stability. Stability and deformation criteria should be established for remedial measure construction, earthquake loading, and post-earthquake cases. In addition to stability and deformation criteria, there may also be limits on acceptable accelerations for the improved ground and supported foundations. Although the stability and deformation values given in the performance criteria section of this paper serve as a good starting point, project specific values may be needed in many cases.

Preliminary Selection

Preliminary selection of potential ground and/or foundation improvement methods is generally made on a qualitative basis, taking into consideration the bridge performance criteria and general knowledge of the level of improvement that can be achieved with different methods, as well as limitations imposed by site and subsurface conditions. Table 1 is one guide that can be used in this selection process. At this stage, the objective is to eliminate only those methods that cannot be used by themselves or in conjunction with other methods to produce the desired bridge performance.

Preliminary Design

Estimates are made of the size, location, and level of improvement needed for the treatment zones at the abutments and piers of the bridge. These estimates can be made using rough design guidelines, as well as some limited analyses to evaluate performance (i.e. — stability and deformations). The analyses performed in preliminary design will be dependent on the criticality and nature of the bridge, with simple analyses generally being performed for non-critical, standard bridges and more complex analyses performed for critical, unique bridges. Although simple analytical techniques are available (Mitchell et al., 1998), they generally have limitations such as difficulty in adequately representing time dependent changes in frequency response and ground conditions. Simple methods are the subject of continued research and verification. Complex methods can include dynamic analyses.

Selection for Final Evaluation

Different improvement schemes are compared to each other in terms of expected performance, relative cost, ease of construction, and other factors the engineer deems relevant. Based on this comparison one or more schemes, which are deemed to be more desirable, are selected for final design.

Final Design

Complete detailed designs of the selected improvement schemes are developed. This work includes more refined stability and deformation analyses to finalize the size, location, and extent of
ground and/or foundation improvements. Based on this work, detailed improvement plans and cost estimates are developed. Analyses for major projects may include the use of finite element or finite difference computer codes with dynamic capabilities. In addition, non-linear, elastic-plastic constitutive soil models and pore water pressure generation formulations can be used in these analyses.

*Comparison and Selection*

The preferred improvement scheme for reducing liquefaction-induced damage to the bridge is selected taking into consideration constructability, expected bridge performance, cost, and other factors relevant to the project.

**VERIFICATION OF IMPROVEMENTS**

Quality assurance and control (QA/QC) procedures to verify that the desired ground and/or foundation improvements have been achieved are essential. These procedures generally incorporate a combination of construction observations, in-situ testing, and laboratory testing to evaluate the treated soils in the field. Construction observations can consist of monitoring the effort or energy expended during improvement operations, types and quantities of materials used, and changes in the ground elevation. These observations provide a preliminary indication of where improvement was or was not successful. In-situ testing, particularly the standard penetration test (SPT), cone penetration test (CPT), and shear wave velocity measurements are used to assess properties of soils improved by densification methods. Laboratory tests are useful for testing the quality of materials produced during the improvement process (e.g. soil-cement from mix-in-place operations).

**CONCLUSIONS**

An important part of the seismic retrofit of existing bridges is the evaluation of the risk of liquefaction-induced damage, and where that risk is judged to be unacceptable, designing and implementing appropriate ground and/or foundation improvements. Procedures have been developed and presented for executing this process in the following three phases: (1) assessing the liquefaction risk and need for remediation, (2) selecting and designing improvement schemes, and (3) verifying that the anticipated improvements are achieved in the field using QA/QC methods. Conclusions regarding these three parts of the process are:

- Assessment of liquefaction risk and the need for remediation can be efficiently performed using a series of evaluations for assessing whether seismic loading and soil conditions are conducive to liquefaction and, if so, estimating stability safety factors and ground deformations. Although the factor of safety for stability and deformations are both evaluated, deformations ultimately determine acceptable performance.

- The feasibility of various ground and foundation improvement methods for liquefaction mitigation are dependent on a combination of site, bridge, and ground conditions. Important characteristics and the potential feasibility of different ground and foundation improvement
methods for liquefaction mitigation have been evaluated and summarized. Grouting is an effective ground improvement method for bridges founded over or partially in liquefiable soils. Vibro system methods are useful for improving soils around deep foundations extending through liquefiable soils. Soil reinforcement methods (Root piles and mix-in-place walls and columns) and surcharge fills may have only moderate to low applicability for liquefaction mitigation in the soils around bridge foundations. Buttress fills can be useful for increasing the post-earthquake stability of abutments. Drains are useful for preventing the migration of excess pore water pressure plumes from unimproved to improved ground zones. Foundation improvement using mini- or pin-piles can potentially be used provided they can be installed and properly tied into the existing foundation system.

- A systematic procedure for selecting and designing ground or foundation improvement schemes to mitigate liquefaction risk include establishment of performance requirements for the bridge, preliminary selection of improvement methods, preliminary design of improvement schemes, selection of schemes for final evaluation, final design of schemes, and comparison and selection of a preferred scheme. Some guidelines are presented for (1) selecting the location, size, and level of ground improvement; (2) accounting for the effects of pore pressure migration; (3) including inertial effects; and (4) estimating lateral spreading forces. Additional study of these factors is needed, as well as the effects of inertial force phasing, dynamic fluid pressures, influence of structures, and ground motion amplification.

- Verification of ground and foundation improvements during construction is an integral part of liquefaction remediation work. In-situ tests are particularly useful for evaluating densification improvement.

Remediation of liquefiable soils is a challenging aspect of the seismic retrofit of existing highway bridges. As additional studies are conducted, progress will continue to be made in developing practical means for designing improvement schemes to mitigate liquefaction effects.

ACKNOWLEDGEMENTS

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REFERENCES


SEISMIC DESIGN OF FOUNDATION

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ABSTRACT

Seismic design of foundations includes analyses of both the dynamic stiffness and damping of individual foundation elements and the overall stability of the foundation system. Evaluation of the dynamic modulus of the soil, including consideration of confining pressure and strain softening effects, is a key consideration in evaluating the dynamic response of shallow foundations. Evaluation of group effects is a key consideration in analyses of the seismic response of deep foundations. Stability considerations include both bearing capacity and translational (sliding) stability. Important considerations with respect to overall stability include load combinations, the appropriate value for the dynamic shear strength of the soil, allowable local yielding of foundation elements, and the reliance on passive earth pressure in evaluation of sliding stability. The residual shear strength of the soil may also be an important consideration in evaluating post-earthquake overall stability.

INTRODUCTION

Seismic design of foundations for bridges and other highway structures involves evaluation of both the dynamic stiffness and damping of the foundation elements and overall stability of the foundation. Due to soil-structure interaction, optimal design requires good communication between the structural and geotechnical engineer. The structural engineer requires input on foundation stiffness in order to calculate the dynamic loads. To calculate foundation stiffness, foundation dimensions must be known. However, optimal dimensioning of the foundation may depend upon the dynamic loads. Therefore, at least one iteration is required to converge on the optimal solution.

Evaluation of the dynamic stiffness and damping of foundation elements relies upon closed form solutions from elasticity theory for shallow foundations and upon “p-y” and “t-z” analyses for deep foundations. Key issues in evaluation of the dynamic stiffness and damping of foundation elements include evaluation of the appropriate soil modulus for shallow foundation response and group effects for lateral loading of deep foundations. Evaluation of overall seismic stability
typically relies upon conventional geotechnical analyses of foundation capacity under vertical and lateral loads. Key issues in evaluation of the overall stability of a foundation subject to earthquake loading include the shear strength of the soil under dynamic loads, the ability of the structural system to accommodate local yielding of foundation elements, and the post-earthquake residual shear strength of liquefiable soil.

THE DESIGN PROCESS

Ideally, seismic design of foundations is a cooperative effort between the structural and geotechnical engineer. Because of the interdependence of foundation dimensions and foundation stiffness, seismic foundation design should also be an iterative process. At least one, and sometimes more than one, cycle of foundation dimensioning is required for optimal foundation design. Figure 1 illustrates the process for seismic design of foundation.

![Seismic Foundation Design Process Diagram](image)

Figure 1 - Seismic Foundation Design Process

As illustrated in Figure 1, initial foundation dimensions are developed based upon preliminary loads provided by the structural engineer. These preliminary loads are often based upon static loads, past experience, and/or typical values. The geotechnical engineer then evaluates the dynamic stiffness and damping of the foundation elements and provides these values to the structural engineer. The structural engineer next performs a dynamic response analysis to evaluate the dynamic loads. The dynamic loads are then provided back to the geotechnical
engineer, who checks the adequacy (overall stability) of the foundation elements subject to the dynamic loads. If necessary, foundation dimensions are adjusted, foundation stiffness is re-evaluated, the new stiffness is passed back to the structural engineer, the dynamic loads are re-evaluated, and the geotechnical engineer evaluates foundation adequacy under the new dynamic loads. This process continues to convergence. However, more than one iteration is rarely needed for convergence and initial dimensions based upon static loads are often sufficient.

STIFFNESS AND DAMPING

Shallow Foundations

Dynamic stiffness and damping for shallow foundations are generally evaluated using closed form solutions for circular footings from elasticity theory. Figure 2 illustrates the six modes of deformation for which stiffness and damping are evaluated. Analytically, the stiffness of a footing is represented by the $6 \times 6$ matrix of stiffness coefficients illustrated in Figure 2.

Figure 2 - Degrees of a Footing and Corresponding Stiffness Matrix

Each coefficient $k_{ij}$ of the stiffness matrix describes the deformation of the footing in the $i$ direction in response to a unit load in the $k$ direction. Table 1 presents the equations from elasticity theory for the stiffness coefficients for a circular footing of radius $R$ for the cases in which $i$ equals $j$ in terms of the soil shear modulus, $G$, and Poisson's ratio, $\nu$. 

3
Table 1 - Stiffness Coefficients for Circular Footings

<table>
<thead>
<tr>
<th>Mode</th>
<th>Vertical</th>
<th>Horizontal</th>
<th>Rocking</th>
<th>Torsion</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(\frac{4GR}{(1-v)})</td>
<td>(\frac{8GR}{(2-v)})</td>
<td>(\frac{8GR^3}{3(1-v)})</td>
<td>(\frac{16GR^3}{3})</td>
</tr>
</tbody>
</table>

In general, as shown in Figure 2, the coefficient \(k_{ij}\) equals zero when \(i\) does not equal \(j\). However, \(k_{ij}\) does not equal zero for the coupled modes of rocking and sliding. Because horizontal forces are not, in general, applied through the center of gravity of the foundation, horizontal seismic forces generally induce rocking moments on the footing (and vice-versa). The coupled stiffness coefficient for rocking induced by sliding, equal to the rotation induced by a unit horizontal force, is easily calculated from the distance between the line of action of the applied unit force and the center of gravity of the foundation (i.e., from the rocking moment induced by the unit horizontal force) and the rocking stiffness. The coupled stiffness coefficient for sliding induced by rocking (the horizontal translation induced by a unit rotation) is easily calculated from foundation geometry.

The stiffness coefficient for a non-circular footing is evaluated based upon \(k_{ECF}\), the stiffness coefficient of a circular foundation of equivalent radius \(r_0\) defined in Table 2 (after Richart, et al. (2)), the shape factors \(\alpha\) the embedment factor, \(\beta\), and the following equation:

\[
k = \alpha \beta k_{ECF}
\]

Values for \(\alpha\) and \(\beta\) can be found in Figures 3 and 4.

![Figure 3 - Shape Factors for Foundation Stiffness](image-url)
<table>
<thead>
<tr>
<th>Mode of Vibration</th>
<th>Mass (or Inertia) Ratio</th>
<th>Damping Coefficient</th>
<th>Damping Ratio</th>
<th>Equivalent Radius</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Translation</td>
<td>$B_z = \frac{(1-v)}{4} \frac{m}{\rho r_o^3}$</td>
<td>$c_z = \frac{3.4 , r_o^2}{1-v} \sqrt{\rho G}$</td>
<td>$D_z = \frac{0.425}{\sqrt{B_z}}$</td>
<td>$r_o = R_z = \sqrt{BL/\pi}$</td>
</tr>
<tr>
<td>Horizontal Translation (Sliding)</td>
<td>$B_x = \frac{(7v-8v)}{32} \frac{m}{(1-v) \rho r_o^3}$</td>
<td>$c_x = \frac{4.6 , r_o^2}{2-v} \sqrt{\rho G}$</td>
<td>$D_x = \frac{0.288}{\sqrt{B_x}}$</td>
<td>$r_o = R_z = \sqrt{BL/\pi}$</td>
</tr>
<tr>
<td>X- and Y-axis Rocking</td>
<td>$B_\phi = \frac{3(1-v)}{8} \frac{I_\phi}{\rho r_o^3}$</td>
<td>$c_\phi = \frac{0.8 , r_o^2 \sqrt{\rho G}}{(1-v)(1+B_\phi)}$</td>
<td>$D_\phi = \frac{0.15}{(1+B_\phi) \sqrt{B_\phi}}$</td>
<td>$r_o = R_\phi = \left[ \frac{16(B)(L)}{3\pi} \right]^{\frac{1}{4}}$</td>
</tr>
<tr>
<td>Z-axis Rotation (Torsion)</td>
<td>$B_\theta = \frac{I_\theta}{\rho r_o^3}$</td>
<td>$c_\theta = \frac{4 \sqrt{B_\theta} \cdot \rho G}{1+2B_\theta}$</td>
<td>$D_\theta = \frac{0.5}{1+2B_\theta}$</td>
<td>$r_o = R_\theta = \left[ \frac{16BL(B^2 + L^2)}{6\pi} \right]^{\frac{1}{4}}$</td>
</tr>
</tbody>
</table>

Notes:
- $m$ = mass of the foundation
- $c$ = damping coefficient ($c_z$, $c_x$, $c_\phi$, $c_\theta$)
- $I$ = moment of inertia of the foundation
- $\rho$ = mass density of foundation soil
- $r_o$ = equivalent radius ($R_z$, $R_\phi$, $R_\theta$)
- $B$ = width of the foundation (along axis of rotation for rocking)
- $L$ = length of the foundation (in the plane of rotation for rocking)
- $G$ = shear modulus of the soil
- $v$ = Poisson's ratio of the soil
- $D$ = damping ratio ($D_z$, $D_x$, $D_\phi$, $D_\theta$)
Figure 4 - Embedment Factors for Foundation Stiffness
The key issue in calculating the stiffness coefficient for a shallow foundation is evaluation of the appropriate value for the shear modulus of the soil. The shear modulus of soil depends upon both confining stress and cyclic strain level. Little guidance is available to the geotechnical engineer in evaluating these factors for seismic foundation response, as no systematic study of representative values exists. Typically, an initial shear modulus is established based upon empirical correlation with soil index properties, field measurements, and/or laboratory testing and then is reduced to account for strain softening during the earthquake. A summary of empirical correlations relating initial modulus to soil type and SPT blow count, CPT tip resistance, and/or confining pressure is presented in the FHWA circular on geotechnical earthquake engineering (2). The modulus is usually evaluated directly below the center of the footing, taking into account the dead load on the footing in evaluating the associated confining pressure. In the absence of formal studies, a depth of approximately 0.75 times the footing width for rocking and vertical vibrations and a depth of 0.5 times the footing width for translation and torsion would appear to be appropriate for evaluation of a representative confining pressure.

The reduction factor applied to the initial shear modulus of a soil depends upon soil type and cyclic shear strain. Little guidance exists on evaluation of appropriate reduction factors for seismic foundation design. However, based upon typical free field ground strains in earthquakes, a reduction factor of 0.9 would seem to be appropriate for events of magnitude less than 6.0, while reduction factors of 0.7 and 0.5 would seem to be appropriate for events between magnitude 6.0 and 7.0 and greater than magnitude 7.0, respectively.

Damping ratios (the fraction of critical damping) and damping coefficients for shallow foundations are also presented in Table 2. Damping for shallow foundations is primarily radiation damping. Damping does not play a major role in the forced response of a foundation system and is oftentimes omitted from the response analysis.

Deep Foundations

Stiffness coefficients for deep foundation systems are generally developed using p-y and t-z curve solutions for laterally and vertically loaded piles and pile groups. Deep foundation stiffness may be represented by the stiffness matrix or by an equivalent cantilever (a freestanding column with a length and stiffness that results in an equivalent deflection under the design load). Lateral and axial response is calculated using p-y and t-z computer programs. The stiffness coefficient is calculated by dividing the calculated deformation by the applied load. As each mode of response has a different stiffness, a different equivalent cantilever will represent each mode. Moment response of a single pile can also be evaluated using available p-y computer programs. For pile groups, it is usually assumed that moment loads on the footing are carried by axial loading in the pile. Torsional loading of a multi-pile deep foundation is usually
accommodated by the lateral resistance of the outside piles and torsional stiffness is evaluated on this basis. The torsional stiffness of a single pile is negligible and is usually ignored. However, the pile cap usually provides torsional resistance. Pile cap resistance may be an important contributor to overall foundation stiffness, particularly for lateral loading. For simplicity, cap stiffness is usually considered to be uncoupled from pile stiffness and is accounted for using the equations for shallow foundations presented in the previous section of this paper.

Key issues in evaluating deep foundation stiffness include assessing the degree of pile head restraint, what load level to use, and group reduction factors. Pile head restraint (i.e., whether the pile head is fixed, free to rotate, or partially restrained) has a major impact on lateral stiffness. Martin and Lam (3) discuss the evaluation of the impact of pile head restraint on lateral loading. Similar to other pseudo-static analyses, use of the peak load to evaluate stiffness is probably not appropriate because the peak load only occurs once during the earthquake. In the absence of any systematic study, it seems appropriate to use a reduced value of the peak load with a reduction factor, \( n \), evaluated using the following equation proposed by Idriss and Sun (4) for the effective strain reduction factor for seismic response analyses as a function of the moment magnitude, \( M_w \), of the design event:

\[
  n = \frac{(M_w - 1)}{10}
\]  

(2)

Damping of pile foundations is generally ignored in dynamic response analyses.

Group reduction factors, sometimes referred to as p-multipliers, are used to account for reductions in the stiffness of a deep foundation system due to interaction among piles. Using this approach, the p axis of the p-y curve for the pile is multiplied by the appropriate p-multiplier. Table 3 presents recommended p-multipliers for lateral loading as a function of pile spacing. For vertical load, group efficiency effects are often ignored.

<table>
<thead>
<tr>
<th>Row Spacing</th>
<th>Front Row</th>
<th>2nd Row</th>
<th>3rd &amp; More Rows</th>
</tr>
</thead>
<tbody>
<tr>
<td>3D</td>
<td>0.8</td>
<td>0.45</td>
<td>0.35</td>
</tr>
<tr>
<td>4D</td>
<td>0.9</td>
<td>0.65</td>
<td>0.55</td>
</tr>
<tr>
<td>5D</td>
<td>1.0</td>
<td>0.85</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Table 3 - Recommended “p-Multiplier” Values for Pile Group Design  
(Brown and Bollmann (5), Hannigan, et al., (6))
Abutment Walls

Elasticity theory can also be used to evaluate the translational and rotational stiffness of abutment walls. Lam and Martin (7) give the following equations for the rotational stiffness, $K_\theta$, and translational stiffness, $K_s$, of an abutment wall of width $B$ and height $H$ as a function of the Young’s modulus of the soil, $E$:

$$K_\theta = 0.072EBH^2$$  
$$K_s = 0.425EB$$

The location of the resultant force due to abutment wall translation may be applied at 0.6H from the base of the wall while the resultant force from wall rotation acts at approximately 0.37H from the base of the wall. Once again, the key issue is evaluation of the soil modulus, $E$. The value of $E$ may be computed from elasticity theory based on the shear modulus, $G$, and Poisson’s ratio, $v$. Assuming a Poisson’s ratio of 0.5 for compacted fill, $E$ may simply be assumed equal to three times $G$. In the absence of formal studies, a confining pressure evaluated at 0.67H seems appropriate for evaluation of the modulus for an abutment wall.

GLOBAL STABILITY

Shallow Foundations

Global stability analyses of shallow foundations subject to dynamic loading are conducted using the same limit equilibrium-type of analyses used to analyze the stability of shallow foundations subject to static loading. Modes of failure considered in dynamic stability analyses of shallow foundations include bearing capacity and sliding. In special cases, post-earthquake stability of shallow foundations may also be a design concern (e.g., for foundation placed upon potentially liquefiable soil).

In evaluating the bearing capacity of a shallow foundation subject to dynamic loading, vertical, horizontal, and moment loading are typically combined into a resultant eccentrically-applied inclined load, as shown in Figure 5. The general bearing capacity equation with adjustments for inclined and eccentric loading as proposed by Meyerhof is often used for this evaluation. Equations developed by Meyerhof for load inclination factors that are applied to the general bearing capacity equation and for calculating the effective width of the footing to account for load eccentricity are summarized in the FHWA geotechnical earthquake engineering circular (2).
Figure 5 - Principle of Superposition of Loads on Footing

Key issues in applying the general bearing capacity equation for dynamic stability are determining appropriate values for the factor of safety, the combinations of vertical, lateral, and moment load, and the dynamic shear strength of the soil for use in the analysis. While a factor of safety of 3.0 is generally used in static bearing capacity analysis, a factor of safety of 1.0 or 1.1 is typically used for dynamic loads. However, the selection of the factor of safety cannot be separated from the issues of load combination and shear strength. If the maximum vertical, horizontal, and moment loads from the dynamic analyses are superimposed simultaneously on the foundation, the resulting combined load is likely to be greater than any load the foundation actually experiences in the earthquake, as it is extremely unlikely that these loads will occur simultaneously. However, the maximum horizontal and moment loads may occur simultaneously. As the maximum load only occurs once during the earthquake, use of the maximum dynamic load on the footing is itself a very conservative approach. Furthermore, some codes allow a 33 percent increase in soil strength for rapid (dynamic) loading. Therefore, the appropriate factor of safety, load combination, and shear strength must be considered together to develop a rational approach to bearing capacity analysis.

For the sake of convenience, it is recommended herein to use a factor of safety of 1.0 and the static shear strength of the soil in evaluating dynamic bearing capacity. Then, in the absence of formal studies, seismic foundation stability may be analyzed for one-half of the maximum vertical load superimposed with one-half of the maximum horizontal load. This approach should provide a conservative assessment of the dynamic bearing capacity, assuming that some local yielding (permanent seismic deformation) is allowable and the foundation soil has low to medium sensitivity. If this conservative approach indicates that dynamic bearing capacity governs the foundation dimensions, a more sophisticated analysis looking at foundation stresses and bearing capacity under various load combinations, including the peak vertical load superimposed with the horizontal and moment loads acting at the time the peak vertical load is applied and the peak horizontal and moment load superimposed with the vertical load acting at
the time the peak horizontal and moment load occurs, may be warranted. Some provision for local yielding and dynamic strength increase may still be warranted when considering the above load combinations.

The sliding stability of shallow foundations subject to dynamic loading requires consideration of the sliding resistance on the base of the footing and active and passive earth pressure. Once again, load combinations, soil shear strength, and factor of safety are key issues in evaluating dynamic stability. Sliding resistance should be based upon the dead load on the footing. Due to the difference in frequency content, vertical loading should not be superimposed on top of the horizontal load in evaluating sliding stability. Moment loading only needs to be considered if there is an adhesive component of sliding resistance on the bottom of the footing, in which case the effective contact area evaluated using Meyerhof’s equations (summarized in the FHWA circular (2)) should be used. In the absence of formal studies, use of the seismic active and passive earth pressures, evaluated using a seismic coefficient equal to one-half the free field peak ground acceleration, in conjunction with the static shear strength of the soil and two-thirds of the peak horizontal force from the dynamic response analysis should provide an appropriate basis for evaluation of sliding stability using a factor of safety of 1.0.

**Deep Foundation**

The dynamic stability of deep foundation systems subject to vertical and lateral loading is generally also evaluated using the same analyses used for static loading. The p-y and t-z analyses used to evaluate load-deformation response can generally also be used to evaluate stability. As the ultimate load is reached, the deformation calculated using these analyses becomes very large. The moment capacity of a deep foundation system is generally evaluated using a limit analysis approach. One key issue in evaluating the stability of deep foundations subject to moment loading is the potential for local yielding of piles within a pile group. In many analyses, as soon as the outside pile in a pile group exceeds its capacity, the group is assumed to have reached its moment capacity. This is the moment capacity at the first yield. However, unrestrained deformation will not occur until all piles in a group have reached their capacity. This represents the ultimate moment capacity of the foundation. The difference in moment capacity based on these two assumptions is significant, as illustrated in Figure 6, from Lam and Martin (8). If the “ultimate capacity” approach is used, the unfactored seismic moment should be used in conjunction with a factor of safety of one, the foundation dead load, and the static shear strength of the soil to minimize the potential for large unrestrained deformations.
Figure 6 - Comparison of Moment Capacity at First Yield to Climate Moment Capacity (8)

Abutment Walls

Abutment walls have historically been the location of significant damage to bridges in earthquakes. While AASHTO calls for design of abutment walls for seismic active earth pressure, analysis of bridge response indicates that abutment walls often are subject to seismic passive earth pressures as they are pushed back into the backfill by the bridge deck (7). As seismic passive earth pressure can be five to 10 times greater than seismic active earth pressures, the observed damage to abutment walls in earthquakes is not surprising. Therefore, it is recommended the abutment walls be designed to resist seismic passive earth pressure calculated using the static shear strength and a seismic coefficient equal to one-half the free field peak horizontal ground acceleration at the bridge site.
Post-Earthquake Stability

Seismic stability evaluation may require assessment of the post-earthquake stability of foundations founded upon liquefiable soil. For shallow foundations, this may include evaluation of bearing capacity using residual shear strength and evaluation of lateral spreading potential using the equations of Bartlett and Youd (9). In evaluating bearing capacity, the residual shear strength is applied to the liquefied zone and a conventional limit equilibrium analysis is conducted using the dead load of the foundation, as illustrated in Figure 7. No seismic load is included in this analysis.

![Diagram of footing and liquefiable layer](image)

Figure 7 - Post-Liquefaction Stability Assessment

For deep foundations, lateral spreading due to liquefaction can apply significant lateral loads to the foundation system. However, only limited information is available on this phenomenon. The available information (10, 11) indicates that, if the liquefiable zone extends up to the surface, the residual shear strength may be applied to the side of the foundation piles as an equivalent fluid pressure. If a non-liquefiable zone caps the potentially liquefiable layer, the available data indicates that the piles should be designed to resist passive earth pressure from the non-liquefiable soil applied to the side of the pile. These loads can be very large. This situation should be avoided, if possible, for new foundations may require ground stabilization measures for existing foundations.

The key issue in evaluating post-earthquake stability is evaluation of the residual shear strength of the liquefied soil. This is a subject of much debate and research at the present time (12). The correlation between residual shear strength and Standard Penetration test blow count, N, developed by Seed and Harder (13) shown in Figure 8 is probably the most widely used
relationship for assessing residual shear strength in current practice. Due to the large band of values shown in Figure 8, considerable judgment and caution is required when using this figure.

Figure 8 - Relationship Between Corrected “Clean Sand” Blow Count \((N_t)_{60-cs}\) and Undrained Residual Strength \((S_r)\) from Case Studies. (Seed and Harder (13)).

CONCLUSIONS

Seismic design of foundations involves evaluation of both the dynamic response characteristics of the foundation system (i.e., dynamic stiffness and damping) and overall foundation stability. Good communication is required between the structural and geotechnical engineer to complete these evaluations in a cost-effective manner. There are many unanswered questions in making these evaluations. Therefore, until further information is available on the seismic behavior of foundation systems, the geotechnical engineer must exercise considerable judgment and discretion in design of these systems.

REFERENCES

ABSTRACT: The foundation soils for a bridge approach and abutment were identified as liquefiable soils. The subsurface soils consist of alluvial deposits comprising 4.5 to 6 m of soft silt over 21 m of loose sand. Vibro-replacement stone columns (0.9 m diameter) were used to improve the top silt deposit. Vibro-compaction was used to densify the top 11 m of the underlying loose sand. The standard penetration test (SPT) was used as an acceptance criterion for vibro-compaction. Out of 370 SPTs conducted, 350 exceeded the minimum required blow count. Liquefaction and slope stability analyses were conducted before and after ground improvement. Significant improvements in soil strength, density and factor of safety against liquefaction, were achieved for both the soft silt and loose sand deposits. The short- and long-term slope stabilities were also improved.

1 INTRODUCTION

A Cable-stayed bridge is currently under construction on the Mississippi River along Illinois State Highway IL 146, at Cape Girardeau (Illinois and Missouri states). The project is located in seismically active area, where the bedrock acceleration is estimated to be 0.36g, with an earthquake magnitude of approximately 8.5. On the Illinois side, a bridge abutment and a 10 m high by 62 m long approach embankment were constructed by Illinois department of Transportation (IDOT). The end slope was designed for 0.24g, with no deformation and no liquefaction. The foundation soils consisted of alluvial deposits which comprise 4.5 to 6 m of soft silt over 21 m of loose sand. The top 15 m of alluvial deposits were identified as highly liquefiable soils.

Several ground improvement alternatives were considered for the project. These include deep dynamic compaction, preloading, removal of silt and replacement with acceptable material, and the vibro-replacement and vibro-compaction techniques. Experience (Lukas, 1995) indicated that the top silt layer, with a plasticity index of 0 to 26, may present an "unfavorable" condition for dynamic compaction. Also, the relatively large depth of influence (about 27 m) would require a high energy input which would make dynamic compaction costly. The preloading was not a viable option because of project schedule. Removal and replacement was estimated to be the most costly alternative.

Vibro-replacement stone columns (0.91 m diameter) were used to improve the top silt deposit. Vibro-compaction was used to densify the top 11 m of the underlying loose sand. In both cases, coarse crushed stone was used as a backfill material. Compaction points were arranged in a uniform triangular grid.

Twenty eight (28) quality assurance soil borings were performed, at an average of 1 boring for every 30 compaction points. The standard penetration test (SPT) was used as an acceptance criterion for vibro-compaction. Approximately, 370 SPT tests were conducted, of which 350 exceeded the minimum required blow count.

Liquefaction analyses were conducted before and after ground improvement using Seed and Idriss' (1982) simplified procedure. Also, static and pseudo-static seismic slope stability analyses were conducted using the simplified Bishop slices method. The computer program SHAKE91 (Idriss and Sun, 1992) was used to identify ground motion. Significant improvements in soil strength, density and factor of safety against liquefaction, were achieved for both the soft silt and loose sand deposits. Slope stabilities were also improved.
2 SITE INFORMATION

Ground modification was accomplished on the east side of the Mississippi River Bridge along Highway IL 146 at Cape Girardeau, Illinois (Figure 1) to improve the foundation soils for the bridge approach and abutment. The east side of the project will be referred to as the "site".

2.1 Subsurface Soil Condition

The subsurface investigation at the site, prior to ground modification, indicated that the foundation soils consisted of 4.5 to 6 m of very soft sandy silt to silt over 21 m of loose, fine to coarse silty sand to sand. The silt to silty loam layer consisted, on the average, of 50% to 70% silt, 20% to 30% sand and 5% to 20% clay. A typical particle size distribution for this silty soil is shown on Figure 2. The majority of the silty soil samples were non-plastic; however, a few samples with the high clay content showed plasticity index (PI) ranging from 6 to 23. The moisture content ranged from 20% to 50% Consolidated-drained (CD) and consolidated-undrained (CU) triaxial tests were conducted on typical silty soil samples. Based on the CD tests, the average effective cohesion ($c'$) and friction angle ($\phi'$) values for the silty soils were 8 kPa and 31°, respectively. Based on the CU tests, the average $c'$ and $\phi'$ values were 3 kPa and 17°, respectively. The average N-value, from the SPT tests, for the underlying 21 m of sandy layer ranged from 3 to 27 blows per 0.3 m penetration at various depths. The average moisture content ranged from 25% to 35%. The ground water was encountered at an average depth of 2.5 m below existing grade for all borings.

2.2 Liquefaction Analysis

The liquefaction potential of the foundation soils was evaluated using the Simplified Procedure originally developed by Seed and Idriss (1982). This method is based on documented field performance of soils subjected to earthquakes, and it uses the SPT N-value to estimate the cyclic stress required to cause liquefaction, called the critical cyclic stress. This stress is then compared to the cyclic stress induced by the design earthquake, to estimate the factor of safety against liquefaction (FOSL). Corrections to the procedure were made for SPT hammer energy and overburden pressure. Also, corrections for the site earthquake magnitude of 8.5 was made. The average corrected N-values, used in the liquefaction analysis, were 5 and 15 for the upper 4.5 to 6 m silty soil and the underlying 21 m sandy soil, respectively. An average ground water depth of 2.5 m below existing grade was used in the analysis. Based on the analysis, it was concluded that the FOSLs for the upper 4.5 to 6 m silty soil and the underlying 21 m sandy soil were 0.4 and 0.7, respectively.
2.3 Slope Stability Analysis

In order to conduct slope stability analysis, the bedrock acceleration at the embankment base was estimated, using SHAKE91 program (Idriss and Sun, 1992). This program is based on a one-dimensional equivalent-linear site response analysis. The average soil parameters used in this program for the upper 4.5 to 6 m silty soil and the underlying 21 m sandy soil, respectively, were: shear moduli of 43 MPa and 66 MPa, mass densities of 1900 and 2000 kg/m³, and an average equivalent viscous damping ratio of 0.04 used for both layers based on an effective strain factor of 0.65 (Kavazanjian et al., 1998). The bedrock acceleration at the site is estimated to be 0.16g for a return period of 475 years (90% non-exceedance in 50 years) and 0.36g for a return period of 2400 years (90% non-exceedance in 250 yrs.). Results of the SHAKE91 analysis indicate amplification of the bedrock motion to be about 0.24g at the base of the embankment for the lower return period. The SHAKE91 analysis did not indicate amplification of the bedrock motion for the higher return period.

Static slope stability analysis was conducted for the side slopes, using a slope stability analysis program called XSTABL (Sharma, 1994). XSTABL is based on the limit equilibrium approach, using the simplified Bishop slices method. Also, pseudostatic seismic stability analysis was conducted for the endslope, using XSTABL and assuming the earthquake loads are applied at the centroid of each individual slice:

Table 1. Summary of slope stability analyses before ground modification.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Soil Parameters</th>
<th>FOS</th>
</tr>
</thead>
<tbody>
<tr>
<td>End of Construction</td>
<td>Silt: $c' = 17$ kPa, $\phi = 0$</td>
<td>0.82</td>
</tr>
<tr>
<td>(Static: $K = 0.0g$)</td>
<td>Sand: $c' = 0$, $\phi = 30'$</td>
<td></td>
</tr>
<tr>
<td>UU Tests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Long-Term</td>
<td>Silt: $c' = 0$, $\phi = 31'$</td>
<td>1.74</td>
</tr>
<tr>
<td>(Static: $K = 0.0g$)</td>
<td>Sand: $c = 0$, $\phi = 30'$</td>
<td></td>
</tr>
<tr>
<td>CD Tests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Long-Term</td>
<td>Silt: $c' = 0$, $\phi = 17'$</td>
<td>0.79</td>
</tr>
<tr>
<td>(Seismic: $K_s = 0.24g$)</td>
<td>Sand: $c' = 0$, $\phi = 30'$</td>
<td></td>
</tr>
<tr>
<td>CU Tests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Long-Term</td>
<td>Silt: $c = 0$, $\phi = 17'$</td>
<td>0.65</td>
</tr>
<tr>
<td>(Seismic: $K_s = 0.36g$)</td>
<td>Sand: $c = 0$, $\phi = 30'$</td>
<td></td>
</tr>
</tbody>
</table>

The end slope was designed to have no deformation at a yield base acceleration of 0.24g, for a factor of safety (FOS) of 1. For base accelerations greater than 0.24g and the FOS is less than 1, the permanent seismic deformation was estimated using the procedure developed by Makdisi and Seed (1978). The procedure is based on two-dimensional finite element analysis of embankments. The end slope was designed to have a tolerable deformation less than 15 cm for earthquakes producing up to 0.36g base acceleration. A cohesive silty clay soil, with an average cohesion of 50 kPa and zero friction angle, was used for the 18.6 m approach embankment. The end and side slopes were constructed at 2H:1V and 3H:1V, respectively, in all analyses. A summary of the subsurface soil parameters, used in the stability analyses for different conditions, and the resulting FOS are summarized in Table 1.

3 CONSTRUCTION PROCEDURES

The objective of ground modification at the site was to improve the shear strength of the upper 4.5 to 6 m silty layer and densify the underlying 21 m of loose sandy layer. This was accomplished by using the vibro-replacement method for the soft silty layer and the vibro-compaction method for the loose sandy layer. As shown on Figure 3, both methods covered the entire area, except vibro-compaction extended 9 m to the west beyond the limits of vibro-replacement to cover the entire end slope. The analyses indicated that stone columns beyond the mid-point of the end slope did not significantly influence the different factors of safety. In the common (shaded) area, both methods were performed as a continuous operation at the same modification points (locations) as shown on Figure 4. All modification points were arranged in a 2.4 m equilateral triangular grid as shown on Figure 5.

The backfill material (stone) used for both vibro-replacement and vibro-compaction holes was an IDOT-specified coarse aggregate, designated as CA-05. This material consists of 97±3% passing US sieve size 37.5 mm, 40±25% passing US sieve size 25 mm, 5±5% passing US sieve size 12.5 mm and 3±3% passing US sieve size 4.75 mm (No. 4). The stone was tested for gradation (AASHTO T 27), specific gravity (ASTM C 127) and the minimum and maximum densities (ASTM C 29). The same stone was also used in a 1-m thick drainage blanket/working platform over the existing ground.

The downhole vibratory probe used in both methods was a 16-metric ton gyratory probe with an average
Figure 3. Limits of vibro-replacement and vibro-compaction.

Figure 4: Cross-section through ground modification points where vibro-replacement stone columns and vibro-compaction were applied.
frequency of 2300 rpm. The axes of modification points (or vibration centers) were near vertical, not exceeding 5° inclination from the vertical or a maximum of 0.3 m from the assigned location. No subsurface obstructions were encountered, during the modification operations, to cause any significant deviation in the location of the modification points.

3.1 Vibro-Compaction

Vibro-compaction commenced at the specified locations by full penetration of the vibratory probe through the treatment zone. After that, the probe was slowly retrieved in 0.6 to 1.2 m increments to allow backfill placement. The backfill stone was then repenetrated at least twice by the probe, in order to densify and force the stone radially into the surrounding insitu soil. The diameter of vibro-compaction holes ranged from 0.45 to 0.6 m.

SPT soil borings were performed concurrently with the vibro-compaction operation, at a frequency of one boring every 30 compaction points. This was done to provide test results as quickly as possible, verify the minimum target blow count of 25 blows per 0.3 m and to perform the necessary corrective measure at the spots not meeting this criterion, while the compaction equipment was on the site.

3.2 Vibro-Replacement Stone Columns

Vibro-replacement commenced at the specified locations, after the vibro-compaction operation at each location. Water was applied at the tip of the vibratory probe to widen the diameter of the stone columns to the required 0.9 m. The flow of water from the bottom jet was maintained at all times during backfilling to prevent caving or collapse of the hole, and to provide a clean stone column. The stone columns were constructed by slowly retrieving the vibratory probe in 0.6 to 1.2 m increments and allowing the stone placement. Also, the backfill stone was then repenetrated at least twice by the probe, in order to densify and force the stone radially into the surrounding insitu soil.

Quality assurance was achieved by maintaining a minimum area replacement ratio [0.907(column diameter/column spacing)]² of 0.128. This ratio was considered critical to the embankment stability, according to Barksdale and Bachus (1982). The average effective stone column diameter was calculated, using the in-place density of the stone and the weight of the stone used to fill a given length of the hole. The in-place density was assumed to equal 95% of the compacted stone density as determined by laboratory testing (ASTM C 29). The weight of the stone required to fill a column was based on the equivalent number of full buckets of stone placed in the hole and the loose stone density determined in the laboratory (ASTM C 29).

4 RESULTS AND DISCUSSION

The shear strength of the upper 4.5 to 6 m of soft silty soil and the density of the underlying 21 m loose sandy soil were both significantly improved by the vibro-replacement and the vibro-compaction methods, respectively. The improvement in the shear strength of the silty soil was estimated using the weighted average shear strength method recommended by Barksdale and Bachus (1982). In this method, the stone columns in row were converted into an equivalent strip whose width is proportional to the total volume of stone in that strip. The stone and soil friction angles were converted into an equivalent shear strength for the entire layer, taking into consideration the effective overburden pressure acting on the potential sliding surface, inclination of this surface, soil plasticity and any possible stress concentration. Based on average friction angles of 45° and 31° assumed for the in-place stone and the silty soil, respectively, the equivalent weighted average undrained shear strength of the improved soil was estimated to be 68 kPa (compared to the unimproved 17 kPa) for no stress concentration condition.

The improvement in the sandy layer was measured in terms of the increase in the SPT blow count. Based on the liquefaction analysis, the minimum required (corrected) N values at different elevations
were as specified on Figure 6 for a FOSL of 1.3. As shown on this figure, the observed N values for the improved sandy soil significantly exceeded the specified values at all depths. Therefore, a friction angle of 35° was conservatively assumed for the improved sandy soil in all analyses.

![Graph showing SPT N value curves](image)

1 FT = 0.30 m

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Observed Specified (FOSL = 1.3)

Figure 6. Specified and observed SPT N value curves after ground improvement.

Liquefaction and slope stability analyses were conducted after accomplishing ground improvement. The liquefaction FOSL increased to the minimum required value of 1.3 for both layers. Also, the FOS against slope failure increased for the different conditions analyzed as shown in Table 2. For the

Table 2. Summary of slope stability analyses after ground modification.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Soil Parameters</th>
<th>FOS</th>
</tr>
</thead>
<tbody>
<tr>
<td>End of Construction</td>
<td>Silt: c = 68 kPa, φ = 0</td>
<td>1.52</td>
</tr>
<tr>
<td>(Static: K = 0.0g)</td>
<td>Sand: c = 0, φ = 35°</td>
<td></td>
</tr>
<tr>
<td>UU Tests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Long-Term</td>
<td>Silt: c' = 0, φ' = 31°/45°</td>
<td>2.31</td>
</tr>
<tr>
<td>(Static: K = 0.0g)</td>
<td>Sand: c' = 0, φ' = 35°</td>
<td></td>
</tr>
<tr>
<td>CD Tests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Long-Term</td>
<td>Silt: c' = 0, φ' = 17°/45°</td>
<td>1.00</td>
</tr>
<tr>
<td>(Seismic: Ks = 0.24g)</td>
<td>Sand: c' = 0, φ' = 35°</td>
<td></td>
</tr>
<tr>
<td>CU Tests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Long-Term</td>
<td>Silt: c' = 0, φ' = 17°/45°</td>
<td>0.81</td>
</tr>
<tr>
<td>(Seismic: Ks = 0.36g)</td>
<td>Sand: c' = 0, φ' = 35°</td>
<td></td>
</tr>
</tbody>
</table>

* With tolerable deformation of 15 cm.

5 CONCLUSIONS

The foundations soils of a bridge approach and abutment were identified as liquefiable soils. Vibro-replacement (stone columns) method was used to increase the shear strength of the upper silty soil, and vibro-compaction was used to densify the underlying loose sandy soil. Liquefaction and slope stability analyses were conducted before and after the ground improvement. The factors of safety against liquefaction and slope failure, both significantly increased after the ground improvement. The two methods proved to be effective for the types of soils considered in this project.

6 REFERENCES


The contents of this paper reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of IDOT. This paper does not constitute a standard, specification or regulation at IDOT. Trademark or proprietary names appear in this paper only because they are considered essential to the object of this document; their use does not constitute an endorsement by IDOT.
SESSION 7
SUBSTRUCTURE & SUPERSTRUCTURE ISSUES
SEISMIC RETROFIT OF POPLAR STREET
ELEVATED COMPLEX NEAR EAST ST. LOUIS, ILLINOIS

ABSTRACT

Upon evaluation, a system of 1960's vintage elevated roadways was found to be prone to major structural damage given seismic demands well below those specified in current AASHTO standards. Due to the system's importance, it was decided to explore retrofit measures capable of maintaining serviceability throughout an AASHTO design level event. In order to meet project objectives efficiently, roadway support piers were modified so that inelastic deformations would be concentrated in the circular column members, which could then be easily retrofitted to achieve the necessary levels of ductility.

INTRODUCTION

In East St. Louis Illinois, an extensive system of elevated roadways handles the interface between several state and federal highways and the Poplar Street Bridge, which crosses the Mississippi River. This Poplar Street complex of roadways includes several thousand feet of plate girder/concrete decks, supported on reinforced concrete piers. Examples of typical roadway elements are shown in figures 1 and 2.

The complex was built in the late 1960's, with no consideration of seismic hazards. Given the nature and extent of the various highways included in the complex, the surrounding area would be greatly affected if a seismic event caused extensive damage. Therefore, the Illinois Department of Transportation (IDOT), decided to investigate the ability of the complex to sustain seismic demands. Initial investigations revealed severe deficiencies with respect to current AASHTO design seismic demands. Based upon this finding and other considerations, IDOT decided to replace the complex's mainline roadways, and to investigate retrofit options for some of the remaining structures.

A roughly 7,000 ft section of elevated roadway called the collector/distributor (C/D) system was examined as part of a preliminary evaluation. This effort included a conceptual study concerning the use of isolation bearings to reduce demands on existing structural elements. The study concluded that isolation bearings could be used to significantly improve seismic performance of the C/D system. The cost of implementing a useful isolation retrofit was estimated to be about five million dollars. In 1995, Wiss, Janney, Elstner Associates, Inc. (WJE) was retained to perform a comprehensive seismic evaluation of the C/D system, and to develop efficient concepts for improving seismic performance. Following the evaluation, WJE was given the task of developing detailed retrofit plans for modifying the C/D system.

SEISMIC EVALUATION

Evaluation Parameters

A necessary early step in any evaluation is establishing performance criteria. Given the important nature of the roadways carried by the C/D system, IDOT specified that the modified roadways must be able to sustain the target seismic demands, and remain essentially serviceable. This level of performance is more restrictive than the typical seismic design objective of protecting life-safety.

The selected project seismic demand was consistent with current AASHTO design requirements. Minor modifications, primarily involving the shape of the design response spectra, were made to reflect local site conditions. Previous geotechnical studies indicated some potential for localized liquefaction. It was therefore decided to consider a reduction in pile capacity consistent with conservative estimates of reduced soil strengths.
Structural Capabilities

The C/D system comprises approximately 27 structures, each including one to four spans of roadway deck. The structures are separated by expansion joints at common piers. In a typical structure, the superstructure is connected to the supporting piers via a combination of fixed, rocker and elastomeric bearings. In most cases, one pier has fixed bearings (restraint in all directions), the expansion joint piers have elastomeric bearings (very little restraint in the horizontal plane), and the remaining piers have steel rocker bearings (no restraint in longitudinal direction). In a typical three-span structure, longitudinal loads are resisted by the fixed bearing pier, while transverse loads are carried by piers with fixed bearings and those with rocker bearings.

Evaluating the existing structures required the determination of the capacity (in terms of strength and ductility) of the components of each load path. It quickly became apparent that the existing structural elements and connections had very little ductility. For example, the reinforced concrete columns, walls and pile caps were not detailed to sustain repeated applications of cyclic inelastic deformations. Therefore, successful seismic performance would require essentially elastic response.

Although the C/D structural systems had significant strength, estimated elastic response to project seismic demands would exceed the capacities of many elements in most of the structures. While this condition is common in most seismic-resistant designs, such designs, if successful, incorporate ductile components that are capable of sustaining anticipated inelastic deformations while protecting the less ductile elements in the structure. Lacking the necessary ductility, the subject structures would probably be severely damaged during a design level seismic event. Brittle failures of bearings and/or rapid degradation of non-ductile reinforced concrete elements (columns, walls and pile caps) could lead to the collapse of many structures.

To illustrate the deficiencies of the existing structure, and to develop a basis for discussing retrofit options, the interaction between design level seismic demand and the response of a typical, well designed, seismic-resistant structure is illustrated graphically in figure 3. The Demand Curve shown in figure 3 is a response spectrum envelope similar to those typically used to define the seismic demands that must be addressed on a project. The line labeled "Spectrum for Elastic Design" represents the minimum amount of elastic capacity that is required for a particular type of ductile structure. In typical seismic resistant designs, elastic capacity requirements are a small fraction of the demands associated with elastic response. As discussed below, the ability to sustain inelastic deformations (ductility) plays a vital role in resolving this apparent "discrepancy" between demand and capacity. The line labeled Capacity Curve in figure 3 represents the relationship between structural capacity (here in terms of spectral acceleration) and fundamental period of vibration. The significance of points A, B and C shown in figure 3 are discussed below:

**Point A:** Point A represents the ultimate elastic capacity of the structure. Therefore, at demand levels equal to or less than Point A, the structure remains elastic and its fundamental period of vibration remains at its original value $T_0$. As long as the structure remains elastic, the design seismic demands are represented by Point A' on the demand curve. Since the demand at Point A considerably exceeds the corresponding capacity, inelastic deformation of the structure will occur during a design level event.

**Point B:** As the structure deforms inelastically, its lateral load resisting system becomes softer. This softening results in an increase in the fundamental period of vibration. Therefore, Point B lies somewhat to the right of Point A. In most well designed structures, inelastic deformation results in at least a small increase in strength. This increase is most often due to a redistribution of forces to as yet unyielded members of the lateral load resisting system. For example, not all frames or shear walls in a particular
system will reach their ultimate elastic strengths at the same time. Therefore, yielding of the most critical element, represented by Point A, will be followed by an increase in strength as additional elements sustain larger and larger forces until they too yield. For this reason, Point B usually (but not necessarily) lies above Point A. Point B' represents the design seismic demands on an elastic structure with a period $T_B$. In this example, however, the structure only possesses a period $T_B$ after a certain amount of inelastic deformation occurs. This inelastic deformation dissipates energy above and beyond that which would be dissipated by an elastic structure. This dissipation of additional energy effectively reduces the force levels associated with the design spectrum envelope. This reduction in seismic demand is reflected by the dashed Demand Curve that passes through Point B". Although at Point B our capacity is greater and our demands are less than at Point A, demand still exceeds capacity. Therefore, additional inelastic deformation is expected.

**Point C**: Point C reflects the effects of inelastic deformation beyond Point B. At this point, the elastic demands represented by Point C' still exceed the structure's capacity. However, as Point C" indicates, the inelastic deformations required to reach Point C have effectively reduced the design seismic demands to a level below the structure's capacity. Therefore, sometime before Point C is reached (see Point R), the demand and capacity curves intersect, which indicates a point of stability.

If the example structure represented in figure 3 had little or no ductility, the capacity curve would have terminated at or near Point A, while the corresponding design demands would have remained at or near Point A'. Such is the case with the original C/D structures.

**RETOFIT CONCEPTS**

To achieve acceptable performance in the C/D structures, the existing load paths would have to either be strengthened to be capable of sustaining the full seismic demands elastically, or they must be modified to incorporate and mobilize some degree of ductile behavior. While effective, the strengthening approach would require modification of almost every component of every load path, including many foundation elements. This would be very costly. In contrast, a ductility-based approach can be used to provide adequate seismic response without such comprehensive modifications.

One approach to incorporate ductility involves the installation of deformable bearings (isolation bearings). These bearings are typically designed to deform at forces less than what would severely damage the rest of the structure. At many locations in the C/D system, however, minimum non-seismic strength requirements and maximum displacement limitations would limit the effectiveness of isolation bearings so that some additional (strengthening) modifications would be required. The concept of isolation bearings was evaluated as part of a previous engineering study, and was found to require extensive modification of the pier/deck interfaces, and strengthening of many existing load path elements. As previously indicated, the estimated cost of implementation was about five million dollars. Given the estimated cost of installing isolation bearings, and the fact that substantial modifications to existing structural elements would still be required to satisfy project performance criteria, the project team decided to investigate alternative ways of providing the necessary ductile capabilities.

To be capable of acceptable inelastic deformations, a given load path requires a single ductile element. If that element is the weakest link in the load path, its yielding can limit the force sustained by the load path to a value less than the elastic capacities of the remaining elements. Therefore, these elements need not be ductile. With this concept in mind, the following retrofit approaches were considered:

1. Ductility modifications only: This approach involves identifying the weakest link in each load path and modifying it to be sufficiently ductile. The most common problem with this approach is the cost of developing adequate ductility if the weak link is difficult to modify. Examples include foundation
elements which are difficult to access and cumbersome to modify appropriately. Many of the existing weak links lie below grade.

2. Ductility and strength modifications: This approach involves strength modifications that create load paths where the weak links are easy to modify as required. As will be discussed, this approach was applied to the C/D system so that no below-grade structural modifications were required. The selected retrofit approach also utilized proven ductility-enhancing modifications on easily accessible members.

In the C/D structures, the circular pier columns represent major elements in the lateral load paths. The pier columns are also very accessible. Fortunately, considerable research work has been devoted to developing ways to improve the ductility of reinforced concrete columns that were not designed to sustain significant, repeated inelastic deformations. Reference 1 includes detailed ductility retrofit procedures for concrete columns. Much of the research work that was used to develop the reference 1 procedures was conducted by Dr. M.J.N. Priestly and others at the University of California at San Diego (UCSD). Examples of related publications are included in the bibliography of reference 1. Research work related specifically to the subject roadway complex was supervised by Dr. Neil Hawkins of the University of Illinois (U of I). A program of full-scale, Poplar Street column tests is summarized in reference 2.

The diagram shown in figure 4 represents a typical existing load path. For this system, the project elastic seismic demand and the ultimate strengths of the various structural components (in terms of inertial load in deck) are shown in figure 4. Since none of the components possess significant ductility, application of design-level demands would compromise the integrity of this system.

The pile cap comprises the weak link in the figure 4 example. Unfortunately, it would be difficult to implement appropriate ductility-enhancing measures at this location. In contrast, the easily accessible and readily modified columns are the strongest elements in the load path. In such situations, it was decided to eliminate the need for below-grade modifications, and to make the load path easy to retrofit by modifying the columns so that they would become the weak link. This is to be accomplished by cutting column base/wall dowel bars until column base flexure controls the ultimate capacities of the load paths. Once a pier's columns are made the weak link, all below-grade elements will be protected from overload (i.e. remain elastic), and load path ductility can be established by proven column retrofit methods.

To ensure inelastic response is confined to the desired locations (in this case the bases of the pier columns), the columns in each pier will be modified to sustain flexural hinges at approximately eighty percent of the load necessary to exhaust the elastic capacity of the next weakest element in the pier. Therefore, in the figure 4 example, the modifications would result in an ultimate lateral load capacity of 160 k. While this represents a forty percent reduction in the lateral load capacity of the columns, it is only a twenty percent reduction in the pier's ultimate capacity. Each C/D structure will be modified so that in all load paths where inelastic deformations are anticipated, pier columns will be the weak links and will have the necessary ductility. Typical retrofit measures include:

- Cutting column base/wall dowel bars to establish column base flexure as the weak link in the load path.

- Wrapping column bases with prestressing strands or other suitable confining materials so that anticipated flexural hinges would be stable.

In many instances, mobilizing available column capacity (even after their strengths were reduced) required supplemental connections to be installed between the deck and the tops of the columns. In other words, even the reduced column capacities could not be fully mobilized in some of the existing systems,
which means that reducing the column strengths did not reduce the capacity of the load path below that of the original.

To ensure that project serviceability criteria were met, analyses of the modified structures were performed in accordance with the concepts shown in Figure 3. In all cases, ultimate column ductility demands were limited to values well within the limits indicated by the UCSD and U of I research, and ultimate deck displacements were kept within the deformation capacities of existing elastomeric bearings. The concerns regarding possible local liquefaction of some soil layers were addressed either by limiting ultimate pile demands to roughly half of their ultimate capacities or, for longitudinal loads, by connecting a structure to at least one adjacent structure with ties at expansion joints.

Contractors have submitted construction bids based upon final contract plans and specifications. Three of the bids received indicated that the C/D seismic retrofit program will cost less than two million dollars.

SUMMARY

Successful seismic performance typically requires a combination of strength and ductility. Recognizing the role of each of these characteristics is an essential step in the design of an efficient seismic retrofit program. An evaluation of the subject structures showed that improving load path ductility would result in substantial seismic performance improvements. The final design used creative structural modifications to both eliminate the need for costly below-grade work, and make state-of-the-art ductility enhancement measures easy to implement.

References:


Figure 3 - Intended interaction of seismic resistant structure and design seismic demands
LONGITUDINAL INERTIAL FORCE (V), ORIGINATING IN DECK
($V_{ELASTIC} = 580K$)

DECK/COLUMN CONNECTION
($V_U = 250K$)

COLUMN
($V_U = 266K$)

GRADE

WALL
($V_U = 230K$)

PILE CAP
($V_U = 200K$)

PILES
($V_U = 210K$)

$V_u$ = Lateral load in deck at which component would reach its ultimate elastic capacity

Figure 4 - Example load path
EXPERIENCE WITH APPLICATION OF CONCRETE PIER COLUMN ENCASEMENT FOR SEISMIC RETROFITTING

By

M. Karshenas and Iraj I. Kaspar

ABSTRACT

Fifteen years' experience in retrofitting concrete columns with advanced composite fibers is discussed, emphasizing adverse effects of freeze-thaw on durability of the wrapped concrete and the composite fiber. A strip-wrap alternative to full encasement of concrete columns has been proposed to allow ventilation of concrete and minimize freeze-thaw effects in cold climates. Findings from tests on the behavior of column specimens retrofitted with strip-wraps, with full encasement, and the as-built condition are presented for comparison.

INTRODUCTION

Full encasement of concrete columns with composite fiber wraps has become a very popular and often preferred method of column repair. Fiber wraps are generally used in seismic areas as a confining element and in non-seismic areas simply to protect the concrete columns against deteriorating elements. One of the concerns in using composite fiber wraps is the entrapment of water inside the wrapping and the subsequent potential for damaging freeze-thaw effects in cold climates. The issue of freeze-thaw of encased concrete columns became the subject of a 1997 investigation, when the Hydroester wrap encasements of a number of columns split and fell off.

Several bridges carrying Archer Avenue and its ramps over US Route 45 in Justice, Illinois were built in 1960s. The pier columns of these structures were repaired in early 1980s. The repair consisted of removing deteriorated concrete along each column, placing flexible Hydroester fiberglass sheets around the columns, and filling the gaps with a superplasticized mortar. The Hydroester wrap, used as a stay-in-place form with no anticipated structural role, extended full height of the columns, up to three inches below the cap-beam shown in figure 1-a. The wraps were placed around the columns with a few inches of overlap, holding one row of bolts along the lap. Evidently, the wrap was expected to protect the concrete columns against direct exposure from splashing of deicing salt by traffic and leakage from the deck joints between adjacent units. Some examples of deteriorated concrete and the wrapping conditions observed in a 1997 inspection are presented in figures 1-b to 1-f. The columns showed different degrees of deterioration. Some wraps had split, disintegrated and dropped off the columns, while others displayed less damage. The extent of damage was generally consistent with the amount of leakage from the transverse joint of the super-elevated deck above.
FAILRE INVESTIGATION AND DIAGNOSIS

Field Observation

An investigation was conducted on the possible causes of the wrap failure and the deterioration of concrete inside the wrap (Francis J. Young 1997). Field observation showed clear evidence of leakage of deicing salt through the transverse joints of the deck. In addition, the columns were exposed to direct splash from the traffic passing under the bridge. The surface of the concrete column exposed after removal of the wrap is shown on figure 3-c, that exhibits the obvious erosion of mortar caused by water flow between the wrap and the concrete. The debris on the outside of the wrap below tears confirmed the passage of water between concrete and wrap where water had leaked. Leaching of the concrete is evidenced from figure 4-f, showing disintegrated concrete at the top of a column that released fine debris after removal of the wraps.

The exposed mortar inside the wrap showed evidence of excessive segregation of the mortar during placement, with a high sand content at the bottom of the column and almost pure paste near the top. Near the bottom of the column shown in figure 1-c there are pockets of a white paste that cracked, disintegrated, and easily removed. Figure 1-c also shows an exposed reinforcement bar inside the wrap. The concrete in such pockets was broken up into small fragments, typical of damage caused by freeze-thaw effects. Areas of disintegrated concrete were located in mortar pockets near the top and the bottom of some columns.

Laboratory Testing

Several cores were taken from three deteriorated columns in 1997 for laboratory testing of the materials. Testing included scanning electron microscopy, X-ray diffraction, and chloride analysis on selected samples. Scanning electron microscopy on several samples consistently indicated an extensive network of fine microcracks, primarily in the paste and the paste-aggregate interface as shown in figures 2-a and 2-b. Chloride was detected in the outer portions of column, and at a few locations, near the top of the columns, chloride penetrated the original concrete as well. Some corrosion was detected on the main reinforcing bars at the top of a column. The coarse aggregate was determined to be mostly dolomite, with some quartz, and its fine aggregate was primarily quartz, dolomite, and calcite.

Diagnosis

Based on field observation and laboratory testing, the primary cause of mortar deterioration was found to be water from leaking bridge deck joints saturating the concrete inside the wrap and resulted in freeze-thaw cycle breaking down the integrity of the concrete and the wrap. The poor initial quality of the mortar including high water content and segregation compounded by infiltration of water containing deicing salts underneath the fiber wrap aggravated the process.

RECOMMENDED MODIFICATIONS TO WRAPPING PROCEDURES

To allow normal evaporation of water from wrapped columns, the use of intermittent fiber wraps instead of full encasement is recommended. This may be achieved by the use of individual bands of fiber wrap spaced along the column. The effectiveness of the proposed method was evaluated by using 5-inch-wide bands placed at 10-inch spacing on test columns. This was adequate to confine the compression zone and prevent de-bonding at a deficient lap splice of the main reinforcing bars. To verify the effectiveness of the intermittent wraps, a series of laboratory tests was performed on
intermittently and fully encased columns. Figure 3-a shows 5-inch wide strips of fiber wraps at 10-inch centers along a test column. The column tests showed results identical with that of the fully encased columns with using an equal wrap thickness.

The testing program was part of research funded by the Illinois Department of Transportation and conducted by the Civil Engineering Department of the University of Illinois at Urbana-Champaign. This project started in 1993 to study seismic retrofitting of bridges and develop rational design procedures complementing current methods (FHWA Seismic Retrofit Manual, 1995) including various column wrap systems. The research is ongoing; however, the results of the completed phase I has been published (Lin, Gamble, Hawkins 1998), and the phase II is to be published soon (Shkurti, Hawkins, 1999). Following is a brief description of test results for an as-built column and three columns retrofitted with fiber warps compensating for inadequate reinforcement bar splice lengths at their bases. Test results indicated the structural effectiveness of intermittently strip-wrapped columns to fully encasing columns and may be considered in cold regions where infiltration of water contaminated with deicing salts is possible.

Test Setup

A sketch of the test setup is shown in figure 3, with 2-ft diameter half scale columns extending from a crash wall anchored to the lab floor. Each column was reinforced with 6- #11 Grade-60 reinforcing bars lapped at the base of the columns to 6- # 11 dowels extending 43 inches out of the crash wall. The main bars and dowels were inside #3 Grade-60 horizontal hoops, with 12-inch lap lengths and placed at 10-inch centers to simulate the as-built conditions for common bridge columns. One of the columns is partially encased with intermittent wraps in the lap-splice area, and the other column was tested as built, with no retrofit. Each column was subjected to monotonically increasing cyclic loading and displacement applied at the top. Figure 4 shows a sketch of the typical cyclic displacements that was applied to the test specimens.

As Built Column Test F-1

The displacement cycles started from 0.2 inch in two opposite directions and increased until the force level dropped off. The loading history for this column was similar to the first few cycles shown in figure 4. The corresponding hysteretic load-deflection response is presented in figure 5. The loading started at about eight kips and increased in eight kips increments, corresponding to 25%, 50%, and 75% of the column's ideal flexural strength, respectively. The measured horizontal column deflection at the load point was about 0.2 inch, 0.5 inch, and 1.0 inch, respectively. The force corresponding to yielding of the main bars was estimated to be 28 kips at a 1.2-inch deflection. The maximum ductility reached in the as-built column was two before the force dropped off. Ductility was defined as ratio of the ultimate deflection to the deflection at first yield ($\Delta_w/\Delta_y$).

Strip-Wrapped Column Test G-1

This test column was identical to test F-1 except it was wrapped with 5-inch wide strips of carbon fiber spaced at 10-inch centers in the vertical reinforcement lap splice area shown in figure 3-a. The hand installed carbon fiber wrap was composed of four layers each with a 0.052-inch thickness for a total thickness of 0.21 inch. The force-displacement results of the retrofitted column are shown in Figure 6. Individual 0.21 inch thickness carbon fiber bands increased column ductility ratio ($\Delta_w/\Delta_y$) to 12 before the test had to be stopped due to the displacement limits of the testing equipment. At 9-inch lateral displacement, the specimen was loaded from zero displacement to an increasing displacement in one direction only.
Encased Column Test G-2

This test column was identical to test G-1 except the 0.21-inch thick carbon fiber wrap was a continuous jacket over the lower half of the column as shown in figure 3-b. Load-displacement behavior of the test specimen was similar to that of the test column G-1 with strip-wrapped column, however, this test had a ductility ratio ($\Delta_w/\Delta_y$) of only 10.

Encased Column E-1

This column was strip-wrapped with four bands of 6-inch wide glass fibers placed at 12-inch centers at lower part of the column test as shown in figure 3-b. The glass wrap was composed of eight layers of 0.05-inch thickness fiberglass fabric for a total wrap thickness of 0.4 inch. The displacement history and the load-displacement behavior of this test specimen are not shown, but it was very close to the results of test column G-1, and it reached a ductility ratio ($\Delta_w/\Delta_y$) of 12 under cyclic loading.

Column Test Summary

A summary of the reported test results is given in table 1. These tests only represent a sample of the total tests performed, and indicate no significant difference between individual fiber bands and full encasement wraps. The bottom line is that the use of bands is beneficial to allow breathing of the concrete and avoid potential freeze-thaw effects on the column and fiber wrap.

<table>
<thead>
<tr>
<th>Column Test</th>
<th>Retrofit Ht.= L (In)</th>
<th>Wrap Thickness (In.)</th>
<th>Ductility Ratio $\Delta_w/\Delta_y$</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-1</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>Splice</td>
</tr>
<tr>
<td>G-1</td>
<td>5&quot; Bands @ 10&quot;=56&quot;</td>
<td>0.2 in. Carbon Fiber</td>
<td>12</td>
<td>Main Bar</td>
</tr>
<tr>
<td>G-2</td>
<td>Full encasement= 61&quot;</td>
<td>0.2 in. Carbon Fiber</td>
<td>10</td>
<td>Main Bar</td>
</tr>
<tr>
<td>E-1</td>
<td>6&quot; Strips @ 12&quot;=43&quot;</td>
<td>0.4 in. Glass Fiber</td>
<td>12</td>
<td>Main Bar</td>
</tr>
</tbody>
</table>

SUMMARY AND CONCLUSIONS

Advanced composite fibers are commonly used for seismic and non-seismic repair of concrete bridges. The question of long term durability of the fibers and components thereof remains to be resolved. Meanwhile, it is very important to apply composite fibers cautiously and monitor their long-term behavior to accumulate a reliable database. This article has discussed the potential problem and a proposed solution to prevent potential freeze-thaw effects on concrete columns encased with composite fibers.

During the repair process of a number of deteriorated concrete columns in the Chicago area, Hydroester stay-in-place forms were used to replace deteriorated concrete. The long-term presence of waterproof forms resulted in moisture entrapment and saturation of concrete inside. Field and laboratory studies found that freeze-thaw cycles, and the poor quality of the superplasticized mortar were the primary causes of the premature failure.
A strip method of fiber wrap application was proposed that allows ventilation of wrapped concrete for the purpose of minimizing the potential for freeze-thaw effects in cold regions. Results of cyclic load tests of 2-ft diameter strip-wrapped test columns showed no significant departure from the behavior of fully encased columns. The testing program included as-built columns (with no wraps), partially encased columns with carbon fiber, and strip-wrapped columns with carbon or glass fibers. The wrapped columns showed substantial improvement in ductility over similar columns with no wrapping. In addition, test results showed almost identical ductility ($\Delta_w \Delta_s$) and strength for both fully encased and strip-wrapped columns that were subjected to lateral cyclic loading.

The results of this investigation and the subsequent testing program clearly showed that encasement of concrete columns when subjected to freeze-thaw cycles could be detrimental. Unless provisions are made to allow natural evaporation of moisture, the use of fiber-wraps in full encasement of columns should be avoided in cold regions. Test results showed that wrapping of concrete with individual strips along the column is as effective as full encasement. The ductility ratio ($\Delta_w \Delta_s$) of columns wrapped with individual strips of glass fiber or carbon fiber significantly improved compared to as-built columns under lateral cyclic loading. Along the fully encased columns, it is a good practice to leave gaps at proper intervals to break the continuity of the wrap to allow natural evaporation of moisture.

LIST OF REFERENCES


Figure 1 - Illinois Route 45, Archer Avenue Bridge in Justice, Illinois
Figure 1 - Illinois Route 45, Archer Avenue Bridge in Justice, Illinois

Figure 2 - Scanning electron microscopy photos of the mortar inside the wrap
(After J. Francis Young, 1997)
Figure 3 - Laboratory test setup (After Shkurti and Hawkins)

Figure 4 - Cyclic load history (After Shkurti and Hawkins)
Figure 5 - Load-deflection curves for as built column
(After Shkurti and Hawkins, 1999)

Figure 6 - Load-deflection curves for retrofitted column G-1
(After Shkurti and Hawkins, 1999)
SEISMIC RETROFITTING OF THE I-57 BRIDGES
OVER ILLINOIS ROUTE 3 IN SOUTHERN ILLINOIS

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ABSTRACT

As part of an ongoing seismic retrofitting program undertaken by the Illinois Department of Transportation (IDOT), PB Booker Associates, Inc. was selected to perform the final seismic retrofit design of dual structures on Interstate 57 over Illinois Route 3 and the Illinois Central Railroad. Located in Alexander County in southern Illinois near the New Madrid seismic zone, these bridges each consist of thirty spans of continuously welded steel plate girders supported on reinforced concrete substructure units. This paper summarizes the evaluation of the existing structures and discusses the development of the retrofitting strategy, including the state-of-the-art structural enhancements incorporated into the superstructures and substructures of these bridges.

INTRODUCTION

Following the Loma Prieta earthquake in Northern California in October 1989, the Illinois Department of Transportation initiated a seismic retrofitting program, due to the seismic hazards posed by the New Madrid seismic region. This program began with a bridge condition survey. The first phase of IDOT’s bridge condition survey provided for the development of a preliminary screening system that ranked the Department’s inventory of over 6,700 bridges according to their seismic vulnerability. The preliminary screening identified six highly ranked structures that were advanced to the second phase. The second phase of IDOT’s bridge condition survey included the seismic evaluation and preliminary retrofit design for these critical structures. The dual bridges on Interstate 57 over Illinois Route 3 were included in the initial six structures that underwent preliminary retrofit design. In August 1994, IDOT issued a request for professional engineering services for the final seismic retrofit design and plan preparation for the pair of structures on Interstate 57. PB Booker Associates, Inc. was selected as the prime consultant, providing structural engineering services for this project. Woodward-Clyde Consultants provided the geotechnical engineering services.

The I-57/Illinois Route 3 bridges are located approximately 6 kilometers northwest of Cairo, Illinois and about 2 kilometers north of the Mississippi River (See figure 1). The structures, which include a northbound and southbound bridge, carry Interstate 57 over Illinois Route 3, the Illinois Central Railroad and the Army Corps of Engineers’ levee. Each structure consists of 30 spans of horizontally curved, welded steel plate girders, composite with a 190-mm reinforced concrete slab. The superstructures are typically divided into 3-span and 4-span continuous units with one single-span unit spanning between Piers 15 and 16. Typical span lengths are about 30 meters long, but vary in length from 25 meters to 46 meters. The out-to-out deck width is
approximately 13 meters. All superstructure units are supported on steel rocker bearings at "expansion" locations and fabricated steel bolsters at "fixed" locations.

Figure 1 – Project location sketch

The substructures consist of reinforced concrete, multi-column piers from Piers 1 to 16. Piers 17 to 25 are composed of reinforced concrete multi-columns supported on reinforced concrete walls. The southernmost piers, Piers 26 to 29, consist of reinforced concrete, wall-type piers. Column diameters for the multi-column piers range from 762 mm to 1219 mm. The thickness of the wall piers varies from 762 mm to 914 mm. The bridge piers are typically skewed to the centerline and vary in magnitude from the north abutment to the south abutment. Both abutments of each bridge are non-integral, seat-type and are situated at the top of the approach embankments. The reinforced concrete wingwalls turn back and are aligned parallel with the roadway.

The bridges are supported on cast-in-place, helically welded, cylindrical metal shell piles with a design load of 312 kN. Pile driving data indicate that the piles were driven to a refusal criterion based on the "Engineering News" formula. The pile capacities calculated by the pile driving formula were generally in a range of 445 kN to 534 kN. Generally, the outer rows of piles are battered outward 2H:12V. Pile caps vary in plan dimensions and were typically 914 mm to 1219 mm thick. The piles are driven through the upper 10 meters of low strength clay soils to bear in medium to very dense granular soils about 10 to 20 meters below grade. These piles are
considered friction piles since they develop capacity primarily from skin friction with some end-bearing in the granular formations.

These bridges were constructed in the early to mid-1970’s and lacked the seismic resistant details recommended in the most recent standards. Current AASHTO specifications recommend a bedrock acceleration of 0.22 g at this bridge site. AASHTO Soil Type III is appropriate, resulting in a ground surface acceleration of approximately 0.33 g. The retrofit design was performed in accordance with AASHTO and FHWA requirements for Seismic Performance Category C.

SEISMIC EVALUATION OF EXISTING STRUCTURES

The evaluation procedure, in general, was based on the May 1995 Federal Highway Administration publication entitled *Seismic Retrofitting Manual for Highway Bridges*\(^1\) (Retrofitting Manual) and on the 1996 AASHTO *Standard Specifications for Highway Bridges*\(^2\). This procedure provides for the identification of deficient bridge elements and the development of an overall retrofitting strategy. According to the Retrofitting Manual, bridge elements are evaluated quantitatively to determine their ability to resist the seismic forces generated by a design seismic event. Utilizing a Capacity/Demand (C/D) approach, an analysis is performed and the resulting forces and displacements, known as demands, are compared with the capacities of the bridge components to evaluate their ability to resist these forces and displacements. C/D ratios, representing the fraction of the design earthquake at which a local failure may occur, are calculated for critical elements of the bridge. C/D ratios less than one indicate failure of the component during the design seismic event. Therefore, an assessment of the consequences of that failure must be made to determine the need for retrofitting. According to the Retrofitting Manual, the components that require evaluation for bridges in Seismic Performance Category C include:

- Expansion joints and bearings;
- Reinforced concrete columns, piers and footings;
- Abutments; and
- Liquefaction.

Based on the requirements presented in the Retrofitting Manual, the multimode spectral method of analysis was used for these structures. The seismic analysis was conducted with the aid of global and local models. The bridges were modeled globally, from abutment to abutment, to depict the “as-built” conditions which include the existing mass, connections, foundation stiffness, enhanced material strengths and the geometry of the structures. Additionally, the global models aided in assessing the impact each unit had on adjacent units and provided data regarding the vibration and displacement characteristics of the structure as a whole. The preliminary multimodal analysis indicated that the displacements for each unit, in general, were not large enough to close the gap at each expansion joint. This led to the conclusion that each unit would behave such that the motion of the adjacent units was neither enhanced nor impeded.
Based on this conclusion, more localized “unit” models were created to model each unit separately. The most significant benefit of these unit models was the reduction of the computational effort required due to the reduced number of degrees of freedom. The structural analysis for the global and unit models was performed using SEISAB\textsuperscript{3}. SEISAB (SEISmic Analysis of Bridges) is a linear elastic, finite element program used for the seismic analysis of bridges which employs both the single mode and multimode analysis methods prescribed in AASHTO.

In addition to the global and unit models, local “pier” models were developed using STAAD-III\textsuperscript{4}, a multi-purpose, three dimensional, finite element program. The pier models were created to verify the displacement characteristics and force distribution within individual piers.

Site-specific dynamic response analyses were performed to evaluate the effects of localized soil conditions on the dynamic response at the ground surface. Response analyses were done for several cases using the program SHAKE\textsuperscript{5}. The analyses were carried out at three locations for two different depth profiles, shallow (40\textpm m) and deep (175\textpm m). The three locations are the north abutment, south abutment and the former river channel near Pier 22. The results of the analyses for the two depths were not significantly different and, thus, only results for the shallow profile are presented herein. The recommended site response spectra for the three locations are shown in figure 2. The response spectrum recommended by AASHTO is also included in figure 2 for comparison.

![Spectral Acceleration vs. Period](image)

Figure 2 – Comparison of site-specific response spectra and AASHTO response spectrum

**Superstructure Evaluation**

In general, superstructure support lengths at expansion joints were found to be inadequate. In addition, the rocker-type bearings (figure 3) created the potential for toppling. When coupled
with inadequate support lengths, toppling of rocker bearings posed the threat of a span drop-type failure.

Lateral restraint of the superstructure was also investigated and found to be inadequate. Typically, the longitudinal restraint for each superstructure unit was provided with a set of "fixed" steel bolsters (figure 3), located at one pier within the unit. The longitudinal restraint provided by the anchor bolts for these steel bolsters was insufficient when compared to the longitudinal forces generated by the elastic seismic analysis. In addition, the transverse restraint provided by the rockers and bolsters was also inadequate when compared with the elastic seismic demands.

![Diagram of expansion pier and fixed pier with labels: Top Plate, Rocker, Bottom Plate, Shim Plate, Lead Plate, Anchor Bolts.]

ELEVATION AT EXISTING EXPANSION PIERS  
ELEVATION AT EXISTING FIXED PIERS

Figure 3 – Typical expansion ( rocker) and fixed ( steel bolster) bearings

Substructure Evaluation

The substructure was investigated based on the assumption that the superstructure would be retrofitted to provide the necessary lateral restraint to transfer the seismic forces from the superstructure to the substructure.

The two primary seismic vulnerabilities associated with the columns included insufficient shear capacity and inadequate ductility to resist plastic hinging. Column shear failure is critical since it results in a brittle-type failure, with a sudden loss of shear strength.

For these structures, the column failure modes associated with the lack of ductility included loss of flexural capacity due to inadequate confining reinforcement and splice failures in the longitudinal column reinforcement. The columns for these structures were typically reinforced laterally with #4 (English) hoops at 300 mm centers (See figure 4). This reinforcement was
insufficient to confine the column cores and prevent buckling of the longitudinal reinforcement, when compared with the design earthquake demands. In addition, the column longitudinal reinforcement was generally spliced at the base of the column, in a region where plastic hinging was expected, with dowel bars that were embedded in the crash walls and/or pile caps (See figure 4). The investigation revealed that, due to the expected degradation of the base of the columns under the anticipated loading, the potential existed for the splices in the longitudinal reinforcement to fail.

Figure 4 – Typical seismic vulnerabilities of existing multi-column piers

Capacity/Demand ratios were also calculated for a variety of modes of pile cap yielding. The two primary failure modes of pile cap yielding included pile pullout and shear failure of the concrete in the pile caps (See figure 4). Pile pullout resulted for piles subjected to tension, since there was no positive connection provided between the piles and pile caps. Similar to a column shear failure, a concrete shear failure in the pile cap could be serious because of its sudden, brittle nature, resulting in a relatively sudden loss of overturning resistance.

Geotechnical Evaluation

The liquefaction potential of the existing soils was evaluated. In the northern half of the structures (north abutment through Pier 19), liquefaction was found to be spotty and not widespread. Consequently, the risk of damage from liquefaction was considered negligible and the bearing capacity of the existing piles was not reduced for investigating the foundations from the north abutment to Pier 19.
In the southern half of the bridges (Pier 20 to the south abutment), liquefaction was found to be widespread in the loose silts and medium dense sands between depths of about 10 to 18 meters below grade. The existing piles typically bear within the liquefiable zone. Coupled with the down-drag from the upper clayey soils and liquefaction in the bearing zone, pile capacity would be essentially negligible in this reach of the bridges. Consequently, bearing capacity failure and/or significant settlement of the bridge piers in this region should be expected unless mitigation is performed.

**RETROFIT STRATEGY**

Once the vulnerabilities associated with superstructure support lengths, rocker bearings and lateral restraint were identified, it became apparent that the connection between the superstructure and substructure (i.e., bearings) would require retrofitting. After investigating several schemes involving modifications to the existing bearings and installation of restraining devices, complete bearing replacement appeared to be the most feasible option. Several replacement systems were investigated including conventional elastomeric low-profile bearings, seismic isolation bearings and various methods of introducing damping into the structural system. These retrofitting systems were incorporated into revised global and local models and analyzed to determine their impact on the overall behavior of the structures. Eventually, the retrofitting strategy developed with two primary objectives in mind. These objectives were:

- Provide a more effective distribution of forces to the substructure, allowing each of the substructure units to participate, to some extent, in the resistance of the seismic loads; and

- Mitigate the inertial effects of the superstructure by a combination of isolation and damping whereby the energy is dissipated through controlled means rather than uncontrolled damage to the structures.

With this retrofitting strategy, the extent of the inelastic behavior in the structures was minimized. Column shear demands were reduced to tolerable levels. Force demands on foundations were also reduced to levels that no longer required retrofitting. Although plastic hinging was still expected to occur in a number of columns, the columns could be retrofitted to withstand the inelastic behavior.

The following sections describe the various methods and details used to implement this retrofitting strategy.

**Superstructure Retrofit**

The retrofit of the superstructures involved two primary modifications to the bridges. These modifications were:

- Replacement of the existing bearings with a seismic protection system; and
Installation of superstructure restraining devices at Span 16.

The concept of seismic isolation may not be the most efficient system where a flexible structure and soft soils exist. Since these structures were relatively flexible and partially located within a zone susceptible to liquefaction, it was undesirable to increase the fundamental period of vibration, which typically accompanies the use of seismic isolation bearings. Therefore, the retrofitting strategy was not solely based on the concept of seismic isolation. Rather, the strategy focused on introducing high levels of damping into the system to limit displacements and dissipate energy, reducing the forces transmitted to the substructure units.

A performance specification was written for the seismic protection system used for this project, outlining displacement, damping and force-level criteria. This specification was prepared in accordance with the AASHTO Guide Specifications for Seismic Isolation Design. Based on gaps at the existing expansion joints, the performance specification limited the maximum displacement of the superstructure, relative to the substructure, to 50 mm. The seismic protection system was also required to have self-centering capabilities.

To evaluate the feasibility of such a seismic protection strategy, a number of systems were investigated on a limited basis, with input provided from the manufacturers of these systems. A "dampened" spectrum was developed based on performance criteria provided by the design team. Figure 5 illustrates the expected level of damping. For a given period of vibration, the spectral acceleration was significantly lower for the "dampened" spectrum when compared to the site-specific spectra developed for this project. This was generally true when the period of vibration falls within the 0.7 second to 4 second range. The moment demands on the columns were generally 2.5 to 3 times less with the seismic protection system in place when compared with a system using conventional "expansion" and "fixed" low-profile bearings.

Figure 5 – Comparison of site-specific response spectra and "dampened" spectrum
Located near the midpoint of each of the bridges, Span 16 is a single span unit of variable length, with the supporting piers at each end skewed in opposite directions. Although large displacements were not anticipated with the retrofitted superstructure, the vulnerability of this skewed configuration, coupled with the lack of redundancy of the single span, led to the decision to install restrainers at Span 16. Figure 6 illustrates the restrainer details installed at Piers 15 and 16.

Figure 6 – Details of restrainers at Piers 15 and 16

Substructure Retrofit

With the seismic protection in place and appropriately modeled, the vulnerabilities associated with pile cap yielding (i.e., pile pullout and shear failure of the concrete in the pile caps) were eliminated. However, resulting column moment Capacity/Demand ratios still remained below 1.0 (as low as 0.46), indicating that the columns will yield sufficiently to require an evaluation of their ability to withstand plastic hinging. Further investigations indicated that column retrofitting was required for flexural confinement of plastic hinges and for flexural integrity of lap splices.

Column retrofitting was accomplished using tensioned prestressing tendons in the form of hoops, spaced at intervals over the required length of column. The prestressing tendons consisted of Grade 1860, 15-mm diameter, high strength, low relaxation 7-wire strand. The strands were
galvanized and encased in plastic sheathing for protection from the environment. In addition, the strands were greased to reduce the friction between the strand and column during stressing, thus providing a more uniform confining stress to the column. Each hoop consisted of a pair of strands and strand anchors. The design of the hoops resulted in a hoop spacing of 150 mm. The strands were tensioned from both ends to an effective prestress of 415 MPa, after losses.

By way of special provision, the Contractor was permitted the option of choosing one of four alternative composite column wrap systems, in lieu of the prestressing tendons. The four systems approved by IDOT were:

- System 1 – An epoxy-resin glass fiber, electrical grade, E-glass, hand applied composite wrap with a painted exterior surface;
- System 2 – An E-glass, prefabricated, segmented composite shell assembled around the column with an epoxy adhesive with a closure gap of 25 mm or less when attached to the column, and with prepainted exterior surfaces;
- System 3 – An epoxy-resin prepreg-carbon fiber composite wrap, machine applied, high-temperature cured, with painted exterior surface; and
- System 4 – An epoxy-resin prepreg-carbon fiber composite wrap, hand applied, ambient temperature cured, with painted exterior surface.

The design of the alternative composite wraps was performed in accordance with Illinois DOT Recommended Procedure for Design of Fiber Reinforced Composite Wraps.

In addition to the column improvements, retrofitting was required to mitigate the effects of the anticipated liquefaction from Piers 20 to 29. Two techniques for liquefaction mitigation were considered. These techniques include:

- Soil improvement at pier locations; and
- Underpinning to transfer foundation loads to bear in the non-liquefiable strata below the liquefiable zone.

Underpinning was recommended because it provides a more positive means of mitigation. Also, IDOT previously had limited success with soil improvement by compaction grouting below foundation piers at another nearby location.

Both driven piles (pipe and H-sections) and drilled-in piles (micro-piles) were considered for the underpinning of the existing foundations. Driven piles had the following two major disadvantages:

- Driven piles are difficult to install where headroom is limited, thus increasing the cost; and
- Pile driving could potentially cause settlement of the existing bridge.

In contrast, micro-piles can be installed at sites with limited headroom with little potential of causing settlement to the existing bridge if properly installed. Generally, micro-piles are more expensive than driven piles. However, considering the possible detrimental effects on the existing bridge with the installation of driven piles and the relatively small additional cost, micro-piles were used to underpin the existing foundations.

The new micro-piles for the foundations from Piers 20 to 29 were designed to support the entire dead load and seismic forces, resulting from the earthquake load case during liquefaction, assuming the existing piles were ineffective. The micro-piles were incorporated into the existing pile cap by drilling and epoxy grouting new reinforcement into the existing footing. The pile caps were encased in new concrete, with 900-mm extensions around the perimeter and a 600-mm overlay. IDOT had previous experience with a similar detail where new footing reinforcement was mechanically spliced to existing reinforcement. However, difficulties were encountered during construction with this detail when removing existing concrete to expose the existing reinforcement. Due to the required amount of concrete removal necessary to properly splice the new reinforcement to the existing reinforcement, an excessive quantity of concrete was removed in the vicinity of the existing piles, which, potentially, compromised the structural integrity of the foundation. Figure 7 depicts the past and current detail used by IDOT for retrofitting of pile-supported foundations.

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Figure 7 – Past and current pile-supported foundation retrofitting details
CONCLUSIONS

The primary goal of seismic retrofitting of existing bridges is to minimize the risk of unacceptable damage during a design earthquake. There are currently a variety of techniques that structural and geotechnical engineers have at their disposal to accomplish this goal. These techniques include seismic isolation, energy absorption, increasing structural continuity and ductility, strengthening, and site stabilization. The seismic retrofit design of the I-57/Illinois Route 3 bridges utilized several of these techniques. The most effective and efficient retrofitting strategy for these bridges, based on the analysis, was one that combined a seismic protection system, incorporating elements of isolation and damping, along with measures for increasing the ductility and strength of critical substructure elements. Overall, the seismic performance of these structures was greatly enhanced with the implementation of state-of-the-art technology.

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GEOTECHNICAL ASPECTS OF THE SEISMIC EVALUATION OF BRIDGES
MIDWEST EXPERIENCE

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ABSTRACT

This paper presents results of detailed site specific response analyses carried out for three major projects in the New Madrid Seismic Zone. The analyses were based on measured shear wave velocity data and were calculated using computer code SHAKE with input time histories developed from earthquake records spectrally matched to empirical rock spectra for each site. Results of the analyses are compared to the common approach for routine projects of using code-based spectra and site effects accounted for by soil coefficients determined from conventional geotechnical data. Results show that the code-based approach is close or generally over predicts response for moderate size (500 year) events, but under predicts, sometimes significantly for larger (2500 year) events. The 97 NEHRP provisions generally provided the best estimate of site response of the three code-based approaches checked.

INTRODUCTION

During the past few years the authors have performed seismic evaluations for several major bridges in the New Madrid Seismic Zone (NMSZ), all located in the floodplain of the Mississippi River. Because these were large important structures, a higher level of effort was used to evaluate ground motions than for smaller projects. In particular, site specific response spectra were developed for each structure based on measured shear wave velocities at each site. In contrast, most routine projects rely on code-based spectra and local site effects using soil profile types based on common geotechnical data such as Standard Penetration Test blow counts, shear strength tests, and soil index properties. This paper compares the results of the more detailed site specific analyses for these projects to the code-based approaches of AASHTO 1996 (1), NEHRP 1994 (2), and NEHRP 1997 (3). While this paper only covers a few projects, it provides some insight into the reasonableness of the code-based spectra.

PROJECT DESCRIPTIONS AND SUBSURFACE CONDITIONS

Analyses were performed at three bridge sites in the Mississippi River Valley; 1) the proposed Bill Emerson Bridge over the Mississippi River at Cape Girardeau, Missouri, 2) the Interstate 57 Bridge over Illinois Route 3 in Cairo, Illinois, and 3) the Interstate 40 Bridge (Hernando DeSoto Bridge) over the Mississippi River at Memphis, Tennessee. (Figure 1) The first project involved new construction, and the other two seismic retrofit of the existing bridges. Seismicity at each site is dominated by the NMSZ, the most active portion of which is shown in Figure 1 based on Johnson and Nava, 1994 (4). Project details are given in Table 1 and generalized subsurface profiles are given in Figure 2.
| **Table 1 - Project descriptions and generalized subsurface conditions** |
| --- | --- | --- | --- |
| **BRIDGE SITE** | **Emerson Bridge**<br>(Mississippi River)<br>Cape Girardeau, MO.<br>Missouri Approach | **Emerson Bridge**<br>(Mississippi River)<br>Cape Girardeau, MO.<br>Illinois Approach | 1-57 over Illinois 3<br>Cairo, IL. | 1-40 over Mississippi River<br>Memphis |
| **Bridge Type and span length** | Main Span --Cable Stayed 350 meters, MO approach on 8 meter high embankment | Illinois Approach -- steel girders, 50 to 60 meters. | Steel girders, 25 to 46 meters | Tied Arch, steel and concrete girder |
| **Foundations** | Rock bearing hydraulic caissons and rock bearing footing- main span. | Illinois Approach on rock bearing drilled shafts | Friction piles in sand | Tied Arch –caissons Steel and concrete girder spans on steel and concrete piles |
| **Soil Conditions** | 11+ meters of firm low plastic clay over limestone bedrock | 30+ meters of loose to very dense sand over limestone bedrock | 27+ meters of loose to dense sand over Mississippi embayment deposits of very dense sand. Bedrock ~ 150+ meters deep | 30 meters of medium to dense sand over Mississippi embayment deposits of hard clay. Bedrock ~ 1000 meters deep |
| **Design bedrock acceleration, return period, and magnitude event (M<sub>s</sub>)** | 0.36g, 2500 yr. M~8.4 | 0.36g, 2500 yr. M~8.4 | 0.28 g, 500 yr, M~6.4 | 0.185 g 500 yr, M~ 6.4 |
| **Shear Wave Velocity Measurements:**<br>-Number of locations<br>-Max. Depth (meters)<br>-Penetration into rock or hard soil (meters) | 1<br>20.8<br>9.1 | 1<br>37.3<br>9.1 | 2<br>43.6 and 47.8<br>22.9 and 22.2 | 2<br>45.7 and 47.2<br>9.1 each |
| **Average Shear Wave Velocity, upper 30 m of native soils (meters/sec)** | 396 (299 with embankment) | 218 | 239 (average two locations) | 207 (average two locations) |
| **'94 and '97 NEHRP Soil Profile Type for native soils** | C (D with embankment) | D | D | D |
| **Average SPT blow count, upper 30 meters (blows/30 cm)** | NA | 29 | 30 (average of two locations) | 34 (average of two locations) |
| **1996 AASHTO Soil Profile Type for native soils** | S3 with and without embankment | S2 | S3 | S2/S3 |
SHEAR WAVE VELOCITY MEASUREMENTS

Down-hole shear wave velocity measurements were made at each site using a conventional down-hole technique (Crice 1980 (5), Fumal and Tinsley, 1985 (6)). This method uses a seismic source located at the ground surface near the borehole and pulses from the source are detected by a sensor positioned in the borehole. The travel times of the detected energy pulses are measured at various depths within the borehole. Seismic velocities are then calculated from the travel times of the pulses. Note that for the Missouri Approach of the Emerson Bridge the table above lists the measured shear wave velocities in the upper 30 meters of native soils as well as with the proposed embankments since the SHAKE analyses was done with the embankment included. The shear wave velocity of the embankment fill material was assumed to be 245 m/s corresponding to a stiff to very stiff compacted low plastic clay.
Figure 2
Generalized Subsurface Conditions
and Shear Wave Velocity at
Bridge Sites
SITE SPECIFIC RESPONSE ANALYSIS, GENERAL APPROACH

The same approach was used to calculate site response analyses at each site and included the following tasks:

1. Establish a median target rock spectrum for rock for the site. The target rock spectrum was based on attenuation relationships by McGuire et al 1988 (7), and Nuttli 1973, 1986 (8, 9), with the Newmark and Hall 1978, 1982 (10,11) amplification factors. The final target rock spectrum chosen was generally between these two relationships and was anchored at the zero period of acceleration for each site. The target rock spectra are shown in Figures 3 through 7.

2. Develop rock spectrum compatible time histories. The spectrum compatible time histories were scaled from available records of earthquakes from similar tectonic environments with similar magnitudes and recorded distances as the design earthquakes. The horizontal components of each recorded motion were used as input and modified to fit the target spectrum using an in-house computer code. Because there are no available records for large eastern US earthquakes, records from the 1985 Michoacan Mexico earthquake (Caleto de Campos Station) were used for the M ~ 8.4 events at Cape Girardeau and Memphis. The 500 year events at I-57/I13 and Memphis used the 1985 Nahanni record, Station No. 3, and 1988 Saguenay record, Station No. 16.

3. Calculate ground surface response spectra. Ground surface response spectra were calculated using the computer program SHAKE (Schnabel et al, 1972, (12) based on the spectrum compatible time histories for each site and the shear wave velocity data. At each site SHAKE runs were made for both horizontal components of the time histories based on the measured shear wave velocities, and the measured shear wave velocities modified by ± 10 to 20 percent to account for variations in the velocities. Therefore a total of at least 6 runs were made for each site for each design earthquake. A final smoothed spectra representing about the mean of the SHAKE results was drawn and recommended for design. These smoothed spectra are shown as the “SHAKE” spectra in Figures 3 through 7.

SITE RESPONSE RESULTS AND COMPARISON TO CODE-BASED SPECTRA

The scale and importance of these projects allowed a more robust site response analysis than used on routine projects which rely on simpler code-based spectra and general soil descriptions to estimate site response. In addition, few analyses such as these based on actual shear wave velocity
measurements are available in the Central US as compared to other regions such as California. Therefore these case histories provide some insight as to the reasonableness of the code-based approach to estimate site response analysis in the Central US.

When comparing the results of the SHAKE analysis to the code-based results, we selected soil profile types on the results of the conventional geotechnical data such as Standard Penetration Test N-values rather than the measured shear wave velocities, since for most routine projects, only the standard geotechnical data are available. For these cases, however, the site classifications based on the conventional geotechnical data and measured shear wave velocities were similar. The bedrock acceleration for the code-based spectra was the same used for the SHAKE analysis and was determined in site specific studies (Memphis, I-57), or taken from published hazard maps (Emerson). For the NEHRP spectra, we also assumed that the rock spectra were on a soft rock material (Soil Profile Type B/C). At the I-57 and I-40 sites, the “rock” motion was on the Mississippi Embayment soils of hard clay/very dense sand as opposed to rock. The plotted response spectra for each code-based method shown in Figures 3 through 7 are computed as follows:

<table>
<thead>
<tr>
<th>Spectral Ordinate Value</th>
<th>96 AASHTO</th>
<th>94 NEHRP</th>
<th>97 NEHRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat Portion of Spectra</td>
<td>C = 2.5 A</td>
<td>C = 2.5 F_s A</td>
<td>C = 2.5 F_s A</td>
</tr>
<tr>
<td>Curved Portion of Spectra</td>
<td>C = 1.2A/T^{2/3}</td>
<td>C = 1.2 F_s A/T^{2/3}</td>
<td>C = 1.2 F_s A/T</td>
</tr>
</tbody>
</table>

Where C = Spectral Ordinate Value, A = bedrock acceleration at site, S=AASHTO Site Coefficient for Soil Profile Types (I through IV), T = period in seconds, F_s and F_v are NEHRP Site Coefficients for generalized Soil Profile Types (A through F).

Note that the 1997 NEHRP approach is recommended for adoption by AASHTO (Friedland, 1997, 13).

Specific results for each site are discussed below.

**Emerson Bridge, Missouri Approach**

The results for this analysis indicate that the code-based spectra typically underestimate the SHAKE spectra, from about 0.4 to 2 seconds (Figure 3a and 3b). Except for the AASHTO spectra, however, the difference is no more than about 20 percent. The 97 NEHRP spectra is closest to the SHAKE spectra in the long period range beyond about 2 seconds. This site includes a stiff embankment fill over firm clay and shallow bedrock. Without the embankment this is clearly a NEHRP “C” category; with the embankment it is near the upper end of the “D” category.
Figure 3a - Response spectra, Missouri Approach, Emerson Bridge

Figure 3b - Percent of SHAKE spectral ordinates by various methods, Emerson Bridge, Missouri Approach

**Emerson Bridge - Illinois Approach**

This is a floodplain site that is a NEHRP "D" category based on both the geotechnical data and the shear wave velocity data. The soil conditions include interbedded sands and clays in the upper portion of the profile, making it difficult to select between Soil Profile Type S2 or S3 in the 96 AASHTO criteria.
In all cases, the code-based spectra underestimate the response, especially beyond 1 second. (Figure 4a and 4b). The '96 AASHTO underestimates by about 50 percent, while the NEHRP spectra are up to about 30 percent lower. Using a NEHRP "E" spectra would improve the estimate in the long period range, but the short period (flat portion) the curve would be lower, thereby reducing the accuracy in the short period range. In this case the 94 NEHRP values are closest to the SHAKE results, especially beyond 1 second.

Figure 4a - Response spectra, Illinois Approach, Emerson Bridge

Figure 4b - Percent of SHAKE spectral ordinates by various methods, Emerson Bridge, Illinois Approach
I-57/Illinois Route 3 Bridge

Site response calculations for this bridge were more complex due to the great depth to bedrock (150 + meters) and need to model the very dense Mississippi Embayment soils above bedrock. To account for the deep embayment soils, two SHAKE analyses were run; one modeling the full depth of soil down to bedrock (150+ meters), and the other to a shallower depth (43 meters) where the shear wave velocity was estimated to be 610 m/s or more. The shear wave velocity of the soils below the depth of the geotechnical investigation was based on work by Bernreuter et al, 1989 (14). The results for the two analyses (deep and shallow) were not appreciably different and we selected the results of the shallow depth.

Results are given below and indicate that the code-based spectra, especially NEHRP, are in good agreement being slightly higher than the SHAKE results for periods less than about 1 second. Beyond 1 second, they tend to over predict significantly except the 97 NEHRP which is about 20 percent above the SHAKE results (Figure 5a and 5b). Note that in this case the design earthquake is a moderate (500 year) event, as opposed to the Emerson Bridge site.

![Figure 5a - Response spectra, I-57/IL3 bridge](image-url)
I-40 Bridge, Memphis, 500 year event

At this site, two analyses were run; one for a moderate event (500 year return period, $M \sim 6.4$) and one for a large event (2500 year return period, $M \sim 8.4$). This allows a comparison of the reasonableness of the code-based spectra for these two events. Results show that for the 500 year event, the NEHRP values greatly over predict the response, especially for periods exceeding 1 second. In this case, the 97 NEHRP is generally a better predictor than the 94 NEHRP (Figure 6a and 6b). 96 AASHTO under predicts less than about 1.5 seconds and slightly over predicts beyond 1.5 seconds.
Figure 6a - Response spectra, Memphis site 500 yr. return period

Figure 6b - Percent of shake spectral ordinates by various methods, Memphis site, 500 yr. return period

1-40 Bridge, Memphis, 2500 year event

Results in this case are significantly different from the 500 year event (Figure 7a and 7b). For the 2500 year event, all code-based methods significantly under predict the response compared to SHAKE. Again, the poorest agreement is with 96 AASHTO which is up to 50 percent below the SHAKE results. The NEHRP spectra are closer, but still 20 to 40 percent below the SHAKE
results except beyond about 2.5 seconds. As with the Emerson Bridge site, the code-based spectra under predicted the response for the 2500 year event. Using an “E” Soil Profile Type at this site would give better prediction for the long period range, but reduce accuracy in the short period range. Also, it would not be consistent with the 500 year event.

Figure 7a - Response spectra, Memphis site 2500 yr. return period

Figure 7b - Percent of SHAKE spectral ordinates by various methods, Memphis site, 2500 yr. return period
CONCLUSIONS

While this paper considers only a few case histories, it points out some interesting findings regarding site response analysis in the Central US. These conclusions need to be confirmed by further studies, but provide some general insight into the reasonableness of the code-based spectra.

1. The code-based spectra, 96 AASHTO, 94 NEHRP, and 97 NEHRP appear to over predict response for the moderate (500 year events), but tend to under predict response, sometimes significantly, for the large (2500 year events). This later finding is significant since codes in the Central and Eastern US are moving away from using the 500 year event as a design basis and toward adopting the 2500 year event.

2. The NEHRP spectra that are based on a quantitative measure of site conditions are a better predictor of site response than the 96 AASHTO method which is not quantitative and relies on general soil profile descriptions.

3. The 97 NEHRP spectra, which is proposed for use by AASHTO, is generally closer to the SHAKE results than the 94 NEHRP approach and is a much closer than 96 AASHTO which had the poorest agreement with SHAKE. In most cases the 96 AASHTO values were non conservative, especially in the short periods.

4. For critical projects, the more robust approach of site specific response analysis based on actual shear wave velocity measurements is appropriate, especially for projects designed for large (2500 year) events.

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Mr. Edward Wasserman P.E., Director of the Structures Division, Tennessee Department of Transportation.
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EFFECTS OF VERTICAL EARTHQUAKE MOTIONS ON THE BEHAVIOR OF HIGHWAY BRIDGES

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ABSTRACT

The design of bridges for seismic loads rarely includes the effects of vertical accelerations. This results in part to a lack of understanding of the occurrence and amplitude of vertical ground motions. Recent compilations and investigation of vertical accelerations are reviewed in this paper. Currently available data indicates that in the near-source region of an earthquake vertical motions can be considerably larger than horizontal motions, and both may be three times or more greater than assumed for design. The response of bridges to vertical ground motion is discussed. In regions of very strong shaking, vertical accelerations acting in conjunction with horizontal motions may significantly increase the amount of damage. Recommendations for design and research are also given.

INTRODUCTION

Design of bridges for seismic loads rarely includes consideration of vertical ground motion. This stems, in part, from past observations of recorded ground motion that indicate that vertical accelerations are almost always smaller than horizontal accelerations. A general "rule of thumb" is that the effective peak acceleration (EPA) for the vertical accelerations recorded at a site will be on average 2/3 of the EPA for the horizontal accelerations. Recent data recorded at certain sites refutes this assumption. In addition, the profession's understanding of the effects of bridge response to vertical ground motion is limited by the lack of actual observations and research on this topic.

This paper will review recent compilations and studies of recorded strong motion data that include vertical accelerations. Results of prior and current research will be summarized. Recommendations for design and future research are also included. Much of this paper is included in the proceedings of a recent workshop (Foutch, 1997).
VERTICAL EARTHQUAKE GROUND MOTION DATA

Prior to the 1971 San Fernando Earthquake, only 22 recorded earthquake accelerograms with vertical components were available to design and research engineers (Naeim and Anderson, 1996). Those that were available indicated that the vertical motions were small in amplitude compared to horizontal motions. The largest vertical peak acceleration (PA) for these was 0.25 g recorded during the Parkfield Earthquake at the Cholame Shandon Array 2 site (Naeim and Anderson, 1996). The San Fernando Earthquake generated an additional 58 strong motion records of vertical accelerations and 153 records of horizontal accelerations. Only the Pacoima Dam record at 696 cm/sec² exceeded 20% g in the vertical direction. Twenty of the horizontal motions generated peak accelerations in excess of 20% g with the largest being the two components recorded at the Pacoima Dam site with PA's of 1125 and 1071 cm/sec² (Naeim and Anderson, 1996).

Naeim and Anderson (1996) recently published a compilation of strong motion data. Much of this data came from the USGS data base of 4,270 strong motion records recorded in North America from 1933 through 1986 (Seekins, et.al., 1992). Naeim and Anderson (1996) include selected North American records through the Northridge Earthquake of January 17, 1996. Their data base includes over 300 accelerograms with PA’s in a horizontal component of between 20% g and 60% g. For vertical motions, over 80 accelerograms had PA’s between 20% and 60% g and 11 had PA’s exceeding 60% g. The largest of these is 1734 cm/sec² recorded at Site 1 during the 1985 Nahanni Earthquake.

In addition to the usual data such as peak acceleration, velocity and displacement generated during the data processing operation, Naeim and Anderson (1996) produced valuable information from each accelerogram such as duration of strong motion, elastic response spectra, energy spectra, constant strength response spectra, constant ductility response spectra, input energy spectra and hysteretic energy spectra for the most significant of the horizontal and vertical accelerograms.

CHARACTERISTICS OF VERTICAL EARTHQUAKE GROUND MOTION

Absolute acceleration response spectra for two typical strong motion records are shown in Figure 1. The peak horizontal and vertical acceleration recorded at the Sylmar County hospital parking lot recorded during the Northridge Earthquake were 827 cm/sec² and 525 cm/sec², respectively. The ratio of the vertical-to-horizontal PA is 0.63. The peak horizontal and vertical acceleration recorded at the Corralitos-Eureka Canyon Road site during the 1989 Loma Prieta Earthquake were 618 cm/sec² and 431 cm/sec², respectively. The ratio of the vertical-to-horizontal PA is 0.70. These fall within the average range of vertical-to-horizontal PA ratio of 2/3 which has become a "rule of thumb" among engineers. Figure 1 indicates that the energy in each vertical component is concentrated in the short period range of 0.1 to 0.2 seconds.

Absolute acceleration response spectra for two stations at near-source locations are shown in Figure 2. The horizontal and vertical peak accelerations recorded at the Differential Array station during the 1979 Imperial Valley Earthquake were 476 cm/sec² and 583 cm/sec², respectively, with a peak vertical-to-horizontal ratio of 1.22. The horizontal and vertical peak accelerations recorded at the Arleta-Nordhoff Ave. Fire Station site during the 1994 Northridge Earthquake were 337
cm/sec² and 541 cm/sec², respectively, with a peak vertical-to-horizontal ratio of 1.60. Again, the energy in the vertical components was concentrated in the short period range of 0.1 to 0.2 seconds. These are also noteworthy because the vertical peak accelerations were substantially larger than the peak horizontal accelerations with ratios of peak vertical-to-horizontal ratios greater than one.

Absolute acceleration response spectra for the Icy Bay site for the 1979 Southern Alaska Earthquake and for the Jensen Filtration Plant site for the 1994 Northridge Earthquake are shown in Figure 3. These are quite irregular in that they possess significant energy content out to the one to two second range. Although the soil conditions at these sites are not known by the author, these were quite likely to be soft soil sites given the significant energy present in the long period range for both horizontal and vertical components of motion. This could be significant since soft soils are abundant along the Mississippi River and other major rivers flowing into it such as the Ohio and Missouri Rivers.

As noted above, it is commonly assumed that the V/H peak acceleration ratio is about 2/3 on average. This is obviously a gross over simplification. The data presented above indicate that this ratio exceeds 1.0 for some near-source sites. This ratio is actually a function of source-to-site distance. Since peak acceleration is usually associated with short period motion, it has very little to do with damage potential. Likewise, the V/H peak acceleration ratio has very little importance when considering the effects of strong ground motion on structural damage. It is a parameter, however, that is recognized by structural engineers and provides simple comparison of relative amplitude of vertical and horizontal accelerograms.

Bozorgina, et.al. (1994), studied selected ground motions recorded during the Northridge Earthquake. Instead of V/H peak acceleration ratio, they investigated the V/H response spectral ratio as a function of period and source-to-site distance. Figure 4 shows their results for alluvium and stiff soil sites.

These data show several trends. The V/H spectral ratios are very dependent on source-to-site distance and period. For a given site condition and source-to-site distance, the V/H ratio is larger near a period of 0.1 seconds than near a period of 1.0 second by roughly a factor of 3 to 4. For stiff-soil near-source (<10 km) sites the V/H ratio is about 1.3. For alluvium near-source sites the V/H ratio is about 0.8. The V/H ratio is about 0.3 in the 1.0 second period range for each of these sites for all source-to-site distances. Thus, main span segments of major river crossings are not expected to be significantly affected.

**DAMAGE POTENTIAL FOR VERTICAL EARTHQUAKE MOTIONS**

As mentioned above, peak acceleration is not a good measure for evaluating the damage potential of earthquake ground motion. However, if it is recognized by design professionals and researchers as a very rough measure of the relative "strength" of a ground motion record.

Peak velocity is commonly recognized as one of the best single-parameter measures of the damage potential of an acceleration record. Others that have been proposed are peak displacement, maximum incremental velocity and maximum incremental displacement. These parameters are
Figure 3.
Figure 4. Vertical-to-horizontal spectral ratio for varying site-to-source distance for Alluvium (top) and stiff soil sites for Northridge Earthquake.
listed in Table 1 for the vertical accelerograms with the largest peak velocities. The parameters for the largest horizontal record for the same site and event are also shown.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Station Name</th>
<th>Vertical Record</th>
<th>Horizontal Record</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>PA</td>
<td>PV</td>
</tr>
<tr>
<td>N. Palm Springs</td>
<td>Devers</td>
<td>816</td>
<td>87</td>
</tr>
<tr>
<td>Northbridge</td>
<td>Tarzana</td>
<td>1028</td>
<td>72</td>
</tr>
<tr>
<td>Petrolia</td>
<td>Cape Mend.</td>
<td>739</td>
<td>60</td>
</tr>
<tr>
<td>Imp. Valley</td>
<td>Array 6</td>
<td>1655</td>
<td>57</td>
</tr>
<tr>
<td>San Fern.</td>
<td>Pacoima</td>
<td>696</td>
<td>56</td>
</tr>
<tr>
<td>Northbridge</td>
<td>Pacoima</td>
<td>1205</td>
<td>49</td>
</tr>
<tr>
<td>Nahanni</td>
<td>Site 1</td>
<td>1734</td>
<td>42</td>
</tr>
<tr>
<td>Imp. Valley</td>
<td>Array 5</td>
<td>377</td>
<td>41</td>
</tr>
<tr>
<td>Point Mugu</td>
<td>Santa Clara</td>
<td>108</td>
<td>39</td>
</tr>
<tr>
<td>Landers</td>
<td>Lucern V</td>
<td>668</td>
<td>36</td>
</tr>
</tbody>
</table>

PA = Peak Acceleration; PV = Peak Velocity; PD = Peak Displacement; IV = Incremental Velocity; ID = Incremental Displacement

Using PV as a measure of the damage potential, the ratio of PV-vertical to PV-horizontal has an average of 0.51 for six of the accelerograms listed in Table 1. This difference between horizontal of vertical motion results from the fact that most of the energy for these vertical records is concentrated in the very short period range (0.10 seconds). For two of the records, however, the ratio of PV-vertical to PV-horizontal is greater than 1.0. For the San Fernando Pacoima dam record the ratio is 2.0. For the Imperial Valley Array 5 record it is 2.2. In fact, this ratio is greater than 1 for all of the parameters except PA. This is a strong indication that vertical motions in the near-source region can have a larger destructive capacity than the horizontal motions.

**BRIDGE RESPONSE TO VERTICAL GROUND MOTIONS**

Most highway bridges are not designed for vertical earthquake accelerations. One reason for this stems from the fact that bridge piers and abutments are designed for factored dead and live loads that are much greater than the dead load that would be considered for seismic motions. Thus, the piers and abutments have a very large margin of safety under seismic excitation. Also, it has been the general belief that the vertical motions are small compared to the horizontal ones.
RESPONSE OF RC COLUMNS AND BRIDGES WITH VARYING HORIZONTAL AND VERTICAL LOADS

The strength of an RC column is a function of the interaction between axial force and moment. A typical interaction curve is shown in Figure 5. So, for a fixed axial force the moment capacity can be obtained. One of the difficulties in assessing the behavior of a column subjected to varying axial load and moment is that the instantaneous location inside the interaction curve changes with time. A bridge will respond to horizontal and vertical motion during an earthquake which in turn causes the moment and axial force to vary with time. It is also true that the shear capacity of the column will increase and decrease with time as the axial force in the column varies. These factors complicate the behavior of the column.

In a study by Saadeghvaziri and Foutch (1988), the effects of varying vertical and transverse loading on column behavior and bridge response were studied. The study was limited to two directions, vertical and transverse. A finite element model of the columns was used which considered the concrete and reinforcing separately. The model was verified by comparison with laboratory tests. The bridge deck was modelled as a flexural beam (Saadeghvaziri and Foutch, 1988).

The load-deformation behavior of a column with constant and varying axial force, \( P \), are shown in Figure 6. The figure indicates that varying the axial force in a column can have a very significant effect on the behavior of the column. Of course, the actual behavior depends greatly on the sequence of loading.

Saadeghvaziri and Foutch (1988) studied the behavior of eight typical two span bridges, some with single- and some with two-column bents. The bridges were similar to the California RC box girder spans cast monolithically with the pier and the abutments. This study demonstrated that for bridges located in regions where the EPA is 0.4 g or less the vertical accelerations had almost no effect on the overall behavior of a bridge. This is due to the large margin of safety for the bridge piers under dead and live load. For regions where the EPA might be greater than 0.7 g, the vertical response acting in conjunction with the horizontal response has a significant effect on the performance of the bridge.

One bridge studied was a 2-span (140 ft/span) 2-lane RC box girder bridge considered to be monolithic with the pier and abutments. The pier was a 2-column bent. The columns were assumed to be 30 ft long. The model was analyzed for spectrum-compatible ground motions with an EPA of 0.7 g. The vertical reactions at the pier and abutments are shown in Figure 7. This result indicates that both the column and abutment will experience net uplift during the earthquake. The damage to the column is 75% greater for the H&V case, where damage is defined as the area of concrete experiencing strains >0.003. The maximum moment occurring at the base of one of the columns was 4450 k-ft for H-only and 6050 k-ft for H&V. The range in column force is 420 k compression to 1900 k compression for H-only and 400 k tension to 3200 compression for H&V (Saadeghvaziri and Foutch, 1988).
Figure 5. Interaction curves for a typical column

Figure 6. Hysteresis behavior with constant and varying axial force
Figure 7. Time history of column axial force (top) and abutment reaction (bottom)
Another bridge that was studied was a 2-span (150 ft/span) 4-lane RC box girder bridge considered to be monolithic with the pier and abutments. The pier was a 2-column bent with 30 ft long columns. For the H-only response the column behavior is nearly linear because the stiff superstructure carries the transverse loads directly to the abutment. The shear capacity varies because of the varying axial load. The shear capacity goes to zero when the axial force is in tension. When this happens the abutments are asked to carry all of the shear. If abutment failure occurs at this time the bridge could collapse.

A summary of observations from this study are as follows.

- For earthquake motions with EPA of 0.4 g or less the vertical accelerations cause relatively minor additional damage compared to the case with H-only excitation.

- For earthquake motions with EPA of 0.7 g or higher, the vertical accelerations result in considerably more additional damage compared with the H-only excitation.

- The varying axial force in the columns results in pinched hysteresis behavior that causes larger horizontal displacement and column end moments and curvature.

- Much larger column forces (compression and tension) occur when vertical motions are considered.

- Tension forces in the columns reduce the shear capacity to zero causing much larger reactions to be carried by the abutments.

- The larger compression forces in the columns increase the shear capacity of the column and results in much larger axial and shear loads to be transmitted to the foundation.

RESPONSE OF A MAJOR TRUSS SPAN TO HORIZONTAL AND VERTICAL GROUND MOTIONS

An elevation view of the Mississippi River crossing of I57 near Cairo, Illinois is shown in Figure 8. This bridge is being studied as part of a Mid-America Earthquake Center project. A schematic of part of the superstructure of the truss is shown in Figure 9. The model of the entire bridge was subjected to the earthquake ground motions shown in Figure 10 which represent motions from a M7.5 earthquake centered 75 miles away. Results for the seven members identified in Figure 9 are shown in Tables 2 and 3 for horizontal shaking and in Table for horizontal plus vertical shaking. Reactions for selected bearings are shown in Figure 11.

The results in Tables 2 and 3 indicate that the vertical motions can increase the member forces by as much as 20%. Similar results are found for the vertical reaction at the bearings shown in Figure 11.
Figure 9 – Location of truss members specified in table 2 and 3.
Figure 10 - Input ground motions.
Table 2 - Response of truss members under dead load and horizontal motions

<table>
<thead>
<tr>
<th>Member (No.)</th>
<th>Dead Load</th>
<th>Horizontal Motions</th>
<th>Interaction factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Axial Force kips</td>
<td>Moment 2-2 kips-ft</td>
<td>Axial Force kips</td>
</tr>
<tr>
<td>1</td>
<td>558</td>
<td>-16</td>
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<tr>
<td>2</td>
<td>695</td>
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<td>-60</td>
</tr>
<tr>
<td>7</td>
<td>-4382</td>
<td>-678</td>
<td>-225</td>
</tr>
</tbody>
</table>

Table 3 - Response of truss members under dead load, horizontal and vertical motions

<table>
<thead>
<tr>
<th>Member (No.)</th>
<th>Dead Load</th>
<th>Horiz. + Verti. Motions</th>
<th>Interaction factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Axial Force kips</td>
<td>Moment 2-2 kips-ft</td>
<td>Axial Force kips</td>
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<td>0</td>
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<tr>
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<td>695</td>
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<tr>
<td>7</td>
<td>-4382</td>
<td>-678</td>
<td>-225</td>
</tr>
</tbody>
</table>
Figure 11 – Bearing reactions.
SUMMARY AND RECOMMENDATIONS

A review of currently available strong motion data reveals that very large vertical motions occur at near-source sites. In many cases, the peak accelerations and damage potential of the vertical motions at a site exceed those of the horizontal motions by as much as a factor of 2. These large vertical motions should be accounted for in the design of highway bridges at potential near-source sites.

An analytical investigation of typical two-span bridges subjected to horizontal and vertical spectrum-compatible ground motion with effective peak accelerations of 0.7 g were conducted. The results revealed that there was a significant difference in the response and accumulated damage between the cases for H-only and V&H excitation. Large uplift forces occur in the columns of the pier and on the abutments. Larger than expected compression forces in the columns result in much larger vertical and horizontal forces to be transmitted to the pier foundation. The results of the analytical study were limited by the fact that only two directions of response, vertical and transverse, were included. The longitudinal response would result in even more damage to the columns. This is particularly true for bridges with free supports at the abutments. The damage was also underestimated because of the perfect fixity assumed at the abutments.

For bridges with stringers supported by bearings at the piers and abutments, the large fluctuations in bearing forces resulting from vertical motions will result in significantly more damage to bearings than expected. The increased transverse reactions at the abutments caused by loss of stiffness of the pier can result in reactions that are 2 or 3 times greater than assumed for design.

It is important that additional research be done. The response of typical bridges with realistic support conditions needs to be done. The analysis should be done for tri-axial loading using some of the large near-source records from recent earthquakes. These analyses will lead to more realistic design procedures for bridges.

In the interim, it is recommended that the current load interaction equations be modified and an additional one be added. The resulting seismic load interactions equations would be:

\[ P_c = P_L + 0.3 P_T + 0.3 P_V \]
\[ P_c = 0.3 P_L + P_T + 0.3 P_V \]
\[ P_c = 0.3 P_L + 0.3 P_T + P_V \]

These would apply to column, abutment and bearing design in potential near-source locations and at sites where the effective peak accelerations at bedrock are expected to exceed 0.5 g.

Research is also needed on the effects of vertical ground motion on longer, multiple span bridges, curved bridges and complex interchanges. The example for a long-span truss indicate that
forces in critical members and bearings may increase by as much as 20% over those produced by horizontal motions acting alone.

ACKNOWLEDGMENT

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REFERENCES


SEISMIC ANALYSIS OF SKEWED BRIDGES WITH SOIL-FOUNDATION INTERACTION

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ABSTRACT

This paper describes several soil-foundation interaction models by which the equivalent linear spring constants for spread footings, pile footings, abutment backwalls, beams and wings are generated for the seismic analysis of highway bridges. For abutments or pile footings, these equivalent linear spring constants are transformed to a master joint through the Rigid Body Transformation method (RBT). The master joint is usually chosen at the center of gravity of the superstructure for abutments, and the center of gravity of the cap for pile footings to reduce the total number of degrees-of-freedom in the analysis and to take into account the geometric relationships among these spring constants. From the response spectrum analysis, the demand forces of these linear springs are calculated and compared with the corresponding structural components' capacities. The capacities include: abutment backfill passive pressure capacity, pile capacity, and footing bearing capacity. If the demand passive pressure of the backwall is greater than the allowable backfill capacity, then the equivalent linear stiffness of the backwall spring is revised and the next iteration is performed in the analysis. Two numerical examples are provided to show that i) the structure's natural periods and responses are strongly influenced by the abutment soil-foundation interaction model and ii) the response of the abutment is sensitive to the bridge skew.

INTRODUCTION

Current AASHTO Specifications in Division I-A\textsuperscript{12} give specific methods for the structural analysis and design of bridges during earthquake loading. It is less specific with respect to the foundation modeling for the analysis. This is in part due to the complexities which are associated with the different foundation systems encountered in bridge structures in combination with the wide variety of soil types encountered in practice. In assessing the overall dynamic response of highway bridges, it is necessary to account for soil-foundation interaction effects. This paper summarizes several soil-foundation interaction models currently used by the Missouri Department of Transportation (MoDOT) for the seismic analysis and design of highway bridges. Based on these models, the equivalent linear spring constants of the spread footings, pile footings, and components of the abutment can be estimated using the soil boring data. Then the stiffness matrix of individual spread footings, pile footings, and abutments can be formulated using the Rigid Body Transformation (RBT) technique. The purpose of using RBT is to reduce the total number of degrees-of-freedom in the analysis and to take into account the geometric relationships among the spring constants. The response spectrum method associated with the iterative approach is
adopted for the dynamic analysis. The purpose of the iterative approach is to find the effective linear stiffness in order to represent the nonlinear behavior of soil-backwall interaction. The effective stiffness of the backwall is adjusted if the calculated passive pressure of the backwall soil exceeds its passive capacity. This approach is also used in the soil-wing interaction. The capacities of the soil-foundation elements including the abutment backfill passive capacity, pile ultimate capacity, and spread footing bearing capacity, are also described in this paper for the analysis. Two numerical examples are presented here. The first one compares the current approach and the conventional approach which assumes 1) fixed foundations at the intermediate bents; 2) RBT is not considered; 3) only 2 stiffnesses corresponding to the abutment longitudinal and transverse directions are considered for modeling the abutments; and 4) one-half of the abutment backwall stiffness is allocated at each abutment and the resulting abutment backwall forces are doubled for design. The second example demonstrates the bridge skew effects on the bridge responses. The skews considered here are 0, 15, 30, and 45 degrees.

SOIL-Foundation INTERACTION MODELS

Spread Footing Spring Constants
The equivalent linear stiffness matrix for spread footings and caps of pile footings is summarized as

\[ [K] = \alpha \cdot \beta \cdot [K_0] \]  

\[ [K_0] = \begin{bmatrix} K_{11} & K_{22} \\ K_{33} & K_{44} \\ K_{55} & K_{66} \end{bmatrix} \]  

in which \(\alpha\) and \(\beta\) are the foundation shape correction factor and the foundation embedment factor, respectively. Their values depend on the L/B and D/R ratios, respectively. L, B, D, and R are the footing length, width, thickness, and the equivalent radius of a circular footing which varies for different modes of displacement. The basic stiffness coefficients are translational stiffnesses \(K_{11}, K_{22},\) and \(K_{33}\) equal to \(\frac{EGR}{2(1-v)}\), \(\frac{EGR}{2(1-v)}\), and \(\frac{4GR}{1-v}\), respectively, and rotational stiffnesses \(K_{44}, K_{55},\) and \(K_{66}\) equal to \(\frac{aGR^3}{3(1-v)}\), \(\frac{aGR^3}{3(1-v)}\), and \(\frac{16GR^3}{3}\), respectively. Note that the off-diagonal terms in \([K_0]\) are neglected because the values of off-diagonal terms are small, especially for shallow footings.

Pile Axial Spring Constants
The pile axial spring constants are evaluated based on the pile vertical ultimate capacity. The vertical ultimate capacity of piles in cohesive and cohesionless soils is first determined by the microcomputer program SPILE using soil properties of soil layers from the boring data with consideration of groundwater table level. The program follows the methods and equations presented by Nordlund, Meyerhof, and Tomlinson. For Cast-In-Place (CIP) friction pile in
compression, the ultimate pile capacity, \( Q_u \), is equal to the sum of the ultimate pile bearing capacity, \( Q_b \), and the ultimate pile friction capacity, \( Q_f \). Based on \( Q_b \) and \( Q_f \), the pile axial bearing load (b) - axial deformation (z) curve and pile axial friction load (f) - axial deformation (z) curve can be estimated by equations 3 and 4 respectively.

\[
b = Q_b \times \left( \frac{z}{Z_c} \right)^{\frac{3}{2}}
\]

\[
f = Q_f \times \left[ 2 \times \sqrt{\frac{f}{Z_c}} - \frac{z}{Z_c} \right]
\]

where \( b \) = pile tip resistance mobilized at a displacement of \( Z \leq Z'_c \), where \( Z'_c \) = the critical pile displacement at which \( Q_b \) is fully mobilized. Use \( Z'_c = 0.05 \times D \) in which \( D \) = diameter of piles. \( f \) = friction mobilized along a pile at a displacement of \( Z \leq Z_c \), where \( Z_c \) = the critical displacement of the pile segment at which \( Q_f \) is fully mobilized. Use \( Z_c = 0.2 \) inches.

The typical pile axial load-axial deformation curve is shown in Figure 1 in which the pile compliance is added to the rigid pile displacement. Then the total pile axial load-axial deformation relationship is determined. As shown in Figure 1, the pile axial load-axial deformation relationship is a nonlinear curve. Since the response spectrum analysis is a linear analysis method, the secant modulus stiffness of pile is used to represent the equivalent linear stiffness for the analysis. The secant modulus stiffness is defined as the slope between two points at which the axial loads are equal to zero and \( \frac{M_0}{z} \) (see Figure 1).

For steel HP bearing piles, the ultimate pile capacity, \( Q_u \), in compression is equal to the ultimate pile bearing capacity, \( Q_b \). This is because pile bears directly on rock, so the friction capacity is not mobilized and the axial stiffness of steel bearing piles is independent of soil properties and equal to \( \frac{AE}{L} \) in which \( A \) is the cross-section of pile; \( E \) is the elastic modulus of pile; and \( L \) is the length of pile.

**Pile Lateral Spring Constants**

The pile lateral spring constants are determined with the aid of the microcomputer program COM624P\(^7\) using soil properties of soil layers with consideration of groundwater table level. COM624P was developed for use in the analyses of stress and deflection of piles under lateral loads. The theory upon which the program is based, is the widely-used p-y curve method which considers the nonlinearities of soils. The program determines pile deflection, rotation, bending moment, and shear by using iterative procedures in order to account for the nonlinear response of the soil. For a given soil profile, the pile lateral force-lateral deflection curve at the top of pile (pile head) can be obtained by applying incremental lateral forces at the pile head, the program then calculates the corresponding lateral deflections at the pile head. The material nonlinearity of pile is also considered in the program.

Since the response spectrum method is based on a linear analysis, the secant modulus stiffness is used to represent the equivalent linear stiffness. The secant modulus stiffness is defined as the slope between two points at which the lateral loads are equal to zero and \( \frac{P(M_0)}{z} \), where \( M_0 \) is
the ultimate moment capacity of the pile; \( P(M_u) \) represents the lateral load at which \( M_u \) is developed in the pile. The ultimate moment capacity of CIP pile or composite concrete-steel shell pile is based on the limit state of concrete strain \( \varepsilon_c = 0.003 \) and steel shell strain \( \varepsilon_s = 0.015 \). The ultimate moment capacity of steel HP pile is equal to the plastic moment, \( M_p \), of the pile under constant axial load due to superstructure and substructure dead loads.

**Abutment Spring Constants for Backwall, Beam, and Wings**

The abutment translational and rotational spring constants for backwall, beam, and wings are determined using estimated soil properties and the Wilson equations\(^8\). The translational spring constant can be expressed as

\[
K_x = \frac{E_s a}{(1-\nu^2)I}
\]  
(5)

in which \( E_s \) = elastic modulus of soil; \( \nu \) = Poisson's ratio for soil (0.35 may be used for cohesionless soils and 0.45 for cohesive soils); \( a \) = the longer dimension of the rectangular backwall, beam, or wing; and \( I \) is the shape factor for the stiffness. The rotational spring constant of the backwall, beam, or wing can be obtained by using the computed translational spring constant, \( K_x \), as

\[
K_{\theta z} = \frac{K_x B^2}{12}
\]  
(6)

where \( B \) is the length of the backwall, beam, or wing.

**Abutment Rigid Body Transformation (RBT)**

An abutment consists of many translational and rotational springs which represent the interaction between soil and backwall, beam, wings, pile caps, and piles as shown in Figure 2. To reduce the total number of degrees-of-freedom in the analysis and to take into account the geometric relationships among the spring constants, it is attractive to lump all the stiffnesses of these springs from "slave" joints to a "master" joint through a rigid body transformation\(^9,10\). Any two joints on the rigid body (e.g. abutment) are constrained such that the deformation of one joint (the slave joint) can be represented by the deformation of the other joint (the master joint). Thus the degrees-of-freedom for the slave joint are transferred to the master joint, and the number of degrees-of-freedom in an abutment is reduced. Figure 3 depicts the typical joints "j" and "m" for slave and master joints, respectively. For integral abutments, usually the center of gravity of the superstructure is chosen as the "master" joint as shown in Figure 2. Summing the forces acting on the slave joint \( j \) about the master joint \( m \), in three dimensions, yields the force transformation matrix for a 3D rigid body as follows,
\[
\begin{bmatrix}
F_{jmx} \\
F_{jmy} \\
F_{jmz} \\
M_{jmx} \\
M_{jmy} \\
M_{jmz}
\end{bmatrix} =
\begin{bmatrix}
1 & 0 & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 & 0 \\
0 & -Z_{ms} & X_{ms} & 1 & 0 & 0 \\
Z_{ms} & 0 & -X_{ms} & 0 & 1 & 0 \\
-Y_{ms} & X_{ms} & 0 & 0 & 0 & 1
\end{bmatrix}
\begin{bmatrix}
F_{jx} \\
F_{jy} \\
F_{jz} \\
M_{jx} \\
M_{jy} \\
M_{jz}
\end{bmatrix}
\]

or
\[
F_{jm} = T_{ms} F_{js}
\]

where \(F_{jm}\) represents the force vector acting on the master joint "m", and \(F_{js}\) represents the force vector acting on the slave joint "j". A similar transformation for displacements can be derived as
\[
D_{js} = T_{ms}^T D_{jm}
\]

From Eqs. 8 and 9, the stiffness matrix at the slave joint "j" (containing stiffnesses from the springs connected at the slave joint "j"), \(K_j\), can be transformed to the master joint "m" as
\[
K_{jm} = T_{ms}^T K_j T_{ms}
\]

The total master joint stiffness matrix, \(K_m\), is the sum of all the transformed stiffness matrices of the slave joints.
\[
K_m = \sum_{j=1}^{n} K_{jm}; \quad n = \text{total no. of slave joints}
\]

RBT can also be applied to the pile footing by transforming all the pile springs and pile cap springs to an assigned master joint. Usually the master joint is placed at the center of gravity of the pile cap. The RBT takes into account the coupling effects between translational and rotational responses of skewed abutments.

**STRUCTURAL ANALYSIS USING THE ITERATIVE APPROACH**

The force-displacement relationship at bridge abutments is a highly complex nonlinear problem affected by the abutment design. The following iterative technique associated with RBT can be used for the analysis of typical bridge structures by using the equivalent linear stiffness matrix of individual bridge components (i.e., abutments, spread footings, pile footing, etc.). The procedure is outlined in the flowchart appearing in Figure 4 and described in the following steps:

1. Calculate bridge components' (i.e., abutments, spread footings, pile footing, etc.) equivalent linear spring stiffnesses based on soil-foundation interaction models described previously.
2. Perform RBT to obtain stiffness matrix at master joints for abutments, pile footings, and/or spread footings.
3. Analyze the bridge by the response spectrum method and determine the forces at the master joints of abutments, pile footings, and/or spread footings.
4. Based on Eqs. (8) and (9), back calculate the force and displacement of each spring at abutments, pile footings, and/or spread footings.
5. Calculate abutment backfill pressure from the abutment backfill spring force from step 4. If the abutment backfill pressure exceeds the acceptable passive capacity of the abutment fill, reduce the abutment backfill spring stiffness to obtain the effective stiffness of the backfill spring. The effective stiffness of the backwall (spring constant) can be obtained by calculating the slope between the origin and the point which corresponds to the displacement of the backfill spring from Eq. (9). (see Figure 5).
6. Check abutment pile forces against pile capacity. The pile interaction equation (Eq. 12) is used as the pile capacity. If the interaction equation for piles is greater than 1, redesign piles by adding more piles or use high yielding stress piles, etc. Recalculate the pile stiffness.
7. Check pile forces of pile footing. If pile axial force is greater than pile-soil ultimate axial capacity, \( Q_u \), redesign pile footing.
8. Calculate spread footing soil bearing pressure distribution. If maximum soil bearing pressure is greater than the ultimate capacity of the soil, redesign spread footing.
9. If all the forces are less than or equal to the corresponding capacities mentioned in steps 5 through 8. Proceed to step 10. If not, go to step 2.
10. Observe the analyzed displacement at the abutment's master joint and take the appropriate following steps:
   (a) If the displacements exceed acceptable levels, then the abutment design is inadequate. Redesign the abutment and return to step 1.
   (b) If the displacements are acceptable, then the last abutment stiffness matrix is consistent with the abutment design.

CAPACITY CRITERIA USED IN THE ANALYSIS

Several capacity criteria used in the analysis are described here.

**Abutment Backfill Capacity**

Recently, abutment backfill passive capacity was studied using large-scale abutment tests at the University of California-Davis\(^\text{11}\). The force-deformation relationship from the test indicated that the maximum soil passive pressure is about 6 kip/ft\(^2\). Caltrans utilizes an abutment capacity based on a soil passive pressure of 5 kip/ft\(^2\), amplified by about 50% to 7.7 kip/ft\(^2\) for earthquake loads. In this study, the abutment backfill passive capacity of 7.7 kip/ft\(^2\) is adopted.
Pile Ultimate Capacities

The ultimate capacity of the pile itself (not the pile-soil ultimate capacity, \( Q_u \)) is also used to check the pile forces. Since steel piles and CIP piles at abutments or pile footings are below the ground level, it is desirable to ensure that piles do not fail. Therefore a response modification factor of \( R = 1 \) is considered in this study although piles have good ductility capacity. The ultimate capacity of CIP pile or composite concrete-steel shell pile at a demand axial load is based on the limit state of \( \varepsilon_c = 0.003 \) and \( \varepsilon_s = 0.015 \). The ultimate capacity of steel H pile is based on the following interaction formula:

\[
\frac{P}{0.85A_{Fy}} + \frac{M_x}{M_{mu}} + \frac{M_y}{M_{my}} = 1.0 \quad \text{from AASHTO}^{12} 10.54.2
\]  

(12)

Spread Footing Soil or Rock Bearing Capacity

The ultimate soil or rock bearing capacities are based on AASHTO Specifications Division I, Section 4. The demand bearing pressures of the footing are calculated based on the finite element method. The procedure for this method is briefly described below.

A typical footing is sketched in Figure 6 in which the footing is divided into many finite elements. Let \( y_0 \) and \( z_0 \) represent the initial principal axes of the footing. The soil is assumed to have no tensile capacity. Therefore the elastic modulus of soil is assumed to be zero for tension.

When loads obtained from the response spectrum analysis are applied to the footing, some elements may separate from the soil, thus the instantaneous centroid location needs to be determined according to the compressive area of the footing (see Figure 6). The instantaneous centroid location \( C(y'_{eo}, z'_{eo}) \) and rotation angle, \( \beta \), are shown in Figure 6. The rotation angle \( \beta \) is the angle between reference axis \( y' \) which is parallel to the initial principal axis \( y_0 \) and instantaneous principal axis \( y \).

The sectional properties corresponding to the instantaneous principal \( y \) and \( z \) axes are calculated. \( EI_y, EI_z, \) and \( EA \) represent flexural rigidities in the \( y \) and \( z \) directions, and axial rigidity in the \( X \) (vertical) direction, respectively. The bearing pressure of each element shown in Figure 6 is computed by an incremental loading procedure. At each incremental step, the incremental loads including axial load, \( \Delta P \), moment in \( y \) direction, \( \Delta M_y \), and moment in \( z \) direction, \( \Delta M_z \), are applied to the footing and the footing sectional properties of \( EI_y, EI_z, \) and \( EA \), and the maximum bearing pressure of footing at that step are calculated. A microcomputer program "SPREAD"\(^{9,10,13}\) has been developed for calculating soil bearing pressures of spread footings under axial load and biaxial bending moments.

NUMERICAL EXAMPLES

Example 1

An existing bridge as shown in Figure 7 was used to study the seismic response with respect to different soil-foundation modeling techniques. This bridge is a five-span prestressed concrete I-girder structure with integral pile cap abutments and concrete round column intermediate bents on pile footings. The total span length is 324'-6" with a skew of 45 degrees. The soil conditions at
the site in general consist of submerged gray fine sand with some clay, loose to medium dense. Based on the original seismic design procedures, four wings were used at each abutment. All piles are cast-in-place with steel shells. This bridge is classified as an essential bridge with an importance classification coefficient (IC)=1, and an acceleration coefficient of 0.36g. Therefore the seismic performance category (SPC)=D is considered in the analysis and design. The original design of this bridge was based on the following conventional design criteria: 1) fixed foundations for all intermediate bents; 2) RBT is not considered; 3) only 2 stiffnesses corresponding to the abutment long. and trans. directions (Figure 2) are considered for the abutments; and 4) one-half of the abutment backwall stiffness is allocated at each abutment and the resulting abutment backwall forces are doubled for the design. Table 1 compares the abutment spring constants between the current MoDOT approach with the conventional approach.

<table>
<thead>
<tr>
<th>Current Approach by MoDOT</th>
<th>Conventional Approach</th>
</tr>
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<tbody>
<tr>
<td>Use &quot;SPILE&quot; to determine pile axial spring constants</td>
<td>Use &quot;SPILE&quot; to determine pile axial spring constants</td>
</tr>
<tr>
<td>Use &quot;COM624P&quot; to determine pile lateral spring constants</td>
<td>Use &quot;COM624P&quot; to determine pile lateral spring constants</td>
</tr>
<tr>
<td>Use Wilson's equations to obtain backwall-soil and wing-soil stiffnesses</td>
<td>Use Wilson's equations to obtain backwall-soil and wing-soil stiffnesses</td>
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<tr>
<td>Consider spring constants (Figure 2) from:</td>
<td>Consider spring constants (Figure 2) from:</td>
</tr>
<tr>
<td>• backwall and beam cap (Fx, Fz, Mx, Mz)</td>
<td>• backwall and beam cap (Fx)</td>
</tr>
<tr>
<td>• beam piles (Fx, Fy, Fz, My, Mz)</td>
<td>• beam piles (Fy, Fz)</td>
</tr>
<tr>
<td>• wings (Fx, Mz)</td>
<td>• wings (Fx)</td>
</tr>
<tr>
<td>• wing piles (Fx, Fy, Fz, My, Mz)</td>
<td>• wing piles (Fz)</td>
</tr>
<tr>
<td>• wing pile caps (Fx, Fy, Fz, Mx, My, Mz)</td>
<td>• wing pile caps (not considered)</td>
</tr>
<tr>
<td>Use rigid body transformation (RBT) to formulate full stiffness matrix (6 x 6) at master joint</td>
<td>Sum individual element stiffnesses to obtain only 2 stiffnesses corresponding to abutment long. and trans. directions</td>
</tr>
</tbody>
</table>

Four load cases are considered in an analysis, Case 1: apply seismic force in the bridge longitudinal direction; Case 2: apply seismic force in the bridge transverse direction; Case 3: 100% of Case 1 + 30% of Case 2; and Case 4: 100% of Case 2 + 30% of Case 1. Five conditions are studied here. They are

Condition 1: assume backfill of abutment #6 is subjected to passive pressure (e.g. bridge moves away from abutment #1). Backwall stiffness at abutment #6 is considered, but not at abutment #1. Flexible foundations are used with consideration of pile axial and translational springs for intermediate bents. Pile flexural spring constants are not considered at abutments or intermediate bents.

Condition 2: assume backfill of abutment #1 is subjected to passive pressure (e.g. bridge moves away from abutment #6). Backwall stiffness at abutment #1 is considered, but not at abutment #6. Flexible foundations are used with consideration of pile axial and...
translational springs for intermediate bents. Pile flexural spring constants are not considered at abutments or intermediate bents.

Condition 3: same as condition 1 but assume fixed foundations for intermediate bents.
Condition 4: same as condition 1 but with consideration of pile flexural spring constants at abutments only. Based on the soil boring data, the flexural stiffness of pile is calculated by the design charts for submerged sand.
Condition 5: use conventional approach (e.g. one-half of the abutment backwall stiffness is allocated at each abutment; fixed foundations for intermediate bents; no RBT).

The natural period of this bridge for conditions 1 - 4 is about 0.37 seconds and the natural period based on condition 5 is 0.26 seconds. The difference is due to a very stiff abutment model in the conventional approach since only two translational stiffness coefficients corresponding to each abutment's longitudinal and transverse directions are considered and the other degrees-of-freedom are completely restrained (Table 1). Figures 8 and 9 show the abutments' forces parallel and perpendicular to the abutment beams and moments at the bottom of a typical column at the individual bents for conditions 1, 2, and 5 under seismic load cases 1 and 2, respectively. In general, the conventional approach overestimates the abutment forces and underestimate the column forces and pile footing forces at intermediate bents. Comparing the envelope of conditions 1 and 2 with condition 5 in Figure 8 for load case 1 shows that the conventional approach is up to 45% higher than the abutment transverse force at the center gravity of the superstructure for the envelope of conditions 1 and 2. However, the abutment longitudinal forces have similar magnitudes. For load case 2 in Figure 9, the conventional approach is about 20% higher than the abutment transverse forces for the envelope of conditions 1 and 2. Figures 8 and 9 also indicate that the conventional approach's column moments are about 55% smaller than the envelope of conditions 1 and 2 for seismic load cases 1 and 2. This study indicates that the different modeling approaches used for conditions 1 and 2 versus those used for condition 5 yield a significant difference in the analysis results. The current modeling approach used by MoDOT in conditions 1 and 2 is believed to give more realistic results because the modeling techniques better reflect the bridge characteristics.

Figures 10 and 11 show the abutment forces parallel and perpendicular to the abutment beams and the moments at the bottom of a typical column at the individual bents for conditions 1, 3, and 4 under seismic load cases 1 and 2, respectively. Comparing conditions 1 and 3, shows that the column end moments of intermediate bents with fixed foundation are about 17% higher than those with flexible pile footings for load cases 1 and 2. Comparing conditions 1 and 4, it is noteworthy that the contribution of pile bending rigidities to the abutment stiffnesses is insignificant and the structural responses from both conditions are very close.

Example 2
The sensitivity of structural responses of the bridge shown in Figure 7 due to skew effects was studied by using the soil-foundation interaction model of condition 1. The original bridge skew of 45 degrees was changed to 0, 15, and 30 degrees for all bents. Figure 12 compares the abutment 6 total transverse shear forces with the backwall passive forces for load cases 1 and 2. It shows that the abutment transverse force decreases when bridge skew increases for case 1 and it increases when bridge skew increases for case 2. It also shows that the majority of the abutment
transverse force is resisted by the backwall (dashed lines in Figure 12). The backwall force is 
greater than the abutment total transverse force when the skew is greater than 25 degrees. This is 
due to the combination of translational and torsional effects on the abutment caused by the 
passive force components of the wings which act opposite to the direction of the backwall passive 
force. Therefore, the torsional and translational effects increase as bridge skew increases. Figure 
13 shows the abutment backwall passive forces due to load cases 3 and 4. As the skew increases, 
the 3 and the 4 approach each other. It also shows that the maximum backwall passive force 
occurs in the region of 0 to 15 degrees. The abutment wing passive forces due to skew effects are 
shown in Figure 14 for two exterior wings. When the skew = 0 degree, both wings resist equal 
amount of forces. When the skew increases, the individual wings can resist significantly different 
forces. This phenomena is also due to the torsional effect of the abutment. Figure 14 also shows 
that the wing force for load case 1 can exceed that for load case 2 depending on the skew and 
wing. Figure 15 shows the total wing forces (i.e. summation of 2 exterior wing forces and two 
interior wing forces) due to load cases 3 and 4 at abutments 1 and 6. It seems that the wing force 
increases as the skew increases. The total wing force resisted by the wings at abutment 6 which 
moves toward the backfill is greater than that for abutment 1 which moves away from the backfill.

SUMMARY

Soil-foundation stiffness formulation is an important factor in seismic analysis and design. More 
accurate soil-foundation stiffnesses will result in more reliable bridge natural periods and 
responses. In order to simplify the analysis, equivalent linear spring constants are considered in 
the response spectrum analysis associated with the iterative approach for estimating abutment 
backwall and wing effective stiffness. To estimate abutment stiffnesses at the center of gravity of 
the superstructure or to formulate foundation stiffnesses at the center of gravity of the pile cap, 
the rigid body transformation approach is recommended to reduce the total number of 
degrees-of-freedom and to take into account the geometric relationships among the spring 
constants. Using conventional one-half of the abutment backwall stiffness approach for bridges 
with large skew and without RBT can result in significant error in the analysis. Using this 
conventional approach for modeling the abutment can significantly underestimate the intermediate 
bents' forces and can not adequately predict the abutment forces because of the restrained 
conditions assumed at abutments. The response of the abutment is also sensitive to the bridge 
skew.

This paper also demonstrates that i) the column end moments of intermediate bents with fixed 
foundations are about 17% more than those with flexible pile footings, ii) the contribution of pile 
bending rigidities to the abutment stiffnesses is insignificant, iii) the total force resisted by the 
wings increases as skew increases; iv) the maximum backwall force occurs when the skew is less 
than 15 degrees; v) the combination of torsional and translational effects on the abutment 
becomes important when skew increases; and vi) the backwall force can be greater than the 
abutment total transverse force when the skew is greater than 25 degrees. Since this study is 
based on only one bridge, more parametric studies are needed to verify the trends and conclusions 
drawn in this paper.
REFERENCES

Figure 1 - Pile axial stiffness model

Figure 2 - Abutment spring constants considered.
Figure 3 - Master joint and slave joint relationship

Figure 4 - Flowchart for structural analysis
Figure 5 - Abutment effective stiffness obtained from iteration

Figure 6 - Instantaneous centroid location, $C'$, and instantaneous principal axes $(y,z)$

Figure 7 - Example: five-span prestressed concrete I-girder bridge
Figure 8 - Comparison of conditions 1, 2, and 5 for load case 1

Figure 9 - Comparison of conditions 1, 2, and 5 for load case 2
Figure 10 - Comparison of conditions 1, 3, and 4 for load case 1

Figure 11 - Comparison of conditions 1, 3, and 4 for load case 2
Figure 12 - Abutment 6 transverse forces and backwall forces for condition 1

Figure 13 - Abutment 6 backwall passive forces for cases 3 and 4 for condition 1
Figure 14 - Abutment 6 wing forces for condition 1

Figure 15 - Total wing forces at abutments 1 and 6 for condition 1
SESSION 8
LESSONS LEARNED FROM OTHERS
"LESSONS LEARNED FROM KOBE AND OTHER EARTHQUAKES"

by

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ABSTRACT

This summary paper presents a list of lessons which have been learned and relearned from major earthquakes dating back to the 1964 Good Friday Alaskan Earthquake. The author has been directly or indirectly involved with the investigations of the 1964 Alaska, 1971 San Fernando, 1976 Guatemala, 1978 Sendai, 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe earthquakes. This experience, tempered by the results of subsequent research, has helped the author formulate and advance a philosophy for the design and construction of new bridges and for the retrofitting of existing structures.

INTRODUCTION

In the United States, it's not just California and Alaska that are at risk. Thirty-nine states are seismically active. However, lack of public awareness of the earthquake potential outside of California for many years caused the inattention to implementing improved design criteria nationally. Also overlooked was the need for retrofitting important bridges with the technologies that have been derived from research stimulated by post earthquake reconnaissance investigations.

Other countries around the world are seismically active, but they too have areas of lower seismicity just as there is in the United States. This is important, as some 39 states are considered seismically active and have the potential of generating significant damage, although none has been experienced in the past several decades. Through the lack of awareness for potential havoc or because of complacency, the infrastructure has been constructed without regard for the hazard exposure or risk at hand. One thinks of California or Alaska as earthquake country, however, there is an active New Madrid fault zone in Missouri, Kentucky, Illinois, and Tennessee that has significant potential for widespread damage and disruption. In addition, The St. Lawrence Seaway; St. Lawrence Valley; upstate New York; Charleston, S.C.- all have the capability of generating very significant earthquakes. It's just a question of when, where and how strong.

Part of the difficulty in the United States is in creating an awareness of the seismic vulnerabilities in those areas outside of California. Californians know they have a problem. Other states are beginning to appreciate the fact that they could also have a potential problem. Although they recognize that they should prepare for possible seismic events, questions remain: What level of
earthquakes to design for? How to select bridges for retrofit? How far to retrofit? What is the actual hazard exposure and risk? Funds to attack the problem are limited, especially when congestion management and maintaining the highway system is paramount in the owner’s mind. Risks and costs must be weighed.

The formula is to identify the hazard, analyze the vulnerability of the network, and selected priority structures to that hazard, and then to fix those structures. Cost must be balanced against risk. Easy to say, but almost impossible to do perfectly. Identification of structures in most need of fixing (retrofitting) is difficult because various sectors of the population have differing priorities. A bridge or portion of the network that is important to one may be of little value to another.

For example, in evaluating which bridges should be retrofitted and to what extent, decision makers representing the owners must evaluate the importance of the bridge to the community as a whole. Where does it go? What are the intermodal links that it affects? Are there redundant routes? What is the level of inconvenience to the community if the bridge becomes unusable? When these questions, and others are addressed, it is time for action.

Since it is impossible to design or retrofit a bridge or structure to be “earthquake proof” - to be totally safe near the epicenter of a large quake - a philosophy is promoted to design for “life safety,” a philosophy that can accept damage and closures, but avoids the structural collapse that causes loss of life. Serviceability, although important, is always secondary to life safety.

Another decision - do nothing - may be just as valid, given existing constraints imposed on the owner. At least a conscious, defendable decision will have been made.

LESSONS FOR U.S. PLANNERS AND DESIGNERS

The lessons of the 1971 San Fernando earthquake led to the development of seismic retrofitting - that is, going back and evaluating structures vulnerable to earthquakes and determining how to strengthen them in a cost effective manner. Some fixes that have been developed are simple. Some are complex. In the San Fernando quake, those bridge spans that were simply supported or that had discontinuities simply fell off during the shaking. To counter this, restrainers or couplers were developed to basically tie the spans together. However, it was learned in subsequent earthquakes that when restrainers forced the bridge to act as a unit, the destructive force was transmitted to another part of the bridge. The lesson: If one prevents failure in one place, it may occur in another -- the weakest link in the chain theory.

Highway engineers must be able to trace demand paths which will result from displacements and forces and then devise appropriate capacity links, the designs or fixes that properly distribute those displacements and forces. The first obvious fix was the use of joint restrainers or couplers. They are very cheap. But in the next big earthquake that came along, it was observed that failures were forced somewhere else. This lead to the design and retrofit of bents and columns. Their retrofit is more expensive. Next came foundations, footings and pilings, which are even more expensive. Somewhere down the line, it is important to balance the significance of the
structure, it's vulnerability, the nature of the hazards, the severity of the expected shaking, and the costs of seismic protection. The goal is to come up with designs and fixes that balance all these factors. This is easy to state, but extremely difficult to do realistically.

Many agencies have been trying to achieve this since 1971. Caltrans is the absolute leader in the field of retrofit development. They are well aware of what their risks and vulnerabilities are, and they are taking a very aggressive approach to investigate the effects of strong ground shaking on bridges. It is gratifying that many other states are taking much more active roles in identifying the hazards, evaluating the risks, and taking positive action to reduce the vulnerabilities within their inventory.

Following is a list of ten specific observations which have resulted from post-earthquake investigations:

(1) Large earthquakes can be very destructive in terms of lives lost, injuries sustained, business losses, construction costs and social disruption. The closure of arterial highways and bridges affects emergency relief and business recovery and can have a major economic impact on a region and its ability to survive such a disaster.

(2) Large damaging earthquakes can occur in areas considered to have, on average, only moderate exposure to seismic hazards.

(3) Capacity design procedures, ductile details and generous seat widths are necessary to prevent catastrophic collapse during large earthquakes.

(4) Minimum connective forces need to be enforced for all seismic zones unless such connections can be shown to be fully protected by acceptable yielding of the substructures. Redundancy in connection detailing is particularly important for essential structures. Alternative load paths are necessary if the primary load path fails due to unforeseen circumstances.

(5) Critically important structures must be designed to a higher level of performance than that provided by current specifications if full service is to be maintained after a large earthquake. Multi-level performance criteria and corresponding design strategies are necessary for important bridges.

(6) Retrofit measures reduce damage, but inappropriate use and/or installation can defeat their purpose and perhaps trigger a collapse. This applies particularly to couplers (restrainers).

(7) Lateral spreading due to liquefaction can lead to span collapse even in modern structures with massive foundations (caissons) and well-engineered fills.
(8) Premature failure of some bearings appear to reduce the seismic loads in their supporting substructures by uncoupling the superstructure from its supports. This fuse-like action may have saved a number of spans from collapse and columns from failure in shear and flexure.

(9) Accelerations in (base) isolated superstructures are less than in conventional structures.

(10) Skewed bridges are susceptible to in-plane rotation, leading to large displacements at their supports and possible unseating of girders at the acute corners.

WHAT THE KOBE “SURPRISE” SHOULD TEACH THE U.S.

Although Japan is a very seismically active country, there are areas of lower seismicity. The Japanese had previously identified the Kobe-Osaka region as an area of comparatively low seismic vulnerability and of low seismic intensity should an earthquake occur. Therefore, bridge and building designs were not held to the same high standards as more earthquake-prone areas such as Tokyo. Furthermore, many of the structures were built in the 1960s before the development of new design criteria derived after the 1971 San Fernando earthquake.

To bridge engineers and owners of bridges in the central and eastern United States, the Kobe earthquake is perhaps of even greater significance than recent earthquakes in California, such as Loma Prieta in 1989 and Northridge in 1994. One reason for this is that in the Japanese case, the possibility of an earthquake larger than the design earthquake was considered unlikely. Only nominal attention had been given to the problem, and then only for structures designed since 1990. This difference between the maximum credible earthquake and the design earthquake is clearly very large for this region of Japan - a situation that exists in the United States to a greater degree in the eastern and central states than in the West.

A second reason that this quake is of greater interest to the central and eastern United States is that the predominant type of bridge in Japan is the steel girder superstructure (simple and/or continuous spans) supported by bearings on concrete columns and foundations. This class of Japanese bridge, largely designed in the 1960's, is found throughout the central and eastern United States, whereas bridges in California tend to be concrete box girders with monolithic bents and integral abutments, especially in shorter bridges. This is why the performance of Japanese bridges is of particular relevance to states east of the Sierra Nevada Mountains and the performance of these non-ductile structures during the Kobe earthquake can and has provided insight into how many older U.S. bridges will respond when another large earthquake occurs.

RETROFIT NOW, DESIGN FOR THE FUTURE

The most cost-effective, long-term program for mitigating earthquake damage to highway bridges is to ensure that new construction is designed even beyond existing specifications in accordance with the latest technology derived from studies of real-world disasters. This is more cost-effective than retrofitting, which can be very expensive. Before 1971, earthquake design criteria were very simple, basic, and fundamental. The usual extent of design guideline was
"consider earthquake forces." Since 1971, extensive research has been done here and abroad, real-world aftermaths of earthquakes at home and abroad have been investigated, and specifications have been updated.

During the ten year period from 1971 to 1981, coupler technology advanced as a result of the San Fernando experience. The importance of subsequent earthquakes was that the community was able to see how this older technology performed, evaluating its successes and failures. A better understanding of structural demands (forces and displacement) had been gained so that structures designed after 1982 performed very well in more recent large earthquakes.

The community has observed how actual structures and new technologies have improved seismic performance when structures were subjected to real-world earthquakes in 1989, 1994, and 1995. There were failures, but many new design/retrofit details have worked well. While there have been shortcomings, much better performance has resulted. As an example, there are a large number of short, stiff bridges in the United States. It wasn't until the late 1970s that designers started designing for ductility (that is, flexible, "forgiving" designs) following the San Fernando experience. The value of ductile design was proven in these later events.

The difficulty now is in trying to decide how much seismic protection we can provide through new design and, more critically, how much retrofitting should be done. There are some 590,000 highway bridges in the United States. They cannot all be retrofitted, certainly not to a uniform level of performance. This implies that there is a need to identify different design levels, based on desired performance. Not every bridge can or needs to be designed to California standards.

Considerable progress has been made since 1971. Better technologies exist today, and the lessons learned in Northridge in 1994 and Kobe in 1995 have boosted our confidence. AASHTO is aggressively working to develop improved technologies and better design methods for both new construction and retrofitting.

RECOMMENDATIONS FOR NEW DESIGN

The following recommendations for the seismic resistant design of new or replacement highway bridges are based on past experience and research results to date.

- Use approach slabs with positive ties to the abutment. This can provide continuity and minimize the effect of soil slumping behind the abutment.

- Use continuous spans rather than simply supported spans; this will reduce the need for expansion joints and thus minimize the potential for span separation. A side benefit of this practice will be a reduction in joint maintenance costs.

- Provide adequate seat widths for simply supported spans at piers, in-span joints, and abutments to prevent girders from becoming unseated.
• Design all bearings for simply supported spans for lateral seismic loads - that is, provide adequate strength in restrained directions. Check the stability of the bearing in the unrestrained direction at its maximum anticipated displacement.

• Provide adequate confinement for bridge columns by using either spiral reinforcement or transverse ties.

• Do not lap or anchor column longitudinal steel in the plastic hinge zones at the column-to-cap connection and/or the column-to-footing connection.

• Design footings to resist the full moment and shear demands transmitted from the column. Do not allow plastic hinging in the footings.

• Use earthquake-protective systems (e.g., seismic isolation) where appropriate, to minimize the seismic demand on bridge members.

• Use soil improvement technologies to reduce the potential for soil failure or liquefaction.

Recommendations for Existing Construction

The following are recommendations for retrofitting existing bridges. Note that some of the recommendations for new designs also can be applied to existing structures (e.g., using soil improvement technologies).

• Identify and rank those bridges in need of retrofitting based on structural vulnerabilities and socioeconomic considerations using one of the many screening and prioritization schemes available.

• Either extend the seat width or add cable couplers restrainers across the joint if seat widths are inadequate in order to prevent spans from becoming unseated at piers and in-span hinges. Check for adverse effects in columns and foundations.

• Consider bearing replacements if bearing failures could result in collapse or loss of function of the superstructure. Older steel rocker bearings with inadequate anchor bolts are known to be particularly vulnerable and should be replaced or strengthened.

• Eliminate expansion joints. This not only improves seismic performance, but also reduces maintenance costs. As an alternative to extending seat widths or adding cable restrainers, a number of simple spans could be made continuous by structural modifications (e.g., by casting a continuous deck slab with or without web connectors on the girders).
"consider earthquake forces." Since 1971, extensive research has been done here and abroad, real-world aftermaths of earthquakes at home and abroad have been investigated, and specifications have been updated.

During the ten year period from 1971 to 1981, coupler technology advanced as a result of the San Fernando experience. The importance of subsequent earthquakes was that the community was able to see how this older technology performed, evaluating its successes and failures. A better understanding of structural demands (forces and displacement) had been gained so that structures designed after 1982 performed very well in more recent large earthquakes.

The community has observed how actual structures and new technologies have improved seismic performance when structures were subjected to real-world earthquakes in 1989, 1994, and 1995. There were failures, but many new design/retrofit details have worked well. While there have been shortcomings, much better performance has resulted. As an example, there are a large number of short, stiff bridges in the United States. It wasn't until the late 1970s that designers started designing for ductility (that is, flexible, "forgiving" designs) following the San Fernando experience. The value of ductile design was proven in these later events.

The difficulty now is in trying to decide how much seismic protection we can provide through new design and, more critically, how much retrofitting should be done. There are some 590,000 highway bridges in the United States. They cannot all be retrofitted, certainly not to a uniform level of performance. This implies that there is a need to identify different design levels, based on desired performance. Not every bridge can or needs to be designed to California standards.

Considerable progress has been made since 1971. Better technologies exist today, and the lessons learned in Northridge in 1994 and Kobe in 1995 have boosted our confidence. AASHTO is aggressively working to develop improved technologies and better design methods for both new construction and retrofitting.

RECOMMENDATIONS FOR NEW DESIGN

The following recommendations for the seismic resistant design of new or replacement highway bridges are based on past experience and research results to date.

- Use approach slabs with positive ties to the abutment. This can provide continuity and minimize the effect of soil slumping behind the abutment.

- Use continuous spans rather than simply supported spans; this will reduce the need for expansion joints and thus minimize the potential for span separation. A side benefit of this practice will be a reduction in joint maintenance costs.

- Provide adequate seat widths for simply supported spans at piers, in-span joints, and abutments to prevent girders from becoming unseated.
・ Design all bearings for simply supported spans for lateral seismic loads - that is, provide adequate strength in restrained directions. Check the stability of the bearing in the unrestrained direction at its maximum anticipated displacement.

・ Provide adequate confinement for bridge columns by using either spiral reinforcement or transverse ties.

・ Do not lap or anchor column longitudinal steel in the plastic hinge zones at the column-to-cap connection and/or the column-to-footing connection.

・ Design footings to resist the full moment and shear demands transmitted from the column. Do not allow plastic hinging in the footings.

・ Use earthquake-protective systems (e.g., seismic isolation) where appropriate, to minimize the seismic demand on bridge members.

・ Use soil improvement technologies to reduce the potential for soil failure or liquefaction.

**Recommendations for Existing Construction**

The following are recommendations for retrofitting existing bridges. Note that some of the recommendations for new designs also can be applied to existing structures (e.g., using soil improvement technologies).

・ Identify and rank those bridges in need of retrofitting based on structural vulnerabilities and socioeconomic considerations using one of the many screening and prioritization schemes available.

・ Either extend the seat width or add cable couplers restrainers across the joint if seat widths are inadequate in order to prevent spans from becoming unseated at piers and in-span hinges. Check for adverse effects in columns and foundations.

・ Consider bearing replacements if bearing failures could result in collapse or loss of function of the superstructure. Older steel rocker bearings with inadequate anchor bolts are known to be particularly vulnerable and should be replaced or strengthened.

・ Eliminate expansion joints. This not only improves seismic performance, but also reduces maintenance costs. As an alternative to extending seat widths or adding cable restrainers, a number of simple spans could be made continuous by structural modifications (e.g., by casting a continuous deck slab with or without web connectors on the girders).
• Provide column jacketing for existing bridges in high seismic zones if there is inadequate confinement in the column (i.e., insufficient transverse hoops or ties) and/or there are splices or laps in hinge zones. In low-to-moderate seismic zones, consider column jacketing if these deficiencies exist and the bridge is judged to be important or essential. Check for adverse effects on other components.

• Provide footing overlays or extensions in high seismic zones if the column-footing connection is intended to be moment-resistant and there is a lack of top reinforcing steel, inadequate shear steel, or insufficient bearing capacity. In low-to-moderate seismic zones, consider footing retrofits if deficiencies exist and the bridge is judged to be important or essential.

• Strengthen cap beams to provide increased resistance to transverse flexure and shear through external concrete jacketing and/or prestressing. Joints between columns and cap beams can also be strengthened by external jacketing.

• Use seismic isolation technologies to retrofit bridges with short, stiff columns. Various isolation technologies exist and have demonstrated good seismic performance.

CONCLUSION: LESSONS LEARNED FROM PAST EARTHQUAKES HAVE PAID OFF, CONTINUED EFFORT IS NEEDED

Investigation of bridge performance from previous earthquakes has shown that we are indeed on the right track with regard to the development of effective seismic-resistant highway bridge design and retrofit procedures and technology. New designs hold up; retrofitting works.

Some people believe that structures can or should be made earthquake proof. Unfortunately, earthquake design and retrofit are still more of an art than a science. At this time, research and engineering have provided the tools to improve the seismic performance of bridges and minimize the likelihood of structural collapse. As we better understand and apply new procedures and practice emanating from lessons learned from past earthquakes, damage and failure will be reduced further.

ACKNOWLEDGMENTS

The author is grateful to Ian G. Buckle, former Deputy Director of the Multidisciplinary Center for Earthquake Engineering Research (MCEER); Ian M. Friedland, Assistant Director for Bridges and Highways, MCEER; Phillip Yen and John O’Fallon, Structural Research Engineers, Federal Highway Administration (FHWA); Toshio Iwasaki, President, Civil Engineering Research Laboratory, Tokyo; Roland Nimis, FHWA Western Resource Center; and Nancy Bobb, FHWA California Division for their contributions to numerous publications which formed the basis for this paper.
CALIFORNIA'S CURRENT BRIDGE SEISMIC RESEARCH BASED MITIGATION

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ABSTRACT

Nearly ten years have passed since the disastrous Loma Prieta earthquake and over nine years have passed since the Governor's Board if Inquiry into the cause of highway structure failures during that earthquake issued its final report with the warning title Competing Against Time. It is the purpose of this paper to discuss the Seismic Design Specifications and Construction Details that have been developed in California as lessons were learned from the 1971 San Fernando earthquake and subsequent seismic events; and to discuss the unprecedented research program that has provided the bridge design community the assurance that the new specifications and details perform reliably. The California Department of Transportation (Caltrans) staff engineers, consulting firms, independent Peer Review Teams, and university researchers have cooperated in this program of Bridge Seismic Design and Retrofit Strengthening to meet the challenge presented in the Board of Inquiry report. The nine-year old Seismic Advisory Board has been an invaluable asset in reviewing the performance criteria, design specifications, and design procedures for both new design and retrofit strengthening of older, non-ductile bridges. In many instances, the Advisory Board positively influenced management decisions to continue financial support of a strong research program to support seismic design and retrofit through its recommendations to the Director of Transportation.

The success of the Bridge Seismic Design and Retrofit Program and the success of future seismic design for California bridges is based, to a large degree, on the accelerated and "problem-focused" seismic research program. That program has been supported at a level more than ten times the pre Loma Prieta level of financial support for all bridge research. The Department has been able to sustain the necessary high level of research support over the past ten years and has adopted a commitment for that level of funding for the foreseeable future. To date, over $40 million U.S. have been expended in this research and proof testing program.

Until recent years, most other states in the United States have not been concerned with seismic design for bridges. However, some 37 states in the U.S. have some level of seismic hazard. The American Association of State Highway and Transportation Officials (AASHTO) is the agency responsible for development of bridge design specifications for nationwide use. AASHTO has typically adopted seismic design criteria modeled after those developed in California. Understandably, there are hundreds of bridges in these other states which have been designed to seismic criteria that are not adequate for seismic forces and displacements that we know today. The seismic retrofit details designed by the California bridge engineers can be of great benefit to
those states who are faced with seismic threats of lesser magnitude, with little financial support for seismic retrofitting, and much less for research and seismic detail development.

The greatest number of large scale tests have been conducted to confirm the calculated ductile performance of older, non-ductile bridge columns that have been strengthened by application of structural steel plate, prestressed strand, epoxy-fiberglass, and carbon fiber composite jackets to provide the confinement necessary to insure ductile performance. Since the spring of 1987 the researchers at UC San Diego have completed more than 100 sets of tests on bridge column models.

The two major considerations in seismic design of foundations are ground motion and foundation and substructure interaction. Caltrans adopted a site specific seismic design philosophy shortly after the 1971 San Fernando earthquake. For the average smaller freeway structures we use the Maximum Credible Earthquake (MCE) for determining the seismic design forces. For major structures we use a site specific probabilistic hazard analysis to determine the most probable design earthquake spectra.

Procedures have been developed to provide for large ground movements in the deck joints and hinges. These details are designed to prevent superstructure elements from collapsing, even though joints may open as much as three feet.

Much research has gone into the development of joint shear, torsion, and moment reinforcement for the large joints common in bridge superstructures.
OREGON'S SEISMIC WAKE UP

Mark Hirota

ABSTRACT

In the past decade the Oregon Department of Transportation seismic design for bridges has undergone extensive growing pains in order to bring our designs up to current standards.

The turning point for us was the Loma Prieta quake in 1989. The high visibility of the event sent a media shockwave through Oregon, with the question being asked, "Could that happen here?" Not only was the answer "YES," but soil samples indicated that there have been periodic events of major proportion in Oregon’s past.

We started a systematic approach to upgrading our system for seismic risk. This approach included a design specification change, development of an Oregon specific bedrock acceleration map, training for designers, lifeline route analysis, and a seismic retrofit prioritization.

We still have a long way to go in order to feel comfortable with the condition of our bridge inventory but have implemented a strategy to get there.
1. INTRODUCTION

Purpose and Scope

The purpose of this paper is to present WASDOT guidelines and procedures for the post-earthquake safety evaluation of bridges. Development of these procedures is intended to promote uniformity in the rating of bridge damage such that two individuals examining the same bridge will arrive at essentially the same conclusion regarding a structure's safety and its posting category.

The scope of this paper deals primarily with the technical aspects of making bridge structural safety evaluations and does not deal with organizing and managing inspection work. Local authorities must provide management and administration of inspections. The three levels of inspection used by WASDOT are discussed and example assessment forms are included. Resources needed and a list of tools recommended for inspection work is given.

Post-earthquake inspection procedures to be followed for inspecting any concrete, steel, or timber bridge, and the specific procedures to follow for inspecting a given bridge element are summarized and illustrated by examples of real earthquake damage.
2. OVERVIEW OF BRIDGE SAFETY EVALUATION PROCEDURE

The safety evaluation procedure for bridges is outlined in the Washington State Disaster Plan, Emergency Response Guide (ERG). An overview of the emergency response procedures is given in the ERG and is reproduced in Fig. 2.1 for reference. Three levels of postearthquake inspection are described in the ERG. Description of the three levels of inspection is shown in Fig. 2.2, which is also reproduced from the ERG.

Level I Inspection

The Level I inspection consists of a rapid visual survey of all bridges in the area affected by the disaster to identify obviously unsafe bridges. Aerial view (helicopter), drive through, and traffic video cameras will be used for this level of inspection. The purpose of this inspection is:

To close obviously unsafe bridges
To identify routes that cannot be traversed
To identify the geographical extent of damage

The Level II inspection will be conducted on all bridges in the affected area except those that had collapsed or suffered partial collapse. Teams of WSDOT personnel or volunteers led by an experienced WSDOT inspection engineer will be chosen to perform this level of inspection. Transportation to the site would probably be by regular DOT van or by helicopter for great event.

Level II Inspection

In addition to the first two objectives of Level I inspection, Level II inspection team will make an assessment of the structural and geotechnical post earthquake condition of the bridge. The inspection team may barricade a bridge to slow the approach speed of vehicles when the only visible damage is settlement by a few inches of the pavement at the approach to the bridge abutment, or may limit bridge use to emergency vehicles only.

Level III Inspection

The Level III inspection will be performed on all bridges recommended for further inspection by Level II teams. In addition to the objectives of the other two levels of inspection, this inspection team will make recommendations for repair, shoring, as well as closure or limited access to damaged bridges.

Assessment Forms

Levels II and III inspection forms are shown in Figs. 2.3 and 2.4. The proper form should be filled out by the inspection team at the conclusion of inspection at the bridge site. Proper actions should be taken to ensure the public safety. According to the ERG, if the structure is in imminent danger of collapse, the inspection team shall: coordinate with the State Patrol to
stop traffic from crossing the bridge, radio for region assistance to provide temporary barricades and inform the Region Command Center of the closing. To ensure public safety and prevent further damage, traffic restrictions on the bridge will be implemented by the regions based on the recommendations of the inspection teams. Emergency Response Communications for Bridge Management is shown in Fig. 2.5, which is reproduced from ERG.

3. RESOURCES

Resources needed for Levels II and III inspection are:

1. Regular (existing) inspection team equipment. See list of tools.
2. Radios and cellular phones for communication.
3. Mountaineering equipment for access under unusual conditions (for those trained on its use).
4. Water, food, tents, shelter and supplies for 72 hours per person.

TOOLS

List of tools used in Regular Inspection.

1. Have on hand, or available, the following general tools:
   ▶ Field book, inspection forms, sketch pad, paper, pencils, and clipboard
   ▶ 100 foot tape, pocket tape or 6 foot folding rule
   ▶ Geologist hammer and/or testing hammer
   ▶ Inspection mirror on swivel head and flashlight (for seeing into inaccessible areas)
   ▶ Keel marker
   ▶ Camera and film
   ▶ Binoculars
   ▶ Tool belt, boots (and hip boots if wading is required)
   ▶ Wire brush, shovel, and whisk broom (for cleaning)
   ▶ Pocket knife
   ▶ Safety harness and lanyard
   ▶ Scraping tool
   ▶ Calipers
   ▶ Ladders
   ▶ Lead lines
   ▶ Hand level
   ▶ Thermometer
   ▶ Pocket or wrist watch
   ▶ Plumb bob
   ▶ Safety vest
   ▶ Hard hat
   ▶ Rope
- Axe
- Machete with sheath
- Tool box
- Life Jacket
- Gloves, PVC coated and leather
- Ear plugs
- Eye wash
- First aid kit
- Cones, traffic safety
- Fire extinguisher
- Sign, flagman's signal

2. Also have on hand, or available, the following special tools for concrete, steel, or timber:
   - Piano wire or some other device for measuring the depths of cracks
   - Chain or other tools for dragging deck areas to locate deteriorating sections
   - Feeler gauges or crack comparator for measuring crack widths
   - Large screwdriver, slotted
   - Screwdriver, Phillips
   - Heavy-duty pliers
   - Open-end wrench, adjustable (crescent)
   - Dye penetrant kit and wiping cloths for finding small cracks and determining their depth
   - Periscope, fluorescent tube trouble light
   - Magnifying glass for viewing small cracks along welds and around connections
   - Geologist hammer for determining depth and degree of rot or decay
   - Pry bar or crow bar for pulling on fittings to determine their tightness and for checking any deterioration between wood surfaces
   - Increment borer, ship auger, or hand drill and bit for taking test borings to determine the extent of internal damage
   - Creosoted plugs to plug test holes
   - Straight edge (protractor with steel ruler)
   - Flagging for marking damaged areas, hub tacks

As a general rule, badly damaged structures will be inspected first so that emergency actions to shore, barricade, or repair the structures can be started as soon as possible.

All inspection teams should take a copy of the bridge inventory log book with them. The log will be very helpful in locating and identifying bridges and will provide other useful information.
4. INSPECTION PROCEDURES

No two bridges will be alike; thus, there is no single set of procedures which can be followed in all cases. Clearly, the procedure to follow when inspecting a 50-year-old steel truss will be much different than those used to inspect a 6-year-old concrete box girder. In addition, certain bridge elements or design features may require other types of inspection altogether. Accomplishing any inspection, therefore, will require the inspector to use his/her best engineering judgement, expertise, and common sense.

The following pages describe the post earthquake inspection procedures to be followed for inspecting any concrete, steel, or timber bridge, and the specific procedures to follow for inspecting a given bridge element (the bridge abutments, for example). These steps can be used by the inspector as a checklist to help accomplish the inspection and to help spot particular types of problems a given bridge or bridge element will be prone to.

General Procedures: Concrete, steel, and timber bridges

When en route to the bridge site

♦ Review the type and location of the bridge and the type of materials to be encountered
♦ Organize an inspection routine to:
  • Establish an effective and efficient sequence for inspection of various bridge elements
  • Assign inspection responsibilities to the appropriate individuals
♦ Anticipate the need for any special tools or equipment and try to arrange to have them available at the bridge site.

Bridge Site Procedures

1. Note arrival and departure times
2. Record the ambient air temperature
3. Make a visual inspection of the entire bridge and note the following:
   • Collapse, partial collapse
   • Superstructure damage (movement, buckling, cracking or failure)
   • Substructure damage (tilting, settlement, sliding, cracking)
   • Bearing damage
   • Geo-Hazard (soil liquefaction, slope failure, fissures)
   • Earthquake restraints failed
4. Inspect the superstructure and substructure elements to confirm their structural condition and note questionable elements that need additional investigation.
5. Discuss observations with team members and arrive at consensus.
6. Fill out the appropriate inspection form and check the rating box at the top right hand corner, Figs. 2.3 and 2.4.
7. As required barricade the bridge approaches or passage below bridge, and inform appropriate authority this has been done.
8. Take photos of inspected bridges showing damage.
9. Get the names and addresses of persons who may have taken photos before you.
10. Be sure to identify any markings made on the bridge, such as the ends of significant cracks with date, time, and your initials. Because of aftershocks someone may have to return to inspect the bridge after you leave.

**Inspection Focus**

The following types of damage to bridge components were common in recent earthquakes:
- Hinge and hinge restrainer damage
- Abutment damage
- Pounding damage at abutment
  - Shear key cracking and failure
  - Spalling or failure of abutment back walls or wing walls
  - Approach settlement
  - Approach slab buckling
- Deck spalling
- Column and girder damage
- Pile cap and pile damage
- Partial or complete collapse

**Damage in steel bridges**

- Pounding damage between adjacent elements
- Buckling of cross-bracing
- Bending of cross-brace gussets
- Bearing damage
- Anchor bolt and restrainer fracture
- Damage to supporting abutments and pier walls
- Cracking of welds or members
- Shifted girders

Damage in structural steel elements is often not as readily apparent as in concrete components. Careful inspection should be made of bearing assemblies, keeper plates, anchor bolts, earthquake restrainers, hinge connections, utility hangers, and other details. Sheared bolts, buckled or bent members, cracked welds, shifted girders, and anything out of the ordinary should be assessed and notes made as to the severity and location of the damage.

Anchor bolts which connect steel caps or columns to concrete components should be checked. They should be examined to assure that they have not failed above or below the concrete surface. The initial inspection should include a visual inspection of the nuts to see if the bolts have elongated (leaving the nut loose). Other methods to determine if the bolt has elongated are: striking the nut with a hammer and observing the sound (a sharp ring indicates the bolt has not broken and a dead sound is an indication of a broken bolt) and by attempting to insert a thin object, such as a knife blade under the nut. If the object can be inserted, this is an indication that the bolt has elongated. Any bolts which appear to have been elongated or
highly stressed should be marked and recorded for later testing, using ultrasonic testing equipment or other non-destructive testing means.

Some structural elements such as hinges in box girders, footings and piles are not visible unless special access is created by cutting holes into box girder cells, excavating around footings, etc. It is obviously not feasible to create special access to all these details. However, a close inspection of the area around and adjacent to these details would give a good indication of whether the structure experienced sufficiently large forces or movement to have caused damage to these components. Major spalling of the concrete at deck expansion joints and bearing seats are indicators of possible internal damage. Large cracks or settlement of the ground over footings are indications of possible substructure problems. Note that in case of large footings these indications may be as far as 10 feet or more away from the bent columns.

Other instruments such as a periscope and a fluorescent tube trouble light may be used to look at hinge diaphragms and girder stems in box girders through the vent holes in the bottom slab. If damage is suspected in these hidden details, access should be created and the detail checked.

The condition of severely damage structures may worsen due to aftershocks, traffic, or simply, gravity. To facilitate monitoring and detecting changes in the condition of a damaged structure, the larger cracks should be marked with paint or other material and, if possible, apply crack width monitors. Other references may also need to be placed on the structure. These references will provide a more definitive means of assessing whether the damage is getting worse. When assessing any bridge, you should assume that an aftershock will occur and consider what effect it may have.
REFERENCES


15. ATC-6, Seismic Design Guidelines for Highway Bridges. Applied Technology Council, Redwood City, California 94065.


17. ATC-6-2, Seismic Retrofitting Guidelines for Highway Bridges, August, 1983. Applied Technology Council, Redwood City, California 94065.


Overview of the Emergency Response Inspection Procedures

Table A-1
<table>
<thead>
<tr>
<th>Application Range</th>
<th>Level I</th>
<th>Level II</th>
<th>Level III</th>
</tr>
</thead>
<tbody>
<tr>
<td>All bridges within the area affected by the earthquake.</td>
<td>All bridges in the affected area except those that collapsed or suffered partial collapse.</td>
<td>All bridges recommended for further inspection by Level II teams.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Method of Inspection</th>
<th>Level I</th>
<th>Level II</th>
<th>Level III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aerial view (Helicopter) or Drive through or Traffic video-camera</td>
<td>For great event, helicopter. For other events, probably regular van-type transportation will be needed.</td>
<td>For great event, helicopter. For other events, probably regular van-type transportation will be needed.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Personnel</th>
<th>Level I</th>
<th>Level II</th>
<th>Level III</th>
</tr>
</thead>
<tbody>
<tr>
<td>To be designated.</td>
<td>Team of WSDOT personnel or volunteers led by an experienced WSDOT inspection engineer.</td>
<td>&quot;Regular&quot; inspection teams.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Objectives</th>
<th>Level I</th>
<th>Level II</th>
<th>Level III</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) To close all unsafe bridges.</td>
<td>(1) To close all unsafe bridges.</td>
<td>(1) To close all unsafe bridges.</td>
<td></td>
</tr>
<tr>
<td>(2) To identify routes that cannot be traversed.</td>
<td>(2) To identify routes that cannot be traversed.</td>
<td>(2) To assess the structural and geotechnical post-earthquake vulnerability of the bridge.</td>
<td></td>
</tr>
<tr>
<td>(3) To identify the geographical extent of damage/affected area.</td>
<td>(3) To assess the structural and geotechnical post-earthquake vulnerability of the bridge.</td>
<td>(3) To limit access to or close damaged bridges.</td>
<td></td>
</tr>
<tr>
<td>(4) To identify routes that should not be traversed.</td>
<td></td>
<td>(4) To identify routes that should not be traversed.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Resources</th>
<th>Level I</th>
<th>Level II</th>
<th>Level III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Helicopter(s), back-up power.</td>
<td>&quot;Regular&quot;/existing inspection team equipment in addition to radios and cellular phones for communications; mountaineering equipment for access under unusual conditions; water, food, and supplies for 72 hours per person.</td>
<td>&quot;Regular&quot;/existing inspection team equipment in addition to radios and cellular phones for communications; mountaineering equipment for access under unusual conditions; water, food, and supplies for 72 hours per person.</td>
<td></td>
</tr>
</tbody>
</table>

Description of Inspection Levels

*Figure A-3*

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Page G10-2

Disaster Plan — Emergency Response Guide
May 1996

Fig. 2.2
## WSDOT Level II - Rapid Assessment Bridge Inspection Report (RABIT)

### Condition

<table>
<thead>
<tr>
<th>Condition</th>
<th>Yes</th>
<th>No</th>
<th>More Review</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>U 1. Collapse, partial collapse</strong></td>
<td>□</td>
<td>□</td>
<td>□</td>
</tr>
<tr>
<td><strong>N 2. Superstr damage (movement, buckling, cracking or failure)</strong></td>
<td>□</td>
<td>□</td>
<td>□</td>
</tr>
<tr>
<td><strong>S 3. Substr damage (tilting, settlement, sliding, cracking)</strong></td>
<td>□</td>
<td>□</td>
<td>□</td>
</tr>
<tr>
<td><strong>A 4. Bearing damage (failure, shearing or pull-out of bolts)</strong></td>
<td>□</td>
<td>□</td>
<td>□</td>
</tr>
<tr>
<td><strong>F 5. Geo-Hazard (soil liquefaction, slope failure, fissures)</strong></td>
<td>□</td>
<td>□</td>
<td>□</td>
</tr>
<tr>
<td><strong>E 6. Earthquake restraints failed</strong></td>
<td>□</td>
<td>□</td>
<td>□</td>
</tr>
</tbody>
</table>

### Hazard

<table>
<thead>
<tr>
<th>Hazard</th>
<th>Yes</th>
<th>No</th>
<th>More Review</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>7. Secondary Structure damage (wingwalls, parapets, etc.)</strong></td>
<td>□</td>
<td>□</td>
<td>□</td>
</tr>
<tr>
<td><strong>8. Other hazard present</strong></td>
<td>□</td>
<td>□</td>
<td>□</td>
</tr>
<tr>
<td><strong>9. Are Major utilities broken</strong></td>
<td>□</td>
<td>□</td>
<td>□</td>
</tr>
</tbody>
</table>

### Recommendations

- □ 10. No further action required
- □ 11. Detailed inspection required (Enter element(s) in notes)
- □ 12. Repair required (Enter element(s) in notes)
- □ 13. Barricades needed in the following areas (Enter notes)
- □ 14. Other: (Enter notes)

**Item Comments and Recommendations**

---

**Fig. 2.3**
<table>
<thead>
<tr>
<th>Overall Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAFE</td>
</tr>
<tr>
<td>LIMITED ENTRY</td>
</tr>
<tr>
<td>UNSAFE</td>
</tr>
</tbody>
</table>

**Instructions:**
Complete inspection and checklist and then summarize the results below.

1. **Recommendations**
- [ ] Barricade
- [ ] Shore and Brace
- [ ] Use Off-Site-Detour
- [ ] Use On-Site-Detour
- [ ] Emergency Vehicle Use
- [ ] Repairs
- [ ] Pedestrian Use
- [ ] Monitor

**COMPLETE THE FOLLOWING SECTION ONLY IF THERE IS DAMAGE OBSERVED (D.O.)**

(Use SWIBS condition codes for primary bridge members)

2. **GEOTECHNICAL**
- [ ] Settlement
- [ ] Liquefaction
- [ ] Landslide
- [ ] Faulting
- [ ] Other

3. **APPROACHES**
- [ ] Aprdwy Condition
- [ ] Operational
- [ ] Roadway Settled
- [ ] Off Bridge Seat

4. **BEARINGS**
- [ ] Bearings
  - [ ] Integral
  - [ ] Contact
  - [ ] Rocker
  - [ ] Elastomeric

5. **SUBSTRUCTURE**
- [ ] AbutmentCond
- [ ] Disturbance
- [ ] Wall Movement
- [ ] Backfill Settlement
- [ ] Foundation Movement
- [ ] Wingwall Movement
- [ ] Wingwall Separation
- [ ] Intermediate Bents
  - [ ] Near Top
  - [ ] Near Middle
  - [ ] Near Bottom
  - [ ] Moment Failure
  - [ ] Shear Failure
  - [ ] Compression Failure
  - [ ] Support Lost
  - [ ] Foundation Failure

6. **SUPERSTRUCTURE**
- [ ] Stringers
- [ ] Floorbeams
- [ ] Longitudinal Beams
- [ ] Truss
- [ ] Arches
- [ ] Girders
- [ ] Shear Cracks
- [ ] Moment Cracks
- [ ] Upper Chord
- [ ] Lower Chord
- [ ] Diagonals

7. **DECK**
- [ ] Deck
  - [ ] Longitude Joints Enlarged
  - [ ] Expansion Joints Opened

**ITEM RECOMMENDATIONS AND NOTES**
Emergency Response Communications for Bridge Management

Figure A-2

Denotes a relationship for all earthquake events

Denotes a relationship for major or great events

Fig. 2.5
SESSION 9
FUTURE INITIATIVES
HITEC FULL-SCALE DYNAMIC TESTING OF
SEISMIC ISOLATION
AND
ENERGY DISSIPATION SYSTEMS

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ABSTRACT

Bridge owners in U.S. have recently become interested in a variety of seismic isolation and energy dissipation systems offered by several manufacturers. There are a number of challenges and barriers to the implementation of these systems however. The most important are the proprietary nature of these systems and the lack of knowledge of their dynamic characteristics and longevity. In addition, there are no uniform design standards and no agreement on testing protocols.

While several of the available systems have undergone static and dynamic testing, the testing often consisted of pseudo static tests or reduced-scale shake-table tests. Some dynamic characteristics can be established from static behavior, but a full-scale dynamic test is the only true test of the response of any system to an earthquake. Full-scale tests are essential to develop higher confidence levels among bridge designers and owners, which will lead to a wider application of seismic isolation in the United States.

In response to the heightened interest and the potential benefits seismic isolation offers, a testing program was developed and executed to provide the bridge community with credible information on the static and dynamic performance and quality of the available seismic isolation and energy dissipation systems. The program is unique for its full-scale dynamic testing.

This paper provides an overview of the testing program and a sampling of test results.

INTRODUCTION

The ductility-design philosophy is the mainframe of modern seismic design guidelines in the U.S. and most other countries. While this design philosophy provides safety against collapse, it tends to be costly due to the damage induced in plastic-hinge zones and severe lateral displacements after even a moderate earthquake. Therefore, in recent years design engineers have sought an alternative design philosophy that avoids or limits damage to bridge elements in order to maintain post-earthquake serviceability. One approach that has shown to be promising in limiting damages to bridge columns due to moderate ground motions is the use of Seismic Isolation.
In the seismic isolation approach, the superstructure mass is uncoupled from seismic ground motions. This is also referred to as “Superstructure” isolation. It uses special types of bearings called “seismic isolation bearings” which are placed below the superstructure and on top of the substructure (piers and/or abutments). Under normal conditions, these bearings behave like conventional bearings. However, in the event of a strong earthquake they add flexibility to the bridge by elongating its period and dissipating energy. This permits the superstructure to oscillate at a lower frequency than the piers. It could also give rise to large relative displacements across the isolator interface, which can be controlled by incorporating damping elements in the bearing or by adding supplemental dampers.

Seismic isolation has also emerged as one of the most promising retrofitting strategies for improving the seismic performance of existing bridges. Japan, New Zealand, and a number of European countries pioneered the use of seismic isolation in civil engineering structures. More recently, the United States has begun to implement isolation technology in bridges. In 1985, the California Department of Transportation (Caltrans) became the first U.S. transportation agency to use seismic isolation on a bridge when it retrofitted the Sierra Point Overpass. The bridge's steel bearings were replaced with seismic isolation bearings (a lead-rubber isolation system) on the existing piers. Since then, approximately 90 U.S. bridges, many of them in the East, have incorporated this technology.

**SHORTCOMINGS**

This wide-spread use of seismic isolation strategy in the U.S., can be credited to the development of the AASHTO Guide Specifications for Seismic Isolation Design in 1991, as well as a partnership among academic researchers, seismic design engineers and industrial entrepreneurs. However, there have been a few shortcomings in the guide specifications. One was its failure to incorporate systems other than elastomeric bearings since at the time of its development, elastomeric systems were the only readily available isolation bearings on the U.S. market. This made it difficult for systems other than elastomeric lead-rubber bearings to be competitive. There was also the lack of sufficient data on the dynamic characteristics and the longevity of isolation systems then in use.

Also, most studies had emphasized building rather than bridge applications and the testing of scale models rather than full-size bearings. There is a fundamental difference between bridges and buildings since bridges require multiple supports, whose spacing range from tens to hundreds of feet. Recent earthquakes have demonstrated that even relatively close points on the soil surface can experience significant relative displacements. In addition, bridge bearings must withstand daily live loads and ambient environments that are much more severe than those in buildings. These concerns gave a sense of urgency to refining and updating the 1991 AASHTO Guide Specifications for Seismic Isolation Design as well as to developing a testing program for evaluation and dynamic characterization of the different available systems.
THE NEWLY DEVELOPED AASHTO GUIDE SPECIFICATIONS FOR SEISMIC ISOLATION DESIGN

In 1995, the AASHTO Subcommittee on Bridges and Structures formed the T-3 Seismic Design Technical Committee with the task of modifying the 1991 Guide Specifications for Seismic Isolation Design. The technical committee formed a task group and developed the new specification by considering the current state-of-practice, results of completed and ongoing technical efforts, and research activities in the field of seismic isolation. The AASHTO Guide Specifications for Seismic Isolation Design was completed in May 1997 and was subsequently balloted and approved by the AASHTO Subcommittee on Bridges and Structures. In addition to numerous modifications to the 1991 guide specifications, the new guide specifications provide recommendations for design of sliding isolation bearings as well as elastomeric bearings.

THE HITEC TESTING PROGRAM

Recognizing the need for impartial verification of the dynamic performance of the readily available seismic isolation and energy dissipation systems, FHWA, Caltrans, and the Highway Innovative Technology Evaluation Center (HITEC) cooperated in launching a full-scale, dynamic-testing program in 1994. FHWA provided financial assistance through the Applied Research and Technology (ART) program, Caltrans' engineering staff provided technical supports and HITEC administered the program.

HITEC is a non-profit organization established under an agreement between the FHWA and the Civil Engineering Research Foundation (CERF), a subsidiary of the American Society of Civil Engineers (ASCE). The mission of HITEC is to evaluate new products, materials, and equipment, for which industry standards or specifications do not exist, and to work towards overcoming barriers to innovation. In fulfilling its mission, HITEC will facilitate the conduct of national, consensus-based evaluations utilizing public highway agencies and other organizations. HITEC came into existence in 1989, but officially began operating in January 1994.

A total of eleven domestic and international manufacturers participated in and completed the HITEC testing program. A test plan was developed with the guidance of a panel appointed by HITEC, which was comprised of experts representing several State Departments of Transportation, FHWA, university researchers, and private industry.

The objectives of the evaluation program were to:

1. Implement a program of full-scale, dynamic testing sufficient to characterize the fundamental properties and performance characteristics of the devices evaluated;
2. Provide guidance on the selection, use, and design of seismic isolation and energy dissipation devices for different levels of performance; and
3. Help with the development of suggested guide specifications for the use of seismic isolation and energy dissipation devices in new bridges and retrofit projects.
The test program examined characteristics such as: stability; range; capacity; resilience; resistance to service and dynamic loads; energy dissipation; functionality in extreme environments; resistance to aging and creep; predictability of response; fatigue and wear; and size effects.

These properties provide the engineer with critical information on the suitability of these devices for specific applications. Furthermore, the program addresses the ability of the vendor or manufacturer to provide a quality system, and to understand and predict system response.

TESTING FACILITY AND ITS CAPACITY

The HITEC full-scale testing program required testing of systems at frequencies of up to 2.0 Hz with large displacements. This demanded a major source of energy, which was not available to most laboratories. The laboratories of the Energy Technology Evaluation Center (ETEC) were selected, since they possessed the necessary capabilities to perform the proposed testing. ETEC is owned and operated by the Rocketdyne Division, a subsidiary of North America Corporation.

The isolator test rig shown in figure 1 had been originally developed for dynamic testing of seismic isolation bearings of nuclear power plants. It has the following capacities:

- Maximum Compressive Load = 800,000 lbs
- Maximum Lateral Dynamic Load = ±240,000 lbs
- Maximum Velocity = ±50 in./sec.
- Maximum Displacement = ±15 in.
- Maximum Component Height = 36 in.
- Carriage Coefficient of Friction = 0.003
- Maximum Lateral Static Load = ±340,000 lbs
- Maximum Operating pressure = 3,000 psi

![Figure 1 - ETEC test rig](image)

SYSTEMS CATEGORIZATION

The isolation and energy dissipation systems submitted for testing were all passive. This means they require no energy input or mechanical interaction with an outside source. Table 1 presents the systems' categorization and their respective manufacturers.
Table 1. Systems’ characterization

<table>
<thead>
<tr>
<th>Category</th>
<th>Manufacturer</th>
<th>Product</th>
</tr>
</thead>
<tbody>
<tr>
<td>A Elastomeric</td>
<td>Dynamic Isolation Systems, Inc.</td>
<td>1- Lead-Rubber Isolation Bearings</td>
</tr>
<tr>
<td></td>
<td>Scougal Rubber Corp.</td>
<td>2- High Damping Rubber Bearings</td>
</tr>
<tr>
<td></td>
<td>Skellerup Industries, Ltd.</td>
<td>3- Lead-Rubber Isolation Bearing</td>
</tr>
<tr>
<td></td>
<td>Tekton Inc.</td>
<td>4- Steel Rubber Bearing</td>
</tr>
<tr>
<td>B Slider/Roller</td>
<td>FIP</td>
<td>5- Slider Bearings</td>
</tr>
<tr>
<td></td>
<td>R.J. Watson, Inc.</td>
<td>6- Eradiquake (Slider Bearings)</td>
</tr>
<tr>
<td>C Spherical Slider</td>
<td>Tekton Inc. Earthquake Protective System, Inc.</td>
<td>7- Roller Bearing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8- Friction Pendulum Bearing</td>
</tr>
<tr>
<td>D Energy Dissipator</td>
<td>Oiles Taylor Enidine</td>
<td>9- Viscous Shear Type Damper</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10- Hydraulic Damper</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11- Hydraulic Damper</td>
</tr>
</tbody>
</table>

Figure 2 shows some of the systems submitted for testing.

SYSTEMS' REQUIREMENTS

Each manufacturer submitted five test articles with predetermined design parameters, specified by the HITEC plan, for testing and evaluation (see tables 2 and 3).

Table 2. Isolator requirements

<table>
<thead>
<tr>
<th>Design Comprehensive Load (+10%) (DCL)</th>
<th>Number of Devices</th>
<th>Test Article Number</th>
<th>Lateral Design Disp. (DD)</th>
<th>Minimum Movement Rating (MR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>150 kips</td>
<td>1</td>
<td>1</td>
<td>6 in.</td>
<td>2.0 in.</td>
</tr>
<tr>
<td>500 kips</td>
<td>3</td>
<td>2,3,4</td>
<td>9 in.</td>
<td>3.0 in.</td>
</tr>
<tr>
<td>750 kips</td>
<td>1</td>
<td>5</td>
<td>12 in.</td>
<td>4.0 in.</td>
</tr>
</tbody>
</table>

Table 3. Energy dissipator requirements

<table>
<thead>
<tr>
<th>Design Rating (+10%) (DR)</th>
<th>Number of Devices</th>
<th>Test Article Number</th>
<th>Axial Design Disp. (DD)</th>
<th>Movement Rating (MR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 kips</td>
<td>1</td>
<td>1</td>
<td>&lt;12.0 in.</td>
<td>2.0 in.</td>
</tr>
<tr>
<td>150 kips</td>
<td>3</td>
<td>2,3,4</td>
<td>&lt;12.0 in.</td>
<td>3.0 in.</td>
</tr>
<tr>
<td>240 kips</td>
<td>1</td>
<td>5</td>
<td>12.0 in.</td>
<td>4.0 in.</td>
</tr>
</tbody>
</table>
TESTING REQUIREMENTS

The testing for the various systems included a combination of the following tests:

*Test #1 – Performance Benchmark*
To verify the stiffness and damping/friction characteristics of the test article and the number of cycles of lateral loading required in stabilizing response.

*Test #2 – Compressive Load Dependent Characterization*
To quantify the effects of varying compressive loads on stiffness, damping and energy dissipation per cycle (EDC) of the test article.

*Test #3 – Frequency Dependent Characterization*
To determine dynamic performance characteristics at varying frequencies.

*Test #4 – Frequency Dependent Characterization*
To determine dynamic performance characteristics at varying frequencies (for dampers).

*Test #5 – Fatigue and Wear*
To evaluate the potential seismic performance changes resulting from 10,000 cycles of service movements (temperature and live load fluctuations).

*Test #6 – Environmental Aging*
To verify seismic performance after exposing the test article to a salt-spray environment.

*Test #7 – Dynamic Performance Characteristics at Temperature Extremes*
To assess the effects of extreme temperatures on performance characteristics (stiffness, damping, and EDC).

*Test #8 – Durability*
To assess component durability after a moderate number of strong motion cycles.

*Test #9 – Ultimate Performance*
To determine ultimate displacements and margins of safety.

PROGRAM STATUS

The HITEC testing program was completed in 1998. A total of 14 reports will be published which include:

1. A test plan requirements report
2. 11 individual reports that provide results for each system tested under the seismic evaluation process (8 for isolator and 3 dampers).

3. A summary of evaluation findings, which synthesizes the performance of all systems submitted for evaluation and provides basic knowledge of seismic isolation and energy dissipation.

4. A test system overview report describing the test system and methodologies used to obtain isolator characterization.

The eight individual reports for isolators have been published by HITEC. The first draft of the three reports on dampers, the summary and the overview report has been completed and the finals are expected to be available by the April 1999.

Figures 3-8 show a sampling of test results for seismic isolation systems.

PROGRAM IMPACTS

The HITEC testing program is expected to have a major impact on the use of seismic isolation systems nationwide. It will increase the confidence level of bridge owners, and lead them to consider and use isolation/damping technology cost effectively to protect otherwise vulnerable structures from more severe earthquake damage. In addition, it will provide an invaluable resource and useful data to academic researchers.

Several states are just beginning to retrofit structures against seismic forces and seismic isolation may very well be a viable strategy for them. It is certain that seismic isolation has reached a level of maturity in U.S. that can provide earthquake hazard mitigation for the entire nation.
Figure 2 - Five of the seismic isolation bearings submitted for testing
Figure 3- DIS force-degradation during test protocol #1

Figure 4- DIS force-degradation during test protocol #8
Figure 5 - Hysteretic behavior of 500 kips elastomeric isolation systems (T=2.0 sec., DD=9")

Figure 6 - Hysteretic behavior of 500 kips slider isolation systems (T=2.0 sec., DD=9")
Figure 7 - Stiffness frequency dependency, number of cycles = 3
(FIP seismic isolation bearings)

Figure 8 - Stiffness frequency dependency, number of cycles = 3
(Scougal-Rubber seismic isolation bearings)
APPLICATION OF SMART MATERIALS TECHNOLOGY TO SEISMIC PROTECTION

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ABSTRACT

Highway bridges are key components in our transportation network. Risks associated with the damage or destruction of these bridges include not only the loss of human life, but a potential for extreme economic implications nationwide. Recent advances in seismic protection systems offer an exciting new means of enhancing the ability of such structures to withstand severe earthquakes. Magnetorheological (MR) fluid devices are a particularly promising type of smart damping system for civil applications. MR dampers offer a variety of desirable characteristics, combining reliable operation and large force capacity with minimal power requirements and environmental robustness. Herein the potential of magnetorheological (MR) dampers for the protection of highway bridges is discussed.

INTRODUCTION

Transportation systems in the United States are generally recognized as being critical to our nation's success and well being. From commerce, to commuting, to travel, Americans rely on the highway systems interconnecting our cities and states. Bridges provide continuity to our highway system. Seismic protection of highway bridges is essential to safeguard lives and maintain a high level of service during and immediately following severe earthquakes. Failure of a bridge will most likely result in the loss of property or, more importantly, of human life for those crossing the bridge. Damaged bridges may compromise emergency response and repair crews ability to respond effectively; traffic may be disrupted or completely stopped for days, weeks and even months. The potential financial loss may far exceed that of simply repairing or rebuilding the bridges.

Bridge damage has been sustained during nearly every major recent seismic event [1]. Failures, such as the collapse of the Nimitz Freeway and Oakland Bay Bridge during the 1989 Loma Prieta earthquake and the numerous highway bridge failures in the Los Angeles area during the 1994 Northridge earthquake, remind us that continued research on the seismic resistance of highway bridges must be pursued and resulting seismic designs implemented. In pursuit of safer and more resistant designs, both traditional methods of seismic protection (i.e., redundancy of members) and non-traditional methods of seismic protection (i.e., structural control) should be studied. The Development of Standards in the Seismic Design section of the American Association of State Highway and Transportation Officials, Inc. (AASHTO) Standard Specifications for Highway Bridges [2] states the philosophy that “Ingenuity of design not be restricted.” It is in this spirit that the application of “smart” materials to seismic protection of bridge structures is presented herein as a viable component of the design of safer highway bridges.
Structural control devices can be categorized as passive, active and semi-active (or “smart”). Passive devices are defined as devices which cannot inject mechanical energy into a controlled structural system and whose dynamic properties cannot be modified in real time. Examples of passive devices are rubber bearings, viscous dampers and tuned mass dampers. Active devices are defined as devices which can inject mechanical energy into a structural system and can be controlled to provide a wide range of dynamical responses to a structure. Active control devices include hydraulic actuators and AC servomotors. “Smart” control devices are defined as devices that can not inject mechanical energy into the structural system, but whose dynamic properties can be changed in real time. Examples of “smart” devices include variable orifice dampers, variable friction dampers, controllable tuned liquid dampers and controllable fluid dampers [3].

Following a brief review of structural control devices considered in the literature for seismic protection of bridge structures, a new class of smart dampers employing magnetorheological (MR) fluid is introduced. To demonstrate the scalability of these devices, the design of monotube, linear MR fluid dampers capable of achieving real-time controlled damping forces of $10^5$ to $10^6$ Newtons is discussed. This MR damper design is shown to be mechanically simple, to require low power and to have high potential for seismic control of highway structures.

**SEISMIC CONTROL OF BRIDGE STRUCTURES**

Passive control strategies have been considered for highway bridges since the early 1970's. In the late 1970's, Blakeley [4], outlined design requirements and provided analytical results for superstructure isolation, a form of base isolation for bridge decks. Throughout the 1980's and 1990's studies have continued on bridge structure isolation demonstrating that this passive form of structural control is an effective means for seismic protection. In 1991, the “Guide Specifications for Seismic Design of Highway Bridges” was incorporated by AASHTO, providing regulations for the design of seismically isolated bridge structures. Numerous seismically isolated bridges have been constructed to date in the United States, as well as New Zealand, Japan and Italy [5].

Viscous dampers have been employed or are planned for six bridges in North America. For example, the seismic retrofit of the Golden Gate Bridge in California is to employ 40 nonlinear viscous dampers with a capacity of 300 tons in connections between the towers and the roadway trusses. [6, 7].

Tuned mass dampers have also been investigated for implementation in highway bridges, but with limited success. Siwieki and Derby [8] concluded in 1972 that tuned mass dampers (TMDs) provided little response reduction to the highway bridge in a Federal Highway Administration experimental project. More recently, Das and Dey [9] performed analytical studies of a simple girder bridge and a Warren truss bridge. They demonstrated that TMDs can reduce the vertical mean square displacements of their model bridge decks. Although promising results were observed for the particular bridge models, generalization of these results was unclear.

To provide more effective response reduction, active and hybrid control strategies have been proposed for highway bridges in recent years. For base-isolated bridge-structures, damaging pounding of adjacent girders may occur during severe seismic activity. A significant challenge is to design an isolation system which avoids strong pounding, yet effectively maintains the benefits of
reduction of the seismic forces on the bridge piers. These goals are in opposition to one another, but can be achieved through the effective use of hybrid control systems. In such systems, a passive device is in a sense a first line of defense and the active component is to insure further response reduction. Reinhorn and Riley [10] performed analytical and experimental studies of a small-scale bridge with a sliding hybrid isolation system in which a control actuator was employed between the sliding surface and the ground to supplement the base isolation system. Yang, et al. [11–13] performed analytical studies and proposed various hybrid systems to reduce the response of a single-span bridge model. The hybrid systems proposed by Yang consist of passive components of rubber and sliding bearings and active components of hydraulic actuators. Nagarajaiah, et al. [14–16] have performed numerous analytical studies of hybrid control for highway bridges using a hybrid system consisting of passive sliding bearings and hydraulic actuators. The hydraulic actuators are used to supplement the passive system in further reducing the deck accelerations.

Active and hybrid systems can provide increased response reduction over the passive systems, but at what cost? The active component of the system is typified by power intensive actuators. These control strategies may not always be feasible in the field. Furthermore, during a large seismic event, when the control system is most needed, the external power source may be severed. Thus researchers have continued to search for control devices that can perform as well as active devices with minimal power requirements.

“Smart” (or semi-active) control devices combine the low power requirements of passive devices with the dynamic adaptability of active devices. There are many types of “smart” dampers being studied today including: variable orifice oil dampers, variable friction dampers, electro rheological (ER) fluid dampers and magnetorheological (MR) fluid dampers.

Researchers to date have focused on the variable orifice dampers for the control of highway bridges. Kawashima, et al. [17–22] have performed numerous analytical tests including variable orifice dampers on single- and three-span girder bridges. Analytical results have been verified with a 20-ton variable orifice damper applied to a 40-ton model single-span bridge. Feng et. al. [23–25] also suggested and studied the use of variable dampers to control bridges. Through analytical studies, Sadek and Mohraz [26] have examined control algorithms for variable damping and stiffness devices to control a single-degree-of-freedom (SDOF) bridge model. Patten et. al. [27, 28] have suggested the use of semi-active vibration dampers to reduce seismic response of bridges. Additionally, Sack and Patten [29] conducted experiments in which a variable orifice damper was implemented on a model bridge to dissipate the energy induced by vehicle traffic. Figure 1 shows a full-scale experiment being conducted by Sack and Patten on a bridge on interstate highway I-35 in Oklahoma to demonstrate this technology.
All of the smart control devices discussed until now in this section have employed some electrically controlled valves or mechanisms. Such mechanical components can be problematic in terms of reliability and maintenance. Another class of smart devices uses controllable fluids. The advantage of controllable fluid dampers is simplicity; they contain no moving parts other than the piston. These devices will be discussed in the next section.

CONTROLLABLE FLUID DAMPERS

Two fluids that are contenders for development of controllable dampers are: (i) electrorheological (ER) fluids and (ii) magnetorheological (MR) fluids. The essential characteristic of these fluids is their ability to reversibly change from a free-flowing, linear viscous fluid to a semi-solid with a controllable yield strength in milliseconds when exposed to an electric (for ER fluids) or magnetic (for MR fluids) field. Although the discovery of both ER and MR fluids dates back to the late 1940’s [30, 31], research programs have to date concentrated primarily on ER fluids. A number of ER fluid dampers (see figure 2) have recently been developed, modeled, and tested for civil engineering applications [32–37].

Recently developed MR fluids appear to be an attractive alternative to ER fluids for use in controllable fluid dampers [38–40] (see also: www.nd.edu/~quake/, www.rheonomic.com/mrfluid/ and www.seas.wustl.edu/research/quake/). MR fluids have an inherent ability to provide a simple and robust interface between electronic controls and mechanical components. Much of the current interest in MR fluids can be traced directly to the need for reliable, fast-acting valves necessary to enable semi-active vibration control systems [41–43]. MR fluid technology provides the means for enabling such a valve.

A typical magnetorheological fluid consists of 20–40% by volume of relatively pure, soft iron particles, e.g. carbonyl iron, suspended in an appropriate carrier liquid such as mineral oil, synthetic oil, water or a glycol. MR fluids made from iron particles exhibit a yield strength of 50–100 kPa for an applied magnetic field of 150–250 kA/m (~2–3 kOe). MR fluids are not highly sensitive to contaminants or impurities such as are commonly encountered during manufacture and usage. Further, because the magnetic polarization mechanism is not affected by the surface chemistry of surfactants and additives, it is relatively straightforward to stabilize MR fluids against particle-liquid separation in spite of the large density mismatch. Antiwear and lubricity additives can also be included in the formulation without affecting strength and power requirements [44,45].

![Figure 2 – Schematic of controllable fluid damper](image-url)
As a controllable fluid, the primary advantage of an MR fluid stems from the large, controlled yield stress it is able to achieve. Typically, the maximum yield stress of an MR fluid is an order of magnitude greater than that of the best ER fluid, while their viscosity is comparable. This has a profound impact on ultimate device size and dynamic range, because the minimum amount of active fluid in a controllable fluid device is proportional to the plastic viscosity and inversely proportional to the square of the maximum field induced yield stress [38,40]. This means that for comparable mechanical performance the amount of active fluid needed in an MR fluid device will be about two orders of magnitude smaller than that of an ER device.

From a practical application perspective, an advantage of MR fluids is the ancillary power supply needed to control the fluid. While the total energy and power requirements for comparably performing MR and ER devices are approximately equal [38,40], only MR devices can be powered directly from common, low voltage sources. Further, standard electrical connectors, wires and feedthroughs can be reliably used, even in mechanically aggressive and dirty environments, without fear of dielectric breakdown. This consideration is particularly important in cost sensitive applications.

Another advantage of MR fluids is their relative insensitivity to temperature extremes and contaminants. Carlson and Weiss [39] indicated that the achievable yield stress of an MR fluid is an order of magnitude greater than its ER counterpart and that MR fluids can operate at temperatures from −40 to 150°C with only slight variations in the yield stress. This arises from the fact that the magnetic polarization of the particles, and therefore the yield stress of the MR fluid, is not strongly influenced by temperature variations. Similarly, contaminants (e.g., moisture) have little effect on the fluid’s magnetic properties. A summary of the properties of both MR and ER fluids is given in table 1.

20-TON MAGNETORHEOLOGICAL DAMPER

Magnetorheological dampers are one of the most promising realizations of semi-active dampers for application to full-scale civil structures. Spencer, et al. [46–48], Dyke, et al. [50–53], Carlson and Spencer [49] and Yi, et al. [54] have recently conducted pilot studies to demonstrate the efficacy of MR dampers for semi-active seismic response control. Through simulations and laboratory model experiments, it has been shown that an MR damper, used in conjunction with recently proposed acceleration feedback strategies, significantly outperforms comparable passive damping configurations, while requiring only a fraction of the input power needed by the active controller. This section demonstrates that the technology can be scaled to devices sufficiently large for implementation in civil engineering structures.

To prove the scalability of MR fluid technology to devices of appropriate size for civil engineering applications, a full-scale, MR fluid damper has been designed and constructed [48,49]. For the nominal design, a maximum damping force of 200,000 N (20-ton) and a dynamic range equal to ten were selected. A schematic of the large-scale MR fluid damper is shown in figure 3. The damper uses a particularly simple geometry in which the outer cylindrical housing is part of the magnetic circuit. The effective fluid orifice is the entire annular space between the piston outside diameter and the inside of the damper cylinder housing. Movement of the piston causes fluid to flow through this entire annular region. The damper is double-ended, i.e. the piston is supported
Table 1 – Summary of the properties of MR and ER fluids [44, 45].

<table>
<thead>
<tr>
<th>Property</th>
<th>MR Fluids</th>
<th>ER Fluids</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. Yield Stress ( \tau_y(\text{field}) )</td>
<td>50–100 kPa</td>
<td>2–5 kPa</td>
</tr>
<tr>
<td>Maximum Field</td>
<td>~250 kA/m</td>
<td>~4 kV/mm</td>
</tr>
<tr>
<td>Plastic Viscosity, ( \eta_p )</td>
<td>0.1–1.0 Pa-s</td>
<td>0.1–1.0 Pa-s</td>
</tr>
<tr>
<td>Operable Temp. Range</td>
<td>-40 to 150°C</td>
<td>+10 to 90°C</td>
</tr>
<tr>
<td>Stability</td>
<td>Unaffected by most impurities</td>
<td>Cannot tolerate impurities</td>
</tr>
<tr>
<td>Response Time</td>
<td>milliseconds</td>
<td>milliseconds</td>
</tr>
<tr>
<td>Density</td>
<td>3 to 4 g/cm³</td>
<td>1 to 2 g/cm³</td>
</tr>
<tr>
<td>( \eta_p/\tau_y^2(\text{field}) )</td>
<td>(10^{-10} – 10^{-11}) s/Pa</td>
<td>(10^{-7} – 10^{-8}) s/Pa</td>
</tr>
<tr>
<td>Maximum Energy Density</td>
<td>0.1 Joules/cm³</td>
<td>0.001 Joules/cm³</td>
</tr>
<tr>
<td>Power Supply (typical)</td>
<td>2–25V, 1–2 A</td>
<td>2000–5000 V, 1–10 mA</td>
</tr>
</tbody>
</table>

Figure 3 – Schematic of 20-ton MR fluid damper.
by a shaft on both ends. This arrangement has the advantage that a rod-volume compensator does not need to be incorporated into the damper, although a small pressurized accumulator is provided to accommodate thermal expansion of the fluid. The damper has an inside diameter of 20.3 cm and a stroke of 8 cm. The electromagnetic coil is wound in three sections on the piston. This results in four effective valve regions as the fluid flows past the piston. The coils contain a total of about 1.5 km of magnetic wire. The completed damper is approximately 1 m long and with a mass of 250 kg. The damper contains approximately 5 liters of MR fluid. The amount of fluid energized by the magnetic field at any given instant is approximately 90 cm$^3$. A summary of the parameters for the 20-ton damper are given in table 2.

Figure 4 shows the experimental setup at the University of Notre Dame for the 20-ton MR fluid damper. The damper was attached to a 7.5 cm thick plate that was grouted to a 2 m thick strong floor. The damper is driven by a 560 kN actuator configured with a 305 lpm servo-valve with a bandwidth of 80 Hz. A Schenck-Pegasus 5910 servo-hydraulic controller is employed in conjunction with a 200 MPa, 340 lpm hydraulic pump.

Figure 5 shows the measured performance for the damper at 5 cm/sec (triangular displacement). The maximum force measured at full magnetic field strength is 201 kN at a piston velocity of 5 cm/sec, which is within 0.5% of the analytically predicted result [48]. Moreover, the dynamic range of the damper is well over the design specification of 10.
Table 2 – Design parameters for 20-ton seismic damper.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stroke</td>
<td>±8 cm</td>
</tr>
<tr>
<td>$F_{\text{max}} / F_{\text{min}}$</td>
<td>10.1 @ 10 cm/s</td>
</tr>
<tr>
<td>Cylinder Bore (ID)</td>
<td>20.32 cm</td>
</tr>
<tr>
<td>Max. Input Power</td>
<td>&lt;50 watts</td>
</tr>
<tr>
<td>Max. Force (nominal)</td>
<td>200,000 N</td>
</tr>
<tr>
<td>Effective Axial Pole Length</td>
<td>8.4 cm</td>
</tr>
<tr>
<td>Coils</td>
<td>$3 \times 1050$ turns</td>
</tr>
<tr>
<td>Fluid $\eta_p / \tau_{y(\text{field})}^2$</td>
<td>$2 \times 10^{-10}$ s/Pa</td>
</tr>
<tr>
<td>Fluid $\eta_p$</td>
<td>1 Pa-s</td>
</tr>
<tr>
<td>Fluid $\tau_{y(\text{field}) \text{ max}}$</td>
<td>70 kPa</td>
</tr>
<tr>
<td>Gap</td>
<td>2 mm</td>
</tr>
<tr>
<td>Active Fluid Volume</td>
<td>~90 cm$^3$</td>
</tr>
<tr>
<td>Wire</td>
<td>16 gauge</td>
</tr>
<tr>
<td>Inductance (L)</td>
<td>6.6 henries</td>
</tr>
<tr>
<td>Coil Resistance (R)</td>
<td>$3 \times 7.3$ ohms</td>
</tr>
</tbody>
</table>

CONCLUSIONS

The essential need for critical transportation arteries to remain operational after severe earthquakes challenges the design engineer to employ innovative seismic protective systems. Because of their mechanical simplicity, low power requirements and high force capacity, magnetorheological (MR) dampers constitute a class of smart dampers that meshes well with the demands and constraints of civil infrastructure applications and will likely see increasing interest from the engineering community as a viable means for mitigating the devastating effects of severe dynamic loads on civil structures.

APPENDIX


SEISMIC INSTRUMENTATION OF THE CAPE GIRARDEAU (MO) CABLE-STAYED BRIDGE

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ABSTRACT

The objective of this paper is to describe the needs and tentative schemes for seismic instrumentation of the Cape Girardeau (MO) Cable-Stayed Bridge now under construction in the New Madrid area, near the location of the 1811–1812 New Madrid earthquakes. The area is seismically active and therefore requiring hazard mitigation programs including those related to investigation of strong shaking of structures and the potential for ground failures in the vicinity of structures.

The instrumentation for the Cape Girardeau bridge will require deployment of sensors on the superstructure and pier foundations, and in the vicinity of the bridge (e.g. free-field both surface and downhole and horizontal spatial arrays to assess the differential motions at the piers of the long span structure). The instrumentation scheme of the bridge will facilitate recording of the seismic response of this important lifeline during mid-size or large events. The response data will be used by researchers and engineers to (1) assess the performance of the bridge, (2) check design parameters including dynamic characteristics with actual response, and (3) better design future similar bridges.

Furthermore, if properly configured, the instrumentation can be used as a “health monitoring” tool (a) to serve as an early warning system and/or (b) to assess if there is damage to the structure. This paper will detail options for possible instrumentation schemes for the subject bridge.

INTRODUCTION

General

The acquisition of structural response data during earthquakes is essential to confirm and develop methodologies for analysis, design, repair and retrofit of earthquake resistant structural systems. Particularly for urban environments in seismically active regions, acquisition of data, from structures including lifelines such as bridges, is one of the basic requirements for a thorough investigation of the effects of earthquakes on the structures. In order to understand the structural responses thoroughly, in addition to structural arrays which should include sensors for soil-structure interaction effects, it is necessary to record ground motions at the free-field in the vicinity of the structures. The New Madrid area, the location of the 1811–1812 New Madrid
earthquakes, is a potentially seismically active region requiring earthquake hazard mitigation programs including those related to investigation of strong shaking of structures and the potential for ground failures in the vicinity of structures (Nuttli, 1974, Woodward-Clyde, 1994).

The USGS, since 1983, has conducted a systematic effort to instrument structures through-out the United States. Further details on the instrumentation of structures program of the USGS can be found in USGS Circular 947 (Çelebi and others, 1987). The effort pertaining specifically for the New Madrid Area is compiled in USGS Open-File Report 87-59 (Çelebi [compiler], 1987) by an advisory committee consisting of seismologists, academicians, and practicing engineers. The report outlines the aims, provides an extensive list of selected and prioritized structures for instrumentation in different parts of the New Madrid area.

Objective

The objective of this paper is to discuss possible schemes of seismic instrumentation to be deployed on and in the vicinity of the new Cape Girardeau Bridge (MO) now under construction. Early planning of seismic instrumentation schemes is a necessity in order to cover installation of basic hardware during the construction phase (such as conduits and cables to interconnect the sensors to recorders and conduits for deployment of downhole accelerographs).

For seismic engineering studies, in general, three different objectives are sought. In planning for the overall scheme, it is important to clearly identify these objectives:

1. Instrumentation of the superstructure and pier foundations.
2. Instrumentation of the free-field in the vicinity of the structure including those related to downhole measurements and horizontal spatial arrays to assess the differential motions at the piers of the long span structure.
3. Ground failure arrays in the vicinity of the structure.

The current plan is to have this work done via the FHWA-sponsored MCEER Highway Project, as part of a contract authorized under the TEA-21 legislation. The intent is that the USGS will complete the system design, a consultant will be employed to check the final design, and that MCEER will acquire the hardware and turn it over to the state of Missouri for installation.

Scope

The scope of this paper is limited to description of seismic instrumentation covering the first two objectives. Details of geotechnical considerations related to the bridge are provided in Woodward-Clyde (1994).

It is important to note also the proposed instrumentation is for strong-motion recording and not for low-amplitude weak motions. For low-amplitude weak motions, special purpose temporary deployment schemes should be adopted. However, by increasing the sensitivities of the recording systems, it may be possible to record ambient response of the subject bridge. This is not within the scope of this paper.
Due to changes in design and construction constraints, the instrumentation schemes presented herein may have to be altered. Hence, the scope presented here could be adjusted to such changes. However, the discussions provide an overall understanding of the requisite instrumentation.

SEISMIC INSTRUMENTATION

Goals

In general, when instrumenting a bridge, the following overall goals are aimed at:

a. Detection of the structural response (of the caissons, tower, deck). Whenever applicable, sufficient additional sensors for soil-structure interaction (SSI) are added.

b. Spatial variation of ground motion along the total span (3956 ft ~ 1206 m) of the bridge (longitudinal, translation and vertical).

Measurements

The goals stated above can be achieved by deploying pre-packaged combinations of uniaxial, bi-axial, tri-axial and tri-axial downhole accelerometers. These will measure acceleration responses at the deployed points of the structure and/or free-field. In general, the accelerometers are common-time connected to recording units. Common-time recording is essential in order to make comparative study of the time-variant responses at one location of the structure with respect to another.

Recently, there are new developments to monitor the responses of structures in real-time or near real-time. This approach is usually adopted when rapid response (in real-time or near real-time) information is a necessity. New approaches to monitoring structures such as “health-monitoring” techniques require sophisticated and relatively expensive configuration of hardware. It may require radio-telemetry from locations such as the top of the tower to a designated and remote recording location. The response information that is retrieved may then be configured to make decisions to secure the safety requirements (“health”) of the structure.

Recent developments in improved sampling rates (~10Hz) of differential Global Positioning Systems (GPS) brings an opportunity to measure real-time relative displacements of long-period structural systems such as the Cape Girardeau Bridge (Çelebi et al, 1999). In general, measuring displacements is a very difficult task. Up to now, to calculate displacements, it was necessary to record accelerations followed by double-integration. This process cannot usually be done directly and normally requires processing techniques. On the other hand, differential GPS tri-axial displacement measurements can now be made with very high accuracy (horizontal error of 1 cm and vertical error of 2 cm). In Figure 2, a recent test performed using differential GPS units shows the clear response signals achieved (Çelebi et al, 1999). Therefore, in instrumenting Cape Girardeau Bridge, differential GPS technology should be considered.
General Locations

In accordance with the above aims and measurement capabilities, Figure 1 shows the general locations where sensors should be deployed for the Cape Girardeau Bridge: (a) to record motions of, and (b) assess the following responses of the bridge and its vicinity:

a. Overall motion of the cable-stayed bridge structure (tower, deck, and caissons).
   1. Motions of the two towers to assess their translational and torsional behavior – relative to the caissons and deck levels.
   2. Motions of the deck to assess the fundamental and higher mode translational (longitudinal, transverse and vertical) and torsional components.

b. Motions of extreme ends of the bridge as well as intermediate pier locations to provide data for the translational, torsional, rocking and translational SSI at the foundation levels. This setup will also provide insight into the horizontal and vertical spatial variation of ground motion along the total span (3956 ft ~ 1206 m) of the bridge (longitudinal, translation and vertical).

c. Free-field motions at the surface and downhole locations reaching competent rock.

In this particular description for instrumenting Cape Girardeau Bridge, no specific downhole arrays below the caissons of Piers 2 and 3 are considered. The argument for this is that the motions at the free-field (surface and downhole) in the vicinity of Bent 1 (on the Missouri side) and Pier 15 (on the Illinois side) can be justifiably used as surrogate motions to perform detailed analyses of the bridge deck, piers with towers, towers, piers without towers and caisson tops. Also, no load cells for the cables are included. If desired, these could be added on to the overall list and the costs associated with the load cells also can be summed up to the rest of the hardware detailed below.

However, because of the known changes in interlayering of soft and hard rock at Piers 4 and 5, we recommend that three triaxial accelerographs (two downhole and one surface accelerographs) be deployed at approximately 100 ft to the North or South of Pier 5.

Strategies for Housing the Recording Systems for the Structural Array

There are several possible strategies for implementation of the structural array:

A. A new approach to seismic instrumentation scheme appears to be feasible by taking advantage of the new state-of-the-art recording hardware that includes or can be ordered with time-measuring GPS capability. This can be used for common-time recording of events for one or more recording system used at different locations of a structure such as the long-span cable-stayed Cape Girardeau bridge. Thus, cables and appropriate connections are minimized. Normally, since a bridge is inherently a long span structure, extensive and expensive cabling (and conduits) would be required to interconnect various sensors to the recorder and various recorders to one another. The proposed or alternative (recording system) hardware can be configured to provide combinations of 3-18 channels at different locations (Bent 1, Pier 2, Pier 3, Pier 4 and Pier 15). The recommended strategy would be more
economical than having all recording systems in one or two locations with various sensors deployed at different piers, caisson tops, towers and deck connected to the recorders with long cables. The downside of this approach is that for maintenance of the recording systems, several stops and access arrangements would be required.

B. All recording systems be placed in one location (e.g. in the room at Pier 2 or 3). This has advantages from maintenance point of view and future transmission of data by phone lines or telemetry such that all data can be downloaded or transmitted from one location. The negative of this approach is the extensive "heavy duty" cabling required between sensors and recorders.

C. All recording systems up to and including Pier 5 be placed on ground in a secure housing at Bent 1 (Missouri side) or Pier 2 or Pier 3. All recording systems from Pier 6 through Pier 15 be located in a secure housing on the Illinois side. This will also require extensive cabling.

D. A more expensive approach is to develop location-specific sensor packages that includes some form of radio telemetry which transmit data from the measuring location to a recording or observation location. In the case where high-accuracy differential GPS units are used for relative displacement measurements on the structure or the site, radio telemetry is the most reasonable option since there is a limit to the cable length for transmittal of GPS signals to its receiver units.

Special Considerations for Weather

In addition, it is essential to consider the general weather requirements for the instrumentation. Cape Girardeau area is often subjected to severe thunderstorms and lightning. Therefore, at each step of the way, lightning protection for the seismic instruments will be required. This is available for the type of recording systems that are cited later on in this paper. However, special transient protection hardware is necessary for the downhole accelerometers.

Specifics

On Bent 1 (Figure 1) only one acceleration (and/or differential GPS) triaxial unit is recommended. However, in the immediate vicinity of Bent 1 and Pier 15, within a distance of 100-300 m., free-field arrays are recommended. These free-field locations, without any feedback from the structure, will be essential in providing the ground motions that may be used as a surrogate for the various piers of the bridge and also for convolution and deconvolution studies of the free-field ground motion.

Also, as repeated above, because of the changes in interlayering of soft and hard rock at Piers 4 and 5, deployment of three triaxial accelerographs (two downhole and one surface accelerographs) at approximately 100 ft to the North or South of Pier 5 is recommended.

Figures 3 shows a sample recommended instrumentation scheme for Pier 3. The specific caisson 3 instrumentation is shown in Figure 4. Caisson 3 will be equipped with one vertical sensors at each of the 3 corners to detect vertical translational and rocking motions, if any and additional horizontal sensors to detect translational (longitudinal and transverse) motions.
Figure 6 shows deck level instrumentation for Pier 4. Essentially, this will be repeated for Piers 5, 10 and 15. At each of the the foundation levels of Piers 5, 10 and 15, a triaxial accelerograph is planned. For specific piers, this might be modified if so desired.

Figures 6 and 7 show general instrumentation scheme for deck location at the centerline (CL) of the cable-stayed deck, and at the “quarter” locations (L1, L2, R1 and R2).

Specific sensors, the particular orientations desired are shown on each figure. Essentially, the orientation of a sensor is meant that the degree of freedom in that direction is important.

Types of Hardware and Approximate Costs

The total cost for the hardware for the instrumentation scheme presented herein is approximately $360,000. This amount does not include any high accuracy differential GPS units to measure relative displacements, should they be considered. The GPS units for common-time acquisition are included. The configuration of the instrumentation can be changed in any other fashion desired and estimates of the total can be easily revised.

<table>
<thead>
<tr>
<th>SENSOR TYPE1</th>
<th>COMMENTS</th>
<th>CHAN/UNIT</th>
<th>QUANTITY</th>
<th>CHANNELS</th>
<th>UNIT COST [$1000]</th>
<th>TOTAL [$1000]</th>
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<tbody>
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<td>FBA-11</td>
<td>uniaxial accelerometer</td>
<td>1</td>
<td>14</td>
<td>14</td>
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<td>FBA-23DH</td>
<td>downhole accelerometer</td>
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<td>6</td>
<td>18</td>
<td>$12.00</td>
<td>$72.00</td>
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<td></td>
<td></td>
<td></td>
<td>98</td>
<td></td>
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<th>RECORDE R</th>
<th>comments</th>
<th>CHAN/UNIT</th>
<th>QUANTITY</th>
<th>CHANNELS</th>
<th>UNIT COST [$1000]</th>
<th>TOTAL [$1000]</th>
</tr>
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<tr>
<td>Mt. Whitney</td>
<td>records accelerations</td>
<td>Up to 18</td>
<td>5</td>
<td></td>
<td>$21.00</td>
<td>$105.00</td>
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<tr>
<td>K-2</td>
<td>records accelerations</td>
<td>3-12 chan.</td>
<td>2</td>
<td></td>
<td>$13.00</td>
<td>$26.00</td>
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<th>QUANTITY</th>
<th>CHANNELS</th>
<th>UNIT COST [$1000]</th>
<th>TOTAL [$1000]</th>
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<tr>
<td>Regular</td>
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<td></td>
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<td></td>
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</tr>
<tr>
<td>Downhole</td>
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<td></td>
<td>1500 ft.</td>
<td></td>
<td>$10.00</td>
<td>$15.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$31.00</td>
</tr>
<tr>
<td>CONDUITS</td>
<td></td>
<td></td>
<td>5000 ft.</td>
<td></td>
<td>$3.00</td>
<td>$15.00</td>
</tr>
<tr>
<td>MISC.</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td>$23.00</td>
</tr>
</tbody>
</table>

|              |           |           |          |          |                   | $360.00       |

1The sensors and recorders mentioned herein are typical products. The statement of these names does not constitute as an endorsements or preferences on other types of commercially available sensors and recorders.
To repeat, the cost estimates do not include installation costs. Similarly, the estimates do not include costs for dedicated phone lines (if desired) or telemetry system (if desired) for communicating with the recording systems and transfer of data to another location. However, these can be detailed and added later on once the scheme of instrumentation is finalized and the location of the recording systems are decided upon.

CONCLUSIONS AND RECOMMENDATIONS

A discussion on possible seismic instrumentation scheme for the Cape Girardeau Cable-Stayed Bridge now under construction is presented in this paper. The scheme described provides extensive strong-motion response recording capability to facilitate different types of studies and to assess the performance of the subject bridge during strong-motion events. The instrumentation scheme recommended herein can be also used to record low-amplitude motions for short periods of time (following the conclusion of construction and deployment of the instrumentation) should it be desired – by raising the gains on the recording system. However, it is recommended that the system be thought of as necessary for recording responses of the structure to moderate to large events. It is envisaged that the scheme described here will be altered due to changes in design and budget considerations.

REFERENCES


DISCLAIMER AND ACKNOWLEDGEMENTS

The deliberations for possible seismic instrumentation schemes of the Cape Girardeau Cable-Stayed Bridge described herein are not final. Furthermore, there is no commitment by any federal and State agencies (Federal Highway Administration [FHWA], United States Geological Survey [USGS]) and State (Missouri Department of Transportation [MODOT]). They are based on previous deliberations of an ad-hoc committee formed in 1996 as outlined in USGS Open File Report 97-80 and additional developments since then. The final instrumentation scheme of the bridge will be based on extensive work to be carried out during 1999 and the level of funding through FHWA. As part of its earthquake hazard reduction program, USGS, will provide input in deployment of instrumentation, establish a mechanism with MODOT and FHWA to maintain the instruments, retrieve, process and disseminate strong-motion response data following events in the future. The contributions to the discussion presented herein of the following individuals are acknowledged: Tom Cooling (Woodward-Clyde, St. Louis, MO), Jerry DiMaggio (US DOT, FHWA HQ Bridge Division), Christopher Dumas (US DOT, FHWA Region 3, Wash. DC), William Forester (US DOT, FHWA Region 7), Ian Friedland (MCEER, Buffalo, NY), Glen Fullerson (FHWA, Jefferson City, MO), Steve Hague (HNTB, Kansas City, MO), Francois Heuze (LLNL, Livermore, CA), James Hummert (Woodward-Clyde, St. Louis, MO), Larry Hutchings (LLNL, Livermore, CA), David McCallen (LLNL, Livermore, CA), Ronald Porcella (USGS, Fresno, CA), Clifford Roblee (Caltrans, Sacramento, CA), Marion Salsman (USGS, Menlo Park, CA), Tom Shantz (Caltrans, Sacramento, CA), Glenn Smith (US DOT, FHWA HQ Bridge Division), William Strossner (Missouri DOT, Jefferson City, MO), Philip Yen (US DOT, FHWA, McLean, VA).

Figure 1. General Locations of Seismic Instrumentation along the Bridge
Figure 2. GPS Monitoring of Long-Period Structures – Results from a Test

Figure 3. Suggested Deployments of Accelerometers for Pier 3 and the Tower (arrows indicate uniaxial accelerometers, dots indicate accelerometers perpendicular to the paper).
Figure 4. Pier 3 Caisson – Suggested Instrumentation (Arrows and dots indicate uniaxial accelerometers with dots perpendicular to the paper).

Figure 5. Suggested Instrumentation for Pier 4.
Figure 6. Instrumentation of Deck of Cable-Stayed Bridge (see Fig. 7)

Figure 7. Instrumentation for Center of Span and for Other Deck Locations (see Fig. 6).
THE USE OF ADVANCED COMPOSITES IN SEISMIC BRIDGE RETROFITTING

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ABSTRACT

Advanced composites known as polymer matrix composites (PMCs) or fiber reinforced polymers (FRPs) have been shown to be structurally very efficient in the seismic retrofit of constructed facilities. Light weight and ease of application also result in cost effective retrofit measures despite higher material costs compared to conventional retrofit systems.

Design and application issues with advanced composite retrofit for bridges are discussed together with structural performance validations from large and full scale laboratory experiments. Examples for seismic response and retrofit design will include laboratory tests and field applications on seismic retrofits of reinforced concrete bridge columns, bridge bents, and superstructures. Detailed design guidelines for seismic retrofits using FRPs are presented together with critical detailing and application issues. New developments in the use of FRPs for seismic bridge protection are also presented.

INTRODUCTION

Advanced composite materials, predominantly developed for use in the aerospace industry, have shown a great potential for strengthening, retrofitting and repair of existing buildings and bridge structures to extend their service life well into the 21st century. With the broader availability of glass, aramid and carbon fibers, as well as automation in the manufacturing process, PMCs can be affordable and competitive with conventional structural rehabilitation materials and processes.

While the low density and the high mechanical characteristics of advanced composite materials were long recognized, applications in the civil engineering sector were limited to date due to (1) high materials and manufacturing costs, and (2) the component by component replacement of existing structural members rather than a comprehensive design approach with these new materials. This paper provides an overview of the use of advanced composites for the seismic retrofit of existing bridge structures.

The structural effectiveness of PMCs in retrofitting existing bridge systems has repeatedly been demonstrated with full or large-scale structural tests at the University of California, San Diego. Carbon fabric overlays have been used to retrofit single columns and multi-column bridge bents for seismic loads and to restore and more than double the displacement capacity of damaged or substandard columns. Bridge columns were retrofitted and repaired with fiberglass, carbon and hybrid composite jackets and these FRP retrofits were shown to be just as effective as conventional steel jacket column retrofit technology.
The realistic range of properties for FRPs in civil engineering applications can be summarized as follows:

**strength:** For quasi-isotropic material considerations or design assumptions strength values comparable to high strength structural steels can be achieved. However, due to the non-ductile failure characteristics only a limited range of these capacities can be utilized. Unidirectional fiber geometry can result in strength characteristics similar to high strength prestressing wires and strands.

**strain:** Failure strains are low and typically limited to the 1 to 3% range with carbon fibers at the lower and glass and aramids at the upper end of the range.

**modulus:** Moduli for quasi-isotropic assumptions range from 10% to 30% of steel and for unidirectional fiber applications from 1/3 to 2/3 of the structural steel modulus.

**cost:** Cost is dominated by the price of the fiber material, ranging currently per pound from $1-3 for glass to $10-25 for carbon (T300, AS4). Typical resins are $1-2 per pound.

With a 40 to 60% typical fiber volume fraction and automated manufacturing techniques, PMC structural component costs in place can range from $3-15 per pound, with glass at the lower and carbon at the upper end of the range.

It should be noted that both strength and modulus decrease rapidly with deviation of the fiber orientation from the loading direction which is largely a function of a very low shear modulus of the matrix or resin system, typically 1% or less of Young's Modulus for steel. For retrofitting of existing concrete structures, the reduced weight of these advanced composite materials in the form of thin overlays is a major benefit, as will be discussed in the following.

While the chemical resistance of these PMCs to acid or corrosive environments is very good in general, durability aspects such as ultra violet degradation of the matrix, alkaline reactions between glass and concrete, water absorption by the resin system as well as fire and vandalism protection typically require a form of external coating. Furthermore, only limited information on the creep and relaxation characteristics of affordable PMCs in the civil engineering environment exist to date and require comprehensive evaluations.

**SEISMIC RETROFITTING OF BRIDGE COLUMNS**

The seismic response of concrete columns is to a large extent controlled by the amount and detailing of the horizontal or transverse reinforcement. Since the transverse reinforcement for many existing columns was not designed based on current design knowledge and criteria but only nominally provided to support the longitudinal column reinforcement during construction, the retrofit solution is to provide the missing horizontal reinforcement externally. External jackets need to be designed to prevent three critical column failure modes, namely (a) shear, (b) flexural hinge confinement/bar buckling, see figure 5, and (c) lap splice clamping. Advanced composite jacket design issues for these three predominant failure modes are discussed in the following.
a) Shear:

The column shear failure is primarily a strength and a dilation problem. Shear strength can be added to concrete columns by hoop or horizontal reinforcement in the form of 90° (from the column axis) oriented composite fibers, where the opening of inclined cracks, and with it the loss of aggregate interlock in these cracks which is one of the key shear force transfer mechanisms, can be controlled by limiting the column dilation in the loading direction to experimentally determined dilation strains of $\varepsilon_d < 0.004$ or 0.4%, [1,3]. The jacket thickness for shear retrofit can be determined based on Eqn. (1) for circular and rectangular columns as

$$
t_{j} = \frac{V_0 - (V_c + V_s + V_p)}{\phi_v \frac{\pi}{2} \times 0.004E_{ij} \cdot D}
$$

for circular columns,

$$
t_{j} = \frac{V_0 - (V_c + V_s + V_p)}{\phi_v \frac{2 \times 0.004E_{ij} \cdot D}{2}}
$$

for rectangular columns,

where $V_0$ is the column shear demand based on full flexural overstrength in the potential plastic hinges, $\phi_v$, a shear capacity reduction factor (typically taken as 0.85), $V_c$, $V_s$ and $V_p$ the three shear capacity contributions from the concrete, horizontal steel reinforcement and axial load based on the UCSD three component shear model [4] with reductions for the concrete component $V_c$ in the flexural plastic hinge region based on the ductility demand, and $E_{ij}$ and $D$ the composite jacket modulus and the column dimension in the loading direction, respectively.
From figure 6 it is obvious that the high strengths in the composite materials can typically not be utilized due to the limitation on the dilation strains to ensure aggregate interlock. Thus, the proportional relationship for composite jacket thickness for shear retrofit can be expressed as

$$t_j^v \sim \frac{1}{E_jD} \times C_v$$  \hspace{1cm} (2)

where $C_v$ denotes the remaining coefficient derived from Eqn. (1). Equation (2) shows that for most advanced composite jacket retrofits the jacket thickness is inversely proportional to the jacket modulus and the column dimension in the loading direction.

![Diagram](image)

**Figure 6 – Typical mechanical characteristics of column jackets in the hoop direction**

**b) Flexural hinge confinement:**

Inelastic deformation capacity of flexural plastic hinge regions can be increased by stabilizing the longitudinal bars against buckling and by confinement of the column concrete with hoop reinforcement from an advanced composite fiber jacket system. For circular columns the required jacket thickness can be expressed as

$$t_j = 0.09 \frac{D(e_{cu} - 0.004)f'_{cc}}{\phi_f \cdot f_{ju} \cdot \varepsilon_{ju}}$$  \hspace{1cm} (3)

where $f'_{cc}$ is the confined concrete compression strength which depends on the effective lateral continuing stress and the nominal concrete strength and can be conservatively taken as 1.5 $f'_{cc}$, for most retrofit designs [4] $f_{ju}$ and $\varepsilon_{ju}$ are the strength and deformation capacity of the composite jacket in the hoop direction, $\phi_f$ is a flexural capacity reduction factor (typically taken as 0.9), $e_{cu}$ is the ultimate concrete strain which depends on the level of confinement provided by the composite jacket and can be determined as
\[ \varepsilon_{cu} = 0.004 + \frac{2.8 \rho_j f_{ju} \varepsilon_{ju}}{f_{cc}} \]  

with \( \rho_j \) representing the volumetric jacket reinforcement ratio. In turn, \( \varepsilon_{cu} \) can be obtained from

\[ \varepsilon_{cu} = \Phi_u \cdot c_u \]  

based on the ultimate section curvature \( \Phi_u \) and the corresponding neutral axis depth \( c_u \) which both can be determined from a sectional moment-curvature analysis and directly related to a structural member ductility factor

\[ \mu_f = 1 + 3 \left( \frac{\Phi_u}{\Phi_y} - 1 \right) \frac{L_p}{L} \left( 1 - 0.5 \frac{L_p}{L} \right) \]  

where \( L_p \) represents a semi empirical plastic hinge length [3], \( L \) represents the shear span to the plastic hinge, and \( \Phi_y \) is the section yield curvature [3].

Expressing the flexural jacket thickness Eqn. (3) for hinge confinement in proportional terms similar to the shear Eqn. (2) the required jacket thickness

\[ t_j^e \sim \frac{D}{f_{ju} \varepsilon_{ju}} \times C_c \]  

is proportional to the column dimension \( D \) in the loading direction and inversely proportional to the product of ultimate jacket stress and strain in the hoop direction.

To ensure that column bar buckling in the plastic hinge does not control the flexural failure mode, additional checks on the transverse reinforcement ratio \( \rho_j \) need to be performed [4] particularly for slender columns where \( M/V \cdot D > 4 \) (with \( M \) and \( V \) the maximum column moment and shear). The expression for the required thickness

\[ t_j^b \sim \frac{D}{E_j} \times C_b \]  

The confinement effects provided by circular jackets originate directly from the radial pressure forces generated by the jacket curvature and the tensile hoop strains in the jacket generated by the dilation of the plastic hinge. For rectangular columns an oval jacket should be provided by means of added precast concrete segments with changing radii of curvature in the different loading directions. An equivalent circular column diameter \( D_c \) can be derived from the average of the oval jacket principal radii of curvature, and the jacket thickness calculations can follow those outlined for circular columns using the equivalent column diameter \( D_e \). It should be noted that mid side or unsupported column bars in rectangular columns cannot be stabilized against buckling with thin FRP overlays.
c) Lap splice clamping:
Finally, lap splice clamping requires sufficient lateral pressure $f_l$ onto the splice region to prevent starter bars and the column reinforcement to slip relative to each other. With this simplified failure model, basic shear friction type considerations for a monolithically cast interface can be employed. Furthermore, experimental test results showed that onset of lap splice debonding or relative slippage starts when measured hoop or dilation strain levels are between 1,000 to 2,000 $\mu$ε. At strain levels of 2,000 $\mu$ε debonding of the lap splice was in progress as indicated by a loss in lateral load carrying capacity of the test columns [4].

Limiting dilation strain levels to 1,000 $\mu$ε, the composite jacket thickness to ensure lap splice clamping can be derived as

$$ t_j = 500 \frac{D(f_r - f_h)}{E_j} \quad (9) $$

where $f_h$ represents the horizontal stress level provided by the existing hoop reinforcement in a circular column at a strain of 0.1% and $f_r$ the lateral clamping pressure over the lap splice $L_s$ can be determined as

$$ f_l = \frac{A_s \cdot f_{sy}}{\left[ \frac{p}{2n} + 2(d_b + cc) \right] L_s} \quad (10) $$

where $p$ is the perimeter line in the column cross-section along the lap spliced bar locations, $n$ is the number of spliced bars along $p$, $A_s$ is the area of one main column reinforcing bar, and $cc$ the concrete cover to the main column reinforcement with diameter $d_b$ [4].

Expressing the splice clamping problem again in terms of a proportionality relationship in the form of

$$ t_j^s \sim \frac{D}{E_j} \times C_s \quad (11) $$

indicates that the required composite jacket thickness is inversely proportional to the jacket modulus $E_j$ in the hoop direction and directly proportional to the column diameter.

**BRIDGE SUPERSTRUCTURE STRENGTHENING**

Bridge superstructure capacity deficiencies have been encountered both for increasing traffic loads and permit overload vehicles, as well as for longitudinal seismic resistance, where current seismic design philosophy requires that sufficient superstructure capacity exists to force plastic hinging into the column. Column hinging is desirable since in the column the plastic hinge region can be (1) confined by spiral reinforcement, and (2) inspected and repaired following an earthquake without superstructure closure or traffic interruptions.
On a 1/3 scale proof-test model of the San Francisco Terminal Separation replacement structure design following the 1989 Loma Prieta earthquake, the concept of carbon fabric soffit overlays was explored [3], see figure 7.

Only two layers of each nominally 0.56 mm thick carbon overlays with fibers along the bridge axis were applied. The effectiveness of strain transfer from the soffit reinforcement to the carbon overlay is depicted in figure 8, which shows for the indicated soffit location both, the reinforcement strains before and after the overlay application, as well as the recorded strains in the carbon fabric overlay. Premature joint failure of the cap/column connection prevented a full development of a plastic column hinge and with it a complete verification of the carbon overlay strengthening concept.

However, the strain transfer shown in figure 8, as well as measured carbon overlay strains of over 1500 με at other locations clearly showed the contribution of the advanced composite overlay strengthening to the superstructure capacity. Buckling of the thin carbon overlay in front of the compression toe of the column, installation and quality assurance procedures, as well as design guidelines, still need to be developed prior to field applications.

Figure 7 - Bridge superstructure strengthening with soffit carbon overlay

Figure 8 - Strain comparison of soffit strains with and without carbon overlay
DESIGN DETAILING

Because composites can be easily applied to existing concrete structures either through in-situ overlay installation on the structure itself or through the adhesion of prefabricated panels or strips, there is a tendency to follow with the FRP the contours of the element to be rehabilitated without sufficient thought given to the consequences of such designs. Common examples of the misuse of ease of conformance are shown in figures 9 and 10 wherein composites are used for the shear strengthening of T and I-beams respectively, through FRP overlays on the beam faces. As can be seen in the right hand side of the figure 9, in new conventional concrete T-beam, shear stirrups are never curtailed in the beam region, but are carried over into the slab section to be anchored in the compression zone and to provide the sought after truss mechanism. When FRPs are curtailed in the vertical section below the slab, there is a distinct possibility that the composite will debond or delaminate at the bottom corner and along the vertical edge below the slab, significantly reducing safety and reliability margins. For good detailing, the composite should be continued or anchored into the slab and wrapped around the beam bottom. Also, re-entrant corners, see figure 10, should be avoided.

Composite plates are routinely bonded with adhesives to the bottom of beams in order to strengthen beams as shown in figure 9. This however disregards the fact that the presence of the beam causes very high shear stresses at the end of the composite plate resulting in peel stresses and ultimate failure due to premature debonding of the plate/strips from the concrete.

![Design detailing for shear strengthening of T-beams](image)

Figure 9 – Design detailing for shear strengthening of T-beams

Appropriate detailing should follow the procedure prescribed for internal rebar which would be carried through to the support. FRP strengthening can be developed through partial embedment of the composite in the concrete so as to provide anchorage and stress buildup over its length and in case termination short of the support region is required strict limits on the maximum allowable strength/capacity enhancement should be applied.

The successful application of composites to the bottom surface of beams has resulted in some use of strips on the soffit of slabs with strips being adhesively bonded to the soffit at wide spacings. Again this is poor detailing practice due to shear lag between tension and compression mechanisms and since it allows for punching shear failure in the large unreinforced gaps between
the external composite strips as shown in figure 12. Correct procedure would be to place the strips closer together similar to the placement of internal steel rebar or to use continuous fabric over the entire soffit.

Figure 10 – Design detailing for shear strengthening of I-beams

Figure 11 – Design detailing for the flexural strengthening of beams

Figure 12 – Design detailing for the flexural strengthening of slabs
Composite sheets can be easily applied to large aspect ratio rectangular columns or pier walls. However due to the long length between corners, the composite does not confine the internal concrete structure if just applied to the surface. In order to achieve confinement, the composite wraps need to be constrained on both sides along the length through the use of dowels or bolts that anchor the composite to the pre-existing structure, thereby creating shorter distances which are confined between bolts. This is actually similar to the technique used in conventional construction (figure 13) wherein the transverse reinforcement on the two side faces are tied together through the use of J-hooks.

As a general rule, external FRP retrofit rehabilitation measures on concrete structures should emulate conventional internal reinforcement detailing as much as possible. In cases where this is not practical, strict limitations on the allowable capacity enhancement for the rehabilitation measure should be applied.

![Rehabilitation of Existing RC Structure](image1)

![Design of New RC Structure](image2)

Figure 13 – Design detailing for the seismic retrofit of rectangular columns/pier walls

**SUMMARY AND CONCLUSIONS**

The use of fiber reinforced composites for the seismic retrofitting of concrete bridges requires that appropriate design philosophies, guidelines and detailing be established and that the design be conducted using a methodology that ensures appropriate use of the material. With the appropriate design criteria for FRPs in structural seismic retrofit significant advantages can be derived form the light weight properties of these new materials and their ease of handling and installation. The structural effectiveness of FRPs in retrofitting of in-plane structural walls, out-of-plane unreinforced walls and concrete columns with shear, flexure and lap-splice problems has been demonstrated with large or splice problems has been demonstrated with large or full scale laboratory tests.

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TRANSPORTATION NETWORKS PROGRAM OF THE MID-AMERICA EARTHQUAKE CENTER

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ABSTRACT:

This paper describes the Transportation Networks Program of the MAE Center and some of the research activities that are being pursued to achieve the objectives of this Program. The transportation network includes substantial portions of the nation’s highway and railroad systems, major waterways and shipping facilities on the Mississippi, Missouri, and Ohio Rivers, and airports that serve as hubs for the nation’s airline and air freight operations. Extensive damage to any of these systems has national economic and security ramifications, and would seriously impact emergency response and recovery operations. As a result, the initial coordinated program in the networks track of the MAE Center concentrates on these transportation networks.

KEY WORDS

Transportation, Lifelines, Earthquakes, Vulnerability, Economic Loss, Retrofit, Bridges, Locks

INTRODUCTION

The Mid-America Earthquake (MAE) Center (http://mae.ce.uiuc.edu) assembles researchers from seven core institutions across six states with complementary talents in seismology, geophysics, geotechnical and structural engineering, social science, economics, risk assessment, and urban planning. The Center’s focus is directed at the infrequent, but high consequence, earthquake hazards of the eastern and central United States. All research efforts are coordinated through two parallel programs on facilities and networks that will evolve over the lifetime of the Center. (Education, implementation, outreach, and collaboration activities are also integrated among the core institutions.)
The Mid-America region hosts an extensive transportation network that serves local, regional, and national interests. Highway and railroad traffic across the major rivers is constrained to relatively few long-span crossings, e.g., there are only nine interstate highway crossings of the Mississippi River. The national economic consequences of just a few outages would be extreme. Experience suggests that these crossings are vulnerable; structural damage, loss of bearing support, damage to approach spans, and loss of foundation support due to liquefaction and lateral spreading may occur.

Some of the largest liquefaction-prone deposits in the nation lie along rivers in the central United States. Damage to river, port, and waterfront structures due to liquefaction, lateral spreading, earth pressures, and subsidence of soils, as well as vibrational damage caused to structures, could cripple the transport of goods across the central United States and impact shipping and distribution systems nationally.

Major airports in the region serve as hubs of many passenger airlines and air freight operations. Lambert International Airport in St. Louis is one of only two passenger hub airports in the U.S. that are in areas of moderate to high potential seismic activity. Federal Express operates its major hub airport in Memphis. Airport control towers, fueling equipment, and terminal support systems are subject to vibrational damage and operation of runways may be disrupted by liquefaction, lateral spreading, and subsidence of soils. Loss of service at these facilities could have major repercussions nationally.

The consequences of failure of a transportation system from earthquake shaking could involve: (a) direct loss of life due to collapse or structural failure, (b) indirect loss of life due to an inability to respond to secondary catastrophes, such as fires, and/or provide emergency medical aid, (c) delayed recovery operations, (d) release of hazardous materials and environmental impact, (e) direct loss of property and utility service, (f) economic losses due to interruption of access to a transportation system, and (g) disruption of economic activity across the region and nation. The Transportation Networks Program will not yield substantive results if all of these items are addressed, since funding constraints would limit the depth of coverage. The program will therefore examine in depth the disruption of economic activity (item g), since losses to the national network of transportation are expected to be a significant percentage of the total loss expected when the next major earthquake strikes in Mid-America.

PROGRAM OBJECTIVES

The primary objectives of the Transportation Networks Program are to: (a) assess vulnerabilities and estimate potential economic losses in the national transportation network, and (b) identify effective retrofit methods for reducing these potential losses. The immediate goal of the program is to focus on the Mid-America earthquake problem. However, the methods developed should also apply to regional transportation networks in other parts of the nation and world.
Because the subject of national transportation networks is broad, priorities have been established among the various modes of transportation. Substantial discussion has taken place within the Mid-America Earthquake Center Leadership Team and at an End User Focus Group Meeting in St. Louis on January 8, 1998, that was attended by representatives of the major transportation modes, including highways, railways, waterways, and airways. Based on this meeting, highway and waterway systems were given the top priority over railway and airway systems. Since these modes interconnect, the total national shipping system depends on all four modal networks. The program will address all of them, but the depth of coverage will be greater for highway and waterway modal networks than for railways and airways.

PROGRAM DESCRIPTION

The Transportation Networks Program is outlined in the flow chart in Figure 1. Relations between individual research (rectangular shapes) and implementation (octagon shapes) projects are shown to illustrate the integration and reliance of one project on another in the Transportation Networks Program. Projects representing the end products of the program are shown at the top of the flow chart. The phasing or sequencing of the projects is shown on the timeline for the Transportation Networks Program in Table 1. A more detailed flow chart and timeline are presented in the MAE Center website (http://mae.cc.uiuc.edu).

Projects in structural and geotechnical engineering will combine with those in hazard identification and economics to study the vulnerability of the national transportation network and develop improved methods of rehabilitation to help reduce earthquake losses on a national scale. Nonlinear force-deflection relations for specific nodes of transportation modal networks (e.g. bridges, port facilities, lock and dam structures) will be identified through structural engineering research. These relations will serve as needed input for development of fragility curves that will define damage probabilities for various ground motion intensities. Synthetic ground motions will be developed and serve as the basis for these fragility curves. Fragility curves for the critical nodes of a modal network will then serve as input for development of network fragility models. Network fragility will then be subsequently used to estimate economic losses.
Figure 1. Coordination of Tasks for Transportation Networks Program
Table 1. Sequence of Research Projects in Transportation Networks Program

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*Research Institutions:*
- UIUC - University of Illinois at Urbana-Champaign
- GT - Georgia Tech
- UM - University of Memphis
- MIT - Massachusetts Institute of Technology
- WU - Washington University
- TAMU - Texas A & M University
The impact of retrofitting a particular group of critical nodes on reducing national economic loss will be studied by examining the differences in network loss estimates for systems that have been retrofitted and those that have not been retrofitted. Information on economic loss will also be used with new hazard maps to investigate national network vulnerability. Structural engineering assessment tools and retrofit methods will also be provided to practicing engineers as a direct product of the engineering research. A general overview of the projects in the Transportation Networks Program is given in the following subsections according to technical discipline. A detailed task statement for each project is given on the MAE Center website.

INDIVIDUAL RESEARCH PROJECTS

The following research projects are planned for the Transportation Networks Program. The sequencing of the projects and the investigator(s) conducting the research are shown in Table 1.

Structural Engineering Projects

Research collaboration will be developed with state departments of transportation in a multi-faceted program to improve the safety, performance, and reliability of major bridges. Using ground motions that reflect regional seismicity, the performance of bridges crossing major rivers will be studied with an aim to develop feasible rehabilitation schemes. Response of bridge structural systems will be researched through laboratory and field testing of components, as well as full-scale structures in the field. Computational models will be developed for seismic analysis and evaluation of existing and retrofitted bridges crossing major rivers. This research will involve both highway and railway bridges. The models will be verified with experimental data from laboratory and field tests. The computational models will also be made available to structural engineers in a user-friendly format for assessment purposes. The highway and railway research is being conducted in cooperation with the Illinois Department of Transportation and American Association of Railroads, respectively.

A multi-state inventory of highway and railroad bridges will be compiled from state departments of transportation and railroad companies, and will be entered into a GIS database. Maps showing potentials for bridge failure will be developed for use by policy makers and emergency response officials. The costs and benefits of retrofitting to various performance levels will be explored. Methods will be developed for establishing retrofit priorities considering network reliability theory. In addition, new retrofit techniques will be sought, e.g., the use of innovative materials or systems to cost effectively retrofit bridges in areas of infrequent, but high consequence, earthquakes.

Geotechnical Engineering Projects

The high liquefaction potential in river basins creates a unique vulnerability for bridges and other transportation facilities in Mid-America. A multidisciplinary investigation will
develop new techniques for assessing site-specific liquefaction potential under major river crossings in Mid-America and its impact on the structural response of bridges. Analytical and insitu research will be conducted to investigate the effectiveness of remedial measures for bridge foundations. This research will lead to new and more cost-effective techniques for improving bridge foundations and/or foundation soils, to reduce structural vulnerabilities caused by liquefaction and earthquake shaking.

Locks, dams, and port facilities are integral parts of the navigable waterways in the Central United States. Seismic response of river and port facilities will be examined with computational models that represent interactions between the structural systems, soil, and water. In particular, the seismic performance of concrete lock structures and waterfront facilities will be investigated. Soil-fluid-structure interaction analyses will be used to estimate the response of major lock systems to seismic motion. Nonlinear soil behavior, separation, gapping, and energy dissipation mechanisms will be considered in the analyses. As a result, improved analytical procedures will be developed to assess seismic performance of shipping locks. Seismic performance of a lock system will be investigated for site specific ground motions as determined from the hazards studies. The analytical results will be verified using case histories and/or physical model tests. This research is being conducted in collaboration with the U.S. Army Corps of Engineers.

Methods for assessing seismic performance of waterfront facilities will also be developed. Improved analytical models will be developed for estimating seismic earth pressures and stability under vertical ground motions. Estimated bedrock acceleration histories will be used to predict surface ground motions using multi-dimensional site response analyses. Anticipated seismic performance of lock and waterfront structures will be incorporated into the fragility models to assess vulnerability of the national transportation network. This research is also being conducted in collaboration with the U.S. Army Corps of Engineers.

Airport pavements can be vulnerable to soil liquefaction. The susceptibility of pavements to ground movements will be studied using a collaborative effort between the Mid-America Earthquake Center and the FAA Center for Excellence at the University of Illinois. The synergism of earthquake engineers and pavement researchers will provide a unique blend of talent that is necessary for this problem. In addition, the seismic behavior of structural and non-structural systems in an airport terminal will be studied. This research is being conducted in cooperation with Lambert International Airport in St. Louis and the Memphis International Airport.

Societal Response and Economic Projects

Network theory concerns the reliability with which a given network will operate under various scenarios by considering redundancy in the network and probabilities that a sufficient number of its nodes and links will be operable during any time frame. Analyses of network reliability allows key points of vulnerability to be established, along with probabilistic descriptions of corresponding losses. Evaluations will be made of performance characteristics
for various bridge, pavement, waterway, and airport facilities. These evaluations will be incorporated into network reliability models. Economic loss estimates based on probabilities will be determined, considering the possible damage states to these facilities. Interactions, such as flooding that results from liquefaction of levees or the failure of water retaining structures, will also be considered.

To accomplish this research, GIS-based inventories of critical transportation facilities will be assembled. Structural characteristics will be identified in a detail necessary to guide future research efforts and to support subsequent damage modeling. Basis capacity and traffic flow information critical to estimating the economic impact associated with their damage will also be collected. Descriptive statistics will be produced describing the number, value, size, function, and structural characteristics (e.g. steel span girder bridge built in 1955) of these facilities by location. A regional economic model will be developed based on the GIS inventory that is sensitive to commodities and personal travel on all modes of the transportation system. National economic impact due to loss of key transportation infrastructure components will be evaluated.

Anticipated economic losses will be estimated based on the inventory information, the network fragility curves, and the information on regional economic flows. The economic loss estimates will be caused by a disturbance of the national network as a result of a scenario earthquake event in Mid-America and will be based primarily on interruptions in shipping and business. The vulnerability of the national transportation network to an earthquake event in Mid-America will be studied using the previously described loss estimation model and hazard maps. The fragilities of and losses to various modal networks will be combined in an inter-modal model representing the national transportation network. This task will serve as a capstone project that will summarize the results of the entire Transportation Networks Program, and will be completed in Year 5.

Mitigation measures necessary to assure various levels of operability of these networks will be defined. Cost-benefit analyses will be performed to prioritize actions. The costs of achieving various levels of network reliability will be compared with corresponding reductions in property damage and business interruption losses, to establish optimal levels of rehabilitation. Methodologies developed for assessing business interruption losses will be applied in future programs on industrial and commercial facilities as well as for telecommunication and power networks.

Risk and Reliability Projects

To support the study of social and economic consequences of damaging earthquakes and to develop reliable economic assessment tools for evaluating potential seismic rehabilitation measures, it is necessary to quantify anticipated damage to transportation network nodes and systems in terms of predicted ground motion intensities, via fragility curves. The development of fragility curves will synthesize anticipated ground motion characteristics with anticipated network damage levels. The damage levels will support the development of cost-benefit models for use in the decision-making process. Damage probabilities will be determined for various
intensities of ground motions for specific nodes of transportation systems. Data on nonlinear force-deflection properties of those nodes studied in depth with structural engineering research projects (e.g. bridges and waterway facilities) will be used to develop these fragility curves along with synthetic ground motions developed in accompanying risk and reliability projects. Fragility of national transportation networks will be determined using the previously described node fragility curves. Network fragility will be based on a scenario earthquake occurring at various locations in Mid-America. These network fragility models will be used to quantify economic loss estimates and study the cost-benefit of retrofit.

SUMMARY

The Transportation Networks Program of the Mid-America Earthquake Center has developed an integrated, multi-disciplinary approach to assess the vulnerabilities, and estimate potential economic losses, in the national transportation network due to an earthquake in the eastern and central United States. To accomplish this integrated and multi-disciplinary research, investigators from seven core institutions with expertise in seismology, geophysics, geotechnical and structural engineering, social science, economics, risk assessment, and urban planning have been assembled. The methods developed by the Transportation Networks Program of the Mid-America Earthquake Center will be applicable to transportation networks in other parts of the world.