

NCHRP

REPORT 472

**NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM**

Comprehensive Specification for the Seismic Design of Bridges

TRANSPORTATION RESEARCH BOARD

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 472

Comprehensive Specification for the Seismic Design of Bridges

Prepared by

ATC/MCEER JOINT VENTURE

Redwood City, CA

and

Buffalo, NY

SUBJECT AREAS

Bridges, Other Structures, and Hydraulics and Hydrology

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the American Association of State Highway and Transportation Officials, or the Federal Highway Administration, U.S. Department of Transportation.

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FOREWORD

*By Staff
Transportation Research
Board*

This report contains the findings of a study to develop recommended specifications for the seismic design of highway bridges. The report describes the research effort leading to the recommended specifications and discusses critical conceptual and technical issues. The material in this report will be of immediate interest to bridge-design specification writers and to bridge engineers concerned with the seismic design of highway bridges.

Recent damaging earthquakes in the United States and abroad have demonstrated the earthquake vulnerability of highway bridges that were designed to existing seismic codes. To address this inadequate performance, extensive research programs have been carried out. These programs have advanced the state of the art to the point where a new specification for seismic design is necessary to take advantage of new insight into ground motion and geotechnical effects, improved performance criteria, and more advanced analytical and design methodologies.

The objective of this research was to enhance safety and economy through the development of new load and resistance factor design (LRFD) specifications and commentary for the seismic design of bridges. The research considered design philosophy and performance criteria, seismic loads and site effects, analysis and modeling, and design requirements. The specifications are nationally applicable with provisions for all seismic zones and are intended to be integrated into the *AASHTO LRFD Bridge Design Specifications*.

The research was performed by a joint venture of the Applied Technology Council and the Multidisciplinary Center for Earthquake Engineering Research. The report fully documents the methodology used to develop the recommended specifications. The recommended specifications provided the technical basis for a stand-alone set of provisions prepared by the ATC/MCEER Joint Venture titled "Recommended LRFD Guidelines for the Seismic Design of Highway Bridges." AASHTO will consider these provisions for adoption as a Guide Specification in 2002.

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CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

1.1 BACKGROUND

In the fall of 1998, the AASHTO-sponsored National Cooperative Highway Research Program (NCHRP) initiated a project to develop a new set of seismic design provisions for highway bridges, intended to be compatible with the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 1998). NCHRP Project 12-49, which was conducted by a joint venture of the Applied Technology Council and the Multidisciplinary Center for Earthquake Engineering Research (the ATC/MCEER Joint Venture), had as its primary objectives the development of seismic design provisions that reflected the latest design philosophies and design approaches that would result in highway bridges with a high level of seismic performance.

NCHRP Project 12-49 was intended to reflect experience gained during recent damaging earthquakes and the results of research programs conducted in the United States and elsewhere over the prior 10 years. The primary focus of the project was on the development of design provisions which reflected the latest information regarding design philosophy and performance criteria; seismic hazard representation, seismic-induced loads and displacements, and site effects; advances in analysis and modeling procedures; and requirements for component design and detailing. The new specifications were to be nationally applicable with provisions for all seismic zones, and for all bridge construction types and materials found throughout the United States.

The current provisions contained in the AASHTO *LRFD Bridge Design Specifications* are, for the most part, based on provisions and approaches carried over from Division I-A, "Seismic Design," of the AASHTO *Standard Specifications for Highway Bridges* (AASHTO, 1996). The Division I-A provisions were originally issued by AASHTO as a Guide Specification in 1983 and were subsequently incorporated with little modification into the *Standard Specifications* in 1991. The current LRFD provisions are, therefore, based on seismic hazard, design criteria, and detailing provisions, that are now considered at least 10 years, and in some cases nearly 20 years, out-of-date.

1.2 NCHRP PROJECT STATEMENT AND RESEARCH TASKS

To address this concern, the AASHTO-sponsored NCHRP developed a Project Statement to conduct NCHRP Project

12-49, which was issued in late 1997. The Project Statement read as follows:

"Recent damaging earthquakes in California, Japan, the Philippines, and Costa Rica have demonstrated the earthquake vulnerability of highway bridges that were designed to existing seismic codes. To address this inadequate performance, extensive research programs have been carried out. These programs have advanced the state of the art to the point where a new specification for seismic design is necessary to take advantage of new insight into ground motion and geotechnical effects, improved performance criteria, and more advanced analytical and design methodologies.

Some of the new procedures from these research programs represent major departures from present practice, and simple revisions to existing AASHTO specifications are not feasible. In addition, new provisions are under development by the Federal Emergency Management Agency (and others) for buildings. New seismic design specifications are required that retain the best features of the present specifications while embracing the results of recent research. The development of the next generation of seismic provisions for bridges will keep the recently adopted AASHTO LRFD Bridge Design Specifications in step with the building community and the state of the art.

The project to be undertaken here is to develop new specifications for the seismic design of bridges, considering all aspects of the design process. These aspects include the following: (1) design philosophy and performance criteria, (2) seismic loads and site effects, (3) analysis and modeling, and (4) design requirements. The new specification must be nationally applicable with provisions for all seismic zones. It is expected that the results of research currently in progress and recently completed will be the principal resource for this project.

The objective of this research is to enhance safety and economy through the development of new LRFD specifications and commentary for the seismic design of bridges. These specifications will be recommended for consideration by the AASHTO Highway Subcommittee on Bridges and Structures and shall reflect experience gained during recent damaging earthquakes and the results of research programs conducted in the United States and elsewhere. This work will focus on designing new bridges rather than on retrofitting existing ones."

The research tasks conducted under NCHRP Project 12-49 included the following:

Task 1. Review and interpret relevant practice, performance data, research findings, and other information related

to seismic design of bridges. Identify any gaps in knowledge. This information shall be assembled from technical literature and from unpublished experiences of engineers and bridge owners.

- Task 2. Based on the findings of Task 1, develop conceptual LRFD design criteria for structural systems and components. Design criteria should address, but not be limited to, the following technical areas: (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.
- Task 3. Based on the results of Tasks 1 and 2, identify and prioritize specification areas that need research and modification beyond what will be accomplished in this project.
- Task 4. Submit an Interim Report, within 9 months of the contract start, that documents the results of Tasks 1, 2, and 3, includes a proposed format for the specification and commentary, and proposes a detailed work plan for the remainder of the project. Following project panel review of the Interim Report, meet with the panel to discuss the Interim Report and the remaining tasks. NCHRP approval of the Interim Report will be required before proceeding.
- Task 5. On approval of the work plan, the following subtasks will be performed: (a) conduct parametric studies to refine the proposed LRFD design criteria and (b) develop a draft of the recommended specifications and commentary (draft 1); The results of subtasks (a), and (b) shall be submitted for panel review.
- Task 6. (a) Conduct liquefaction case studies to determine the impact of various return periods on the seismic design provisions; (b) Revise the draft recommended specifications and commentary; (c) Prepare design examples demonstrating the application of the proposed specifications; and (d) Prepare an analysis of impacts and benefits of implementation of the proposed specifications. The results of this task shall be submitted for panel review.

Task 7. Submit a final report describing the entire research effort.

1.3 PROJECT ACTIVITIES AND DELIVERABLES

NCHRP Project 12-49 developed a preliminary set of comprehensive specification provisions and commentary intended for incorporation into the AASHTO LRFD specifications. In so doing, three drafts of the Project 12-49 LRFD-based specifications and commentary were prepared and reviewed by an outside expert advisory panel convened by ATC (the ATC Project Engineering Panel), NCHRP Project Panel C12-49, and the AASHTO Highway Subcommittee on Bridges and Structures' seismic design technical committee (T-3), which was chaired by James Roberts of Caltrans.

A number of other important reports were also prepared and submitted to the NCHRP during the course of the project. These were based on the following project activities:

- A survey of state bridge design agencies, bridge engineering consultants, and researchers was conducted to assess current seismic design and analysis approaches and practices, needs, and concerns.
- Research leading up to the submission of the project Interim Report, which was submitted in March 1999. The efforts under Tasks 1 through 4 provided a review of current and past research, data, and a review and assessment of current practices in seismic design of bridges and other structures; identification and discussion of major issues that needed to be considered in the development of seismic design criteria; a work plan for the remainder of the project; and a list of specific research needs for criteria and issues which could not be adequately addressed in this project with current knowledge.
- Research to identify and document the major issues that were required to be considered during development of the LRFD-based seismic design criteria. This addressed all of the major issues related to the development of the conceptual LRFD design criteria, which were then recommended by the ATC/MCEER Joint Venture Project Team to the NCHRP Project Panel and AASHTO T-3 committee for consideration during each draft of the specification development effort.
- A detailed parameter study was conducted to determine the impact of the proposed provisions, to benchmark the results of the recommended provisions against those produced by existing seismic design provisions, and to ascertain the effects of key parameters and fine-tune them relative to good engineering practice.
- A workshop supported and organized by the FHWA and MCEER in support of the work on NCHRP Project 12-49 on geotechnical performance requirements

and provisions was conducted in September 1999. The MCEER-sponsored workshop brought together researchers, practicing geotechnical engineers, and structural engineers, to address issues related to foundation performance requirements and tolerable displacements that structures and foundations can adequately accommodate during seismic-induced ground movements.

- A second workshop supported by the FHWA and MCEER on seismic provisions for the design of steel bridges was conducted in July 2000. The MCEER-sponsored workshop brought together researchers, practicing engineers, and steel industry representatives in an attempt to identify and formulate provisions to advance knowledge in the seismic design and performance of steel highway bridges.
- A major study on liquefaction hazard assessment and impacts was conducted to assess the effects of liquefaction and associated hazards, including lateral spreading and ground flow failures. The study investigated liquefaction hazard implications for the design of bridges using two real bridges and sites in relatively high regions of seismicity: one in the Western United States in Washington State, and the second in the Central United States in Missouri.
- Two detailed bridge design examples demonstrating the application of the recommended design provisions were developed.

Throughout the course of the project, members of the Project Team interacted regularly with the ATC Project Engineering Panel, the NCHRP Project Panel, members of the AASHTO Bridge Committee's seismic design technical committee (T-3), and with many other researchers and state, federal, and practicing engineers. Project Team members made annual presentations during the meetings of the AASHTO T-3 committee and at various other meetings and venues.

1.4 RESEARCH APPROACH

To the extent possible, NCHRP Project 12-49 was intended to incorporate then-current research results and engineering practice in the development of the new LRFD-based seismic design specifications. However, during the course of the project, it became apparent that knowledge gaps needed to be addressed before uniformly applicable provisions could be developed that recognize the variations in seismic hazard, local soil and bedrock conditions, and design and construction practices found throughout the United States. Therefore, a number of special studies were conducted under this project and through the assistance of the FHWA-sponsored MCEER Highway Project.

The initial efforts under this project were primarily related to information gathering. Specification provisions and philoso-

phies used in current AASHTO documents and by various States were obtained and assessed, as were specifications from a number of foreign countries, including those of Japan, New Zealand, as well as the new Eurocodes. These materials were augmented by a detailed review of recently completed and ongoing research projects and programs, sponsored by State transportation agencies, FHWA, and others. Eventually, a series of recommendations were developed by the Project Team regarding the overall philosophy and intent that the new specifications should address, and these were widely discussed with the ATC Project Engineering Panel, NCHRP Project Panel, and AASHTO T-3 seismic design technical committee.

Once there was general agreement on the underlying philosophy on which the new provisions should be based, the Project Team developed three distinct drafts of the LRFD-based seismic design provisions. Each draft was circulated for review by the ATC Project Engineering Panel, NCHRP Project Panel, and AASHTO T-3 committee. Drafts 1 and 2 were also discussed in detail during meetings between the Project Team, NCHRP Project Panel, and AASHTO T-3. Where required, special studies were conducted and specialty workshops were held to develop new provisions or resolve questions or problems as they arose.

1.5 REPORT ORGANIZATION

This report provides a summary of the work conducted under NCHRP Project 12-49. The specific LRFD-based design provisions were submitted to the NCHRP in separate reports, and are not reproduced herein. Other major reports and project deliverables were also prepared and submitted, as described above, and these too are not reproduced herein.

Chapter 1 of this report (this chapter) provides an overview of the project background and objectives; how the project was organized; summary of results from the project; and a list of project participants.

Chapter 2 contains a description of the philosophy used in developing the new seismic design provisions and a discussion of new and important concepts and design approach changes made in the proposed LRFD specifications.

Chapter 3 provides a review and discussion of some of the major technical issues addressed during the course of NCHRP Project 12-49. Section 3.1 discusses issues involved in selection of the seismic hazard (representation and design levels) used in the LRFD provisions; Section 3.2 presents a summary of the liquefaction assessment study and bridge assessment case studies that were conducted; and Section 3.3 presents an assessment of the impacts (engineering impacts, potential costs, and benefits) that can be expected if the provisions developed under the project are implemented.

Chapter 4 presents a summary of this report and a suggested plan to implement the proposed LRFD seismic design provisions.

The results of this project represent a significant change in philosophy, approach, and methodologies from those currently used for highway bridge analysis and design for seismic performance. It is, therefore, possible that the AASHTO Highway Subcommittee on Bridges and Structures may prefer, in the short term, to adopt the recommended provisions

as a Guide Specification. However, regardless of whether the proposed LRFD provisions are incorporated directly into the AASHTO *LRFD Bridge Design Specifications* or as a stand-alone Guide Specification, there will be a need to educate and train bridge designers and engineers on the use of the new provisions and approaches.

CHAPTER 2

FINDINGS

2.1 BASIC PHILOSOPHY EMPLOYED IN DEVELOPING THE PROPOSED SEISMIC DESIGN PROVISIONS

In keeping with current seismic design approaches employed both nationally and internationally, the development of the LRFD specifications was predicated on the following basic philosophy:

- Loss of life and serious injuries due to unacceptable bridge performance should, to the extent possible and economically feasible, be minimized.
- Bridges may suffer damage and may need to be replaced, but they should have low probabilities of collapse due to earthquake motions.
- Full functionality for essential bridges (so-called “critical lifeline” structures) should be maintained even after a major earthquake.
- Upper-level-event ground motions used in design should have a low probability of being exceeded during the approximately 75-year design life of the bridge.
- The provisions should be applicable to all regions of the United States.
- The designer is encouraged to consider and employ new and ingenious design approaches and details and should not be restricted to approaches contained solely within the provisions.

2.2 NEW CONCEPTS AND MAJOR CHANGES IN THE PROPOSED SEISMIC DESIGN PROVISIONS

In comparison to the current AASHTO *Standard Specifications for Highway Bridges* (AASHTO, 1996) and the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 1998), the proposed NCHRP 12-49 specifications contain a number of new concepts and additions, as well as some major modifications to existing provisions. These are summarized in the following sections.

2.2.1 1996 USGS Maps

The national earthquake ground motion map used in the existing AASHTO provisions is a probabilistic map of peak

ground acceleration (PGA) on rock which was developed by the U.S. Geological Survey (USGS) (Algermisson et al., 1990). The map provides contours of PGA for a probability of exceedance (PE) of 10 percent in 50 years, which is approximately equal to a 15 percent PE in the 75-year design life predicated for a typical highway bridge.

In 1993, the USGS embarked on a major project to prepare updated national earthquake ground motion maps. The result of that project was a set of probabilistic maps published in 1996 for the conterminous United States (Frankel, et al., 1996) and subsequently for Alaska and Hawaii, that cover several rock ground motion parameters and three different probability levels or return periods. The maps are available from the USGS as large fold-out paper maps, smaller maps that can be obtained via the Internet, and as digitized values obtained from the Internet or a CD-ROM published by USGS.

Parameters of rock ground motions that have been mapped by the USGS include peak ground acceleration (PGA) and elastic response spectral accelerations for periods of vibration of 0.2, 0.3, and 1.0 sec. Contour maps for these parameters have been prepared for three different probabilities of exceedance (PE): 10 percent PE in 50 years, 5 percent PE in 50 years, and 2 percent PE in 50 years (which is approximately equal to 3 percent PE in 75 years). In addition to these contour maps, the ground motion values at any specified latitude and longitude can be obtained via the Internet for these three probability levels for PGA, and spectral accelerations for periods of vibration of 0.2, 0.3, and 1.0 sec. The CD-ROM published by the USGS also provides spectral accelerations at additional periods of 0.1, 0.5, and 2.0 sec. In addition, the CD-ROM contains both the PGA and spectral acceleration values at three probability levels as well as the complete hazard curves on which they are based.¹ Therefore, the ground motion values for all of these ground motion parameters can be obtained for any return period or probability of exceedance from the hazard curves.

These maps form the rock ground motion basis for seismic design using these proposed LRFD provisions. Upper bound limits on ground motions obtained by deterministic methods, as described in the following section, have been applied in

¹ Hazard curves present relationships between the amplitude of a ground motion parameter and its annual frequency of exceedance for specified locations.

the proposed provisions, in order to limit probabilistic ground motions in the western United States.

2.2.2 Design Earthquakes and Performance Objectives

The current AASHTO provisions have three implied performance objectives for small, moderate, and large earthquakes with detailed design provisions for a 10 percent PE in 50-year event (which is approximately equal to a 15 percent PE in 75-year event) to achieve its stated performance objectives. The proposed LRFD provisions provide more definitive performance objectives and damage states for two design earthquakes with explicit design checks to ensure that the performance objectives are met.

The upper-level event, termed the “rare” or Maximum Considered Earthquake (MCE), describes ground motions that, for most locations, are defined probabilistically and have a probability of exceedance of 3 percent in 75 years. However, for locations close to highly active faults, the mapped MCE ground motions are deterministically bounded so that the levels of ground motions do not become unreasonably high. Deterministic bounds on the ground motions are calculated by assuming the occurrence of maximum magnitude earthquakes on the highly active faults. These are equal to 150 percent of the median ground motions for the maximum magnitude earthquake, but not less than 1.5 g for the short-period spectral acceleration plateau and 0.6 g for 1.0-sec spectra acceleration. On the current MCE maps, deterministic bounds are applied in high-seismicity portions of California, in local areas along the California-Nevada border, along coastal Oregon and Washington State, and in high-seismicity portions of Alaska and Hawaii. In areas where deterministic bounds are imposed, the result is design ground motions that are lower than ground motions for 3 percent PE in 75 years. The MCE governs the limits on the inelastic deformation in the substructures and the design displacements for the support of the superstructure.

The lower level design event, termed the “expected” earthquake, has ground motions corresponding to 50 percent PE in 75 years. This event ensures that essentially elastic response is achieved in the bridge substructure for the more frequent or expected earthquake. This design level is similar to the 100-year flood and has similar performance objectives. An explicit check on the strength capacity of the bridge substructure is required. Parameter studies performed as part of the development of the provisions show that the lower level event will only impact the strength of columns in parts of the western United States.

2.2.3 Design Incentives

The proposed LRFD design provisions contain an incentive from a design and construction perspective for perform-

ing a more sophisticated “pushover analysis.” The response-modification factor (R-Factor) increases approximately 50 percent when a pushover analysis is performed, primarily because the analysis results will provide a greater understanding of the demands on the seismic resisting elements. The analysis results are assessed using additional plastic rotation limits on the deformation of the substructure elements to ensure adequate performance.

2.2.4 New Soil Site Factors

The site classes and site factors incorporated in these new provisions were originally recommended at a site response workshop in 1992 (Martin, ed., 1994; Martin and Dobry, 1994; Rinne, 1994; Dobry et al., 2000). They were subsequently adopted in the seismic design criteria of Caltrans (1999), the 1994 and 1997 NEHRP Provisions (BSSC, 1995; BSSC, 1998), the 1997 Uniform Building Code (ICBO, 1997), and the 2000 International Building Code (ICC, 2000). This is one of the most significant changes with regard to its impact on the level of seismic design forces. It should be noted that the recommended site factors affect both the peak (i.e., flat) portion of the response spectra as well as the long period ($1/T$) portion of the spectra. The soil site factors increase with decreasing accelerations due to the nonlinear response effects of soils. Soils are more linear in their response to lower acceleration events and display more nonlinear response as the acceleration levels increase. The effects of soil nonlinearity are also more significant for soft soils than for stiff soils.

2.2.5 New Spectral Shapes

The long-period portion of the current AASHTO acceleration response spectrum is governed by a spectrum shape that decays as $1/T^{2/3}$. During the original development of this decay function, there was considerable massaging of the factors that affect the long period portion of the spectra in order to produce a level of approximately 50 percent conservatism in the design spectra when compared with the ground motion spectra beyond a 1-sec period. The proposed LRFD provisions remove this arbitrary conservatism and provide a spectral shape that decays as $1/T$.

2.2.6 Earthquake Resisting Systems and Earthquake Resisting Elements

The proposed LRFD provisions provide a mechanism to permit the use of some seismic resisting systems and elements (termed earthquake resisting systems (ERS) and earthquake resisting elements (ERE) that are not permitted in current AASHTO provisions. Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. To this end, the identification of the lateral-force-resisting concept and the selection of the necessary elements to facilitate the

concept should be accomplished in the conceptual design (also known as the type, selection, and layout—TS&L) phase of the project. Seismic performance is typically better in systems with regular configurations and evenly distributed stiffness and strength. Thus, typical geometric configuration constraints, such as skew, unequal pier heights, and horizontally curved bridges, conflict, to some degree, with these seismic design goals. For this reason, it is advisable to resolve potential conflicts between configuration and seismic performance early in the design effort.

The classification of ERS and ERE into three categories: permissible, permissible with owner's approval, and not recommended, is done to trigger due consideration of seismic performance that leads to the most desirable outcome—that is, seismic performance that ensures, wherever possible, post-earthquake serviceability. It is not the objective of the specification to discourage the use of systems that require owner approval. Instead, such systems may be used, but additional design effort and consensus regarding the amount and type of damage acceptance or system performance will be necessary between the designer and owner in order to implement such systems.

2.2.7 “No Seismic Demand Analysis” Design Concept

The “no seismic demand analysis” design procedure is an important new addition to the proposed LRFD provisions. It applies to “regular” bridges in low-to-moderate seismic hazard areas. The bridge is designed for all non-seismic loads and does not require a seismic demand analysis. Capacity design procedures are used to determine detailing requirements in columns and the forces in columns-to-footing and column-to-superstructure connections. There are no explicit seismic design requirements for abutments, except that integral abutments are to be designed for passive pressures.

2.2.8 Capacity Spectrum Design Procedure

The capacity spectrum design method is a new addition to the provisions and is conceptually similar to the new Caltrans' displacement design method. The primary difference is that the capacity spectrum design procedure begins with the non-seismic capacity of the columns and then assesses the adequacy of the resulting displacements. At this time, the capacity spectrum method may be used for “very regular” bridges that respond essentially as single-degree-of-freedom systems, although future research should expand the range of applicability. The capacity spectrum approach uses the elastic response spectrum for the site, which is then reduced to account for the dissipation of energy in the earthquake resisting elements. The advantage of the approach is that the period of vibration does not need to be calculated, and the designer sees an explicit trade-off between design forces and resulting

displacements. The method is also quite useful as a preliminary design tool for bridges that may not satisfy the current regularity limitations of the approach.

2.2.9 Displacement Capacity Verification (“Pushover”) Analysis

The “pushover” method of analysis has seen increasing use since the early 1990s and is widely employed in the building industry and by some transportation agencies both for seismic design and retrofit. This analysis method provides the designer with additional information on the expected deformation demands of columns and foundations and, therefore, with a greater understanding of the expected performance of the bridge.

The method is employed in two different ways in these recommended LRFD provisions. First, it provides a mechanism under which the highest R-Factor for preliminary design of a column can be justified, because there are additional limits on the column plastic rotations that the results of the pushover analysis must satisfy. Second, it provides a mechanism to allow incorporation of earthquake resisting elements (ERE) that require an owner's approval. There is a trade-off associated with this, in that a more sophisticated analysis is needed so that the expected deformations in critical elements can be adequately assessed. The ERE can then be used, provided that the appropriate plastic deformation limits are met.

2.2.10 Foundation Design

In the area of foundation design, the proposed LRFD provisions are essentially an update of the existing AASHTO LRFD provisions, incorporating both current practice and recent research results. The primary changes in the proposed LRFD provisions include the addition of specific guidance for the development of spring constants for spread footings and deep foundations (i.e., driven piles and drilled shafts), as well as approaches for defining the capacity of the foundation system when exposed to overturning moments. The capacity provisions specifically address issues such as uplift and plunging (i.e., yield) limits within the foundation. Procedures for including the pile cap in the lateral capacity and displacement evaluation are also provided. The implications of liquefaction of the soil, either below or around the foundation system, is also described.

2.2.11 Abutment Design

The proposed LRFD provisions incorporate a significant body of research that has been conducted on bridge abutments over the past 10 years. Current design practice varies considerably on the use of the abutments as part of the ERS. Some states design their bridges so that the substructures are

capable of resisting all of the seismic loads without any contribution from the abutment. Other states use the abutment as a key component of the ERS. Both design approaches are permitted in these provisions.

The abutments can be designed as part of the ERS and, in so doing, become additional components for dissipating the earthquake energy. In the longitudinal direction, the abutment may be designed to resist forces elastically utilizing the passive pressure of the backfill. However, in some cases, passive pressure at the abutment will be exceeded, resulting in larger soil movements in the abutment backfill. This will require a more refined analysis to determine the amount of expected movement, and procedures are provided in the provisions to incorporate this nonlinear behavior. In the transverse direction, the abutment is generally designed to resist loads elastically. The proposed LRFD provisions therefore recognize that the abutment can be an important part of the ERS and considerable attention is given to abutment contributions on the global response of the bridge. It should be noted, however, that for the abutments to be able to effectively contribute to the ERS, a continuous superstructure is required.

2.2.12 Liquefaction Hazard Assessment and Design

Liquefaction has been one of the most significant causes of damage to bridge structures during past earthquakes. Most of the damage has been related to lateral movement of soil at the bridge abutments. However, cases involving the loss of lateral and vertical bearing support of foundations at bridge piers have also occurred. Considerable research has been conducted over the past 10 years in the areas of liquefaction potential and effects, and much of this information has been incorporated in the proposed LRFD provisions. For example, the new provisions outline procedures for estimating liquefaction potential using methods developed in 1997 as part of a national workshop on the evaluation of liquefaction (Youd and Idriss, 1997). Procedures for quantifying the consequences of liquefaction, such as lateral flow or spreading of approach fills and settlement of liquefied soils, are also given. The provisions also provide specific reference to methods for treating deep foundations extending through soils that are spreading or flowing laterally as a result of liquefaction.

For sites with mean earthquake magnitudes contributing to the seismic hazard less than 6.0, the effects of liquefaction on dynamic response can be neglected. When liquefaction occurs during an earthquake, vibration and permanent ground movement will occur simultaneously. The recommended methodology in the LRFD provisions is to consider the two effects independently; i.e., in a decoupled manner.

If lateral flow occurs, significant movement of the abutment and foundation system can result and this can be a difficult problem to mitigate. The range of design options include (1) designing the piles for the flow forces and (2) acceptance

of the predicted lateral flow movements, provided that inelastic hinge rotations in the piles remain within a specified limit. The acceptance of plastic hinging in the piles is a deviation from past provisions in that damage to piles can now be accepted when lateral flow occurs; however, the designer and owner are effectively acknowledging that the bridge may need to be replaced if this were to occur when this option is selected.

Structural or soil mitigation measures to minimize the amount of movement to meet higher performance objectives are also described in the new provisions. Due to concerns regarding the difficulty and cost impacts of liquefaction assessment and mitigation when coupled with the higher level design events, two detailed case studies on the application of the recommended design methods for both liquefaction and lateral flow design were performed. The case studies demonstrated that, for some soil profiles, application of the new provisions would not be significantly more costly than the application of the current AASHTO provisions. These case studies are summarized in Chapter 3.

2.2.13 Steel Design Requirements

The existing AASHTO specifications do not have explicit seismic requirements for steel bridges, except for the provision that a continuous load path be identified and included in the design (for strength). Consequently, a comprehensive set of special detailing requirements for steel components, which are expected to yield and dissipate energy in a stable and ductile manner during earthquakes, were developed. These include provisions for ductile moment-resisting frame substructures, concentrically braced frame substructures, and end-diaphragms for steel girder and truss superstructures. These provisions now provide a moderate amount of guidance for the seismic design of steel bridges.

2.2.14 Concrete Design Requirements

Although there are no major additions to the concrete provisions contained in the proposed LRFD provisions, there are important updates for key design parameters. These are based on a major body of research that was conducted over the past decade. The minimum amount of longitudinal steel was reduced from the current AASHTO requirements of 1.0 percent to 0.8 percent; this is expected to result in significant material cost savings when used with capacity design procedures. An implicit shear equation was also added, for cases where no seismic demand has been determined. Modifications to the explicit shear equation and confinement requirements were also provided, and a provision to address global buckling of columns and beam-columns was added. Plastic rotation limits were also added in order to conduct pushover analyses.

2.2.15 Superstructure Design Requirements

Detailed design requirements for bridge superstructures are not included in the current AASHTO seismic design provisions, other than those required by the generic load path requirement. In reviewing current bridge seismic design practices, it was noted that there are wide discrepancies in the application of this load path requirement. Therefore, for the higher hazard levels, explicit load path design requirements have been added and discussed in the proposed LRFD provisions.

2.2.16 Bearing Design Requirements

One of the significant issues that arose during development of the steel provisions was the critical importance of bearings as part of the overall bridge load path. The 1995 Kobe, Japan earthquake and other more recent earthquakes clearly showed the very poor performance of some bearing types and the disastrous consequence that a bearing failure can have on the overall performance of the bridge. To address this concern, the proposed LRFD provisions require the designer to select from one of three design options for bridge bearings: (1) testing of the bearings, (2) ensuring restraint of the bearings, or (3) the use of a design concept that permits the girders to land and slide on a flat surface available within the bridge system if the bearings fail.

2.2.17 Seismic Isolation Provisions

The *Guide Specifications for Seismic Isolation Design* were first adopted by AASHTO in 1991; they were significantly revised and reissued in 1999 (AASHTO, 1999). As directed

by the NCHRP Project Panel, the 1999 *Guide Specifications* provisions were incorporated into the recommended LRFD provisions. The net result of this is that it will require a new chapter 15 in the AASHTO LRFD specifications.

2.3 DESIGN EFFORT AND COST IMPLICATIONS OF THE NEW PROVISIONS

A parameter study was performed as part of the NCHRP 12-49 project and the results are summarized in Chapter 3. Based on a study that looked at 2,400 different column configurations and five different seismic hazard locations (i.e., five different cities located throughout the United States), it was shown that the net effect on the cost of a column and spread footing system is, on average, 2 percent less than that designed with the current Division I-A provisions for multi-column bents and 16 percent less than Division I-A provisions for single-column bents. These cost comparisons are based on the use of the more refined method for calculating overstrength factors provided in the proposed LRFD provisions and the assumption that the P- Δ assessment requirements included in the LRFD provisions are included in the Division I-A designs.

One factor that caused a cost increase in some of the lower seismic hazard areas and for some column configurations was the short-period modifier. Since this provision is considered to be an important part of any new code, the cumulative effect of all other changes (including the use of the MCE event; new soil site factors, spectral shape, R-Factors, and phi-factors; and the use of cracked section properties for analysis) would have resulted in even lower average costs had the short-period modifier been a part of current AASHTO LRFD and Division I-A provisions.

CHAPTER 3

INTERPRETATION, APPRAISAL, AND APPLICATION

3.1 SEISMIC HAZARD REPRESENTATION AND SELECTION OF DESIGN EARTHQUAKES

This section describes the design earthquakes and associated ground motions that have been adopted for the proposed revisions to the AASHTO *LRFD Bridge Seismic Design Specifications*. For applicability to most bridges, the objectives in selecting design earthquakes and developing the design provisions of the specifications are to preserve life safety and prevent bridge collapse during rare earthquakes, and to provide immediate post-earthquake serviceability, following an inspection of the bridge, with minimal damage during expected earthquakes. For bridges of special importance as determined by the bridge owner, performance objectives may be higher than those specified in the proposed provisions.

Additional discussion and analyses of earthquake ground motion maps, site factors, and response spectrum construction procedures may be found in ATC/MCEER (1999a, 1999b).

3.1.1 Current AASHTO Map (1988 USGS Map)

The national earthquake ground motion map contained in both Division I-A of the current AASHTO *Standard Specifications for Highway Bridges* and the AASHTO *LRFD Bridge Seismic Design Specifications* is a probabilistic map of peak ground acceleration (PGA) on rock, which was developed by the U.S. Geological Survey (USGS) in 1988. This map provides contours of PGA for a probability of exceedence (PE) of 10 percent in 50 years, which is equivalent to a 500-year earthquake return period. The PGA map is used with rules contained in the AASHTO specifications for obtaining seismic response coefficients or response spectral accelerations.

3.1.2 New USGS Maps

In 1993, the USGS embarked on a major project to prepare updated national earthquake ground motion maps. In California, the mapping project was a joint effort between the USGS and the California Division of Mines and Geology (CDMG). The result of that project was a set of probabilistic maps published in 1996 for the conterminous United States and, subsequently, for Alaska and Hawaii. These maps cover several rock ground motion parameters and three different earthquake

probability levels or return periods (Frankel et al., 1996, 1997a, 1997b, 1997c, 2000; Frankel and Leyendecker, 2000; Klein et al., 1999; Peterson et al., 1996; Wesson et al., 1999a, 1999b). The maps are available as large-scale printed paper maps, small-scale paper maps that can be downloaded from the Internet, and as digitized values that can be obtained from the Internet or on a CD-ROM published by the USGS (Frankel and Leyendecker, 2000).

Parameters of rock ground motions that were mapped by the USGS include PGA and response spectral accelerations, for periods of vibration of 0.2, 0.3, and 1.0 sec. Contour maps for these parameters were prepared for three different PE levels: 10 percent PE in 50 years, 5 percent PE in 50 years, and 2 percent PE in 50 years (which is approximately equal to 3 percent PE in 75 years). These correspond, respectively, to approximate ground motion return periods of 500 years, 1,000 years, and 2,500 years. In addition to these contour maps, ground motion values at locations specified by latitude and longitude can be obtained via the Internet for these same three probability levels for PGA, and for spectral accelerations for periods of vibration of 0.2, 0.3, and 1.0 sec. The USGS CD-ROM provides these and spectral accelerations at additional periods of 0.1, 0.5, and 2.0 sec. The CD-ROM also provides PGA and spectral acceleration values at three probability levels, as well as the complete hazard curves (i.e., relationships between the amplitude of a ground motion parameter and its annual frequency of exceedence² for specified latitudes and longitudes). Therefore, ground motion values can be obtained or derived for any return period or probability of exceedence from the hazard curves on the CD-ROM.

The 1996 USGS national ground motion mapping effort incorporated inputs for seismic source models and ground motion attenuation models that represent major improvements over the models used to develop the current AASHTO maps. Among these improvements are the following.

- Much more extensive inclusion of identified discrete active faults and geologic slip rate data. Approximately 500 faults were incorporated in the mapping. Geologic slip rates for these faults were used to determine earthquake recurrence rates for the faults.

² Annual frequency of exceedence is the reciprocal of return period.

- Improved and updated seismicity catalogs were used to determine earthquake recurrence rates for seismic sources not identified as discrete faults. In the central and eastern United States (CEUS), these catalogs were based on updated assessments of the magnitudes that had been determined for older earthquakes (originally characterized by their Modified Mercalli Intensity). These assessments had the effect of reducing the estimated rate of occurrence for larger earthquakes in the CEUS (equal to or greater than approximately magnitude 5.0).
- In the Pacific Northwest (Washington, Oregon, and northwest California), the Cascadia subduction zone seismic source was explicitly included. Geologic and paleoseismic data were used to characterize the recurrence rate of very large earthquakes (potentially magnitudes 8.0 to 9.0) occurring in the coastal and offshore regions of the Pacific Northwest.
- Geologic and paleoseismic data were used to characterize the recurrence rate of large earthquakes occurring in the New Madrid seismic zone (in the vicinity of New Madrid, Missouri) and the Charleston seismic zone (in the vicinity of Charleston, South Carolina).
- Updated and recently developed ground motion attenuation relationships were obtained and applied. These relationships incorporated the developing knowledge of differences in ground motion attenuation relationships in different regions and tectonic environments of the United States. As a result, different attenuation relationships were used in the CEUS, shallow-crustal faulting regions of the western United States (WUS), and subduction zone regions of the Pacific Northwest and Alaska.

The 1996 probabilistic maps developed by the USGS have been widely accepted by the engineering and earth sciences communities as providing a greatly improved scientific portrayal of probabilistic ground motions in the United States, when compared with earlier maps. These maps were assessed for possible use in the seismic design of bridges and other highway facilities by the 1997 workshop sponsored by FHWA and MCEER on the national characterization of seismic ground motion for new and existing highway facilities (Friedland, et al., 1997). The workshop concluded that “. . . these new maps represent a major step forward in the characterization of national seismic ground motion. The maps are in substantially better agreement with current scientific understanding of seismic sources and ground motion attenuation throughout the United States than are the current AASHTO maps. . . . the new USGS maps should provide the basis for a new national seismic hazard portrayal for highway facilities . . .”

The USGS has a systematic process for periodically updating the seismic hazard maps to reflect continuing advances in knowledge of earthquake sources and ground motions. Therefore organizations using these maps (or maps adapted from

the USGS maps as described below) have the opportunity to update the maps in their seismic criteria documents as appropriate.

3.1.3 National Earthquake Hazard Reduction Program (NEHRP) Maximum Considered Earthquake Maps

The federally sponsored Building Seismic Safety Council (BSSC) adopted a modified version of the 1996 USGS Maps for 2 percent PE in 50 years to define the recommended ground motion basis for the seismic design of buildings in the 2000 NEHRP Provisions (BSSC, 2001; Leyendecker, et al., 2000a, 2000b). These maps are termed the Maximum Considered Earthquake (MCE) maps and are presented in Figures 3.10.2.1-1(a) through 3.10.2.1-1(l) of the proposed LRFD specification provisions. The maps are for 0.2-sec and 1.0-sec response spectral accelerations.

The 1997 NEHRP MCE maps are identical to the 1996 USGS maps for 2 percent PE in 50 years (return period of approximately 2,500 years), except that in areas close to highly active faults, “deterministic bounds” were placed on the ground motions with the intent that ground motions are limited to levels calculated assuming the occurrence of maximum magnitude earthquakes on the faults. The deterministic bounds are defined as 150 percent of the median ground motions calculated using appropriate ground motion attenuation relationships (the same relationships as used in the USGS probabilistic mapping) assuming the occurrence of maximum magnitude earthquakes on the faults, but not less than 1.5 *g* for 0.2-sec spectral acceleration and 0.6 *g* for 1.0-sec spectral acceleration. Increasing the median ground motions by 150 percent results in ground motions that are approximately at a median plus one-standard-deviation level (actually somewhat lower, in general, because the ratio of median plus one-standard-deviation ground motions to median ground motions usually exceeds 1.5). The deterministic bounds limit ground motions to values that are lower than those for 2 percent PE in 50 years in areas near highly active faults in California, western Nevada, coastal Oregon and Washington, and parts of Alaska and Hawaii.

3.1.4 Design Earthquakes

Two design earthquakes are defined in the proposed LRFD specifications. The upper level earthquake is the “rare” earthquake and is defined as the MCE. For a bridge design life of 75 years, the ground motions for the MCE correspond to 3 percent PE in 75 years, except that lower ground motions are defined in areas of deterministic bounds as described above. The lower level earthquake is the “expected” earthquake and is defined as ground motions corresponding to 50 percent PE in 75 years.

3.1.4.1 The Rare MCE Event

The intent of the MCE is to reasonably capture the maximum earthquake potential and ground motions that are possible throughout the United States. The design objective is to preserve life safety and prevent collapse of the bridge, although some bridges may suffer considerable damage and may need to be replaced following the MCE.

In the current AASHTO specifications, a 10 percent PE in 50 years (approximately a 500-year return period) is used. Based on a detailed analysis of the 1996 USGS maps (ATC/MCEER, 1999a; ATC/MCEER, 1999b), the ground motions over much of the United States increase substantially for probability levels lower than 10 percent PE in 50 years; i.e.,

for return period longer than 500 years. The increase in ground motions with return period is illustrated in Figures 1 and 2. In these figures, ratios of 0.2-sec and 1.0-sec spectral accelerations for given return periods to 0.2-sec and 1.0-sec spectral accelerations for an approximate 500-year return period are plotted versus the return period for selected cities in three regions of the conterminous United States: CEUS), WUS, and California. In California, and coastal Oregon and Washington, the effects of deterministic bounds, as described in Section 3.1.3, on the ground motion ratios are included where applicable. The curves in Figures 1 and 2 illustrate that MCE ground motions in highly seismically active areas of California where deterministic bounds control do not significantly exceed 500-year ground motions; ratios of MCE to

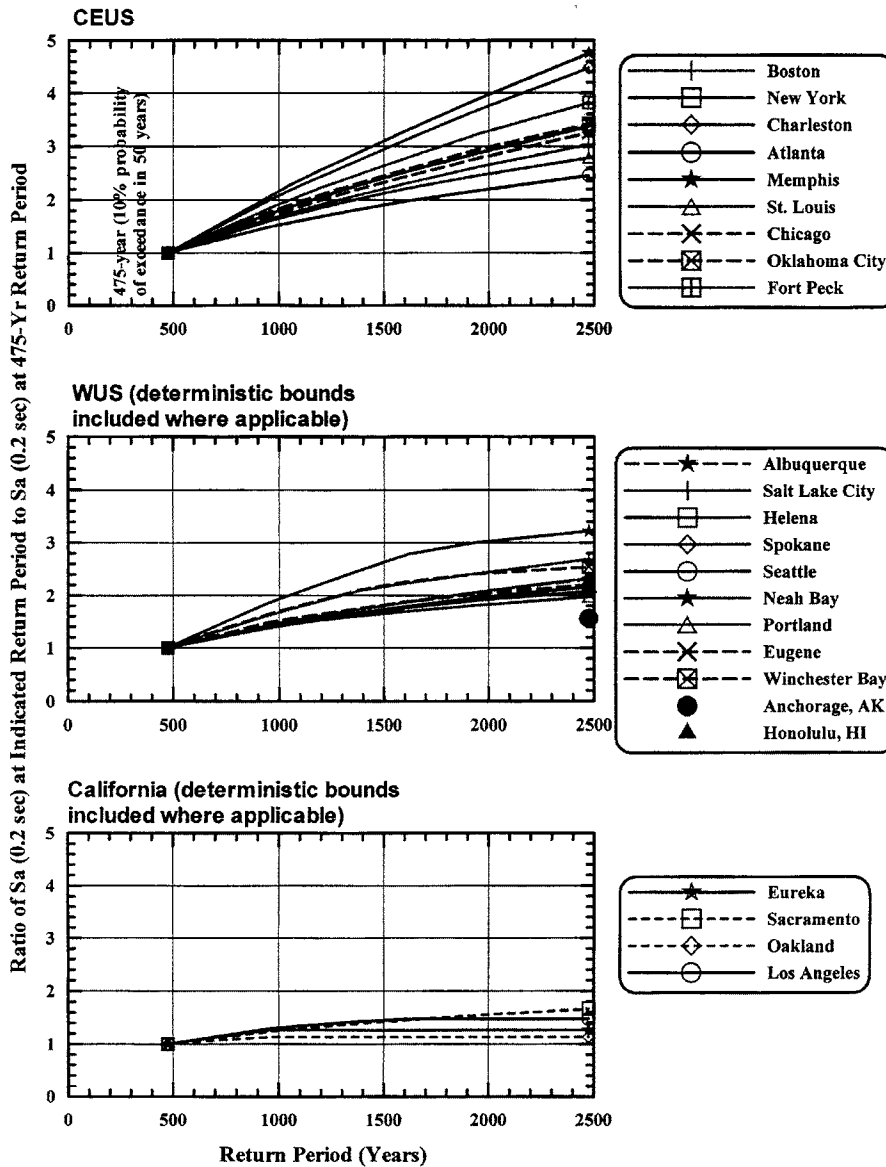


Figure 1. Ratios of 0.2-sec spectral acceleration at return period to 0.2-sec spectral acceleration at 475-year return period.

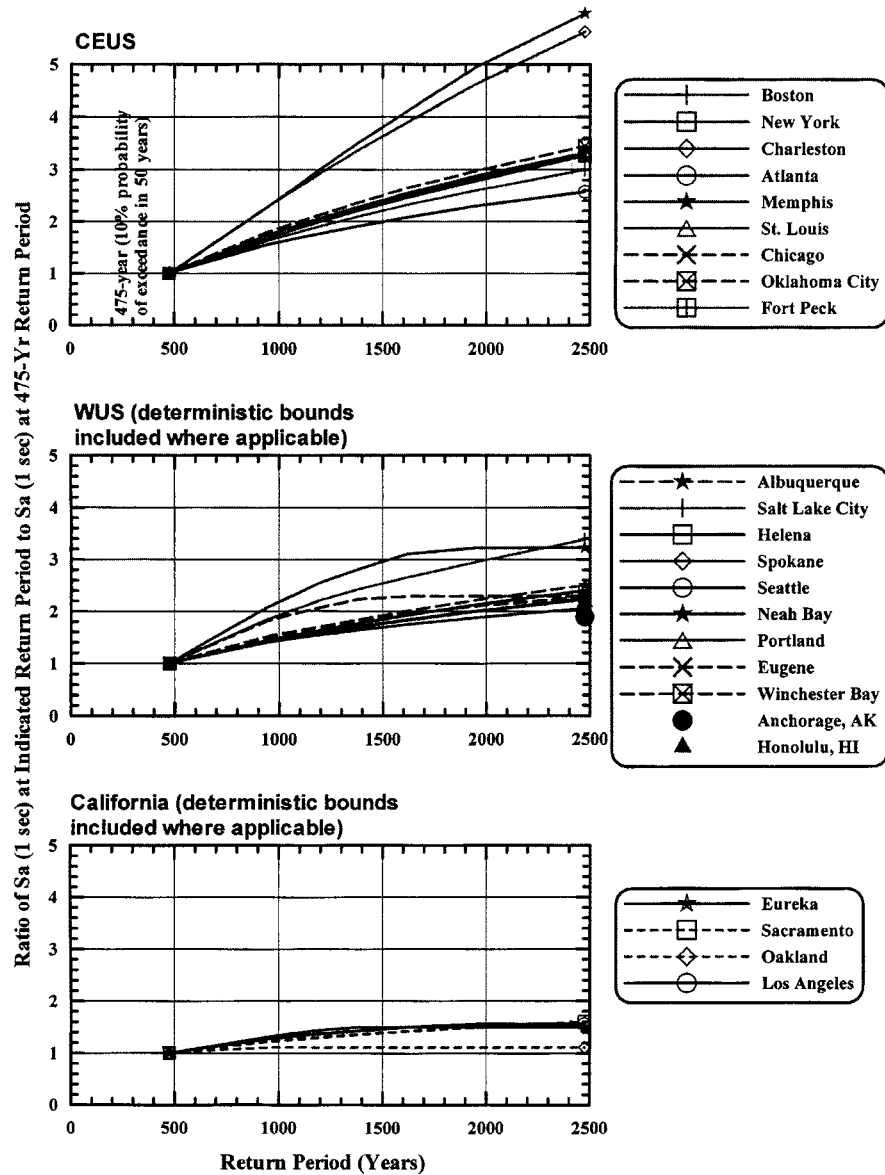


Figure 2. Ratios of 1.0-sec spectral acceleration at return period to 1.0-sec spectral acceleration at 475-year return period.

500-year ground motions are typically in the range of about 1.2 to 1.5.

In other parts of the WUS and in the CEUS, ratios of MCE ground motions (i.e., approximately 2,500-year ground motions except where deterministically bounded) to 500-year ground motions typically range from about 2.0 to 2.5, and 2.5 to 3.5, in the WUS and CEUS respectively. Even higher ratios are obtained for some areas exposed to large magnitude characteristic earthquakes that have moderately long recurrence intervals, as defined by paleoseismic data (e.g., areas associated with Charleston, New Madrid, the Wasatch Front, and coastal Oregon and Washington). These results provide the primary justification for adoption of the MCE ground motions as a design basis for a “no collapse”

performance criterion for bridges during rare but scientifically credible earthquakes.

Analysis of 1996 USGS map ground motions in the Charleston, South Carolina, and New Madrid, Missouri, regions also indicate that 500-year return period ground motions within 75 km of the source region of the 1811–1812 New Madrid earthquakes and the 1886 Charleston earthquake are far below the ground motions that are likely to have occurred during these historic earthquakes. Instead, data suggest that 2,500-year return period ground motions are in much better agreement with ground motions estimated for these earthquakes. If deterministic estimates of ground motions are made for the historic New Madrid earthquake of estimated moment magnitude 8.0 using the same ground

motion attenuation relationships used in the USGS probabilistic ground motion mapping, then the 500-year mapped ground motions are at or below the deterministic median-minus-standard-deviation ground motions estimated for the historic events within 75 km of the earthquake sources, whereas 2,500-year ground motions range from less than median to less than median-plus-standard-deviation ground motion. Similarly, 500-year ground motions range from less than median-minus-standard-deviation to less than median ground motions deterministically estimated for the 1886 Charleston earthquake of estimated moment magnitude 7.3 within 75 km of the earthquake source; whereas 2,500-year ground motions range from less than median to slightly above median-plus-standard-deviation ground motions for this event. From a design perspective, it is important that design ground motions reasonably capture the ground motions estimated for historically occurring earthquakes.

Adoption of the MCE as the design earthquake for a collapse-prevention performance criterion is consistent with the 1997 and 2000 NEHRP Provisions for new building (BSSC, 1998; BSSC, 2001), the 2000 International Building Code (IBC) (ICC, 2000), and the NEHRP *Guidelines for the Seismic Rehabilitation of Buildings* (BSSC, 1997). In the 1997 and 2000 NEHRP provisions for new buildings and the 2000 IBC, the MCE ground motions are defined as “collapse prevention” motions, but design is conducted for two-thirds of the MCE ground motions on the basis that the provisions in those documents (including the R-Factors) provide a minimum margin of safety of 1.5 against collapse. On the other hand, in the NEHRP *Guidelines for Seismic Rehabilitation of Buildings*, the MCE ground motions are not factored by two-thirds, but rather are directly used as collapse prevention motions for design. The approach proposed in the proposed LRFD specifications is similar to that of the NEHRP seismic rehabilitation guidelines in that the design provisions for the MCE (including the proposed R-Factors) have been explicitly developed for a collapse-prevention performance criterion.

The decision to use the 3 percent PE in 75-year event with deterministic bounds rather than two-thirds of this event (as used in the 2000 NEHRP provisions) was to more properly accommodate the design displacements associated with the MCE event. Displacements are much more important in bridge design than building design, since displacements govern the determination of required pier and in-span hinge seat widths, and are thus critically important in preventing collapse.

3.1.4.2 The “Expected” Earthquake

The intent of the “expected” earthquake is to describe ground motions that are likely to occur during a 75-year bridge design life (with a 50 percent probability of being exceeded during this 75-year life). The design criteria provides for minimal damage and normal service following a post-earthquake inspection. Expected earthquake ground motions are defined

by the 1996 USGS probabilistic ground motion mapping described in Section 3.1.2.

Figures 3 and 4 illustrate ratios of 0.2-sec and 1.0-sec response spectral accelerations at various return periods to 0.2-sec and 1.0-sec spectral accelerations at a 108-year return period (corresponding to 50 percent PE in 75 years) for selected cities in California, the WUS outside of California, and the CEUS, based on 1996 USGS mapping. Deterministic bounds on ground motions for long return periods have been incorporated, where applicable, in the curves in these figures. The curves indicate that ratios of MCE to expected earthquake ground motions in highly seismically active regions of California are typically equal to or less than 3.0 but typically exceed 4.0 to 5.0 in other parts of the WUS, and 7.0 to 10.0 in the CEUS. As shown in these figures, the spectral ratios of the MCE to that of frequent earthquakes may exceed 10.0 to 20.0 in some locales of low seismicity, and in environments of characteristic large magnitude earthquakes having moderately long recurrence intervals.

The decision to incorporate explicit design checks for this lower-level design event was provided in order to provide some parity with other infrequent hazards, such as wind and flood. The current AASHTO LRFD provisions require essentially elastic design for the 100-year flood and 100-mph wind which, in many parts of the country, is close to a 100-year earthquake load. Although the 50 percent PE in 75-year earthquake (108-year return period) only controls column design in parts of the western United States, this recommendation provides some consistency in the expected performance of 100-year return period design events. The significant difference in magnitude of earthquake loads with longer return periods is another reason why seismic design must consider much longer return period events. Both wind and flood loads tend to asymptotic values as the return period increases and, in fact, the ratio of a 2,000-year to 50-year wind load is in the range of 1.7 to 2.1 (Heckert, Simiu, and Whalen, 1998).

3.1.5 Impact Studies

Current AASHTO design uses a 500-year return period for defining the design earthquake. A more meaningful way to express this earthquake is in terms of probability of exceedence (PE). A 500-year earthquake is one for which there is a 15 percent chance of exceedence in the 75-year life of the bridge. In other words, there is a 15 percent chance that an earthquake larger than the design earthquake will occur during the life of the bridge. Whether this risk is considered acceptable or not depends on the probability of occurrence of the event, the consequences of the event larger than the design event, and the cost of reducing these consequences. A 15 percent PE in 75 years provides, by most standards, a low factor of safety against exceeding the design load. However, in order to evaluate if we should reduce the PE, the conse-

quences must be known or understood. To address this, two questions must be addressed: (1) By how much will the design earthquake be exceeded? and (2) What is the bridge’s available reserve capacity due to conservative design provisions?

Most highway bridges have at least some capacity in reserve for extreme events. The current AASHTO specifications provide varying levels of conservatism due, among other factors, to the use of relatively low R-Factors, a spectral shape based on $1/T^{2.3}$, generous seat widths, uncracked sections for analysis, low ϕ factors, and Mononabe-Okabe coefficients for abutment wall design. These criteria are based on engineering judgment and provide a measure of protection against large but infrequent earthquakes. However, the degree of conservatism is actually unknown and the consequences of earthquakes larger than the design event are uncertain and

may be considerable. If the actual event is only 20 percent larger than the design event, damage is likely to be slight, the consequences tolerable, and the risk acceptable. On the other hand, if the actual earthquake is 200 percent to 400 percent larger than the design earthquake, the reserve capacity is likely to be exceeded, damage is likely to be extensive, and loss of access or use of the bridge likely. In this case, the risk may be considered unacceptable. If MCE 0.2-sec and 1.0-sec values of spectral acceleration shown in Figures 1 and 2 are used as a measure of earthquake size, actual forces may exceed the design 500-year earthquake forces by factors that range from 1.5 in Los Angeles to 4.5 (or more) in Charleston, South Carolina. Similar ratios based on the 108-year “expected” earthquake forces shown in Figures 3 and 4 are approximately 3.0 for Los Angeles and greater than 20.0 for Charleston.

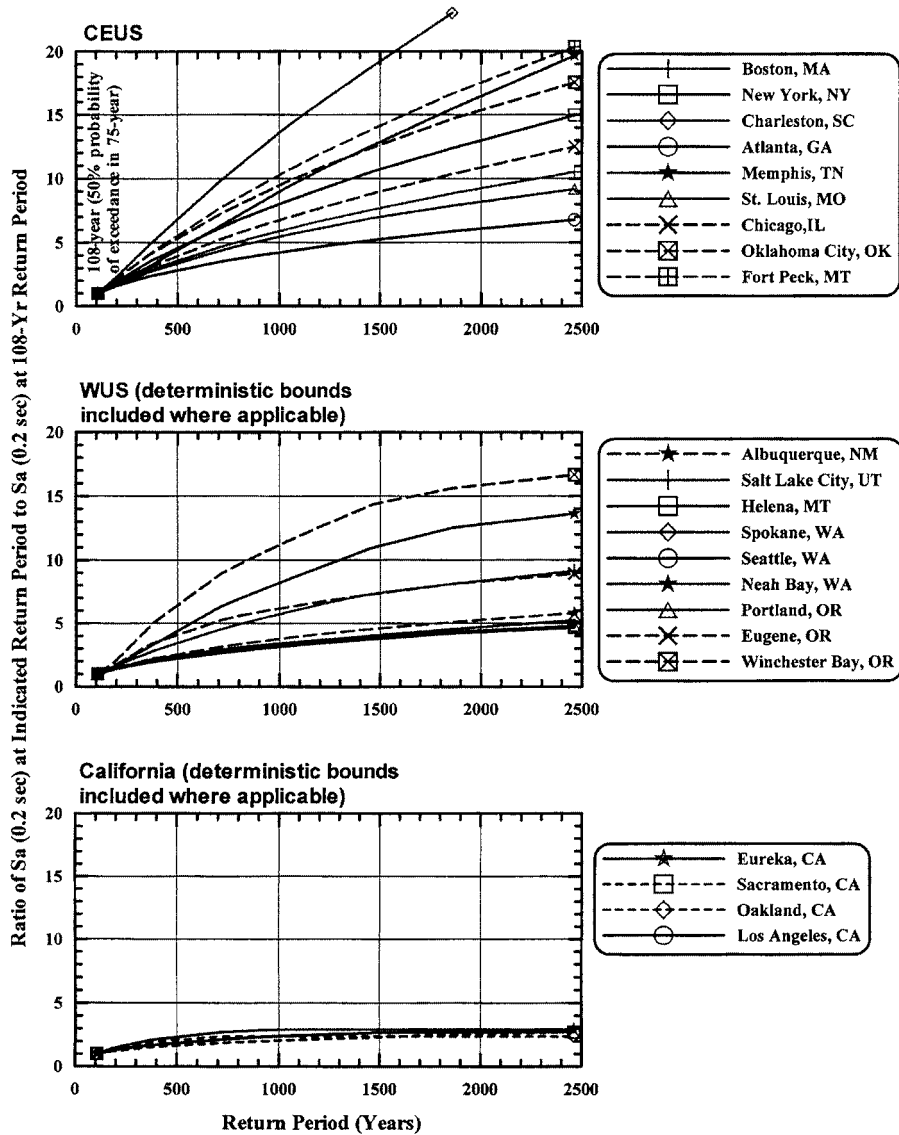


Figure 3. Ratios of 0.2-sec spectral acceleration at return period to 1.0-sec spectral acceleration at 108-year return period.

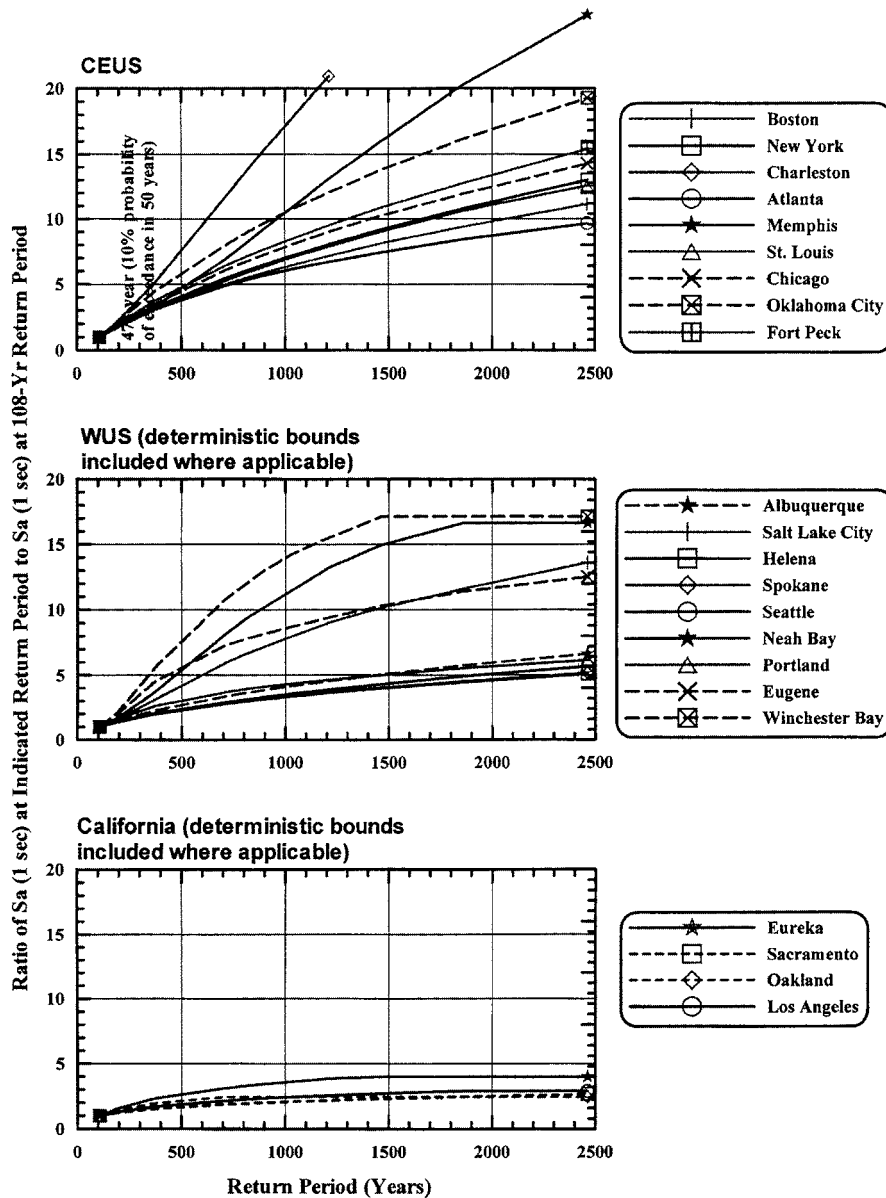


Figure 4. Ratios of 1.0-sec spectral acceleration at return period to 1.0-sec spectral acceleration at 108-year return period.

Reserve capacities as high as 4.5 are not likely embodied in the current AASHTO specifications, and no assurance can be given regarding bridge damage and access in these situations.

With this as background, there were therefore two options for the development of the new LRFD seismic design provisions. The first option was to design explicitly for a larger event (3 percent PE in 75 years) but refine the provisions to reduce the conservatism and thus keep costs about the same as those associated with the current AASHTO provisions. Under this scenario, the degree of protection against larger earthquakes is quantified and is based on scientific principles and engineering experience. The second option was to design for a more moderately sized event (say 15 percent PE in

75 years), and maintain the current conservative provisions as a measure of protection against larger events. In this scenario, the degree of protection is still unknown and depends to a large extent on intuition and engineering judgment. The first option was selected for the proposed LRFD provisions and, as part of the development process, a series of parameter studies were performed to assess the potential cost impacts of designing for the higher level event. These studies are summarized in Section 3.3. In brief, however, they indicate that the net effect on the cost of a column and spread footing system is on the average 2 percent less than the current AASHTO Division I-A provisions for multi-column bents and 16 percent less than the current Division I-A provisions for single-

column bents. These cost comparisons are based on the use of more refined methods for calculating overstrength factors and 2,400 different column configurations that were analyzed based on seismic input from five different cities throughout the United States.

Another cost concern that arose during the development of the provisions was the impact of the longer return period on liquefaction. Two detailed case studies were performed using both the proposed LRFD and existing AASHTO provisions, and are summarized in the following section. These case studies demonstrated that the application of the proposed LRFD provisions, when applied with the inclusion of inelastic deformation in the piles as a result of lateral flow, would not be significantly more costly than the application of the current provisions. Hence, the objective of having a quantifiable degree of protection against larger earthquakes, while attempting to maintain a similar level of cost, was achieved.

3.2 LIQUEFACTION ASSESSMENT AND CASE STUDIES

A study of the effects of liquefaction and the associated hazards of lateral spreading and flow was undertaken as part of NCHRP Project 12-49. The motivation for the study was to assess the impact of the MCE event, when compared with current AASHTO criteria. As noted earlier, the recommended LRFD provisions are based on a 3 percent PE in 75 years for most of the United States, with the exception of areas where ground motions are deterministically bounded. In contrast, the design ground motion hazard in the current AASHTO Division I-A has a 10 percent PE in 50 years (which is approximately equal to a 15 percent PE in 75 years). With this increase in return period comes an increase in the potential for liquefaction and liquefaction-induced ground movements, and the resulting potential for significant bridge damage.

Current methods of analysis that are typically used for liquefaction evaluation have a level of conservatism built into them. As a result, the use of state-of-the-art design procedures could lead to designs that perform satisfactorily in larger earthquakes, and still produce designs that cost on par with those developed under current AASHTO seismic provisions.

The scope of the liquefaction study was limited to two sites in relatively high areas of seismicity; one was located in the western United States in Washington State, and the other was located in the central United States in Missouri. The Washington State site is located near the Cascadia subduction zone, while the Missouri site is located near the New Madrid seismic zone. Actual site geologies and bridge configurations were provided by the two states and were used in the initial stages of the study. The site geologies were subsequently idealized through limited simplifications, although the overall geologic character of each site was preserved.

The investigation of the two sites and their respective bridges focused on the resulting response and design differences between the recommended ground shaking level (3 per-

cent PE in 75 years) and that corresponding to the current AASHTO Division I-A provisions (approximately 15 percent PE in 75 years). The scope of the study for each of the two sites and bridges included

- Development of both 15 percent PE in 75-year and 3 percent PE in 75-year acceleration time-histories;
- Simplified, conventional liquefaction analyses;
- Nonlinear assessment of the site response to these accelerations including determining the time history of pore pressure increases;
- Assessment of abutment end slope stability;
- Estimation of lateral spreading and flow conditions at each site;
- Design of the structural systems to withstand the predicted response and flow conditions;
- Evaluation of geotechnical mitigation measures for liquefaction-related ground displacements; and
- Evaluation of cost impacts due to the structural and geotechnical mitigation strategies.

The results for the 15 percent PE in 75-year and 3 percent PE in 75-year events were compared to assess the implications of using the larger event for design. Additionally, the study helped synthesize an overall approach for handling liquefaction-induced movements in the recommended design provisions.

3.2.1 Design Approach

The design approach used in the study and recommended for the new AASHTO LRFD provisions involves four basic elements:

1. Conduct a stability analysis;
2. Conduct a Newmark sliding block analysis;
3. Perform assessments of the passive forces that can ultimately develop ahead of the piles or foundation systems as liquefaction induces lateral spread; and
4. Perform an assessment of the likely plastic mechanisms that may develop in the foundations and substructure.

The intent of this approach is to determine the expected magnitude of lateral soil movement and to assess the structure's ability to accommodate this movement or limit it. The approach is based on use of a deep foundation system, such as piles or drilled shafts, as spread footing foundations will not typically be used when soil conditions lead to the possibility of liquefaction and associated lateral spreading or settlement.

It should be noted that the concept of considering a plastic mechanism or hinging in the piles under the action of spreading forces is tantamount to accepting foundation damage. This is a departure from seismic design for structural inertia loading alone, but is believed to be reasonable for the rare MCE event because it is unlikely that the formation of plastic

hinges in the foundation will lead to overall structure collapse. The reasoning behind this is that lateral spreading is essentially a displacement-controlled process. Thus, the estimated soil displacements represent a limit on the overall structure displacement, excluding buckling of piles or shafts below grade and increased displacements that can be produced by large $P-\Delta$ effects. However, buckling should be checked, and methods that include the soil residual resistance should be used. Meyersohn et al. (1992) provides a method for checking buckling as an example. The effects of $P-\Delta$ amplification are discussed later.

3.2.2 Washington State Bridge Case-Study Site Selection and Characterization

As noted earlier, two sites were chosen for this study: a western United States site and a site located within the New Madrid seismic zone. The western site analyses and results are discussed herein; a brief summary of the results of the New Madrid site assessment are given in Section 3.2.9 and are discussed in more detail in ATC/MCEER (2000). The Washington State site is located just north of Olympia, Washington, in the Nisqually River valley.³ The location is within a large river basin in the Puget Sound area of Washington State and is situated near the mouth of the Nisqually River in the estuary zone. The basin is an area that was overridden by glaciers during the last ice age and therefore has over-consolidated material at depth. Additionally, the basin contains significant amounts of recently deposited loose material over the glacially consolidated materials.

Soil conditions for the site were developed from data provided by the Washington State Department of Transportation (WSDOT) for another well-characterized site located in a geologically similar setting near Seattle. The actual site was “moved” to the Olympia area to avoid the effects of the Seattle Fault. At the prototype site, the material at depths less than 45 m (150 ft) are characterized by alluvial deposits. At greater depths, some estuarine materials exist and below about 60 m (200 ft) dense glacial materials are found. This then produces a site with the potential for deep liquefiable soils.

For the purposes of this study, the site profile was simplified so that fewer layers were considered to exist, and the profile was assumed as uniform across the entire site. The simplified profile retains features and layering that reproduce responses of the actual site. The simplified soil profile is shown in Figure 5. This figure also includes relevant properties of the soil layers that have been used for the seismic response assessments and bridge design. Shear wave velocity (V_s), undrained shearing strength (c_u), soil friction angle (ϕ), and residual soil strength (S_{ur}) were interpreted from the

field and laboratory data provided by WSDOT. The cyclic resistance ratio (CRR) was obtained by conducting simplified liquefaction analyses using both the SPT and CPT methods to obtain CRR values. Average CRR values were determined for liquefiable materials and represent clean sand values for a magnitude 7.5 earthquake.

The prototype site profile with structure elevation is shown in Figure 6. The modified site is a smaller river crossing than that of the original bridge since the total length of the bridge was substantially shortened for the study. A length sufficient to illustrate the issues of soil movement and design was used. In this study, the total length of the bridge is 152 m (500 ft). The ground surface is shown as the 0-foot elevation. As can be seen in the figure, approach fills are present at both ends of the bridge and, in this case, they are relatively tall at 9 m (30 ft) each. An approach fill comprised of a relatively clean sandy gravel was assumed at each abutment, with an assigned friction angle of 37 deg.

3.2.3 Washington State Bridge Description

The prototype bridge from which the study data were drawn is a river crossing with several superstructure and foundation types along the length of the structure. For this study, the actual structure was simplified. The 152-m (500-ft) long structure is comprised of a 1.8-m (6-ft) deep concrete box girder that is continuous between the two abutments. The intermediate piers are two-column bents supported on pile caps and 0.6-m (24-in.) steel piles filled with reinforced concrete. The roadway is 12 m (40 ft) wide. The two 1.2-m (4-ft) diameter columns for each pier are spaced approximately 7 m (23 ft) apart and, because of the relatively large size of the pile caps, a single combined pile cap was used for both columns at each pier. Figure 7 shows the general arrangement of an intermediate pier.

The centermost pier in this example is located at the deepest point of the river channel, as shown in Figure 6. While this is somewhat unusual in that a longer span might often be used to avoid such an arrangement, the river pier was used here as a simplification. The columns of this pier are also relatively slender, and they were deliberately left that way so as to allow any negative seismic effects due to slenderness, like $P-\Delta$, to be assessed. In a final design, the size of these columns might likely be increased. In fact, non-seismic load combinations or conditions are likely to require larger columns.

The abutments are of the overhanging stub abutment type. Figure 8 shows the transverse and longitudinal elevations of the abutments used in this bridge. For this type of abutment, the backfill is placed directly against the end diaphragm of the superstructure. This has the seismic advantage of providing significant longitudinal resistance for all displacement levels, since the passive resistance of the backfill is mobilized as the superstructure moves. This type of abutment also eliminates the need for expansion joints at the ends of the

³ This site was selected for, and the liquefaction evaluation completed prior to, the February 2001 Nisqually earthquake. Ground motions associated with the Nisqually earthquake were considerably less than those used in this study. While liquefaction occurred at some locations near the selected site, little or no bridge damage apparently occurred, likely due to the limited extent of ground shaking and liquefaction.

**WASHINGTON SITE
Non-Liquefied Soil Profile**

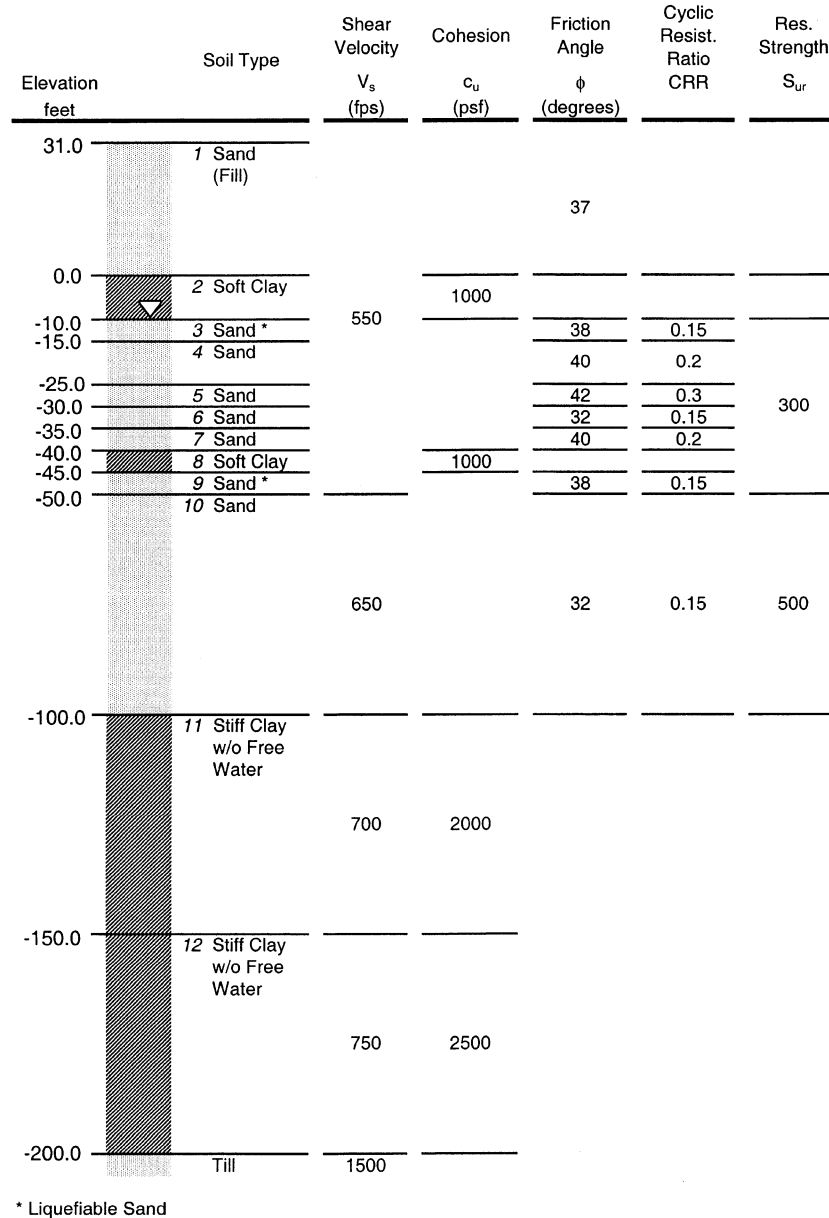


Figure 5. Simplified soil profile for the western U.S. site.

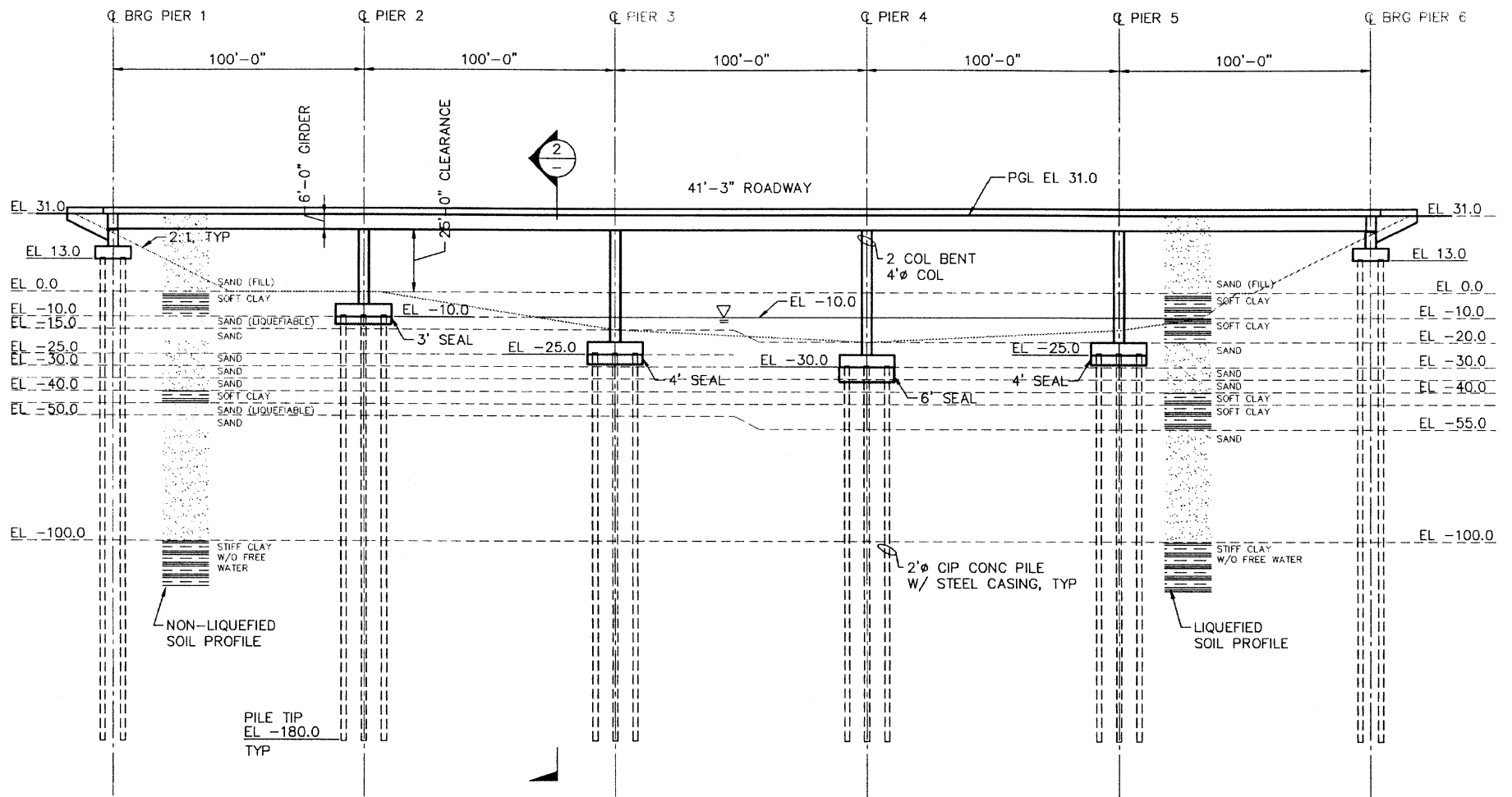
structure and, for this reason, is limited to shorter total length structures.

3.2.4 Design Response Spectra and Time Histories

Design response spectra from current AASHTO specifications and from the recommended LRFD provisions were constructed using the procedures and site factors described in the respective specifications. For the current AASHTO specifications, the hazard level of 10 percent PE in 50 years was used.

For the recommended LRFD specifications, both the rare earthquake (MCE) with 3 percent PE in 75 years with deterministic bounds near highly active faults, and the expected (frequent) earthquake with 50 percent PE in 75 years, were used as design earthquakes.

Design response spectra based on current AASHTO specifications were constructed using a peak ground acceleration (PGA) on rock of 0.24 g for the Olympia, Washington, site, on the basis of the map contained in the current AASHTO specifications. Design spectra for the MCE of the recommended LRFD specifications were constructed using rock



1
-
ELEVATION - WASHINGTON BRIDGE
SCALE: 1" = 30'-0"

Figure 6. Site profile and structure elevation.

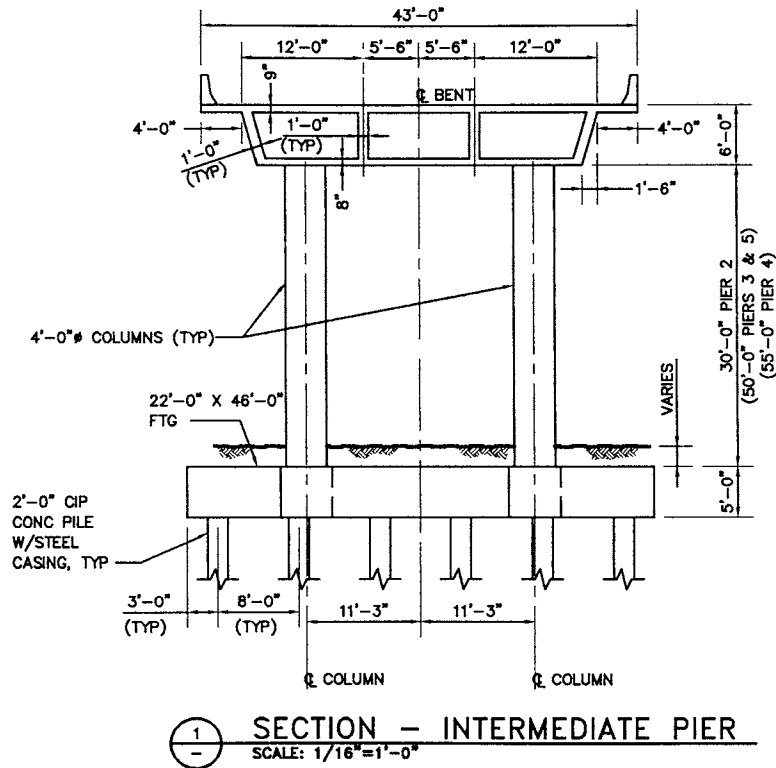


Figure 7. Elevation of an intermediate pier.

(Site Class B) spectral accelerations at 0.2-sec and 1.0-sec periods. These spectral values were obtained from the maps published by the USGS. The PGA for the MCE was defined as 40 percent of the spectral acceleration at 0.2 sec, as recommended by the LRFD provisions. Design spectral accelerations for the expected earthquake were obtained from the hazard curves of probabilistic ground motions, which are on the CD-ROM also published by the USGS.

Rock spectra based on the AASHTO specifications and the proposed LRFD provisions were adjusted for local site soil conditions. According to the current AASHTO specifications, this site is classified as soil profile Type III; the LRFD provisions define the site as Site Class E. Figure 9 presents the design response spectra of the current AASHTO specifications on soil profile Type III, and for the MCE and the frequent earthquake of the proposed LRFD specifications on Site Class E. These site classifications represent the assessed soil profile below the ground surface where response spectra are defined for structural vibration design, and peak ground accelerations are used for simplified liquefaction potential analyses. Note that in the figure the short-period branch of the AASHTO spectrum are assumed to drop from the acceleration plateau at a period of 0.096 sec to the peak ground acceleration at 0.02-sec period, which is the same as for the MCE spectra. Also note that, because the long-period branch of the current AASHTO spectra declines more slowly with period than those of the MCE (i.e., $1/T^{2/3}$ versus $1/T$), the cur-

rent AASHTO and MCE spectra are converging as the period increases.

Acceleration time histories consistent with the current AASHTO specifications and with MCE ground motions of the proposed LRFD specifications were developed as firm soil outcropping motions for input to the one-dimensional, nonlinear site response analyses to assess the liquefaction hazard of the site. These time histories were developed in accordance with the requirements and guidelines of the proposed LRFD specifications. Deaggregation of the probabilistic results for the Olympia, Washington, site indicates that significant contributions to the ground motion hazard come from three magnitude–distance ranges: (1) magnitude 8.0 to 9.0 earthquakes occurring at distances of 70 to 80 km; (2) magnitude 5.0 to 7.0 events occurring at distances of 40 to 70 km; and (3) magnitude 5.0 to 6.5 earthquakes occurring at distances less than 20 km. These three magnitude–distance ranges are associated, respectively, with (1) large-magnitude subduction zone interface earthquakes, (2) moderate magnitude earthquakes occurring within the subducting slab of the Juan de Fuca plate at depth beneath western Washington State and in the shallow crust of the North American plate at relatively large distances from the site, and (3) moderate magnitude earthquakes occurring in the shallow crust of the North American plate in the near vicinity of the site.

Time histories were developed for each of these three earthquake types. The selected source for (1) was the 1985 Chile

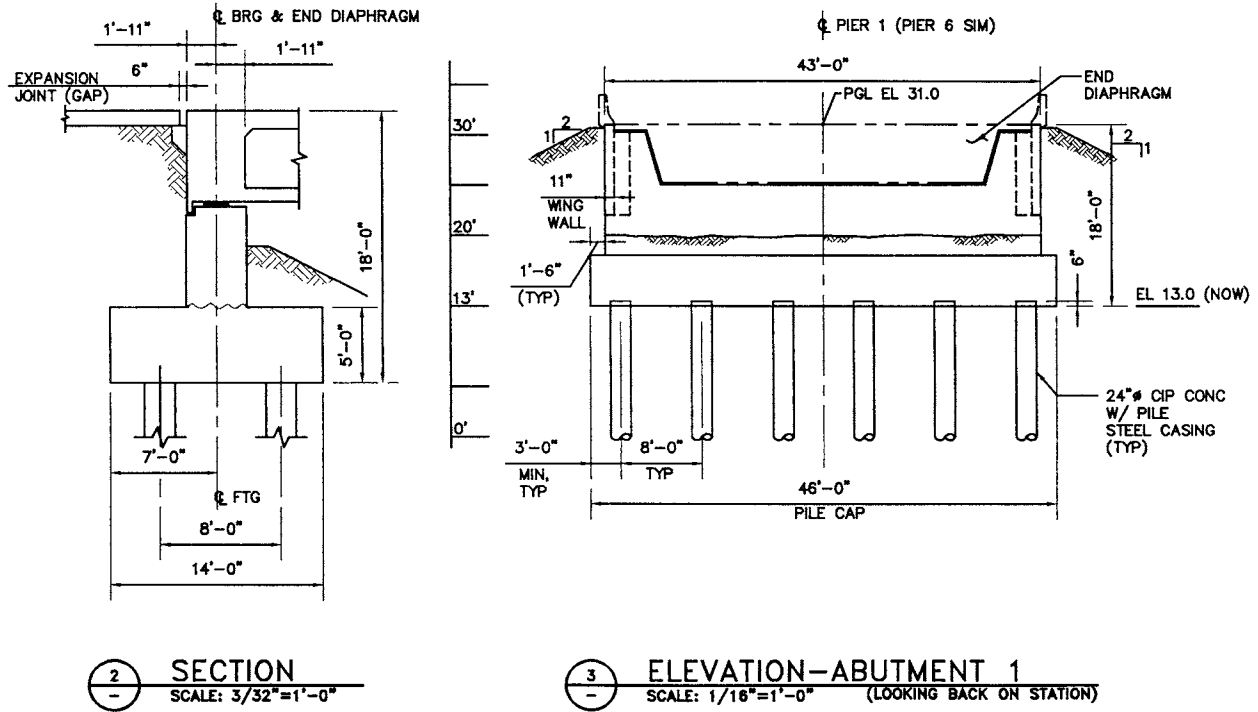


Figure 8. Elevation of the abutment.

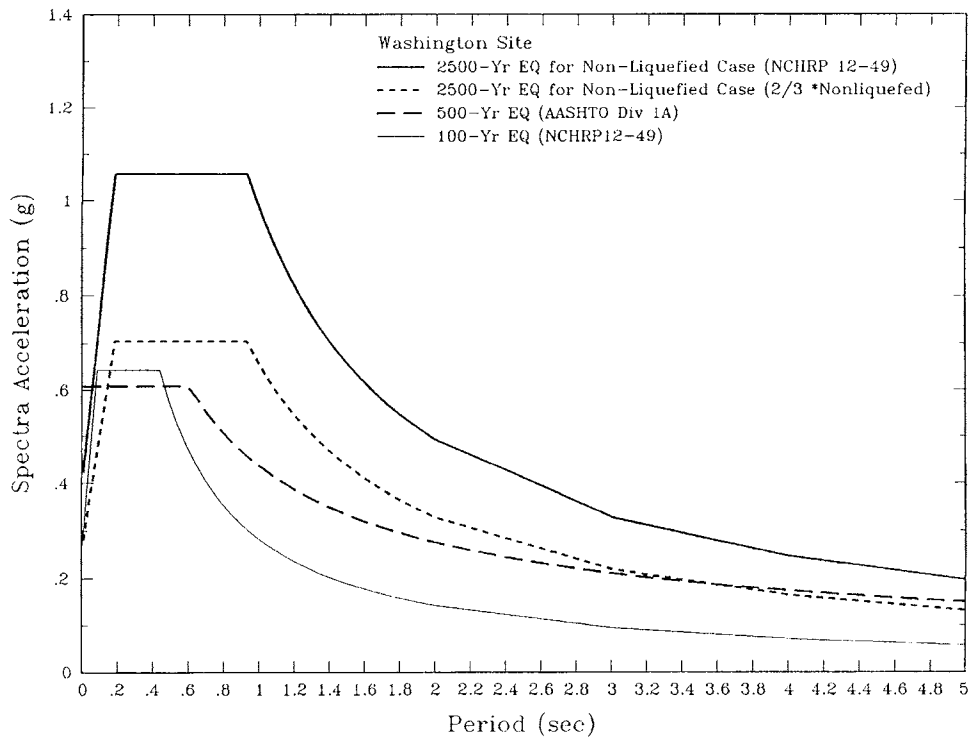


Figure 9. Comparison of design response spectra based on current AASHTO Specifications (Site Class III), with the MCE and Frequent Earthquake of the proposed LRFD specifications (Site Class E), for the Washington State site.

earthquake; (2) was representative of the events occurring within the subducting slab of the type that occurred near Olympia in 1949 and during the 2001 Nisqually earthquake; and (3) was the 1986 Desert Hot Springs earthquake, which was a moderate magnitude local shallow crustal event.

3.2.5 Liquefaction Studies

The liquefaction study for the Washington State bridge site involved two phases. In the first phase, a series of liquefaction analyses were conducted using the SPT and CPT simplified methods. Results of these analyses were used to determine the depths at which liquefaction could occur during the 15 percent PE in 75-year and 3 percent PE in 75-year earthquakes. These results were also used as a basis for determining the residual strength of the soil. Concurrent with these analyses, a series of one-dimensional nonlinear, effective stress analyses were conducted to more explicitly define the mechanisms for pore water pressure increase within the soil profile, and to define the changes in ground accelerations and deformations resulting from the development of liquefaction.

3.2.5.1 Simplified Liquefaction Analyses

The first step of the procedure is to determine whether liquefaction is predicted to occur.

Simplified liquefaction analyses were conducted using the procedures given in Youd and Idriss (1997). Two levels of PGA were used, one representing accelerations from the current AASHTO specifications with its 10 percent PE in 50-year event and the other representing the proposed LRFD specification 3 percent PE in 75-year event. The PGA for the 10 percent in 50-year event was not adjusted for site effects; this is consistent with the approach recommended in the current AASHTO specifications.⁴ Ground motions for the 3 percent PE in 75-year event were adjusted to Site Class E, as recommended in Article 3.4 of the proposed LRFD provisions. The resulting PGA values for each case are shown in Table 1.

The magnitude of the design earthquake was required for the SPT and CPT simplified analyses. Results of deaggregation studies from the USGS suggest that the mean magnitude for PGA for the 10 percent PE in 50-year and 3 percent PE in 75-year events is 6.5. This mean magnitude reflects contributions from the different seismic sources discussed above. However, common practice within Washington State has been to use a magnitude 7.5 event as that representative of the likely size of a subduction zone event occurring directly

below the Puget Sound area. In view of this practice in Washington State, a range of magnitudes (6.5, 7.0, and 7.5) was used during the liquefaction analyses.

For these analyses, ground water was assumed to occur 3 m (10 ft) below the ground surface for the case without approach fill. Evaluations were also performed using a simplified model to evaluate the effects of approach fill. For the fill model, the soil profile with the associated soil properties was the same as for the free-field case. However, an additional 9 m (30 ft) of embankment was added to the soil profile. This change results in a lower imposed shearing stress (i.e., demand) because of the lower soil flexibility factor (R_d). No adjustments were made to the normalized CRR values for the greater overburden. As discussed in Youd and Idriss (1997), the recommended approach for a site where fill is added is to use the pre-fill CRR value, under the assumption that the overburden effects from the fill will not have an appreciable effect on the density of the material.

Liquefaction potential factors of safety (FOS) at the three earthquake magnitudes (6.5, 7.0, and 7.5) are shown in Figures 10 and 11 for the 10 percent PE in 50-year and 3 percent PE in 75-year events, respectively, for the case of no approach fill. These results indicate that liquefaction could occur at two depths within the soil profile for the 10 percent PE in 50-year ground motion, depending somewhat on the assumed earthquake magnitude. For the 3 percent PE in 75-year event, liquefaction is predicted to depths of 23 m (75 ft), regardless of the assumption on the earthquake magnitude.⁵

Detailed results of the liquefaction analyses with the approach fill are presented in the liquefaction study report (ATC/MCEER, 2000). The fill case results in somewhat lower liquefaction potential (i.e., higher FOS) due to the lower imposed shearing stress.

3.2.5.2 DESRA-MUSC Ground Response Studies

A nonlinear dynamic effect stress approach can be used as a more detailed and refined way to assess whether liquefaction occurs and what the resulting ground motions are. For this study, one-dimensional nonlinear effective stress site response analyses were conducted using the program DESRA-MUSC (Martin and Qiu, 2001).

The idealized site profile and related soil properties adopted for the response analyses are again those shown in Figure 5. Response analyses were performed for the three ground motions, assuming a transmitting boundary input at a depth

⁴ Common practice is to adjust the PGA for the site soil factors as given in the current AASHTO specifications. While this adjustment may be intuitively correct, these site factors are not explicitly applied to the PGA. If the site coefficient were applied at the Washington State site, the PGA would be increased by a factor of 1.5, making it only slightly less than the PGA for the MCE 3 percent PE in 75-year event.

⁵ The maximum depth of liquefaction was limited to 23 m (75 ft), consistent with WSDOT's normal practice. However, there is some controversy as to whether a maximum depth of liquefaction exists; some researchers have suggested that liquefaction does not occur below 17 m (55 ft). Unfortunately, quantitative evidence supporting liquefaction below 17 m (55 ft) on level ground is difficult to find even though cases of deep liquefaction were recorded in the 1964 Alaskan earthquake. For expediency, liquefaction in the simplified analysis was limited to 23 m (75 ft).

TABLE 1 Washington State bridge case study PGA values

| Input Parameter | 10% PE in 50 Years | 3% PE in 75 Years |
|--------------------------|--------------------|-------------------|
| Peak ground acceleration | 0.24 g | 0.42 g |
| Mean magnitude | 6.5 | 6.5 |

of 61 m (200 ft), corresponding to the till interface. Analyses were conducted for both the 10 percent PE in 50-year and 3 percent PE in 75-year events and for site profiles with and without embankment fill. The DESRA-MUSC parameters used for analyses for the various soil strata (G/G_{max} , backbone, and liquefaction strength curves) are documented in the case study report (ATC/MCEER, 2000) together with the results of the response analyses for all cases defined above. A representative set of results for the time history matching the site spec-

tra, but based on the 1985 Chile Earthquake—which has the highest energy levels of the three events used for analyses (representative of a magnitude 8 event)—are described below.

3.2.5.2a Nonlinear Analysis Without Embankment Fill

The site response was analyzed for the 10 percent in 50-year and 3 percent in 75-year events without embankment

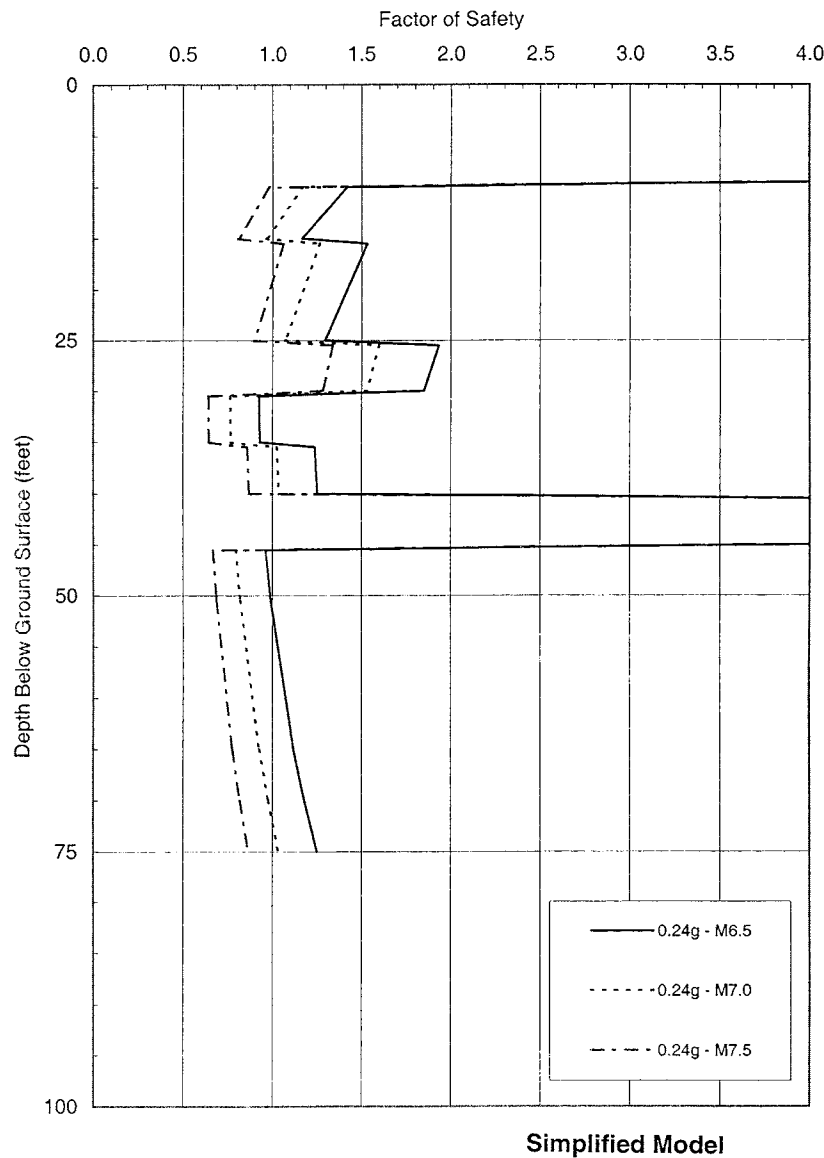


Figure 10. Liquefaction potential of the 475-year return period earthquake.

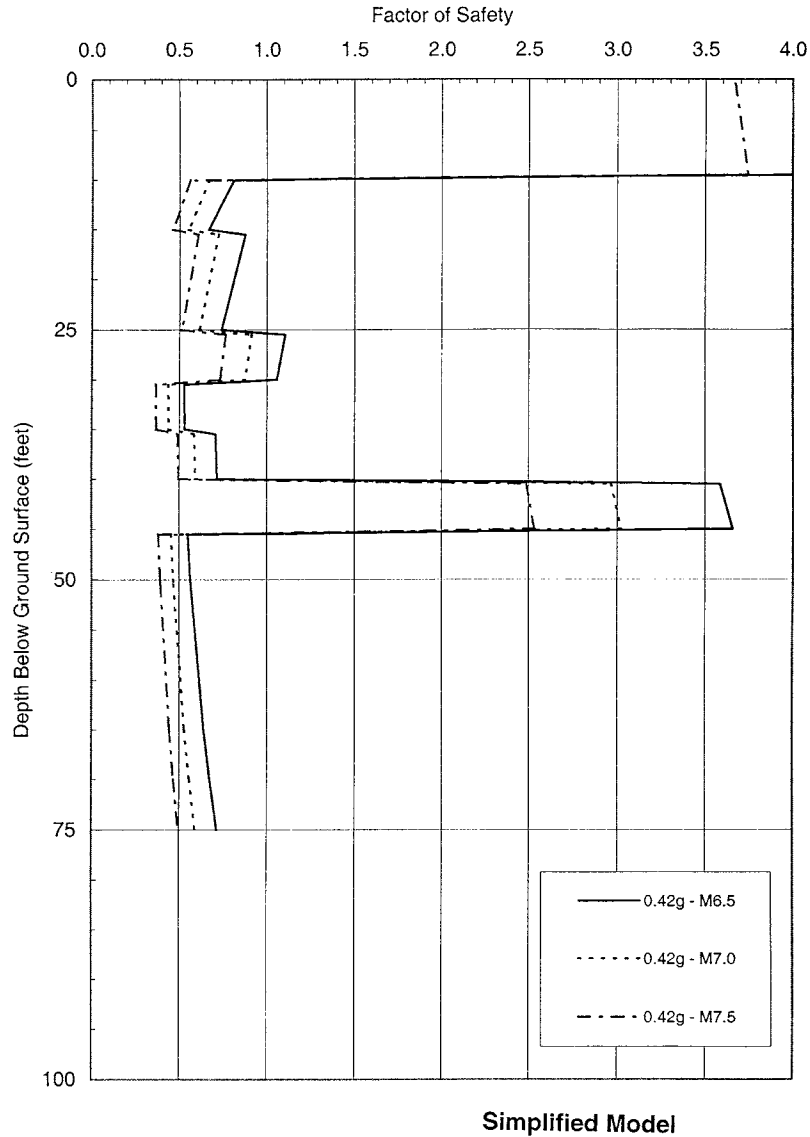


Figure 11. Liquefaction potential of the 2,475-year return period earthquake.

fill. A detailed discussion of these analyses is presented in ATC/MCEER (2000). The following are important observations resulting from these analyses:

- The pore water pressure time history response and output accelerations are very similar for the 10 percent PE in 50-year and 3 percent PE in 75-year cases. The underlying reason for this is the fact that the higher input accelerations for the 3 percent PE in 75-year case are more strongly attenuated when transmitted through the clayey silts between 30 and 60 m (100 to 200 ft), such that input accelerations at the 30 m (100 ft) level for both cases are of the order of 0.25 *g*.
- All liquefiable layers between 3 and 30 m (10 to 100 ft) eventually liquefied in both cases. Liquefaction was first

triggered in the 14- to 15-m (45- to 50-ft) layer, which became the focal point for shear distortion and associated ground lurch. Maximum shear strains of about 6 percent and 10 percent for the 10 percent PE in 50-year and 3 percent PE in 75-year events, respectively, over the 1.5-m (5-ft) depth of this layer, would suggest maximum ground lurches of about 0.1 m and 0.15 m (0.3 and 0.5 ft), respectively. Liquefaction also occurred at about the same time for the layer between 3 and 6 m (10 and 20 ft). Maximum shear strains in this and other layers were relatively small, but still sufficient to eventually generate liquefaction. The strong focal point for shear strains for the 14- to 15-m (45- to 50-ft) layer suggests that this layer would also be the primary location of lateral spread distortion.

- Liquefaction at the 14- to 15-m (45- to 50-ft) layer, which was triggered at about a time of 17 sec, effectively acted as a base-isolation layer, subsequently suppressing the transmission of accelerations above that depth, and generating a much “softer” soil profile. Such behavior is representative of observations at sites that liquefied during the Niigata and Kobe, Japan, earthquakes.

Similar trends to these were seen for the other two time histories based on the Olympia and Desert Hot Springs earthquakes. However, for the Desert Hot Spring event, which is more representative of a magnitude 6.5 event, liquefaction did not occur at depths greater than 17 m (55 ft) and just barely occurred at depths between 6 m and 9 m (20 to 30 ft) for the 10 percent PE in 50-year event.

The above results are generally consistent with the factor of safety calculations conducted using the simplified method. However, one notable difference was the observation that the sand layer between 7.5 m and 9 m (25 to 30 ft) with $CRR = 0.3$ tends to build up pore water pressure and liquefy in a manner similar to the layers above with $CRR = 0.2$ and below with $CRR = 0.15$ because of pore water pressure redistribution effects, which are considered in DESRA-MUSC. Meanwhile, the simplified method, which assumes no drainage during earthquake shaking, indicates factors of safety greater than 1.0 for 50 percent PE in 50-year events. The effects of redistribution also tend to suppress the rate of pore water pressure build up in the layer between 9 m and 10.7 m (30 to 35 ft).

3.2.5.2b Nonlinear Analysis with Embankment Fill

Analyses with embankment fill for the 10 percent PE in 50-year and 3 percent PE in 75-year events were conducted in a manner similar to that for the case without embankment fill. As in the simplified method, the effect of the fill is to suppress the rate of pore water pressure buildup in the DESRA-MUSC analyses (or increase the factor of safety in the case of the simplified method). However, the overall response is similar for both the 10 percent PE in 50-year and 3 percent PE in 75-year cases as that of the case without embankment fill.

Liquefaction was first triggered in the 14- to 15-m (45- to 50-ft) layer, which became the focal point for shear distortion (similar to the no fill case). Liquefaction also occurred at about the same time for layers between 3 m and 6 m (10 to 20 ft). However, liquefaction was suppressed in layers between 6 m and 12 m (20 to 40 ft). The strong focal point for shear strains for the 14- to 15-m (45- to 50-ft) layer again suggests that this layer will be the primary location of lateral spread distortion. Similar trends to those described above were also seen for the time histories based on the Olympia and Desert Hot Spring earthquakes although, as for the no embankment fill case, liquefaction did not occur at depths greater than 17 m (55 ft) for the 10 percent PE in 50-year Desert Hot Springs event.

The above results are again generally consistent with the factor of safety calculations using the simplified method, but with the notable differences that for the 10 percent PE in 50-year Olympia and Chile events, liquefaction occurred at depths between 21 m and 30 m (70 to 100 ft), whereas factors of safety would have been greater than 1.0 based on the simplified method. This reflects the “bottom-up” wave propagation used in DESRA-MUSC, versus the “top-down” inertial loading from the simplified method.

3.2.5.3 Lateral Ground Displacement Assessment

Based on the results of the simplified liquefaction studies, two liquefiable zones were identified for stability and displacement evaluations. One extends from a depth of 3 m to 6 m (10 to 20 ft) below the ground surface. The other extends from 14 m to 17 m (45 to 55 ft) below the ground surface. The residual strength of these two liquefied zones was selected as 14 kPa (300 psf), which was based on SPT blow counts in each layer. Soils between 6 m and 12 m (20 to 40 ft) below the ground surface and between 17 m and 30 m (55 to 100 ft) below the ground surface were assumed to have a partial build-up in pore water pressure, resulting in some reduction in the friction angle of the nonliquefied sand layers; this was readily apparent in the DESRA-MUSC analyses. For these conditions, the response of the end slope at the approach fill on each side of the river channel was estimated by conducting pseudo-static stability analyses followed by simplified deformation analyses using chart-based Newmark analyses.

3.2.5.3a Initial Stability Analyses

Once liquefaction had been determined to occur, a stability analysis was performed to assess the potential for soil movement. The computer program PCSTABL (Purdue University, 1995) was used during these analyses. Most analyses were conducted using a simplified Janbu failure method of analysis with a wedge failure surface, as this geometry was believed to be most representative of what would likely develop during an earthquake. Checks were also performed for a circular failure surface, using the modified Bishop and Spencer methods of analysis. Both pre- and post-liquefaction strengths were used during these analyses.

Results of the pre-liquefaction studies indicate that the static FOS for the end slopes on each side of the channel were 1.5 or greater, confirming that acceptable static conditions existed. Yield accelerations (i.e., accelerations that produce FOSs of 1.0 on postulated failure surfaces in the pre-liquefaction state) were typically greater than 0.15 g , suggesting that some deformation would occur within the end slopes, even without liquefaction.

FOS values dropped significantly when residual strengths were assigned to the two liquefied layers, as summarized in Table 2. For these analyses, the geometry of the failure sur-

TABLE 2 Factors of safety resulting from stability analyses

| Case | Abutment | Factor of Safety | Analysis Method |
|--------------|----------|------------------|-----------------|
| Upper wedge | Right | 0.71 | Modified Janbu |
| Lower wedge | Right | 0.79 | Modified Janbu |
| Upper circle | Right | 0.81 | Modified Bishop |
| Lower circle | Right | 0.86 | Modified Bishop |

faces was constrained to force failure through the upper or lower liquefied zone. Results given in Table 2 are for post-liquefaction conditions.

Results of the stability analyses for the right-hand abutment indicate that, for liquefied conditions and no inertial force in the fill (i.e., after the earthquake), the FOS's range from 0.7 to 0.9 for the different assumptions of failure surface location and method of analysis. FOS values less than 1.0 indicate that lateral flow failure of the material is expected during any event that causes liquefaction in the two layers, whether it is associated with the 10 percent PE in 50-year or 3 percent PE in 75-year event. The potential for instability is similar for failure surfaces through the upper and lower layers of liquefied soil, suggesting that any mitigation procedure would have to consider displacements through each layer. In other words, it would not be sufficient to improve only the upper 6 m (20 ft) of soil where the FOS was lower, as a liquefaction-related failure could also occur at greater depths.

Given the predicted occurrence of a liquefaction-induced flow failure, it would be desirable to quantify the amount of displacement expected during this flow. Unfortunately, this is quite difficult when flow failures are predicted to occur. Neither simplified chart methods nor Newmark time history analyses can be used to compute displacements associated with flow failures. However, flow displacements could be large, and such large displacements would indicate that mitigation may be required. More detailed analyses considering both structural pinning effects and ground modifications for mitigation of displacements are discussed in the following sections.

3.2.5.3b Lateral Spread Implications Based on DESRA-MUSC Analyses

A key conclusion from the DESRA-MUSC analyses was that there is a strong likelihood that lateral spread deformations will be controlled by a failure zone in the 14-m to 15-m (45- to 50-ft) layer. Displacement time histories for a rigid block sliding on this layer (assuming a Newmark sliding block analogy) were generated for a range of yield accelerations, using input acceleration time histories generated at the base of the 15-m to 17-m (50- to 55-ft) layer. The analyses were performed using the DISPMNT computer program (Houston et al., 1987). "Upslope" deformations were suppressed, assuming a strong one-directional driving force from the embankment. At time zero, drained strengths for the liquefied layer were assumed. Strengths were degraded as a func-

tion of pore water pressure increase and reduced to the assumed residual strength of 14 kPa (300 psf) when liquefaction was triggered. As would be expected, most of the computed displacements occurred subsequent to triggering.

Results showing displacement time history plots for the 3 percent PE in 75-year event, based on the Chile earthquake, as a function of yield acceleration are shown in Figure 12. Total accumulated displacements as a function of yield acceleration are shown in Figure 13 for the three earthquake records discussed previously. These results were the basis for the remediation analyses, as described in the next section. Similar analyses conducted to assess potential failure surfaces in the depth zone of 3 m to 6 m (10 to 20 ft) gave a maximum displacement of only 18 mm (0.7 in.).

3.2.5.3c Stability Analyses with Mitigation Measures

Because it was determined in this case study that significant soil movements will occur, the liquefaction design procedure requires an evaluation of measures that will reduce the amount of movement. Two procedures were evaluated for mitigating the potential for lateral flow or spreading: (1) structural pinning and (2) ground improvement. For these analyses, the additional resistance provided by the improved ground or by the structural pinning of the soil was incorporated into the stability analyses described above. If the FOS for the revised analysis is then greater than 1.0, the yield

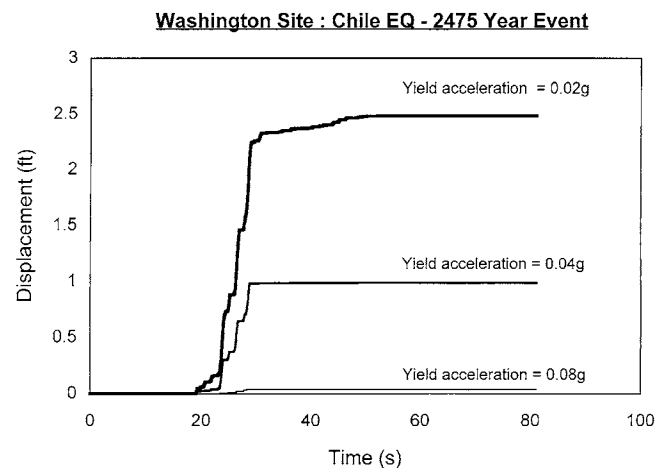


Figure 12. Displacement versus time for the 2,475-year earthquake.

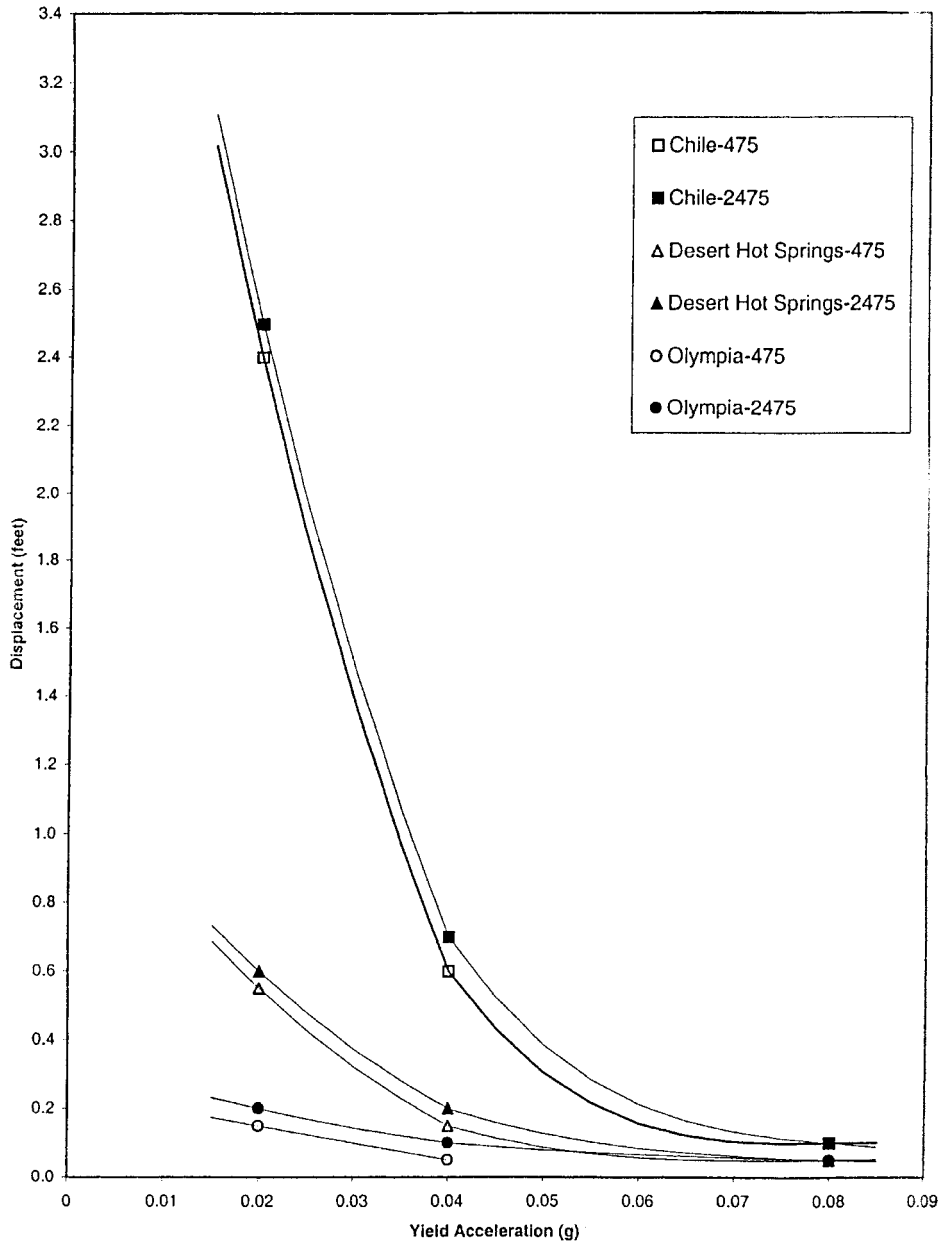


Figure 13. Displacement versus yield acceleration for the deep sliding surface of the western U.S. site.

acceleration for the mitigated condition is determined; this is then used to estimate displacements. If the FOS is still less than 1.0, then flow will still occur and additional mitigation measures are required.

For the structural pinning evaluation, shear forces were calculated as 0.4 MN (90 kips) per pile for sliding on either the upper or lower failure surfaces. (Note that procedures for determining the amount of pinning force are discussed in Section 3.2.6.2c). The abutment has 12 piles that extend through the sliding zone, resulting in 4.8 MN (1,080 kips) of additional shear reaction to sliding. Pier 5 of the bridge has 16 piles that produce 6.4 MN (1,440 kips) of pinning force.

The abutment and columns for Pier 5 are expected to develop reaction forces from passive pressure and column plastic shear. These forces were calculated to be 1.8 MN and 1.9 MN (400 and 420 kips), respectively. This reaction occurs over the 14.6-m (48-ft) width of the abutment and pile cap, resulting in a total resistance of 0.45 MN and 1.02 MN per meter (31 and 70 kips per foot) of width (or 6.6 MN and 14.9 MN (1,480 and 3,340 kips), total) for displacement along the upper and lower liquefied zones, respectively.

This reaction force was introduced into the slope stability analysis using two different methods; both procedures gave generally similar results. The two methods were as follows:

- A thin vertical slice the width of the pile group was placed at the location of the pile. This slice was then assigned a strength that gives the same total pile resistance per unit width.
- The resistance per unit width was converted into an equivalent shear strength along the shear plane in the liquefied zone, and this equivalent strength was added to the residual strength of 14.4 kPa (300 psf). For these analyses, the upper failure plane was determined to be 31.7 m (104 ft) in length, giving an added component to the liquefied strength of 14.4 kPa (300 psf). The resulting strength assigned to the liquefied layer was therefore 28.8 kPa (600 psf). For the lower zone, the surface is 40.2 m (132 ft) in length, resulting in an average pinning resistance of 25.4 kPa (530 psf) and a total resistance of 39.7 kPa (830 psf).

The FOS for the lower surface is greater than 1.0 for the post-liquefaction case, indicating that a post-earthquake flow failure would not occur. However, under the slope inertial loading, displacement of the slope could occur, and this can be assessed using the Newmark sliding block analysis once the yield acceleration is determined. The upper surface has a FOS of 1.0, indicating that flow failure is on the verge of occurring.

The yield acceleration for the lower surface was determined by varying the seismic coefficient within the slope stability analysis until the factor of safety was 1.0. This analysis resulted in the lower surface yield acceleration of 0.02 g. For the upper surface, it was assumed that the yield acceleration was zero, since the FOS was 1.0 without any additional inertial force.

For the ground improvement case, different widths of improved ground were used below the abutment. The improved ground extended through each of the liquefied zones. Soil in the improved ground was assigned a friction angle of 45 deg. This increase in strength was assumed to be characteristic of mitigation through stone columns or from a similar improvement procedure. As in the structural pinning case, two procedures were used to represent the improved zone. One was to model it explicitly; the second involved “smearing” the reaction from the improved strength zone across the failure surface by increasing the strength of the soil in the liquefied zone to give the same reaction. The resulting FOS was greater than 1.0 for all cases, indicating that flow would not occur. This allowed yield accelerations to be computed as a function of the width of the improved zone as shown in Table 3, which can then be used to estimate the displacements that can occur.

TABLE 3 Computed yield accelerations

| Width | Yield Acceleration |
|--------------|--------------------|
| 9 m (30 ft) | 0.12 g |
| 15 m (50 ft) | 0.33 g |
| 21 m (70 ft) | 0.65 g |

3.2.5.3d Displacement Estimates Based on Simplified Methods

Once lateral flow has been prevented, the amount of displacement that occurs from inertial loading on the failure wedge is estimated. Displacements were estimated for the yield accelerations given above using simplified methods. For these estimates, methods recommended by Franklin and Chang (1977), Hynes and Franklin (1984), Wong and Whitman (1982), and Martin and Qiu (1994) were used. All the methods approach the problem similarly. However, the Hynes and Franklin, Wong and Whitman, and Martin and Qiu methods eliminate some of the conservatism that is implicit in the Franklin and Chang method. For the Franklin and Chang method, it is necessary to define both the peak acceleration and velocity. The ratio of velocity to acceleration was assumed to be 30 for this study, based on typical observations from recordings of more distant events. For near-source events with epicentral distances less than about 15 km, this ratio can be as high as 60. In the case of the Hynes and Franklin method, displacements can be obtained for the mean, mean plus one sigma, and upper bound displacements. The mean displacements are used for this study.

The Martin and Qiu study was based on the Hynes and Franklin database, but included the peak ground acceleration as an additional variable in the data regression analyses. Mean values were also used in their regressions. Each of these simplified methods relates displacement to the ratio of yield acceleration to peak ground acceleration (k_{max}). For these evaluations, k_{max} was 0.24 g and 0.42 g for the 10 percent PE in 50-year and 3 percent PE in 75-year events, respectively.

The approximate displacement from the Martin and Qiu method for the 10 percent PE in 50-year event is 0.7 m (28 in.). For the 3 percent PE in 75-year event, the displacement is 1.1 m (42 in.).

The proposed LRFD provisions recommend that the Martin and Qiu results be used. The Franklin and Chang and the Wong and Whitman results provide possible upper and lower bound ranges on the displacements, but they are not thought to be as credible as the Hynes and Franklin and the Martin and Qiu results.

3.2.5.3e Displacement Estimates Using Site Response Analysis Results

Similar estimates to those obtained from the simplified methods described above may be made using nonlinear effective stress methods. These are based on displacement versus yield acceleration curves, as shown in Figure 13. As the curves are essentially identical for the 10 percent PE in 50-year and 3 percent PE in 75-year events, the displacement estimates shown in Table 4 are for both events and for the lower yield surface at the 14-m to 17-m (45- to 55-ft) depth.

These estimates are generally consistent with the estimates from the simplified methods, although the site-specific results

TABLE 4 Displacement estimates for site response analyses

| Case | Displacements in mm (in.) | | |
|--------------------------------------|---------------------------|----------|--------------------|
| | Chile | Olympia | Desert Hot Springs |
| Pile Pinning | 740 (29) | 180 (7) | 75 (3) |
| Stone Columns > 9-m (30-ft) width | < 25 (1) | < 25 (1) | < 25 (1) |

indicate that the event representative of the large mega-thrust subduction zone earthquake (Chile) will produce the largest displacements. The displacements from a moderate magnitude subduction zone intraslab earthquake (Olympia) and a moderate magnitude local shallow crustal earthquake (Desert Hot Springs) produce much more modest displacements that could be accommodated by the foundations.

3.2.6 Structural Analysis and Design

The procedures for the design of bridges for liquefaction effects generally require two components. The first is that the bridge must perform adequately with just the liquefaction-induced soil changes. This means that the mechanical properties of the soil that may liquefy are changed to reflect their liquefied values (i.e., properties such as stiffness are reduced). Design for these cases is essentially a design for structural vibration effects, and these are the effects that code-based procedures typically cover for design. The second component of the design is the consideration of liquefaction-induced ground movements.

The potential interaction or combination of these effects must be addressed in the design and, at the present time, there is insufficient understanding of the phenomena to warrant performing a combined analysis. Therefore, the methodology recommended in the proposed LRFD provisions is to consider the two effects independently; that is, in a decoupled manner. The reasoning behind this is that it is not likely that the peak vibration response and peak spreading or flow effects will occur simultaneously. In fact, for most earthquakes, the peak vibration response is likely to occur somewhat in advance of the maximum ground movement loading. Furthermore, response decoupling allows the flexibility to use separate and different performance criteria for design to accommodate the two phenomena. However, in areas where extended shaking could result in the two phenomena occurring concurrently, it may be desirable to use more rigorous coupled effective stress computer models.

3.2.6.1 Vibration Design

Vibration design was done using both the current AASHTO specifications and the proposed LRFD provisions. For the proposed LRFD approach, both the 3 percent PE and 50 percent PE in 75-year events were considered. Because the primary objective of the study was to compare the results obtained from the existing and proposed provisions, the designs were

done in a preliminary manner, which was thought to be sufficient to highlight the major differences in the two approaches. A brief summary of this assessment follows; a more detailed discussion can be found in ATC/MCEER (2000).

The bridge is comprised of multi-column bents; the existing AASHTO provisions therefore use an R-Factor of 5, while the proposed LRFD provisions allow an R-Factor of 6 (provided that a nonlinear static displacement check is done). For the 50 percent PE in 75-year event, the proposed LRFD provisions allow an R-Factor of 1.3.

For the longest columns, the proposed LRFD provisions for the 3 percent PE in 75-year event require a column steel content of 1.4 percent, which was therefore controlled by the 50 percent PE in 75-year event. The 50 percent PE in 75-year event also produced a design moment that was approximately 20 percent larger than the 3 percent PE in 75-year event. This is due to the relative magnitudes of the R-Factors and of the input spectra. For the current AASHTO provisions, the design required 1 percent steel in the columns. Similar results were obtained for Pier 2.

The foundation configuration used as starting point for both the existing AASHTO and the proposed LRFD provisions was the same. This is because one objective of the study was to evaluate a system that worked for the existing provisions when it was subjected to the effects of the larger design earthquake contained in the proposed LRFD provisions.

The pier designs were checked for displacement capacity, using an approximate “pushover” analysis. The assessment considered the superstructure and the pile caps as rigid restraints against rotation for simplicity. While this check is only explicitly required in the proposed LRFD provisions, the checks were also performed on the designs based on the current AASHTO provisions. All columns met the checks (i.e., the displacement capacity exceeded the demands).

The recommended specification also requires that the displacements be checked for P- Δ effects. In other words, the lateral shear capacity of each bent defines a maximum displacement that can be accommodated without experiencing problems resulting from displacement amplification due to P- Δ effects. It was determined that both piers are adequate with respect to P- Δ effects.

3.2.6.2 Lateral Spreading Structural Assessment and Design

In Section 3.2.5.3, the tendency for the soil near Piers 5 and 6 to move during or after a major earthquake was assessed.

Once it was determined that lateral spreading would occur, the next step was to evaluate the beneficial pinning action of the foundation system in the analysis. This section describes the method of determining the pinning force to add to the stability analyses of Section 3.2.5.3, and it describes the process of determining whether flow around the foundation will occur or whether the foundation will move with the soil.

3.2.6.2a Modes of Deformation

As discussed above, there are two potential sliding surfaces for the end of the bridge nearest Piers 5 and 6 that can occur during liquefaction. One is at the base of the upper liquefiable layer and the other is at the base of the lower liquefiable layer. These potential deformation modes must be assessed to evaluate the forces that develop in the piles and to determine the overall resistance of the bridge.

The overall foundation deformation modes may be formally assessed using models that consider both the nonlinear nature of the soil resistance and the nonlinear behavior of the piles and foundations, when subject to prescribed soil displacement profiles. In this study, the deformations and structural behavior were approximated using assumed displaced structural configurations that are approximately compatible with the constraints provided by the soil. Examples of these configurations are given in Figures 14, 15, and 16. In this example, the abutment foundation will move in a manner similar to that shown in Figure 14 because there are sliding bearings at the substructure-to-superstructure interface. In the figure, the frictional forces transferred through these bearings have been conservatively ignored.

Pier 5 will move similar to the mode shown in Figure 15. Under this displaced shape, both the columns and the piles contribute to the lateral resistance of the foundation. The columns contribute because there is an integral connection between them and the superstructure. In the current assessment, any residual displacements have been ignored. Reductions in resistance due to P-Δ effects are also shown in Figure 15, but for many of the deformations and column height

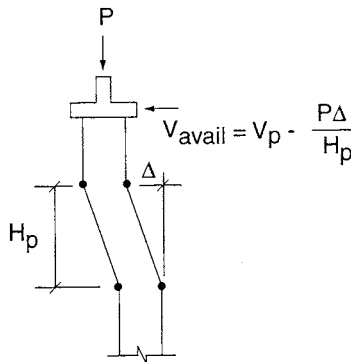


Figure 14. P-delta effects on a stub abutment.

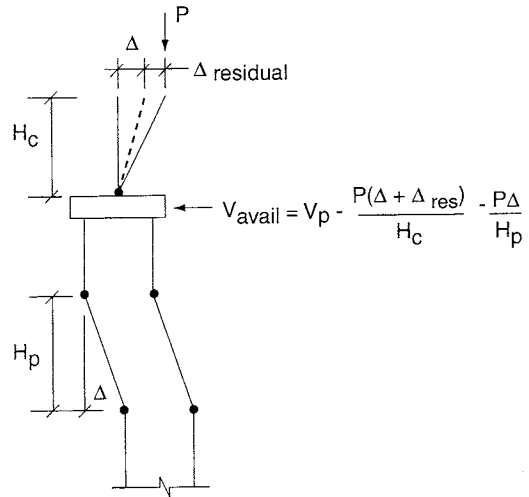


Figure 15. P-delta effects for an intermediate pier with piles and pile cap.

combinations considered in this study, this reduction is small; it has not therefore been included in the calculations.

3.2.6.2b Foundation Movement Assessment

An assessment should also be made as to whether the soil will move around the foundation or whether it will move the foundation with it. Passive capacities of the various layered soils were extracted from the p-y curves generated by conducting LPILE (Reece and Wang, 1997) analyses for the piles.⁶ These forces represent the maximum force that is exerted against the piles as the soil moves around the pile. This, then, is the upper bound limit state of the soil force that can be developed. Additionally, the maximum passive forces that can be developed against the pile caps and abutment stem walls were developed. Two total forces were developed; one for shallow-seated soil failure and one for deep failure. The shallow failure will develop approximately 4.9 MN (1,100 kips) per pile and the deep failure approximately 15.6 MN (3,500 kips) per pile at the point where the soil is moving around the foundation. By comparison, one pile with a clear distance of 9 m (30 ft) between plastic hinges can develop about 0.4 MN (90 kips) of shear at the point where a full plastic mechanism has formed in the pile. The conclusion from this comparison is that there is no practical likelihood that the soil will move around the piles. Instead, the foundations will be pushed along with the soil as it displaces toward the river channel beneath the bridge.

Intuitively, it is reasonable to expect that soil will move around a pile if there is no crust of non-liquefied material

⁶ LPILE is a computer program used to evaluate lateral response of piles subjected to loads and moments at the pile head.

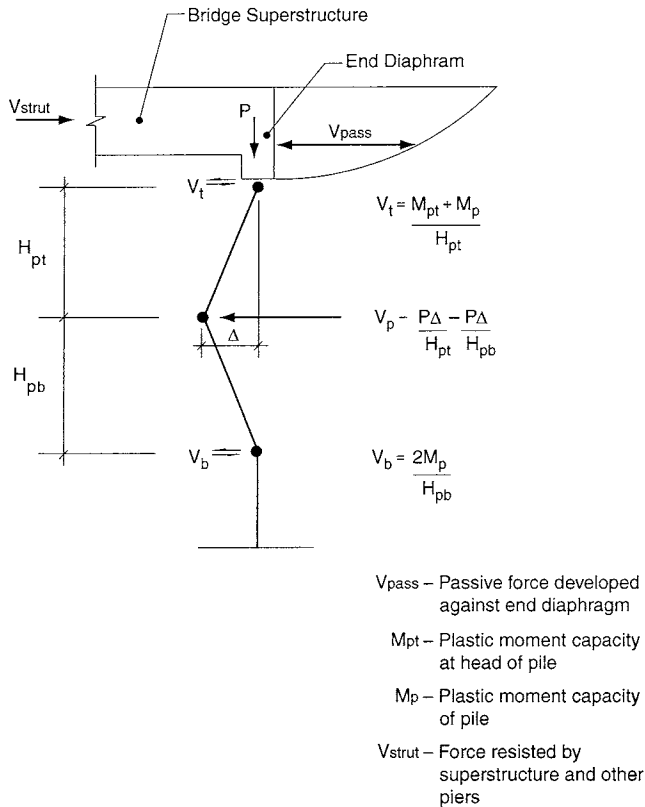


Figure 16. Plastic mechanism for an integral abutment supported on piles.

being carried along with the displacing soil. In the case examined here, there are significant depths of non-liquefied material above the liquefiable material, and it is that material which contributes to the high passive forces. Thus, if a defined crust exists, the foundations are likely to move with the soil.

There are now two questions that must be considered: (1) can the foundation systems endure the displacement that the soil produces? And (2) can the foundations appreciably reduce the soil movement via pinning action?

3.2.6.2c Pinning Force Calculation

Various types of pinning forces were discussed above and were included in the stability analyses to investigate the effectiveness of including the existing foundation pinning. The following discussion describes how the pinning force values were determined.

Figure 17 illustrates qualitatively the forces developed against the foundations and how they are reacted using the bridge deck as a strut. Two soil blocks are shown, Block A on the right and Block B on the left. Block A represents a postulated deep-seated slide that affects both Piers 5 and 6. Shears V_{p5} and V_{p6} represent the pinning shear force developed by the piles of Pier 5 and 6, respectively. Shear V_{c5} is

the shear contributed by the Pier 5 columns, while V_{pas} is the passive resistance provided by the backfill acting against the end diaphragm.

While Block A is the most likely of the two to move, Block B is shown in this example to illustrate where and how the forces transferred into the bridge by Block A are resisted. In this case, the bridge deck acts as a strut. Note that if a significant skew exists, then these forces cannot be resisted without some overall restraint provided to resist rotation of the bridge about a vertical axis.

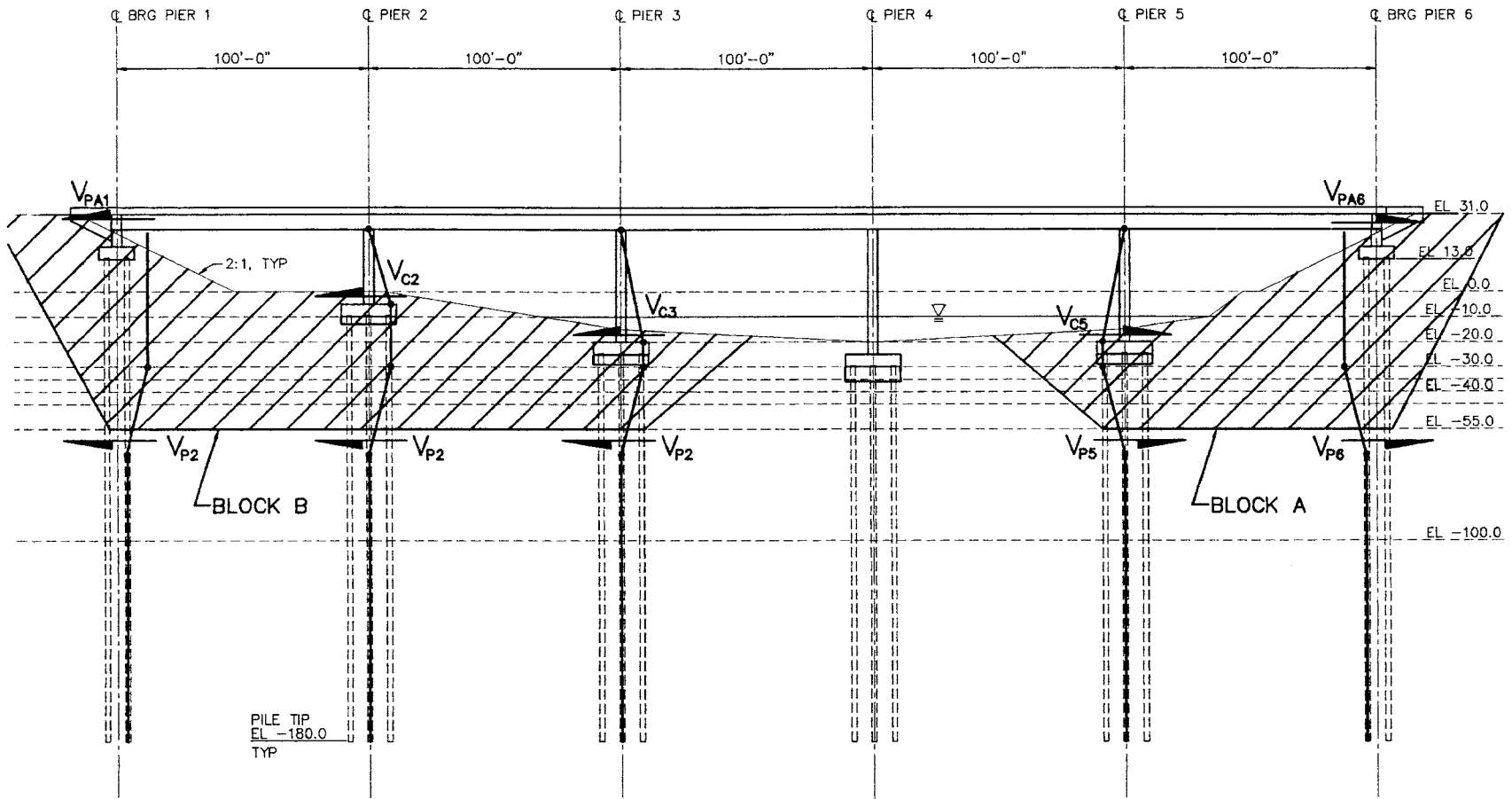
Figure 18 illustrates the pinning forces acting on a soil block sliding on the lower liquefiable layer. In this case, the piles at the abutment and Pier 5 each contribute about 0.4 MN (90 kips), the abutment itself about 1.8 MN (400 kips), and the columns at Pier 5 about 1.9 MN (420 kips). The total abutment pile resistance is 4.8 MN (1,080 kips), which corresponds to the approximate plastic mechanism shear with 9-m (30-ft) clear between points of assumed fixity in the piles. This comprises 3 m (10 ft) of liquefiable material and 5D (where D = the pile diameter) to fixity above and below that layer.⁷ The upper portion of the soil block is assumed to move essentially as a rigid body and, therefore, the piles are assumed to be restrained by the integrity of this upper block. The pile resistance at Pier 5 is determined in a similar manner, and the shear that the Pier 5 piles contribute is 6.4 MN (1,440 kips). The abutment passive resistance corresponds to half of the prescribed passive capacity of the backfill and is assumed to act against the end diaphragm. The abutment fill is assumed to have slumped somewhat due to the movement of the soil block, and thus half of the nominal resistance was judged to be reasonable. The column resistance at Pier 5 is 1.9 MN (420 kips), which suggests that plastic hinging has occurred at the tops and bottoms of the columns at this pier.

These forces [14.9 MN (3,360 kips)] represent maximum values that occur only after significant plasticity develops. In the case of Pier 5, the approximate displacement limit is 550 mm (22 in.), which comprises 100 mm (4 in.) to yield and 450 mm (18 in.) of plastic drift. The plastic drift limit is taken as 0.05 radians. The 550-mm (22-in.) displacement limit of Pier 5 is controlled by the piles. Because the piles of Pier 6 are the same, their limit is also 550 mm (22 in.) of displacement.

Because the Pier 5 columns are longer than the distance between hinges of the piles, the column displacement limits are 175 mm (7 in.) at yield, and 860 mm (34 in.) total. The fact that the piles control the displacement limit in this case implies that some margin is available in the column to accommodate any residual plastic hinge rotations that remain in the column after strong shaking stops.

Figure 19 shows the displaced shape of the foundations for a shallow (upper layer) soil failure. In this case, the distance

⁷ Fixity was assumed to develop 5D above the liquefied layer. In an actual design case, a lateral analysis using a computer code such as LPILE could be conducted to be more rigorous about the distance to fixity.



$$V_{PA1} + V_{C2} + V_{C3} = V_{C5} + V_{PA6}$$

BLOCK B < BLOCK A

Figure 17. Forces provided by bridge and foundation piling for resisting lateral spreading.

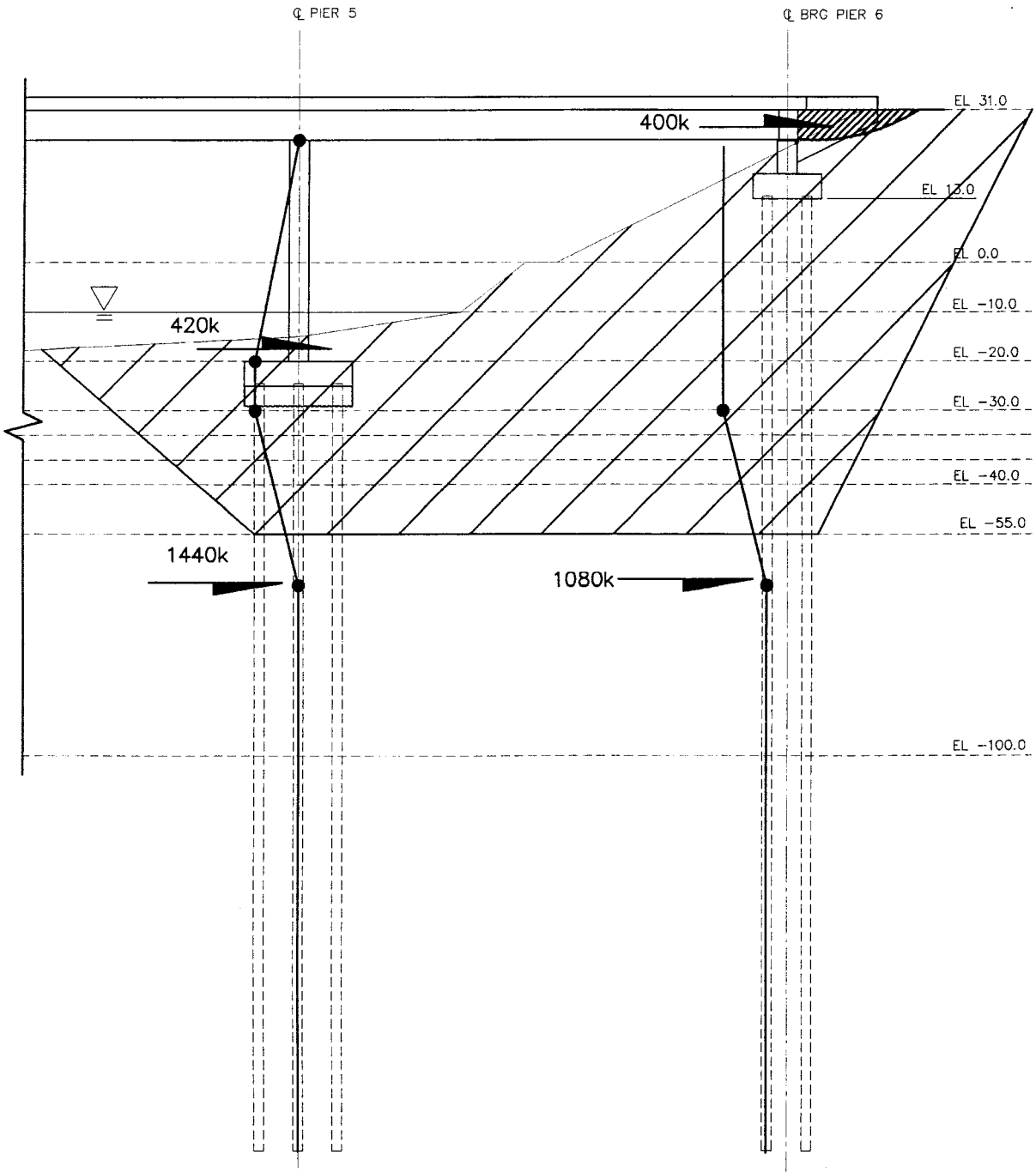


Figure 18. Piers 5 and 6 resisting lateral spreading through a deep wedge.

between plastic hinges in the piles is 9 m (30 ft), just as with the deeper failure, and thus the plastic shear per pile is 0.4 MN (90 kips). The total contributed by the piles is therefore 4.8 MN (1,080 kips) as before.

As discussed in Section 3.2.5.3, the estimated displacements for the lower or deeper failure wedges were 0.7 m (28 in.) for the 10 percent PE in 50-year event and 1.1 m (42 in.) for the 3 percent PE in 75-year event. Neither of these are within the plastic capacity of the piles, and either addi-

tional piles could be added as “pinch” piles⁸ or ground remediation could be used. It will be recalled that the yield acceleration for the upper failure was essentially zero for both the 10 percent PE in 50-year and 3 percent PE in 75-year events, which indicates that some remediation would be required to stabilize the fill and its toe for both design events.

⁸ Pinch piles refer to piles driven at close spacing to increase the shear resistance or density of a soil mass. In the Pacific Northwest, these are often timber piles.

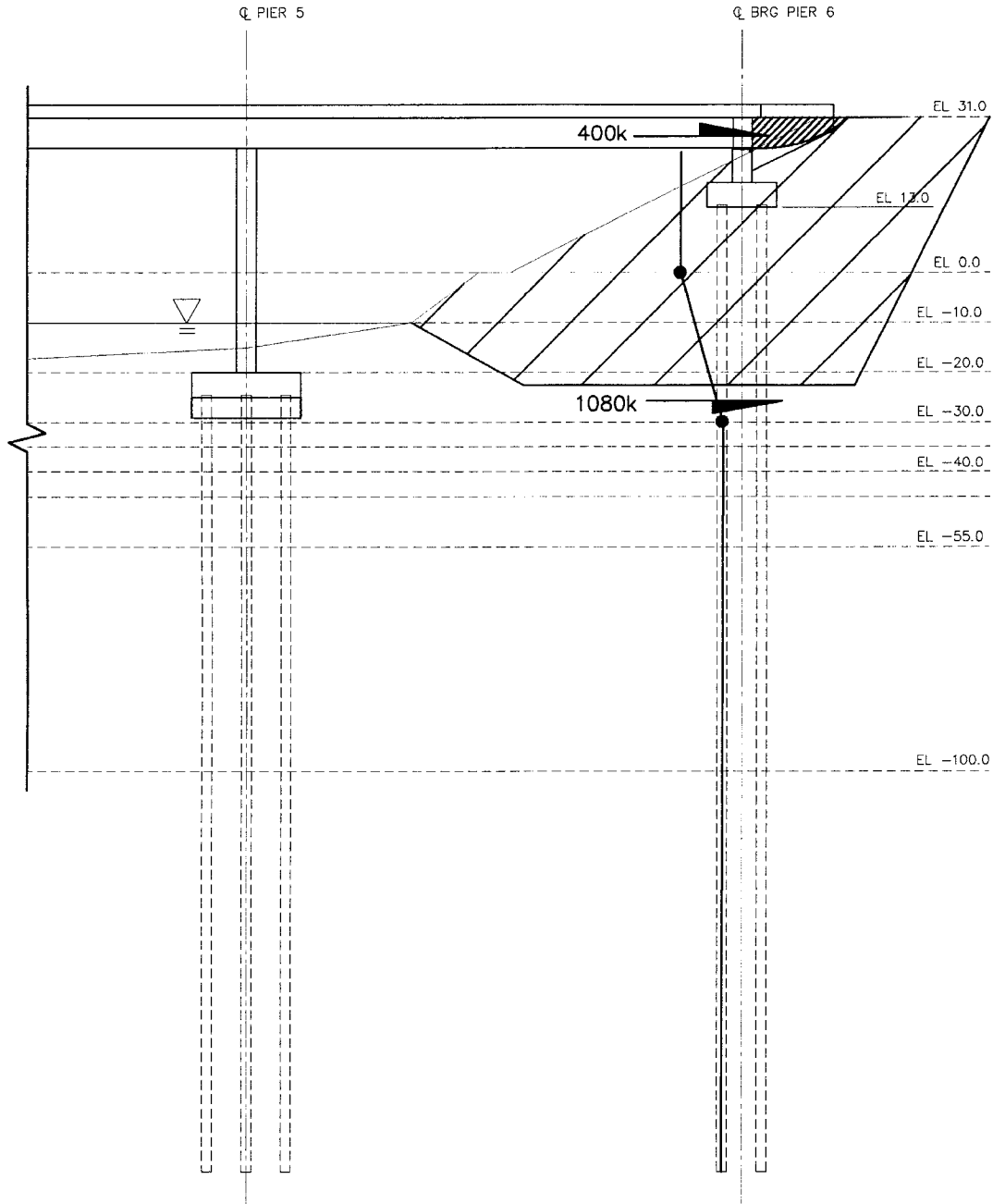


Figure 19. Pier 6 resisting lateral spreading through a shallow wedge.

3.2.7 Comparison of Remediation Alternatives

The primary intent of these analyses was to determine the potential effects of increasing the seismic design criteria from the current AASHTO 10 percent PE in 50-year criteria to the proposed LRFD 3 percent PE in 75-year MCE criteria. Liquefaction was predicted to occur for both events and, as a consequence, there is little difference in the amount of remedial work required for either the current AASHTO or proposed LRFD MCE criteria.

3.2.7.1 Summary of Structural and Geotechnical Options

Mitigation measures are assessed based on the desired performance requirement of the bridge. The first option is to assess the performance in its as-designed configuration. If this results in unacceptable performance, a range of mitigation measures is then assessed.

For this example, some form of structural or geotechnical remediation is required at the right-hand abutment because

the yield acceleration for the upper failure wedge is zero. This implies that this wedge is unstable under static conditions after the soil liquefies, which it does in both the proposed LRFD 3 percent PE in 75-year and current AASHTO 10 percent in 50-year events.⁹ Two choices for improving the conditions were considered: the use of additional piles or ground improvement with stone columns. Since the yield acceleration for the upper failure surface is so low, the more effective choice of the two is to use stone columns. These provide the combined advantage of increasing the residual shear strength of the sliding interface and they can reduce pore water pressure build up, thereby postponing or possibly eliminating the onset of liquefaction.

Because the lower failure wedge also has a relatively low yield acceleration of 0.02 *g*, it makes sense to extend the mitigation deep enough to improve the deeper soil layers as well. This low yield acceleration results in displacements of 0.7 m (28 in.) and 1.1 m (42 in.) for the current AASHTO and proposed LRFD provisions, respectively, from the simplified analyses, and displacements of approximately 0.7 m (29 in.) for both events for the time history corresponding to the mega-thrust subduction zone earthquake for the site-specific Newmark analyses. The decision to improve the deeper layers requires that stone columns extend on the order of 15 m (50 ft) deep. However, stone column remediation work will limit displacements to less than 100 mm (4 in.). This will keep the piles within their elastic range and will meet the highest level of operational performance objectives in the foundation system.

Although in this example the left-hand abutment was not evaluated in detail because the FOS from the initial stability analyses was greater than 1.0, a cost-benefit assessment would typically be made to determine if some remediation work on the left-hand abutment is cost-effective. Once a contractor is mobilized on the site, it is logical to provide improvement on both sides of the river. It may be that, upon more in-depth investigation, the stone columns could be spaced further apart or applied over a smaller width on the left-hand bank.

3.2.7.2 Comparison of Remediation Costs

As noted above, remedial work will be required for both the current AASHTO and proposed LRFD MCE events. The stone column option would likely be applied over a 9 m (30 ft) length in the longitudinal direction of bridge, since that produced acceptable displacements (i.e., within the elastic capacity of the piles) of less than 100 mm (4 in.) for the site-specific results. The width would, at a minimum, be 15 m (50 ft) and the depth would be the same. If the columns were spaced

roughly on 2 m (7 ft) centers, then 40 stone columns would be required. At approximately \$98 per linear meter (\$30 per linear ft), the overall cost of stone columns per approach fill would be on the order of \$60,000; i.e., about \$120,000 for both sides of the river if the left-hand fill were judged to require remediation.

As a rough estimate, the overall cost of the bridge, based on finished deck square-footage costs of \$100 to \$150 in Washington State, is between \$2 million and \$3 million. If the higher amount is used, due to the fact that the bridge is over water and the foundation system is relatively expensive because of its depth, the cost to install stone columns on the right-hand side is on the order of 2 percent of the overall bridge cost. If both sides were remediated, then the stone column costs would add about 4 percent to the total bridge costs. It should be noted that this additional cost will produce a foundation performance level that meets the operational criteria for both earthquake return period events.

If pinch piles are used to augment the foundation piles, the pinch piles would not need to be connected to the foundation, nor would they need to extend as deep as the load-bearing foundation piles. The per-pile costs for the foundation piles were estimated to be on the order of \$10,000 to \$12,000 each for 55-m (180-ft) long piles. If shorter piles on the order of 24-m (80-ft) long are used, their costs would be about half as much. Thus, if pinch piles were used, about 10 to 12 piles per side could be installed for the same cost as the stone column remediation option. Although detailed analyses have not been performed with these pinch piles, the amount of movement anticipated would be in the range of 150 mm to 300 mm (6 to 12 in.), rather than the 100 mm (4 in.) obtained with the stone columns. Therefore, the stone column option would appear to be the more structurally and cost-efficient option in this situation. On a specific project, combinations of the two options should be evaluated in detail.

It is useful to recognize that in this situation some remediation would be required for both the current AASHTO and proposed LRFD MCE events, because of the predicted instability of the upper failure wedge. In the case of the former, remediation is required to a depth of 15 m (50 ft) because the anticipated movement of the lower failure wedge would be on the order of 0.7 m (28 in.) for the simplified analyses and 0.8 m (30 in.) for the site-specific analysis, both of which exceed the 0.6 m (22 in.) limit. For the proposed LRFD MCE event, movement on the order of 1.1 m (42 in.) is predicted by the simplified analysis and 0.8 m (30 in.) by the site-specific analyses. Consequently, remediation is required to a depth of 15 m (50 ft) for both events. Hence the difference in cost for this site and bridge between the two design earthquakes is minimal.

3.2.8 Missouri Bridge Example

As noted earlier, a second bridge, which is located in the New Madrid earthquake seismic zone in the lower southeast

⁹ The approach fill and ground profile conditions for the bridge considered in this study are more severe than that used in the actual bridge that this example was modeled after. Thus, the implication of instability here does not imply instability in the prototype structure.

corner of Missouri, was also assessed in this study. This location was selected as it is one where a significant seismic hazard occurs, and there are numerous stream crossings and low-lying areas where the potential for liquefaction also exists. This central United States site was also included in the study since the effects of different source mechanisms and differences in shaking levels between the current AASHTO and proposed LRFD MCE events could be assessed. Since the design process and procedures used for this example were similar to that done for the Washington State example, a detailed discussion of this example is not provided herein; however, the next section provides a discussion of some of the results of the Missouri case study, in comparison to the Washington State example. Details of the Missouri case study can be found in ATC/MCEER (2000).

3.2.9 Summary and Conclusions Resulting from the Liquefaction Case Studies

There are two phenomena that must be considered in the design of a bridge on a liquefiable site. The first is the traditional vibration design based effectively on the response spectra for the site, which corresponds to the design process considered in the current AASHTO provisions. The second phenomenon is lateral forces induced by lateral movement in the soil, either by flow sliding or lateral spreading.

For the proposed LRFD MCE event, the recommended performance objective is “life-safety” and, for many bridges, deformation can be allowed in the foundation for the lateral spreading case. Mitigation measures are able to achieve higher levels of performance, so that piles remain within their elastic capacity when desired. The vibration cases are designed for inelastic response above ground that occurs at inspectable locations. It is believed that allowing some inelastic action in the presence of large spreading movements during the MCE event is necessary. Because spreading-induced deformations are displacement-controlled, instability of the system is unlikely even though some damage may exist in the foundations. The implication of this decision is that a bridge and its foundations may need to be replaced after an MCE event; this, however, avoids a significant expenditure of funds to prevent displacements from occurring in the first place.

Design for vibration and for lateral spreading is done independently, as coupling of the vibration load case and the spreading load case is not usually warranted. The vibration design is considered separately from the spreading design because it is unlikely that the maximum vibration effect and the maximum lateral spreading forces occur simultaneously. This de-coupled approach is considered reasonable with respect to the current state of knowledge and practice.

The recommended approach is to determine the ground movements that are likely to occur at the site. These should include the effects of altered site configurations (e.g., through the addition of fills) and the beneficial effects that can occur due to pinning of piles. The prediction of lateral spreading

can be made using currently accepted simplified methods or site-specific analyses, as outlined in ATC/MCEER (2000). It was noted in the two cases studied here that there can be significant variation in the predicted displacements using different methods; this indicates that the designer should be aware that there can be a significant range in anticipated movements. Although refined accuracy is not considered warranted, the beneficial resistance of the substructure should be included in the assessment of movements. The substructure is then assessed for the predicted movements and, if it cannot tolerate the predicted displacements, ground or structural remediation should be used.

It is important to recognize that the two case studies conducted in this project are based on conditions whereby lateral spreading is parallel to the superstructure, which is typically in the “strong” direction of the bridge. If the spreading effect is skewed with respect to the superstructure or piers, then the skew must be accounted for in determining the likely plastic mechanism that will control.

The conclusions from this study on the effects of liquefaction when the design earthquake return period is increased from the existing AASHTO 10 percent PE in 50-year earthquake to that of the proposed LRFD 3 percent PE in 75-year MCE earthquake are summarized as follows:

- For both the Washington State and Missouri examples, there were additional costs required to accommodate liquefaction when the bridge was properly designed for the current AASHTO earthquake and was then subjected to the proposed LRFD MCE earthquake for the life-safety level of performance.
- For the Washington State example, liquefaction occurred for the current AASHTO event, and it was necessary to provide stone column mitigation measures in the upper 9 m (30 ft) or so. This would also most likely be necessary at both abutments (only one was assessed in this example). The cost for stone columns at both abutments was estimated to be on the order of 2 percent of the total bridge cost. For the proposed LRFD MCE event, similar measures were required with the depth of the stone columns extended to 15 m (50 ft). The estimated cost of this remediation is on the order of 4 percent of the total bridge cost.
- For the Missouri example, liquefaction did not occur during the current AASHTO 10 percent PE in 50-year event. In addition, the bridge was capable of meeting the liquefaction requirements for the proposed LRFD 3 percent PE in 75-year MCE event, with liquefaction occurring at a depth of 6 m to 12 m (20 to 40 ft), through pinning action of the piles. By allowing some inelastic deformations in the piles, no ground improvement was required.
- For both the Missouri and Washington State sites, the highest operational level of performance can be achieved in the foundation system (i.e., piles remain elastic) for the proposed LRFD 3 percent PE in 75-year MCE event by

improving the ground with stone columns. This improvement can be achieved for less than 5 percent additional cost in the case of the Washington State site and less than 10 percent additional cost in the case of the Missouri site.

- This study demonstrated the beneficial effects of considering the resistance that the bridge substructure provides against lateral movement of soil by pile pinning. These effects can be significant and should be considered in assessing lateral soil movements. The study also showed the benefit from allowing inelastic behavior in foundations under the action of lateral ground movement. For many cases, relatively large ground displacements can be accommodated by the bridge without collapse.

There have been considerable advances in the state of the art in assessing impacts of liquefaction since the current AASHTO seismic design provisions were initially developed. Many of these advanced methods have been included in the proposed LRFD provisions and were used in the two case studies. They are relatively easy to use and permit a much better understanding of the effects of liquefaction and lateral spreading. Among the benefits of applying these methods are

- Improved ability to estimate the displacements that may occur as a result of lateral spreading.
- The ability to incorporate the beneficial effects of pile pinning and ground improvement methods in resisting lateral flow movements.
- The ability to perform nonlinear stress analysis time-history studies to better understand the sequence of events that occur during liquefaction and changes in ground motions that occur as a result.

There are, however, significant implications in implementing the proposed LRFD 3 percent PE in 75-year earthquake as the upper-level design event. Among these are the fact that a larger number of areas in the United States will require detailed seismic design and a liquefaction assessment. In general, liquefaction should be considered for bridges classified as Seismic Design Requirement (SDR) 3 or greater when a site has a mean earthquake magnitude contributing to the seismic hazard greater than 6.4. If the mean magnitude is less than 6.0, then liquefaction does not need to be considered. Between mean magnitudes of 6.0 and 6.4, liquefaction may or may not need to be considered, depending on the combinations of soil type and acceleration levels. Although liquefaction must be assessed in certain designs, the Missouri bridge example demonstrated that a bridge may meet the recommended performance requirements of the new provisions without a significant expenditure of funds. However, due to the limited number of bridges assessed in the study, it is difficult to draw wider implications from it.

It should be recognized that the approach recommended herein for very large, infrequent earthquakes is a departure

from the traditional approach of preventing damage in the foundation. For ground movements on the order of those assessed in this study, either remediation will be necessary or allowance of some inelastic action in the foundation will be required. It is recognized that only two examples were considered in this study, and that, with time, refinement will be possible as more structures are studied and designed. It is also recognized that the prediction of earthquake-induced ground movement is approximate at best, and much remains to be learned with respect to how one can make more accurate predictions. In seismic design, the greatest uncertainty lies in the methods of predicting ground displacements; this can be seen in the variations resulting from the simplified methods and the more precise nonlinear analyses conducted in this study. However, it is thought that the proposed approach is a reasonable beginning for rationally designing for such earthquake-induced hazards.

3.3 ASSESSMENT OF IMPACTS

The proposed LRFD specifications constitute a significant advance over the existing AASHTO specifications for the seismic design of bridges. They also represent a major departure from the philosophy and design approach employed in the existing specifications. Several key differences are the use of the MCE ground motions for design, multi-level seismic performance criteria, nonlinear displacement capacity checks, and, most importantly, the incorporation of a more comprehensive design specification and companion commentary. This last item is significant because, at first glance, the new provisions appear to be more complex than those currently in use. However, the design process is similar to that currently in use, and the appearance of a higher level of complexity in the provisions is the result of providing more guidance to designers with the goal of producing greater consistency in design than is currently being achieved.

The assessment of the impacts of the proposed LRFD specifications on the seismic design of bridges is split into two topics: the impact on engineering design effort, and the impact on the resulting sizes and proportions of bridge components (and related materials and construction cost impacts).

Over the course of the NCHRP Project 12-49, a number of activities were undertaken to assess these two types of impact. Two fully detailed design examples were prepared to illustrate the application of the provisions, and a parametric study was conducted to assess the impact of the provisions on the sizes and configurations of typical bridge substructures. The development of the design examples and the conduct of the parametric study both provided insight into the engineering effort required by the proposed LRFD specifications and on the differences the specifications will make on overall bridge designs. Additionally, a study of impacts from liquefaction was conducted to aid the development of those provisions, a problem which has historically challenged bridge designers.

The two design examples that were prepared under NCHRP Project 12-49 follow the format and approach used in the seven design examples prepared under FHWA sponsorship in 1997, which were based on the AASHTO *Standard Specifications* Division I-A provisions. One bridge from that earlier set of seven examples (Design Example 2 (FHWA, 1997)) was re-worked on the basis of the proposed LRFD specifications. The re-worked version has been titled Design Example 2LRFD to denote its relationship to the previous Division I-A example and its conformance to the proposed LRFD specifications. The second bridge design example was developed as part of the liquefaction study effort; it provided the basis for part of that study and for Design Example 8, which was so named to distinguish it from the previous seven FHWA examples.

3.3.1 Design Engineering Effort Impacts

3.3.1.1 Format

The overall process inherent in applying the proposed LRFD specifications is similar to that of the current AASHTO specifications. For design, the traditional force-based approach has been retained, although nonlinear displacement-based checking is encouraged in order to assess the ability of a bridge to adequately perform under the design earthquake. The retention of the current approach should help designers apply the proposed LRFD provisions with minimal training. This is important because there are two major changes that designers will be exposed to: first, that these provisions are intended to be integrated into the AASHTO *LRFD Bridge Design Specifications*, which itself is relatively new to many designers (particularly for substructure design); and second, that the proposed seismic design specifications are substantially different in format and content than existing AASHTO provisions. Therefore a complete departure in design approach would not be simple or inexpensive to implement for many agencies.

3.3.1.2 Ground Motions

The proposed LRFD specifications have adopted the design spectra, soil site factors, and site classifications used by NEHRP and many of the more recent building codes. This provides overall consistency between code agencies and structure types and should provide a framework that can be used and maintained in a more uniform manner. For example, many bridge design agencies use geotechnical and structural consultants who also are involved with building-related design work. Consistency between the building, port and harbor, industrial, and bridge design communities should lead to more uniform and simpler site seismic response characterizations, and less ambiguity in interpretation of likely ground motions. Furthermore, with appropriate levels of bridge community input and oversight, the national mapping effort conducted and regularly updated by the USGS will better address bridge

design-community needs, possibly alleviating local bridge-owning agencies from having to develop their own seismic hazard maps.

3.3.1.3 Hazard Categories

The current AASHTO design provisions primarily delineate the seismic hazard through the use of a firm soil/rock acceleration coefficient and only minimally recognize the effects of the soil on overall site and bridge response. Consequently, AASHTO's current Seismic Performance Categories (SPCs), which essentially control the design process and are based on a peak acceleration coefficient, can be mapped throughout the country based only on that coefficient. This implies that the SPCs are effectively constant for all bridges in a given region. By contrast, the approach incorporated into the NEHRP provisions and also employed in the proposed LRFD specifications consider soil effects in establishing seismic design requirements. Therefore, a given region may have multiple seismic hazard levels depending on the soil types present. To this end, a state or other transportation agency that essentially did not have to consider the higher seismic design requirements of the current AASHTO provisions may find that more rigorous requirements apply for some sites with softer soils in the proposed provisions. This change in seismic design procedures is rational and defensible, given the observed seismic performance of structures on soft sites in recent years. The result of this is that some bridge designs in some locations may require significantly more attention to seismic performance than previously considered.

3.3.1.4 "No Seismic Demand Analysis" Provisions

The proposed LRFD specifications provide what should amount to considerable relief in design effort in regions of the country with low to moderate seismic demand over that required by the current AASHTO design provisions. In the current AASHTO provisions, the classification by SPC required either minimal detailing (e.g., minimum seat widths or connection forces), or a full seismic analysis and design. In the proposed LRFD specifications, an intermediate category has been added that is based on satisfying capacity design principles to ensure a proper hierarchy of structural strength, but does not require a formal seismic demand analysis. This should make the design of bridges that qualify for this category simpler, while turning the focus of the design effort from that of pure analysis to one of providing a satisfactory seismic resisting system with minimal design effort.

3.3.1.5 Capacity Spectrum Procedure

Another new design procedure has been added for structures that qualify as "very regular" in geometric configuration.

The procedure is called the capacity spectrum method, and it recognizes the single-degree-of-freedom behavior inherent in such regular structures. This method greatly simplifies the analysis process, and directly incorporates the effects of inelastic action in the demand side of the analysis. The method also is that used for seismic isolation design, which has been included in the proposed LRFD specifications. Thus, there is now a consistent basis for design among conventional structures and those that employ base isolation.

3.3.1.6 *Displacement Capacity Verification (Pushover Analysis)*

The proposed LRFD specifications provide procedures for conducting a displacement capacity verification, more commonly referred to as “pushover analysis.” This procedure provides direct checks of a structure’s ability to accommodate expected seismic displacements. It also allows a limited reduction of the effective lateral design force, if it can be shown that the structure still has adequate displacement capacity. This feature can potentially provide reduced structure costs, although the design engineering effort will typically be increased in order to verify displacement capacity.

Currently, few guidelines exist for conducting a pushover analysis. Thus, design criteria and procedures are established by an agency for almost every project. For instance, Caltrans almost exclusively uses pushover analysis procedures, and they have developed specific criteria consistent with their design approach (Caltrans, 1999). The proposed LRFD specifications provide a formal methodology for performing pushover analyses that are consistent with the rest of the provisions. They also incorporate many of the criteria and approaches that Caltrans uses. As a result, the proposed LRFD specifications should provide savings in engineering effort in cases where displacement verification is to be employed, because the criteria and approach have been standardized and are readily available.

Additionally, pushover analyses are often analytically intensive. Codification of the approach should encourage software developers to include standardized procedures within their software products, which could result in additional economy in engineering effort. For example, several of the most widely used structural analysis programs have added pushover modules to their software packages, and others will likely do the same if the proposed LRFD specifications are adopted by AASHTO (as either a guide specification or within the AASHTO LRFD specifications). This has occurred within the building analysis and design community where FEMA guidelines outline design procedures and acceptance criteria, and these have been incorporated into recent software releases.

3.3.1.7 *Two-Level Earthquake Design and Performance Criteria*

The proposed LRFD specifications include requirements to design for two levels of earthquakes with two levels of min-

imum performance based on these design ground motions. For the upper-level MCE event, the bridge will likely sustain significant damage in components that are designated to yield. For the frequent (expected) earthquake, the structure should remain essentially elastic and undamaged. Typically, one of these two conditions will control the design (although in some cases non-seismic load combinations may actually control); therefore, once designers are familiar with the new provisions, they should be able to quickly identify the controlling case and will spend most of their effort satisfying the structural requirements for this event while simply checking the adequacy of the other. The controlling event will depend on the relative magnitudes of the design ground motion and on the chosen lateral-force resisting system, which ultimately governs how much inelastic action is permitted. For instance, it is expected that the frequent event will rarely, if ever, control in the eastern United States, but may in some cases control in the western United States. Therefore, the fact that two design earthquakes are used does not necessarily mean that the design time will be doubled; at most, only a slight increase in design time is anticipated.

3.3.1.8 *Geotechnical Engineering and Guidance*

There is a significant amount of geotechnical guidance provided in the proposed LRFD specifications and commentary (very limited guidance is provided in the current AASHTO provisions). This information is based on and integrates a significant body of state-of-the-art data and methodologies. Many geotechnical engineers are currently applying some of this material in piecemeal ways, as the material is not integrated into a single, easily accessible document. The proposed LRFD specifications will provide consistent information that can be used in a uniform manner throughout the United States.

3.3.1.9 *Load Combinations*

As a “service” level earthquake is included in the design process and since the LRFD load combination methodology includes the possibility of extreme events in combination with other service loads, the proposed load combinations for seismic design have been kept as simple as possible. This was an explicit attempt to avoid undue design effort for loads that are imprecisely known. Additionally, superstructure forces must be assessed for seismic loading but, again, simplified load combinations were provided to avoid unnecessary effort. Therefore, the load combinations in the proposed LRFD specifications are not expected to significantly add to the design effort.

3.3.1.10 *Response Modification Factors (R-Factors)*

The use of R-Factors to reduce elastically derived seismic forces has been retained for several of the design methods

allowed by the proposed LRFD specifications. Overall, the factors used for design in the MCE event are higher than those allowed by current AASHTO provisions. However, recent research demonstrates that the use of R-Factors that are higher for multi-column structures versus those for single-column structures, as used in the current AASHTO provisions, may not be correct. Thus, the proposed LRFD specifications specify the same R-Factor for both single- and multi-column structures. Specifically, in the proposed LRFD provisions, R is set equal to 4 for a modal analysis and equal to 6 if a displacement verification check is performed. By contrast, current AASHTO provisions allow an R equal to 3 for single-column bridge bents and equal to 5 for multi-column bents. The design effort will not be impacted by these changes in R-Factors.

3.3.1.11 Level of Detail in the Proposed LRFD Specifications

Throughout the proposed LRFD specifications, more definitive guidance is provided than in current or previous AASHTO seismic design specifications. Alternate approaches are provided for cases where ideal seismic design approaches cannot be met, providing the designer with a significant amount of flexibility. An example of this is when it becomes difficult or costly to keep all column plastic hinging above ground.

In order to accommodate the increased level of flexibility, the proposed LRFD specifications are more voluminous than existing specifications. However, this should allow a designer more options and guidance, and should save time that would have been spent trying to develop or customize an approach for cases not covered explicitly by the provisions. Significant time is often expended on a small portion of a design project, as limited or no guidance is provided in the governing design provisions. The need to develop custom criteria or approaches should be reduced through use of the proposed LRFD specifications.

3.3.2 Structural Configuration, and Material and Construction Cost Impacts

This section discusses the impacts to structural configurations resulting from use of the proposed LRFD specifications. These impacts are discussed in terms of the results of the parameter study, liquefaction case study, and design examples prepared under NCHRP Project 12-49.

3.3.2.1 Impacts Based on the Parameter Study

The parameter study considered both single- and multiple-column bents founded on spread footings, with five locations throughout the United States assessed for their regional seismic hazard. Varying height, diameter, and end restraint con-

ditions were considered in the study, as were varying restraint conditions imposed by the soil at the abutments. In general, the comparisons of size, longitudinal steel content, and costs showed that the current AASHTO provisions and the proposed LRFD specifications resulted in designs that were remarkably similar. In some cases, one specification produced designs requiring more concrete or steel, but in other cases, the other specification controlled. It is important to remember, however, that the current AASHTO specifications are based on a ground motion with a 10 percent PE in 50 years (equivalent to a 500-year return period earthquake) while the proposed LRFD specifications are based on design ground motions with a 3 percent PE in 75 years (the MCE event, which is equivalent to a 2,500-year return period earthquake).

Although the design ground motions for the MCE case have been effectively increased in the proposed LRFD specifications in terms of return period, there are counteracting actions that result in relatively similar design configurations when compared to designs produced by the current AASHTO specifications. These include the fact that mapped acceleration values have actually decreased in large areas of the country at the same equivalent return period; response modification factors (R-Factors) are larger for many substructure types and configurations; the use of effective (cracked) section properties have been included in the proposed provisions; ϕ factors have been increased; minimum longitudinal steel ratios have been reduced; and overstrength factors have been significantly revised.

Thus, for the cases investigated, the range in material and construction cost difference are generally estimated to be less than about 10 percent for the proposed LRFD MCE design case. In some cases, design and analysis options, such as those accounting for the passive resistance of the soil behind the abutment, had a larger impact on the final design than using the MCE earthquake.

3.3.2.2 Impacts Based on the Liquefaction Study

The parameter study illustrated that the controlling elements of a bridge design produced using the proposed LRFD specifications may not be all that different than the elements controlling designs produced by the current AASHTO specifications. The liquefaction study, which had among its goals the development of rational methods for assessing and designing to mitigate liquefaction impacts, demonstrated that many of the effects of liquefaction could likely be handled rather economically by applying the higher-order design approaches recommended in the proposed LRFD provisions. This requires, however, a shift in design approach from that of maintaining a simple factor of safety against the occurrence of liquefaction during the design earthquake, to one of rationally accommodating the ground softening and displacements that accompany liquefaction. In order to do so, the designer must account for a bridge foundation's beneficial impact on restraining liquefaction-induced ground movement and include

allowances for inelastic action in the foundation. Allowing inelastic action in the foundation provides the greatest displacement capacity and the most resistance for limiting liquefaction effects. The proposed specifications also recognized that few recent bridge failures can be directly attributed to liquefaction; therefore, allowing significant inelastic action within the foundation as a means of withstanding liquefaction effects is deemed rational and cost-effective.

Overall, it is expected that the impact on in-place design and construction costs should be minimal; however, in some cases, ground improvement techniques such as the use of stone columns may be required.

The liquefaction study compared the assessment of liquefaction-induced effects by simple semi-empirical methods with those of more advanced and computationally intensive methods. The simpler methods yielded reasonable results and are therefore considered viable for use in design to resist liquefaction effects.

For the cases that were investigated in the study, the overall cost impacts on bridge designs, when compared similar current AASHTO designs, were generally less than 10 percent of the total estimated bridge cost; in some cases, the overall costs were less than 5 percent different. The benefit to calculated bridge performance, however, was substantial, resulting in little or no damage to foundations, even at the MCE earthquake event (3 percent PE in 75 years).

3.3.2.3 Impacts Based on the Design Examples

The two design examples produced under NCHRP 12-49 show the application of the proposed LRFD specifications for designing new bridges. One of the design examples is a steel plate girder bridge founded on wall piers in a relatively low seismic hazard region of the country. This example also provides an analysis of the same bridge when supported on multi-column bents. The second bridge design example is a concrete box girder structure on multi-column piers with piles in liquefiable soils, located in a region of relatively high seismicity. Both bridges had been previously designed using the current AASHTO specifications, making a direct comparison of impacts between the two specifications relatively easy.

The first example, designated Design Example 2LRFD, is the steel plate girder structure in the low seismic hazard zone. Because the location of the original bridge design example conducted using the AASHTO Division I-A provisions was so close to the seismic hazard level boundaries in the 1996 USGS maps, two seismic analysis and design procedures (SDAP), as specified in the proposed LRFD provisions, were used (SDAP A2 and SDAP C). These help illustrate differences between the various SDAP requirements in the proposed provisions. Furthermore, as noted above, Design Example 2LRFD was prepared assuming two different types of substructures: (1) wall piers with elastomeric bearings and spread foundations and (2) multi-column bents with conventional bearings and spread footings. By preparing the design example in this

fashion, one could contrast the requirements and results of different substructure systems. This is important for the proposed LRFD specifications because elastomeric bearings are subject to seismic design requirements that involve many of the principles of seismic isolation design.

The original Design Example 2 (based on current AASHTO Division I-A provisions) had been prepared using the multi-modal analysis method, which considered the stiffness of the elastomeric bearings in the analytical model. This is somewhat more complex than that required for either SDAP A2 or SDAP C of the proposed LRFD specifications. In fact, the engineering effort required by the proposed LRFD specifications for this bridge example is much simpler than that required by current AASHTO seismic design specifications. The requirements for SDAP A2 are similar to those of the current AASHTO Division I-A's seismic performance category (SPC) A, in that only seat width considerations and bearing connection force design are explicitly required. The proposed LRFD SDAP C provisions require demand analysis using only single-degree-of-freedom (SDOF) principles for regular bridges. This procedure greatly simplifies the demand analysis, especially with respect to multi-modal analysis. To this end, the design effort for this type of regular bridge in seismic zones with relatively low seismic hazard is substantially easier and less time consuming than that required by even the SPC B provisions of AASHTO Division I-A. Therefore, the engineering effort required by the proposed LRFD specifications is expected to be less than that required by the current AASHTO provisions for many bridges in low-to-moderate seismic hazard areas within the United States.

The proposed LRFD specifications also account for the beneficial aspects of elastomeric bearings when used between the superstructure and substructure. Effectively, elastomeric bearings behave as isolators, resulting in a reduction of the forces that are transmitted to the superstructure from ground shaking. It is therefore rational to take advantage of seismic isolation criteria when designing these elements.

The resulting size of substructures and magnitude of design forces are less for the proposed LRFD specifications than those obtained from the Division I-A design, despite the use of the larger MCE design event in the LRFD provisions. This is partially a result of decreases in acceleration values in the proposed LRFD specifications at similar levels of seismic hazard in northeastern regions of the country. Pier wall sizes were controlled by minimum longitudinal steel requirements for both designs; these minimums have not changed from current AASHTO design requirements. Bearing sizes could have been reduced, but this would have required design refinements that were not incorporated into the design example.

The second design example, designated as Design Example 8, was the same bridge as that used in the liquefaction case study conducted on the Washington State bridge. Design Example 8, however, illustrates the application of the provisions for the nonliquefied condition of the soils around the bridge foundation.

The design effort required for Design Example 8 was not significantly more involved than that required by the current AASHTO provisions. For both methods, significant effort is required to develop foundation properties that reflect or bound expected behavior. For this design example, the displacement verification check (i.e., the pushover analysis) was included, and the application of the method was relatively simple and straightforward due to the restraint provided by the pile caps. Commercially available software was used to execute the pushover analysis and, while additional effort was required beyond a basic modal analysis, the additional effort was not considered significant (i.e., on the order of 10 percent to 15 percent of the overall design effort).

For this structure, the same column sizes were used for both the current AASHTO Division I-A design and the proposed LRFD design. The column longitudinal steel requirements were higher for the proposed LRFD specifications than for the Division I-A case, so steel costs would be higher. Column heights varied from bent to bent, and several of the shorter bents also required more shear steel, due primarily to the antibuckling requirements that were added in the proposed LRFD specifications. These provisions have been calibrated to provide constructible volumes of steel, however; thus, even though more longitudinal and transverse steel may be required, overall material costs should not be that different than current AASHTO designs require.

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS FOR IMPLEMENTATION

4.1 CONCLUSIONS

NCHRP Project 12-49 has resulted in the development of an advanced set of specifications for the seismic design of highway bridges, compatible with the current AASHTO *LRFD Bridge Design Specifications* (AASHTO, 1998). These proposed specifications should provide more uniformity and a more rational design basis for bridge seismic designs conducted throughout the United States.

The proposed LRFD seismic design specifications are based on the most current scientific and engineering knowledge regarding seismic hazard representation, foundation and soil behavior, design and analysis methods, material performance, and component and system detailing. They also provide a level of uniformity with the philosophies and design approaches contained in the codes and standards that have been implemented for the seismic design and detailing of other structure types, including buildings, ports and harbors, and pipeline systems.

The approaches and provisions contained in the proposed LRFD specifications are, in some cases, quite different than the approaches and provisions contained in the current AASHTO specifications. In some cases, this new material may be more liberal than existing provisions; in other cases, more restrictive. Overall, it is believed that the proposed LRFD specifications will result in bridge designs that are similar in cost to those prepared with the current AASHTO provisions. There will be cases where designs constructed with the proposed provisions cost more than existing provisions cost, but this is directly related to the advances made in seismic hazard knowledge in some regions of the country along with advances in the knowledge of site response on ground motion, and the fact that the seismic design input is significantly larger than the current AASHTO specifications recognize. However, in these cases, the proposed LRFD specifications will help ensure that bridges constructed in these locations perform adequately under the high levels of ground shaking that have been actually experienced in past earthquakes, and are likely to occur again in the future.

However, to actually implement the proposed LRFD specifications, there will be a need for training by bridge designers so that they can become fully conversant in and proficient with the new provisions. After this initial learning curve, it is expected that the differences in design engineering effort will

be minimal when compared with the effort currently required for seismic design.

The proposed LRFD specifications provide a great deal of overall guidance that is not included in current AASHTO specifications. One benefit of directly providing such guidance in the specifications is reduced design time, which could result in engineering cost savings in cases where the design engineer was previously left to develop project-specific criteria and methodologies.

4.2 RECOMMENDATIONS FOR IMPLEMENTATION

It is believed that the proposed LRFD seismic design specifications are as advanced and complete as can be developed at this time. However, as with any new and comprehensive set of engineering procedures or specifications, there is likely the need for a “shakedown” period during which the proposed specifications are tested and used in a series of trial designs. A trial design program conducted prior to adoption of the proposed specifications will help identify areas in the specifications for which provisions or guidance is either unclear or, in extreme cases, inadequate, and therefore allow for an opportunity to improve the specifications before they become mandatory.

It is therefore suggested that the proposed specifications be considered for adoption by AASHTO as a *Guide Specification* in the interim, during this testing and shakedown period. To assist in this, the ATC/MCEER Joint Venture with funding provided by the Federal Highway Administration (FHWA) has prepared a “stand-alone” version of the proposed LRFD provisions, similar in structure to the stand-alone seismic design specifications contained in the AASHTO *Standard Specifications* Division I-A provisions. In addition, MCEER (again with FHWA funding), may initiate a “trial design” process involving a number of AASHTO states in the fall of 2001 and spring of 2002, in order to provide the initial shakedown required to gain confidence in the proposed provisions.

In order to adequately implement the *Guide Specification* and ultimate LRFD provisions, there will be a need to develop and offer a comprehensive training course on the use of the new provisions. It is recommended that the FHWA pursue this need. In addition, the two design examples produced

under NCHRP Project 12-49 can assist designers in understanding aspects of the new provisions. These, supplemented by the trial designs produced by state transportation agencies under the MCEER-sponsored shake-down process, should be augmented with additional design examples exercising all key provisions of the specifications in various seismic hazard regions with a range of soil conditions, and for a wide variety of structural types (materials and system load path configurations).

There is one concern that must be raised regarding the potential adoption of the proposed LRFD provisions into the current AASHTO *LRFD Bridge Design Specifications*. The scope of work for NCHRP Project 12-49 was limited to highway bridges and components directly attached to high-

way bridges (e.g., abutments and wing walls). However, the work on the project did not look at the impact that the new provisions will have on free-standing retaining structures and walls, buried structures, or components attached to the bridge like light standards and railings. Each of these structure types and provisions for their design contained in the AASHTO LRFD specifications will need to be assessed before the MCE provisions can be adopted uniformly by AASHTO. This is another reason why it is encouraged that the proposed LRFD provisions be adopted as a *Guide Specification*—the *Guide Specification* would allow the use of the advanced seismic design approaches and methodologies for highway bridges, but would not affect the mandatory design requirement for these other structures in the short term.

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Abbreviations used without definitions in TRB publications:

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| AASHO | American Association of State Highway Officials |
| AASHTO | American Association of State Highway and Transportation Officials |
| ASCE | American Society of Civil Engineers |
| ASME | American Society of Mechanical Engineers |
| ASTM | American Society for Testing and Materials |
| FAA | Federal Aviation Administration |
| FHWA | Federal Highway Administration |
| FRA | Federal Railroad Administration |
| FTA | Federal Transit Administration |
| IEEE | Institute of Electrical and Electronics Engineers |
| ITE | Institute of Transportation Engineers |
| NCHRP | National Cooperative Highway Research Program |
| NCTRP | National Cooperative Transit Research and Development Program |
| NHTSA | National Highway Traffic Safety Administration |
| SAE | Society of Automotive Engineers |
| TCRP | Transit Cooperative Research Program |
| TRB | Transportation Research Board |
| U.S.DOT | United States Department of Transportation |

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