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C. Rojahn, R. Mayes, D.G. Anderson, J. Clark, J.H. Hom, R.V. Nutt and M.J. O'Rourke

Technical Report NCEER-97-0002

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NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

Headquartered at the State University of New York at Buffalo

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C. Rojahn[1], R. Mayes[2], D.G. Anderson[3], J. Clark[4], J.H. Hom[5], R.V. Nutt[6] and M.J. O'Rourke[7]

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1 Executive Director, Applied Technology Council
2 President, Dynamic Isolation Systems, Inc.
3 Consulting Engineer, Woodward-Clyde Consultants
4 Vice President, Anderson Bjornstad Kane Jacobs
5 President, SOH and Associates, Structural Engineers
6 Structural Engineering Consultant
7 Professor, Department of Civil and Environmental Engineering,Rensselaer Polytechnic Institute

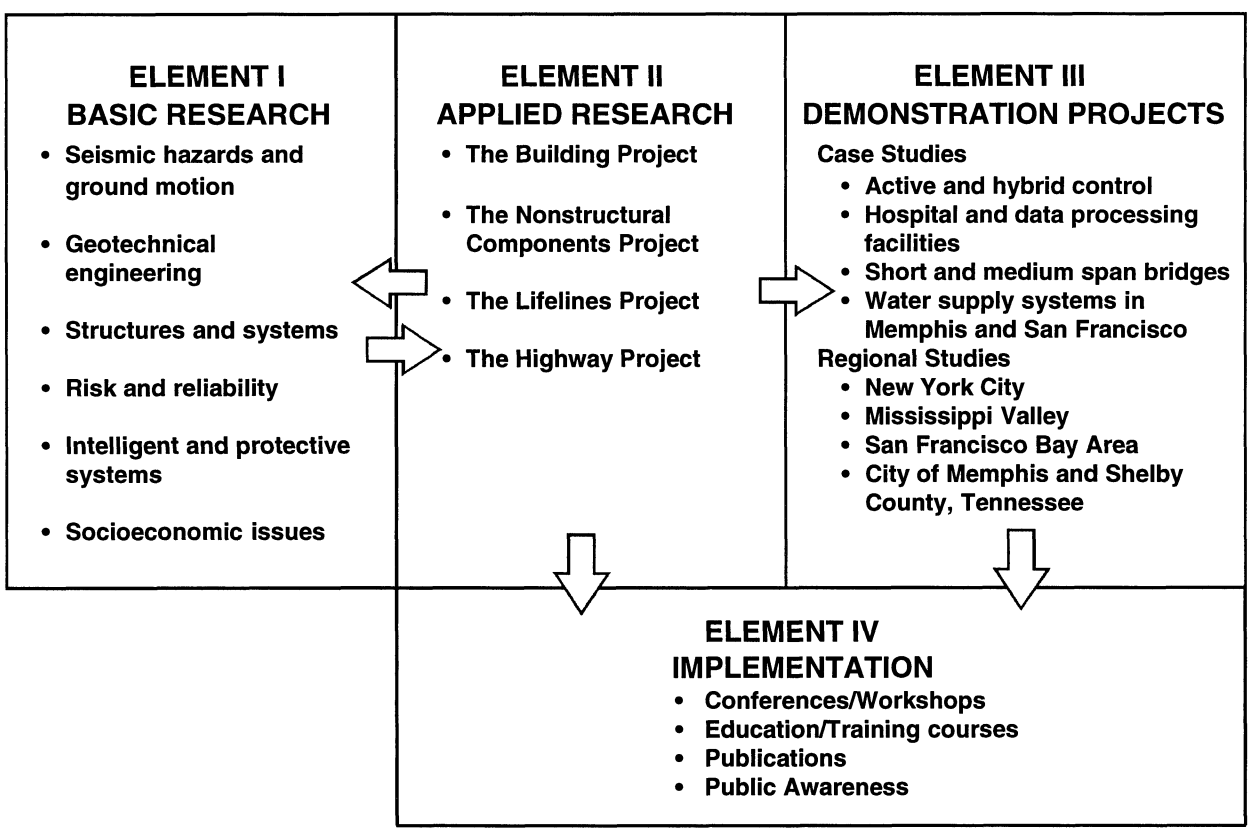
NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
State University of New York at Buffalo
Red Jacket Quadrangle, Buffalo, NY 14261

PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established in 1986 to develop and disseminate new knowledge about earthquakes, earthquake-resistant design and seismic hazard mitigation procedures to minimize loss of life and property. The emphasis of the Center is on eastern and central United States *structures*, and *lifelines* throughout the country that may be exposed to any level of earthquake hazard.

NCEER's research is conducted under one of four Projects: the Building Project, the Nonstructural Components Project, and the Lifelines Project, all three of which are principally supported by the National Science Foundation, and the Highway Project which is primarily sponsored by the Federal Highway Administration.

The research and implementation plan in years six through ten (1991-1996) for the Building, Nonstructural Components, and Lifelines Projects comprises four interdependent elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten for these three projects. Demonstration Projects under Element III have been planned to support the Applied Research projects and include individual case studies and regional studies. Element IV, Implementation, will result from activity in the Applied Research projects, and from Demonstration Projects.

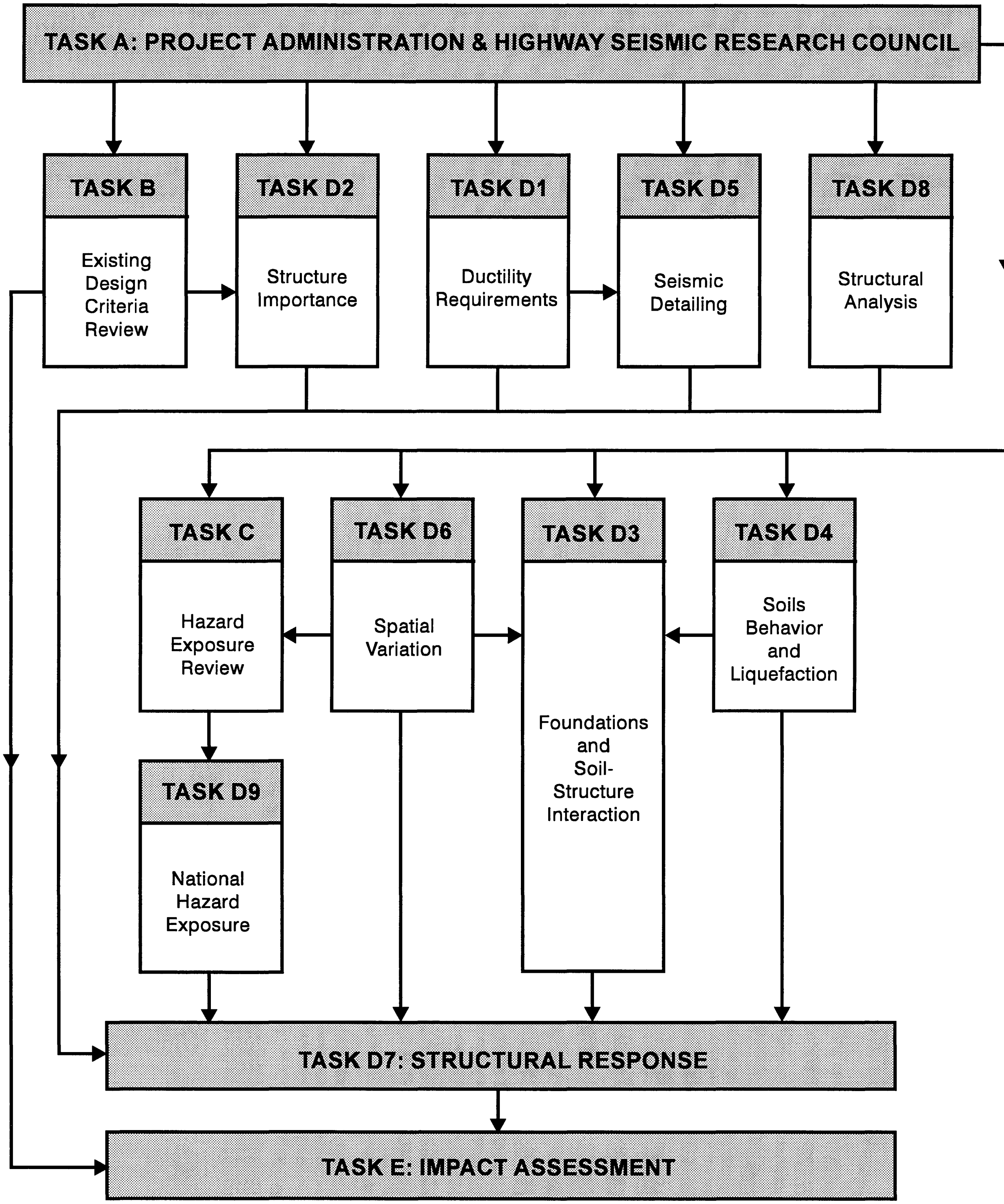


Research under the **Highway Project** develops retrofit and evaluation methodologies for existing bridges and other highway structures (including tunnels, retaining structures, slopes, culverts, and pavements), and develops improved seismic design criteria and procedures for bridges and other highway structures. Specifically, tasks are being conducted to: (1) assess the vulnerability of highway systems and structures; (2) develop concepts for retrofitting vulnerable highway structures and components; (3) develop improved design and analysis methodologies for bridges, tunnels, and retaining structures, with particular emphasis on soil-structure interaction mechanisms and their influence on structural response; and (4) review and improve seismic design and performance criteria for new highway systems and structures.

Highway Project research focuses on one of two distinct areas: the development of improved design criteria and philosophies for new or future highway construction, and the development of improved analysis and retrofitting methodologies for existing highway systems and structures. The research discussed in this report is a result of work conducted under the new highway construction project, and was performed within Task 112-B, "Review Existing Design Criteria and Philosophies" of the project as shown in the flowchart.

The overall objective of this task is to review current domestic and foreign design practice and criteria, ongoing research in seismic design criteria development, and philosophies behind the seismic resistant design of highway structures, including bridges, tunnels, abutments, retaining wall structures and foundations. In addition, recommendations for the future direction of seismic code requirements for transportation structures in the U.S. are to be developed. This report provides the results from this extensive review. A project engineering panel was assembled by the Applied Technology Council to conduct the review. The project, named ATC-18, was conducted over a four-year period.

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



FOREWORD

In 1992 the National Center for Earthquake Engineering Research (NCEER), in conjunction with the Applied Technology Council (ATC) and several other institutions under contract to NCEER, commenced the NCEER Highway Project, which consists of two projects sponsored by the U.S. Federal Highway Administration (FHWA). Both projects include research studies on the seismic vulnerability of U.S. highway construction including bridges, tunnels, retaining structures, slopes and embankments. The goal of the first project is to review current seismic retrofit guidelines in order to provide cost-effective tools for improved evaluation and seismic upgrading of the existing highway network. The goal of the second project is to develop improved seismic design guidelines for new highway construction.

ATC, as one of the subcontractors on the second project (FHWA Project DTFH61-92-C-00112) had responsibility for conducting a review and assessment of current seismic design criteria for new highway construction (ATC-18 project). This report presents the findings and recommendations developed during that effort.

The overall scope of work on the ATC-18 project consisted of a review of current domestic and foreign design practice and criteria, ongoing research in seismic design criteria development, and philosophies behind the seismically resistant design of highway structures including bridges, tunnels, abutments, retaining wall structures, and foundations. In addition, recommendations for the future direction of seismic code requirements for transportation structures in the United States were developed.

The ATC-18 project was conducted over a 4-year period during which some of the documents being reviewed were in a state of flux. The Japanese code that was reviewed in this report, for example, is based on decisions and knowledge developed prior to the January 1995 Kobe earthquake. Changes in the Japanese code as a result of that earthquake are likely under consideration.

This report was prepared by the ATC-18 Project Engineering Panel, whose membership included Ronald L. Mayes (Chairman and Project Director), Donald G. Anderson, John Clark, Ian G. Buckle (NCEER Highway Project Principal Investigator), John H. Hom, Richard V. Nutt, Michael J. O'Rourke, and Charles Thornton (ATC Board Contact). Technical editing and report production services were provided, respectively, by Nancy Sauer and Peter Mork. The affiliations of these individuals are provided in the list of project participants (Appendix A).

Applied Technology Council gratefully acknowledges the funding provided by FHWA and the support and cooperation provided by Ian M. Friedland, NCEER Asst. Director for Bridges and Highways.

Christopher Rojahn
ATC Executive Director

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INTRODUCTION

1.1 Background

In the United States, highway transportation systems consist primarily of roadbeds, bridges, tunnels, and retaining structures. Of all the highway transportation system components, highway bridges have been the most closely studied for seismic vulnerability, and they are the only component for which national seismic design standards have been prepared.

Prior to the 1971 San Fernando, California earthquake, which severely damaged bridges in the Los Angeles region, the American Association of State Highway and Transportation Officials (AASHTO) specifications for the seismic design of bridges were based in part on the lateral force requirements for buildings developed by the Structural Engineers Association of California. In 1973, prompted by the 1971 San Fernando earthquake, the California Department of Transportation (Caltrans) introduced new seismic design criteria for bridges, which included the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of bridges. In 1975 AASHTO adopted Interim Specifications, which were a slightly modified version of the 1973 Caltrans provisions, and made them applicable to all regions of the United States. At about the same time, FHWA awarded Applied Technology Council a contract to develop nationally applicable guidelines for the seismic design of bridges. These guidelines were completed in 1981 and are known as ATC-6 (ATC, 1981).

AASHTO subsequently adopted the ATC-6 guidelines, first as a set of Guide Specifications in 1983 and later by incorporating them into the *Standard Specifications for Highway Bridges* in 1991. (At the same time, the 1981 map of acceleration coefficients was replaced with a revised version prepared by the U.S. Geological Survey in 1988.) The 16th edition of AASHTO's Standard Specifications for Highway Bridges was published in 1995.

In the fifteen years since the original ATC-6 report was prepared, there have been numerous advances in earthquake engineering. Also, a considerable body of experience has been accumulated both through the use of the 1983 Guide Specifications and the performance of bridges during the 1989 Loma Prieta, California earthquake, the 1990 and 1991 Costa Rica and Philippine earthquakes, and the 1994 Northridge, California earthquake.

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

The NCEER project was prompted in part because significant progress has been made in understanding seismic risk throughout the United States, in geotechnical engineering, and in seismically resistant design. However, at the time the project was initiated, there were still many gaps in basic knowledge and some of the recently developed information and data required additional study before applying them directly to highway engineering applications. Consequently, the NCEER project includes both analytical and experimental studies related to the seismic design and performance of bridges, tunnels, slopes, embankments, and retaining structures. It focuses on the following elements:

- seismic hazard exposure of the U.S. highway system
- foundation design and soil behavior
- structural issues of importance, response, and analysis

- design issues
- design criteria review and impact assessment.

1.2 Purpose and Scope

This ATC-18 report presents the results of one component of the FHWA project DTFH61-92-00112, namely design criteria review. The overall scope of work on the ATC-18 project consisted of a review of current design practice and criteria, ongoing research in seismic design criteria development, and philosophies behind the seismically resistant design of highway structures including bridges, tunnels, and retaining structures. Particular attention was given to foundation design requirements. Roadbeds were excluded, however, because they are generally not the concern of structural engineering practitioners and researchers, the primary intended audience for this report.

The ATC-18 project team initially focused on a review of current design practice and criteria for new bridge design, as well as the philosophies on which they are based. This involved a review of existing U.S. standards along with the latest codes of Japan, New Zealand, and Europe. Guidelines developed for the Transportation Corridor Agencies (TCA) for Orange County, California, and in-progress work such as the new AASHTO Load and Resistance Factor Design (LRFD) Bridge Specifications were also reviewed. Issues that received attention included performance criteria, importance classification, definitions of seismic hazard for areas where damaging earthquakes have longer return periods, design ground motion, duration effects, site effects, structural response modification factors (*R* factors), ductility demand, design procedures, foundation and abutment modeling, soil-structure interaction, seat widths, joint details, and detailing reinforced concrete for limited ductility in areas with low-to-moderate seismic activity.

The ATC project team also reviewed information emanating from the ongoing ATC-32 project, an extensive effort under which ATC is reviewing and recommending revisions to Caltrans *Bridge Design Specifications* (Caltrans, 1993), which are a modified version of the 1983 AASHTO specifications and subsequent interim specifications dated 1984, 1985, and 1986. Issues under investigation in the ATC-32 project include performance criteria for bridges that consider the importance of the bridge, life-safety preservation in all earthquakes, and bridge functionality after lesser earthquakes; foundation and abutment design; concrete design; steel design; analysis methods; and bearing design.

The final phase of the ATC-18 project was to develop recommendations for the future direction of seismic code requirements for bridge structures in the United States. This was achieved by developing an outline of a future standard together with issues that need to be developed and resolved. An important part of the recommendations is a two-level design approach.

1.3 Report Organization

An overview of the design philosophies of various codes throughout the world is discussed in Section 2 along with a summary of their seismic input requirements. Section 2 also includes key issues that need to be addressed in future codes.

Sections 3 to 7 summarize the current worldwide design requirements for transportation structures. U. S. bridge requirements are given in Section 3 and New Zealand, Japanese, and European bridge requirements are presented in Section 4. A detailed summary of worldwide bridge code requirements is presented in tabular form in Section 4.4. A summary of worldwide foundation design requirements is given in Section 5 and abutment and retaining wall requirements are presented in Section 6. U. S. and foreign design requirements for tunnels are presented in Section 7.

A detailed discussion of a two-level design approach is presented in Section 8, a proposed outline for a future code is presented in Section 9, and topics for additional research and development are presented in Section 10.

SECTION 2
DESIGN PHILOSOPHIES, STRATEGIES AND SEISMIC INPUT FOR BRIDGE STRUCTURES

There are a number of key issues that must be established before developing detailed design provisions. Two of the most crucial are the design philosophy and the associated seismic ground motions. These issues are discussed in Sections 2.1 and 2.2. Also of interest but extremely difficult to codify, are design strategies or "good design practice," which is addressed in Section 2.3. Some of the more significant design issues that need to be addressed by future codes are discussed in Sections. 2.4 through 2.6.

2.1 Design Philosophies

The following sections summarize the design philosophies of current U.S. and foreign bridge codes.

2.1.1 United States: AASHTO

The American Association of Highway and Transportation Officials (AASHTO) currently publishes two codes for the design of highway bridges: *Standard Specifications for Highway Bridges, 16th edition* (1995), and *LRFD Bridge Design Specifications*, *1st edition* (1994).

AASHTO specifications call for designing bridges to resist design earthquake motions and forces that have a low probability of being exceeded during the normal life expectancy of a bridge (10 percent probability of exceedance in 50 years). Bridges that are designed and detailed in accordance with the provisions of these specifications may suffer damage, but should have low probability of collapse due to seismically-induced ground shaking.

The principles used for the development of these specifications are:
- Realistic seismic ground motion intensities and forces should be used in the design procedures.
- Small-to-moderate earthquakes should be resisted within the elastic range of the structural components without significant damage.
- Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge.
- Where possible, damage that does occur should be of a nature that can be easily confined to locations that are easily detected and accessible for inspection and repair.

2.1.2 Current Caltrans Requirements

The current Caltrans Bridge Design Specification (BDS) design philosophy (Caltrans, 1993) is to identify the most destructive level of earthquake that can occur at a given site, which is referred to by Caltrans as the maximum credible earthquake (MCE). Caltrans then designs the bridge at that site to limit and control damage from the MCE. The paramount goal is the preservation of human life. Protecting the bridge from damage in the MCE is not considered to be economically feasible in most cases. Significant damage to the bridge during the MCE is accepted, although it is kept confined to failure modes and locations where inspection and repair are facilitated. Design of the bridge to protect it from significant damage from more frequent earthquakes is not performed directly, but is assumed to occur as a natural result of designing for the MCE.

Design forces for ductile components of the bridge such as columns are calculated by applying ductility and risk (Z) factors to the component forces obtained from the response-spectrum analyses. Ductile performance of these components is assured through proper reinforcement detailing. Components and failure modes that are nonductile or for which damage is unacceptable are protected by designing them to force levels greater than those produced by the yielding of ductile components. Therefore, ductile components are intended to act as "fuses" that can be used to limit and control damage.

Special attention is given to expansion joints. The provisions specify minimum seat widths and restrainer cables are used to limit relative displacements at such joints.

2.1.3 ATC-32 Recommendations to Caltrans

The ATC-32 project was conceived in the wake of the 1989 Loma Prieta earthquake, which focused attention on the seismic design procedures used for California bridges. The purpose of the ATC-32 project, which was sponsored by Caltrans, was to review and recommend revisions to the current Caltrans seismic design standards and criteria for new highway bridges.

The ATC-32 report (ATC, 1996a) includes new performance criteria proposed by Caltrans. These criteria specifically identify two design earthquakes. One is a safety-evaluation earthquake, which is either the current MCE or is defined in probabilistic terms as an event with a 1000- to 2000-year return period. The other is the functional-evaluation earthquake, which has a 60 percent probability of not being exceeded during the useful life of the bridge, which is assumed to be 200 years to 300 years for Important Bridges. This corresponds to a return period of approximately 500 years. Important Bridges, as defined by Caltrans, should sustain only minimal damage in the functional-evaluation earthquake and a "repairable" level of damage in the safety-evaluation earthquake. The seismic performance goals for Important Bridges are to be achieved by a two-level design approach (i.e., a direct design for each of the two earthquakes). In the case of other bridges, it is Caltrans' intent to continue to use a single-level approach using the safety-evaluation earthquake as the design event. It is noted, however, that it may not be possible to achieve performance goals with a single-level approach in areas of high seismicity (i.e., where the functional-evaluation earthquake is relatively large compared to the safety-evaluation earthquake). This may be an additional reason for using site-dependent performance criteria.

The performance criteria for the two levels of earthquakes are summarized in Table 2-1. The performance criteria are defined as follows:

Service Level—Immediate: Full access to normal traffic available almost immediately.

Service Level—Limited: Limited access possible within days (reduced lanes, light emergency traffic). Full service restorable within months.

Minimal Damage: Essentially elastic performance.

Repairable Damage: Damage that can be repaired with a minimum risk of losing functionality.

Significant Damage: A minimum risk of collapse, but damage that may require closure to repair.

Important Bridge (one or more of the following):
- Bridge required to provide secondary life-safety (e.g., access to emergency facilities)
- Time to restore bridge function creates major economic impact
- Bridge formally identified as critical by a local emergency plan

Table 2-1 Caltrans Seismic Performance Criteria for the Design and Evaluation of Bridges (ATC, 1996a)

Ground Motion at Site	*Minimum Performance Level*	*Important Bridge Performance Level*
Functional-Evaluation Earthquake	Service Level—Immediate "Repairable" Damage	Service Level—Immediate "Minimal" Damage
Safety-Evaluation Earthquake	Service Level—Limited "Significant" Damage	Service Level—Immediate "Repairable" Damage

Safety-Evaluation Ground Motion (up to two methods of defining ground motions may be used):

- Deterministically-assessed ground motions from the MCE as defined by the Division of Mines and Geology Open File Report 92-1 (1992)
- Probabilistically-assessed ground motions with a long return period (approx. 1000–2000 years)

For Important Bridges, both methods will be considered; however, the probabilistic evaluation must be reviewed by a Caltrans-approved consensus group. For all other bridges, the motions are based only on the deterministic evaluation. In the future, the role of the two methods for other bridges will be reviewed by a Caltrans-approved consensus group.

Function—Evaluation Ground Motion: These are probabilistically-assessed ground motions that have a reasonable (60 percent) probability of not being exceeded during the useful life of the bridge, which is assumed to be 200 to 300 years for Important Bridges. The determination of this event will be reviewed by a Caltrans-approved consensus group. A separate functional evaluation is required only for the two-level design procedure used for Important Bridges.

A major criticism of the current Caltrans design approach has been its overemphasis on design forces and its failure to consider displacements adequately. The proposals in the ATC-32 report attempt to address some of these criticisms. Design spectra are presented in the form of spectral displacements as well as spectral accelerations. Design forces will be determined using Z factors that consider displacement ductility in a more rational way. Provisions for considering P-Δ effects are also included. These are just a few of the improvements made.

2.1.4 New Zealand

The primary objective of seismic design, as specified in the New Zealand Bridge Manual (TNZ, 1994), is to ensure that the bridge will perform its function of maintaining communications after a seismic event. The extent to which this is possible will depend on the severity of the event and thus, by implication, on its return period. For design purposes, bridges are categorized according to their importance and assigned a risk factor related to the seismic return period. This will then result in an equivalent design seismic response and consequent loading. If the behavior at this design intensity meets the criteria of (a) below, it is expected that with appropriate detailing, the behavior at other intensities, as in (b) and (c) below, will also be satisfactory and no further specific check is required.

The seismic performance requirements are as follows:

(a) After the design event, the bridge will be useable by emergency traffic, although damage may have occurred and some temporary repairs may be required. Permanent repair to reinstate the design capacities for both vehicle and seismic loading should be feasible.

(b) After a smaller event with a return period significantly less than the design event, damage should be minor, and there should be no disruption to traffic.

(c) After a larger event with a return period significantly greater than the design event, the bridge should not collapse, although damage may be extensive. It should be rendered useable by emergency repair, although a lower level of loading may be required.

For bridges that must be located on liquefiable foundations or over Class 1 active faults, recognition must be given to the large movements that may result from settlement, rotation, or translation of piers. A Class 1 active fault is defined as one that has shown repeated movements over the last 5,000 years or a single movement within the last 5,000 years and repeated movements over the last 50,000 years.

2.1.5 Eurocode

Eurocode (CEN, 1996) is referred to as ENV—European Prestandard. The lifetime of the current document is three years, after which it will be revised. Committee members from the CEN (Comite European de Normalization) represent Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland, and the United Kingdom.

The design philosophy of the Eurocode is based on the general requirement that communications should be maintained, with appropriate reliability, after the design seismic event.

The design seismic action defined in Section 3 of this code reflects a design seismic event with a return period of approximately 475 years. The design event has a probability of exceedance ranging between 10 percent and 20 percent for a design life ranging between 50 and 100 years. This level of design action is applicable to the majority of the bridges, which are considered to be of average importance.

The Eurocode specifies two performance criteria – one for the design event and one for ground motions with a high probability of occurring during the life of the bridge. These criteria are defined as follows:

Non-Collapse Requirement (Ultimate Limit State)

- After the occurrence of the design seismic event, the bridge will retain its structural integrity and adequate residual resistance, although at some parts of the bridge considerable damage may occur.
- The bridge must be damage-tolerant; i.e., those parts of the bridge susceptible to damage by their contribution to energy dissipation during the design seismic event should be designed in such a manner as to ensure that the structure can sustain the actions from emergency traffic and that inspections and repairs can be performed easily.
- Flexural yielding of specific sections (i.e., the formation of plastic hinges) is allowed in the piers and is necessary in regions of high seismicity in order to reduce the design seismic action to a level requiring reasonable additional construction costs.
- The bridge deck should be protected from the formation of plastic hinges and from unseating under extreme seismic displacements.

Minimizing Damage (Serviceability Limit State)

After seismic actions with high probability of occurrence during the design life of the bridge, the parts of the bridge intended to contribute to energy dissipation during the design seismic event should only undergo minor damage without giving rise to any reduction of the allowable traffic or the need for immediate repair.

2.1.6 Japan

The best description of the Japanese design philosophy was obtained from a paper by Iwasaki et al. (1990) and Kawashima (1995). Two design force levels are considered although neither is defined in terms of return period. Bridges are required to maintain their function without structural damage after small-to-moderate earthquakes and to avoid critical failure after significant earthquakes, such as the 1923 Kanto earthquake. The evaluation of maintaining function after small-to-moderate earthquakes is made based on the allowable-stress design approach, and the evaluation for avoiding critical failure after significant earthquakes is made based on the bearing capacity or ductility of structural members. Presently, the latter evaluation is only made for reinforced concrete piers, but various research programs are being performed to develop evaluation methods for steel piers and overturning/sliding of rigid foundations and piles.

The Japanese do not like the probabilistic approach for evaluating the design force. Although it is generally assumed that the occurrence of earthquakes follows a Poisson distribution, they do not believe this distribution holds true for earthquakes of larger magnitudes, which are the major contributors when determining seismic force. The average return period of earthquakes with magnitudes greater than 8 along the Pacific side of Japan is said to be approximately 136 years, but for most areas the exact recurrence times are not known. It is meaningless, therefore, to talk about a 90 percent probability of not being exceeded in a 50-year timespan.

The Japanese prefer to evaluate the design force based on a combination of earthquake magnitude and epicentral distance. A force level with a combination of $M = 7$ and epicentral distance of 50 km may be close to the design force level based on the allowable-stress design approach, and a force level with a combination of $M = 8$ and epicentral distance of 50 km may be close to the design force level for the ductility check.

2.2 Seismic Input Requirements

The seismic input requirements for the design of bridge structures typically involve three components: (1) an acceleration coefficient to represent the design ground motion shaking level, (2) a site coefficient to account for local soil amplification/attenuation effects at the site, and (3) an elastic response spectrum to show the change in acceleration with predominant period of ground motion. Procedures to determine the seismic inputs in the different codes and guidelines in the United States and in the European, Japanese, and New Zealand codes are summarized below.

2.2.1 Acceleration Coefficient

Two procedures can be used to determine the acceleration coefficient for a site. One procedure involves a deterministic process, in which the acceleration level is computed on the basis of specific seismic source location and size (i.e., scenario design event). Given the location and seismic source size (magnitude), empirically derived attenuation relationships are used to estimate ground acceleration at the site. The other approach involves probabilistic procedures, in which the acceleration level is computed as a function of the likelihood of occurrence for all active earthquake sources in the vicinity of the site. This computation considers the design life of the bridge, the size and locations of all sources in proximity to the site, the activity of these seismic sources, and the attenuation of ground motion between the source and the site. The probability of occurrence calculated through this process is directly related to the average return period of the event. In other words, a level of ground motion that has a 10 per cent probability of exceedance in a 50-year design life has a return period of approximately 500 years[1]. For ease in determining the acceleration coefficient during design, many countries have developed maps, using either deterministic or probabilistic procedures, that show acceleration coefficients for different geographic areas within their respective countries.

United States

The acceleration map most often used in the United States for bridge design is that given in Division I-A, Seismic Design, of the AASHTO standard specifications (AASHTO, 1995). At the time this map was being developed for the ATC-6 report (ATC, 1982), the only nationally applicable maps available provided ground motions with a 10% probability of being exceeded in 50 years. Consequently, the 1995 *AASHTO Standard Specifications for Highway Bridges, Division I-A: Seismic Design* have a similar map showing horizontal acceleration coefficients having a 10 % probability of exceedance in 50 years.

The AASHTO specifications include a requirement that special studies by a qualified professional be used to determine site- and structure-specific acceleration coefficients rather than using the AASHTO maps if any one of the following conditions exist:

- the site is located close to an active fault
- long-duration earthquakes are expected in the region
- the importance of the bridge is such that a longer exposure period (and therefore return period) should be considered

Some states have also developed maps specific to the state showing acceleration contours. These include Arizona, Nevada, Oregon, and Washington, which developed theirs on a probabilistic basis. Most of these states use a 475-year return period. Arizona includes maps for return periods of 475 and 2,375 years; Nevada gives the acceleration coefficients for return periods of approximately 95, 475, and 975 years. Oregon provides maps for 500 and 1000 years.

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1 The actual return period is determined from the equation, $P(a_{max}>a) = 1 - e^{-T/R}$ where T is the design life and R is the return period. This results in a 475-year return period for a 10 percent probability of exceedance in 50 years.

Caltrans has traditionally used an acceleration map developed by the California Divison of Mines and Geology (CDMG) to select acceleration coefficients for design. CDMG developed its map using a deterministic process: acceleration levels represent the maximum probable acceleration for the site, determined on the basis of the maximum credible earthquake (MCE) for nearby faults and on the basis of average attenuation relationships.

Several other codes and guidelines also identify return periods that should be used in design. The commentary within the AASHTO LRFD specifications (AASHTO, 1994) refers to a return period of 2,500 years for a maximum credible earthquake. The commentary suggests that critical structures might be designed for this return period. The 1991 National Earthquake Hazards Reduction Program (NEHRP) maps (BSSC, 1991) presented acceleration values with both a 10 percent probability of exceedance in 50 years and 250 years, which correspond to return periods of approximately 475 and 2,375 years, respectively. The 1991 NEHRP map set differs significantly from previous maps in that contours of spectral acceleration at periods of 0.3 seconds and 1.0 second are given, in addition to maps showing peak effective acceleration or peak effective velocity-related acceleration.

The base isolation requirements of the Uniform Building Code require that some structural response analyses be conducted for the MCE, defined as the ground motion with a 10 percent probability of exceedance in 100 years.

The Tri-Services manual (DoD, 1986) uses two levels of design for essential buildings: (1) a maximum probable earthquake with a 50 percent probability of exceedance in 50 years (72-year return period), and (2) a maximum capable earthquake with a 10 percent probability of exceedance in 100 years (950-year return period).

Eurocode 8

As noted in Section 2.1.5, the return period for the design seismic event in the Eurocode is 475 years, which is stated as having a probability of exceedance ranging from 10 percent to nearly 20 percent for design lives ranging between 50 and 100 years, respectively. Each country covered by Eurocode 8 is required to develop its own hazards zonation, where hazards within a zone are constant. However, provisions of Eurocode 8 do not need to be observed in areas with acceleration coefficients less than 0.04 g.

Annex A to the Eurocode provides recommendations for selecting the design seismic event during the construction phase of the project. The minimum recommended return period for construction is 100 years, assuming a construction period equal to or less than five years. The design acceleration level during construction is roughly 50 percent of the value with a return period of 475 years.

Annex D of Eurocode 8 presents guidance on spatial variability during earthquake loading. Spatial variability of the motion is identified as an important issue if the length of the bridge is over 2,000 feet or if certain geological discontinuities or topographical features exist.

Japanese Code

The seismic coefficient used in the 1990 Japanese Code is the product of a standard horizontal seismic coefficient, equal to 0.20, multiplied by a zone factor that ranges from 1.0 on the eastern side of Japan to 0.7 on the western tip and northern tip of Japan. A map shows the variation in zone factor with area. The seismic coefficient then varies between 0.14 and 0.2.

Iwasaki et al. (1990) and Kawashima (1995) indicate that the standard horizontal seismic coefficient k_{hor} was determined to represent a "realistic ground motion developed during a significant earthquake with a magnitude as large as 8." No information is given by Kawashima about the method used to determine the "realistic" ground motion; i.e., whether the resulting coefficient represents maximum credible shaking levels or a probabilistic acceleration level for a given return period. The range in horizontal seismic coefficients is, however, somewhat lower than, say, acceleration coefficients for a 475-year return period in California and Washington. For equal levels of seismicity, this suggests that the return period for the design earthquake in the Japanese Code is somewhat less than 475 years.

New Zealand Code

The New Zealand Code uses a base acceleration coefficient, a zone factor, and a risk factor. The base coefficient at zero period (i.e., $T = 0$) is equal to 0.4 for flexible and normal soils; the zone factor varies from 0.6 to 1.2, and the risk factor varies between 1.0 and 1.3. The return period associated with a risk factor equal to one is stated to be 450 years. A plot of the risk factor versus the return period is provided in the code (TNZ, 1994).

2.2.2 Site Coefficient

Most seismic design codes and guidelines apply some type of site coefficient to modify the acceleration coefficient described above for local site effects. Local site effects result when the seismic wave propagates upward from crystalline rock into softer rock and soil layers. The ground motion is either amplified or deamplified, depending on the thickness and stiffness of the geologic material and the strength of the motion at the crystalline rock layer. Two methods are used to estimate the local site effects. The first involves numerical modeling. The amount of amplification or deamplification for a specific site is estimated using one-dimensional, equivalent linear computer codes such as SHAKE (Schnabel, et al., 1972), one-dimensional, nonlinear codes such as DESRA (Lee and Finn, 1978), or any one of several other similar codes. The other approach is to use generic site amplification factors. These factors have been developed by empirical and analytical studies, and are normally given in the codes and guidelines for general soil categories. The site coefficients are normally multiplied by the expected ground acceleration to estimate a seismic coefficient more consistent with local site conditions.

United States

The 1992 edition of the AASHTO specifications identifies three soil types: The values of the site coefficients are (1) 1.0 for any type of rock and stiff soil less than 200 feet in depth, (2) 1.2 for stiff soil greater than 200 feet in thickness, and (3) 1.5 for soft soil. In the 1995 edition, the number of site categories was increased from three to four. Table 2-2 gives the values of the site coefficients, with the description of the soil profiles. Similar information is presented in the "LRFD Bridge Design Specifications and Commentary" (AASHTO, 1994).

Caltrans currently introduces local site effects in their design process through the use of a set of response spectra, referred to as *ARS* spectra. *ARS* spectra were developed by conducting one-dimensional, equivalent linear ground response analyses with SHAKE for four geologic conditions: (1) 0 to 10 feet of alluvium, (2) 10 to 80 feet of alluvium, (3) 80 to 150 feet of alluvium, and (4) greater than 150 feet of alluvium. Spectra were developed for peak rock accelerations that range from 0.1 to 0.7g, based on earthquake magnitudes less than 7. Work being performed for Caltrans as part of ATC-32 recommends use of a new set of spectra for handling local site effects. The new spectra are defined for a rock site and three soil sites at rock accelerations from 0.1 to 0.7g based on earthquakes of magnitude 6.5., 7.25, and 8. The four site conditions are defined using the procedures defined in the NEHRP provisions for new buildings (NEHRP, 1994) as described in the next paragraph. Figure 2-1 shows the shapes of these spectra.

The 1991 NEHRP provisions for new buildings (BSSC, 1991) also provided guidance in the selection of site coefficients. Values given in this document are identical to those listed in Table 2-2 for the 1995 AASHTO standard specifications. The 1994 NEHRP provisions for new buildings modified the previous (1991) approach to selection of site coefficients by defining five site categories, based on geologic condition in the upper 100 feet of soil or rock profile, then assigning an acceleration-based site coefficient at 0.3 second period (F_a) and velocity-based site coefficient at 1.0 second period (F_v) depending on the acceleration level. Values of F_a and F_v are used to develop response spectra that

Table 2-2 Site Coefficients in AASHTO Standard Specifications (1995)

Soil Type	*Description*	*Site Coefficient (S)*
I	Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 fps, or by other appropriate classification) or stiff soil conditions where the soil depth is less than 200 ft and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays	1.0
II	Stiff clay or deep cohesionless conditions where the soil depth exceeds 200 ft and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays	1.2
III	Soft or medium-stiff clays and sands, characterized by 30 ft or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils	1.5
IV	Soft clays or silts greater than 40 ft in depth. These materials may be characterized by a shear wave velocity less than 500 fps and might include loose natural deposits or man-made, nonengineered fill.	2

implicitly incorporate the effects of local site geology. A sixth site type is also identified, which requires site-specific evaluations. The characteristics of the six soil types are summarized in Table 2-3.

Additional information about the F_a and F_v coefficients, including their variation with acceleration level and soil classification, is given in the discussion on response spectra. (As a word of caution, it must be remembered that coefficients developed for one code cannot be used in another, unless the design basis is the same.)

Eurocode 8

Eurocode 8 identifies three soil classes based on the shear wave velocity of the soil. These soil classes are summarized in Table 2-4. Sites not falling into these categories may require special studies.

These site categories are used to define a site coefficient S and shape factors for elastic response spectra. Values of S are 1.0 for Classes A and B and 0.9 for Class C. Eurocode 8 also provides guidance for topographic amplification effects in Annex 5 of Part 5. These adjustments are recommended for important structures in high seismic zones.

Japanese Code

The Japanese Code identifies three values of the modification factor c_g for ground conditions: (1) c_g = 0.8 for ground group I, (2) c_g = 1.0 for ground group II, and (3) c_g = 1.2 for ground group III. The ground group is defined on the basis of the predominant period (T_g) of the site, where $T_g = 4H/V_s$. Ground group I has a $T_g < 2$ seconds and is typically tertiary or older in age; ground group III has $T_g >$ 0.6 seconds and consists of soft alluvium. Ground group II falls between these two conditions and consists of alluvium or diluvium.

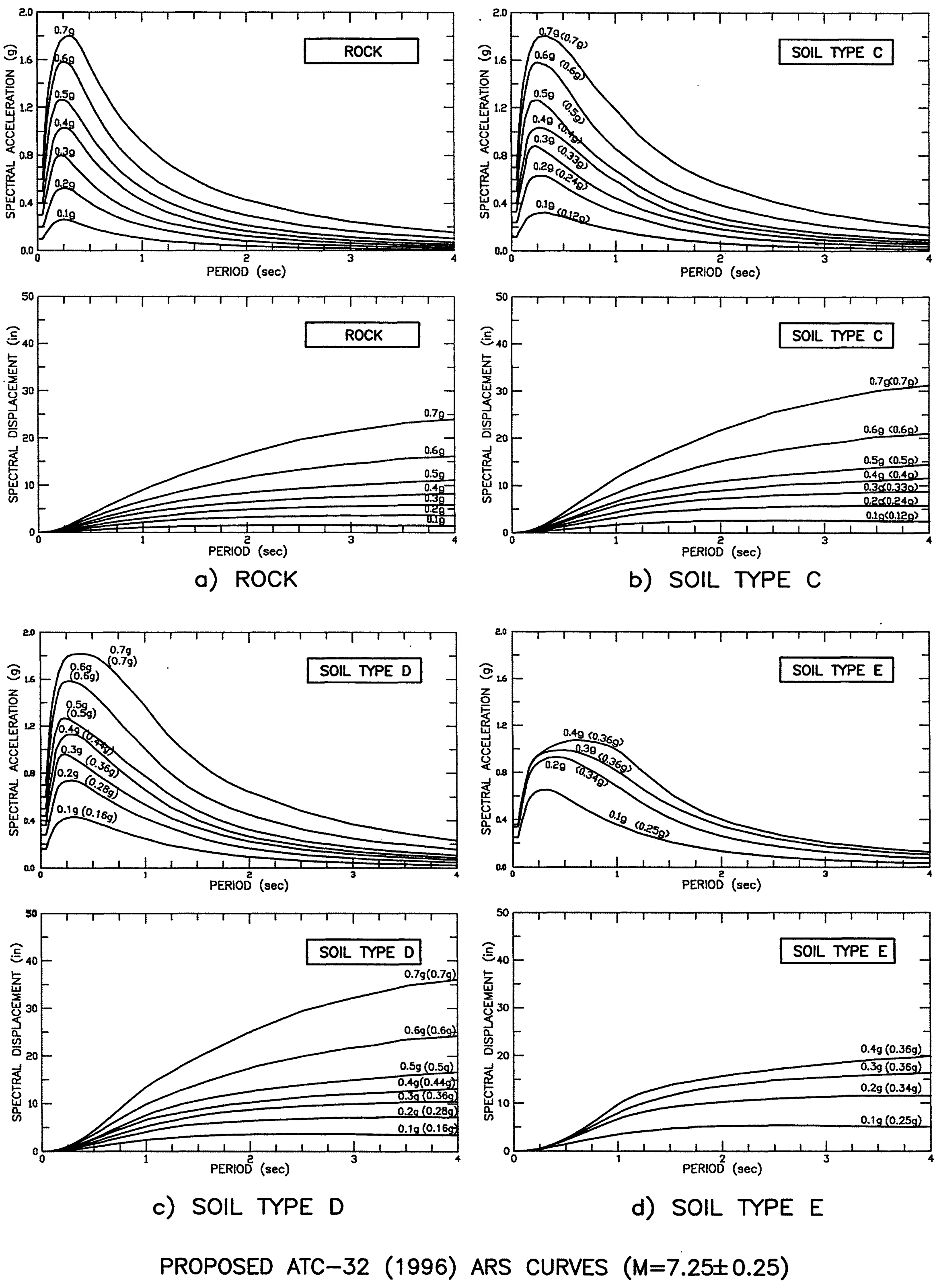


Figure 2-1 Spectral acceleration values from ATC-32 (ATC, 1996a).

Table 2-3 Soil Categories, NEHRP Provisions for New Buildings (BSSC, 1994)

Soil Profile Type	*Description*	*Shear Wave Velocity (fps)*	*N-Value (bpf)*	*Undrained Strength (psf)*
A	Hard Rock	>5,000	-	-
B	Rock	2,500 to 5,000	-	-
C	Very dense soil and soft rock	1,200 to 2,500	>50	>2,000
D	Stiff soil	600 to 1,200	15 to 50	1,000 to 2,000
E	Any profile with more than 10 feet of soft clay defined as soil with PI>20, w>40%	<600	<15	<600
F	1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils 2. Peats and/or highly organic clays (H>10 ft of peat and/or highly organic clay where H= thickness of soil) 3. Very high plasticity clays (H>25 ft with PI>75) 4. Very thick soft/medium stiff clays (H>120 ft)	-	-	-

Table 2-4 Soil Categories in Eurocode 8

Soil Class	*Description*
A	Rock deposits or other geological formation characterized by a shear wave velocity of at least 800 mps, including at most 5 m of weaker material at the surface; and stiff deposits of sand, gravel or overconsolidated clay, at least several tens of meters thick, characterized by a gradual increase of the mechanical properties with depth and by shear wave velocity values of at least 400 mps at a depth of 10 m
B	Deep deposits of medium dense sand, gravel, or medium stiff clays with thickness from several tens to many hundreds of meters, characterized by shear wave velocities of at least 200 mps at a depth of 10 m, increasing to at least 350 mps at a depth of 50 m
C	Loose cohesionless soil deposits with or without some soft cohesive layers, characterized by shear wave velocity values below 200 mps in the uppermost 20 m; and deposits with predominant soft-to-medium stiff cohesive soils, characterized by shear wave velocity values below 200 mps in the uppermost 20 m

Iwasaki et al. (1990) note that shear wave velocities used in the determination of T_g are generally estimated from empirical correlations based on N values from the Standard Penetration Test. Procedures are given in the Japanese Code for determining an average site period where layered soils exist.

New Zealand Code

The New Zealand code (TNZ, 1994 and Chapman, 1995) identifies three types of soil: rock or very stiff, intermediate, and flexible or deep:

"Site subsoil category (a) (Rock or very stiff soil sites):

Sites where the low amplitude natural period is less than 0.25 s, or sites with bedrock, including weathered rock, with unconfined compressive strength greater than or equal to 500 kPa, or with bedrock overlain by:

(i) Less than 20 m of very stiff cohesive material with undrained shear strength exceeding 100 kPa; or

(ii) Less than 20 m of very dense sand, with N_1>30, where N_1 is the SPT (N) value corrected to an effective overburden pressure of 100 kPa; or

(iii) Less than 25 m of dense sandy gravel with N_1>30

Site subsoil category (b) (Intermediate soil sites):

Sites not described as category (a) or (c) may be taken as intermediate soil sites.

Site subsoil category (c) (Flexible or deep soil sites):

Sites where the low amplitude natural period exceeds 0.6 s, or sites with depths of soils exceeding the values given" in a companion table that provides the maximum depths allowed for a specific site.

2.2.3 Response Spectra

Each code or guideline presents methods for determining the acceleration of the structure, given its predominant period of vibration. These methods typically are based on one of two approaches. One approach is a graphic representation of the response spectra. The other approach is an equation that relates the acceleration coefficient to the period of vibration. Some of the codes and guidelines also allow determination of site-specific spectra using computer modeling methods or scaling of earthquake records obtained for similar seismo-tectonic conditions.

United States

The 1995 AASHTO specifications use a simplified relationship

$$C_s = 1.2\left(\frac{AS}{T^{\frac{2}{3}}}\right) \tag{2-1}$$

for determining the lateral force design coefficient, where A = the acceleration coefficient, S = the site coefficient, and T is the period of the bridge. The maximum value of C_s does not have to exceed 2.5A, and for soil profile types III and IV, (see Table 2-2), for which A > 0.3g, C_s does not have to exceed 2.0A. For multimodal analyses, other restrictions are placed on C_s for Type III and IV soils. As noted previously, the number of soil categories has been changed from three to four (relative to the 1992 edition of the AASHTO standard specifications), and the S values have been adjusted for the new categories. Figure 2-2 shows typical spectral shapes obtained with this method. Recommendations given in the "LRFD Bridge Design Specifications and Commentary" (AASHTO, 1994) also follow this approach. These specifications also allow time-history analyses for important bridges, but do not provide specific guidance on selecting earthquake records for the analyses.

Caltrans presently uses four sets of standard *ARS* spectra. As noted in Section 2.2.2, these spectra cover peak ground accelerations from 0.1 to 0.7 g and four soil categories. The proposed revision to the current Caltrans procedure (ATC, 1996a) includes spectra for four site categories and earthquake magnitudes ranging from 6.5 to 8.

The 1991 NEHRP provisions define the seismic design coefficient as

$$C_s = 1.2 A_v \left(\frac{S}{R\,T^{\frac{2}{3}}} \right) \tag{2-2}$$

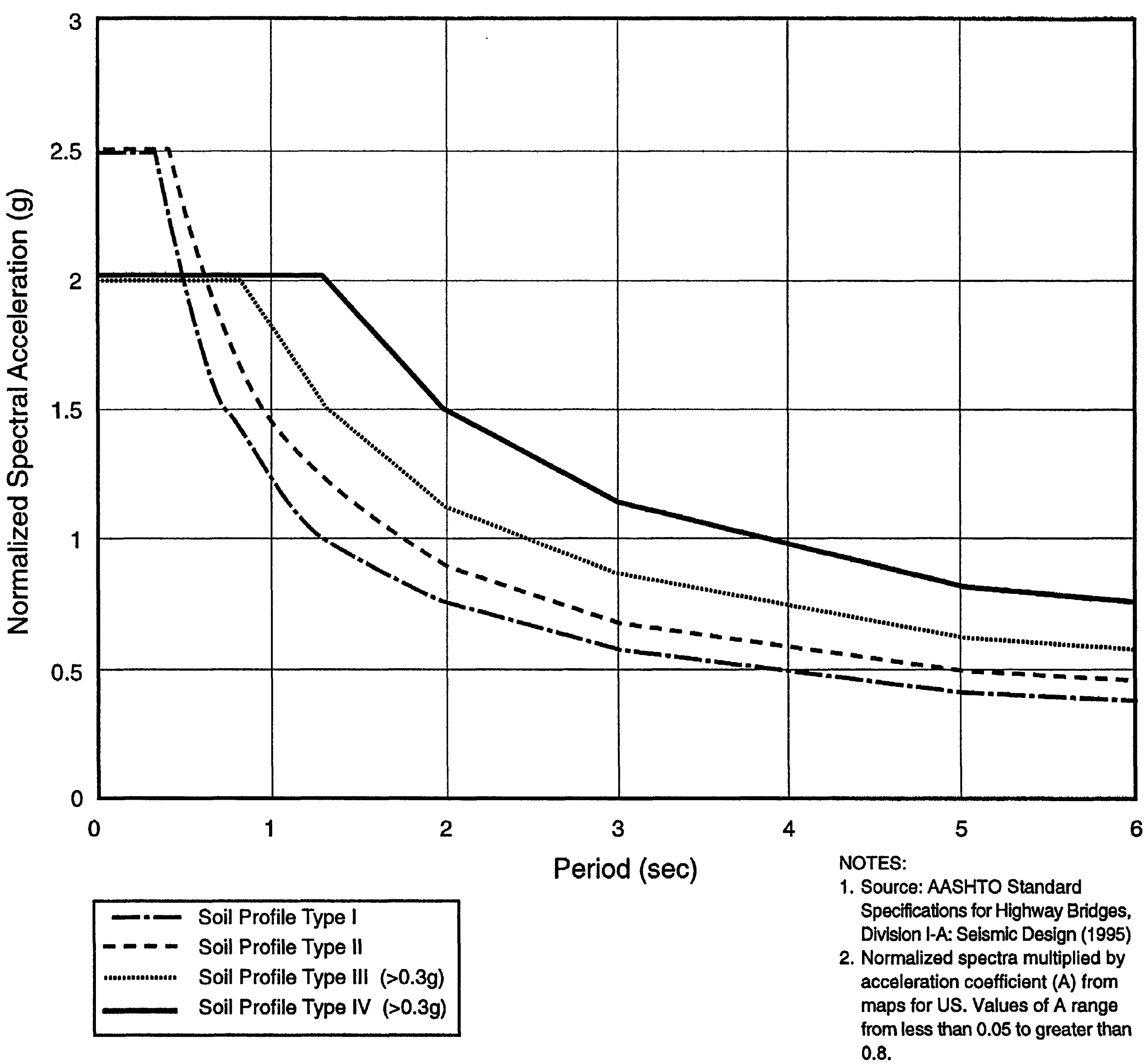


Figure 2-2 AASHTO response spectra for a = 0.4g.

with the S factor defined above. The maximum value of C_s does not have to be greater than $2.5A_a/R$. The 1994 revisions to the NEHRP provisions modifies the seismic design coefficient:

$$C_s = 1.2A_v\left(\frac{F_v}{R\,T^{\frac{2}{3}}}\right) \tag{2-3}$$

with a maximum

$$C_s = \frac{2.5A_aF_a}{R} \tag{2-4}$$

where F_a and F_v are acceleration- and velocity-based site coefficients described in Tables 2-5 and 2-6.

Figures 2-3 and 2-4 show the spectral shapes obtained with this two-factor approach to local site response for $A_a = 0.1$g and 0.4g, respectively.

Table 2-5 Acceleration-Based Site Coefficients from 1994 NEHRP Provisions

	Shaking Intensity			
Soil Profile Type	*$A_a = 0.1g$*	*$A_a = 0.2g$*	*$A_a = 0.3g$*	*$A_a = 0.4g$*
A	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0
D	1.6	1.4	1.2	1.1
E	2.5	1.7	1.2	0.9

Table 2-6 Velocity-Based Site Coefficients from 1994 NEHRP Provisions

	Shaking Intensity			
Soil Profile Type	*$A_v = 0.1g$*	*$A_v = 0.2g$*	*$A_v = 0.3g$*	*$A_v = 0.4g$*
A	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4
D	2.4	2.0	1.8	1.6
E	3.5	3.2	2.8	2.4

Eurocode 8

Eurocode 8 provides equations for computing spectral accelerations as a function of structural period. Coefficients used to establish the shape of the spectra are selected on the basis of subsoil class, as summarized in Table 2-4. Figure 2-5 shows the normalized spectra determined using these equations. Parameters used to develop these spectral shapes are reported to have a uniform probability of exceedance of 50 percent over all periods. These spectra are 5 percent damped spectra; provisions are included in Eurocode for modifying the spectra for other values of damping. The form of the damping adjustment equation is

$$\eta = \sqrt{\frac{7}{2+\xi}} \tag{2-5}$$

where η is the damping correction factor and ξ is the viscous damping ratio of the structure. Values of η are given in the relevant parts of Eurocode. The Eurocode also allows alternative representations of the ground motion using power-spectrum or time-history analysis. In the case of a power-spectrum analysis, the power spectrum must be consistent with the elastic response spectrum shown in Figure 2-5. For a time-history analysis, specific guidance is provided on the number and form of the time histories. For

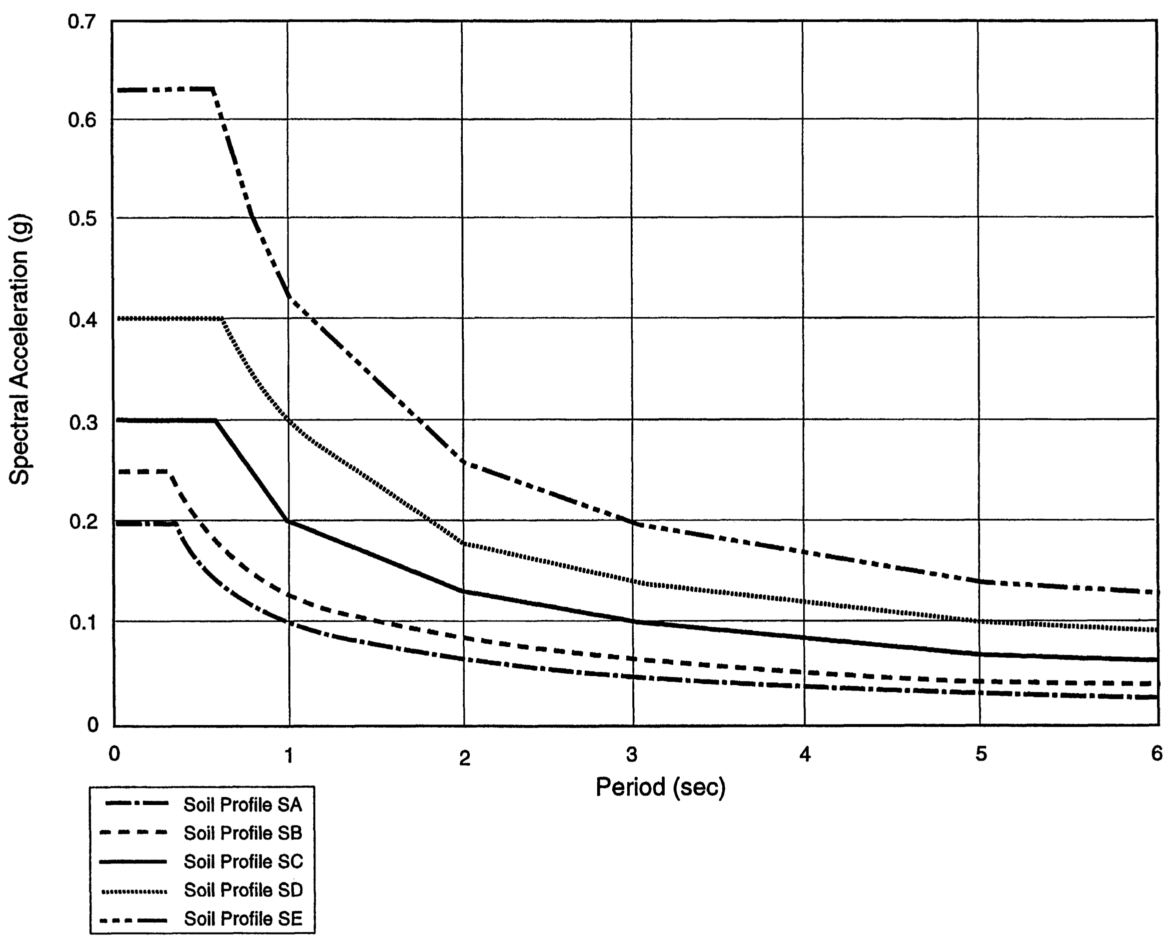


Figure 2-3 NEHRP response spectra for $A_a = A_v$ = 0.1g (BSSC, 1994).

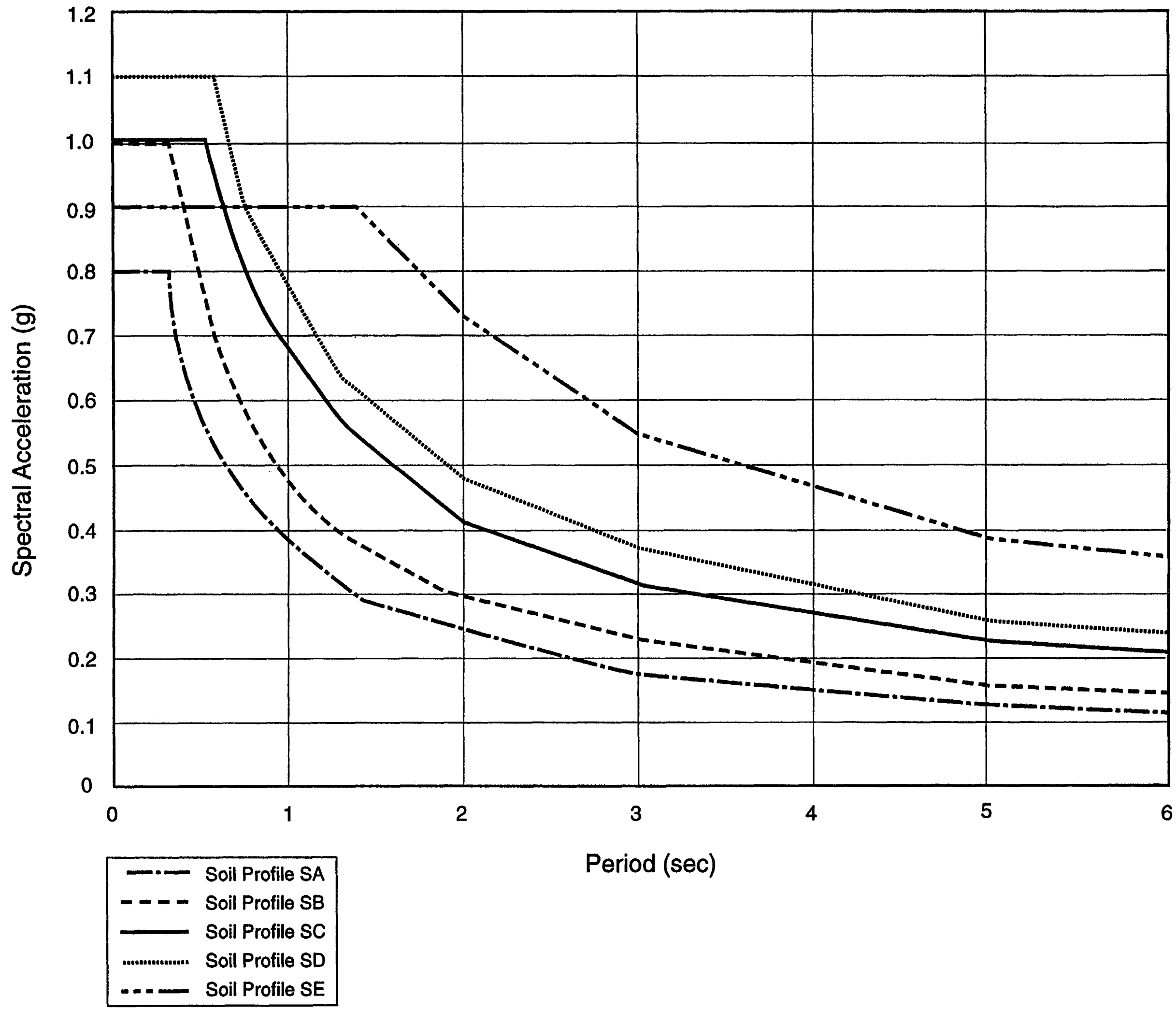


Figure 2-4 NEHRP response spectra for $A_a = A_v$ = 0.4g (BSSC, 1994).

example, five accelerograms must be used. The mean of these spectra must be similar to the elastic spectra shown in Figure 2-5.

Eurocode 8 also provides specific guidance on modifying design spectra for vertical motions. For vertical seismic action, the response spectrum developed for horizontal seismic action is modified by the following factors:

- for vibration periods T smaller than 0.15 seconds, the ordinates are multiplied by a factor of 0.7
- for vibration periods T greater than 0.5 seconds, the ordinates are multiplied by a factor of 0.5
- for vibration periods T between 0.15 and 0.5 seconds, a linear interpolation is used

Japanese Code

The Japanese Code also provides a set of equations for computing structural response coefficients as a function of period. Different equations are used for each of the three ground groups recognized in the code. The modification factors used in these equations were determined from an analysis of earthquake response spectra based on 394 components of strong-motion records obtained in Japan. Figure 2-6 shows the resulting spectra at five percent damping for each of the ground conditions. The Japanese Code includes a damping adjustment factor

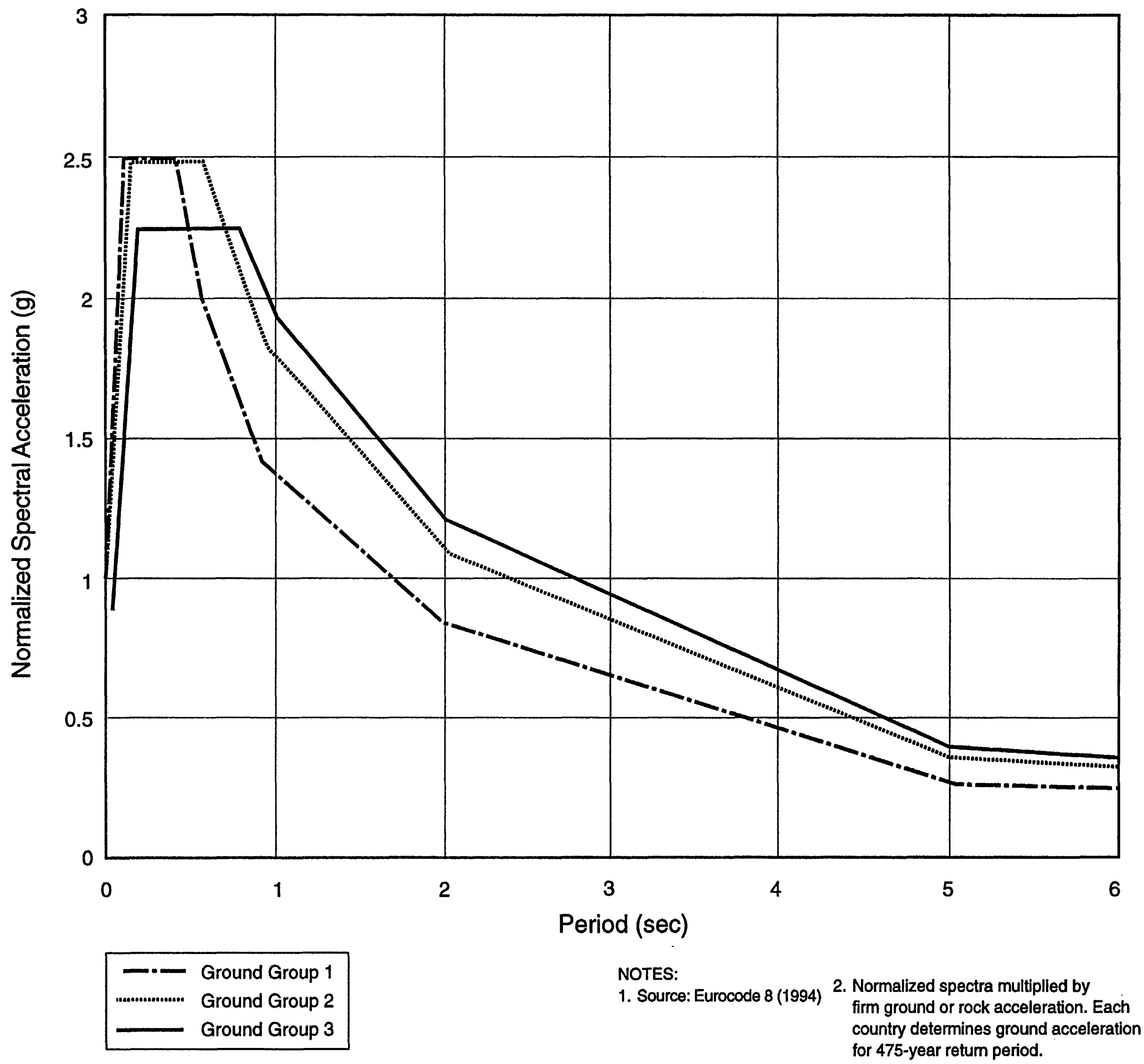


Figure 2-5 Eurocode 8 response spectra.

$$C_D = \left(\frac{1.5}{40h+1}\right) + 0.5 \tag{2-6}$$

where h is the damping ratio.

Iwasaki et al. (1990) indicate that when a time-history analysis is required, strong-motion records that have characteristics similar to the spectra shown in Figure 2-6 must be used. These time histories must also consider site conditions and the structural response of the bridge. The Japanese Code apparently provides ground acceleration records that were modified in the frequency domain so that their response characteristics match the response spectra. The Japanese Code also stipulates that the results of a time-history analysis must be checked against allowable stresses and displacements computed from the seismic coefficient method because of uncertainties in the model and properties used in the time-history analyses.

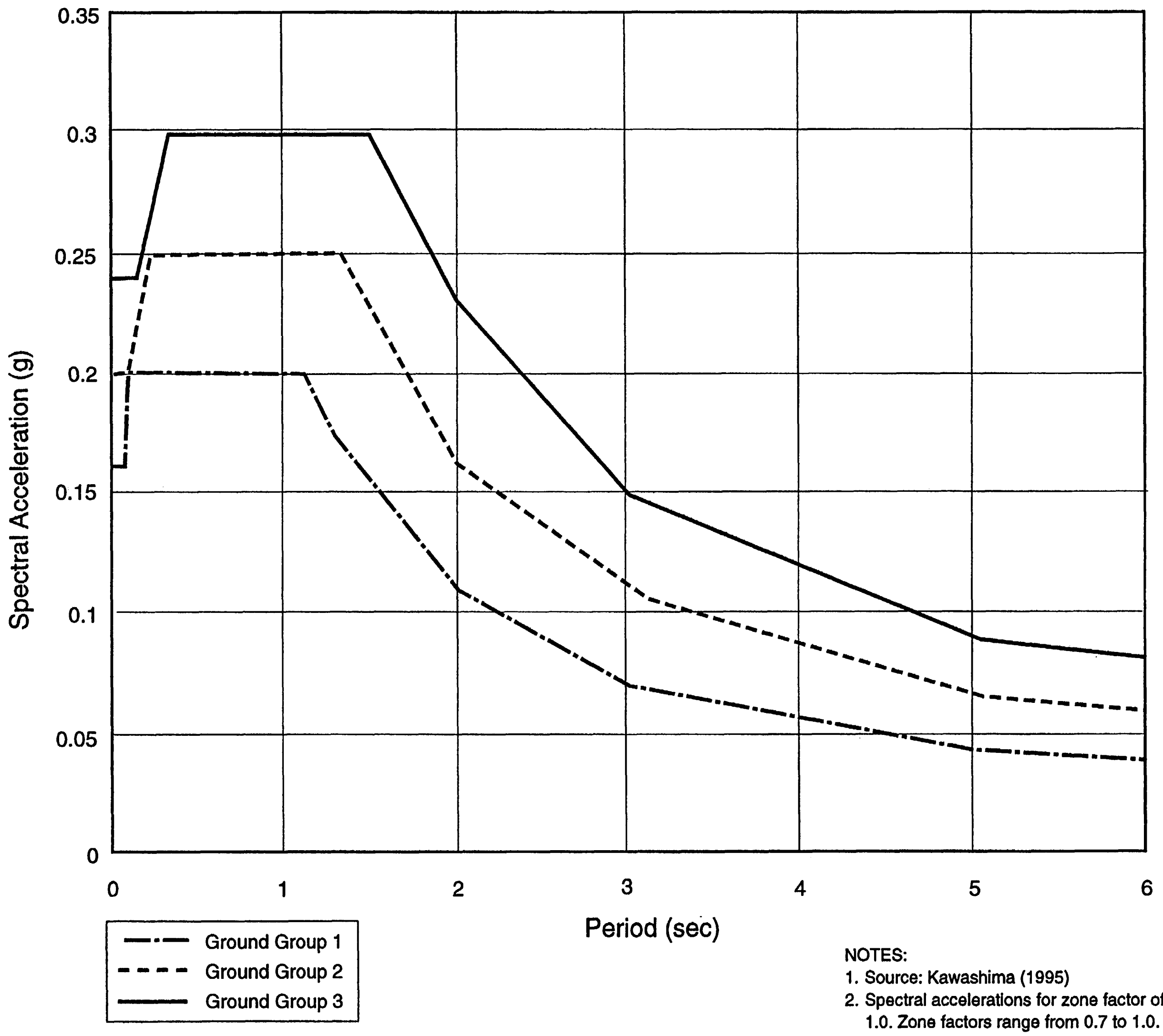


Figure 2-6 Japanese response spectra.

Iwasaki et al. (1990) indicate that the vertical design seismic coefficient may generally be considered as zero, except for bearing supports.

New Zealand Code

The New Zealand Code gives three sets of spectra, one for each soil condition. Within each set, curves are given as a function of a structural ductility displacement factor. Figure 2-7 shows the curves for the three soil conditions at a ductility factor of 1.0.

The New Zealand Code provides some direction on vertical response. This Code notes that the bridge structure should be designed to remain elastic under both positive and negative vertical acceleration. It then goes on to state that the vertical acceleration coefficient should be set at two-thirds the horizontal coefficient.

2.3 Design Strategies and Good Design Practice

Codes provide specific requirements that need to be met as part of the design process. It is difficult, however, to codify good design strategies. Some efforts are being made in this direction, though. The

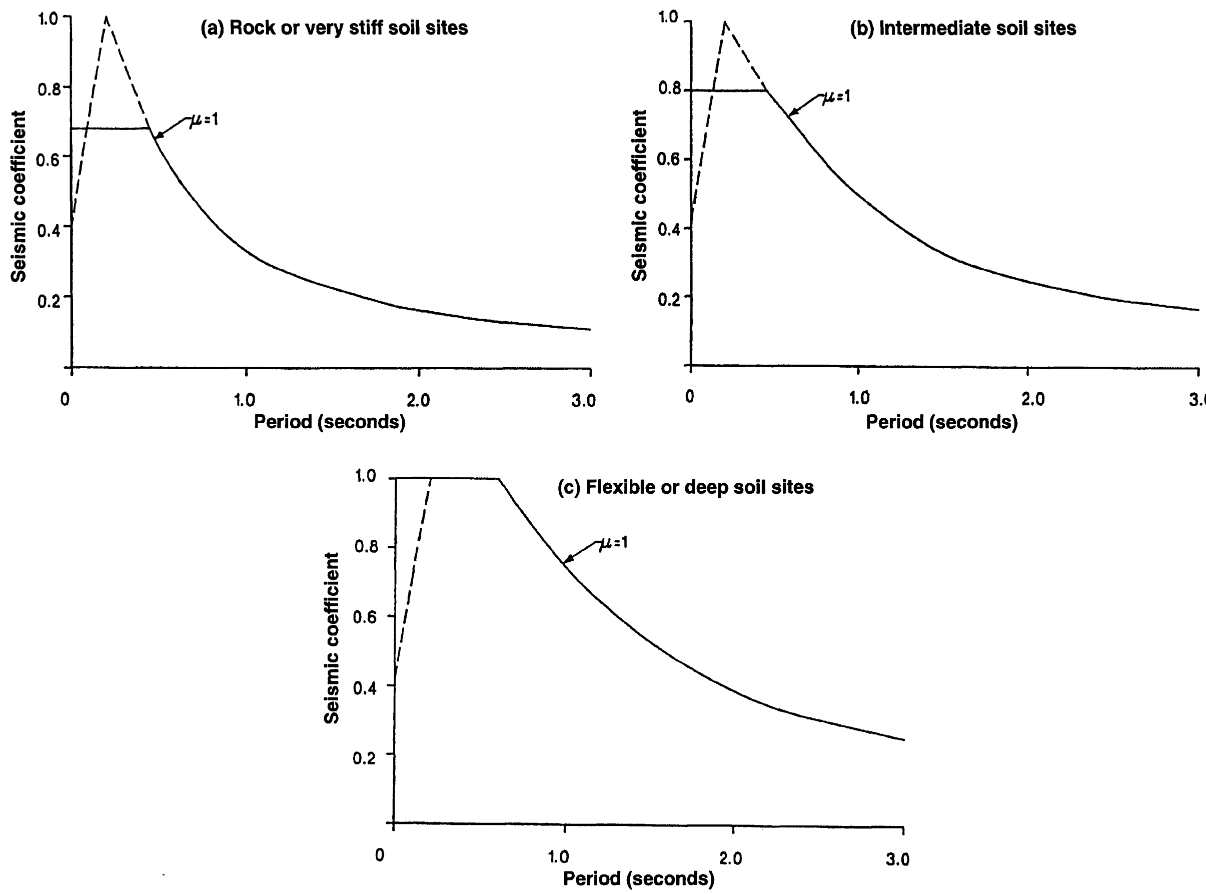


Figure 2-7 New Zealand response spectra.

FHWA Seismic Design and Retrofit Manual, (Buckle et al., 1986) addresses this issue as does the ATC-32-1 report (ATC 1996b).

2.3.1 FHWA Seismic Design and Retrofit Manual

The first attempt in the United States to incorporate a discussion on good and bad structural design concepts was given in the *FHWA Seismic Design and Retrofit Manual* (Buckle et al., 1986). Subsection 5.1.5 of the report containes a series of figures of acceptable and unacceptable structural form with specific comments on each of the concepts presented. Also included are sections on special considerations for bridges with unusual characteristics such as severe curvature, severe skew, piers of differing heights, substructures in deep water, and several other issues.

2.3.2 ATC-32 Caltrans Design Strategies

ATC-32 (ATC, 1996a) states that selection of the appropriate seismic design strategy is something that is best done early in the design process. This seems obvious, but unfortunately in the past, earthquake design was often considered almost as an afterthought. Within California, the earthquake-resistant design of most bridges has involved the use of ductile concrete components as the main lateral-force-resisting members. Use of more innovative approaches such as base isolation and energy dissipation has been limited. This may change, however, if higher performance standards are required for certain bridges. These more innovative methods may be the most cost-effective means of achieving higher performance.

The seismic resistance of typical bridge structures is facilitated by the initial selection of the proper framing scheme. Unfortunately, there are numerous factors besides seismic resistance that often constrain framing options for modern urban bridge structures. Usually, these consist of geometric constraints resulting from the presence of other facilities that are difficult or expensive to relocate. Although these guidelines do not attempt to limit the options of the designer by dictating framing requirements, there are a number of issues relative to framing that should be considered. These are summarized in the ATC-32-1 report (ATC, 1996b).

2.4 Design Level Earthquake Versus Maximum Credible Earthquake

Seismic design in the AASHTO specifications is based on the use of a design event, defined as a 475-year return period event . It is assumed by many design professionals that the use of a uniform hazard in the design process results in uniform risk and that the relationship between hazard and risk is independent of regional seismicity.

Consider the relationship between peak ground accelerations in various cities in the United States, computed for 10% probabilities of exceedance in 50, 150 and 250 years (Reaveley and Nordenson, 1992) presented in Table 2-7 (corresponding to return periods of 475 years, 1300 years, and 2500 years, respectively).

In Table 2-7, the peak ground accelerations for the three probabilities of exceedance are normalized to the 10 percent-in-50-year value for comparison purposes. The Maximum Credible Event (MCE) is often characterized by response spectrum ordinates with a 10 percent probability of exceedance in 100 years in regions of high seismicity and 250 years in regions of low seismicity. Table 2-7 shows that the mean ratio of

Table 2-7 Peak Ground Acceleration (PGA) Comparison by Region and Return Period

City	*UBC Seismic Zone*	*PGA with 10% Probability of Exceedance in 50 years (g)*	*Acceleration Ratios Normalized by 10%-in-50-Year Value*		
			10% in 50 Years	*10% in 150 Years*	*10% in 250 Years*
San Francisco	4	0.40	1.00	1.04	1.06
San Diego	3	0.30	1.00	1.15	1.28
San Bernardino	4	0.40	1.00	1.32	1.41
Anchorage	4	0.40	1.00	1.35	1.53
New Madrid	3	0.30	1.00	2.04	2.70
Wasatch Front	3	0.30	1.00	1.84	2.60
Seattle	3	0.30	1.00	1.32	1.43
Memphis	3	0.30	1.00	1.97	2.33
Boston	2A	0.15	1.00	1.97	2.33
Charleston	2A	0.15	1.00	1.82	2.10
St. Louis	2A	0.15	1.00	1.53	1.83

peak ground acceleration for the 10 percent-in-250-year event to the 10 percent-in-50-year event, for Zone 4, the regions of highest seismicity, is 1.3. The mean ratio for Zone 2A, regions of lower seismicity, is 2.1.

Noting the increase in spectral demands from the 10 percent-in-50-year event to the 10 percent-in-250-year event in Zone 4 is on the order of only 30 percent, it is plausible that a bridge designed for life safety in the 10 percent-in-50-year event in Zone 4 will not collapse in the 10 percent-in-250-year event. However, given the significant difference in spectral demand between the two events in Zone 2A, it is unlikely that a bridge designed for life safety in the 10 percent-in-50-year event in Zone 2A will perform acceptably during the 10 percent-in-250-year event. Expressed in terms of risk, the risk of a bridge collapsing during a 10 percent-in-250-year event is higher if the bridge is located in Zone 2A than in Zone 4. This issue is an important one for future codes to consider.

2.5 Inelastic Versus Elastic Displacement Values

Current U.S. seismic design requirements are based on the premise that displacements caused by the inelastic response of a bridge are approximately equal to the displacements obtained from an analysis using the unreduced elastic response spectrum. As diagrammatically shown in Figure 2-8, this assumes that Δ_{max} (or $\Delta_{inelastic}$) is equal to Δ_e (or $\Delta_{elastic}$).

Recent work by Miranda and Bertero (1994) and by Chang and Mander (1994), indicates that this is a reasonable assumption, except for short period structures for which it is nonconservative. A plot of the results for Miranda and Bertero's work is given in Figure 2-9, where the $\Delta_{inelastic}/\Delta_{elastic}$ ratio is plotted as a function of period or period ratios for various values of the system ductility μ. The ATC-32 recommendations to date are the only U.S. design provisions that include an adjustment factor for short-period structures.

2.6 R-Factor Approach and Nonlinear Response Spectra

Response-modification factors (R in AASHTO and Z in Caltrans) are used to compute the design forces of various components by reducing the elastic force demand. The most important R factor is that of the supporting substructures. Since a bridge closely approximates a single-degree-of-freedom (SDOF) system the design process is shown schematically in Figure 2-8, where the elastic force F_e divided by the R factor determines the design force F_b. A column designed for F_b will have a yield strength F_y in excess of F_b. The amount of overstrength will vary depending on many factors (e.g., real material strength and architectural oversizing), but is expected to be in the range of 1.3 - 2.0.

There has been a considerable amount of research over the past ten years on the relationship between the ductility demand of a SDOF system and its design strength. For example, if we assume an element has a displacement ductility capacity μ of a given value, we would like to know the design force necessary to ensure that this ductility capacity is not exceeded. Results from the work of Miranda and Bertero (1994), Nasser and Krawinkler (1991), and Chang and Mander (1994) will be used to illustrate what we currently know on this issue. Figure 2-8 illustrates the current design process in which the elastic force F_e is divided by the R factor. The actual yield strength or overstrength force level is given by F_y with the overstrength ratio given by F_y/F_b. The relationship of Δ_{max} to Δ_e was discussed in Section 2.5. Figure 2-10 shows the schematic relationship of the overstrength force F_y required for a constant displacement ductility μ_j. The ductility reduction factor R_μ corresponding to a given overstrength F_y value is defined as:

$$R_\mu = \frac{F_e\ (\mu = 1)}{F_y\ (\mu = \mu_j)} \tag{2-7}$$

Figure 2-11 shows a comparison of the design base shear value using R=1 and R=8 compared to inelastic response spectra of μ = 1, 2, 4, and 6 for both rock and soft soil sites. Figure 2-12 is a comparison of the R_μ factor required to achieve displacement ductility values of 3 and 5, respectively, on rock, alluvium, and soft

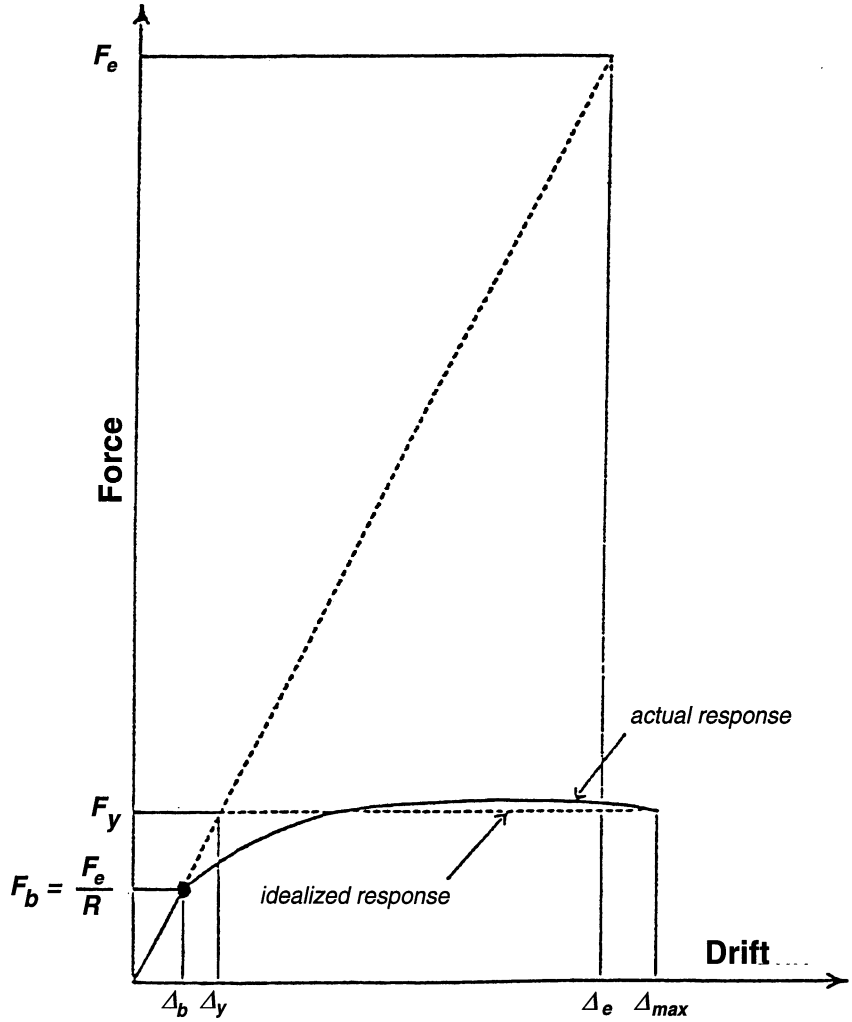


Figure 2-8 Idealized Force-Displacement Relationship.

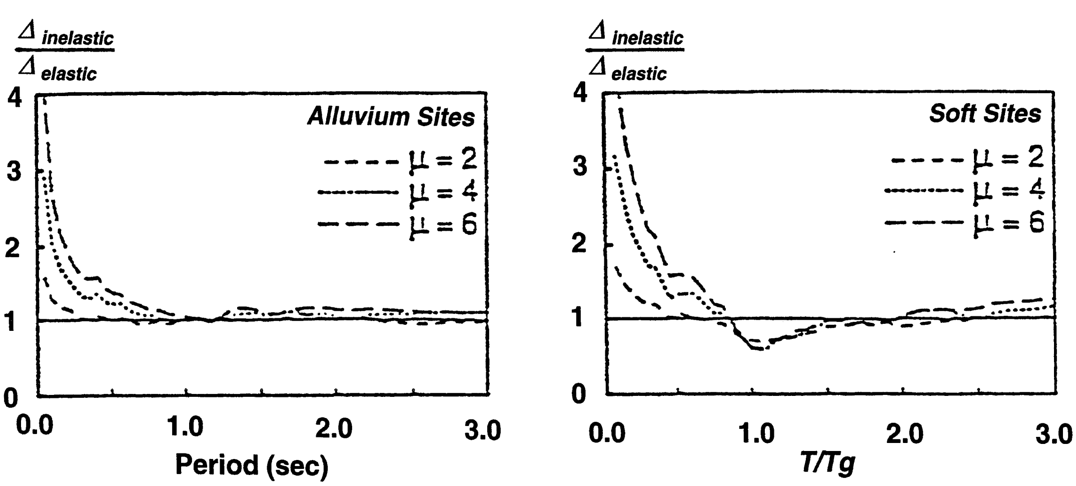


Figure 2-9 Mean Displacement Ratios for Alluvium and Soft Soil Sites (after Miranda and Bertero, 1994)

soil sites. For rock and alluvium sites, note that R_μ is approximately equal to μ when the period exceeds about one second. R_μ is less than μ for periods less than one second. Figures 2-13 and 2-14 show a smoothed relationship developed by Miranda and Bertero (1994) for the three different soil sites. Again, R_μ is less then μ for periods less than one second. In contrast to Figure 2-12 for the soft sites, the plot is a function of T/T_g, where T_g is the predominant period of the soil site. Figure 2-15 is a plot developed by Nasser and Krawinkler (1994) for S_1 sites. It is similar to the rock and alluvium plots of Figures 2-12 and 2-14. This, in fact, shows that R_μ is a little in excess of μ for periods greater than one second.

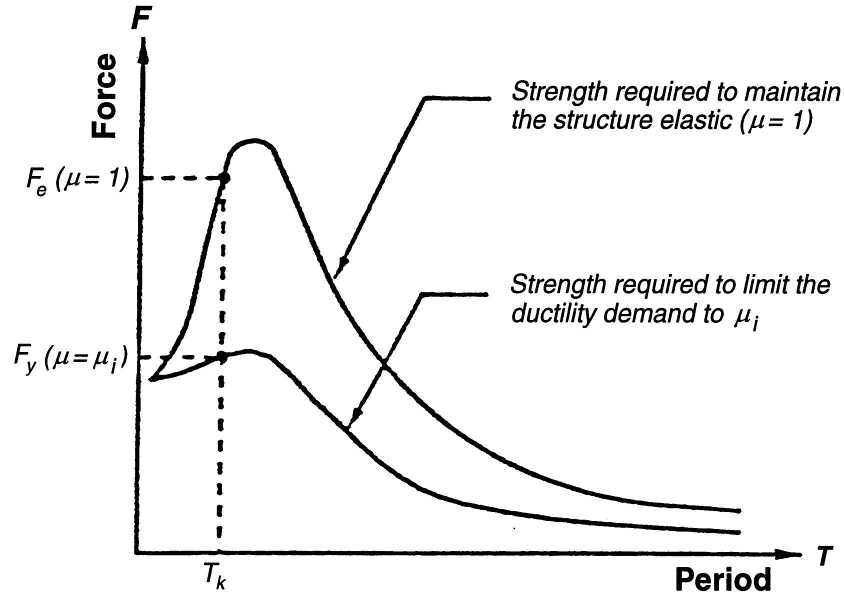


Figure 2-10 Constant Displacement Ductility Nonlinear Response Spectra (after Miranda and Bertero, 1994)

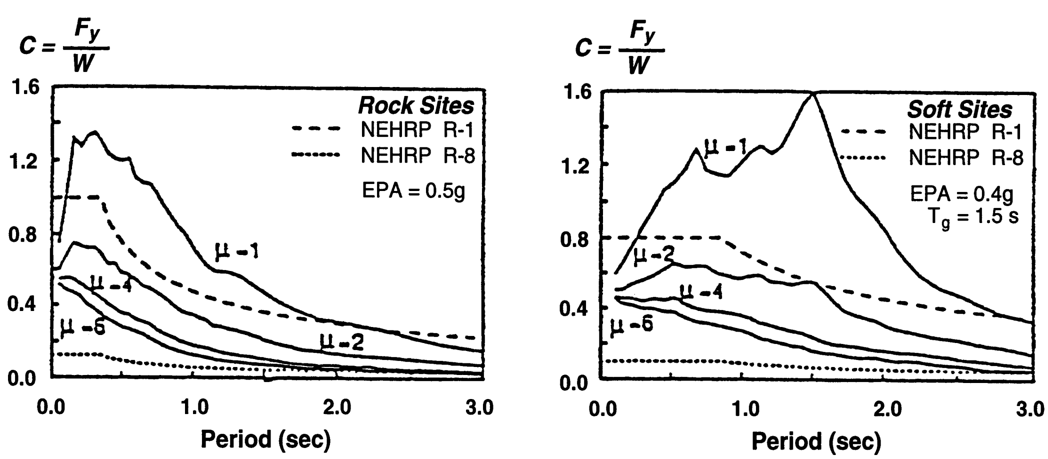


Figure 2-11 Comparison of Mean Strength Demands with Those Required by 1988 NEHRP for New Buildings for Special Moment Resisting Frame Structures (Note: (EPA = effective peak acceleration) (after Miranda and Bertero, 1994)

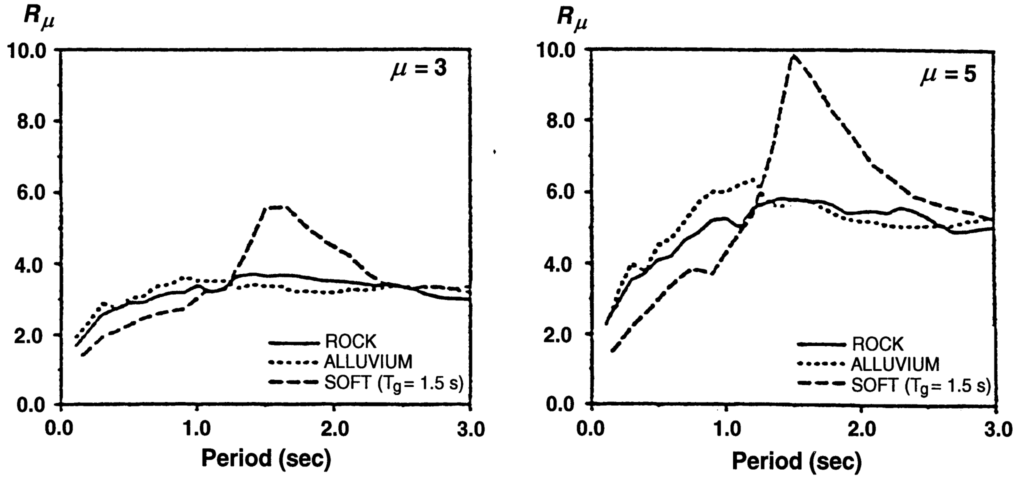


Figure 2-12 Influence of Local Site Conditions on Strength-Reduction Factors (after Miranda and Bertero, 1994)

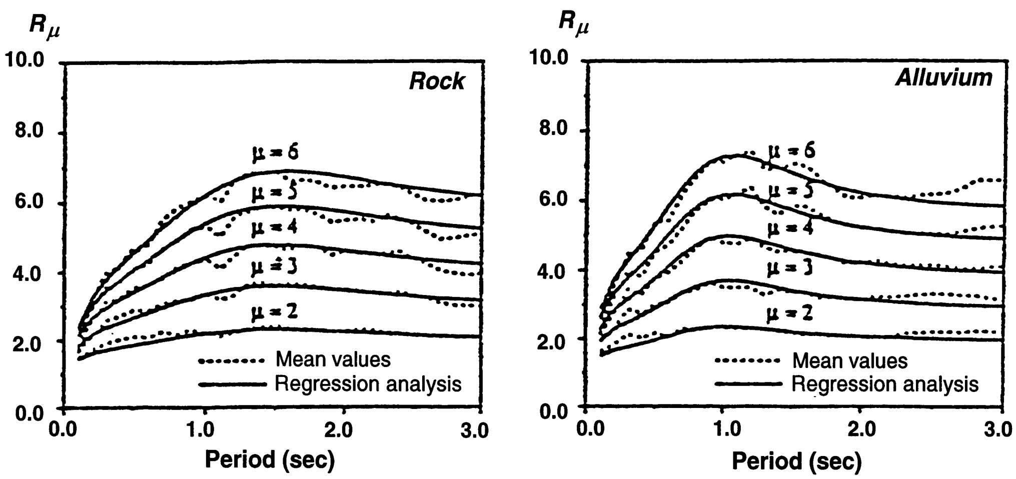


Figure 2-13 Comparison of Mean Strength-Reduction Factors of Rock and Alluvium Sites with Regression Analysis (after Miranda and Bertero, 1994)

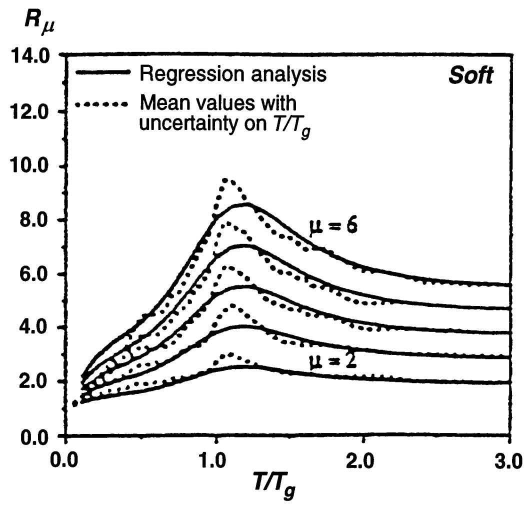


Figure 2-14 Mean Strength-Reduction Factors of Soft Soil Sites Considering 10% Error in Estimation of T/T_g Ratio Compared to Regression Analysis (after Miranda and Bertero, 1994)

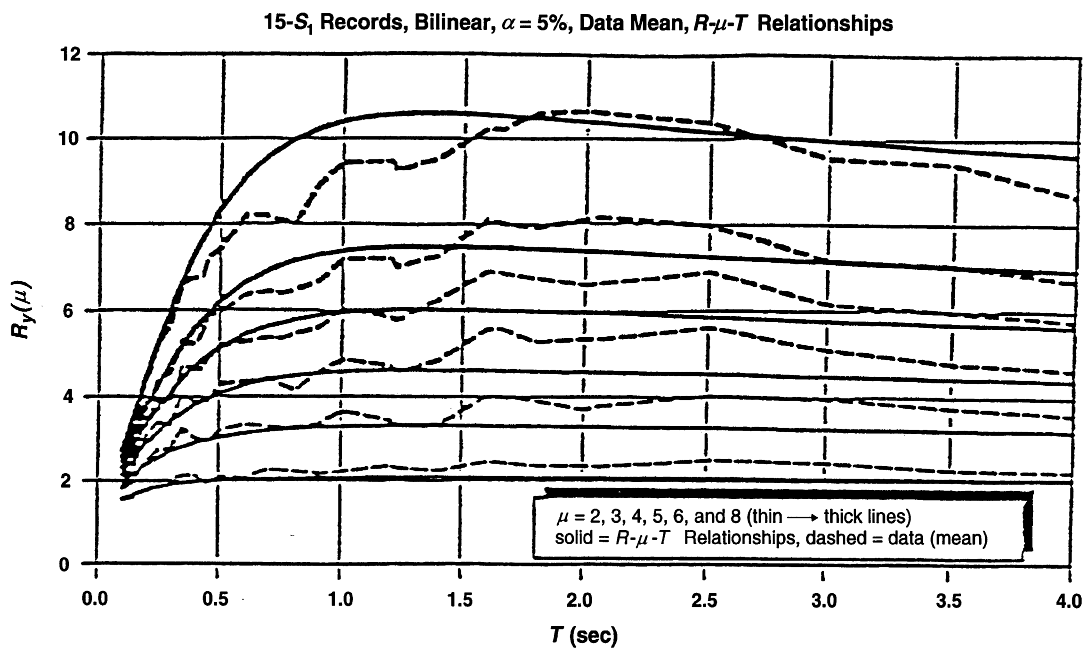


Figure 2-15 Strength-Reduction Factors, $R_y(\mu)$ (after Nasser and Krawinkler 1991)

SECTION 3
UNITED STATES BRIDGE CODE REQUIREMENTS

This section summarizes four seismic design specifications for bridges used in the United States. The AASHTO requirements are presented in Section 3.1. The current Caltrans requirements and the revisions proposed by the ATC-32 project are summarized in Sections 3.2 and 3.3, respectively. The two-level design criteria developed for the Transportation Corridor Agencies in Southern California are presented in Section 3.4.

3.1 AASHTO

The sponsor of a highway bridge project has the choice of which AASHTO code to use: *Standard Specifications for Highway Bridges, 16th Edition* (1995) or *LRFD Design Specifications, 1st Edition* (1994). It is anticipated that AASHTO will decide between the two codes in the future. The seismic design provisions in the 16th Edition are collected in a special supplement, Division I-A, whereas the seismic design provisions of the LRFD code are an integral part of the specifications.

3.1.1 Overview

The AASHTO seismic design goals are: the prevention of collapse in major earthquakes, repairable damage limited to plastic hinge zones in the columns in the design-level earthquake, and minimal damage in earthquakes with a substantial probability of being exceeded within an assumed design life. Analysis is carried out only for the design-level earthquake (defined as an intensity of shaking with a 10 percent probability of being exceeded in 50 years). A map showing contours of the effective peak ground acceleration representing this intensity are included in the specifications.

The design force of plastic hinge zones in columns is determined by dividing the predicted elastic response forces (combined with dead and other permanent load forces as specified for Group VII) by a modification factor R. The value of R is always greater than one and depends on the framing system. The reliable strength of the column must be greater than these design forces. The strength of critical connection elements must be greater than the elastic response forces (R is less than or equal to one for these elements). Foundations are designed using the lesser of (1) the forces calculated using one-half the response modification factor ($R/2$) or (2) the forces resulting from plastic hinging at the columns, considering possible material overstrength. Specific detailing rules are included to ensure the formation of ductile plastic hinges in columns prior to shear failure. "Capacity" design (i.e., prevention of brittle failure modes) is an intended outcome of these specifications but calculations are not explicitly required.

3.1.2 Seismic Input

In the AASHTO procedure, effective peak ground acceleration is taken from a contour map, then modified by a site coefficient and a spectral amplification factor to obtain a seismic response coefficient for use with either single-mode pseudo-static analysis or multi-mode dynamic analysis. The seismic response coefficient is calculated as follows:

$$C_{sm} = \frac{1.2AS}{T^{2/3}} \tag{3-1}$$

where:
C_{sm} = seismic response coefficient
A = effective peak ground acceleration, (a_{max})
S = site soil type coefficient
T = structure period (in seconds)

The maximum value of C_{sm} is limited to 2.5 times A and to 2.0 times A for soil types III (soft to medium-stiff clays and sands) and IV (soft clays and silts) when A exceeds 0.30.

3.1.3 Site Soil Type Coefficient

The site soil type coefficient is determined from a qualitative description of the soil column given in Table 3-1.

An alternative site-specific response coefficient or spectrum may be used in lieu of the soil type factors.

3.1.4 Seismic Performance Category

Bridges are assigned to one of four possible Seismic Performance Categories (SPC) as shown in Table 3-2, based on the value of A and the importance category (essential or other). Analytical methods and detailing requirements vary by the SPC of the bridge.

3.1.5 Analysis Methods

Single-span bridges do not require analysis except that connections between the superstructure and the substructure are designed for the dead load reaction multiplied by the product of the effective peak acceleration A and the site coefficient S. Seismic analysis is not required for bridges in Seismic Performance Category A.

The acceptable analytical methods for multi-span bridges in Seismic Performance Categories B, C, and D range from pseudo-static, single-mode analysis for "regular" bridges to time history analyses for "critical" bridges. Requirements for a bridge to be considered as "regular" are given in Table 3-3.

A curved bridge may be analyzed as a straight bridge with equal span lengths if both of the following are true:

- The bridge is regular as defined in Table 3-3, except that for a two-span bridge the maximum span length ratio from span to span must not exceed two.
- The subtended angle in plan is not greater than 30 degrees.

Table 3-1 Current AASHTO (1995) Site Soil Coefficients

Description	*Soil Profile Type*	*Site Coefficient S*
Rock characterized by a shear wave velocity >2500 ft/sec (750 m/sec)	I	1.0
Stiff soil conditions where the depth to rock is <200 ft (60 m) and the overlaying soil is stable sand, gravel, or stiff clay		
Stiff clay or cohesionless material where the depth to rock is >200 ft (60 m) and the overlaying soil is stable sand, gravel, or stiff clay	II	1.2
Soft to medium-stiff clays and sands characterized by 30 ft (9m) or more of soft to medium-stiff materials without intervening layers of sand	III	1.5
Soft clays or silts >40 ft. (12 m) in depth characterized by a shear wave velocity <500 ft/sec (150 m/sec)	IV	2.0

Table 3-2 AASHTO (1995) Seismic Performance Categories

Acceleration Coefficient	*Importance Classification*	
A	*Essential*	*Other*
$A \leq 0.09$	A	A
$0.9 < A \leq 0.19$	B	B
$0.19 < A \leq 0.29$	C	C
$0.29 < A$	D	C

Suitable analytical methods include:

Uniform Load Method. The uniform load method is for bridges where the response is primarily in the first mode in each of two orthogonal directions (usually parallel and perpendicular to the longitudinal centerline of the bridge). It can be used for regular bridges and is a conservative approximation for cases in which the generalized stiffness of the bridge is calculated as the ratio of the total applied load to the maximum displacement under a uniform load. The natural period of the bridge is calculated from the total weight of the bridge and the generalized stiffness. The seismic response coefficient is calculated from Equation 3-1 for this period and the seismic loading is then a uniform load given by the product of the distributed weight and the seismic coefficient.

Single-Mode Spectral Analysis. This method can be used for regular bridges and is essentially a Rayleigh analysis for the anticipated first mode response in each direction. Equations are given for the generalized stiffness, mass, and the mass participation for a given mode shape. This method is a refinement of the uniform load method.

Multi-Modal Response Spectrum Analysis. This method is required for all irregular bridges. A lumped-mass model is used with sufficient nodes in the deck and columns to capture all relevant motions. The suggested number of modes is three times the number of spans (but not more than 25) or the number of modes required to obtain a mass participation factor of 95 percent of the total mass. Modal responses are combined using the complete quadratic combination (CQC) method.

Table 3-3 AASHTO (1995) Requirements for a Regular Bridge

	Number of Spans				
	2	3	4	5	6+
Maximum subtended angle (curved bridge)	90°	90°	90°	90°	90°
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span	-	4	4	3	2

Time History Analysis. For this method, a minimum of five spectrum-compatible time histories are to be used. If an inelastic time-history analysis is performed, the response modification factor R used to calculate required component strength values must be 1.0. Suggestions for modeling are given in the code and commentary.

3.1.6 Component Design

The design loading of a component is determined by the lesser of (1) the elastic response force divided by a response modification factor R, or (2) the maximum force that can be produced by development of the plastic hinge mechanism. It is intended that inelastic energy dissipation be limited to plastic hinging in the columns.

To account for directional uncertainty of the seismic input motions, the elastic forces from the analysis in each of two orthogonal directions (longitudinal, L, and transverse, T) are combined as 100 percent of the response in one direction plus 30 percent of the response in the orthogonal direction; i.e., $L + 0.3T$ and $0.3L + T$.

3.1.7 Response-Modification Factors

Response-modification factors to account for inelastic action in selected areas are based on general framing types or component location and/or function, as detailed in Table 3-4.

Table 3-4 AASHTO (1995) Response-Modification Factors

Substructure[1]	*R*	*Connections*[3]	*R*
Wall-type pier[2]	2	Superstructure to abutment	0.8
Reinforced concrete pile bents		Expansion joints within a span of the superstructure	0.8
a. Vertical piles only	3		
b. One or more batter piles	2		
Single Columns	3	Columns, piers, or pile bents to cap beam or superstructure[4]	1.0
Steel or steel & composite steel & concrete pile bents		Columns or piers to foundations[4]	1.0
a. Vertical piles only	5		
b. One or more batter piles	3		
Single columns	3		
Multiple-column bent	5		

1. R factor to be used for both orthogonal axes of the substructure.
2. Wall-type pier may be designed as a column in the weak direction using the R factor for a single column if appropriate ties are provided.
3. Mechanical devices designed to transfer shear and axial forces between components (e.g., bearings and shear keys).
4. Connections may be designed for the maximum forces capable of being developed by plastic hinging in the columns, considering an overstrength value in the plastic hinges of 1.30 times the nominal moment capacity corresponding to the axial load present.

Response-modification factors for abutments and foundations are equal to one-half the value used for the columns, except that foundations need not be designed for forces larger than those that can be transferred. Response-modification factors for pile bents are not reduced. The forces from plastic hinging in the columns will usually control foundation design in seismic performance Categories C and D. It is also a safe and conservative method for all categories.

3.1.8 Component Design

Reduced seismic response forces are combined with permanent forces from dead load and other long-term permanent forces in Group VII, with a group factor of 1.0 for use in component design.

Concrete Columns. Component design for concrete members is based on strength design principles. Component nominal strength is reduced by a capacity-reduction factor (less than one). This capacity-reduction factor is 0.90 for spiral reinforced columns and 0.85 for tied columns with an axial load ratio less than 0.10. The axial load ratio is the axial load divided by the product of the gross area and the concrete compressive axial load divided by the product of the gross area and the concrete compressive cylinder strength f_c. For tied columns with an axial load ratio of 0.20 or greater, the capacity-reduction factor is 0.50. Linear interpolation is specified for axial load ratios between 0.10 and 0.20. This provision is not used in all jurisdictions because it is thought by some to be overly conservative.

Concrete columns in Seismic Performance Categories B, C, and D are subject to detailing rules intended to ensure formation of plastic hinges prior to shear or compression failure and to ensure adequate ductility for earthquakes larger than anticipated.

Longitudinal reinforcement is required to be between 1.0 percent and 6.0 percent of the gross area except where, for architectural reasons, the column has a section greater than required, in which case the section of similar shape for which the reinforcement is equal to 0.01 A_g (A_g = gross concrete area) must have the required nominal strength.

Sufficient transverse reinforcement is required in the potential plastic hinging regions to ensure that the axial strength of the confined core is equal to the maximum axial strength of the gross section prior to spalling of the cover. The required volumetric spiral reinforcement is the greater of

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c}{f_{yh}} \tag{3-2}$$

or

$$\rho_s = 0.12\frac{f_c}{f_{yh}} \tag{3-3}$$

where

A_g = gross concrete area
A_c = area of confined concrete core
f_c = concrete compressive cylinder strength
f_{yh} = yield strength of transverse reinforcement

The total cross-sectional area of hoop reinforcement in tied columns in each direction is the greater of:

$$A_{sh} = 0.30ah_c\frac{f_c}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right) \tag{3-4}$$

or

$$A_{sh} = 0.12ah_c \frac{f_c}{f_{yh}} \tag{3-5}$$

where

a = spacing of the ties in vertical direction
h_c = dimension of the column in the direction of the tie.

Ties are required to have a 135-degree hook on each end with an extension not less than ten tie diameters or six inches. This requirement often leads to constructibility problems in tied columns and hooks are often provided with a 135-degree hook on one end and a 90-degree hook on the other to facilitate placement. The 135-degree hook ends are alternated.

Spacing of transverse reinforcement depends on the Seismic Performance Category and may be decreased by 100 percent in regions outside of the potential plastic hinging regions. These are defined to exist within a length of column that is the greater of (1) the cross-sectional column dimension or (2) one-sixth of the clear height at the top and bottom of the column, but not less than 18 inches (450 mm). Transverse reinforcement in the columns must also be checked to see that premature shear failure does not occur when overstrength (130 Percent M_n) plastic hinges occur in the column. M_n is the nominal moment capacity calculated without ϕ factors.

Lapping of longitudinal reinforcing within the potential plastic hinging regions is not allowed. Splicing of spiral reinforcement is required to be such as to develop the full strength by welding or by mechanical splices.

*Steel Columns***.** Component design for steel members follows either load-factor or service-load design principles. If service-load design is used, the allowable stresses are permitted to increase by 50 percent for Group VII loads. Provisions for inelastic action in noncompact sections are not well codified.

P-Δ effects are required to be considered in component design.

Longitudinal Linkages. For Seismic Performance Categories C and D, positive horizontal linkages must be provided between adjacent superstructure units and between the superstructure and the abutment. The strength of the linkage must be equal to the weight of the lighter unit times the effective peak ground acceleration.

Hold-down Devices. Hold down devices must be provided at all supports where the upward elastic seismic force exceeds 50 percent of the dead-load reaction. For a vertical seismic force between 50 percent and 100 percent of the dead load reactions, the device must be designed for a force equal to 10 percent of the dead-load reaction assuming simple spans.. For a vertical seismic force exceeding 100 percent of the dead-load reactions the device must be designed for 1.2 times the net vertical force, but not less than a force equal to 10 percent of the dead-load reaction assuming simple spans.

Displacements. Requiring minimum seat widths protects against displacements causing loss of vertical support. These seat widths are empirical. The required seat width for Seismic Performance Categories C and D is

$$N = (12 + 0.03L + 0.12H)(1 + 0.000125\,S^2) \tag{3-6}$$

where

N = seat width perpendicular to the skew (in inches)
L = length (in feet) of the bridge deck to the adjacent expansion joint or the end of the deck
H = average height (in feet) of the columns supporting the units on either side of the support or joint under consideration
S = skew angle of support (degrees) measured from a line normal to the longitudinal centerline of the span(s).

For Seismic Performance Categories A and B, the seat length is two-thirds of that for seismic performance Categories C and D.

The corresponding metric formulation is

$$N = (305 + 2.5L + 10H)(1 + 0.000125\, S^2) \tag{3-7}$$

where

N is in mm and L and H are in meters.

This seat length provision is an attempt to provide for out-of-phase motion of the two units on either side of the support or joint under consideration. Predicted seismic displacement values are based on the assumption that displacements of the inelastic structure will be the same as the displacements of the elastic structure.

3.2 Caltrans BDS

3.2.1 Introduction

Current Caltrans design criteria for new bridges (Caltrans, 1993) is supplemented by a number of internal design standards described at various locations within the following internal publications:

1. *Memos to Designers Manual*: A collection of design directives related to various types of bridge construction.

2. *Bridge Design Aids Manual*: A manual that contains design standards, charts, and tables used for production design.

3. *Bridge Design Details Manual*: A manual of bridge detailing standards.

Bridge construction is also governed by standard specifications for construction and by standard special provisions that are used to generate project-specific special provisions for construction. The design manuals and construction specification standards are continuously updated to reflect new developments.

Since the 1989 Loma Prieta earthquake, Caltrans bridge design standards have been modified on a project-by-project basis. These modifications reflect research findings, postearthquake observations, and input from a number of external committees that were established as a result of that earthquake and the subsequent investigation by the Governor's Task Force. These committees include the Seismic Advisory Board, project-specific Peer Review Committees, and the Seismic Research Advisory Committee formed to help guide Caltrans' new seismic research program.

The following paragraphs summarize the current official Caltrans BDS with some comments about changes that are being made internally on a project-specific basis.

3.2.2 Overview

The current BDS uses a single-level force design approach based on the concept of a maximum credible earthquake (MCE). Seismic forces are determined from a multi-modal, response-spectrum analysis of a model with gross section properties using elastic (*ARS*) design spectra. Elastic forces are reduced for ductility and risk using *Z* factors that are prescribed on a component-by-component basis.

$$F_{EQ} = \frac{ARS}{Z} \tag{3-8}$$

These forces are included in AASHTO Group VII loading and are combined with dead load and prestress forces for component design. Design is based on the load-factor method and utilizes nominal ultimate capacities modified by capacity-reduction factors (ϕ) that are often greater than unity. When applicable, a capacity-design concept is used for the design of certain components that are either subject to brittle failure (e.g., column shear) or are to be otherwise protected from failure (e.g., foundations). Design loads for these types of components are based on the maximum force that can be generated in adjacent ductile components.

3.2.3 Seismic Loading

Standard elastic design spectra are based on peak rock accelerations *A*, normalized rock spectra *R*, and soil amplification factors *S* and are known as *ARS* curves. A typical *ARS* curve is shown in Figure 3-1.

A statewide risk map of expected peak rock acceleration values was originally prepared by the California Department of Mines and Geology (CDMG). This map was developed by identifying all known faults within the state capable of generating significant earthquakes. The current version reflects potential earthquakes from over 200 faults and fault zones. Each fault has been evaluated to estimate the magnitude of its MCE based on the estimated maximum length of fault rupture. Maximum peak rock accelerations for each location in California are determined by using average attenuation relationships based on the magnitude of the MCE and the distance from the causative fault. The value of *A* to be used in design is taken from the risk map unless otherwise determined by site-specific studies. It should be noted that *A* is not the highest possible peak acceleration since an average attenuation model was used. Also, because of variations in the frequency of seismic activity on different faults, these peak accelerations do not reflect a uniform probability of exceedance for all California sites.

The current *R* factors used by Caltrans are based on normalized, smoothed, five-percent damped acceleration spectra on rock derived from five separate California earthquakes. The shape of these spectra have been adjusted by lengthening the period region of maximum response for lower peak rock accelerations. This reflects the increase in predominant periods at increased distance from the causative fault, which is implied by lower peak rock accelerations.

The current standard Caltrans spectra consider four different types of sites. These sites are distinguished by the depth of cohesionless alluvium (sands and gravels) over "rock-like" material. Peak acceleration and period-dependent *S* factors were originally derived from the SHAKE (Schnabel et al., 1972) computer program. Although research is currently ongoing to develop spectra for deep clay sites, these are not described in the current BDS.

In the wake of the Loma Prieta earthquake, it has become more common for site-specific spectra to be developed for design.

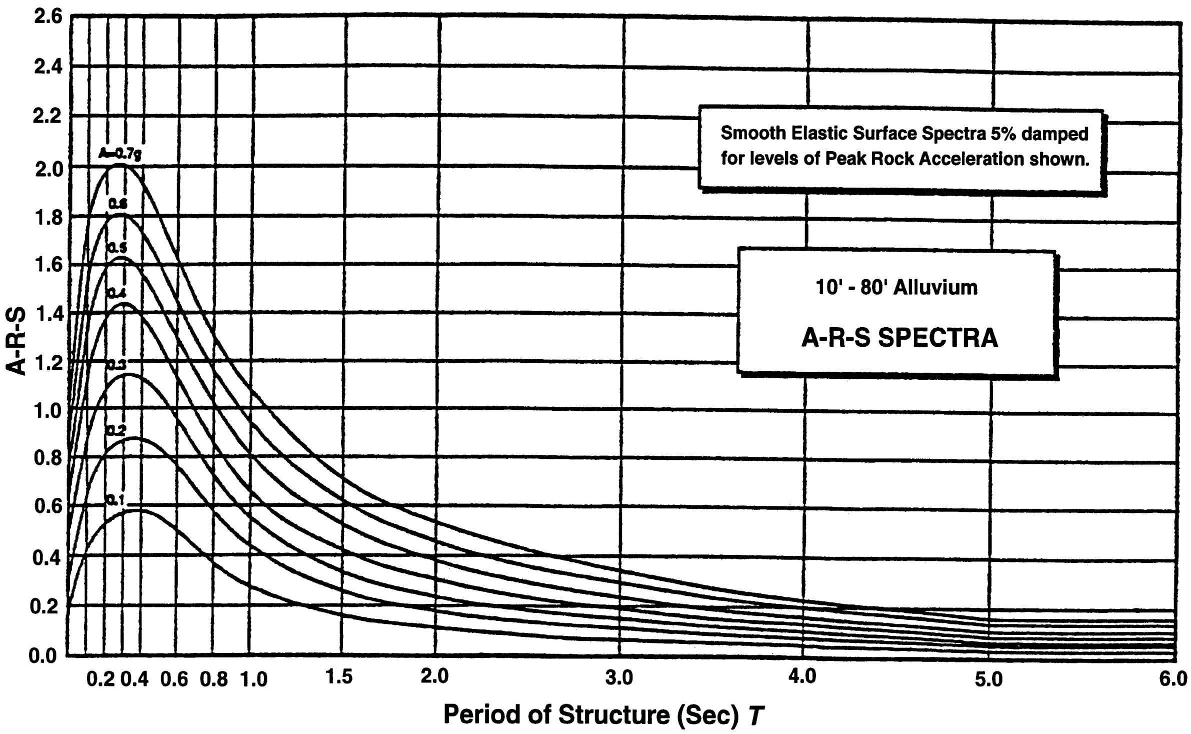


Figure 3-1 Typical Caltrans ARS curve.

3.2.4 Analysis

Although the BDS allows an equivalent static force approach for the seismic design of very regular structures, most bridges in California are designed using dynamic analysis. This approach typically involves a multi-modal, response-spectrum computer analysis of a 3-D lumped-mass space frame. This is usually done even when it is not required, because the computer resources (hardware and software) are readily and economically available to most designers.

Computer modeling of bridge structures for elastic dynamic analysis generally follows several rules of thumb. Mass is often lumped at three or more locations within the center portion of spans as well as at the ends. Similarly, columns will have lumped masses placed at two or more internal nodes. Gross-section properties are commonly used when design forces are being sought, while cracked-section properties are used to determine displacements at the location of plastic hinges. The use of foundation springs to simulate soil-structure interaction has become increasingly popular. Trial and error adjustment of spring constants to achieve force-displacement compatibility is usually performed, particularly at abutments.

An attempt to envelope the effect of the nonlinear behavior of expansion joint hinges is accomplished through the use of "tension" and "compression" models. In the tension model, longitudinal stiffness equivalent to the stiffness of restrainer cables is provided across a joint. In the compression model, expansion joints are locked together longitudinally.

To account for the directional uncertainty of earthquake motion, the Caltrans BDS requires that forces resulting from longitudinally and transversely directed earthquakes be combined. The 30 percent rule, in which forces from the primary direction are combined with 30 percent of the forces from the orthogonal direction, is specified. No provision is made for vertical motion.

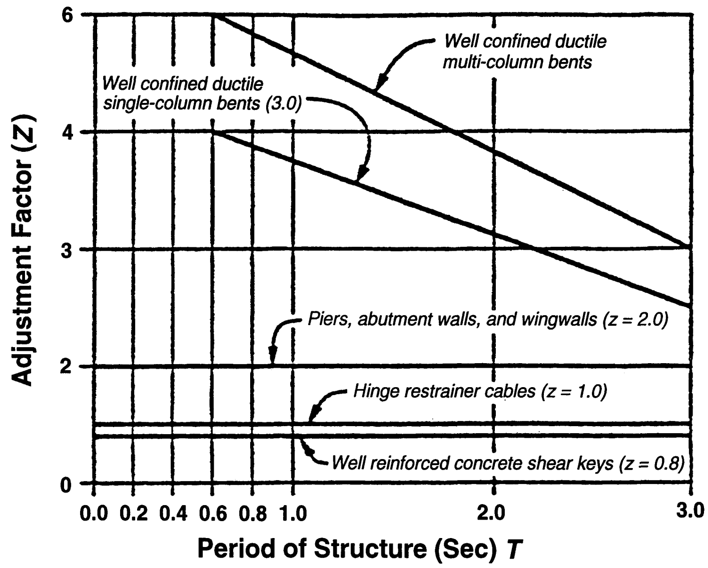


Figure 3-2 Caltrans Adjustment factors for ductility and risk (Caltrans, 1993).

Elastic forces from a dynamic or equivalent static analysis are modified by component-and period-dependent Z factors to account for ductility and risk. The Z factors used in the current BDS are shown in Figure 3-2. It should be noted, however, that following the Loma Prieta earthquake most designers abandoned these Z factors. Recently, in the interest of uniformity, Caltrans has tentatively adopted factors proposed by ATC-32 for use in design.

Design of restrainer units is not based on computer dynamic analysis, but on a simplified equivalent static procedure that considers the longitudinal response of individual frames on either side of the expansion joint (and the transverse response for curved structures), the elastic displacement capacity of restrainers, the seat width, and the expansion joint gap. In this procedure, the displacement capacity of the joint is determined from either the physical dimensions of the bearing seat or the elastic displacement limit of the restrainers. This value is compared with the displacement demand at the joint, which is determined from an equivalent static analysis that may consider the effect of adjacent frames. If capacity exceeds demand, then nominal restrainers are indicated. If the converse is true, then restrainers must be provided to limit restrainer forces and displacements to an acceptable level. With new minimum seat width values, this analysis will most often result in a nominal number of longitudinal restrainers for new bridges. It is generally preferable to increase the seat width and restrainer displacement capacity rather than to modify structural response through the use of extra restrainers.

3.2.5 Design

Caltrans typically uses reinforced or prestressed concrete superstructures that are continuous with reinforced concrete columns and/or piers. Seismic design of Caltrans bridges has traditionally focused on columns and piers, abutments, foundations, and expansion joints. Recently, more attention has been directed toward superstructure components. There has been a particular focus on assuring ductile behavior of superstructure/column joints and preventing plastic hinging in continuous bent caps and girder stem/deck systems. Also, unusual bridge configurations such as double-deck structures and structures with outrigger and/or "C"-bents have been the subject of considerable research.

Columns And Piers. Caltrans uses several design methods to assure ductile behavior in columns and piers. These include:

- Longitudinal reinforcement is limited to between one and eight percent of the gross concrete area of columns. This minimum percentage is reduced to 0.25 percent for piers.
- The proportion of transverse shear reinforcement is determined by capacity design that considers the probable overstrength of column moment capacity. The design shear capacity of concrete is reduced in plastic hinge zones with low axial load.
- Transverse confinement reinforcement ratio for spirals is determined by the following equations:

$$= 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y}\left(0.5 + \frac{1.25P_e}{f'_c A_g}\right) \tag{3-9}$$

for columns ≤ 3 feet in diameter;

$$= 0.12\frac{f'_c}{f_y}\left(0.5 + \frac{P_e}{f'_c A_g}\right) \tag{3-10}$$

for columns > 3 feet in diameter.

- Tied column/pier confinement reinforcement requirements are similar to those for spiral reinforcement.
- Reduced confinement and reduced ductility are allowed for some piers.
- Spirals are to be welded or mechanically spliced.
- A706 reinforcement is specified.
- No longitudinal lap splices are allowed in plastic hinge zones. Splices are generally only allowed in the middle one-third of the column length.
- ACI anchorage requirements, which are more conservative than those in the 1993 BDS, are being met.
- Maximum spacing of longitudinal and transverse steel is specified.
- Nominal column moment capacities are increased by a factor ϕ that ranges in value between 1.0 and 1.2 depending on the axial force in the column.
- Z factors currently in use result in higher ductility demands for multi-column bents than for single-column bents.

Abutments

Abutments are generally designed for limited displacement values of about three inches of soil movement. Required seat widths are similar to those specified in ATC-6 with an added factor for skew. The ultimate capacity of the soil behind abutments is given as 7.7 ksf for typical abutments eight feet or more in height. Ultimate lateral pile capacity for typical Caltrans piles is nominally taken as 40 kips, although higher capacities are allowed for heavy steel piles. Abutment shear keys are designed to fail prior to failure at abutment foundations. Caltrans policy is to not allow lateral seismic loads to govern the number of piles. Abutment walls are generally detailed to provide a nominal amount of ductility.

Foundations

Controlling design forces for bent and pier foundations are usually determined from the plastic moment capacity at the base of the column or pier. In rare cases, the elastic forces derived from the analysis of *ARS* spectral loading may control the foundation design. In general, damage to foundations is to be prevented whenever possible.

Spread footings are allowed to experience some uplift on the edges due to rocking during an earthquake, but this behavior is generally limited to approximately one-third to one-half of the bearing area. Soil bearing capacities during uplift are limited to values that do not cause excessive settlement or materially affect subsequent footing behavior.

Pile footings are seismically designed for ultimate pile loads that are generally twice the nominal capacities in compression and 50 percent of this value in temporary uplift. Pile anchorage to develop uplift must be given adequate consideration. Past Caltrans policy has been to not allow the number of piles to be governed by lateral pile capacity during seismic loading, although this practice is changing. Because batter piles are generally discouraged, it is usually necessary to use larger-diameter piles or rely on passive pressure against the pile cap to obtain the required capacity. Standard concrete pile types are reinforced for ductile behavior.

The thickness and reinforcement ratios of all pile caps and spread footings are selected to prevent damage to the foundation. Caltrans has always required lateral column reinforcement to extend into footings, and recent peer review panels have encouraged the use of additional lateral and vertical reinforcement in the footing/column joint to improve the ductile behavior of these joints.

Large-diameter drilled shafts have become a popular foundation type in recent years. These elements are generally designed as extensions of the columns. When they are of the same or similar diameter to the columns and continuous with them, plastic hinges will generally occur below grade in conflict with the design philosophy of preventing damage to the foundation. When this type of foundation is used, the added foundation flexibility is included in the analysis and a computerized *P-y* curve analysis is used to determine internal design forces. Minimum longitudinal and transverse steel spacing is preferred in these shafts for constructibility reasons.

Expansion Joints

Mid-span expansion joints are common in California bridge construction, although the current trend is to eliminate these joints whenever possible. Minimum seat widths are based on the ATC-6 requirements, with an added factor for joint skew. When longitudinal expansion joint restrainers are required, their capacity is determined from a simplified analysis. These restrainers are designed to remain elastic during an earthquake which usually requires that they be lengthened and anchored behind the adjacent bent cap. Vertical restrainers are also required when the uplift due to predicted earthquake motions exceeds 50 percent of the dead-load reaction. Transverse shear keys at joints are designed for 125 percent of the elastic load determined from dynamic analysis.

Bent Caps

Standard Caltrans practice is to place additional superstructure reinforcement in the top and bottom of prestressed box girders at continuous bents. There are similar requirements for bent caps. This practice is intended to prevent plastic hinging of the superstructure. In addition, bridge replacement projects following the Northridge earthquake required that the superstructures be designed for vertical earthquake-induced forces equivalent to 50 percent of the dead load in uplift and an additional 50 percent of the dead load in the direction of gravity. Special joint shear reinforcing requirements for bent caps have been specified for some peer-reviewed design projects. These requirements include extra lateral and vertical reinforcement within the joint zone to handle joint shear. The latest of these requirements is based on research conducted at the University of California at San Diego.

3.2.6 Summary

The current Caltrans BDS, which was essentially developed in 1975, represents a pioneering effort and it incorporates many ideas that are still well accepted today. Nevertheless, recent earthquakes have pointed out a number of weaknesses that need correction. Corrections have been made in the form of interim provisions that are often project- and/or design-section-specific, thus resulting in a certain degree of confusion among California bridge designers. The ATC-32 project is intended to overcome this fragmentation by consolidating many changes into a single, coherent seismic design specification.

3.3 ATC-32 Recommendations to Caltrans

3.3.1 Introduction

The principal change in the ATC-32 recommended specifications (ATC, 1996a) is the recognition that analyzing relative displacements is key to evaluating the seismic performance of bridges. Therefore, the recommendations provide for a more realistic determination of displacement response as well as a more scientifically precise consideration of displacement capacity. Requirements vary depending on the type of structural action upon which the design is based. Additional improvements were made to the design response spectra, foundation design provisions, and reinforced concrete criteria. New provisions were added for steel structures and conventional bridge bearings.

The following summary describes draft recommendations. Unless otherwise noted, the ATC-32 recommendations are similar to the current Caltrans BDS.

3.3.2 Overview

One of the starting points for the ATC-32 project was the specification by Caltrans of seismic performance criteria for new bridges. These criteria are summarized in Table 2-1.

In these criteria, design earthquakes are defined for both safety and functional evaluation. The safety evaluation earthquake, which Caltrans currently defines deterministically as the "Maximum Credible Earthquake", has only a small probability of occurring during the useful life of the bridge. In the newly defined performance criteria, this earthquake may alternately be defined probabilistically as an earthquake with a 1000- to 2000-year return period.

The functional evaluation earthquake is intended to represent an earthquake that has a reasonable probability of occurring during the life of the bridge. Because no statewide risk map such earthquakes has been developed at this time, the seismic inputs for functional evaluation must be determined on a case-by-case basis through site-specific studies.

Performance is defined in terms of both the service level or usability of the structure immediately following the earthquake and the extent or repairability of physical damage. The required performance depends on the earthquake loadings (safety-evaluation or functional-evaluation) and whether the bridge is classified as Important or Ordinary. As described in section 2.1.3, an Important Bridge is defined as a bridge that meets one or more of the following criteria

- it is essential for secondary life safety
- it has been formally identified as critical in a local emergency plan
- its loss would create a major economic impact

For Important Bridges, it is recommended that the seismic performance goals be assured through a two-level design approach (i.e., a direct design for each of the two earthquake levels). For Ordinary Bridges, Caltrans intends to use a single-level design approach using only the safety evaluation earthquake. Although the important bridge classification is currently applied somewhat subjectively, Caltrans is working to identify a statewide system of critical lifeline routes, which will help state officials classify bridges as Important or Ordinary.

3.3.3 Seismic Loading

For Ordinary Bridges, the ATC-32 recommendations are based on the deterministic design earthquake loadings currently used by Caltrans. Although probabilistic risk maps have been developed by others (U.S. Geological Survey & California Division of Mines and Geology), Caltrans does not have enough confidence in these maps to switch to them at the present time. However, at some locations in California, these maps indicate that ground motions with a 475-year return period can equal or exceed the Maximum Credible Earthquake (MCE) ground motions. Many ATC-32 Project Engineering Panel (PEP) members feel that Caltrans would be well advised to use probalistically based maps to complement their current approach.

The MCE is less than the 475-year earthquake in some cases because ground motions attributed to the MCE are derived using mean ground attenuation. When mean-plus-one-standard deviation attenuation is used, the motions are approximately 50 percent more severe than those resulting from mean attenuation. At sites where frequent large earthquakes can occur, there is a significant probability that ground motions in excess of the current Caltrans design ground motions will be experienced.

New design spectra for three earthquake magnitude ranges were developed as part of the ATC-32 project. This approach may require design for multiple spectra in some cases, since high ground accelerations at a given site can often be produced by different sources with different earthquake magnitudes.

Recent studies of strong-motion instrumentation results have yielded information that make it possible to refine the current Caltrans *ARS* curves. The current family of site-dependent design spectra developed as part of the ATC-32 project are based on standard sites in the NEHRP *Provision for New Buildings* (BSSC, 1994). These sites are characterized in part by the shear wave velocity of the soil, as shown in Table 3-5.

Site spectra for other site types must be developed on a site-specific basis.

As a result of recent earthquakes, the ATC-32 project considered near-fault effects such as high-velocity pulses. This phenomenon is associated with the mechanism of fault rupture and is typically strongest in the direction perpendicular to the fault. Although the ATC-32 elastic spectra were derived from a database of actual earthquake ground motion records, the spectra do not adequately address the effects of a velocity pulse.

Nonlinear studies were carried out that subjected several columns designed according to the ATC-32 provisions to several different real and synthetic earthquakes containing velocity pulses. These pulses significantly magnified the displacements due to structural yielding. The most extreme effects occurred

Table 3-5 ATC-32 Site Characteristics (ATC, 1996a)

Site Designation	*Site Description*	*Shear Wave Velocity Range*
B	Medium rock	2500 to 5000 fps
C	Dense/stiff soil	1200 to 2500 fps
D	Medium soil	600 to 1200 fps
E	Soft soil	< 600 fps

Table 3-6 ATC-32 Minimum Required Analysis

Bridge Category	*Configuration Type*	*Functional Evaluation*	*Safety Evaluation*
Ordinary	Type I	None Required	A
	Type II	None Required	B
Important	Type I	A	A
	Type II	B	B and C

A = Equivalent Static Analysis
B = Elastic Dynamic Analysis
C = Inelastic Static Analysis

with synthetic ground motions. To overcome this magnification, the extent of yielding needs to be minimized by increasing the column yield strength. The capacity of columns to accommodate increased displacements also needs to be enhanced. However, because of schedule constraints and a lack of consensus among project participants, no specific design recommendations were made.

The ATC-32 project also summarized the current state of practice for developing vertical design spectra and spectrum-compatible time histories for design.

3.3.4 Analysis

Although the ATC-32 recommendations retain a force design approach, some of the inherent shortcomings of this approach have been overcome. This was accomplished through the use of new response modification factors and modeling techniques that more accurately consider seismic displacement. The ATC-32 procedures also provide specific means for considering geometric and material nonlinearity directly in special cases.

As shown in Table 3-6, the ATC-32 project has developed recommendations for the minimum type of analysis that should be used under various circumstances. The type of analysis depends on whether or not the bridge is classified as Important and on the complexity of the structural configuration (Type I = simple and Type II = complex). These analysis types include equivalent static, elastic dynamic, and inelastic static analysis. Basic requirements for each type are also included.

Equivalent static analysis allows an equivalent static force to be applied to the structure. The magnitude of this force is determined from the value of the ARS design spectra at the structure's period of vibration. This force is applied at the vertical center of mass and distributed in the horizontal plane based on the distribution of mass in the structure or on the product of mass distribution and displacement.

Elastic dynamic analysis is required when configuration of the bridge or the distribution of stiffness and/or mass within the structure is complex enough to preclude reliably predicting the response with static analysis. In most cases, a multi-modal response-spectrum analysis using a lumped-mass stick model satisfies these requirements.

Inelastic static analysis is required only when the bridge is classified as Important and it has a complex configuration. The analysis, commonly referred to as a "push-over" analysis, is done in conjunction with elastic dynamic analysis and requires a preliminary determination of the strength and stiffness of critical members. In this analysis, loads are applied incrementally until the structure has achieved ultimate displacements. At each step, changes in the structure's characteristics due to geometric and material nonlinearity are considered. Results of this analysis are used to confirm that the structure is capable of accommodating the displacement demands determined from dynamic analysis.

Although inelastic dynamic analysis is not required for any structure type, the ATC-32 recommendations provide guidelines for conducting such an analysis. Because knowing member strength and stiffness values is a prerequisite of the analysis, this technique is used primarily for verifying a completed design. Both geometric and material nonlinearity should be considered. In general a lumped-mass stick model with five percent of critical damping is appropriate.

The ATC-32 document also provides a method for adjusting the displacement results obtained from elastic dynamic analysis to better reflect the actual maximum inelastic displacements that are likely to occur during an earthquake. The adjustment is given by the following formula:

$$R_d = \left(1 - \frac{1}{Z}\right)\frac{T^*}{T} + \frac{1}{Z} \geq 1 \qquad (3\text{-}11)$$

where

$T =$ natural period of the structure
$T^* =$ predominant period of ground motion

To account for the directional uncertainty of earthquake motion, ATC-32 has increased the 30 percent rule of the current BDS to 40 percent; that is, the forces from the primary direction are combined with 40 percent of the forces from the orthogonal direction and 40 percent from the vertical direction. The addition of the vertical load case is new.

The ATC-32 project developed revised response-modification factors, referred to as Z factors by Caltrans. One consideration in developing these factors was the relationship between elastic and inelastic response. This relationship varies as a function of the natural period of vibration of the structure and the predominant period of ground motion at the site. These are among the same factors used to modify seismically induced displacements obtained from an elastic analysis.

A second factor considered in developing Z factors, particularly for ductile columns, is the distribution of elastic and inelastic deformations within the structural component being designed. This distribution is a function of the geometric properties of the component and its framing into the remainder of the structure. Nonlinear dynamic analysis studies demonstrated that very little, if anything, was lost by using a constant Z factor as opposed to a more complicated factor based on column aspect ratios. Therefore, the ATC-32 recommendations include a constant Z factor for columns and other components, which is decreased linearly starting at a structural period of T^* to a minimum value of 1.0 at a period of zero.

Charts showing Z factors are included in Figure 3-3. The factors recommended in ATC-32 are typically lower than those defined in the current Caltrans BDS. These values assume that cracked section properties have been used to determine the elastic response of the bridge.

Trial designs and column studies performed using the 1993 ATC-32 draft specifications indicated that P-Δ effects could be significant. Nonlinear studies were also performed to determine the dynamic effect of P-Δ moments. It was found that a biased response could be prevented if yielding was limited by keeping the design base shear at a high enough level. The following drift limit was established to keep bridge columns from being significantly affected by P-Δ moments:

$$\frac{\delta_u}{L} = 0.25\frac{V_o}{P} \qquad (3\text{-}12)$$

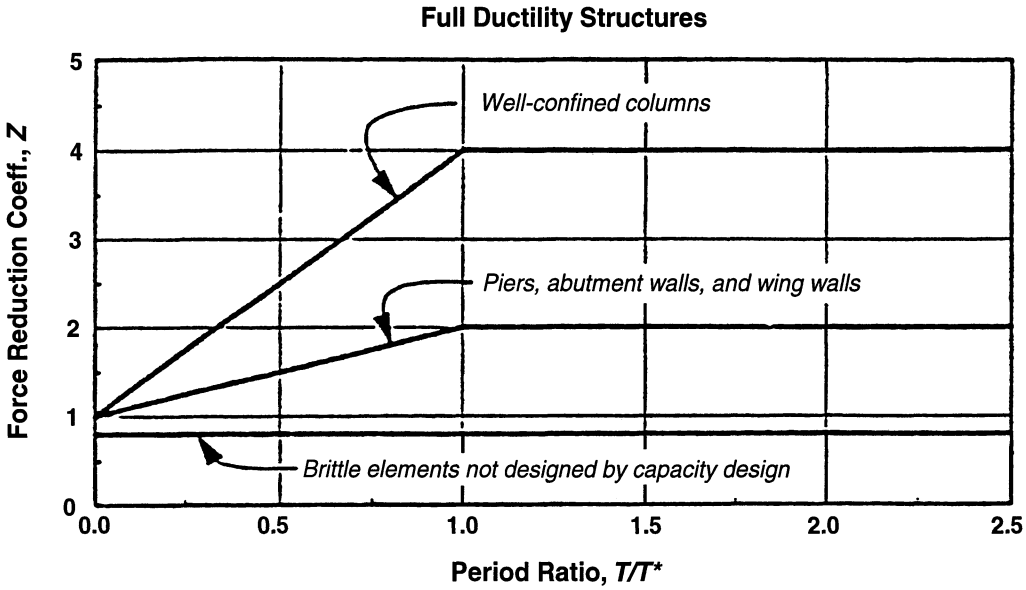


Figure 3-3 ATC-32 Force reduction divisor Z (full-ductility structures).

This equation can be rewritten in terms of the required base design shear V_o. Because a typical bridge column will have a drift ratio value of approximately 0.03, this implies a minimum base shear coefficient of approximately 0.12 to prevent unacceptable P-Δ effects.

The ATC-32 project also reviewed simplified design procedures for restrainer cables. Although the current Caltrans approach is not technically correct, no other simplified method seems to give any better results. Because of this, and the general feeling that restrainers are a back-up to the practice of providing adequate seat widths at expansion joints, no change in the current Caltrans method was recommended.

3.3.5 Design

Modifications to several aspects of reinforced concrete design have been proposed by the ATC-32 project. These include the design of ductile elements, the design of nonductile elements using a capacity design approach, and the detailing of reinforced concrete for seismic resistance.

Columns

The recommended ATC-32 design specifications place a lower limit of 0.01 on the longitudinal column reinforcing steel ratio and an upper limit of 0.04. In addition, the longitudinal steel requirements are based on expected material strengths rather than nominal strengths. Trial designs and column design studies using the ATC-32 recommendations showed that longitudinal column reinforcing steel is actually less than that required by the current Caltrans design specifications in most cases. This occurred despite lower Z factors. The reduction was primarily attributable to the use of cracked-section properties for analysis, which resulted in lower elastic force demands.

Evaluation of the displacement capacity/demand ratio for the columns designed with these factors indicates that their expected performance is actually improved in most cases, although the performance varied significantly from column to column. The improvement was primarily due to increased displacement capacities resulting from more stringent confinement requirements. The recommended requirements for the volumetric reinforcement ratio of spirally reinforced columns are given by the following equation:

$$\rho_s = 0.16 \frac{f'_{ce}}{f_{ye}} \left(0.5 + \frac{1.25P}{f'_{ce} A_g} \right) + 0.13(\rho_l - 0.01) \qquad (3.13)$$

where:

f'_{ce} = expected concrete strength
f_{ye} = expected yield strength of the reinforcement

An additional requirement, which is designed to prevent inelastic buckling of the reinforcing, requires a volumetric ratio for spirals that is linearly related to the number of longitudinal reinforcing bars, regardless of their size. Improved provisions for transverse reinforcement of tied columns and piers have also been proposed .

Revised criteria for column shear design are recommended, which are consistent with the format of current American Concrete Institute (ACI) proposals. Column shear strength is suggested to be the sum of a contribution from concrete V_c and a contribution from reinforcing steel V_s. The nominal concrete contribution to shear resistance in English units is given by:

$$V_c = 2\left(0.5 + \frac{N_u}{2000A_g}\right)\sqrt{f'_c}A_e \qquad (3.14)$$

within plastic hinge zones and by

$$V_c = 2\left(1.0 + \frac{N_u}{2000A_g}\right)\sqrt{f'_c}A_e \qquad (3.15)$$

outside of plastic hinges.

In both cases:
N_u = ultimate axial compressive force in the column
A_g = gross cross-sectional area
A_e = effective shear area, which is $0.8A_g$ for columns

Slightly modified versions of these formulas are recommended for columns subjected to axial tension.

The nominal shear contribution from reinforcing is given by:

$$V_s = \frac{A_v f_{yh} d}{s} \qquad (3.16)$$

for tied rectangular sections and by:

$$V_s = \frac{\pi}{2}\frac{A_h f_{yh} D'}{s} \qquad (3.17)$$

for spirally reinforced circular sections.

Where

A_v = total area of shear reinforcement parallel to the applied shear force
A_h = area of a single hoop
f_{yh} = yield stress of horizontal reinforcement
d = distance from extreme compression fiber to centroid of torsion reinforcement
D' = diameter of a circular hoop
s = spacing of horizontal reinforcement along the axis of the member

New recommendations for determining the anchorage length of column reinforcement in footings and caps have been developed. These new provisions, which are based on recent laboratory experiments, reduce current ACI anchorage length requirements within bent caps, thus making it practical to use large-diameter bars (#14 and #18). To achieve these reduced lengths, significant confinement reinforcement is required within anchorage zones.

Joints

New design procedures for calculating shear forces and moments within footing and superstructure joints are also recommended in the proposed ATC-32 design specifications. These procedures are based on recent laboratory test results and have been used in the design of replacement bridges following the Northridge earthquake. The proposal requires that joints be capable of resisting plastic column moments through a combination of concrete and reinforcing steel action. Reinforcing requirements are based on the magnitude of principal tensile stresses within the joint. When principal tensile stresses are below $3.5\sqrt{f_c'}$ steel reinforcement is required to carry 50 percent of these stresses. For tensile stresses above $3.5\sqrt{f_c'}$ specific reinforcement is required. Principal compressive stresses within the joint are limited to $0.25f_c'$.

In addition to reviewing and modifying provisions currently used by Caltrans for concrete design, ATC-32 has also added new provisions in subject areas related to ductile design. These provisions include design procedures for preventing flexural yielding of bent caps and superstructures near columns, and torsion within columns, knee joints, and bent caps.

Steel Components

Steel design guidelines have been developed for determining the seismic design forces for ductile steel bridge components. Detailing requirements for steel framing and various types of steel joints likely to be used in bridge work are spelled out. These requirements are directed toward moment-resisting beam-to-column connections, slip-critical bolted connections, concentrically braced steel frames, and stiffened as well as unstiffened box sections. These provisions are designed to prevent localized failures within these components and to assure that steel structures will behave satisfactorily when subjected to yielding during a strong earthquake. In addition, recommendations for the seismic design of steel cross-bracing and conventional bridge bearings have been developed.

Foundations

Foundation design guidelines include provisions for modeling and designing abutments, pile and spread footings, and drilled shafts. These recommendations tend to validate current Caltrans practice. They have been updated to include the latest results of research (e.g., abutment research at University of California, Davis). It should be pointed out, however, that many issues related to the effect of foundations on total system response are still not fully understood.

The foundation design guidelines also include site-specific issues such as liquefaction and lateral spreading, soft clay sites, unstable slopes, and fault rupture zones. Design guidelines for earth retaining structures have also been developed. These are discussed in more detail in Section 5.

3.3.6 Conclusion

The ATC-32 recommendations represent a significant step forward in bridge seismic design specifications. They are built upon the previous efforts of Caltrans and thus retain many of the features of the current Caltrans BDS. However, they do differ from the Caltrans approach in some fundamental ways.

It would be a mistake to assume that the ATC-32 recommendations fully address all issues. Even during the course of the project, ongoing research efforts and experience from actual earthquakes were advancing the knowledge about bridge seismic performance, requiring that the draft recommendations be updated. As Caltrans begins to implement these recommendations, it is expected that further modifications may be required.

3.4 Transportation Corridor Agencies

3.4.1 Introduction

The Transportation Corridor Agencies (TCA), which is made up of a number of local government agencies, was formed to facilitate the construction of a new toll road system within Orange County, California. Because uninterrupted income from tolls was essential to financing this project, it was decided early on that special seismic design criteria for toll road bridges was required to ensure that these roads remain operational following an earthquake. This summary is based on the report describing the development of the original criteria and the actual criteria used in the design of the first set of toll road bridges. In 1995, TCA had the criteria reviewed as a result of the ATC-32 developments.

3.4.2 Overview

To accomplish TCA's goals, a two-level design approach was developed. This approach is essentially a force design approach that uses seismic inputs from a Maximum Credible Earthquake (MCE) similar to the one used by Caltrans and a lower-level earthquake with a 72- to 150-year return period. Structures are designed to suffer no damage during the lower-level event. During the higher-level event, some abutment damage is allowed and column damage is controlled by placing relatively low allowable limits on ductility demand. Because Caltrans will eventually take over the maintenance of these highways, other design requirements must comply with those of Caltrans.

3.4.3 Seismic Loading

The upper-level event is defined as the 84th percentile Maximum Credible Spectra. Because these spectra are similar to Caltrans spectra for some of the sites with lower peak accelerations, Caltrans spectra have been used in practice for these sites. For some of the sites with higher peak accelerations, the newly defined spectra are higher. This is because the 84th percentile attenuation model was used instead of the 50th percentile used by Caltrans.

The lower-level seismic loading, defined by a spectrum that represents an event with a 72-year return period, has been used on most of the TCA bridges constructed to date. A 150-year return period spectra has been adopted as the lower-level design event for current and future projects.

3.4.4 Analysis

A multi-modal response spectrum computer analysis of a 3-D space frame model similar to the type used by Caltrans is generally used for analysis of both events. The complete quadratic combination (CQC) method of statistically combining modal responses is specified. Special modeling considerations are given below.

Lower-Level Event

- Abutment keys are assumed to remain intact.
- Transverse abutment displacements are to be compatible with force capacity.
- Because of large gaps, freedom of longitudinal movement at the abutments is generally assumed.
- Gross column section properties are used. This was changed in the 1995 review to the effective stiffness properties over the column height.

Upper-Level Event

- Abutment keys are assumed to have failed unless pile and wing wall capacity is sufficient.
- Longitudinal abutment stiffness values are to be displacement- and force-compatible, considering the effect of longitudinal gaps.
- Effective section properties are used over the full height of the column.

3.4.5 Design

Column design at the lower level event was based on a Caltrans *Z* factor of one, which implies little or no column damage. This was changed to a *Z* factor of 1.25 in the 1995 review. Column reinforcement is limited to four percent of the gross column area. Abutment shear keys are designed to carry the lower-level loads elastically, as are abutment piles and wing walls. The longitudinal gap at the abutments is typically made large enough to prevent pounding during the lower-level event. In 1995, a displacement demand/capacity check was added for the upper-level event. The demand/capacity ratio for this event was limited to 0.8.

Column ductility demands are verified in the upper-level event by requiring that column design be checked with a *Z* factor of three for single-column bents and four for multi-column bents. In 1995, a displacement demand/capacity check was added for the upper-level event. The demand/capacity ratio was limited to 0.8. The $P-\Delta$ effect is to be considered where appropriate. Abutment shear keys are to be designed as fuses unless the capacity of the piles and wing walls is sufficient to carry the seismic loads without damage. The movement of the abutment backwall into the approach soil is limited to 2-1/2 inches to prevent serious abutment damage. Superstructure damage due to column yielding is to be avoided. To accomplish this objective, an interim design memo proposed by Caltrans has been used. This memo requires minimum levels of reinforcement at or near the column/superstructure joint.

Otherwise, design is similar to Caltrans in that foundation damage is avoided, column shear and confinement requirements apply, and Caltrans abutment stiffness and capacity values may be used. In general, the use of internal expansion joint hinges is avoided.

3.4.6 Summary

The TCA requirements are the first U.S. highway bridge design specification to require a two-level design approach. Because these criteria apply to a relatively small jurisdiction, they are continuously under review, based on the experience of applying these criteria to actual bridge designs and also to incorporate new knowledge about bridge seismic performance. The sequence of changes that have occurred in the criteria, including the 1995 review recommendations, are given in Table 3-7. Experience has shown that very little additional construction cost is added by the more stringent TCA performance criteria.

Table 3-7 Evolution of TCA Design Criteria

								Design Criteria in Current Use		
		Caltrans 3/94 Practice	*TCA Special Criteria As Bid in 9/94*		*Caltrans 2/95 Practice*	*SEQAD 6/95 Recommendations (Similar to ATC-32, 5/95)*		*TCA Special Criteria As Revised 7 & 9/95*		*Caltrans Criteria for Review Purposes 9/95*
			Lower Level	**Upper Level**		**Lower Level**	**Upper Level**	**Lower Level**	**Upper Level**	
Objective		No Collapse	No Damage	Repairable	No Collapse	Minor Damage	No Collapse	No Damage	Repairable	No Collapse Max
Design Spectra		Max Credible	150 Year	Max Credible	Max Credible	150 yr.	Max Credible	150 yr.	Max Credible	Credible
Col. Stiff.		Effective EI	Gross EI	Effective EI	Effective EI	Effective EI	Effective EI	Effective EI	Effective EI	Effective EI
Abut. Restraint	Transverse	Optional	Full	Optional	Optional	Not Specified	Not Specified	Full	Optional	Optional
	Longitude	Optional	None	Optional	Optional	Not Specified	Not Specified	None	Optional	Optional
Analysis & Design Methods	Demand	Multi-mode Psuedo Elastic	Multi-mode Psuedo Elastic	Multi-mode Psuedo Elastic	Multi-mode Psuedo Elastic	Multi-mode Psuedo Elastic	Multi-mode Psuedo Elastic	Multi-mode Psuedo Elastic	Multi-mode Psuedo Elastic	Multi-mode Psuedo Elastic
	Capacity	Strength	Strength	Strength	Displacement	Strength	Strength or Displacement	Strength	Displace-ment	Strength or Displacement
Column Design Strength		ATC-32(93) Q Factors	Z=1	ATC-32(93) Q Factors for Important Bridges	Q=4 for initial column sizing	Z=1.5	Z per ATC-32 (95)	Z=1.25	Pushover Analysis	Z=5.0
Displacement Demand to Capacity Ratio					*D/C*=1.0, max,	*ec*=.004, *es*=.012	*ec*=*ecu*, *es*=.08, *D/C*=1.0, max		*ec*=*ecu*, *es*=.09, *D/C*=.80, max (.90 w/site ARS)	*ec*=*ecu*, *es*=.09 *D/C*=1.0 max
Displacement Capacity					μ=4, min.		Not Explicit		μ=4 min.	μ=4, min.
Vertical Acceleration		Not req.	Not req.	Required	Required.	Not Required	Required	Not req.	Superstr only	Superstr only
ARS for Flexure				2/3 Horiz.	1.2 Mcr		2/3 Horiz.		2/3 Horiz.	2/3 Horiz.
Girder/Cap Shear					1.5 D		1.5D		1.5D	1.5D
Flex Capac @supports	In Superstr	Memo 20-6	Gr VII Moment	Col *Mp*=1.3*Mn*	20-6 or Col *Mp*	Not specified	Col *Mp*=1.4*Mn*	Gr VII Moment	Col Mp=1.4Mn	Col *Mp*=1.3*Mn*
	Foundation	Col *Mp*=1.3*Mn*	Gr VII Moment	Col *Mp*=1.3*Mn*	Col *Mp*=1.3 *Mn*	Not specified	Col *Mp*=1.4*Mn*	Gr VII Moment	Col Mp=1.4Mn	Col *Mp*=1.3*Mn*
Joint Shear Design	@Cap Bm	Memo 20-6	N/A	Memo20-6	.20 Asc tot.	N/A	.16Asc-face	N/A	.16Asc-face	.20 Asc tot.
	@Footing	None	N/A	50% of cap	None	N/A	.16Asc-face	N/A	.16Asc-face	None
Material Strength	For *Mp* calc	Nominal	N/A	Nominal	Nominal	N/A	Expected	N/A	Expected	Nominal
	Otherwise	Nominal	Nominal	Nominal	Nominal	Expected	Expected	Nominal	Nominal	Nominal

SECTION 4
FOREIGN BRIDGE CODE REQUIREMENTS

This section summarizes three foreign seismic design requirements for bridges. The New Zealand requirements are presented in Section 4.1, the Japanese requirements in Section 4.2, and the European requirements in Section 4.3.

4.1 New Zealand

The Transit New Zealand (TNZ) Bridge Manual addresses seismic design in Section 5, "Earthquake Resistant Design of the Transit New Zealand Bridge Design Manual" (TNZ, 1994 and Chapman, 1995). The manual states that the primary objective is stated as "to ensure that the bridge can perform its function of maintaining communications after a seismic event". Design is performed using one level of ground motion intensity, an event that has a 10 percent probability of exceedance in 50 years. There is an assumption that the design procedure also minimizes damage in lesser events. Additionally, the specified detailing and the capacity design approach are intended to provide a reserve of ductility to prevent collapse in a greater event. The potential for large movements in bridges built on liquefiable soils or crossing active faults is to be recognized and the affected bridges adequately designed and detailed.

4.1.1 Structural Action

For design purposes, each bridge must be categorized according to its structural action under horizontal seismic loading. Categories are defined with reference to the relationship between the total applied horizontal loading and the resulting displacement of the center of mass of the whole supersructure. Figure 4-1 illustrates the force/displacement relationships, yield force, yield displacement (Δ_y), and displacement ductility factor (μ) for the various categories.

In cases where large ductility demands are placed on concrete members due to flexibility of foundations or bearings, special analyses must be made and steps taken to limit the likelihood of damage during less severe shaking.

*Ductile Structure***.** Under horizontal loading, a plastic mechanism develops. After yield, increasing horizontal displacement is accompanied by an approximately constant total resisting force. A ductile structure must be capable of sustaining a ductility factor of at least six through at least four cycles to maximum design displacement, with no more than 20 percent reduction in horizontal resistance. For the purpose of determining the design load, the design ductility value is restricted to six or less.

*Partially Ductile (Types I and II)***.** Under horizontal loading, a plastic mechanism forms in only part of the structure, so that after yield there is a significant upward slope in the force/displacement relationship. In a Type I structure, this upward slope continues until the design displacement value is reached. In a Type II structure, a complete mechanism will form after further displacement, but the load at which this happens may not be predictable if it is due to hinging in piles.

Structure of Limited Ductility Demand. This structure is subjected to limited ductility demand under the design earthquake. It may otherwise qualify as Ductile or Partially Ductile, but its proportions are such that its yield strength exceeds the design load, and consequently the ductility demand is less than six.

Structure of Limited Ductility Capacity. This structure may otherwise qualify as Ductile or Partially Ductile, but its proportions or detailing are such that its ductility capacity is less than six. The design load must be determined according to the relevant curve on the seismic hazard response spectrum.

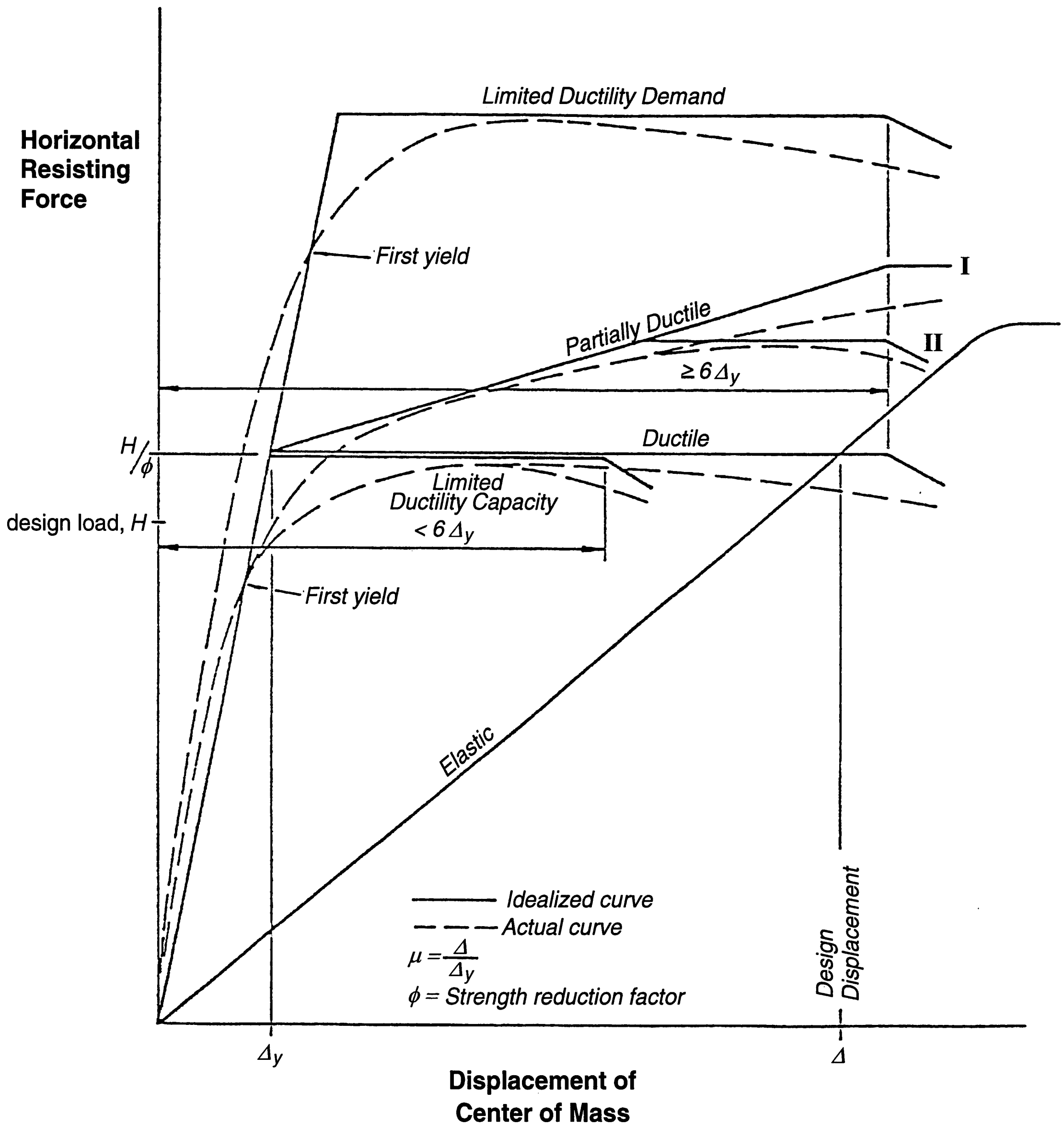


Figure 4-1 Idealized force/displacement relationships for various categories. TNZ Bridge Manual (1994)

Elastic Structure - This structure remains elastic up to or above the design load. It might have little or no reserve ductility after reaching its load capacity, which, while undesirable, may be unavoidable. In Elastic Structures, detailing must ensure that the risk of collapse is not greater than for a ductile structure, even though there may be a low standard of post-elastic behavior.

Structure Incorporating Mechanical Energy Dissipating Devices. This structure may be ductile, partially ductile, or of limited ductility demand, depending on the type of dissipator or mounting used.

Structure "Locked In" to the Ground. This is an elastic structure that relies on the integrity of the abutment approach material for longitudinal seismic resistance. It is assumed to move with ground acceleration.

Structure on Rocking Piers. This is a special case of ductile structure, in which spread footing foundations tend to lift at alternate edges. The deformation of the soil and the impact effects provide energy dissipation. Because of the lack of experimental or practical experience with such systems, a maximum displacement ductility value of $\mu = 3$ must be adopted, unless a larger value can be specifically justified.

Maximum values of the design ductility factor are given in Table 4-1 and illustrated in Figure 4-2:

4.1.2 Design Earthquake Loading

The design seismic ground motion is defined by acceleration response spectra for three types of site soil conditions. The spectra are modified by a zone factor to account for seismicity and attenuation which has an implicit relationship to the return period for the design event. Finally, the spectra are modified by a risk (importance) factor, which is based on the sponsor's jurisdiction and on traffic. The direction of the earthquake loading may be from any direction.

4.1.3 Analysis Methods

Three methods of analysis can be used. The choice of method is largely determined by the form of the structure.

Equivalent Static Analysis

Equivalent static analysis is acceptable when the bridge can be idealized as a single-degree-of-freedom oscillator. The masses of the superstructure and the pier caps and one-half of the mass of the piers are considered to act at the centroid of the superstructure and the transverse mass distribution is taken into account for transverse analysis. Accidental horizontal torsion is provided for by including a torsional

Table 4-1 TNZ Maximum Values for the Design Ductility Factor (Ratio)

Energy Dissipation System	*Factor*
Ductile or Partially Ductile Structure (Type I), in which plastic hinges form at design load intensity, above ground or normal water	6
Ductile or Partially Ductile Structure (Type I), in which plastic hinges form in accessible locations; e.g., less than 2 m below ground but not below normal water level	4
Ductile or Partially Ductile Structure (Type I), in which plastic hinges form in inaccessible locations; e.g., more than 2 m below ground or below normal water level, or at a level reasonably predictable	3
Partially Ductile Structure (Type II)	3
Structure with spread footings designed to rock	3
Hinging in raked (batter) piles in which earthquake load induces large axial forces	2
"Locked in" structure (T=0)	1
Elastic structure	1

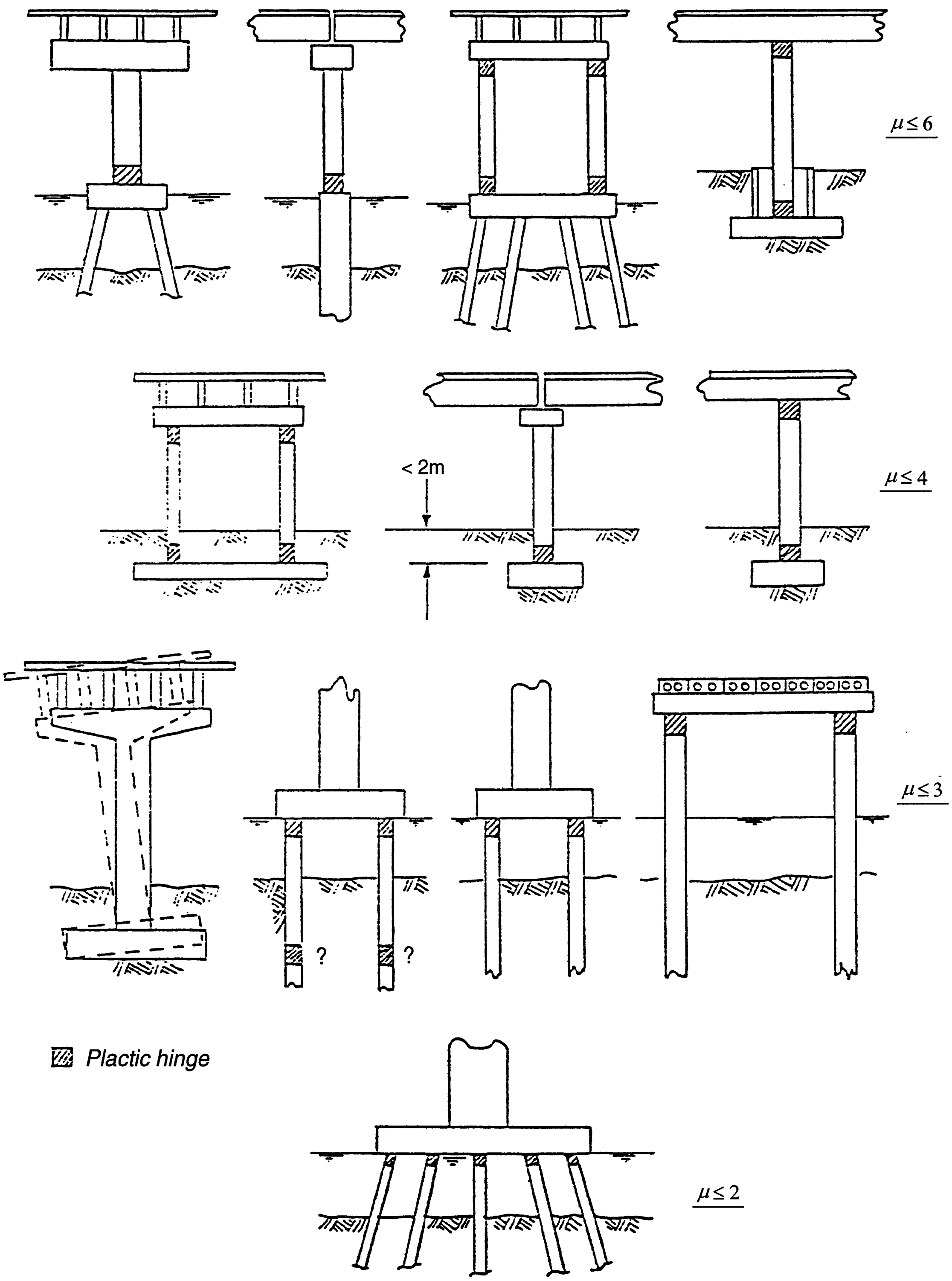


Figure 4-2 Examples of maximum values of μ allowed by Table 4-1 TNZ Bridge Manual (1994)

moment in the horizontal plane equal to the total horizontal force multiplied by $(1.00 + 0.025\ b)$ where b is the dimension of the structure, in meters, perpendicular to the direction of motion. The design seismic effect is not reduced by this torsional moment. Rotational inertia effects are included for wide superstructures on single-column piers.

Dynamic Analysis

Dynamic analysis is required whenever equivalent static analysis alone is not appropriate. Examples of such bridges are: a bridge in which the mass of the pier stems (including hydrodynamic mass) is greater than 20 percent of the superstructure mass tributary to any pier; structures that have discontinuities in mass or stiffness distribution or geometry for transverse analysis; bridges curved through an arc more than 45 degrees; bridges incorporating seismic energy dissipation devices other than conventional piers and abutments; suspension, cable-stayed, or arched bridges; and bridges with piers designed to rock.

Dynamic analysis is performed in two orthogonal directions. Concrete section properties are taken for the cracked section in "highly stressed" regions (piers and piles) and gross section in "lightly stressed" regions or for prestressed concrete members. The results of the dynamic analysis are compared to the equivalent static analysis based on the first mode. The dynamic analysis results may not be used to reduce the requirements determined from static analysis by more than 20 percent.

In dynamic analysis, the response spectrum (varies according to ductility ratio) is factored by the zone factor and the risk factor as described above for the seismic inputs. Response forces are thus used directly for design of the components. Displacements must be multiplied by the ductility factor to predict actual seismic response displacements. Modal responses are combined by any appropriate method, (e.g., square root of the sum of squares or complete quadratic combination). Sufficient modes should be taken to achieve participation of at least 90 percent of the total mass.

Inelastic Time-History Analysis

Inelastic time-history analysis may be used for any structure. At least two appropriate input motions must be used for each direction and the maximum response must be used for design purposes. Motions are defined as "appropriate" or not by comparing them with the response spectrum for the site. The ordinates of the input ground motion spectra must be no less than 90 percent of the design spectrum over the range of the first three periods of vibrations. The spectra must contain a duration of strong shaking of 15 seconds or five times the first period, whichever is greater.

Inelastic moment curvature and force idealizations must be appropriate to the material and the expected level of response. Structural damping of five percent of critical is assumed to account for both structural damping and for damping arising from radiation and inelastic behavior in the foundation. The overall ductility demand accepted for the structure may not exceed the limits for the type of structure (see Table 4-1).

4.1.4 Member Design Criteria

General principles of member design are given for each of the structure categories. In general, capacity design is used, wherein the plastic hinging mechanism is analyzed and members outside the plastic hinge regions are designed to fully develop the hinges before other kinds of failure can occur. Other documents provide formulae for determining the strength of the sections and detailing rules.

Ductile Structure. Strength of the hinge regions should not be less than the response forces calculated from the seismic analysis. Capacity design requirements are considered to be met if the overstrength hinges are developed by the ideal (i.e., nominal or unreduced) shear strength of the member between the hinges and by the ideal strength of the bounding members.

Partially Ductile Structures. Strength of the plastic hinges is specified in the same way as for Ductile Structures where practical. Rotational capacity of the lower (i.e., first to form) plastic hinges must be verified and detailed to ensure the necessary rotational capacity.

Structure Remaining Elastic at Design Earthquake Loading. If economically feasible, the structure should be designed so that damage from earthquakes larger than the design event should occur in accessible locations. The dependable (reduced) strength of members below the ground should at least match the ideal strength of the member above the ground. If hinge formation is anticipated during larger earthquakes, then capacity design principles should be followed.

Structure Anchored to a Friction Slab. Friction slabs (i.e., approach slabs anchored to the abutment and buried in the fill) may be used to control longitudinal seismic displacements if the integrity of the embankment can be relied upon during an earthquake (i.e., it is not susceptible to slope failure or liquefaction). The design forces on the abutment include response forces from the superstructure, soil pressure, and abutment inertial forces. The dependable restraint force furnished by the friction slab is assessed by considering sliding at the base of the friction slab or at the interface between the fill under the friction slab and the natural ground. The action of inertial forces on the slab and the overlying material are considered. The connection between the friction slab and the abutment is to be at least 1.2 times the dependable force provided by the friction slab.

"Locked In" Structures. Structures "locked in" to the ground must have integral or semi-integral abutments. Soil pressure values for the design of the abutments are those required to resist the seismic load, up to a maximum of the available passive pressure.

Structure with Pile/Cylinder (Drilled Shaft) Foundation. Pile stiffness values used for analysis must account for the probable range of values of the relevant soil parameters, possible residual scour, separation between the soil and the pile for a depth of two pile diameters, and liquefaction.

The design of the foundation must consider pile group action and the strength of the piles as governed by the soil in which the piles are embedded. The minimum dependable strength of the pile-footing connection must be 10 percent of the tensile strength of the pile. The region immediately below the pile cap (one pile diameter, but not less than 500 mm) must be provided with confinement reinforcing, as required for plastic hinges. Steel shells may be used for both confinement and shear reinforcing in the region of the connection. However the shells cannot be counted on to resist moment unless they are adequately anchored to the foundation.

Structure on Spread Footing Foundations. Maximum soil pressures must be less than the ideal bearing capacity of the soil, multiplied by an appropriate reduction factor. (Note: This is interpreted to mean the bearing capacity is determined by gross soil failure rather than by settlement considerations.) The minimum factor of safety against sliding is 1.2.

Structure on Rocking Foundation. Time-history dynamic analysis is required for these structures. The ideal moment strength of the base of the pier must be greater than 1.3 times the seismic response forces determined by analysis. Capacity design requirements are satisfied if the overstrength flexural capacity of the pier hinge is matched by the ideal shear strength of the pier and the dependable moment and shear capacity of the footing. The requirements for induced soil pressure at the initiation of rocking and the stability against sliding are the same as for spread footings. The response of the bridge to larger earthquakes should be assessed to ensure that structural integrity is maintained.

Structure with Energy Dissipating Devices. The energy dissipating devices must be treated similarly to plastic hinges in a Ductile Structure. In particular, they should be designed according to capacity design principles. Energy dissipating devices must have their performance substantiated by tests. The devices must be accessible for maintenance and repair or replacement if necessary. Design of lead rubber bearings is covered in a separate document.

4.1.5 Structural Integrity and Provisions for Relative Displacements

The TNZ requirements address the overall structural integrity of the bridges through provisions regarding adequate clearance and seat widths, as well as the need for restrainer cables.

Clearances

At locations where relative motion between structural elements is designed to occur, sufficient clearance must be provided to allow for seismic displacements without damage. Design displacement values of two components that may be out of phase are combined by the root-mean-square method. Appropriate long-term effects such as creep, shrinkage, and one-third of the maximum thermal displacement from the median temperature are added to the predicted seismic displacement for this calculation.

Clearance between the superstructure and the abutments for short skew bridges are increased by 25 percent to allow for torsional motions of the superstructure.

Clearance values may be lower at deck joints, provided that damage due to impact is limited to sacrificial devices. In any case, the range of movement to be accommodated must be not less than the sum of one-quarter of the predicted seismic response motion, long-term permanent shortening effects, and one-third of the maximum thermally induced motion from median temperature position. Damage to the joint seal is acceptable, but mechanical damage to expansion devices should be avoided.

Horizontal Linkage Systems (Restrainers)

All spans must be secured against the loss of vertical support under seismic movement by either a positive horizontal linkage (restrainer) or by providing adequate seat length to safely allow large relative displacements. Linkage is classified as either tight or loose depending on whether relative motion is intended or not. Tight linkage refers to the case in which relative movement is not intended. Rubber pads may be installed between elements to provide for relative rotation where applicable.

Longitudinal linkage is required between all simply-supported spans and their piers and between the two parts of the superstructure at an in-span hinge or expansion joint. Longitudinal linkage is not required at the abutment provided that the seat length requirements are met. Longitudinal linkage is not required where a fully continuous beam crosses a pier, provided that the relative displacement of the support does not cause local distress.

Transverse linkage is not required for any type of superstructure, provided that the superstructure is capable of supporting the exterior girder should it be displaced from its support (i.e., through load transfer by the diaphragm or cross frame to interior girder supports).

Loose linkage is intended as a security measure against loss of vertical support in an earthquake larger than the design event. Where loose linkage is appropriate, bumper elements must be placed between superstructure units to absorb impact shocks. The dependable strength of loose linkage elements is specified to be not less than 0.20 times the dead load of the contributing lesser length of the two adjacent superstructure elements. However, in the case of a short length (e.g., suspended span) between two longer lengths, the longer length is used to determine the required strength of the linkage.

Overlap Requirements (Seat Lengths)

Minimum span/support overlap (seat length) and bearing overlap are specified to prevent loss of vertical support under earthquake conditions in excess of the design event. Reduction is permitted at the designer's discretion if the requirements are not economically feasible. Minimum values are given in Table 4-2.

Table 4-2 TNZ Minimum Overlap Requirements

Linkage System	*Span/Support Overlap*	*Bearing Overlap*
None	$2.0\,E + 100$ mm (400 mm minimum)	$1.25\,E$
Loose	$1.5\,E^1 + 100$ mm (300 mm min)	$1.0\,E^1$
Tight	200 mm	N/A

Where

E = relative movement from median temperature position at construction under design seismic conditions = $EQ + SG + TP/3$

E^1 = equivalent relative movement at which loose linkage operates ($E^1 > E$)

EQ = predicted relative seismic displacement

SG = relative displacement due to permanent actions (e.g., dead load, creep shrinkage)

TP = relative displacement due to thermal movements from the median temperature position

Hold Down Devices

Hold down devices are required between the superstructure and all supports (and at in-span hinges) for which the net vertical reaction under design seismic conditions is less than 50 percent of the dead-load reaction. The hold down device must have sufficient strength to prevent separation of the superstructure elements at in-span hinges or uplift of these elements from the support. The minimum dependable strength of the device must be 20 percent of the dead-load reaction. In the case of a free or propped cantilever span, the dependable strength of the hold down device (if required) at the end of the cantilever must be at least 20 percent of the reaction that would exist if the cantilever span were simply supported.

Effects of Concurrent Orthogonal Movements

Linkages, bearing assemblies, and hold down devices must be designed to accommodate relative horizontal movements that occur simultaneously in the longitudinal and transverse directions.

4.2 Japan

4.2.1 Introduction

The Japanese bridge design code began specifically addressing seismic design following the 1923 Kanto earthquake that destroyed many bridges. Over the years, this code has been revised to address deficiencies observed in subsequent earthquakes. This summary is based on the 1990 code described in several papers written in English by Japanese bridge engineers (Iwasaki et al. 1990 and Kawashima, 1995).

4.2.2 Overview

Code requirements for Japanese bridges appear to be directed primarily at simple spans supported on bearings. Design is based on an allowable-stress approach. The use of simplified equivalent static analysis methods that consider the predominant period of the bridge is the norm, with dynamic analysis being the exception. The code includes special requirements for dealing with liquefaction, which has historically been a problem for Japan.

4.2.3 Seismic Loading

Design spectra used in Japan are prereduced to account for ductility. They are presented as a design seismic coefficient that is obtained by multiplying the standard coefficient by several "c" factors.

$$k_h = c_z c_g c_i c_t k_{ho} \tag{4-1}$$

where

k_h = design seismic coefficient
c_z = seismic zone factor
c_g = ground conditions factor
c_i = structural importance factor
c_t = structural response factor
k_{ho} = standard horizontal seismic coefficient (0.2)

Three seismic zones are identified in Japan. Zone A is located on the more seismically active east coast (c_z = 1.0), while zones B (c_z = 0.85) and C (c_z = 0.7) make up the rest of the island chain.

Three ground conditions are also defined. These are rock (c_g = 0.8), alluvium/diluvium (c_g = 1.0) and soft alluvium (c_g = 1.2).

Bridges are assigned one of two importance classes. First-class bridges (c_i =1.0) consist of all bridges on national roads and principal local roads and important bridges on other local roads. Second-class bridges (c_i = 0.8) are all other bridges.

The structure response factor c_t depends on period and ground condition, and it defines the shape of the design response spectrum. This shape may be seen in Figure 4-3, a plot of k_h versus period with c_z and c_i set to 1.0.

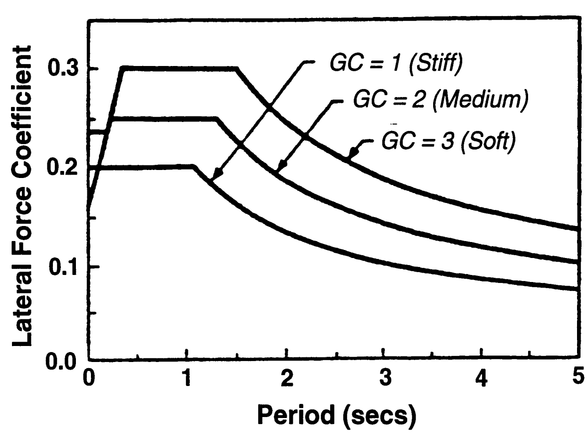
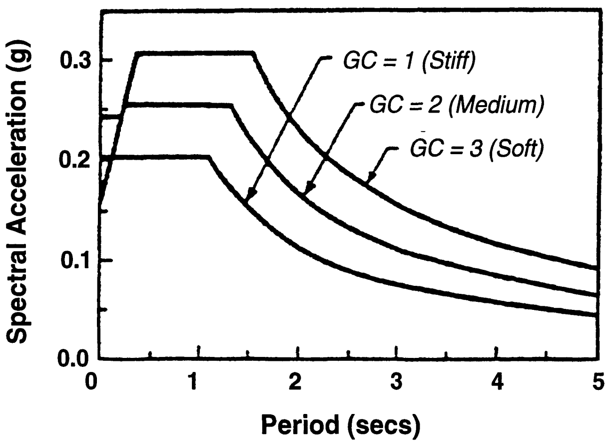


Figure 4-3 Japan Code Normalized design seismic coefficient.



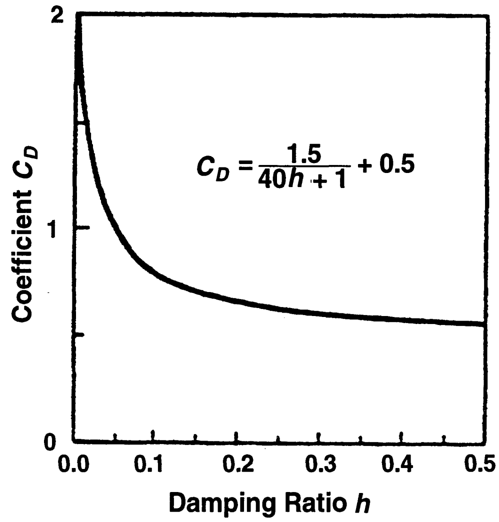


Figure 4-4 Japan Code Normalized design spectra.

Figure 4-5 Japan Code Ductility adjustment factor.

The design spectra used for dynamic response analysis shown in Figure 4-4, are similar to the spectra in Figure 4-3, except in the higher-period range where design coefficients are lower. These design spectra must still be adjusted for zone (c_z) and structure importance (c_i). In addition, an adjustment for structural damping other than five percent of critical is allowed. A plot of the damping modification factor c_d is shown in Figure 4-5.

Three ground acceleration time histories that have been modified so that their response characteristics are consistent with the design spectra are also included in the specifications. Vertical motion is generally not considered in bridge design.

4.2.4 Analysis

The Japanese use the equivalent static method as the normal analysis method for a typical bridge. Dynamic response analysis of any type is still considered a special case. Two types of equivalent static analyses are used.

1. *Reaction Force Method.* This method has sometimes been referred to as the "lollipop" method, because mass is assumed to be lumped at the top of a single column based on the mass that is tributary to that column. This single column is then analyzed as a unit. Since all columns carry transverse load, tributary mass in the transverse direction is generally found from the reaction force at the column. In the longitudinal direction, the effect of expansion joints and bearings must be taken into consideration when determining tributary mass. Longitudinal and transverse earthquake forces are considered separately.

2. *Static Frame Method.* The static frame method is a refinement of the reaction force method that includes a consideration of frame action in distributing seismic forces. It is similar to the single-mode spectral analysis method (Procedure 1) specified in ATC-6. In this method, the structure is divided into design structural units based on the structural configuration of the bridge. This approach is generally required for bridges that are constructed with continuous girders and those that have a significant difference in the

stiffness of supporting piers. It is recommended that forces obtained from the reaction force method not be reduced due to the redistribution of forces obtained from the static frame method.

Dynamic response analysis is generally used as a check on the equivalent static methods and generally cannot be used as a justification to reduce design forces. This method is reserved for bridges with complicated response characteristics or for new types of bridges where the response is not well understood. In general, dynamic response analysis consists of a response-spectrum analysis of a 3-D lumped mass frame, but time-history analysis is also used.

A check for ductility demand on reinforced concrete columns and piers is also provided in the code. This method uses an equivalent horizontal seismic coefficient that is calculated from the allowable displacement ductility factor and a standard horizontal seismic coefficient that is indicative of a magnitude 8 earthquake. This equivalent horizontal seismic coefficient must yield lateral forces that are less than the ultimate lateral force capacity of the column or pier. The values for ultimate lateral force capacities and allowable displacement ductility factors depend on the anticipated failure mode.

4.2.5 Design

Seismic design is basically done using the allowable stress method. Seismic forces such as seismically induced hydrodynamic pressure and earth pressure are added to the forces obtained from the equivalent static or dynamic analysis methods. Total seismic design forces are combined with other forces due to primary loads (e.g., dead load) and design is performed using 1.5 times the allowable stress.

The 1990 Japanese Code also includes structural details for seismic design. Although these were not described in detail in the papers reviewed, they include devices to prevent the dislodging of bearings, minimum bearing seat widths, and column/pier reinforcing details.

The Japanese are very active in pursuing innovative design strategies such as base isolation and hybrid active/passive control. A draft design guideline for base-isolated highway bridges has been developed.

4.2.6 Summary

The Japanese have an approach to seismic design that is fundamentally different from U.S. practice in many ways. Design forces are generally higher, resulting in higher yielding thresholds. Less reliance is placed on dynamic analysis and the effectiveness of high levels of ductile behavior as a design strategy. They seem to be more inclined to employ special devices such as dampers, base isolators and active/passive control in their designs.

4.3 Eurocode

4.3.1 Introduction

Eurocode 8 (CEN, 1994 and Pinto, 1995) contains the provisions for structures in seismic regions. Part 2 of this code covers bridge design. Other parts of the code that are referenced include seismic actions and general requirements, Part 1.1; material provisions, Part 1.3; and foundation and geotechnical provisions, Part 5. General design requirements are also found in other parts of the overall Eurocode.

Part 2 of Eurocode 8 primarily addresses the seismic design of bridges in which the horizontal actions are mainly resisted either at the abutments or through bending of the piers, that is, bridges composed of vertical or near-vertical pier systems supporting the traffic deck structure. The code states that it can also be applied to arch bridges, portal or tied bridges, and cable-stayed bridges. Suspension bridges are specifically excluded. It recommends that bridges with extreme skew and curvature be designed by conservative capacity design principles. Seismic isolation is included in a special chapter.

The general requirement of the seismic design is to maintain communications (with appropriate reliability) after the design seismic event.

4.3.2 Design Seismic Event

The design seismic event for bridges with an importance factor of 1.0 is stated as having a probability of exceedance ranging between 10 percent and 19 percent for bridges having a design life ranging from 50 to 100 years respectively. This is stated as being equivalent to a seismic event with a return period of approximately 475 years.

Bridges with "greater than average" importance (i.e., those bridges critical for maintaining communications after the design seismic event) use an importance factor of 1.3. Bridges with "less than average" importance (i.e., those not critical for maintaining post-event communications) use an importance factor of 0.70. An importance factor of 1.3 increases the return period of the design event and an importance factor of 0.7 decreases the return period.

Recommendations for a design seismic event appropriate for the construction period are given in Annex A, calculated from the duration of the construction period t_c and an acceptable probability of exceedance p.

When specifying the spectra of the design earthquake, six degrees of freedom are theoretically possible but these are normally reduced to three translations. Results from each of the three possible motions are combined according to the square root sum of the squares (SRSS) or the complete quadratic combination (CQC) method.

Each horizontal component is represented by a response spectrum, defined as follows:

$$R(T) = a_g S[1 + \frac{T}{T_B}(h\beta_o - 1)] \qquad 0 < T < T_B \qquad (4\text{-}3)$$

$$R(T) = a_g\, h\, S\, \beta_0 \qquad T_B < T < T_C \qquad (4\text{-}4)$$

$$R(T) = a_g h S \beta_0 \left(\frac{T_C}{T}\right)^{k_l} \qquad T_C < T < T_D \qquad (4\text{-}5)$$

$$R(T) = a_g h S \beta_0 \left(\frac{T_C}{T}\right)^{k_l} \left(\frac{T_D}{T}\right)^{k_2} \qquad T_D < T \qquad (4\text{-}6)$$

where:

$R(T)$ = acceleration response-spectrum ordinate
T = structure period
S = soil modification factor
h = damping correction factor
β_0 = spectral-acceleration amplification factor
k_1 = spectrum-shaping exponents

k_2 = spectrum-shaping exponents

Values of S, T_B, T_C, T_D, β_0, k_1 and k_2 are given in Part 1.1 of the Eurocode and repeated in Table 4-3 below. The value of the peak ground acceleration a_g is set by the National Authorities in each jurisdiction.

The period ranges above may be described as follows:

$0 < T < T_B$	increasing spectral amplification range
$T_B < T < T_C$	constant acceleration range
$T_C < T < T_D$	constant velocity range (except as modified by k_1)
$T_D < T$	constant displacement range (except as modified by k_2)

R(T) is expressed in the same units as a_g. There is no lower limit at the long-period end.

Table 4-3 Variation of Values with Different Soil Classes, Eurocode

Sub Soil Class	*S*	β_o	k_1	k_2	T_B *[sec]*	T_C *[sec]*	T_D *[sec]*
A	1.0	2.5	1.0	2.0	0.10	0.40	3.0
B	1.0	2.5	1.0	2.0	0.15	0.60	3.0
C	0.9	2.5	1.0	2.0	0.20	0.80	3.0

Site-Specific Effects

The values of the soil modification factor, period ranges, and spectrum-shaping parameters vary with the soil class as given in Table 4-3. These soil types are defined as follows.

Type A. Two separate conditions qualify as Type A: (1) less than 5 meters of overburden on rock-like material with a shear wave velocity not less than 800 m/s; (2) stiff deposit of sands, gravels, or over-consolidated clay several tens of meters thick with increasing shear modulus with depth and a shear modulus at 10 meters not less than 400 m/s.

Type B. These sites are deep deposits (several tens or hundreds of meters thick) of dense or medium-dense sand, gravel, or medium-stiff clays characterized by a shear wave velocity at 10 meters of not less than 200 m/s, increasing to at least 350 m/s at a depth of 50 meters.

Type C. These sites are loose, cohesionless deposits or soft to medium-stiff cohesive soils characterized by shear wave velocities below 200 m/s in the top 20 meters.

When the subsoil contains an alluvial layer 5 to 20 meters thick underlain by Type A material, the spectrum for Type B may be used, with the spectral ordinates multiplied by 1.40.

Several types of sites require special investigation: (1) sites underlain by Type C deposits, (2) sites with a soft clay layer at least 10 meters thick, and (3) sites with silts having a high plastic index ($PI > 40$) and a high water content. Such deposits have been shown to have low internal damping and an abnormally extended range of linear behavior.

An inelastic response spectrum is used to represent the motion for ductile structures and structures of limited ductility by dividing the spectral amplification factor β_0 by a behavior modification factor q (see section 4.3.5). In this case, the spectrum-shaping exponents k_1 and k_2 are taken as 2/3 and 5/3, respectively.

When the site response varies due to different soil conditions, the site-dependent response spectrum may be either an envelope of the various applicable spectra or a weighted average, where the weighting factor is the fixed-base stiffness of the pier.

The code states that even for a long bridge (< 600 m), spatial variability may be disregarded unless there are significant discontinuities in the soil type or topography along the length of the bridge. Annex D presents a method of modeling spatial variability.

The vertical component of motion is specified to be 70 percent of the horizontal components. Vertical motions are generally required to be investigated in regions of high seismicity and in bridges where the piers may be subjected to high bending moments from nonequilibrated permanent actions of the deck (unbalanced spans), prestressed concrete decks where unloading may cause unfavorable response, or for bearings and links.

4.3.3 Basic Requirements

Two design limit states must be satisfied: an ultimate limit state and a serviceability limit state.

At the ultimate limit state, the bridge should retain its structural integrity and have adequate residual resistance. Damage is permitted as long as the bridge is able to carry at least emergency traffic. The structure must be designed for damage to occur in areas that can be easily inspected and repaired.

Flexural hinging in the piers is anticipated in regions of high seismicity. In all cases, the bridge deck should be protected from the formation of plastic hinges and loss of vertical support.

The serviceability limit refers to the expected performance of the bridge following seismic events that have a high probability of occurring within the design life of the bridge. The serviceability limit criteria are such that the bridge should only sustain minor damage and be able to carry full traffic without the need for immediate repair.

Compliance with these two limit states is demonstrated by strength verification, capacity design, provisions for ductility, and control of displacements at the ultimate limit state. Compliance with the serviceability limit state is then assumed to be satisfied.

Strength Verification

All sections must have sufficient capacity to resist either the design seismic event or the capacity design effects (for ductile structures). Strength requirements are specified in Part 5.5 of the overall Eurocode.

Capacity Design

In regions of high seismicity (not defined) it is usually preferable to design a bridge for ductile behavior; i.e., to provide means of dissipating seismic input energy. This is accomplished either by specific energy-dissipating devices (isolators or dampers) or by designing for the formation of flexural plastic hinges. Brittle failure modes are to be avoided. Capacity design includes designing for the upper fractile of possible overstrength in the plastic hinges of the intended plastic mechanism.

Provisions for Ductility

Ductility is defined as the ratio of the displacement at the center of mass at the ultimate limit state to the displacement at yield, with reference to a single-degree-of-freedom (SDOF) structure. The ultimate limit state is defined as the point on the force-displacement curve where the descending branch of the curve is not less than 20 percent below the nominal value. The structure should be able to sustain at least five cycles of this level of displacement.

Compliance with the detailing rules given in Section 6 of this report are considered adequate to meet this requirement. Specific ductility checks may be required in special cases based on required curvature ductility and the length of the plastic hinges. Specific procedures are given in Annex B. Ductility demands must be specifically assessed when nonlinear dynamic analysis is used.

Control of Displacements

The secant stiffness at yield is specified as the appropriate stiffness value to use for linear elastic analysis of ductile components. Prestressed concrete elements are assumed to have the stiffness of the uncracked section.

Simplified Criteria

In regions of low seismicity ($a_g < 0.10$ g), simplified criteria may be based on elastic response. The ductility requirements are waived for bridges in such areas.

4.3.4 Structural Action

The Eurocode contains a section giving additional general principles to be observed in seismic design. The main emphasis is on determining the type of behavior desired in response to the design seismic event.

The three types of behavior recognized in the design process are: essentially elastic, partially ductile, and ductile. The forces and displacements determined by linear elastic analysis are modified by different q values (see Section 4.3.5), depending on the type of behavior: for essentially elastic structures, $q = 1.0$; for partially ductile structures, $q = 1.5$; and for ductile structures, $q < 3.5$.

Note that the response forces are divided by the q factor and the response displacements are multiplied by this factor.

Ductility is preferably confined to flexural plastic hinges in the piers. It is prohibited where the axial load ratio ($N/f'_c A_g$) is greater than 0.6. Global force-displacement curves for ductile bridges should show a significant yield plateau, be reversible, and be stable over at least five cycles of deformation.

Supporting elements connected to the deck through sliding bearings or flexible elastomeric bearings should remain elastic.

4.3.5 Behavior Modification Factors for Linear Analysis

Behavior modification factors are specified for structures designed to respond with limited or full ductility. The behavior factor q is analogous to the response modification factor R of the AASHTO code, although q generally has much lower values. The q factor is defined as a correction factor for post-elastic behavior that is to be applied to the results of a linear elastic analysis. The Eurocode explicitly states that these factors can only be applied to structures capable of forming energy-dissipating flexural plastic hinges.

Maximum values of the behavior modification factor q are a function of structural element (similar to AASHTO R factor). These maximum values are given in Table 4-4.

Table 4-4 Maximum Values of the Eurocode Behavior Modification Factor q

	Anticipated Seismic Behavior (Ductility)	
Ductile Element	*Limited*	*Full*
Reinforced concrete piers		
Vertical piers in bending	1.5	3.5
Inclined piers in bending	1.2	2.0
Squat piers*	1.0	1.0
Steel piers		
Vertical piers in bending	1.5	3.0
Inclined piers in bending	1.2	2.0
Piers with traditional bracing	1.5	2.5
Piers with eccentric bracing	NA	3.5
Abutments	1.0	1.0
Arches	1.2	2.0

*Squat piers are defined as piers where the shear span (M/V_d) is less than 2.0.

If the location of a potential plastic hinge is not readily accessible for inspection and repair, the behavior modification factor q should be divided by 1.4, except that the resulting q must not be less than 1.0. Plastic hinges are permitted in piles that have been designed and detailed for plastic behavior where the hinges are not accessible. The q factor in this case is 2.5 for vertical piles and 1.5 for batter piles.

The q factor for motion in the vertical direction is 1.0.

When the seismic action is resisted entirely by elastomeric bearings, the behavior factor is 1.0 and the bridge should be designed as an isolated structure.

The q factors given above for reinforced concrete members are to be adjusted (downward) if the axial load divided by the product of the characteristic strength of the concrete and the gross-sectional area is between 0.3 and 0.6. If the axial load ratio is less than 0.3, no adjustment is required. If the axial load ratio is 0.6, the behavior modification factor is 1.0 (elastic response is required). Linear interpolation is used for intermediate ratios.

The behavior factor for "special" bridges requires special justification. These special bridges include cable-stayed bridges, arch bridges, bridges with inclined struts, bridges with extreme geometry (heavy skew or sharp curvature), and bridges for which the piers have markedly different yield resistances.

4.3.6 Analysis Methods

General Modeling Considerations

The mass to be included in the analysis includes all the structural mass and a portion of the live load distributed uniformly over the deck area. The percentage of live load to be used in the model depends on the anticipated traffic levels.

- Intense traffic, urban bridges 20%
- Normal traffic, footbridges 0%
- Railway bridges 30%

Hydrodynamic mass should be added to piers submerged in water whenever it is significant. In the absence of a rigorous analysis, this mass may be estimated as the mass of a cylinder of water with a diameter equal to the width of the pier, perpendicular to the direction of motion and a height equal to the immersed height of the pier.

Uncracked section properties are recommended to give greater earthquake forces.

Torsional effects in bridges with heavy skew (< 70 degrees) or a low ratio of span-to-width ($L/B < 2$) are incorporated by assuming that the seismic response inertial force is applied at an eccentricity equal to eight percent of the length or width of the bridge, depending on the direction of motion. This eccentricity is stated as being three percent accidental eccentricity and five percent for the interaction of translational and rotational modes. In a full 3-D analysis, the eccentricity is to be applied by displacing the center of mass by the three percent accidental eccentricity in the most unfavorable direction and sense. Torsional resistance should not be dependent upon a single pier. The bearings of single-span bridges should be designed to resist torsional effects.

Ductile and partially ductile structures may be designed using an inelastic response spectrum where the ordinates of the site-dependent elastic response spectrum $R(T)$ are divided by the behavior modification factor q. For $q > 1.0$, the effects of any viscous damping correction are assumed to be included in q.

Response-Spectrum Analysis

Response-spectrum analysis is recommended as the primary analysis method using a full dynamic, discrete linearly elastic model.

The overall response is determined by combining the response of the individual modes using either the SRSS or the CQC method. Sufficient modes should be combined to include participation of 90 percent of the total mass.

Directional uncertainty is accounted for by combining the response from the three orthogonal directions according to the most adverse of the following combinations:

- $1.00\ E_x + 0.30\ E_y + 0.30\ E_z$
- $0.30\ E_x + 1.00\ E_y + 0.30\ E_z$

- $0.30\, E_x + 0.30\, E_y + 1.00\, E_z$

Note that the vertical motion is included in these combinations.

Fundamental Mode Method

Eurocode describes three fundamental mode methods and specifies how they can be used. The three methods are the rigid deck model, the flexible deck model, and the individual pier model. They are applicable to bridges for which the fundamental mode is the principal contributor to seismic response. The following are the specific situations in which one of the fundamental mode methods can be applied.

- In the longitudinal direction of approximately straight bridges with continuous decks. The seismic forces are carried by piers whose total mass is not greater than one-fifth the mass of the deck.
- In the transverse direction when the structural system is approximately symmetrical about the center of the deck, that is, when the eccentricity between the center of mass and the center of rigidity is not greater than five percent of the length of the deck.
- In the case of piers carrying simply-supported spans, when no significant interaction between the piers is expected and the total effective mass of each pier is not greater than one-fifth the mass of the tributary superstructure.

Of the three fundamental mode methods, the rigid deck model is always applicable in the longitudinal direction. It is also applicable in the transverse direction when the span-to-width ratio L/B is less than 4.0 or when the ratio of the maximum difference in displacement and the average displacement of all piers (using the seismic action or an arbitrary transverse load of similar distribution) is not greater than 1/5.

The seismic response forces are determined by applying an equivalent horizontal force to the deck. This force is given by the following expression:

$$F = \gamma_I\, M\, R(T) \tag{4-7}$$

where:

γ_I = importance factor
M = total effective mass
$R(T)$ = response spectrum ordinate corresponding to the period T

In the transverse direction, the force F is distributed similarly to the mass.

The flexible deck model uses a Rayleigh analysis to determine the fundamental period and participating mass. The seismic response forces are determined by loading the deck at each nodal point with a force:

$$F_i = \left(\frac{4\pi^2 \gamma_I}{T^2}\right)\left(\frac{R(T)}{\mathrm{g}}\right) d_i m_i \tag{4-8}$$

where:

γ_I = importance factor
$R(T)$ = response spectrum ordinate corresponding to the period (T)
g = acceleration of gravity
d_i = ordinate of the assumed mode shape at the ith nodal point
m_i = mass at the ith nodal point.

Nonlinear Time-History Analysis

The earthquake effects are defined as the sample average (over the suite of time histories employed) of the extreme response. If fewer than 10 input motions are used, the results are increased by the following factor:

$$S_E = (1 + 0.352N^{-0.5})\, S_A \qquad (4\text{-}9)$$

where:
S_E = design seismic action
S_A = average extreme response
N = number of time-history inputs used

Either real or artificially generated accelerograms may be used as input.

The validity of the time-history input is demonstrated by computing the average response spectra $R(T)$ using a geometric series of five-percent damped SDOF oscillators. One hundred oscillators are used with a period ratio of 1.064786 and periods ranging between 0.04 and 20 seconds. The input ensemble is acceptable if for at least half of the periods, the average of the ensemble is greater than or equal to the site-dependent response spectrum. The average of the response for the 13 periods nearest to the fundamental period of the structure to the ordinate of the site-dependent response spectrum must be greater than one.

Real accelerograms from sites with similar seismo-tectonic characteristics may be scaled by a factor between 0.5 and 2.0.

Nonlinear time-history analysis is used in conjunction with a linear response-spectrum analysis. The seismic effects used for design are the lesser of the two, but not less than 80 percent of the linear response-spectrum analysis results.

4.3.7 Strength Verification

Material safety (capacity-reduction) factors are used to ensure reliable performance. The nominal or characteristic strength of the material is to be divided by the appropriate factor.

Reinforced concrete	*Factor*
• Concrete	$\gamma_C = 1.5$
• Reinforcing steel	$\gamma_S = 1.15$
Structural steel	
• Axial or flexural yield	$\gamma_{M0} = 1.1$
• Plastic resistance of gross section	$\gamma_{M1} = 1.1$
• Resistance of net section at bolt holes	$\gamma_{M2} = 1.25$
• Bolts, rivets, pins, welds, slip	$\gamma_M = 1.25$

Capacity design effects are determined by applying an overstrength factor to the flexural plastic hinges of the energy-dissipating mechanism (collapse mechanism) of ductile structures. The general value of the overstrength factor is 1.40. The exception is reinforced concrete sections with special confinement reinforcing and an axial load ratio greater than 0.10, where the overstrength factor γ_0 is increased to:

$$\gamma_0 = 1.40\ [1 + 2(\eta_K - 0.1)^2] \tag{4-10}$$

where η_K = ratio of the axial load to the product of the gross area and the characteristic concrete strength

When sliding bearings participate in the plastic mechanism, the transmitted lateral force must be calculated assuming a 30-percent increase in the coefficient of friction to account for aging effects. A similar increase in the shear stiffness of elastomeric bearings is specified.

Seismic effects are combined with other load effects for the purpose of strength verification.

$$S_d = G_k + P_f + E + \psi_{21} Q_{1k} \tag{4-11}$$

where
S_d = design force effect
G_k = effect of permanent loads at their characteristic value
P_f = final prestress effects
E = most unfavorable combination of seismic effects
Q_{1k} = characteristic value of the traffic load effect
ψ_{21} = fraction of the live load included in the mass for seismic effects

In general, seismic actions are not combined with other deformation-induced effects such as temperature, creep, shrinkage, or settlement of supports. This provision does not apply to elastomeric bearings, where the forces generated are the result of all displacements.

The value of ψ_{21} is taken as 0.0 for the case of wind and snow loads.

Strength Verification of Concrete Sections

The flexural strength of concrete sections M_{rd} in the plastic hinge regions is to be greater than M_{sd}, the design moment under the seismic load combination including second-order effects. The flexural strength of concrete sections outside the plastic hinge zones R_d is to be greater than S_C the capacity design effects including the additional axial load due to capacity design effects in the plastic mechanism.

The designer must verify that the shear strength of sections outside the plastic hinge zones is larger than the capacity design effects. Comparison is made on the basis of diagonal web strength and the shear strength of the concrete and the reinforcing steel.

For diagonal web compression strength,

$$V_C < 0.5(n\, f_{cK}\, b_w)\,(0.9d) \tag{4-12}$$

where
V_C = capacity design shear,
n = modular ratio
f_{cK} = characteristic (cube) strength of the concrete
b_w = web width
d = section depth

For shear reinforcement,

$$V_C < [b_w\, d\, (\tau_{Rd}\, k(1.2 + 40\rho_l) + 0.15\sigma_{cp})] + 0.9d\, (\frac{A_{sw}f_{yw}}{s}) \tag{4-13}$$

where

τ_{Rd}	$= 0.035\ f_{cK}^{2/3}$
k	$= 1.6 - d$, but not less than 1
ρ_l	$= A_{sl} / b_w\, d$ the longitudinal reinforcement ratio (not < 0.02)
σ_{cp}	$= N_C / A_C$, the nominal axial stress under capacity design effects
$A_{sw} f_{yw}$	= product of area and yield stress of transverse reinforcing steel
s	= transverse reinforcing steel spacing
A_{sl}	= area of longitudinal reinforcing steel

The designer must also verify that V_{rde}, the shear strength of concrete sections within the plastic hinge zones, is greater than the capacity design effects using the following equation:

$$V_{rde} = 0.275\ v f_{cK}\, b_w\, d \quad \textit{where } v = 0.7 - f_{cK}/200 \quad \text{(not} < 0.5)$$

V_{rde} is equal to the shear strength after degradation. Note that this is approximately 60 percent of the strength outside the plastic hinge zones. The concrete contribution V_{cde} to the combined shear resistance is also reduced.

$$V_{cde} = 0 \text{ for axial load ratio } \eta_K < 0.10$$

$$V_{cde} = 2.5\ \tau_{Rd}\ b_w\, d \text{ for } \eta_K > 0.10$$

Sliding shear (shear friction with $\mu = 1.0$) is also checked using the following equation.

$$V_C < A_v f_{yd} + \min N_C \tag{4-14}$$

where

A_v	= area
f_{yd}	= yield stress of the total longitudinal reinforcing
N_C	= axial load due to capacity design effects.

This verification does not apply to squat piers.

4.3.8 Foundations

Plastic hinges are usually not intended to form in foundations. The foundations of partially ductile structures are designed for elastic seismic response forces. Foundations of ductile structures are designed by capacity design principles.

4.3.9 Specific Detailing Rules

Specific detailing rules apply to structures designed to undergo ductile behavior in response to seismic action. The rules aim at ensuring a minimum level of curvature/rotation ductility in the plastic hinges. Since no plastic hinges are assumed to form in the superstructure, the special detailing rules do not apply to this portion of the bridge.

Confinement of the plastic hinge region is required whenever the axial load ratio exceeds 0.08. The required quantity of confinement reinforcing is specified by:

$$w_{wd} = \rho_w \frac{f_{yd}}{f_{cK}} \tag{4-15}$$

where ρ_w = volumetric ratio of the confinement reinforcing to the area of the confined core

For rectangular sections

$$\rho_w = \frac{A_{sw}}{sb} \tag{4-16}$$

where
s = spacing of the transverse reinforcing
b = dimension perpendicular to the direction of confinement, measured to the outside of the perimeter hoop
A_{sw} = total area of hoops in the confining direction

For spiral reinforcement,

$$\rho_w = \frac{4A_{sp}}{D_{sp}s} \tag{4-17}$$

where
s = pitch of the spiral
D_{sp} = diameter of the spiral
A_{sp} = area of the spiral bar

The maximum spacing of the hoops or pitch of the spiral is limited to no more than six times the longitudinal bar diameter nor one-fifth of the least dimension of the confined core. The required amount of confinement reinforcing is determined by the following expressions:

$$w_{wd} > 1.30\,(0.15 + 0.01\,\mu_c)(A_c/A_{cc})(\eta_k - 0.08) \tag{4-18}$$

but not less than 0.08 for rectangular hoops

or

$$w_{wd} > 1.90\,(0.15 + 0.01\,\mu_c)(A_c/A_{cc})(\eta_k - 0.08) \tag{4-19}$$

but not less than 0.12 for spirals

where
A_c = gross area
A_{cc} = area of the confined core
μ_c = required curvature ductility (not < 15)

Interlocking spirals are recommended for approximately rectangular sections. The distance between the centers of the spirals should not exceed 60 percent of the diameter D_{sp}.

The length of the plastic hinge zone is estimated as the greater of (1) the depth of the section perpendicular to the axis of the hinge, or (2) the distance from the point of maximum moment to the point where the moment is reduced by 20 percent. If the axial load ratio η_k is between 0.30 and 0.60, the above length should be increased by 50 percent. This estimate of the plastic hinge length L_h is to be used to determine the required length over which to place the confining reinforcing. It is not sufficient for estimating the extent of rotation due to plastic hinging. The amount of the confinement reinforcing provided in the length L_h beyond the plastic hinge zone should not be less than 50 percent of that used in the plastic hinge zone. Beyond this length, the confining reinforcement is governed by other considerations (e.g., capacity design shear or longitudinal bar buckling).

In order to ensure that the longitudinal bars in the plastic hinge zone may undergo several cycles of inelastic behavior without outward buckling, the confinement reinforcing must not be spaced more than six times the diameter of the longitudinal bars. Along straight sections of rectangular sections, the perimeter hoops must be restrained by ties anchored in the confined core. These hoops must be spaced not greater than 20 centimeters apart. The minimum amount of this transverse reinforcing is:

$$\frac{A_t}{s} = \frac{f_{ys}\sum A_s}{1.6 f_{yt}} \tag{4-20}$$

where

A_t = area of one tie

$\sum A_s$ = area of the longitudinal bars restrained by the tie

s = spacing of the ties

f_{ys} = yield strength of the longitudinal bars

f_{yt} = yield strength of the ties

Confinement reinforcing hooks must be 135 degrees and extend an additional eight diameters into the confined core. Similar anchorage or full-strength welding is required at splices in spirals. Lap splices are not permitted in plastic hinge zones. Confinement reinforcing is not required in piers with flanged sections (box or I-shape) if a curvature ductility μ_c of 15 or greater can be attained under seismic load conditions, with a maximum concrete compressive strain ε_{cu} not exceeding 0.35%.

In the case of deep compressive zones, the confining reinforcing may be limited to the depth at which the concrete compressive strain exceeds 0.50 ε_{cu}.

Structures of Limited Ductility

In partially ductile structures, critical sections that may become plastic hinges must be provided with 50 percent of the confining reinforcing specified above if the structure is located in a region of high seismicity ($a_g > 0.10$g) and a q factor of 1.5 or less was used in the design. These sections must also attain a curvature ductility of 7.5. A critical section is defined as one in which the ratio of the maximum design seismic moment to the minimum flexural strength exceeds 1.30.

Foundations

Direct foundation structures (e.g., spread footings, rafts, box-type caissons, and piers) are not allowed to enter the inelastic range under design seismic actions and are therefore exempt from the ductility detailing requirements.

Piles in which inelastic action cannot be avoided must be detailed to ensure ductile behavior. Plastic hinge zones in piles can potentially develop near the pile heads at the junction with the foundation mat when the upper layer of soils has low strength and at the interface between soil layers of markedly different stiffness or strength. Near the pile head, confining reinforcing as specified for ductile structures should be provided over a length equal to five pile diameters. At the interface between soil layers, the confining reinforcing above should be provided over a length equal to two pile diameters on either side of the interface, unless a more rigorous analysis shows that hinging is not likely.

4.3.10 Structural Integrity and Relative Displacement

General

Displacement values calculated from a linear elastic analysis, either static or dynamic, are modified by the q factor (behavior factor).

Connections between supporting and supported elements are designed to ensure structural integrity, particularly maintenance of support under extreme displacements. Bearing, link, and hold down devices are to be designed using capacity design principles. Positive linkage (i.e., restrainers) may be used in cases in which it is not possible to provide the necessary seat lengths. Restrainers can also be used as a second line of defense.

Clearance between structural elements must be sufficient to accommodate the total seismic displacement d_{td}. The total seismic design displacement is defined as:

$$d_{td} = d_{sd} + d_G \pm \psi_2 d_T \tag{4-21}$$

where
d_{sd} = design seismic displacement
d_G = displacement due to permanent actions
d_T = displacement due to thermal effects
ψ_2 = factor normally taken as 0.5

Elastomeric bumpers between superstructure elements are recommended for structures of limited ductility to reduce impact forces. Clearance between superstructure elements at movement joints should accommodate frequently occurring seismic events. The displacement to be accomodated at these joints is as given above, using 40 percent of the design seismic displacement and 50 percent of the thermal displacement.

Bearings and Links

The design seismic action may be transmitted through either the bearings or through seismic links (restrainers) provided with proper slack so that they become effective only in strong seismic events. Seismic links (restrainers) may be shear keys, buffers, and/or linkage bolts or cables. They are required in the following cases:

- with flexible (elastomeric) bearings
- with fixed bearings not designed for capacity effects
- between the deck and the abutment and at the piers when the minimum seat lengths are not provided
- between adjacent parts of the superstructure at expansion joints

Vertical hold down devices are required whenever the seismic action opposes permanent load action and is greater than 80 percent of the magnitude of the permanent load action in ductile structures and 50 percent in partially ductile structures. This requirement does not apply to individual bearings, but no uplift of individual bearings should occur under the design seismic action.

Minimum Overlap Lengths (or Seat Width)

At supports where relative displacement between the supporting and the supported elements is intended to occur under seismic loading, a minimum overlap length is specified. (Similar to N in AASHTO). This length is:

$$l_{ov} = L\frac{v_g}{c_p} + d_{ef} + m \tag{4-22}$$

where
l_{ov} = minimum overlap length
L = effective length of the deck
v_g = maximum ground velocity (depends on soil classification)
c_p = low estimate of the compressive wave velocity of the surface soils
d_{ef} = predicted seismic relative displacement
m = 40 cm

In cases for which the contributing lengths are different, the root-mean-square value is used.

The maximum soil velocity v_g may be estimated from the maximum ground acceleration and soil type according to:

$$v_g = \alpha\, a_g g \tag{4-23}$$

where
α = 0.10, 0.14, or 0.18 for Type A, B, or C soils, respectively
g = acceleration of gravity

The low estimate of compressive wave velocity c_p may be taken as 500, 300, or 150 m/s for Type A, B, or C soils, respectively.

The predicted relative seismic displacement d_{ef} is the total design seismic displacement d_{td} or, in the case of abutments where slack is provided in the restrainers or links, $d_{ef} = d_{td}$ + slack.

4.3.11 Bridges with Seismic Isolation Devices

A linear response-spectrum analysis can be used for the design of seismically isolated bridges if the following criteria are met:

- Distance from the site to the nearest active fault is greater than 15 km.
- Soil at the site is Type A or B.
- Effective first natural period is less than 3.0 seconds and is at least three times the first natural period of the elastic fixed-based structure (without isolators).
- Bridge is "regular".
- Effective stiffness of the isolation system at design displacement d_d is at least 50 percent of the effective stiffness at 20 percent of d_d.
- The force-displacement characteristics of the isolation system are not dependent on the rate of loading or on the vertical or bidirectional load.

- The isolation system provides a restoring force such that the increase in force between 0.5 d_d and d_d is at least 2.5 percent of the total gravity load above the isolators.

When these criteria are not met, a properly substantiated, site-specific, five-percent-damped spectrum is required. If all of the foregoing criteria are met, a fundamental mode analysis is permitted. If all but the soil type, structure regularity, and isolator characteristic restrictions are met, then a multi-mode response-spectrum analysis is required. In other cases, a time-history analysis is required.

If a fundamental mode or multi-mode response-spectrum analysis is used, the normal site spectrum is modified for the increased damping provided by the isolators for the period range above 0.8 times the fundamental period of the structure without isolators. The modification factor η_I by which the spectral values are multiplied for damping ξ_I other than five percent is given by:

$$\eta_I = [7.0 / (2 + \xi_I)]^{0.35} \tag{4-24}$$

Torsional effects are to be considered and combinations of the response orthogonal motions are combined as specified in section 4.3.2.

Verification forces in the deck and piers are the forces from the analysis increased by a factor equal to 1.1 times the ratio of the maximum and minimum isolator stiffness k_{max} / k_{min}.

A factor of safety is applied to the design displacement at the isolation interface to provide for larger earthquakes. The extreme earthquake displacement d_{ed} is found from the analysis result displacement d_{sd} by the following expression:

$$d_{ed} = \gamma_e \, d_{sd} \tag{4.25}$$

where γ_e is a function of the maximum ground acceleration a_g. Values for γ_e are given in Table 4-5.

Table 4-5 Values of γ_e, Eurocode

Maximum ground acceleration, a_g [g]	γ_e
> 0.40	1.25
0.30	1.50
0.20	1.75
0.15	2.00
0.10	2.50
0.05	3.00

A separate set of requirements is given for the case in which the isolation is achieved by means of elastomeric bearings that do not provide increased damping, except as a fundamental characteristic of the elastomer. Bonded metal shims are required in such cases. The design is based on a total design shear strain less than 75 percent of the minimum elongation at break strain of the elastomer (as determined by tests) and seismic shear strain less than 200 percent. The total design shear strain includes strain due to compression, shear displacement, and angular rotation.

Conditions under which friction may be relied to prevent sliding of the bearings are specified.

$$\frac{V_{sd}}{N_{sd}} < 0.1 + \frac{k_f}{\sigma_m} \tag{4-26}$$

where
V_{sd} = maximum shear across the bearing
N_{sd} = simultaneous vertical force
k_f = friction factor equal to 0.60 for concrete and 0.20 for all other surfaces
σ_m = mean normal stress on the area of the bonded shims [N/mm^2]

If these criteria are not met, then positive means of fixing are to be provided to resist the total force V_{sd}.

4.4 Comparison of Bridge Seismic Design Specifications

Table 4-6 provides a comparison of selected provisions in the following bridge codes and specifications: AASHTO Standard Specifications for Highway Bridges, 16th Edition, Division I-A (AASHTO, 1995); AASHTO LRFD Bridge Design Specifications (AASHTO, 1994); Caltrans Bridge Design Specifications Manual (Caltrans, 1993); ATC-32 Improved Seismic Design Criteria for California Bridges: Provisional Recommendations (ATC, 1996); Orange County, California, Transportation Corridor Agencies (TCA) criteria; Transit New Zealand Bridge Manual (TNZ, 1994); Eurocode 8 (CEN, 1994); and the Japan bridge code (Iwasaki et.al., 1990; and Kawashima, 1995). Symbols are defined in the relevant text material, or in the Definitions of Selected Symbols.

Table 4-6 Comparison of

Provision	AASHTO 16th Ed, Division I-A	AASHTO LRFD	Caltrans	ATC-32
1. General				
a. Performance Criteria	Implied. Maintain structural integrity and prevent collapse during strong shaking.	Implied. Maintain structural integrity and prevent collapse during strong shaking.	Implied. Maintain structural integrity and prevent collapse during strong shaking.	Specifically spelled out for two levels of seismic loading. Criteria defined in terms of physical damage and disruption to service. Varies based on bridge importance.
b. Design Philosophy	Provide adequate ductility capacity and prevent failure of non-ductile elements and elements not easily inspected or repaired.	Provide adequate ductility capacity and prevent failure of non-ductile elements and elements not easily inspected or repaired.	Provide adequate ductility capacity and prevent failure of non-ductile elements and elements not easily inspected or repaired.	Similar to existing Caltrans.
c. Design Approach	Single-level design. Desired performance at lower earthquake (EQ) loads is implied.	Single-level design. Desired performance at lower EQ loads is implied.	Single-level design. Desired performance at lower EQ loads is implied.	Two-level design for important bridges. Single level for ordinary bridges using upper-level EQ; desired performance at lower EQ loads is implied.

TCA	New Zealand	Eurocode 8	Japan
In addition to limiting damage during strong shaking, no damage is to occur during events with a high probability of occurring during life of bridge.	Implied. Maintain structural integrity and prevent collapse during strong shaking.	No collapse under safety-level event (ultimate limit state). No damage under event w/ higher probability of occurrence (service limit state).	Roadway function maintained in small-to-moderate earthquakes. Total collapse to be avoided during significant (historical) earthquake.
Similar to Caltrans. Damage specifically allowed to protect abutment foundations.	Provide adequate ductility capacity and prevent failure of non-ductile elements and elements not easily inspected or repaired.	Elastic structures should have sufficient strength to avoid damage. Brittle types of failure to be avoided in all structures.	Components to perform elastically under functional earthquake. Details provided to mitigate unacceptable damage to certain components.
Two-level design for upper and lower level EQs.	Single-level design. Desired performance at lower EQ loads is implied.	Single-level design. Desired performance at lower EQ loads is implied.	Design bridge for functional-level earthquake using working stress methods. Check column ductility for significant event. Provide details to prevent falling of girders, etc.

Provision	AASHTO 16th Ed, Division I-A	AASHTO LRFD	Caltrans	ATC-32
2. Seismic Loading	Elastic design spectra.	Elastic design spectra.	Elastic design spectra.	Elastic design spectra for 3 different magnitude events or site-specific design spectra. Final provisions provide guidelines for developing standard time histories, and loading and re-sponse adjustment factors to consider near source effects.
a. Return Period	475 year.	475 year.	Not defined Maximum Credible Earthquake (MCE) with mean attenuation given by *ARS* curves.	Safety Evaluation - 1,000 year. Functional Evaluation not well defined - 150 to 300 years.
b. Geographic Variation	Equal risk-based map giving values of effective peak acceleration.	Equal risk-based map giving values of effective peak acceleration. Based on maps in 1998 NEHRP Provisions for New Buildings (BSSC, 1991)	Contour maps of peak rock acceleration developed for MCE and average attenuation as a function of distance from fault.	Caltrans peak rock acceleration maps.
c. Importance Considera-tions	Two categories.	Three categories.	Not addressed directly by design specifications.	Site-specific. Two categories; studies to be considered for important bridges.

Bridge Seismic Design Specifications (continued)

TCA	New Zealand	Eurocode 8	Japan
Elastic design spectra.	Normalized inelastic design spectra for 3 soil conditions and ductility factors between 3 and 6.	Normalized elastic response spectra with variable cut-off frequencies and frequency vs. acceleration relationships. Values set by national agencies. Power spectrum representation permitted.	Elastic design spectra for magnitude 8± event.
Upper level - 1,000 year. Lower level - 150 year.	450 year. Adjustment by Poisson distribution for different return periods and design life.	475 year. Adjustment by Poisson distribution for different return periods and design life.	Not defined.
Site-specific based on route location.	Zone factors (map).	Set by national agencies.	Zone modification factor from map.
All structures on the design corridors to which these criteria apply are considered important.	Three categories based on traffic and importance (jurisdiction).	Three categories based on national definitions.	Importance modification factor based on type of route or for particularly important bridges.

Provision	AASHTO 16th Ed, Division I-A	AASHTO LRFD	Caltrans	ATC-32
d. Site Effects	Three soil type factors.	Four soil type factors.	Four soil profile types based on alluvial depth to bedrock.	Standard NEHRP (BSSC, 1994) soil profile types (B,C,D & E).
e. Damping	5% critical.	5% critical.	5% critical.	5% critical.
f. Duration Considera-tions	Not accounted for.	Not accounted for.	Not considered directly.	Three (3) sets of design spectra allow future implicit consideration.
3. Analysis				
a. Selection Guidelines	Selection based on Seismic Performance Category (SPC), which is function of a_{max} and importance.	Selection based on Seismic Performance Category (SPC), which is function of a_{max} and importance.	Based on structure complexity only. Structure with $T > 3.0$ sec treated as special case.	Selection matrix based on structure complexity and importance and on type of evaluation being performed.
b. Equivalent Static	May be used for "regular" bridges.	May be used for "regular" bridges.	Uniform lateral load—preferred for hinge restrainer design.	Apply static load in proportion to mass or product of mass times displacement.

Bridge Seismic Design Specifications (continued)

TCA	New Zealand	Eurocode 8	Japan
Site-specific.	Inelastic spectra for three site conditions.	Three soil types defined. Spatial variability must be considered for bridges with total length > 600m and where abrupt change in soil type.	Three types of ground condition defined.
5% critical.	5% critical.	5% critical normally. Damping multiplicative factor on response spectra.	5% critical implied.
Not considered directly.	Not considered.	Specifications for time histories.	Not considered directly.
Multi-modal response spectrum analysis except for single-span bridges.	Based on ductility capacity of structure.	Three types of structure performance defined: ductile; limited ductile; and elastic	Based on structure complexity.
Single-span bridges only.	May be used for all but "irregular" bridges. Accidental torsion provided for.	Applicable for "regular" bridges.	Reaction force method (lollipop). Static frame method with static load proportional to the product of mass times displacement.

Provision	AASHTO 16th Ed, Division I-A	AASHTO LRFD	Caltrans	ATC-32
c. Elastic Dynamic	Primary analysis method.	Primary analysis method.	Multi-modal response spectrum analysis. 3-D lumped mass space frame with gross section properties.	Multi-modal response spectrum analysis. 3-D lumped mass space frame with cracked member properties.
d. Inelastic Static	Not required.	Not required.	Not required.	3-D incremental "push-over" analysis. Combine with the effects of dead load and partial live load.
e. Inelastic Dynamic	Recommended for "unusual" or "critical" bridges.	Required for "unusual" or "critical" bridges.	No guidelines.	Special case—may be substituted for push-over analysis—guidelines provided.
f. Directional Uncertainty	100% of forces for one direction with 30% of forces for orthogonal direction.	100% of forces for one direction with 30% of forces for orthogonal direction.	Case I = $L + .3\ T$ Case II = $.3\ L + T$	Case 1 = $0.4L + T + 0.4V$ Case 2 = $L + 0.4T + 0.4V$ Case 3 = $0.4L + 0.4T + V$
g. Load Combinations	Not explicitly stated. Implied = $DL + PS + b_E E + EQ$	Not explicitly stated. Implied = $DL + PS + b_E E + EQ$	AASHTO Group VII = $DL + PS + b_E E + EQ$	AASHTO Group VII = $DL + PS + b_E E + EQ$

TCA	New Zealand	Eurocode 8	Japan
Multi-modal response spectrum analysis. 3-D lumped-mass space frame. Cracked properties and "free" abutments for safety earthquake.	Not used.	Multi-modal response spectrum analysis. 3-D lumped mass space frame.	Considered special case.
Not required.	Not used except that collapse mechanism must be insured.	Capacity design required for ductile structures.	Considered special case.
No guidelines.	SDOF or multi-mode inelastic response spectrum analysis.	Permitted but results may not relax requirements from elastic response spectrum analysis except for seismically isolated bridges.	Considered special case.
Case I = $L + .3\ T$ Case II = $.3\ L + T$	Analyze for two directions; combining results not required.	Case 1 = $.3\ L + T + .3\ V$ Case 2 = $L + .3\ T + .3V$ Case 3 = $.3\ L + .3\ T + V$	Longitudinal and transverse loads examined separately.
AASHTO Group VII = $DL + PS + b_E E + EQ$	Not stated.		EQ combined with "Primary loads".

Provision	AASHTO 16th Ed, Division I-A	AASHTO LRFD	Caltrans	ATC-32
4. Seismic Effects				
a. Design Forces	Component basis.	Component basis.	Consider ductility and "risk".	Consider ductility and structure importance.
i. Ductile Components	Elastic design forces divided by force-reduction factor *R*.	Elastic design forces divided by force-reduction factor *R*.	Component and period-dependent *Z* factors.	Component, structure and site period, and importance dependent *Z* factors.
ii. Nonductile Components	Elastic design forces divided by force-reduction factor *R*.	Elastic design forces divided by force reduction factor *R*.	Capacity design or use $Z = 0.8$.	Capacity design or use $Z = 0.8$.
b. Displacements	Determined from elastic analysis. No guidance on applicable moment of inertia.	Determined from elastic analysis. Cracked section moment of inertia recommended.	Use cracked model.	Use elastic analysis with cracked member properties -adjust based on T/T_g.
c. Minimum Seat Width	SPC A & B: $(8+0.02L+0.08H)$. SPC C & D: $(12+.03L+.12H)$.	Same as for AASHTO 15th Ed $\times (1+S^2/800)$.	$(12+.03L+.12H) \times (1+S^2/800)$.	$(12+.03L+.12H) \times (1+S^2/800)$.

Bridge Seismic Design Specifications (continued)

TCA	New Zealand	Eurocode 8	Japan
Consider ductility.	Consider ductility.		Elastic forces for functional earthquake. Forces adjusted for ductility when ductile behavior is required.
Component and loading-type dependent Z factors.	EQ forces from inelastic response spectrum analysis but not less than 80% of equivalent static force analysis results based on first natural period.	Elastic force reduction factor depends on structure type and component $1.0 < q < 3.5$. (q analogous to R)	Revise design coefficient considering ductility and magnitude 8± ground motion and check ultimate force capacity.
Capacity design or use $Z = 0.8$.	Capacity design.	Capacity design.	Force design using functional earthquake and ductility check with reduced ductility capacity.
Use cracked model for upper-level earthquake. Uncracked for lower-level earthquake.	Inelastic response-spectrum analysis displacements multiplied by ductility factor. Use cracked sections for columns.	Use secant stiffness at yield (i.e., cracked sections) for members with plastic hinges.	
$(12 + .03L + .12H)$ — Skew limited.	without restrainers: $2.0 \times$ displ +100 mm (400 mm min.) with loose restrainers: $1.5 \times$ displ + 100 mm (300 mm min.) with tight restrainers: 200 mm.	Based on estimated seismic displacements that depend on maximum ground velocity, P-wave velocity, and contributing length. 400mm minimum.	$70 + 0.5l$ $l \leq 100$ meters $80 + 0.4\,l$ $l \geq 100$ meters

Provision	AASHTO 16th Ed, Division I-A	AASHTO LRFD	Caltrans	ATC-32
5. Concrete Design				
a. Columns (Flexure)	Ultimate strength design using appropriate force-reduction factor and capacity-reduction factor between 0.50 and 0.90 depending on axial load ratio. Longitudinal reinforcing.	Ultimate strength design using appropriate force-reduction factor and capacity-reduction factor between 0.50 and 0.90, depending on axial load ratio. Longitudinal reinforcing.	Ultimate strength design using nominal material strengths and a ϕ factor between 1.0 and 1.2 depending on axial stress. $0.01 \leq \rho_s \leq 0.08$	Ultimate strength design using expected material strengths and ϕ=1.0 (M_E). $0.007 \leq \rho_s \leq .04$
b. Columns (Shear)	Shear demand based on capacity design with $M_P = 1.3M_N$. Shear capacity based on contribution from concrete ($V_c = 2\sqrt{f_c'}$) and transverse reinforcing. Concrete contribution at plastic hinges is reduced to zero as axial stress decreases from $.10f'_c$ to zero.	Same as AASHTO, 16th Ed.	Same as AASHTO, 16th Ed.	Shear demand based on capacity design with $M_P = 1.4M_E$. Shear capacity based on contribution from concrete and transverse reinforcing. Concrete contribution based on presence or absence of plastic hinging.

TCA	New Zealand	Eurocode 8	Japan
	.		
Same as Caltrans.	Covered by other documents	Ultimate strength design with capacity-reduction factors.	Allowable stress design with 1.5 × normal design stresses.
Same as Caltrans.	Covered by other documents	Verification of capacity of diagonal compression, shear reinforcement, and sliding shear required.	Allowable stress design with 1.5 × normal design stresses.

Provision	AASHTO 16th Ed, Division I-A	AASHTO LRFD	Caltrans	ATC-32
c. Column spirals (Confinement)	*Outside plastic hinge zone:* $\rho_s = 0.45(A_g/A_c - 1)\, f'_c/f_y$ *Within plastic hinge zone:* Columns 3' or less $\rho_s = 0.45(A_g/A_c - 1)(f'_c/f_y)(0.5 + 1.25P_e/(f'_c A_g))$ Columns larger than 3': $\rho_s = 0.12(f'_c/f_y)(0.5 + 1.25P_e/(f'_c A_g))$	*Outside plastic hinge zone:* $\rho_s = 0.5 \times \rho_s$(plastic hinge) *Within plastic hinge zone:* $\rho_s = 0.16(f'_{ce}/f_{ye})(0.5 + 1.25P_e/(f'_{ce} A_g)) + 0.13(\rho_l - 0.01)$	Same as AASHTO, 16th Ed	Same as AASHTO LRFD
d. Column ties (Confinement)	*Outside plastic hinge zone:* $A_{sh} = 0.3 s_t h_c (f'_c/f_y)(A_g/A_c - 1)$ *Within plastic hinge zone:* $A_{sh} = 0.12 s_t h_c (f'_c/f_y)(0.5 + 1.25P_e/(f'_c A_g))$	*Outside plastic hinge zone:* $A_{sh} = 0.3 s_t h_c (f'_c/f_y)(A_g/A_c - 1)$ *Within plastic hinge zone:* $A_{sh} = 0.12 s_t h_c (f'_c f_y)(0.5 + 1.25P_e/(f'_c A_g)) + 0.13\, s_t h_c (\rho_l - 0.01)$	Same as AASHTO, 16th Ed.	Same as AASHTO LRFD.
e. Piers	Force-reduction factor *R*=2 in strong direction. Design as column in weak direction.	Force-reduction factor *R*=2 in strong direction. Design as column in weak direction.	$\rho_s \geq 0.0025$ Nominal detailing with reduced ductility allowed.	$\rho_s \geq 0.01$ Shear design similar to beams and columns.
f. Footings	Design for lesser of elastic EQ forces or overstrength plastic hinging forces (capacity design). Use ultimate strength design with normal capacity-reduction factors.	Design for lesser of elastic EQ forces or overstrength plastic hinging forces (capacity design). Use ultimate strength design with normal capacity-reduction factors.	Capacity design for seismic loading. Ultimate strength design with $\phi = 1.0$.	Same as Caltrans.

Bridge Seismic Design Specifications (continued)

TCA	New Zealand	Eurocode 8	Japan
Same as AASHTO, 16th Ed.	Covered by other documents	Not required where axial load ratio < 0.08. Volumetric requirement function of required curvature ductility and axial load ratio.	Column design ductility factors are based on tests of typically reinforced Japanese bridge columns.
Same as AASHTO, 16th Ed.	Covered by other documents	Not required where axial load ratio < 0.08 or in flanged or box sections where curvature ductility of 15 attainable with maximum concrete compressive strain <.35%. Volumetric requirement function of required curvature ductility and axial load ratio.	Column design ductility factors are based on tests of typically reinforced Japanese bridge columns.
Same as Caltrans.	Covered by other documents	Covered as tied columns.	Column design ductility factors are based on tests of typically reinforced Japanese bridge columns.
Same as Caltrans.	Covered by other documents	Detailing rules.	Allowable stress design with 1.5 × normal design stresses.

Provision	AASHTO 16th Ed, Division I-A	AASHTO LRFD	Caltrans	ATC-32
g. Superstructure and Column Joints	Confinement reinforcement extended into cap.	Maximum joint shear: $V_c = 12\sqrt{f'_c}$ Confinement reinforcement extended into cap.	Joint shear: $V_c = 12\sqrt{f'_c}$ Spirals extended into cap.	Rational analysis of joint stresses required. Tensile stresses above $3.5\sqrt{f'_c}$ to be resisted by special reinforcing. Compression stresses not to exceed $0.25\,f'_c$.
h. Caps	Design for lesser of elastic forces (R=1) or over-strength plastic hinging forces (capacity design). Normal capacity-reduction factors apply.	Design for lesser of elastic forces (R=1) or over-strength plastic hinging forces (capacity design). Normal capacity-reduction factors apply.	Post-Loma Prieta interim design rules require capacity design techniques to assure plastic hinges form in columns rather than caps.	Capacity design techniques to assure plastic hinges form in columns rather than caps.
I. Superstructure	Design for lesser of elastic forces (R=1) or overstrength plastic hinging forces (capacity design). Normal capacity-reduction factors apply.	Design for lesser of elastic forces (R=1) or overstrength plastic hinging forces (capacity design). Normal capacity-reduction factors apply.	Post Loma Prieta interim design rules require capacity design techniques to assure plastic hinges form in columns rather than superstructure.	Capacity design techniques to assure plastic hinges form in columns rather than superstructure.
j. Shear Keys	Force-reduction factor R=0.8 Shear-friction design with capacity - reduction factor 0.85.	Force-reduction factor R=0.8 Shear-friction design with capacity- reduction factor 0.85.	Designed as corbels. Abutment shear keys designed as sacrificial elements to protect stability of abutment.	Same as Caltrans
k. Column Anchorage	Splice lengths as per Section 8, Division I (Similar to ACI 318-89).	Splice lengths as per Section 8, Division I (Similar to ACI 318-89).	Since Loma Prieta Caltrans has been using ACI anchorage criteria. $l_a = 0.028 d_{bl} f_y$	Column Anchorage Length: $l_a = 0.028 d_{bl}$ $f_y/\sqrt{f'_c}$ when confined by spirals where: $\rho_s = 0.6\rho_l D/l_s$

Bridge Seismic Design Specifications (continued)

TCA	New Zealand	Eurocode 8	Japan
Use of nominal supplementary mild reinforcement to prevent shear failures.	Covered by other documents	Not covered.	Unknown.
Same as Caltrans.	Covered by other documents	Capacity design in ductile structures.	Unknown.
Use of nominal supplementary mild reinforcement to prevent hinging in superstructure.	Covered by other documents	Capacity design in ductile structures.	Unknown.
Same as Caltrans.	Covered by other documents	Capacity design in ductile structures.	Allowable stress design with 1.5 × normal design stresses.
Same as Caltrans.	Covered by other documents	Covered in other documents.	Unknown.

Provision	AASHTO 16th Ed, Division I-A	AASHTO LRFD	Caltrans	ATC-32
6. Column Splices	Splices in longitudinal reinforcement in central 1/2 of column height only. 60 bar diameters or 16 in (400 mm) minimum.	Splices in longitudinal reinforcement in central 1/2 of column height only. 60 bar diameters or 16 in (400 mm) minimum.	Lap splices are not allowed in transverse column reinforcement or in longitudinal reinforcing within plastic hinge zones.	Lap splices are not allowed in transverse column reinforcement or in longitudinal reinforcing within plastic hinge zones. Splice length given by $l_s = 0.04 d_b f_s / \sqrt{f_c'}$ Lap splices must be confined by hoops or spirals where: $\rho_s = 6A_{bl}/Dl_s$.
7. Steel Design	No specification provisions for seismic design of steel members.	No specification provisions for seismic design of steel members.	No specification provisions for seismic design of steel members.	Detailing rules to assure plastic hinges form properly in compression members. Provisions assuring a viable load path through cross-bracing and bearings.
8. Foundation Design				
a. Spread Footings	Separation of up to 50% width allowed. Ultimate soil pressures used with normal capacity-reduction factors.	Separation of up to 50% width allowed. Ultimate soil pressures used with normal capacity-reduction factors.	Designed for ultimate soil bearing capacity under seismic loading . Some uplift of edges of footing allowed.	Designed for ultimate soil bearing capacity considering the effect of overturning moment and inclined loading. Allowable uplift of edges of footing based on type of bent.
b. Pile Footings	Separation of up to 50% width allowed. Ultimate end bearing pile capacity used with normal capacity-reduction factors.	Separation of up to 50% width allowed. Ultimate end bearing pile capacity used with normal capacity-reduction factors.	Designed for ultimate compressive capacity and 50% uplift capacity (friction piles) under seismic loads. Batter piles discouraged.	Similar to Caltrans.

Bridge Seismic Design Specifications (continued)

TCA	New Zealand	Eurocode 8	Japan
Same as Caltrans.	Covered by other documents	No splices permitted in plastic hinging regions.	Unknown.
No specification provisions for seismic design of steel members.		No specific provisions.	Unknown.
		Foundations not to be used as source of energy dissipation.	
Same as Caltrans.	"Rocking" foundations permitted (time-history dynamic analysis required). Design for ultimate foundation pressures with capacity reduction factor. Minimum factor of safety against sliding 1.2.	No specific provisions for spread footings.	Unknown.
Same as Caltrans.	Allowance made for liquefiable layers, residual scour, and pile/soil separation in cohesive soils to depth of 2 pile diameters.	No specific provisions for pile footings.	Unknown.

Provision	**AASHTO 16th Ed, Division I-A**	**AASHTO LRFD**	**Caltrans**	**ATC-32**
c. Liquefaction	Empirical assessment procedure in commentary.	Not mentioned.	Not addressed by specifications.	Procedures within guidelines for calculating lateral displacements due to liquefaction.
9. Misc. Design				
a. Restrainers	Required at all joints. Empirical design for a_{max} times weight of lighter adjacent unit.	Required at joints where minimum seat width not met. Empirical design for a_{max} times weight of lighter adjacent unit.	Designed to remain elastic and restrict movement to allowable levels at expansion joints. Nominal restrainers required when ample seat width provided.	Same as Caltrans.
b. Base isolation	Grade Specification Adopted in 1990.	Grade Specification Adopted in 1990.	Special case.	Not addressed.
c. Active/ passive control	Not covered.	Not covered.	Not used.	Not addressed.

Bridge Seismic Design Specifications (continued)

TCA	New Zealand	Eurocode 8	Japan
Not addressed by specifications.	See above.	Not mentioned.	Method provided for determining liquefaction resistance factor $F_L = R/L$
Same as Caltrans. Expansion joints avoided if possible.	Required if minimum seat length without restrainers not met.	Required if minimum seat length without restrainers not met in ductile structures. Required in structures of limited ductility. Required in base-isolated structures.	Standard devices available for tying superstructure and substructure together and for preventing the dislodging of moveable bearings (stoppers).
To be considered as an alternative design approach.	General guidance only. Details covered by other documents.	Specific chapters devoted to seismic isolation and elastomeric bearings.	Draft guidelines available.
Not used.	Not mentioned.	Not mentioned.	Draft guidelines available.

SECTION 5
U.S. AND FOREIGN REQUIREMENTS FOR GEOTECHNICAL DESIGN OF FOUNDATIONS

This section covers both U.S. and foreign seismic design requirements for bridge foundations. U.S. requirements are presented in Section 5.2 and the New Zealand, Japanese, and European requirements are presented in Section 5.3.

5.1 Introduction

This section summarizes the results of a review of geotechnical requirements for the seismic design of bridge foundations given in guidelines and codes in the United States, Europe, Japan, and New Zealand. The documents reviewed include:

- AASHTO *Standard Specifications for Highway Bridges*, Division I-A: Seismic Design (1995)
- Sections 3 and 4 from ATC-32, the recommended revisions to the Caltrans Bridge Design Specifications (BDS).
- AASHTO *LRFD Bridge Design and Specifications and Commentary* (1994)
- Section 2.1.2 of the 1994 Edition *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*
- Articles by Iwasaki et al. (1990) and Kawashima, (1995) summarizing recent changes to the Japanese code (1990)
- *New Zealand Bridge Manual: Design and Evaluation* (TNZ, 1994)
- Draft Eurocode 8: *Structures in Seismic Regions - Design, Part 5: Bridges* (1994)

The design philosophy within each of these codes and guidelines is generally similar from the standpoint that each endeavors to protect important structures such as essential bridges from earthquake-induced damage. The intent of the foundation provisions in these codes and commentaries is to provide foundations that have the capability to support loads that may develop during the inelastic response of the structure to major earthquake ground motions.

5.2 U.S. Foundation Design Requirements

Historically, codes and guidelines for foundation design have dealt primarily with the structural aspect of the foundation, with little attention given to geotechnical considerations. Traditionally, the only geotechnical direction provided by codes and guidelines was limited to requirements that appropriate consideration be given to the soil surrounding the foundation during seismic design. Within the last decade or so, codes and guidelines have become much more specific regarding the manner in which the geotechnical aspects of foundation design should be handled. Issues covered within the newer codes often include topics such as exploration methods, methods of ground response analysis, and procedures for evaluating soil-foundation interaction.

5.2.1 AASHTO Standard Specifications

The AASHTO *Standard Specifications for Highway Bridges*, Division I-A: Seismic Design gives specific requirements for foundation design for Seismic Performance Categories B, C, and D. These requirements include consideration of slope instability, liquefaction, fill settlement, and increases in lateral earth pressures as a result of seismic loading. Methods for addressing these issues are identified in the commentaries to Division 1-A, Seismic Design.

These commentaries provide information on the selection of ground motions and response spectra for a project, evaluation of liquefaction potential, and foundation design. The foundation design discussion covers topics such as lateral loading to piles and earth pressures on abutments.

5.2.2 AASHTO LRFD Specifications

The LRFD Bridge Design Specifications and Commentary do not provide any specific direction for foundation design. The commentary does, however, indicate that essential bridges should be designed for an earthquake with a 475-year return period, whereas critical bridges should be designed for an event with a 2,500-year return period, which is referred to as the maximum credible earthquake. As with the AASHTO standard specifications, four soil profile types are identified for establishing site coefficients and soil spectra. These specifications discuss the need to design walls for static earth pressures compatible with wall deformation, but make no mention of modifications for seismic loads.

5.2.3 NEHRP Provisions

The NEHRP *Recommended Provisions for Development of Seismic Regulations for New Buildings* (Parts 1 and 2) provide direction on methods for incorporating soil-structure interaction when determining design earthquake forces and the corresponding displacements of buildings. These provisions include information on the reduction of soil stiffness in terms of soil shear modulus and effective (geometric) damping for different levels of velocity-related ground acceleration (A_v). The commentary to the NEHRP provisions provides specific direction on evaluating fault hazards, liquefaction, and ground displacement (settlement, slope instability, and lateral spreading). Limited information is presented on the design of pile foundations and no specific information is provided regarding earth pressures.

5.2.4 ATC-32 Recommendations

Proposed revisions to Section 4 of the specifications for the seismic design of abutments and foundations within the Caltrans *Bridge Design Specifications* (BDS) are included in the ATC-32 report (ATC, 1996a). The proposed revisions provide detailed information about geotechnical considerations during the seismic design of bridge foundations. This information includes requirements for site investigations, site stability analyses, abutment and wing wall design requirements, pile and spread footing design requirements, and retaining structure design. The commentary to the proposed revision notes that foundation failures were the main cause of bridge failure in some large (magnitude > 7) earthquakes, as was evident from the 1964 Alaska earthquake, the 1964 Niigata earthquake, the 1990 Philippines earthquake, and the 1991 Costa Rica earthquake. In all these earthquakes, foundation failure related to liquefaction-induced lateral spreading and subsequent loss of bearing capacity were the cause of bridge collapse. Given these observations of past failure, detailed direction is provided in the commentary for evaluating the potential for liquefaction and lateral spreading.

The commentary to the proposed revisions also provides direction for evaluating site response at soft clay sites, in response to large ground motion amplification observed during the 1985 Mexico City and 1989 Loma Prieta earthquakes. This direction includes the need to conduct site-specific response analyses. While not specifically identified in the commentary, Caltrans' approach to these site-specific analyses has involved a one-dimensional, equivalent linear modeling with SHAKE (Schnabel et al, 1972). More recent projects have used nonlinear, effective stress models such as DESRA (Lee and Finn, 1978).

The design of pile foundations receives very extensive coverage in the commentary, including the need to consider the cyclic degradation of soil strength. Recommendations are given for evaluating the stiffness of piles and large-diameter shafts, stiffness and capacity of a pile-footing, batter pile design, and pile-group effects (e.g., to neglect group effects at center-to-center spacings greater than three diameters).

Spread footing design recommendations are provided, These deal primarily with handling the combined effects of dead, moment, and lateral loads under large earthquake-induced loading.

For retaining structure design, the commentary notes that free-standing retaining walls appear to have performed well during past earthquakes, even though most retaining walls have been designed only for relatively low static active earth-pressure coefficients. This good performance is attributed to the ability of the wall to deform, thereby relieving excessive wall pressures. This document describes an iterative process for the design of abutments and wing walls that accounts for soil capacity and wall flexibility. A resource document (ATC, 1996b) that accompanies the proposed revisions includes design charts for determining earth-pressure coefficients and for determining displacements associated with the use of the Newmark sliding block model for the evaluation of dynamic wall loading.

5.2.5 Other Guidelines for Seismic Response of Bridge Foundations

The Federal Highway Administration (FHWA) sponsored the preparation of a valuable guideline for seismic design of bridge foundations entitled "Seismic Design of Bridge Foundations" (Lam and Martin, 1986). This document was published in 1986 as a supplement to the then-current FHWA/AASHTO guidelines, with the intent of providing simplified, explicit approaches for the seismic design of bridge foundations and abutments. The document now serves as one of the basic design references for state highway agencies and consultants involved in bridge foundation design. Although the report was published in 1986, it still represents one of the most thorough and useful treatments of bridge foundations and now serves as a starting point for new code development. For example, the foundation portions of ATC-32 generally built on the 1986 FHWA document.

One of the key values of the 1986 FHWA bridge foundation design guidelines is that it covers the entire spectrum of issues related to seismic design for bridge foundations. Section 3 of the document provides a summary of soil property evaluation, where reference is made to use of the Standard Penetration Test (SPT) and the Cone Penetrometer Test (CPT) methods for dynamic property assessments. This is followed by Section 4, which discusses liquefaction, slope stability, and dynamic settlement analyses. The remainder of the document provides guidelines for the determination of:

- Determination of stiffness coefficients for spread footings (Section 5)
- Analysis of the axial and lateral behavior of driven piles (Section 6)
- Analysis of the axial and lateral behavior of drilled shafts (Section 7)
- Analysis of abutment response (Section 8)

General design and analysis procedures for piles and pile groups are also summarized. These design and analysis procedures cover topics such as linear versus nonlinear foundation design, influence of moment distribution, effects of soil property variation, liquefaction, rotational stiffness of the pile group, batter effect in a pile group, pile group settlement, uplift of a pile in a group, and many other related areas. A set of example problems is included in Volume III of the document to demonstrate the use of the recommended foundation design techniques.

5.3 Foreign Foundation Design Requirements

This section summarizes foundation design requirements given in codes from Europe, Japan, and New Zealand. While each of the codes covers geotechnical seismic design requirements for foundations in different degrees of detail, the issues covered within the three codes are generally similar to those used in the United States.

5.3.1 Eurocode 8

Eurocode 8 includes a specific section for geotechnical issues related to earthquake-resistant design. This section is referred to as Part 5: Foundations, Retaining Structures, and Geotechnical Aspects. The general requirements for foundation systems are:

- Seismic forces are transferred to the ground without substantial permanent deformation.

- The seismically induced ground deformations are compatible with the essential functional requirements of the structure.
- The risk associated with the uncertainty in seismic response is considered.

Part 5 includes guidance on strength parameters, stiffness, and damping properties applicable during seismic loading. With regard to siting facilities, it states that structures in Importance Categories I, II, and III should not be erected in the immediate vicinity of seismically active faults, which is defined as absence of movement in Late Quaternary. The definition of "immediate vicinity" is not provided.

The code provides guidance for evaluating slope stability. Dynamic analysis involving finite element or rigid block models, as well as pseudo-static methods are allowed. For pseudo-static analyses, $F_H = 0.5a_gW$ is applied in the horizontal direction and $F_v = \pm 0.5F_H$ in the vertical direction, where a_g is the applicable design ground acceleration ratio and W is the weight of the sliding mass. A topographic amplification factor for a_g is also provided in Annex A to Part 5. Pseudo-static methods are not used for sites at which the soils are capable of developing high pore water pressures or where significant stiffness degradation occurs under cyclic loading. Guidance is also provided for the safety verification of the pseudo-static method, particularly in areas with $a_g > 0.15g$ where strength degradation and pore-pressure buildup can occur.

With regard to liquefaction assessment, the Eurocode focuses on the Standard Penetration Test (SPT) and the Cone Penetrometer Test (CPT). Impact energy and overburden effects are discussed. For structures on shallow foundations, the code states that liquefaction susceptibility evaluation can be omitted when the saturated sandy soils are found at depths greater than 15 meters. Neglecting the liquefaction hazard is also allowed when $a_g < 0.15g$ and one or more of the following conditions exist:

- The sands have a clay content greater than 20 percent with plasticity index > 10.
- The sands have a silt content greater than 10 percent and $N_{1(60)} > 20$.
- The sands are clean, with $N_{1(60)} > 25$.

Annex B to Part 5 provides charts showing the liquefaction strengths for clean and silty sand, and also as a function of the normalized shear wave velocity. If the field correlation approach is used, a soil is considered susceptible to liquefaction if the earthquake-induced stress exceeds 80 percent of the critical stress known to have caused liquefaction in previous earthquakes, implying a safety factor of 1.25. Settlement of soils under cyclic loads is also addressed in Part 5.

The code includes guidance for soil investigations. CPT is recommended whenever feasible, because it provides a continuous record of soil mechanical characteristics. The profile of shear wave propagation velocity is identified as the most reliable predictor of the site-dependent seismic characteristics. The strain dependency of the soil is defined in Table 5-1. These values are for depths to 20 meters and for average v_{smax}< 300 mps.

Provisions are given for strain dependency, combinations of foundation types (discouraged), and energy transfer to the ground. Bearing capacity must consider load eccentricity and inclination, as well as the possible inertial forces in the supporting soil itself. Provisions are also provided for foundation horizontal connections (tie beams and foundation slab), raft foundations, box-type foundations, and piles and piers.

Piles and piers must be designed to withstand inertial forces from the superstructure and kinematic forces arising from soil deformation caused by seismic waves. Pile head stiffness expressions are provided in Annex C for three soil models. Part 5 also clearly states that the piles must be designed to remain elastic and that when this cannot be achieved, potential plastic hinging must be assumed for a depth of two diameters from the pile cap and an area within two diameters from either side of an interface between two layers with markedly different shear stiffness (ratio of shear moduli > 6). Effects of dynamic soil-structure interaction are also addressed.

Table 5-1 Strain Dependency of the Soil, Eurocode

Ground Acceleration a_g	*Damping Factor*	v_s/v_{smax}	G/G_{max}
0.1	0.03	0.9 (±0.07)	0.8 (±0.1)
0.2	0.06	0.7 (±0.15)	0.5 (±0.2)
0.3	0.1	0.6 (±0.15)	0.35 (±0.2)

5.3.2 Japanese Code

The latest specifications for seismic design of bridge foundations in Japan were issued in February of 1990 by the Ministry of Construction. Kawashima (1995) and Iwasaki et al. (1990) provide the table of contents for the 1990 code, which indicates that geotechnical aspects of foundation design are covered within the code.

As an introduction to his overview, Kawashima cites cases of highway bridge damage due to inadequate strength of foundations and soil liquefaction. He goes on to indicate that Japanese codes began including methods for evaluating liquefaction in the early 1970s. The table of contents given in these review papers indicates that Chapter 3 of the code covers methods for estimating dynamic earth pressures, as well as the reduction in soil bearing capacity during seismic loading. Chapter 6 gives guidance on selecting ground motions for dynamic response analysis.

According to Kawashima, the procedures for determining whether liquefaction is of concern rely on (SPT) blowcount methods, and they include corrections for the mean grain size and fines content of the soil. It is interesting to note that the standard horizontal seismic coefficient used for the check on liquefaction is 0.15, which is 75 percent of the standard design horizontal seismic coefficient. Common practice in the United States is to use a factor of 65 percent. For those soils that are expected to liquefy within approximately 67 feet (20 meters) of the ground surface, spring stiffness values for pile foundations are multiplied by a factor that ranges from 0 to 1. The value used depends on the liquefaction resistance factor, which is equivalent to the factor of safety against liquefaction, and the depth below the ground surface. This approach is much more explicit than recommendations found in the U.S. codes and guidelines.

5.3.3 Transit New Zealand Code

Section 5 of the 1991 Transit New Zealand Code (TNZ, 1994) covers earthquake-resistant design of bridge structures. For design of pile/cylinder foundations, a range of soil stiffness parameters typical for the site are to be considered when estimating the natural period and curvature demands of the structure. Allowances are to be made for liquefaction of cohesionless soil layers and pile-soil separation in the upper two pile diameters in cohesive soil. If liquefaction is likely under the design earthquake, no horizontal restraint should be assigned to the liquefied layers. This differs somewhat from practice in the United States, where a residual strength value is normally assigned to the liquefied layer. Guidance is also provided for structures on spread footing foundations and for foundations subjected to rocking.

The design of earth retaining structures is also covered in Section 5 of the TNZ code. Both bridge abutments and free-standing retaining structures are addressed. A factor of 25 percent is applied to the product of the zone factor (0.6 to 1.2) and the risk factor (1.0 to 1.3) to determine the peak ground acceleration coefficient for design. Vertical accelerations are neglected. Walls must be designed to account for the dynamic increment of earth pressures caused by an earthquake. Guidance is also provided for the design and performance of free-standing walls, tied-back walls, reinforced earth walls, culverts, and subways.

SECTION 6
U.S. AND FOREIGN ABUTMENTS AND RETAINING WALL REQUIREMENTS

This section covers both U.S. and foreign design requirements for abutments and retaining walls. The requirements for each country are addressed separately.

6.1 Introduction

In many situations, abutments and retaining walls represent critical links within a transportation system. During an earthquake, the ability of the abutment to develop sufficient lateral resistance to inertial loading from the bridge deck and its ability to maintain support of end spans will often determine whether structural damage or even collapse of the bridge deck occurs. Performance of the abutment or the retaining structure during an earthquake will also determine whether there is access to the bridge, and hence whether the transportation system is intact following the earthquake.

While the primary function of both abutments and retaining walls is to retain earth on one side of the wall, these two types of walls serve different functions for highway systems:

- *Abutments.* The abutment supports the beginning (or the end) span of a bridge at the juncture where the earth roadway meets the bridge. In addition to retaining the roadway grade, the abutment also acts to support the vertical weight of the end span and to restrain or damp the seismic displacement of the end span during ground shaking. Wing walls on each side of abutments are usually built to make a smooth transition of grade.
- *Retaining Wall.* A retaining wall provides lateral support to ground in areas where a difference in grade occurs. Generally, retaining walls (such as wing walls) retain hillside slopes when stable slopes cannot be achieved otherwise. Also, retaining walls are used as middle dividers for divided highways when roadways are at different grades. Unlike abutments, retaining walls have little, if any, vertical load support requirements.

Although some functions of the abutment and the retaining wall may differ, these systems involve many similar characteristics from a design and performance standpoint. Specifically, the amount of resistance developed by the abutment during inertial loading will be determined by the dynamic earth pressure mobilized behind the wall and the width over which the resistance will be mobilized. The type of soil plays an integral part in this determination. The stiffness of the wall determines the amount and distribution of the resisting loads. Likewise, a retaining structure will have to withstand an increase in lateral earth pressure during an earthquake. The amount of this increase will depend on the rigidity of the wall—seismic earth pressures on a rigid wall could be more than twice those on a flexible wall.

The close relationship between soil behavior and structural characteristics of the wall creates a difficult mechanism to adequately account for when evaluating seismic response. If the structural system is assumed to be too flexible or if deformations are assumed to be too large during active loading conditions, then loads on a retaining wall may be underestimated. Likewise, if the wall is assumed to be too stiff during passive loading conditions, then the resisting capabilities of the wall may be overestimated.

Further complicating this situation is the variety of structural systems that are being used to construct or retrofit abutments and retaining walls. In additional to conventional cantilever or gravity walls that can be supported on spread footings or piles, it is not uncommon to deal with nailed walls, tie-back walls, or mechanically stabilized earth (MSE) walls. To retrofit a foundation, pin piles or compaction grouting might be used to enhance foundation performance. Each of these systems is characterized by different system stiffness values, and each responds differently during seismic loading. As a result, generalization of abutment and retaining wall response into design guidelines is only now being addressed.

6.2 U.S. Design Requirements

The United States has made some progress on developing seismic design guidelines for abutment foundations. These were originally presented in a commentary to Chapter 6 of ATC-6 (ATC, 1981). The introduction to this commentary points out that there have been numerous case histories of bridge damage or failure caused by abutment failure or displacement during earthquakes. Damage was typically associated with fill settlement or slumping, displacements induced by high lateral earth pressures, or the transfer of high longitudinal or transverse inertial forces from the bridge structure itself. While guidelines for retaining wall design have not been explicitly treated in highway design guidelines, foundation engineers responsible for the foundation design recommendations generally use many of the same methods for determining retaining wall pressures as they use for the design of abutments.

6.2.1 AASHTO Standard Specifications

AASHTO has adopted the guidelines given in ATC-6 (ATC, 1981). These guidelines are presented in Division I-A, Seismic Design, of AASHTO's 1995 specifications. They focus on free-standing abutments; i.e., gravity or cantilever walls, which are able to yield laterally during an earthquake. The Mononobe-Okabe pseudo-static approach is given as a means of computing the earth pressures induced by earthquakes.

Several important assumptions are given for this method:

- The abutment is free to yield sufficiently to enable full soil strength or active pressure conditions to be mobilized. It is noted that if the abutment is rigidly fixed and unable to move, the soil forces will be much higher than those predicted by the Mononobe-Okabe analysis.
- The backfill is cohesionless, with a friction angle of ϕ.
- The backfill is unsaturated, so liquefaction problems will not arise.

Clearly, when these assumptions do not apply, the complexity of the design process increases. Other important decisions must also be made by the designer when applying the Mononobe-Okabe method of analysis given in the AASHTO specifications. For example, the specifications do not spell out how to determine the horizontal and vertical seismic coefficients for use in the Mononobe-Okabe equations. Typically, vertical acceleration is ignored, and the horizontal acceleration coefficient is set at 50 percent to 80 percent of the predicted peak ground acceleration. Determining the effects of abutment inertia are also not explicitly required in the Mononobe-Okabe approach. The AASHTO specifications point out that neglect of the inertial forces from the abutment is not conservative.

It was recognized in ATC-6 that it is unrealistic to design for free-standing abutments to provide zero displacement under peak ground accelerations in high seismic areas. This led to the development of a displacement-based design method. By adopting this "Newmark" approach, it can be shown that for a horizontal seismic coefficient equal to 50 percent of the predicted peak ground acceleration, the outward displacement of the abutment will be approximately ten times the peak ground acceleration (in g's), in inches. Given an estimate of displacement magnitude, it is then possible to detail the abutment to perform properly during and after this displacement. This approach is also included in the present AASHTO specifications.

Behavior of nonyielding abutments is also addressed in Division I-A of the AASHTO specifications. This condition is expected to occur to some degree when tie-backs or batter piles are used. Pressures induced by inertial forces in the backfill will be greater than those given by a Mononobe-Okabe analysis. Using a factor of 1.5 in conjunction with peak ground accelerations is suggested for design whenever doubt exists that an abutment can yield sufficiently to mobilize soil strength.

The 1995 AASHTO specifications provide additional direction for the seismic design of mechanically stabilized earth (MSE) walls. With respect to external stability of the wall, the dynamic horizontal force

P_{AE} and the static force are added to 60 percent of the inertial force P_{IR} to evaluate equilibrium conditions. The dynamic thrust is applied at a height of $0.6H$ from the base, and the horizontal inertial force is applied at mid-height. The following equations are used to compute the dynamic thrust, the inertial force, and the maximum wall acceleration coefficient C_m.

$$P_{AE} = 0.375\ C_m \gamma H^2 \tag{6-1}$$

$$P_{IR} = 0.5\ C_m \gamma\ H^2 \tag{6-2}$$

$$C_m = (145 - C)\ C \tag{6-3}$$

where

γ = unit weight of soil

H = wall height

C = seismic coefficient from the acceleration contour maps in the AASHTO specifications

The period of the MSE structure is estimated by

$$T = \frac{H}{125} \tag{6-4}$$

For an MSE structure, factors of safety against sliding and overturning failure under combined loading are taken as 75 percent of the static factors of safety. Guidance is also provided for designing reinforcement (i.e., internal stability), including the distribution of the inertial force to the reinforcement. Reduction factors are given for various design coefficients used to determine reinforcement requirements and factors of safety for stability.

6.2.2 AASHTO LRFD Specifications

Section 11 of the AASHTO LRFD Bridge Design Specifications and Commentary (AASHTO, 1994) also provides a summary of procedures for calculating earth pressures on walls. These procedures generally follow those given previously in AASHTO specifications. For example, the Mononobe-Okabe method is used to estimate equivalent static forces for gravity and semi-gravity retaining walls. The estimate is supposed to account for wall inertial forces in addition to the equivalent static forces, unless the wall is a flexible cantilevered wall, in which case the inertial force can be ignored.

For anchored walls, the commentary to the seismic design procedure states that the Mononobe and Okabe method may be used to estimate the equivalent static forces, provided that the maximum lateral earth pressure is computed using a seismic coefficient $k_h = 1.5\ A$. This approach is consistent with the recommendation in ATC-6 for wall pressures in which the wall is restrained from movement.

The seismic design for MSE walls is similar to that given in AASHTO (1995) with one modification. Rather than using 60 percent of the dynamic horizontal inertial force P_{IR}, the specification states that 50 percent of the dynamic horizontal thrust P_{AE} should be used. Equations for computing P_{AE} and P_{IR} are the same. The specification also provides guidance for computing the inertial force for sloping backfill conditions.

Appendix A11 of the LRFD specifications summarizes the Mononobe-Okabe method of determining wall pressures. This summary is similar to that given in ATC-6.

6.2.3 ATC-32 Design Recommendations

ATC-32 presents a thorough discussion on abutment and foundation design in Section 4.5 and its associated commentary. Abutments, wing walls, and retaining structures are included in the discussion.

The starting point for the ATC-32 discussion is the determination of earth pressures. Caltrans summarizes this determination in the BDS: pressures are determined by Rankine's formula, or "any commonly recognized soil mechanics formula using an active earth pressure coefficient of not less than $K_a = 0.3$, except the maximum load on the heels of wall footings is determined using $K_a = 0.225$." A trapezoidal earth pressure diagram is specified if deflections sufficient to create an active wedge cannot occur. Guidance is also provided for reduction in positive moments for rigid frames carrying fills, for highway traffic within one-half the wall height from the face of the wall or abutment, and the applicable live load when an approach slab is supported on one end of the bridge.

The commentary to ATC-32 notes that most free-standing retaining walls not associated with other structures have performed well during past earthquakes, but that abutments (especially skewed abutments) are highly prone to damage. The drastic difference in performance between retaining and abutment walls is attributed to the fact that free-standing retaining walls are not restricted from movement that relieves the earthquake-induced soil pressure, whereas movement of abutment walls is typically restricted to some degree by the bridge structure. In addition, the inertial load of the bridge forces the abutment wall to move into the backfill soil, creating a passive earth pressure loading condition. This passive pressure is noted as being over 30 times the active pressure, the value for which most retaining walls and abutments are designed.

ATC-32 notes the following basic design requirements for abutments:

- The stiffness of the abutment must be incorporated in the dynamic response analysis of the overall bridge, which should reflect the structural configuration, the load-transfer mechanism from the bridge to the abutment system, and the stiffness and capacity of the wall-soil system.
- The capacity of the abutment to resist the bridge inertial loading must be compatible with the structural design of the abutment wall (i.e., whether part of the wall will be damaged by the design earthquake) as well as the soil resistance that can be mobilized. The soil capacity is to be evaluated based on an applicable passive earth pressure theory.

The ATC-32 suggests that damage to the abutments of Ordinary Bridges be accepted during the safety-evaluation earthquake as long as the abutment damage does not lead to unacceptable performance of the bridge structure. For Important bridges, where the functionality of the bridge must be ensured during and immediately following an earthquake, the structural components of the abutments need to be designed to accommodate the earthquake load or the passive pressure condition that might develop. According to the commentary, guidelines for designing abutments so that they avoid damage during large earthquakes, cannot be prepared until additional research is completed.

ATC-32 provides guidance for characterizing the abutment stiffness as part of the dynamic response analysis of the overall bridge. This guidance includes a valuable summary of the importance of the flexibility of the abutment. For example, it is noted that an 18-inch concrete wall cannot mobilize soil resistance beyond approximately five feet from the point of loading. Consequently, for typical box-girder bridges, the Caltrans practice of limiting the soil resistance to an eight-foot wall height at the back wall is reasonable. For other bridge types (e.g., slab bridges) the commentary notes that a smaller wall height may be appropriate. Additional comments are provided on the extent of displacement required to mobilize passive-pressure, transverse stiffness of wing walls, stiffness of skewed abutments, and stiffness of abutments supported by piles.

For retaining wall design, ATC-32 identifies two basic requirements:

- The overall size and configuration of the retaining structure should be specified to satisfy the overall stability (i.e., sliding and overturning) requirements.
- The structural design of the retaining wall must be able to withstand earth pressure exerted on the wall.

The guidelines note that retaining walls are traditionally designed only for a static active earth pressure, with a factor of safety that varies from 1.5 to 2. This approach assumes that there is sufficient reserve capacity (associated with the factor of safety) to avoid unacceptable earthquake damage. Additionally, excessive soil pressure on the wall can be relieved by a small amount of movement, which is generally of little consequence for retaining walls not associated with other structures. On the other hand, retaining walls very close to nearby structures that can restrain the movement of the wall should be carefully analyzed. If a nearby structure restricts the retaining wall movement, the wall should be designed for higher earth pressure (e.g., at rest or dynamic active pressure).

The ATC-32 guidelines also note that some retaining walls have better seismic performance characteristics than others. For example, crib walls have relatively poor seismic performance, whereas retaining walls that can tolerate ground displacements (e.g., MSE walls) generally perform better during earthquakes. The commentary points out that most of the case histories of retaining wall failure have been associated with clay soils, either as retained fill or as foundation soils, rather than seismic loading *per se*. Given this difference in performance, ATC-32 suggests that typical retaining walls (i.e., less than 30 feet high and not associated with adjacent structures) should be designed using traditional non dynamic procedures using a factor of safety of 1.5 for sandy soils and a factor of safety of 2 for clayey soils. The resource document, ATC-32-1, provides information on active and passive earth pressure theories, including dynamic active pressure theories. Design charts are also provided for determining earth pressure coefficients, and permanent displacements for use in the Newmark sliding block approach.

ATC-32 notes that for the design of retaining walls, the active pressure value represents the minimum earth pressure exerted on the wall, and that there are situations where the earth pressure will exceed the static active earth pressure. In these situations, the static active earth pressure may be adequate for evaluating the overall stability of the retaining wall, but it may not provide an adequate margin of safety for structural design. The commentary recommends that for unusual conditions (i.e., wall height greater than 30 feet or adjacent to other structures), the structural design of the retaining wall be based on other more refined methods or more conservative loading. The alternatives for this situation are either to use the dynamic active earth pressure as described in AASHTO or to use the at-rest pressure condition with an appropriate larger factor of safety.

6.2.4 Other Guidelines

Several other published guidelines address the design of abutment and retaining walls. These include an FHWA document, *Seismic Design of Highway Bridge Foundations* (Lam and Martin, 1986). This document includes a summary of the Mononobe-Okabe method that is virtually the same as that given in ATC-6. However, it goes beyond ATC-6 by suggesting methods for determining the stiffness of integral abutments. (An integral abutment is sometimes referred to as a monolithic or end-diaphragm abutment.) The FHWA document notes that the design of retaining walls for seismic loads generally applies some form of the Mononobe-Okabe procedure for estimating wall pressures. It acknowledges that some amount of wall deformationis considered tolerable, which is one of the key assumptions of the Mononobe-Okabe method.

Whereas various specifications and design guides address abutment wall design, the seismic design of retaining walls is generally not explicitly treated. In these situations, determining the design loading is left to the professional judgment of the foundation engineer. For retaining wall design, as with abutment wall design, it is common to use a seismic coefficient that is lower than the peak ground acceleration.

Depending on soil conditions and wall type, the foundation engineer may reduce the peak ground acceleration by 30 to 70 percent; i.e., 0.7 a_g to 0.3 a_g. While the logic in the reduction is reasonable, the large range in reduction values is probably unreasonable and most likely reflects the lack of published guidelines in this area. Further complicating the design of retaining walls, and specifically the application of the Mononobe-Okabe procedure, is the types of soil retained by the wall. In many cases the soil will have some degree of cohesion. While the principles of the Mononobe-Okabe method can still be applied for soils with a cohesive content, equations are not readily available for these situations, often leading to misuse of the method.

Other documents that present relatively detailed discussions of the design of retaining structures for seismic loading include the ASCE Technical Engineering and Design Guide, *Retaining and Flood Walls* (ASCE, 1994), and the Navy's publication, *The Seismic Design of Waterfront Retaining Structures* (NCEL, 1993).

6.3 Foreign Design Requirements

Abutment and retaining wall design requirements are covered in Eurocode 8, and in the Japanese and New Zealand bridge design codes. Procedures used in each of these codes are similar. For example, each defines wall pressures using the Mononobe-Okabe equations. The methods differ, though, in terms of how the seismic coefficient is used and how water pressures are treated.

6.3.1 Eurocode 8

Part 5 of Eurocode 8 provides in-depth guidance on the design of retaining structures for seismic loading. The general philosophy for design is that the retaining structure should not suffer significant structural damage after the design earthquake. Permanent displacements in the form of combined sliding and tilting are identified as being acceptable if the displacements are compatible with functional and/or aesthetic requirements.

When simplified pseudo-static analyses are conducted, the general approach for computing earth pressures involves use of the Mononobe and Okabe equations,. In the absence of specific studies, the horizontal (k_h) and vertical (k_v) seismic coefficients are defined as the following:

$$k_h = \frac{a_g}{r} \qquad (6\text{-}5)$$

$$k_v = 0.5k_h \qquad (6\text{-}6)$$

where

a_g = peak ground acceleration
r = factor dependent on the type of retaining wall
= 2 for free gravity walls that can accept a displacement equal to or less than 300 a_g (mm)
= 1.5 for free gravity walls that can accept a displacement equal to or less than 200 a_g (mm)
= 1 for reinforced concrete walls, anchored or braced walls, reinforced concrete walls founded on vertical piles, and restrained abutment and retaining walls

The vertical acceleration is taken as upward or downward, whichever produces the most unfavorable effect.

Additional conditions apply to r if soils are liquefiable or walls are greater than 10 meters in height. Specifically, a freefield, one-dimensional analysis of vertically propagating waves can be carried out and a more refined estimate of a_g determined. Guidance is also provided on the location of wall forces, hydrostatic pressures on walls (water moves with soil if permeability less than 5 x 10^{-2} cm/s). Hydrostatic water pressures are added for more pervious conditions using the Westergaard method.

Additional dynamic earth pressure acts on rigid structures that are completely restrained so that an active state cannot develop in the soil and for a vertical wall and horizontal backfill. In these situations, the dynamic earth pressure increment is defined as

$$\Delta P_d = a_g \gamma H^2 \tag{6-7}$$

where

γ = unit weight of soil
H = wall height

The code provides some direction for tie-back and anchored walls. The primary provision is that these walls have enough strength to assure equilibrium of the critical soil wedge under seismic conditions and sufficient capacity to adapt to the seismic deformations of the soil. The length of the anchor for seismic loading is defined as follows:

$$L_e = L_s (1 + 15\, a_g) \tag{6-8}$$

where

L_s = length of anchors required for static loading

It is apparent from this relationship that for highly seismic areas where a_g is greater than 0.3g, the length of the tie-back anchor could be increased by nearly 50 percent.

6.3.2 Japanese Code

Part V of the 1990 Japanese Code includes two sections related to the design of abutments and retaining walls: 1) Section 3.4: "Dynamic Earth Pressure" and 2) Section 3.5: "Hydrodynamic Pressure."

For active pressure conditions, Section 3.4 gives the following equation for computating active earth pressures:

$$p_{EA} = \gamma x\, K_{EA} - 2\, c \sqrt{K_{EA}} + q'\, K_{EA} \tag{6-9}$$

where

p_{EA} = earthquake-induced active earth pressure at depth x
γ = soil unit weight
x = distance from the top of the wall
K_{EA} = seismic active earth pressure coefficient
c = soil cohesion
q' = surcharge load

The value of K_{EA} is determined using the Mononobe-Okabe equation, which is similar to that outlined in the AASHTO specifications. It is interesting to note that in contrast to procedures conventionally given in the United States, the Japanese equation is more complete in that it accounts for the effects of cohesion in the soil and the normal frictional effects. Consistent with the standard Mononobe-Okabe equation, factors such as backfill inclination, wall inclination, and wall friction can be accounted for when calculating K_{EA}.

The corresponding equation for passive earth pressures is also given in Section 3.4 of the Japanese Code. The equation for passive earth pressure is:

$$P_{EP} = \gamma x\, K_{EP} + 2\, c\, \sqrt{K_{EP}} + q'\, K_{EP} \tag{6-10}$$

where

P_{EP}	= earthquake-induced passive earth pressure at depth x
K_{EP}	= seismic passive earth pressure coefficient
γ, x, c, q'	= as defined above

Similar to the active condition, the value of K_{EP} is determined using the Mononobe-Okabe equation, which is the same as that outlined in the AASHTO specifications. Also consistent with the active case, factors such as backfill inclination, wall inclination, and wall friction can be accounted for when calculating K_{EP}.

The determination of K_{EA} and K_{EP} are based on the seismic coefficient described in Section 2 of this report. No mention is made of reducing this coefficient to represent an effective acceleration value (i.e., the peak acceleration is not reduced by a factor of 0.5 to 0.65, as is often done in the United States), thereby implying that the seismic coefficient already represents a more sustained loading condition rather than the peak ground acceleration. Vertical accelerations are assumed to be zero when computing active earth pressures.

Section 3.5 of the Japanese Bridge Code presents an approach for computing hydrodynamic forces on bridges for two cases: (1) free-standing water located on one side of the abutment, as in the case of an abutment wall at a river or embayment, and (2) free-standing water on both sides of a pier or column, as will occur for a pier or column in the center of a river or embayment.

The equation for computing the hydrodynamic force for the case of water on only one side of the wall is given by:

$$P = \frac{7}{12}\, k_h\, \gamma_w\, b\, h^2 \tag{6-11}$$

where

P	= hydrodynamic force
k_h	= seismic coefficient
γ_w	= unit weight of water
b	= abutment width
h	= water depth

This force is applied at a height of 2/5 h above the river or harbor bottom.

For the case of a center pier or column in a river or harbor, the Japanese Code provides equations for estimating the hydrodynamic force for three conditions defined by the ratio of b/h, where b is the pier or column width and h is the water depth:

For $b/h \leq 2.0$

$$P = \frac{3}{4} k_h \gamma_w A_0 h \left(\frac{b}{a}\right)\left(1 - \frac{b}{4h}\right) \tag{6-12}$$

where

P = force
k_h = seismic coefficient
A_0 = projected area
a = width perpendicular to the water pressure

For $2.0 < b/h \leq 4.0$

$$P = \frac{3}{4} k_h \gamma_w A_0 h \left(\frac{b}{a}\right)\left(0.7 - \frac{b}{10h}\right) \tag{6-13}$$

For $b/h > 4.0$

$$P = \frac{9}{40} k_h \gamma_w A_0 h \left(\frac{b}{a}\right) \tag{6-14}$$

This force is applied at a height of 3h/7 above the river or harbor bottom. On one side of the column or pier this dynamic force value is added to the hydrostatic force value. On the opposite side, the dynamic force is subtracted from the static force. These provisions for water forces are not applied at locations where soils are very soft or where soil will liquefy.

6.3.3 Transit New Zealand

Transit New Zealand (TNZ) follows a pseudo-static procedure for evaluating the response of abutments and free-standing retaining walls. Wing walls are considered part of the abutment if they are integral with it.

The values of peak horizontal ground acceleration and velocity to be used for computing inertial forces and wall displacement values in the TNZ codes are given by the following equations:

$$C_0 = 0.25\, Z R \tag{6-15}$$

$$v_0 = 0.36\, Z R \tag{6-16}$$

where

C_0 = peak ground acceleration coefficient (g)
v_0 = peak ground velocity (m/s)
Z = zone factor
R = risk factor

Vertical acceleration is neglected in the computation. As noted in Section 2 of this report, the zone factor Z ranges from a minimum value of 0.6 to a maximum value of 1.2. The risk factors for retaining walls are given in Table 6-1.

From a structural standpoint, the TNZ code requires that all structural components of the abutment or wall must have a dependable strength not less than the forces calculated using the relevant strength method load combinations specified in the code. The wall has to be checked for stability subject to the appropriate load combinations with a strength-reduction factor for the soil not exceeding 0.9. In cases in which the wall is designed to sustain permanent displacements during the earthquake, the load factor can be taken as unity.

Table 6-1 TNZ Minimum Allowable Risk Factors for Abutment and Retaining Walls

Importance Category	*Risk Factor R*
Abutment walls for bridges carrying more than 2,500 vehicles per day (vpd), for bridges over or under motorways or railways, or for bridges on national State highways	1.3
Abutment walls for bridges carrying between 250 and 2,500 vpd, or abutment walls for bridges on provincial State highways if not in above category. Retaining walls supporting roadways carrying more than 2,500 vpd, providing protection or support to motorways or railways, supporting national State highways, or providing protection to adjacent property, the consequential reinstatement cost of which would exceed NZ$450,000[1]	1.15
Retaining walls supporting roadways carrying between 250 and 2500 vpd, providing support to provincial State highways (if not in above category), or providing protection to adjacent property, the consequential reinstatement cost of which would be less than NZ$400,000. Retaining walls exceeding 3 m in height.	1.0
Abutments for bridges carrying less than 250 vpd or for nonpermanent bridges	0.9
Other wall not covered above - specific seismic design not required	N/A

1: Values quoted are referenced to a New Zealand Producers Price Index and are not corrected to the current index.

Earth pressure forces are accounted for in the TNZ design approach. Three forces are used: 1) static earth pressure force P_s, including compaction effects where applicable; 2) the increment (or decrement) in earth pressure ΔP_e due to seismic effects; and 3) the increment of force P_f on the wall due to its displacement towards the static backfill. When assessing earth pressure effects, the relative stiffnesses of the wall, backfill, foundations, and any tie-back anchors must be accounted for.

Structural inertial forces are also included in design. These include the inertial force P_I on the abutment or wall due to ground acceleration acting on the wall, including the soil block above the heel of the wall, the inertial force C_0W on a locked-in superstructure, and the force P_B transmitted between the superstructure and the abutment by the bearings and any force-limiting device. The force transmitted by sliding bearings is based on the maximum likely friction coefficient, 0.15, unless another value can be justified. The force due to other bearings is determined from the product of the total support stiffness and the seismic displacement Δ with the calculation of Δ taking into account the relative stiffness of the various supports and the relative stiffness of the abutment bearings and foundations.

The structural components of the abutments are designed to remain elastic under all design loads (including seismic).

Free-standing walls are designed so that one of two conditions is met.

- The wall remains elastic and does not undergo any permanent displacement under the design earthquake loading.
- Limited permanent outward displacement due to soil deformation is accepted and the wall is designed to avoid yielding of the structural elements. In this case, provisions must be made to accommodate the calculated displacement with minimal damage and without encroaching on

clearances. Walls other than those on piles are designed to slide rather than rotate. Dependable resistance to overturning (using a strength-reduction factor of 0.8) must be greater than the overturning moment derived from the upper-bound force required to cause sliding. Walls designed for significant permanent outward movement that requires yielding of the wall stem or piles must be designed to remain elastic up to at least 0.6 times the design load.

Tie-back walls are also addressed in the TNZ Code. These walls must be designed to remain elastic under seismic loading. During design, consideration must be given to the consequences of tie and wall flexibility under design conditions. Acceptable wall displacements can be used to reduce design loading. Sudden failure of the wall must be avoided. Postearthquake effectiveness of the corrosion protection system of the ties must be ensured.

In the TNZ Code, MSE walls that are intended to undergo permanent displacement under seismic loading must be designed so that the outward movement results from pullout or ductile extension of the reinforcing strips. Where design is for pullout, the ideal strength of the connection must be at least twice the pullout force calculated from the probable coefficient of friction. Upper and lower bounds on the threshold acceleration required to produce incipient failure are calculated by considering the ties acting both horizontally and along the failure surface and by allowing for probable variations in the pullout resistance and yield strength of the ties. Stability is checked under the upper-bound acceleration. If the wall is required to avoid permanent displacements, it must be designed to remain elastic and stable under the design loading.

The TNZ guideline also provides some direction for the design of culverts or similar small "locked-in" structures. Seismic design is not required for structures with a maximum cross-section dimension of less than three meters. Reinforcement detailing must provide tolerance to ground deformation, with particular attention to corner details and bar terminations. For structures with maximum cross-section dimensions of three meters or more and where the soil cover is less than the height of the structure, rigid structures are designed for ΔP_e. According to TNZ, flexible structures such as steel plate culverts can be assumed to be capable of deforming so that the soil pressures remain uniform around the periphery.

6.4 Special Design Considerations

A summary of design philosophies and requirements for retaining walls and abutments for codes of different countries is given in Table 6-2. Comparing these code requirements and a number of research projects, it is apparent that a dynamic increment of pressure added to the static wall pressure has become a fairly common approach for the design of abutments and *vital* retaining walls. This requirement is normally handled through use of the Mononobe-Okabe method.

For conventional cantilevered or gravity retaining wall design, the use of an added horizontal force creates two major design problems:

- *Overturning.* The ratio of the base dimension to retaining wall height for a statically designed wall is usually 0.65 to 1. Due to the additional dynamic load (i.e., dynamic increment), this ratio will have to be increased to approximately one (1) in order to keep the vertical resultant within the center half of the base and maintain a reasonable factor of safety.
  Meeting this requirement imposes significant burdens on the design. An alternative approach would be to relax the normal requirements for maintaining the resultant in the center half of the foundation base, as long as it can be demonstrated that stability will be maintained with an adequate margin of safety. Earth Mechanics Inc. points out in a progress report to NCEER (EMI, 1994) that the overturning moment requirements for many bridge configurations (diaphragm or seat-type abutments) may not be an overriding issue for many abutment foundation systems. Nevertheless, additional design guidance appears to be warranted in this area.
- *Sliding.* Sliding resistance usually relies on two components: friction and passive pressure. The frictional force developed at the base of a wall by the gravitational weight of the wall will be reduced

Table 6-2 Comparison of Abutment and Retaining Wall Specifications

PROVISIONS			AASHTO 16th Ed., Div. I-A	AASHTO LRFD	ATC-32	TCA	New Zealand	Eurocode 8	Japan
10. Abutments									
Lateral Load	Static	Soil Pressure	Free Standing Type Active Pressure Monolithic Type Active Pressure	Active Pressure and At-rest condition.	At-rest earth pressure for rigid foundation. Active earth pressure for yielding foundation.	(Not in Study)	Active earth pressure.	Earth pressure according to Eurocode 7. Information not available.	Active earth pressure.
Lateral Load	Dynamic	Soil Pressure	(Same as above) Seismic earth pressure. Passive pressure theory required.	(Same as above) Seismic earth pressure. Passive pressure theory required.	The pseudo-static seismic earth pressure. Passive pressure theory required.	(Not in Study)	Active earth pressure. Increment or decrement pressure due to earthquake. Increment passive pressure.	Earth pressure including seismic effects determined according to Eurocode 7. Passive pressure theory required.	Active earth pressure. Seismic earth pressure. Passive pressure design required.
Lateral Load	Dynamic	Normal to Plane (Longitudinal)	Inertial load from wall end bridge structure.	Inertial load from wall and bridge structure. (Modification factor R use in different zones.)	Inertial load from wall and bridge structure.	For Lower Level Design Earthquake: Avoid engagement of abutment backward and end span.	Inertial load from wall and bridge structure. (All structural components shall be designed to remain elastic under all loads).	Inertial force of wall and bridge structure. (All components of abutment shall be designed to remain elastic under the seismic action).	Inertial load from wall and bridge structure.
Lateral Load	Dynamic	Parallel to Plane (Transverse)	(Same as above)	(Same as above)	Inertial load from wall and bridge structure.	For Upper Level Design Earthquake: Limits on the displacement of end span are provided to avoid significant damage.	(Same as above)	(Same as above)	(Same as above)
Vertical Load	Static		Bridge load. (Group Load VII)	Bridge Load and wall weight.	Bridge Load (Group Load VII)	(Not in study)	Bridge Load	Bridge Load and wall weight.	Bridge Load and wall weight.
Vertical Load	Dynamic		Bridge Load (Group Load VII) (Vertical acceleration omitted for Free-Standing Wall)	(Same as above)	Bridge Load (Group Load VII)	(Not in study)	Bridge Load (R-factor for Important Bridge Category).	(Same as above)	(Same as above)

Table 6-2 Comparison of Abutment and Retaining Wall Specifications (continued)

PROVISIONS			AASHTO 16th Ed., Div. I-A	AASHTO LRFD	ATC-32	TCA	New Zealand	Eurocode 8	Japan
11. Retaining Wall (Wing Wall)									
Lateral Load	Static	Soil Pressure	(See Abutment Requirements).	(See Abutment Requirements).	Active earth pressure for yielding foundation.	(Not in Study)	Active earth pressure. (Tieback wall to be designed to remain elastic under seismic loadings).	(See Abutment Requirements).	Active earth pressure.
	Dynamic	Soil Pressure	(See Abutment Requirements except passive pressure).	(See Abutment Requirements except passive pressure).	Dynamic earth pressure effects.	(Not in Study)	Active earth pressure. Increment pressure due to earthquake.	(See Abutment Requirements except passive pressure).	(See Abutment Requirements except passive pressure).
		Normal to Plane (Longitudinal)	Inertial load from wall.	Inertial load from wall.	Inertial load from wall.	(Not in Study)	Inertial load from wall. (Walls are allowed to be designed yield or not yield condition).	Inertial load from wall.	Inertial force of wall.
		Parallel to Plane (Transverse)	Not required.	Not required.	Not required.	(Not in Study)	Not required.	Not required.	Not required.
Vertical Load	Static		Group Load I thru IV.	Wall weight	Group Load I thru IV	(Not in study)	Wall weight	Wall weight	Wall weight
	Dynamic		Group Load VII	Wall weight	Group Load I thru VII	(Not in study)	Wall weight	Wall weight	Wall weight

during an earthquake. by the vertical component of ground acceleration. Specific guidance on applying the vertical component of ground acceleration for gravity walls appears to be needed. It should be noted that if the structure is supported by a pile foundation, friction at the base of the foundation should not be used, to allow for the possibility that the soil settles away from the foundation base as the piles maintain the vertical position of the foundation.
Passive resistance is developed at the face of a gravity wall during seismic loading. In situations in which the face is unable to develop sufficient resistance, it is common practice to use a shear key to increase the horizontal resistance. Increases in horizontal force will require the wall to have a *deeper* key to meet a reasonable factor of safety against sliding. While this analysis appears to be relatively straightforward, at least from the standpoint of limiting equilibrium, careful consideration must be given during design to the manner in which each component (i.e., face versus shear key) is developed. EMI (1994) suggests that it may be advantageous to allow the shear key to break in order to take full advantage of soil capacities at the abutment in some situations. Additional guidance appears to be warranted in this area. If the structure is supported on piles, the piles provide additional lateral resistance to seismic forces. The designer must also determine the manner in which passive resistance is developed for the wall and the piles, as a function of deformation.

It was also apparent from this review that most codes and research projects discuss only the nonrestrained type of retaining structures such as cantilevered or gravity retaining walls. It is not an easy task for the designer to conceive an economical and constructable solution to meet the dynamic pressure requirements using only these types of walls. The "two-level design concept" especially requires that all elements maintain function in the lower-level earthquake and suffer damage without collapsing in the upper-level event. These stricter performance criteria have led to designers using alternative walls to meet seismic loading requirements. Unfortunately, codes and guidelines are less specific for these wall types. The tendency is to apply conventional design methods such as the Mononobe-Okabe equations, irrespective of assumptions under which the conventional methods were developed. In more than one case, failure to adequately consider the assumptions associated with these simplified methods has led to design approaches that are not conservative.

Another challenge that must be met during seismic design of abutments and retaining structures is appropriate design methods to use for hybrid wall systems. These hybrid systems have been introduced to provide cost-effective, constructable walls. For example, permanent tie-backs are used where conventional cantilevered walls cannot be physically or economically built (see Figure 6-1). These walls are attractive because they can be designed to develop increased resistance to loading during earthquakes. In recent years, a variety of other retaining wall systems and innovative construction procedures have been developed to offer improved resistance to seismic loading, for example "nailed" walls and MSE walls. Design procedures for these hybrid wall systems are not normally covered in codes and guidelines. This means that the foundation design consultant must decide what design methods are applicable. Unfortunately, in today's commodity-driven approach to design, budgets are often insufficient to give more than a rudimentary treatment of the actual response mechanism.

From this review of existing codes and guidelines, it is apparent that abutment design requires more attention in the following areas:

- Displacement of abutments in the plane of and normal to the wall. These displacement values must be limited to acceptable amounts.
- The connection between the bridge and superstructure and the abutment. These connections must be carefully detailed so that damage (e.g., pounding, pull-away, and relative displacement) is minimal and no collapse occurs.

As for future code development for abutment and *vital* retaining wall design, the following topics require attention:

- Codes should include a list of approved and acceptable wall systems. For example, ATC-6 lists a few acceptable systems to be used in designing abutments and wing walls. Such a list will streamline the design and approval process. A conventional, single-wall system may not be the only solution for a state-of-the-art design procedure.

- Seismic design procedures (e.g., dynamic or pseudo-static) should apply to bridge abutments and *vital* retaining walls. It may be possible to relax or waive seismic design procedures for ordinary walls as long as factors of safety applied during static design of the wall are greater than 1.5 to 2.
- Research projects and field tests should be directed towards hybrid systems of retaining structures. This research will be of benefit for designing new structures, as well as for retrofitting projects.

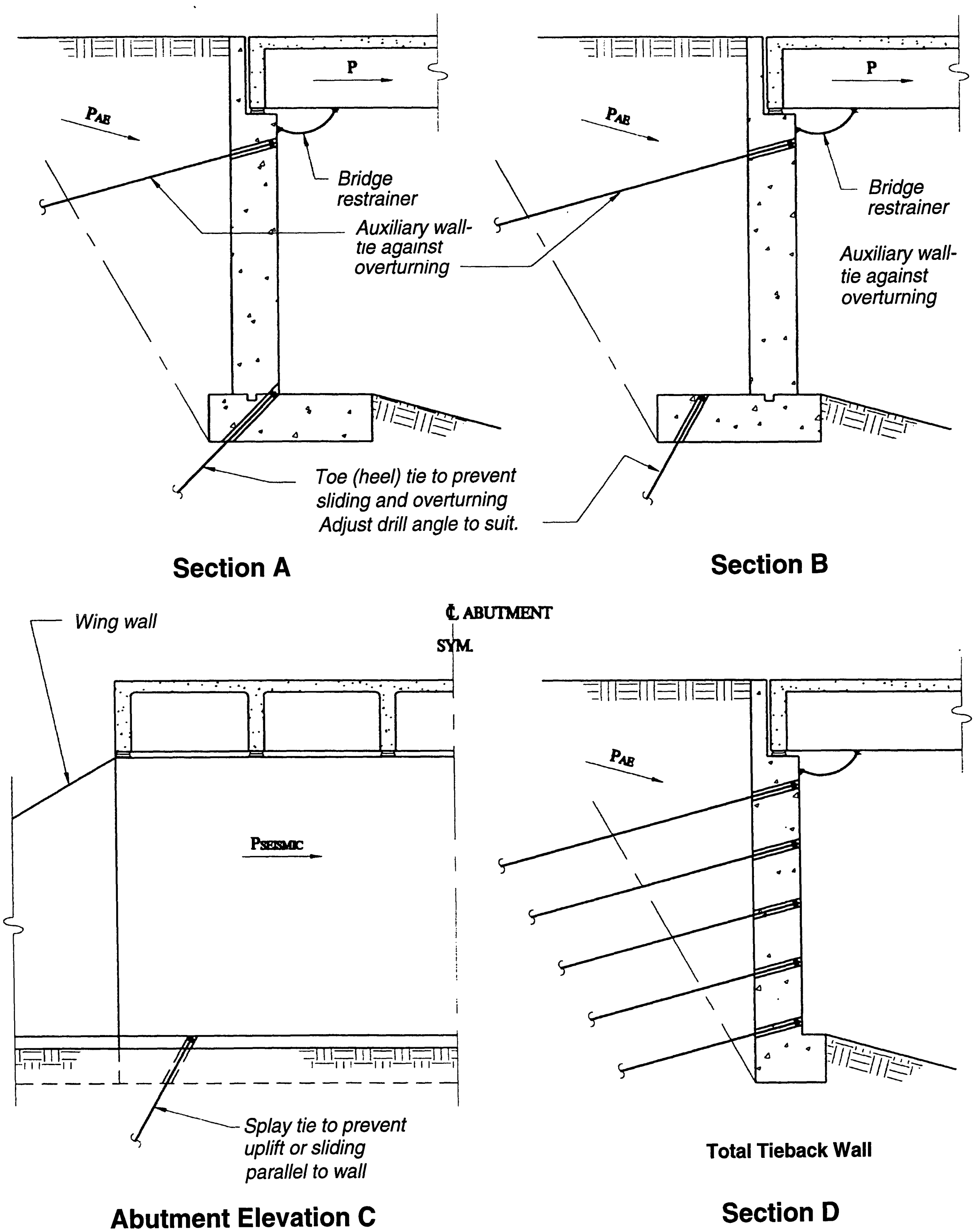


Figure 6-1 Sample tie-back wall designs.

SECTION 7
U.S. AND FOREIGN TUNNEL REQUIREMENTS

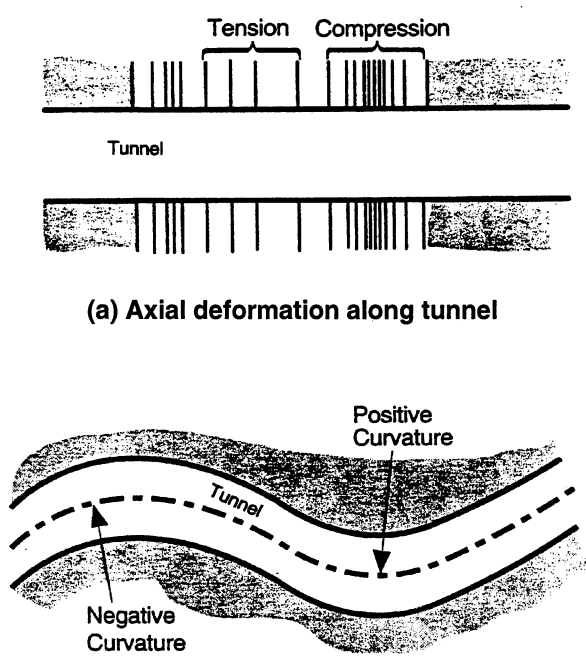
This section covers both U.S. and foreign seismic design requirements for tunnels. U.S. requirements are presented in Section 7.1 and foreign requirements in Section 7.2.

7.1 Current U.S. Seismic Design Procedures For Tunnels

There currently is no U.S. code for seismic design of tunnels. However, noncodified seismic design procedures have been developed and used successfully since the mid-1960s. Examples of the successful tunnel design include the BART system in the San Francisco Bay area, which survived the 1989 Loma Prieta event with no significant damage, and the LA Metro system in Los Angeles, which survived the 1994 Northridge event with no significant damage. This good performance is not surprising since buried tunnels typically sustain much less damage than above-ground structures when subjected to seismic loading.

This section summarizes current U.S. tunnel design procedures. It is based on a report titled "Seismic Design of Tunnels" (Wang, 1993). The procedure is based on a two-level design approach. The lower-level event, the Operating Design Earthquake (ODE) is reasonably expected to occur at least once during the design life of the facility. The performance criterion for this lower-level event is that the system remain operational during and after the ODE. The upper-level event, the Maximum Design Earthquake (MDE) has a small probability of exceedance during the facility life (e.g., 5 percent). The performance criterion for this upper-level event is that public safety be maintained during and after the MDE.

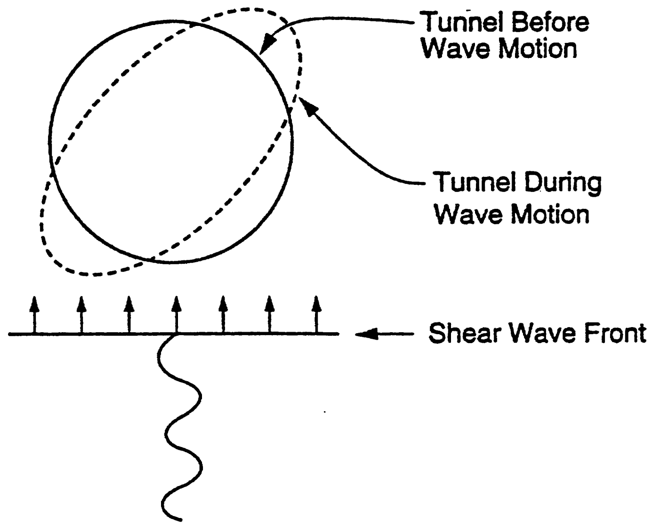
There are two primary considerations for each design event: axial and curvature deformation, and racking or ovaling deformation. Axial and curvature deformation refer to longitudinal strain and bending of the running line tunnel due to seismic wave propagation effects in a horizontal plane, as shown in Figure 7-1. Racking and ovaling deformation refers to shearing of the tunnel or station cross-section in a vertical plane due to vertical propagation of seismic waves, as shown in Figure 7-2. Note that for both types of response, a deformation or strain is applied to the tunnel as opposed to a load.



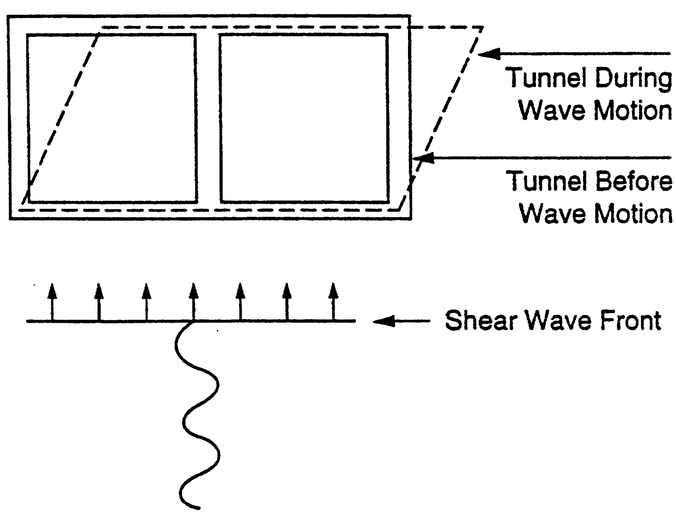
(a) Axial deformation along tunnel

(b) Curvature deformation along tunnel

Figure 7-1 Axial and curvature deformation of a tunnel (after Wang, 1993).



(a) Ovaling deformation of a circular cross section



(b) Racking deformation of a rectangular cross section

Figure 7-2 Racking and ovaling deformation of a tunnel (after Wang, 1993).

For both types of response, various levels of analysis are described. For axial and curvature deformation, the simplest level of analysis assumes that the tunnel axial strain exactly matches that for the freefield ground. For example, for Rayleigh wave propagation, the maximum deformation occurs when the direction of wave travel is parallel to the longitudinal axis of the tunnel. In that case, the axial strain, ε, and the curvature $1/r$ become

$$\varepsilon = \frac{V_r}{C_r} \tag{7-1}$$

and

$$\frac{1}{r} = A_r C_r^2 \tag{7-2}$$

where

V_r = peak particle velocity for the Rayleigh wave
C_r = effective propagation velocity for the Rayleigh wave
A_r = peak particle acceleration for the Rayleigh wave

In the more advanced level of analysis, the tunnel strain is given by the freefield ground strain multiplied by a reduction factor. The reduction factor is based upon a beam-on-elastic-foundation type analysis. For axial strain effects, it is a function of the relative stiffness of the soil/rock medium with respect to the longitudinal stiffness (i.e., modulus of elasticity, (E) times area, (A)) of the tunnel itself. The reduction factor is close to one (i.e., little reduction) if the soil/rock is very stiff compared to the tunnel while the reduction factor is significantly less than one (i.e., large reduction) if the tunnel is stiff compared to the soil/rock medium. In addition, an upper limit, based upon the maximum friction force per unit length at the tunnel/medium interface, is applied to the axial force on the tunnel.

For racking or ovaling deformation, the simplest level of analysis assumes that the tunnel racking strain (i.e., relative lateral displacement of the top of the tunnel with respect to the bottom, divided by the tunnel height) is comparable to that of the freefield ground. That is, for a circular tunnel, the maximum change in the tunnel diameter ΔD is given by

$$\Delta D = \frac{\gamma}{2D} \tag{7-3}$$

where

D = tunnel diameter
γ = maximum freefield shear strain due to vertically propagating shear waves

$$\gamma = \frac{V_s}{C_s} \tag{7-4}$$

where

V_s = peak particle velocity due to shear waves
C_s = effective propagation velocity due to shear waves

In the more advanced level of analysis, the tunnel strain is given by the freefield shear strain multiplied by a modification factor. The modification factor is greater or less than one depending on the relative in-plane flexural stiffness of the tunnel (i.e., EI) with respect to the soil it replaced. If the tunnel is stiff, the modification factor is small (i.e., <1.0). If the tunnel matches the soil, the modification factor equals one. If the tunnel is more flexible than the soil, the modification factor is large (i.e., >1.0). For a very flexible tunnel, the modification factor is such that the tunnel racking response matches that for the freefield ground with a "tunnel-size" hole. Graphs are provided to facilitate evaluation of the racking modification factor.

Finally, current U.S. procedures also address special considerations such as unstable ground, faulting and abrupt changes in structural stiffness by suggesting various mitigation measures. For unstable ground (including ground susceptible to landslides and/or liquefaction) the suggested mitigation measures are rerouting, removal and replacement of problem soil, or ground stabilization.

7.2 Foreign Code Requirements

This section presents a brief review of tunnel and "tunnel-related" provisions in selected foreign codes. The tunnel-related provisions are liquefaction, fault crossing, and seat width requirements. Both liquefaction and fault crossing are potential hazards for tunnels. In some cases, seat width requirements are based on seismic wave propagation concepts that have direct application to axial and curvature deformation in tunnels.

7.2.1 Eurocode

Tunnels are not formally addressed in Part 2, Bridges. However, there is a fairly extensive section on spatial variability involving spectral density functions that appears intended for the analysis of above-ground "inertia" loading (i.e., not wave propagation effects).

In Section 6.6.4 there is a more direct application of wave propagation concepts in relation to "minimum overlap lengths" (minimum seat widths). Procedures for estimating the traveling wave contribution to minimum seat-width are presented: peak ground velocity as a function of A_{max} for various soil types as well as prescriptive values for the propagation velocity.

In Section 2.4, Conceptual Design, the potential for fault movement and conceptual design approaches to deal with this movement (i.e., adequate flexibility) are noted. The same section mentions potential liquefaction problems, which are to be investigated according to Eurocode 8: Part 5.

7.2.2 New Zealand Code

Tunnel-like structures are addressed in this code. No special earthquake analysis is required if the maximum cross-sectional dimension is less than three meters. For larger structures, the design loads are a function of the soil cover, as shown in Figure 7-3.

In the figure,

W_s = static force due to weight of soil above the structure
P_s = static earth pressure force, including compaction force where appropriate
ΔP_E = increment ($\pm$) in earth pressure force due to the earthquake.

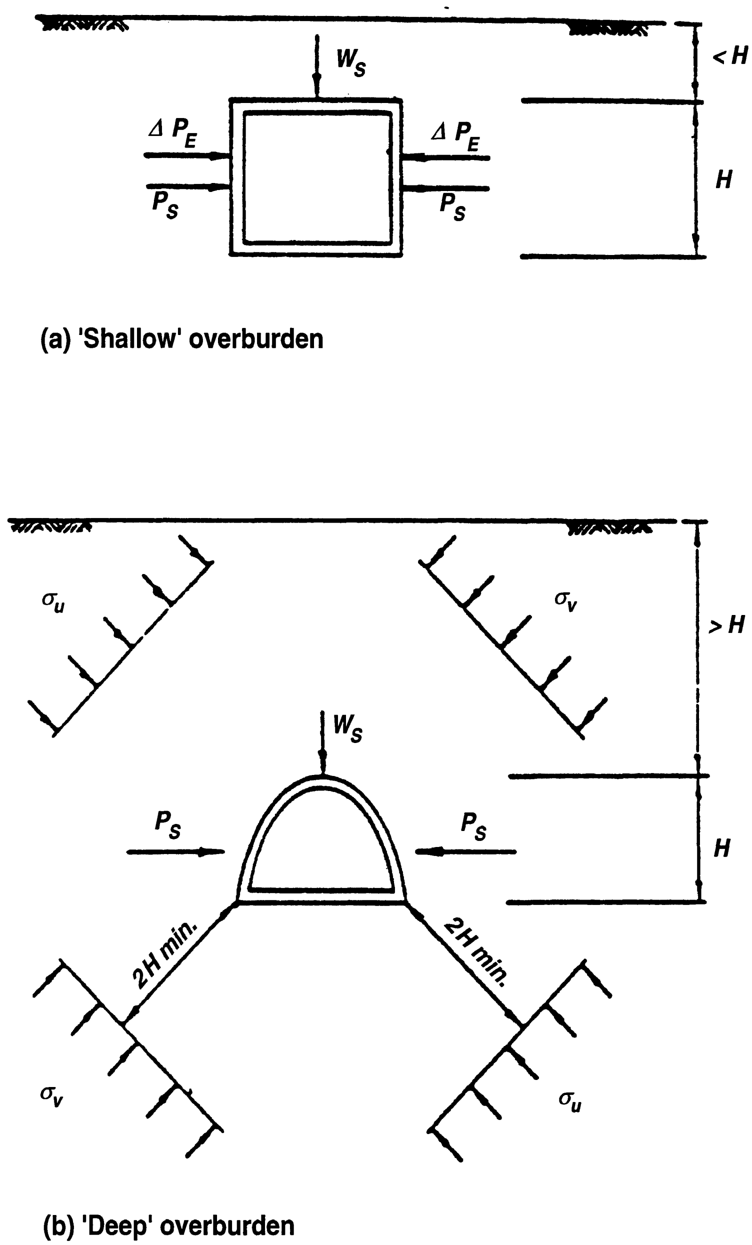
Procedures for estimating these forces and the equivalent stress values σ_u and σ_v are not identified, but are given in NZNSEE (1980).

Spatial variability of ground motion is not mentioned specifically in the code. However, Section 5.3.5 requires that allowance for "out-of-phase" response at movement joints be made, while Section 5.5.3 presents semi-prescriptive requirements for overlaps (i.e., seat widths). Apparently more detail is provided in Section 3.

The Manual requires that "recognition" be given to bridges over active faults or on liquefiable soils (see Section 5.1.1). Sections 5.2.1 and 5.4.6 identify requirements or assumptions for sites with liquefiable soils.

7.2.3 Japan

In Kawashima's 1990 article on the Japanese Bridge Code, no specific mention is made of tunnels, faulting, or spatial variation of ground motion. There is an extensive discussion of evaluating liquefaction potential, and the implications of liquefaction for design. Prescriptive requirements for seat length are given as a function of span length.



σ_u *and* σ_v **are equivalent static stresses from seismic compression wave.**

Figure 7-3 Tunnel design loads in New Zealand Code.

Although not mentioned in Kawashima's paper, the Japanese do have seismic design procedures for tunnels. For example, the Kobe Rapid Transit Railway (KRTR) was designed following procedures derived from the Japan Railway and Tokyo Metropolitan Subway Corp. In the KRTR design, dynamic earth pressures (i.e., loads) were considered, using a Mononobe-Okabe analysis.

In a sense, the Japanese procedures are similar to those in the New Zealand Code in that they both specify a load, in contrast to the current U.S. procedures, which specify a deformation.

7.3 Comparison Of Approaches With Observed Behavior

As noted previously, buried tunnels (whether designed for earthquakes or not) have performed well in past earthquakes. In this regard, damage to subway stations and running line tunnels in the 1995 Kobe earthquake is particularly important in that it provides a benchmark as to which design approach (i.e., load or deformation) may be the most effective.

The KRTR facilities in Kobe are reinforced concrete rectangular box structures constructed by cut-and-cover techniques. Postearthquake inspections indicated racking of the box with flexural failure at the joint between the exterior wall and the base slab. In addition, flexural failures were observed at the bottom of the central support columns (i.e., at the joint with the base slab). This behavior is consistent with a racking deformation of the tunnel due to vertical propagation of seismic waves.

Hence, the Kobe case history suggests that the current U.S. procedures in which racking deformation is explicitly considered may be a more effective approach than current foreign procedures that specify a load or pressure on the tunnel walls. In other words, the earthquake loads for shallow overburden given in Figure 7-3 would not result in racking deformation of a tunnel, which appears to have been the damage mechanism for the KRTR facilities.

SECTION 8
TWO-LEVEL DESIGN APPROACH

After assessing current code requirements and evaluating the direction of future design requirements, the ATC-18 Project Engineering Panel (PEP) believes that a two-level design approach has some significant benefits. Such an approach should produce designs that have higher levels of reliability for meeting desired performance requirements.

A two-level design approach has been used by the nuclear industry since the mid-1970's. The American Petroleum Institute (API) has used two-level criteria for offshore platforms since the early 1980s. The Port of Los Angeles has been using two-level criteria since the late 1980s.

A two-level design approach was first considered in the early 1970s for buildings. An extensive study performed by Applied Technology Council (ATC, 1974) titled *An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings* (ATC-2), produced two-level design examples that were intended to form the basis for the next generation of building codes. In fact, this did not occur and the ATC-3 (ATC, 1978) recommendations that followed the ATC-2 study were based on a single-level design approach. It was not until 1986 that a two-level design approach was included in U.S. building design codes. It was part of the Tri Services Manual for Building Design (DoD, 1986) which is still in use today.

The first U.S. bridge design code to embody a two-level design approach was that adopted by the Transportation Corridor Agencies (TCA) in Southern California in the early 1990s. Since these toll roads and bridges were the first in California to be financed by commercial bonds, TCA believed that a higher level of performance was necessary to secure the bonds. The two-level design approach used by TCA is summarized in Section 3.4 of this report. In the ATC-32 project (Section 3.3), a two-level design approach has been adopted for Important Bridges. However the current one-level approach will continue to be used for the majority of bridges.

The ATC-18 PEP recommends that future codes include a two-level design approach as a minimum for important bridges. As a consequence, the pros and cons of a two-level design approach were enumerated by the ATC-18 PEP in order to focus the debate by groups that will be responsible for developing future bridge codes. This assessment includes feedback from designers that are currently using the TCA two-level design approach discussed in Section 3.4.

8.1 TCA Experience With the Two-Level Design Approach

The experience of bridge designers on the Orange County Toll Road projects using the TCA two-level design criteria is summarized by the following points (oral commun., S. Gloyd, TCA chief Bridge Engineer):

Construction and Cost Impact

- The lower-level design event (72 year return period) was fairly easily incorporated in the design, both with regard to design procedure and construction cost. This allowed TCA to consider the 150-year return period event as the lower-level design event for later portions of the project as shown in Table 3-8.
- The construction cost impact of the TCA two-level design criteria has not been a problem. They have probably had less of an impact than did the additional reinforcement for joint shear and superstructure strength exceeding column plastic moment that Caltrans has required during this same period.
- The use of the two-level criteria has been successful in helping TCA deal with bond financing issues.

Ease of Use

- Designer attitude varied, but with workshops and a little experience, the two-level design practice was learned rather easily.
- The more progressive designers welcomed the two-level design approach. Some commented that it gave them a better feel for the way their bridge would respond. Others who prefer to stay very close to Caltrans practice, or are dependent on it, were less inclined to explore the issues, but were generally cooperative. There were a few complaints about the additional engineering work involved.

Effects on Designs and Detailing

- So far, there have been no major changes in bridge configuration caused by the two-level design approach. With the exception that a few columns were larger than otherwise would have been required, no adjustments to bridge type, fixity, or geometry occurred. In the latest TCA criteria, the lower-level design event controls the longitudinal reinforcement in columns and the upperlevel event controls the lateral reinforcement.
- Abutments seem to be a problem area for design, requiring that assumptions be made regarding modeling and limitations for interaction, capacity, fusing, and other factors. The abutment participation in the overall seismic response is not as well understood or defined as column participation, yet it can be a very significant element for shorter bridges. This is a problem area for either single- or two-level design, but the two-level criteria require more decisions regarding abutment participation, so it becomes more noticeable in this approach.
- Displacement criteria were not established for the two-level design, but some bridge designs were more flexible than TCA felt comfortable with. Some limitations to both structure and foundation displacement would be logical and helpful.
- In attempting to provide a large enough gap at seat-type abutments to avoid closure during the lower-level event, expansion joints and bearings sometimes need to be designed for overtravel, as compared to the usual Caltrans criteria that disregard earthquakes in sizing joints and bearings. This created a "policy" conflict where Caltrans tries to avoid modular type expansion joints. This conflict was resolved by using larger glands in "strip seal" joints and considering them expendable if displacement exceeds their capacity.

Looking ahead, with the use of displacement capacity as the primary factor for evaluating performance at the upper-end event, the two-level design approach provides an advantage by assuring that there is also a strength design for seismic loading. However, conflicts may develop between the two: the lower-level design tends to call for larger columns and/or more vertical reinforcement in columns and the upper-level displacement ductility goals may then require transverse reinforcement in the columns beyond practical limits. On the other hand, a single-level design, focused on displacement capacity may have such a low threshold of damage that it becomes a maintenance problem after moderate earthquakes.

8.2 Advantages and Disadvantages Of Two-Level Design Approach For Bridges

8.2.1 Advantages

- A major advantage of the two-level approach is that it directly addresses safety and functional performance separately. This better assures that the performance goals are met, especially since the performance criteria are such that either of the levels could control design.
- A two-level design procedure helps to insure that the risk of failing to meet both safety and functional requirements is consistent in areas of different seismic activity.
- Two-level design that includes both force and displacement requirements is a logical way to approach the design of a bridge. Some structural elements such as column longitudinal steel and sacrificial shear keys are best sized using a force-design approach at the lower level, whereas other performance issues such as ductility and abutment movement are better addressed through a

displacement design approach at the upper level. A two-level design procedure also helps the designer focus on what is really important for the performance of the structure.

- By having a logical two-level design approach that does not bury one set of requirements within a single-level design method, future advancements in engineering knowledge can be more easily and understandably incorporated into subsequent improvements of the design criteria. For example a great deal of progress toward understanding the seismic behavior of bridges has been made in recent years. This progress, plus the availability of related computer tools, now makes it more feasible to directly consider displacement response and performance. This ability is the key to developing a two-level design approach.
- The two-level approach could be useful in accounting for the difference between the ground motions implied by the two levels of inputs, which varies throughout the country.
- The two-level design approach can help target effort and expenditure of funds to ensure that critical structures are designed to suffer less damage under a given level of ground motion. It might also allow the designer (and sponsor) to quantify the cost of providing a higher level of performance.

8.2.2 Disadvantages

- The use of a two-level approach involves extra design effort and more design documentation. In general, a specification that requires increased design effort should be based on one of two criteria:
  - Increased design effort justifies lower margins of safety and thus, less material,
    or
  - Current levels design effort lead to deficient performance.
- Seismic hazard maps suitable for a lower level design (e.g., 72 to 150 yr.) event do not exist on a nationwide basis, and the seismicity of the eastern United States is not well understood.
- There are still many unknowns with respect to seismic design. With our current level of knowledge, fine-tuning seismic design procedures may create a false sense of security.
- Designing for a specific level of earthquake performance must be translated into specific design requirements in order to achieve the given levels of performance. This is a difficult task due to all the inherent uncertainties in the design process.
- There is no general agreement on the service life of a bridge, although 50 years is often stated. More bridges have been replaced because of functional obsolescence than for structural deterioration (e.g., for narrow width rather than lack of load capacity). In general, the larger or more expensive the bridge the longer the service life should be. However, it is difficult to predict functional requirements far into the future.
- Many two-level criteria call for "immediate functionality" following the lower-level earthquake. However, there is no general agreement as to what constitutes immediate functionality for a given structure. Does this mean that it is usable with no emergency repairs? With emergency repairs that can be accomplished within 24 hours? The required traffic level immediately following the earthquake needs to be defined. Is it sufficient that the bridge can be used by a single emergency vehicle? Limited traffic? Normal traffic? Perhaps these problems could be handled by making a general statement in the code and requiring the agency with jurisdiction over the bridge (i.e., the funding agency) to set the specific performance parameters. This is assuming that critical performance parameters can be identified and quantified.

8.3 Two-Level Design Approach For Foundations

The behavior of soil changes as the level and duration of earthquake loading changes. For saturated, cohesionless soil, the combination of a high acceleration level and a relatively long duration of earthquake loading can lead to liquefaction and dynamic settlement. In all soils, the stiffness decreases and the material damping increases as shearing strains caused by ground acceleration increase. Consequently, implicit in a two-level design procedure is the need to consider how the soil will behave under both levels of loading. The design ground motions for the two design earthquakes differ in terms of duration, frequency content, and amplitude.

8.3.1 Advantages

- The primary benefit of the two-level approach to geotechnical design of foundations is that soil behavior during the lower-level event is likely to be better for many sites. Liquefaction potential will be lower, slope deformations will generally be small-to-negligible, and nonlinear soil behavior that could affect soil-structure interaction will generally be negligible. Using the lower-level earthquake motions can eliminate the need for costly ground improvement or special structural design procedures for bridges that are not essential.
- The two-level approach to design provides a rational basis for deciding what form of ground modification or structural system should be used to achieve satisfactory bridge performance.
- The two-level design approach provides a rational basis for deciding the geotechnical conditions for temporary construction or other uses of limited duration.
- The two-level approach is generally well accepted within industry and government agencies. Geotechnical engineers generally have experience in performing analyses for two-levels of design.
- The two-level approach to design provides more flexibility regarding the analysis method to be used. For the lower level of acceleration, it may be sufficient to use simplified analysis methods, since soil nonlinearity will generally be less important under those conditions.
- The stiffness of the soil and the amount of ground deformation differ appreciably for the two design events. A two-level approach to design provides a more rational basis for evaluating behavior of a structure for these different conditions.

8.3.2 Disadvantages

- Two-level design could double the requirements for geotechnical analyses.
- A second level of design requires additional judgment regarding the appropriate earthquake magnitude and acceleration records to use in the analyses. If a probabilistic approach is taken, decisions must be made on the probability of occurrence and the design life that should be used in the analyses.
- The two-level design approach forces additional decisions to be made regarding what soil behavior to accept, what factors of safety to require, and what type of analyses to conduct.
- Two-levels of design requires closer scrutiny by the transportation agency when reviewing consultants' submittals. It probably also requires more knowledgeable individuals within the agency for conducting these reviews or for performing their own analyses.

8.4 Two-Level Design Approach For Abutments And Retaining Walls

Many of the pro and con issues for abutments and retaining walls are similar to those for foundations and these are not repeated.

8.4.1 Advantages

The chief additional benefit of a two-level design procedure for abutments and retaining walls is that seismic safety would be enhanced

8.4.2 Disadvantages

- Increase in the designer's time and cost. In addition to the analysis for the lower-level design, several more analyses are needed to determine the most suitable abutment design to meet the upper-level design requirements. These include a trial design with a shear key at a pre-determined failure value and a trial design with a fully restrained connection between the bridge superstructure and the abutment in case the shear key fails to yield.
- Construction budgets may increase to meet the upper level demand unless "the-State-of-the-Art" construction systems are implemented to reduce cost.

8.5 Two-Level Design Approach For Tunnels

The current situation is that there is no U.S. code for seismic design of tunnels, but there are design procedures that use a two-level design approach. Hence, the question for tunnels is whether the current design procedures should be codified, not whether the codified approach should be for one or two-levels of design earthquake. That is, it is assumed that if a U.S. code for tunnels is developed, it will follow the current U.S. design procedures.

Presented below is a list of advantages and disadvantages regarding the need to codify current U.S. seismic design procedures for tunnels.

8.5.1 Advantages

- Current U.S. procedures are somewhat unclear as to what are acceptable levels of strain or deformation for both the lower- and upper-level events. Hopefully, a code would clarify this issue.
- Current U.S. procedures do not clearly specify what vertical load should be used in combination with the racking/ovaling deformation. Specifically, should vertical dead load be increased for the effects of vertical acceleration? A code could address this issue.
- Current U.S. procedures do not provide a great deal of guidance regarding faulting and liquefaction issues. A code could address these issues in greater detail.
- Although U.S. procedures are rational, they have not been subject to a consensus code-development process.
- In the future, the level of federal reimbursement to communities and agencies may be tied to the degree those communities and agencies are seismically proactive. Developing a code for seismic design of tunnels would likely be considered as a proactive activity.

8.5.2 Disadvantages

- Historically, whether designed for earthquakes or not, tunnels have performed fairly well in earthquakes, Kobe being a notable exception.
- Tunnels designed according to existing U.S. procedures have performed very well in moderate earthquakes. Hence, the need for a code is questionable.
- In a sense, a code is a technology transfer device. Individuals with specialized expertise can develop a code that can then be used by a much larger number of engineers who may not have expertise in each of the areas addressed by the code. For bridges, a code tends to ensure that the public gets the same general level of safety, irrespective of whether the facility is designed by a large firm with a great deal of experience, or by a smaller firm with more limited experience. However, in the United States, tunnels are typically designed by large firms that already have experience and expertise. Hence, the need for a seismic code as a tunnel design technology transfer tool may be limited.

SECTION 9
FUTURE DIRECTIONS FOR CODE DEVELOPMENT

The ATC-18 Project Engineering Panel (PEP) believes that one method of attempting to reach consensus on recommendations for the development of future codes is to recommend specific and/or alternate key elements of the code and provide a discussion of issues that need to be addressed. This section provides the ATC-18 PEP recommendations.

The ATC-18 PEP believes that a two-level design approach should be included in future codes, as a minimum, for important bridges in the higher seismic zones. A detailed discussion on the advantages and disadvantages of a two-level design approach is given in Section 8.

9.1 Performance Criteria

The performance criteria to be included in any future code should be approved by a group that includes legislative policy makers. In order for this review to be effective, a significant amount of cost and technical data will need to be developed before the performance criteria can be intelligently discussed by a non-technical panel.

The other key requirement for the recommended performance criteria is a specific and unambiguous definition of *Important* and *Ordinary* bridges. Future codes may have significantly different design requirements for these two bridge categories.

9.1.1 Two-level Design Approach

The recommended performance criteria for a two-level design approach is given in Table 9-1. The terms used in the table are defined below. Both the functional- and safety-evaluation earthquakes will have specific definitions in terms of probabilistic return periods; i.e., 72, 150, or 250 years for a functional evaluation design event and 950 or 2475 years for a safety-evaluation event. The expected performance of Ordinary bridges will be similar to those of the typical one-level procedure; i.e., collapse will be prevented, but significant damage may occur. Bridges will be designed so that damage occurs in visible locations.

Table 9-1 Performance Criteria

Ground Motion At Site	*Ordinary Bridges*	*Important Bridges*
FUNCTIONAL EVALUATION	Service Level—Immediate Repairable Damage	Service Level—Immediate Minimal Damage
SAFETY EVALUATION	Service Level—Limited Significant Damage	Service Level—Immediate Repairable Damage

9.1.2 One-level Design Approach

The performance criteria for a one-level design approach cannot be as specific as for a two-level design approach, because the performance in the lower level events can only be implied from the design requirements of the upper-level event. (This is consistent with current design philosophies). A suggested philosophy follows:

1. For a low level earthquake there should be only minimal damage.

2. For a significant earthquake, collapse should be prevented but significant damage may occur. Damage should occur in visible locations.

3. An addition to Item 2 is required if different *R* or *Z* factors are used for Important and Ordinary bridges in a one-level design procedure. Item 2 as it stands would apply to Ordinary bridges. The following would be added to Item 2 for Important bridges:

   For a significant earthquake, only repairable damage would be expected for Important bridges, and full access to the bridge should be possible within three days after the earthquake.

9.1.3 Two-levels Of Design Earthquakes

Two-levels of design earthquakes are required if a two-level procedure is developed.

Functional-evaluation ground motion. This is an event that is determined to have a reasonable (approximately 30 percent to 50 percent) probability of occurring during the useful life of a bridge. (Note that this may eventually become a 72-, 150-,or 250-year return-period event, depending on the chosen probability of exceedance and the definition of useful life of the bridge.) This definition may also result in more than one design return period since some bridges will have longer useful lives.

Safety-evaluation ground motion. This is an event with only a small probability of occurring during the useful life of the bridge (i.e., 10 percent probability of exceedance for a design life of 100 or 250 years. This results in a return period of 950 years or 2475 years, respectively). Note: The current AASHTO specifications use a 10 percent probability of exceedance in 50 years or 475-year return period event as the design earthquake.

9.1.4 Service Levels

Two definitions of service levels are recommended for the two-level design approach.

Service Level — Immediate. Full access to normal traffic is available almost immediately (e.g., within hours) following the earthquake. (It may be necessary to allow 24 hours or so for inspection of the bridge.)

Service Level — Limited. Limited access (reduced lanes, light emergency traffic) is possible within three days of the earthquake. Full service can be restored within months.

9.1.5 Damage Levels

A significant amount of work is required to develop reasonably specific definitions for levels of damage to columns, caissons, abutments and retaining walls. The following definitions are based on the ATC-32 criteria. The PEP believes that they should be augmented wherever possible with closure time frames, ductility levels, and, if feasible, allowable steel and concrete strain levels.

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

- *Significant Damage*. Although there is no collapse, permanent offsets may occur and damage consisting of cracking, reinforcement yield, and major spalling of concrete may require closure to repair. Partial or complete replacement may be required in some cases. Criteria need to be developed for abutments, wing walls, and steel members.

9.1.6 Issues To Be Resolved

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

1. Probabilistic versus Deterministic ground motion design parameters

   The PEP recommends the use of probabilistic design parameters but recognizes that in some seismic zones deterministic values of ground acceleration may exceed probabilistic values. A decision needs to be made on how to handle such cases.

2. Definition of *Important* and *Ordinary* Bridges

   Unambiguous definitions of these categories are needed.

   It is possible that a third category called "critical" may be needed. Also, the design life of *Important* and *Ordinary* bridges needs to be resolved. The PEP recognizes this may be difficult. A further complication occurs with toll bridges funded by bonds. These may be categorized as important or critical not because of their size or role in the transportation system, but because of financing requirements in the bond market.

3. Limited Service Level

   The PEP recommends that a limited service level be defined as being able to open a bridge to some trafficwithin three days after it had been inspected. This definition is based on the premise that no shoring would be required to restore service. A firm definition of limited service must be established, since it will impact the design requirements and performance levels.

4. Definition of functional-evaluation and safety-evaluation earthquakes

   The specific design earthquakes could be different for *Important* and *Ordinary* bridges. In all likelihood the final definition will depend on the availability of seismic risk maps for the selected return periods.

5. Significant damage.

   The definition of significant damage needs to address the issue of what level of aftershock the damaged structure needs to resist.

9.2 Design Approach

It is recommended that the current AASHTO Seismic Performance Category approach be continued in future codes, since it is a good method of varying design requirements in different seismic zones. It is also recommended that a two-level design approach be adopted at least for Important Bridges in higher seismic zones. The lower-level design requirements should be based on elastic design principles to ensure that there is no damage. The upper-level analysis should be deformation-based using nonlinear static (pushover) analysis procedures with strength and stiffness requirements being derived from appropriate nonlinear response spectra.

If a single-level design procedure is adopted for *Ordinary* bridges, it is recommended that the design approach include a nonlinear static analysis as part of the design procedure.

9.3 Seismic Loading

9.3.1 Two-level Design Approach

The upper-level design event is recommended to be a 2475-year return-period event. (Note: It may be necessary to have different return periods for the eastern and western portions of the United States; 2475 years and 950 years, respectively). The lower-level design event would be in the range of a 72-year to 250-year return-period earthquake. The return period would be based on the same probability of exceedance (e.g., 30 to 50 percent) and may be different for Ordinary and Important Bridges because Important Bridges would have a longer design life (i.e., 200 years versus 50 years).

The mathematical relationship between probability of exceedance and return period is given by the following equation:

$$P(a_{max} > a) = 1 - e^{-T/R} \qquad (9\text{-}1)$$

where:

$P(a_{max}>a)$ = probability that a given acceleration or response spectrum will be exceeded.
T = time frame in which the probability is expressed (e.g., 50 years, 100 years)
R = return period

Table 9-2 presents relationships between return-period and probabilities of exceedance in 50 years and 100 years.

Table 9-2 Return Period and Probability of Exceedance

Return Period	*Probability of Exceedance*	
(Years)	*In 50 Years Design Life*	*In 100 Years Design Life*
25	86%	98%
50	63%	86%
72	50%	75%
150	28%	49%
250	18%	33%
475	10%	19%
950	5%	10%
2475	2%	4%

9.3.2 *One-level Design Approach*

It is recommended that a single level design procedure be based on 2475-year return-period risk maps if they are available. (Note: It may be necessary to have different return periods for the eastern and western portions of the United States of 2475 years and 950 years, respectively.

9.3.3 *Issues To Be Resolved*

1. For design ground motions specified by a probabilistic method, a decision must be made whether to use mean or mean-plus-one standard-deviation values for attenuation relationships and the shape of response spectra. Whatever decision is made should be clearly stated. Current AASHTO spectral shapes are based on a mean-plus-one standard deviation values.

2. The other key issue that needs to be addressed is how near-fault effects should be included in the design ground motion. The "fling" or "pulse" effect is a recently discovered phenomenon but it has a significant impact on the response of yielding structures. The 1994 NEHRP Provisions for New Buildings (BSSC, 1994) and 1995 UBC requirements for seismically isolated buildings and the ATC-32 requirements for bridges are the only current code requirements that attempt to address this important issue.

9.4 Site Effects

The ATC-18 PEP recommends that the 1994 NEHRP (BSSC, 1994) soil factors or their derivatives be the basis for the response spectra used for design.

9.5 Damping

Current practice is to use a five-percent damped response spectra for design. If nonlinear static analysis is adopted for the upper-level design event, then nonlinear spectra will have to be derived from the elastic spectra. The nonlinear spectra will incorporate whatever levels of energy dissipation (hysteretic and viscous) are deemed appropriate. If nonlinear static analysis is not adopted, higher damping levels in combination with elastic response spectra may be appropriate for the upper-level design event.

The only current U.S. code requirements that consider the impact of higher levels of damping on the response of a structure are those for seismically isolated bridges and buildings (AASHTO, 1995 and BSSC, 1994). The values of the reduction factor used to modify the five-percent damped response spectra are given in Table 9-3.

9.6 Analysis

9.6.1 *Two-level Design Approach*

Current elastic analysis procedures (equivalent static and multi-modal) are appropriate for the lower-level event. It is envisioned that the lower-level design procedures would use component stiffness values consistent with "little or no" damage.

Table 9-3 Damping Reduction Coefficient β

EFFECTIVE DAMPING A (Percentage of Critical)	< 2%	5%	10%	20%	30%	40%	> 50%
Reduction Factor β	0.8	1.0	1.2	1.5	1.7	1.9	2.0

The recommended procedure for the upper-level event should include a nonlinear static analysis. However, this analysis method requires additional development work before it can be used as a standard office procedure.

9.6.2 One-level Approach

There is no consensus on the best method of implementing a one-level procedure. The PEP believes that it is desirable for nonlinear static analysis to be a part of any analysis requirements. At a minimum, nonlinear static analysis should be required for Important Bridges. Current elastic design and analysis procedures may be sufficient for smaller Ordinary Bridges.

It is possible that the analysis procedure in a one-level design approach could incorporate both the current *R*-Factor elastic procedure and nonlinear static analysis. Possible options are given in Table 9-4. Nevertheless, requiring different analysis procedures for different seismic input levels in a two-level approach seems more rational than requiring two analysis procedures for the same seismic input level in a single-level design approach.

Table 9-4 Minimum Analysis Options

OPTIONS	*ORDINARY BRIDGES*		*IMPORTANT BRIDGES*
	Small Regular	*Other(s)*	
Option 1	*R* Factor and Pushover	*R* Factor and Pushover	*R* Factor and Pushover
Option 2	*R* Factor only	*R* Factor and Pushover	*R* Factor and Pushover
Option 3	*R* Factor only	*R* Factor only	*R* Factor and Pushover

9.7 Design Forces

9.7.1 Two-level Design Approach

1. *Ductile components*. These should be sized to remain undamaged for the lower-level event and to have adequate ductility to meet the performance criteria for the upper-level event. A definition is required for undamaged; e.g., elastic response and uncracked, or cracked with limited inelastic response but no spalling. Concrete and steel strains and ductility levels for minimal, repairable, and significant damage need to be developed.

2. *Non-ductile components.* These should be sized to remain undamaged for the lower-level event. The design requirement for the upper-level event would depend on whether or not the element was sacrificial. For sacrificial elements, the ultimate strength should be close to but larger than that required for the lower-level event. The actual capacity of sacrificial elements must be low enough to ensure that these elements will fail in the upper-level event. If they do not, they may make the consequences of the upper-level event worse. Nonsacrificial essential elements should be designed either for elastic demands from the upper-level event or by the use of capacity design procedures.

3. *Foundations.* A capacity design procedure should be used for all foundation designs to ensure there is no damage in either design event. Special studies are required for pile bents, drilled shafts, and

caissons to determine appropriate design criteria. As with the evaluation of the columns and abutments, more attention is required of the deformation capacities and demands in foundations.

9.7.2 One-level Design Approach

1. *Ductile components.* These should be sized by the *R*-Factor elastic design procedure or by nonlinear static analysis. The R Factors or permissible ductility levels in a nonlinear static analysis may vary depending on the desired performance for the upper-level event.

2. *Non-ductile components.* Nonsacrificial elements should be designed for elastic demands from the upper-level event or by the use of capacity design procedures. Sacrificial elements would have to be designed using a guideline that would somewhat correspond to the design level of an unspecified lower-level event. For example, the sacrificial elements could be designed to withstand one-half or one-third of the force required for the upper-level event.

3. *Foundations.* A capacity design procedure should be used for all foundation designs to ensure that there is no damage. Special studies are required for pile bents, drilled shafts, and caissons to determine appropriate design criteria.

9.8 Design Displacements

Design displacement values should be determined from the upper-level design event, using stiffness properties appropriate to the expected level of displacement. These values could be determined directly from a nonlinear static analysis, but in a one-level elastic approach, an iterative analysis procedure may be needed to ensure that the appropriate stiffness values are used.

It is recommended that the current minimum seat width requirements remain since it is not considered practical in a design office environment to accurately calculate the relative displacement for all applicable parameters (e.g., surface wave effects).

It is also recommended that overall drift limits be incorporated to avoid P-Δ effects on long period structures.

9.9 Concrete And Steel Design

It is recommended that capacity design procedures be used to prevent brittle modes of failure in all critical members. Confinement, shear, joint shear, torsion, anchorage, and splice reinforcement requirements will be improved as research progresses over the next several years. It is recommended that the requirements be updated to reflect new research findings.

9.10 Foundation Design

9.10.1 Two-level Design Approach

Geotechnical analyses should be conducted for both levels of design acceleration to confirm that response of the soil will not adversely affect the performance of structures supported on or within the soil. These analyses should include assessments of the potential for liquefaction, lateral spreading, and slope instability; dynamic earth pressures on buried walls; soil-structure interaction; and uplift and rocking of the foundation. Since some of the analyses depend on the magnitude of the earthquake causing the design acceleration, care must be used in selecting an earthquake magnitude that is compatible with the seismo-tectonics in the area, both in terms of source mechanism and source distance. Where partial or total liquefaction of saturated, cohesionless soil layers is predicted, the effects of loss in soil strength on the stability of sloping ground, in particular, and the vertical and horizontal bearing capacity of the soil, must be addressed to confirm that structures supported on or within the soil can tolerate these effects. The evaluation should also consider the amount and effects of settlement resulting from liquefaction or densification of granular soil and the increased soil downdrag forces on pile foundations resulting from liquefaction or densification of the soil.

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

9.10.2 One-level Design Approach

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

SECTION 10
RECOMMENDED RESEARCH AND DEVELOPMENT TOPICS

10.1 Abutments

Throughout the project, there was considerable discussion on issues related to the design and analysis of abutments. It was agreed that the state of knowledge and the ability to accurately model abutments was significantly behind that of columns and foundations. The PEP concluded that for many bridges, abutment performance would have a significant impact on the overall response of a bridge at different levels of shaking. Furthermore, it appears that abutment design strategies vary significantly due to the inadequacies in the current state of knowledge. Concerns were expressed over the ability to properly design abutments for high-level accelerations from longer return-period events.

As a consequence it is the unanimous recommendation of the ATC-18 PEP that the NCEER program fund an in-depth study on abutments. It should include the following three aspects.

1. *Design strategies.* This project should determine the pros and cons of the various abutment strategies currently in use. It should include options for protecting piles, back walls, and wing walls. It should also address the pros and cons of shear keys, knock-off devices, and other similar elements.

2. *Analysis issues.* The project should recommend how abutments should be modeled for various levels of shaking, directions of shaking (i.e., longitudinal vs. transverse), and different design strategies. It is envisioned that further experimental and analytical work may be warranted as part of this project.

3. *Sensitivity studies.* If items (1) and (2) are addressed in the first part of a comprehensive program, then analytical sensitivity studies could be performed to identify the research areas that will provide the most useful information from a design and analysis perspective.

10.2 Retaining Walls

Results from the research recommended in Section 10.1 could also be used in retaining wall design. Designing a retaining wall is generally less complicated than designing an abutment. Soil pressure (including an appropriate dynamic increment) acting normal to the wall is the only additional force to be considered in design.

10.3 Seismic Hazard Maps

The PEP recommends a two-level design procedure, at least for Important Bridges in higher seismic zones. The U.S. Geological Survey is developing 475-year, 950-year, and 2475-year return period maps. These will most likely cover whatever is selected as an upper-level design event for new highway transportation structure design provisions. In addition to these maps, maps with lower return periods (e.g., 72 years, 150 years or 250 years) or a mechanism to scale the 475-year return period maps to obtain the lower-level events for the various regions of the United States are needed. The other key issue that must be resolved with respect to both hazard maps and the associated response spectra is whether to use mean or mean-plus-one-standard-deviation values of attenuation.

10.4 Ordinary And Important Bridges

It is envisioned that future codes will have differentiated design requirements between *Ordinary* and *Important* bridges. The PEP recommends that specific and unambiguous definitions be developed for these categories and that a third "critical" category be considered.

10.5 Damage Levels

In order to develop more definitive performance criteria for future codes, a detailed study is needed to develop specific definitions for minimal, repairable, and significant damage. These definitions should include ductility levels, concrete and steel strain levels, and approximate downtimes. Section 9 on the recommended directions for future codes incorporates definitions used in ATC-32, but the PEP believes these need further refinement.

10.6 Pushover Analysis

The PEP is recommending that pushover (i.e., inelastic static) analysis be a significant part of any future analysis requirements. In order to achieve this goal, it is necessary to develop the analytical tools to allow this method to be used in a design office environment. We therefore recommend that this development work be performed. Use of pushover analysis will also require the availability of appropriate nonlinear spectra. The PEP recommends that an effort be undertaken to develop tools to convert elastic spectra to nonlinear spectra that can be used in design.

10.7 Performance Criteria

The PEP believes that any future codes should be based on performance criteria that are developed with the input of transportation policy makers as well as government and public officials. In order for such an effort to be productive, a significant amount of cost/benefit data would need to be developed to facilitate the discussion of performance criteria with nontechnical groups. The PEP recommends that the appropriate cost/benefit data be developed.

10.8 Redundancy

It is difficult to quantify the redundancy of a seismically resistant structure. It has long been considered good design practice to incorporate as much redundancy as possible, although code provisions have never addressed the issue. However, there has been a trend over the last 20 years to provide less redundancy in the seismic framing system. This was an area of concern in the 1994 Northridge earthquake, when steel moment frames suffered significant damage. As a consequence of Northridge, both the 1997 NEHRP Provisions for new buildings and the UBC address the issue of redundancy: less redundant structures must be designed for higher force levels.

Redundancy in the design of vertical-load-carrying elements requires the provision of as many alternate vertical load paths as possible. Redundancy in seismic design involves providing as many energy-dissipating mechanisms (usually joints) as possible. In a building, this means that as many frame lines as possible should be part of the lateral-load-resisting system. In a bridge, it means providing as many locations at the top and bottom of columns as possible that can dissipate energy.

In bridge structures, seismic redundancy is an issue of concern because of the trend to design zero- or nominal-moment hinges at the base of columns in multi-column bents in order to reduce foundation costs. This design philosophy results in the removal of a joint that acts as one of a very limited number of energy-dissipating mechanisms. For example, if the base of the columns in a two-column bent were designed for such hinges, then there are only two locations (tops of columns) where energy can be dissipated. Thus, this two-column bent has the same number of energy-dissipating elements as a single column bent, but is designed for significantly lower forces than is a single column bent. The value of the R factor in a multi-column bent is five compared to three in a single column bent due primarily to the increased redundancy.

The PEP believes the issue of redundancy requires further study in order that it can be included in the next generation of seismic codes.

10.9 Tunnels

The PEP recommends that a consensus code for two-level design of tunnels be developed based on the current noncodified design procedure used in the United States.

SECTION 11
SYMBOLS AND ACRONYMS

11.1 Definitions of Selected Symbols

This list contains only those symbols which are not defined close to where they appear. In some cases, the relevant organization or publication is noted. See the following Acronyms and Section 12, References.

A	acceleration coeff.; effective peak ground acceleration (AASHTO); peak rock acceleration
A_a	effective peak acceleration (1994 NEHRP)
A_C	applicable column area for capacity design
A_c	area of confined concrete core
A_e	effective shear area, $= 0.8A_g$ for columns
A_g	gross concrete cross-sectional area
A_{sh}	total cross-sectional area of hoop reinforcement in tied columns, each direction
$A_{sh}/s_t h_c$	transverse reinforcement ratio
A_v	effective peak velocity-related acceleration (1994 NEHRP); total area of shear reinforcement parallel to the applied shear force
a	spacing of column reinforcing ties in vertical direction
a_g	peak ground acceleration (E8)
a_{max}	peak ground acceleration
B	width of a bridge span
C_D	damping adjustment factor (Japan), same as c_d
C_s	lateral force seismic design coeff. (AASHTO)
C_{sm}	seismic response coeff. (AASHTO)
c_d	damping modification factor (Japan), same as C_D
c_g	ground conditions modification factor (Japan)
c_i	structural importance factor (Japan)
c_t	structural response factor (Japan)
c_z	seismic zone factor (Japan)
D	column diameter
D/C	displacement demand to capacity ratio
D'	diameter of a circular hoop
d	distance from extreme compression fiber to centroid of torsion reinforcement
d_i	ordinate of the assumed mode shape at the ith nodal point
E_x, E_y, E_z	displacement response from the three orthogonal directions (E8)
ec	concrete compressive strain ($= \varepsilon_s$)
ecu	maximum concrete compressive strain ($= \varepsilon_{cu}$)
es	reinforcing steel strain ($= \varepsilon_s$)
F	an equivalent horizontal force applied to the deck (E8)
F_a	acceleration-based site coeff. at 0.3-sec period
F_b	$= F_e / R$, design force
F_e	elastic force
F_{EQ}	earthquake design forces
F_i	force applied to the ith nodal point

F_L = R/L, liquefaction resistance factor (Japan)
F_v velocity-based site coeff. at 1.0-sec period
F_y yield strength; overstrength force level
F_y / F_b overstrength ratio
f_c concrete cylinder compressive strength
f'_c concrete design compressive stress
f'_{ce} expected concrete strength
f_s max tensile stress in column longitudinal reinforcement
f_y reinforcement yield strength
f_{ye} expected reinforcement yield strength
f_{yh} yield strength, or stress, of transverse, or horizontal, reinforcement
G shear modulus
g acceleration of gravity
H thickness of soil; average height of adjacent columns in a seat width calculation; design load (NZ)
h damping ratio (Japan); damping correction factor
h_c dimension of column in direction of the reinforcing tie
k horizontal seismic coeff.
k_h design seismic coeff. (Japan); lateral force coeff. (Japan)
k_{ho} standard horizontal seismic coeff., = 0.2 (Japan),
k_1, k_2 spectrum-shaping exponents
L length of bridge deck in a seat width calculation; column length; bridge span length
l_a anchorage length in a column
l_s splice length
M earthquake magnitude (Richter or local); mass
M_{cr} moment at first cracking of concrete
M_N nominal plastic moment (= M_n); nominal moment capacity calculated without ϕ factors
M_P overstrength plastic moment for design (= M_p)
m_i mass at the ith nodal point
N number of blows per foot in SPT; seat width for bridge girders (AASHTO); number of time-history inputs used; axial force at a plastic hinge (E8)
N_u ultimate axial compressive force in column
N_1 SPT N-value corrected to an effective overburden pressure of 100 kPa
$N_{1(60)}$ SPT blow count normalized to 1 ton per sq ft confining pressure (or effective stress), and equivalent hammer energy efficiency of 60%
P column load
P_{AE} dynamic seismic horizontal force on abutment or retaining wall
P_e earth pressure due to seismic effects (TNZ); column axial force
$P(a_{max} > a)$
the probability that a_{max} is greater than a specific value of a
$P\text{-}\Delta$ the effect on additional overturning from displaced vertical load
q behavior modification factor (E8)
R return period; structural response modification factor; normalized rock spectrum
$R(T)$ acceleration response spectrum (E8)
R_d adjustment to displacement from elastic dynamic analysis
$R_y(\mu)$ strength-reduction factor
R_μ ductility reduction factor
r radius of curvature; 1/r is curvature

S	site coeff.; site soil type coeff.; spectrum coeff.; support skew angle in a seat width calculation; soil amplification, or modification, factor;
S_A	average extreme response (E8)
S_E	design seismic action (E8)
s, s_t	spacing of horizontal reinforcement along the axis of the member (= a)
T	design life; period; period of bridge; natural period of the structure
T^*	predominant period of ground motion
T_g	predominant period of the site (Japan)
V_0	base design shear
V_c	concrete contribution to column shear strength
V_s	shear wave velocity; reinforcing steel contribution to column shear strength
v_s	shear wave propagation velocity;
w	water content (%); moisture content
Z	force reduction divisor or factor or coeff.; risk factor; response modification factor (Caltrans); adjustment factor for ductility and risk (Caltrans)
α	5%
β	damping reduction coeff.
β_0	spectral acceleration amplification factor
γ_I	importance factor (E8)
γ_e	a factor of safety dependent on a_g applied to the design displacement (E8)
Δ	an increment
Δ_e	= $\Delta_{elastic}$, displacement using unreduced elastic response spectrum
Δ_{max}	= $\Delta_{inelastic}$, displacement during inelastic response
Δ_y	yield displacement (NZ)
δ_u	column drift limit
ε_{cu}	maximum concrete compressive strain
η_κ	normalized axial load; ratio of the axial load to gross area x characteristic concrete strength; (= η_k, η_K)
μ	displacement ductility factor (NZ); system ductility; displacement ductility capacity
ρ_l	column longitudinal reinforcement ratio
ρ_s	column spiral reinforcement ratio (volumetric)
φ, ϕ	capacity, or strength, reduction factor

11.2 Acronyms

This list contains those acronyms not spelled out close to where they appear.

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ARS	acceleration response spectrum
ARS curves	standard elastic design spectra based on *A, R*, and *S* (Caltrans); *ARS* spectra
BDS	Bridge Design Specification (Caltrans)
$b_E E$	load due to earth pressure
Caltrans	California Department of Transportation
CDMG	California Division of Mines and Geology
CPT	Cone Penetrometer Test
CQC	complete quadratic combination
DL	dead load
EI	Young's modulus x moment of inertia, proportional to bending stiffness; flexural stiffness
EPA	effective peak acceleration
EQ	elastic seismic force
E8	Eurocode 8
FHWA	Federal Highway Administration
fps	feet per second
GC	ground condition
kPa	kiloPascal
KRTR	Kobe Rapid Transit Railway
LRFD	Load and Resistance Factor Design
MCE	maximum credible earthquake
mps	meters per second
MSE	mechanically stabilized earth
NCEER	National Center for Earthquake Engineering Research
NEHRP	National Earthquake Hazard Reduction Program
NZ	New Zealand
P-Δ effect	additional overturning from displaced vertical load
PGA	peak ground acceleration
PI	plasticity index
PS	load due to buoyancy or stream flow
R-factor	structural response modification factor
SDOF	single degree of freedom
SHAKE	computer code for site amplification
SPC	Seismic Performance Category
SPT	Standard Penetration Test
SRSS	square root of the sum of the squares
TCA	Transportation Corridor Agencies
TNZ	Transit New Zealand
UBC	Uniform Building Code

SECTION 12
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APPENDIX A
PROJECT PARTICIPANTS

APPLIED TECHNOLOGY COUNCIL

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

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

Dr. Ian G. Buckle, Highway Project Principal Investigator
National Center for Earthquake Engineering Research
State University of New York, Red Jacket Quad
Buffalo, New York 14261

PROJECT ENGINEERING PANEL

Mr. Donald G. Anderson
Woodward-Clyde Consultants
1501 Fourth Avenue
Seattle, Washington 98101

Mr. John Clark
Anderson Bjornstad Kane Jacobs
800 Fifth Avenue, Suite 800
Seattle, Washington 98104

Mr. John H. Hom
SOH & Associates, Structural Engineers
550 Kearny Street, #200
San Francisco, California 94108

Dr. Ronald L. Mayes, Project Director
Dynamic Isolations Systems, Inc.
3470 Mt. Diablo Blvd., Suite A200
Lafayette, California 94549

Mr. Richard V. Nutt
9048 Hazel Oak Court
Orangevale, California 95662

Prof. Michael J. O'Rourke
Rensselaer Polytechnic Institute
Civil Engineering Department
Troy, New York 12180

Dr. Charles Thornton*
Thornton-Tomasetti
641 Avenue of the Americas
New York, New York 10011

*ATC Board Representative

REPORT PREPARATION SERVICES

Mr. Peter Mork
Applied Technology Council
555 Twin Dolphin Drive, Suite 550
Redwood City, California 94065

Ms. Nancy Sauer
RDD Consultants
1163 Franklin Avenue
Louisville, Colorado 80027

APPENDIX B
APPLIED TECHNOLOGY COUNCIL PROJECTS AND REPORT INFORMATION

One of the primary purposes of Applied Technology Council is to develop resource documents that translate and summarize useful information to practicing engineers. This includes the development of guidelines and manuals, as well as the development of research recommendations for specific areas determined by the profession. ATC is not a code development organization, although several of the ATC project reports serve as resource documents for the development of codes, standards and specifications.

Applied Technology Council conducts projects that meet the following criteria:

1. The primary audience or benefactor is the design practitioner in structural engineering.

2. A cross section or consensus of engineering opinion is required to be obtained and presented by a neutral source.

3. The project fosters the advancement of structural engineering practice.

A brief description of several major completed projects and reports is given in the following section. Funding for projects is obtained from government agencies and tax-deductible contributions from the private sector.

ATC-1: This project resulted in five papers that were published as part of *Building Practices for Disaster Mitigation, Building Science Series 46*, proceedings of a workshop sponsored by the National Science Foundation (NSF) and the National Bureau of Standards (NBS). Available through the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, VA 22151, as NTIS report No. COM-73-50188.

ATC-2: The report, *An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings*, was funded by NSF and NBS and was conducted as part of the Cooperative Federal Program in Building Practices for Disaster Mitigation. Available through the ATC office. (Published 1974, 270 Pages)

> ABSTRACT: This study evaluated the applicability and cost of the response spectrum approach to seismic analysis and design that was proposed by various segments of the engineering profession. Specific building designs, design procedures and parameter values were evaluated for future application. Eleven existing buildings of varying dimensions were redesigned according to the procedures.

ATC-3: The report, *Tentative Provisions for the Development of Seismic Regulations for Buildings* (ATC-3-06), was funded by NSF and NBS. The second printing of this report, which includes proposed amendments, is available through the ATC office. (Published 1978, amended 1982, 505 pages plus proposed amendments)

> ABSTRACT: The tentative provisions in this document represent the results of a concerted effort by a multi-disciplinary team of 85 nationally recognized experts in earthquake engineering. The provisions embody several new concepts that were significant departures from then existing design codes. The provisions serve as the basis for the seismic provisions of the 1988 *Uniform Building Code* and the 1988 and subsequent issues of the *NEHRP Recommended Provisions for the Development of Seismic Regulation for New Buildings*. The second printing of this document contains proposed amendments prepared by a joint committee of the Building Seismic Safety Council (BSSC) and the NBS.

ATC-3-2: The project, Comparative Test Designs of Buildings Using ATC-3-06 Tentative Provisions, was funded by NSF. The project consisted of a study to develop and plan a program for making comparative test designs of the ATC-3-06 Tentative Provisions. The project report was written to be used by the Building Seismic Safety Council in its refinement of the ATC-3-06 Tentative Provisions.

ATC-3-4: The report, *Redesign of Three Multistory Buildings: A Comparison Using ATC-3-06 and 1982 Uniform Building Code Design Provisions*, was published under a grant from NSF. Available through the ATC office. (Published 1984, 112 pages)

ABSTRACT: This report evaluates the cost and technical impact of using the 1978 ATC-3-06 report, Tentative Provisions for the Development of Seismic Regulations for Buildings, as amended by a joint committee of the Building Seismic Safety Council and the National Bureau of Standards in 1982. The evaluations are based on studies of three existing California buildings redesigned in accordance with the ATC-3-06 Tentative Provisions and the 1982 Uniform Building Code. Included in the report are recommendations to code implementing bodies.

ATC-3-5: This project, Assistance for First Phase of ATC-3-06 Trial Design Program Being Conducted by the Building Seismic Safety Council, was funded by the Buildings Seismic Safety Council and provided the services of the ATC Senior Consultant and other ATC personnel to assist the BSSC in the conduct of the first phase of its Trial Design Program. The first phase provided for trial designs conducted for buildings in Los Angeles, Seattle, Phoenix, and Memphis.

ATC-3-6: This project, Assistance for Second Phase of ATC-3-06 Trial Design Program Being Conducted by the Building Seismic Safety Council, was funded by the Building Seismic Safety Council and provided the services of the ATC Senior Consultant and other ATC personnel to assist the BSSC in the conduct of the second phase of its Trial Design Program. The second phase provided for trial designs conducted for buildings in New York, Chicago, St. Louis, Charleston, and Fort Worth.

ATC-4: The report, *A Methodology for Seismic Design and Construction of Single-Family Dwellings*, was published under a contract with the Department of Housing and Urban Development (HUD). Available through the ATC office. (Published 1976, 576 pages)

ABSTRACT: This report presents the results of an in-depth effort to develop design and construction details for single-family residences that minimize the potential economic loss and life-loss risk associated with earthquakes. The report: (1) discusses the ways structures behave when subjected to seismic forces, (2) sets forth suggested design criteria for conventional layouts of dwellings constructed with conventional materials, (3) presents construction details that do not require the designer to perform analytical calculations, (4) suggests procedures for efficient plan-checking, and (5) presents recommendations including details and schedules for use in the field by construction personnel and building inspectors.

ATC-4-1: The report, *The Home Builders Guide for Earthquake Design*, was published under a contract with HUD. Available through the ATC office. (Published 1980, 57 pages)

ABSTRACT: This report is an abridged version of the ATC-4 report. The concise, easily understood text of the Guide is supplemented with illustrations and 46 construction details. The details are provided to ensure that houses contain structural features that are properly positioned, dimensioned and constructed to resist earthquake forces. A brief description is included on how earthquake forces impact on houses and some precautionary constraints are given with respect to site selection and architectural designs.

ATC-5: The report, *Guidelines for Seismic Design and Construction of Single-Story Masonry Dwellings in Seismic Zone 2*, was developed under a contract with HUD. Available through the ATC office. (Published 1986, 38 pages)

ABSTRACT: The report offers a concise methodology for the earthquake design and construction of single-story masonry dwellings in Seismic Zone 2 of the United States, as defined by the 1973 Uniform Building Code. The guidelines are based in part on shaking table tests of masonry construction conducted at the University of California at Berkeley Earthquake Engineering Research

Center. The report is written in simple language and includes basic house plans, wall evaluations, detail drawings, and material specifications.

ATC-6: The report, *Seismic Design Guidelines for Highway Bridges*, was published under a contract with the Federal Highway Administration (FHWA). Available through the ATC office. (Published 1981, 210 pages)

ABSTRACT: The Guidelines are the recommendations of a team of sixteen nationally recognized experts that included consulting engineers, academics, state and federal agency representatives from throughout the United States. The Guidelines embody several new concepts that were significant departures from then existing design provisions. Included in the Guidelines are an extensive commentary, an example demonstrating the use of the Guidelines, and summary redesign reports on 21 bridges redesigned in accordance with the Guidelines. The Guidelines have been adopted by the American Association of Highway and Transportation Officials as a standard specification.

ATC-6-1: The report, *Proceedings of a Workshop on Earthquake Resistance of Highway Bridges*, was published under a grant from NSF. Available through the ATC office. (Published 1979, 625 pages)

ABSTRACT: The report includes 23 state-of-the-art and state-of-practice papers on earthquake resistance of highway bridges. Seven of the twenty-three papers were authored by participants from Japan, New Zealand and Portugal. The Proceedings also contain recommendations for future research that were developed by the 45 workshop participants.

ATC-6-2: *The report, Seismic Retrofitting Guidelines for Highway Bridges*, was published under a contract with FHWA. Available through the ATC office. (Published 1983, 220 pages)

ABSTRACT: The Guidelines are the recommendations of a team of thirteen nationally recognized experts that included consulting engineers, academics, state highway engineers, and federal agency representatives. The Guidelines, applicable for use in all parts of the U.S., include a preliminary screening procedure, methods for evaluating an existing bridge in detail, and potential retrofitting measures for the most common seismic deficiencies. Also included are special design requirements for various retrofitting measures.

ATC-7: The report, *Guidelines for the Design of Horizontal Wood Diaphragms*, was published under a grant from NSF. Available through the ATC office. (Published 1981, 190 pages)

ABSTRACT: Guidelines are presented for designing roof and floor systems so these can function as horizontal diaphragms in a lateral force resisting system. Analytical procedures, connection details and design examples are included in the Guidelines.

ATC-7-1: The report, *Proceedings of a Workshop of Design of Horizontal Wood Diaphragms*, was published under a grant from NSF. Available through the ATC office. (Published 1980, 302 pages)

ABSTRACT: The report includes seven papers on state-of-the-practice and two papers on recent research. Also included are recommendations for future research that were developed by the 35 participants.

ATC-8: This report, *Proceedings of a Workshop on the Design of Prefabricated Concrete Buildings for Earthquake Loads*, was funded by NSF. Available through the ATC office. (Published 1981, 400 pages)

ABSTRACT: The report includes eighteen state-of-the-art papers and six summary papers. Also included are recommendations for future research that were developed by the 43 workshop participants.

ATC-9: The report, *An Evaluation of the Imperial County Services Building Earthquake Response and Associated Damage*, was published under a grant from NSF. Available through the ATC office. (Published 1984, 231 pages)

ABSTRACT: The report presents the results of an in-depth evaluation of the Imperial County Services Building, a 6-story reinforced concrete frame and shear wall building severely damaged by the October 15, 1979 Imperial Valley, California, earthquake. The report contains a review and evaluation of earthquake damage to the building; a review and evaluation of the seismic design; a comparison of the requirements of various building codes as they relate to the building; and conclusions and recommendations pertaining to future building code provisions and future research needs.

ATC-10: This report, *An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance*, was funded by the U.S. Geological Survey (USGS). Available through the ATC office. (Published 1982, 114 pages)

ABSTRACT: The report contains an in-depth analytical evaluation of the ultimate or limit capacity of selected representative building framing types, a discussion of the factors affecting the seismic performance of buildings, and a summary and comparison of seismic design and seismic risk parameters currently in widespread use.

ATC-10-1: This report, *Critical Aspects of Earthquake Ground Motion and Building Damage Potential*, was co-funded by the USGS and the NSF. Available through the ATC office. (Published 1984, 259 pages)

ABSTRACT: This document contains 19 state-of-the-art papers on ground motion, structural response, and structural design issues presented by prominent engineers and earth scientists in an ATC seminar. The main theme of the papers is to identify the critical aspects of ground motion and building performance that currently are not being considered in building design. The report also contains conclusions and recommendations of working groups convened after the Seminar.

ATC-11: The report, *Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers*, was published under a grant from NSF. Available through the ATC office. (Published 1983, 184 pages)

ABSTRACT: This document presents the results of an in-depth review and synthesis of research reports pertaining to cyclic loading of reinforced concrete shear walls and cyclic loading of joint reinforced concrete frames. More than 125 research reports published since 1971 are reviewed and evaluated in this report. The preparation of the report included a consensus process involving numerous experienced design professionals from throughout the United States. The report contains reviews of current and past design practices, summaries of research developments, and in-depth discussions of design implications of recent research results.

ATC-12: This report, *Comparison of United States and New Zealand Seismic Design Practices for Highway Bridges*, was published under a grant from NSF. Available through the ATC office. (Published 1982, 270 pages)

ABSTRACT: The report contains summaries of all aspects and innovative design procedures used in New Zealand as well as comparison of United States and New Zealand design practice. Also included are research recommendations developed at a 3-day workshop in New Zealand attended by 16 U.S. and 35 New Zealand bridge design engineers and researchers.

ATC-12-1: This report, *Proceedings of Second Joint U.S.-New Zealand Workshop on Seismic Resistance of Highway Bridges*, was published under a grant from NSF. Available through the ATC office. (Published 1986, 272 pages)

ABSTRACT: This report contains written versions of the papers presented at this 1985 Workshop as well as a list and prioritization of workshop recommendations. Included are summaries of research projects being conducted in both countries as well as state-of-the-practice papers on various aspects of design practice. Topics discussed include bridge design philosophy and loadings; design of columns, footings, piles, abutments and retaining structures; geotechnical aspects of foundation design; seismic analysis techniques; seismic retrofitting; case studies using base isolation; strong-motion data acquisition and interpretation; and testing of bridge components and bridge systems.

ATC-13: The report, *Earthquake Damage Evaluation Data for California*, was developed under a contract with the Federal Emergency Management Agency (FEMA). Available through the ATC office. (Published 1985, 492 pages)

ABSTRACT: This report presents expert-opinion earthquake damage and loss estimates for industrial, commercial, residential, utility and transportation facilities in California. Included are damage probability matrices for 78 classes of structures and estimates of time required to restore damaged facilities to pre-earthquake usability. The report also describes the inventory information essential for estimating economic losses and the methodology used to develop loss estimates on a regional basis.

ATC-14: The report, *Evaluating the Seismic Resistance of Existing Buildings*, was developed under a grant from the NSF. Available through the ATC office. (Published 1987, 370 pages)

ABSTRACT: This report, written for practicing structural engineers, describes a methodology for performing preliminary and detailed building seismic evaluations. The report contains a state-of-practice review; seismic loading criteria; data collection procedures; a detailed description of the building classification system; preliminary and detailed analysis procedures; and example case studies, including non-structural considerations.

ATC-15: This report, *Comparison of Seismic Design Practices in the United States and Japan*, was published under a grant from NSF. Available through the ATC office. (Published 1984, 317 pages)

ABSTRACT: The report contains detailed technical papers describing design practices in the United States and Japan as well as recommendations emanating from a joint U.S.-Japan workshop held in Hawaii in March, 1984. Included are detailed descriptions of new seismic design methods for buildings in Japan and case studies of the design of specific buildings (in both countries). The report also contains an overview of the history and objectives of the Japan Structural Consultants Association.

ATC-15-1: The report, *Proceedings of Second U.S.-Japan Workshop on Improvement of Building Seismic Design and Construction Practices*, was published under a grant from NSF. Available through the ATC office. (Published 1987, 412 pages)

ABSTRACT: This report contains 23 technical papers presented at this San Francisco workshop in August, 1986, by practitioners and researchers from the U.S. and Japan. Included are state-of-the-practice papers and case studies of actual building designs and information on regulatory, contractual, and licensing issues.

ATC-15-2: The report, *Proceedings of Third U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices*, was published jointly by ATC and the Japan Structural Consultants Association. Available through the ATC office. (Published 1989, 358 pages)

ABSTRACT: This report contains 21 technical papers presented at this Tokyo, Japan, workshop in July, 1988, by practitioners and researchers from the U.S., Japan, China, and New Zealand. Included are state-of-the-practice papers on various topics, including braced steel frame buildings, beam-column joints in reinforced concrete buildings, summaries of comparative U. S. and Japanese designs, and base isolation and passive energy dissipation devices.

ATC-15-3: The report, *Proceedings of Fourth U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices*, was published jointly by ATC and the Japan Structural Consultants Association. Available through the ATC office. (Published 1992, 484 pages)

ABSTRACT: This report contains 22 technical papers presented at this Kailua-Kona, Hawaii, workshop in August, 1990 by practitioners and researchers from the United States, Japan, and Peru. Included are papers on postearthquake building damage assessment; acceptable earth-quake damage; repair and retrofit of earthquake damaged buildings; base-isolated buildings, including Architectural Institute of Japan recommendations for design; active damping systems; wind-resistant design; and summaries of working group conclusions and recommendations.

ATC-15-4: The report, *Proceedings of Fifth U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices*, was published jointly by ATC and the Japan Structural Consultants Association. Available through the ATC office. (Published 1994, 360 pages)

ABSTRACT: This report contains 20 technical papers presented at this San Diego, California workshop in September, 1992. Included are papers on performance goals/acceptable damage in seismic design; seismic design procedures and case studies; construction influences on design; seismic isolation and passive energy dissipation; design of irregular structures; seismic evaluation, repair and upgrading; quality control for design and construction; and summaries of working group discussions and recommendations.

ATC-16: This project, Development of a 5-Year Plan for Reducing the Earthquake Hazards Posed by Existing Nonfederal Buildings, was funded by FEMA and was conducted by a joint venture of ATC, the Building Seismic Safety Council and the Earthquake Engineering Research Institute. The project involved a workshop in Phoenix, Arizona, where approximately 50 earthquake specialists met to identify the major tasks and goals for reducing the earthquake hazards posed by existing nonfederal buildings nationwide. The plan was developed on the basis of nine issue papers presented at the workshop and workshop working group discussions. The Workshop Proceedings and Five-Year Plan are available through the Federal Emergency Management Agency, 500 "C" Street, S.W., Washington, DC 20472.

ATC-17: This report , *Proceedings of a Seminar and Workshop on Base Isolation and Passive Energy Dissipation*, was published under a grant from NSF. Available through the ATC office. (Published 1986, 478 pages)

ABSTRACT: The report contains 42 papers describing the state-of-the-art and state-of-the-practice in base-isolation and passive energy-dissipation technology. Included are papers describing case studies in the United States, applications and developments worldwide, recent innovations in technology development, and structural and ground motion issues. Also included is a proposed 5-year research agenda that addresses the following specific issues: (1) strong ground motion; (2) design criteria; (3) materials, quality control, and long-term reliability; (4) life cycle cost methodology; and (5) system response.

ATC-17-1: This report, *Proceedings of a Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control*, was published under a grant from NSF. Available through the ATC office. (Published 1993, 841 pages)

ABSTRACT: The 2-volume report documents 70 technical papers presented during a two-day seminar in San Francisco in early 1993. Included are invited theme papers and competitively selected papers on issues related to seismic isolation systems, passive energy dissipation systems, active control systems and hybrid systems.

ATC-19: The report, *Structural Response Modification factors* was funded by NSF and NCEER. Available through the ATC office. (Published 1995, 70 pages)

ABSTRACT: This report addresses structural response modification factors (R factors), which are used to reduce the seismic forces associated with elastic response to obtain design forces. The report documents the basis for current R values, how R factors are used for seismic design in other countries, a rational means for decomposing R into key components, a framework (and methods) for evaluating the key components of R, and the research necessary to improve the reliability of engineered construction designed using R factors.

ATC-20: The report, *Procedures for Postearthquake Safety Evaluation of Buildings*, was developed under a contract from the California Office of Emergency Services (OES), California Office of Statewide Health Planning and Development (OSHPD) and FEMA. Available through the ATC office (Published 1989, 152 pages)

ABSTRACT: This report provides procedures and guidelines for making on-the-spot evaluations and decisions regarding continued use and occupancy of earthquake damaged buildings. Written specifically for volunteer structural engineers and building inspectors, the report includes rapid and detailed evaluation procedures for inspecting buildings and posting them as "inspected" (apparently safe), "limited entry" or "unsafe". Also included are special procedures for evaluation of essential buildings (e.g., hospitals), and evaluation procedures for nonstructural elements, and geotechnical hazards.

ATC-20-1: The report, *Field Manual: Postearthquake Safety Evaluation of Buildings*, was developed under a contract from OES and OSHPD. Available through the ATC office (Published 1989, 114 pages)

ABSTRACT: This report, a companion Field Manual for the ATC-20 report, summarizes the postearthquake safety evaluation procedures in brief concise format designed for ease of use in the field.

ATC-20-2: The report, *Addendum to the ATC-20 Postearthquake Building Safety Procedures* was published under a grant from the National Science Foundation and funded by the USGS. Available through the ATC office. (Published 1995, 94 pages)

ABSTRACT: This report provides updated assessment forms, placards, and procedures that are based on an in-depth review and evaluation of the widespread application of the ATC-20 procedures following five earthquakes occurring since the initial release of the ATC-20 report in 1989.

ATC-20-T: The report, *Postearthquake Safety Evaluation of Buildings Training Manual* was developed under a contract with FEMA. Available through the ATC office. (Published 1993, 177 pages; 160 slides)

ABSTRACT: This training manual is intended to facilitate the presentation of the contents of the ATC-20 and ATC-20-1 reports. The training materials consist of 160 slides of photographs, schematic drawings and textual information and a companion training presentation narrative coordinated with the slides. Topics covered include: posting system; evaluation procedures; structural basics; wood frame, masonry, concrete, and steel frame structures; nonstructural elements; geotechnical hazards; hazardous materials; and field safety.

ATC-21: The report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook*, was developed under a contract from FEMA. Available through the ATC office. (Published 1988, 185 pages)

ABSTRACT: This report describes a rapid visual screening procedure for identifying those buildings that might pose serious risk of loss of life and injury, or of severe curtailment of community services, in case of a damaging earthquake. The screening procedure utilizes a methodology based on a "sidewalk survey" approach that involves identification of the primary structural load resisting system and building materials, and assignment of a basic structural hazards score and performance modification factors based on observed building characteristics. Application of the methodology identifies those buildings that are potentially hazardous and should be analyzed in more detail by a professional engineer experienced in seismic design.

ATC-21-1: The report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentation*, was developed under a contract from FEMA. Available through the ATC office. (Published 1988, 137 pages)

> ABSTRACT: Included in this report are (1) a review and evaluation of existing procedures; (2) a listing of attributes considered ideal for a rapid visual screening procedures; and (3) a technical discussion of the recommended rapid visual screening procedure that is documented in the ATC-21 report.

ATC-21-2: The report, *Earthquake Damaged Buildings: An Overview of Heavy Debris and Victim Extrication*, was developed under a contract from FEMA. Available through the ATC office. (Published 1988, 95 pages)

> ABSTRACT: Included in this report, a companion volume to the ATC-21 and ATC-21-1 reports, is state-of-the-art information on (1) the identification of those buildings that might collapse and trap victims in debris or generate debris of such a size that its handling would require special or heavy lifting equipment; (2) guidance in identifying these types of buildings, on the basis of their major exterior features, and (3) the types and life capacities of equipment required to remove the heavy portion of the debris that might result from the collapse of such buildings.

ATC-21-T: The report, Rapid Visual Screening of Buildings for Potential Seismic Hazards Training Manual was developed under a contract with FEMA. Available through the ATC office. (Published 1996, 135 pages; 120 slides)

> ABSTRACT: This training manual is intended to facilitate the presentation of the contents of the ATC-21 report. The training materials consist of 120 slides and a companion training presentation narrative coordinated with the slides. Topics covered include: description of procedure, building behavior, building types, building scores, occupancy and falling hazards, and implementation.

ATC-22: The report, *A Handbook for Seismic Evaluation of Existing Buildings* (Preliminary), was developed under a contract from FEMA. Available through the ATC office. (Originally published in 1989; revised by BSSC and published as the NEHRP Handbook for Seismic Evaluation of Existing Buildings in 1992, 211 pages)

> ABSTRACT: This handbook provides a methodology for seismic evaluation of existing buildings of different types and occupancies in areas of different seismicity throughout the United States. The methodology, which has been field tested in several programs nationwide, utilizes the information and procedures developed for and documented in the ATC-14 report. The handbook includes checklists, diagrams, and sketches designed to assist the user.

ATC-22-1: The report, *Seismic Evaluation of Existing Buildings: Supporting Documentation*, was developed under a contract from FEMA. Available through the ATC office. (Published 1989, 160 pages)

> ABSTRACT: Included in this report, a companion volume to the ATC-22 report, are (1) a review and evaluation of existing buildings seismic evaluation methodologies; (2) results from field tests of the ATC-14 methodology; and (3) summaries of evaluations of ATC-14 conducted by the National Center for Earthquake Engineering Research (State University of New York at Buffalo) and the City of San Francisco.

ATC-23A: The report, *General Acute Care Hospital Earthquake Survivability Inventory for California, Part A: Survey Description, Summary of Results, Data Analysis and Interpretation*, was developed under a contract from the Office of Statewide Health Planning and Development (OSHPD), State of California. Available through the ATC office. (Published 1991, 58 pages)

> ABSTRACT: This report, completed in 1991, summarizes results from a seismic survey of 490 California acute care hospitals. Included are a description of the survey procedures and data collected, a summary of the data, and an illustrative discussion of data analysis and interpretation that has been provided to demonstrate potential applications of the ATC-23 database.

ATC-23B: The report, *General Acute Care Hospital Earthquake Survivability Inventory for California, Part B: Raw Data*, is a companion document to the ATC-23A Report and was developed under the same contract from OSHPD. Available through the ATC office. (Published 1991, 377 pages)

ABSTRACT: Included in this report, completed in 1991, are tabulations of raw general site and building data for 490 acute care California hospitals in California.

ATC-24: The report, *Guidelines for Seismic Testing of Components of Steel Structures*, was jointly funded by the American Iron and Steel Institute (AISI), American Institute of Steel Construction (AISC), National Center for Earthquake Engineering Research (NCEER), and NSF. Available through the ATC office. (Published 1992, 57 pages)

ABSTRACT: This report, completed in 1992, provides guidance for most cyclic experiments on components of steel structures for the purpose of consistency in experimental procedures. The report contains recommendations and companion commentary pertaining to loading histories, presentation of test results, and other aspects of experimentation. The recommendations are written specifically for experiments with slow cyclic load application.

ATC-25: The report, *Seismic Vulnerability and Impact of Disruption of Lifelines in the Conterminous United States*, was developed under a contract from FEMA. Available through the ATC office. (Published 1991, 440 pages)

ABSTRACT: Documented in this report is a national overview of lifeline seismic vulnerability and impact of disruption. Lifelines considered include electric systems, water systems, transportation systems, gas and liquid fuel supply systems, and emergency service facilities (hospitals, fire and police stations). Vulnerability estimates and impacts developed are presented in terms of estimated first approximation direct damage losses and indirect economic losses.

ATC-25-1: The report, *A Model Methodology for Assessment of Seismic Vulnerability and Impact of Disruption of Water Supply Systems*, was developed under a contract from FEMA. Available through the ATC office. (Published 1992, 147 pages)

ABSTRACT: This report contains a practical methodology for the detailed assessment of seismic vulnerability and impact of disruption of water supply systems. The methodology has been designed for use by water system operators. Application of the methodology enables the user to develop estimates of direct damage to system components and the time required to restore damaged facilities to pre-earthquake usability. Suggested measures for mitigation of seismic hazards are also provided.

ATC-28: The report, *Development of Recommended Guidelines for Seismic Strengthening of Existing Buildings, Phase I: Issues Identification and Resolution*, was developed under a contract with FEMA. Available through the ATC office. (Published 1992, 150 pages)

ABSTRACT: This report identifies and provides resolutions for issues that will affect the development of guidelines for the seismic strengthening of existing buildings. Issues addressed include: implementation and format, coordination with other efforts, legal and political, social, economic, historic buildings, research and technology, seismicity and mapping, engineering philosophy and goals, issues related to the development of specific provisions, and nonstructural element issues.

ATC-29: The report, *Proceedings of Seminar and Workshop on Seismic Design and Performance of Equipment and Nonstructural Elements in Buildings and Industrial Structures*, was developed under a grant from NCEER and NSF. Available through the ATC office. (Published 1992, 470 pages)

ABSTRACT: These Proceedings contain 35 papers describing state-of-the-art technical information pertaining to the seismic design and performance of equipment and nonstructural elements in buildings and industrial structures. The papers were presented at a Seminar in Irvine, California in 1990. Included are papers describing current practice, codes and regulations; earthquake performance; analytical and experimental investigations; development of new seismic qualification methods; and research, practice, and code development needs for specific elements and systems. The report also includes a summary of a proposed 5-year research agenda for NCEER.

ATC-30: The report, *Proceedings of Workshop for Utilization of Research on Engineering and Socioeconomic Aspects of 1985 Chile and Mexico Earthquakes*, was developed under a grant from NSF. Available through the ATC office. (Published 1991, 113 pages)

ABSTRACT: This report documents the findings of a 1990 technology transfer workshop in San Diego, California, co-sponsored by ATC and the Earthquake Engineering Research Institute. Included in the report are invited papers and working group recommendations on geotechnical issues, structural response issues, architectural and urban design considerations, emergency response planning, search and rescue, and reconstruction policy issues.

ATC-31: The report, *Evaluation of the Performance of Seismically Retrofitted Buildings*, was developed under a contract from the National Institute of Standards and Technology (NIST, formerly NBS) and funded by the U. S. Geological Survey. Available through the ATC office. (Published 1992, 75 pages)

ABSTRACT: This report summarizes the results from an investigation of the effectiveness of 229 seismically retrofitted buildings, primarily unreinforced masonry and concrete tilt-up buildings. All buildings were located in the areas affected by the 1987 Whittier Narrows, California, and 1989 Loma Prieta, California, earthquakes.

ATC-32: The report, *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations,* was funded by the California Department of Transportation (Caltrans). Available through the ATC office. (Published 1996, 215 Pages)

ABSTRACT: This report provides recommended revisions to the current *Caltrans Bridge Design Specifications* (BDS) pertaining to seismic loading, structural response analysis, and component design. Special attention is given to design issues related to reinforced concrete components, steel components, foundations, and conventional bearings. The recommendations are based on recent research in the field of bridge seismic design and the performance of Caltrans-designed bridges in the 1989 Loma Prieta and other recent California earthquakes.

ATC-35: This report, *Enhancing the Transfer of U.S. Geological Survey Research Results into Engineering Practice* was developed under a contract with the USGS. Available through the ATC office. (Published 1996, 120 pages)

ABSTRACT: The report provides a program of recommended "technology transfer" activities for the USGS; included are recommendations pertaining to management actions, communications with practicing engineers, and research activities to enhance development and transfer of information that is vital to engineering practice.

ATC-35-1: The report, *Proceedings of Seminar on New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice*, was developed under a cooperative agreement with USGS. Available through the ATC office. (Published 1994, 478 pages)

ABSTRACT: These Proceedings contain 22 technical papers describing state-of-the-art information on regional earthquake risk (focused on five specific regions--California, Pacific Northwest, Central United States, and northeastern North America); new techniques for estimating strong ground motions as a function of earthquake source, travel path, and site parameters; and new developments specifically applicable to geotechnical engineering and the seismic design of buildings and bridges.

ATC-R-1: The report, *Cyclic Testing of Narrow Plywood Shear Walls,* was developed with funding from the Henry J. Degenkolb Endowment Fund of the Applied Technology Council. Available through the ATC office (Published 1995, 64 pages)

ABSTRACT: This report documents ATC's first self-directed research program: a series of static and dynamic tests of narrow plywood wall panels having the standard 3.5-to-1 height-to-width ratio and anchored to the sill plate using typical bolted, 9-inch, 5000-lb. capacity hold-down devices. The report provides a description of the testing program and a summary of results, including comparisons of drift ratios found during testing with those specified in the seismic provisions of the 1991 *Uniform Building Code*.

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NCEER-87-0001 "First-Year Program in Research, Education and Technology Transfer," 3/5/87, (PB88-134275, A04, MF-A01).

NCEER-87-0002 "Experimental Evaluation of Instantaneous Optimal Algorithms for Structural Control," by R.C. Lin, T.T. Soong and A.M. Reinhorn, 4/20/87, (PB88-134341, A04, MF-A01).

NCEER-87-0003 "Experimentation Using the Earthquake Simulation Facilities at University at Buffalo," by A.M. Reinhorn and R.L. Ketter, to be published.

NCEER-87-0004 "The System Characteristics and Performance of a Shaking Table," by J.S. Hwang, K.C. Chang and G.C. Lee, 6/1/87, (PB88-134259, A03, MF-A01). This report is available only through NTIS (see address given above).

NCEER-87-0005 "A Finite Element Formulation for Nonlinear Viscoplastic Material Using a Q Model," by O. Gyebi and G. Dasgupta, 11/2/87, (PB88-213764, A08, MF-A01).

NCEER-87-0006 "Symbolic Manipulation Program (SMP) - Algebraic Codes for Two and Three Dimensional Finite Element Formulations," by X. Lee and G. Dasgupta, 11/9/87, (PB88-218522, A05, MF-A01).

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NCEER-87-0008 "IDARC: Inelastic Damage Analysis of Reinforced Concrete Frame - Shear-Wall Structures," by Y.J. Park, A.M. Reinhorn and S.K. Kunnath, 7/20/87, (PB88-134325, A09, MF-A01). This report is only available through NTIS (see address given above).

NCEER-87-0009 "Liquefaction Potential for New York State: A Preliminary Report on Sites in Manhattan and Buffalo," by M. Budhu, V. Vijayakumar, R.F. Giese and L. Baumgras, 8/31/87, (PB88-163704, A03, MF-A01). This report is available only through NTIS (see address given above).

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NCEER-87-0011 "Seismic Probabilistic Risk Assessment and Seismic Margins Studies for Nuclear Power Plants," by Howard H.M. Hwang, 6/15/87, (PB88-134267, A03, MF-A01). This report is only available through NTIS (see address given above).

NCEER-87-0012 "Parametric Studies of Frequency Response of Secondary Systems Under Ground-Acceleration Excitations," by Y. Yong and Y.K. Lin, 6/10/87, (PB88-134309, A03, MF-A01). This report is only available through NTIS (see address given above).

NCEER-87-0013 "Frequency Response of Secondary Systems Under Seismic Excitation," by J.A. HoLung, J. Cai and Y.K. Lin, 7/31/87, (PB88-134317, A05, MF-A01). This report is only available through NTIS (see address given above).

NCEER-87-0014 "Modelling Earthquake Ground Motions in Seismically Active Regions Using Parametric Time Series Methods," by G.W. Ellis and A.S. Cakmak, 8/25/87, (PB88-134283, A08, MF-A01). This report is only available through NTIS (see address given above).

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NCEER-87-0017 "Digital Simulation of Seismic Ground Motion," by M. Shinozuka, G. Deodatis and T. Harada, 8/31/87, (PB88-155197, A04, MF-A01). This report is available only through NTIS (see address given above).

NCEER-87-0018 "Practical Considerations for Structural Control: System Uncertainty, System Time Delay and Truncation of Small Control Forces," J.N. Yang and A. Akbarpour, 8/10/87, (PB88-163738, A08, MF-A01). This report is only available through NTIS (see address given above).

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NCEER-87-0023 "Active Structural Control in Civil Engineering," by T.T. Soong, 11/11/87, (PB88-187778, A03, MF-A01).

NCEER-87-0024 "Vertical and Torsional Impedances for Radially Inhomogeneous Viscoelastic Soil Layers," by K.W. Dotson and A.S. Veletsos, 12/87, (PB88-187786, A03, MF-A01).

NCEER-87-0025 "Proceedings from the Symposium on Seismic Hazards, Ground Motions, Soil-Liquefaction and Engineering Practice in Eastern North America," October 20-22, 1987, edited by K.H. Jacob, 12/87, (PB88-188115, A23, MF-A01).

NCEER-87-0026 "Report on the Whittier-Narrows, California, Earthquake of October 1, 1987," by J. Pantelic and A. Reinhorn, 11/87, (PB88-187752, A03, MF-A01). This report is available only through NTIS (see address given above).

NCEER-87-0027 "Design of a Modular Program for Transient Nonlinear Analysis of Large 3-D Building Structures," by S. Srivastav and J.F. Abel, 12/30/87, (PB88-187950, A05, MF-A01). This report is only available through NTIS (see address given above).

NCEER-87-0028 "Second-Year Program in Research, Education and Technology Transfer," 3/8/88, (PB88-219480, A04, MF-A01).

NCEER-88-0001 "Workshop on Seismic Computer Analysis and Design of Buildings With Interactive Graphics," by W. McGuire, J.F. Abel and C.H. Conley, 1/18/88, (PB88-187760, A03, MF-A01). This report is only available through NTIS (see address given above).

NCEER-88-0002 "Optimal Control of Nonlinear Flexible Structures," by J.N. Yang, F.X. Long and D. Wong, 1/22/88, (PB88-213772, A06, MF-A01).

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NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

Headquartered at the State University of New York at Buffalo

State University of New York at Buffalo
Red Jacket Quadrangle
Buffalo, New York 14261
Telephone: 716/645-3391
FAX: 716/645-3399

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