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Report on Analysis and Design of Seismic- Resistant Concrete Bridge Systems

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Report on Analysis and Design of Seismic-Resistant Concrete Bridge Systems

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Report on Analysis and Design of Seismic-Resistant Concrete Bridge Systems

Reported by ACI Committee 341

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This report is intended for use by practicing engineers and provides a summary of the state-of-the-art analysis, modeling, and design of concrete bridges subjected to strong earthquakes. It is intended to supplement and complement existing documents from the American Association of State Highway and Transportation Officials (AASHTO), California Department of Transportation (Caltrans), and various building codes and guidelines. Procedures and philosophies of codes and guidelines are summarized. Linear and nonlinear seismic analysis methods are also discussed, and important modeling considerations for different bridge elements, including curved girders and skewed abutments, are highlighted. The report also includes a summary of general seismic-resistant design and construction considerations for concrete bridges, as well as analysis and design considerations for bridges with seismic isolation.

Keywords: abutment; bridge; column; connections; design; earthquake; footing; girder; hinge; restrainer; seismic; seismic analysis; seismic isolation.

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CHAPTER 1—INTRODUCTION

1.1—General

The stated objectives of seismic design provisions in major codes have evolved considerably over the last 20 years. The initial focus of preventing structural collapse under the design earthquake to prevent loss of life has shifted to broader design objectives, such as achieving a level of serviceability following a major earthquake that allows for emergency response and ensures that transportation lifelines remain operational. These newer design objectives focus on the need for structures to remain operational after an earthquake, particularly for structures important to emergency response and those housing emergency and high-risk facilities. Critical structures include bridges on key response routes, hospitals, public safety headquarters, communication centers, and nuclear power stations.

Bridge seismic design philosophies may use a traditional single seismic design level (AASHTO 2012; AASHTO LRFDSEIS-2-M) or a two-level approach (MCEER-ATC-49) where both functional-level and safety-level hazards are considered. Performance objectives for each level are composed of a performance level or functional requirement at a seismic hazard level. The functional-level event considered in this two-level approach is typically a lower-level event

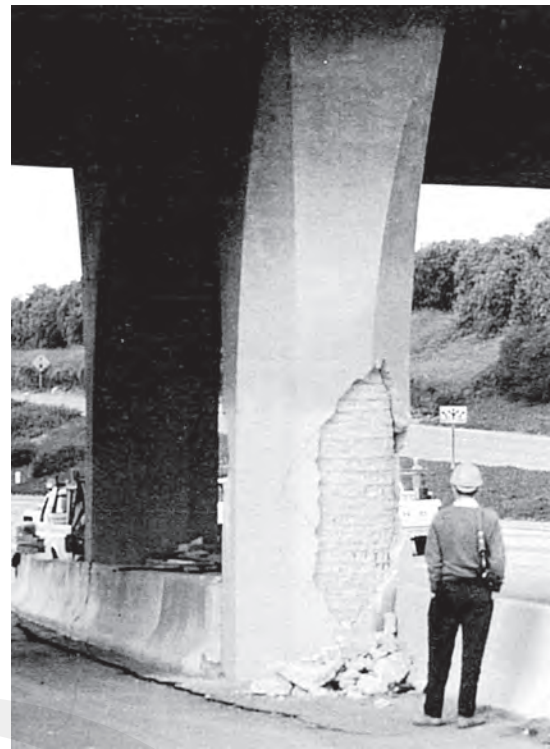


Fig. 1.1—Acceptable damage (spalling of cover concrete) to a bridge column for large earthquake.

with relatively high probability of exceedance (PE), and the safety-level event is typically a major seismic event with a very low PE. The typical performance objectives for the two-level approach tolerate only slight damage to ensure uninterrupted service of the bridge under the lower-level event, and allow only easily repairable damage under the higher-level event to ensure minimal or no disruption of lifelines.

In setting minimum performance standards, design codes recognize that it is not practical to design a structure to resist a large earthquake elastically; therefore, some degree of damage is typically permitted under the higher-level event (Fig. 1.1). For critical structures, however, depending on expectations of how quickly the particular structure can be put back in service and repaired, the damage can be further restricted by tighter requirements defined by the owner.

Design performance level requirements have become more general and are not always tied to traditional notions of force and strength. Thus, analysis requirements have also evolved beyond the traditional methods involving equivalent static forces representing the design event. The extent of damage in different bridge components is commonly quantified using performance quantities such as strains, curvatures, and displacements. Limiting damage requires imposing appropriate limits on these parameters in the critical sections of the structural members. In addition, the response of the structural system should be evaluated as a whole to assess functionality and operability. This requires a higher level of sophistication in both system modeling as well as sectional and material-level analysis. Reinforced concrete structural members, in particular, require greater attention to detail when moving beyond elastic or equivalent elastic analysis.



Fig. 1.2a—1971 San Fernando Earthquake.

because of the interaction of concrete and reinforcing bar, nonhomogeneity of the concrete material, and the progression of cracking and yielding of the section with increasing strains. For example, a pushover analysis accounting for the pier or bent moment-curvature relationships at different axial loads is commonly used to develop a better understanding of the nonlinear behavior of the structure and the type of damage that might be expected.

For bridge structures, damage permitted under the design seismic event is limited primarily to elements with ductile capacity that can experience dependable flexural inelastic response such as the columns or pier walls. In addition, nominal damage may be tolerated in other parts of the bridge such as at the abutment, shear keys, and in-span hinges or expansion joints. These bridge elements are easy to inspect and repair, should damage be sustained during a seismic event. The use of a capacity design approach is also intended to prevent damage to elements, such as foundation piling, that are difficult to inspect and repair following a seismic event. Acceptable damage depends on the parameters discussed previously and the expectations of the bridge owners and stakeholders; however, in all cases, loss of girder support, column failure, foundation failure, and connection failure are unacceptable.

Performance-driven design requirements, especially for structures in areas of high seismicity, make the modeling of the bridge structural system very important. The bridge structural system being modeled should not only include the columns, but also the abutment and foundation systems. Modeling should account for interaction between these different components (for example, superstructure impact on abutment gap closure) as well as with the surrounding soil (for example, transmission of base rock motion to the foundation elements through the surrounding soil).

Although the discussions presented herein are in principle applicable to all bridges, the intent is to address short- and medium-span bridges with span lengths less than 500 ft (150 m). Long-span and specialty bridges involving additional design considerations are outside the scope of this document. The information presented in this document has largely been extracted from design specifications, codes, and



Fig. 1.2b—1999 Ji-Ji Earthquake Taiwan (USGS 2008a).



Fig. 1.2c—Arterial bridge in Kobe, Japan, earthquake.

other references. This document should be considered as a guide to be used by a responsible design professional with the proper background and in conjunction with experience and judgment of the designer.

1.2—Lessons learned from earthquake damage to bridges

Earthquakes over the last 40 years, including the 1971 San Fernando, 1989 Loma Prieta, and 1994 Northridge earthquakes in the United States; the 1995 Hyogo-Ken Nanbu Earthquake in Japan; the 1999 Ji-Ji Earthquake in Taiwan; and the 1999 Kocaeli Earthquake in Turkey, are reminders of the vulnerability of existing bridges and their impacts on society when they perform poorly during earthquakes (Chen and Duan 2014; Yashinsky 2000). Collapse or severe damage to several bridges occurred in each of these events, some examples of which are shown in Fig. 1.2a, 1.2b, and 1.2c. Although research to improve bridge seismic performance has been ongoing for several decades, research with significant experimental emphasis intensified in the United States following the 1989 Loma Prieta Earthquake (Housner 1990; Caltrans 1991, 1993, 1994, 1996, 1998, 2001; FHWA 1995, 1997, 2000; Astaneh-Asl and Roberts 1993, 1997; Housner 1994; Priestley et al. 1996). A similar trend was followed in several other countries around the world, including Japan, New Zealand, and Taiwan (Kawashima and Unjoh 1995; Park 1994).



Fig. 1.2.1a—Unseating of bridge spans, Kocaeli earthquake (Turkey) (Erdik 2008).



Fig. 1.2.1b—Unseating of Nishinomiya-Ko Bridge approach span during 1995 Kobe earthquake. (University of Washington 2008).

Experimental bridge research initially focused on evaluating the expected performance of critical components, such as columns, cap beams, joints, and foundation elements, with typical standard details. Next, the focus of research shifted to studying cost-effective means for improving seismic performance of poorly detailed bridge components and implementing the outcomes of these research studies to strategically retrofit the most vulnerable bridges in high seismic regions. Finally, the third phase of research has begun, in which the focus has been on the design of critical bridge components based on alternative and efficient details, innovative materials, new concepts such as performance-based design, or any combination of these.

Observations of damage to existing bridges caused by earthquakes and research investigations have confirmed that the damage to bridge components or the entire bridge can take several different forms. The specific details—input ground motion, conditions of foundation soils, structural condition, configuration, and structural redundancy—can contribute to the seismic response of the bridge (Chen and Lui 2005). Generally, there is little evidence that bridge damage was initiated by damage to superstructure or foundation elements, although it should be noted that limited inspection of foundations typically occurs after a damaging earthquake. Most common types of bridge failure have stemmed from one of the following.



Fig. 1.2.1c—Unseating of bridge deck, JiJi earthquake (Kochi University of Technology 2008).



Fig. 1.2.1d—Unseating of curved bridge, I-5 Antelope Valley Freeway collapse, Northridge earthquake (USGS 1996).

1.2.1 Unseating of bridge decks—The lack of seismic design considerations in older bridges commonly resulted in narrower supports that were susceptible to unseating. In addition, unseating of bridge decks in earthquakes has also been commonly attributed to the improper use of elastic methods in design that may underestimate lateral displacements due to the assumption of linear behavior, use of gross section properties of columns, and underestimation of ground movement (Priestley et al. 1996). Unseating of bridge decks in earthquakes can be exacerbated in curved, skewed, or complex bridges. Figure 1.2.1a, 1.2.1b, and 1.2.1c show unseating of bridge decks in various earthquakes, whereas Fig. 1.2.1d and 1.2.1e depict similar damage to a curved and skewed bridge.



Fig. 1.2.1e—Unseating of skewed bridge, freeway collapse, Northridge earthquake (Heney et al. 1999).



Fig. 1.2.2a—Inadequate confinement, 1971 San Fernando earthquake (USGS 2008b).



Fig. 1.2.2b—Shear failure, 1994 Northridge earthquake (FEMA 2009a).

1.2.2 Nonductile failure of columns—The main causes of nonductile column failure in past earthquakes have been due to insufficient confinement reinforcement, inadequate shear



Fig. 1.2.2c—Anchorage failure of column bars, 1971 San Fernando earthquake (Priestley et al. 1996).



Fig. 1.2.3—Bridge joint damage, 1989 Loma Prieta earthquake (Priestley et al. 1996).

design, or poor anchorage of column longitudinal reinforcement into the adjoining cap or foundation member (Priestley et al. 1996; ACI 341.3R). Examples illustrating each failure mode are shown in Fig. 1.2.2a, 1.2.2b, and 1.2.2c. In all cases, the column failure led to collapse or near-collapse of the bridge and replacement of the bridge following the earthquake was inevitable.

1.2.3 Failure of column-beam joints—Failure of bridge joints can easily lead to damage (Fig. 1.2.3) and collapse of bridges. Joint failures in concrete bridges typically occur due to insufficient anchorage of column reinforcement, beam longitudinal reinforcement, or both; insufficient shear reinforcement; or inadequate splicing between the column and beam longitudinal reinforcement (Sriharan and Ingham 2003). The potential for joint failure, which is nonductile in nature, is more prevalent in older bridges due to the common design practice of not using joint shear reinforcement until the 1970s. Inadequate detailing of bridge columns, however, has limited the demand imposed on joints in past earthquakes, resulting in premature column failure and thereby not exposing the potential inadequacy of details used in more recent bridge joints.



Fig. 1.2.4a—I-880 Cypress double deck viaduct before failure.



Fig. 1.2.4b—I-880 Cypress double deck viaduct after failure, 1989 Loma Prieta earthquake.



Fig. 1.2.4c—Aerial view of 5/14 Interchange, Los Angeles, CA, 1994 Northridge earthquake (FEMA 2009b).

1.2.4 Failure resulting from complex nature of structures—

In addition to the routine failure of bridges discussed previously, collapse or severe damage to bridges has occurred due to the complex nature of the structure (Priestley et al. 1996; ACI 341.3R). The complexity has been typically introduced through geometry (for example, use of C-bent, outrigger bent, flared column), nonuniform stiffness distribution, and

multiple-deck configuration. Figures 1.2.4a, 1.2.4b, and 1.2.4c show examples of bridge damage resulting from the complex nature of structures.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A_{jh}	=	effective horizontal joint area, in. ² (mm ²)
A_{jv}	=	effective vertical joint area, in. ² (mm ²)
B_{cap}	=	bent cap width, in. (mm)
C_v	=	dead load multiplier
C_s	=	elastic seismic response coefficient
D_c	=	cross-sectional dimension of column in the direction of bending, in. ² (mm ²)
D_s	=	depth of superstructure at the bent cap, in. (mm)
DR	=	dead-load or permanent load reaction, lb (N)
d	=	tip axial displacement, in. (mm)
h	=	distance from the center of gravity of tensile force to the center of gravity of compressive force within a column section, in. (mm)
k_s	=	subgrade reaction, lb/in. ² /in. (MPa/m)
L	=	length, in. (mm)
L_{ac}	=	length of column reinforcement embedded into the bent cap, in. (mm)
M_o^{col}	=	column overstrength moment, in.-lb (N-mm)
P_c	=	column axial force including overturning effects, lb (N)
P_b	=	beam axial force at the center of the joint including prestressing, lb (N)
Q	=	axial resistance at the pile tip, lb (N)
T	=	soil characteristic length, in. (mm)
T_c	=	column tensile force defined as M_o^{col}/h , lb (N)
t	=	axial strength per unit of length, lb/in. (N/mm)
z	=	axial displacement as a nonlinear function of the tip axial displacement d , resulting in the Q - d curves, in. (mm)

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, “ACI Concrete Terminology,” <http://www.concrete.org/tools/ConcreteTerminology>. Definitions provided herein complement that resource.

girder seating—distance from the end of the girder to the face of the supporting element.

beam-column joint—interface region of column that extends into the supporting foundation or cap element.

seismic force levels—threshold-inertia-derived force levels set by governing bodies for determining the appropriate capacity, in terms of strength and ductility of a structure, to resist the design earthquake; designers and managing entities may choose single or multiple seismic force levels for the structure’s design based on behavior objectives and experience.

pushover analysis—method of nonlinear analysis whereby the loading event is scaled and the structure’s element behavior is modified when events such as member cracking, hinge formation, and soil yielding occur.

CHAPTER 3—CODES

3.1—Historical perspective

The first U.S. code specifically addressing highway bridge design was published in 1931 by the American Association of State Highway and Transportation Officials (AASHTO). That code, and subsequent editions before 1941, did not address seismic design. While some later AASHTO specifications mentioned seismic loading, they gave no guidance or criteria as to how the earthquake forces were to be determined or applied to the structure.

The California Department of Transportation (Caltrans) was the first organization within the United States to develop specific seismic design criteria for bridges. In 1965, the Structural Engineers Association of California (SEAOC) adopted provisions with building seismic force levels that varied according to the structure type (Shea 1959). Following the 1971 San Fernando earthquake, which caused several freeway structures to collapse, Caltrans made substantial amendments to their seismic design criteria and more stringent seismic force levels were introduced. Research was conducted that helped develop a more scientifically-based seismic code, including ground motion attenuation, soil effects, and structural dynamic response. Those efforts led to development of the ARS Spectra, where A, R, and S refer to the maximum expected bedrock acceleration (A), the normalized rock response (R), and the amplification ratio for the soil spectrum (S).

3.1.1 No-collapse-based design—As discussed by Chen and Lui (2005), basic no-collapse design philosophy is to prevent bridge collapse during severe earthquakes (Caltrans 1990; MCEER-ATC-49) that have small probabilities of occurrence during the useful life of the bridge. The ATC-6 recommendations, the first widely accepted force-based approach, incorporated an elastic response spectrum analysis. Response modification factors (*R*-factors) were used to reduce the elastic forces and considered redundancy, ductility, and over-strength provided by the various structural systems. In the ATC-6 guidelines, the primary departure from previous practice was: 1) an emphasis on ductile details to enable safe accommodation of plastic deformations; 2) an emphasis on realistic survivability of structures with minor damage allowed; and 3) specification of girder seating requirements that were substantially longer than practiced at the time. Later, Caltrans (1990) and ATC-32 used an adjustment (*Z*-factor) for ductility and risk assessment that was similarly applied to the elastic forces derived from elastic response spectrum analysis. ATC-32 sought to provide the design community with seismic design criteria that could be applied uniformly to all bridges. An alternative displacement-based approach (Caltrans 1995a, 2004b, 2006; AASHTO LRFDSEIS-2-M), defined by element and frame ductility, has been developed to prevent bridge collapse.

3.1.2 Performance-based design—Following the 1989 Loma Prieta earthquake, bridge engineers faced three essential challenges (Housner 1990):

1. Ensuring that the vulnerability of new bridge construction under earthquake risks is minimized

2. Identifying and upgrading unacceptable details that may make existing bridges vulnerable to seismic events

3. Formulating and rapidly implementing effective and economical response mechanisms for preserving structural integrity in damaging earthquakes

Chen and Lui (2005) states that “performance-based, project-specific criteria (Caltrans 1997, 1999b; IAI 1995) and design memoranda (Caltrans 1995a) have been developed and implemented for the design and retrofitting of important bridges in California. These performance-based criteria include guidelines for development of site-specific ground motion estimates, ductile design details to preclude nonductile failure modes, rational procedures for beam-column joint shear design, and the definition of limit states for various performance objectives (Housner 1994). The performance-based criteria usually require a two-level design approach. The first level of design is to ensure the performance (that is, serviceability) of a bridge in small-magnitude earthquake events that may occur several times during the life of the bridge. The second level of design is to achieve the performance (no collapse) of a bridge under severe earthquakes that have only a small probability of occurrence during the useful life of the bridge.”

CHAPTER 4—SEISMIC HAZARDS

4.1—Introduction

The advances in plate tectonics theory in the 1960s and the ensuing decades of intensive research in earth sciences has produced an understanding of the long-term earthquake potential of most parts of the world, especially near plate boundaries. With few exceptions, however, moderate to great earthquakes do not appear to occur at uniform time intervals, nor do successive earthquakes rupture exactly the same segment of a fault. Consequently, even at sites near the best-documented faults, it is difficult to predict where and when the next damaging earthquake is going to occur. Thus, a probabilistic approach versus a deterministic approach to seismic hazard evaluation is generally most compatible with the state of knowledge of earthquake potential.

For other regions where the locations of potentially active faults are poorly known or unknown, there is even greater uncertainty regarding seismic potential. At any particular site, there is also a large degree of random variability in the level of the strong ground motion shaking that a specified earthquake will generate, and this usually dominates the uncertainty in the seismic hazard estimate. In addition, there are other site-specific hazards associated with fault rupture, such as permanent soil or ground deformation, landslides, and tsunami wave generation.

For some purposes and sites, the expected ground motions and seismic hazard are derived using a deterministic approach (based on a single earthquake), and this approach may complement a probabilistic approach. For example, seismic hazards for the design of critical facilities are sometimes characterized using a deterministic approach. Deterministic estimates of ground motions are typically based on a single controlling earthquake whose magnitude and closest

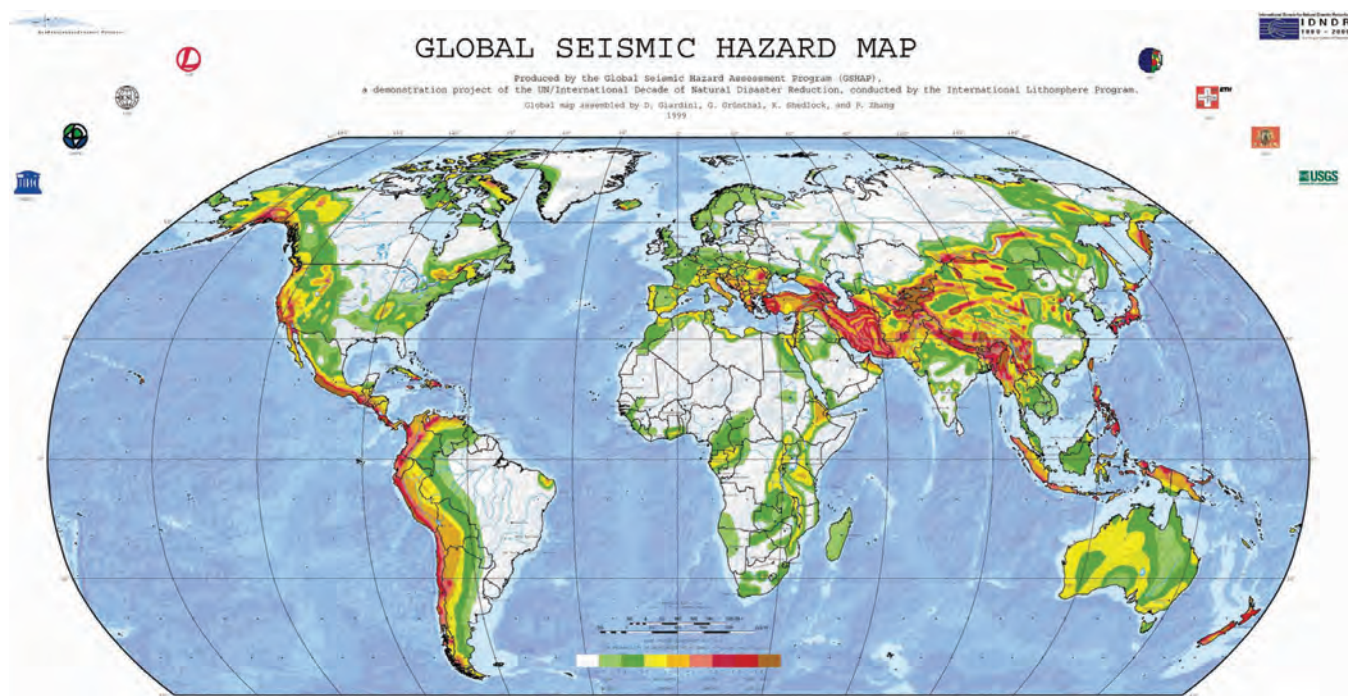


Fig. 4.1—Global seismic hazard map (GSHAP 2007).

distance are specified. Because the ground motions from only one earthquake are considered, the estimated ground motions are very sensitive to detailed aspects of the earthquake source, the wave propagation path between the source and the site, and the site response. Uncertainties in these input parameters give rise to uncertainty in the estimated ground motions. In particular, this approach requires a decision whether to use the median ground motion level for the combination of magnitude and distance represented by the scenario earthquake, or some higher statistical percentile. Thus, the deterministic approach has limitations because it cannot provide an upper bound on the ground motion level, nor can it avoid making assumptions about its frequency of occurrence.

Uncertainties in each of the input parameters can have a large effect on the results and thus need to be reflected in the analysis. These include:

- a) The location, geometry, and style of faulting and maximum magnitude of earthquake sources
- b) Fault slip rates and earthquake recurrence relationships
- c) Ground motion prediction methods

For most engineering purposes, it is more meaningful to use a probabilistic approach than a scenario earthquake to characterize the ground motion that a given site will experience in the future. Probabilistic seismic hazard analysis takes into account the ground motions from the full range of earthquake magnitudes that can occur at each fault or source zone affecting the site, and their frequency of occurrence. This is done through the use of logic trees (Kulkarni et al. 1984), which assign probability values to alternative values of these parameters. From the hazard curves, response spectra corresponding to any specified return period can be constructed.

Probabilistic seismic hazard analysis also accounts for the random variability in ground motion level about the median value for each magnitude-distance pair. This information is numerically integrated using probability theory to produce the annual frequency of exceedance of each different ground motion level (Cornell 1968; Kulkarni et al. 1979). Knowing this frequency enables designers and owners to make rational decisions about the trade-off between hazards and costs.

Ground motion hazard maps play an extremely important role in urban planning and mitigation activities. A probabilistic ground shaking hazard map of the world, showing the peak acceleration having a return period of 500 years, is shown in Fig. 4.1. Most of the earthquakes occur around the rim of the Pacific Ocean, with many earthquakes also occurring in the Alpine belt extending from the Mediterranean region, eastward through Turkey, Iran, northern India, and China. As shown in Table 4.1, the United States Geological Survey (USGS 2014) estimates that approximately 500,000 earthquakes are recorded worldwide each year. Approximately 100,000 of these will be felt. Of those, 100 to 150 will have a magnitude higher than 6.0 and will produce structural damage. In the United States, the seismic hazard is moderate to high for locations near plate boundaries in California, Nevada, Oregon, Washington, and Alaska. Also, major intraplate earthquakes (within the North American plate) have historically occurred near New Madrid, MO, and Charleston, SC.

4.2—Probabilistic seismic hazard analysis

4.2.1 Physical assessments and hazard levels—The three primary categories of earthquakes contributing to ground shaking hazards in the United States are earthquakes occur-

Table 4.1—Worldwide frequency of earthquake activity (USGS 2014)

Descriptor	Richter magnitude	Number of earthquakes per year	
Great	8 and higher	1	Damage
Major	7 to 7.9	15	
Strong	6 to 6.9	134	
Moderate	5 to 5.9	1319	No damage
Light	4 to 4.9	13,000 (estimated)	
Minor	3 to 3.9	130,000 (estimated)	
Very minor	2 to 2.9	1,300,00 (estimated)	

ring in the shallow crust, within subducting slabs, and on the subduction interface between a subducting plate and an overriding plate. Crustal earthquakes can potentially occur anywhere in the United States, whereas interface and slab earthquakes are confined to the subduction zones of the Pacific Northwest and Puerto Rico.

In a probabilistic seismic hazard evaluation for a given site, it is necessary to identify the seismic sources where future earthquakes are likely to occur, to characterize the magnitudes and frequency of occurrence of earthquakes at each seismic source, and to identify the distance and orientation of each seismic source in relation to the site. The expected frequencies of occurrence of earthquakes of various magnitudes can be derived from one or more of the following three distinct categories of data.

1. Historical seismicity is most reliable in tectonically active regions that have a relatively long historical earthquake record, and may be applicable to both individual faults and to a region. Historical seismicity can also be used to constrain the distribution of earthquake magnitudes, especially in the low magnitude range.

2. Slip rates of active faults are usually only available in tectonically active regions, and are applicable to individual faults. Additional geological data, such as slip per event and average return period of events, may be used to constrain the distribution of earthquake magnitudes, especially in the high magnitude range.

3. Geodetic strain rates are, in principle, available in all tectonic environments and can be used to constrain the earthquake occurrence rate over an extended region. Additional information is needed to partition the geodetic strain rate among individual faults, and to constrain the distribution of earthquake magnitudes.

The return period of an earthquake at a specific fault is the average interval of time between successive earthquakes of specified magnitude. It is equal to the reciprocal of the annual rate of occurrence of earthquakes of that magnitude on that fault. The return period of a ground motion level at a specific site is the interval of time between successive occurrences of that ground motion level at that site. It is equal to the reciprocal of the annual rate of occurrence of that ground motion level.

4.2.2 Ground motion prediction equations—A set of ground motion prediction equations for shallow crustal earthquakes in tectonically active regions (such as the Western

United States) has been developed by the Pacific Earthquake Engineering Research (PEER) in partnership with the USGS and Southern California Earthquake Center (SCEC). These models were derived from a very large and carefully documented data set of strong ground motion recordings, including recordings from recent earthquakes such as the 1999 Kocaeli, 1999 Ji-Ji, and 2000 Denali earthquakes. The vastly larger size of the data set that has been used, and the diligence that has been applied by the modelers, has resulted in a set of ground motion models that has a much more substantial basis than the 1997 generation of models (Abrahamson and Shedlock 1997). The NGA ground motion models were used in generating the 2008 USGS National Seismic Hazard Maps (Petersen et al. 2008). Ground motion prediction equations for the latter two regions are described in Abrahamson and Shedlock (1997).

4.3—Multi-level earthquake ground motions

Various organizations have recognized that an economical bridge design can be achieved by targeting near-elastic behavior of the bridge for a ground motion having a relatively short return period, such as 50 years, from an expected earthquake. This ensures survivability of the bridge for a ground motion having a relatively long return period such as 2500 years (from a rare earthquake), referred to as the two-level design approach.

An expected earthquake ground motion with a 50-year return period is quite likely to occur during a structure's design life and, for this ground motion level, the structure is designed to remain essentially elastic without experiencing any plastic strains and should not sustain any damage requiring repair. Additionally, the traffic-carrying capacity of the structure should not be interrupted or compromised during or after the expected earthquake ground motion.

The maximum considered earthquake (MCE) ground motion is used in the International Building Code (IBC) (International Code Council 2006a) to describe the largest earthquake ground motion that need be considered for design and, in most locations, represents the ground motion having a return period of 2500 years. A bridge is designed to survive this rare earthquake ground motion by allowing formation of plastic hinges and associated damage (ductile response), but the structure remains stable and, in some cases, repairable. The MCE was previously used to describe this event, but with the understanding of the probabilistic nature of earthquake ground motions, as described in 4.1, this term is not applicable.

4.4—USGS probabilistic ground motion maps and design value maps

The USGS generates maps of probabilistic ground motions for the United States for the following three probabilities of exceedance (PE): 10 percent PE in 50 years, 5 percent PE in 50 years, and 2 percent PE in 50 years, with approximate return periods of 500, 1000, and 2500 years, respectively. The USGS provides maps of these ground motions for peak acceleration and response spectral acceleration at periods of 0.2 and 1 second, and also provides probabilistic hazard

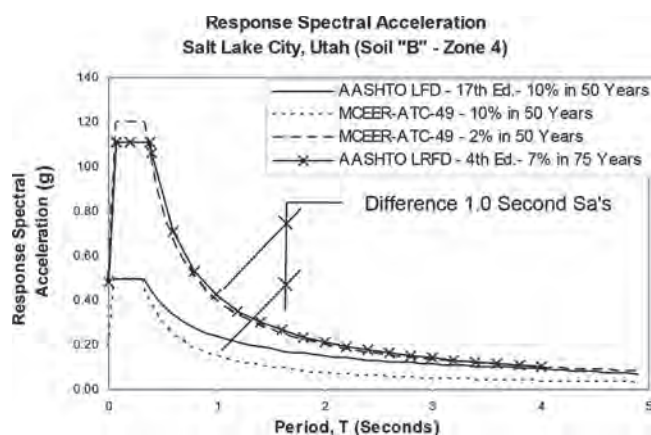


Fig. 4.4—Response spectral accelerations for AASHTO HB-17 and AASHTO (2012).

curves and uniform hazard response spectra at any location in the United States.

Design value maps are derived from the USGS probabilistic seismic hazard maps, but in some regions, a floor or ceiling ground motion level is applied. The USGS (2013) has developed a program that provides hazard curves, uniform hazard response spectra, and design parameters for all 50 of the United States, Puerto Rico, and the U.S. Virgin Islands. Design parameters are also available for Guam and American Samoa. Parameters are searchable by zip code or latitude and longitude, can be graphed, saved, and printed for later use. The design values that are addressed also include the National Institute of Building Sciences (2003); ASCE 7; IBC (International Code Council 2006a); the International Residential Code (International Code Council 2006b); and NFPA 5000.

AASHTO HB-17 is based on USGS contour maps from 1988, giving the acceleration coefficient for a 10 percent PE within a 50-year return period. The seismic accelerations from the AASHTO maps give the spectral response acceleration value for a 0.2 second period with 5 percent critical damping. The provisions of AASHTO (2012) use the 5 percent in 50-year USGS data sets from the following regions: 48 contiguous states (2002), Alaska (2006), Hawaii (1998), and Puerto Rico and the Virgin Islands (2003). Because the design life of any new bridge structure is expected to be 75 years, the AASHTO (2012) maps are labeled with a PE of 7 percent in 75 years that is approximately equal to the 5 percent in 50-year data (approximately 1000-year return period).

The response spectra in AASHTO HB-17 and AASHTO (2012) for a selected location (Salt Lake City, UT) are compared in Fig. 4.4. This figure shows that the response spectral accelerations for AASHTO (2012) are significantly higher for periods of approximately 2 seconds and less when compared with those from AASHTO HB-17. The response spectral accelerations show an increase due to the longer return period for the design earthquake.

4.5—Vertical accelerations

Measurements of earthquake ground motions indicate that, during a seismic event, structures are subjected to simultaneous ground motions in three orthogonal directions. In the past, it was common engineering practice to estimate the vertical response spectrum by multiplying the horizontal response spectrum ground motions by a factor of two-thirds. Studies (Abrahamson and Silva 1997; Silva 1997; Campbell and Bozorgnia 2000) show the vertical response is dependent on many variables and the two-thirds ratio may be conservative for long-period motions (greater than 0.3 seconds), and unconservative for other cases, such as in the near-fault region.

The AASHTO HB-17 and AASHTO (2012) specifications do not present guidelines on the effects of vertical ground motions. The provisions of MCEER-ATC-49 state that the effects of vertical ground motion may be ignored if the bridge site is more than 30 miles (50 km) from an active fault. This generally implies that vertical acceleration need not be considered for bridges in the Central and Eastern United States because few active faults have been located accurately in this part of the country.

For bridge locations within 30 miles (50 km) of faults, however, MCEER-ATC-49 provides guidance related to vertical acceleration effects in terms of a dead load multiplier C_v . For various distances and for magnitudes above and below 7.0, values of the dead load multiplier C_v are given for different structural components. The magnitude referenced herein is based on the moment magnitude—not the Richter or local magnitude—and the maximum moment magnitude should be estimated by a knowledgeable geologist or seismologist.

4.6—Near-fault ground motions and residual ground displacements near faults

An earthquake occurs when elastic strain that has gradually accumulated across a fault is suddenly released in the process of elastic rebound. The elastic energy stored on either side of the fault drives the motion on the fault. The elastic rebound generates dynamic ground motions that last for a few seconds to a few minutes, constituting a primary seismic hazard. The elastic rebound also generates residual deformation of the ground. The residual deformation of the ground consists of a discontinuity in displacement on the fault itself, and a gradual decrease in this displacement away from the fault on either side of the fault. If there is surface faulting, the residual displacements are discontinuous across the fault at the ground surface, constituting a primary seismic hazard. Even if the fault does not break the surface, there may be residual deformation of the ground surface due to subsurface faulting.

Near-fault ground motions differ from more distant ground motions in that they often contain strong coherent dynamic long-period pulses and permanent ground displacements. The dynamic motions are often dominated by a large long-period pulse of motion that occurs along the horizontal component that is perpendicular to the orientation of the fault, caused by rupture directivity effects (Somerville et

al. 1997; Abrahamson 2000). Near-fault recordings from recent earthquakes indicate that this pulse is a narrow-band pulse with a period that increases with magnitude (Somerville 2003a). This magnitude dependence of the pulse period causes the response spectrum to have a peak with a period that increases with magnitude, such that the near-fault ground motions from moderate magnitude earthquakes may exceed those of larger earthquakes at intermediate periods (approximately 1 second).

The residual ground displacements in near-fault ground motions, caused by the relative movement of the two sides of the fault on which the earthquake occurs, are discontinuous across a fault having surface rupture, and can subject a bridge crossing a fault to significant differential displacements. The residual ground displacements occur at approximately the same time as the large dynamic motions, indicating that the residual and dynamic displacements need to be treated as coincident loads.

Special aspects of the engineering description of near-fault ground motions and residual displacements have been outlined by Somerville (2003b). These near-fault ground motions may have residual ground displacements, and the fault-normal component of ground motion may be larger than the fault parallel component. Consequently, at near-fault locations, the usual treatment of the ground motion components as having random orientation, described in 4.8, may need to be modified. In particular, the ground motions should be specified in their fault-normal and fault-parallel orientations, and these ground motion components are not interchangeable because, in general, they will have different response spectral levels, different waveforms, and different residual ground displacements.

Residual ground displacements due to faulting can be estimated using probabilistic procedures such as those used for probabilistic ground motion hazard analysis. Such procedures are described by Youngs et al. (2003), and guidelines for their application to bridges are described by Ashford et al. (2001).

4.7—Load combinations

AASHTO (2012) requires that bridge components be designed to withstand the forces resulting from each of the specified load combinations. In accordance with AASHTO (2012), the load group that accounts for seismic loading is:

$$\text{Extreme Event I} = \gamma_p \cdot \text{permanent loads} + 1.00 \cdot \text{WA} + \gamma_{EQ} \cdot \text{live loads} + 1.00 \cdot \text{EQ}$$

where

permanent loads = DC, DD, DW, EH, EV, ES, EL, PS, CR, and SH;

WA = water load and stream pressure;

live loads = LL, IM, CE, BR, PL, and LS;

EQ = earthquake load

γ_p = load factor for permanent loads per AASHTO (2012)

γ_{EQ} = load factor for live loads

The AASHTO (2012) commentary suggests γ_{EQ} , the load factor applied to live loads for the Extreme Event I load case

equal to 0.50, is reasonable for a wide range of average daily truck traffic. This issue, however, is still not resolved, and it is suggested that the value of γ_{EQ} be determined on a project-specific basis.

4.8—Combining effects of orthogonal components of earthquakes

Generally, it is recommended, as stated by both AASHTO HB-17 and MCEER-ATC-49, to use a complete quadratic combination (CQC) or square root sum of the squares (SRSS) combination (also discussed in 5.1.1) to obtain a summary of the individual modal forces for each orthogonal direction. The combination of orthogonal modes may be obtained by using 100 and 30 percent (AASHTO 2012) for the individual directions. This combination accounts to some degree for the fact that the true direction of the design earthquake is not entirely known. These load combinations should be used for design of all substructure and foundation elements. For near-fault ground motions, the fault-normal and fault-parallel horizontal ground motions have systematic differences as described in 4.6, and these components of ground motion should be applied in their actual orientations, resolved into the longitudinal and transverse directions. These ground motions may include residual ground displacements.

As noted in 5.5, when considering plastic hinging of columns or pier walls as permitted by AASHTO (2012), Seismic Performance Zones (SPZs) 3 and 4, the column, pier wall, and foundation elements should be proportioned in the direction of the hinging considering the forces resulting from plastic hinging in that direction only, and the above load combinations need not be considered. The specified combinations, however, should be used in the strong direction of a wall-type concrete pier that is designed as a column.

4.9—Ground motion time histories

Depending on the analysis and design requirements, the design ground motions may need to be specified not only as response spectra but also by suites of strong motion time histories for input into time domain nonlinear analyses of structures. The probabilistic response spectra represent the aggregated contribution of a range of earthquake magnitudes occurring at various rates on each of several discrete faults or seismic source zones located at various distances from the site, and include the effect of random variability in the ground motions for a given magnitude and distance. To provide ground motion time histories that represent the response spectrum, one or more discrete combinations of magnitude, distance, and severity (epsilon) should be chosen to represent the probabilistic ground motion. The parameter epsilon is defined as the number of standard deviations above or below the median ground motion level predicted by a ground motion prediction equation for the given magnitude, distance, and period. The magnitude, distance, and epsilon values are estimated through disaggregation of the probabilistic seismic hazard (Bazzurro and Cornell 1999; Chapman 1995; Cramer and Peterson 1996; Harmsen et al. 1999; McGuire 1995).



Documents

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4.10—Geotechnical considerations

Aside from determining the appropriate response acceleration coefficients for the bridge, the most significant effect on the bridge response is related to the modifications that should be applied to the acceleration coefficients to account for local soil conditions.

As the soil conditions deteriorate from hard rock to dense and stiff soils to loose soils and peat, the potential for soil amplification of the design earthquake increases. **AASHTO (2012)** accounts for this behavior by employing site coefficients that are used to factor the design acceleration, and offers prescriptive recommendations regarding which soil characteristics are appropriate to determine the site-specific soil response.

Beyond the general guidance of **AASHTO HB-17**, liquefaction and lateral spreading potentials of the underlying soil strata should be considered. Refer to **MCEER-ATC-49-1** for specific recommendations for these phenomena. These aspects of soil-structure interaction are further discussed in **6.4.2**.

CHAPTER 5—ANALYSIS

5.1—Overview

Dynamic response of bridges can be estimated using a variety of methods. The two general analysis types include mode-based analysis and time-history analysis. Following an overview of these methods and benefits, more details regarding the different types of analyses that fall under these categories are presented. While historically the mode-based methods, and seismic analysis in general, used elastic analysis, progression is toward use of nonlinear analysis techniques to better predict bridge response for analysis and design. This progression is reflected in an expanded presentation of types and sources of nonlinearities discussed in **Section 5.6** and necessary nonlinear modeling details discussed in **Chapter 6**.

For more detailed information on the topics of modeling and analysis of bridges, refer to **Aviram et al. (2008)** and **Kappos et al. (2012)**. Guidelines for nonlinear analysis of bridge structures in California were discussed by **Aviram et al. (2008)**. The seismic analysis of bridges document by **Kappos et al. (2012)** provides a more detailed summary of analytical bridge modeling, method of analysis, and comparison examples demonstrating the effectiveness of the different analytical methods.

Examples of commonly used computer models for inelastic seismic analysis of highway bridges are discussed in the literature (**Imbsen and Schamber 1983b**; **Ghusn and Saiidi 1986**; **IAI 1992**; **Saiidi et al. 1984**; **Mackie and Stojadinovic 2003, 2005**; **Mackie et al. 2008**). The available analytical models and software, however, do not always adequately model all the necessary nonlinearities that effect dynamic response, and preparing model input and output may be time-consuming. Due to the uniqueness of each bridge structure, a standard reliable method of nonlinear analysis applicable to all cases does not yet exist. Therefore, application of prudent engineering is especially important when conducting a nonlinear analysis and it is recommended

the engineer should have a strong background in structural dynamics.

5.1.1 Mode-based analysis—The complete response history is seldom needed for design of bridges and the maximum values of response to the earthquake will usually suffice. The response in each mode of vibration can be calculated using a generalized single-degree-of-freedom (SDOF) system. The maximum response in each individual mode can be computed directly from the earthquake response spectrum, and the modal maxima can be combined to obtain estimates of the maximum total response. It is emphasized that these are not the exact values of the total response, but are estimates. Inelastic behavior can be incorporated by using an inelastic response spectrum or reduced elastic spectrum.

A sufficient number of modes should be included in the analysis to ensure that the effective mass included in the model is at least 90 percent of the total mass of the structure. This can usually be verified by investigation of the mass participation factors in the analysis. Additionally, it should be confirmed that all important elements (in terms of mass and stiffness) of the structure are represented in the response model. For example, if a long structure with many piers has been modeled as a single unit, each pier base shear obtained from the response spectrum analysis should be compared with the product of the elastic seismic response coefficient, C_s , and the tributary mass of that pier. If the response spectrum analysis result is considerably lower than the result from hand calculations, one possible adjustment would be to increase the number of modes considered.

It would be unnecessarily conservative to directly add the contribution of each mode because the modal maxima do not occur at the same time. A widely accepted modal combination rule is the square root of the sum of the squares (SRSS) method. This method is considered to provide an acceptable approximation of the structural response for structures with well-separated natural periods, where coupling is unlikely to occur. When closely-spaced modes occur (a common occurrence for bridges), a preferred combination technique is the complete quadratic combination (CQC) method, which accounts for the statistical correlation among the various modal responses. For more information on the CQC method and other combination techniques, refer to **Wilson et al. (1981)**.

5.1.2 Time-history analysis—Time-history analysis requires a detailed description of the time variation of the ground accelerations at all supports. It is obviously not possible to predict the precise nature of the future ground accelerations at a particular site. This uncertainty is accommodated by using multiple ground motions that best represent the seismicity of the site based on the recommendations of a geophysicist (refer to **4.9**). Further discussion of time-history analysis is made in **5.5**.

The benefit of using time-history analysis methods is that the model of the bridge is solved using coupled equations of motion. Therefore, no considerations are needed on the number of modes to include in the analysis and no modal combination rules are required; time-history analysis produces the response history directly at any monitoring

point on the bridge. In addition, the bridge models used for time-history analysis are typically more detailed than those used in mode-based analysis, and therefore, nonlinear behavior can be more easily included.

5.2—Single-mode spectral analysis

Single-mode spectral-analysis methods may be used for final design of simple bridges and for preliminary design of complex bridges. This approach is reasonably accurate for response of straight bridges without a high degree of stiffness or mass irregularity. A bridge is considered regular if it is two to six spans and has no abrupt or unusual changes in mass, stiffness, or geometry along its length and has no large differences in these parameters between adjacent supports, excluding abutments. The geometry of a regular bridge is straight or defines a sector of an arc not exceeding 90 degrees and has adjacent columns, or piers, that do not differ in stiffness by more than 25 percent ([AASHTO HB-17](#)).

Three general types of single-mode analysis techniques have been used in past codes: the inverted pendulum method, the uniform load method, and the generalized coordinate method.

5.2.1 Inverted pendulum method—The inverted pendulum method models the entire structural mass and stiffness as a single lumped mass at the top of a single-degree-of-freedom rod element with zero mass. The main advantages of this method are its simplicity and that it does not require a computer. The disadvantages are that it neglects the effects of continuity of the structure and cannot predict torsional effects. Accordingly, it may not properly account for the distribution of seismic forces within the structure and may introduce inaccuracies in the structural period that may give unrealistic values for the seismic forces. This method was widely used before the San Fernando earthquake of 1971, but is no longer in general use for final design. It may be adequate, however, for preliminary analysis or as a check on complex dynamic response analysis.

5.2.2 Uniform load method—The uniform load method is recommended by AASHTO HB-17, [AASHTO \(2012\)](#), and [LRFDSEIS-2-M](#). The basic procedure is to determine the equivalent total structural stiffness by computing the uniform horizontal load that will produce a maximum displacement of 1 in. (25 mm) in the structure. This stiffness is used in conjunction with the total mass to predict the fundamental period, which, in turn, is used in conjunction with a response spectrum to determine an equivalent seismic force. This force is converted to a uniform load and is reapplied to the structure to determine member seismic forces. The method provides a more representative distribution of seismic forces within the structure, as compared with the inverted pendulum method, and accounts for the interaction among the structural members. It requires more effort, however, than the inverted pendulum method and may require a computer-based analysis. The uniform load method may not give acceptable results for skewed bridges, curved bridges, and bridges with intermediate expansion joints.

5.2.3 Generalized coordinate method—The generalized coordinate method provides the best approximation of

dynamic seismic responses using equivalent static methods, and is the method recommended by AASHTO (2012) and AASHTO HB-17. The method is based on Rayleigh energy principles. It differs from the uniform load method primarily in that the natural frequency is based on an assumed vibrational shape of the structure. This assumed shape can be approximated by determining the deflected shape associated with the dead load of the structure applied in the direction of interest. The loads should be applied in the same direction as the anticipated deflection. The maximum potential and kinetic energies associated with this deflected shape are equated to calculate the natural frequency, which is then used with a response spectrum (similar to the uniform load method) to determine an equivalent seismic force. This force is reapplied to the structure as a distributed load (with a shape and sense corresponding to the load used to calculate the assumed vibrational shape) to determine member equivalent static seismic forces.

The generalized coordinate method provides a more representative distribution of seismic forces, as compared with the uniform load method, and accounts for variations in mass distribution along the structure. This method is considerably more involved than either the inverted pendulum or the uniform load methods because it requires an assumption of the vibrational shape and a computer analysis.

5.3—Pushover analysis

With increasing emphasis on evaluation of displacement capacity of the structure, and the emergence and growing popularity of displacement-based design methods, pushover analysis has become an important design tool to understand expected bridge system performance. An example of a pushover analysis is presented by [Priestley et al. \(1996\)](#). Pushover analysis is a nonlinear static method of determining the structural system response due to incrementally increasing displacement demand. It consists of a sequence of linear static analyses where the stiffness of the structural system is continuously updated to reflect the nonlinearity in the material constitutive relationship. Nonlinear soil springs; cracking; slip of reinforcement due to strain penetration effects, including crushing and spalling of cover concrete; and flexural yielding are inelastic events that can be considered in the analysis.

For bridges, the analysis is typically used to determine the displacement capacity of a pier or bent system as well as the corresponding deformation demands on inelastic components and the force demands on capacity protected elements. It also provides information about the sequence of inelastic actions, and about the formation of local and global collapse mechanisms.

For noncomplex structures such as a single column bent on a rigid foundation, this analysis can be done using relatively simple hand methods. For complex systems, such as a multi-column bent with a foundation where nonlinearity can occur in multiple locations and at different degrees of system deformation, computer analysis is usually used. A nonlinear static (pushover) analysis is performed to determine the complete pushover load-deflection relationship for