

Seismic Evaluation and Retrofit Techniques for Concrete Bridges

Reported by ACI Committee 341



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Seismic Evaluation and Retrofit Techniques for Concrete Bridges

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Seismic Evaluation and Retrofit Techniques for Concrete Bridges

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This document provides a summary of seismic evaluation and retrofit techniques for reinforced concrete bridges. The document is intended to be useful to practicing engineers and academic researchers. Three primary phases of a retrofit program are described: seismic vulnerability evaluation, evaluation of the seismic demands and capacities, and selection and design of the retrofit measures. General descriptions of appropriate linear and nonlinear analysis methods to evaluate the seismic response of an existing bridge are provided. Various retrofit measures for individual bridge components are described. In all cases, the information is presented at the conceptual level rather than providing detailed descriptions of the design method. A rich resource of references is included in each section of the document for obtaining more specific information on the subject matter.

Keywords: abutment; bridges; column; expansion joint; footing; hinge; joint; pier; pile; seismic analysis; seismic evaluation; seismic isolation; seismic retrofit.

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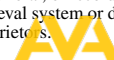
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CHAPTER 1—INTRODUCTION

Performance of bridges in past earthquakes indicates that existing bridge structures can be susceptible to severe structural damage. This vulnerability is evident in regions of high seismic risk, as demonstrated by extensive damage in bridge structures in the 1971 San Fernando Earthquake (Fung et al. 1971), the 1989 Loma Prieta Earthquake (EERI 1989) and the 1994 Northridge Earthquake (Moehle 1995). In those earthquakes, damage included pounding at expansion joints, severe spalling and cracking in bridge columns and joints, and structural collapse. The 2001 Nisqually Earthquake in the state of Washington resulted in damage to columns, restrainers, and the superstructure due to pounding, indicating that some bridges in the United States may be susceptible to damage even in moderate earthquakes (Ranf and Eberhard 2002).

The bridge damage resulting from the San Fernando earthquake caused concern about the seismic vulnerability of bridges and initiated research into and development of seismic retrofit guidelines and measures (Applied Technology Council (ATC) 1983; Zelinski 1985; Buckle et al. 1986; Selna et al. 1989a,b). These earlier guidelines and procedures for seismic retrofit of bridges used strength-based evaluation approaches in which the forces were used as a basis for the evaluation. If the seismic force demand exceeds the elastic strength of the structure, the structural system may be subjected to large inelastic displacements and subsequent strength degradation, instability, or both, of the system that could lead to structural collapse. In this case, retrofit measures solely based on a strength-based approach may not provide adequate deformation capacity to ensure structural stability. Damage to bridges in the Loma Prieta and Northridge earthquakes emphasized the need to address both strength and deformation capacities in bridge seismic retrofit programs, which has resulted in more comprehensive seismic retrofit prioritization schemes as well as improved evaluation procedures and retrofit measures.

A comprehensive retrofit measure for a concrete bridge requires detailed evaluation of the probable strength and stiffness characteristics at member and structure levels, structural displacement and component deformation capacities, and earthquake hazard potential. As such, deformation-based retrofit approaches may be more appropriate to ensure survival of the structure without experiencing collapse under extreme earthquakes. Alternatively, energy-based approaches may be adopted as long as these approaches sufficiently

address all required elements of the complete retrofit plan. Seismic retrofit guidelines started to include these approaches in the early 1990s (Maroney 1990; Lwin and Henley 1993).

Retrofit measures have traditionally been developed to improve seismic performance in extreme events where the primary concern was ensuring structural stability to prevent collapse. More recently, engineers have focused on designing to reduce damage in more frequent events (Lehman et al. 2004; MCEER-ATC 2003). The pairing of a capacity or performance level with a seismic hazard level is called a performance objective. Engineering a structure using multiple performance objectives is termed performance-based earthquake engineering. For example, in addition to ensuring structural stability at the maximum considered earthquake, the performance of the structure at the operational limit states (that is, no damage needing repair) and delayed operational limit states (that is, permitting repairable damage) may also be considered to ensure satisfactory structural performance under the appropriate seismic hazard levels (for example, frequent and moderate earthquakes, respectively). Using a performance-based approach may be advantageous for the retrofit of existing structures in that a designer, for economical reasons, may choose to upgrade the structure to a performance level that is less than that implied by the current code. Performance-based engineering procedures are under development, and the performance of available retrofit strategies under different multiple hazard levels has yet to be evaluated and, therefore, is not directly addressed herein.

This document presents a summary of seismic evaluation and retrofit techniques suitable for ensuring structural stability. A comprehensive seismic retrofit program consisting of multiple retrofit stages will permit efficient and cost-effective retrofit solutions where each stage consists of bridges that will meet a state-specific prioritization criteria (Lwin and Henley 1993). As illustrated in Fig. 1.1, the primary phases of a seismic bridge retrofit program should include:

1. Seismic vulnerability evaluation;
 2. Seismic demand-capacity evaluation;
 3. Selection of efficient retrofit measures and their design;
- and
4. Implementation.

This document briefly describes the phases of a seismic retrofit program followed by sections that provide a more thorough treatment of key aspects of the first three phases. The vulnerability evaluation, demand-capacity evaluation, and retrofit measures presented are described for monolithic reinforced concrete bridges, but may be applicable to other bridge types. In the subsequent sections, emphasis is placed on providing a general understanding of the development and execution of each phase, with a focus on achieving structural stability performance. Seismic retrofit measures are presented at a conceptual level for the critical members responsible for ensuring ductile seismic response. Design and analysis methods vary within the research and design communities, and therefore specifics of each method are not provided in this document. A rich resource of appropriate references, however, is given in each section for more specific and detailed retrofit design and analysis information.

the reader is referred to the recommended references. Information regarding the definition of common terms used to describe seismic retrofit of bridges may be found in Yashinsky and Karshenas (2003).

1.1—Seismic vulnerability evaluation

The first phase of a seismic retrofit program consists of identifying vulnerable bridges and developing a prioritization scheme that accounts for their vulnerability and impact of bridge closure. In this phase, an approximate, rather than a detailed and accurate, vulnerability assessment is carried out for each bridge or bridge type. Issues that should be given consideration in the seismic vulnerability evaluation phase, as indicated in Fig. 1.1, are:

1. Evaluation of the site-specific seismic hazards including the influence of the local soil and the likelihood of lateral spreading, settlement, and liquefaction;
2. Evaluation of the structural vulnerability including the effects of physical geometry, date of design and construction, structural detailing, the actual physical condition, and the foundation soil conditions. Information regarding the performance of comparable laboratory specimens or bridges in previous earthquakes should be considered; and
3. Evaluation of the socioeconomic consequences of damage or failure including issues related to potential casualties, rescue and recovery operations, lifeline interruption, detours, and economic impact in case of temporary closure or failure.

The aforementioned information required should be aggregated to develop a retrofit prioritization scheme for bridges in a seismic region. General aspects of seismic retrofit prioritization schemes have been studied (Basoz and Kiremidjian 1997); however, they may differ from state to state. The California Department of Transportation based their prioritization scheme on outcomes from a workshop sponsored by the Applied Technology Council (ATC 1983) and implemented several seismic retrofit procedures following the Loma Prieta Earthquake (Roberts 1990b, 1993). Other western states, including Nevada, Washington (Lwin and Henley 1993), and Oregon (ODOT 1999) also maintain active bridge seismic retrofit programs. Nationwide, many other states, including Kentucky, Missouri, Illinois, Indiana, and New York, have programs at various stages of development. For example, the Kentucky Transportation Cabinet, through research studies by the Kentucky Transportation Center at the University of Kentucky, has supported efforts to prioritize needed seismic upgrades and to conduct more in-depth analysis of bridges that have been identified as requiring further assessment (Fleckenstien and Drnevich 1990; Sutterer et al. 2000). Many other departments of transportation, however, evaluate a bridge for seismic retrofit only when the bridge is being evaluated for other rehabilitation work.

Chapter 2 provides further discussion about the structural vulnerability evaluation procedures for bridge components. Additional information on seismic hazard analysis and socioeconomic consequences of damage or failure may be found elsewhere (Sundstorm and Maroney 1992; Chang and

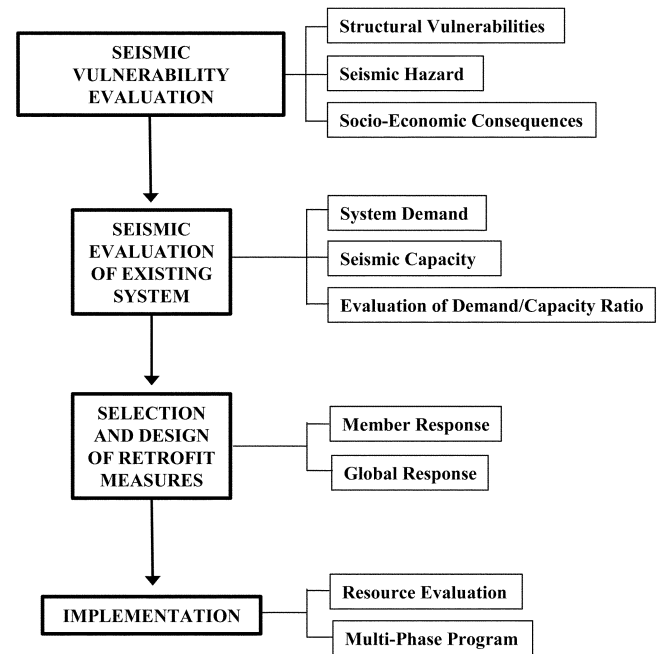


Fig. 1.1—Primary phases of seismic retrofit program and their components.

Shinozuka 1995; Petersen et al. 1996; Kiremidjian and Basoz 1998; Frankel et al. 2000; Sommerville et al. 2001; Ballantyne et al. 2002; FHWA 2006).

1.2—Seismic demand-capacity evaluation

For a bridge that has been identified as a candidate for seismic retrofit from the vulnerability evaluation phase, a detailed demand-capacity evaluation of the existing condition should be performed. This is an important and difficult phase because the extent of the retrofit should be based on a reasonable estimate of the seismic hazard, soil conditions, bridge geometry, and material properties using standard engineering assumptions, research findings, or both. Issues that should be given consideration in the seismic demand-capacity evaluation phase, as indicated in Fig. 1.1, are as follows:

1. Demand evaluation;
2. Capacity evaluation; and
3. Evaluation of the demand-capacity ratios.

The seismic demand may be estimated using established analysis methods. The selection of the analysis method depends on the complexity of the bridge and the performance state of interest. For simple structures or for performance states in which limited inelastic action is expected, single or multimodal response spectra evaluations may be appropriate. Structures expected to sustain significant inelastic action, complex structures, or structures with different performance considerations will require more sophisticated analyses such as inelastic time-history analyses. The models should account for realistic member properties, incoherent (out-of-phase) ground motion for long bridge structures, and boundary and hinge/expansion joint conditions.

The seismic capacity evaluation is intended to provide a realistic estimate of various strengths of the critical components

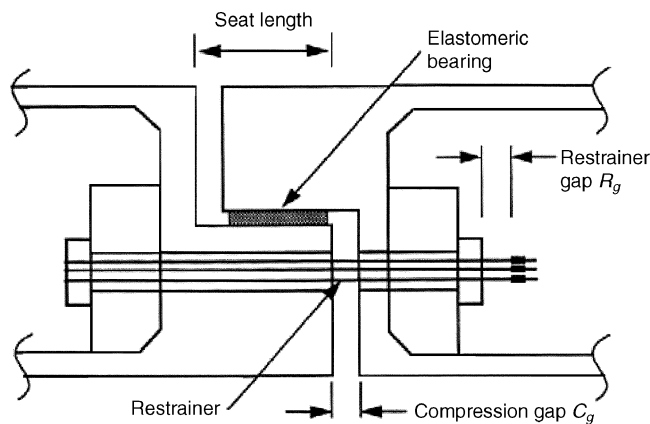


Fig. 1.2—Typical cable restrainer system. (Courtesy of the University of Washington).

as well as the structure using the as-built (or as-existing for structures that have been partially retrofitted) details and realistic material properties. Because seismic demands may induce inelastic action in the structure, an evaluation of both the probable strength and deformation capacities is required.

The demand-capacity ratio evaluation is performed by comparing the results of the capacity and demand analyses for components as well as the entire bridge. More discussion on this topic is presented in [Chapter 3](#).

1.3—Seismic retrofit measures

The selection of appropriate retrofit measures should be based on the results of the demand-capacity evaluation of the bridge. Multiple evaluations may be required if the retrofit strategy is required to satisfy multiple performance objectives. The retrofit strategy should always address the global performance of the structure, although retrofit measures may be selected at the member level. This is because eliminating deficiencies in one member or member type may increase its capacity and subsequently increase the demand on other members, making them vulnerable to seismic damage. To avoid such a scenario and minimize overall seismic vulnerability, the retrofit measures should ensure satisfactory global performance of the structure in accordance with capacity design principles. In the critical regions (such as regions expected to sustain significant inelastic action), satisfactory local responses should also be ensured. When new retrofit concepts are used, it is recommended that the technique and analysis results be verified using large-scale experiments. Retrofit concepts for different types of structural members are discussed in [Chapter 4](#).

1.4—Implementation

After the seismic demand-capacity ratios have been evaluated and the retrofit design phases have been completed, the available resources should be used to determine the order and extent of the retrofit implementation. In an effort to maximize the seismic risk reduction with limited resources, many state transportation agencies have adopted multi-phase implementation programs. The initial phase may consist of implementing simpler, cost-efficient retrofit measures such as seat-width extensions, expansion-joint

restrainers, or both. Figure 1.2 illustrates one such method in which cable restrainers are added to an expansion joint in a typical box-girder bridge. Such measures can be economically implemented in a timelier manner than other component-based retrofit measures, reducing potential risk of collapse. If designed appropriately without introducing other modes of brittle failure (Selna et al. 1989a,b), these retrofit measures can avoid unseating of bridge decks while contributing to energy dissipation through inelastic deformation of the restrainers (ATC 1983). The overall seismic risk for most individual bridge structures cannot be eliminated completely using these simple measures. As time and resources permit, the next phases may include implementation of retrofit measures designed for other members so that the bridge response in future earthquakes will meet the performance criteria adopted in the retrofit design. Decisions about the form and execution of the implementation phase are made by the individual state transportation administration. Examples of implementation plans may be found in Roberts (1990a,b) and Lwin and Henley (1993). Because these plans depend on the economic and political environment of the state and the highway agencies, further discussion of the implementation stage is not provided in this document.

CHAPTER 2—SEISMIC VULNERABILITY EVALUATION

Recent seismic events have increased the awareness of the seismic vulnerability of a large number of bridges in the United States and internationally. This has led to the realization that a careful screening and prioritization procedure is needed to identify the most vulnerable structures. Numerous risk assessment algorithms exist or are under development (Sundstrom and Maroney 1992; Maroney and Gates 1992; Sexsmith 1994; Werner et al. 1995; Basoz and Kiremidjian 1997; Augusti and Ciampoli 1999) that evaluate the seismic hazard, structural vulnerability, and socioeconomic consequences of severe damage or collapse.

Assessment of the level of the seismic hazard is critical to determining the demand on the bridge and its structural components. Issues such as recurrence intervals and expected ground motion intensity and durations should be carefully considered. In the western United States, the differences in seismic demands between a 500-year event and a 2500-year event are not as significant when compared with similar events in the eastern and central United States (Leyendecker et al. 2000). The design-level earthquake(s) should be site- and state-specific, and be set by the appropriate governing agency. Aspects of the site that can impact the response of a structure include the fault location and type, local soil conditions, and the possibility of soil liquefaction or failure. In a long bridge structure, the soil profile and corresponding ground motions may vary over the length of the bridge. This varying condition will impact the bridge behavior and should be taken into consideration.

One of the most important parameters to consider in seismic retrofitting is the socio-economic consequences of severe damage or collapse. Recent earthquakes have demonstrated that the long-term disruptions from an earth-

quake can be much more costly than the physical damage of the earthquake. The closure of the San Francisco-Oakland Bay Bridge after the Loma Prieta Earthquake caused significant economic hardship for the San Francisco Bay Area. This led to a substantial investment by Caltrans in retrofitting and replacing bridges in the region that were deemed critical. The retrofit measures were designed so that the bridges would remain operational after a significant earthquake. For further information on the seismic hazard evaluation including the socioeconomic consequences of bridge damage or collapse, the reader is referred to Sundstrom and Maroney (1992).

2.1—Structural vulnerability indicators

Full evaluation of the structural vulnerability should be performed with regards to its type, overall geometry, structural redundancy, expansion joint configuration and details, age/design and detailing practices, structural condition, and conditions of foundation soils.

2.1.1 Bridge geometry—Construction practices between the 1950s and the 1980s have often led to bridge configurations and design details that have been observed to be problematic in recent earthquakes. Irregular bridge geometries, which are common in congested urban regions, can also be structurally vulnerable. The following are typical geometric characteristics that may indicate potentially problematic structures:

- Bent configurations that include squat columns, flared columns, outrigger bents, C-bents, and columns and bents with dissimilar stiffnesses in the same structure;
- High degree of skew;
- High degree of curvature;
- Short seat widths;
- Multiple deck levels; and
- Multiple superstructure types.

Squat columns (columns that have aspect ratios of less than three [Paulay and Priestley 1992]) often result in stiffer structural elements with fundamental periods for the bridge near the peak of the acceleration response spectrum (ARS) curve, causing the bridge to attract high seismic forces. Squat columns or bents in frames with taller and more slender columns tend to attract a disproportionate amount of the inertial forces, making them more vulnerable to seismic damage (Priestley et al. 1994b; Saiidi et al. 2001b). Additionally, the low height-depth ratios make these columns more susceptible to shear damage, preventing ductile performance of the bridge. This response can lead to the loss of the gravity-load-carrying capacity of the bridge system.

Outrigger bents (bents in which the bent cap projects beyond the superstructure to span over a feature beneath the bridge), as shown in Fig. 2.1, and C-bents (those with an eccentric, single column under one side of the structure) have complex internal force states because of their geometry (Thewalt and Stojadinovic 1995; Griezic et al. 2001). The original design of the bent may not have accounted for the actual force-transfer mechanism, which may increase its seismic vulnerability.

Skewed spans have a tendency to rotate about the longitudinal axis of a bridge during seismic response, increasing

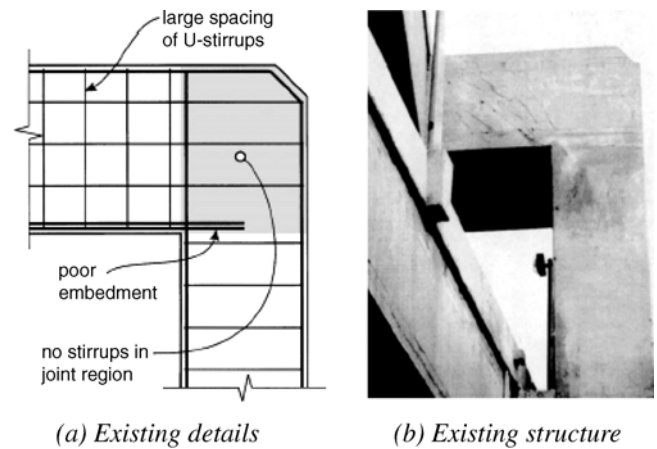


Fig. 2.1—Outrigger joint (Griezic et al. 2001).



Fig. 2.2—Damage to superstructure due to rotation of skewed spans (courtesy of San Francisco Chronicle).

their vulnerability to unseating at piers and abutments (Maragakis and Jennings 1987; Björnsson et al. 1997). The vulnerability of these bridges depends on the degree of skew in that the amount of rotation increases with the amount of skew and the seat width. As shown in Fig. 2.2, with increased rotation, the skewed spans can also collapse due to self-weight (Priestley et al. 1994b).

Highly curved bridges and those with geometrically varying sections, such as widening roadways to accommodate on- or off-ramps, have unique dynamic response characteristics that require analytical procedures that account for three-dimensional response (DesRoches and Fenves 1997). Bridges designed without explicit consideration of the maximum seismic demand may have seat widths that are not sufficient to accommodate the lateral movement resulting from seismic demands. Short seat widths increase the potential for the bridge to experience unseating and collapse of the superstructure.

Bridges with multiple levels have vertical mass and stiffness distributions that may make it difficult to predict the force distributions to the columns (Fig. 2.3). Multilevel structures frequently have unusual connection details between the decks and the supporting columns. Many of these problems were emphasized by damage in the Loma Prieta (EERI 1989;



Fig. 2.3—A multiple-level bridge with distributed mass and stiffness. (Courtesy of Marc O. Eberhard.)



Fig. 2.4—Collapse of a bridge superstructure due to inadequate expansion joint details. (Courtesy of University of California, San Diego.)

Nims et al. 1989; Housner 1990; Lew 1990), Northridge (Housner 1994; Priestley et al. 1994b), Kobe (Fujino et al. 1998; Seible et al. 1995) and Taiwan (Chi-Chi) (Mahin 1999) earthquakes.

Older and modern bridges may have adjacent systems that have different types of superstructures. The demands on each type of superstructure may be different, and may not have been considered in the original design. The coupling of the demands on different superstructures may result in localized structural damage in one system.

2.1.2 Structural redundancy—Bridges with structural redundancy need to be identified during the risk assessment phase to account for internal force redistribution capabilities and inelastic energy dissipation at multiple locations. Retrofitting all components may not be necessary in a system with redundant seismic components. For these structures, certain portions of the bridge may be retrofitted to carry a higher proportion of the seismic demand. Redundancy,

however, can be used only if appropriate ductile detailing allows internal force redistribution and satisfactory energy dissipation.

2.1.3 Expansion joint configuration and details—Key factors that affect the dynamic response of bridges are seat widths at abutments and midspan hinges, and force and displacement capacities of joint restrainers, shear keys, and abutment components. Retrofitting measures that address inadequacies in expansion joints, including expansion of seat widths and addition of restrainers, shear keys, or both, can improve the seismic response of the bridge by preventing loss of support during an earthquake (Fig. 2.4).

2.1.4 Age/design and detailing practices—The age of a bridge structure is an indicator of its seismic vulnerability. The type of construction depends on the prevailing design and construction practice during the period the bridge was built. Before the 1970s, for example, bridge columns were not designed to sustain the plastic shear demand resulting from column flexural hinging, as typified by bridges in California designed before the early 1970s. Although changes in seismic design criteria following the 1971 San Fernando earthquake resulted in bridges with reduced structural vulnerability (EERI 1986; Maroney and Gates 1992) seismically vulnerable bridges from the post-1971 era are widespread.

The adequacy of the reinforcement detailing should be evaluated to determine the structural vulnerability in terms of seismic performance. Reinforcement detailing practices, including size and spacing of column transverse reinforcement, splice length and location, anchorage of column and cap beam main reinforcement, and transverse reinforcement in cap beams and joints, have a large impact on the ductility of a bridge structure. Therefore, the adequacy of the reinforcement detailing should be evaluated to determine structural vulnerability in terms of seismic performance. As an initial assessment technique, the existing details can be compared with existing prescriptive criteria in the AASHTO Standard Specifications for Highway Bridges, AASHTO LRFD Bridge Design Specifications, ACI 318, or by using performance-based criteria available in the literature (ATC 1996; MCEER-ATC 2003). More reliable evaluation may be provided by comparison with relevant existing experimental research results.

2.1.5 Structural condition—The physical condition of a structure can affect its seismic vulnerability. While the strength of sound concrete in older bridges can be significantly greater than the design strength, the opposite may be true for deteriorated or cracked concrete. Similarly, corrosion of reinforcement can result in degradation of the strength and ductility of adequately designed structural members. Less obvious conditions, such as long-term creep, shrinkage, and thermal movements, can result in excessive joint openings that may contribute to unseating of superstructure elements during a seismic event.

2.1.6 Conditions of foundation soils—Critical soil conditions, such as liquefiable soils or poorly compacted soils (fills or soft sediments) where large settlement or ground motion amplifications can be expected, have to be evaluated as part of the seismic vulnerability evaluation. Typically, foundation

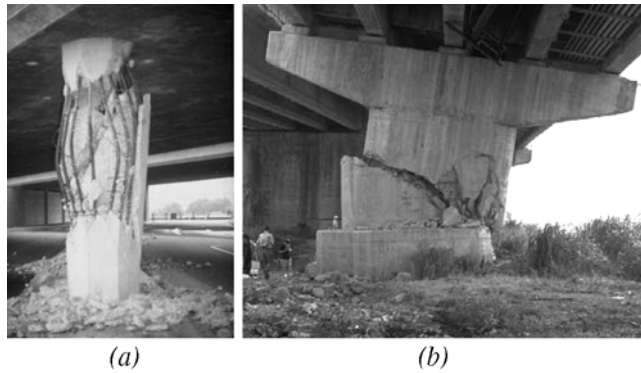


Fig. 2.5—(a) Column damage, Route 5/210, San Fernando earthquake; and (b) damage to squat column, Wu-Hsi Bridge, Chi-Chi earthquake.

soil conditions are assessed in conjunction with a site-specific seismic hazard evaluation.

2.2—Vulnerable structural elements

A seismic vulnerability assessment at a global level can provide initial indications of the seismic response and retrofit needs of existing concrete bridge structures. Once a bridge structure has been identified as a candidate for seismic retrofit, a detailed seismic evaluation to identify vulnerable structural elements is needed before selecting suitable retrofit measures.

Section 2.1 discussed structural vulnerability indicators at a global level. This section focuses on seismic vulnerability at the element (or member) level. Vulnerable structural elements in seismically deficient concrete bridges include: columns (including pier walls), cap beams, cap beam-column joints, foundations including footings and piles, hinges and supports, superstructures, and abutments and embankments. Discussion of seismic deficiencies in each of these elements is presented in Sections 2.2.1 to 2.2.7, and suitable retrofit measures for these deficiencies are presented in **Chapter 4**.

2.2.1 Columns—Concrete columns in older bridges can lack reinforcement details that permit development of plastic hinges required to achieve the desired ductile structural behavior (Fig. 2.5(a)). Older columns can have limited transverse reinforcement that is not adequate to sustain the plastic shear or flexural ductility demands. Columns may have all of the main longitudinal bars lapped immediately above the footing, which is typically a region with the highest moments due to lateral loads, and therefore, a probable location for plastic hinging. Provided that the column-footing connection is intended to sustain flexural demands, the column strength and deformability may be reduced if the length of the column lap splice and the transverse reinforcement provided are not adequate to sustain the bond demand. Laps of spliced transverse reinforcement will also be critical in the plastic hinge region because such reinforcement will not be effective for confining concrete or resisting shear once spalling of the cover concrete occurs.

As noted previously, column geometry can also lead to seismic vulnerability (Fig. 2.5(b)). Short, squat columns are more susceptible to shear failure than more slender columns

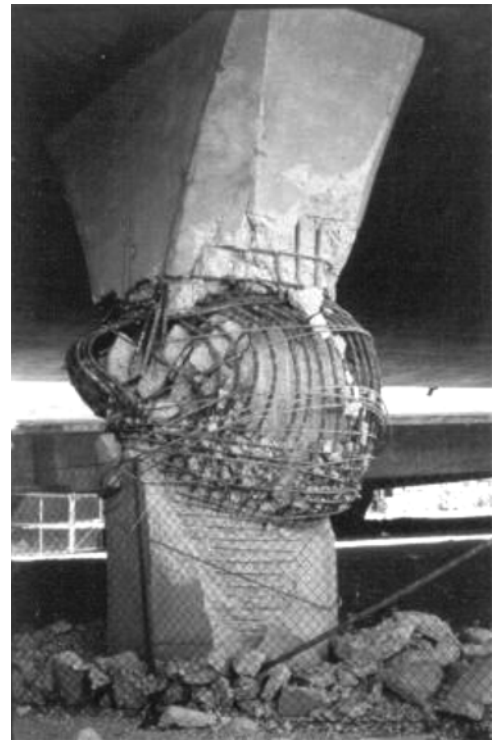


Fig. 2.6—Damage to column mainly due to flares near the column top. (Courtesy of University of California, San Diego.)

for two reasons. First, their effective elastic stiffness is greater than that of their more slender counterparts. For bridges with both tall and short columns, the short columns will attract a larger portion of the lateral load during elastic response. Second, short columns will have higher shear demand than taller columns with the same moment capacities at the critical sections. The interaction between the higher shear demand and the plastic hinging in the column increases the potential for premature failure in short columns.

Columns with architectural or structural flares can force inelastic action in the column to concentrate below the flared portion (Fig. 2.6). This location may be problematic for two reasons. First, the region of the column away from the ends, a location of lower moment that is not typically associated with plastic hinging, may be lightly reinforced in the longitudinal and transverse directions, and therefore, the lateral strength and the deformation capacity of the system may be less than that estimated. Second, plastic hinging in the column away from the cap beam interface reduces the effective length, which increases the moment gradient and the shear demand in the column (Wehbe and Saiidi 1999).

2.2.2 Cap beams—Cap beams in reinforced concrete bridges may have inadequate flexural, shear, or deformation capacity. Flexural resistance of the cap beams will be reduced when the top or bottom longitudinal reinforcement is anchored insufficiently into the joint, which in turn may inhibit the development of the full flexural capacity in the column plastic hinges. This condition is particularly true for cases in which the bottom longitudinal reinforcement is minimal and discontinuous through the joint due to underestimation of the beam positive moments for seismic



Fig. 2.7—Cap beam damage in 1999 Chi-Chi earthquake. (Courtesy of University of Washington.)

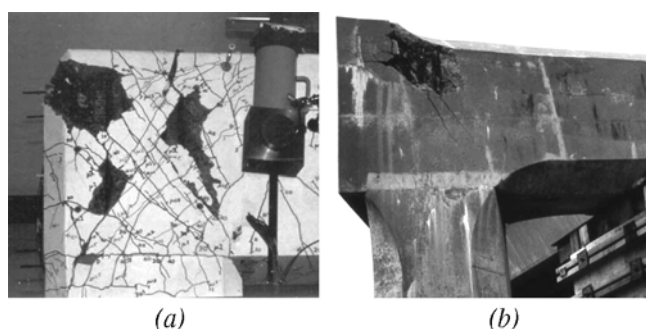


Fig. 2.8—Damage of beam-column joints: (a) joint damage in laboratory specimen (Ingham et al. 1997); and (b) damage in 1989 Loma Prieta earthquake. (Courtesy of University of California, San Diego.)

conditions. Discontinuous longitudinal beam reinforcement is also common in older bridges in which the cap beams were designed primarily for gravity loads. Cap beams of multi-column bents often have limited shear resistance due to inadequate transverse reinforcement. Figure 2.7 shows damage to a cap beam including diagonal cracking that is indicative of damage due to large shear or torsion forces.

2.2.3 Cap beam-column joints—Many older bridge structures have cap beam-column joints that are susceptible to seismic damage (Fig. 2.8). California bridges before 1971, for example, were designed without transverse reinforcement in the beam-column joints. Research has shown that joints without transverse reinforcement sustain greater levels of damage relative to well-confined joints (Ingham et al. 1998; Priestley et al. 1996; Sritharan et al. 1999; Lowes and Moehle 1999). In addition, joints in older bridges with or without limited transverse reinforcement may not be sufficient to carry the large shear demand associated with column plastic hinging. Another deficiency found in many bridges is the premature termination of column longitudinal reinforcement into the joint, which may restrict the development of the theoretical column moment capacity.

These conditions are especially critical in structures with outrigger or C-bents because these bents experience large torsional forces in conjunction with high shear demands associated with typical cap beam-column joints. Premature

termination of the column longitudinal reinforcement can help to limit the joint shear stress demands, although the corresponding flexural capacity of the column may be reduced. In a bent test that modeled construction before 1970, a joint was detailed with longitudinal column bars with insufficient development length, and the resulting bar slippage allowed the specimen to reach a moderate level of displacement ductility (Moore et al. 2001).

Research shows that joints designed without transverse reinforcement or containing prematurely terminated column bars may impose additional demands on the cap beam transverse reinforcement adjacent to the joint and on the bottom beam longitudinal reinforcement in the joint region (Ingham et al. 1997; Sritharan and Ingham 2003; Nada et al. 2003). The additional demand resulting from engaging the beam reinforcement in the joint force transfer can cause yielding of the beam reinforcement. Therefore, it is critical that the expected joint behavior is given consideration when identifying locations of inelastic action.

2.2.4 Footings—Bridges supported on liquefiable or poorly compacted soils are vulnerable to excessive or differential settlements that can reduce the seismic structural performance. Large differential settlements can cause global instability, leading to collapse of the structure, or can result in large movements of supports that lead to unseating of the superstructure at articulated supports (for example, bearings and shear keys). Liquefaction, lateral spreading, or differential settlements can result in magnification of the seismic demands beyond the capacity of the structure.

Several seismic deficiencies may be found in footings in older bridges. For instance, footings in bridges constructed before the 1970s in California were built without a top mat of reinforcement because the lateral design forces calculated per the code at the time did not indicate a potential for reversal of bending moments in the footing. The design shear force for these footings may have been estimated using the design lateral forces, and therefore, the actual shear demand (based on a capacity-design approach) may exceed the shear strength. Therefore, older footings may not have adequate capacity to sustain the shear demand corresponding to plastic hinging in the column.

The bearing area of many existing footings may be insufficient because they were designed primarily for vertical loads. Higher moments caused by earthquake forces can lead to higher soil bearing pressures. To meet the demands, the footing plan area and thickness may need to be increased, and additional reinforcement may need to be provided. There is also potential for pullout failure of the column main reinforcement due to insufficient anchorage, as shown in Fig. 2.9(a).

Piles have also sustained damage in previous earthquakes (Fig. 2.9(b)). Observed response indicates that precast concrete batter piles are particularly vulnerable. The pile capacity may be limited either by the strength of surrounding soils, the capacity of the pile, or by the capacity of the pile-to-pile cap connection. Damage to the pile is also influenced by the soil profile; significant variation in the soil properties along the length of the pile may result in concentration of damage. The damage is not easy to detect, and it is difficult to repair.