ACI 341.3R-07

Seismic Evaluation and Retrofit Techniques for Concrete Bridges

Reported by ACI Committee 341

American Concrete Institute®

First Printing
March 2007

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Seismic Evaluation and Retrofit Techniques for Concrete Bridges

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ISBN 978-0-87031-255-7

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.

CONTENTS

ACI 341.3R-07 was adopted and published March 2007.

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

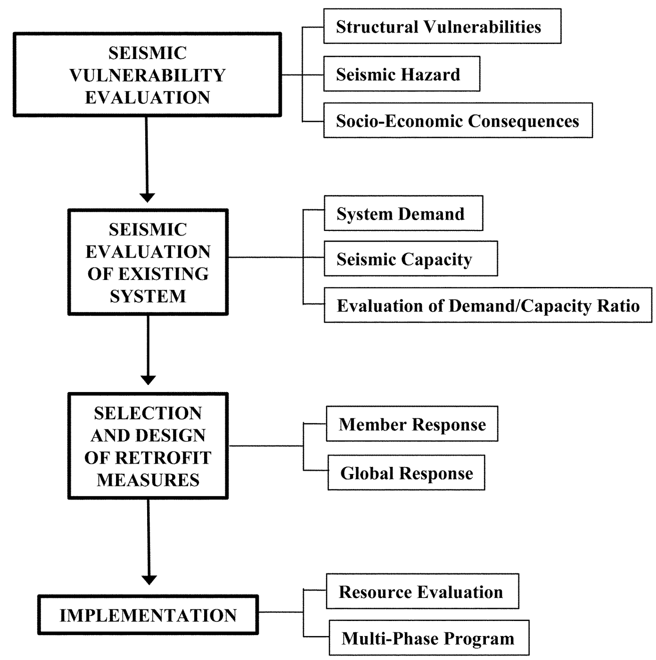


Fig. 1.1—Primary phases of seismic retrofit program and their components.

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

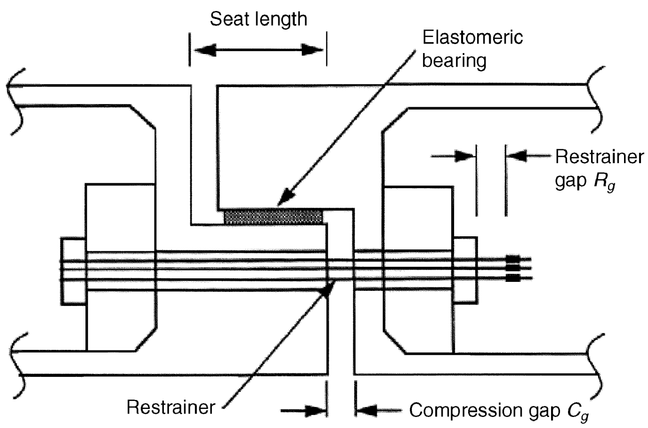


Fig. 1.2—Typical cable restrainer system. (Courtesy of the University of Washington).

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 earthquake 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.

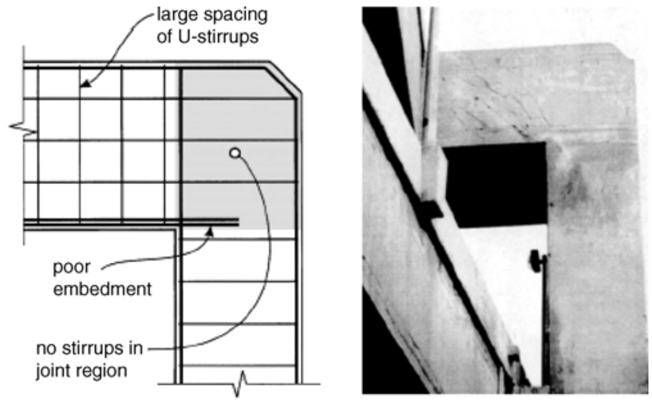
2.1.1 *Bridge geometry*—Construction practices between the 1950s and the 1980s have often led to bridge configurations and design details that have been observed to be problematic in recent earthquakes. Irregular bridge geometries, which are common in congested urban regions, can also be structurally 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



(a) Existing details *(b) Existing structure*

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; Bjèornsson 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

(a) (b)

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

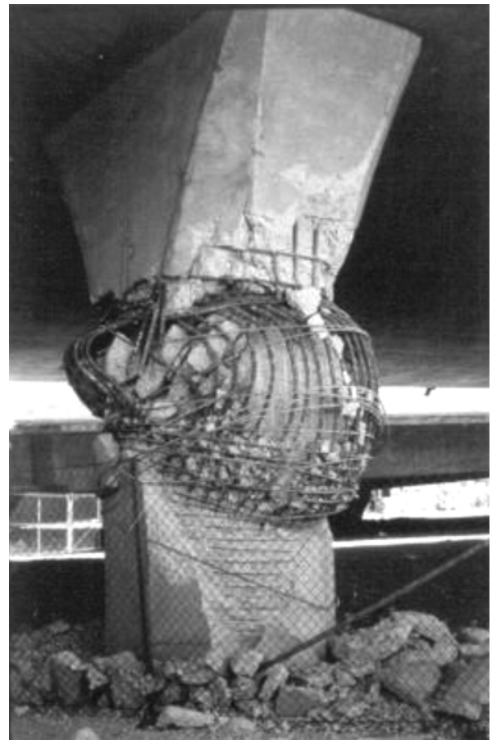


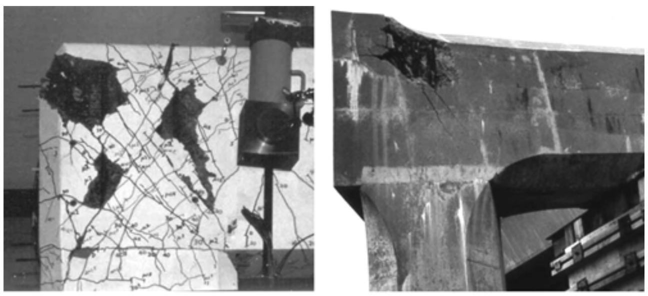
Fig. 2.6—Damage to column mainly due to flares near the column top. (Courtesy of University of California, San Diego.)

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 joints 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.)



(a) *(b)*

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).

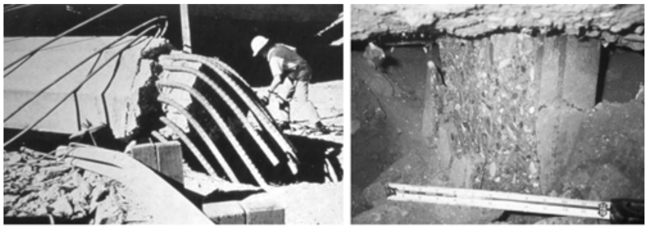
Research shows that joints designed without transverse reinforcement or containing prematurely terminated column bars may impose additional demands on the cap beam transverse reinforcement adjacent to the joint and on the bottom beam longitudinal reinforcement in the joint region (Ingham et al. 1997; 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. Pile damage is not easy to detect, and it is difficult to repair.



(a) (b)

Fig. 2.9—Foundation damage: (a) pullout of column bars (Priestley et al. 1996); and (b) damage to pile and underside of the pile cap. (Courtesy of Santa Clara University.)

Fig. 2.10—Showa Bridge collapse in 1964 Niigata earthquake (NiSEE EQIIS image database).

2.2.5 *Hinges and supports*—Failures due to unseating of superstructures have, historically, been some of the most common and disastrous (Fig. 2.10). In an effort to minimize nonseismic forces, many bridges were designed with seat widths that were just large enough to accommodate movements due to creep, shrinkage, and temperature changes. For example, to minimize the moment in a dapped end of a hinge, the length of the dapped end would be designed to be as short as possible. Structures supported at hinges and abutments by steel rocker bearings or by elastomeric bearing pads with insufficient displacement capacity are susceptible to unseat completely, which may lead to collapse. These bearings are also prone to instability when subjected to large displacements, which can lead to the superstructure dropping a maximum distance equal to the height of the bearings, leaving the bridge out of service. Even moderate damage, such as damage to the bearings or concrete at the expansion joint, may result in temporary bridge closure (Fig. 2.11).

Some bridges built or retrofitted in the 1970s after the San Fernando Earthquake have cable restrainers or high-strength bolts at hinges. Some of these retrofits have insufficient strength or elastic deformation capacity, which can result in failure of the restrainers or diaphragms in a seismic event (Klosek et al. 1995; Ranf and Eberhard 2002; Selna et al. 1989a,b).

Fig. 2.11—Damage at Fourth Avenue on-ramp in 2001 Nisqually earthquake. (Courtesy of Washington Department of Transportation.)

Fig. 2.12—Damage to superstructure in 1971 San Fernando earthquake. (Courtesy of the University of Washington.)

In bridge design, shear keys may be used to restrain the excessive movement of the superstructure at the abutments. Many older bridges either lack transverse shear keys or have shear keys with inadequate strength. There are cases when a shear key is designed to fail so that it prevents damage to a critical portion of the structure in an extreme event. Poorly designed shear keys, however, can reduce the seismic performance of the bridge. Shear keys with inadequate strength may fail to restrain the superstructure, while those with excessive strength can result in unintended failures by overstressing adjacent structural elements (Silva et al. 2003).

2.2.6 *Superstructure*—The superstructure in older bridges may not have been designed considering the force demands associated with formation of plastic hinges at the top of the columns, which can result in damage to the superstructure (Fig. 2.12). Although failure due to insufficient superstructure strength in a seismic event has not been observed as often as other failure modes, this outcome may reflect the vulnerability of other elements, particularly columns, rather than the capacity of the superstructure. For example, in box-girder construction with integral bent caps, the superstructure should be checked to ensure that both the moment and the shear capacities are sufficient to permit development of full

Fig. 2.13—Abutment damage at I5-14 Interchange, 1994 Northridge earthquake. (Courtesy of the University of Washington.)

flexural hinge capacities in the columns without significant damage to the superstructure. Because of the influence of the axial load on the seismic shear strength of columns, attention should be paid to the column axial load demand and capacity (Kowalsky and Priestley 2000; Moehle et al. 2001). Consequently, in retrofitting bridges, it is critical to ensure that the superstructure does not become the weak link. There are three main reasons for this requirement:

1. Damage to columns can be repaired under traffic, whereas superstructure damage will typically require closure of the structure for repair;

2. Significant inelastic action and related damage in the superstructure can render the bridge unusable, if not unstable; and

3. The cost of repairing or replacing the superstructure exceeds the cost of repairing or replacing the columns.

Ductile inelastic action in the superstructure may not result in collapse of the structure. Consequently, it may be appropriate to permit ductile inelastic flexural action in the superstructure if the bridge is to be replaced after experiencing a major earthquake. Satisfactory response of the bridge structure in the expected major earthquake should be ensured by including suppression of nonductile response mechanisms such as shear or bond. Consideration should be given to the location of damage and its impact on bridge serviceability after the earthquake.

2.2.7 *Abutments and embankments*—Damage to abutments has been observed in previous earthquakes (Fig. 2.13). Damage may result from unseating, bridge geometry, abutment geometry and reinforcement, pile failure, and response of the surrounding soil. Abutment seats, like hinges, should be large enough to prevent unseating of the superstructure in a seismic event. Older abutments frequently lack transverse shear keys. Where the shear key capacity exceeds the pile capacity, failure of the piles may result, and the resulting damage is not easy to detect or repair after an earthquake.

As noted previously, highly skewed superstructures are prone to rotate in the horizontal plane, which may result in uneven demands and damage along the face of the abutment. Design of restrainers in these systems should account for these effects.

Abutments can be susceptible to settlement/lateral movement, passive pressure failure, pile failure, and pile-to-abutment connection failure. The response of the abutment influences the bridge performance. Reliable seismic evaluation requires realistic modeling of the abutment stiffness, the abutment-structure, and soil-abutment interaction effects (Werner et al. 1994; Price and Eberhard 1998, 2005; Inel and Aschheim 2002).

CHAPTER 3—SEISMIC EVALUATION

The seismic evaluation phase is carried out to provide a detailed estimate of the behavior of individual bridges in the existing condition. This is an important and difficult phase of the seismic bridge retrofit program because the extent of the retrofit should be based on a reasonable estimate of the seismic demands and capacities. The seismic evaluation phase includes:

1. Evaluation of the seismic demand in the structural components of the bridge;

2. Evaluation of the capacity of each component; and

3. A demand-capacity evaluation to determine the need for retrofit measures based on 1 and 2.

The demand-capacity evaluation is carried out using parameters, such as strength and deformation, to verify that the demand remains within acceptable limits for individual members, bents and frames, and for the overall bridge structure. Examination of demand-capacity ratios will help identify the yield mechanisms and failure modes of the system.

There are many critical locations in the structure where capacity and demand are linked and should not be determined independently. For example, the plastic shear demand V_p of a flexural member may depend on the maximum flexural moment capacity at the predicted plastic hinge locations of that member. Therefore, the shear demand (discussed in Section 3.2) of a bridge column that is designed to form flexural plastic hinges should correspond to V_p, which is determined by the plastic flexural capacity at the appropriate plastic hinge locations, and should be shown to be below the shear capacity ϕV_n, where ϕ is the strength-reduction factor and V_n is the nominal shear strength. Note that expressions available to estimate ϕV_n suggest that it decreases with increasing ductility demand (Priestley et al. 1994a; Moehle et al. 1994; Kowalsky and Priestley 2000; Moehle et al. 2001). The reduction in ϕV_n is mainly due to the reduction in the aggregate interlock capacity with widening of inclined shear cracks. If $\phi V_n > V_p$, it is likely that shear failure is prevented. If $V_p > \phi V_n$, the column may fail in a brittle shear mode before reaching its full flexural mechanism. This concept is illustrated and described in ACI 318-05, Section R21.3.4.2 and Fig. R21.3.4.

A combined gravity and lateral load analysis (discussed in Section 3.1) of the structural system using appropriate means will establish the sequential formation of a structural response mechanism, locations of critical regions, and overall system deformation characteristics, possibly including the system collapse mode.

3.1—Seismic demand evaluation

A variety of methods with different levels of complexity are available to evaluate the seismic demand on bridge structures. Commonly used methods are linear demand analyses, including response spectrum analysis methods, and nonlinear demand analyses, such as limit analysis, static pushover analysis, and fully nonlinear dynamic procedures. Additional information regarding the use of these methods for bridge analysis can be found in the following references: AASHTO Standard Specifications for Highway Bridges; AASHTO LFRD Bridge Design Specifications; Priestley et al. 1996; ATC 1996; FEMA 273; FEMA 356; MCEER-ATC 2003.

Depending on the complexity and the importance of the bridge structure, the seismic demand can be calculated by the uniform load method, single-mode response spectrum method, multimode response spectrum method, linear dynamic analyses based on site-specific ground motion records, or nonlinear analyses.

3.1.1 *Linear elastic analysis methods*—The uniform load and the single-mode response spectrum methods are essentially equivalent lateral-load procedures that are based on the assumption that the fundamental mode of vibration provides a reasonable approximation of the response (AASHTO Standard Specifications for Highway Bridges; AASHTO LFRD Bridge Design Specifications). For bridges with irregular distributions of mass or stiffness, having a significantly curved alignment, or with a large number of spans, a single-mode response spectrum does not provide an adequate representation of the seismic response, and therefore, the use of multimode response spectrum or linear time-history analysis methods is preferable.

When analyses based on ground motion records are used, the model should be subjected to a suite of recorded or synthetic earthquake records that match an appropriate response spectrum. A suite of ground motion histories is preferable to using a single record because the suite of records takes the natural event-to-event variation of earthquakes into account, whereas a single record may be a somewhat unrealistic event with energy concentrated at selected frequency bands. For most bridge structures, it is reasonable to use three to five records, but for bridges of higher importance or with more complex geometries, more investigation may be needed.

One of the simplest evaluation methods consists of comparing forces obtained from an elastic analysis with the calculated strengths to obtain a demand-capacity ratio for each structural element (Buckle and Friedland 1995; ATC 1996). In this method, force demands obtained from linear elastic analysis models are typically reduced by a response modification factor, R (sometimes designated as Z) and compared with available strengths (AASHTO Standard Specifications for Highway Bridges; AASHTO LFRD Bridge Design Specifications; Paulay and Priestley 1992). AASHTO R-factors range from 0.8 to 5.0. R-factors that exceed 1 are used to account for the ductility and redundancy of the member when the structure is subjected to a design-level earthquake. An R-factor of 1 or lower is used for nonductile members with the intention of keeping these components within the elastic range of response and for members whose failure would cause undesirable consequences. For example, an R-factor value of 0.8 is used for connections of superstructure to abutment and expansion joints within a span of the superstructure. Even lower reduction factors may be used for members in critical areas of the structure where it is important to avoid severe damage such as the substructure components. In addition to this type of strength evaluation, both global and individual member deformation demands should be estimated to prevent collapse due to severe structural damage in the members or second-order effects.

The previously described method rests on the assumption that the calculated demand-capacity ratio(s) for each structural element is a good indication of the ductility demand. The main disadvantages of the method are that the use of R-factors may not guarantee that the probability of undesirable performance is sufficiently low, and that the method does not recognize that seismic demands are affected by interaction between structural elements. For structures that have considerable overstrength or in cases where the element demand is limited by the flexural capacity of adjoining members, an elastic analysis may result in unrealistic force demands for some structural elements.

3.1.2 *Nonlinear analyses*—A more accurate estimate of seismic demands can be obtained by directly accounting for the nonlinear behavior of the elements. These methods include static nonlinear procedures (such as limit or pushover analyses paired with linear dynamic analyses) and fully nonlinear dynamic analysis procedures (such as acceleration time-history analyses). Estimates of element forces and deformations obtained with these methods more accurately account for the effects of concrete cracking and yielding of the reinforcement and encompass the nonlinear range of response.

In these analyses, nonlinearity is typically included in the supporting elements (such as columns), and the bridge superstructure is assumed to act as an elastic, in-plane diaphragm. Studies indicate that the accuracy of results of theses analyses depends on the formulation of a suitable model for the superstructure and that using one or multiple spine beams to capture the effect of the superstructure may not be adequate. In those cases, the deck should be modeled using plate or shell elements (Inouye et al. 2003). In addition, if the deck sustains inelastic action, this source of nonlinearity should be adequately accounted for in the analysis model.

Limit analysis is a simplified static nonlinear procedure used to estimate the distribution of the force demand and is based on the principle of virtual work. By assuming appropriate locations of plastic hinges in the superstructure, substructure, or both, a full yield mechanism of the system can be evaluated. Because the system is in equilibrium, the summation of work done by the external and internal forces acting on the structure under any virtual displacement demand should be equal to zero. The work done by external forces is calculated as the product of the applied lateral forces and virtual displacements for the assumed mechanism. The internal virtual work is the result of the flexural moments at plastic hinge locations (remaining constant throughout the virtual displacement) undergoing inelastic rotations. The structural elements are assumed to remain rigid outside the hinging regions so that no additional strain energy is introduced into the system. For each assumed mechanism, the total lateral force that is required to form that mechanism is calculated by equating the internal and external virtual work. The demand on the structure is the minimum total lateral force required to form a collapse mechanism. The main limitations of this method are that it does not provide information about the displacement demands on the structure, and for complex structures, there is no systematic method to ensure that the lowest lateral load yield mechanism has been considered (Massonnet and Save 1965).

A pushover analysis is a more complex nonlinear static procedure that is used to estimate the member demands, the monotonic force-displacement relationship, and the displacement capacity of the system. Within the context of a pushover analysis, independent analyses of stand-alone frame models can be helpful for estimating seismic demands in critical members. In a pushover analysis, monotonically increasing lateral displacements are typically applied to a structural model that includes nonlinear effects such as degradation in stiffness and strength with increasing displacement (FEMA 356). For a demand analysis, the method needs to be paired with a dynamic or response-spectrum analysis method to estimate the system displacement demand (FEMA 273; FEMA 356; Browning 2001). Maximum deformations expected in the elements correspond to the expected maximum displacement in the system.

Unless realistic assumptions are made about expansion joint behavior, abutment characteristics, deck stiffness, and coherence of the ground motions, pushover analyses may lead to unrealistic seismic demands in members. In such cases, fully nonlinear dynamic analysis of the entire multi-frame bridge structure may be necessary to assess the seismic demand on individual members or components. Full nonlinear global models assist in evaluating the dynamic interaction between different parts of a bridge system as a final check on the response characterization. When both component level and global models of a bridge are analyzed, it is an important task left to the designer to ascertain which model reflects the most critical demands if comparable results are not obtained from the analyses due to the simplifications adopted in the models.

In a nonlinear modeling effort, the member and component properties should be based on expected material and effective section properties. To provide an adequate estimate for the foundation flexibility and soil damping, soil springs and dampers with properties established from geotechnical data can be used (Wilson 1988; Wolf 1988). In addition, the effect of structural pounding may need to be included in the analysis, and gap elements may be used to estimate this effect. For example, when investigating the need for restrainers at supports, displacement demands can be accurately calculated using an iterative procedure that accounts for pounding between the abutment and the superstructure (Buckle and Friedland 1995). Simplified procedures have also been developed to simulate the nonlinear abutment-superstructure interaction in design (DesRoches and Fenves 2001).

3.2—Seismic capacity evaluation

To determine the necessity and the extent of a seismic retrofit, both the component and system capacities of the existing system should be estimated. The current design philosophy adopted in the United States for bridges in seismic regions assumes that the earthquake will mobilize the inherent strength and, therefore, a key retrofit design consideration is to ensure the formation of ductile mechanisms (suppress brittle shear and anchorage failure mechanisms) to permit inelastic structural deformations without significant loss of lateral load resistance. This design philosophy requires realistic capacity estimates and comparisons of capacities of local mechanisms within an element with respect to the adjacent elements to ensure a global ductile mechanism. The evaluation of actual member capacities requires that realistic bounds on strength be determined for individual structural components. To guard against brittle modes of failure, conservative estimates of demands should be compared with lower-bound estimates of capacities (ATC 1996). For example, shear demands computed from upper-bound flexural strengths are compared with lower-bound shear capacities. This approach can be applied to assess the seismic vulnerability of existing bridge structures and to determine, if necessary, appropriate retrofit schemes.

The key component in a comprehensive capacity-based seismic design approach is adequate evaluation of the component and system behavior under combined gravity and seismic loads. A number of research efforts describe procedures and provide equations for estimating the critical capacities (Priestley 1991; Priestley and Seible 1991; McLean and Marsh 1999; ATC 1996; Priestley et al. 1996; California Department of Transportation 1999a,b; FHWA 2006). A realistic estimate of the component capacities and critical mechanisms of an existing bridge structure may be based on the following steps.

1. *Determine the expected material properties*. Due to the conservative nature of concrete mixture proportions and the increase in compressive strength that commonly occurs over time, the actual concrete strength can be significantly higher than the specified design compressive strength $f'_{c,design}$ for existing concrete structures. Similarly, to limit the probability of yielding occurring below the nominal yield strength, mills produce reinforcement that typically exceeds the specified yield force. In addition, the ultimate strength of the reinforcement will be considerably higher than the yield strength because of strain hardening of the steel. Steel strengths, due to overstrength and strain hardening effects, may be increased by 50% over the specified yield strength (Moehle et al. 1994). Studies have shown that the concrete strength may increase substantially (from 30 to 100%) due to aging and environmental effects (Wood 1991). This discrepancy

between the design and actual material properties is of a concern because it can lead to members with flexural strengths that will exceed the nominal strength, which will impose higher shear demands on the structural elements. If material testing has not been performed on the existing structure, a 50% increase in the specified concrete strength $f'_{c,design}$ and a 10% overstrength in reinforcement yield strength $f_{y,design}$ are commonly used in practice as estimates of existing conditions (ATC 1996). Somewhat higher material strengths (that is, 70% increase in concrete strength and 10% increase in reinforcement yield strength) have been suggested in MCEER-ATC (2003). To more realistically account for the properties of the reinforcement and the concrete, including the effect of strain hardening, mixture proportion, environmental conditions, and age effects, an accurate assessment of the material properties should be made through testing and a detailed evaluation of the condition of the structure should be conducted.

2. *Calculate the flexural capacities for individual beam and column members from moment-curvature analyses*. The capacities of critical regions should be determined using realistic estimates of the axial loads, material properties, and stress-strain relationships that account for the confinement effects in the concrete and strain-hardening behavior in the longitudinal reinforcing steel. Because the cap-beam flexural strength is enhanced by composite action with an integral superstructure, the moment capacity depends on the effective width of the superstructure used in the analysis. The value of the effective width should be appropriately determined to accurately model the strength and stiffness of the cap beam.

Insufficient development lengths of the main reinforcement, lap splices with insufficient lap length or confinement, or both, may limit the moment capacity of a flexural member under cyclic loading. In such circumstances, flexural capacities should be adjusted appropriately using guidelines available on the evaluation of reinforcement development under seismic load conditions (Priestley et al. 1992a, 1996). Because the development length depends on the cyclic loading demand, capacity design may require an upper-bound estimate of the flexural strength.

3. *Calculate the lower-bound member shear capacities based on specified, not expected, material properties*. Typically, expressions to account for the shear strength of a member with shear reinforcement explicitly account for axial load effects and degradation of the shear strength with increasing ductility demand. Several equations have been proposed for estimating shear capacity, and these equations do provide different estimates (Aschheim and Moehle 1992; ATC 1996; Priestley et al. 1996; Pujol et al. 1999; Kowalsky and Priestley 2000; Saiidi et al. 2001c; Sezen and Moehle 2004). Although there is no consensus within the engineering community concerning which equation is the best, most provide conservative estimates for the shear capacity of members tested in the laboratory. To model the shear response more realistically, the expressions should account for additional uncertainties such as axial loads that vary dynamically and complex loading conditions (six components of forces as opposed to two or three that are typically modeled in the lab).

4. *Evaluate bar anchorage and joint shear strength of beam-column and column-footing joints*. Expressions to determine the anchorage and joint strengths have been developed (Priestley et al. 1992a, 1996; Sritharan et al. 1998; Sritharan and Ingham 2003). These expressions are empirical to a large extent; however, they have been shown to be adequate in large-scale laboratory tests (Ingham et al. 1997, 1998; Sritharan 2005a,b).

5. *Determine strengths of substructure elements (footings, pile-cap connections, and piles)*. In a similar manner to Step 4, the strengths of the substructure elements should be calculated and compared with the demands on them (Priestley et al. 1992a; Xiao et al. 1993; Roeder et al. 2005).

6. *Determine response of the bridge using the behavior characteristics of the individual framing systems*. The load-deformation characteristics for individual bents together with the predicted behavior of the superstructure at the abutments can be used to calculate the load-deformation response of the entire bridge structure if substructuring is used in the structural analysis. Substructuring consists of modeling the structure as a series of elements and super-elements (or substructures). In the case of a bridge, the individual bents may be modeled as super-elements using matrix condensation to reduce the number of degrees of freedom in the structural model. Therefore, the force-displacement response is expressed in terms of the degrees of freedom at the interface. The calculated lateral load and expected deformation capacity or displacement ductility ratio, $\mu_\Delta = \Delta_u/\Delta_y$, in which Δ_u and Δ_y are the ultimate and yield displacements of the frame system, respectively, can now be compared with the required seismic load demand and the associated deformation or ductility demands. If the analysis indicates that some elements will sustain inelastic action, capacity evaluation procedures can be used to determine the plastic force demands on that and adjacent elements.

3.3—Evaluation of demand-capacity ratios

The results of the capacity and demand evaluations are used to determine the demand-capacity ratios for each component and the overall structure. These results can be used to determine potential locations of damage and develop appropriate local and global retrofit measures.

As indicated previously, the global seismic demands on the bridge are usually based on the linear or nonlinear analysis results. A global linear or nonlinear analysis will provide estimates of the overall displacement or drift demand. For elastic systems, results of the linear dynamic analysis can be used to estimate the local element demands. For yielding systems, the local deformation and force demands can be estimated directly from a nonlinear dynamic analysis or by pairing the results of a linear dynamic analysis with a static nonlinear analysis procedure. (Note that the latter approach uses the equal-displacement rule because this is typically an appropriate assumption for bridges that tend to be long-period structures.) These results can be compared with local and global displacement capacities. Damage should be expected in elements and systems for which the demand/capacity ratios exceed 1.

The results of the demand-capacity ratio evaluation should provide insight into the local and global retrofit needs and effective retrofit measures, which are discussed in the next chapter.

CHAPTER 4—SEISMIC RETROFIT MEASURES

The current bridge design philosophy in high seismic regions in the United States, similar to seismic design philosophies in New Zealand and Europe, is to design the structural system to meet the strength and displacement demands that result from the design-level earthquake(s). Initially, a suitable yield mechanism is selected for the system. Typically, the bridge is designed to restrict seismic inelastic action in locations that are accessible, inspectable, and allow for convenient and efficient repair following a large seismic event without extended periods of bridge closure (Calvi and Priestley 1991; Priestley et al. 1996; Hipley 1997).

Consistent with the capacity-design approach, adjacent, nonyielding elements are designed to remain elastic when subjected to the demand resulting from plastic action in the yielding elements. The yielding elements, typically the columns, are designed to suppress nonductile response modes (shear and bond failure), and are detailed to meet the implied displacement demands. Contrary to current building design practice in which a weak beam/strong column concept is used to force plastic hinging into the beams, in bridge design, plastic hinging in the columns is preferred because columns are easier to inspect, retrofit, and repair than the superstructure elements in bridges.

The aforementioned design philosophy can also be used to choose and design retrofit measures for an existing bridge system. In this case, regions of inelastic action are selected, ensuring a desirable yield mechanism for the system. The elements that are expected to sustain the cyclic, inelastic action are retrofitted to sustain the seismic force and ductility demands such that brittle response mechanisms are suppressed. All other members and connections should be retrofitted to respond in an essentially elastic manner when subjected to the forces resulting from plastic action in the yielding elements. Typically, this type of retrofit design will focus on providing sufficient ductility capacity to the potential plastic hinge regions in columns while strengthening all other structural members based on capacity-design principles using appropriate overstrength factors for the column flexural strength.

In some cases, as an alternative or in addition to strengthening, additional elements may be added or other measures can be used to reduce the seismic demand and thereby avoid significant inelastic response of structural members. These retrofit strategies may be more cost effective than an element-by-element retrofit approach.

The most common seismic vulnerability indicators and seismic deficiencies in structural members of existing concrete bridge structures were presented in Chapter 2. These findings can be combined with a demand analysis procedure described in Chapter 3 to identify the damage potential, failure modes, or both, of the bridge. The retrofit measures are then selected to minimize damage and prevent collapse of the bridge system under the design level or greater-intensity earthquakes.

Once retrofit measures at the member level have been developed, a good retrofit design must consider the desired seismic response of the structure as a whole. The system response should be re-evaluated through an analysis of the retrofitted system considering the response of the retrofitted, unretrofitted, and new elements. Redesign of the retrofit measures or a new retrofit scheme may be selected if the system response does not adequately meet the strength and displacement demands expected under the design-level earthquake(s).

Conceptual retrofit measures for deficiencies in the elements discussed in Section 2.2 are presented in Sections 4.1 to 4.9. The dynamic isolation concept for seismic retrofit of bridges and some general retrofit considerations are also presented. Retrofit measures for vulnerabilities related to poor soil conditions may require soil improvement, which is beyond the scope of this document and can be found in Kramer and Holtz (1991) and Kramer (1996).

4.1—Columns

In concrete columns, locations wherein the moment demand exceeds the yield moment are potential plastic hinge regions. In these locations, adequate lateral confinement should be provided to ensure satisfactory inelastic behavior of the plastic hinges and to prevent the longitudinal bars from buckling prematurely or lap splices from debonding. Flexural strength and concrete compressive strain capacity depend on the level of lateral confining pressure (Richart et al. 1929; Park and Paulay 1975; Sheikh and Uzumeri 1980; Mander et al. 1988; Mander and Chen 1992; Saatcioglu and Razvi 1992; Seible et al. 1990; Priestley et al. 1992a).

Retrofit concepts that provide an approximately uniform confining pressure over the column core, such as wraps or jackets, are most effective. Circular columns are more easily retrofitted with circular wraps or jackets. For circular column retrofit with jackets, a small annular gap is left between a preformed jacket and column, and then filled with nonshrink, cementitious grout. Rectangular or oblong columns can be outfitted with elliptical jackets to improve the confinement to the concrete and enhance the shear resistance (Fig. 4.1). Elliptical jackets will form a large gap between the rectangular column and jacket, which may be filled with regular concrete. Figure 4.2 shows laboratory proof tests for columns with circular and oval steel jackets. Alternatively, rectangular column sections may be modified to desirable shapes using precast concrete panels (Fig. 4.3) before implementing a desired retrofit technique to improve the confinement.

Retrofit measures using circular and elliptical steel jackets, advanced composite jackets, and high-strength prestressed wire wraps have proven to be effective in providing large and stable hysteretic response to displacement ductilities of six to eight (Priestley and Seible 1991; Priestley et al. 1992a, 1993; Wipf et al. 1997; Gergely et al. 1998).

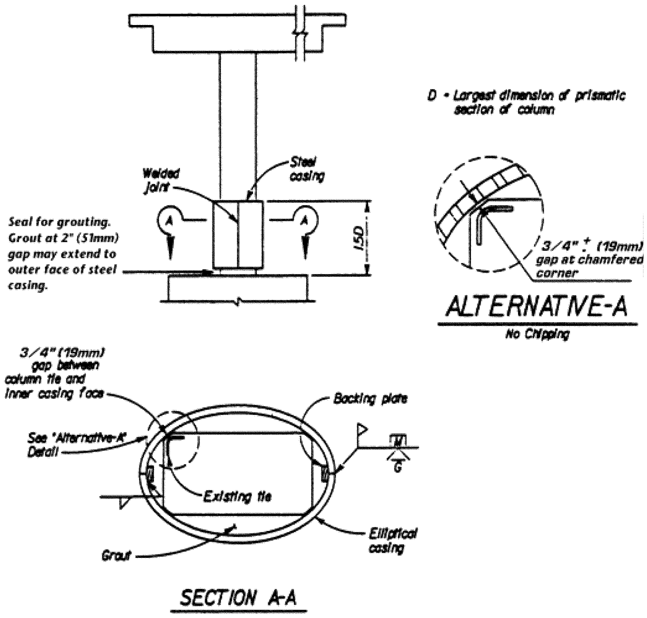
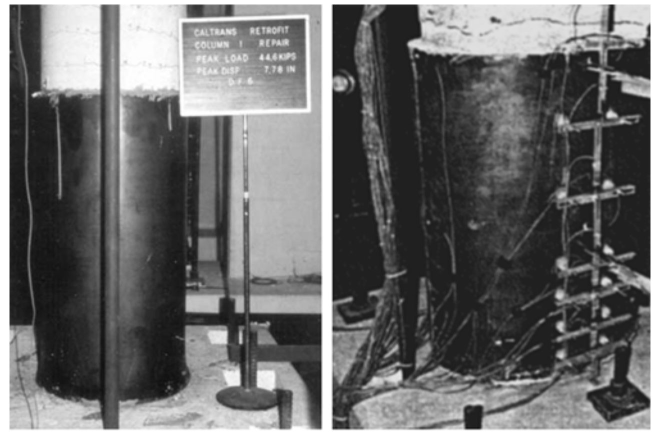


Fig. 4.1—Elliptical steel jacket retrofit for a rectangular column section.

(Advanced composite refers to materials consisting of a polymer matrix reinforced with high-strength continuous fibers of a predefined orientation.)

For both steel and advanced composite jackets, a small gap is provided between the jacket and the adjacent members, such as footings or cap beams, to allow rotation of the column without unnecessary flexural strength increase from the jacket retrofit. Steel jackets are commonly provided over a height equal to 1-1/2 to 2 times the plastic hinge zone or the full column length. They are assembled from two rolled half-shells and welded in place along two vertical seams. Full-height column jacketing may be employed in cases where aesthetic appearance is important or when additional shear capacity is needed. Advanced composite jackets can be applied either manually in the form of woven fabrics in a polymer matrix or automated by using preimpregnated tows and a winding machine. Application of prefabricated advanced composite shells to retrofit column confinement regions has also been proven to be successful (Nanni and Bradford 1995; Xiao and Ma 1997).

With an improved confinement, the column section is expected to sustain large curvature ductility demands (of up to 20) as implied by current seismic design provisions (ATC 1996), where the curvature ductility of a concrete section is defined by the ratio between the ultimate curvature and yield curvature. The effective elastic stiffness of the column may be increased by 10 to 75%, depending on the original shape of the column section, retrofit material, jacketed cross-section dimensions, and extent of retrofit (Priestley et al. 1996). The appropriate stiffness of the column, which may be determined using section analysis results and proper accounting for the strain penetration effects of the bars into the footing (Elwood and Eberhard 2006), should be used in the system level analysis of the retrofitted bridge.



(a) Circular column *(b) Rectangular column*

Fig. 4.2—Laboratory testing of retrofitted columns. (Courtesy of University of California, San Diego.)

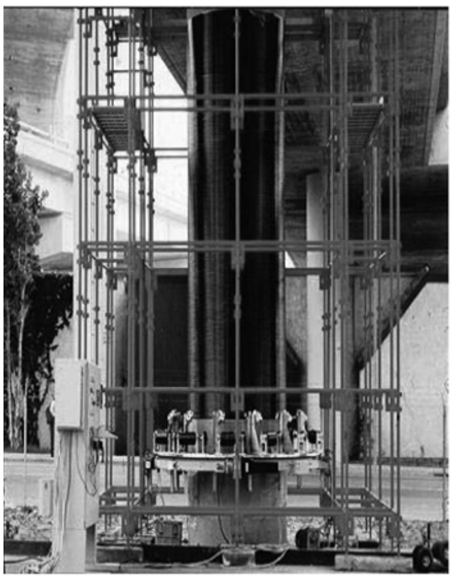
Fig. 4.3—Modifying a column section using precast concrete panels (US40/I64 double-deck seismic retrofit, St. Louis, Mo.). (Courtesy of Jacobs Civil Inc.)

Detailed test data and design equations to determine the jacket thickness for different retrofit types can be found in the cited references (Priestley and Seible 1991; Chai et al. 1991; Stone and Taylor 1993; Priestley et al. 1996; Xiao and Ma 1997; FHWA 2006). Field examples showing application of different retrofit measures that improve lateral confinement to the plastic hinge region or the entire column are illustrated in Fig. 4.4 through 4.6.

In columns where the variation of the column section along the height is not linear, multi-segment jackets may be used (Saiidi et al. 2001c). Installing unidirectional composite fabrics on nonprismatic columns can be problematic because the direction of the fibers changes as the fabrics wrap around the member. This reduces the effectiveness of fibers in the transverse direction while introducing undesirable flexural strength enhancement. A method to apply U-shaped overlapping fiber-reinforced polymer (FRP) straps has been developed and verified experimentally (Saiidi and Cheng 2004). Figures 4.7(b) and (c) show the installation of the FRP and glass fiber-reinforced polymer (GFRP) straps on a test column and on a bridge column in Reno, Nev., respectively. The bridge completed with GFRP strap column retrofit in Reno is shown in Fig. 4.8.

(a) Steel jacket retrofit in critical column regions (*Courtesy of Jacobs Civil Inc.*)

(b) Steel jacket retrofit over the entire column height *(Courtesy of University of California, San Diego)*



(c) Retrofit using carbon fiber reinforced polymer (CFRP) (*Courtesy of FHWA*)

(d) Retrofit using active prestressed wire wraps and use of welded wire fabric in cover concrete (*Courtesy of Jacobs Civil Inc.*)

Fig. 4.4—Field examples of column retrofit.

(a)

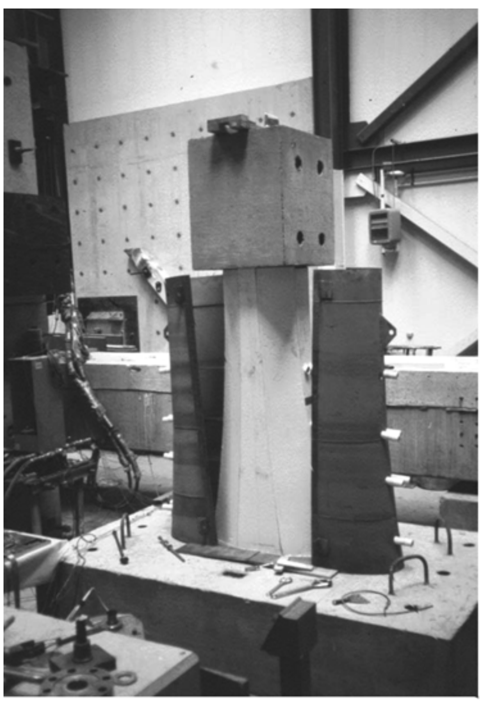
(b)

Fig. 4.5—Field application of external prestress wire-wrap retrofit: (a) I70/55 Poplar Street Approach Complex, East St. Louis, Ill. (Courtesy of Jacobs Civil Inc.); and (b) a close-up view. (Courtesy of St. Louis Bridge Co.)

A retrofit alternative for improving behavior of multi-column bents, especially in the transverse direction of the bridge, is the addition of infill walls between columns, as seen in Fig. 4.9. The walls are anchored to the footing, to the columns, and, generally, to the cap beam. Because of the high strength and stiffness of walls, the seismic demands in the cap beam and columns can be significantly reduced and the overall performance of the bent can be improved (Pulido et al. 2004). If the top of the wall is not connected to the cap beam, to facilitate construction, the lateral inertial load path can pass through the columns before the wall is mobilized, which can lead to column damage within localized regions.

Inadequate lap splices in columns can be retrofitted by providing sufficient confinement to the spliced region using steel or composite jackets. This retrofit measure effectively controls dilatational strain in the radial direction and prevents premature splice failure. When the lap splice is

Fig. 4.6—Field application of prefabricated composite jacketing of column. (Courtesy of University of Southern California.)



(a) Installation of half shell steel jackets

(b) Installation of FRP straps

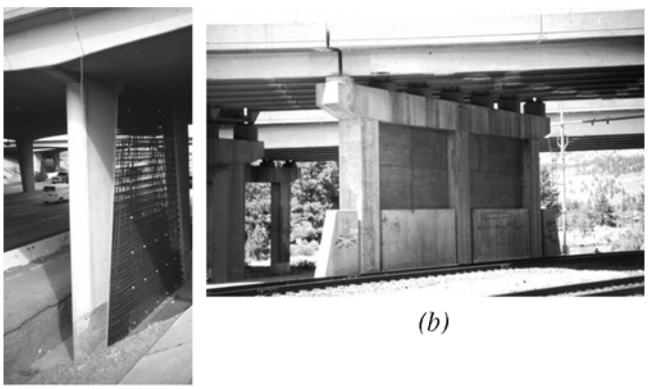
(c) Installation of U-shaped GFRP straps

Fig. 4.7—Seismic retrofit of nonprismatic columns. (Courtesy of University of Nevada, Reno.)

located within a plastic hinge region, the total amount of confinement required for enhancing the inelastic capacity of the plastic hinge and splice retrofit need not be considered as the sum of the individually required amounts because the individual deficiencies are critical on opposite sides, the compression region, and tension region, respectively (Priestley et al. 1996; Darwish et al. 1999).

Circular wraps or jackets for circular columns and elliptical jackets for rectangular columns have been shown to improve flexural capacity of columns with inadequate lap splices (Chai et al. 1991; Innamorato et al. 1996; Saadatmanesh et al. 1996; Wipf et al. 1997). Research on rectangular building columns with inadequate lap splices found that rectangular steel jackets with lengths 1-1/2 times the splice length

Fig. 4.8—Flared columns retrofitted with U-shaped GFRP straps and painted. Bridge 1250, Reno, Nev. (Courtesy of University of Nevada, Reno.)



(b)

(a)

Fig. 4.9—Infill wall retrofit in multi-column bridge bents: (a) during construction at US 395/I 80 interchange, Reno, Nev.; and (b) view of a completed bent. (Courtesy of University of Nevada, Reno.)

improved the response (Aboutaha et al. 1996). Others have found that stiffened rectangular jackets used in columns with lap splices in the plastic hinge zones were less effective in preventing debonding of the splice than the elliptical steel and advanced composite jackets (Seible et al. 1990). Additionally, research on rectangular building columns has involved methods of strengthening lap splices by welding the spliced bars for continuity and enhancing confinement through external ties, internal ties (which involved removal and replacing of cover concrete), or a combination of steel angles and straps. Steel angles were positioned at the column corners and connected by welding the steel straps to the angles. All but the method using internal ties, which involved removal of existing concrete cover and placing new ties internally, performed well (Valluvan et al. 1993).

For multiple column bents with inadequate lap splices at the base of the columns, an alternative retrofit measure can be considered. To decrease the stiffness and flexural demands on the column, the flexural resistance of the column can be reduced by removing some of the concrete and severing some of the longitudinal reinforcement at the column end region and adding additional material to prevent corrosion. Reducing the moment resistance at the base to a significantly small value may allow the column-footing connection to behave more like a pin and limit the seismic demands into the footings as well. Close attention should be given to ensure that the reduced section can sustain the shear and axial force demands.

Columns with inadequate shear capacities will need to be retrofitted to sustain the maximum possible shear demands. Once the column end retrofits are designed to meet the flexural ductility demands, the shear demand in the column needs to be evaluated assuming full development of column plastic moments (including material overstrength and strain hardening effects). If a satisfactory yield mechanism is chosen that does not require development of hinges in the column, the shear demand should be determined with full development of plastic hinges at the appropriate locations. In cases where shear strength of the existing column is not adequate, placement of additional confinement along the full height of the column is recommended (as shown in Fig. 4.4(b) and (c) and Fig. 4.8). The column shear strength can be increased by using retrofit measures including external hoop reinforcement as well as elliptical, circular, or rectangular jackets using steel or composite materials. Detailed design criteria for shear jacketing can be found in Chai et al. (1991) and Priestley et al. (1996). The use of rectangular steel jackets with pressure grout between the concrete and jacket is not recommended to improve the shear strength of large bridge columns because there is a tendency for the jacket to bow out (Priestley et al. 1996).

4.2—Cap beams

Cap beams of multiple column bents may require retrofit to increase shear resistance, to develop the capacity of insufficiently anchored top and bottom reinforcement, or both. In many cases, the bottom reinforcement at the connection to the column is minimal and discontinuous. These seismic deficiencies are critical under transverse seismic response, and can be greatly improved by using external unbonded post-tensioning as shown in Fig. 4.10. This retrofit strategy, which may require an increase in joint size, also improves the joint performance (Priestley et al. 1992b; Ingham et al. 1998; Lowes and Moehle 1999; Sritharan and Ingham 2003). Anchorage of the post-tensioning tendons at the beam ends may require the addition of end blocks. In box-girder bridges with integral bent caps, tendons may be placed along the existing cap beam via ducts placed in holes cored through the longitudinal girder webs.

Where the use of prestress alone is not sufficient to provide the required shear capacity, or where prestressing is not a practical solution, concrete bolsters with shear reinforcement on either side of the cap can be provided by doweling into the existing beam (Lowes and Moehle 1999). To ensure satisfactory cap-beam performance, a sufficient number of closed ties and dowel bars connecting the bolsters to the beam should be provided based on the demands estimated using the overstrength column moment capacity.

Fig. 4.10—Cap beam retrofit using unbonded post-tensioning. (Courtesy of Jacobs Civil Inc.)

Fig. 4.11—Cap beam and joint retrofit using link beams. (Courtesy of University of California, San Diego.)

Research shows that if such a modification is used along with beam post-tensioning, it is possible to develop the plastic moment capacity in the column hinge (Lowes and Moehle 1999). When concrete bolsters are added to the sides of the beam, additional beam longitudinal reinforcement can also be introduced to enhance the flexural resistance of the cap (Thewalt and Stojadinovic 1995; Lowes and Moehle 1999).

A steel jacket connected to the concrete with epoxy and high-strength through-bolts was demonstrated to be an effective cap beam shear retrofit method (Thewalt and Stojadinovic 1995). Recent research has shown that shear and flexural deficiencies of the cap beam can also be significantly improved by providing composite wraps around the beam (Gergely et al. 1998).

Under longitudinal seismic response, integral cap beams can have insufficient torsional resistance. Torsional strength enhancement can be achieved by introducing prestressing in the longitudinal direction of the beam (Fig. 4.10) by providing a reinforced concrete or steel jacket, or both, around the existing cap beam (Priestley et al. 1996). In a series of tests, jackets with high-strength through-bolts improved the torsional resistance of cap beams (Taylor and Stone 1993). The use of post-tensioning to enhance cap beam torsional resistance has also been demonstrated through large-scale bridge model testing with precast concrete girders (Holombo et al. 2000).

Several of the aforementioned retrofit measures can be expensive and difficult to perform on integral cap beams of multiple column bents. An alternative approach that reduces the demand on the cap beam should also be considered wherever possible. For example, introducing a link beam below the cap will reduce the demand on the cap beam if the column plastic hinging is forced to develop adjacent to the soffit of the link beam. The shifting of the plastic hinge location will lead to an increase in the column shear demand, which should be adequately addressed in the assessment of the column behavior. This approach, which increases the lateral stiffness of the column, was successfully demonstrated in a full-scale, two-column bent test unit (Innamorato et al. 1996). A field implementation of this retrofit technique is depicted in Fig. 4.11.

4.3—Cap beam-column joints

Cap beam-column joints in concrete bridge bents may require retrofit measures to limit joint damage, ensure transfer of flexural forces from the cap beam to the columns, and ensure sufficient anchorage of the column longitudinal reinforcement into the joint so that the column plastic moment capacity can be developed adjacent to the joint. Solely meeting the minimum development length requirements for the column bars into the joint may not be adequate to develop the column plastic moment capacity. In addition to satisfying the development length requirements, the column bars should be extended as close as possible to the beam top longitudinal reinforcement to ensure satisfactory force transfer across the joint (Sritharan et al. 1998). Premature termination of column bars into the joint, which was a common practice until recently, was found to be inadequate for establishing satisfactory seismic response (Ingham et al. 1998; Sritharan et al. 1998).

Joint retrofit measures investigated to date consist of complete replacement of the joint region with a replaced joint of increased dimensions; jacketing of the joint using concrete, steel or composite materials; and external prestressing of the joint region. Both joint and cap behavior can be significantly improved with these measures to the point where plastic hinging can be forced into the column with limited distress in the cap beam and joint region. Additional information on joint retrofit measures can be found in Thewalt and Stojadinovic (1995), Priestley et al. (1996), Lowes and Moehle (1999), Ingham et al. (1997, 1998), Gergely et al. (1998), and Sanders et al. (1998).

Replacement of the joint region requires removal of joint concrete, increasing joint dimensions, and placing an appropriate amount of joint reinforcement (Fig. 4.12(a)). Retrofitting with concrete jackets involves addition of concrete to the existing joint and possibly to the cap beam using dowels, new reinforcement, or both, placed through the existing joint concrete, as shown in Fig. 4.12(b) and (c).

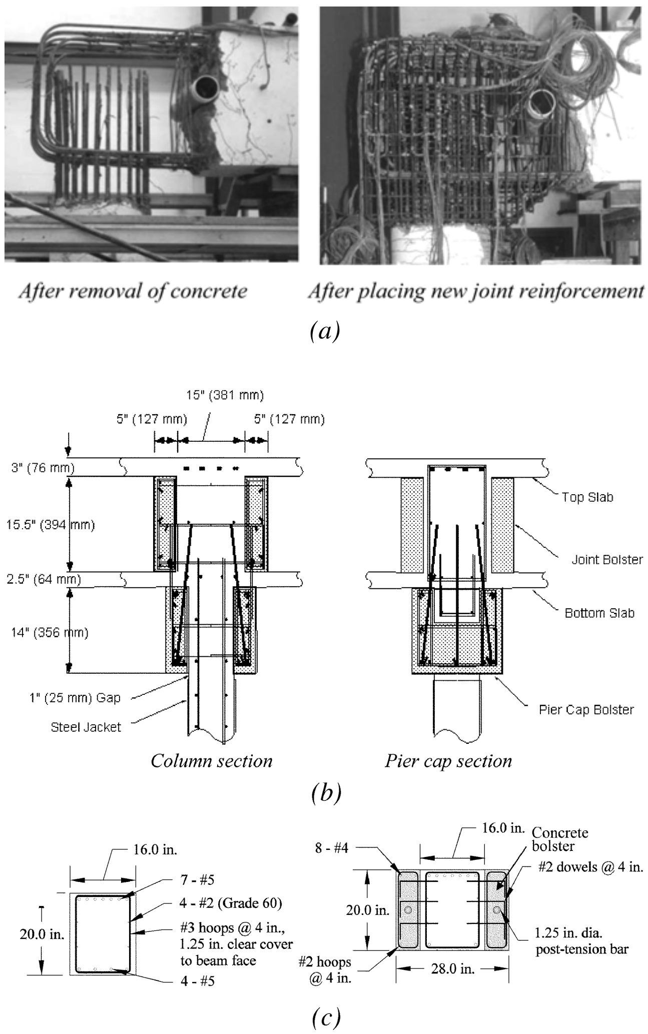


Fig. 4.12—Retrofitting measures for beam-to-column joints: (a) joint replacement (Ingham et al. 1998); (b) adding concrete bolsters to joint and cap beam (Sanders et al. 1998); and (c) adding concrete bolsters to joint and cap beam (Lowes and Moehle 1999).

These retrofitting approaches, which typically include a drop cap in the column region (Fig. 4.12(a)) or an enlargement of the cap beam and joint depth (Fig. 4.12(b)), increase the joint dimensions all around. As a result, joint shear stresses are reduced, the embedment length of the column longitudinal bars is increased, and anchorage conditions for the column and beam reinforcement into the joint are enhanced, all of which improve the force transfer across the joint. The shear reinforcement required for the force transfer across the joint is placed in the added concrete with the new concrete tied to the existing concrete by dowels. Figure 4.12 shows strengthening details used in an experimental investigation to improve poor joint condition, weak cap beam, and insufficient column confinement, which resulted in satisfactory system response up to and beyond a displacement ductility of 7.

Flat or semicircular steel jacketing of the joint and cap region can be tied to the existing concrete with epoxy and through-bolts; it is also an effective means for improving joint performance (Thewalt and Stojadinovic 1995).

Fig. 4.13—Retrofit of the joint region using carbon fibers. (Courtesy of University of Utah.)

Successful retrofit of joints using advanced composite material, as shown in Fig. 4.13, has been demonstrated in the laboratory (Gergely et al. 1998; Pantelides and Gergely 2002) with subsequent field application.

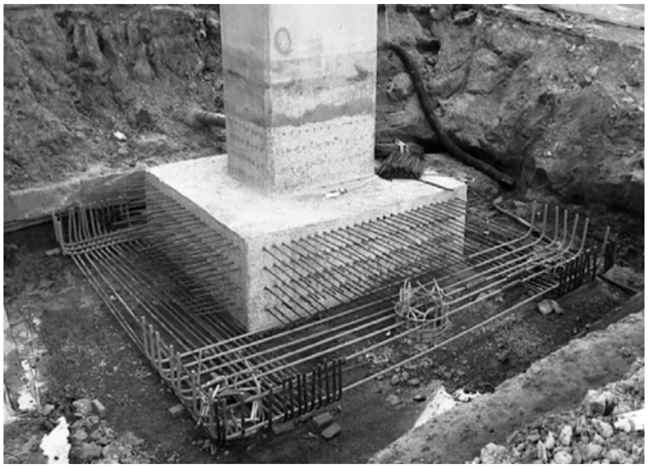
Retrofitting joints with external prestressing can be accomplished relatively easily compared with the alternatives described previously. Prestressed joints have several advantages over equivalent nonprestressed joints. Prestressed joints will experience a relatively low amount of joint cracking and subsequent damage, have an improved anchorage condition for the column bars into the joint, and show improved hysteresis behavior of the bridge system (Sritharan et al. 1999; Sritharan 2005b). As prestressing elevates the joint principal compressive stresses, there is a tendency for the joint to fail due to crushing of concrete, which should be avoided. Increasing cap width in the joint region with concrete bolsters is recommended to improve the effectiveness of an external prestressing retrofit for joint response in the longitudinal direction (Priestley et al. 1996).

An alternative retrofit measure to reduce joint damage is to reduce the shear demand using a link beam; it is similar to the procedure discussed for cap beams (Innamorato et al. 1996; Priestley et al. 1996).

4.4—Footings

Inadequate flexural resistance, shear resistance, and stability problems are typical seismic deficiencies of bridge footings. Adding a new layer of reinforced concrete to the existing concrete with dowels, as illustrated in Fig. 4.14, enhances the flexural resistance of the footing (Xiao et al. 1993). Coring through the existing footing and applying prestressing also improves the flexural resistance, but is relatively more expensive due to the high cost associated with concrete coring.

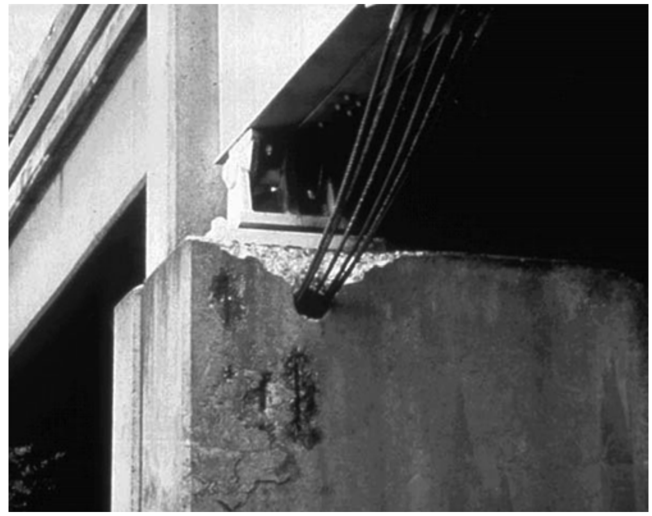
In multi-column configurations, modifying the column end regions to represent pinned base conditions, as discussed in Section 4.1, can eliminate the column-to-footing joint shear problems. This solution is desirable in all cases because it reduces the lateral strength of the bridge bent in both the transverse and longitudinal directions.



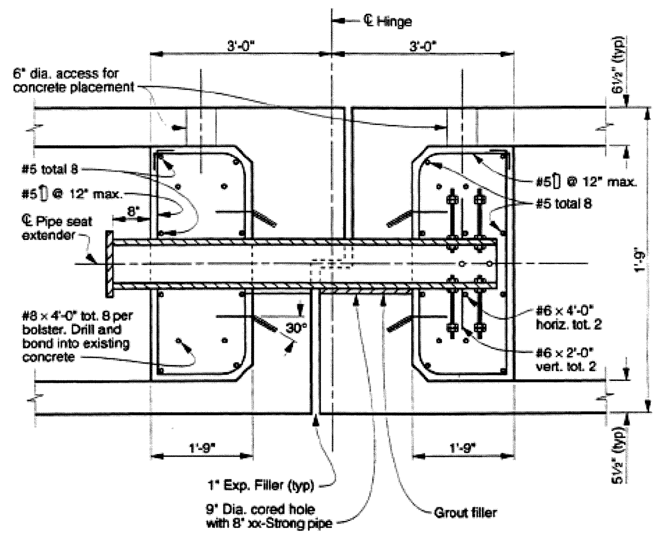
(a)

(b)

Fig. 4.14—Foundation retrofit of a bridge footing: (a) adding new piles, footing dowels, and bottom reinforcement; and (b) view of the existing footing and new reinforcement prior to placing concrete. (Courtesy of Jacobs Civil Inc.)



(a)



(b)

Fig. 4.15—Retrofit measures using restrainers: (a) cable restrainer retrofit (Courtesy of Georgia Institute of Technology.); and (b) typical pipe restrainer detail.

For seismically deficient footings, a retrofit solution to improve the joint shear transfer between column and footing may be necessary. Implementation of such a solution can be difficult and costly. Investigating possible alternative paths to transfer forces across the joint can eliminate the need for shear retrofit of footings.

Increasing footing depth by adding a new layer of concrete to the existing footing, prestressing the footing, or both, may increase the flexural resistance and minimize joint shear cracking by reducing the principal tensile stresses in the column-footing connection. When a concrete layer is added with dowels extended to the bottom of the existing footing, the joint shear transfer can be improved by accounting for the contribution of the dowels (Priestley et al. 1996; McLean and Marsh 1999; Saiidi et al. 2001d). Again, investigating alternative paths for the joint force transfer is emphasized for efficient use of the dowels.

When footings have inadequate overturning capacity, rocking occurs. This response mechanism is not necessarily detrimental to seismic performance of the structure. The rocking potential of spread footings may protect the footing and structure from excessive forces by serving as a form of seismic isolation (Xiao et al. 1993; Priestley et al. 1996; Saiidi et al. 2002). Rocking increases the lateral displacement of the superstructure. If necessary, rocking can be eliminated using soil anchor tie-downs.

4.5—Hinges and supports

High-strength threaded rods (Fig. 1.2) and cable restrainers (Fig. 4.15(a)), and pipe seat extenders (Fig. 4.15(b)) are typically used to retrofit hinges, where hinges refer to locations of pins or other mechanical devices designed to permit rotation, with inadequate seat widths (DesRoches and Fenves 1996; Selna et al. 1989a,b; Pfeifer et al. 2002). Restrainers limit the longitudinal opening of the hinge, while seat extenders increase the effective width of the seat and function as shear keys resisting relative lateral displacements. Both restrainers and seat extenders are installed in cored horizontal holes through the end diaphragms and hinge. Concrete bolsters are usually doweled into the end diaphragms to facilitate distribution of forces into the girder webs and deck as well as the bottom slab if it exists.

Vertical cable restrainers are added where uplift is considered likely during a seismic event. These restrainers work in much the same manner as horizontal restrainers. They should be detailed to allow horizontal movements.

A range of design methods have been developed for the design of restrainers (Saiidi et al. 1996; Hudgings 1996; Trochalakis et al. 1997; DesRoches and Fenves 2001; Saiidi et al. 2001a). Research has shown that accurate design of restrainers may require substantial effort, while simplified methods may suffice in most cases.

4.6—Superstructures

In cases where the superstructure has insufficient capacity to force plastic hinging into the column under longitudinal seismic action, superstructure capacities can be improved by locking hinges and movement joints to balance capacities on both sides of the bent, adding bottom soffit slabs inside existing box sections with additional, continuous reinforcement through the cap, or adding longitudinal post-tensioning. These retrofit solutions are not always feasible because they can be complex and expensive (Priestley et al. 1996). Alternative solutions may need to be considered. For example, a retrofit scheme that was proposed and proof tested for the San Francisco double-deck viaduct retrofits used external edge girders in the longitudinal direction, which were connected to the exterior columns and superstructure at the lower deck level (Priestley et al. 1992b, 1996). The connection between the additional edge girders and the existing columns was detailed to transfer shear and moment and designed to carry a major portion of the longitudinal seismic framing action directly into the columns, thus protecting the superstructure and cap-beam region from large flexural and torsional forces. This retrofit concept, which is similar to the link-beam concept shown in Fig. 4.11, has been reported to have cost as much as that estimated for a replacement structure (Priestley et al. 1996).

4.7—Abutments

Abutment seats may be widened to prevent the superstructure from falling off the seat where cable or high-strength rod restrainers and pipe seat extenders may not be practical. Widening of abutments is often accomplished by doweling into the face of the abutment stem and adding a block of concrete either cantilevered from the stem or supported directly off of the abutment footing. Such widening is particularly useful where capacity of the abutment is limited and strengthening the abutment is not appropriate. Elastomeric or isolation bearings, as detailed in the subsequent section, may be installed as an alternative measure to limit transfer of forces to the abutment or piles. The use of elastomeric bearings at the abutments requires that the adjacent bents have adequate seismic resistance and can accommodate the increased displacements resulting from the abutment retrofit solution.

Lateral displacements of the superstructure can be restrained by adding shear keys on the abutment seat (Silva et al. 2003) or through the addition of large-diameter pile shafts. The piles can be used to improve the lateral load resistance of the abutments or be tied directly into the end diaphragms to reduce the lateral loads on the abutments.

4.8—Dynamic isolation and mechanical devices

Retrofit techniques that change the dynamic characteristics of bridge structures, such as damping systems, isolation bearing systems, and lock-up devices, have been implemented in existing bridge structures (AASHTO Standard Specifications for Highway Bridges; AASHTO LFRD Bridge Design Specifications; Mayes 1993; Stanton et al. 1993; Zayas et al. 2002). These devices, many of which are proprietary in nature, may influence the structural demand by decreasing the structural stiffness or increasing the effective damping ratio, altering the structural capacity by increasing the drift capacity, or modifying both.

Isolation bearings typically function by reducing the effective stiffness of a structure, increasing the fundamental period (away from the peak in the acceleration response spectrum), thereby reducing the force transmitted to supporting elements of the structure while serving as the seismic energy dissipaters. Damping devices absorb energy, thereby reducing the seismic forces. For nonseismic load effects, the devices should be designed to meet serviceability requirements and achieve the required force transfer.

Damping devices have been used mostly for very large, critical, or essential structures, but may have application for more typical bridges. There are four basic types of damping devices: friction, metallic, visco-elastic, and fluid viscous. Friction dampers rely on frictional forces to dissipate energy. Metallic dampers use the deformation of metal elements within the damper to dissipate energy. Visco-elastic dampers rely on the controlled shearing of solids such as rubber or neoprene. Fluid viscous dampers use the forced movement of fluids within a confined cylinder, similar to automobile shock absorbers.

Lock-up devices provide a method for distributing forces from seismic events (and other fast-acting loads) to substructure elements that would otherwise allow free movement, while still allowing essentially unhindered movement under slow-acting loads (such as thermal, shrinkage, and creep). Lock-up devices act as rigid links under fast-acting forces, but do not transfer force under slow-acting forces. They are not, however, energy absorption devices.

At least three types of isolation bearings have been used for bridge seismic retrofits in recent years: lead-rubber bearings, friction pendulum bearings, and damped sliding isolation bearings. Lead-rubber bearings, shown in Fig. 4.16, make use of the unique properties of lead to absorb energy from a seismic event, while still allowing movement resulting from slow-acting loads. In these bearings, the lead core is surrounded by a steel-laminated rubber bearing that

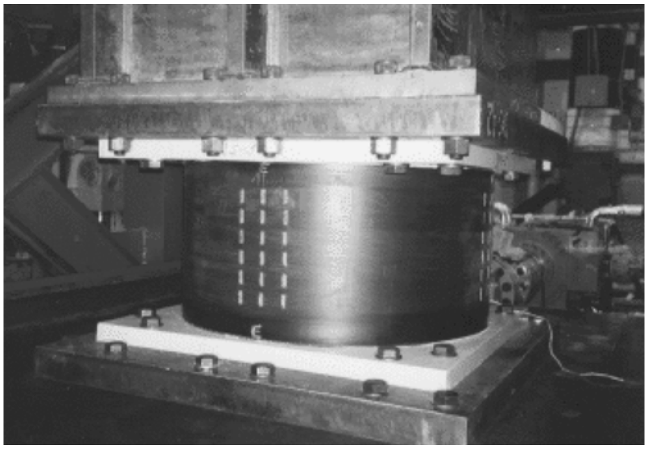


Fig. 4.16—Lead-rubber isolation bearing in testing machine. (Courtesy of FHWA.)

Fig. 4.17—Friction-pendulum bearings used on a concrete box-girder bridge. (Courtesy of HDR Engineering, Inc.)

provides flexibility and acts as a spring to help re-center the bearing after a seismic event.

Friction-pendulum bearings, shown in Fig. 4.17 and 4.18, use a concave dish to convert the lateral motion of the supported structure into vertical motion, which is resisted by gravity. These bearings reduce forces to the substructure by lengthening the structure's period. They provide damping through friction and rely on gravity for self-centering.

Damped sliding bearings, including the one depicted in Fig. 4.19, combine a shear-inhibited disc bearing assembly with a damping device to limit forces and displacements. In damped sliding bearings, kinetic energy is dissipated through friction and converted into potential energy in horizontal springs to limit forces and displacements and return the structure to its original position.

Damping systems and isolation bearings are most commonly used to retrofit structures that were originally built with articulation between the superstructure and substructure (Fig. 4.20), and therefore, they have not been widely used as retrofit measures for monolithic concrete box-girder bridges. Also, the nonductile detailing of most retrofit candidate bridges requires that the isolation system reduces the seismic force levels significantly below the

Fig. 4.18—Friction-pendulum bearing. (Courtesy of California Department of Transportation.)

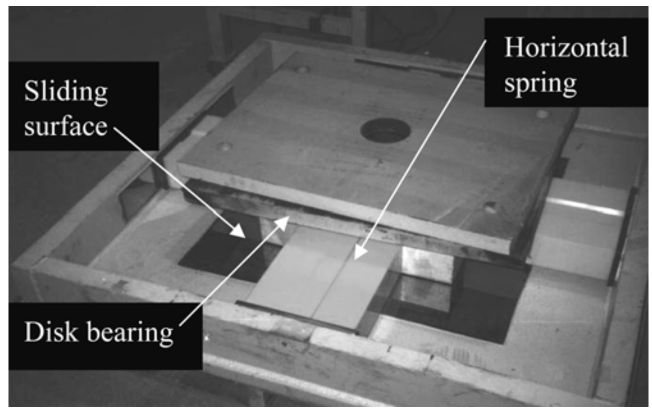


Fig. 4.19—Sliding isolation bearing with an 8 in. (200 mm) movement stroke. (Courtesy of R.J. Watson, Inc.)

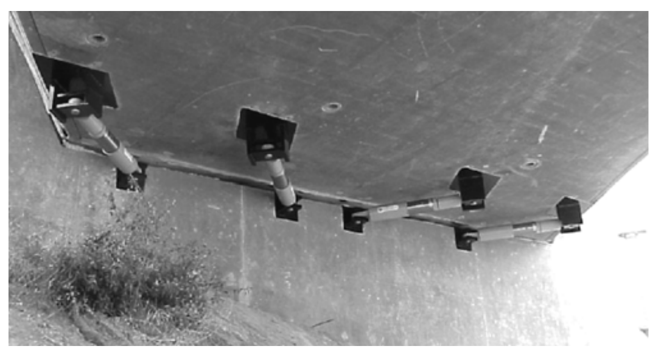


Fig. 4.20—Fluid viscous damping devices at abutment of a skewed box-girder bridge. (Courtesy of California Department of Transportation.)

proportional limit or the yield limit state. This level of reduction is not always feasible. Finally, continued inspection and maintenance may be required with some types of isolation bearings.

4.9—General retrofit considerations

Retrofit measures modify the existing structural system through either replacement or addition of components. To establish the effectiveness of the new composite action between the existing structure and the retrofit components, large or full-scale experimental testing is recommended to provide the necessary behavior verification. While most of the previously described retrofit measures were tested in large-scale laboratory experiments, extrapolations from these tests should carefully consider all parameters. New retrofit measures or technologies should be validated experimentally before installation unless their functional effectiveness can be clearly shown analytically.

Laboratory testing provides essential information to evaluate the vulnerability of elements in existing concrete bridges and to design appropriate and efficient retrofit measures for elements of the superstructure and substructure of older bridges. In addition, some field validations of retrofit techniques have taken place. In the 1994 Northridge Earthquake, all of the bridges that were retrofitted since 1989 did not sustain significant damage (Priestley et al. 1994b). Of the 84 retrofitted bridges in the Northridge area, 24 were in regions that experienced very strong ground shaking and had peak accelerations of over 0.25 g.

CHAPTER 5—CONCLUSIONS

This document has provided an overview of seismic retrofitting of concrete bridges, including discussion of vulnerable elements and existing retrofitting technologies. The seismic response of retrofitted bridges measured in the laboratory and observed after earthquakes has demonstrated that bridge performance may be improved by implementing retrofitting measures. Although several of the techniques described in this document have been examined through large-scale component-level testing typically under laboratory conditions, many of them have not been tested in real-world conditions under dynamic loading resulting from large earthquakes. Seismic demands that differ from those imposed in the laboratory or by the Northridge earthquake may result in unpredicted response of retrofitted bridges. The performance of retrofitted bridges in future earthquakes will, therefore, be of a significant value and will help to further improve seismic retrofit programs in the United States as well as in seismic regions around the world.

CHAPTER 6—REFERENCES

6.1—Referenced standards and reports

The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

American Association of State Highway and Transportation Officials (AASHTO)

AASHTO Standard Specifications for Highway Bridges

AASHTO LRFD Bridge Design Specifications

AASHTO Guide Specifications for Seismic Isolation Design

American Concrete Institute (ACI)

318 Building Code Requirements for Structural Concrete and Commentary

Federal Emergency Management Agency (FEMA)

FEMA 273 Guidelines for the Seismic Rehabilitation of Buildings

FEMA 356 Prestandard and Commentary for the Seismic Rehabilitation of Buildings

The above publications may be obtained from the following organizations:

American Association of State Highway and Transportation Officials
444 North Capitol Street NW, Suite 225
Washington, DC 20001

American Concrete Institute
P.O. Box 9094
Farmington Hills, MI 48333-9094
www.concrete.org

Federal Emergency Management Agency
500 C Street
Southwest Washington, DC 20472

6.2—Cited references

Aboutaha, R. S.; Engelhardt, M. D.; Jirsa, J. O.; and Kreger, M. E., 1996, "Retrofit of Concrete Columns with Inadequate Lap Splices by the use of Rectangular Steel Jackets," *Earthquake Spectra*, V. 12, No. 4, Nov., pp. 693-714.

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Aschheim, M., and Moehle, J. P., 1992, "Shear Strength and Deformability of RC Bridge Columns Subjected to Inelastic Cyclic Displacements," UCB/EERC-92/04, University of California, Mar., 100 pp.

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