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Earthquake Damage to Bridges

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34.1 Introduction

Earthquake damage to a bridge can have severe consequences. Clearly, the collapse of a bridge places people on or below the bridge at risk, and it must be replaced after the earthquake unless alternative transportation paths are identified. The consequences of less severe damage are less obvious and dramatic, but they are nonetheless important. A bridge closure, even if it is temporary, can have tremendous consequences, because bridges often provide vital links in a transportation system. In the immediate aftermath of an earthquake, closure of a bridge can impair emergency response operations. Later, the economic impact of a bridge closure increases with the length of time the bridge is closed, the economic importance of the traffic using the route, the traffic delay caused by following alternate routes, and the replacement cost for the bridge.

The purpose of this chapter is to identify and classify types of damage to bridges that earthquakes commonly induce and, where possible, to identify the causes of the damage. This task is not straightforward. Damage usually results from a complex and interacting set of contributing variables. The details of damage often are obscured by the damage itself, so that some speculation is required in reconstructing the event. In many cases, the cause of damage can be understood only after detailed analysis, and, even then, the actual causes and effects may be elusive.

Even when the cause of a particular collapse is well understood, it is difficult to generalize about the causes of bridge damage. In past earthquakes, the nature and extent of damage that each bridge suffered have varied with the characteristics of the ground motion at the particular site and the construction details of the particular bridge. No two earthquakes or bridge sites are identical. Design and construction practices vary extensively throughout the world and even within the United States. These practices have evolved with time, and, in particular, seismic design practice improved significantly in the western United States during the 1970s as a result of experience gained from the 1971 San Fernando earthquake.

Despite these uncertainties and variations, one can learn from past earthquake damage, because many types of damage occur repeatedly. By being aware of typical vulnerabilities that bridges have experienced, it is possible to gain insight into structural behavior and to identify potential weaknesses in existing and new bridges. Historically, observed damage has provided the impetus for many improvements in earthquake engineering codes and practice.

An effort is made to distinguish damage according to two classes, as follows:

Primary damage — Damage caused by earthquake ground shaking or deformation that was the primary cause of damage to the bridge, and that may have triggered other damage or collapse.

Secondary damage — Damage caused by earthquake ground shaking or deformation that was the result of structural failures elsewhere in the bridge, and was caused by redistribution of internal actions for which the structure was not designed.

The emphasis in this chapter is on primary damage. It must be accepted, however, that in many cases the distinction between primary and secondary damage is obscure because the bridge geometry is complex or, in the case of collapse, because it is difficult to reconstruct the failure sequence.

The following sections are organized according to which element in the overall set of contributing factors appears to be the primary cause of the bridge damage. The first three sections address general issues related to the site conditions, construction era, and current condition of the bridge. The next section focuses on the effects of structural configuration, including curved layout, skew, and redundancy. Unseating of superstructures at expansion joints is discussed in the subsequent section. Then, the chapter describes typical types of damage to the superstructure, followed by discussion of damage related to bearings and restrainers supporting or interconnecting segments of the superstructure. The final section describes damage associated with the substructure, including the foundation.

34.2 Effects of Site Conditions

Performance of a bridge structure during an earthquake is likely to be influenced by proximity of the bridge to the fault and site conditions. Both of these factors affect the intensity of ground shaking and ground deformations, as well as the variability of those effects along the length of the bridge.

The influence of site conditions on bridge response became widely recognized following the 1989 Loma Prieta earthquake. Figure 34.1 plots the locations of minor and major bridge damage from the Loma Prieta earthquake [16]. With some exceptions, the most significant damage occurred around the perimeter or within San Francisco Bay where relatively deep and soft soil deposits amplified the bedrock ground motion. In the same earthquake, the locations of collapse of the Cypress Street Viaduct nearly coincided with zones of natural and artificial fill where ground shaking was likely to have been the strongest (Figure 34.2) [10]. A major conclusion to be drawn from this and other earthquakes is the significant impact that local site conditions have on amplifying strong ground motion, and the subsequent increased vulnerability of bridges on soft soil sites. This observation is important because many bridges and elevated roadways traverse bodies of water where soft soil deposits are common.

During the 1995 Hyogo-Ken Nanbu (Kobe) earthquake, significant damage and collapse likewise occurred in elevated roadways and bridges founded adjacent to or within Osaka Bay [2]. Several types of site conditions contributed to the failures. First, many of the bridges were founded on sand–gravel terraces (alluvial deposits) overlying gravel–sand–mud deposits at depths of less than 33 ft (10 m), a condition which is believed to have led to site amplification of the bedrock motions.

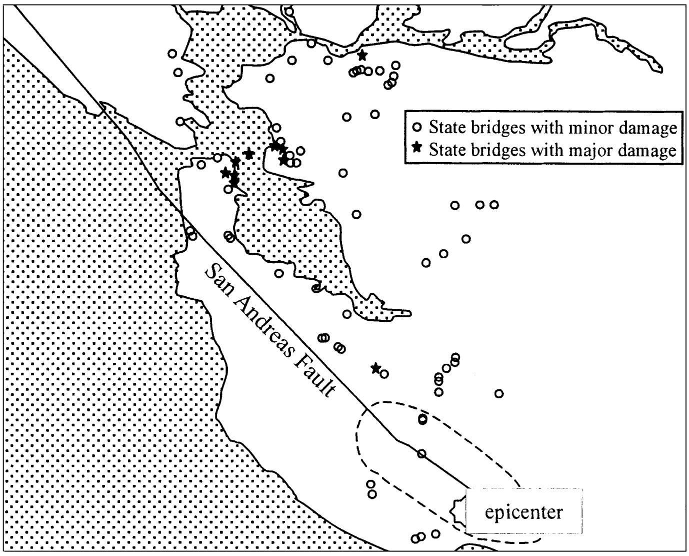


FIGURE 34.1 Incidence of minor and major damage in the 1989 Loma Prieta earthquake [modified from Zelinski, 16].

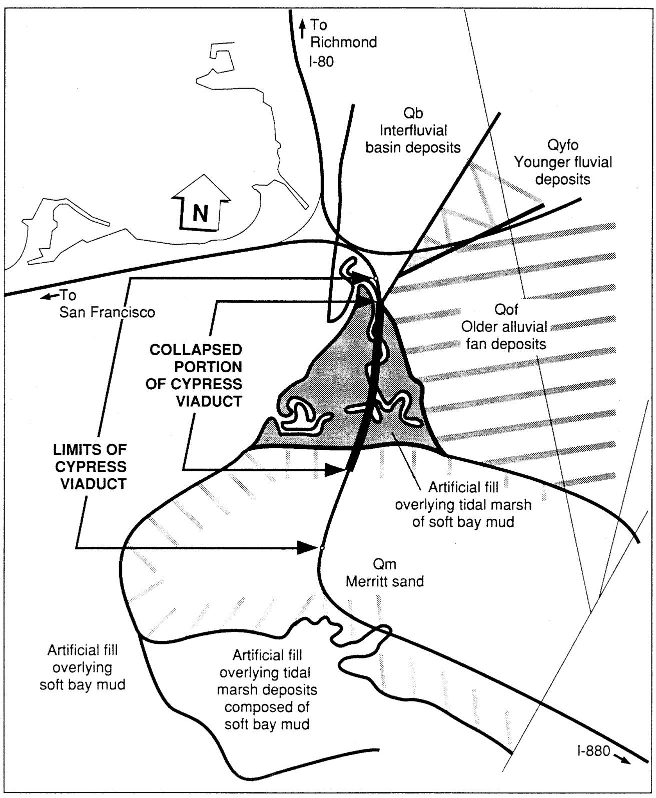


FIGURE 34.2 Geologic map of Cypress Street Viaduct site. (*Source:* Housner, G., Report to the Governor, Office of Planning and Research, State of California, 1990.)

FIGURE 34.3 Nishinomiya-ko Bridge approach span collapse in the 1995 Hyogo-Ken Nanbu earthquake [Kobe Collection, EERC Library, University of California, Berkeley].

Furthermore, many of the sites were subject to liquefaction and lateral spreading, resulting in permanent substructure deformations and loss of superstructure support (Figure 34.3). Finally, the site was directly above the fault rupture, resulting in ground motions having high horizontal and vertical ground accelerations as well as large velocity pulses. Near-fault ground motions can impose large deformation demands on yielding structures, as was evident in the overturning collapse of all 17 bents of the Higashi-Nada Viaduct of the Hanshin Expressway, Route 3, in Kobe (Figure 34.4). Other factors contributed to the behavior of structures in Kobe; several of these will be discussed in subsequent portions of this chapter.

34.3 Correlation of Damage with Construction Era

Bridge seismic design practices have changed over the years, largely reflecting lessons learned from performance in past earthquakes. Several examples in the literature demonstrate that the construction

FIGURE 34.4 Higashi-Nada Viaduct collapse in the 1995 Hyogo-Ken Nanbu earthquake. (*Source:* EERI, The Hyogo-Ren Nambu Earthquake, January 17, 1995, Preliminary Reconnaissance Report, Feb. 1995.)

era of a bridge is a good indicator of likely performance, with higher damage levels expected in older construction than in newer construction.

An excellent example of the effect of construction era is provided by observing the relative performances of bridges on Routes 3 and 5 of the Hanshin Expressway in Kobe. Route 3 was constructed from 1965 through 1970, while Route 5 was completed in the early to mid-1990s [2]. The two routes are parallel to one another, with Route 3 being farther inland and Route 5 being built largely on reclaimed land. Despite the potentially worse soil conditions for Route 5, it performed far better than Route 3, losing only a single span owing apparently to permanent ground deformation and span unseating (Figure 34.3). In contrast, Route 3 has been estimated to have sustained moderate-to-large-scale damage in 637 piers, with damage in over 1300 spans, and approximately 50 spans requiring replacement (see, for example, Figure 34.4).

The superior performance of newer construction in the Hyogo-Ken Nanbu earthquake and other earthquakes [2,8,10] has led to the use of benchmark years as a crude but effective method for rapidly assessing the likely performance of bridge construction. This method has been an effective tool for bridge assessment in California. The reason for its success there is the rapid change in bridge construction practice following the 1971 San Fernando earthquake [8]. Before that time, California design and construction practice was based on significantly lower design forces and less stringent detailing requirements compared with current requirements. In the period following that earthquake, the California Department of Transportation (Caltrans) developed new design approaches requiring increased strength and improved detailing for ductile response.

The 1994 Northridge earthquake provides an insightful study on the use of benchmarking. Over 2500 bridges existed in the metropolitan Los Angeles freeway system at that time. Table 34.1 summarizes cases of major damage and collapse [8]. All these cases correspond to bridges designed before or around the time of the major change in the Caltrans specifications. It is interesting to note that some bridges constructed as late as 1976 appear in this table. This reflects the fact that the new design provisions did not take full effect until a few years after the earthquake and that these did not govern construction of some bridges that were at an advanced design stage at that time. Some caution is therefore required in establishing and interpreting the concept of benchmark years.

34.4 Effects of Changes in Condition

Changes in the condition of a bridge can greatly affect its seismic performance. In many regions of North America, extensive deterioration of bridge superstructures, bearings, and substructures has

TABLE 34.1 Summary of Bridges with Major Damage — Northridge Earthquake

Bridge Name	Route	Construction Year	Prominent Damage
		Collapse	
La Cienega-Venice Undercrossing	I-10	1964	Column failures
Gavin Canyon Undercrossing	I-5	1967	Unseating at skewed expansion hinges
Route 14/5 Separation and Overhead	I-5/SR14	1971/1974	Column failure
North Connector	I-5/SR14	1975	Column failure
Mission-Gothic Undercrossing	SR118	1976	Column failures
		Major Damage	
Fairfax-Washington Undercrossing	I-10	1964	Column failures
South Connector Overcrossing	I-5/SR14	1971/1972	Pounding at expansion hinges
Route 14/5 Separation and Overhead	I-5/SR14	1971/1974	Pounding at expansion hinges
Bull Creek Canyon Channel Bridge	SR118	1976	Column failures

accumulated. It is evident that the current conditions will lead to reduced seismic performance in future earthquakes, although hard evidence is lacking because of a paucity of earthquakes in these regions in modern times.

Construction modifications, either during the original construction or during the service life, can also have a major effect on bridge performance. Several graphic examples were provided by the Northridge earthquake [8]. Figure 34.5 shows a bridge column that was unintentionally restrained by a reinforced concrete channel wall. The wall shortened the effective length of the column, increased the column shear force, and shifted nonlinear response from a zone of heavy confinement upward to a zone of light transverse reinforcement, where the ductility capacity was inadequate. Failures of this type illustrate the importance of careful inspection during construction and during the service life of a bridge.

34.5 Effects of Structural Configuration

Ideally, earthquake-resistant construction should be designed to have a regular configuration so that the behavior is simple to conceptualize and analyze, and so that inelastic energy dissipation is promoted in a large number of readily identified yielding components. This ideal often is not achievable in bridge construction because of irregularities imposed by site conditions and traffic flow requirements. In theory, any member or joint can be configured to resist the induced force and deformation demands. However, in practice, bridges with certain configurations are more vulnerable to earthquakes than others.

Experience indicates that a bridge is most likely to be vulnerable if (1) excessive deformation demands occur in a few brittle elements, (2) the structural configuration is complex, or (3) a bridge lacks redundancy. The bridge designer needs to recognize the potential consequences of these irregularities and to design accordingly either to reduce the irregularity or to toughen the structure to compensate for it.

A common form of irregularity arises when a bridge traverses a basin requiring columns of nonuniform length. Although the response of the superstructure may be relatively uniform, the deformation demands on the individual substructure piers are highly irregular; the largest strains are imposed on the shortest columns. In some cases, the deformation demands on the short columns can induce their failure before longer, more flexible adjacent columns can fully participate. The Route 14/5 Separation and Overhead structure provides an example of these phenomena. The structure comprised a box-girder monolithic with single-column bents that varied in height depending on the road and grade elevations (Figure 34.6a). Apparently, the short column at Bent 2 failed in shear because of large deformation demands in that column, resulting in the collapse of the adjacent spans (Figure 34.6b).

FIGURE 34.5 Bull Creek Canyon Channel Bridge damage in the 1994 Northridge earthquake.

The effects identified above can be exacerbated in long-span bridges. In addition to changes in subgrade and structural irregularities that may be required to resolve complex foundation and transportation requirements, long bridges can be affected by spatial and temporal variations in the ground motions. Expressed in simple terms, different piers are subjected to different ground motions at any one time, because seismic waves take time to travel from one bridge pier to another. This effect can result in one pier being pulled in one direction while the other is being pushed in the opposite direction. This complex behavior is not accounted for directly in conventional bridge design. An example where this behavior may have resulted in increased damage and collapse is the eastern portion of the San Francisco–Oakland Bay Bridge (Figure 34.7a). This bridge includes a variety of different superstructure and substructure configurations, traverses variable subsoils, and is long enough for spatial and temporal variations in ground motions to induce large relative displacements between adjacent bridge segments. The bridge lost two spans, one upper and one lower, at a location where the superstructure was required to accommodate differential movements of adjacent bridge segments (Figure 34.7b).

34.6 Unseating at Expansion Joints

Expansion joints introduce a structural irregularity that can have catastrophic consequences. Such joints are commonly provided in bridges to alleviate stresses associated with volume changes that occur as a bridge ages and as the temperature changes. These joints can occur within a span (in-span hinges), or they can occur at the supports, as is the case for simply supported bridges.

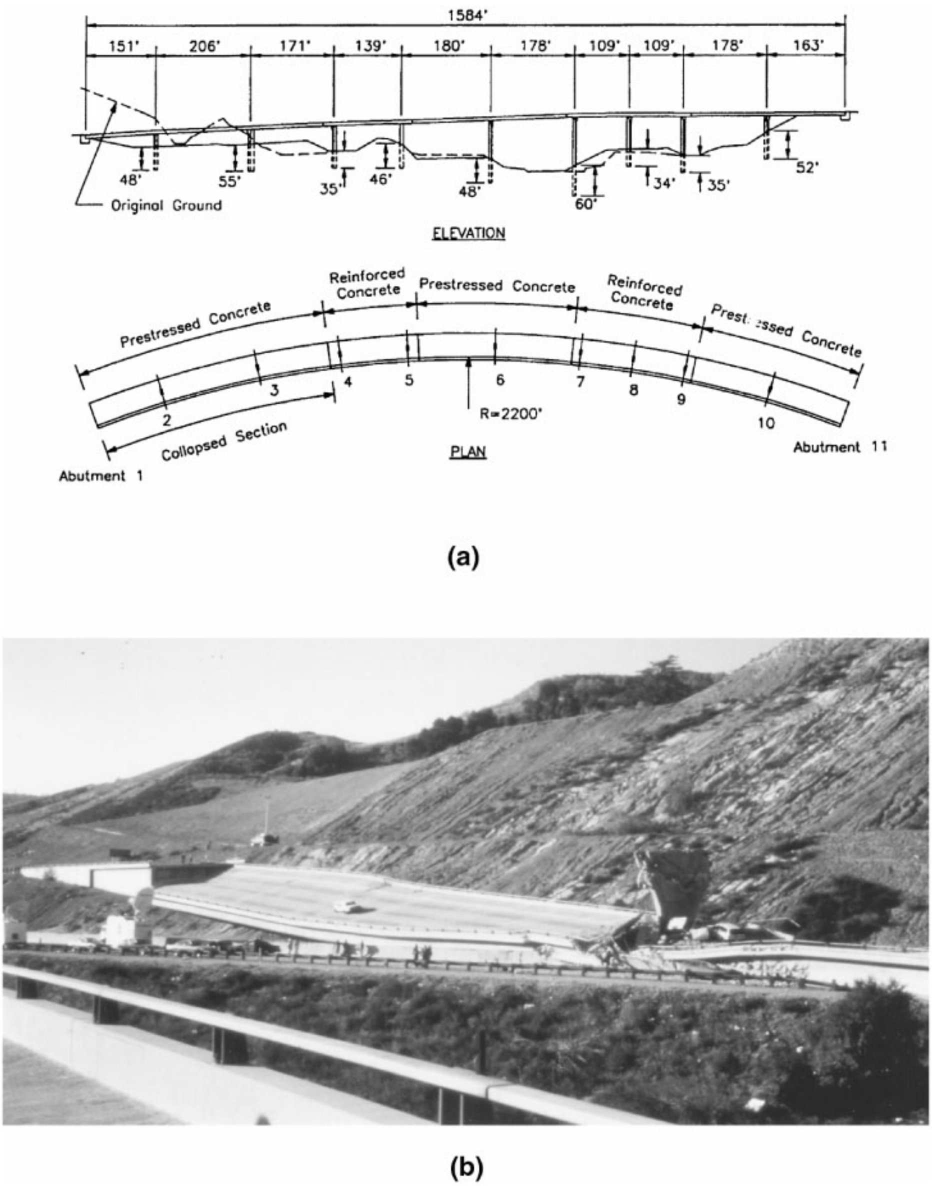
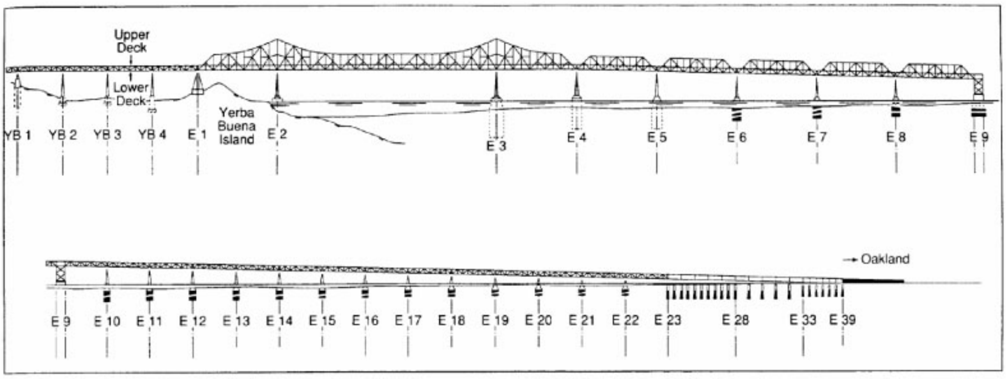


FIGURE 34.6 Geometry and collapse of the Route 14/5 Separation and Overhead in the 1994 Northridge earthquake. (a) Configuration [8]; (b) photograph of collapse.

Earthquake ground shaking, or transient or permanent ground deformations resulting from the earthquake, can induce superstructure movements that cause the supported span to unseat. Unseating is especially a problem with the shorter seats that were common in older construction (e.g., References [2,6–8,12]).

Bridges with Short Seats and Simple Spans

In much of the United States and in many other areas of the world, bridges often comprise a series of simple spans supported on bents. These spans are prone to being toppled from their supporting substructures either due to shaking or differential support movement associated with ground



(a)

(b)

FIGURE 34.7 San Francisco–Oakland Bay Bridge, east crossing; geometry and collapse in the 1989 Loma Prieta earthquake. (a) Configuration [10]; (b) photograph of collapse.

deformation. Unseating of simple spans was observed in California in earlier earthquakes, leading in recent decades to development of bridge construction practices based on monolithic box-girder-substructure construction. Problems of unseating still occur with older bridge construction and with new bridges in regions where simple spans are still common. For example, during the 1991 Costa Rica earthquake, widespread liquefaction led to abutment and internal bent rotations, resulting in the collapse of no fewer than four bridges with simple supports [7]. The collapse of the Showa Bridge in the 1964 Niigata earthquake demonstrates one result of the unseating of simple spans (Figure 34.8).

FIGURE 34.8 Showa Bridge collapse in 1964 Niigata earthquake.

Skewed Bridges

Skewed bridges are defined as those having supports that are not perpendicular to the alignment of the bridge. Collisions between a skewed bridge and its abutments (or adjacent frames) can cause a bridge to rotate about a vertical axis. Because the abutments resist compression but not tension, the sense of this rotation is the same (for a given bridge configuration) regardless of whether the

FIGURE 34.9 Rio Bananito Bridge collapse in the 1991 Costa Rica earthquake. (*Source*: EERI, *Earthquake Spectra,* Special Suppl. to Vol. 7, 1991.)

bridge collides with one abutment or the other. If the rotations are large and the seat lengths small, a bridge can come unseated at the acute corners of the decks.

Several examples of skewed bridge damage and collapse can be found in the literature [7,8,12]. A typical example is the Rio Bananito Bridge, in which the bridge and central slab pier were skewed at 30°, which lost both spans off the central pier in the direction of the skew during the 1991 Costa Rica earthquake (Figure 34.9) [7]. Another example of skewed bridge failure is the Gavin Canyon Undercrossing, which failed during the 1994 Northridge earthquake [8]. Both skewed hinges became unseated during the earthquake, resulting in collapse of the unseated spans (Figure 34.10).

Curved Bridges

Curved bridges can have asymmetrical response similar to that of skewed bridges. For loading in one direction, an in-span hinge tends to close, while for loading in the other direction, the hinge opens. An example in which the curved alignment may have contributed to bridge collapse is the curved ramps of the I-5/SR14 interchange, which sustained collapses in both the 1971 San Fernando earthquake [13] and the 1994 Northridge earthquake (see Figure 34.6) [8]. Other factors that may have contributed to the failures include inadequate hinge seats and column deformability.

Hinge Restrainers

Hinge restrainers appear to have been effective in preventing unseating in both the Loma Prieta [10] and Northridge earthquakes [8]. In some other cases, hinge restrainers were not fully effective in preventing unseating. For example, the hinge restrainers in the Gavin Canyon Undercrossing, which were aligned parallel the bridge alignment, did not prevent unseating (see Figure 34.10).

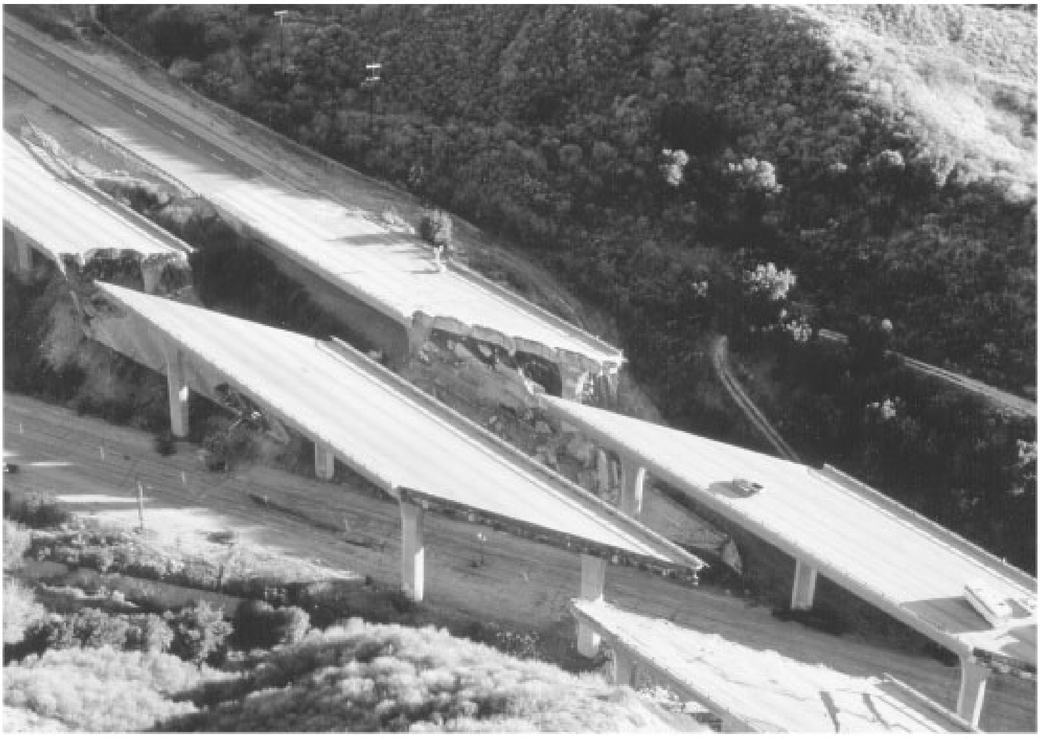


FIGURE 34.10 Gavin Canyon Undercrossing collapse in the 1994 Northridge earthquake.

34.7 Damage to Superstructures

Superstructures are designed to support service gravity loads elastically, and, for seismic applications, they are usually designed to be a strong link in the earthquake-resisting system. As a result, superstructures tend to be sufficiently strong to remain essentially elastic during earthquakes. In general, superstructure damage is unlikely to be the primary cause of collapse of a span.

Instead, damage typically is focused in bearings and substructures. The superstructure may rest on elastomeric pads, pin supports, or rocker bearings, or may be monolithic with the substructure. As bearings and substructures are damaged and in some cases collapse, a wide range of damage and failure of superstructures may result, but these failures are often secondary; that is, they result from failures elsewhere in the bridge. There are, however, some cases of primary superstructure damage as well. Some examples are highlighted below.

With the exception of bridge superstructures that come unseated and collapse, the most common form of damage to superstructures is due to pounding of adjacent segments at the expansion hinges. This type of damage occurs in bridges of all construction materials. Figure 34.11a shows pounding damage at an in-span expansion joint of the Santa Clara River Bridge during the 1994 Northridge earthquake, and Figure 34.11b shows pounding damage at an abutment of the same structure.

Following the 1971 San Fernando earthquake, Caltrans initiated the first phase of its retrofit program, which involved installation of hinge and joint restrainers to prevent deck joints from separating. Both cable restrainers and pipe restrainers (the former intended only to restrain longitudinal movement and the latter intended also to restrain transverse motions) were installed in bridge superstructures. The restrainers extended through end diaphragms that had not been designed originally for the forces associated with restraint. Some punching shear damage to end diaphragms retrofitted with cable restrainers was observed in the I-580/I-980/SR24 connectors following the 1989 Loma Prieta earthquake [15].

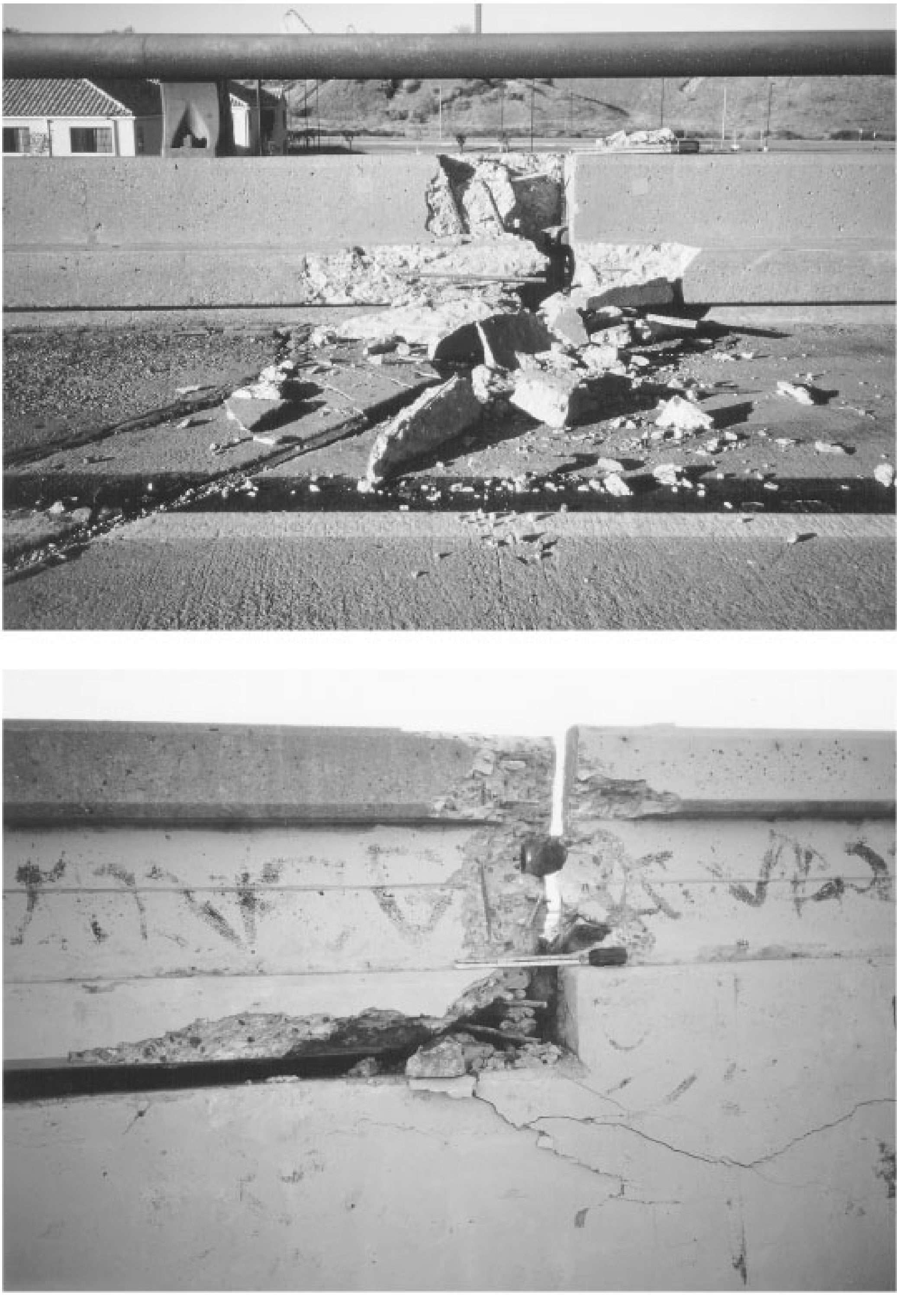


FIGURE 34.11 Santa Clara River Bridge pounding damage in 1994 Northridge earthquake. (a) Barrier rail pounding damage; (b) abutment pounding damage.

FIGURE 34.12 Buckling of braces near pier 209 of the Hanshin Expressway in the 1995 Hyogo-Ken Nanbu earthquake.

Steel superstructures commonly comprise lighter framing elements, especially for transverse bracing. These have been found to be susceptible to damage due to transverse loading, especially following failure of bearings [1,8]. Several cases of steel superstructure damage occurred in the Hyogo-Ken Nanbu earthquake. Figure 34.12 shows buckling of cross braces beneath the roadway of a typical steel girder bridge span of the Hanshin Expressway. Figure 34.13 shows girder damage in the same expressway due to excessive lateral movement at the support. Figure 34.14 shows buckled cross-members between the upper chords of the Rokko Island Bridge. That single-span, 710-ft (217-m) tied-arch span bridge slipped from its expansion bearings, allowing the bridge to move laterally about 10 ft (3 m). The movement was sufficient for one end of one arch to drop off the cap beam, twisting the superstructure and apparently resulting in the buckling of the top chord bracing members [2,3].

A spectacular example of steel superstructure failure and collapse is that of the eastern portion of the San Francisco–Oakland Bay Bridge during the 1989 Loma Prieta earthquake (see Figure 34.7) [10]. In this bridge, a 50-ft (15-m) span over tower E9 was a transition point between 506-ft (154-m) truss spans to the west and 290-ft (88-m) truss spans to the east, serving to transmit longitudinal forces among the adjacent spans and the massive steel tower at E9. Failure of a bolted connection between the 290-ft (88-m) span truss and the tower resulted in sliding of the span and unseating of the transition span over tower E9. This collapse resulted in closure for 1 month of this critical link between San Francisco and the East Bay.

34.8 Damage to Bearings

In some regions of the world, the prevalent bridge construction consists of steel superstructures supported on bearings, which, in turn, rest on a substructure. In the United States, this form of

FIGURE 34.13 Girder damage at Bent 351 of the Hanshin Expressway apparently due to transverse movement during the 1995 Hyogo-Ken Nanbu earthquake.

construction is common in new bridges east of the Sierra Nevada Mountains as well as throughout the country in older existing bridges. In such bridges, the bearings commonly consist of steel components designed to provide restraint in one or more directions and, in some cases, to permit movement in one or more directions. Failure of these bearings in an earthquake can cause redistribution of internal forces, which may overload either the superstructure or substructure, or both. Collapse is also possible when bearing support is lost.

The predominant type of bridge construction in Japan involves steel superstructures supported on bearings, which, in turn, are supported on concrete substructures. The Hyogo-Ken Nanbu earthquake provides several examples of bearing failures in these types of bridges [2,3]. One example is provided by the Hamate Bypass, which was a double-deck elevated roadway comprising steel box girders on either fixed or expansion steel bearings. Bearing failure at several locations led to large superstructure rotations that can be seen in Figure 34.15. Another example is provided by the Nishinomiya-ko Bridge, a 830-ft (252-m) span-arch bridge supported on two fixed bearings at one end and two expansion bearings at the other end. The fixed-end bearings, which apparently were designed to have a capacity of approximately 70% of the bridge weight [2], failed, apparently leading to unseating of the adjacent approach span (see Figure 34.3). The failed bearing is shown in Figure 34.16.

FIGURE 34.14 Buckling of cross-members in the upper chord of the Rokko Island Bridge in the 1995 Hyogo-Ken Nanbu earthquake.

FIGURE 34.15 Hamate Bypass superstructure rotations as a result of bearing failures in the 1995 Hyogo-Ken Nanbu earthquake.

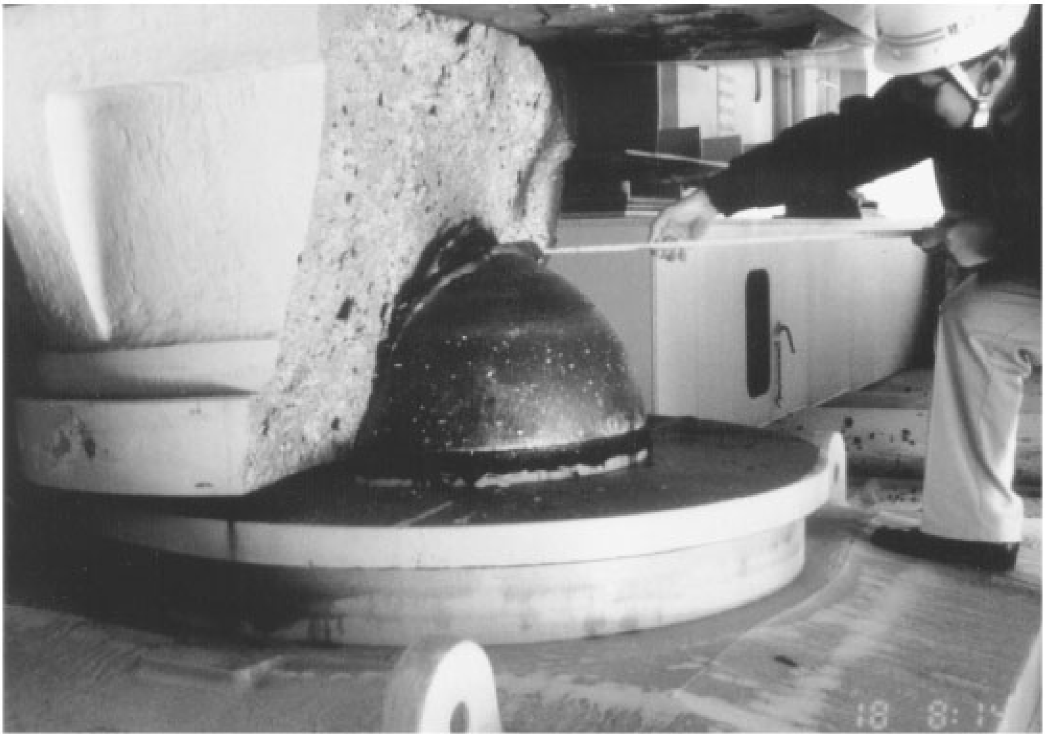


FIGURE 34.16 Nishinomiya-ko Bridge bearing failure in the 1995 Hyogo-Ken Nanbu earthquake.

34.9 Damage to Substructures

Columns

Unlike building design, current practice in bridge design is to proportion members of a frame (bent) such that its lateral-load capacity is limited by the flexural strength of its columns. For this strategy to be successful, the connecting elements (e.g., footings, joints, cross-beams) need to be strong enough to force yielding into the columns, and the columns need to be sufficiently ductile (or tough) to sustain the imposed deformations. Even in older bridges, where the "weak column" design approach may not have been adopted explicitly, columns tend to be weaker than the beam–diaphragm–slab assembly to which they connect. Consequently, columns can be subjected to large inelastic demands during strong earthquakes. Failure of a column can result in loss of vertical load-carrying capacity; column failure is often the primary cause of bridge collapse.

Most damage to columns can be attributed to inadequate detailing, which limits the ability of the column to deform inelastically. In concrete columns, the detailing inadequacies can produce flexural, shear, splice, or anchorage failures, or as is often the case, a failure that combines several mechanisms. In steel columns, local buckling has been observed to lead progressively to collapse.

Ideally, a concrete column should be designed such that the lateral load strength is controlled by flexure. However, even if most of the inelastic action is flexural, a column may not be sufficiently tough to sustain the imposed flexural deformations without failure. Such failures are particularly common in older bridges. In the United States, the transverse reinforcement of reinforced concrete columns designed before 1971 commonly consists of #4 hoops ($\phi = 13$ mm) or ties at 12-in. (305-mm) spacing. Moreover, the ends of the transverse reinforcement rarely are anchored into the

FIGURE 34.17 San Fernando Road Overhead damage in the 1971 San Fernando earthquake.

core of these columns. This amount and type of reinforcement provides negligible confinement to the concrete, particularly in large columns. Figure 34.17 shows bridge columns that had insufficient flexural ductility to withstand the 1971 San Fernando earthquake. Figure 34.18 shows similar damage in a circular cross section column in the 1995 Hyogo-Ken Nanbu earthquake.

Other detailing practices (in addition to providing little confinement) may lead to flexural failure in reinforced concrete columns. A common practice in Japan has been to terminate some of the longitudinal reinforcement within the column height. The resulting development length of the terminated reinforcement can be inadequate, and may lead to splitting failure along the terminated bars or to flexural and shear distress near the cutoff point. Figure 34.19 illustrates failure of a column with bars terminated near the column midheight. In the case of the Hanshin Expressway Route 3, which collapsed during the 1995 Hyogo-Ken Nanbu earthquake (see Figure 34.4), the curtailment of one third of the main reinforcement was accompanied by the use of gas-pressure butt welding of the continuing longitudinal reinforcement. In tests following the earthquake, approximately half of the undamaged, butt-welded bars failed at the welds [14].

Shear failures of concrete bridge columns have occurred in many earthquakes (e.g., [5,8]). Such failures can occur at relatively low structural displacements, at which point the longitudinal reinforcement may not yet have yielded. Alternatively, because shear strength degrades with inelastic loading cycles, shear failures can occur after flexural yielding. Examples of shear failure can be found in several of the references provided at the end of this chapter. Figure 34.20 illustrates shear failure of a column having relatively light transverse reinforcement typical of bridges constructed in the western United States prior to the mid-1970s. The failure features a steeply inclined diagonal crack and dilation of the core into discrete blocks of concrete. Under the action of several deformation

FIGURE 34.18 Hanshin Expressway, Pier 46, damage in the 1995 Hyogo-Ken Nanbu earthquake.

cycles combined with vertical loads, a column can degrade to nearly complete loss of load-carrying capacity, as suggested by the heavily damaged column in Figure 34.21. Provision of closely spaced transverse reinforcement as required in some modern codes is required to prevent this type of failure.

Shear failures in reinforced concrete columns can be induced by interactions with "nonstructural" elements. These elements can decrease the distance between locations of flexural yielding, and therefore increase the shear demand for a column. Figure 34.5, discussed previously, shows a case

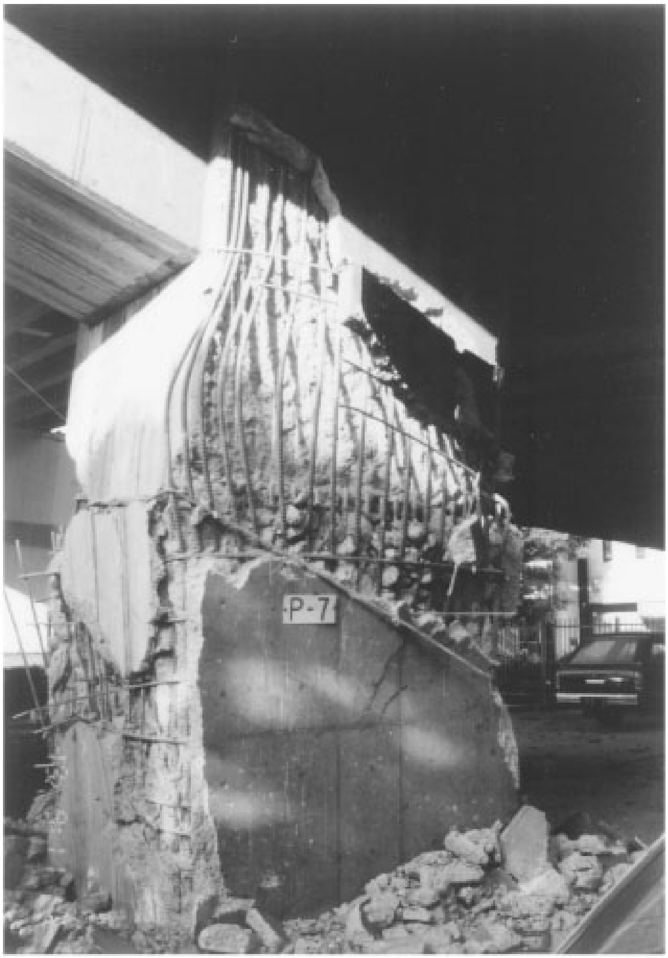


FIGURE 34.19 Failure of column with longitudinal reinforcement cutoffs near midheight in the 1995 Hyogo-Ken Nanbu earthquake.

in which a channel wall restrained the column at the base and forced the location of yielding to occur higher in the column than was anticipated in design [8]. Figure 34.22 shows a case in which an architectural flare strengthened the upper portion of the column, forcing yielding to occur lower than was intended [8]. In both cases, an element that was not considered in designing the column forced failure to occur in a lightly confined portion of the column that was incapable of resisting the force and deformation demands.

Figure 34.23 illustrates the failure of a stout, two-column bent on a spur just to the north of the Hanshin Expressway in the 1995 Hyogo-Ken Nanbu earthquake. The failure involves shattering of the columns, bent cap, and joints, and shatters the notion that strength alone is an adequate provision for bridge seismic design.

Lap splices of longitudinal reinforcement in older reinforced-concrete bridges may be vulnerable because, typically, the splices are short (on the order of 20 to 30 bar diameters), poorly confined, and are located in regions of high flexural demand. In particular, for construction convenience, splices are often located directly above a footing. With these details, the splices may be unable to develop the flexural capacity of the column, and they may be more vulnerable to shear failure. Despite these vulnerabilities, there is little field evidence of lap splice failures at the bases of bridge columns. However, failures associated with welded splices and terminated longitudinal reinforcement were identified in the 1995 Hyogo-Ken Nanbu earthquake (Figures 34.4 and 34.19), as discussed previously.

FIGURE 34.20 Failure of columns of the Route 5/210 interchange during the 1971 San Fernando earthquake.

Concrete columns also can fail if the anchorage of the longitudinal reinforcement is inadequate. Such failures can occur both at the top of a column at the connection with the bent cap and at the bottom of a column at the connection with the foundation. Figure 34.24 shows a column that failed at its base during the 1971 San Fernando earthquake (12). The column had been supported on a single 6-ft (1.8-m)-diameter, cast-in-drilled-hole pile. Other columns having hooked longitudinal reinforcement anchored in footings also failed with similar results in that earthquake. The consequences of foundation anchorage failures perhaps are larger in single-column bents than multicolumn bents, because the lateral-force resistance of single-column bents depends on the column developing its flexural strength at the base.

The record of steel column failures is sparse, because few bridges with steel columns have been subjected to strong earthquakes elsewhere than Japan. In the 1995 Hyogo-Ken Nanbu earthquake, failures apparently were associated with local buckling and subsequent splitting at welds or tearing of steel near the buckle. In columns with circular cross sections, the local buckling sometimes occurred at locations where section thicknesses changed. Figure 34.25 illustrates the formation of a local buckle in a circular cross section column accompanied by visible plastic deformation. In rectangular columns, local buckling of web and flange plates was insufficiently restrained by small web stiffeners [14]. Figure 34.26 illustrates the collapse of a rectangular column. A nearby column sustained local buckling at the base and tearing of the vertical welded seam between the two steel plates forming a corner of the column, suggesting the nature of failure that resulted in the collapse shown in Figure 34.26.

FIGURE 34.21 Failure of columns of Interstate 10, La Cienega-Venice Undercrossing in the 1994 Northridge earthquake. (Masonry walls of storage units are supporting the collapsed frame.)

Beams

Beams traditionally have received much less attention than columns in seismic design and evaluation. In many bridges, the transverse beams are stronger than the columns because of gravity load requirements and composite action with the superstructure. Also, in many bridges, the consequences of beam failures are less severe than the consequences of column failures. In bridges with outriggers, however, the beams can be critical components of the bent and can be subjected to loadings that may result in failure. An example illustrating possible damage to an outrigger beam is in Figure 34.27. This outrigger beam was monolithically framed with the superstructure and supporting column such that, under longitudinal load, significant torsion was required to be resisted by the outrigger portion of the beam. In some modern designs, torsion is reduced by providing nominal "pinned" connections between the beams and columns.

Joints

As with beams, joints traditionally have received little attention in seismic design, and they similarly may be exposed to critically damaging actions when the joints lie outside of the superstructure. Although joint failures occurred in previous earthquakes (e.g., Jennings [13]), significant attention was not paid to joints until several spectacular failures were observed following the 1989 Loma Prieta earthquake [6,10]. Figure 34.28 shows joint damage to the Embarcadero Viaduct in San Francisco during the 1989 Loma Prieta earthquake. The occurrence of damage at the relatively large epicentral distance of approximately 60 miles (100 km) is attributed in part to site amplification and focusing of seismic waves as well as the vulnerability of the framing.

The collapse of the Cypress Street Viaduct during the Loma Prieta earthquake had more severe consequences (Figure 34.29). Failure of a concrete pedestal located just above the first-level joint

FIGURE 34.22 Failure of flared column in the Route 118, Mission-Gothic Undercrossing, in the 1994 Northridge earthquake.

led to the collapse of the upper deck on the lower deck, at a cost of 42 lives. Such pedestals are not common, but this collapse demonstrates that each earthquake has the potential to reveal a mode of failure that has not yet been considered routinely.

The Loma Prieta earthquake also identified an apparent weakness of a modern design. For example, damage occurred to the outrigger knee joints of the Route 980/880 connector, which had been constructed just a few years before the earthquake. This damage identified the need for special details in bridge construction, which has been the subject of important studies identified elsewhere in this book.

Abutments

The types of failures that can occur at abutments vary from one bridge to the next. The foundation type varies greatly (e.g., spread footing, pile-supported footing, drilled shafts), and the properties of the soil can be important, particularly if the soil liquefies during an earthquake. The situation is further complicated by the interaction of the backwalls, wingwalls, footings and piles with the surrounding soil. A common practice has been to treat abutments or abutment components as sacrificial elements, acting as fuses to relieve large seismic forces arriving at the stiff abutment. The occurrence of widespread and extensive damage in the 1994 Northridge earthquake [8] suggests that an alternative approach might be economical.

In most seat-type abutments, longitudinal motion is unrestrained, because there is a joint at the interface of the superstructure and abutment backwall. This configuration is attractive, because it

FIGURE 34.23 Failure of a two-column bent in the 1995 Hyogo-Ken Nanbu earthquake.

reduces the superstructure forces induced by temperature and shrinkage-induced displacements. The most important vulnerability of such abutments is span unseating, which can occur when there are large relative displacements between the superstructure and abutment seat. Abutment unseating failures are often attributable to displacement or rotation of the abutment, usually the result of liquefaction or lateral spreading [7].

Shear keys can be damaged also. Shear keys are components that restrain relative displacements (usually in the transverse direction) between the superstructure and the abutments. External shear keys are located outside of the superstructure cross section, while internal shear keys are located within the superstructure cross section. Since these elements are stocky, it is nearly impossible to make them ductile, and they will fail if their strength is exceeded.

Shear key failures were widespread during the 1994 Northridge earthquake [8]. Figure 34.30 shows a typical failure in which the external shear keys failed. Figure 34.31 shows a failed internal shear key. It appears that these failures can occur with small transverse displacement and little energy dissipation. Damage to internal shear keys usually is accompanied by damage to the interlocking backwall. In seat-type abutments, damage has also occurred in seat abutments due to pounding of backwalls by the superstructure. This type of damage is similar to that shown in Figure 34.11.

In monolithic abutments, the superstructure is cast monolithically with the abutments. This configuration is attractive, because it reduces the likelihood of span unseating. However, the abutment can be damaged as the superstructure displaces in the longitudinal direction away from the abutment. Also, depending on the geometry and details of the abutment, the wingwall may serve as an external shear key. In such cases, the wingwall can fail in the same manner as an external shear key.

FIGURE 34.24 Failure at the base of a column supported on a single cast-in-place pile in the 1971 San Fernando earthquake [Steinbrugge Collection, EERC Library, University of California, Berkeley].

Foundations

Reports of foundation failures during earthquakes are relatively rare, with the notable exception of situations in which liquefaction occurred. It is not clear whether failures are indeed that rare or whether many foundation failures are undetected because they remain underground. There are many reasons why older foundations might be vulnerable. Piles might have little confinement reinforcement, yet be subjected to large deformation demands. Older spread and pile-supported footings rarely have top flexural reinforcement or any shear reinforcement.

The 1995 Hyogo-Ken Nanbu earthquake resulted in extensive damage to superstructures and substructures above the ground, as reported elsewhere in this chapter. The occurrence of that damage provided impetus to conduct extensive investigations of the conditions of foundation components [11]. Along the older inland Route 3, an investigation of 109 foundations identified only cases of "small" flexural cracks in piles. Along the newer coastal Route 5, more extensive liquefaction occurred, resulting in lateral spreading in several cases. An investigation of 153 foundations for this route found cases of flexural cracks in piles where large residual displacements occurred, but the investigators found no spalling or reinforcement buckling. The absence of extensive damage was attributed to the spread of deformations along a significant length of the piles.

Foundation damage associated with liquefaction-induced lateral spreading has probably been the single greatest cause of extreme distress and collapse of bridges [12]. The problem is especially critical for bridges with simple spans (see Figure 34.8). The 1991 Costa Rica earthquake provides many examples of foundation damage [7]. For example, Figure 34.32 shows an abutment that

FIGURE 34.25 Local buckling of a circular cross-section column of the Hanshin Expressway in the 1995 Hyogo-Ken Nanbu earthquake.

rotated due to liquefaction and lateral spreading. Figure 34.33 shows a situation in which soil movements have led to extensive damage to the batter piles. Use of batter piles should be considered carefully in design in light of the extensive damage observed in these piles in this and other earthquakes [6].

Approaches

Even if the bridge structure remains intact, a bridge may be placed out of service if the roadway leading to it settles significantly. For example, during the 1971 San Fernando earthquake [12] and the 1985 Chile earthquake [4], settlement of the backfill abutments led to abrupt differential settlements in many locations. Such settlements can be large enough to pose a hazard to the traveling public. Approach or settlement slabs can be effective means of spanning across backfills, as shown in Figure 34.34.

34.10 Summary

This chapter has reviewed various types of damage that can occur in bridges during earthquakes. Damage to a bridge can have severe consequences for a local economy, because bridges provide vital links in the transportation system of a region. In general, the likelihood of damage increases

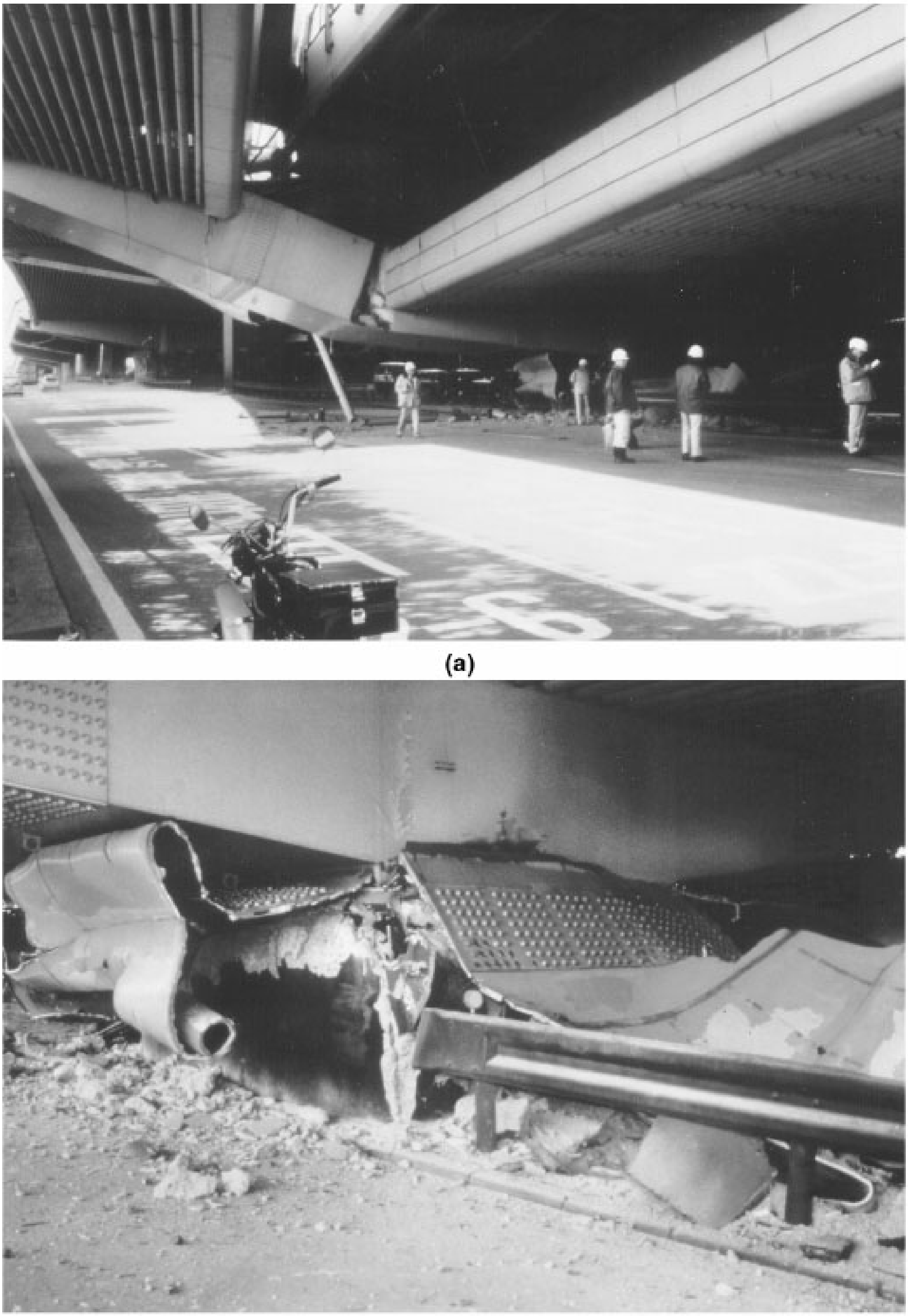


FIGURE 34.26 Collapse of a rectangular cross section steel column in the 1995 Hyogo-Ken Nanbu earthquake. (a) Collapsed bent and superstructure; (b) close-up of collapsed column.

FIGURE 34.27 Outrigger damage in the 1989 Loma Prieta earthquake.

if the ground motion is particularly intense, the soils are soft, the bridge was constructed before modern codes were implemented, or the bridge configuration is irregular. Even a well-designed bridge can suffer damage if nonstructural modifications and structural deterioration have increased the vulnerability of the bridge.

Depending on the ground motion, site conditions, overall configuration, and specific details of the bridge, the damage induced in a particular bridge can take many forms. Despite these complexities, the record is clear. Damage within the superstructure is rarely the primary cause of collapse. Though exceptions abound, most of the severe damage to bridges has taken one of the following forms:

- Unseating of superstructure at in-span hinges or simple supports attributable to inadequate seat lengths or restraint. The presence of a skewed or curved configuration further exacerbates the vulnerability. For simply supported bridges, these failures are most likely when ground failure induces relative motion between the spans and their supports.
- Column failure attributable to inadequate ductility (toughness). In reinforced-concrete columns, the inadequate ductility usually stems from inadequate confinement reinforcement. In steel columns, the inadequate ductility usually stems from local buckling, which progresses to collapse.
- Damage to shear keys at abutments. Because of their geometry, it is nearly impossible to make these stiff elements ductile.
- Unique failures in complex structures. In the Cypress Street Viaduct, the unique vulnerability was the inadequately reinforced pedestal above the first level. In outrigger column bents, the vulnerability may be in the cross-beam or the beam–column joint.

FIGURE 34.28 Embarcadero Viaduct damage during the 1989 Loma Prieta earthquake.

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FIGURE 34.29 Cypress Street Viaduct collapse in the 1989 Loma Prieta earthquake.

FIGURE 34.30 Damage to external shear key in an abutment in the 1994 Northridge earthquake. (*Source*: EERI, *Earthquake Spectra*, Special Suppl. to Vol. II, 1995.)

FIGURE 34.31 Damage to internal shear key in an abutment in the 1994 Northridge earthquake. (*Source:* EERI, *Earthquake Spectra*, Special Suppl. to Vol. II, 1995.)

FIGURE 34.32 Rotation of abutment due to liquefaction and lateral spreading during the 1991 Costa Rica earthquake. (*Source:* EERI, *Earthquake Spectra*, Special Suppl. to Vol. 7, 1991.)

FIGURE 34.33 Abutment piles damaged during the 1991 Costa Rica earthquake. (*Source:* EERI, *Earthquake Spectra,* Special Suppl. to Vol. 7, 1991.)

FIGURE 34.34 Settlement slab spanning across slumped abutment fill material at the Rio Quebrada Calderon bridge in the 1991 Costa Rica earthquake. (*Source:* EERI, *Earthquake Spectra,* Special Suppl. to Vol. 7, 1991.)

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