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Earthquake Damage to Structures

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30.1 Introduction

Earthquakes

Most earthquakes occur due to the movement of faults. Faults slowly build up stresses that are suddenly released during an earthquake. We measure the size of earthquakes using moment magnitude as defined in Equation 30.1.

$$M = (2/3)[\log(M_o) - 16.05] \tag{30.1}$$

where M_o is the seismic moment, as defined in Equation 30.2:

$$M_o = GAD \text{ (in dyne-cm)} \tag{30.2}$$

where G is the shear modulus of the rock (dyne/cm^2), A is the area of the fault (cm^2), and D is the amount of slip or movement of the fault (cm).

The largest magnitude earthquake that can occur on a particular fault is the product of the fault length times its depth (A), the average slip rate times the recurrence interval of the earthquake (D), and the hardness of the rock (G). For instance, the northern half of the Hayward Fault (in the San Francisco Bay Area) has an annual slip rate of 9 mm/yr (Figure 30.1). It has an earthquake recurrence interval of 200 years. It is 50 km long and 14 km deep. G is taken as 3×10^{11} dyne/cm^2:

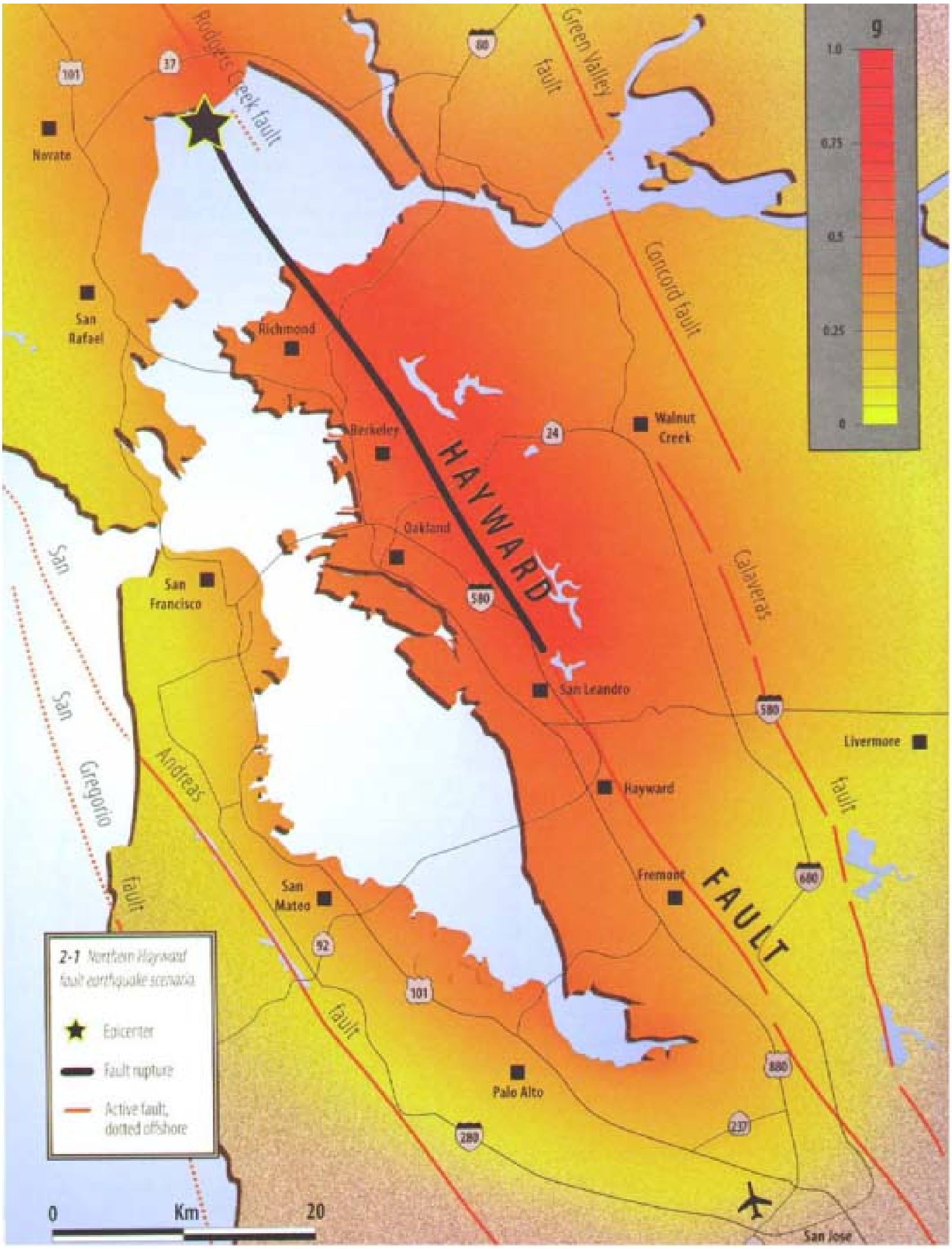

FIGURE 30.1 Map of Hayward Fault. (Courtesy of EERI [1].)

$$M_o = (.9 \times 200)\ (5 \times 10^6)\ (1.4 \times 10^6)\ (3 \times 10^{11}) = 3.78 \times 10^{26}$$

$$M = (2/3)[\log 3.78 \times 10^{26} - 16.05] = 7.01$$

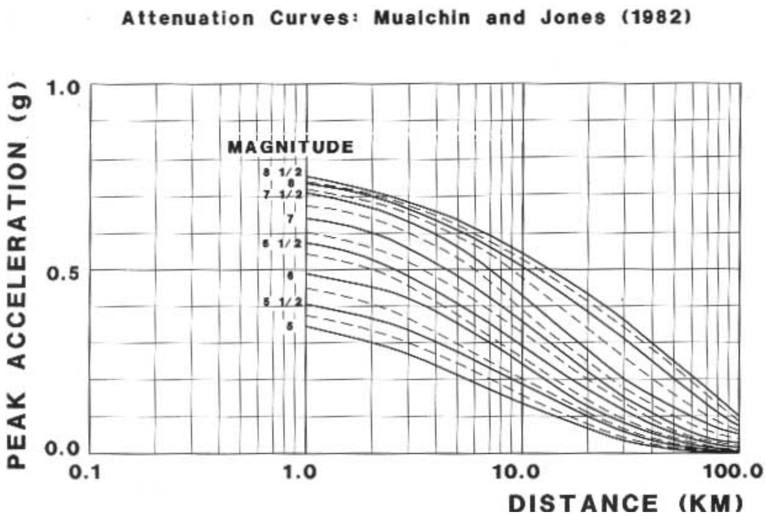

FIGURE 30.2 Attenuation curve developed by Mualchin and Jones [7].

Therefore, an earthquake of a magnitude about 7.0 is the maximum event that can occur on the northern section of the Hayward Fault. Because *G* is a constant, the average slip is usually a few meters, and the depth of the crust is fairly constant, the size of the earthquake is usually controlled by the length of the fault.

Magnitude is not particularly revealing to the structural engineer. Engineers design structures for the peak accelerations and displacements at the site. After every earthquake, seismologists assemble the recordings of acceleration vs. distance to create attenuation curves that relate the peak ground acceleration (PGA) to the magnitude of earthquakes based on distance from the fault rupture (Figure 30.2). All of the data available on active faults are assembled to create a seismic hazard map. The map has contour lines that provide the peak acceleration based on attenuation curves that indicate the reduction in acceleration due to the distance from a fault. The map is based on deterministic-derived earthquakes or on earthquakes with the same return period.

Structural Damage

Every day, regions of high seismicity experience many small earthquakes; however, structural damage does not usually occur until the magnitude approaches 5.0. Most structural damage during earthquakes is caused by the failure of the surrounding soil or from strong shaking. Damage also results from surface ruptures, from the failure of nearby lifelines, or from the collapse of more vulnerable structures. We consider these effects as secondary, because they are not always present during an earthquake; however, when there is a long surface rupture (such as that which accompanied the 1999 Ji Ji, Taiwan earthquake), secondary effects can dominate.

Because damage can mean anything from minor cracks to total collapse, categories of damage have been developed, as shown in Table 30.1. These levels of damage give engineers a choice for the performance of their structure during earthquakes. Most engineered structures are designed only to prevent

TABLE 30.1 Categories of Structural Damage

Damage State	Functionality	Repairs Required	Expected Outage
(1) None (pre-yield)	No loss	None	None
(2) Minor/slight	Slight loss	Inspect, adjust, patch	<3 days
(3) Moderate	Some loss	Repair components	<3 weeks
(4) Major/extensive	Considerable loss	Rebuild components	<3 months
(5) Complete/collapse	Total loss	Rebuild structure	>3 months

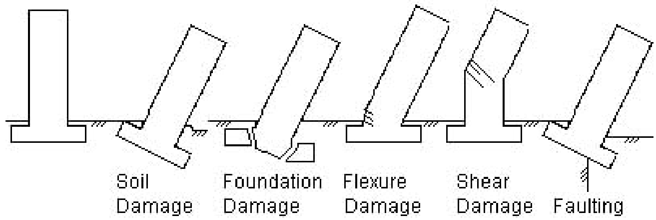

FIGURE 30.3 Common types of damage during large earthquakes.

collapse. This is done to save money, but also because as a structure becomes stronger it attracts larger forces, thus most structures are designed to have sufficient ductility to survive an earthquake. This means that elements will yield and deform, but they will be strong in shear and continue to support their load during and after the earthquake. As shown in Table 30.1, the time that is required to repair damaged structures is an important parameter that weighs heavily on the decision-making process. When a structure must be repaired quickly or must remain in service, a different damage state should be chosen.

During large earthquakes the ground is jerked back and forth, causing damage to the element whose capacity is furthest below the earthquake demand. Figure 30.3 illustrates that the cause may be the supporting soil, the foundation, weak flexural or shear elements, or secondary hazards such as surface faulting or failure of a nearby structure. Damage also frequently occurs due to the failure of connections or from large torsional moments, tension and compression, buckling, pounding, etc. In this chapter, structural damage as a result of soil problems, structural shaking, and secondary causes will be discussed. These types of damage illustrate the most common structural hazards that have been seen during recent earthquakes.

30.2 Damage as a Result of Problem Soils

Liquefaction

One of the most common causes of damage to structures is the result of liquefaction of the surrounding soil. When loose, saturated sands, silts, or gravel are shaken, the material consolidates, reducing the porosity and increasing pore water pressure. The ground settles, often unevenly, tilting and toppling structures that were formerly supported by the soil. During the 1955 Niigata, Japan earthquake, several four-story apartment buildings toppled over due to liquefaction (Figure 30.4). These buildings fell when the liquefied soil lost its ability to support them. As can be seen clearly in Figure 30.5, there was little damage to these buildings and it was reported that their collapse took place over several hours.

Partial liquefaction of the soil in Adapazari during the 1999 Kocaeli, Turkey earthquake caused several buildings to settle or fall over. Figure 30.6 shows a building that settled as pore water was pushed to the surface, reducing the bearing capacity of the soil. Note that the weight of the building squeezed the weakened soil under the adjacent roadway.

Another problem resulting from liquefaction is that the increased pore pressure pushes quay walls, riverbanks, and the piers of bridges toward adjacent bodies of water, often dropping the end spans in the process. The Shukugawa Bridge is a three-span, continuous, steel box girder superstructure with a concrete deck. The end spans are 87.5 m, and the center span is 135 m. The superstructure is supported by steel, multi-column bents with dropped-bent caps. It is part of a long, elevated viaduct and has expansion joints at Pier 131 and Pier 134. The columns are supported by steel piles embedded in reclaimed land along Osaka Bay.

During the 1995 Kobe, Japan earthquake, increased pore pressure pushed the quay wall near the west end of the bridge toward the river, allowing the soil and western-most pier (Pier 134) to move one meter eastward (Figure 30.7). This resulted in the girders falling off their bearings, which damaged the expansion joint devices and made the bridge inaccessible. The eastern-most pier (Pier 131) moved half a meter

FIGURE 30.4 Liquefaction-caused building failure in Niigata, Japan. (Photograph by Joseph Penzien and courtesy of Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

FIGURE 30.5 Liquefaction-caused building failure in Niigata, Japan. (Photograph by Joseph Penzien and courtesy of Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

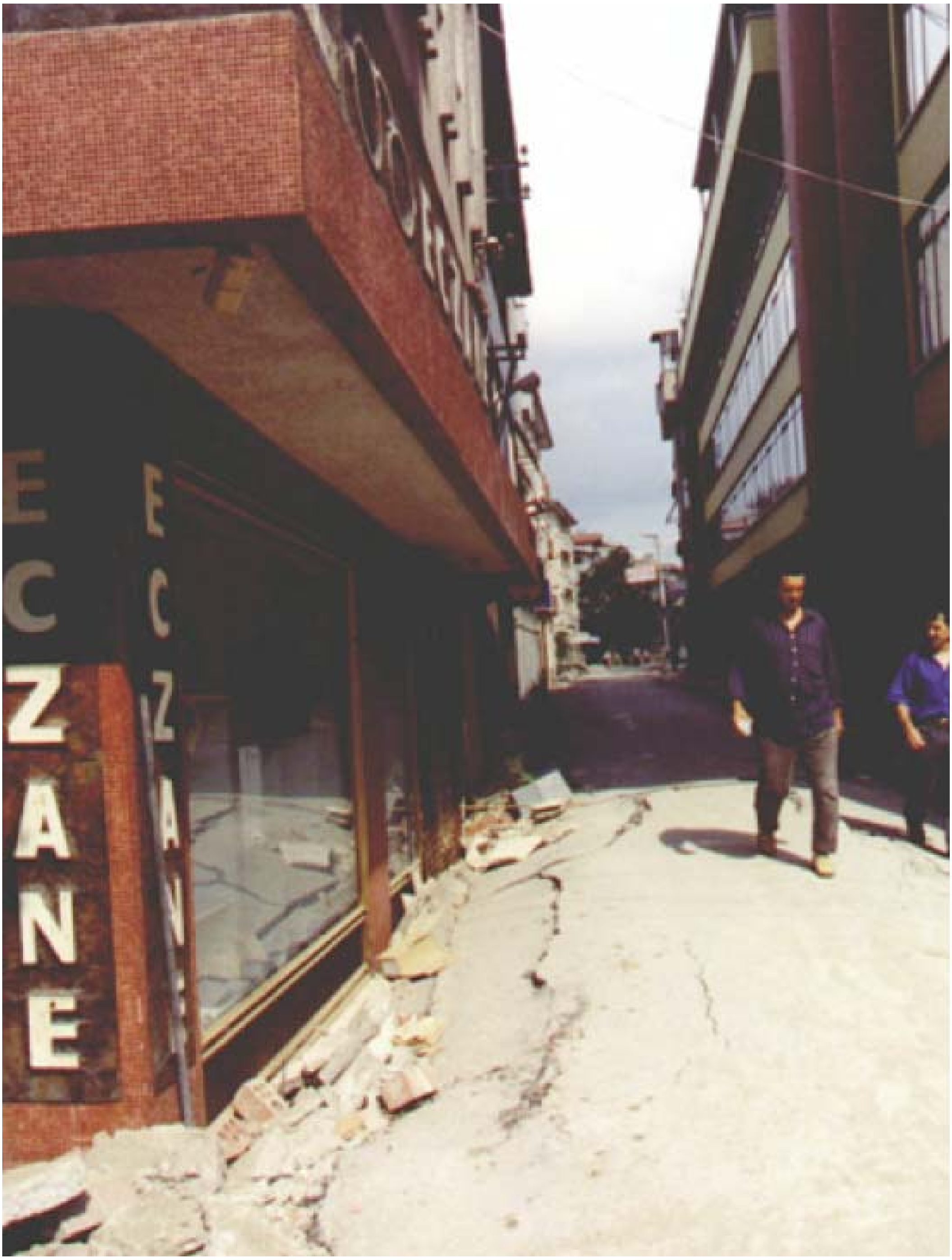

FIGURE 30.6 Settlement of building due to loss of bearing during the 1999 Kocaeli earthquake.

toward the river. It appears that the restrainers were the only thing that kept the superstructure together at the expansion joint above Pier 134, thus preventing the collapse of the west span. The expansion joint had a 0.6-m vertical offset, and excavation showed that the piles at Pier 134 were also damaged due to the longitudinal movement.

FIGURE 30.7 Liquefaction-caused bridge damage during Kobe earthquake.

Structures supported on liquefied soil topple, structures that retain liquefied soil are pushed forward, and structures buried in liquefied soil (such as culverts and tunnels) float to the surface in the newly buoyant medium. The Webster and Posey Street Tube Crossings are 4500-ft-long tubes carrying two lanes of traffic under the Oakland, CA estuary. The Posey Street Tube was built in the 1920s (Figure 30.8), while the Webster Street Tube was built in the 1960s (Figure 30.9). They are both reinforced concrete

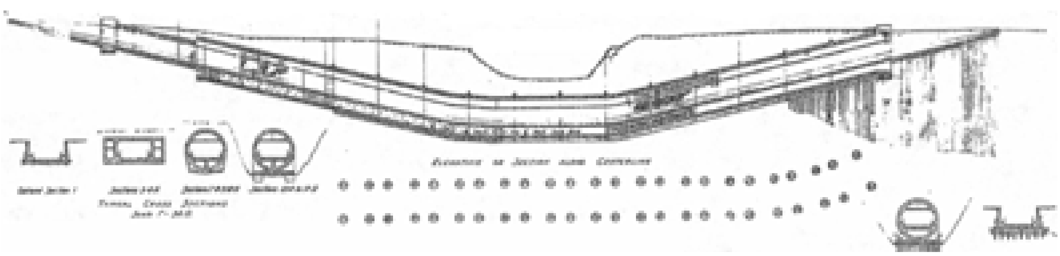

FIGURE 30.8 Elevation view of the Posey Street Tube.

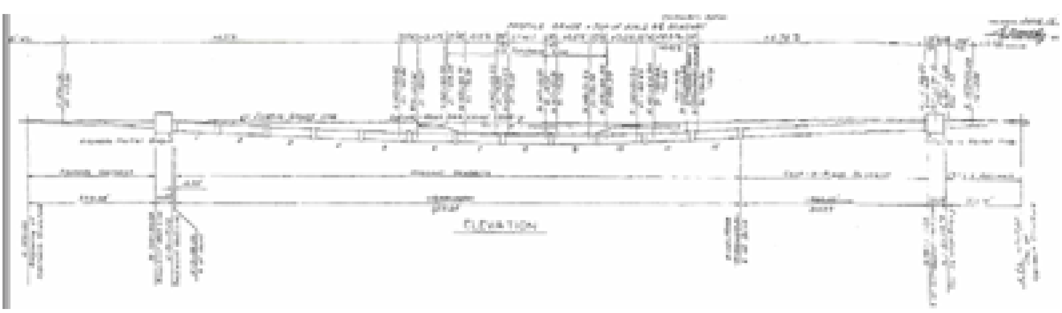

FIGURE 30.9 Elevation view of the Webster Street Tube.

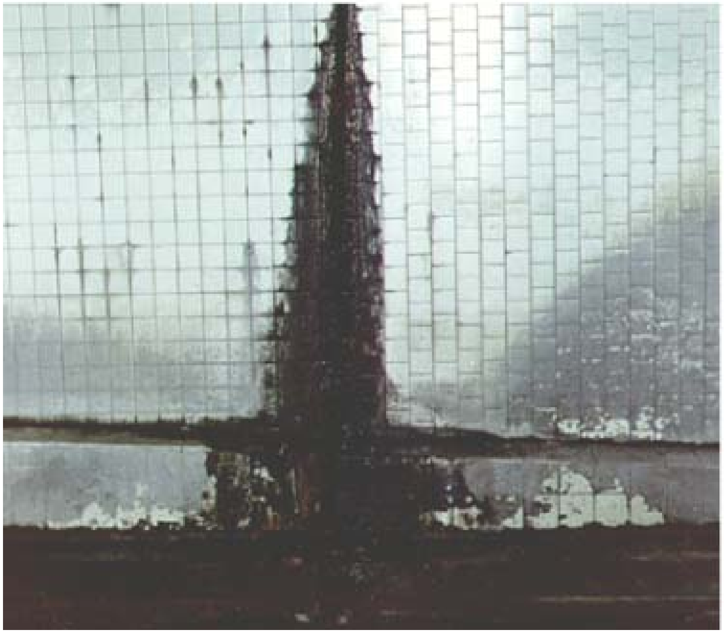

FIGURE 30.10 Liquefaction-induced damage to Webster Street Tube tunnel.

tubes with a bituminous coating for waterproofing. The ground was excavated, and each tube section was joined to the previously laid section. Both tubes descend to 70 ft below sea level. During the 1989 Loma Prieta, CA, earthquake, the soil surrounding the Webster and Posey Tubes liquefied. The tunnels began to float to the surface, the joints between sections broke, and they slowly filled with water (Figures 30.10 and 30.11).

FIGURE 30.11 Liquefaction-induced damage to Webster Street Tube tunnel.

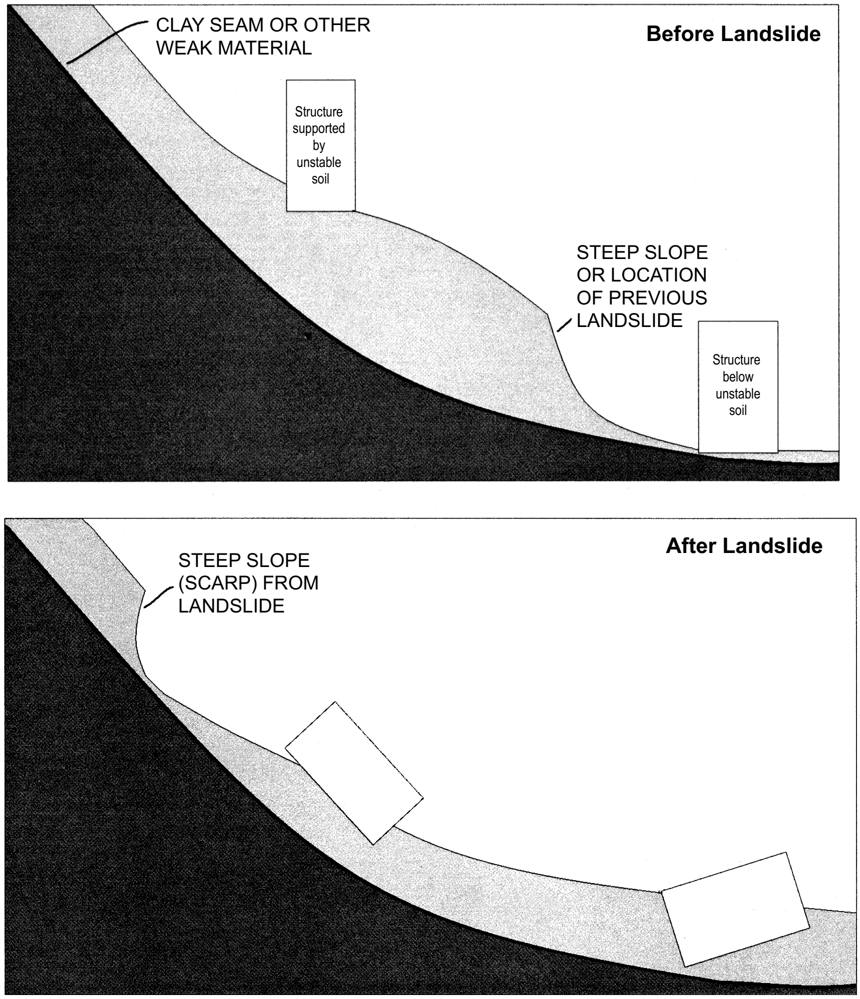

FIGURE 30.12 Diagram showing typical features of landslides.

Landslides

When a steeply inclined mass of soil is suddenly shaken, a slip-plane can form, and the material slides downhill. During a landslide, structures sitting on the slide move downward and structures below the slide are hit by falling debris (Figure 30.12). Landslides frequently occur in canyons, along cliffs and mountains, and anywhere else that unstable soil exists. Landslides can occur without earthquakes (they often occur during heavy rains, which increase the weight and reduce the friction of the soil), but the number of landslides is greatly increased wherever large earthquakes occur. Landslides can move a few inches or hundreds of feet. They can be the result of liquefaction, weak clays, erosion, subsidence, ground shaking, etc.

During the 1999 Ji Ji, Taiwan earthquake, many of the mountain slopes were denuded by slides which continued to be a hazard for people traveling the mountain roads in the weeks following the earthquake. The many reinforced concrete gravity retaining walls that supported the road embankments in the mountainous terrain were all damaged, either from being pushed downhill by the slide (Figure 30.13) or, in some cases, breaking when the retaining wall was restrained from moving downhill (Figure 30.14).

One of the more interesting retaining wall failures during the Ji Ji earthquake involved a geogrid fabric/mechanically stabilized earth (MSE) wall at the entrance to Southern International University (Figure 30.15). This wall was quite long and tall, and its failure was a surprise, as MSE walls have a good

FIGURE 30.13 Gravity retaining wall pushed outward by landslide.

performance record during earthquakes. It was speculated that the geogrid retaining system had insufficient embedment into the soil; also, it was unclear why a MSE wall would be used in a cut roadway section.

One of the best known and largest landslides occurred at Turnagain Heights in Anchorage during the 1964 Great Alaska earthquake. The area of the slide was about 8500 ft wide by 1200 ft long. The average drop was about 35 ft. This slide was complex, but the primary cause was the failure of the weak clay layer and the unhindered movement of the ground down the wet mud flats to the sea. Figures 30.16 and 30.17 provide a section and plan view of the slide. The soil failed due to the intense shaking, and the whole neighborhood of houses, schools, and other buildings slid hundreds of yards downhill, many remaining intact during the fall (Figure 30.18).

Bridges are also severely damaged by landslides. During the 1999 Ji Ji, Taiwan earthquake, landslides caused the collapse of two bridges. The Tsu Wei Bridges were two parallel, three-span structures that cross a tributary of the Dajia River near the city of Juolan. The superstructure was simply supported "T" girders on hammerhead single-column bents with "drum"-type footings and seat-type abutments. The girders sat on elastomeric pads between transverse shear keys. The spans were about 80 ft long by 46 ft wide, and had a 30-degree skew. The head scarp was clearly visible on the hillside above the bridge. During the earthquake, the south abutment was pushed forward by the landslide, the first spans fell off the bent caps on the (far) north side, and the second span of the left bridge also fell off of the far bent cap (Figure 30.19). Also, the tops of the columns at Bent 2 had rotated away from the (south) Abutment 1. Therefore, it appears that both the top of Abutment 1 and the top of Bent 2 moved away from the slide, while the remaining spans were restrained by Bent 3 and Abutment 4 and remained in place. Perhaps the landslide originally had pushed against Bent 2, rotating the columns forward, and the debris had since been removed by the current or by a construction crew. Perhaps the skew had rotated the spans to the right as they fell, pushing them against the shear keys at Bent 2, which rotated the top of the columns forward and eventually pushed the spans off the top of Bent 2 and Bent 3. Or, perhaps there was an element of strong shaking that combined with the landslide to create the column rotation and fallen spans.

FIGURE 30.14 Gravity retaining wall with shear damage from landslide.

FIGURE 30.15 Fabric retaining wall damaged during the 1999 Ji Ji, Taiwan earthquake.

Dams are particularly vulnerable to landslides as they are frequently built to hold back the water in canyons and mountain streams. Moreover, inspection of the dam after an earthquake is often difficult when slides block the roads leading to the dam. When the Pacoima concrete arch dam was built in the 1920s, a covered tunnel was constructed to allow access to the dam. However, this tunnel and the roads and a tramway to the dam were damaged by massive landslides during and for several days after the 1971 San Fernando earthquake (Figure 30.20).

The Lower San Fernando Dam for the Van Norman Reservoir was also severely damaged during the 1971 San Fernando earthquake. It was fortunate that water levels were low, as the concrete crest on this earthen dam collapsed due to a large landslide along both the upstream (Figure 30.21) and downstream (Figure 30.22) faces. Considering the vulnerability of thousands of residences in the San Fernando Valley below (Figure 30.23), a dam failure can be extremely costly in terms of human lives and property damage.

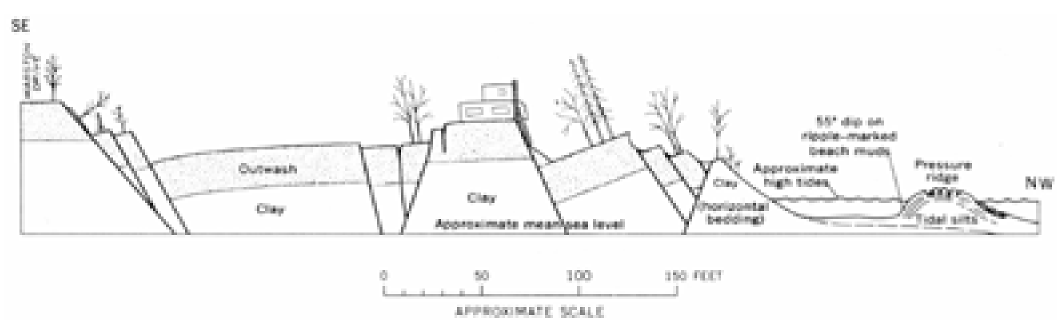

FIGURE 30.16 Section through eastern part of Turnagain Heights slide. (Courtesy of the National Academy of Sciences [8].)

FIGURE 30.17 Aerial view of Turnagain Heights slide. (Courtesy of the Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

FIGURE 30.18 One of about 75 homes damaged as a result of the Turnagain Heights slide. (Courtesy of the Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

FIGURE 30.19 Collapse of Tsu Wei Bridge due to landslide during the Ji Ji, Taiwan earthquake.

FIGURE 30.20 Landslides at Pacoima Dam following the 1971 San Fernando earthquake. (Courtesy of Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

FIGURE 30.21 Damage to the Lower San Fernando Dam. (Courtesy of Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

FIGURE 30.22 Closer view of damage to the Lower San Fernando Dam. (Courtesy of Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

FIGURE 30.23 Aerial view of Lower San Fernando Dam and San Fernando Valley. (Courtesy of Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

Weak Clay

The problems encountered at soft clay sites include amplification of the ground motion as well as vigorous soil movement, both of which can damage foundations. Several bridges suffered collapse during the 1989 Loma Prieta earthquake due to the poor performance of weak clay. Two parallel bridges were built in 1965 to carry Highway 1 over Struve Slough near Watsonville, CA. Each bridge was 800 ft long with spans ranging from 80 to 120 ft. The superstructures were continuous for several spans with transverse hinges located in spans 6, 11, and 17 on the right bridge and in spans 6, 11, and 16 on the left bridge (they are both 21-span structures). Each bent was composed of four 14″-diameter concrete piles extending above the ground into a cap beam acting as an end diaphragm for the superstructure. The surrounding soil was a very soft clay (Figure 30.24). The bridges were retrofit in 1984 by adding cable restrainers to tie the structure together at the transverse hinges.

During the earthquake, the soft saturated soil in Struve Slough was violently shaken. The soil pushed against the piles, breaking their connection to the superstructure (Figure 30.25) and pushing them away from the cap beam so that they punctured the bridge deck (Figure 30.26). Investigators arriving at the bridge found shear damage at the top of the piles, indicating that the soil limited the point of fixity of the piles to near the surface. They also found long, oblong holes in the soil, indicating that the piles were dragged from their initial position during the earthquake. There was some thought that the damage at Struve Slough was the result of vertical acceleration, but the structure's vertical period of 0.20 seconds was too short to be excited by the ground motion at this site.

Similarly, the Cypress Street Viaduct collapsed only at those locations that were underlain by weak Bay mud. This was a very long, two-level structure with a cast-in-place, reinforced-concrete, box-girder superstructure with spans of 68 to 90 ft. The substructure was multi-column bents with many different configurations, including some prestressed top bent caps. Most of the bents had pins (shear keys) at the top or bottom of the top columns, and all the bents were pinned above the pile caps, as well. There was a superstructure hinge at every third span on both superstructures. Design began on the Cypress Street Viaduct in 1949, and construction was completed in 1957. The pins and hinges were used to simplify the

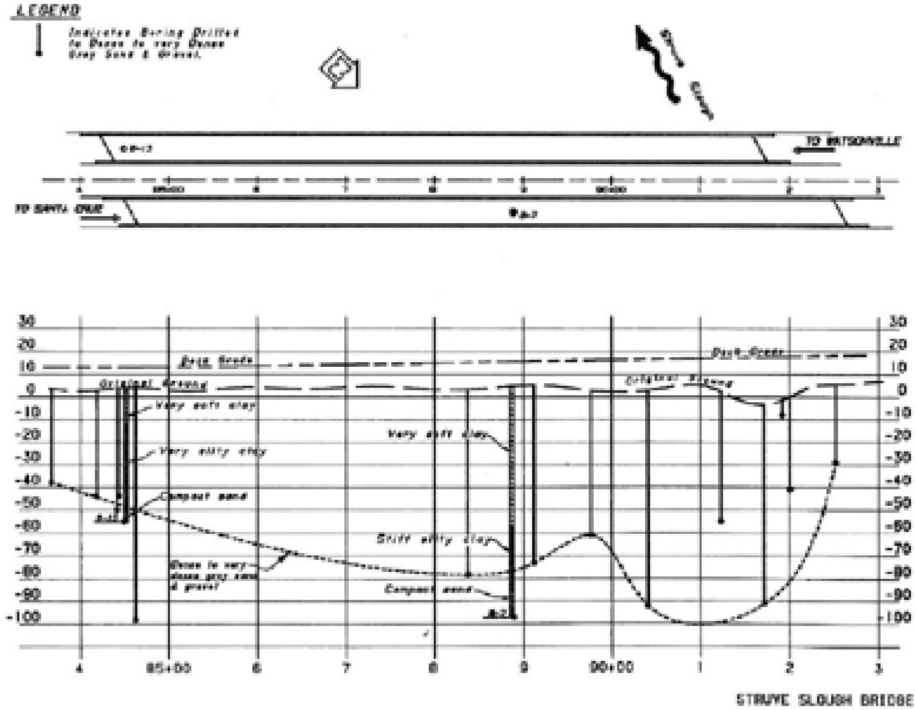

FIGURE 30.24 Soil profile for Struve Slough bridges.

FIGURE 30.25 Broken piles under the bridge.

FIGURE 30.26 Piles penetrating the bridge deck.

analysis for this long, complicated structure. The northern two thirds of the Cypress Street Viaduct was on Bay mud and had 50-ft piles, while the southern third was on Merritt sand with 20-ft piles (Figure 30.27).

During the 1989 Loma Prieta earthquake, the upper deck of the Cypress Street Viaduct collapsed from Bent 63 in the south all the way to Bent 112 in the north. Only Bents 96 and 97 remained standing. This

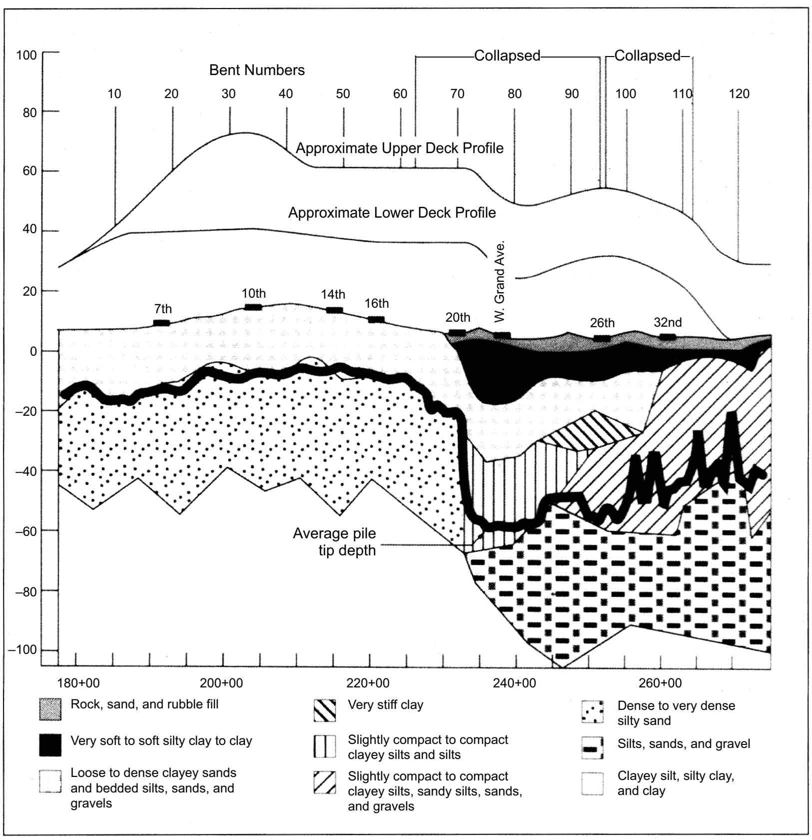

FIGURE 30.27 Geology of Cypress Street viaduct site [4].

FIGURE 30.28 Damage to Cypress Street viaduct.

collapse was the result of the weak pin connections at the base of the columns of the upper frame (Figure 30.28). There was inadequate confinement around the four #10 bars to restrain them during the earthquake; however, the soft Bay mud also played a role in the collapse. The southern portion of the bridge with the same vulnerable details, but supported on sand, remained standing. The northern portion was supported by soft Bay mud that was sensitive to the long period motion and caused large movements that overstressed the pinned connections (Figure 30.29).

Buildings on weak clay also are susceptible to earthquake damage. Mexico City was located 350 km from the epicenter of the 1985 magnitude 8.1 earthquake, but the city is underlain by an old lakebed composed of soft silts and clays (Figure 30.30). This material was extremely sensitive to the long-period (about 2 seconds) ground motion arriving from the distant but high-magnitude (8.1) source, as were the many medium-height (10- to 14-story) buildings that were damaged or collapsed during the earthquake (Figure 30.31). Many much taller and shorter buildings were undamaged due to the difference in their fundamental period of vibration.

FIGURE 30.29 Aerial view of Cypress Street Viaduct showing collapse at north end (left) where the structure crossed over Bay mud, while the southern part (right), supported by dense sand, remained standing.

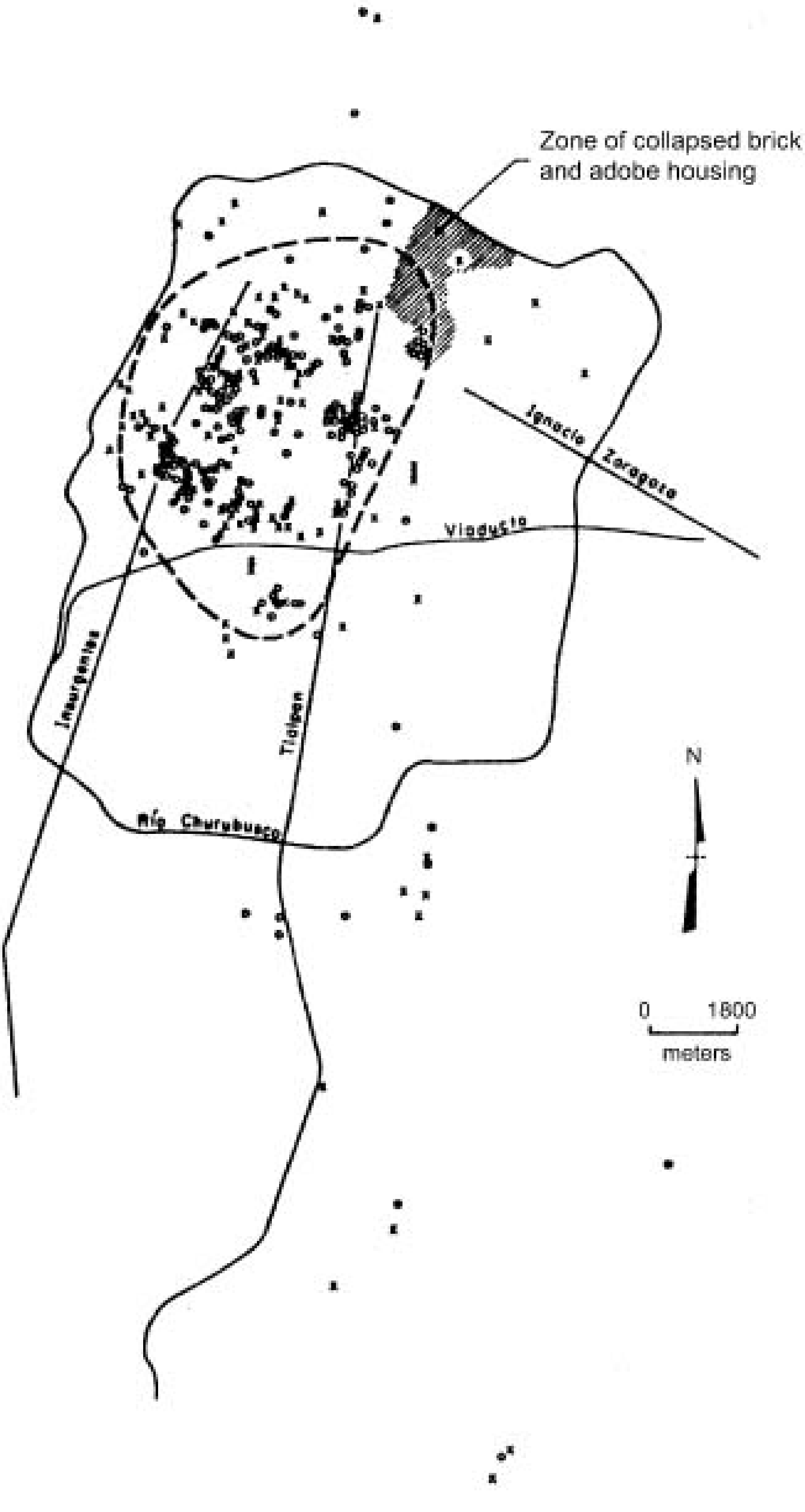

FIGURE 30.30 Locations of building damage at Old Lake Bed in Mexico City. (Courtesy of EERI [2].)

FIGURE 30.31 Damaged 10-story building between the Plaza de la Constitution and Zona Rosa in Mexico City. (Photograph by Karl Steinbrugge and courtesy of the Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

30.3 Damage as a Result of Structural Problems

Foundation Failure

Usually, it is the connection to the foundation or an adjacent member rather than the foundation itself that is damaged during a large earthquake; however, materials that cannot resist lateral forces (such as hollow masonry blocks) make a poor foundation and their use should be avoided (Figure 30.32). Engineers

FIGURE 30.32 Hollow concrete-block foundation that failed during the magnitude 6.0 1987 Whittier, CA earthquake. (Photograph by Karl Steinbrugge and courtesy of the Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

will occasionally design foundations to rock during earthquakes as a way of dissipating energy and of reducing the demand on the structure; however, when the foundation is too small, it can become unstable and rock over. During the 1999 Ji Ji, Taiwan earthquake (magnitude 7.6), a local three-span bridge rocked over transversely due to small, drum-shaped footings that provided little lateral stability (Figure 30.33).

FIGURE 30.33 Three-span bridge rocked over during the 1999 Ji Ji, Taiwan earthquake.

We have already seen pile damage as a result of weak clay on the Struve Slough bridges during the 1989 Loma Prieta earthquake. Similar damage occurred during the 1964 Great Alaska earthquake. After the 1971 San Fernando and 1995 Kobe earthquakes, an inspection was made of bridge foundations, but only a little damage to the tops of piles was found. As long as the foundation is embedded in good material, it usually has ample strength and ductility to survive large earthquakes. Usually, it is the more vulnerable elements above the foundation that can fail or become damaged during earthquakes. Still, as structures are designed to resist larger and larger earthquakes, we may begin to see more foundation damage.

Foundation Connections

The major cause of damage to electrical transformers, to storage bins, and to a variety of other structures and lifeline facilities during earthquakes is the lack of a secure connection to the foundation. Houses need to be anchored to the foundation with hold-downs connected to the stud walls and anchor bolts connected to the sill plates. Otherwise, the house will fall off its foundation, as shown in Figure 30.34.

The connections to bridge foundations also need to be carefully designed. Route 210/5 Separation and Overhead was a seven-span, reinforced-concrete, box-girder bridge with a hinge at Span 3 and seat-type abutments. The superstructure was 770 ft long on a 800-ft radius curve. The piers were 4 × 6-ft single-column bents. Piers 2 and 3 were on piles, while Piers 4 to 7 were supported by 6-ft-diameter drilled shafts. This interchange was built on consolidated sand.

During the 1971 San Fernando earthquake, a structure collapsed onto its west (outer) side, breaking into several pieces and causing considerable damage to two lower-level bridges. Close examination of the fallen structure revealed that the collapse was due to a pull-out of the column reinforcement from the foundations. There was no top mat of reinforcement (and no ties) in the pile caps at Piers 2 and 3. The column longitudinal reinforcement (22 #18 bars) was placed in the footing with 12″ 90° bends at the bottom of the reinforcement. Transverse reinforcement was #4 bars at 12″ around the longitudinal reinforcement. During the earthquake, the longitudinal reinforcement did not have sufficient development length to transfer the force to the footings. Insufficient confinement reinforcement in the footings and columns and the lack of a top mat of reinforcement resulted in the rebar (and columns) pulling out

FIGURE 30.34 House that fell from its foundation during the 1971 San Fernando earthquake. (Courtesy of Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

FIGURE 30.35 Failure of connection of column to pile shaft.

FIGURE 30.36 Failure of connection of column to footing.

of the footing (Figure 30.35). Piers 5 to 7 had straight #18-bars embedded 6 ft into pile shafts, and they also pulled cleanly out during the earthquake (Figure 30.36). After the San Fernando earthquake, the development length of large-diameter bars was increased, splices to longitudinal rebars were no longer allowed in the plastic hinge area, and more confinement steel was provided in footings and columns.

Soft Story

During the 1989 Loma Prieta earthquake, many houses and apartment buildings in the San Francisco area had severe damage to the ground floor. These structures had less lateral support on the ground floor to allow room for cars to park under the structure. The remaining supports could not support the movement of the upper stories, which dropped onto the ground (Figure 30.37).

FIGURE 30.37 Soft-story collapse in San Francisco during the 1989 Loma Prieta earthquake. (Courtesy of the U.S. Geological Service [10].)

A soft story, however, does not always occur on the bottom floor. During the 1995 Kobe, Japan earthquake, many tall buildings had mid-story damage, often due to designing the upper floors for a reduced seismic load. Most buildings in Japan are either built of reinforced concrete (RC) or of steel and reinforced concrete (SRC). These SRC buildings, when correctly designed, provide a great deal of ductility and more fire protection during large earthquakes. However, the design practice in Japan was to discontinue either the reinforced concrete or the steel above a certain floor. Figure 30.38 shows typical details used in SRC buildings. Figure 30.39 shows a ten-story SRC building where the third story collapsed during the 1995 Kobe earthquake.

Torsional Moments

Curved, skewed, and eccentrically supported structures often experience a torsional moment during earthquakes. A nine-story building in Kobe, Japan, consisted of shear walls along three sides and a moment-resisting frame on the fourth (east) side (Figure 30.40). Shaking during the 1995 Kobe earthquake caused a torsional moment in the building. The first-story columns on the east side failed in shear, the building leaned to the east (Figure 30.41), and it eventually collapsed (Figure 30.42).

Because rivers, railroad tracks, and other obstacles do not usually cross perpendicularly under bridge alignments, the columns and abutments must be built on a skew to accommodate them. These skewed bridges are vulnerable to torsion.

The Gavin Canyon Undercrossing consisted of two bridges over 70 ft tall, with a 67° skew, and composed of three frames. An integral abutment and a two-column bent supported each end-frame. The center-frame was supported by two two-column bents while supporting the cantilevered end-frames. The superstructure was reinforced-concrete box girders at the end-frames and post-tensioned concrete box girders at the center-frame. Each column was a 6×10-ft rectangular section, fixed at the top and bottom and with a flare at the top. The bridges were retrofitted in 1974 with cable restrainer units at transverse in-span hinges with an 8″ seat width that connected the frames.

During the 1994 Northridge earthquake, the superstructures unseated due to the following factors. The tall, center-frame had a large, long-period motion that was out of phase with the stiff end-frames. The center of stiffness of the end-frame was near the abutment, while its mass was near the bent, causing the end-frame to twist about the abutment. The sharp skew allowed the acute corners to slide off the

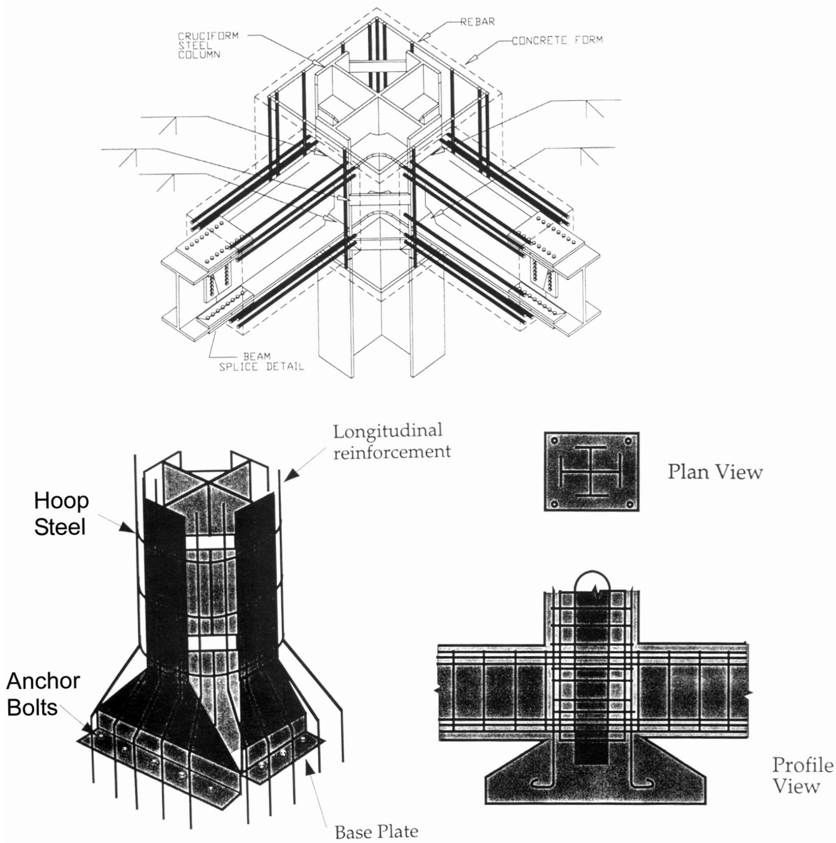

FIGURE 30.38 Examples of steel and reinforced-concrete (SRC) construction. (Courtesy of NIST [9].)

FIGURE 30.39 Ten-story steel and reinforced-concrete (SRC) building with third-floor collapse during the Kobe earthquake. (Courtesy of NIST [9].)

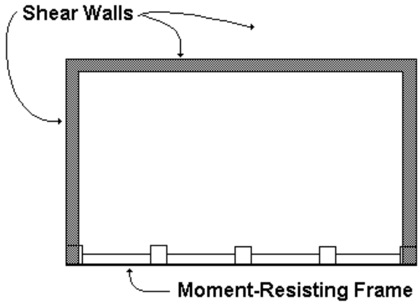

FIGURE 30.40 Plan view of nine-story steel and reinforced-concrete (SRC) building in Kobe.

FIGURE 30.41 Nine-story steel and reinforced-concrete (SRC) building immediately after the 1995 Kobe earthquake. (Courtesy of NIST [9].)

narrow seats. The cable restrainers, one of the first used in this country, were grouted in the ducts, making them too brittle and prone to failure. Both bridges failed, as shown in Figure 30.43.

Another interesting example of torsional damage occurred at the Ji Lu Bridge during the 1999 Ji Ji, Taiwan earthquake. This is a cable-stayed bridge with a single tower and cast-in-place, 102-m box girder spans sitting on two-column end bents that connect the structure to precast "I"-girder approach spans

FIGURE 30.42 Eventual collapse of nine-story steel and reinforced-concrete (SRC) building after the Kobe earthquake. (Courtesy of NIST [9].)

FIGURE 30.43 Damage to Gavin Canyon Undercrossing during the 1995 Northridge earthquake.

(Figure 30.44). The distance from the top of the deck to the top of the tower is 58 m; from top of the footing to the soffit, 20 m. All the foundations are supported on driven piles. Construction was almost completed on this bridge at the time of the earthquake. All of the cables had been tensioned and all but one had been permanently socketed into the tower. The false work had been pretty much removed except for a few final pours for the portion of the superstructure where it connects to the tower.

The dominant mode of shaking for this structure was twisting of the tower as the two cantilever spans moved back and forth. Looking at Figure 30.45, we can see that the key at the end of the spans walked up and down the bent seat almost to the end of the support. T.Y. Lin engineers explained that because the final pour around the tower had not been completed, this structure was extremely flexible in this direction.

Similar damage occurred to the center piers of curved ramps to the Minatogawa Interchange during the Kobe earthquake. In this case, the superstructure swung off its end-supports and the center column

FIGURE 30.44 The Ji Lu cable-stayed bridge after the 1999 Taiwan earthquake.

FIGURE 30.45 Damage at end-supports to Ji Lu cable-stayed bridge.

suffered severe torsional damage (Figure 30.46). As one member of a bridge goes into flexure it can create large torsional moments in adjacent members. For instance, flexure in the columns of outrigger bents causes large torsional moments in the bent cap (Figure 30.47). All of these examples reinforce the idea that most structures require consideration of all three translations and rotations.

FIGURE 30.46 Column damage to the Minatogawa Interchange during the Kobe earthquake.

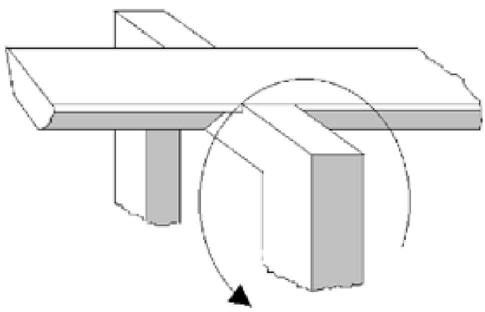

FIGURE 30.47 Column flexure causing torsion in the bent cap.

Shear

Most building structures use shear walls or moment-resisting frames to resist lateral forces during earthquakes. Damage to these systems varies from minor cracks to complete collapse. Figure 30.48 is a photo of the Mt. McKinley Apartments after the 1964 Great Alaska earthquake. The 14-story, reinforced-concrete building was composed of narrow exterior shear walls and spandrel beams (Figure 30.48), as well as interior and exterior columns and a central tower. During the 1964 earthquake, this structure suffered major structural damage to most of the load-bearing members.

The most serious damage was to a shear wall on the north side of the building (Figure 30.49). A very wide shear crack split the wall in two, directly under a horizontal beam. This crack was a result of not having enough transverse reinforcement to hold the wall together as it moved transversely and was also due to a cold joint in the concrete at that location. The spandrel beams between the walls had large "X" cracks associated with shear damage as the building moved back and forth. These cracks decreased in size on the upper floors.

There was also shear damage to many of the columns. Figure 30.50 is one of the exterior columns on the south side of the building with a diagonal shear crack. Again, the problem was insufficient transverse reinforcement to resist the large shear forces that occurred during the earthquake. The central tower was

FIGURE 30.48 West elevation of Mt. McKinley Apartments after the 1964 Great Alaska earthquake. (Photograph by Karl Steinbrugge and courtesy of the Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

FIGURE 30.49 Damage to north side of Mt. McKinley Apartments. (Photograph by Karl Steinbrugge and courtesy of the Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

FIGURE 30.50 Damage to the south side of Mt. McKinley Apartments. (Photograph by Karl Steinbrugge and courtesy of the Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

also damaged. The Mt. McKinley Apartments, though, can be viewed as a success, as there was no collapse and no lives lost during this extremely large earthquake.

Bridges are equally susceptible to shear damage. For instance, there was considerable shear damage to piers on elevated Route 3 during the Kobe earthquake. The superstructure is mostly steel girders, and the substructure is reinforced-concrete, single-column bents. Between Piers 148 and 150, the superstructure

FIGURE 30.51 Shear failure of Pier 150 on Kobe Route 3.

is a three-span, continuous, double-steel box with span lengths of 45 m, 75 m (between Pier 149 and 150), and 45 m. Pier 149 is a 10-m tall × 3.5-m square, reinforced-concrete, single-column bent supported on a 12-m square pile cap. Pier 150 is a 9.1-m tall × 3.5-m square, reinforced-concrete, single-column bent supported on a 14.5 × 12-m rectangular pile cap.

Figure 30.51 shows the shear failure at Pier 150. This damage was the result of insufficient transverse reinforcement and poor details. During the large initial jolt (amplified by near-field directivity effects), the transverse reinforcement came apart and resulted in failure of the column due to shear. Pier 149 was also severely damaged, but, because it was taller (and more flexible), most of the force went to stiff Pier 150. The three-span continuous superstructure survived the collapse of Pier 150 with minor damage.

Flexural Failure

Flexural members are often designed to form plastic hinges during large earthquakes. A plastic hinge allows a member to yield and deform while continuing to support its load; however, when there is insufficient confinement for reinforced concrete members (and insufficient unsupported width to thickness [*b/t*] ratios for steel plates), a flexural failure will occur instead. Often, flexural damage is accompanied by compression or shear damage, as the capacity of the damaged area has been lowered.

The Dakkai subway station in Japan is a two-story, underground, reinforced-concrete structure. It was constructed by removing the ground, building the structure, and then covering it (the-cut-and-cover method). During the 1995 Kobe earthquake, the center columns on both levels suffered a combination of flexural and compression damage that caused both roofs to collapse along with a roadway that ran above the station (Figure 30.52). Figure 30.53 shows the rather slender center columns at the lower level after the earthquake. The columns had insufficient transverse reinforcement at the location of maximum moment. The transverse reinforcement broke as the columns were displaced, allowing the longitudinal reinforcement to buckle and the concrete to fall out of the column.

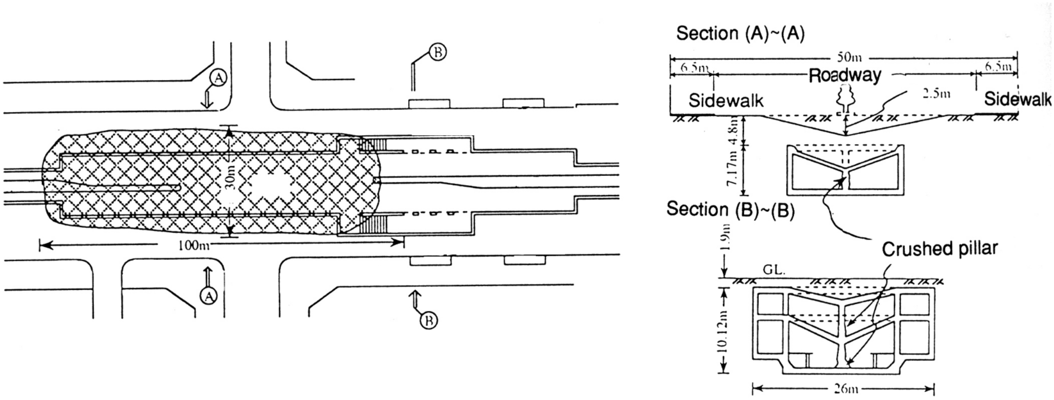

FIGURE 30.52 Plan and section drawings of the Dakkai Subway Station after the Kobe earthquake. (Courtesy of Japan Society of Civil Engineers [5].)

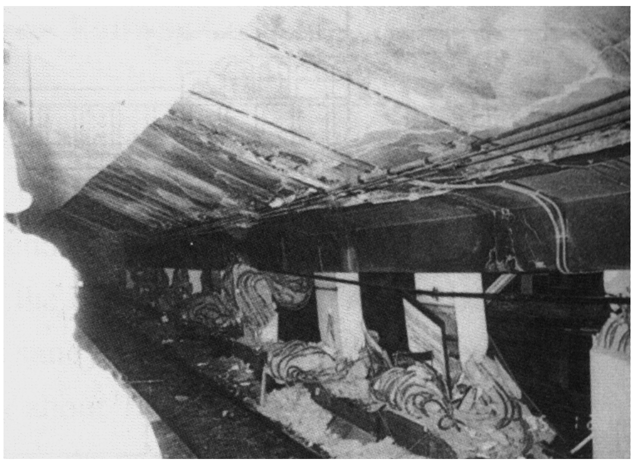

FIGURE 30.53 Flexural damage to columns at lower-level of Dakkai Subway during the 1995 Kobe earthquake. (Courtesy of Japan Society of Civil Engineers [5].)

Steel columns experience similar damage when the flexural demand exceeds the capacity. In downtown Kobe, in the Nagata District, Route 3 splits into two parallel structures, with the superstructure composed of 50-m long, simple spans with steel girders and a three-span continuous steel box girder section between Piers 585 and 588. The substructure is comprised of 2.2-m diameter, 14-m tall, steel, hammerhead single columns. The steel columns are bolted onto 4-m diameter, 20-m long, hollow concrete shafts. The column bottoms are filled with concrete to protect against vehicular impact. During the Kobe earthquake, these steel columns had damage varying from local buckles to a complete section buckle, and, at a few locations, the steel shell had torn, splitting the column. Most of the buckling occurred in a thinner section of the column. In some cases (Figure 30.54), the column underwent an excursion in only one direction and consequently had a buckle on one side of the column. In some cases, the buckled face tore in the tension cycle (Figure 30.55). The tears occurred in low ductility welds. After the earthquake, the columns were tilting dangerously to the side. Buckling occurred before a plastic hinge was formed. A few columns remained undamaged as a result of failed bearings. Although local buckling cannot be completely eliminated, its spread can be prevented by maintaining smaller b/t ratios. This is accomplished through thicker sections, more frequent stiffeners and diaphragms, or filling the steel shells with lightweight concrete.

FIGURE 30.54 Pier 585 on Kobe Route 3 during the 1995 Kobe earthquake. (Courtesy of Hanshin Expressway Public Corporation.)

FIGURE 30.55 Torn buckle of steel column on Route 3 after the 1995 Kobe earthquake. (Courtesy of Hanshin Expressway Public Corporation.)