The Performance of Highway Bridges in the Northridge, California, Earthquake

Bridge seismic retrofit programs exhibited a high degree of success. However, additional requirements for retrofit programs and initial design are needed.

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

During the Northridge earthquake of January 17, 1994, in Los Angeles, seven highway bridges suffered partial collapses and another 170 bridges suffered damage ranging from minor cracking to the slumping of abutment fills. Many of the damaged structures were closed only temporarily for inspection and/or shoring. However, some were closed permanently and have since been demolished and are awaiting replacement. Of those bridges with collapsed spans, all were designed and constructed from the mid-1960s to the mid-1970s. None were "new" in the sense of being built to current codes. Most had been retrofitted with cable restrainers (where appropriate). Some bridge columns in the epicentral region had also been strengthened with steel-jackets. Whereas several cable restrainer units failed, none of the steel-jacketed columns showed distress despite strong ground shaking in some cases.

Eight bridges suffered major damage:

- Gavin Canyon Undercrossing
- Route SR14/I-5 Separation and Overhead (Southbound)
- Route SR14/I-5 North Connector Overcrossing
- Bull Creek Canyon Channel Bridge
- Mission-Gothic Undercrossing
- Balboa Boulevard Overcrossing
- Fairfax-Washington Undercrossing
- La Cienega-Venice Undercrossing

The earthquake damage to the first four structures listed above is described here. Figure 1 shows the location of all these bridges relative to the epicenter of the earthquake in Northridge.[1]

From Ref. 1

FIGURE 1. Location map of bridges with major damage.

The damages that were sustained by these and other bridges can be summarized as follows:

- Abutment back-fill settlement and erosion
- Abutment and shear key structural damage

- Flexural failures in plastic hinges with inadequate confinement
- Pounding and unseating at hinge seats and girder supports
- Shear failures in short single columns, piers, multi-columns bents, columns with flares and other accidental restraints, and columns in skewed bridges.

Seismological Observations

On the morning of January 17, 1994, at 04:30:55.4 (PST), an earthquake occurred near Northridge in the San Fernando Valley about 35 kilometers northwest of the Los Angeles central business district in Southern California (coordinates: 34° 12.7′ N 118° 32.3′ W, and at a depth of 18 kilometers). The preliminary moment estimate determined from regional surface waves and teleseismic recordings was 1 to 1.5×10^{26} dyne-cm, which gives a moment magnitude, M_w, of 6.7. The preliminary local magnitude, M_L, determined from telemetered strong-motion instruments in Southern California, was 6.4. Both the first-motion focal mechanism from local stations and the teleseismic mechanism indicated that there was a thrust fault on a plane trending in a northwest direction. The pattern of aftershocks revealed that the plane dipping toward the southwest was the fault plane, and its dip was 40 to 50 degrees.

A contour map of the maximum component of peak horizontal acceleration for the San Fernando Valley and Los Angeles Basin is shown in Figure 2. A similar map for the vertical component is shown in Figure 3. These maps were constructed by the University of Southern California (USC) using data from the Los Angeles Strong Motion Accelerograph Network.[2] Other sources included the California Strong Motion Instrumentation Program (CSMIP) and the United States Geological Survey's National Strong Motion Program (USGS/NSMP).

The strong ground motions from the Northridge earthquake were recorded on many instruments within the Los Angeles area. Peak accelerations of free-field instruments were generally 0.5 to 1.0 g in the aftershock area and decreased to 0.1 g at distances of about 50 kilometers. Several sites close to the epicentral area recorded accelerations over 1 g. The extensive damage caused by this earthquake emphasizes the need for better understanding of the local site conditions that affect ground motion. More than 75 instruments were deployed following the mainshock to study these site effects. Seismic instruments were placed at many of the strong-motion instrument sites that produced significant records of the mainshock. Also, many of the severely damaged areas in Northridge, Sherman Oaks and Santa Monica were instrumented, as well as the collapsed bridge sites at the State Route 14/Interstate 5 interchange, State Route 118 near Woodley and the Interstate 10 freeway near La Cienega Boulevard.

Thousands of aftershocks occurred in the two-month period following the earthquake including six magnitude 5, forty-three magnitude 4 and 284 magnitude 3 events as of March 15, 1994. The locations of the aftershocks were distributed across an area about 30 by 20 kilometers. These locations were clearly deeper toward the south. In cross-section, they revealed a plane dipping toward the southwest (which is interpreted to be the fault plane for the earthquake). This plane extends from the mainshock hypocenter at 18 kilometers upward toward the surface. Preliminary analysis of teleseismic data indicated that most of the slip on the fault plane occurred at depths below five to 10 kilometers, with relatively little slip of the shallow portions of the fault.

The location of the fault plane, as inferred from the aftershock distribution, did not correspond to any mapped geologic fault. The earthquake did occur, however, within a system of known thrust faults that extend along the northern edge of the San Fernando Valley. Most of the mapped faults have northerly dips, although there are several structures, such as the nearby Oak Ridge system, that have southerly dips.

Many of the aftershocks were located on or close to the rupture plane. However, there were also many off-fault events that had a variety of focal mechanisms. One example is the magnitude 5.1 earthquake of January 29, which was a shallow strike-slip event above the main rupture plane. Portable instruments recorded accelerations of up to 0.8 g from this earthquake.

Real-time information about the mainshock and aftershocks were broadcast to 15 members

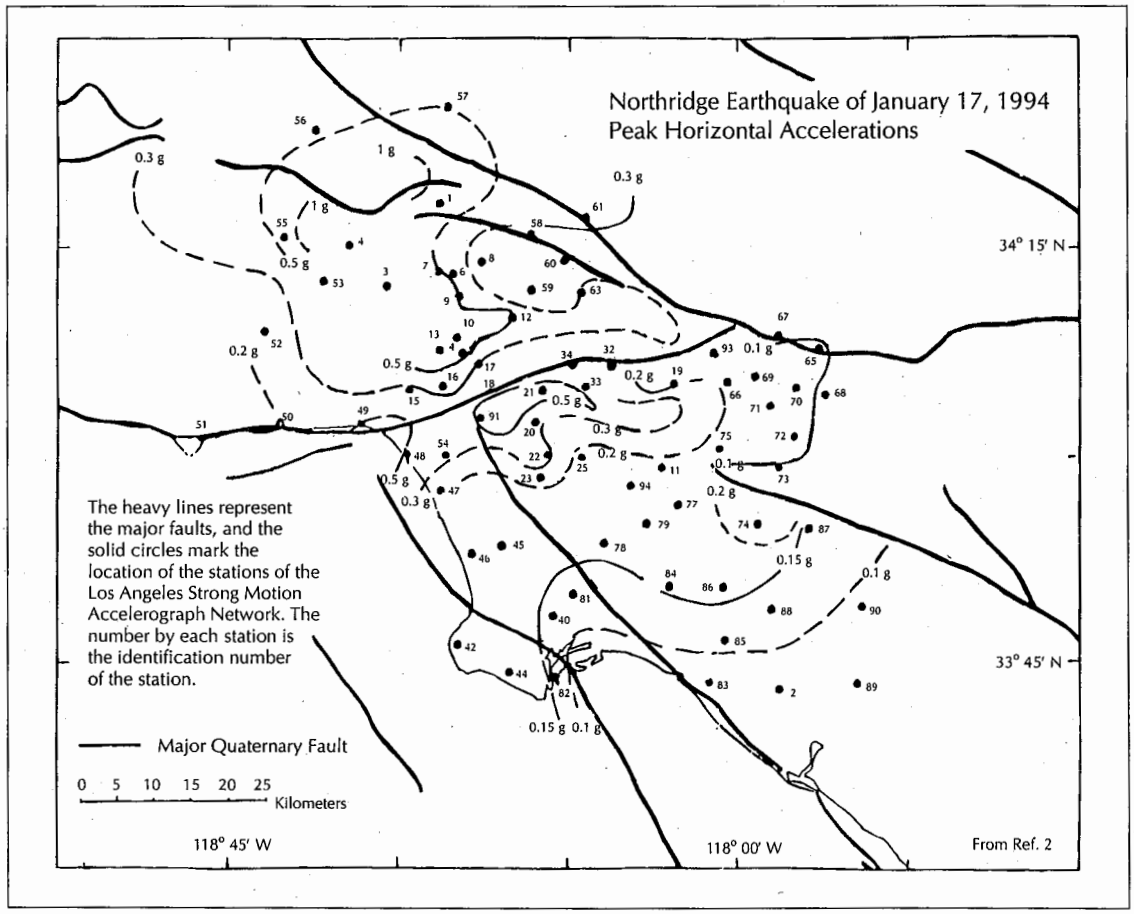

FIGURE 2. Contours of peak horizontal ground acceleration observed during the Northridge earthquake, expressed as a fraction of the acceleration of gravity (as of February 1, 1994).

of the Caltech-USGS Broadcast of Earthquakes (CUBE) program. This project is a cooperative effort to provide rapid earthquake information in southern California. CUBE participants include governmental emergency response agencies, water and power utilities, railroads and other private sector organizations. Earthquake locations and magnitudes are disseminated via pagers and computer displays throughout southern California and to other parts of the country.

Generally, information is received within five to eight minutes of the earthquake occurrence. Because of various problems encountered at the time of the Northridge earthquake, information about the mainshock was relatively slow in being released. However, data on the first aftershocks were being broadcast within 15 minutes of the mainshock.

Aftershocks were recorded at several sites by California Department of Mines and Geology, USGS and Lamont-Doherty Earth Observatory (LDEO) personnel.

Geodetic Observations

Results from re-surveys of benchmarks, using the Global Positioning System (GPS), revealed the extent of static displacements due to the earthquake at 15 sites. In the aftershock region, there were vertical uplifts of 40 to 50 centimeters and horizontal motions of 2 to 20 centimeters. These movements are consistent with the fault geometry derived from seismological observations of a plane dipping toward the southwest at about 40 degrees. Preliminary modeling of the data indicated that there was a slip of 2.5 to 3.5 meters on a 10 by 10 kilometer patch of the fault. The motion was primarily thrust

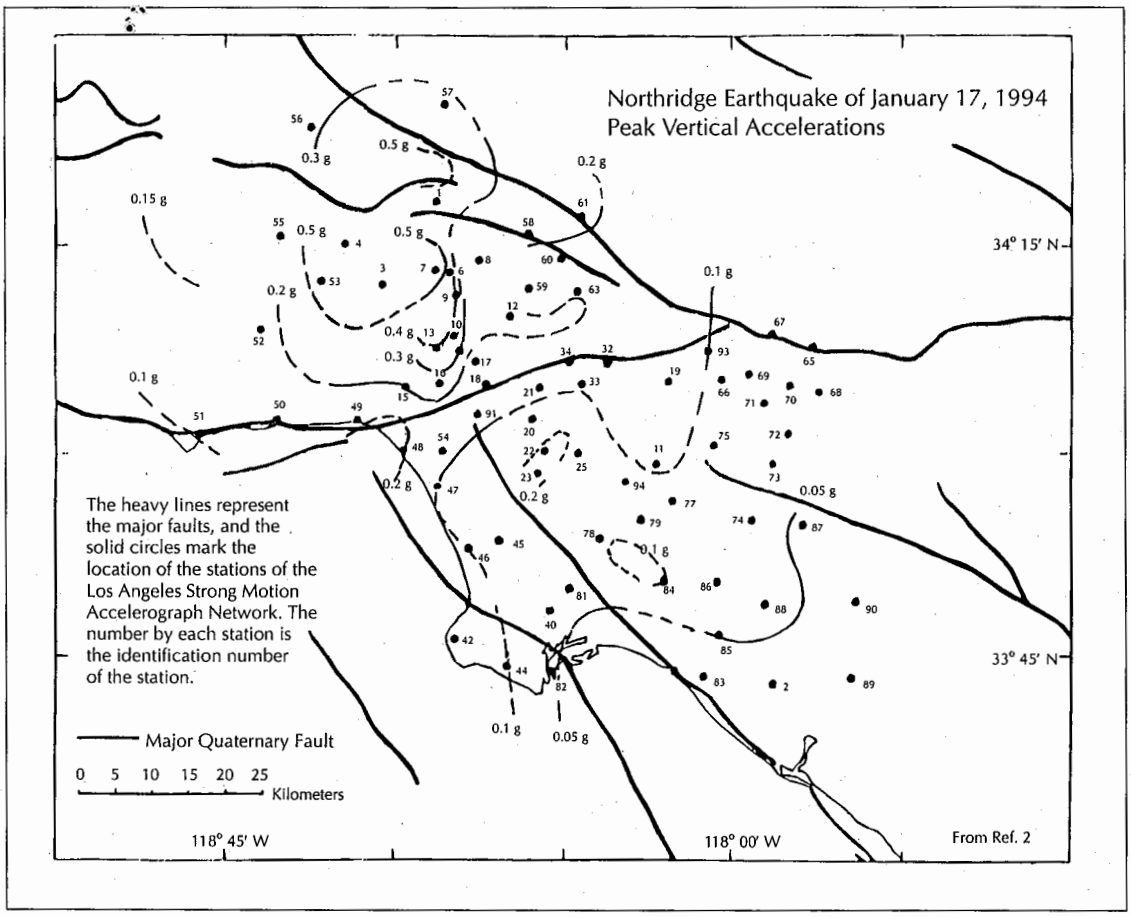

FIGURE 3. Contours of peak vertical ground acceleration observed during the Northridge earthquake, expressed as a fraction of the acceleration of gravity (as of February 1, 1994).

faulting, and most of the slip occurred at depths of greater than six kilometers.

Geological Observations

Two areas of surface cracking that had been observed immediately after the earthquake are currently being studied. It is unclear if these cracks are the direct result of tectonic faulting or whether they are due to ground shaking. The small amount of observed surface cracking, however, is consistent with the geodetic results that indicated that there was not a large amount of slip on shallow portions of the fault.

The most extensive area of ground deformation was in Potrero Canyon on the north side of the Santa Susana mountains near the northern edge of the aftershock zone. A series of discontinuous tension cracks and normal faults with displacements of up to 60 centimeters were observed on both the north and south sides of the canyon extending for about three kilometers. Evidence for compressional features with vertical displacements of eight to 20 centimeters were also observed along the south margin of the canyon. None of the deformations were associated with any previously mapped surface fault.

A second system of small cracks was studied along a five-kilometer zone in Granada Hills, a region that had numerous water and gas main ruptures caused by the earthquake. The complex series of cracks had both extensional and left-lateral features. Some of the deformation occurred in association with buried stream channels but may also represent secondary faulting on the Mission Hills fault.

There were extensive occurrences in the younger sediments of the western Santa Susana

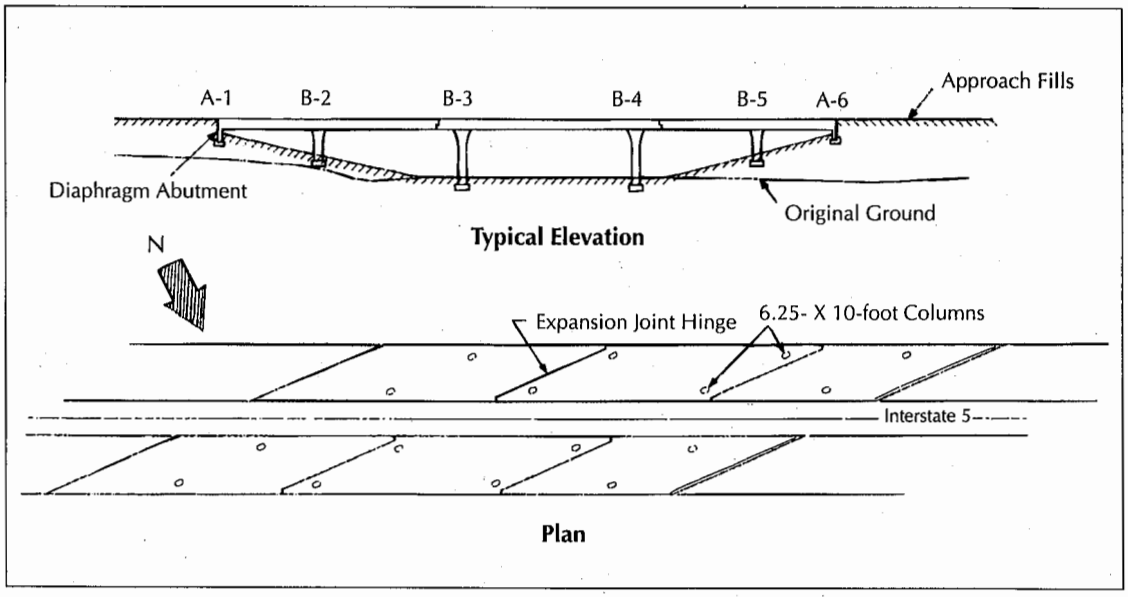

FIGURE 4. General plan and elevation for the Gavin Canyon crossing.

Mountains, Oak Ridge and Big Mountain areas. Rock falls have choked the ravine bottoms of many canyons in the Santa Susana Mountains. These rock falls were of some concern following the earthquake since heavy rains could saturate the material, causing it to mobilize into debris flows that could threaten structures near the mouths of the canyons.

Geotechnical Aspects

The geotechnical aspects of the Northridge earthquake include soil amplification, topographic effects on the intensity, frequency and duration of ground motion, soil liquefaction, permanent ground deformation and landslides. In general, however, the geotechnical effects on the performance of bridges during the earthquake were relatively minor. Goltz describes the earthquake's effects on buildings and buried lifeline systems.[1]

Gavin Canyon Undercrossing

Description. This undercrossing carries the northbound and southbound lanes of Interstate 5 (I-5) over Gavin Canyon Road using two separate bridges. I-5 is the main link between northern and southern California. The bridges are located approximately two and one-half miles northwest of the I-5 and State Route (SR) 14 interchange. They were originally constructed in 1967 and were retrofitted with expansion joint hinge restrainers in 1974.

Each bridge is five spans in length and consists of three frames separated by expansion joints with eight-inch bearing seats. Span lengths on the northbound bridge are 120, 170, 208, 145 and 98 feet. The southbound bridge has span lengths of 128, 170, 208, 145 and 90 feet. Both bridges are 68 feet wide.

The two outside frames are composed of cast-in-place reinforced concrete box girder construction. Each frame is supported on a monolithic end diaphragm abutment and a single two-column bent. The center frame is a cast-in-place prestressed concrete box girder supported on two bents, each with two columns. The abutments, expansion joints and bents are all oriented at a relatively large skew that is approximately 67 degrees to normal. The bridges are separated by a 42-foot wide median. A schematic of the bridges is shown in Figure 4.

The structure is located in mountainous terrain. Large approach fills exist on each end of the bridge. Bent footings are supported on steel "H" piles that penetrate through approximately 20 feet of loose to very dense sands and gravels down to siltstone and shale. These footings are relatively compact in size and provide limited resistance to rotation. Abutments are

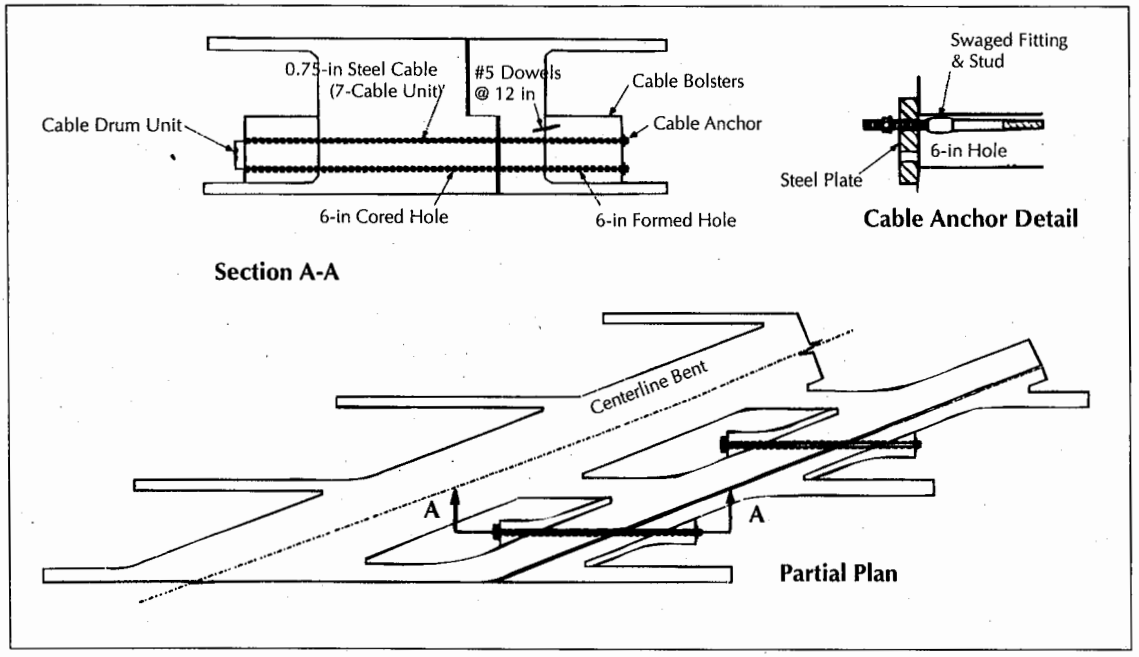

FIGURE 5. Hinge restrainer details for the Gavin Canyon Undercrossing.

supported on spread footings located in the approach fills.

The bridge's reinforced concrete columns measure 6.25 by 10 feet in cross-section at the base and have large, but lightly reinforced, architectural flares that widen out to over 20 feet at the soffit. All columns are poorly confined with nominal half-inch diameter ties at 12 inches on center over the full length of the column. Main column steel is spliced at the footing. At bents 2 and 5 the clear column heights are generally shorter, with all but two columns measuring between 28.3 and 38.9 feet. One column within each of these bents is noticeably longer. These columns measure 53.6 and 67.3 feet, respectively. The columns in bents 3 and 4 are typically longer than those in bents 2 and 5 and measure between 65.9 and 73 feet in length.

The bridge survived the 1971 San Fernando earthquake with only minor damage. Most damage was confined to the expansion joint hinges and consisted primarily of minor spalling and cracking of the concrete near acute corners of the supporting portion of the hinges. The expansion joints were displaced transversely up to approximately one and three-quarter inches. Damage to barrier rails at the hinges indicated that there was longitudinal movement during the earthquake of approximately three inches. Settlement of the bridge, particularly at the west end, was noticed after the earthquake.[3,4]

Expansion joint hinge restrainers were not present during the 1971 earthquake. They were installed after the earthquake. The restrainers are unusual by modern standards because they are oriented along the centerline of the bridge rather than normal to the expansion joints. Restrainer details are shown in Figure 5.

Ground Motion. The ground shaking that occurred at this bridge site is thought to have been severe. The bridge is located in one of the regions where extensional surface fractures were noticed. Other preliminary seismological analyses indicate the bridge site is near the surface projection of the Oak Ridge ("Newhall") fault that is thought to be the source of the earthquake.[5]

The nearest strong-motion data available at the time of this writing is from CSMIP Station Number 24279, a free-field instrument located approximately two and one-half miles to the north at the Newhall Fire Station.[6] This instrument is founded on alluvium of unknown depth. Peak accelerations in excess of 0.6 g were

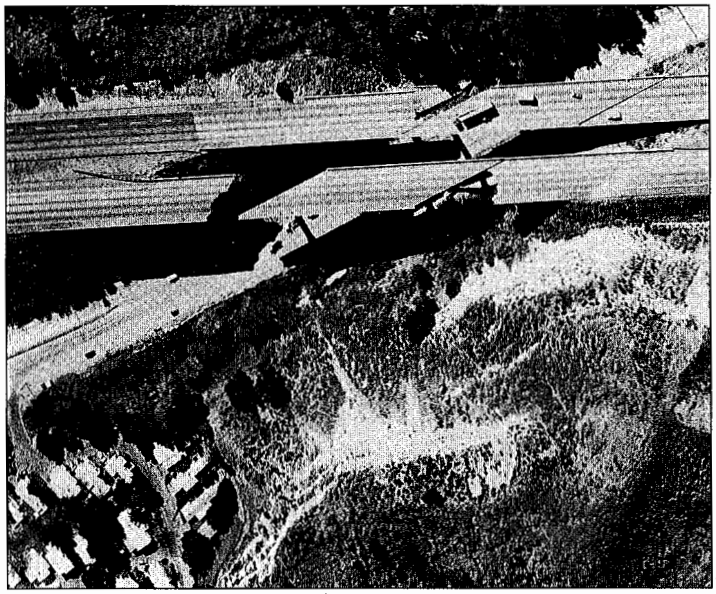

FIGURE 6. An aerial view of the damage at the Gavin Canyon Undercrossing.

recorded in all three directions. The motions for the peak horizontal accelerations were nearly in phase, indicating that there was a horizontal ground acceleration peak of approximately 0.8 g in a northeasterly direction. Very strong shaking lasted for approximately six to seven seconds.

High horizontal accelerations were also recorded at the Olive View Hospital parking lot free-field instrument in Sylmar, which is approximately seven miles east of the bridge site. A peak horizontal acceleration of 0.91 g was recorded in the easterly direction. Strong shaking lasted for approximately nine seconds.

Records from both instruments have been processed and indicated very high spectral accelerations for the frequency range of these structures.[7] In fact, in the case of the Newhall record, the five percent damped spectral accelerations for the north-south component of ground motion exceed the California Department of Transportation's (Caltrans) design spectrum for the bridge site for the period range between one half to two seconds. (Caltrans uses a smoothed elastic design spectrum based on average motions for the "maximum credible earthquake" on the most critical known fault.[8])

Observed Earthquake Damage. Both structures suffered failures due to total or partial loss of support at the expansion joint hinges. The acute, corner of the supported span tended to become unseated first due to a counterclockwise rotation of each of the structural sections about a vertical axis (see Figure 6).

The movement of the superstructure caused restrainer cables to be pulled at an angle to their principal axis as evidenced by spalling at the edges of cored holes through which the restrainers passed. In some cases, restrainer cables snapped as the expansion joints separated. In other cases, they pulled through the expansion joint diaphragm. Some cables remained intact, helping support the partially unseated spans and preventing the unseating of the span at one of the hinges.

Despite the strong ground shaking at the site, the structure suffered very little column damage as can be observed in Figure 7. Only minor cracking was observed at the base of some columns. Cracked pavement at the bridge approaches is evidence that the fills shifted during the earthquake. There was some minor abutment damage.

Failure Analysis. The failure of this bridge can be attributed to large lateral and rotational movements of the superstructure sections that caused the narrow expansion joint hinges to become unseated. Rotational movement of the end spans was probably induced by the eccentricity between the center of mass of the superstructure and the center of stiffness of the end frame that tended to be shifted toward the rather stiff diaphragm abutments. The strong shock measured at the Newhall Fire Station was oriented in a direction transverse to the bridge. Motion in this direction would have resulted in maximum excitation of the rotational response of the end spans.

An additional factor in the response of the structure was the large difference in the vibra-

tional characteristics of the center frame compared to the two end frames. The center frame was much more flexible and had a natural period of vibration that was much longer than the end frames. Calculated relative displacements between the two frames based on a 0.6 g Caltrans design spectra loading exceeded the eight-inch hinge seat width by several inches. It is unlikely that restrainers would have been able to prevent the out-of-phase movement between the three frames that most likely occurred during the earthquake.

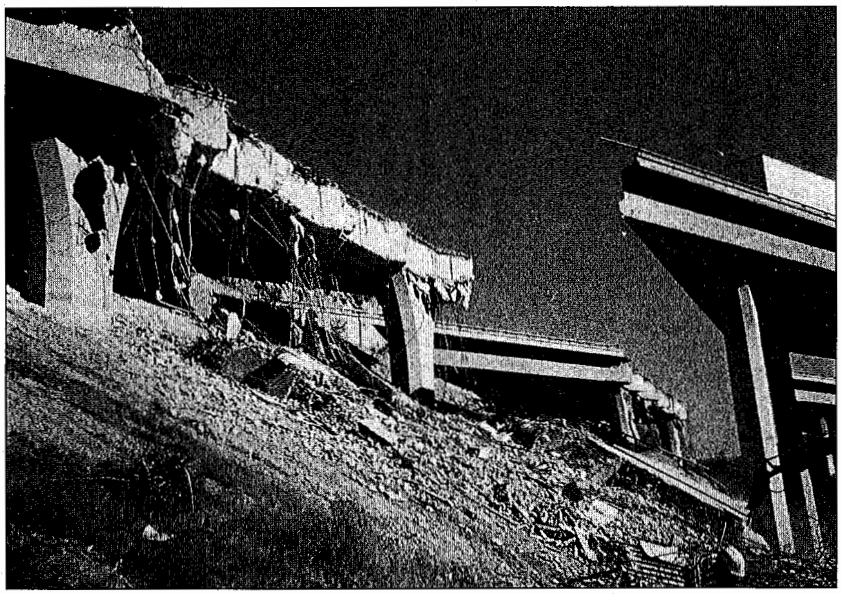

FIGURE 7. Collapsed end spans at the Gavin Canyon Undercrossing after demolition.

Rough calculations show that the columns within this bridge were likely to have been exposed to relatively high ductility demands if it is assumed that the footings remained fixed against rotation. The satisfactory performance of the columns at bents 3 and 4 can be attributed to the relatively low shear forces resulting from their long length and the low percentage of longitudinal reinforcement. However, more damage was expected at bents 2 and 5 due to high calculated shear forces. The fact that these columns remained relatively undamaged may be explained by the rocking of the footings. Calculations showed that these footings were unable to develop the ultimate moment capacity of the column. Most likely, they rocked during the earthquake, thus preventing the development of high shear forces. Moreover, such rocking most probably aggravated the relative superstructure displacements at the hinges.

Issues/Questions. This failure illustrates the difficulties associated with large skews and with multiple expansion joints. A major lesson is the importance of eliminating and/or minimizing skew and expansion joints, especially in high structures where the vibration characteristics of adjacent frames are different from one another.

This failure also illustrates that just because a bridge survives one earthquake, it does not necessarily mean it would be immune to damage in future earthquakes.

The unique response of skewed bridges to earthquakes has been observed in the past. As of yet there is no widely accepted design procedure that adequately accounts for all of the effects of skew during an earthquake. Additional research is needed to better understand this phenomenon and to develop a methodology for designing skewed supports.

With respect to column behavior, this bridge is another example of the importance of relative shear strength. The absence of damage in the presence of ground motions that most likely exceeded those for the "maximum credible earthquake" illustrates the need for more research on the effects of shear on column behavior.

State Route 14/I-5 Separation & Overhead (Southbound)

Description. Route 5 (I-5) is a U.S. Interstate running north-south along the west coast and Route 14 (SR14) is a state highway beginning at the SR14/I-5 interchange and running northeast to U.S. Route 395 in Kern County. The

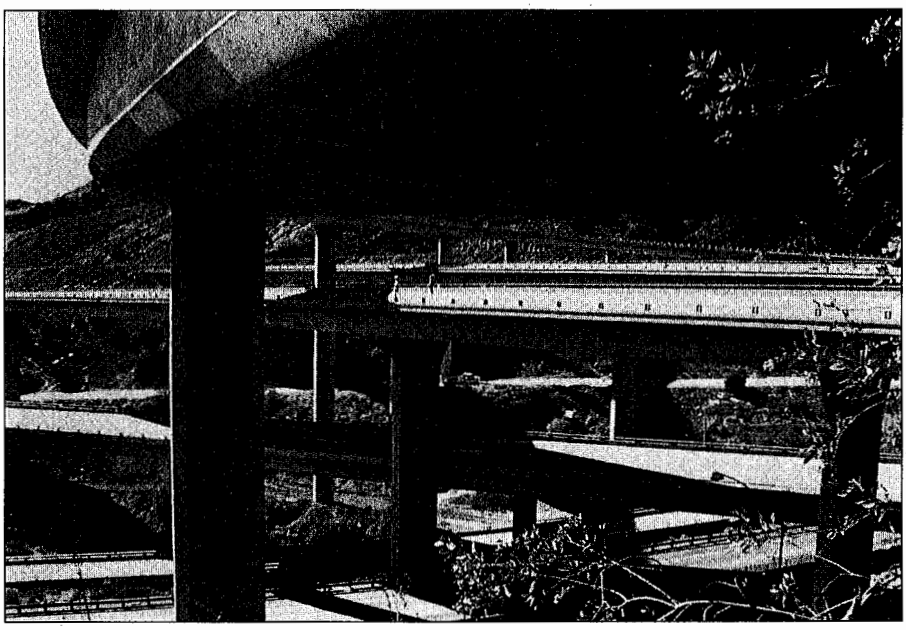

FIGURE 8. A view from abutment 10 of the south connector looking southwest at the SR14/I-5 interchange.

southbound SR14/I-5 separation and overhead structure is located at mile post 24.5 on I-5 in Los Angeles County approximately 24 miles to the northwest of downtown Los Angeles and is generally aligned in a north-south direction. The structure carries traffic from southbound SR14 to southbound I-5 spanning the I-5 southbound and northbound mainlines.

The SR14/I-5 interchange is located on steeply dipping, well consolidated sandstone of the Towsley formation. Alluvium is present locally. Structures are founded in the sandstone with abutment fills present at approaches. A general view of this interchange immediately following the earthquake of January 17 is shown in Figure 8. A simplified plan view of this interchange is given in Figure 9. In this view, the four viaducts that allow the interchange of traffic between I-5 and SR14 are shown. A fifth structure, which carries southbound truck traffic from SR14 to I-5, is not shown. It suffered only minor shear key damage.

The southbound SR14/I-5 separation and overhead is a 10-span continuous, cast-in-place, five-cell concrete box girder bridge. It has seat-type abutments and single column bents. The total length is 1,582 feet, with an overall width of 53 feet. There is no skew but the structure is curved to a radius of 2,200 feet. The bridge is constructed in five segments with four intermediate hinges. See Figure 10 for details of the span geometry.

The structure was under construction at the time of the 1971 San Fernando earthquake. The bottom slab and stem concrete had been placed from abutment 1 to the hinge of span 3 when this earlier earthquake occurred. (These spans subsequently collapsed in the Northridge earthquake of 1994.) Concrete from the hinge of span 9 to abutment 11 had been completely placed but the spans had not been prestressed. The remainder of the superstructure had not been constructed. Most of the damage to the superstructure was the result of falsework settlement.[9]

Reconstruction began in 1972. At the same time, Type 2 hinge restrainers were installed at hinge locations in spans 5 and 6. Type 1 hinge restrainers were added at the hinge located in span 9.

Ground Motion. This interchange is approximately two and one-half miles from the Gavin Canyon Undercrossing. The ground motions here can be assumed to be similar to those experienced at the Gavin Canyon site. However, one difference may be the effect of spatial variation in the ground motion for a viaduct of this length (in excess of 1,500 feet).

During the week immediately following the earthquake, several agencies recorded aftershocks in the area for the purpose of quantifying spatial effects. These agencies included the USGS, the California State Department of

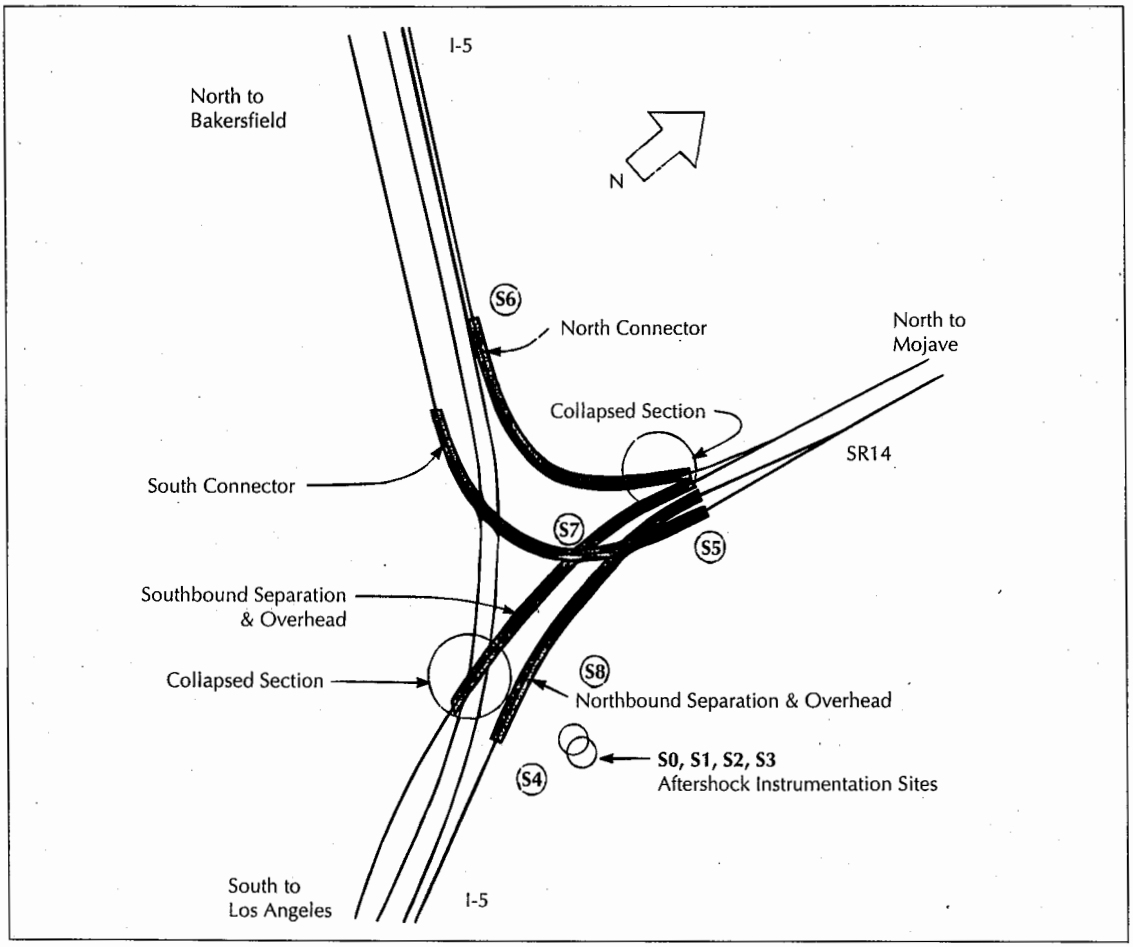

FIGURE 9. Simplified plan of the SR14/I-5 interchange and location of sites where aftershock ground motions were recorded.

Mines and Geology and the Lamont-Doherty Earth Observatory.

The Lamont-Doherty Earth Observatory deployment consisted of eight sites (denoted as circles in Figure 9) arranged in two lines at roughly a 35-degree angle. At each site, the ground acceleration in three orthogonal directions was digitally recorded at 200 samples per second. The stations were arranged at unequal increments along each line with the closest spacing being one meter and the largest around 750 meters. The sites were "hard" rock (conglomerate, sandstone and siltstone) or very shallow soils. Several hundred aftershocks were recorded, including several over magnitude four earthquakes.

An example of the spatial variability of ground motion recorded at this location is shown in Figures 11 and 12. Figure 11 shows the vertical components of P-Wave from a magnitude 4.7 aftershock recorded at four sites within 150 meters. Figure 12 shows the difference between site S0 and sites S2-4 (see Figure 9 for site locations). The first two cycles of P-Wave motion (5.5 to 6.1 seconds) are well correlated at all four sites with correspondingly small differential motions. After 6.1 seconds, the waveforms are much less correlated and differential motions increase. The differential motion increases with distance. The peak difference between sites S0 and S4 (150 meters apart) is twice the difference between S0 and S2 (10 meters apart). Furthermore, the peak differential motion can be larger than the peak ground motion and, in this example, it occurs later in the waveform.

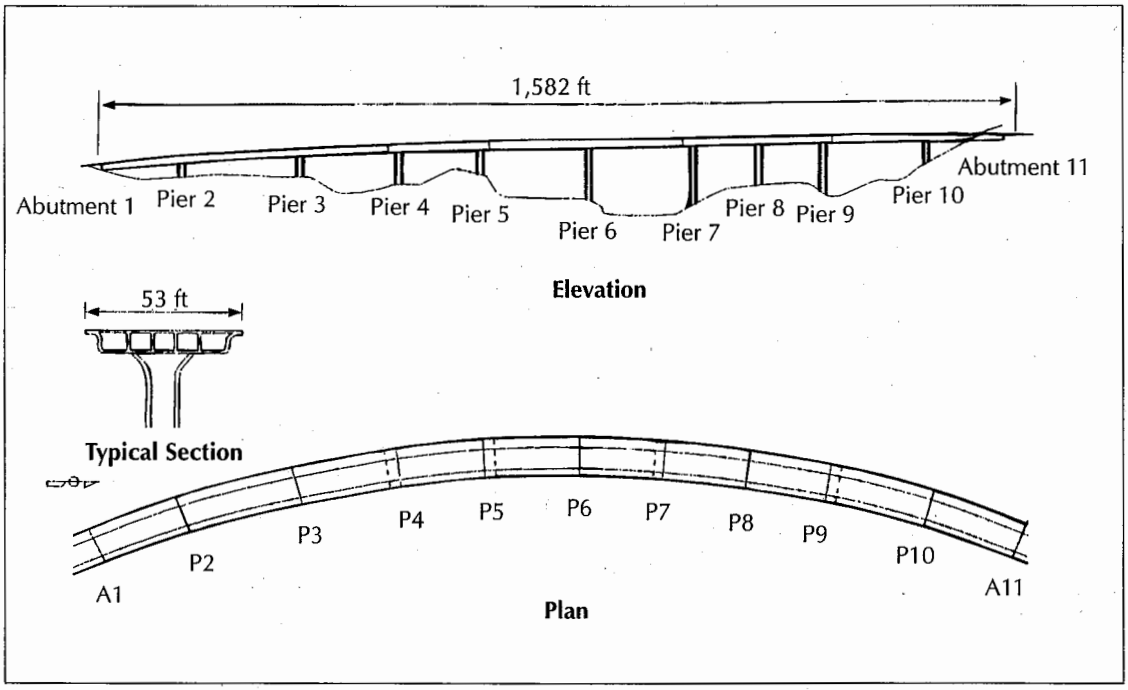

FIGURE 10. General plan and elevation for the SR14/I-5 separation and overhead (southbound).

The impact of these variations in ground motion on structural performance should be the subject of future research.

Observed Earthquake Damage. As shown in Figure 13, spans 1, 2 and 3 collapsed. In addition, pier 2 was completely crushed and pier 3 sheared through the superstructure. The mode of failure at pier 2 could not be determined by visual observation since it was completely crushed under the superstructure. The three spans that collapsed made up the first frame with piers 2 and 3. Span 1 was supported on a 24-inch seat abutment and frame 1 terminated at an in-span hinge approximately 22 feet from pier 4 in span 3. The in-span hinge seat was 14 inches wide and Type 1 restrainers tied frames 1 and 2 together. Restrainers were not used at either abutment.

The right exterior shear key at abutment 1 was damaged. However, a visual inspection of the left shear key revealed little damage. The superstructure at abutment 1 displaced approximately 10 feet up-station (north) and 10 feet right (east). The pier 3 bent cap was inclined towards pier 4 and the measured ground separation from the column at pier 4 was approximately six inches north-south and four inches east-west. At the in-span hinge 1, cantilever side, the transverse shear key was damaged. However, the shear key was still in place. At this same hinge, the equalizing bolts failed in tension and the nuts on the restrainer cable studs were missing. The vertical shear planes noted on the faces of pier 3 bent cap appeared to be similar to the web stem and soffit cracks documented at these locations after the 1971 San Fernando earthquake. The piers were not detailed as ductile elements. However, a visual inspection of piers 3 and 4 above grade did not indicate that there was any structural damage. The predominant motion of the structure appeared to be in the north-south direction.

Failure Analysis. There are three main possible failure mechanisms at the SR14/I-5 interchange:

1. Seat loss at in-span hinge 1 caused span 3 to collapse, which overloaded the bent cap and web interfaces at pier 3. The subsequent collapse of span 2 overloaded pier 2, which was followed by the collapse of span 1 and unseating at abutment 1.

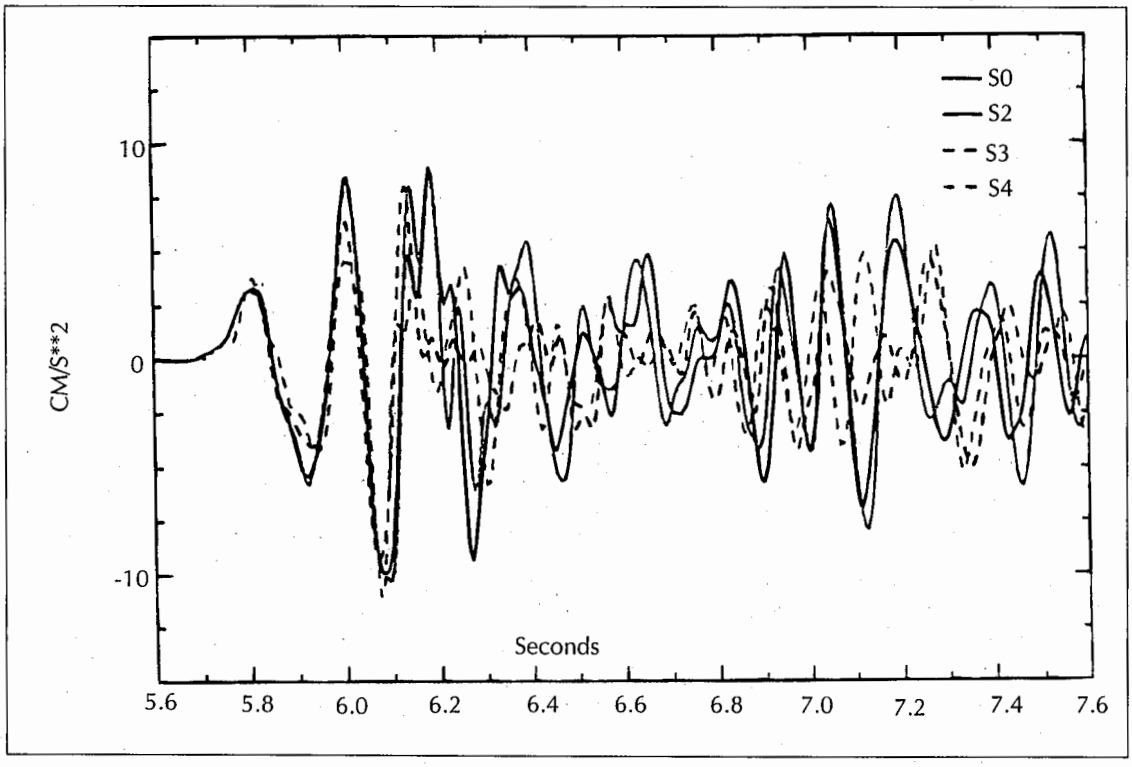

FIGURE 11. Vertical components of P-wave from a magnitude 4.7 aftershock recorded at four temporary sites (S0, S2-4) at the SR14/I-5 interchange.

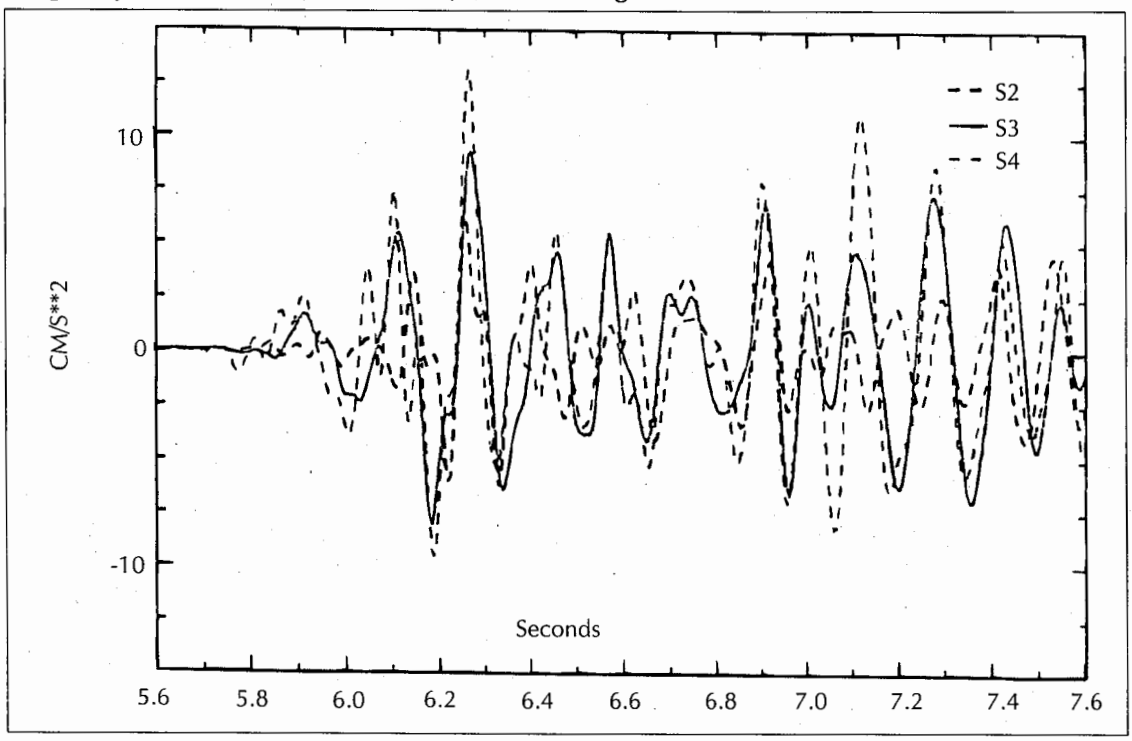

FIGURE 12. The difference between site S0 and sites S2-4 at the SR14/I-5 interchange.

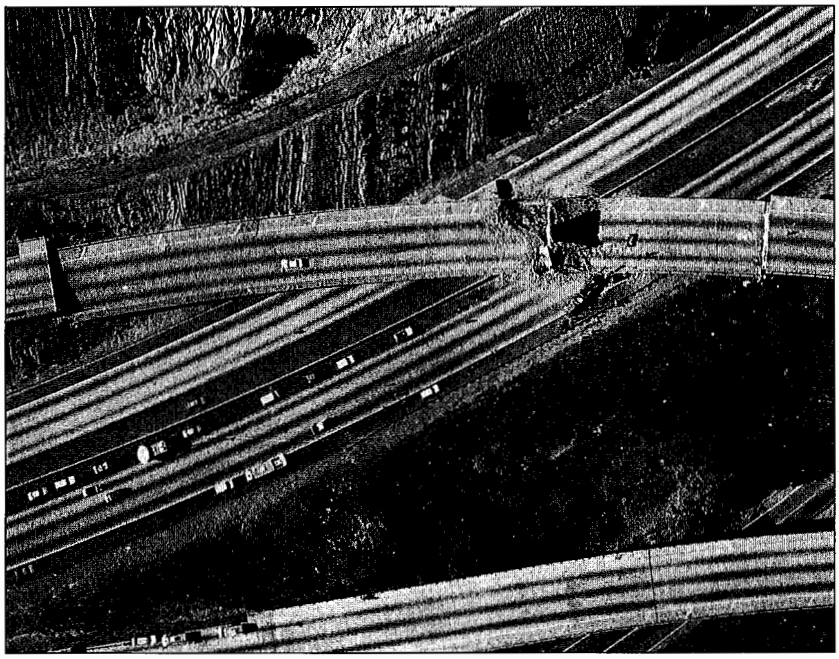

FIGURE 13. Aerial view of the collapse of the SR14/I-5 separation and overhead (southbound).

2. Shear failure at the interface between the web stems and pier 3 bent cap (towards pier 4) led to the collapse of span 3, and the unseating of span 3 at the in-span hinge. The subsequent collapse of span 2 caused the overloading and collapse of pier 2 followed by the collapse of span 1 and the unseating at abutment 1.

3. Non-ductile failure of pier 2 caused the collapse of spans 1 and 2, as well as the failure of the bent cap and web interfaces at pier 3 (with the subsequent collapse of span 3). However, the bent cap at pier 3 is inclined towards pier 4. Also, crushing of column concrete below the bent cap towards pier 4 and no crushing of column concrete below the bent cap towards pier 2 would indicate that perhaps span 3 collapsed first. Furthermore, span 2 was 206 feet long versus span 3 being 149 feet to the in-span hinge. Thus, if span 2 collapsed first, it would have pulled the pier 3 cap towards pier 2.

The circumstantial evidence available at the site, especially the damage sustained by the face of pier 3, suggests that the first mechanism cited above is the most likely failure mode.

Route 14/I-5 North Connector Overcrossing

Description. The SR 14/I-5 north connector overcrossing structure is located at mile post 24.92 on I-5 in Los Angeles County, approximately 24 miles to the northwest of downtown Los Angeles. The structure is curved, with a radius of 550 feet and a subtended angle of approximately 101 degrees. The structure carries traffic from southbound SR14 to northbound I-5, spanning I-5 (truck), Weldon Canyon Road, and Southern Pacific Railroad tracks. The SR14/I-5 interchange is located on steeply dipping, well consolidated sandstone of the Towsley formation. A general view and plan of the interchange are shown in Figures 8 and 9.

The north connector in the SR14/I-5 interchange is a 10-span, continuous, cast-in-place, three-cell, concrete box girder bridge. It has seat type abutments and single column bents supported on both spread footings and piled footings. The total length is 1,532 feet, with an overall width of 34 feet. The bridge is constructed in five segments with four intermediate hinges. See Figure 14 for details of the span geometry.

This bridge was under construction at the time of the 1971 San Fernando earthquake. Approximately 80 percent of the superstructure from abutment 1 to hinge 4 in span 8 was complete. During this earlier earthquake, the deck profile altered a total of 0.4 feet in a reasonably uniform change from abutment 1 to the hinge at pier 8. The hinges experienced the following minor damage: crushed expansion material, slight spalling, separated waterstops and failed equalizing bolts. Pier 2 exhibited signs of move-

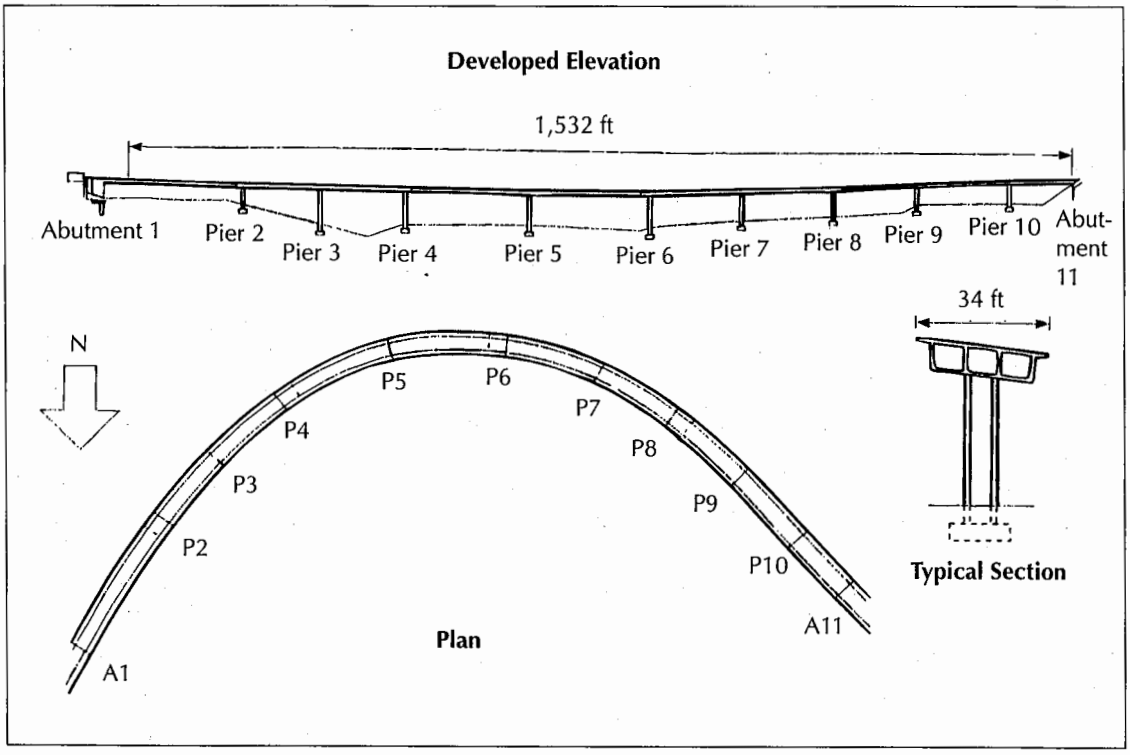

FIGURE 14. General plan and elevation for the SR14/I-5 north connector.

ment at the ground level having an embankment separation of two inches around the column. A crack was also noted on the superstructure soffit near pier 3.[9]

Reconstruction began in 1972 to build the substructures at pier 10 and abutment 11, and the superstructure between the hinge in span 8 to abutment 11. Type 1 hinge restrainers (four units per hinge) were added at the exterior cells. The construction of pier 10 incorporated double #4 spirals at a 3.5-inch pitch along the entire length of the column.

Ground Motion. This interchange is approximately two and one-half miles from the Gavin Canyon Undercrossing. The ground motions here can be assumed to be similar to those experienced at the Gavin Canyon site. However, one difference may be the effect of spatial variation in the ground motion for a viaduct of this length (in excess of 1,500 feet).

During the week immediately following the earthquake, several agencies recorded aftershocks in the area for the purpose of quantifying spatial effects (as noted in the discussion of the southbound interchange above).

Observed Earthquake Damage. As shown in Figures 15 and 16, spans 1 and 2 collapsed and pier 2 was completely crushed. The mode of failure for this pier could not be determined by visual observation. Span 1 was 188 feet long, and was simply supported at the seat abutment and at the in-span 1 hinge. The hinge and abutment seats were both 14 inches wide and hinge restrainers had been installed at the hinge. (Current Caltrans design specifications require that abutment seat minimum widths be 24 inches). The shear keys at abutment 1 were not severely damaged, indicating that motion was primarily in the longitudinal direction of the bridge at abutment 1 — *i.e.*, in a north-south direction (see Figure 16). Equalizing bolts at abutment 1 failed (see Figure 16). The collapsed pier 2 was supported on a spread footing and was approximately 21 feet high. The other two piers making up frame 1 were pier 3 (which was approximately 73 feet high and supported on CIDH piles) and pier 4 (which was approximately 60 feet high and supported on a spread footing). These pier columns were not detailed for ductile behavior since they utilized #4 ties at 12-inch centers.

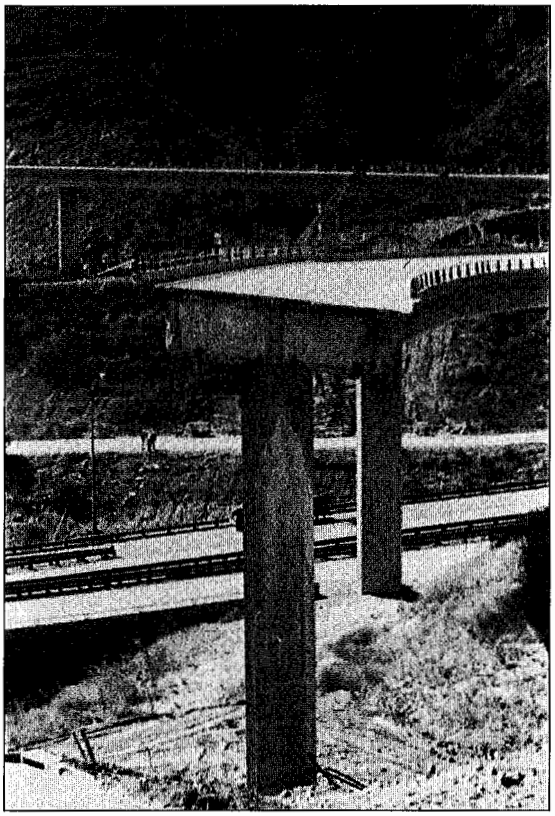

FIGURE 15. Collapsed end spans after demolition at the SR14/I-5 north connector.

Failure Analysis. Possible failure mechanisms appear to include:

1. Seat loss at abutment 1 that caused span 1 to collapse, with subsequent failure of pier 2 followed by collapse of span 2 to pier 3.
2. Non-ductile shear failure of pier 2, followed by the collapse of spans 1 and 2.

Without analysis, the sequence of failure is uncertain. However, it seems likely that pier 2 failed first and then was followed by the collapse of the two spans. It is possible that pier 2 attracted a much higher proportion of the lateral load than assumed in design because of its relatively short height. Furthermore, there was probably a significant reduction in the axial load in the pier due to both the severe curvature in the bridge and the high vertical accelerations in the ground motion. This reduction in axial load may have reduced the capacity of the column to less than the demand and, thus, led to the failure of the pier.

Bull Creek Canyon Channel Bridge

Description. This bridge carries 10 lanes of traffic on State Route 118 (SR118), as well as off ramps to Woodley Avenue, over a small drainage canal in the northern San Fernando Valley. It is located approximately one mile west of the interchange with I-405 and is adjacent to the Mission-Gothic Undercrossing, which suffered a partial collapse during the earthquake. The bridge was constructed in 1976.

The bridge is essentially two parallel structures separated by a longitudinal expansion joint that runs down the median of the freeway. The superstructure for each bridge consists of a three-span, multi-cell, cast-in-

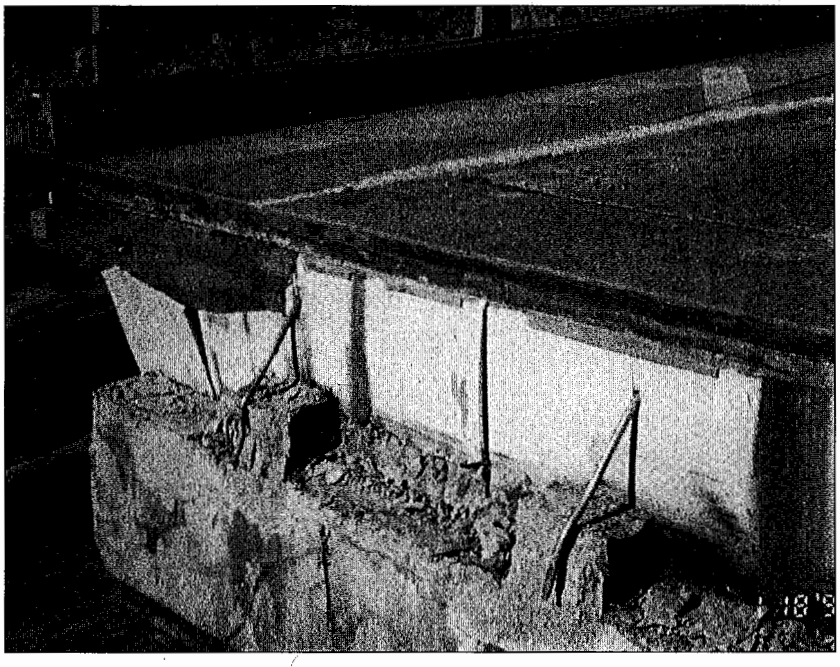

FIGURE 16. Seat and transverse shear keys at abutment 1, SR14/I-5 north connector.

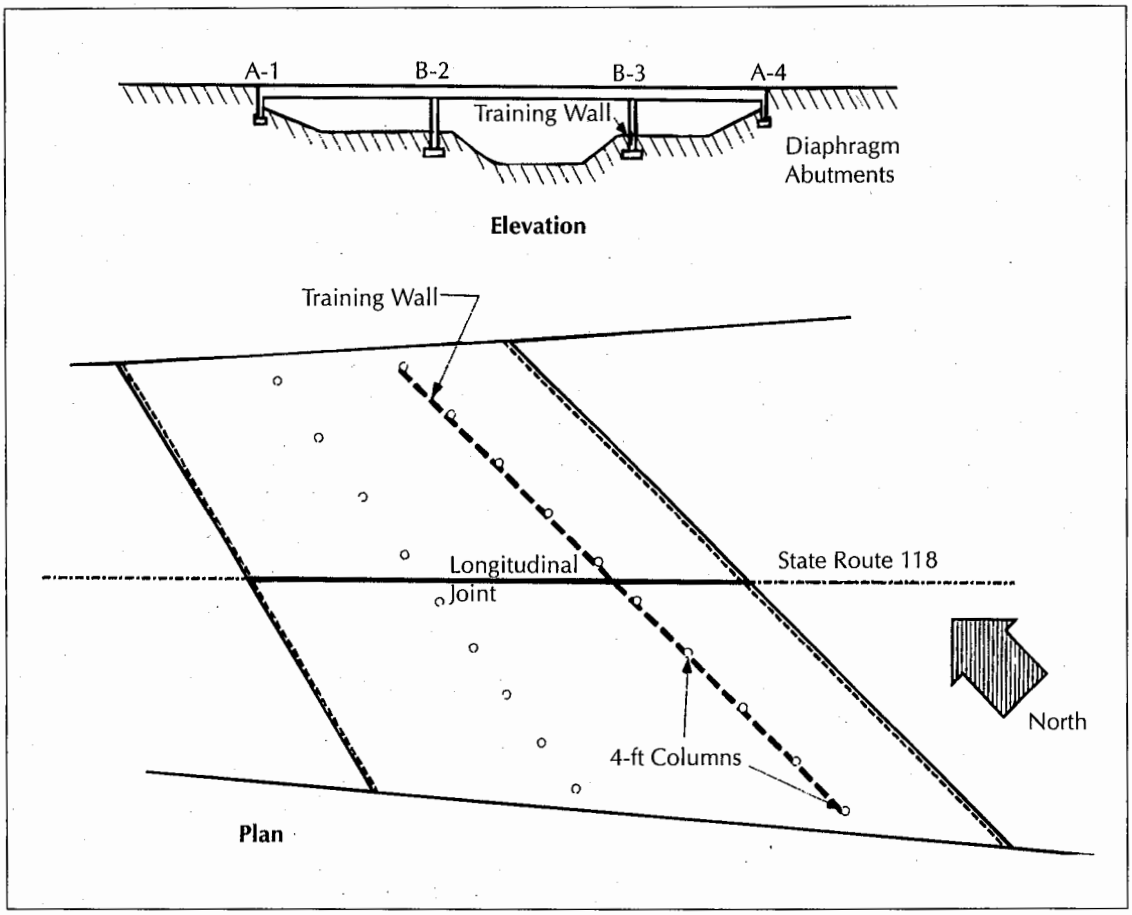

FIGURE 17. General plan and elevation of the Bull Creek Canyon Channel Bridge.

place prestressed concrete box girder that is typical of bridges built in California in the 1970s. At the center of the freeway, the three spans measure 90.5, 101 and 65 feet in length. Because of the off ramps, the bridge is considerably wider at the east end. A schematic of the bridge is shown in Figure 17.

The abutments and bents are not quite parallel and are laid out on skews that vary from approximately 37 to 47 degrees from normal. Due to this skew, the span lengths for the westbound bridge are slightly shorter than those on the eastbound bridge. Reinforced concrete columns, which are fixed against rotation at both the top and the bottom, are approximately 24 feet long and have a four-foot-wide regular octagon-shaped cross-section. Longitudinal reinforcement varies from 2.0 to 3.5 percent of the cross-sectional area. Transverse reinforcement consists of #5 spirals that are spaced at a pitch of three inches over a length of four feet just above the footings and just below the soffit. The remaining transverse reinforcement is spaced at a pitch of 12 inches. A transverse reinforced concrete training wall is located several feet above the footings between the columns in bent 3. Figure 18 shows column details.

Monolithic end diaphragm abutments are supported on greased pads that allow for prestress shortening and temperature movement. Transverse movement at the abutments is restrained by concrete shear keys built into the abutment footings. Abutment and bent footings are supported on 16- and 24-inch diameter cast-in-drilled-hole piles, respectively. Piles are approximately 40 feet long at the abutments and 30 feet long at the bents. The soil profile is alluvium consisting of slightly compact to dense silts, sands and gravels in excess of 90 feet deep.

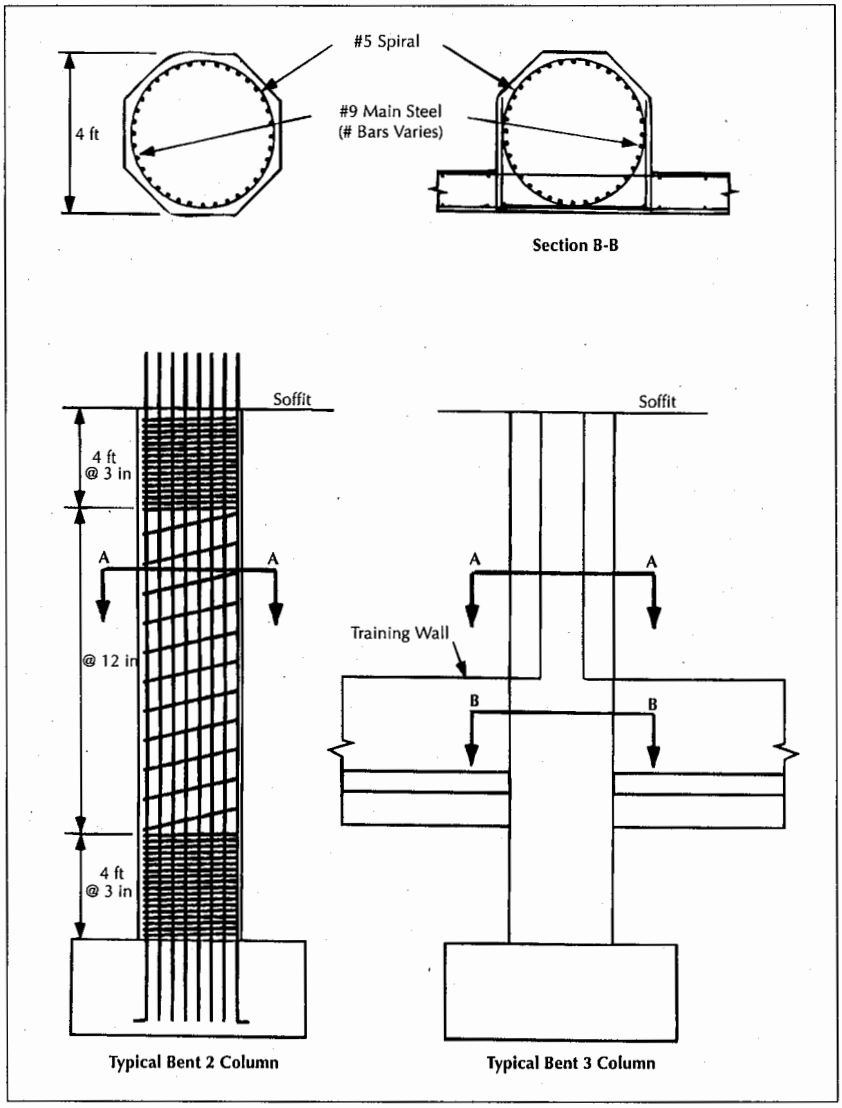

FIGURE 18. Column details for the Bull Creek Canyon Channel Bridge.

Ground Motion. This bridge site is located approximately five miles northeast of the epicenter of the earthquake. Strong-motion instruments that recorded the highest levels of motion during this earthquake are located within a north-south oriented band that passes through the epicenter and includes the region where this bridge is located. A large portion of the damage to manmade facilities, including several bridges on SR118 and I-5, was concentrated within this band.

The nearest two strong-motion instruments are each located on the edge of this hypothetical band approximately four miles southeast of the Bull Creek Canyon Channel Bridge. Both instruments recorded strong shaking that lasted approximately 10 seconds.[6]

A triaxial free-field instrument located at Arleta on deep alluvium recorded peak accelerations of 0.34 g in the east-west direction and 0.31 g in the north-south direction. These records have been processed and reveal five percent damped response spectra in both horizontal directions comparable to the Caltrans' smoothed elastic design spectra for peak rock accelerations between 0.2 and 0.3 g.[7,8] An unusual characteristic of the records obtained from this site was the relatively high peak accelerations of 0.55 g measured in the vertical direction.

A second instrument located at the base of a seven-story hotel in Van Nuys recorded peak accelerations of 0.47 g in the north-south direction and 0.41 g in the east-west direction. Peak vertical accelerations of 0.3 g obtained at this site are more typical of those recorded in past earthquakes, being approximately two-thirds of the peak horizontal accelerations.

It is speculated that the ground motions at the bridge were higher than those that were recorded by either of the above two instruments. This speculation is based on the extent of damage to other structures in the vicinity,

and on the location of the hypothetical band of strong motion mentioned above.

Observed Earthquake Damage. As shown in Figure 19, this bridge sustained irreparable damage to its columns during the earthquake. At bent 2, the two southernmost columns in the eastbound bridge failed just below the well confined section near the superstructure soffit. These failures caused the superstructure to drop several inches at these locations, resulting in cracking and spalling in the soffit near the location of the failed columns. Some remaining columns in the eastbound bridge exhibited diagonal cracking. Bent 2 columns on the westbound structure sustained far less damage.

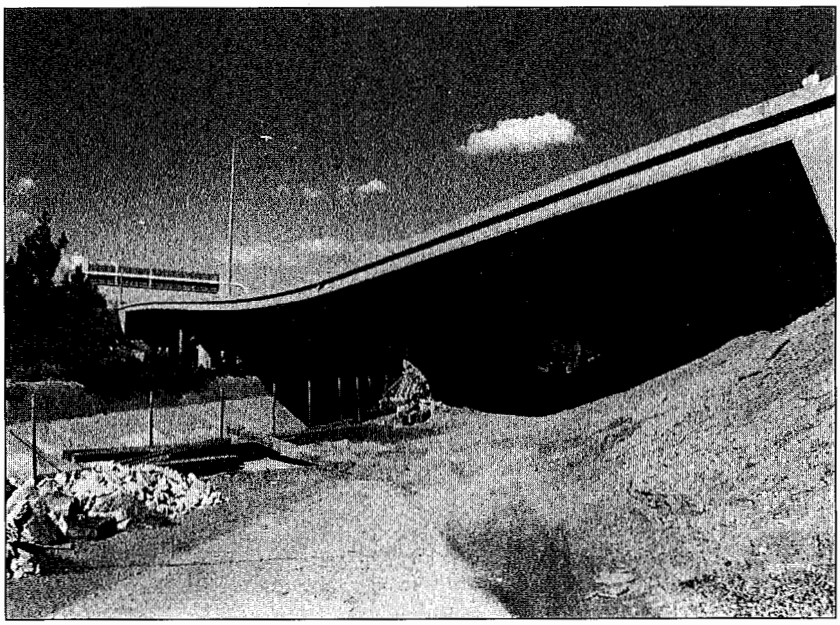

FIGURE 19. Side view of the Bull Creek Canyon Channel Bridge.

At bent 3 all columns in both bridges failed just above the training wall as shown in Figure 20. These failures occurred in an area above the well confined section of the column that was near the footing. Disintegration of these columns during the earthquake resulted in a significant drop in elevation at bent 3.

There was no visible sign of damage to the abutment, but there was an offset of the easterly approach pavement of approximately 15 inches – suggesting a failure of the buried transverse shear keys in the abutment footing.

Failure Analysis. The failures at bent 3 may be explained by the presence of the training wall that forced the plastic hinge to form at the top of the wall and, thus, above the confined section of the column. The training wall also shortened the effective height of the column, thus increasing the shear forces in the columns due to flexural demands. Some torsional shear forces were also induced because of the eccentricity of the training wall to the centerline of the column.

Bent 2 failures and possible abutment shear key failures were probably a result of the failed columns at bent 3 and the subsequent shifting of lateral forces to the remaining supports. The bent 2 column failures were essentially shear failures aggravated by the degradation of shear strength that resulted from flexural yielding. This conclusion is supported by the behavior of the various columns in bent 2.

The eastbound and westbound bridges are very similar in geometry. The tributary deadload for each of the columns in bent 2 is approximately equal, and the column heights and cross-sectional dimensions are essentially the same. Therefore, hypothetical elastic moment demands in each of the columns as a result of earthquake loading is expected to be similar. However, columns on the westbound bridge are more lightly reinforced than those on the eastbound structure, thus resulting in lower ultimate moment capacities and higher ductility demands.

On the other hand, ultimate shear forces, which are directly proportional to column flexural strength, are approximately 30 percent higher in the columns on the eastbound bridge. Since columns on this bridge failed while those on the westbound bridge did not (despite their potentially higher ductility demands), these failures resulted because of higher shear forces and are not solely due to flexural yielding in the

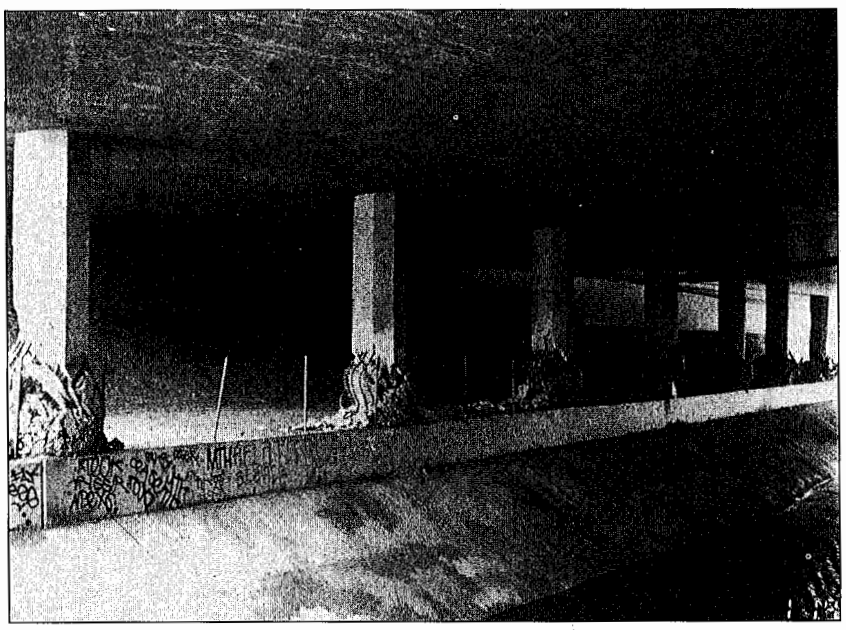

FIGURE 20. Hinges in all columns of bent 3 immediately above the channel training wall at the Bull Creek Canyon Channel Bridge.

region below the well confined zone of the column.

Issues/Questions. The column failures demonstrate the risks of attempting to optimize column designs by trying to predict the location of plastic hinging, even in relatively simple columns such as these. The placement of a training wall forced hinging to occur at a location unforeseen by the designer. Also, it does not appear that the columns were adequately reinforced for shear. Therefore, it would seem prudent to place closely spaced transverse reinforcement over the full length of the columns. It is relatively inexpensive to do so and it will help minimize the effects of unforeseen conditions.

As with several other bridge failures during this earthquake, the failure of the Bull Creek Canyon columns raises a question about the interaction between allowable ductility demands and the relative shear strength of the column. More research is needed in this area.

Retrofitted Bridge Performance

Many bridges in the epicentral region had been retrofitted with cable restrainers and a number of single column bents had been strengthened with steel jackets. Most of these retrofits performed adequately despite strong ground motion at some sites.

In some instances, cable restrainers lacked sufficient capacity to prevent the unseating of girders. Failures in the cables at the swaged fitting were common in these situations. In other instances, fastening details were shown to be inadequate and/or punching failure of the concrete diaphragm occurred. In at least two cases, anchorage nuts were found to be missing. It is not clear whether these nuts were left off during construction, worked loose over time or stripped off during the earthquake. In most cases, these failures were in units with details that had since been superseded by Caltrans.

It is also clear that many restrainers worked as expected and significantly reduced the number of collapsed spans. All of the columns with steel jackets appeared to perform without distress even though some were installed on bridges close to other structures that did partially collapse.

Other Bridge Damage

As of February 1, 1994, there were 176 bridges noted to have been damaged from the earthquake and aftershocks. Damage ranged from minor spalling to collapsed spans. Some examples of the damage include:

- *I-5 at the San Fernando Road Undercrossings:* Abutment and wing wall damage and minor column spalling
- *Southwest Connector at the I-5/Route 118 Interchange:* Column shear cracks
- *I-5 at the Santa Clara River Bridge:* a steel plate-girder bridge with sheared anchor bolts and failed cable restrainers

- *Route 101 at Los Virgenes:* Pile damage
- *I-405 at the Jefferson Boulevard Undercrossing:* Outrigger joint cracking
- *The South Connector in the interchange between I-5 and SR14* suffered severe pounding at the hinge seats and substantial damage to abutment 10. Structural damage at the hinge seats appeared to be more severe on the inside face of the curved girder, indicating greater movements in the radially outwards direction than in the opposite direction.
- Abutment fills slumped behind the back walls of many bridges (for example, at the Bull Creek and Havenhurst Bridges on SR118). In instances where approach slabs were used (and tied to the back walls), access was not impaired. However, slumping in the shoulders and emergency stopping lanes will require repair.

In addition, the city of Los Angeles reported several instances of approach fill settlement, abutment damage, and bearing and shear key damage. The Southern Pacific Railway reported inconsequential damage to bridge structures.

Conclusions & Recommendations

The performance of bridges during the Northridge earthquake has shown that moden bridges designed to current codes performed well. Bridges designed to earlier codes suffered damage. An accelerated retrofit program is necessary for these older bridges.

On the whole, this earthquake showed that bridge retrofit programs are effective, but that priorization algorithms for bridge retrofit should be reexamined. One common failure that occurred on retrofitted bridges was with cable restrainers. Most restrainer failures occurred either because they were based on a design that has since been superseded by Caltrans or were forced to carry the weight of several spans of the bridge due to the collapse of nearby columns. However, some restrainers might have failed due to improper installation. In at least one case, the nuts on several restrainer cable studs were found to be missing and there was no evidence of stripped threads. Column jackets appeared to work well and none exhibited signs of damage or distress despite strong ground shaking nearby.

Prioritization algorithms for bridge retrofit need to be reexamined. At least one bridge that partially collapsed would probably pass current screening procedures and not be identified as vulnerable. Structural attributes such as skew and the unintended participation of nonstructural elements (for example, walls and flares) need to be further addressed. Multicolumn bents should also be elevated in current priority ranking procedures.

Other observations and recommendations include:

- The assessment of bridge vulnerabilities should not overlook the vulnerabilities of co-located pipelines.
- Abutments and internal hinge seats must be generously proportioned to accommodate large relative movements in flexible structures.
- The combination of high vertical ground accelerations in bridges with high curvature may significantly decrease column axial loads and adversely affect shear capacities.
- Approach slabs that are tied to abutment back walls can successfully bridge slumped fills behind those walls and provide continued access.

ACKNOWLEDGMENTS — *This article is an abridged version of a report, "The Northridge, California Earthquake of January 17, 1994: Performance of Highway Bridges," edited by Ian G. Buckle and published by the National Center for Earthquake Engineering Research (NCEER) as Technical Report NCEER-94-0008. The subject was presented as a lecture to the Boston Society of Civil Engineers in the fall of 1994. Except where otherwise acknowledged, the material presented in the report was obtained during the reconnaissance efforts of several NCEER-sponsored teams in the Los Angeles area shortly after the earthquake occurred. Sponsored by the Federal Highway Administration through NCEER, Chris Rojahn of the Applied Technology Council (ATC) organized a team which consisted of Ronald Mayes (Team Leader), Computech Engineering Services; Bruce Douglas, University of Nevada at Reno; Richard Nutt, Structural Engineer,*

Sacramento; and Stephen Thoman, CH2M Hill. At the same time, Steven Horton and Noel Barstow from the Lamont-Doherty Earth Observatory recorded aftershocks at the site of the SR14/I-5 interchange. Ian Buckle, from the State University of New York, also visited the region and compiled the report based on the observations, records and analyses of the various team members. The generous assistance of Caltrans to these reconnaissance efforts is greatly appreciated. The sections on Seismology and Geotechnical Aspects contain contributions from James Mori, USGS; Ken Campbell, EQE International; George Gazetas, State University of New York at Buffalo; and Mishac Yegian, Northeastern University. Most of the photographs and illustrations in this article were either provided by the reconnaissance team members or prepared by NCEER staff. However, Figures 2 and 3 were provided by the University of Southern California; Figures 6 and 13 were made available by Caltrans; and, Figures 7 and 20 were from the Earthquake Engineering Research Center at the University of California at Berkeley. A copy of the full report can be obtained from NCEER, State University of New York at Buffalo, Red Jacket Quadrangle, Box 610025, Buffalo, NY 14261. BSCES acknowledges NCEER staff Ian Buckle, Deputy Director, and Jane Stoyle, Publications Manager, for graciously permitting us to publish this abridged report.

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