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The Northridge, California Earthquake of January 17, 1994: Performance of Highway Bridges

by

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NOTICE

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ABSTRACT

On January 17, 1994 at 4:31 a.m., a magnitude 6.6 earthquake struck the Los Angeles metropolitan area. Epicentered in the San Fernando Valley town of Northridge, California, the earthquake caused serious damage to buildings and sections of elevated freeways; ignited at least one hundred fires as it ruptured gas pipelines; and disrupted water supply systems. As a consequence, 57 people died, another 1,500 were seriously injured, and 22,000 were left homeless. Over 3,000 buildings, most of which were residential structures, were declared unsafe for reentry due to earthquake damage. Los Angeles, a city which has extensively prepared itself for earthquakes, found that it had experienced the most destructive event since the 1906 San Francisco earthquake. Direct economic losses are estimated currently at over $20 billion.

This reconnaissance report provides an analysis of major bridge damage which occurred during the earthquake. Eight highway bridges are described in terms of their geometry, site ground motions, observed damage and likely failure modes.

This report is one of three NCEER reports resulting from reconnaissance activities following the Northridge, California earthquake. The other two reports are: **The Northridge, California Earthquake of January 17, 1994: General Reconnaissance Report** and **The Northridge, California Earthquake of January 17, 1994: Performance of Gas Transmission Pipelines**.

ACKNOWLEDGMENTS

Except where otherwise acknowledged, the material presented in this 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 Lamont-Doherty Earth Observatory recorded aftershocks at the site of the Route 14/I-5 interchange. Ian Buckle from the State University of New York also visited the region and compiled this report based on the observations, records and analyses of the various team members. The generous assistance of the California Department of Transportation (Caltrans) to these reconnaissance efforts is greatly appreciated.

Section 2, Seismology and Geotechnical Aspects, contains contributions from James Mori, U.S. Geological Survey; Ken Campbell, EQE International; George Gazetas, University at Buffalo; and Mishac Yegian, Northeastern University.

Most of the photographs and drawings used in this report were either provided by the reconnaissance team members or prepared by NCEER staff. However, Figures 2-1 and 2-2 were provided by the University of Southern California, Figures 3-6, 6-5 and 9-3 were obtained from the Earthquake Engineering Research Center at UC Berkeley (EERC) and Figures 3-4, 3-5, 4-7 , 9-4, 9-5, 9-7, 9-8, 9-9, 10-8, 10-9, 10-10, 10-11, 10-12, 10-13, 10-14, 10-15, 10-16, and Table 12-I were made available by Caltrans.

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SECTION 1
INTRODUCTION

During the Northridge earthquake of January 17 in Los Angeles, California, 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 but some were closed permanently and have since been demolished pending replacement. Of those bridges with collapsed spans, all were designed and constructed from the mid-sixties to the mid-seventies. 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.

This report contains a detailed summary of the performance of eight bridges that suffered major damage. These bridges are as follows:

1. Gavin Canyon Undercrossing
2. Route SR14/I-5 Separation and Overhead (Southbound)
3. Route SR14/I-5 North Connector Overcrossing
4. Bull Creek Canyon Channel Bridge
5. Mission-Gothic Undercrossing
6. Balboa Boulevard Overcrossing
7. Fairfax-Washington Undercrossing
8. La Cienega-Venice Undercrossing

Figure 1-1 shows the location of these bridges relative to the epicenter of the earthquake in Northridge [1].

Damage 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-column bents, columns with flares and other accidental restraints, and columns in skewed bridges.

The performance of buildings and lifeline systems is described in a companion NCEER report entitled **The Northridge, California Earthquake of January 17, 1994: General Reconnaissance Report [1]**.

v

FIGURE 1-1 Location Map of Bridges with Major Damage

SECTION 2
SEISMOLOGY AND GEOTECHNICAL ASPECTS

2.1 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 35 km northwest of the Los Angeles central business district in Southern California (34° 12.7'N 118° 32.3', depth 18km). The preliminary moment estimate determined from regional surface waves and teleseismic recordings was 1 to 1.5 X 10^{26} dyne-cm, which gives a moment magnitude of M_w=6.7. The preliminary local magnitude determined from telemetered strong-motion instruments in Southern California was M_L=6.4. Both the first-motion focal mechanism from local stations and the teleseismic mechanism show a thrust fault on a plane trending in a northwest direction. The pattern of aftershocks reveals that the plane dipping toward the southwest is the fault plane, and its dip is 40° to 50°.

A contour map of the maximum component of peak horizontal acceleration for the San Fernando Valley and Los Angeles Basin developed by Todorovska and others [2] is shown in Figure 2-1. A similar map for the vertical component is shown in Figure 2-2. These maps were constructed by the University of Southern California (USC) using data from the Los Angeles Strong Motion Accelerograph Network. Other sources included the California Strong Motion Instrumentation Program (CSMIP) and U.S. 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 0.5g to 1.0g in the aftershock area and decreased to 0.1g at distances of about 50km. Several sites close to the epicentral area recorded accelerations over 1g. The extensive damage caused by this earthquake emphasizes the need for better understanding of 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 Route 14/I-5 interchange, Route 118 near Woodley, and the I-10 freeway near La Cienega Boulevard.

Thousands of aftershocks occurred in the two month period following the earthquake including six M5, forty-three M4 and 284 M3 events as of March 15. The locations of the aftershocks are distributed across an area about 30 x 20km. These locations are clearly deeper toward the south and in cross-section reveal 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 18km upward toward the surface. Preliminary analysis of teleseismic data indicates that most of the slip on the fault plane occurred at depths below 5 to 10km with relatively little slip of the shallow portions of the fault.

The location of the fault plane, as inferred from the aftershock distribution, does 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.

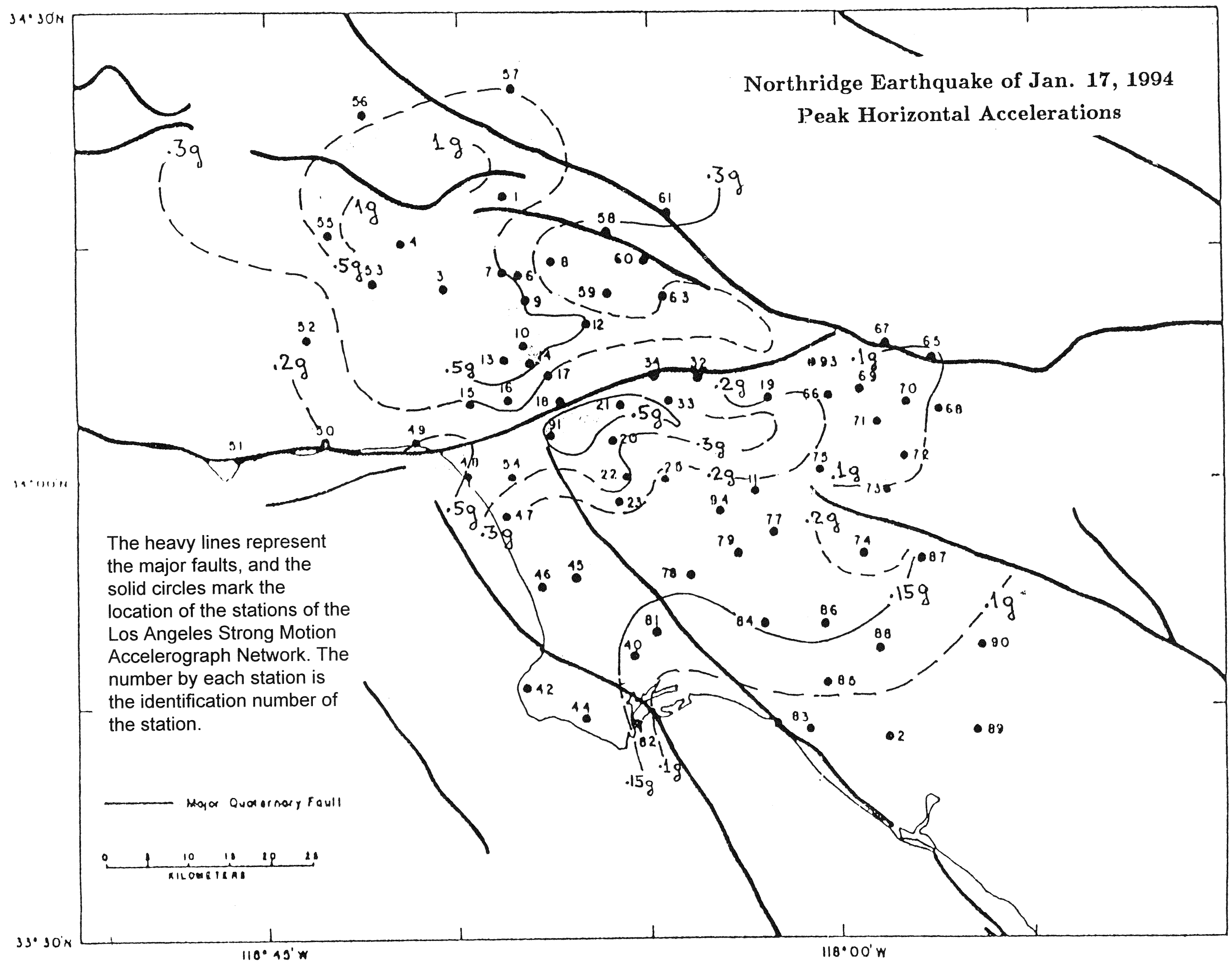


FIGURE 2-1 **Contours of Peak Horizontal Ground Acceleration Observed During the Northridge Earthquake, Expressed as a Fraction of the Acceleration of Gravity [2] (as of February 1, 1994)**

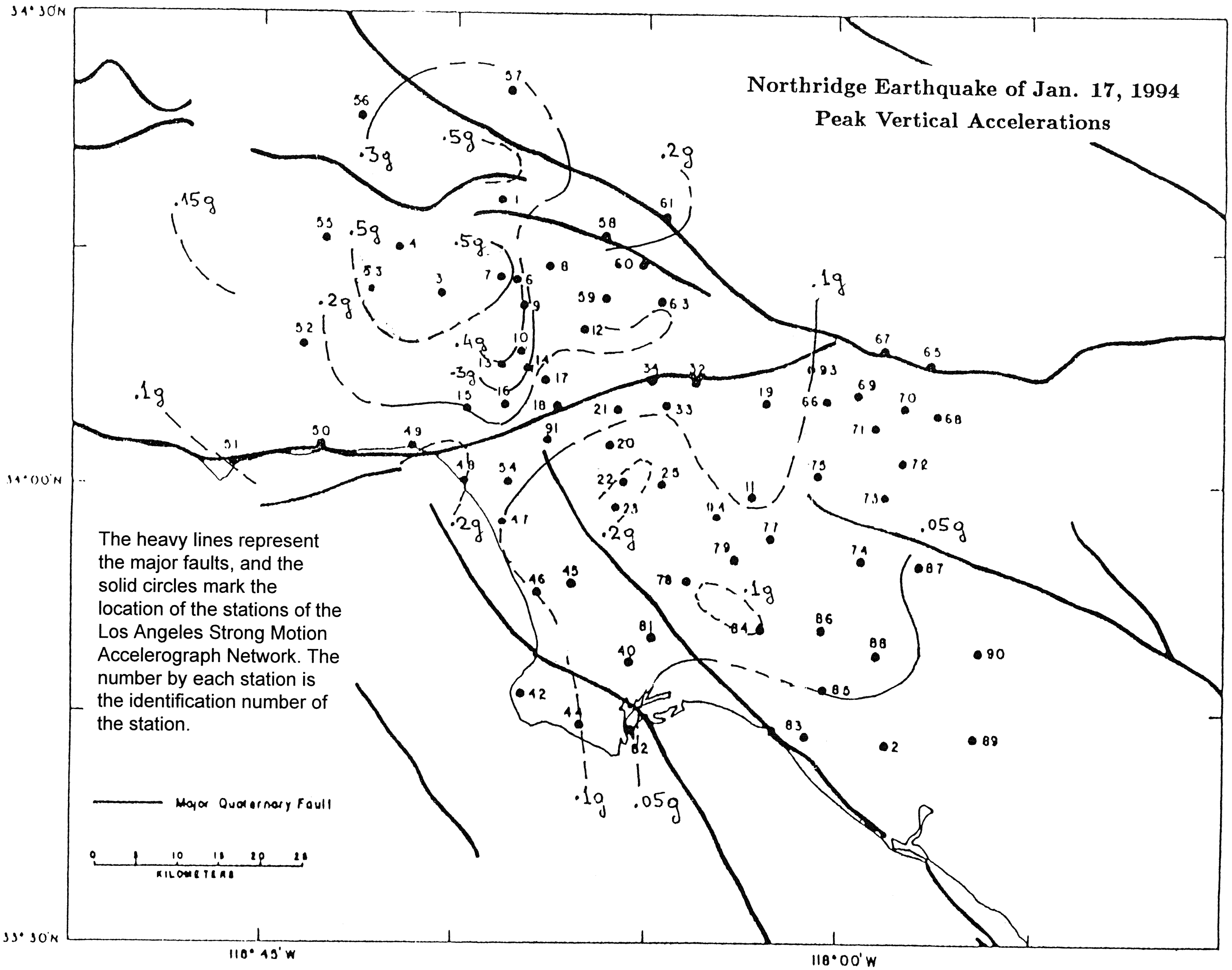


FIGURE 2-2 **Contours of Peak Vertical Ground Acceleration Observed During the Northridge Earthquake, Expressed as a Fraction of the Acceleration of Gravity [2] (as of February 1, 1994)**

Many of the aftershocks are located on or close to the rupture plane. However, there are also many off-fault events that have a variety of focal mechanisms. One example is the M5.1 earthquake of January 29 which was a shallow strike-slip event above the main rupture plane. Portable instruments recorded accelerations of up to 0.8g from this earthquake.

Real-time information about the mainshock and aftershocks were broadcast to 15 members 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, U.S. Geological Survey and Lamont-Doherty Earth Observatory (LDEO) personnel. Some of the LDEO data are presented in Section 4.2.

2.2 Geodetic Observations

Results from re-surveys of benchmarks, using the Global Positioning System (GPS), reveal the static displacements due to the earthquake at 15 sites. In the aftershock region, there were vertical uplifts of 40 to 50cm and horizontal motions of 2 to 20cm. These movements are consistent with the fault geometry derived from seismological observations of a plane dipping toward the southwest at about 40°. Preliminary modeling of the data indicate that there was a slip of 2.5 to 3.5 meters on a 10 x 10km patch of the fault. The motion was primarily thrust faulting, and most of the slip occurred at depths of greater than 6km.

2.3 Geological Observations

Two areas of surface cracking observed immediately after the earthquake are being studied. It is unclear if these cracks are direct results of tectonic faulting or due to ground shaking. The small amount of observed surface cracking, however, is consistent with the geodetic results 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 60cm were observed on both the north and south sides of the canyon extending for about 3km. Evidence for compressional features with vertical displacements of 8 to 20cm were also observed along the south margin of the canyon. None of the deformations was associated with any previously mapped surface fault.

A second system of small cracks was studied along a 5km 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 landslide occurrences in the younger sediments of the western Santa Susana Mountains, Oak Ridge and Big Mountain areas. Rock falls have choked the ravine bottoms of many canyons in the Santa Susana Mountains. These were of some concern following the earthquake since heavy rains could saturate the material, causing it to mobilize into debris flows that threaten structures near the mouths of the canyons.

2.4 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. Effects on buildings and buried lifeline systems are described in [1].

SECTION 3
GAVIN CANYON UNDERCROSSING -
BRIDGE NUMBER 53-1797R & L

3.1 Description

This undercrossing carries the north and south bound lanes of Interstate 5 over Gavin Canyon Road using two separate bridges. Interstate 5 is the main link between Northern and Southern California. The bridges are located approximately 2-1/2 miles northwest of the I-5/SR14 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 8 inch bearing seats. Span lengths on the northbound bridge are 120.0, 170.0, 208.0, 145.0 and 98.0 feet. The southbound bridge has span lengths of 128.0, 170.0, 208.0, 145.0 and 90.0 feet. Both bridges are 68.0 feet wide.

The two outside frames are of cast-in-place reinforced concrete box girder construction, each 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 42.0 foot wide median. A schematic of the bridges is shown in Figure 3-1.

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 supported on spread footings located in the approach fills.

Reinforced concrete columns measure 6.25 ft x 10.0 ft in cross-section at the base and have large, but lightly reinforced, architectural flares that widen out to over 20 ft at the soffit. All columns are poorly confined with nominal 1/2 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 73.0 feet in length. Column details are shown in Figure 3-2.

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 1-3/4 inches and damage to barrier rails at the hinges indicated a 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].

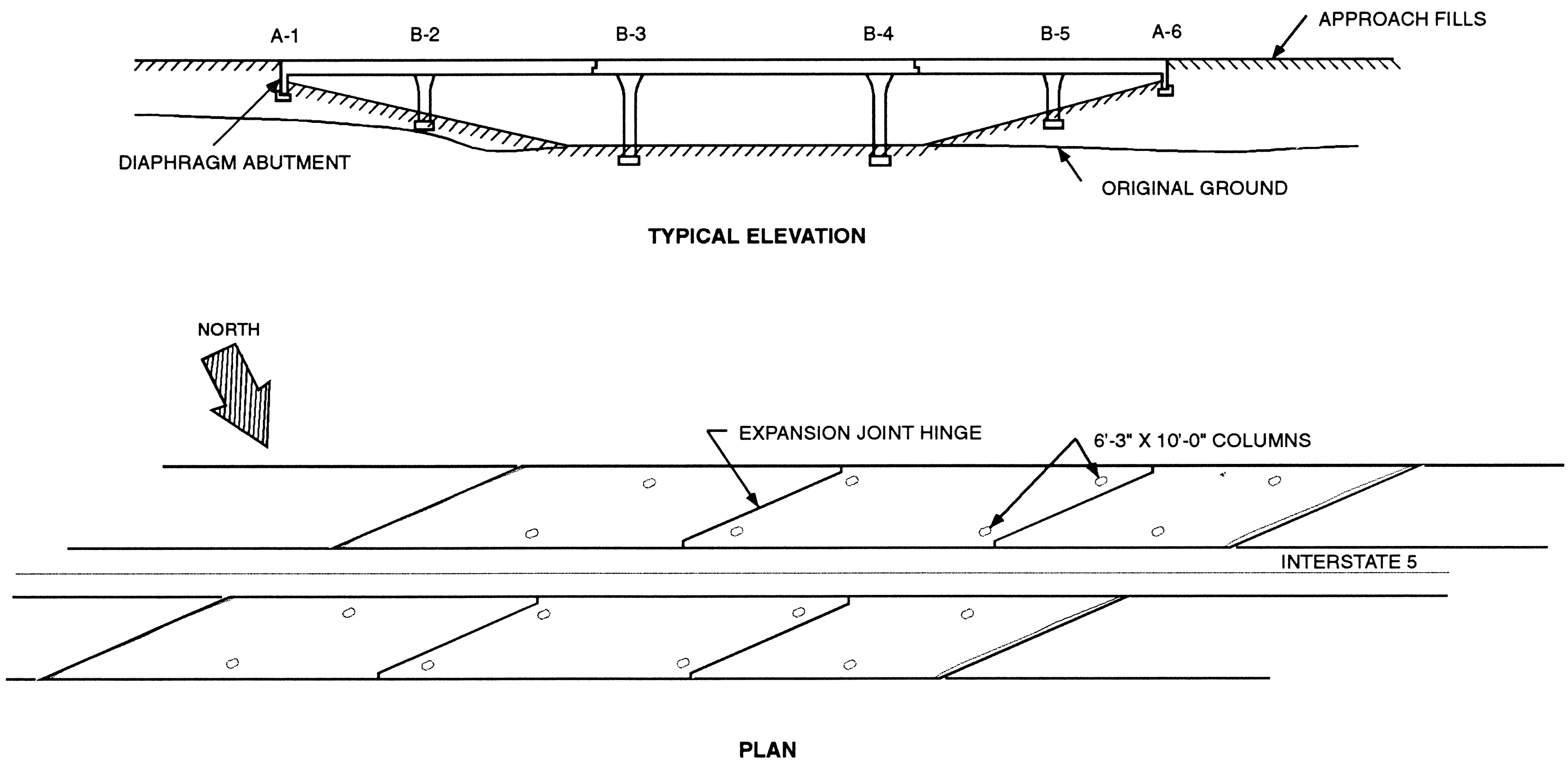


FIGURE 3-1 Gavin Canyon Undercrossing - General Plan and Elevation

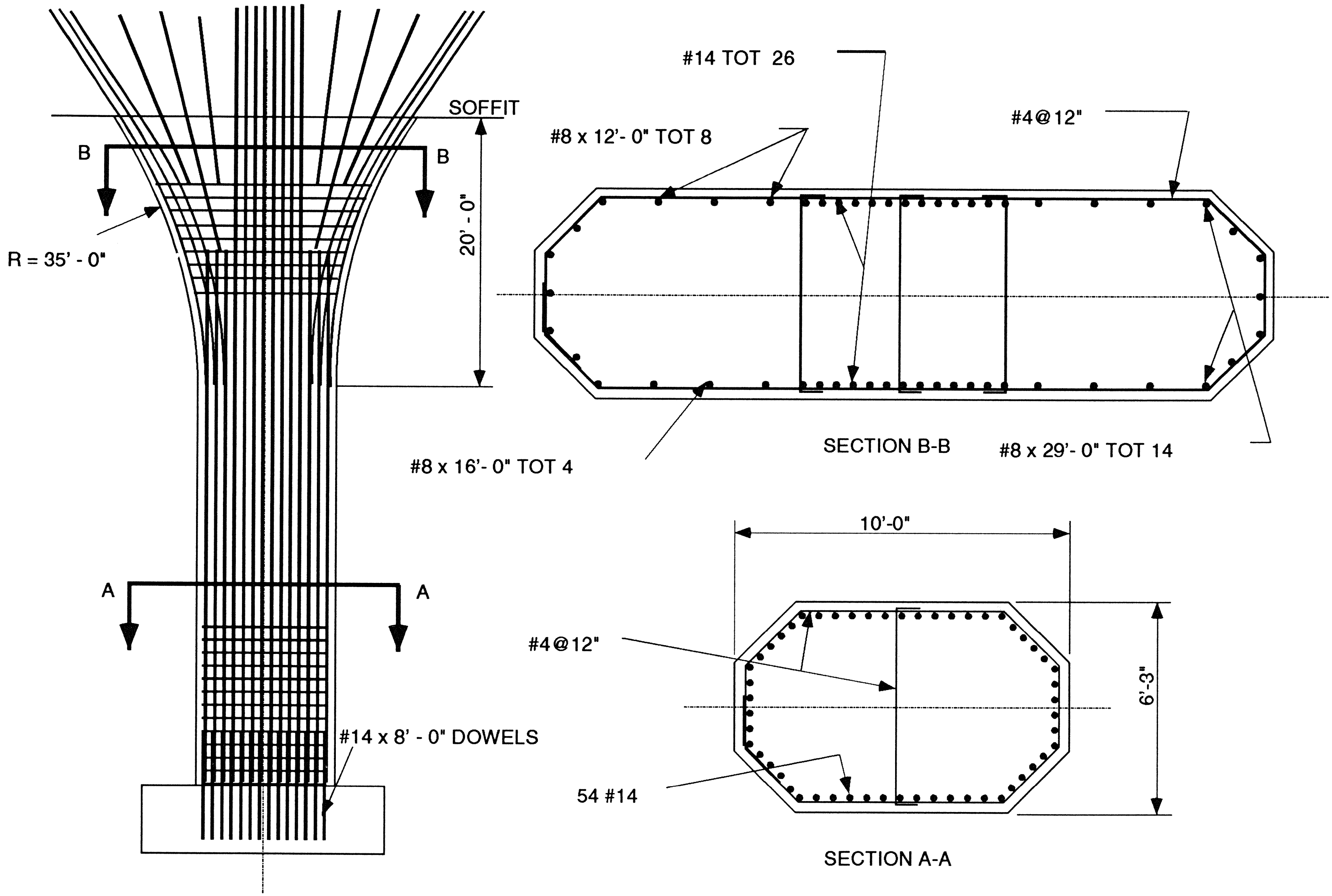


FIGURE 3-2 Gavin Canyon Undercrossing - Column Details

Expansion joint hinge restrainers were not present during the 1971 earthquake. They were placed after the earthquake and are unusual by modern standards in that they are oriented along the centerline of the bridge rather than normal to the expansion joints. Restrainer details are shown in Figure 3-3.

3.2 Ground Motion

Ground shaking 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. This and other preliminary seismological analysis indicates the bridge site is near the surface projection of the Oak Ridge - "Newhall" fault which 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 Sta. No. 24279, a free field instrument located approximately 2-1/2 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 recorded in all three directions. The motions for the peak horizontal accelerations were nearly in phase indicating 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 (CSMIP Sta. No. 24514) 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 indicate 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 Caltrans' design spectrum for the bridge site for the period range between 1/2 to 2 seconds. (Caltrans uses a smoothed elastic design spectrum based on average motions for the "maximum credible earthquake" on the most critical known fault [8].)

3.3 Observed Earthquake Damage

The ATC/NCEER reconnaissance team did not visit this site because demolition of the unstable side spans had progressed to the point that collapse evidence had been destroyed and little would have been gained from a site visit. However, other teams from Caltrans and the University of California at Berkeley were on the scene earlier, and this description of damage is drawn from their observations and an interpretation of available photographs.

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 the structural sections about a vertical axis (Figure 3-4).

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 while in other cases they pulled through the expansion joint diaphragm (Figure 3-5). Some cables remained intact

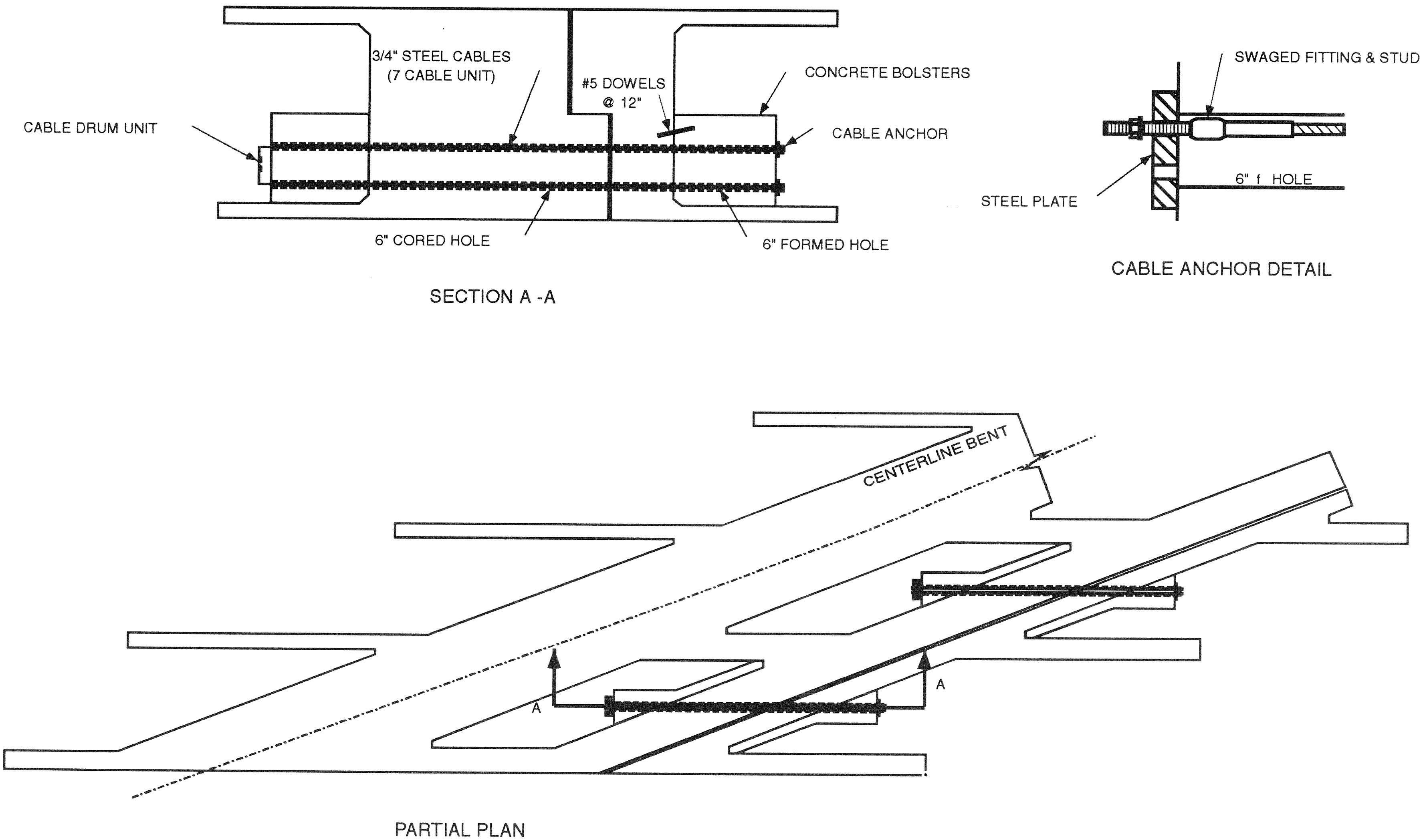


FIGURE 3-3 Gavin Canyon Undercrossing - Hinge Restrainer Details

FIGURE 3-4 Gavin Canyon Undercrossing - Aerial View of Collapse

helping to support the partially unseated spans and preventing 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 3-6. 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.

3.4 Failure Analysis

The failure of this bridge is attributed to large lateral and rotational movements of the superstructure sections which 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 which tended to be shifted toward the rather stiff diaphragm abutments. It should be noted that 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 rotational response of the end spans.

An additional factor in the response of the structure is the large difference in the vibrational characteristics of the center frame compared to the two end frames. The center frame is much more flexible and has a natural period of vibration that is much longer than the end frames. Calculated relative displacements between the two frames based on a 0.6g Caltrans design spectra loading exceed 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.

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 footings remained fixed against rotation. The satisfactory performance of the columns at bents 3 and 4 is attributable to the relatively low shear forces resulting from their long length and 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 rocking of the footings. Calculations show that these footings are unable to develop the ultimate moment capacity of the column and most likely rocked during the earthquake thus preventing the development of high shear forces. However, such rocking most likely aggravated the relative superstructure displacements at the hinges.

3.5 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 (Sylmar, 1971), it does not necessarily mean it is immune to damage in future earthquakes.

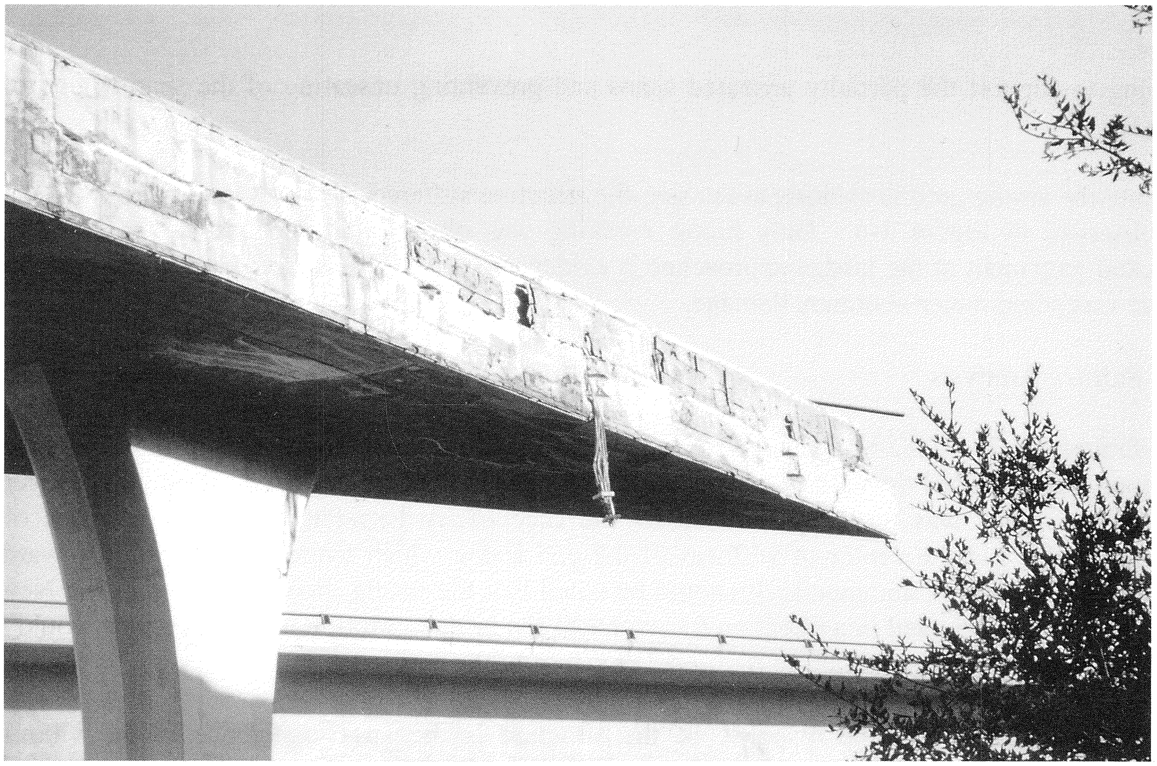


FIGURE 3-5 Gavin Canyon Undercrossing - Hinge Seat and Failed Cable Restrainer

FIGURE 3-6 Gavin Canyon Undercrossing - Collapsed End Spans After Demolition

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. 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.

SECTION 4
ROUTE 14/I-5 SEPARATION AND OVERHEAD (SOUTHBOUND) - BRIDGE NUMBER 53-1960F

4.1 Description

The southbound SR14/I-5 separation and overhead structure is located at mile post 24.5 on Route 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 Route 14 to southbound Route 5 spanning the Route 5 southbound and northbound mainlines. Route 5 is a U.S. Interstate running north-south along the west coast and Route 14 is a State highway beginning at the SR14/I-5 interchange and running northeast to U.S. Route 395 in Kern County.

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 4-1. A simplified plan view of this interchange is given in Figure 4-2. In this view, the four viaducts which 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 and is not discussed further in this report.

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 1582 feet with an overall width of 53 feet. There is no skew but the structure is curved to a radius of 2200 feet. The bridge is constructed in five segments with four intermediate hinges. See Figure 4-3 for details of 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 was completely placed but the spans had not yet 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. (See Figure 4-4 for typical restrainer details).

4.2 Ground Motion

This interchange is approximately 2-1/2 miles from the Gavin Canyon Undercrossing. The ground motions here can be assumed to be similar to those experienced at the Gavin Canyon site (Section 3.2). However, one difference may be the effect of spatial variation in the ground motion for a viaduct of this length (in excess of 1500 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 U.S. Geological

FIGURE 4-1 SR14/I-5 Interchange -
General View from Abutment 10 of the South Connector Looking Southwest

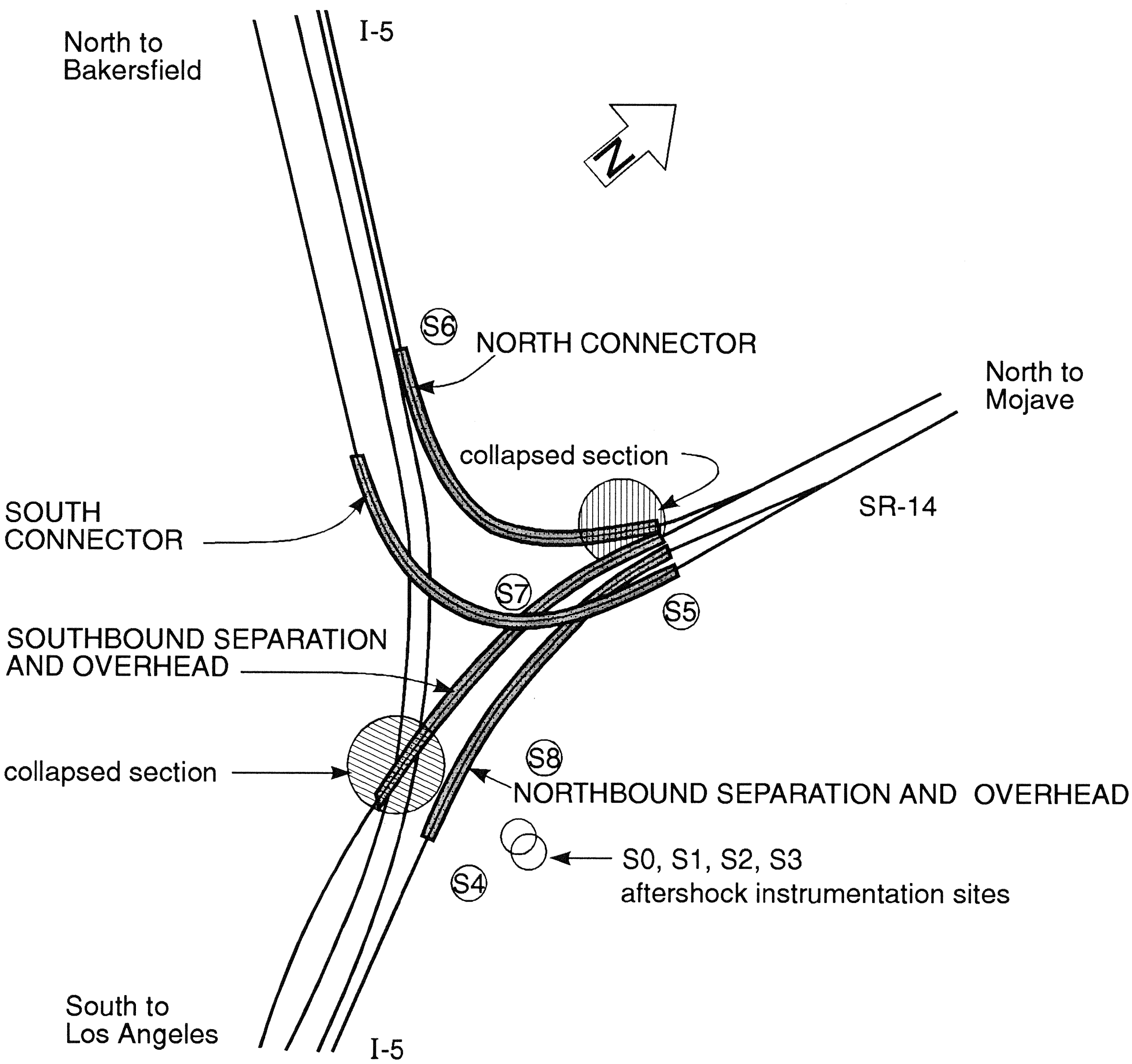


FIGURE 4-2 SR14/I-5 Interchange - Simplified Plan and Location of Sites Where Aftershock Ground Motions Were Recorded

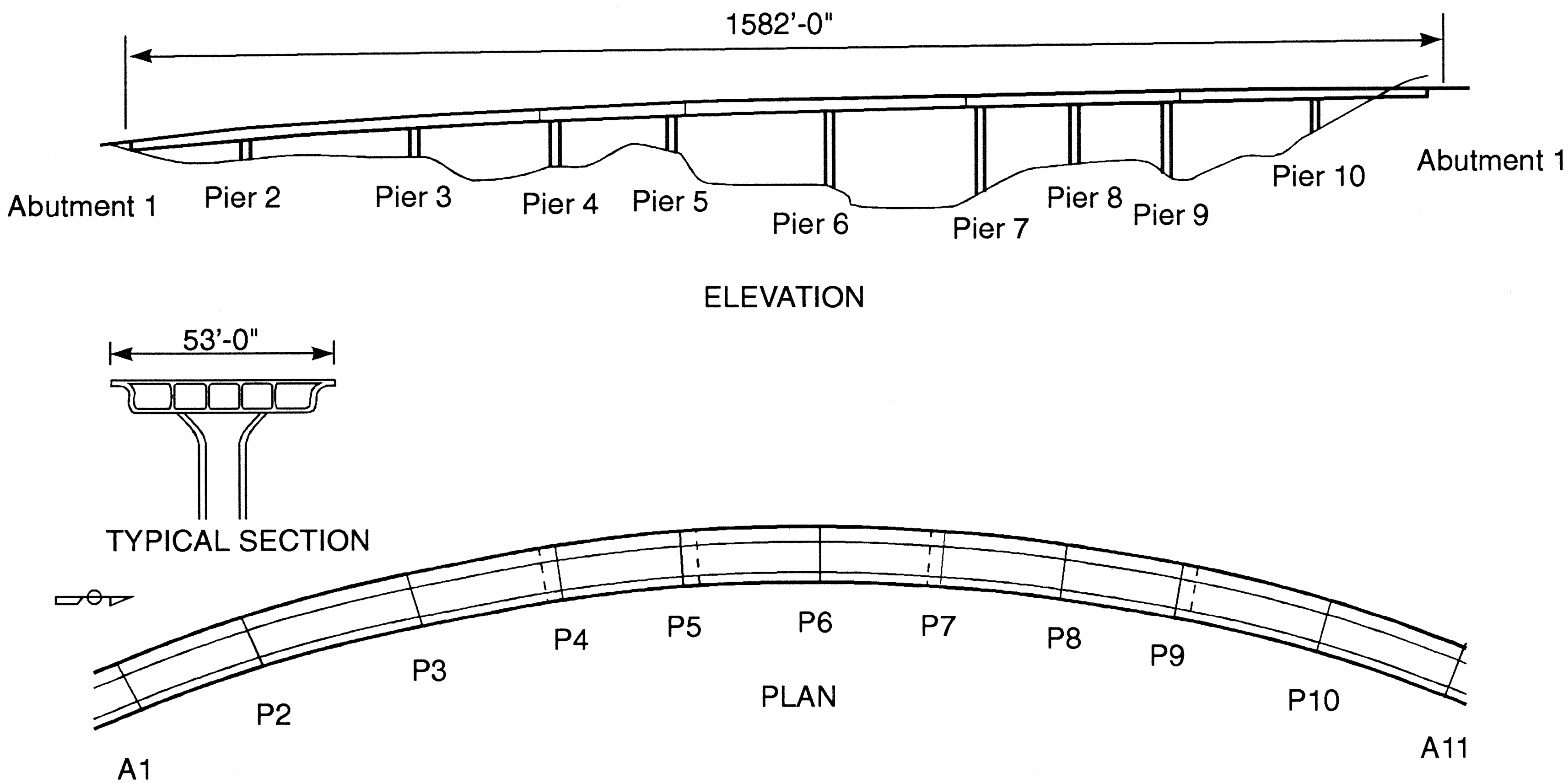
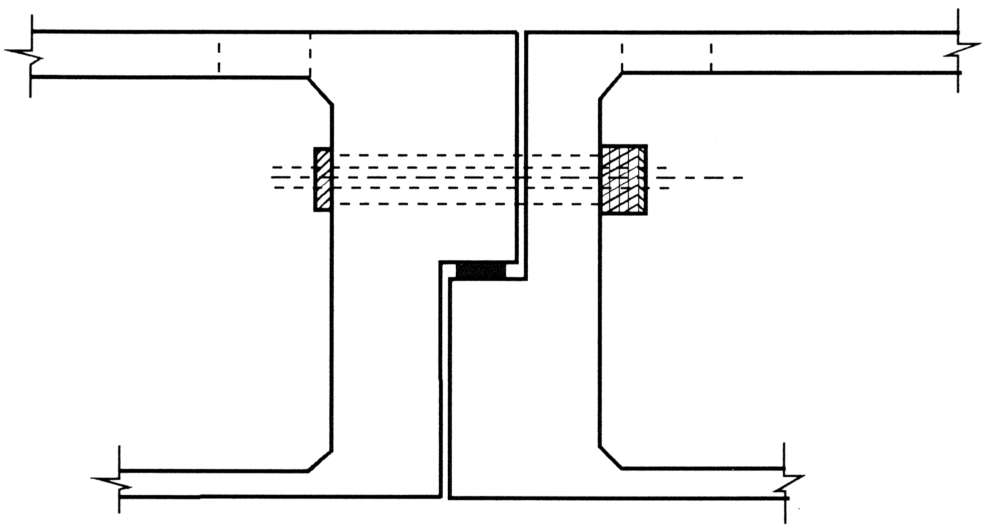


FIGURE 4-3 SR14/I-5 Separation and Overhead (Southbound) - General Plan and Elevation



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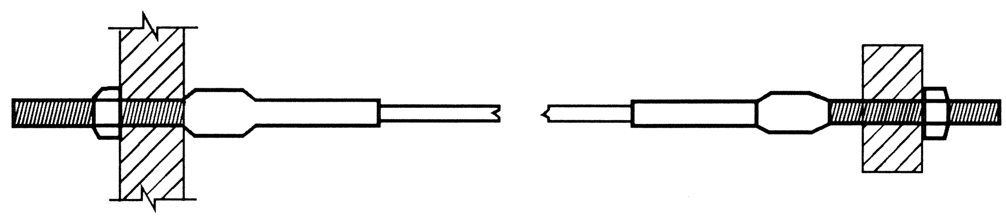


SWAGED FITTING AND STUD

FIGURE 4-4 SR14/I-5 Separation and Overhead (Southbound) - Type I Hinge Restrainer

Survey, the California State Department of Mines and Geology and the Lamont-Doherty Earth Observatory.

The Lamont-Doherty Earth Observatory deployment consisted of eight sites (denoted as circles in Figure 4-2) arranged in two lines at roughly a 35° 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 750m. 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 4-5 and 4-6. Figure 4-5 shows the vertical components of the P-wave from a M4.7 aftershock recorded at four sites within 150 meters. Figure 4-6 shows the difference between site S0 and sites S2-4. It is seen that 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. It will also be noted that the differential motion increases with distance. The peak difference between S0 and S4 (150 m apart) is twice the difference between S0 and S2 (10 m apart). Further, the peak differential motion can be larger than the peak ground motion and, in this example, it occurs later in the waveform.

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

4.3 Observed Earthquake Damage

As shown in Figure 4-7, spans 1, 2 and 3 collapsed, 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 as it was completely crushed under the superstructure. The three spans which 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 in width and Type 1 restrainers (Figure 4-4) tied frames 1 and 2 together. Restrainers were not used at either abutment.

The right exterior shear key at abutment 1 was damaged; however the left shear key had little visual 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 (Figure 4-8). At this same hinge, the equalizing bolts failed in tension (Figure 4-9) and the nuts on the restrainer cable studs were missing (Figures 4-10 and 4-11). 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 visual inspection of piers 3 and 4 above grade did not indicate structural damage. The predominant motion of the structure appeared to be in the north-south direction.

4.4 Failure Analysis

The possible failure mechanisms appear to be:

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 followed by the collapse of span 1 and unseating at abutment 1.

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 overloading and collapse of pier 2 followed by collapse of span 1 and unseating at abutment 1.

3. Non-ductile failure of pier 2 caused collapse of spans 1 and 2, 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 (Figure 4-12) and no crushing of column concrete below the bent cap towards pier 2 (Figure 4-13) would indicate that perhaps span 3 collapsed first. Further, 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 mechanism 1. is the likely failure mode.

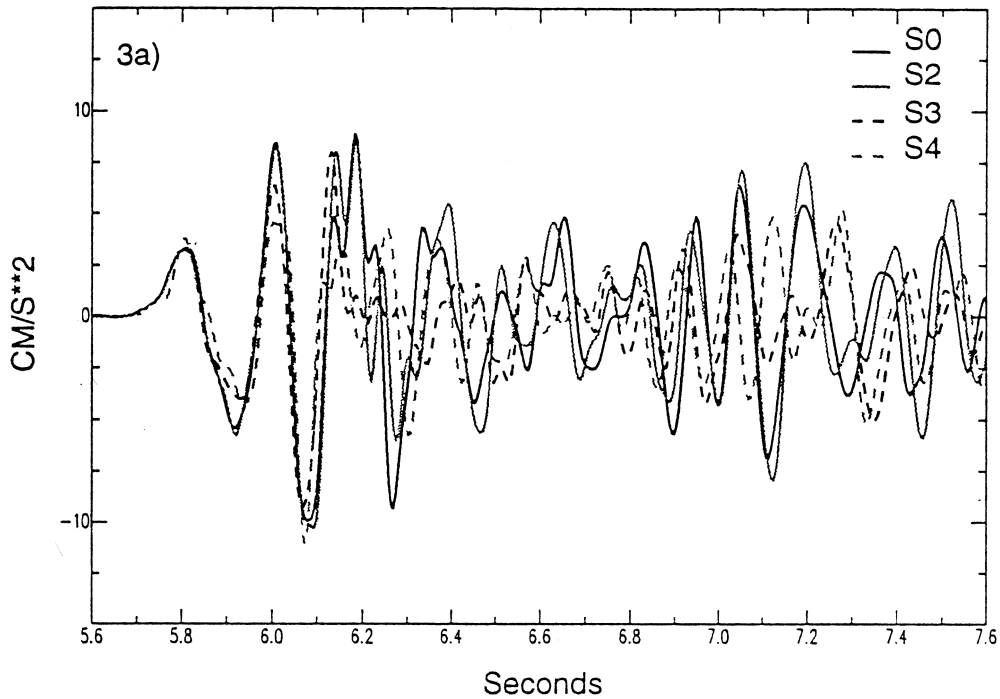


**FIGURE 4-5 SR14/I-5 Interchange -
Vertical Components of P-wave from a M4.7 Aftershock Recorded
at Four Temporary Sites (S0, 2-4)**

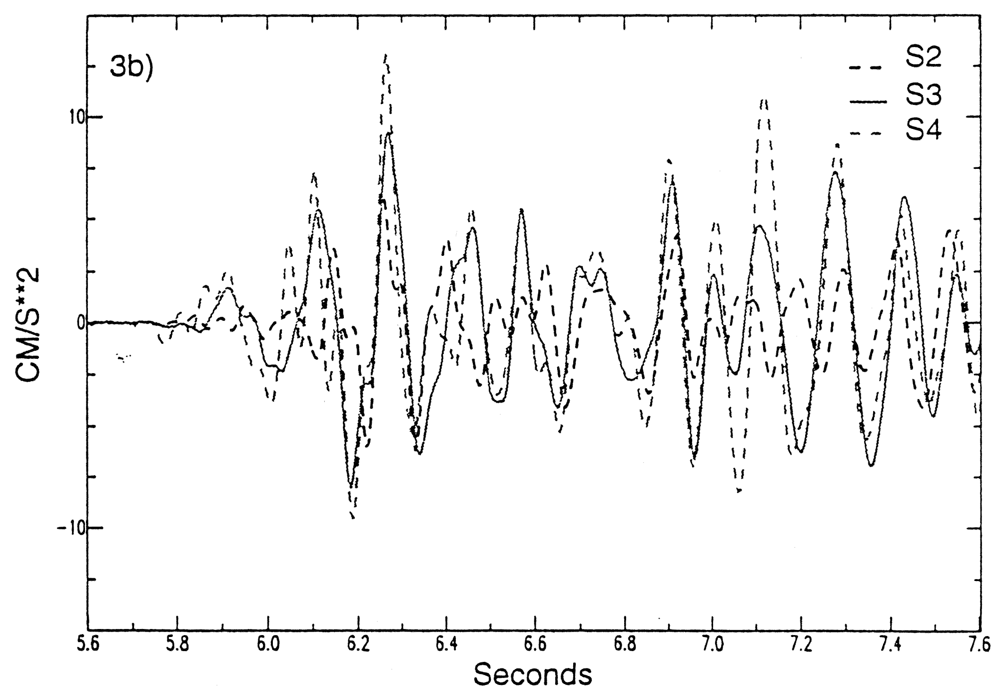


**FIGURE 4-6 SR14/I-5 Interchange -
Difference between Site S0 and Sites S2-4**

FIGURE 4-7 SR14/I-5 Separation and Overhead (Southbound) - Aerial View of Collapse

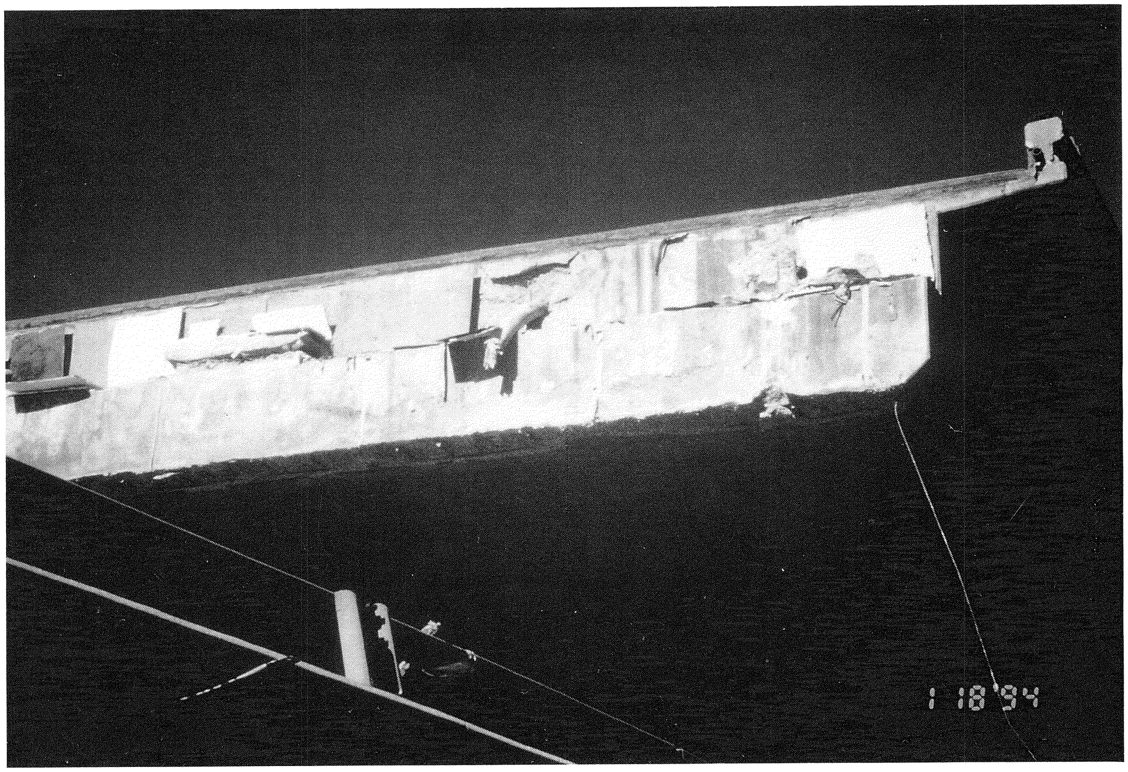


FIGURE 4-8 SR14/I-5 Separation and Overhead (Southbound) - Hinge Seat and Shear Key in Span 3

FIGURE 4-9 SR14/I-5 Separation and Overhead (Southbound) - Equalizing Bolt at Hinge in Span 3

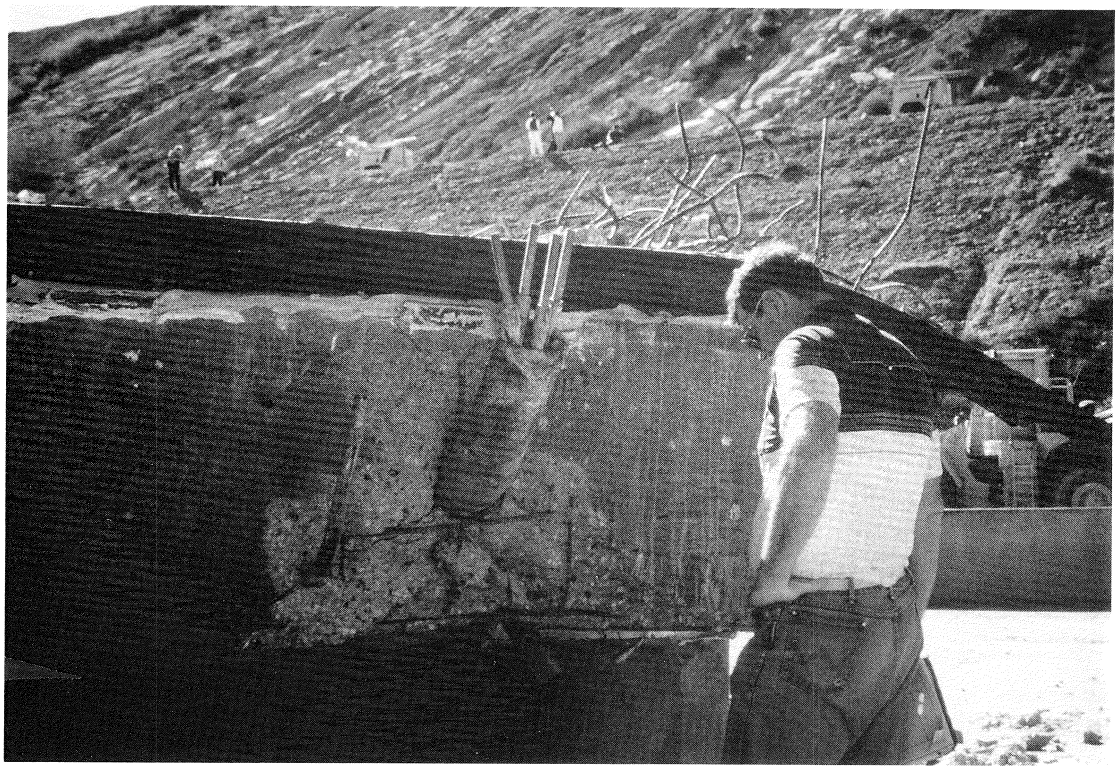


FIGURE 4-10 SR14/I-5 Separation and Overhead (Southbound) - Equalizing Bolt and Restrainers at Hinge in Span 3

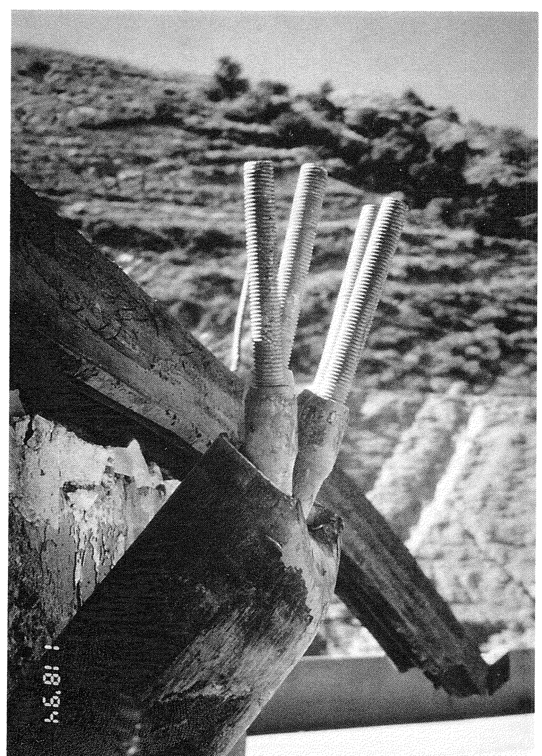


FIGURE 4-11 SR14/I-5 Separation and Overhead (Southbound) - Restrainer Studs at Hinge in Span 3

FIGURE 4-12 SR14/I-5 Separation and Overhead (Southbound) - North Side of Pier 3 and Cap

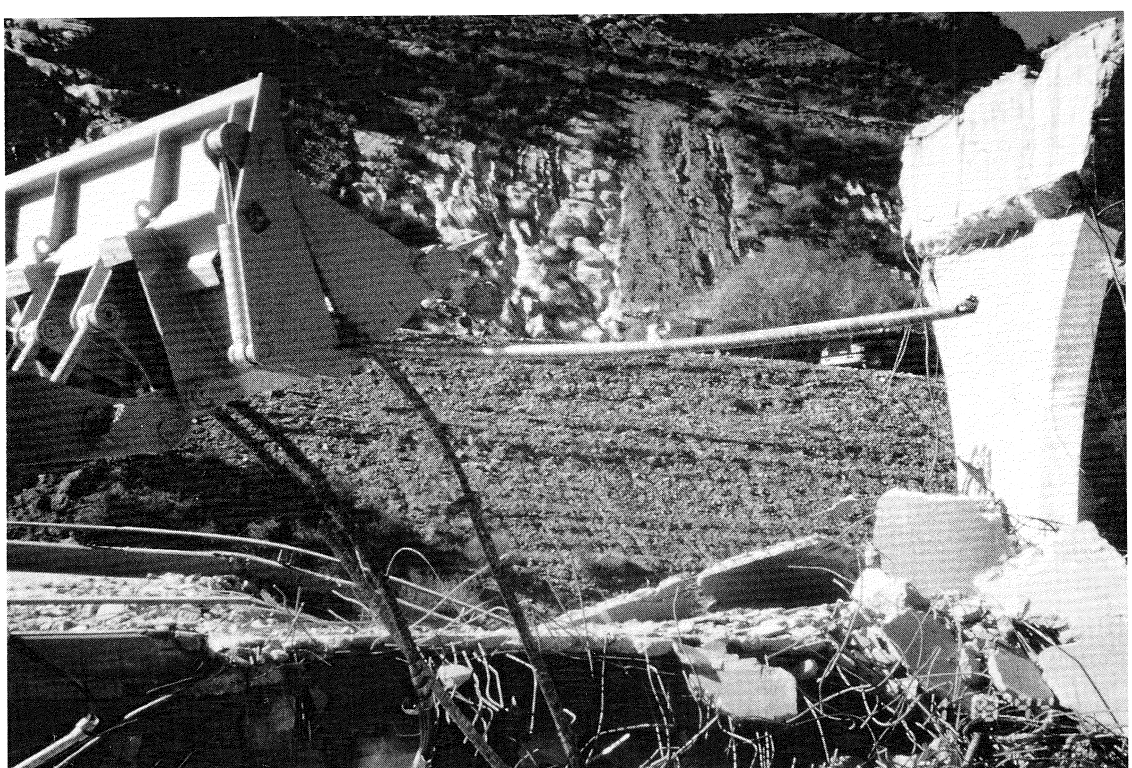


FIGURE 4-13 SR14/I-5 Separation and Overhead (Southbound) - South Side of Pier 3 and Cap

SECTION 5
ROUTE 14/I-5 NORTH CONNECTOR OVERCROSSING - BRIDGE NUMBER 53-1964F

5.1 Description

The SR14/I-5 north connector overcrossing structure is located at mile post 24.92 on Route 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 Route 14 to northbound Route 5, spanning Route 5 (truck), Weldon Canyon Road, and Southern Pacific Railroad tracks. Route 5 is a U.S. Interstate running north-south along the west coast and Route 14 is a State highway beginning at the SR14/I-5 interchange and runs northeast to U.S. Route 395 in Kern County. 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 4-1 and 4-2.

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 5-1 for details of span geometry.

This bridge was under construction at the time of the 1971 San Fernando earthquake. Approximately 80% 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 had minor damage of crushed expansion material, slight spalling, separated waterstops, and failed equalizing bolts. Pier 2 showed signs of movement at the ground level having an embankment separation of 2-inch 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, (4 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.

5.2 Ground Motion

This interchange is approximately 2-1/2 miles from the Gavin Canyon Undercrossing. The ground motions here can be assumed to be similar to those experienced at the Gavin Canyon site (Section 3.2). However, as noted in Section 4.2, 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 U.S. Geological Survey, the California State Department of Mines and Geology and the Lamont-Doherty Earth Observatory. Results obtained in the vicinity of the southbound SR14/I-5 separation and overhead are discussed in Section 4.2.

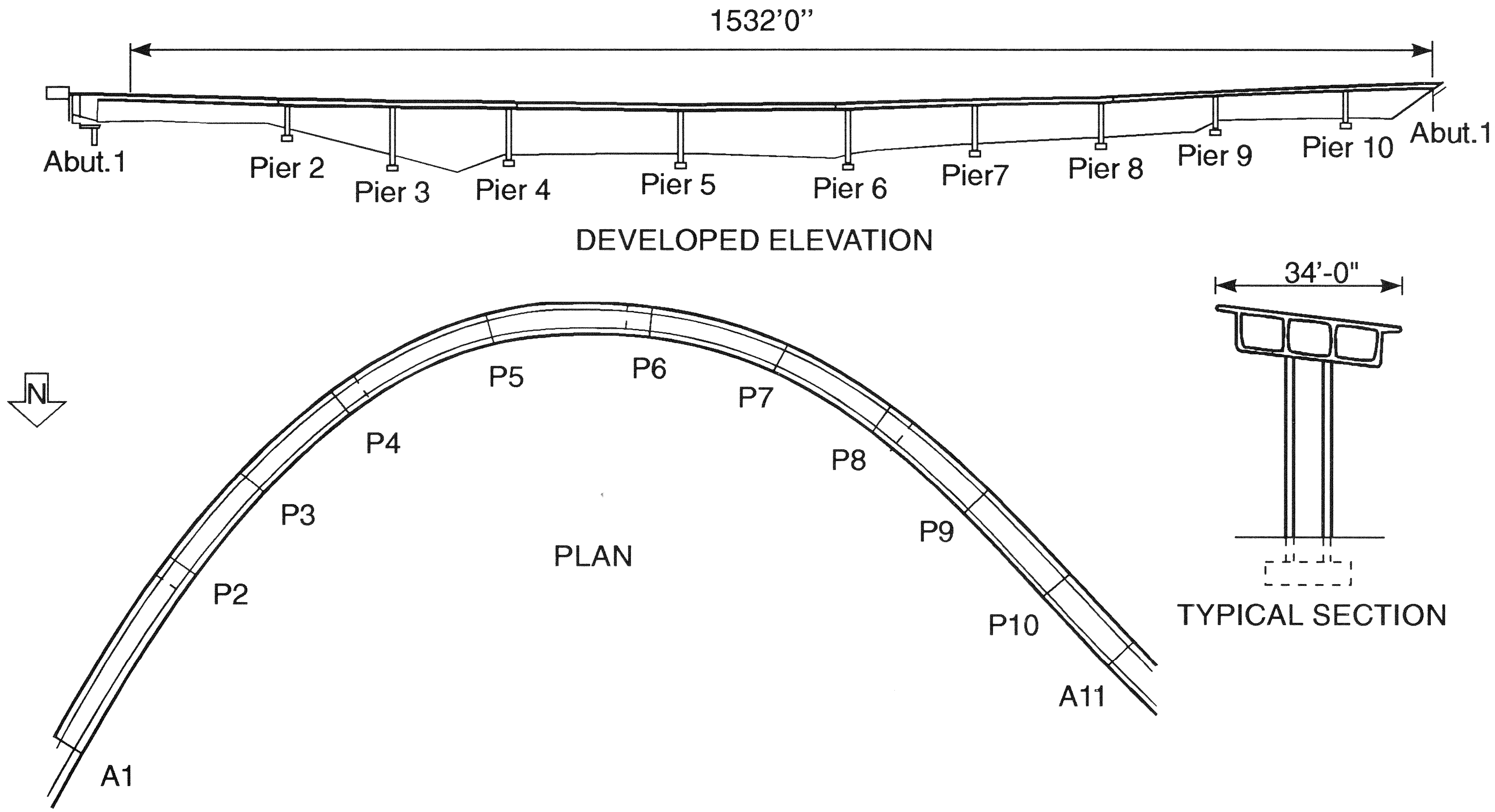


FIGURE 5-1 SR14/I-5 North Connector - General Plan and Elevation

5.3 Observed Earthquake Damage

As shown in Figures 5-2 and 5-3, 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 foot long, simply supported at the seat abutment and at the in-span 1 hinge. The hinge and abutment seats were both 14 inches in width and hinge restrainers had been installed at the hinge. (Current Caltrans design specifications require that abutment seat minimum widths be 24".) 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 (Figure 5-3). Equalizing bolts at abutment 1, shown in Figure 5-3, failed. Pier 2, which collapsed, was supported on a spread footing and was approximately 21 feet high. The other two piers making up frame 1 were pier 3, approximately 73 feet high supported on CIDH piles and pier 4, approximately 60 feet high supported on a spread footing. These pier columns were not detailed for ductile behavior having #4 ties at 12 inch centers.

5.4 Failure Analysis

Possible failure mechanisms appear to include: (1) seat loss at abutment 1 causing 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 but it seems likely that pier 2 failed first 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. Further, 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 leading to the failure of the pier.

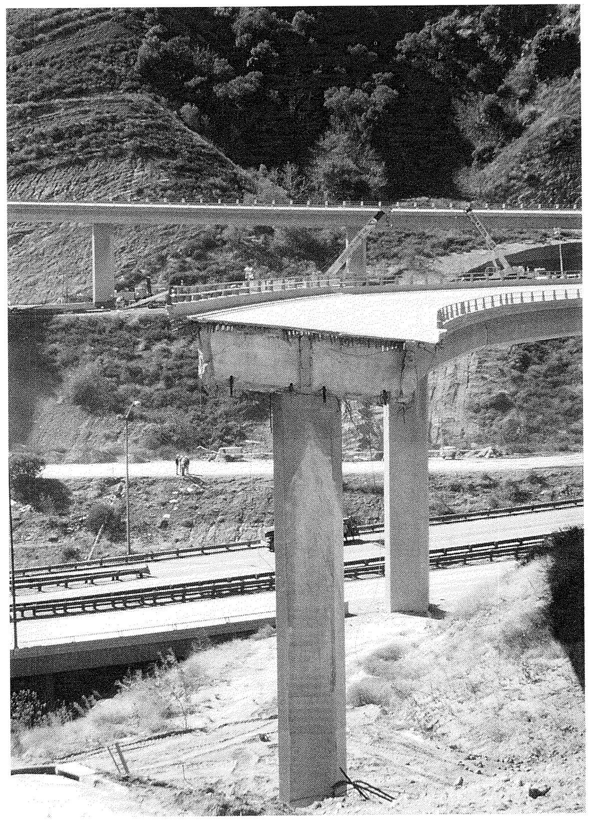


FIGURE 5-2 SR14/I-5 North Connector - Collapsed End Spans After Demolition

FIGURE 5-3 SR14/I-5 North Connector - Seat and Transverse Shear Keys at Abutment 1

SECTION 6
BULL CREEK CANYON CHANNEL BRIDGE - BRIDGE NUMBER 53-2206

6.1 Description

This bridge carries 10 lanes of traffic on State Route 118 and 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 (Br. No. 53-2205) which suffered a partial collapse during the earthquake. The bridge is relatively new, having been 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-place prestressed concrete box girder that is typical of bridges built in California in the seventies. At the center of the freeway, the three spans measure 90.5, 101.0, and 65.0 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 6-1.

The abutments and bents are not quite parallel and are laid out on skews which vary from approximately 37 to 47 degrees from normal. This results in the span lengths for the westbound bridge being 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 in length 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 3" over a length of 4'-0" 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. Column details are shown in Figure 6-2.

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.

6.2 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].

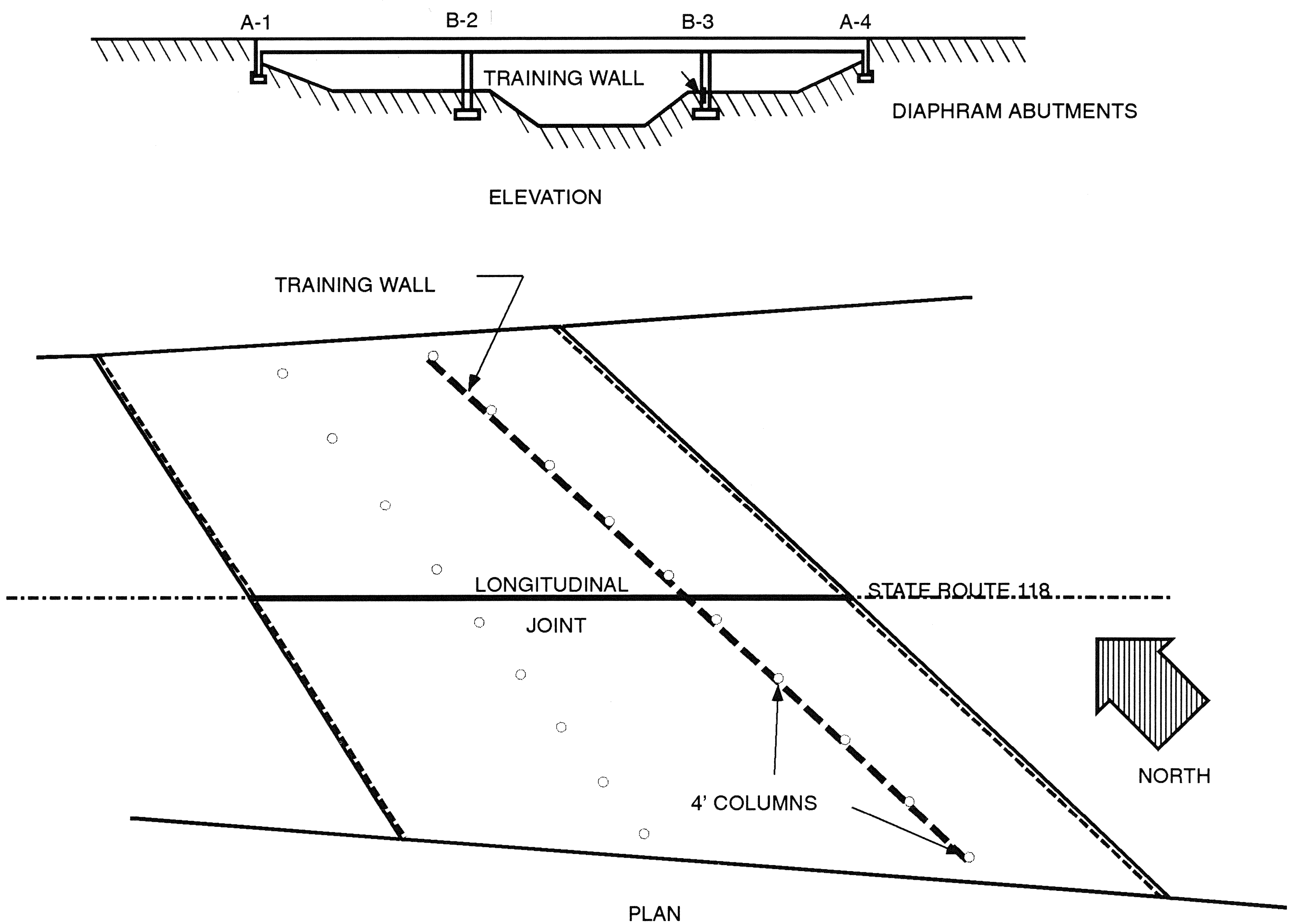
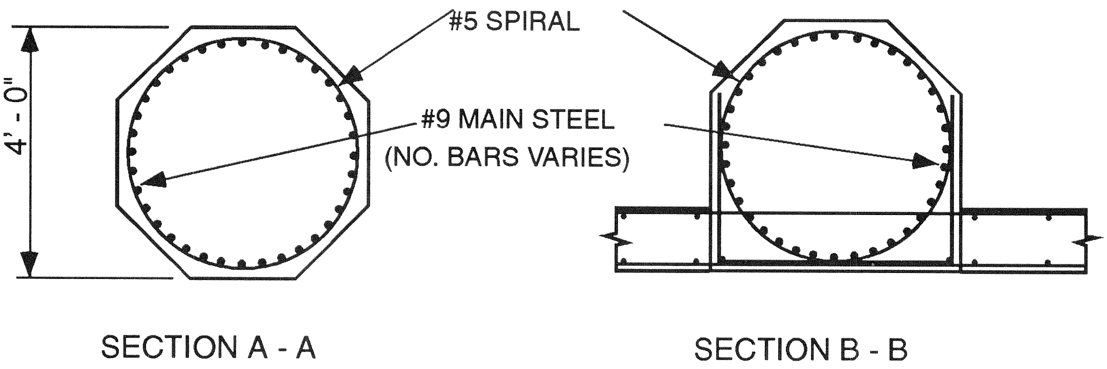


FIGURE 6-1 Bull Creek Canyon Channel Bridge - General Plan and Elevation



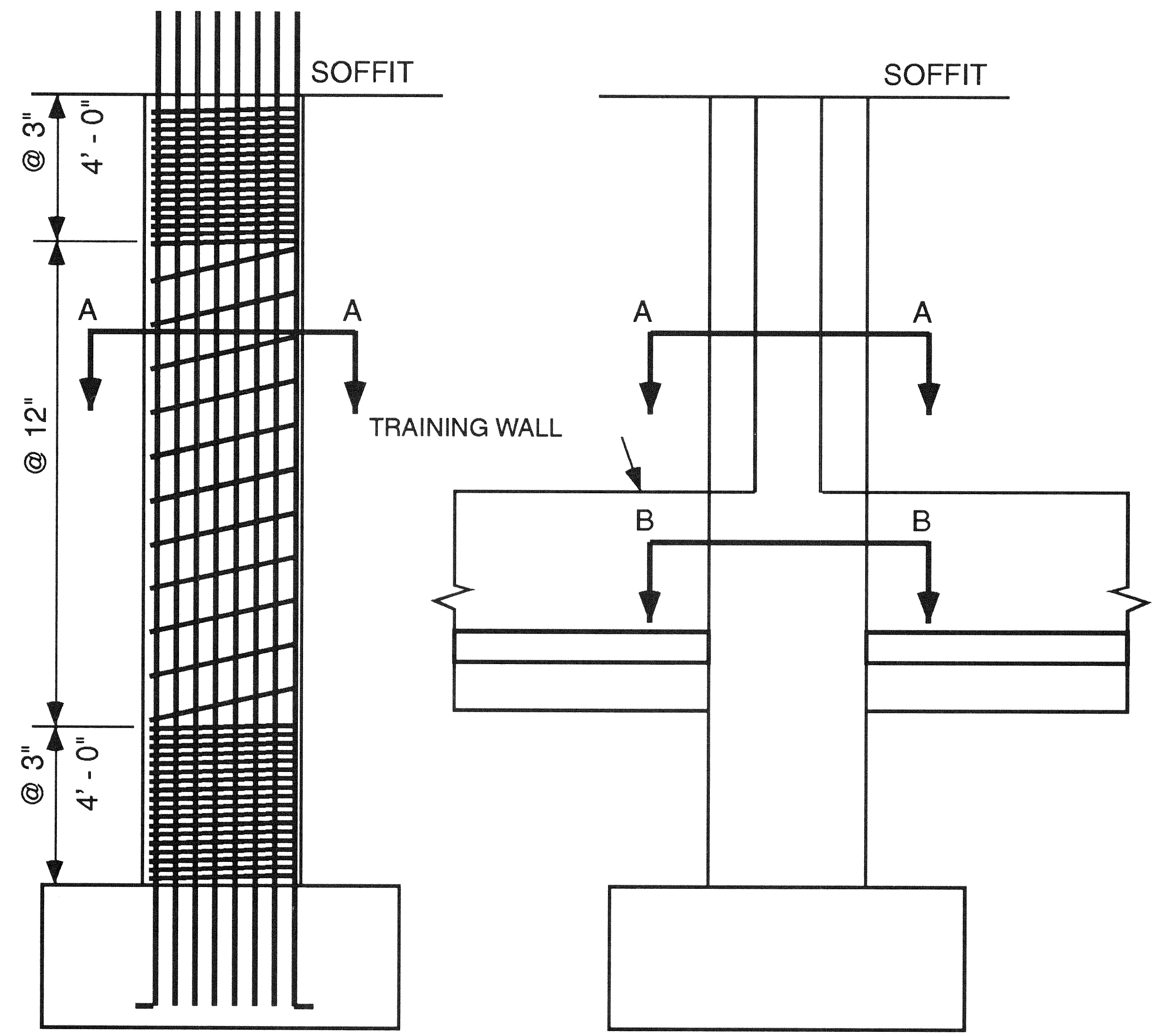


FIGURE 6-2 Bull Creek Canyon Channel Bridge - Column Details

A triaxial free field instrument located at Arleta on deep alluvium (CSMIP Sta. No. 24087) recorded peak accelerations of 0.34g in the east-west direction and 0.31g 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 0.3g. [7,8]. An unusual characteristic of the records obtained from this site was the relatively high peak accelerations of 0.55g measured in the vertical direction.

A second instrument located at the base of a seven story hotel in Van Nuys recorded peak accelerations of 0.47g in the north-south direction and 0.41g in the east-west direction. Peak vertical accelerations of 0.3g 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 ground motions at the bridge were higher than those 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.

6.3 Observed Earthquake Damage

As shown in Figure 6-3, 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 as shown in Figure 6-4. This 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.

At bent 3 all columns in both bridges failed just above the training wall as shown in Figure 6-5. 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 abutments, 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.

6.4 Failure Analysis

The failures at bent 3 may be explained by the presence of the training wall which 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. It is felt that 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.

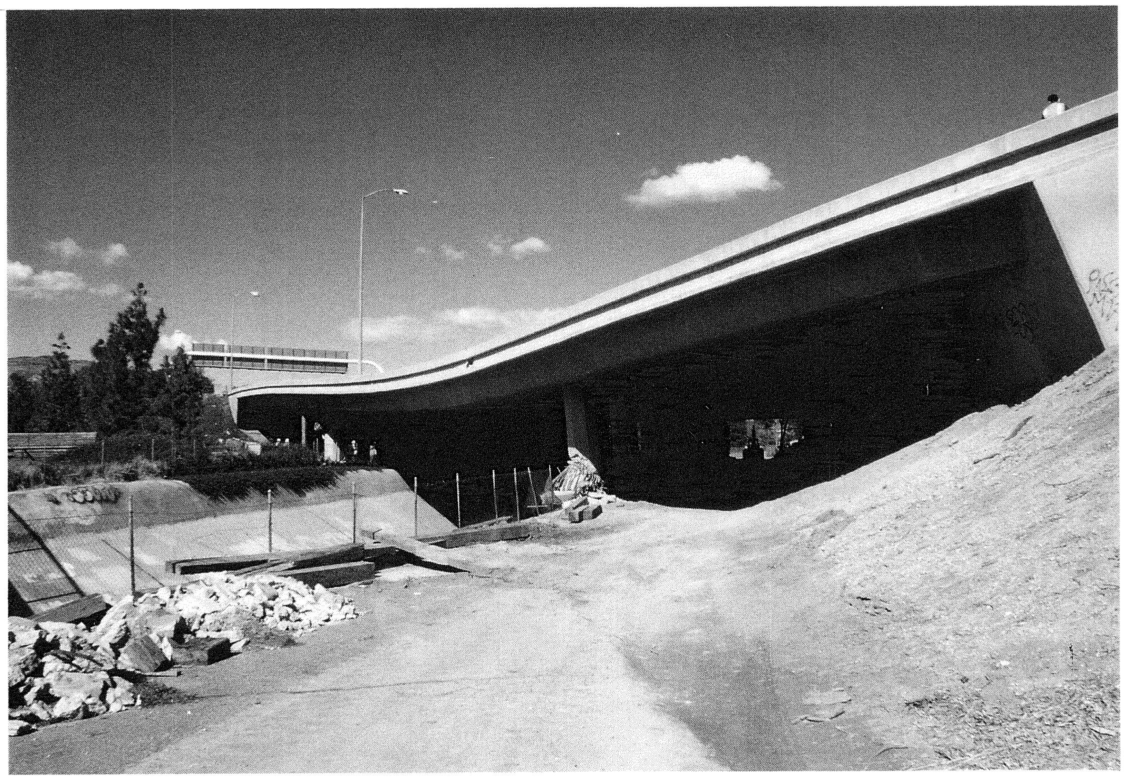


FIGURE 6-3 Bull Creek Canyon Channel Bridge - Side View

FIGURE 6-4 Bull Creek Canyon Channel Bridge -
Hinge Formation Below Well Confined Section in Column of Bent 2 (Eastbound Bridge)

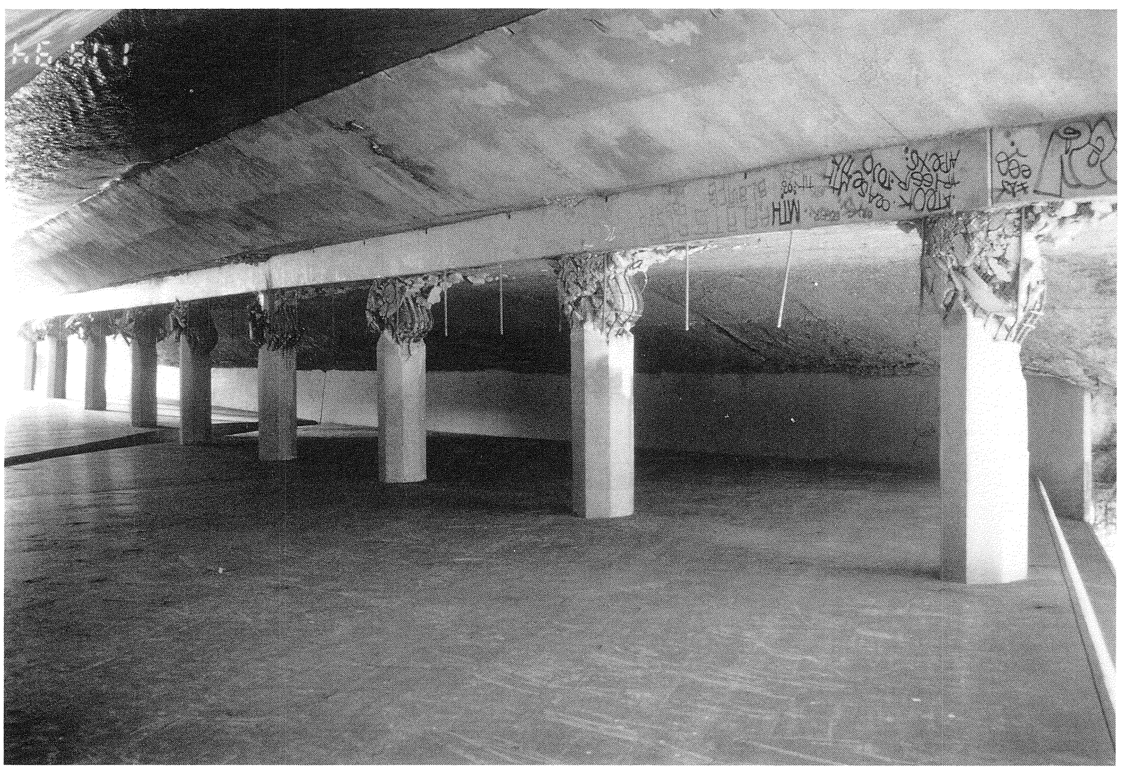


FIGURE 6-5 Bull Creek Canyon Channel Bridge -
Hinges in All Columns of Bent 3 Immediately Above the Channel Training Wall

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, it is concluded that the failures resulted because of higher shear forces and are not solely due to flexural yielding in the region below the well confined zone of the column.

6.5 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 the columns were adequately reinforced for shear. Therefore, it would seem prudent to place closely spaced transverse reinforcement over the full length of the column. 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.

SECTION 7
MISSION-GOTHIC UNDERCROSSING - BRIDGE NUMBER 53-2205

7.1 Description

This bridge is located on State Route 118 at postmile mark 8.63 at the intersection of San Fernando Mission Boulevard and Gothic Avenue in the Northridge area. Designed in 1973, the bridge is in fact two separate side-by-side structures carrying east and west bound traffic through San Fernando Valley. The structure is 536 feet long at the centerline and skewed 45.9° clockwise at abutment 1 and 44.1° counterclockwise at abutments 4 and 5. Since the abutments are almost at right angles to each other, one bridge is approximately 438 feet long on its centerline and the other is 634 feet on its centerline. The shorter, westbound bridge is supported on two bents, while the longer, eastbound bridge is supported on three bents. The superstructure is a cast-in-place prestressed concrete 7.5-foot deep box girder. Both structures are 98 feet wide with slight variations to accommodate nearby ramps. Some of these details are shown in Figure 7-1. Each bent of both bridges consists of two columns, flared at the top, as shown in Figure 7-2. The total depth of the flare is 12 feet; the average column height is 22 feet.

The columns are six-foot diameter octagonal (circular core) reinforced concrete columns wrapped with No. 5 spirals spaced at 3.5 inches. This level of confining shear reinforcement almost satisfies today's standards. The main longitudinal column reinforcement consists of 45, number 11 bars embedded 6.5 feet into the cap. The flares outside of the central 6-foot diameter core are reinforced with 22, number 11 longitudinal bars tied with number 5 bars shown in Figure 7-2. These flares are presumed to have been provided for architectural reasons so as to improve the aesthetics of the column-to-girder connections. Caltrans has used this detail for this purpose since the early seventies, especially on single column bents. These "flares" are not intended to add to the structural strength of the column but are expected to be damaged and later repaired as necessary. Although well-reinforced in the vertical direction they are usually lightly reinforced in the transverse direction outside of the central core (No. 5 bars at 12 inches). As a consequence, these architectural elements are not considered to supplement or compromise the seismic performance of the column in any significant way.

The bases of all the columns are supported on piled footings through ¼-inch thick neoprene sheets coated with molylube grease covered with 16 gage sheet metal. No reinforcing steel runs through the bottom of the column into the pile cap. The lateral support for the column bases was provided by a concrete collar cast at least 120 days after the superstructure was prestressed (Figure 7-2). The ratios of the column lengths to the column widths, measured from the "pin" at the base to the bottom of the flare range, from 1.7 to 2.1, thus technically making them pier walls in the transverse direction.

The first 20 to 30 feet of the soil at the site consists of primarily loose to slightly compact brown very fine to fine grained sand and sandy silt. The bents and the abutments were all supported on cast-in-drilled hole concrete piles.

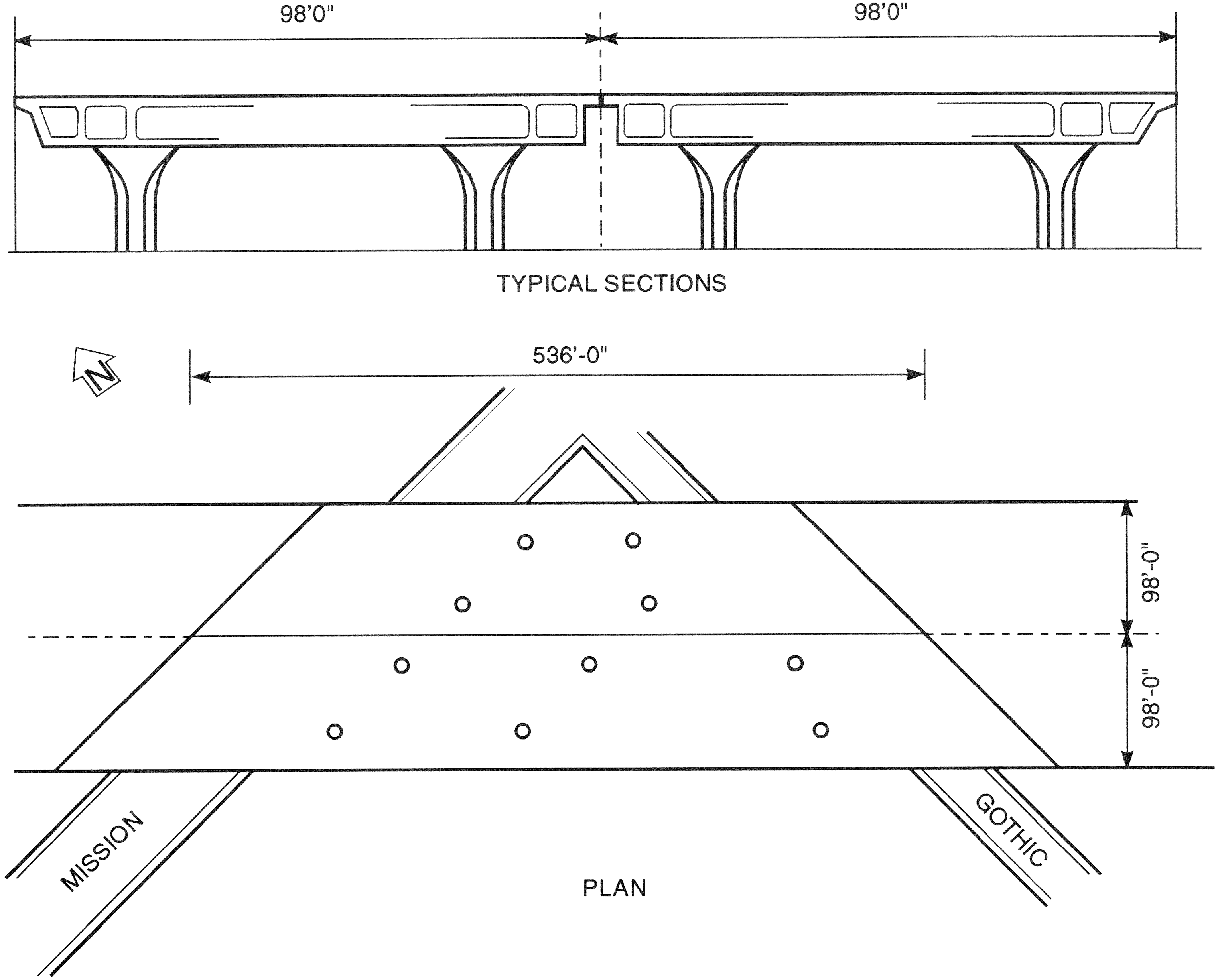


FIGURE 7-1 Mission-Gothic Undercrossing - General Plan and Elevation

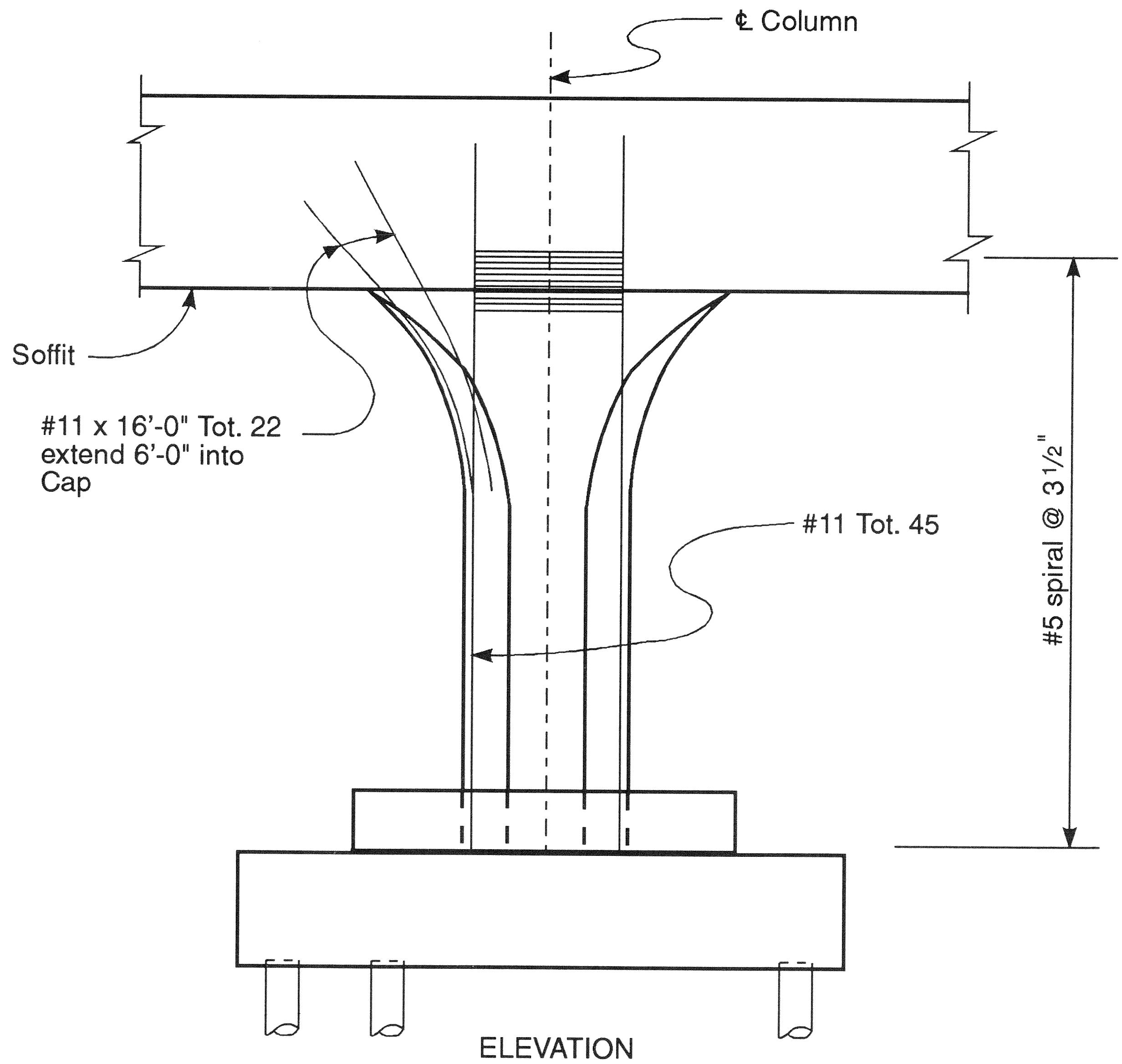


FIGURE 7-2 Mission-Gothic Undercrossing - Column and Footing Details

7.2 Ground Motion

No strong motion accelerograph data in the immediate vicinity of the site are available at this time. It should be noted, however, that intense ground motions were recorded in this general area as noted for the nearby Bull Creek Bridge (Section 6.2). This structure is about five miles from the epicenter, and four miles from the Sylmar accelerograph which recorded a free-field horizontal acceleration of 0.91g horizontally. A "back of the envelope" calculation for this structure indicates that the ground motions would have had to have been at least about 0.35g to initiate the observed damage at this structure. It is quite likely that the ground motions at this site were larger than this estimate. In fact, preliminary contour maps of peak ground accelerations from the Los Angeles Strong Motion Array [2] indicate that both the vertical and the horizontal peak accelerations at the site may have been on the order of 0.5g or greater.

7.3 Observed Earthquake Damage

The eastbound bridge collapsed completely at the end near abutment 5 and fell off the abutment seat (Figure 7-3 and 7-4). The deck of this bridge also rotated clockwise when looking down in plan (Figure 7-5), and was left partially standing on its severely damaged columns.

The westbound bridge (which was shorter in overall length than the eastbound bridge) did not collapse. However, most of the two-column bents were severely damaged (Figures 7-6 and 7-7) with failures occurring at mid-height just below the flare. Similar distress was observed in the eastbound bridge but the columns near abutment 5 also sustained large lateral offsets (up to five feet) due to the clockwise rotation of the deck (in plan). Transverse restraint at the abutments was provided by external shear keys and these clearly failed at abutment 5 on the eastbound bridge. Evidence of similar damage to the restraint at abutment 1, for the same bridge, is shown in Figure 7-8.

As noted above and shown in Figure 7-2, each column is pinned to its footing and the required shear connection is provided by a collar, placed around the column, that is tied to the footing and buried just below the surface. Figure 7-9 shows the right hand column of bent 2 of the eastbound bridge and a substantial upheaval of soil immediately to the south of the column. Excavation by Caltrans (and a demolition contractor) showed this disturbance to be due to the collar breaking free from the footing and being driven into the soil by the column. Figure 7-10 shows a portion of the excavated collar; note the absence of a positive connection to the footing. It is likely that the lack of an effective shear connection to the footing substantially reduced the forces in the column and may explain the absence of damage in this particular column.

7.4 Failure Analysis

This bridge was well designed and detailed and would probably pass all current screening tests for vulnerability and possible retrofit. It is therefore of particular interest to consider the possible failure mode.

It is likely that several factors contributed to its collapse; one possible scenario is as follows. Since the abutments have opposing skew angles, and are thus at right angles to one another, excessive shear demands could have been placed on the restraints at the east abutments which then failed and triggered the partial collapse of the bridge.

FIGURE 7-3 Mission-Gothic Undercrossing -
Side View of Partially Collapsed Eastbound Bridge

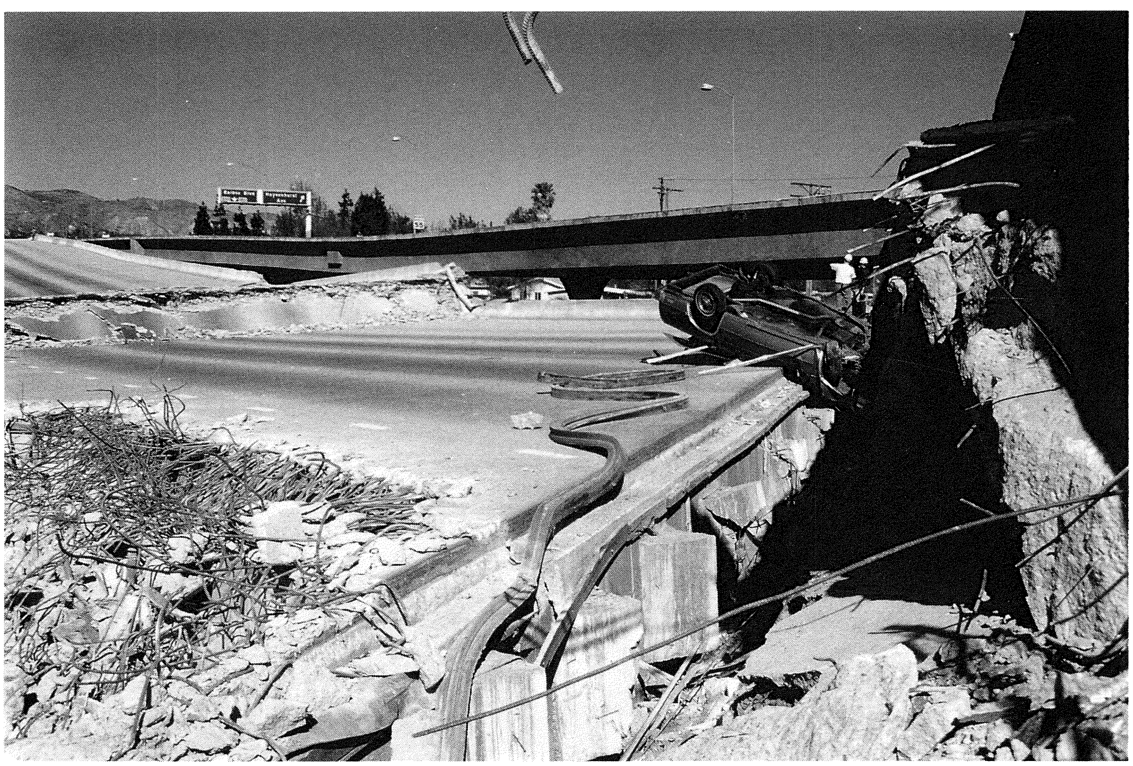


FIGURE 7-4 Mission-Gothic Undercrossing -
Span 4 and East Abutment of Eastbound Bridge

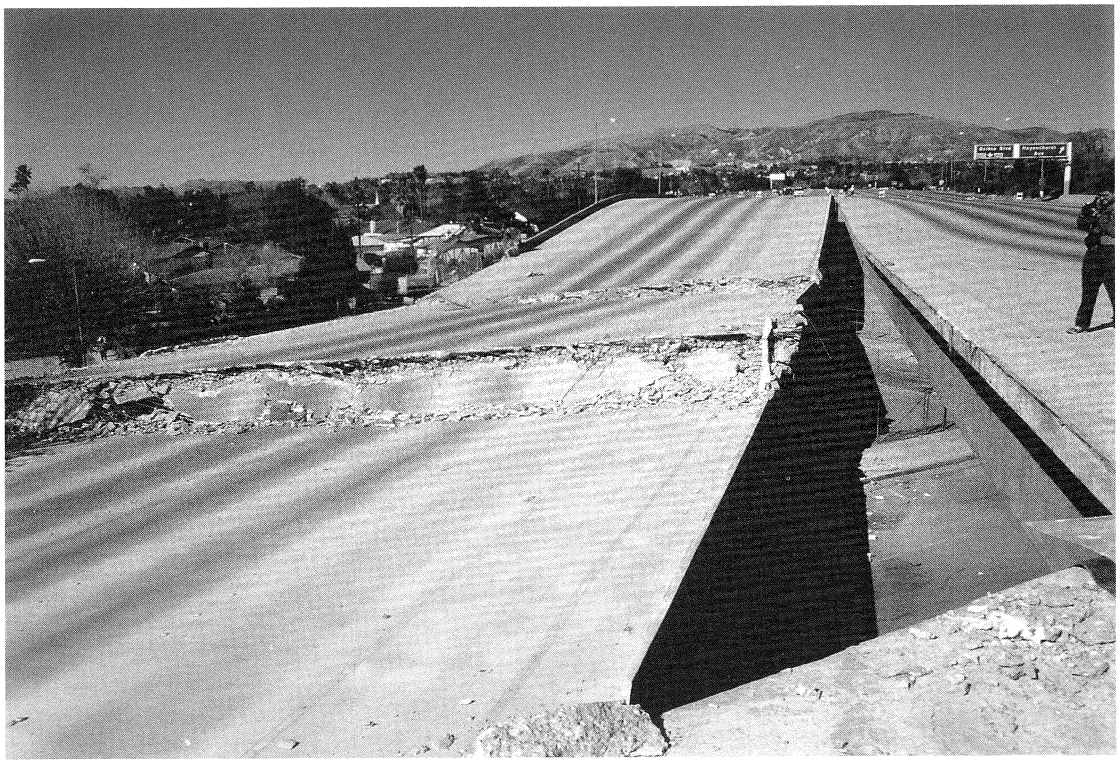


FIGURE 7-5 Mission-Gothic Undercrossing -
View from East Abutment Showing Rotation of Eastbound Superstructure Away from Westbound Bridge

FIGURE 7-6 Mission-Gothic Undercrossing -
Hinge Formation Below Flare in Column of Westbound Bridge (Face View)

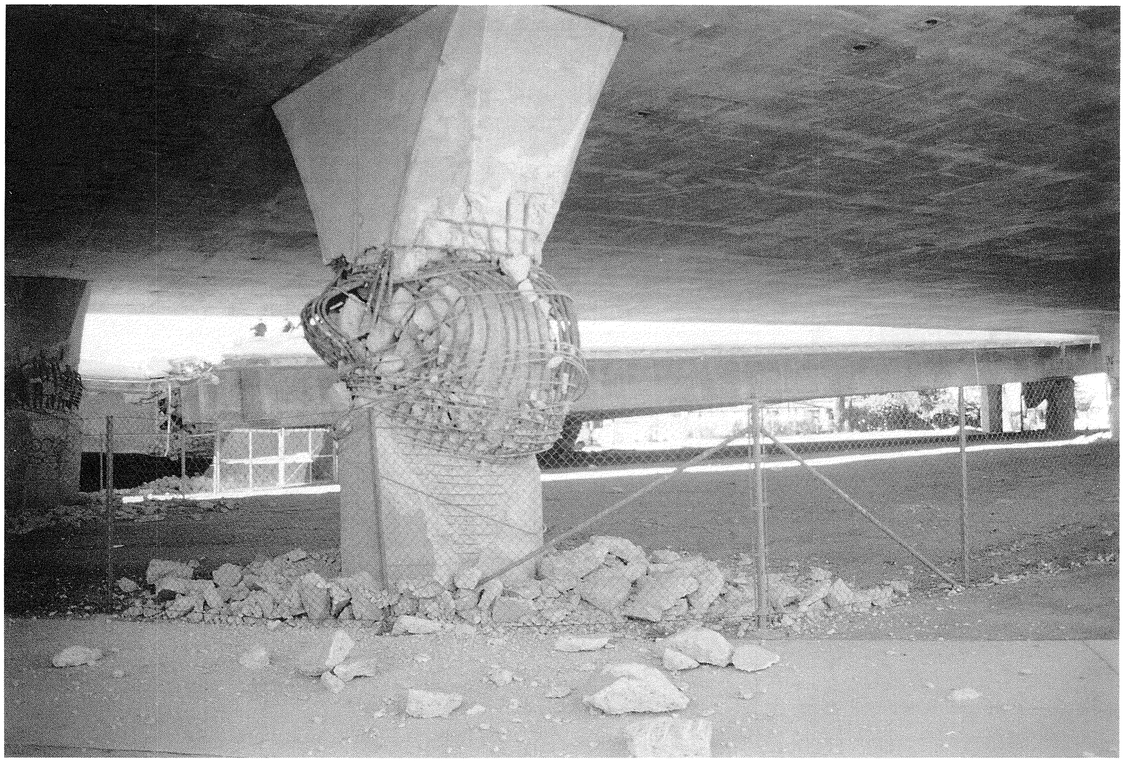


FIGURE 7-7 Mission-Gothic Undercrossing -
Hinge Formation Below Flare in Column of Eastbound Bridge (Side View)

FIGURE 7-8 Mission-Gothic Undercrossing -
Shear Key Damage at West Abutment of Eastbound Bridge

FIGURE 7-9 Mission-Gothic Undercrossing -
Soil Upheaval Above Footing of Right Hand Column in Bent 2 of Eastbound Bridge

FIGURE 7-10 Mission-Gothic Undercrossing -
Excavated Collar from Footing of Right Hand Column in Bent 2 of Eastbound Bridge

This is illustrated in Figure 7-11 which shows the eastbound bridge of the Mission-Gothic undercrossing. For shaking predominantly in the north-south and/or east-west direction, the bridge was "locked" in one direction but "free" in the other. For loading towards the north and east (Figure 7-11a), the bridge was restrained by the backwalls and fills behind the north and east abutments. In this locked position, the columns and shear keys were protected by the resistance provided by the backfills. For loading towards the south and west (Figure 7-11b), the bridge was not restrained by either abutment and the full lateral loads were resisted by the columns until the transverse restraints at the abutments engaged.

Under these loads, the columns began to develop plastic hinges. But because of their flared geometry and the accidental strength that these provide to the upperhalf of the column, these hinges formed near mid-height. The corresponding shear forces were then almost 83% larger than if the hinges had formed just under the soffit (as assumed in design) leading to excessive shear demands in the plastic hinge zones. Degradation of the columns transferred more load to the abutment restraints which subsequently failed leaving the bridge free to translate and rotate about a vertical axis. If the north-south component dominated the ground motion, then the east abutment restraints would likely fail before those at the north abutment leading to a clockwise rotation about the north abutment, as shown in Figure 7-11c.

In fact, the damage patterns and residual displacements at the west end suggest that the center of torsion was probably nearer bent 2 than abutment 1. The corresponding rotation imposed large horizontal deformations in the bents that increased with distance from the west end (Figures 7-11c and 7-5) and culminated in the unseating at the east abutment and the large permanent offsets in the columns of bent 4. Partial collapse of the eastbound bridge followed. The westbound bridge was not as extensively damaged probably because it is shorter in length and the shears in the columns and at the abutments were not sufficient to trigger collapse.

While it is recognized that this may not have been the exact sequence of events, it is almost certain that three factors played a contributing role in the collapse of the bridge. In summary these are:

1. The unusual geometry and in particular the effect of orthogonal skew, and variations in skew from one substructure to the next;

2. The unintended actions of architectural elements such as the flares on the columns which may have inadvertently enhanced the flexural strength of the columns leading to premature shear failures, particularly in these short columns; and

3. The inadequate capacity of shear keys and other transverse restraints at the abutments.

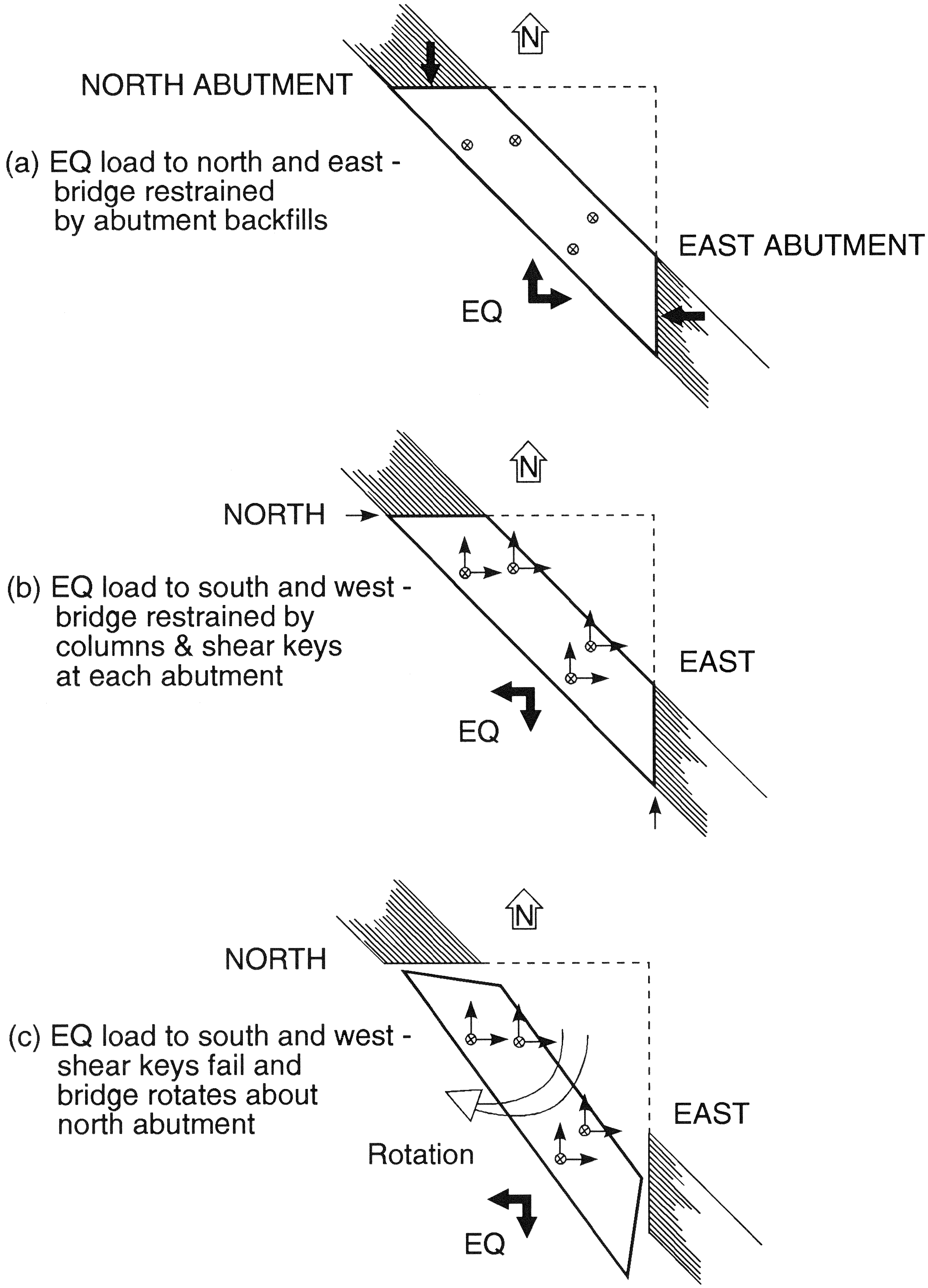


FIGURE 7-11 Mission-Gothic Undercrossing (Eastbound Bridge) - Possible Load Distribution and Failure Sequence

SECTION 8
BALBOA BOULEVARD OVERCROSSING - BRIDGE NUMBER 53-2395

8.1 Description

The Balboa Boulevard overcrossing spans State Route 118 at mile post 7.8 in Los Angeles County, approximately 21 miles to the northwest of downtown Los Angeles. The structure is on a north-south tangent alignment. Balboa Boulevard is a paved undivided arterial which runs from the city of Encino to the Route 5/210 interchange.

Built in 1976, the bridge is a two-span continuous cast-in-place box girder with 13 cells. Seat abutments and one three-column bent support the structure. The southern reinforced concrete bin abutment spans between the seat abutment and a diaphragm abutment, both of which are supported on 16-inch diameter CIDH concrete piles.

The total length is 282.5 feet, with an overall width of 117 feet. There is negligible skew and curvature. The only intermediate joint is located between the bin abutment and the box girder. Overall geometry and span arrangements are given in Figure 8-1, and a view of the bridge is shown in Figure 8-2.

Of particular note is the number of utilities carried by this bridge. These are summarized in Table 8-I.

TABLE 8-I Utilities Carried by the Balboa Boulevard Overcrossing

Utility	Service	Lifelines	
Los Angeles Department of Water and Power	power water water	6, 5" 1, 12" 1, 8"	ducts pipeline pipeline
Southern California Gas Company	gas gas	22" 4"	line in 26" casing line in 8" casing
General Telephone Company AT&T	telephone telephone	4, 4" ducts 6, 4" ducts	
Unassigned	unspecified	4, 3" ducts	

8.2 Ground Motion

This bridge is less than a mile west of the Mission-Gothic Undercrossing. The ground motions here can be assumed to be similar to those experienced at the Mission-Gothic site (Section 7.2).

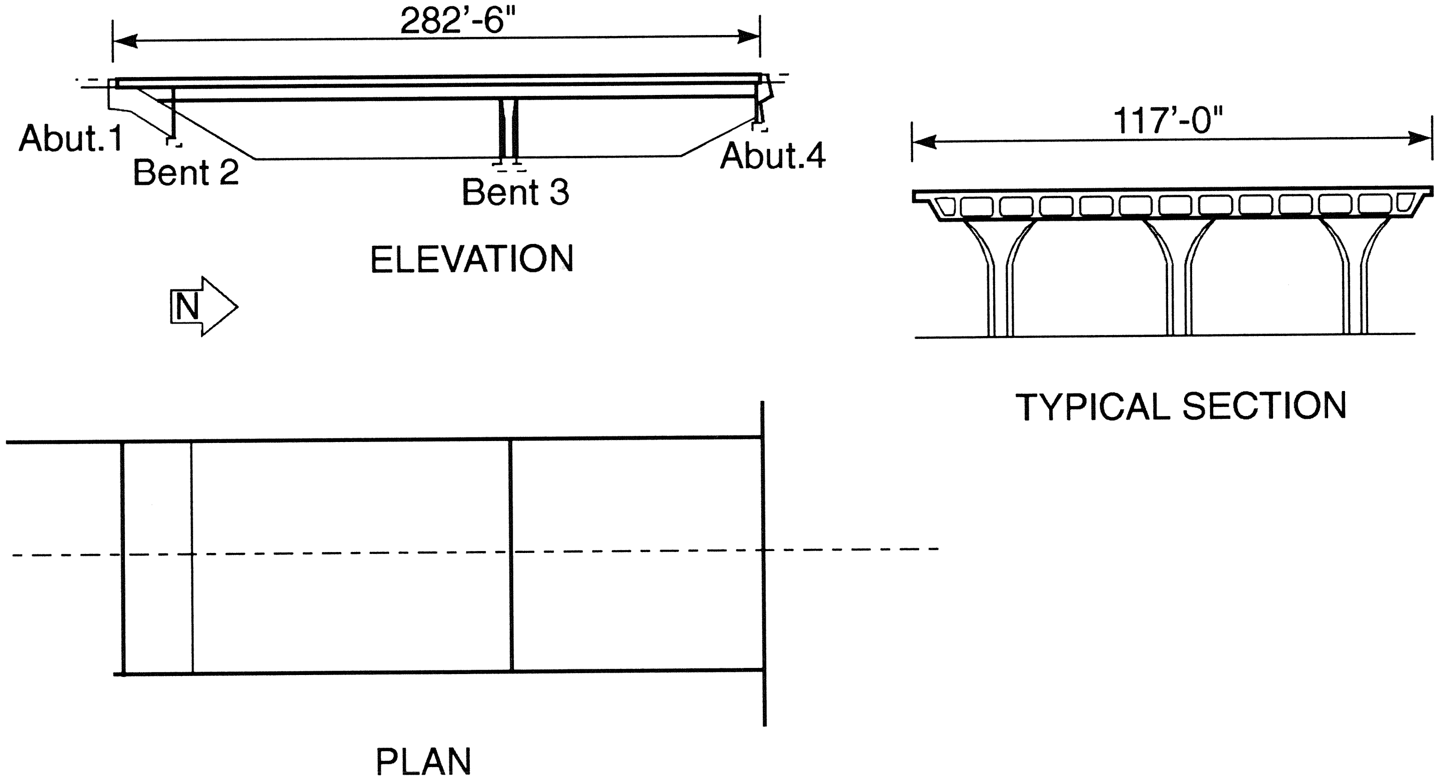


FIGURE 8-1 Balboa Boulevard Overcrossing - General Plan and Elevation

8.3 Observed Earthquake Damage

Damage to the structure was primarily located at bin abutment 1. Waterlines supported from the structure ruptured and subsequently washed out the soil beneath the approach and pile caps causing collapse of approach pavement and undermining of the diaphragm abutment 1 and bent 2 (part of the bin abutment). Some of this damage is shown in Figure 8-3. The seat abutment sustained damage at the left front wall (Figure 8-4) and the adjacent curtain wall near the pile cap (Figure 8-5). The damage appeared to be shear cracks resulting from longitudinal forces being resisted at the bin abutment. Damage to the 16-inch exposed CIDH piles was not noted. Movement of the structure appeared to be primarily in the north-south direction which is the longitudinal direction of the overcrossing.

8.4 Failure Analysis

This bridge performed satisfactorily. It resisted the longitudinal seismic forces by soil plowing at the abutments and frame action at bent 3. Undermining at the bin abutment occurred after the waterline(s) carried by the bridge, ruptured during the seismic event. The principal damage to the bridge (and its closure) was therefore the result of colocation of lifelines - a reminder that the vulnerability of a lifeline that shares right-of-way with other lifelines is determined by the most fragile member; which may not necessarily be the lifeline under study.

FIGURE 8-2 Balboa Boulevard Overcrossing - General View

FIGURE 8-3 Balboa Boulevard Overcrossing - Soil Erosion at South Abutment Due to Ruptured Water Lines

FIGURE 8-4 Balboa Boulevard Overcrossing - Wall and Expansion Joint at Abutment 2

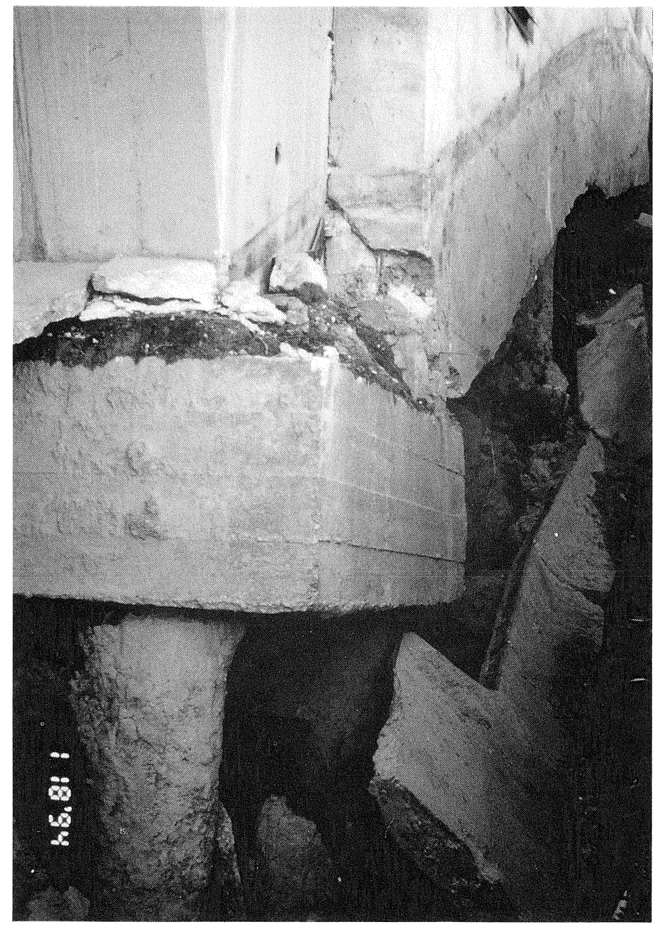


FIGURE 8-5 Balboa Boulevard Overcrossing - CIDH Pile and Cap at Abutment 2

SECTION 9
FAIRFAX-WASHINGTON UNDERCROSSING - BRIDGE NUMBER 53-1580

9.1 Description

This urban undercrossing carries both east and west bound Interstate 10 over Fairfax Avenue and Washington Boulevard near Culver City in west-central Los Angeles County, California. Interstate 10 is reported to be the busiest freeway in the United States. The bridge was originally constructed in 1964 and subsequently retrofitted in 1974 with expansion joint restrainers. At the time of the earthquake the bridge was scheduled for Phase 2 retrofitting consisting of abutment shear keys, steel column shells, bent cap and footing strengthening, and expansion joint modifications.

The bridge superstructure is a multi-cell, cast-in-place, reinforced concrete box girder typical of California construction. It is separated by a longitudinal expansion joint that runs down the median of the freeway. The western abutment is nearly normal to the centerline of the freeway while the eastern abutment is skewed at approximately 47 degrees to normal. The structure is slightly wider at the western end to accommodate an off-ramp. A single expansion joint hinge with a seat width of six inches is located in span 3. The structure to the south of the longitudinal expansion joint is approximately 73 feet wide and consists of seven spans (lengths: 45.8, 112.0, 98.0, 77.6, 83.0, 81.0, and 46.1 feet). The structure to the north flares from between 107 feet wide at the west abutment to 70 feet wide at the east abutment and has eight spans (lengths: 45.8, 112.0, 98.0, 76.1,76.1, 83.0, 81.0, and 46.1 feet). The above span lengths are measured along the approximate centerline of each of the respective structures.

The ends of the structure are supported on steel rocker bearings approximately six inches high at seat type abutments. The supports adjacent to the abutments consist of solid concrete pier walls designed to hinge in the longitudinal direction at both the top and bottom. The remaining supports are multi-column bents with poorly confined four foot round reinforced concrete columns spaced at between 25 to 31 feet on center. The number and location of columns is dictated by the geometry of the city streets which cross under the bridge at different angles. A schematic plan and elevation of the bridge is shown in Figure 9-1.

Although all the reinforced concrete columns have a clear length between the top of footing and the bottom of soffit of approximately 20 feet, the amount of main reinforcement in the columns varies between 1.0 and 5.3 percent of the cross-section area. The end condition at the base of the column also varies depending on the bent. Transverse reinforcement consists of single hoops of 1/2 inch bars spaced at 12 inches along the full length of the column. Hoops are lap spliced. Column details are shown in Figure 9-2.

Abutment foundations bear directly on soil while all interior foundations are supported on 16 inch cast-in-drilled-hole piles approximately 20 to 25 feet in length. The soil profile at the site consists mostly of slightly compact to dense sands and gravels in excess of 70 feet deep.

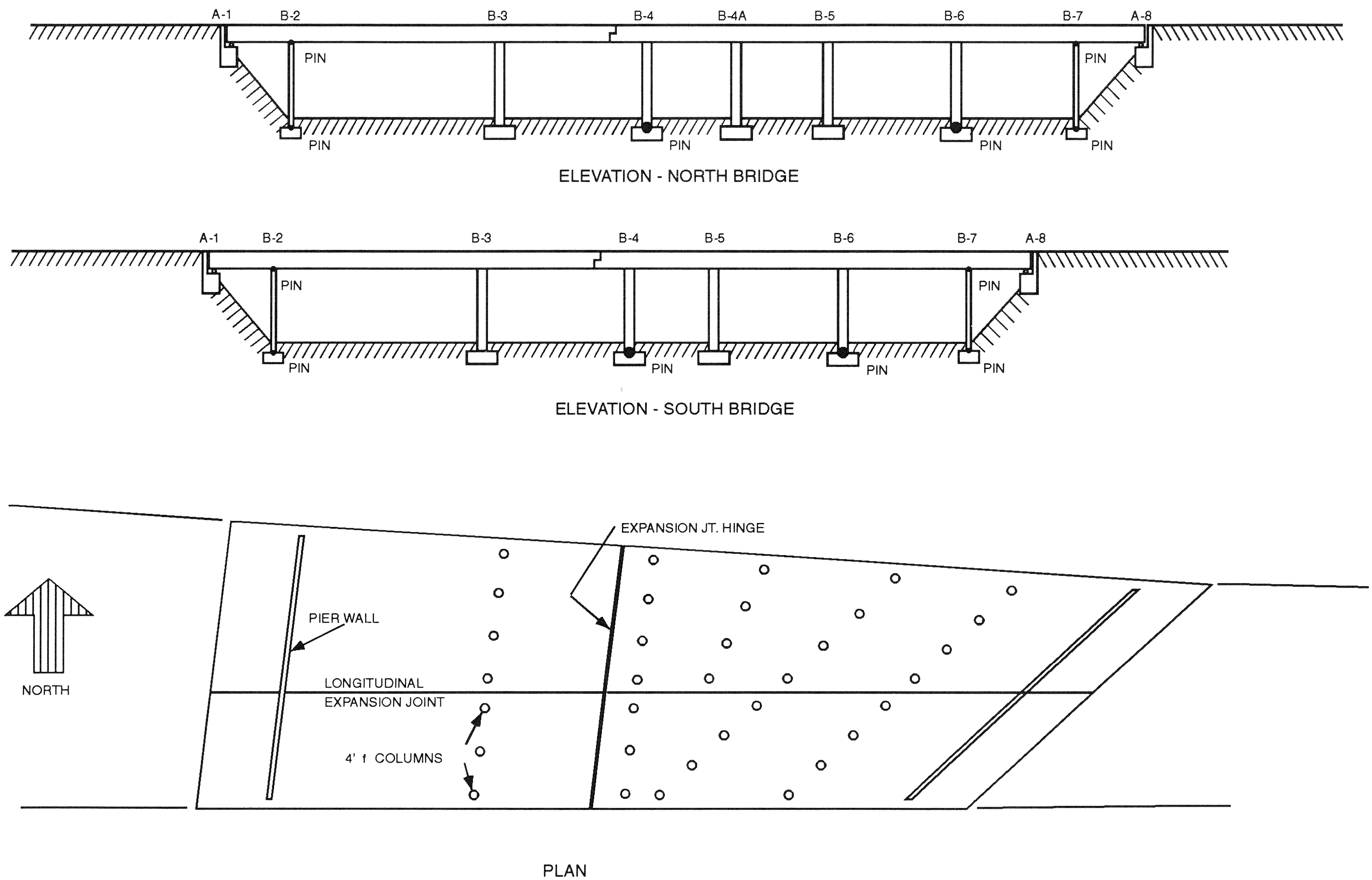


**FIGURE 9-1 Fairfax-Washington Undercrossing -
General Plan and Elevation**

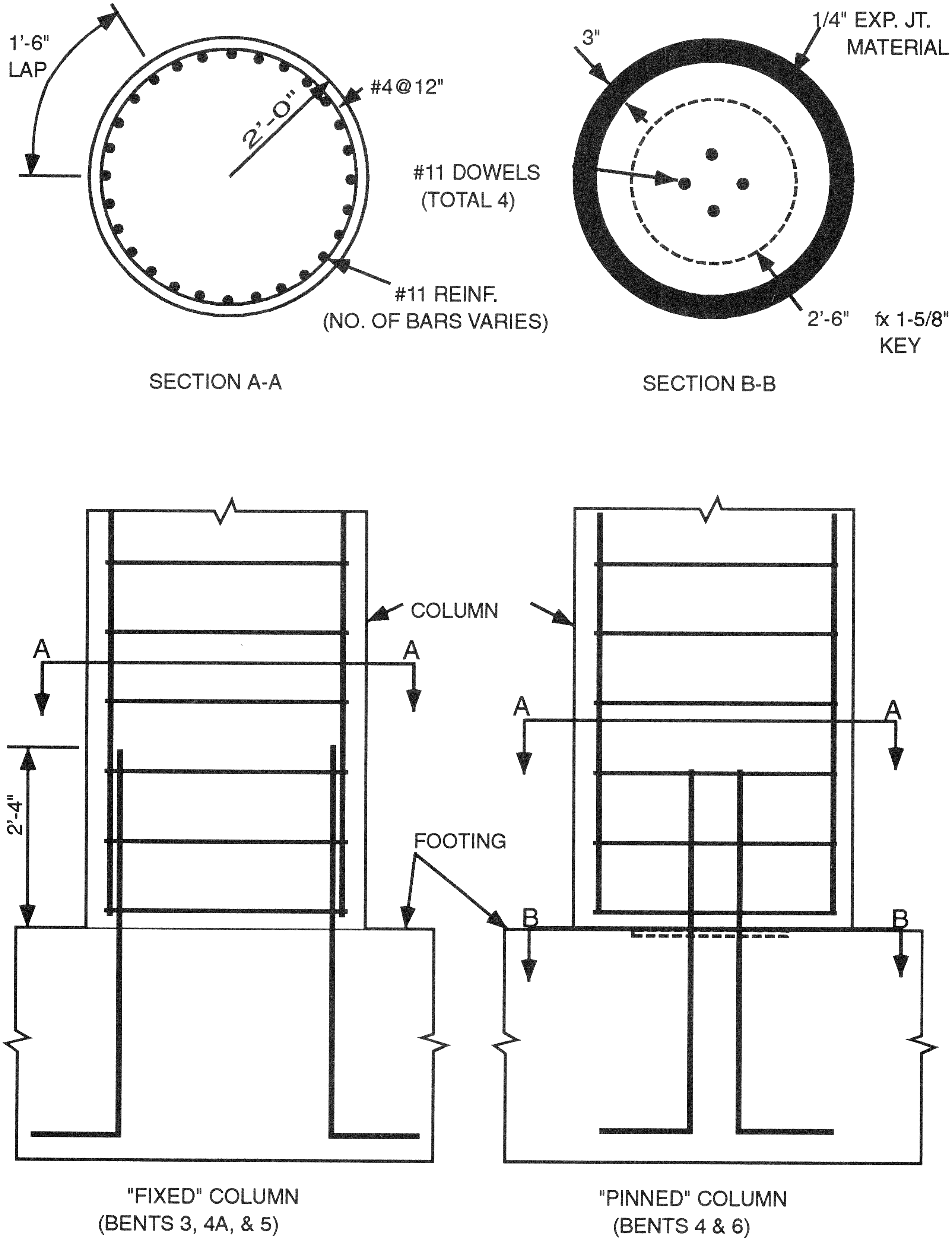


FIGURE 9-2 Fairfax-Washington Undercrossing - Column Details

9.2 Ground Motion

The structure is located approximately 17 miles southeast of the epicenter of the main shock. Two free field strong motion recorders are located within a few miles of the bridge site [6].

The first instrument (CSMIP Sta. No. 24157), which is located two miles to the southeast in Baldwin Hills on one meter of fill over shale/sandstone, recorded approximately 10 seconds of strong shaking with a peak horizontal acceleration in the east-west direction of 0.24g and 0.17g in the north-south direction.

The second instrument (CSMIP Sta. No. 24389) also recorded approximately 10 seconds of strong shaking. The peak horizontal acceleration was 0.27g in the east-west direction and 0.24g in the north-south direction. This instrument is located approximately four miles northwest of the bridge and is supported on terrace deposits.

Motions recorded approximately four miles away on a bridge structure within the I-10/I-405 interchange are consistent with the magnitude of horizontal ground motion recorded at the above two free field sites.

9.3 Observed Earthquake Damage

This structure suffered major damage during the earthquake. All of the columns in bent 3 failed resulting in a several foot drop of the superstructure at this support (Figure 9-3). This drop caused the superstructure to rotate about the pier wall at bent 2 and to lift off the seat at abutment 1 (Figure 9-4). As a result, flexural yielding occurred in the superstructure near bent 3. The expansion joint hinge in span 3 remained effective despite the narrow seat width (Figure 9-5). Observation of the hinge restrainers after demolition of the expansion joint (Figure 9-6) showed that they remained intact during the earthquake and helped prevent total loss of support.

The columns in bent 3 failed near the top at the expected location of plastic hinging. Transverse steel hoops were ineffective in containing the column concrete and in preventing buckling of the main column steel. The columns rapidly disintegrated resulting in the damage shown in Figure 9-7.

Damage in the form of shear cracking was also observed in some of the columns in bent 4 (Figure 9-8), but visible damage in the remaining columns of the bridge was limited to minor spalling and flexural cracking near the top of the columns (Figure 9-9). Distribution of column damage is shown in Figure 9-10.

9.4 Failure Analysis

The column failures at bent 3 were most likely caused by excessive shear forces. Flexural yielding in the columns resulted in degradation of shear capacity while producing relatively high ultimate shear demands. To demonstrate this effect, the columns in the right half of the bridge were analyzed for both moment and shear.

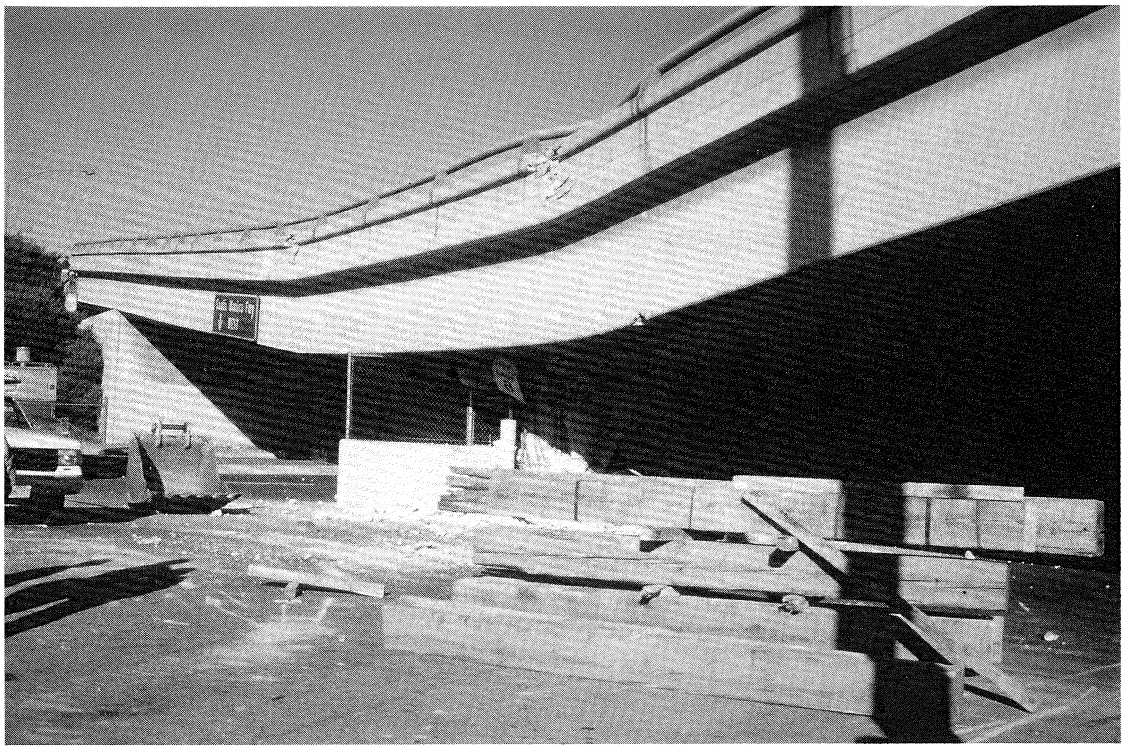


FIGURE 9-3 Fairfax-Washington Undercrossing - Side View of Partially Collapsed Eastbound Bridge

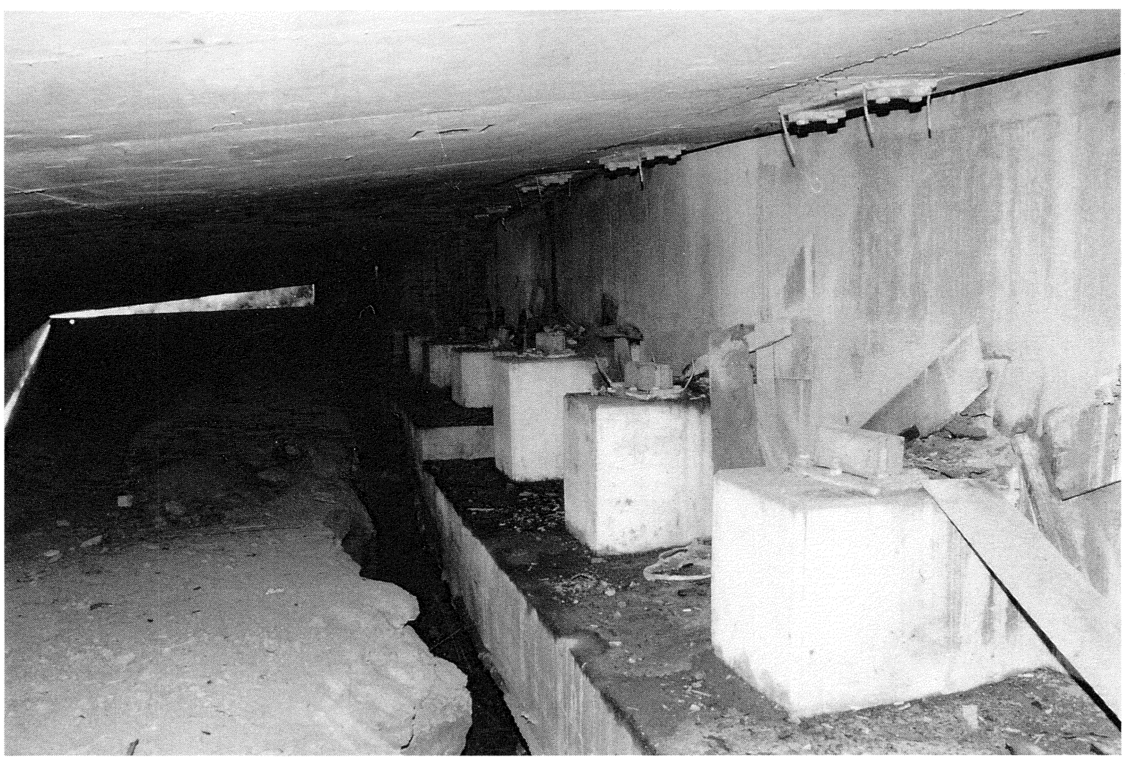


FIGURE 9-4 Fairfax-Washington Undercrossing - Uplift from Seat at Abutment 1

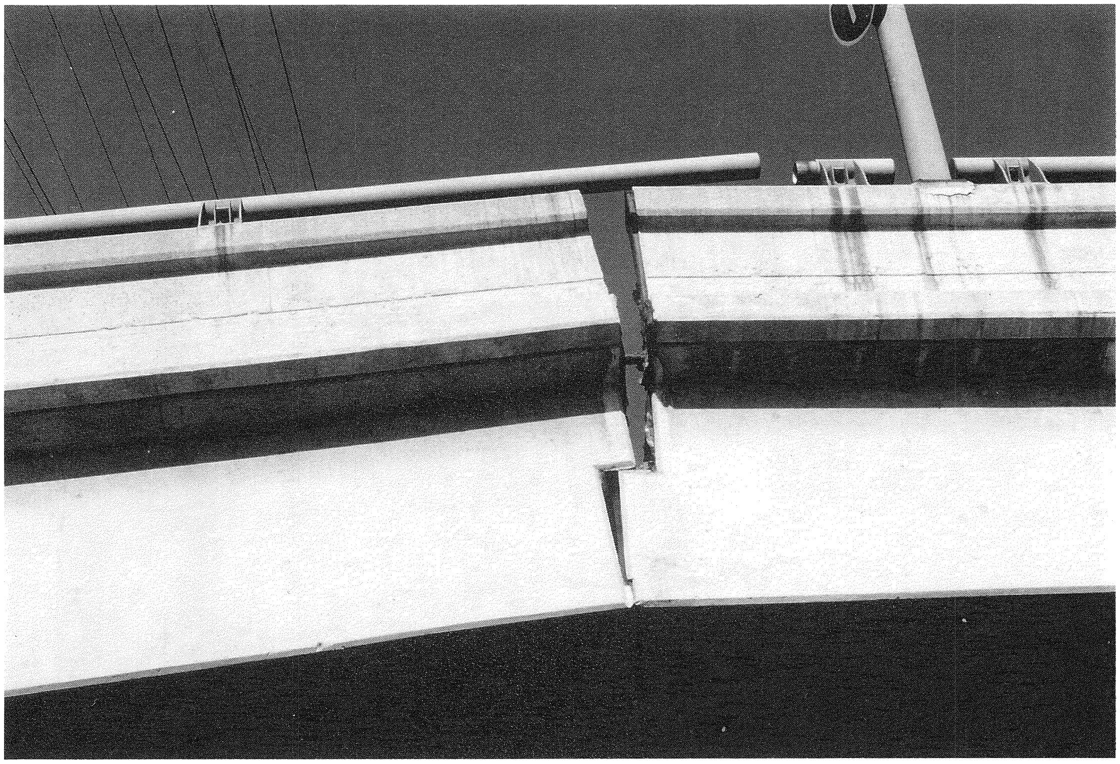


FIGURE 9-5 Fairfax-Washington Undercrossing - Rotation at Hinge in Span 3

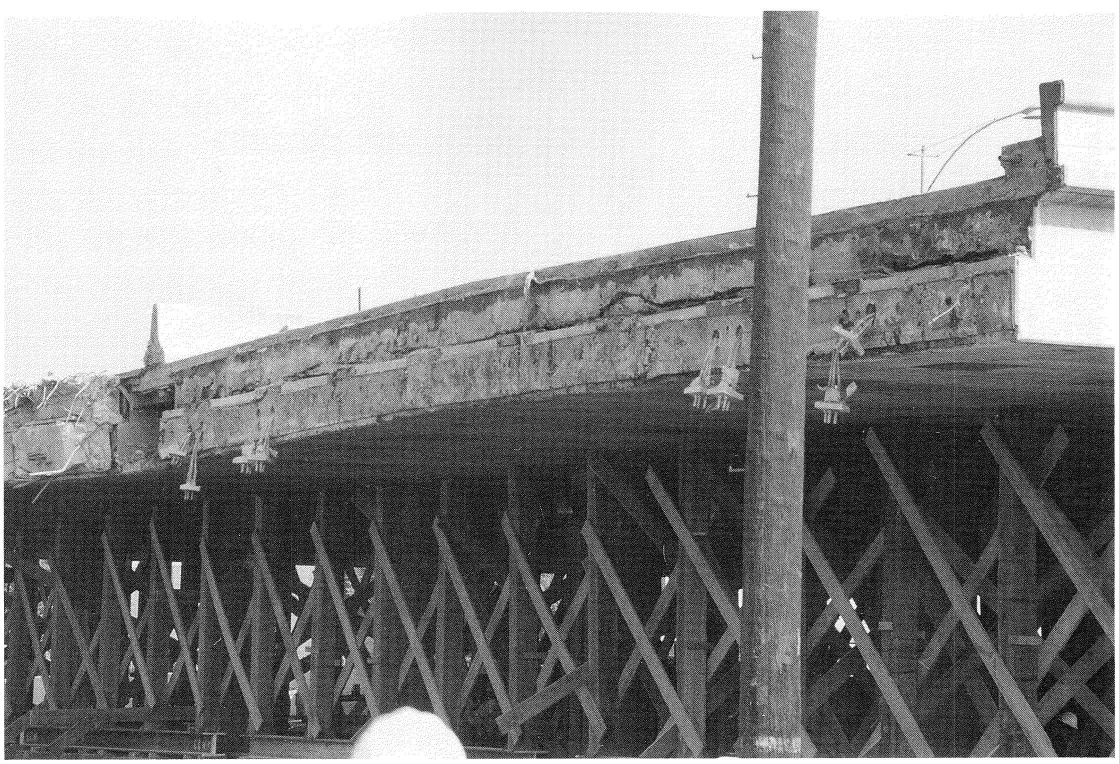


FIGURE 9-6 Fairfax-Washington Undercrossing - Hinge Seat and Restrainers in Span 3 after Demolition

FIGURE 9-7 Fairfax-Washington Undercrossing - Total Column Failure in Bent 3

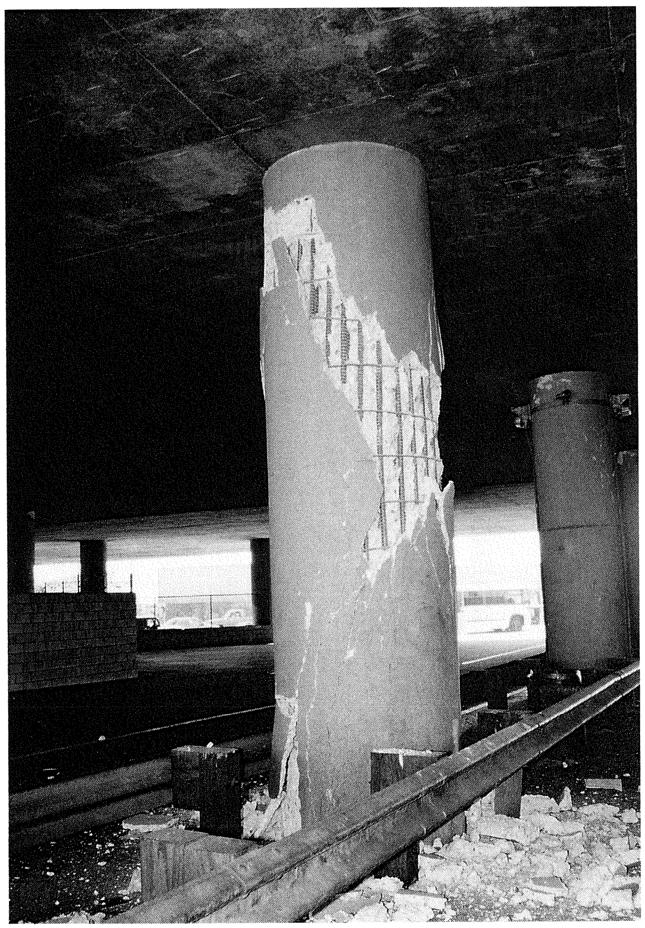


FIGURE 9-8 Fairfax-Washington Undercrossing - Shear Cracking in Column in Bent 4

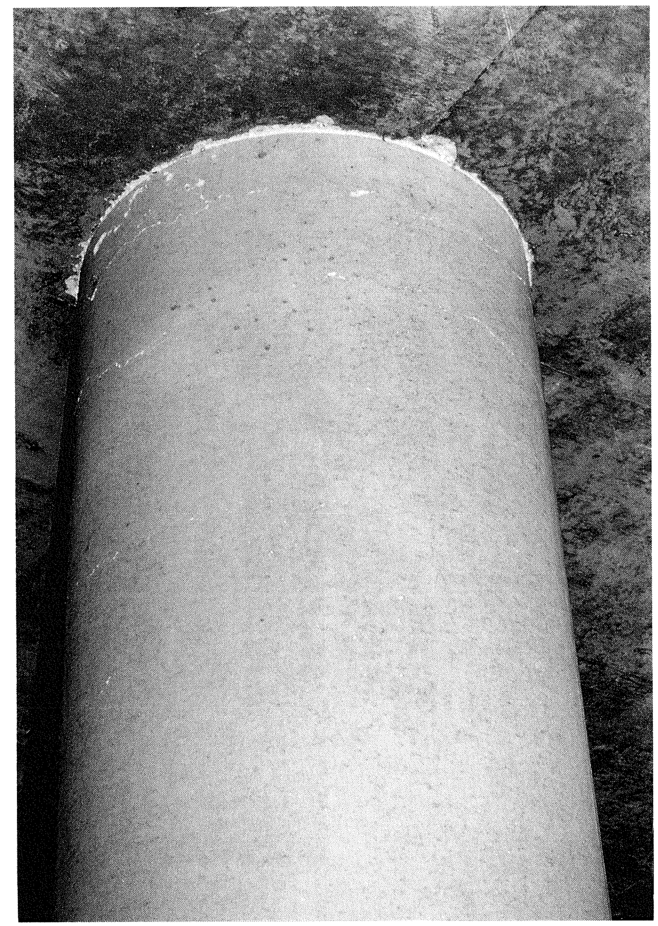


FIGURE 9-9 Fairfax-Washington Undercrossing - Minor Column Damage in Bent 5

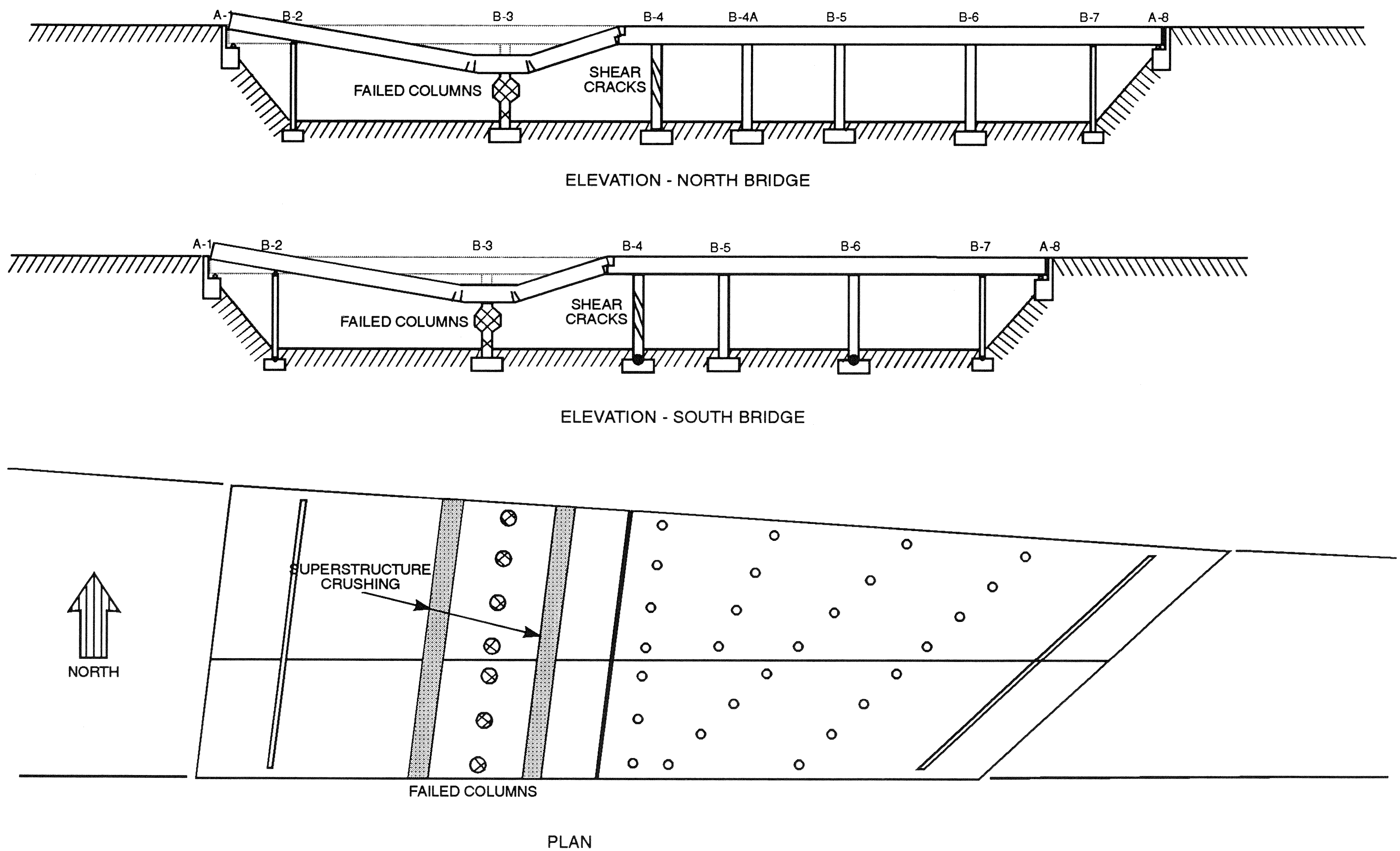


FIGURE 9-10 Fairfax-Washington Undercrossing - Distribution of Column Damage

A response spectrum analysis of the bridge was performed to determine the approximate extent of flexural yielding in the columns during the earthquake. The Caltrans' design spectra for 0.2g peak rock acceleration and 80 to 150 feet of alluvium [8] was used to simulate the ground motion thought to be most likely to have occurred at the bridge site.

Columns were assumed fixed at the base even though the main reinforcement was spliced at this location. Those columns for which the main steel stopped at the footing were assumed to be pinned. The ratio of the elastic moment from the response spectrum analysis to the ultimate moment capacity at the axial compression produced by dead load was calculated as an indicator of ductility demand. This is similar to the "Z" factor defined in the Caltrans Bridge Design Specifications [8].

Column shear capacity was calculated using methods proposed by researchers at the University of California at San Diego [10]. This method considers shear capacity contributions from concrete, steel and axial force. Ultimate shear demands were calculated using statics by assuming that the nominal ultimate moment was present at all potential plastic hinge locations. No capacity reduction factors were used in calculating shear capacity and no multiplier to account for strain hardening of the steel was used in calculating ultimate shear demands. The results of the column analysis are shown in Table 9-I.

TABLE 9-I Column Shear Capacities in Eastbound Bridge at Fairfax-Washington

Bent No.	**Column Shear (kips)**		**Capacity/Demand for Shear**	**Effective "Z"**
	Ultimate Demand	Capacity (UCSD)		
3	370	292	0.79	3.45
4	290	391	1.35	1.31
5	300/194	303/196	1.01/1.01	2.95/6.02
6	99	176	1.78	4.33

The capacity/demand ratio for shear at bent 3 clearly shows the potential for a shear failure. Although bent 4 columns would have been expected to perform well based on the above analysis, the column disintegration at bent 3 would have shifted forces to this bent resulting in higher ductility demands and a disintegration of column shear strength. This would explain the bent 4 column failures.

Bents 5 and 6 performed well despite relatively high calculated ductility demands and physical evidence of flexural yielding. Both of these bents apparently experienced ductility demands similar to bent 3. However, they each had better shear capacity/demand ratios. This seems to suggest that even poorly confined columns are capable of sustaining relatively high ductility demands when shear forces are low.

The notion of relative shear strength was the basis for the column screening procedure presented in the FHWA Seismic Retrofitting Guidelines for Highway Bridges (also known as ATC-6-2) [11]. This procedure uses a column base vulnerability rating (BVR) based on simple column parameters that can be quickly determined from the plans without doing an analysis. This method predicted the vulnerability of columns in this bridge with good accuracy as shown in Table 9-II.

TABLE 9-II FHWA Column Vulnerability Ratings of Eastbound Bridge at Fairfax-Washington

Bent No.	BVR
3	6.8 to 8.2
4	6.7 to 7.5
5	0 to 5.7
6	0

Note BVR =	0	implies no damage
	2.5	implies minor damage (spalling)
	5	implies moderate damage
	7.5	implies major damage
	10	implies severe damage

9.5 Issues/Questions

The lesson from this failure and that of the adjacent La Cienega-Venice Undercrossing is that structures with multi-column bents may be vulnerable to severe damage even when ground motions are well below current design levels. Once the shear capacity of a poorly confined column is exceeded, it disintegrates rapidly. Clearly higher priority must be given to screening and retrofitting bridges with vulnerable multi-column bents, especially those on important routes.

The column failures at bent 3, and thus the failure of this bridge, can be explained by high shear forces. However, the high bending ductility demands in the lightly reinforced columns at bents 5 and 6 coupled with their minimal transverse reinforcement would seem to have been the formula for serious column failure. The fact that these columns suffered very little damage suggests an interaction between relative column shear strength and the bending ductility demands that columns are capable of sustaining. Laboratory research is needed that focuses on this issue and seeks to develop a rational procedure for practitioners to better evaluate existing columns.

This bridge failure also demonstrates the importance of expansion joint hinge restrainers in protecting life. Although the bridge was severely damaged, total collapse was prevented. The situation may have been different if hinge restrainers had not been used, since the failure of bent 3 would have resulted in movements at the expansion joint hinge sufficient to cause it to become unseated.

SECTION 10
LA CIENEGA-VENICE UNDERCROSSING - BRIDGE NUMBERS 53-1609 AND 1609S

10.1 Description

This undercrossing carries both east- and west-bound Interstate 10 over Venice and La Cienega Boulevards near Culver City in west-central Los Angeles County. It is approximately one-half mile west of the Fairfax-Washington Undercrossing that also partially collapsed. The bridge was originally constructed in 1964, and subsequently retrofitted with cable restrainers in 1978.

The bridge superstructure, between piers 2 and 9, consists of two nine-cell cast-in-place, reinforced-concrete box girders, varying from 70 feet to 94 feet in width. A general plan and typical section is given in Figure 10-1. Damage sustained during the earthquake is illustrated in Figures 10-2 through 10-7. Detailed plans, sections and elevations are presented in Figures 10-8 to 10-16. These details are generally representative of Caltrans' design and construction practice in the mid-sixties.

The east- and west-bound bridges are separated by a longitudinal expansion joint that runs down the median of the freeway. Both end spans consist of slab and girder construction with a span length of approximately 51 feet (Figure 10-9). Abutment 1, pier 2 and bent 3 are nearly normal to the center line of the bridge, whereas from bent 4 to abutment 10 there is an increasing skew that is approximately 45° at abutment 10 (Figure 10-10). There is an off-ramp adjacent but structurally separate from the undercrossing on the south side. There are two expansion hinges, one between bents 3 and 4, and the other between bents 6 and 7 (Figure 10-10). From bents 3 to 7 there are storage facilities located underneath most of the superstructure, and the concrete block walls enclosing these facilities prevented the total collapse of the superstructure. The total length of the bridge was 870 feet, with span lengths of 51, 116, 93, 111, 112, 105, 116, 115, and 51 feet, measured from abutment 1.

Bents 3 to 7 have three columns of support for each 70 feet to 90 feet width of superstructure, and bent 8 has four columns of support (Figures. 10-11 through 10-16). All columns are four ft. in diameter, but with significant variations in the longitudinal steel. Confinement/shear reinforcement is a No. 4 bar at 12 inches or a No. 4 spiral with a 12 inch pitch at each column location. All columns are supported on piled foundations of approximately 35 ft. depth.

The soil profile consists of an increasingly dense, sandy, silt soil with gravel down at least 70 ft. where the site borings ended.

10.2 Ground Motion

The structure is located approximately 17 miles southeast of the epicenter of the main shock. Two free-field strong motion recorders are located within a few miles of the bridge site.

The first instrument (CSMIP Sta. No. 24157), which is located two miles to the southeast in Baldwin Hills on one meter of fill over shale/sandstone, recorded approximately ten seconds of strong shaking with a peak horizontal acceleration in the east-west direction of 0.24g and 0.17g in the north-south direction.

The second instrument (CSMIP Sta. No. 24389) also recorded approximately ten seconds of strong shaking. The peak horizontal acceleration was 0.27g in the east-west direction and 0.24g in the north-south direction. This instrument is located approximately four miles northwest of the bridge and is supported on terrace deposits.

Motions recorded approximately four miles away on a bridge structure within the I-10/I-405 Interchange are consistent with the magnitude of horizontal ground motion recorded at the above two free-field sites.

10.3 Observed Earthquake Damage

This structure suffered major damage. Several or all columns at each bent, from bent 3 to bent 8, suffered shear cracking, flexural hinging, crushing of concrete, and symmetrical, longitudinal bar-buckling. The shorter columns on the south side of bent 3 failed (Figure 10-2), and concrete spalling was evident at the top of the single-column support of the off ramp (Figure 10-2). All columns at bent 3 were designed to be fixed at their base. The majority of the columns at bents 4, 5 and 6 appeared to have failed, but most were inside the storage facility and not able to be inspected. Figure 10-3 is typical of the damage to the north side columns at these bents. This figure also shows the storage facility supporting the superstructure between the hinge locations (bents 4 to 7). A view of the superstructure is shown in Figure 10-4. The damage to the columns at bent 7 was unusual (Figure 10-5). In this bent, flexural/shear hinges had developed at the tops of the first (Type M) and third (Type H) columns from the north side, and at the base of the second (Type L) column. Figure 10-16 is not explicit with regard to the column fixity (Types M and H), but it does show that Type L is fixed at its base. The three columns on the south structure did not show the same significant damage, but they were shored as shown in Figure 10-5. The hinge adjacent to bent 6 separated (Figure 10-6) due to the collapse of the central segment of the bridge. It was noticed that some of the high strength rod restrainers had failed while anchor nuts were missing on others. The columns at bent 8 (four columns per superstructure) showed significant shear cracks and concrete spalling (Figure 10-7) and were shored.

10.4 Issues/Questions

1. The lesson from this failure is that multi-columns bents are vulnerable to severe damage even at relatively low ground accelerations. Once the shear capacity is exceeded, the column disintegrates rapidly.

2. The columns on this bridge suffered very significant damage. The bridge immediately to the east carrying I-10 over Ballona Creek (Bridge Number 53-1579) suffered negligible damage to its columns. It appeared to be of similar construction, but the columns were perhaps 20% to 30% taller. This bridge is actually located between La Cienega-Venice and Fairfax-Washington on I-10. The input ground motions can therefore be assumed to be similar but the performance is markedly different. A detailed study of the La Cienega-Venice Undercrossing should therefore also include a comparison with the performance at Ballona Creek.

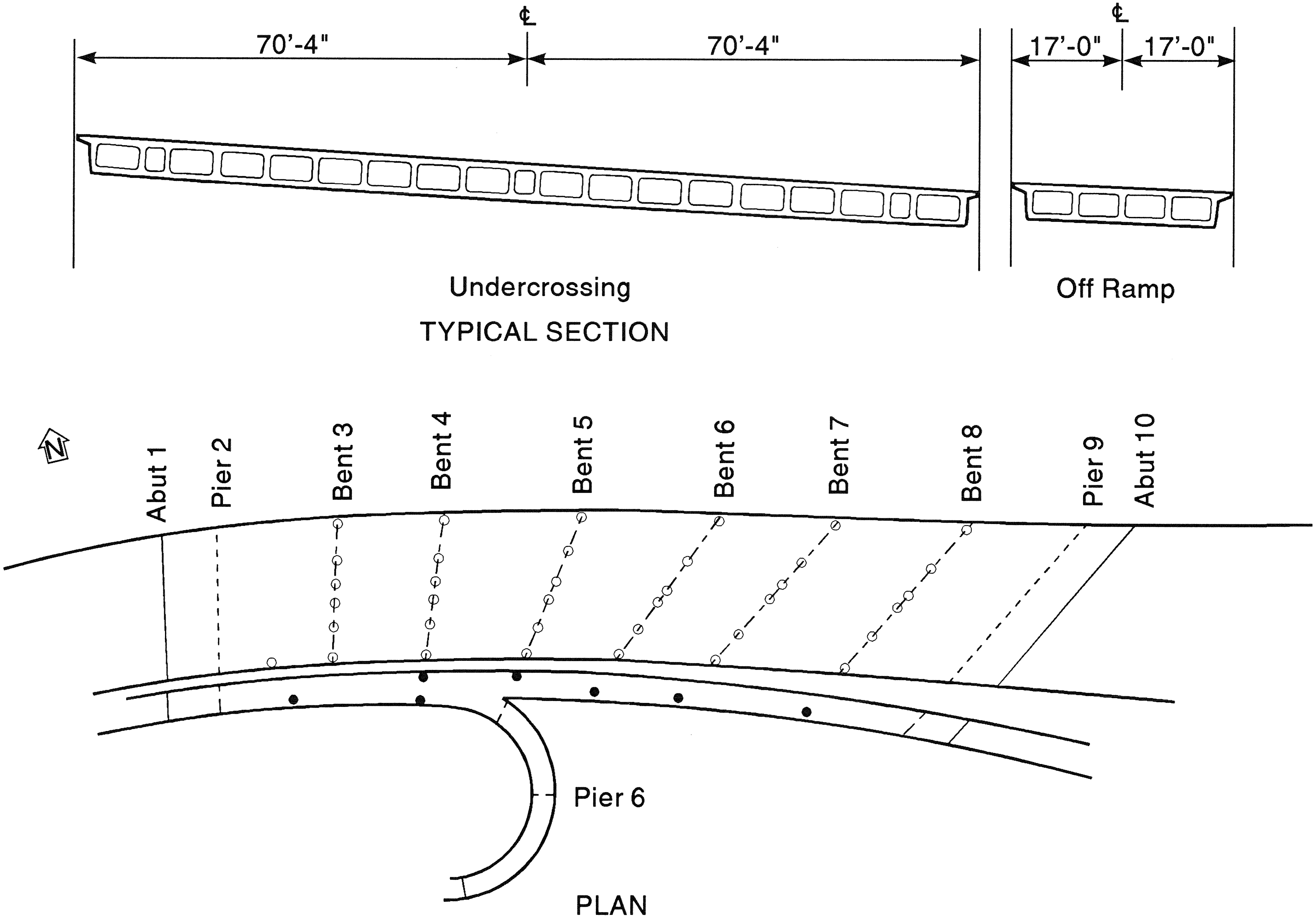


FIGURE 10-1 La Cienega-Venice Undercrossing - General Plan and Elevation

FIGURE 10-2 La Cienega-Venice Undercrossing - Column Failure in Bent 3

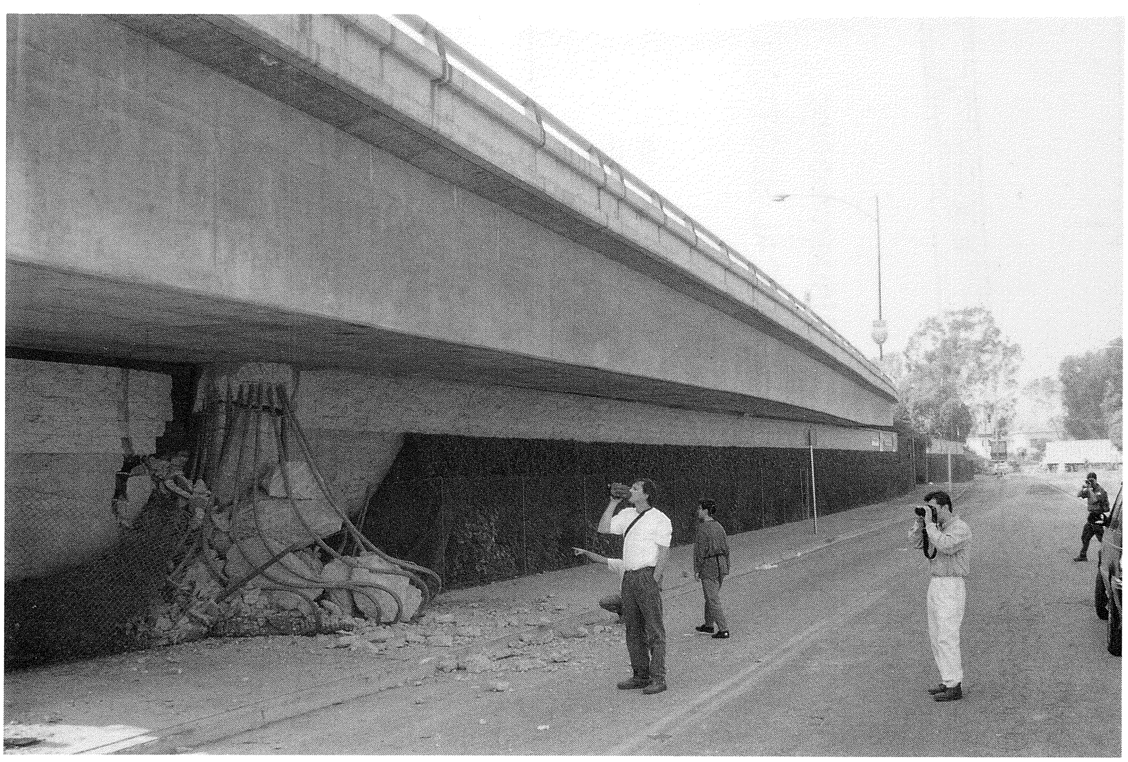


FIGURE 10-3 La Cienega-Venice Undercrossing - Column Damage in Bents 4, 5 and 6 Partially Hidden from View by Storage Facility Under Bridge

FIGURE 10-4 La Cienega-Venice Undercrossing -
Superstructure Settlement Due to Column Failures

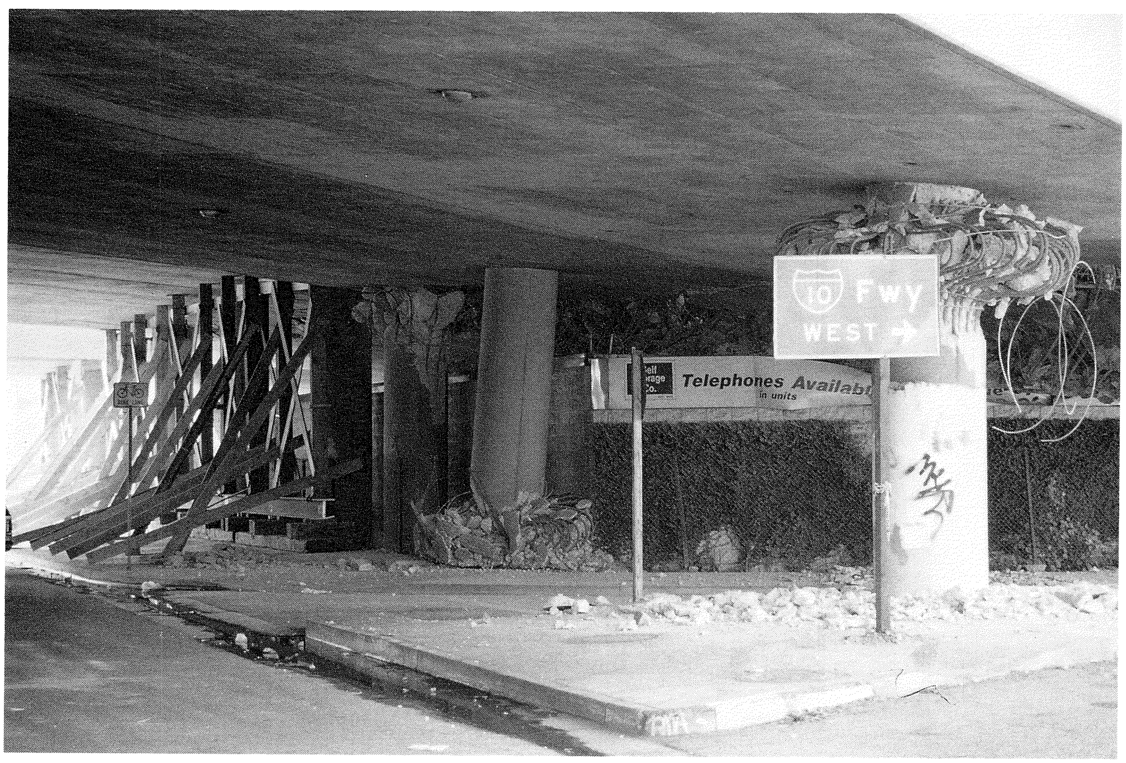


FIGURE 10-5 La Cienega-Venice Undercrossing -
Alternating Locations of Hinges in Columns of Bent 7

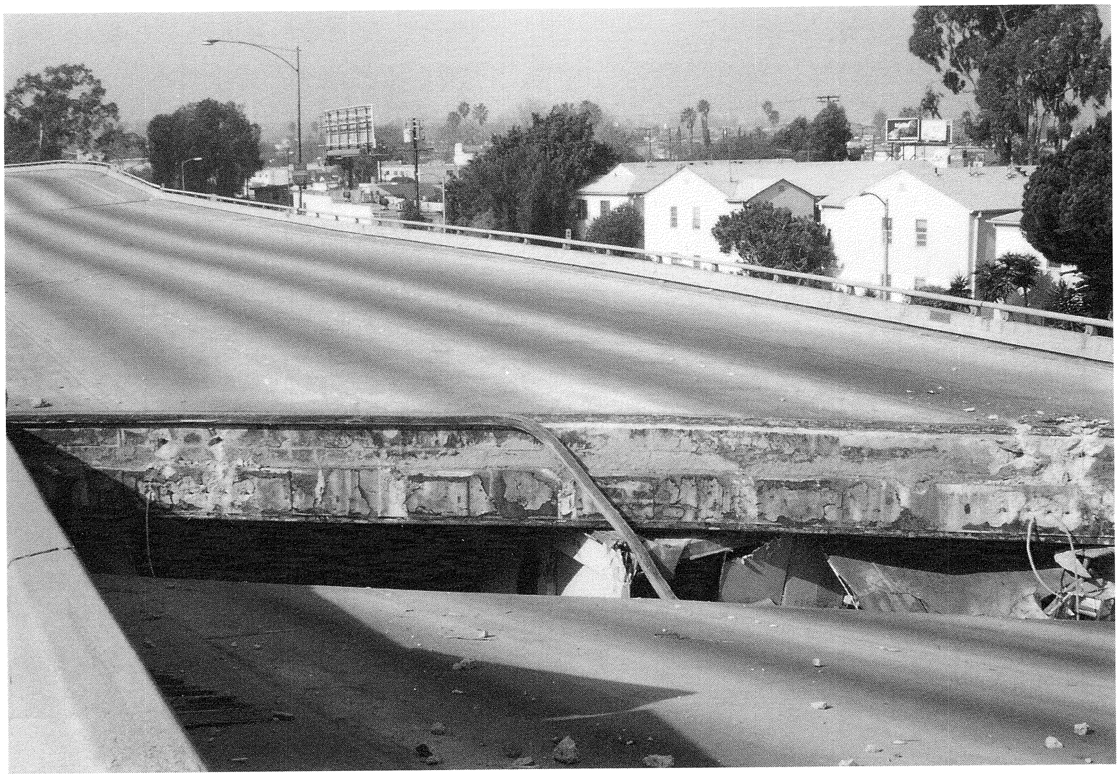


FIGURE 10-6 La Cienega-Venice Undercrossing - Unseated Hinge Near Bent 6

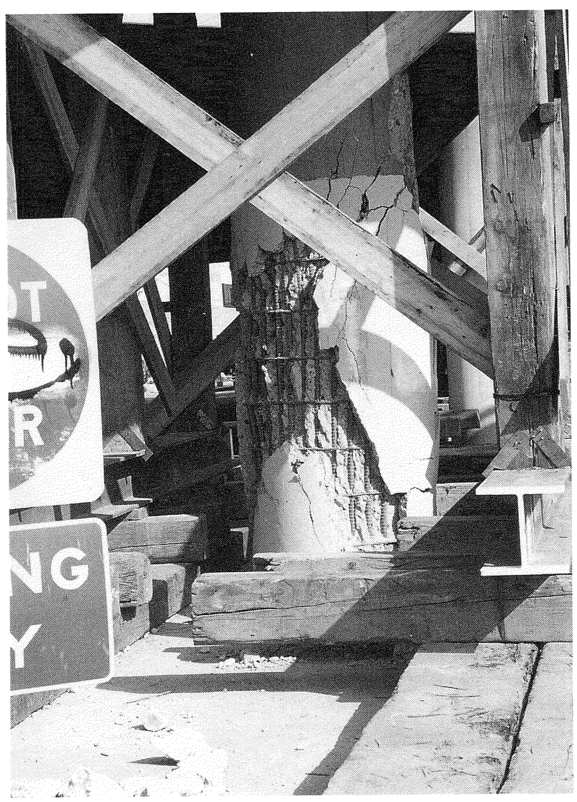


FIGURE 10-7 La Cienega-Venice Undercrossing - Shear Cracking in Column in Bent 8

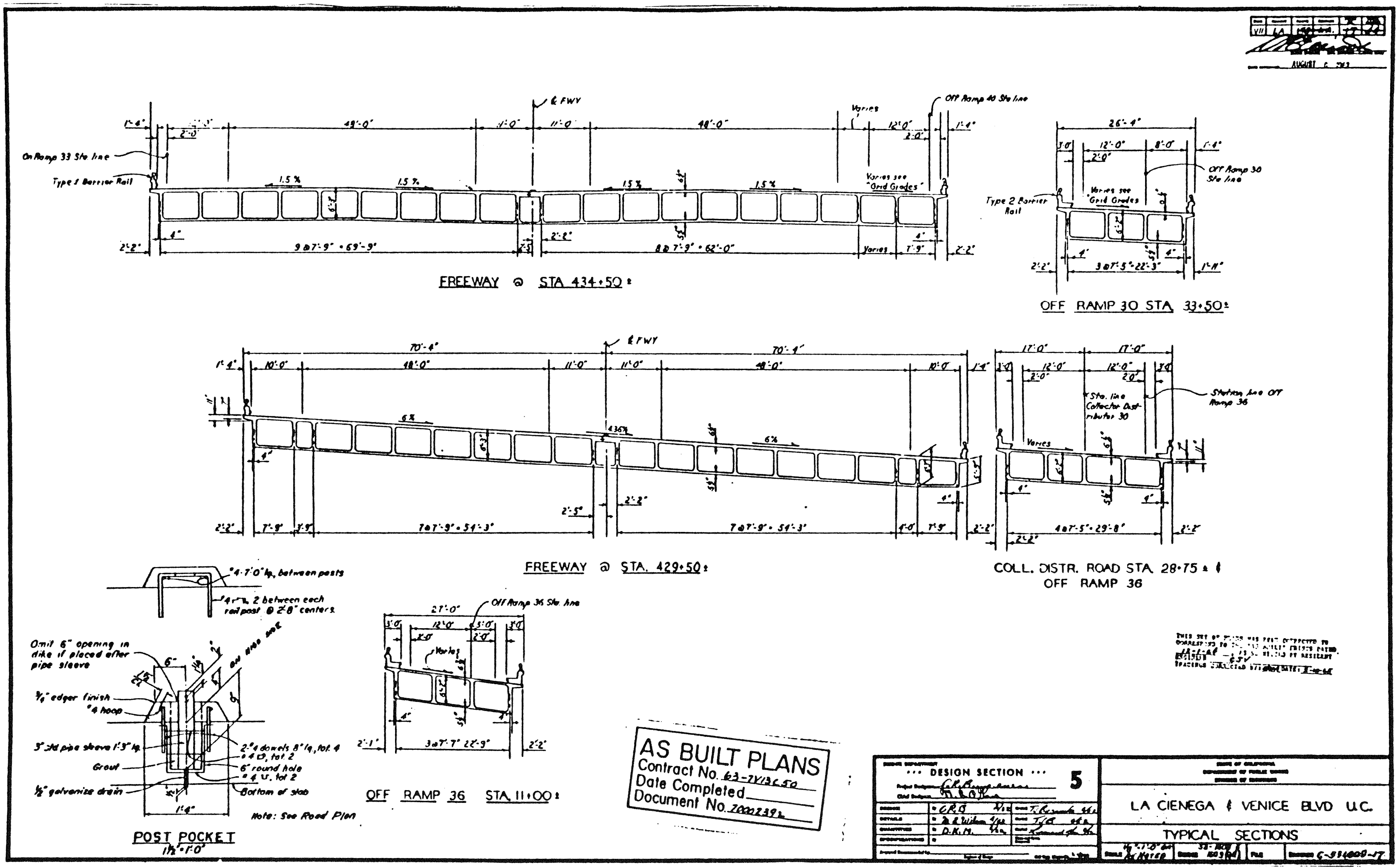


FIGURE 10-8 La Cienega-Venice Undercrossing - Typical Sections

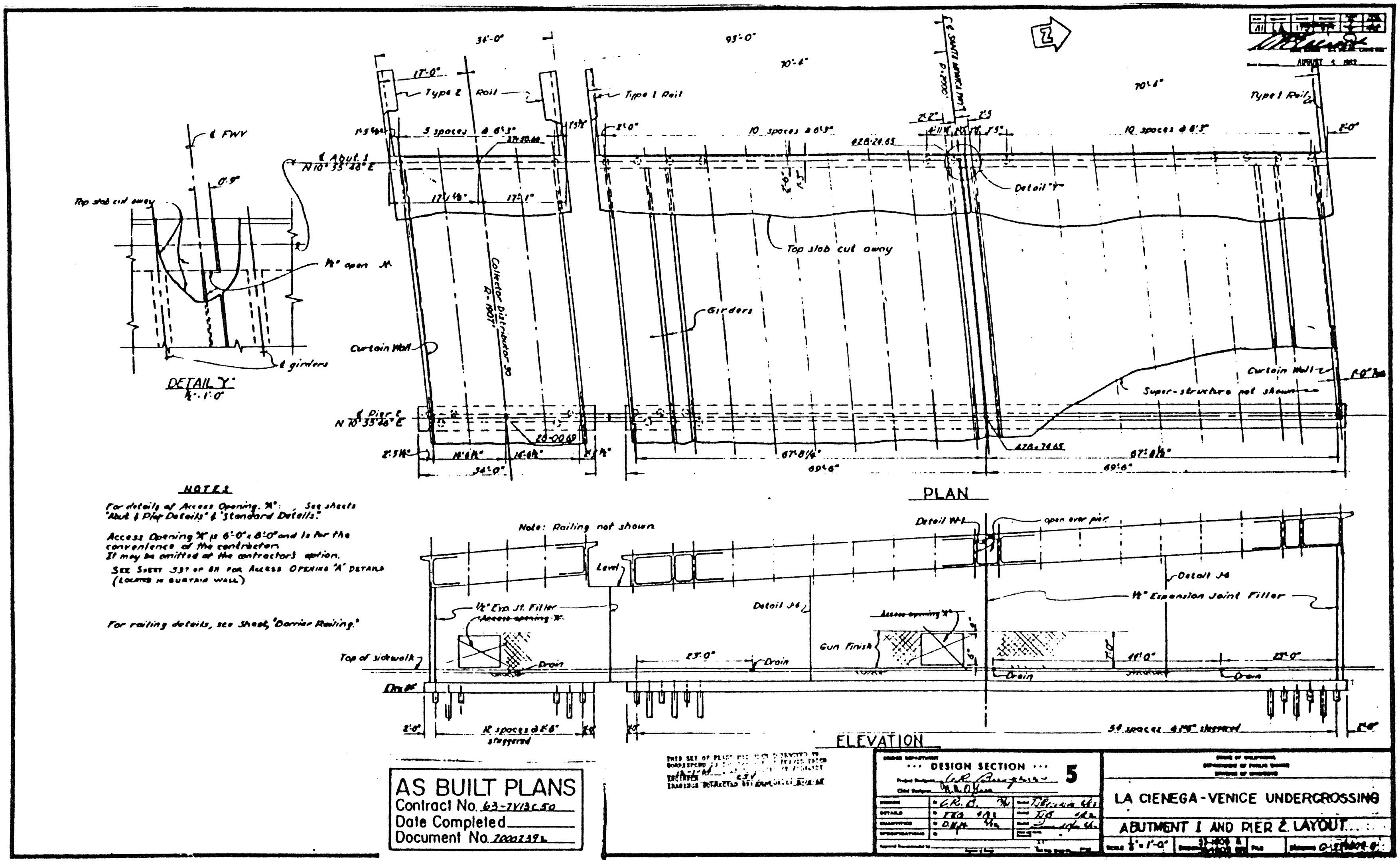


FIGURE 10-9 La Cienega-Venice Undercrossing - Layout of Abutment 1 and Pier 2

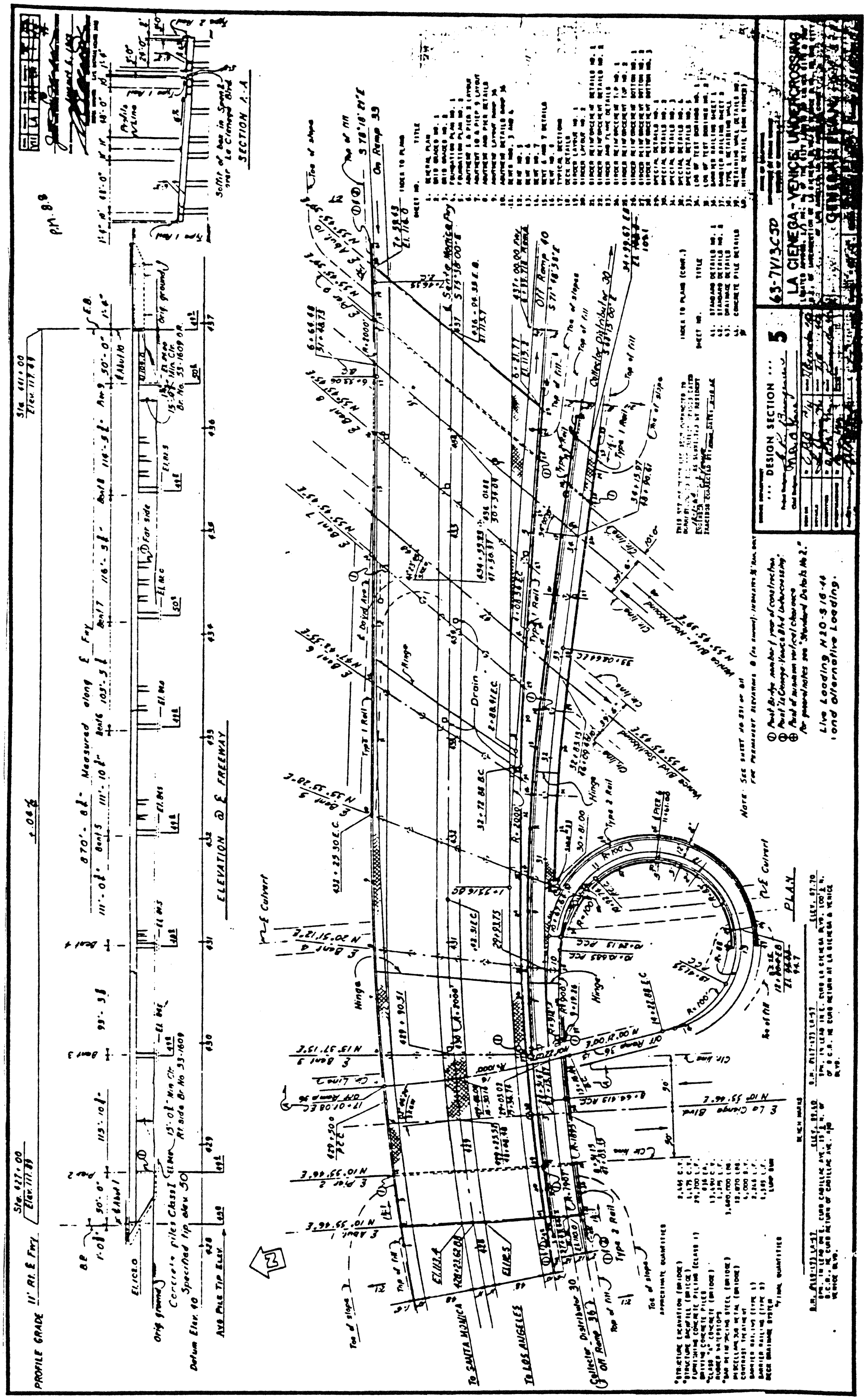


FIGURE 10-10 La Cienega-Venice Undercrossing - Plan and Elevation

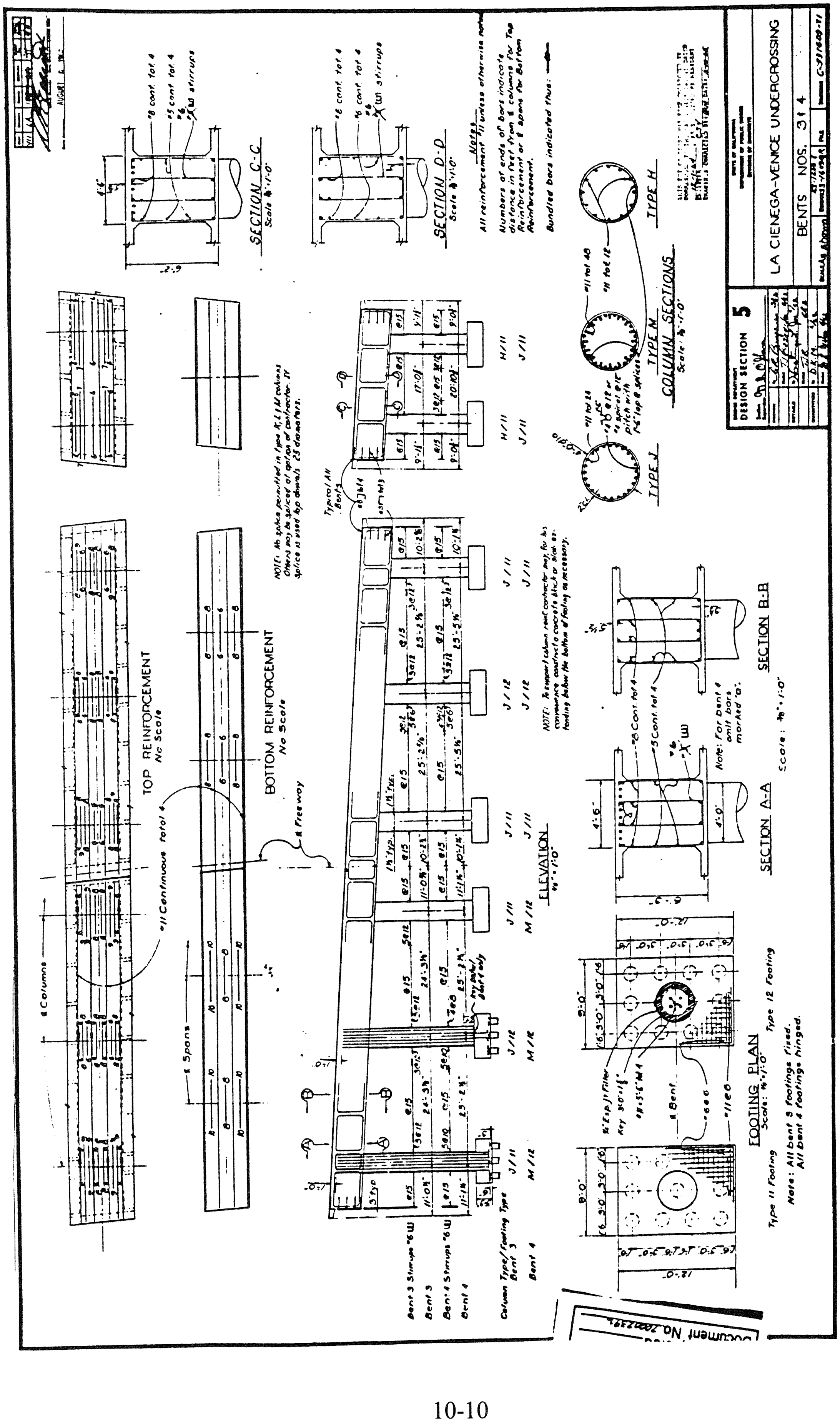


FIGURE 10-11 La Cienega-Venice Undercrossing - Bents 3 and 4 Details

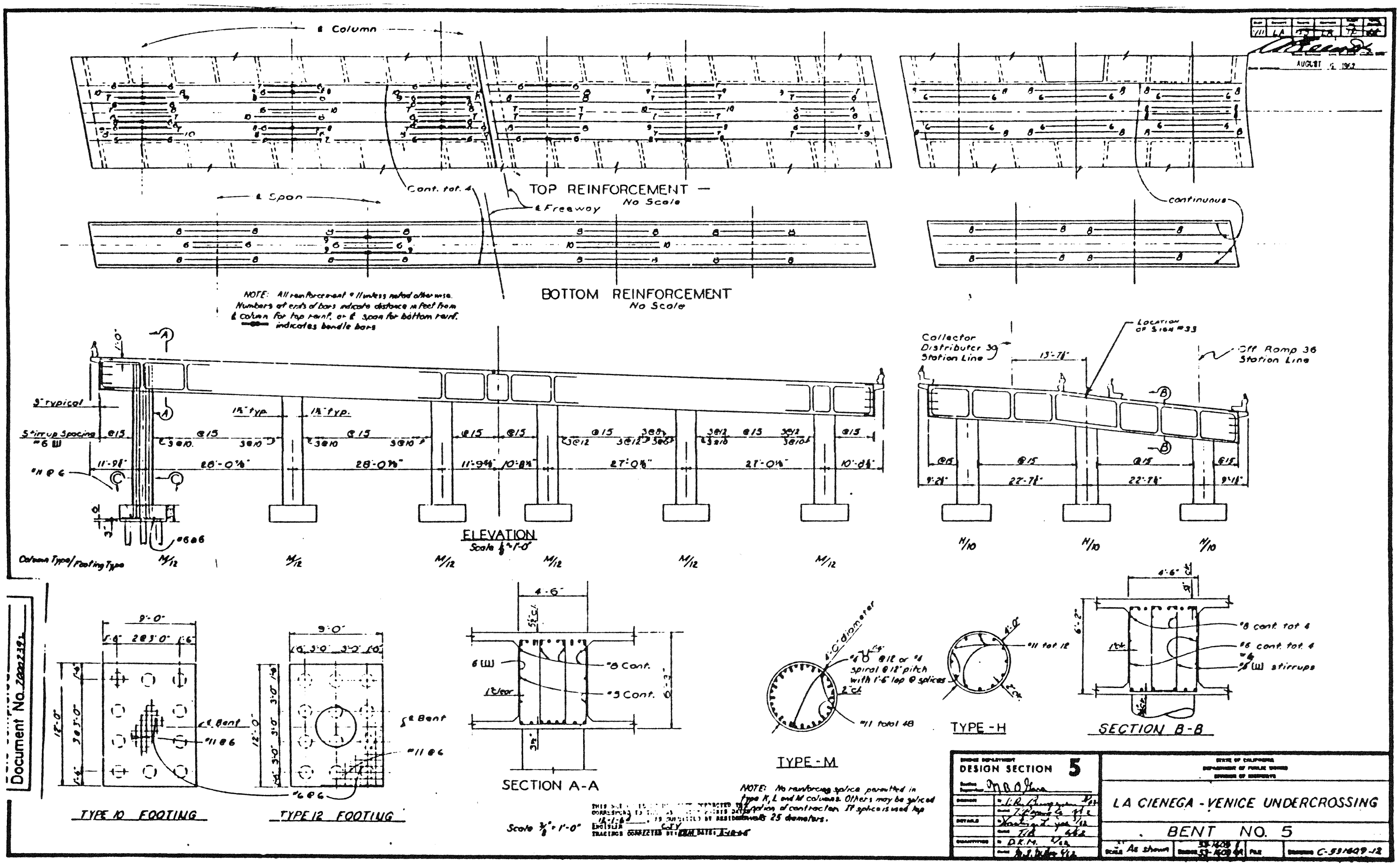


FIGURE 10-12 La Cienega-Venice Undercrossing - Bent 5 Details

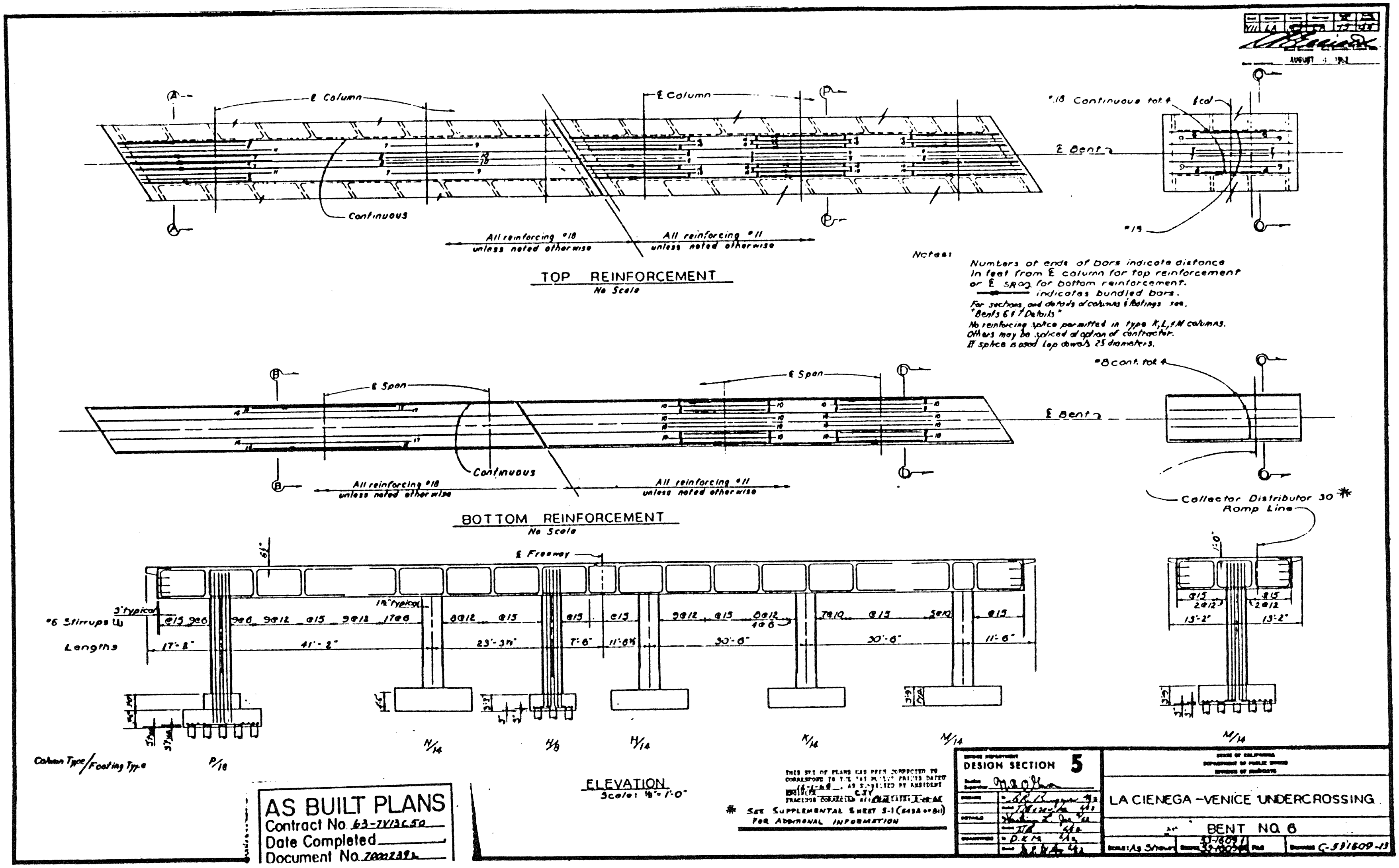


FIGURE 10-13 La Cienega-Venice Undercrossing - Bent 6 Details

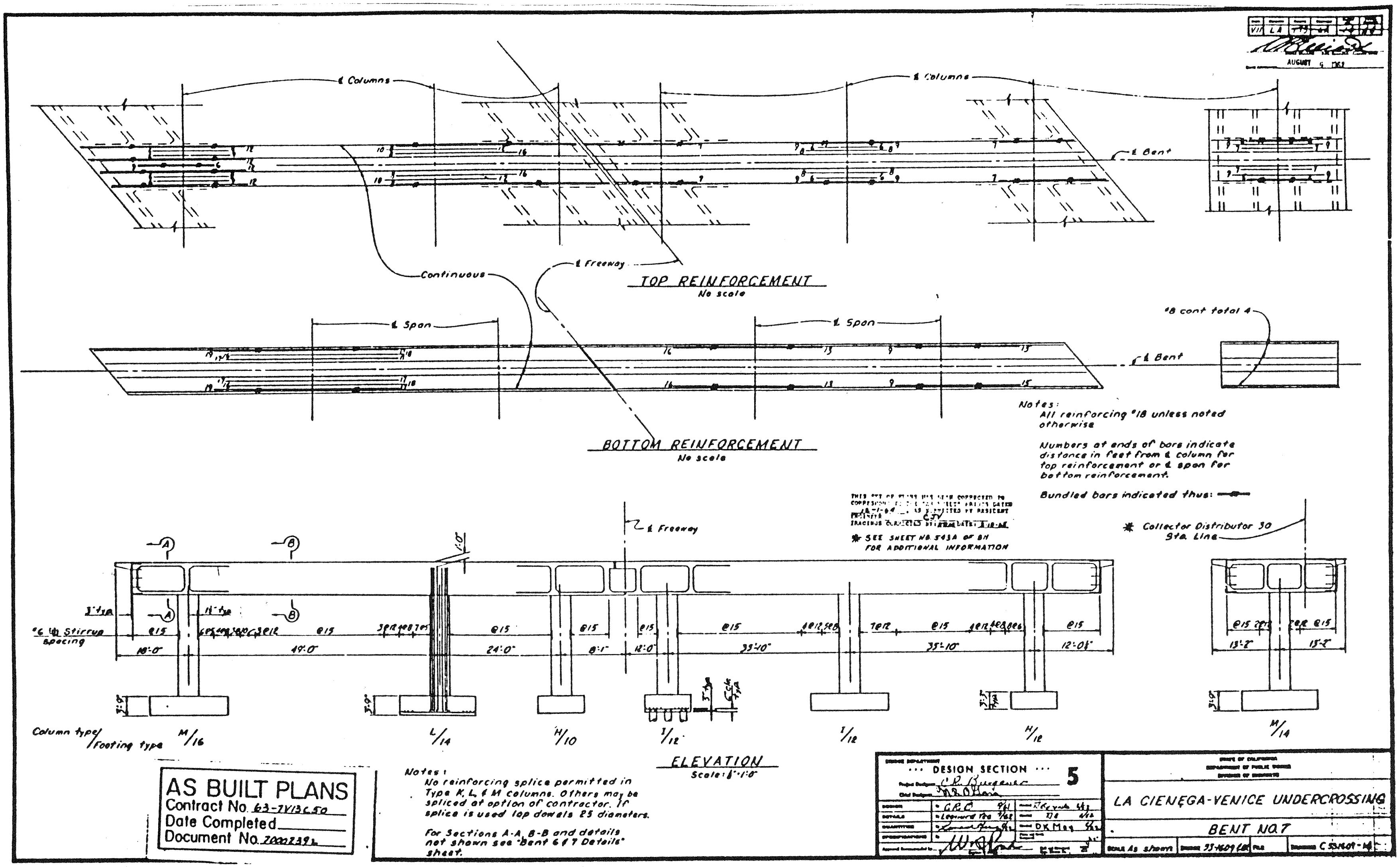


FIGURE 10-14 La Cienega-Venice Undercrossing - Bent 7 Details

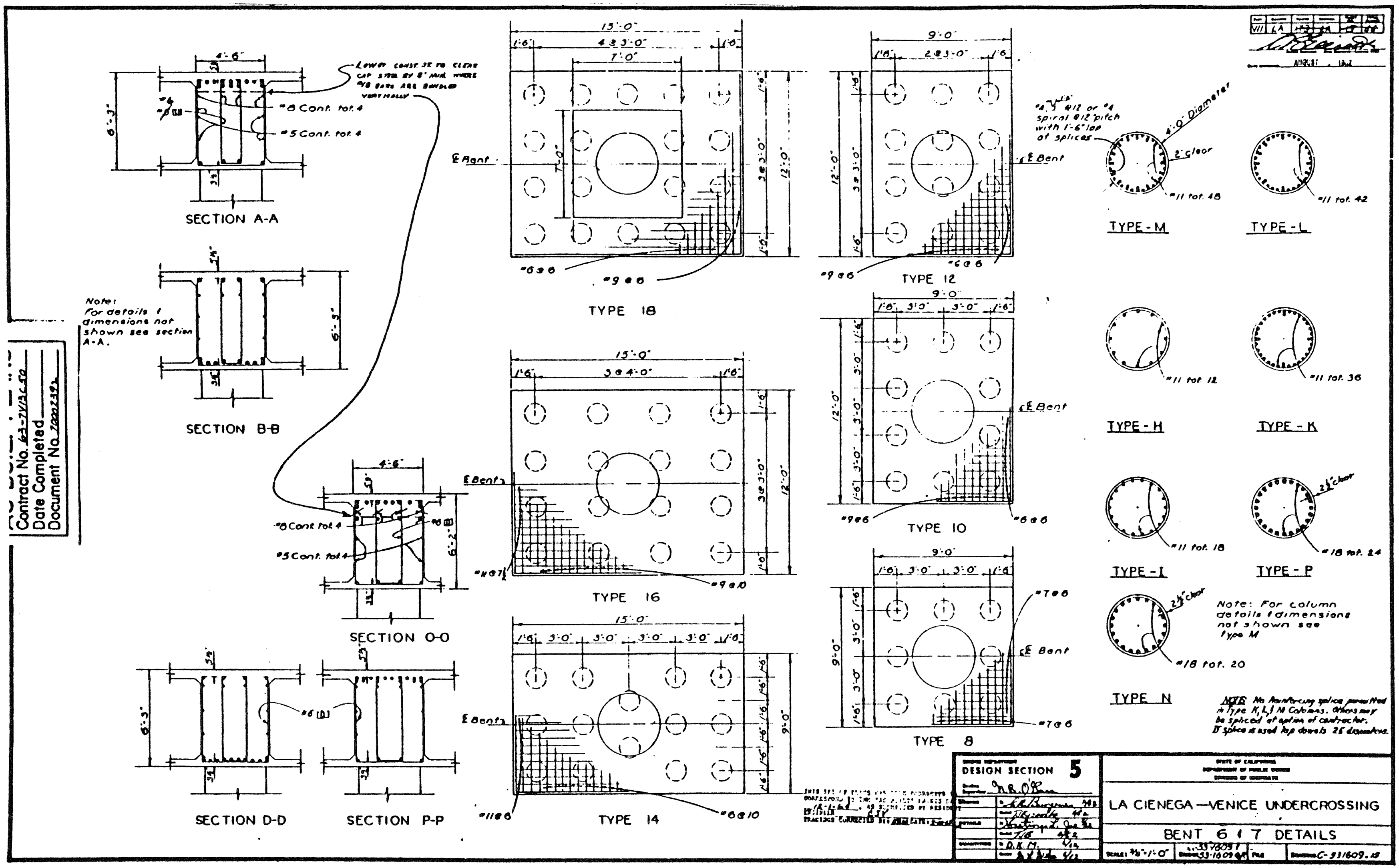


FIGURE 10-15 La Cienega-Venice Undercrossing - Bent 6 and 7 Details (continued)

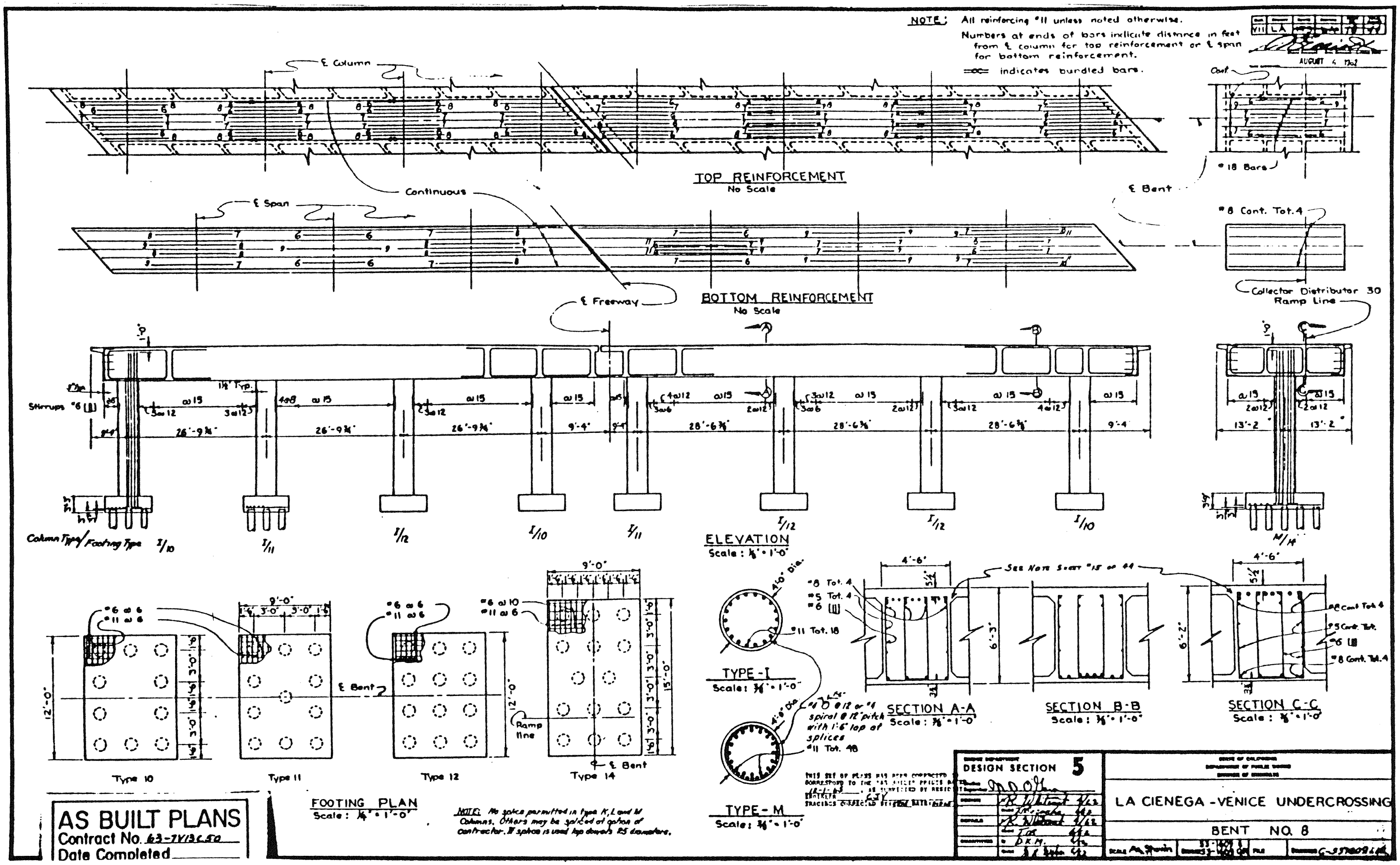


FIGURE 10-16 La Cienega-Venice Undercrossing - Bent 8 Details

SECTION 11
RETROFITTED BRIDGES

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 (Figures 4-4 and 11-1) 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. At the present time, it is not clear as to 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 have 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. Some were on bridges close to other structures which did partially collapse. One example is shown in Figure 11-2 which is a view of a jacketed single column under an off-ramp across Ballona Creek on I-10. It is located between the La Cienega-Venice and Fairfax-Washington structures. Close examination found no sign of damage.

FIGURE 11-1 SR14/I-5 Interchange -
Failure of a Type I Hinge Restrainer at the Swaged Fitting

FIGURE 11-2 Ballona Creek Undercrossing on I-10 -
Single Column Bent Retrofitted with Steel Jacket

SECTION 12
OTHER BRIDGE DAMAGE

A complete list of damaged bridges as of February 1, 1994 is given in Table 12-I. There are 176 bridges on this list with damage ranging from minor spalling to collapsed spans. Some examples include:

- Interstate 5 at the San Fernando Road Undercrossings (abutment and wingwall damage and minor column spalling);
- Southwest Connector at the Interstate 5/Route 118 Interchange (column shear cracks);
- Interstate 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);
- Interstate 405 at the Jefferson Boulevard Undercrossing (outrigger joint cracking);
- The South Connector in the interchange between Interstate 5 and State Route 14 suffered severe pounding at the hinge seats and substantial damage to abutment 10 (Figures 12-1 and 12-2); 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 backwalls of many bridges (e.g., at the Bull Creek and Havenhurst Bridge sites on SR118). In instances where approach slabs were used, and tied to the backwalls, access was not impaired. However, slumping in the shoulders and emergency stopping lanes will require repair (Figures 12-3 and 12-4).

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

FIGURE 12-1 SR14/I-5 South Connector - Pounding and Damage to Hinge Seat in Span 6

FIGURE 12-2 SR14/I-5 South Connector - Damage to Abutment 10

(a) Single Span Bridge with Integral Abutments Sustained Minor Damage

(b) Slumping of Abutment Fills under Roadway Shoulders; Approach Slab in Foreground Successfully Spans Settlement

FIGURE 12-3 Havenhurst Undercrossing on SR 118

FIGURE 12-4 Bull Creek Canyon Channel Bridge - Slumping of Abutment Fills under Roadway Shoulders

TABLE 12-I List of Damaged Bridges
(From Caltrans as of February 1, 1994)

LA	1	39.62	CASTELMARE POC	53-0068	Very minor spall at east abutment seat.
LA	2	R018.81	NE CONNECTR OC	53-1921F	Shear keys cracked and spalled.
LA	5	17.21	HOLLENBECK LAK	53-1359	Keeper plate bolts sheared at abutment 1.
LA	5	17.56	FOURTH ST UC	53-1304	Soundwall damage adjacent to bridge.
LA	5	18.38	S CONNECTOR UC	53-1316	Minor cracks at exterior shear keys at both abutments.
LA	5	18.52	ECHANDIA OH	53-1333	Minor Bearing damage.
LA	5	18.53	ECHANDIA OH	53-1332F	Minor crack at abut 9 curtain wall.
LA	5	18.62	MISSION RD R U	53-1317F	Spalls at abutment 1 backwall. Sheared keeper plate bolts.
LA	5	18.78	MISSION RD UC	53-1312	Broken 2" utility pipe supports. Wingwall separated from abutment.
LA	5	18.96	ALHAMBRA AV OH	53-0368	Spalls under three girders at abutment 5.
LA	5	20.31	ELYSIAN VIAD	53-1424	Spalls at columns 3 and 4 at bent 4. Minor soffit and shear key spalls.
LA	5	24.61	GRFTH PK OR OC	53-1181S	Joint seal (Type A) damage at hinge in span 3.
LA	5	29.16	OLIVE AVE OC	53-1087	Minor spalling at hinge.
LA	5	29.39	MAGNOLIA BV OC	53-1088	Minor damage at pier cap.
LA	5	34.65	TUXFORD ST UC	53-1117	Minor damage of closure wall at abutment.
LA	5	36.34	SHARP AVE ONRP OC	53-1222k	Sheared keeper plate anchor bolts. Minor spalls on outrigger.
LA	5	38.5	VAN NYS BL UC	53-1126	Abutment 2 embankment washed out due to broken water line.
LA	5	39.19	PACOIMA WASH	53-1128	Columns damaged. Shoring required.
LA	5	39.26	PACOIMA WASH	53-2346F	Minor spalling at abutment 1.
LA	5	39.30	SOEAST CONN OC	53-2327F	Moderate damage at both abutments. Damage at hinges.
LA	5	39.31	SOWEST CONN OC	53-2329G	Major abutment damage.
LA	5	39.36	NE CONN OC	53-2330F	Hinge damage in span 5. Spalls at abutment joints.
LA	5	39.98	BRAND BLVD UC	53-1130	Shear cracks and spalling in external shear keys at bents 2 & 5.
LA	5	40.46	RINALDI ST UC	53-1132	Shear cracks and spalling in external shear keys atabutment 1.

TABLE 12-I List of Damaged Bridges (continued)

LA	5	41.55	RTE 5T/405 SEP	53-1548	Cracks in columns.
LA	5	41.57	RTE 5/405 SEP	53-1133	Minor cracks in wingwall at abutment1.
LA	5	42.65	ROXFORD ST UC	53-1115	Minor cracks at curtain wall and shear key. Minor approach settlement.
LA	5	R043.84	SAN FERN RD OH	53-0730	Damage at all abutment shear keys. Minor top of column fractures.
LA	5	R044.01	RTE 210/5 SEP	53-1985F	Abutments damaged. Spalls at hinge joints.
LA	5	R044.43	BALBOA BV OC	53-1986	Minor column damage. Approach slab buckled.
LA	5	R044.87	W SYLMAR OH	53-1984L	Abutment and shear key damage. Spalling at hinges.
LA	5	R044.87	W SYLMAR OH	53-1984R	Abutment and shear key damage. Spalling at hinges.
LA	5	C045.49	SIERRA HWY SEP	53-0848	Bearing and shear key damage.
LA	5	C045.49	SIERRA HWY SEP	53-0848G	Minor shear key damage at both abutments.
LA	5	R046.58	WELDON CYN OC	53-1796	Minor column cracks.
LA	5	R047.83	GAVIN CANYON U	53-1797L	Partial collapse. Removal complete.
LA	5	R047.83	GAVIN CANYON U	53-1797R	Partial collapse. Removal complete.
LA	5	R049.03	CALGROVE BV UC	53-1792L	Approach settlement.
LA	5	R050.33	PICO LYONS OC	53-1783	Bearing damage.Cracks in girder web.Restrainer damage.
LA	5	R50.80	BUTTE CANYON	53-0387	Cracks and spalls in abutments and wingwalls.
LA	5	R051.44	MCBEAN PKWY OC	53-2057	Crack in abutment.
LA	5	R052.47	VALENCIA BV OC	53-1815	Major abutment damage.
LA	5	R053.70	SANTA CLARA R	53-0687L	Abutment and bearing damage at both abutments. Cracks in all piers.
LA	5	R053.70	SANTA CLARA R	53-0687R	Abutment and bearing damage at both abutments. Cracks in all piers.
LA	5	R056.12	HONOR RHO R OC	53-1807	Cracks and spalls at abutments and bents.
LA	5	R056.60	HASLEY CYN R O	53-1809	Cracks in columns.
LA	5	R059.49	LK HUGHES R UC	53-1908L	Minor approach settlement.
LA	10	R002.61	14TH STREET OC	53-1596	Minor bearing damage.

TABLE 12-I List of Damaged Bridges (continued)

LA	10	R003.34	CLOVERFIELD OC	53-1599	Abutment and shear key damage.
LA	10	R004.24	CENTLA-PICO UC	53-1603	Cracks and spalls in columns.
LA	10	R005.28	NORTHW CONN OC	53-1627G	Longitudinal displacement of two spans. Minor spalls.
LA	10	R005.65	SE CONNECTR OC	53-1637F	Cracks in bent 7 exterior girders. Unknown interior damage.
LA	10	R005.99	COVENTRY PL UC	53-1634	Curtain wall damage.
LA	10	R006.31	NATIONAL BL OC	53-1615	Bearing damage. Cracks in wing walls. Spalls on columns.
LA	10	R007.08	MANNING A R OC	53-1553S	Cracks in abutment diaphragm.
LA	10	R007.92	ROBERTSON-N UC	53-1557	Bearing damage. Cracked wingwall.
LA	10	R008.83	LA CNG-VEN SEP	53-1609S	Column damaged. Shoring required at four bents. Removal underway.
LA	10	R008.83	LA CNG-VEN SEP	53-1609	Partial collapse. Complete removal underway.
LA	10	R009.12	BALLONA CREEK	53-1579	Minor column spalls. Crack in wingwall. Retaining wall damage.
LA	10	R009.22	CADILLAC RP SP	53-1485F	Crack in slope paving. Approach settlement.
LA	10	R009.31	FAIRFAX WASH U	53-1580	Partial collapse. Removal complete.
LA	10	R009.74	HAUSER BLVD UC	53-1582	Cracks and spalls in wingwalls.
LA	10	R010.43	LA BREA AVE UC	53-1586	Minor spalls at bent 2 and curtain wall.
LA	10	R10.72	HARCOURT AVE UC	53-1587	Minor crack at bent 3 curtain wall.
LA	10	R011.03	WEST BLVD OC	53-1572	Minor damage.
LA	10	R11.39	CRENSHAW BLVD. OC	53-1571	Minor cracks at the tops of all columns.
LA	10	R011.70	TENTH AVE OC	53-1570	Minor cracks at the tops of all columns.
LA	10	R12.10	FOURTH AVE POC	53-1590	Minor spalling of abutment shear keys.
LA	10	14.23	SANTA MONICA VI	53-1301	Minor spalls at tops of columns. Possible internal hinge damage.
LA	10	18.41	ECHANDIA OH	53-1333F	Sheared keeper plate bolts at abutment 1 rocker bearings.
LA	10	19.98	CITY TERRC POC	53-1856	Minor cracks near hinge 2.
LA	10	20.95	CAMPUS RD RAMP	53-2055K	Minor cracks at bottom of columns in bent 2.

TABLE 12-I List of Damaged Bridges (continued)

LA	10	21	CAMPUS ROAD OC	53-2054	Fine horizontal cracks in all 4 columns.
LA	10	21.33	RAMONA BLVD UC	53-1459G	Minor cracks at top of columns in bent 2.
LA	10	31.72	BESS AVE POC	53-1288	Minor cracks in superstructure. Rail damaged.
LA	14	R024.73	RTE 14/5 SOH	53-1960F	Partial collapse. Replacement possible.
LA	14	R24.77	RTE 14/5 SOH	53-1960G	Severe hinge and abutment damage. Replacement possible.
LA	14	R024.81	TRUCK CONN OC	53-1962F	Unknown hinge damage. Replacement possible.
LA	14	R24.82	SOUTH CONN OC	53-1963F	Hinge damage span 4. Severe abutment damage. Replacement
LA	14	R024.97	NORTH CONN OC	53-1964F	Partial collapse. Replacement possible.
LA	14	R028.08	PLACERITA R UC	53-2076L	Minor spall at southern abutment.
LA	14	R028.08	PLACERITA R UC	53-2076R	Minor spalls at abutment.
LA	14	R030.55	CEDAR VL WY OC	53-2171	Minor spalls on columns at bent 2 and at abutments.
LA	14	R030.81	RTE 126 /14 SEP	53-2200S	Major column damage.
LA	14	R030.90	VIA PRNCSSA UC	53-2166R	Wingwall damage.
LA	14	R030.90	VIA PRNCSSA UC	53-2166L	Wingwall damage.
LA	14	R030.91	VIA PRNCSSA UC	53-2201K	Cracks in abutments and rails.
LA	14	R031.62	HUMPHREYS OH	53-2029L	Damage at acute wingwall / abutment corners.
LA	14	R031.62	HUMPHREYS OH	53-2029R	Damage at acute wingwall / abutment corners.
LA	14	R031.88	SANTA CLARA RI	53-2027R	Possible hinge damage.
LA	14	R031.88	SANTA CLARA RI	53-2027L	Possible hinge damage.
LA	22	1.42	SAN GABRIEL R	53-0302R	Minor spalls at pier wall.
LA	60	R003.88	BELVEDERE POC	53-1728	Minor column spalls.
LA	71	R000.58	E CONNECTOR OC	53-1987F	Crack at hinge near span 2.
LA	90	2.54	RTE 90 405 SEP	53-1851	Damage to bent common with Bridge Number 53-1255.
LA	90	2.54	RTE 90 405 SEP	53-1851L	Shear key damage at bent.

TABLE 12-I List of Damaged Bridges (continued)

LA	90	2.55	NW CONNECT OC	53-1854G	Minor spalls at abutment and overhang near hinge.
LA	90	2.73	JEFFERSON B UC	53-1855F	Minor rail damage.
LA	101	6.15	WILTON PLACE O	53-0731	Sheared keeper plate bolts. Cracks in curtainrwall.
LA	101	6.41	VAN NESS A RP	53-0732K	Damaged curtain wall at abutment 4.
LA	101	11.63	134/101,170SEP	53-1339F	Column damage. Bearing and approach slab damage.
LA	101	11.75	101/134,170 SP	53-1336R	Column and abutment damage. Minor spalls at hinges.
LA	101	13.27	TUJUNGA WASH	53-1337	Buckled cross frames. Bent hanger plate. Cracks at wingwall.
LA	101	15.38	LOS ANGELES R	53-1371	Sheared keeper bolts and spalls at abut 6. Spalling at tops of columns.
LA	101	25.88	SHOUP AVE UC	53-1095	Damaged soundwall 300' adjacent to structure.
LA	101	31.05	LAS VIRGENE OC	53-1442	Abutment damage.Possible pile damage.
LA	105	R7.39	RTE 105/110 SEP	53-2405R	Bearing damage.
LA	118	R002.55	BROWNS CYN WA	53-2182S	Cracks at shear keys. Delaminated bearing pads.
LA	118	R002.55	BROWNS CYN WA	53-2182	Cracks at shear keys. Delaminated bearing pads.
LA	118	R003.13	RINALDI ST OC	53-2498	Possible pile damage. Wingwall displacement. Cracked slope paving.
LA	118	R006.58	ZELZAH AVE OC	53-2513	Cracks and spalls in slope paving.
LA	118	R006.80	WHITE OAK A OC	53-2464	Damaged shear keys.
LA	118	R007.05	ENCINO AVE OC	53-2465	Broken bearing. Curtain wall damage. Approach settlement.
LA	118	R007.80	BALBOA BLVD OC	53-2395	Washed out abutment from broken water main. Approach slab damage.
LA	118	R008.05	RUFFNER AVE OC	53-2396	Major column damage.
LA	118	R008.34	HAVENHURST UC	53-2204	Aproach slab damage.Minor cracking at abutments.
LA	118	R008.63	MISSN GOTHIC U	53-2205	Partial collapse. East bound removal complete.
LA	118	R008.84	BULL CR CYN CH	53-2206	Partial collapse. Major column damage. Replacement possible.
LA	118	R009.04	WOODLEY AVE UC	53-2207	Spalls at abutments. Severe approach slab damage.
LA	118	R009.33	GAYNOR AVE UC	53-2208	Slope paving and city sidewalk damaged

TABLE 12-I List of Damaged Bridges (continued)

LA	118	R009.57	HASKELL AVE UC	53-2209	Approach slab damage.
LA	118	R009.70	CHATSWTH ST UC	53-2210G	Spalls at abutments.Cracks in wingwalls.
LA	118	R009.74	DEVONSHIRE UC	53-2217h	Minor cracks at abutments and soffit. Wingwall displacement.
LA	118	9.85	CENTER CONN OC	53-2212F	Pile cap damage at abutment 7. Possible pile damage.
LA	118	R010.07	SEPULVEDA B UC	53-2213	Approach slab damage.
LA	118	R010.51	CHATSWORTH D U	53-2214	Cracks at abutments. Wingwall displacement.
LA	118	R010.83	FOX STREET UC	53-2215	Minor approach slab damage.
LA	118	R011.05	ARLETA AVE UC	53-2357	Wingwall displacements. Cracked slope paving.
LA	118	R011.31	SHARP AVE UC	53-2342L	Minor Abutment Damage.
LA	118	R011.32	SHARP AVE UC	53-2342R	Cracks at abutment 2. Joint seal damage. Approach settlement.
LA	118	R011.32	SHARP AVE UC	53-2343G	Cracks at abutments. Approach settlement.
LA	118	R011.41	PACOIMA WASH	53-2328G	Abutment and shear key damage. Cracks in columns. Barrier rail
LA	118	R012.27	PAXTON ST UC	53-2354S	Minor spalls at abutment. Approach settlement.
LA	118	R012.40	SAN FERN RD OH	53-2095	Damage at abutment 3. Spalls at soffit.
LA	118	R013.89	PAXTON-FTHL UC	53-2103G	Minor spalling at hinge seats.
LA	118	R013.94	RTE118/210 SEP	53-2102G	Minor damage at abutment 11. Broken electrolier bases.
LA	126	R005.80	RTE 126/ 5 SEP	53-2694G	Cracks at column tops and shear keys. Approach slab damage.
LA	126	8.2	SFK SANTA CLAR	53-0015	Sheared anchor bolts at all supports. Slight permanent displacement.
LA	134	0	RIVERSIDE D OC	53-1493S	Bearing damage. Minor spalls at columns. Joint seal failure.
LA	134	.03	RIVERSIDE DR U	53-1452F	Damaged external shear keys. Bearing damage.
LA	134	.04	RIVERSIDE DR U	53-1345F	Bearing damage at abut 1. Damaged parapet wall at abutment 3.
LA	134	1.36	FORMAN AVE UC	53-1276	Shear key damage. Damaged parapet wall at abutment 1.
LA	134	2.24	OLIVE AVE OC	53-1280	Sheared keeper plate bolts.
LA	134	R005.67	LA RIV BOH	53-1790H	Minor rail and joint damage.

TABLE 12-I List of Damaged Bridges (continued)

LA	134	L009.72	MONTE BONITO U	53-1023R	Crack in abutment 1 external shear key.
LA	134	L009.91	FIGUEROA ST UC	53-1024R	Sheared keeper plates at both abutments. Cracks at abut 1.
LA	134	L009.91	FIGUEROA ST UC	53-1024L	Sheared keeper plates at both abutments.
LA	138	51.06	PEARLAND UP	53-2033	High load hit.
LA	170	R014.78	RIVERSIDE T UC	53-1344	Broken keeper plates. Minor rail damage.
LA	170	R015.63	CHANDLER BV OH	53-1644	Minor rail damage.
LA	170	R018.65	WHITSETT AV OC	53-0490	Superstructure rotated. Column damage. Minimal bearing.
LA	170	R020.52	RTE 170/5 SEP	53-1122G	Bearing damage abutment 1. Large spall near hinge. Joint seal failure.
LA	210	R000.06	SW CONNECTR OC	53-1989F	Major damage both abutments. Damaged shear keys.Column damage.
LA	210	R003.01	TYLER ST POC	53-1925	Spalling at abutment 1. Minimal Bearing.
LA	210	R006.08	RTE118/210 SEP	53-2104F	Spalls at hinge seats. Joint seals failed.
LA	210	R007.16	TERRA BEL ST U	53-2117	Approach slabs have settled.
LA	405	23.71	LA CIEN BV S O	53-1250	Sheared keeper plate bolts.
LA	405	25.93	JEFFERSON BV U	53-1255	Damage to bent common with Bridge Number 53-1851.
LA	405	29.42	SEPULVEDA B UC	53-1638G	Sidewalk damaged at east abutment.
LA	405	29.43	SW CONNECTR OC	53-1630G	Rocker bearing damage at both abutments. Cracks in abutment
LA	405	29.62	NE CONNECTR OC	53-1629F	Rocker bearing damage. Approach slabs have settled.
LA	405	29.85	EXPOSITION OH	53-0704	Minor cracks and spalls in columns. Cracks in bent caps.
LA	405	36.72	RIMERTON RD OC	53-1490	Crack in closure wall at abutment 1.
LA	405	37.03	MULHOLLAND D O	53-0739	Bearing damage. Joint seals failed. Spalls at wingwall.
LA	405	38.59	SEPULVEDA BV U	53-0740	Spall at bent 2.
LA	405	41.27	W VAN NUYS OH	53-1362	Minor cracks in old spall area at abutment 1.
LA	405	41.36	VICTORY BV UC	53-1449	Bearing pad failure at bent 2.
LA	405	44.24	PARTHENIA ST U	53-1439	Approach slab damage. Spalls in rail at bent 2.

TABLE 12-I List of Damaged Bridges (continued)

LA	405	46.24	DEVONSHIRE UC	53-1500	Minor cracks at abutment.
LA	405	46.74	CHATSWRTH ST U	53-1501	Minor cracking. Approach slab damage.
LA	405	46.8	SE CONN OC	53-2216G	Major damage at abut.1. Damage at hinges 1 & 2. Approach settlement.
LA	405	46.83	RT 405 118 SEP	53-2211	Minor cracks at bent walls and columns. Incipient spalls at shear keys.
LA	405	47.24	SAN FERN BL UC	53-1507	Cracks in abutments and wingwalls. Joint spalls. Aproach settlement.
LA	405	47.75	RINALDI ST UC	53-1506	Major damage at bent 3. Cracks in closure wall.
VEN	101	7.89	WENDY DRIVE OC	52-0266	Minor spalls at exterior girder.
VEN	118	R029.56	YOSEMITE ST OC	52-0300	Cracking at tops and bottoms of columns. Approach settlement.

SECTION 13
CONCLUSIONS

The following general conclusions can be made from the performance of bridges during the Northridge earthquake.

Bridge retrofit programs are effective. Although many cable restrainers failed, they were generally of a design that has since been superseded by Caltrans. Also, some that failed catastrophically did so after collapse of nearby columns and loss of support for gravity loads. In these instances, the cable loads far exceeded their design forces because they were then supporting the self-weight of several spans of the bridge. However, some restrainers might have failed due to improper installation. In at least one instance, the nuts on several restrainer cable studs were found to be missing with no evidence of stripped threads. Column jackets appeared to work well and none showed 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 the current screening procedures and not be identified as vulnerable. Structure attributes such as skew and the unintended participation of nonstructural elements (e.g., walls and flares) need to be further addressed. Multicolumn bents should also be elevated in the priority ranking procedures.

Other conclusions include:

- 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 these walls and provide continued access.

SECTION 14
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LIST OF TECHNICAL REPORTS

The National Center for Earthquake Engineering Research (NCEER) publishes technical reports on a variety of subjects related to earthquake engineering written by authors funded through NCEER. These reports are available from both NCEER's Publications Department and the National Technical Information Service (NTIS). Requests for reports should be directed to the Publications Department, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Red Jacket Quadrangle, Buffalo, New York 14261. Reports can also be requested through NTIS, 5285 Port Royal Road, Springfield, Virginia 22161. NTIS accession numbers are shown in parenthesis, if available.

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