DAMAGE TO FREEWAY STRUCTURES IN THE SAN FERNANDO EARTHQUAKE

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1. Introduction

The extensive freeway system of the Los Angeles basin utilizes a very large number of modern bridge structures to distribute traffic at freeway interchanges and to carry the freeways over and under the city street systems. Most of the bridges are of prestressed concrete or reinforced concrete design and commonly box girder construction is used although some arch and girder type bridges are employed. In general bridges are the major structures on the freeway system and high earth retaining structures and tunnels are not common. Retaining walls are used on some of the older sections in the central area of Los Angeles City.

The magnitude 6.6 main shock of the February 9th, 1971, San Fernando earthquake resulted from displacements on a thrust type fault. The fault displacements originated at a point approximately eight miles beneath the epicenter, which was located in an uninhabited region of the San Gabriel Mountains, and propagated southward along a fault plane inclined at about 45° breaking the surface in the built-up area of Sylmar. Because of the thrust type faulting, the location of the surface traces of the fault, and the location of the epicenters of the aftershocks, the center of energy release has been placed about five miles south of the epicenter of the main shock, in the vicinity of the Pacoima Dam (Fig. 1). A maximum peak ground acceleration of approximately 1.2g was recorded by an accelerograph instrument located on a steep rock ridge near one of the dam abutments. Some of the maximum peak ground accelerations recorded on the alluvial valleys of the Los Angeles area were:

San Fernando Valley, 8 miles south of Pacoima Dam -	0.26g
North Hollywood, 13 miles south of Pacoima Dam -	0.18g
Pasadena, 21 miles southeast of Pacoima Dam -	0.20g
Central Los Angeles, 22 miles south of Pacoima Dam -	0.17g
Castaic, 14 miles northwest of Pacoima Dam -	0.33g
Lake Hughes, 24 miles north of Pacoima Dam -	0.35g

Approximately 70 freeway bridge structures (counting twin freeway bridges as single structures) were located within a 10 mile radius of the Pacoima Dam and of these approximately 40 received significant damage, including five bridges that collapsed. These bridges were subjected to approximately 10 seconds of very intense ground motion with maximum peak accelerations perhaps in the range 0.25-0.5g. In general, damage to bridge structures outside the 10 mile radius was relatively minor and was mainly confined to damage at superstructure joints and settlement of approach fills. Probably more than 400 freeway bridges lie within a 25 mile radius of the earthquake's center and many of these were subjected to peak ground accelerations in excess of 0.15g.

The most severe damage occurred to overpass structures at three major interchanges; the Golden State (Interstate 5) and the Antelope Valley (California 14) freeways; the Golden State and the Foothill (Interstate 210) freeways; and the Golden State and the San Diego (Interstate 405) freeways. These three interchanges were all located within seven miles of the Pacoima Dam (Fig. 1) and were probably in the region of strongest ground shaking. Most of the other damaged bridges were located on the Antelope Valley, Foothill and Golden State freeways. The total damage to the freeway system has been estimated to be 15 million dollars.

The freeway bridges in the Los Angeles area are constructed under the supervision of the California Division of Highways. A uniformly high standard of constructions is achieved.

2. Golden State-Antelope Valley Freeway Interchange

The interchange between the Golden State freeway and the Antelope Valley freeway was under construction at the time of the earthquake. An aerial view taken after the earthquake is shown in Fig. 2. Two interior spans of the nine-span South Connector Overcrossing collapsed. The other bridges in the interchange, including the bridges under construction, escaped major damage. A prestressed concrete box girder that had not been stressed at the time of the earthquake was cracked by settlement of the falsework. Several bridges were damaged by the falling overcrossing.

2.1 South Connector Overcrossing

Dimensions and typical details of the two-lane 1349 ft long concrete box overcrossing are given in Fig. 3 and Fig. 4. Further details of the collapsed section and the damage to the remaining sections can be seen in illustrations

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Fig. 5 to Fig. 10.

The bridge was structurally complete at the time of the earthquake but was not open to traffic. The collapsed section consisted of a two-span, 380 ft long, prestressed concrete box girder supported by a central 140 ft high reinforced concrete column which collapsed with the superstructure. The ends of the collapsed section were supported on the adjoining reinforced concrete box spans by means of hinge joints. A standard (California Division of Highways) elastomeric type hinge was used and details can be seen in Fig. 6. At the joint the superstructure seats on elastomeric bearing pads which are located on a 14 in wide ledge. A reinforced concrete shear key prevents relative transverse displacements between the bridge sections. Three 1-1/2 in diameter mild steel bolts connect the bridge sections in the longitudinal direction. Apparently these bolts are used to equalize the longitudinal creep and temperature deflections at the joints and are not related to earthquake resistant design. No vertical hold-down bolts are used at the joints.

At present the reason for collapse is unresolved and a detailed study of possible failure mechanisms is required. However, the pattern of the damage and the details obtained by inspection of the site merit some discussion. It is not known whether an accurate survey of the freeway setting-out points has been undertaken but no permanent ground displacements were detected by visual inspection of the site by a number of investigators. The remaining sections of the bridge sustained only minor damage and with the exception of the very short column near the western abutment, which received shear cracking, the columns supporting the remaining sections were undamaged. The two columns nearest to the collapsed spans (piers 3 and 5) had the same cross-section as the collapsed column but were significantly shorter and consequently stiffer. Had the lateral vibrations of the bridge been large enough to produce a bending failure in the column supporting the collapsed section then it seems likely that the columns in the adjoining spans would have shown some signs of distress. Sections of the superstructure fell across the base of the collapsed column and so it was not possible to inspect that point for evidence of any weakness in the construction materials.

It is conceivable that combined transverse and longitudinal vibrations were sufficient to allow one of the hinge joints to open sufficiently for the girder end to fall. The 14 in seating width appears small in relation to the 190 ft long spans. The two joints in the remaining western section of the bridge showed evidence of pounding but it was not possible to determine whether these joints had separated appreciably during the earthquake. Although the monolithic connections between the superstructure and abutments showed signs of overstressing no large relative displacement between the superstructure and abutments, or abutments and the ground, appeared to have occurred.

The inability of prestressed concrete bridges to carry large upward forces suggests another possible failure mechanism. The prestressed box girders have an appreciable reserve of strength for downward forces but normally are not designed for large uplift forces and if vertical vibrations of the span were sufficient to effectively remove a large percentage of the dead load then a sudden catastrophic compression failure could occur in the girder. The magnitude of the vertical ground accelerations and the resultant structural motion can only be estimated and further investigations are clearly necessary.

3. Golden State-Foothill Freeway Interchange

The extensive damage to the large complex of overcrossings and bridges at the Golden State-Foothill freeway interchange is shown in the aerial photographs Fig. 11 and Fig. 12. Some of the major structures are identified in Fig. 11. The interchange was in the final stages of construction at the time of the earthquake and the major structures were complete but not all were open to traffic. All but one of the bridge structures at this interchange suffered collapse or extensive damage.

There were obvious signs of permanent ground movement in the general area of the interchange and although only relatively minor ground cracking was found in the immediate locality of the bridges, large ground movements were found at the nearby San Fernando Juvenile Facility, the Pacific Intertie Converter Station and the Metropolitan Water District treatment plant (Fig. 1). The degree to which the permanent ground movements contributed to the bridge failures is not known.

The highest level ramp, the Separation and Overhead structure, experienced a total collapse and fell on the Northwest Connector Overcrossing and on several of the spans of the San Fernando Road Overhead. The central, simply-supported steel girder spans of the San Fernando Road Overhead fell onto the Southern Pacific railroad. The Northbound Truck Route Undercrossing and the two-lane transition bridge of the San Fernando Road Overhead were seriously damaged and the latter bridge was demolished soon after the earthquake.

3.1 Separation and Overhead Bridge

Typical dimensions and details of this seven-span 770 ft long reinforced concrete box bridge are given in Fig. 13. The photographs shown in Fig. 14 to Fig. 15b illustrate some of the construction details and the total collapse of the bridge.

The bridge was constructed with one standard elastomeric hinge joint that was similar in detail to those used on the previously described South Connector Overcrossing bridge. The longitudinal 1-1/2 in diameter bolts were not used; apparently they are not required on bridges with a single joint. At each abutment the girder was seated on elastomeric bearing pads that were located on concrete pedestals. The length of the bridge seating on the pedestals was approximately 1 ft 10 in. Eighteen inch thick abutment wing walls provided transverse restraint but no vertical tie down bolts or longitudinal fixing to the abutments were used. The wing walls were seriously damaged by the movements of the bridge and one wall separated from the southern abutment.

Security restrictions and demolition work which followed soon after the earthquake prevented a close inspection of the failures at the bases of the columns. However, many photographs show what appear to be bond type failures of the No. 18 bars (No. 4 ties at 12 in centers). The design specifications require that the No. 18 bars be spliced within the column section by butt welding so apparently the failures resulted from the bars pulling free from the pile caps at piers 2 and 3 (Fig. 13) or from the failure of the splice to the 6 ft diameter drilled concrete piles at the other piers. It is possible that bending failures at the base of some of the columns (a photograph of column 3 indicated this type of failure) initiated the collapse and that the bond failures occurred as a secondary result of the collapsing structure.

The combined influence of permanent ground movements and the severe vibrations of the long bridge essentially unrestrained at its abutments could have caused separation of the 14 in hinge joint with subsequent collapse of the bridge spans.

3.2 The San Fernando Road Overhead

The two San Fernando Road Overhead bridges are seven-span skew structures that carry the Golden State freeway over San Fernando Road and the Southern Pacific railroad. Initially the bridges were constructed with a simply-supported central steel girder span and remaining spans of continuous reinforced concrete box construction. At a later date the bridges were widened to provide additional lanes and transition ramps at the new interchange. Precast, prestressed, girders were used to widen the central spans across the railroad and the other spans were widened with concrete box construction. The steel girder spans were approximately 73 ft long and were aligned 45° skew.

The collapse of the central spans can be seen in Fig. 12 and Fig. 17. Details of the damage to the concrete box sections are shown in Fig. 16. The severe vibrations caused failures in many of the columns permitting large horizontal deformations of the superstructure. Possibly the steel girders fell from their steel bearings and then with the onset of large horizontal deformations the girder spans were rotated in a horizontal plane by the pounding at the skew joints until some of the girders slipped free from the pier capping beams. Permanent ground displacements also may have contributed to the opening of the joints. The effective length of seating of the girders on the concrete pier capping beams was approximately 16 in.

A deck joint was constructed at only one end of the central widening spans (Fig. 12). Probably as a result of the restraint against rotation and displacement provided by the deck these spans remained intact.

3.2 San Fernando Road Overhead Transition

This two-span, continuous, prestressed concrete box bridge carries two lanes of northbound Golden State freeway traffic to the eastbound Foothill freeway and crosses both the railroad and San Fernando Road. Dimensions and details of the 240 ft long bridge are given in Fig. 18 and the structure can be seen in aerial view Fig. 12.

Severe damage occurred at the base of the central column (Fig. 12) but unfortunately the bridge was quickly demolished before an inspection could be made. Photographs that were unavailable for this report indicated that the failure in the column was basically from overstress in bending. The relative simplicity of this particular structure would make it suitable for a detailed analysis.

3.3 Northwest Connector Overcrossing

As a result of the impact of the Overhead and Separation structure this continuous reinforced concrete box bridge was seriously damaged. Some of the structural failures were probably augmented by ground shaking and soil movements. Details of the failures can be seen in Fig. 19 to Fig. 20b.

3.4 Northbound Truck Route Undercrossing

Details of the extensive damage to the three-span continuous concrete box bridge are shown in Fig. 21 to Fig. 25. The bridge is approximately 225 ft long and has large angles of skew at each abutment. The abutment and column foundations are spread footings. Because of the highway geometry the abutment at the western end of the bridge is about 1.4 times longer than the abutment at the eastern end, and the western pier has four columns compared with the three columns at the other pier. The bridge rotated in a horizontal plane about the western end resulting in a large permanent displacement of the superstructure at the eastern abutment and severe bending failures at the tops of the columns in the eastern pier. The backfilling at the eastern abutment settled approximately 1 ft. The tendency of this bridge to rotate in a horizontal plane, effectively becoming more skew, was typical of many of the skew bridges in the area. This phenomenon is discussed in more detail in some of the following sections.

4. Golden State-San Diego Freeway Interchange

At this interchange the Southbound Truck Ramp bridge collapsed onto the San Diego freeway. The collapsed structure was quickly demolished and removed in order to open the freeway to traffic and consequently details of the failure were not obtained. The collapsed bridge was a two-span prestressed concrete box girder approximately 350 ft long.

5. Foothill Freeway Bridges

A newly-constructed 4.5 mile length of Foothill freeway (Fig. 1) was in the zone of strongest shaking and most of the bridges were damaged to some extent. Several of the most severely damaged bridges are discussed in the following sections.

5.1 Foothill Boulevard Undercrossing

The freeway lanes are carried across Foothill Boulevard on two separate four-span reinforced concrete bridges. As a result of the rather complex freeway geometry in this area the dimensions of the two bridges are significantly different. The general layout and representative dimensions of the southeastern bridge are given in Fig. 26.

Details of the damage to the southeastern bridge are illustrated by the photographs Fig. 27 to Fig. 29. As seen in the figures, severe failures occurred in three of the four columns of the central pier. The central columns of the northwestern bridge appeared undamaged but fractures in the street concrete median strip indicated relatively large movements of the columns. Spalling occurred at the tops of two of the end span columns of the northwestern bridge and cracking occurred at the tops of some of the other end span columns. The end span columns are located between retaining walls which were structurally separated from the columns by a 1/2 in layer of soft material. Apparently the walls exerted a stiffening influence on the damaged columns under the northwestern bridge.

A number of factors probably contributed to the overloading of the central pier of the southeastern bridge. The abutments and piers have spread footing foundations and possibly differential settlements occurred producing an increase in vertical loading on some of the columns. The bridge rotated in the horizontal plane and a permanent offset of about 4 in in the direction of increasing skewness was observed at the abutments. Rotation would effectively increase the lateral load carried by the outer columns at the central pier but it is not apparent why the columns at the other piers were not damaged by this displacement.

The less extensive damage to the northwestern bridge columns may have been a consequence of the significant difference in dimensions between the two bridges. Also columns at two of the piers of the northwestern bridge (including the central pier) had pile foundations rather than the spread footings of the southeastern bridge.

It is clear that the extent of the column failures in the southeastern bridge was aggravated by inadequate ties. The use of 1/2 in diameter lapped ties at 12 in spacing does not appear adequate for large bridge columns. Laps are unsatisfactory because spalling of the outer concrete shell renders the ties ineffective.

5.2 Roxford Street Undercrossing

The Roxford Street undercrossing bridges are twin single-span prestressed concrete box structures. The abutments of the 151 ft long bridges are located on filled embankments and have 15 in diameter concrete pile foundations. Dimensions of the bridges are given in Fig. 30.

Both bridges suffered lateral pile failures and the large displacements of the bridges caused extensive damage to the abutment wingwalls and approach pavement. Details of the damage and failures can be seen in Fig. 31 to Fig. 34. A 5 ft section of some of the piles was exposed for inspection and it appeared that the piles had hinged at the underside of the abutments and sheared through the soil. The eastern abutment of the southern bridge received a permanent transverse displacement of approximately 2 ft 6 in and the western abutment moved several inches in the same direction. The eastern abutment of the northern bridge displaced transversely approximately 1 ft but no significant permanent displacement occurred at the other abutment.

5.3 Bledsoe Street Overcrossing

The Bledsoe Street overcrossing is a two-span, 208 ft long, reinforced concrete box girder bridge located about 1/4 mile from the Olive View Hospital. Dimensions of the bridge are given in Fig. 35 and an elevation view is shown in Fig. 36.

The central pier and the abutments have spread footing foundations in original ground. The bridge abutments settled 2 in with respect to the structurally separated wingwalls located on the approach filling and the total settlement of the abutments may have been greater. Distinctive cracks (1/32 in width) were found in the sidewalks at the regions over the central pier and these extended into the roadway deck slab. The cracking indicated that quite appreciable differential settlement of the bridge foundations had occurred.

Although no permanent horizontal displacements of the bridge were observed there was evidence that the bridge had experienced longitudinal vibrations with a maximum amplitude of several inches. The approach sidewalks were damaged by the movements and spalling occurred at the tops of the two columns of the central pier. The column damage is shown in Fig. 37a and Fig. 37b.

5.4 Tyler Street Footbridge

Details of the Tyler Street footbridge can be seen in Fig. 38 to Fig. 40. The bridge is of continuous reinforced concrete box construction and the longest span is approximately 70 ft.

All of the columns, including the tallest and presumably most flexible, spalled at their tops, indicating overstress in bending. The primary motion indicated by the column damage was longitudinal and the extent of this motion was illustrated by the 10 in gap that opened between the bridge seating and the abutment structure at the northeastern end of the bridge. The seatings at both ends of the bridge were not tied to the abutment structures and presumably considerable pounding occurred which resulted in displacement of the abutments.

6. Miscellaneous Damage

Abutment damage was widespread and although mainly confined to bridges within a 10 mile radius of the Pacoima Dam, minor abutment damage occurred at more distant bridges. Some examples of abutment damage can be seen in the illustrations referred to in the previous sections. Other examples are shown in Fig. 41a to Fig. 43. In particular abutment damage was severe on skew bridges and perhaps related to the tendency of skew bridges to rotate in a horizontal plane. A number of abutments were damaged by basically shear type failures in the components that either restrained or provided a connection for the superstructure. In general it appeared that abutments located on firm ground were relatively more rigid than the pier structures and consequently they initially carried a large proportion of the horizontal inertia loads.

Settlement of the backfilling at abutment approaches was common and generally the settlement was most severe at bridges that showed evidence of appreciable horizontal movements. Typically the unreinforced concrete freeway pavement is continuous through to the backface of the abutment where it is seated on a 4 in wide recess. In a number of cases (Fig. 25) movement of both the bridges and the pavement was sufficient for the pavement to slip from the abutment recess and settle with the backfilling. It seems desirable to avoid this type of failure to ensure that the freeways are immediately available for the operation of emergency vehicles following an earthquake. This particular detail would be improved by using a wider recess and a length of reinforced concrete slab across the approach filling.

On a number of bridges a concrete apron was used to pave the slope under the bridge between the city street level and the bridge abutment (Fig. 32). The aprons were not connected to the bridge abutments and many slipped down the slope damaging the apron and the sidewalk. This damage, although minor, would probably have been largely avoided by tieing the aprons to the abutments.

Pounding damage at superstructure and abutment joints was a relatively common form of damage. Some examples are shown in Fig. 43 and Fig. 44. Elimination of this damage could probably be achieved by using thicker elastomeric pads to increase the separations at the joints and by the use of more positive tieing between the components.

7. Conclusions

(1) With the exception of the short steel girder spans that fell at the San Fernando Road Overhead, the structures that collapsed wére long span inverted pendulum structures with relatively tall columns. Thus, these structures appear to be vulnerable to seismic excitation. On the other hand, the short span continuous bridges performed relatively well, indicating inherently high lateral resistance.

(2) Several moderate span bridges with relatively large skew angles received extensive damage to their lateral load-resisting components. The damage appeared to be aggravated by the tendency of these bridges to rotate in a horizontal plane in a direction that increased their skewness. It is thought that the rotation is a result of interaction between the structure and the approach fill. More investigation of this behaviour is required.

(3) The concrete box girder superstructures of continuous bridges experienced very little damage even on bridges where the damage to sub-structures was severe. Minor superstructure cracking occurred on several two-span reinforced concrete box girder bridges with spread footing foundations where abutments settled relative to the central pier (e.g. the Bledsoe Street bridge).

(4) On long bridges the 14 in length of seating provided at the hinge joints appears too short. More positive methods of tieing the sections together at superstructure joints are required to prevent collapse even in the event of the strongest shaking.

(5) Most of the damaged columns were tied with No. 4 bars spaced at 12 in and in general column failures were severe with considerable loss of concrete from the central region of the column. The area of tie steel should be significantly increased over the full height of the columns. More adequate ties are needed to provide ductility and hence reduce the severity of earthquake damage. Also by increasing the area of tie steel additional shear strength would be obtained. In critically stressed regions of columns full confinement of the concrete is desirable and can provide a significant increase in earthquake resistance at a small additional cost.

The ties on the damaged columns were lapped and with the spalling of the outer concrete shell they became ineffective. A more positive anchorage of ties, such as a 135° bend and an extension into the central region of the column, is required.

(6) The influence of relative rigidity on the distribution of horizontal loads between the abutments and piers is important and needs design consideration. If abutment structures are to provide a significant proportion of the horizontal load resistance, it is necessary to use higher lateral load coefficients for the design of abutment components loaded basically in shear to account for the lack of ductility in shear type failures.

(7) Freeway bridges in California are currently being designed to resist earthquake forces in accordance with the Bridge Planning and Design Manual, California Division of Highways, March 1968. The following equations summarize the method used.

$$EQ = KCD$$

EQ = The force applied horizontally at the center of gravity of the structure. (Not less than 0.02D).

K = 1.33 For bridges where a wall with a height to length ratio Of 2.5 or less resists horizontal forces applied along the wall.

K = 1.00 For bridges where single columns or piers with a height to length ratio greater than 2.5 resist the horizontal forces.

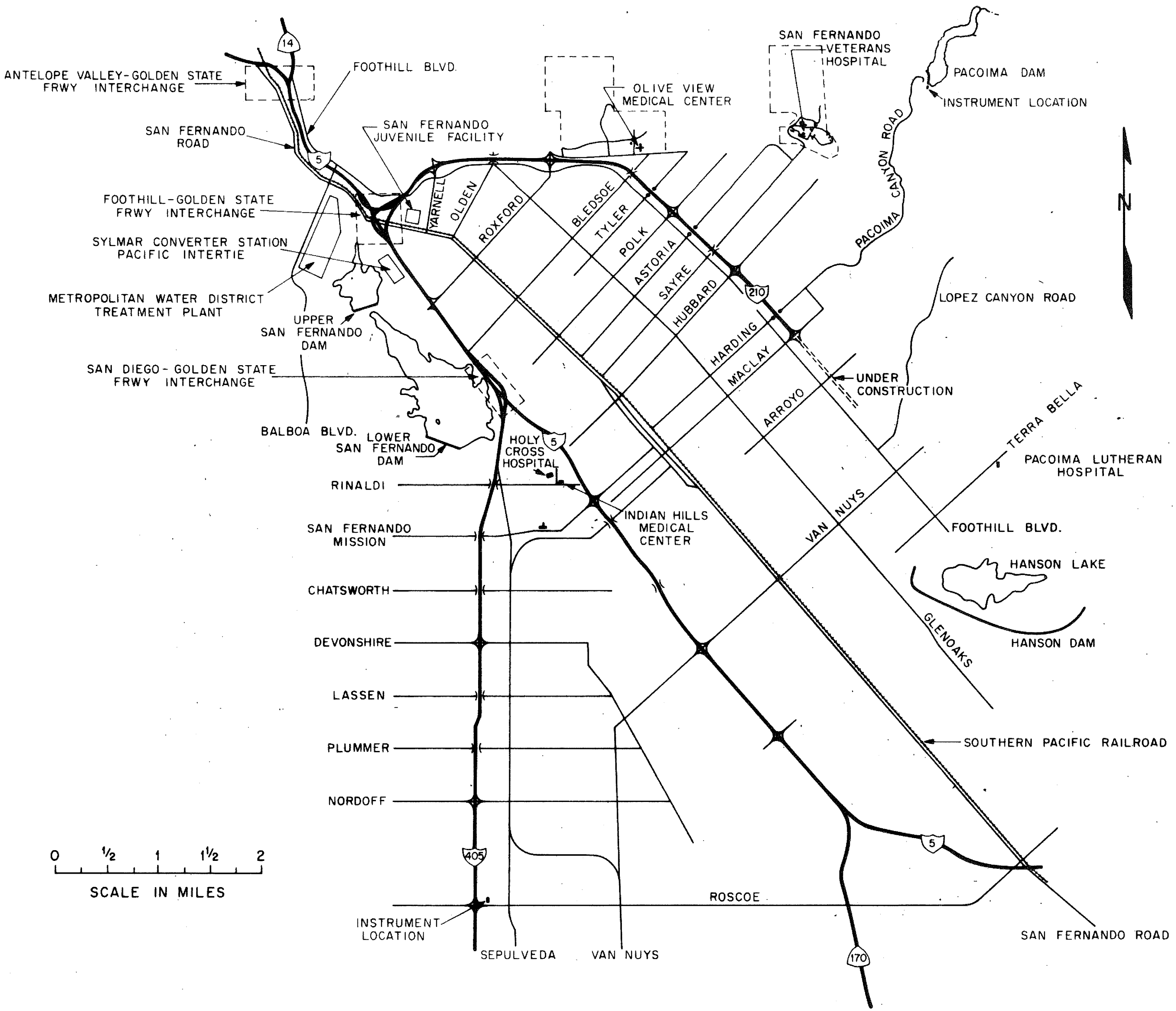
K = 0.67 For bridges where continuous frames resist horizontal forces applied along the frame.

$$C = \frac{0.05}{\sqrt[3]{T}} \quad \text{(Maximum value of } C = 0.10\text{)}.$$

$$T = 0.32\sqrt{\frac{D}{P}} \quad \text{for single story structures}$$

T = Period of vibration of structure
D = Dead load reaction of structure
P = Force required for one inch horizontal deflection of structure.

This manual and the AASHO code are not satisfactory for the earthquake resistant design of bridges. In particular, higher coefficients are clearly required for inverted pendulum structures. Furthermore, the design of large bridges essential to the functioning of the freeway system merits special investigation and dynamic analyses.



NORTHERN SAN FERNANDO VALLEY

Fig. 1 Location of major structures. The epicenter was 5 miles due north of Pacoima Dam, but the dam site was the approximate center of energy release of the magnitude 6.6 earthquake.

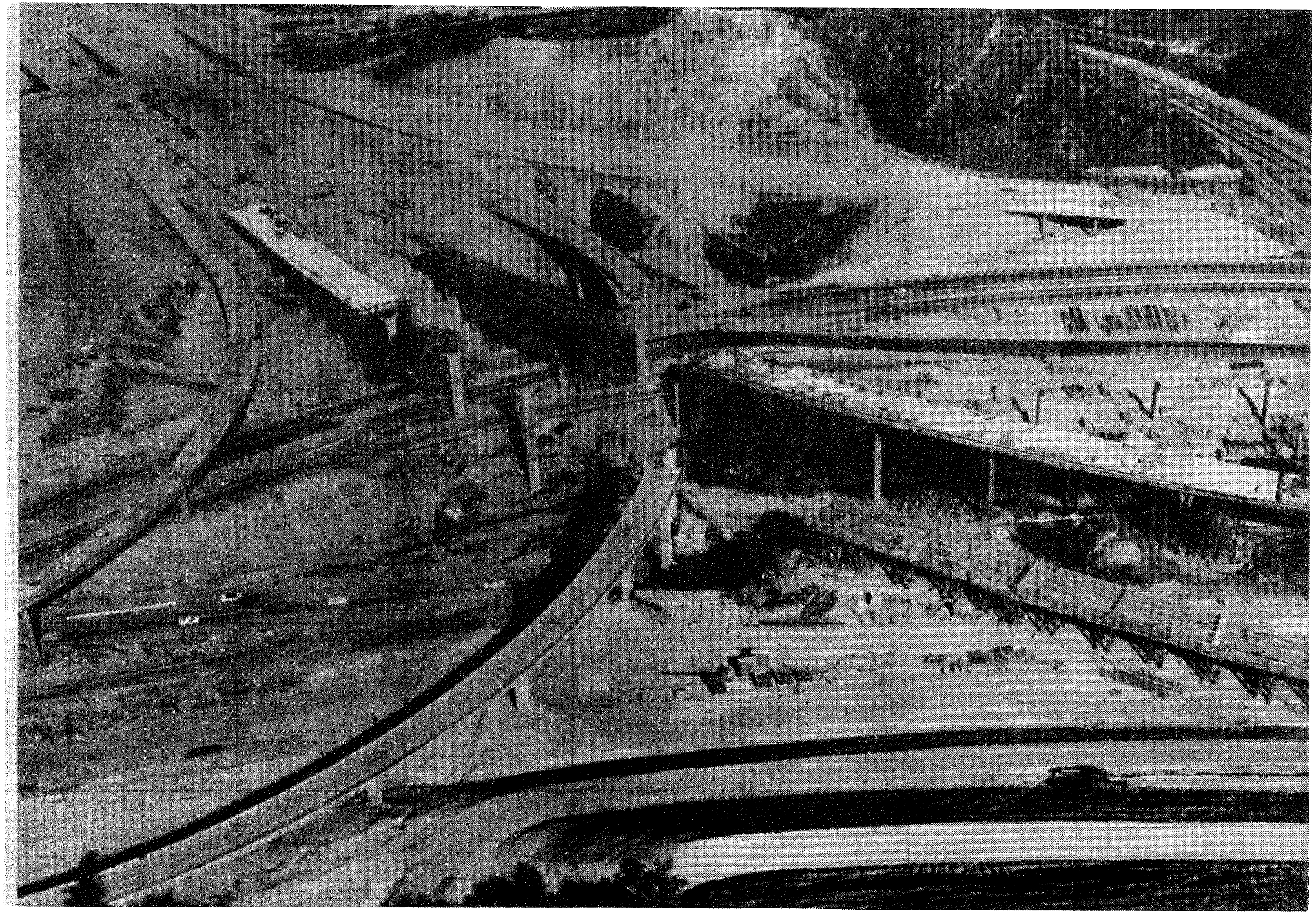
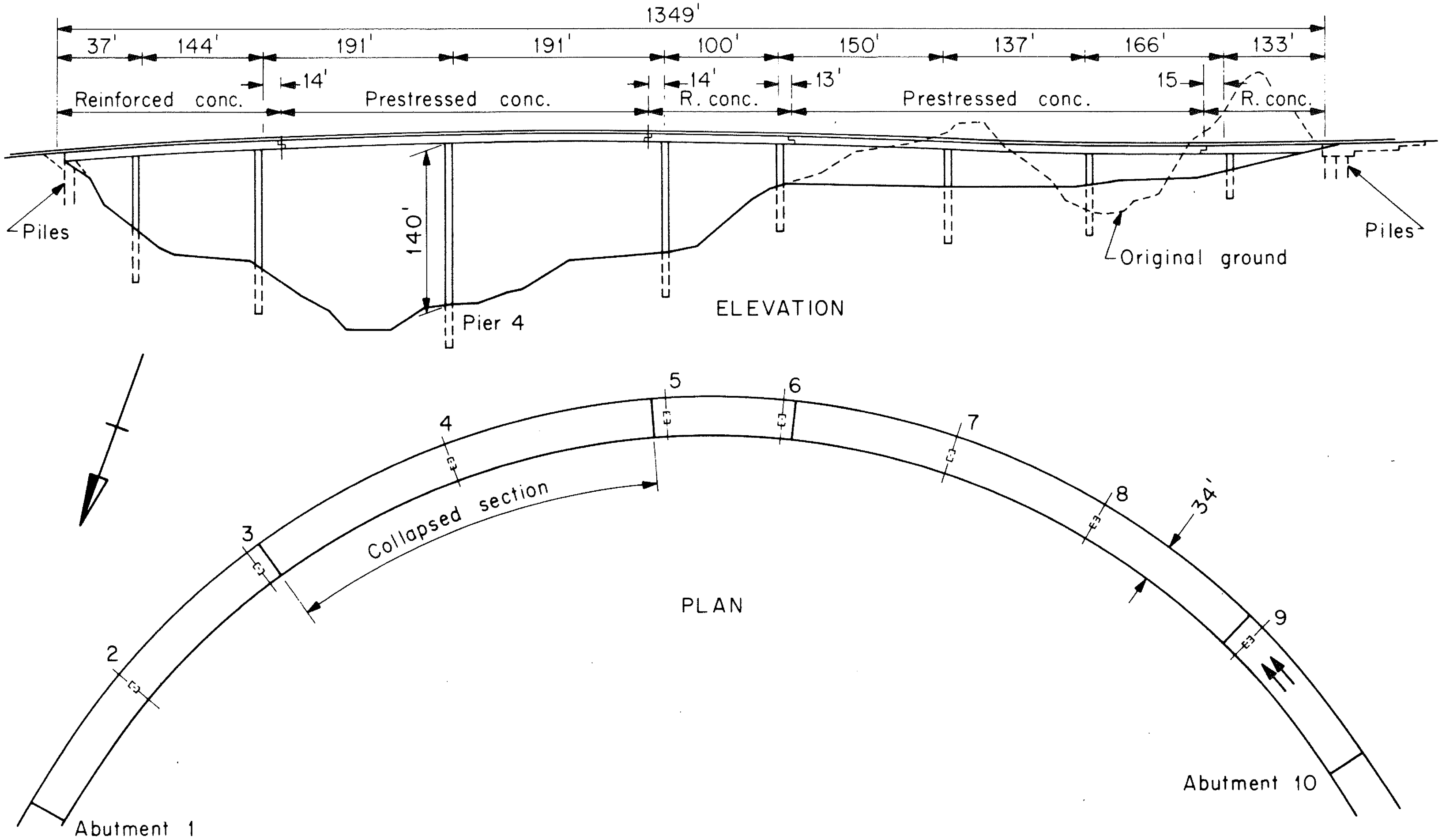


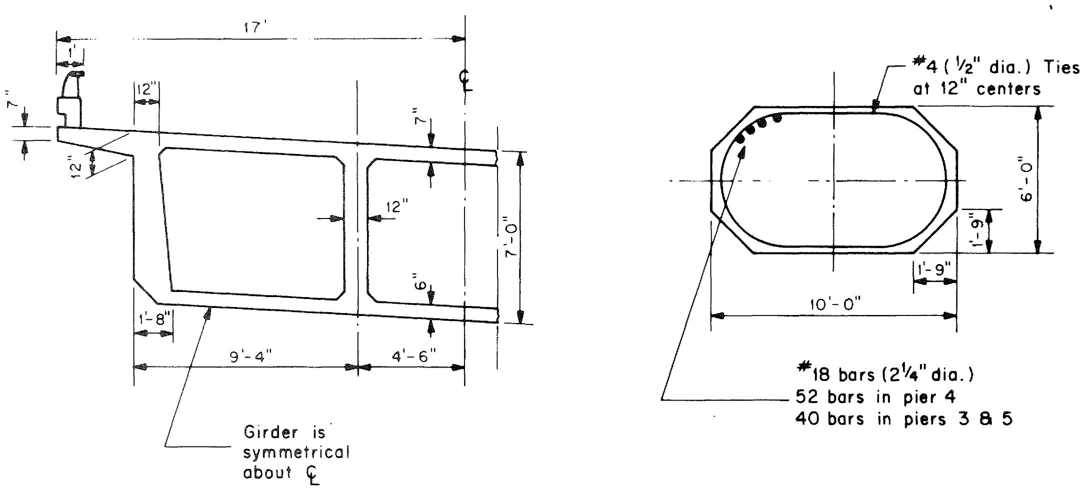
Fig. 2 The Golden State - Antelope Valley freeway interchange, looking east. The South Connector Overcrossing curves from lower left and the missing spans which collapsed can be seen in the center of the photograph. Other bridges were damaged by falling debris and settlement of falsework.



SOUTH CONNECTOR OVERCROSSING

GOLDEN STATE – ANTELOPE VALLEY FREEWAY INTERCHANGE

Fig. 3



PRESTRESSED BOX GIRDER SECTION

COLUMN SECTION – PIERS 3,4 & 5

SOUTH CONNECTOR OVERCROSSING

GOLDEN STATE-ANTELOPE VALLEY FREEWAY INTERCHANGE

Fig. 4

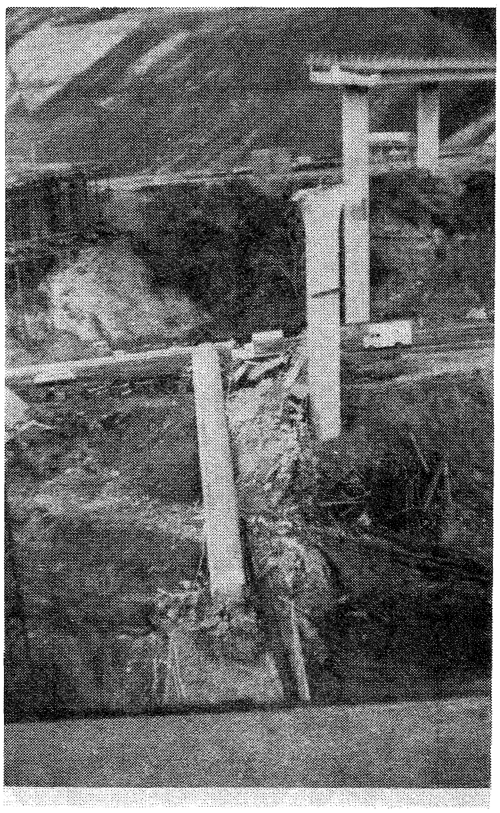


Fig. 5 The collapsed pier and superstructure of the South Connector Overcrossing. This view was taken from the remaining eastern section of the bridge looking towards the western section. The pier standing between the two sections is for an overcrossing under construction.

Fig. 6 The western end of the collapsed superstructure of the South Connector Overcrossing. An elastomeric hinge-joint was constructed at both ends of the collapsed section. Details of the 14 in wide notch used at the joints can be seen in this view.

Fig. 7 Top of the collapsed column of the South Connector Overcrossing. Apparently the deck was cast on top of the flat surface at the column end.* Remains of the capping beam were not visible and must have stripped from the column during failure.

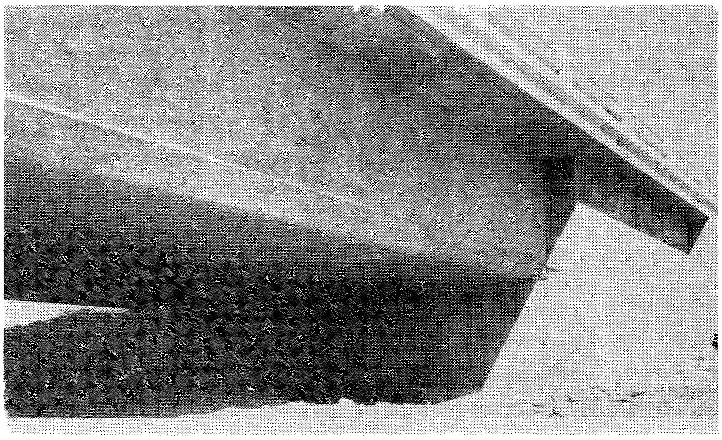


Fig. 8 Abutment of the remaining eastern section of the South Connector Overcrossing. Spalling occurred at the monolithic joint between superstructure and abutment but there was no evidence of appreciable displacement.

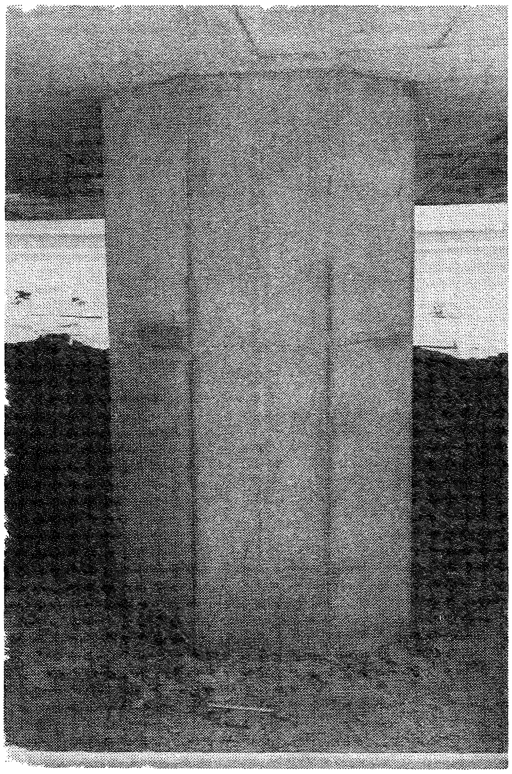


Fig. 9 Shear cracking in Column 9, the shortest column (7 ft) supporting the South Connector Overcrossing. Other remaining columns were undamaged.

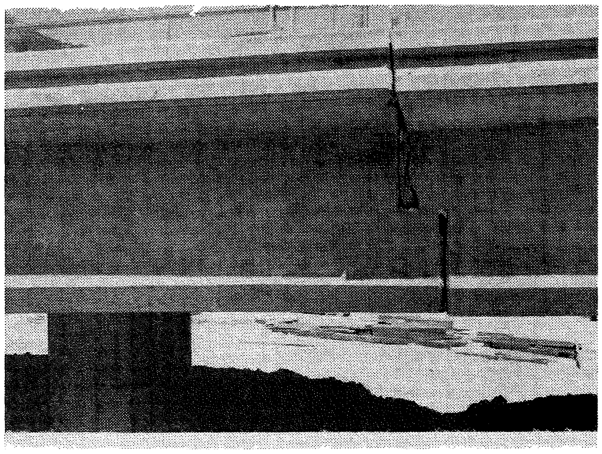


Fig. 10 The hinge-joint near Column 9, South Connector Overcrossing. Both joints on the remaining western section were damaged by pounding.

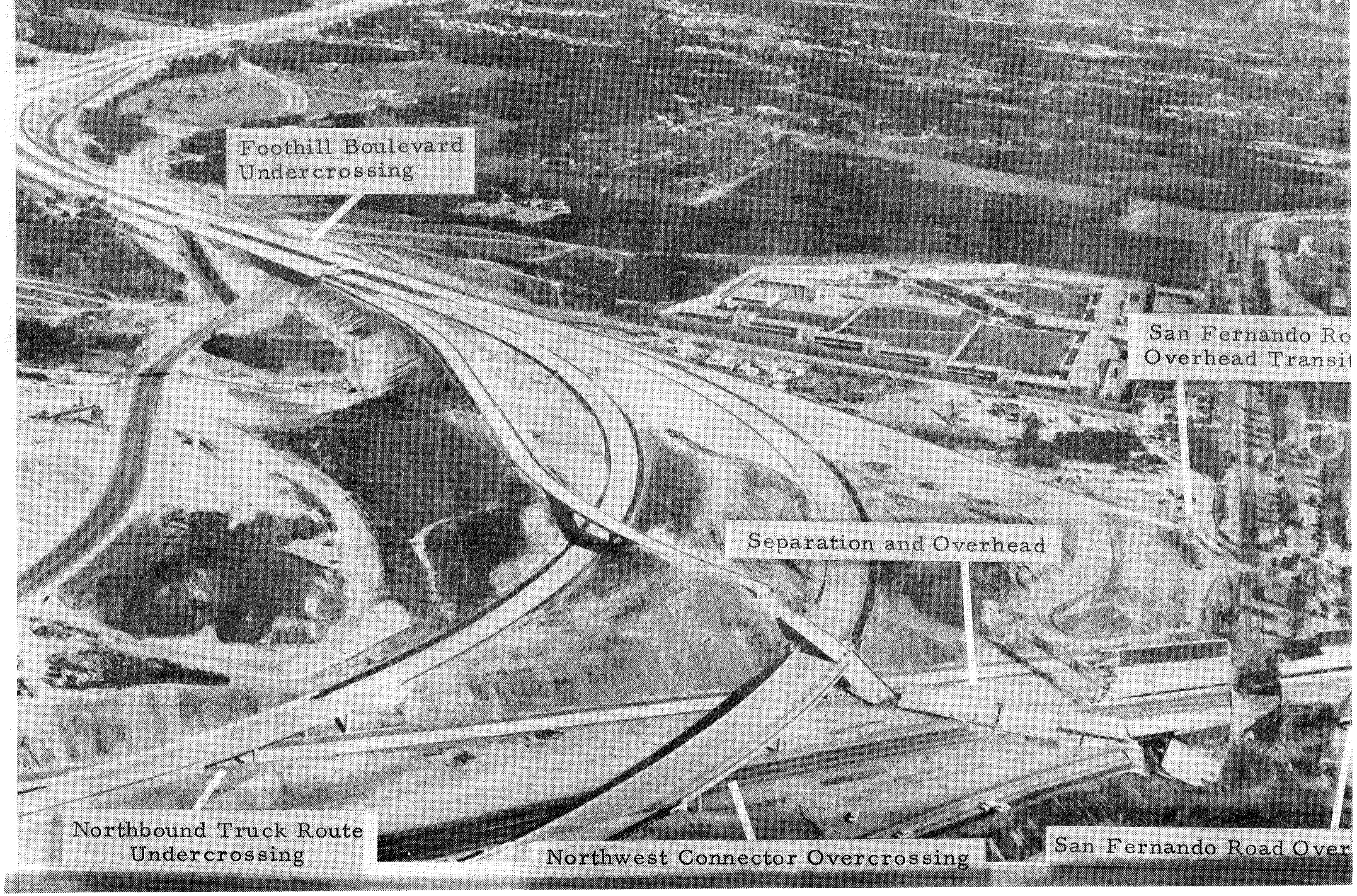
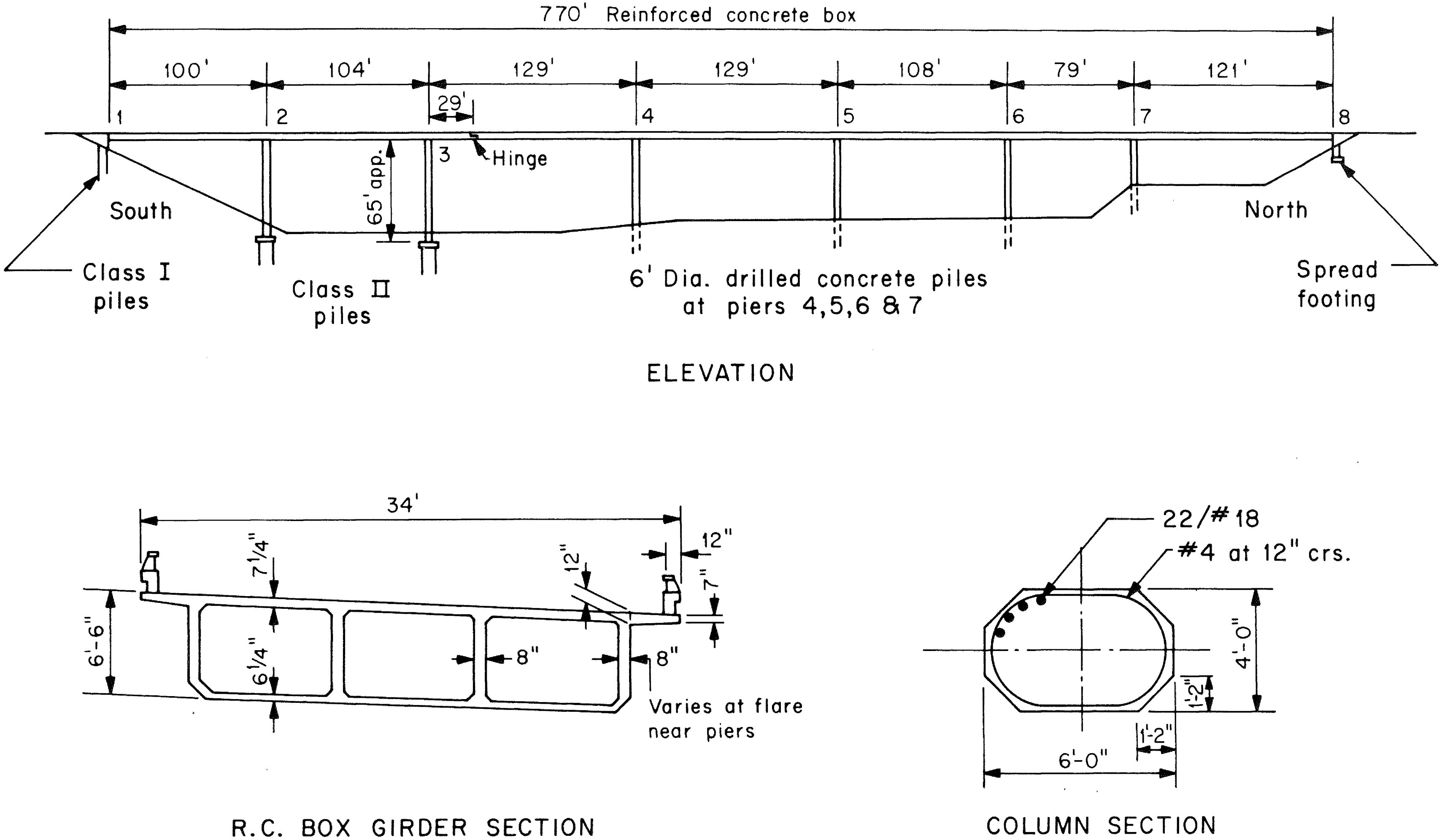


Fig. 11 The Golden State - Foothill freeway interchange, looking southeast. This view was taken several days after the earthquake and considerable demolition work had been carried out in the vicinity of the San Fernando Road overhead bridges.

Fig. 12 The Golden State - Foothill freeway interchange, looking west. This view was taken several hours after the earthquake. The San Fernando Road transition bridge, lower left, was still intact at this stage but note the vertical sag produced by a failure at the column base.



SEPARATION AND OVERHEAD STRUCTURE

GOLDEN STATE - FOOTHILL FREEWAY INTERCHANGE

Fig. 13

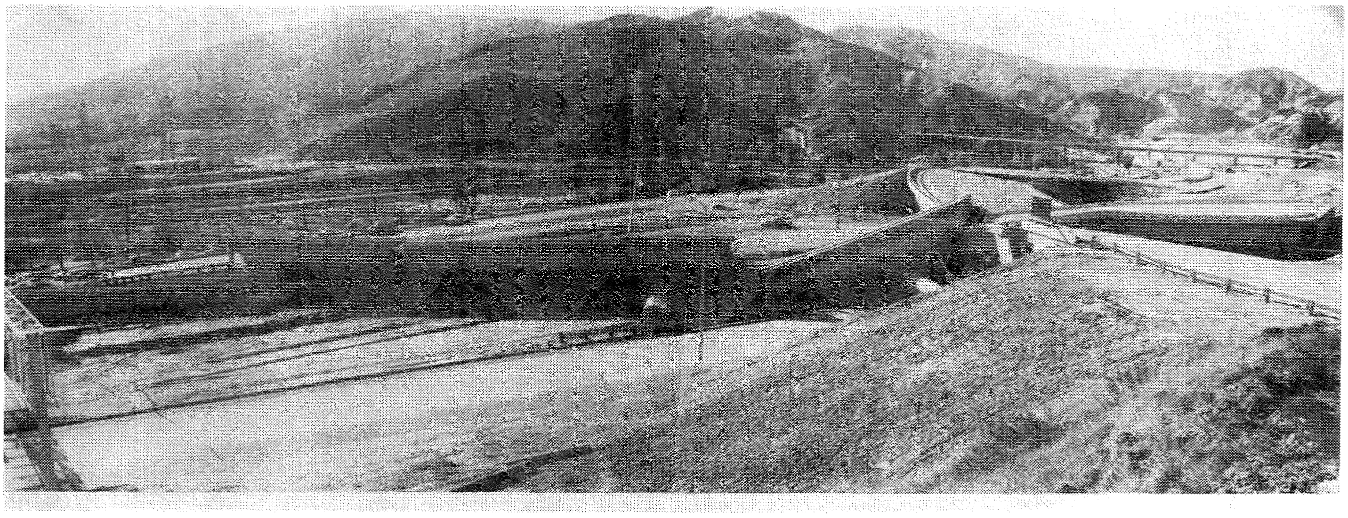
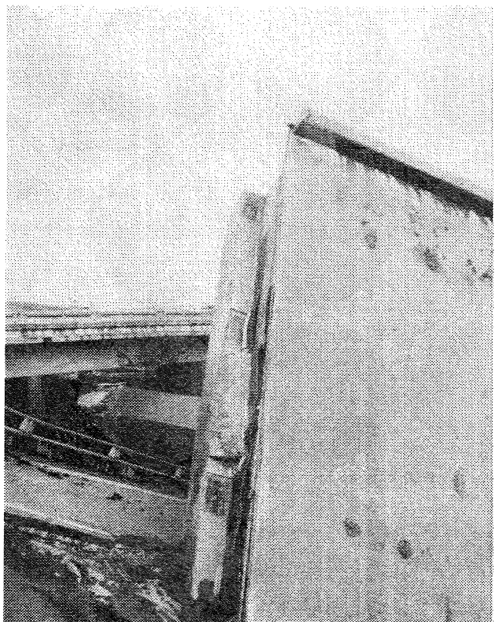


Fig. 14 Collapse of the Separation and Overhead bridge at the Golden State - Foothill freeway interchange. The northernmost end of the superstructure is visible at the far right. The bridge was a 770 ft R.C. box with a single hinge-joint.



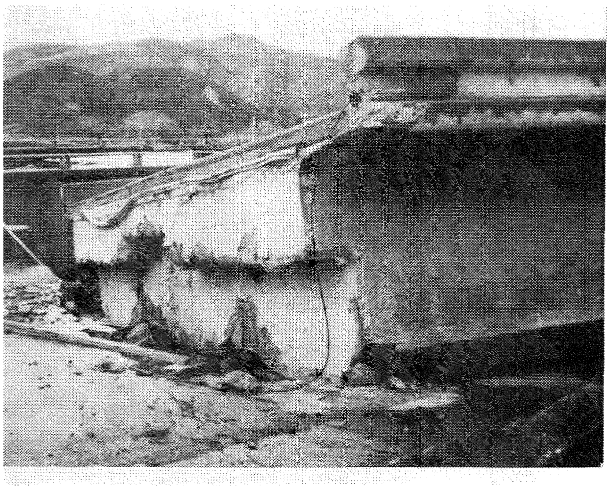


Fig. 15b

The two views show the separated sections at the hinge-joint of the Separation and Overhead bridge. A standard 14 in wide elastomeric hinge was used. Note the shear-key and the absence of hold-down or linkage bolts.

Fig. 15a

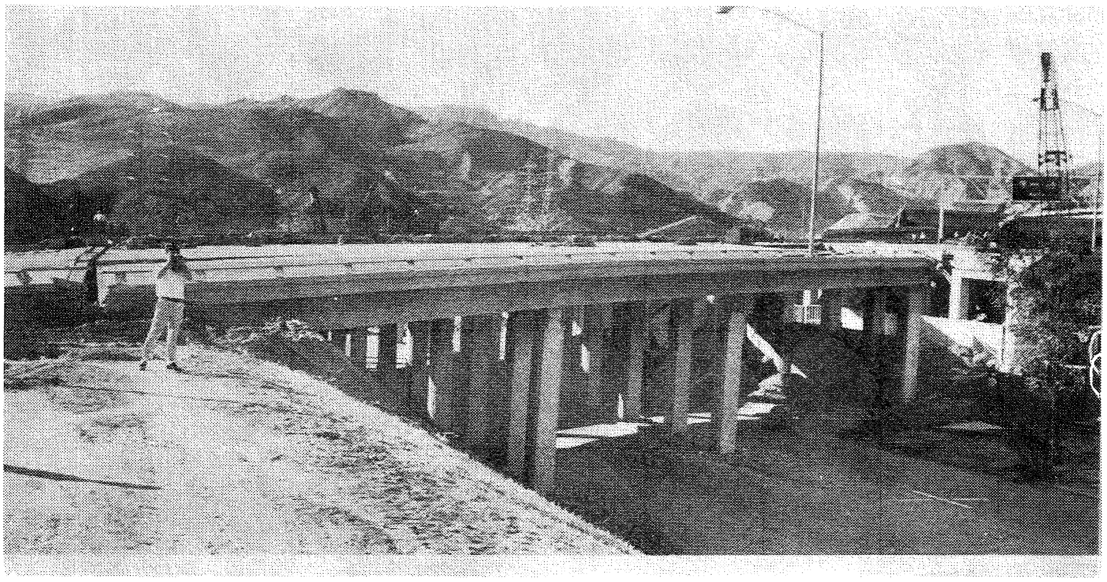


Fig. 16 The San Fernando Road Overhead bridges, Golden State - Foothill freeway interchange, looking northwest. Sections of the central span of the seven-span bridges collapsed. Note the separation at the abutment joint and the column failures.

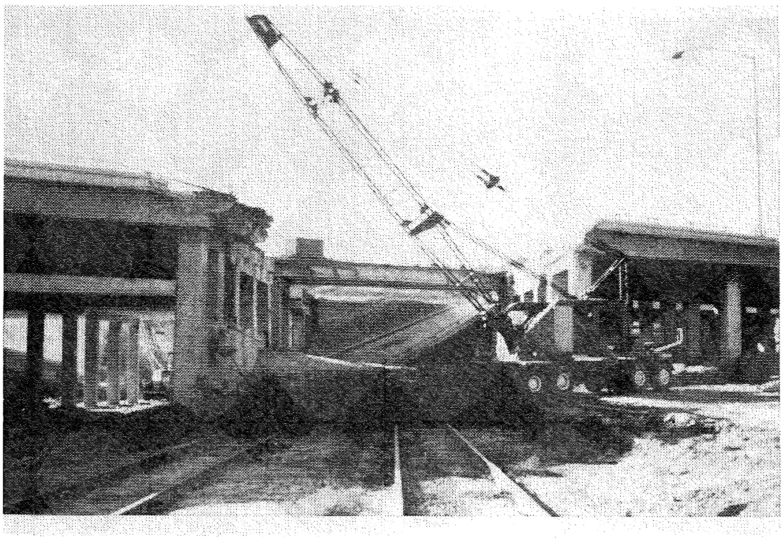
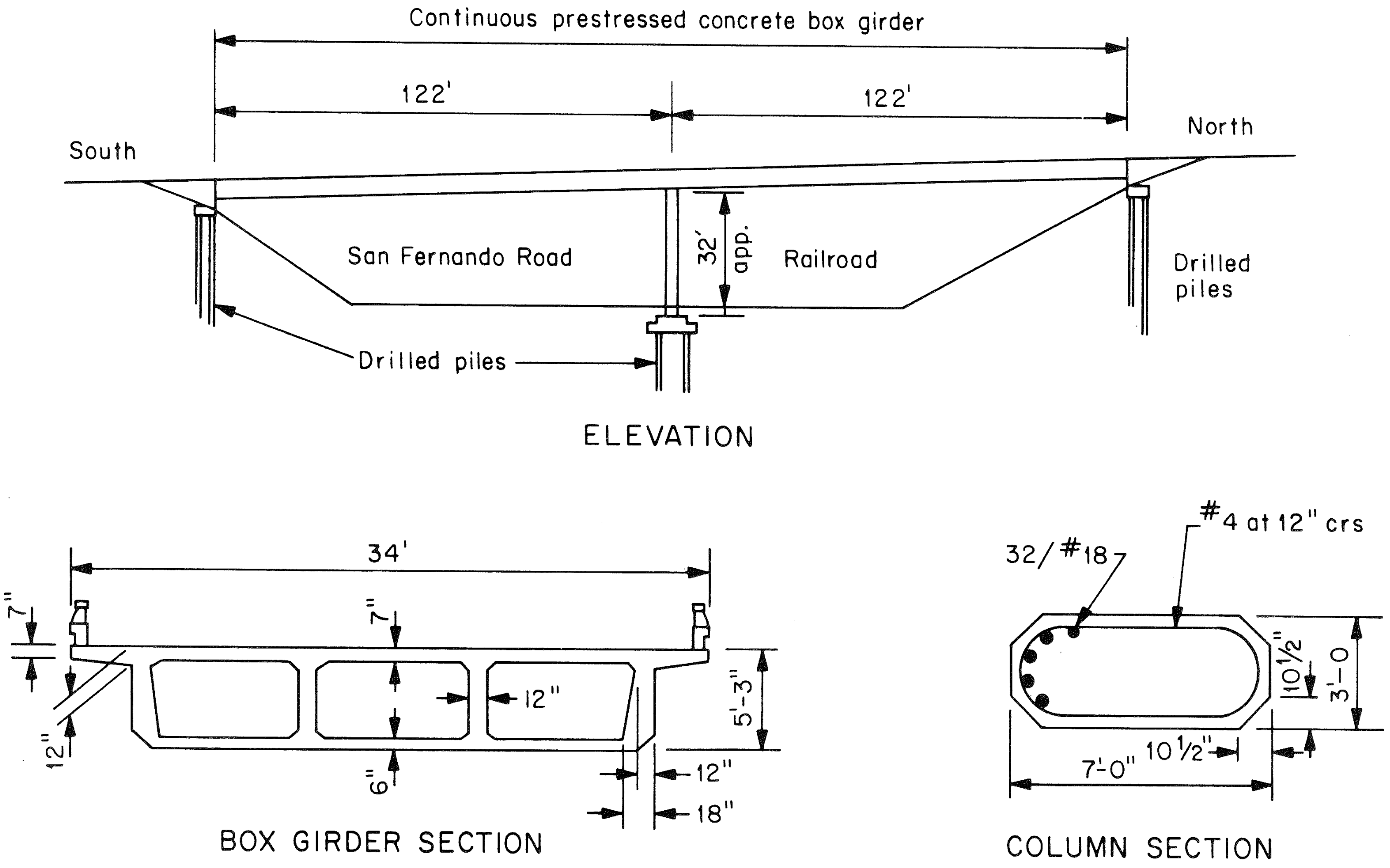


Fig. 17 Collapse of the simply-supported steel girder spans of the San Fernando Road overhead. The newer widening sections of prestressed girder construction remained intact. This view, the day following the earthquake, shows demolition work in progress to open the railway.



SAN FERNANDO ROAD OVERHEAD (TRANSITION)

GOLDEN STATE - FOOTHILL FREEWAY INTERCHANGE

Fig. 18

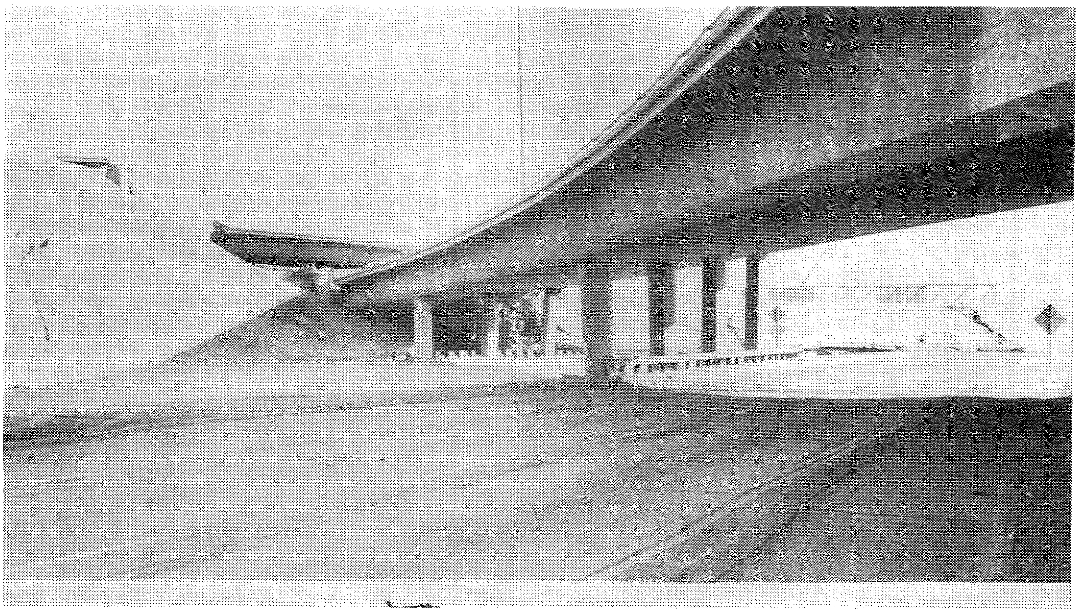


Fig. 19 The Northwest Connector Overcrossing, Golden State - Foothill freeway interchange, looking east. The Separation and Overhead bridge fell on the span adjacent to the eastern abutment of this bridge.

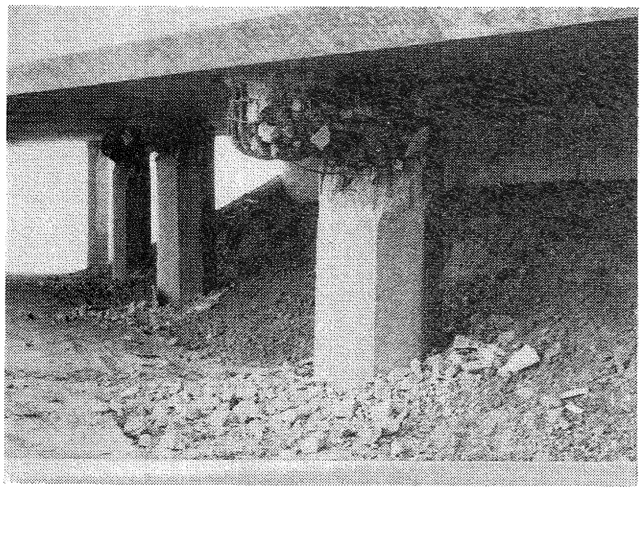


Fig. 20a

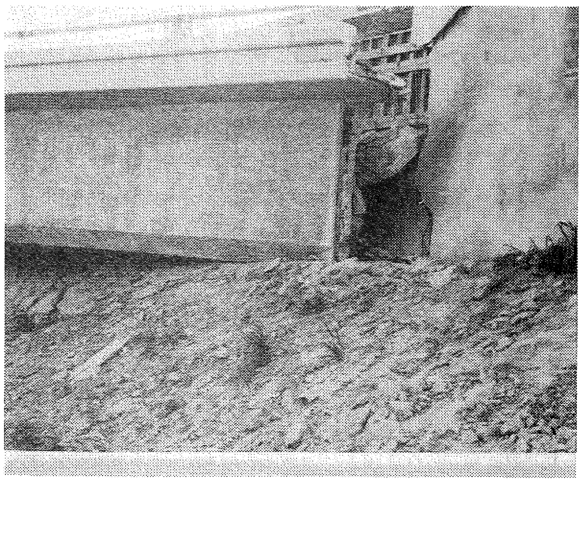


Fig. 20b

Pier and abutment failures at the western end of the Northwest Connector Overcrossing. The failures may have resulted from the combined influence of ground shaking, permanent displacement and the impact at the eastern end of the bridge.

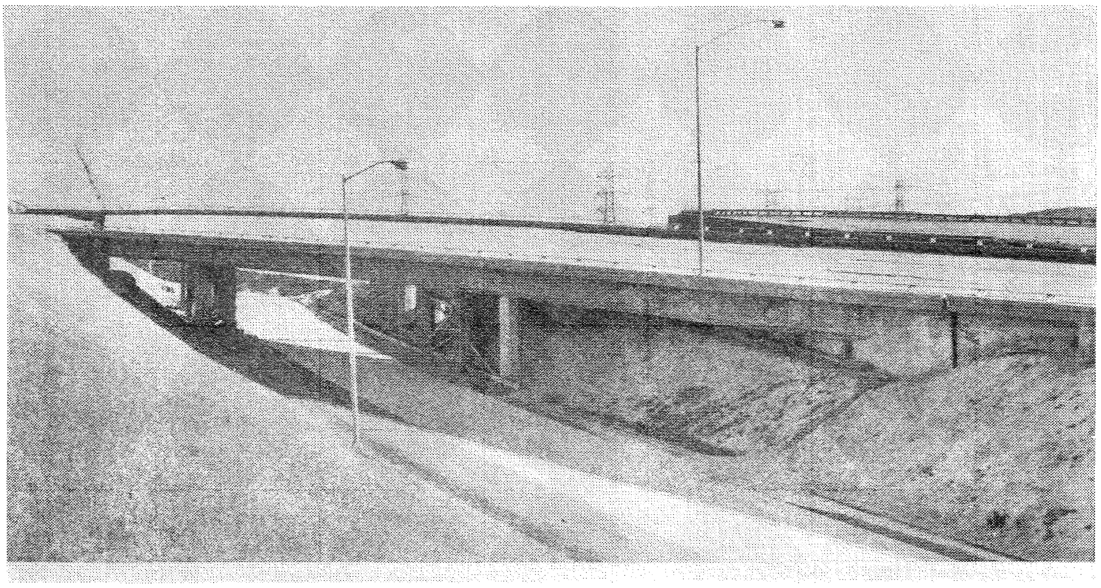


Fig. 21 The Northbound Truck Route Undercrossing, Golden State - Foothill freeway interchange, looking south. The eastern abutment has a skew of 55° and the western abutment is approximately 70° skew.

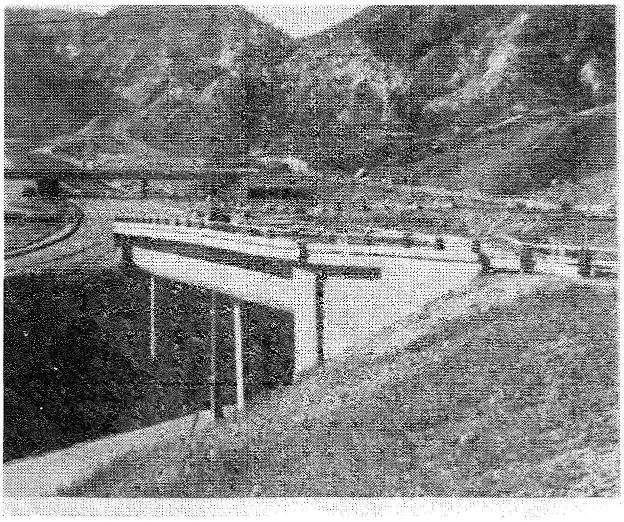


Fig. 22 The eastern abutment Northbound Truck Route Undercrossing. The bridge tended to rotate in a horizontal plane resulting in a 4 in displacement of the superstructure relative to the abutment wingwalls.

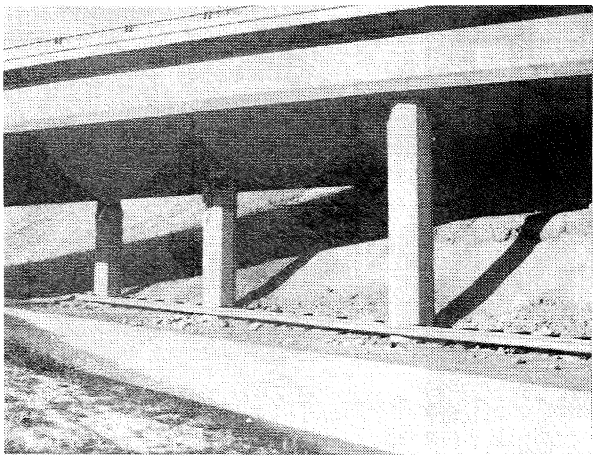


Fig. 23 Bending failures in the columns of the eastern pier of the Northbound Truck Route Undercrossing. The columns are on spread footings.

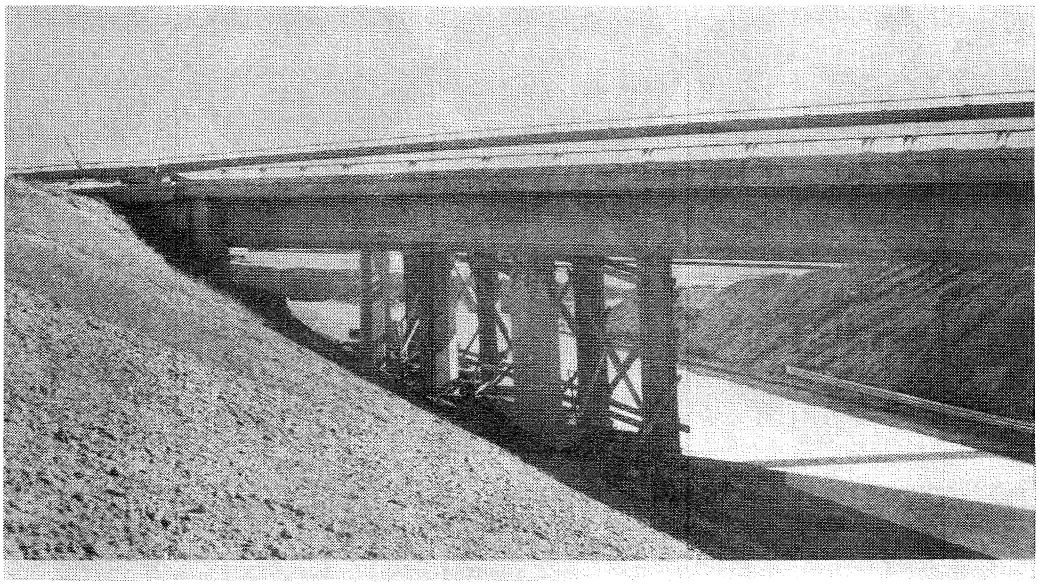


Fig. 24 Pier and abutment damage at the eastern end of the Northbound Truck Route Undercrossing. The continuous R. C. box bridge tended to rotate about the western end producing large displacements at the eastern end.

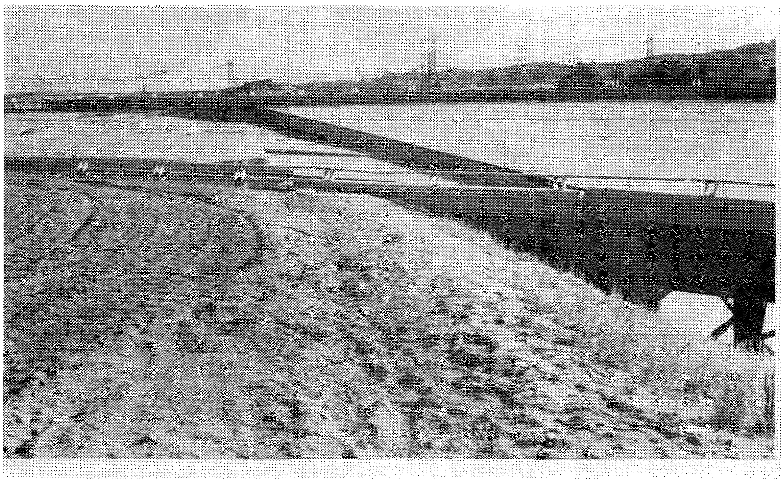
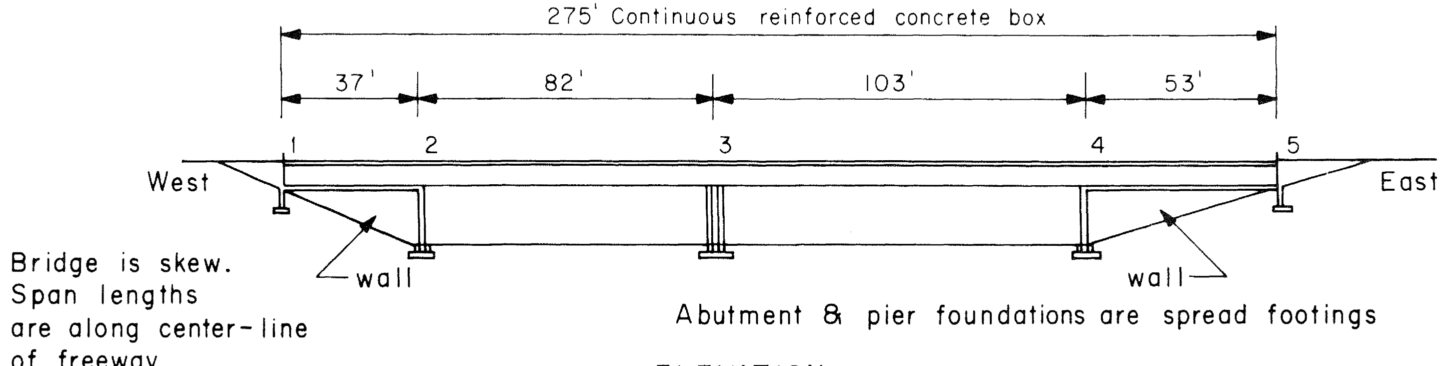
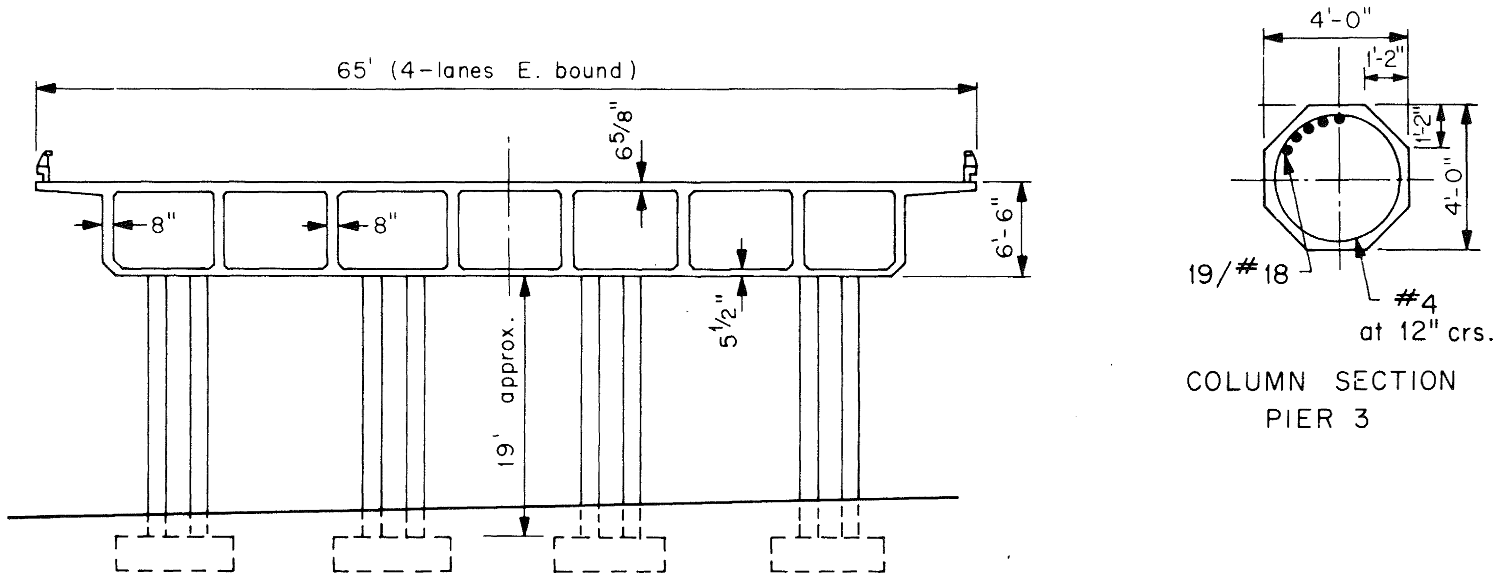


Fig. 25 Backfilling settlement and pavement failure at the eastern abutment of the Northbound Truck Route Undercrossing. Note the separation of the pavement from the abutment recess resulting in a potential barrier for emergency vehicles.



ELEVATION



CROSS-SECTION NEAR PIER 3 (NW Bridge not shown)

COLUMN SECTION
PIER 3

FOOTHILL BLVD. UNDERCROSSING - S.E. BRIDGE

FOOTHILL FREEWAY

Fig. 26

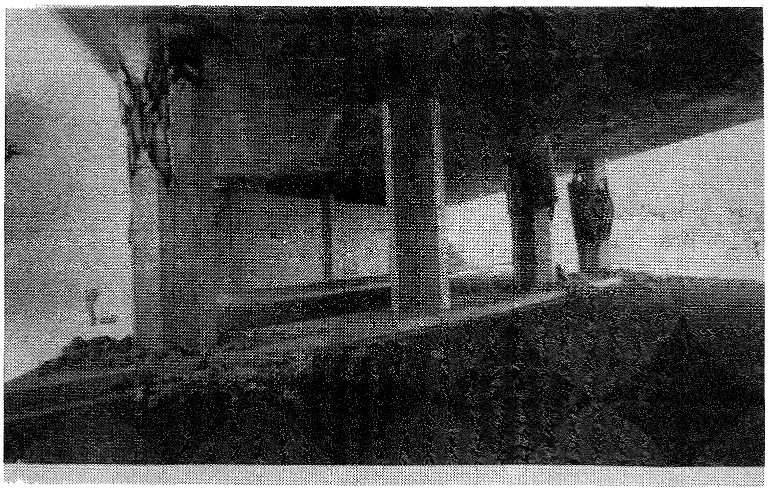


Fig. 27 Foothill Boulevard Undercrossing, southeastern bridge, Foothill freeway. Failures were severe in three of the central columns but the columns located between the walls at either end of this bridge were undamaged.

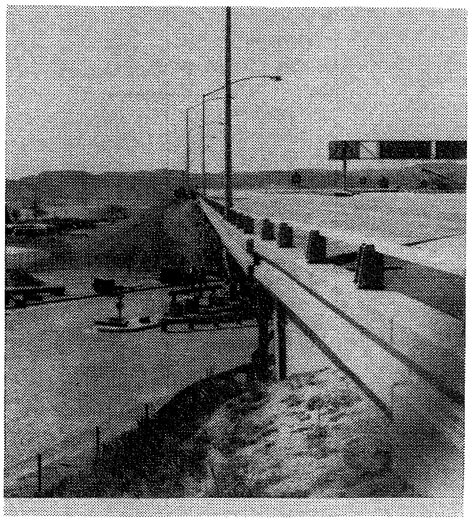


Fig. 28 The skew bridges of the Foothill Boulevard Undercrossing tended to rotate producing displacements of the superstructures relative to the abutment wingwalls of about 4 in.

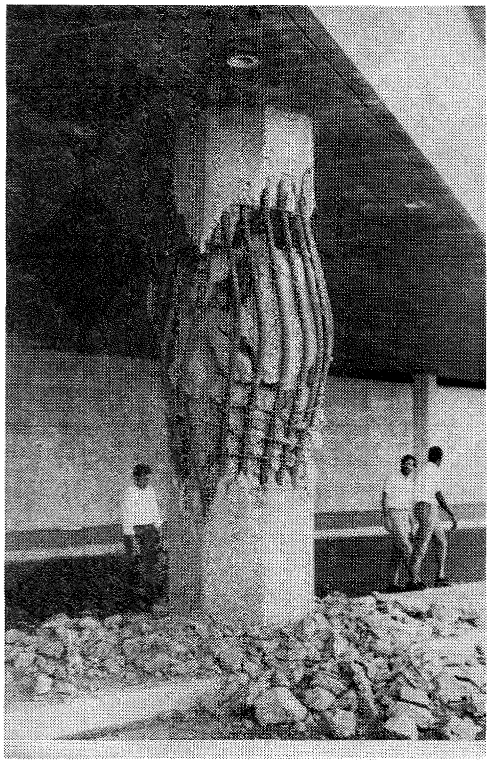
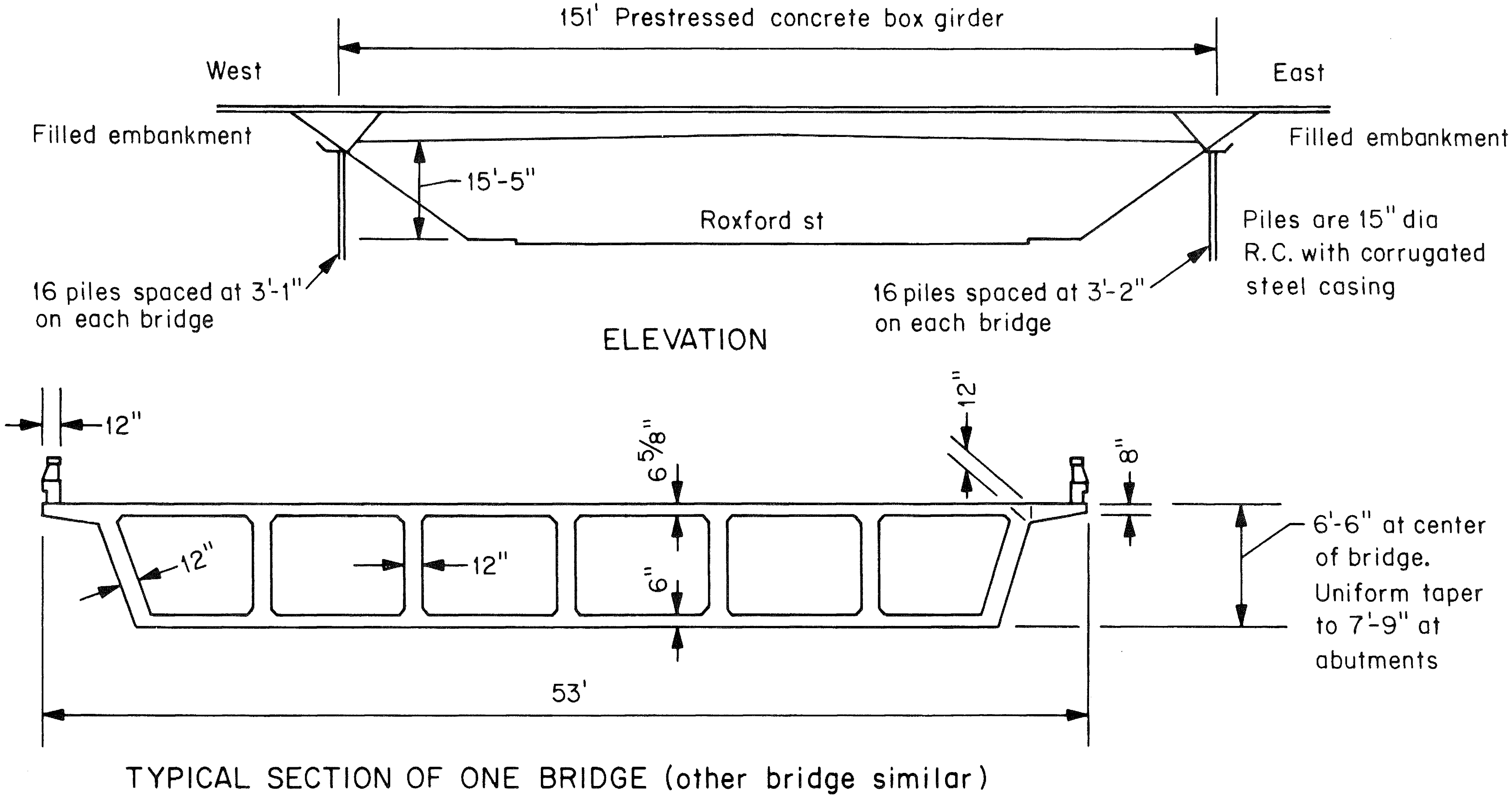


Fig. 29 Details of a central column failure, Foothill Boulevard Undercrossing. The No. 18 bars were tied with No. 4 bars spaced at 12 in.



ROXFORD STREET UNDERCROSSING

FOOTHILL FREEWAY

Fig. 30

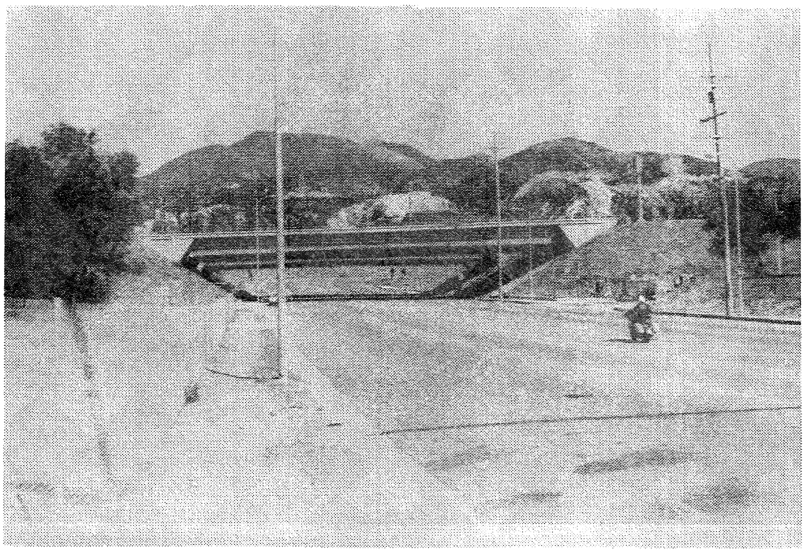


Fig. 31 The Roxford Street Undercrossing, Foothill freeway. Twin 151 ft long prestressed box girder bridges. Horizontal load pile failures occurred on both bridges.

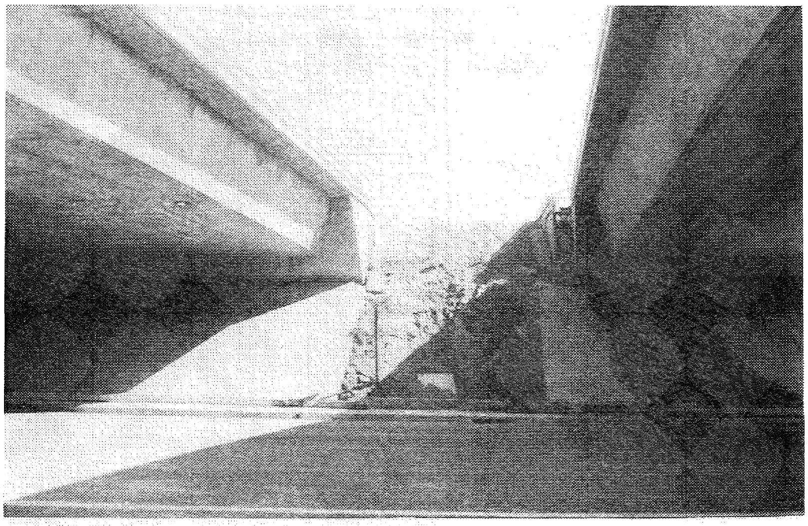


Fig. 32 Eastern abutments of the Roxford Street Undercrossing. At this end the southern bridge displaced transversely 2 ft 6 in and the other bridge moved about 1 ft.

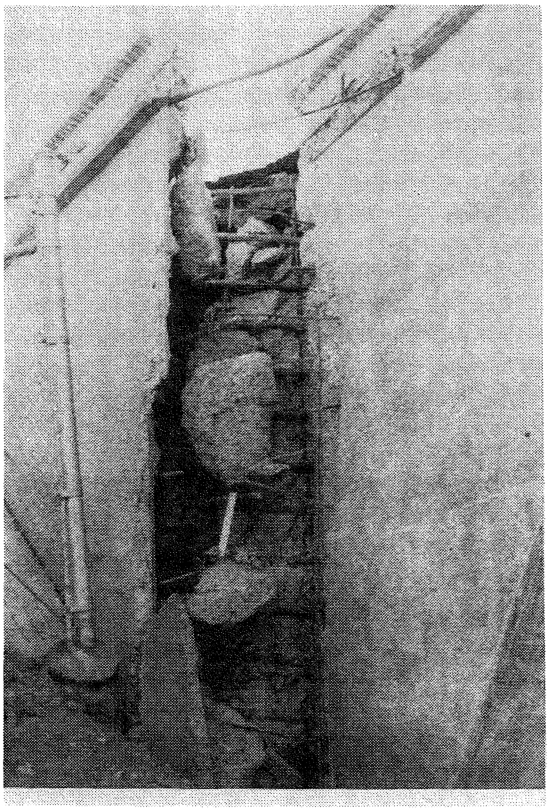


Fig. 33 Fracture of the wing-wall at the eastern abutment of the southern bridge, Roxford Street Undercrossing. A severe lateral failure of the piles at this abutment resulted in a 2 ft 6 in horizontal displacement.

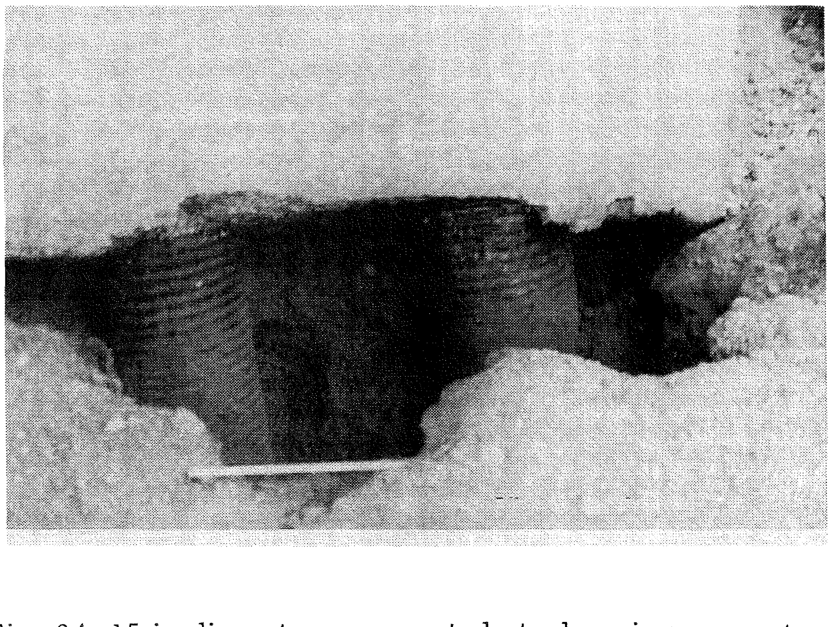
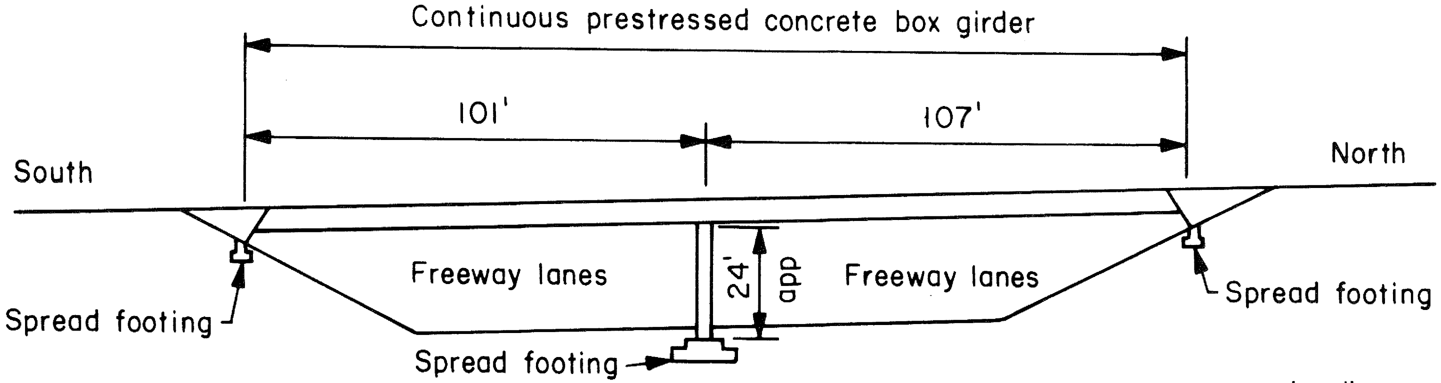
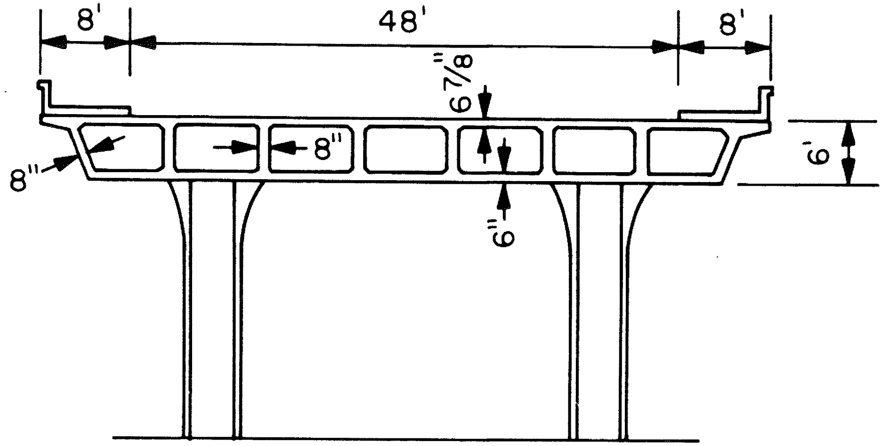


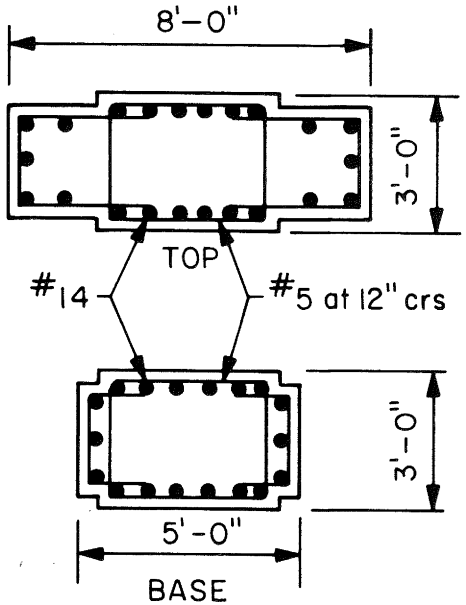
Fig. 34 15 in diameter corrugated steel casing concrete piles at the western abutment of the southern bridge, Roxford Street Undercrossing. Permanent displacement at this abutment was about 6 in. Apparently the piles failed by hinging at their tops and shearing through the soil.



ELEVATION



TYPICAL SECTION



COLUMN SECTIONS

BLEDSOE STREET OVERCROSSING

FOOTHILL FREEWAY

Fig. 35

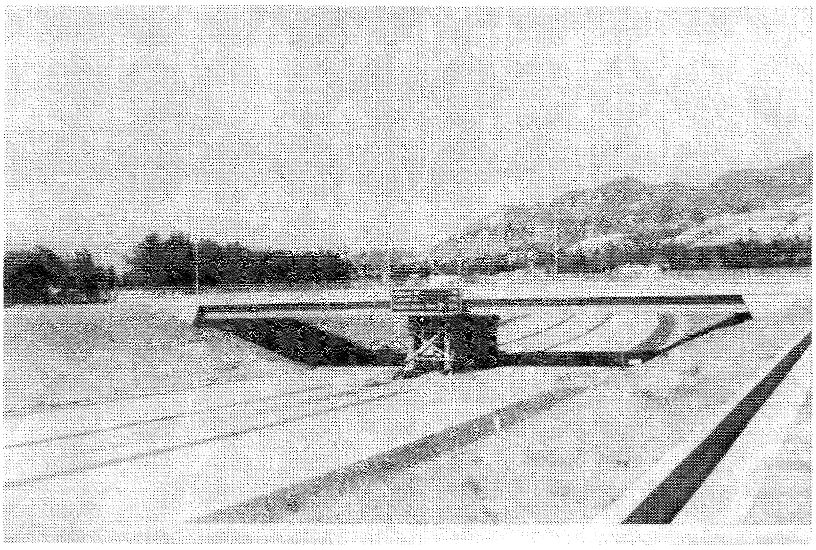


Fig. 36 Bledsoe Street Overcrossing, Foothill freeway. A continuous R. C. box bridge with spread footing foundations. The abutments settled relative to the central pier cracking the superstructure deck.

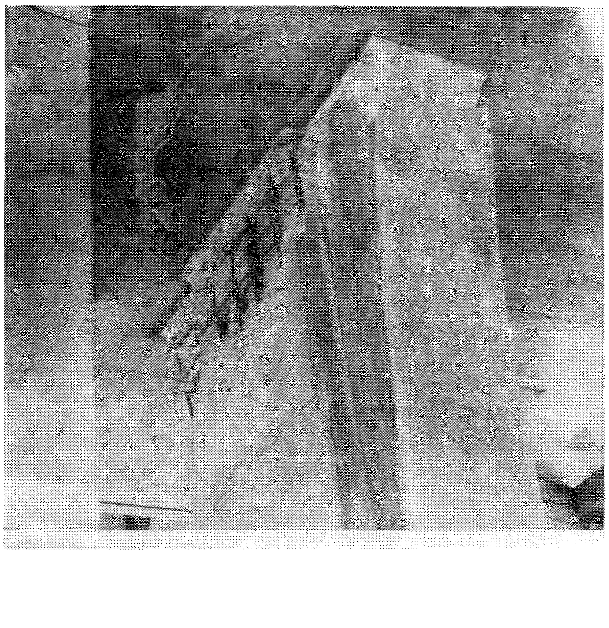


Fig. 37a

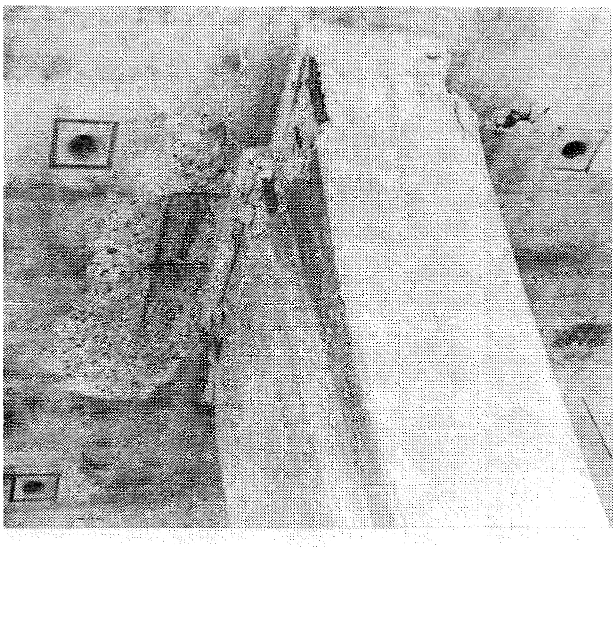


Fig. 37b

Spalling damage at the tops of the twin central columns of the Bledsoe Street Overcrossing. The damage was probably caused by excessive longitudinal vibrations and aggravated by the increase in axial load due to the differential settlement.

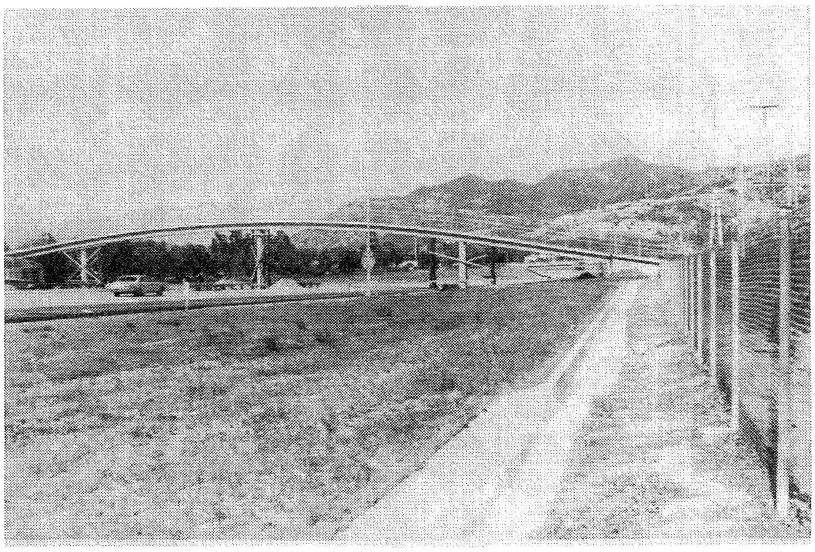


Fig. 38 Tyler Street Pedestrian Overcrossing, Foothill freeway. Longitudinal movements caused bending failures at the column tops and separation at the abutments.

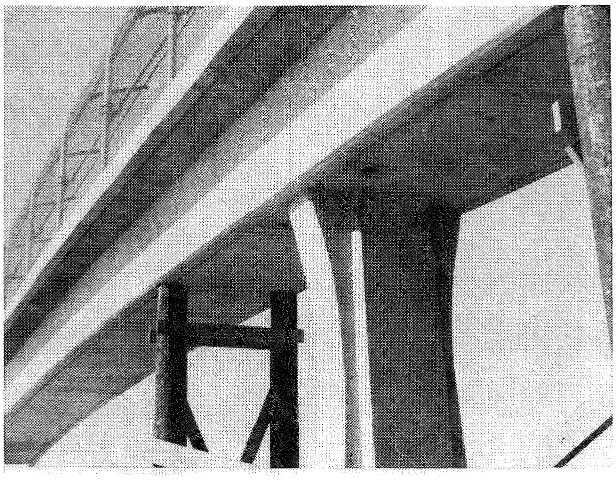


Fig. 39 Spalling of the top of the tallest column. Tyler Street Pedestrian Overcrossing

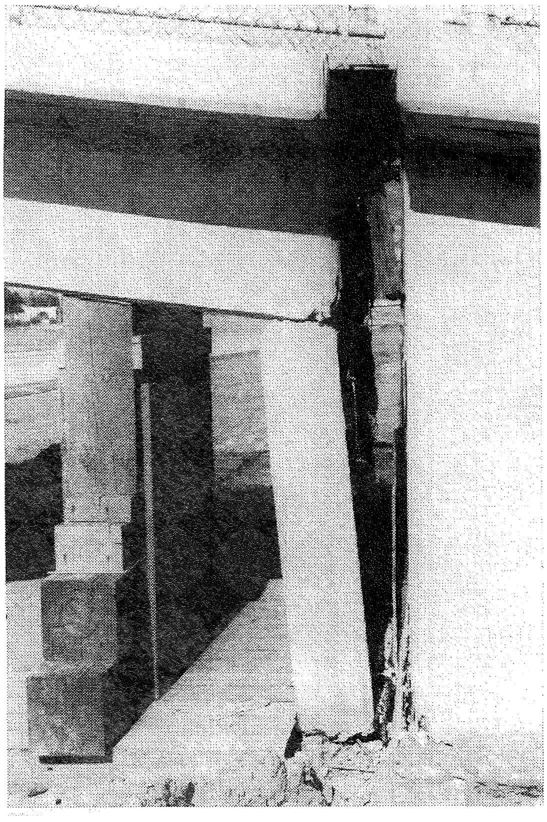


Fig. 40 Separation of the seating from the abutment structure at the northern end of the Tyler Street Pedestrian Overcrossing.

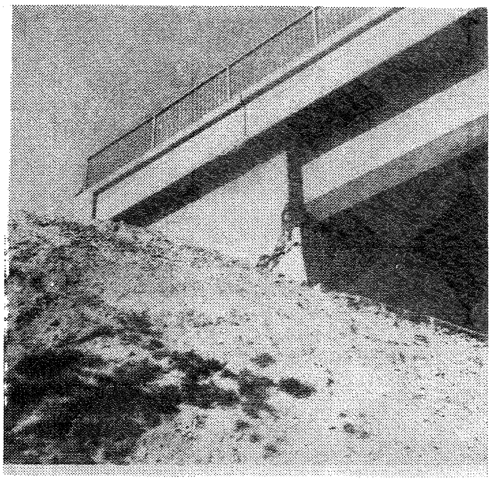


Fig. 41a

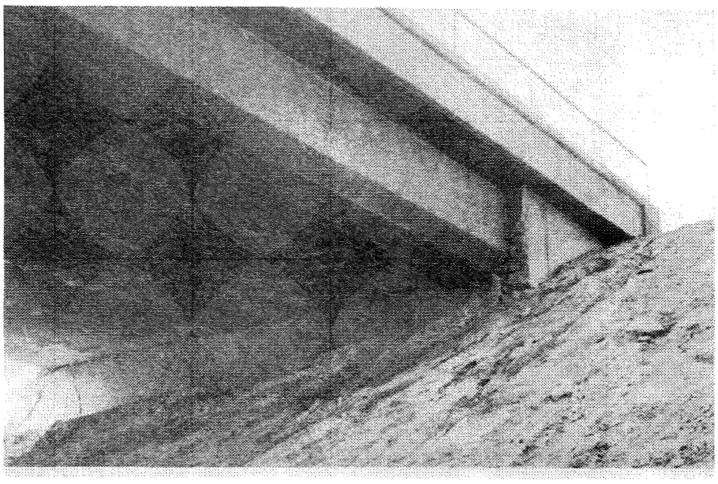


Fig. 41b

Abutment damage at the western end of the Balboa Boulevard Overcrossing, Golden State freeway. The 674 ft long R. C. box superstructure was cast monolithically with the spread footing abutment structure.

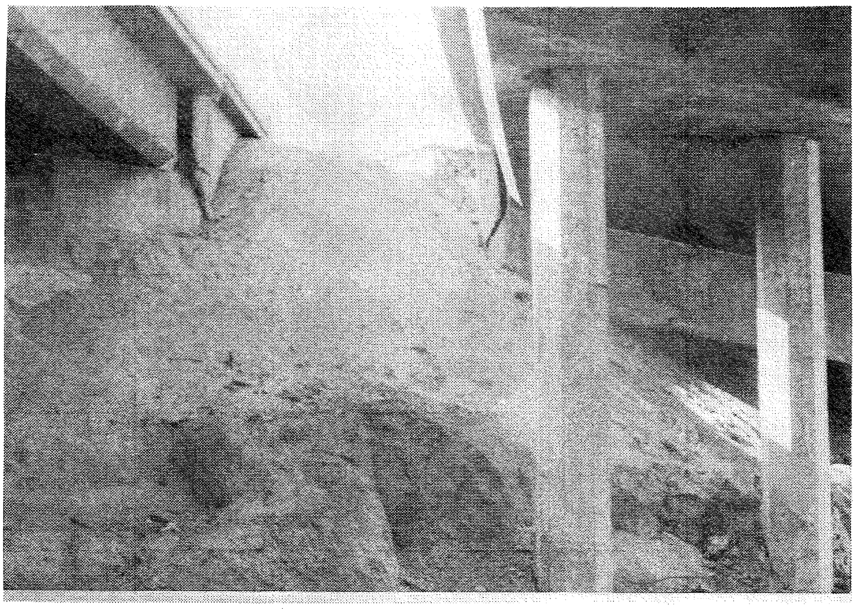


Fig. 42 Abutment damage at the northern end of the west Sylmar Overhead bridges, Golden State freeway. Apparently the skew superstructures were structurally separated from the abutments and seated on elastomeric pads. The abutment wingwalls failed at both ends of the bridges.

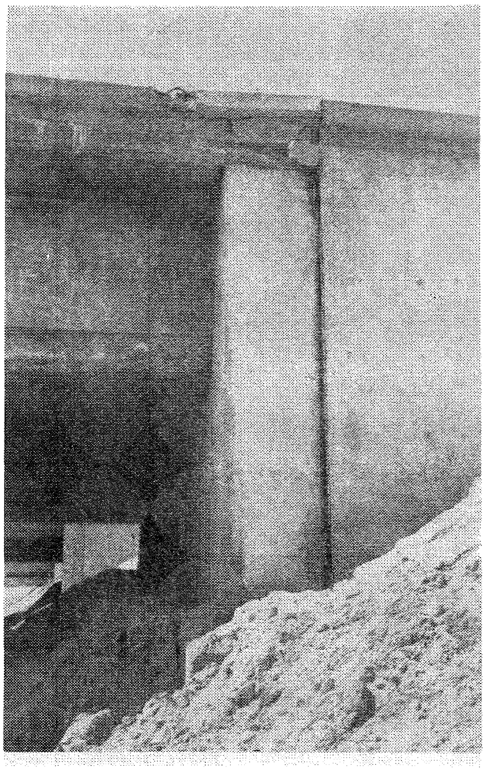


Fig. 43 Pounding damage at the joint between the abutment structure and wingwall at the eastern end of the Sierra Highway undercrossing, Angelope Valley freeway.

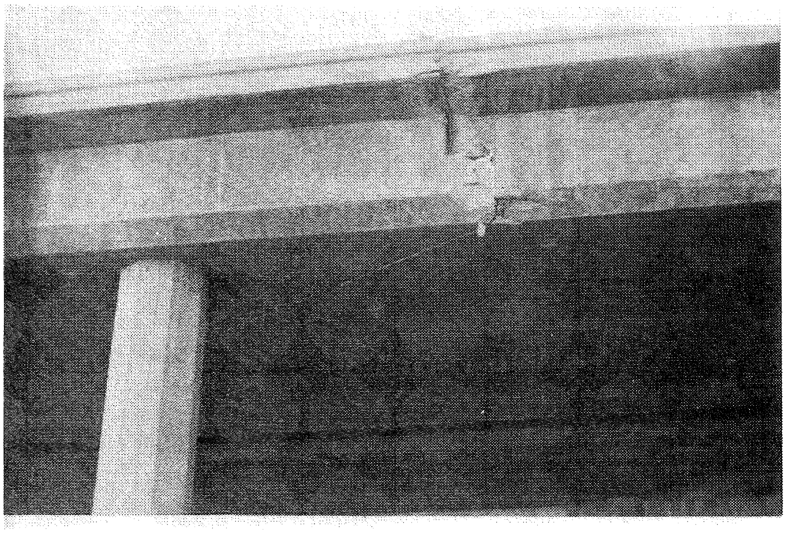


Fig. 44 Pounding damage at a skew joint in the superstructure of the West Sylmar Overhead, Golden State freeway.