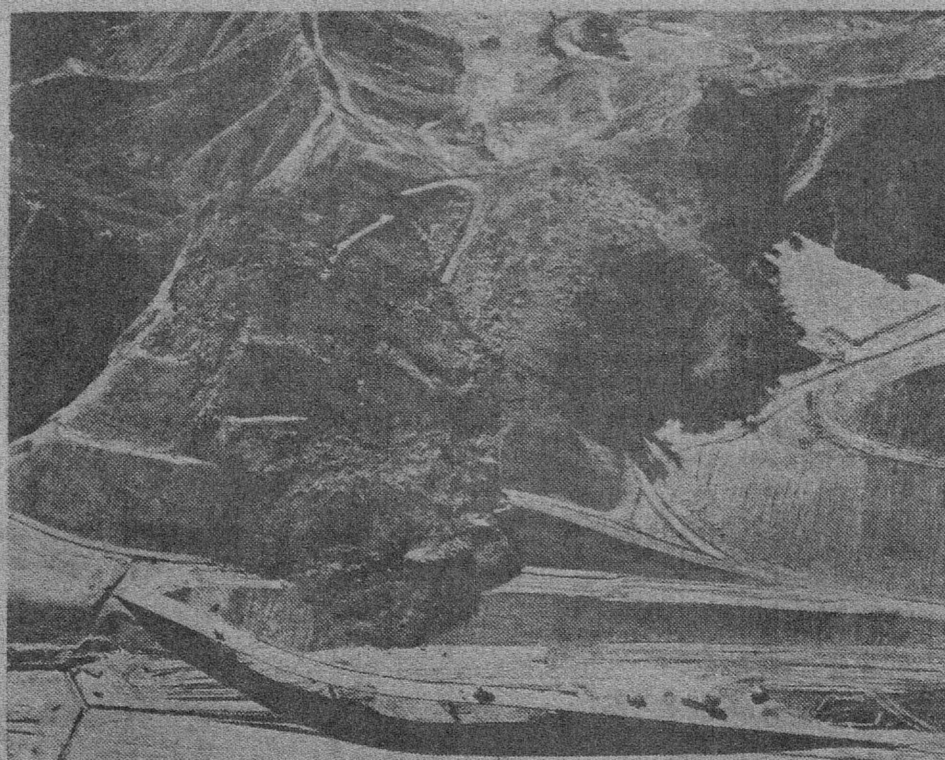
REPORT NO. FHWA/CA/TL-80/17

ROADWAY DAMAGE DURING THE SAN FERNANDO, CALIFORNIA EARTHQUAKE OF FEB. 9, 1971



FINAL REPORT

FEB., 1981

NOTICE

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This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration, under the title "Earthquake Induced Embankment Distress".

16. ABSTRACT

Roadway damage during the San Fernando, California, Earthquake of February 9, 1971, is described and illustrated based on in-depth field damage surveys made immediately after the quake. The damage is documented by Route No. and then discussed relative to roadway elements such as embankments, embankment foundations, cut slopes, pavement, drainage structures, and bridge abutment backfill. The damage is further discussed with regard to soil type and behavior during the strong ground motion. Recommendations for minimizing certain types of damage on future projects are presented. Bridge damage was investigated separately and is not described in this report.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
Earthquakes, earthquake effects, roadway damage, cut slopes, embankments, pavement, geological faults, slope stability, surveys, data collection, landslides, bridge approaches.	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

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DEPARTMENT OF TRANSPORTATION
DIVISION OF CONSTRUCTION
OFFICE OF TRANSPORTATION LABORATORY

REPORT NO. FHWA/CA/TL-80/17

ROADWAY DAMAGE DURING THE SAN FERNANDO, CALIFORNIA, EARTHQUAKE OF FEB. 9, 1971

Study Supervised by .... Raymond A. Forsyth, P.E.

Co-Principal Investigators .... R. H. Prysock, P.E.
J. P. Egan, Jr.

Report Prepared by .... R. H. Prysock, P.E.
J. P. Egan, Jr.

ROBERT O. WATKINS, P.E.
Chief, Office of Transportation Laboratory

CONVERSION FACTORS

English to Metric System (SI) of Measurement

Quanity	English unit	Multiply by	To get metric equivalent
Length	inches (in)or(")	25.40	millimetres (mm)
		.02540	metres (m)
	feet (ft)or(')	.3048	metres (m)
	miles (mi)	1.609	kilometres (km)
Area	square inches (in^2)	6.432×10^{-4}	square metres (m^2)
	square feet (ft^2)	.09290	square metres (m^2)
	acres	.4047	hectares (ha)
Volume	gallons (gal)	3.785	litres (l)
	cubic feet (ft^3)	.02832	cubic metres (m^3)
	cubic yards (yd^3)	.7646	cubic metres (m^3)
Volume/Time			
(Flow)	cubic feet per second (ft^3/s)	28.317	litres per second (l/s)
	gallons per minute (gal/min)	.06309	litres per second (l/s)
Mass	pounds (lb)	.4536	kilograms (kg)
Velocity	miles per hour(mph)	.4470	metres per second (m/s)
	feet per second(fps)	.3048	metres per second (m/s)
Acceleration	feet per second squared (ft/s^2)	.3048	metres per second squared (m/s^2)
	acceleration due to force of gravity(G)	9.807	metres per second squared (m/s^2)
Weight Density	pounds per cubic (lb/ft^3)	16.02	kilograms per cubic metre (kg/m^2)
Force	pounds (lbs)	4.448	newtons(N)
	kips (1000 lbs)	4448	newtons (N)
Thermal Energy	British thermal unit (BTU)	1055	joules (J)
Mechanical Energy	foot-pounds(ft-lb)	1.356	joules (J)
	foot-kips (ft-k)	1356	joules (J)
Bending Moment or Torque	inch-pounds(ft-lbs)	.1130	newton-metres (Nm)
	foot-pounds(ft-lbs)	1.356	newton-metres (Nm)
Pressure	pounds per square inch (psi)	6895	pascals (Pa)
	pounds per square foot (psf)	47.88	pascals (Pa)
Stress Intensity	kips per square inch square root inch (ksi $\sqrt{in}$)	1.0988	mega pascals $\sqrt{metre}$ (MPa $\sqrt{m}$)
	pounds per square inch square root inch (psi $\sqrt{in}$)	1.0988	kilo pascals $\sqrt{metre}$ (KPa $\sqrt{m}$)
Plane Angle	degrees (°)	0.0175	radians (rad)
Temperature	degrees fahrenheit (F)	$\frac{tF - 32}{1.8} = tC$	degrees celsius (°C)

ACKNOWLEDGEMENTS

Thanks are extended to those individuals of Caltrans' District 7, and the local governments and utility companies who provided assistance when needed and were completely cooperative during the field investigation reported herein.

Appreciation also is extended to Bob Branch of the District 7 Materials Section, who prepared the "Effects of the Earthquake" portion of this report. Typing was by Lydia Burgin, and editorial assistance was by John Campbell and Max Alexander.

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INTRODUCTION

The San Fernando earthquake of February 9, 1971, was the most damaging quake to occur in California since the Long Beach earthquake of March 10, 1933. Although of moderate magnitude (6.4), it inflicted greater damage on the California freeway system than any previous earthquake. The major north-south link (Interstate 5) between the San Joaquin Valley and Los Angeles, which served 80,000 to 90,000 vehicles daily, was rendered inoperable and hence closed to traffic for a period of 15 hours. Sixty-six days elapsed before a 6-lane, long term detour could be provided to allow free movement of traffic. Other routes that sustained damage included Interstates 210 and 405 and State Routes 2, 14, and 118; the latter two being in various stages of construction and not open to traffic in the affected area at the time of the earthquake. Major road damage was concentrated within a remarkably small area of about 12 square miles in the northern portion of the San Fernando Valley and involved only about 10 miles of operating freeways. The above routes and area of major damage are shown in Figure 1, and to a larger scale in Figure 2.

Purposes and Scope of Study

The purposes of this study were to inspect and document roadway damage; determine, if possible, the mechanisms by which different types of damage occurred; and determine if the damage was related to design practices and/or construction methods then employed.

The scope of the study included the following:

1. A cursory review of roads and adjacent areas to delineate limits of major roadway damage and to determine overall damage patterns.

2. A review of ground breakage to provide insight to road damage and to select individual areas for further study.

3. A detailed inspection of road damage to provide photographic documentation and field notes of observations.

4. Special work at a few locations to obtain quantitative data primarily for aids in interpreting observations.

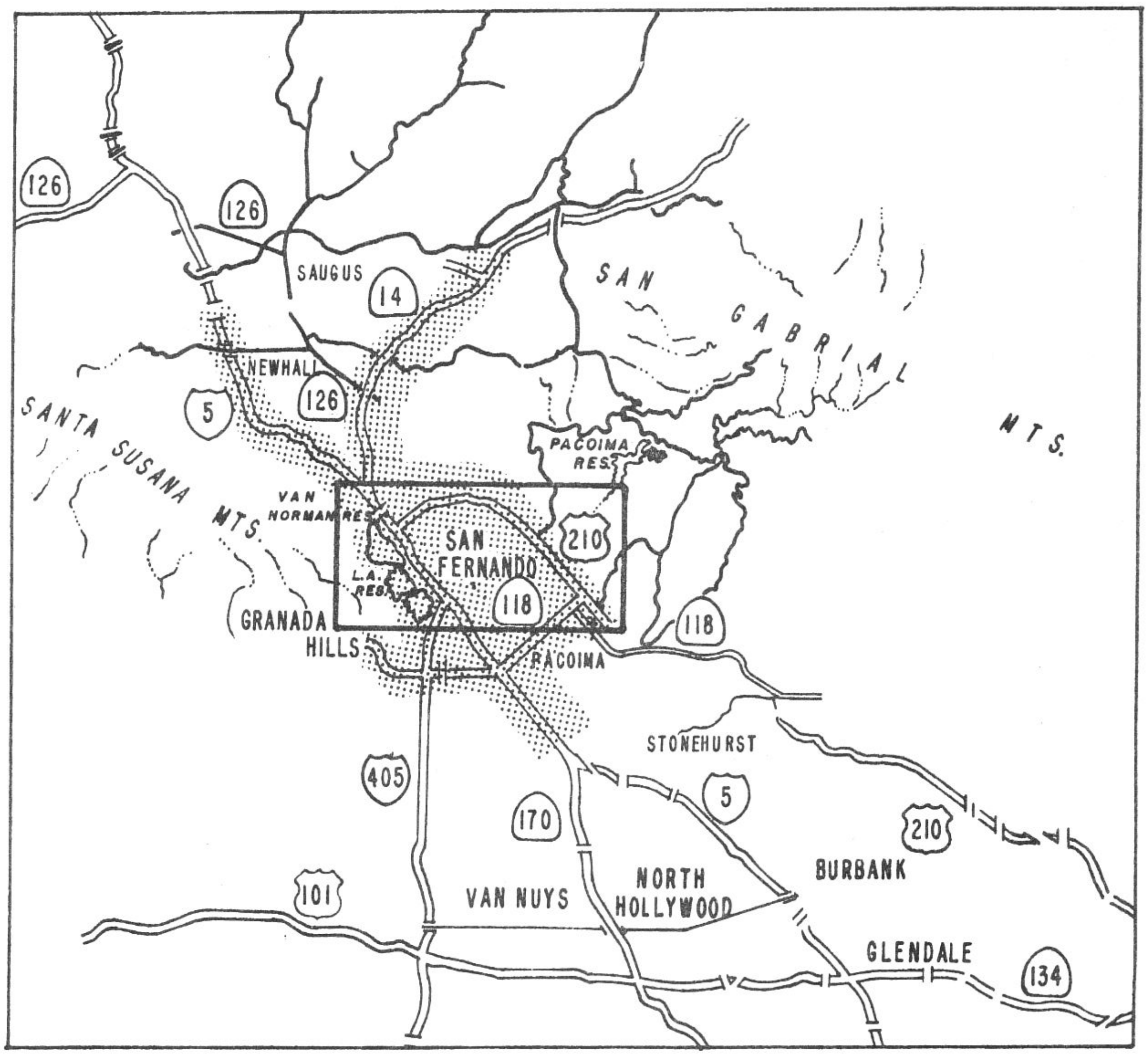
Although this study was oriented toward reviewing roadway earthwork damage (notably embankments and cut slopes), other elements of the roadway were reviewed for an overall appraisal of damage and for the sake of completeness. Furthermore, areas adjacent to, but outside, the highway rights of way were studied to provide more information regarding soil and earthwork behavior.

This Report

Much of the information presented in this report was included in earlier interim reports (1, 2). The purpose of the present report is to provide more photographic documentation, to describe the repair and reconstruction operations, and to discuss briefly the effect of the San Fernando event on Caltrans' position regarding earthquake engineering. It should be noted that a detailed description of bridge damage and subsequent changes in Departmental bridge aseismic design procedures are outside the scope of this project and may be found elsewhere (3,4).

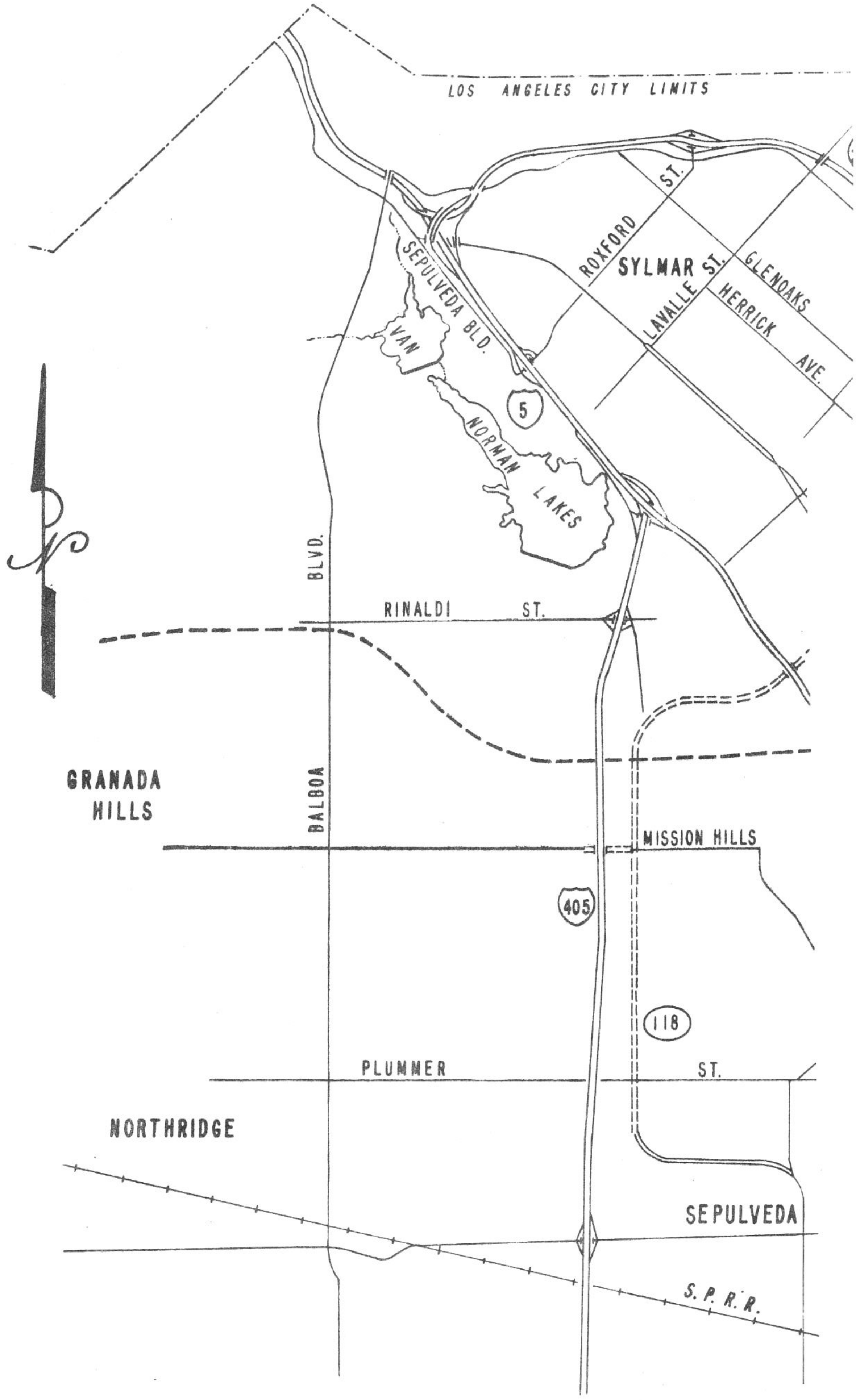
Information regarding seismological and geological aspects of the earthquake, as well as descriptions of damage to other facilities, may be found in References 5-7.

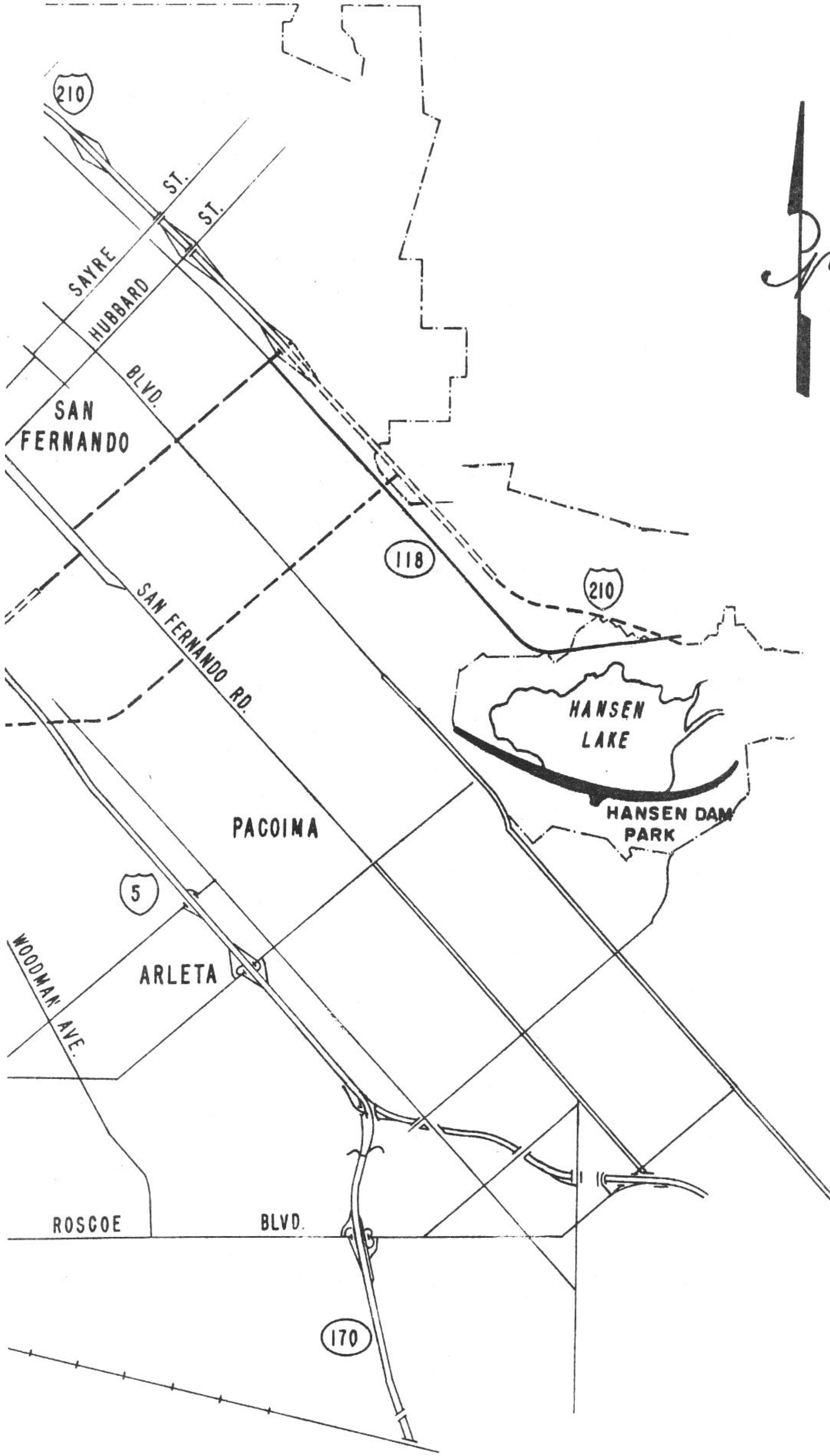
Figure 1
Vicinity Map of Freeway Damage Area



STREET MAP OF FREEWAY DAMAGE AREA

FIGURE 2




210
SAYRE ST.
HUBBARD ST.
BLVD.
SAN FERNANDO
118
210
SAN FERNANDO RD.
HANSEN LAKE
HANSEN DAM PARK
PACOIMA
5
ARLETA
WOODMAN AVE.
ROSCOE BLVD.
170

ROADWAY DAMAGE

Within the field review area shown in Figure 1, all principal elements of the freeways sustained damage to varying degrees. The following descriptions and photographs of damage by route are included as documentation of field observations and as illustrations of the behavior of earthworks and other structural elements of roadways when subjected to ground shocks similar to those experienced during the San Fernando earthquake.

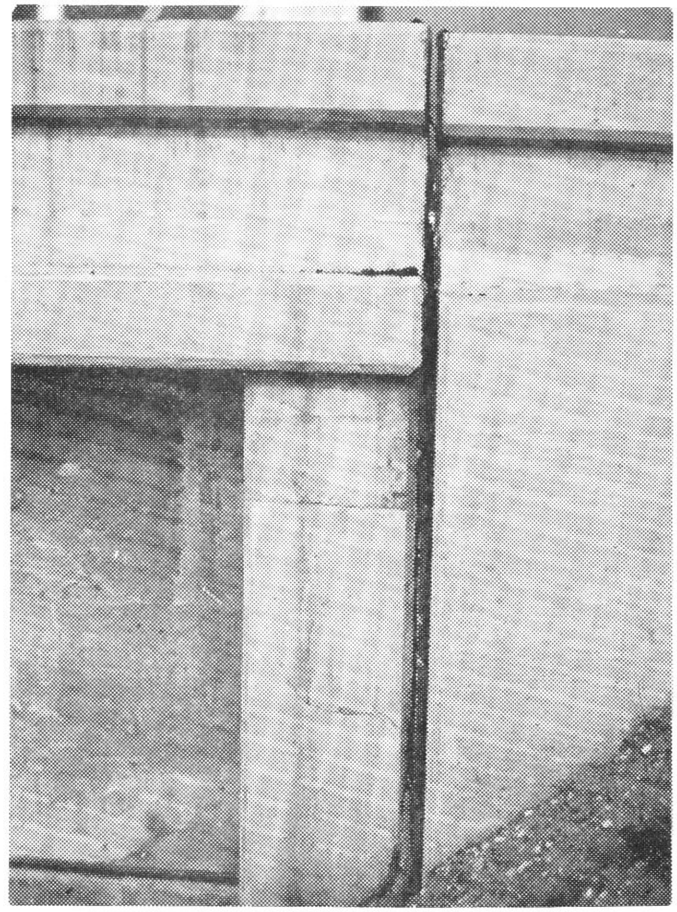
ROUTE 5, FROM OSBORNE STREET TO I-5/405 SEPARATION

Damage to the Route 5 freeway facility south of the Route I-5/405 Separation was slight, with no emergency repairs to the roadway or bridges required. With the exception of the Rinaldi Street Bridge and the East Canyon Channel structure, damage consisted of superficial cracking of curbs and sidewalks, some settlement of sidewalks at undercrossings relative to the bridge curtain walls, some settlement and separation of wingwalls, slight rotation of some bridge decks, spalling at construction or expansion joints, and minor soil cracking around abutments, both in fill and original ground.

Damage to the Rinaldi Street Undercrossing consisted of abutment settlement, shattering of the abutment corners, and wingwall movement. Settlement and separation of the south abutment wingwall are shown in Photo No. 1.

Reinforced concrete box pedestrian undercrossings suffered minor cracking and separation at construction joints.

Photo 1. Looking easterly at south abutment of Rinaldi Street Undercrossing. Note settlement of wingwall, separation at construction joint, and crack in diaphram.



At the East Canyon Channel, just north of Rinaldi Street, a reinforced concrete double 12x14 box crosses beneath the freeway. The box sustained cracks up to 1/4-inch wide in all walls. At the box crossing a slight rise was noted in the roadway profile due to fill settlement on either side of the box.

About 1/2 mile north of Rinaldi Street an AC patch had been placed across the southbound lanes prior to this investigation. Immediately west of the patch a concrete gutter had undergone compression spalling, which suggests the pavement had buckled.

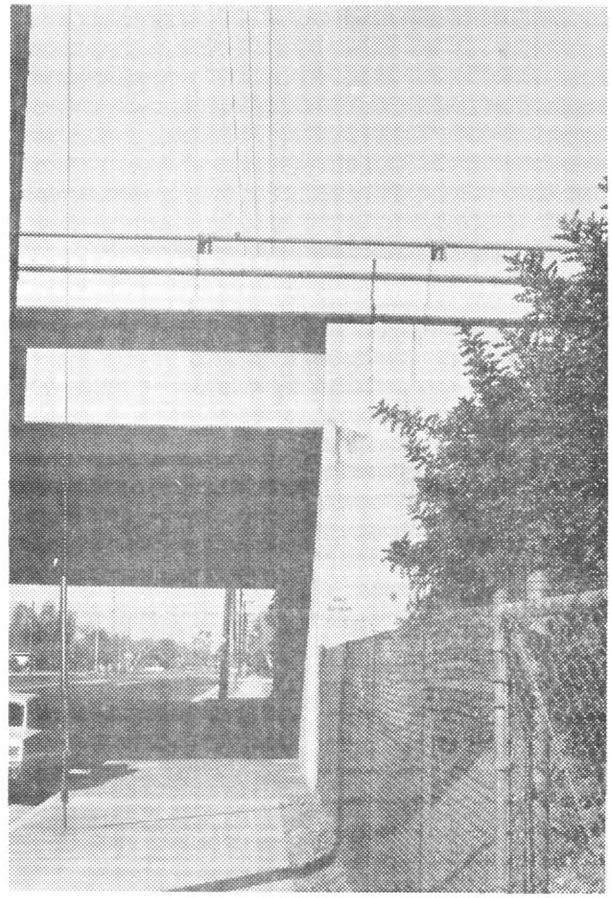
ROUTE 405, FROM ROSCOE BOULEVARD TO I-5/405 SEPARATION

South of the Route I-5/405 Separation, sixteen structure sites were reviewed, which were of two basic types: pedestrian undercrossings and street undercrossings. Each type of structure was of one design; consequently, the distress patterns were similar for all structures within each group. Damage with respect to structural integrity was considered to be negligible and traffic was using all surface streets and freeway structures, none of which required any type of shoring or immediate repair. Asphalt patches up to ±150 feet long had been placed at bridge approaches at some sites to eliminate bumps due to fill settlement. At several of these sites mud-jacking had been attempted but its effectiveness was not determined. There was no evidence of any other type of repair or remedial work.

A typical freeway undercrossing structure is shown in Photo No. 2. As may be noted, the entire fill end slope is enclosed by concrete curtain walls. Also, the concrete sidewalk at the surface street level extends full-width between curb and curtain wall. As these structures rotated in response to ground motion, forces were exerted through the sidewalks to the curbs causing shear cracks at numerous locations as illustrated in Photo Nos. 3 and 4.

At the San Fernando Mission Boulevard Undercrossing 3-1/2 inches of fill settlement occurred at the northeast wingwall, as shown in Photo No. 5. At this location, the evidence showed the structure tended to rotate counterclockwise.

Photo 2. Typical undercrossing structure on Route 405.



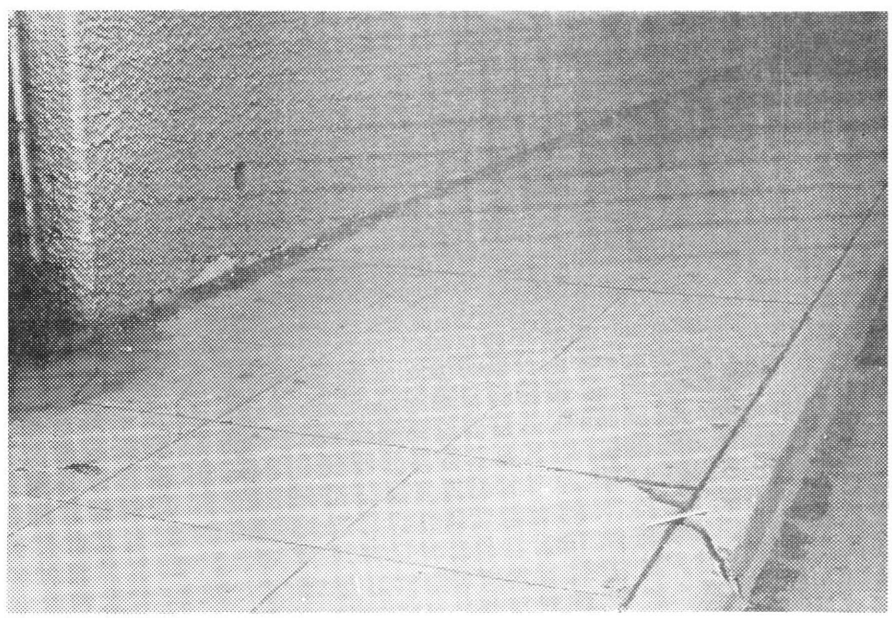


Photo 3. Typical curb crack and displacement caused by forces from curtain wall movement. Note cracks in sidewalk ajacent to curtain wall.

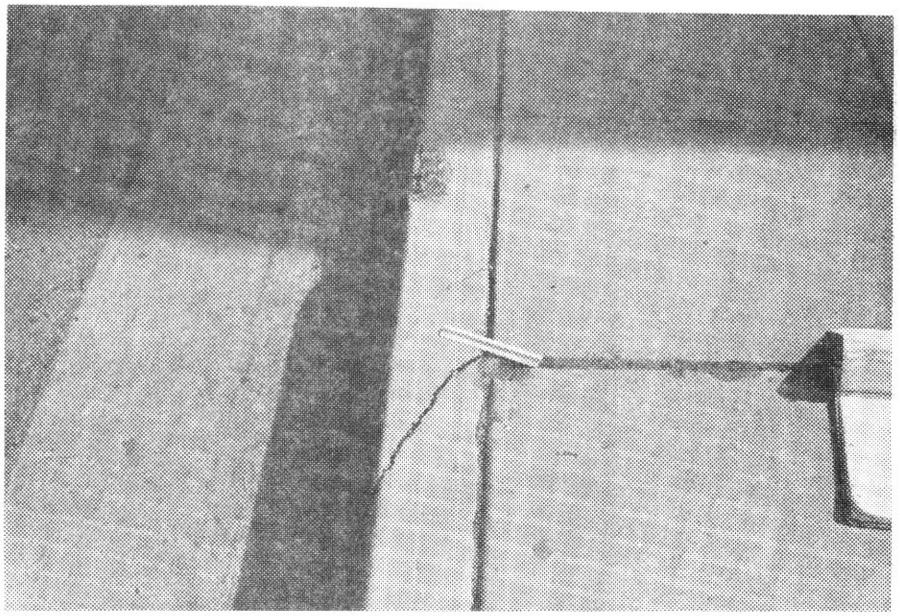


Photo 4. Typical curb crack and displacement caused by forces from curtain wall movement.

Photo 5. Looking westerly at northeast wingwall of San Fernando Mission Boulevard Undercrossing. Note fill settlement relative to wingwall.

It was interesting to observe that, south of Rinaldi Street, Route 405 fills clearly experienced more settlement than those of Route 5.

A definite progression of damage severity was noted on both Routes 5 and 405 north from Osborne Street and Roscoe Boulevard, respectively. South of San Fernando Mission Boulevard, damage was negligible and permanent displacements often were difficult to detect. The San Fernando Mission Boulevard structures were the first to show more than negligible (although still considered very minor) damage. Even more damage was noted at the Rinaldi structures; but it, too, was considered minor. In the Rinaldi area, damage to other structures became more apparent, the Holy Cross Hospital and the Indian Hills Medical Center, for example. It is of interest to note that the geological map of the San Fernando Quadrangle shows the Mission Hills Fault trending east-west and closely paralleling Rinaldi Avenue in this area (8).

ROUTE 5/405 SEPARATION AREA

The Route 5/405 Separation, shown in Figure 3, and Photo 6, is constructed in cut section immediately east of the lower San Fernando Reservoir. The terrain is composed of marine siltstone and mudstone (extreme right side of Photo 6) in contact with younger, nonmarine sandstones.

The following damage descriptions are keyed to location numbers on Figure 3.

Location 1

The most notable damage in the separation area was the collapse of the Route 5 southbound truck freeway bridge over Route 405 as shown in Photos 6, 7 and 8. A discussion of bridge damage may be found in Reference 5.

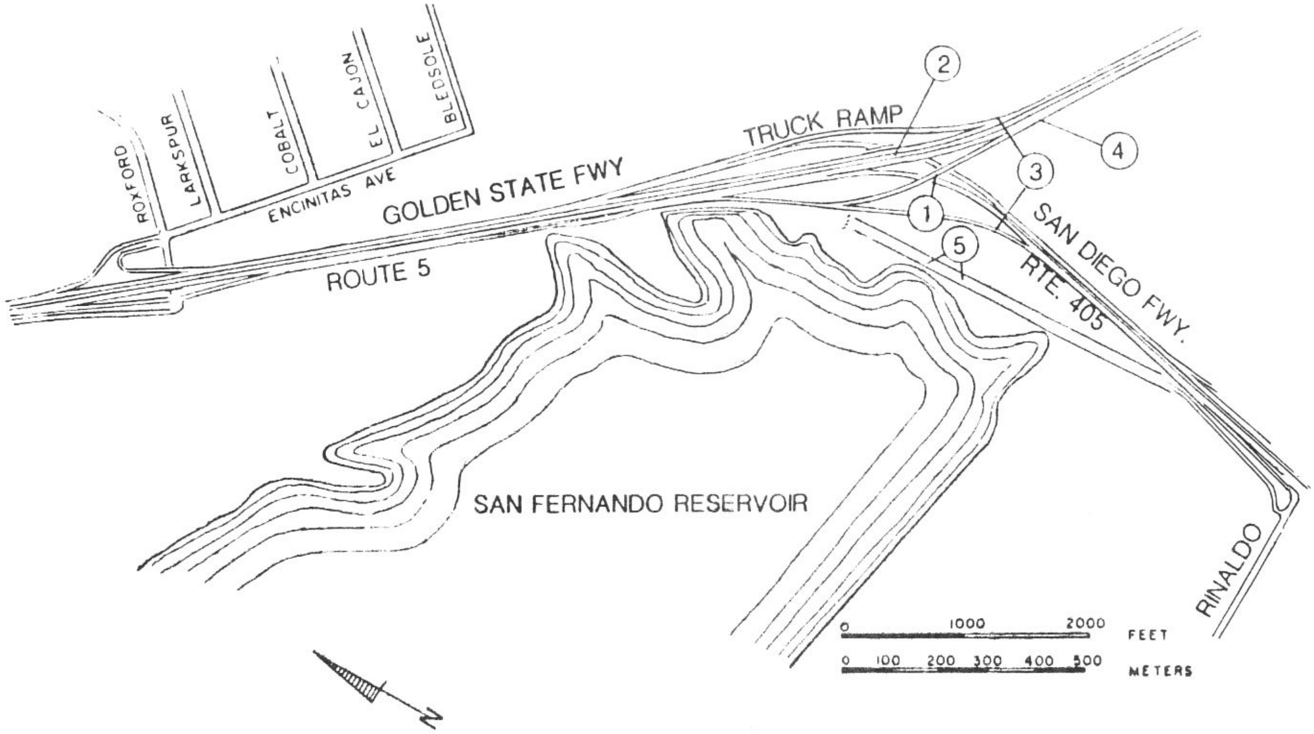
Location 2

At this location, northbound Route 405 traffic passes through a concrete box tunnel structure, shown in the left center of Photo 6. No visible disturbance of the soils surrounding the structure and the roadway within the structure was detected. Minor cracking in the walls and 1/4- to 1/2-inch separations in some expansion joints were observed. Also, a small amount of wingwall separation from the top deck at the southwest corner of the structure occurred. A report of the structure appears in Reference 5.

It is interesting to note that the tunnel structure suffered only very minor damage, yet 500 feet away a bridge was destroyed.

Figure 3

Layout of Route 5/405 Separation



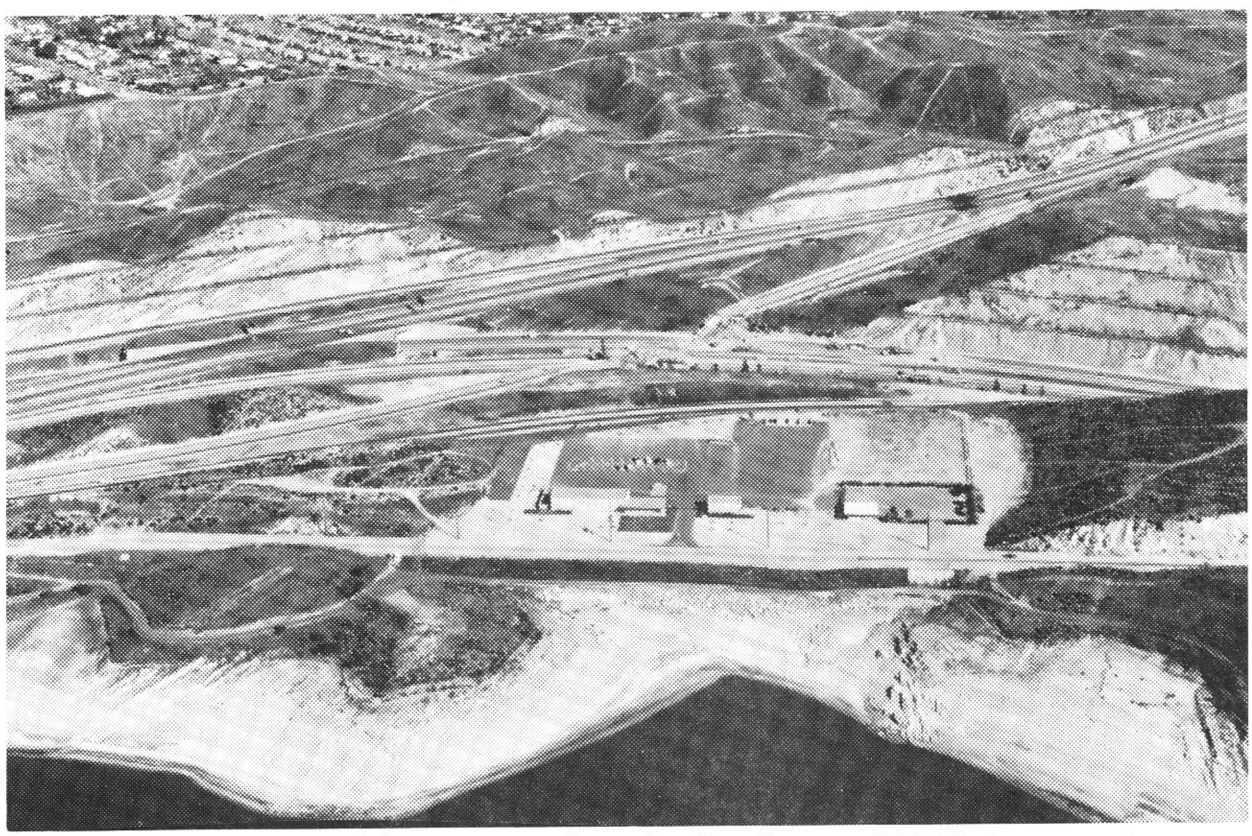


Photo 6. Looking easterly at the Route 5/405 Separation Area.

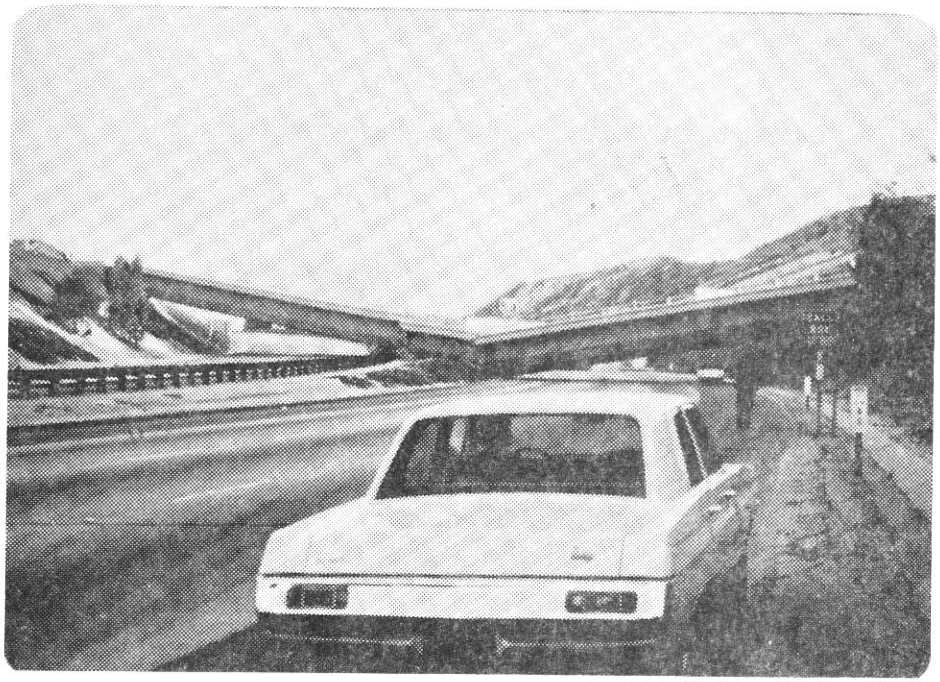


Photo 7. Looking northerly at the collapsed Route 5 southbound truck freeway bridge over Route 405.

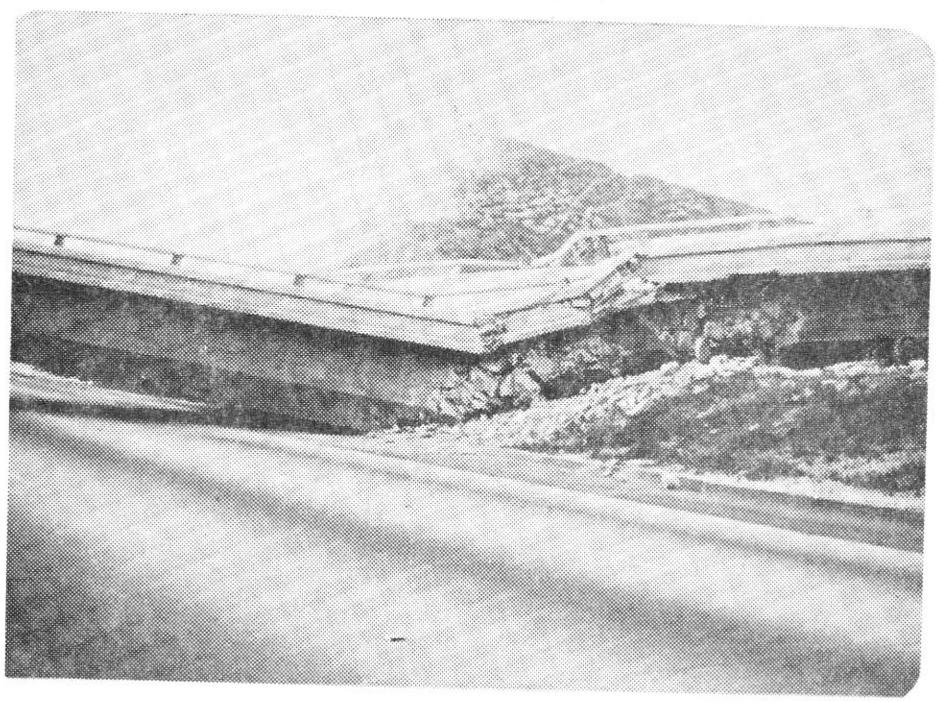


Photo 8. The center portion of the collapsed Route 5 southbound truck freeway bridge.

Location 3

Faulting occurred at this location in an east-west trend through the dark patch across Route 5 shown in Photo 6, upper right. The faulting is shown in more detail in Photos 9, 10, and 11. Two feet of reverse fault displacement, north over south, occurred. The writers believe the faulting continued in a westerly direction across Route 405 in a zone several hundred feet wide (see Ref. 2, p. 19, Item 6).

Undulations of the pavement, either visible or detectable when riding over the zone, were noted. Photo 12 is a southerly view of Route 405 across this zone. Immediately to the west of the photo a one-half foot rise in the ground was noted, northside up, but an actual break could not be found. A detail of the pavement buckle in the center of the photo is shown in Photo 13.

Although the above faulting may be localized, it is interesting to note that on a regional map (Reference 5) it appears to be an extension of the Sylmar Fault break of 1971.

Location 4

An earthquake-induced slump failure occurred at this location in a cut slope, shown in the center of Photo 14. The failure was confined entirely to the upper three-fourths of the 100-foot high cut.

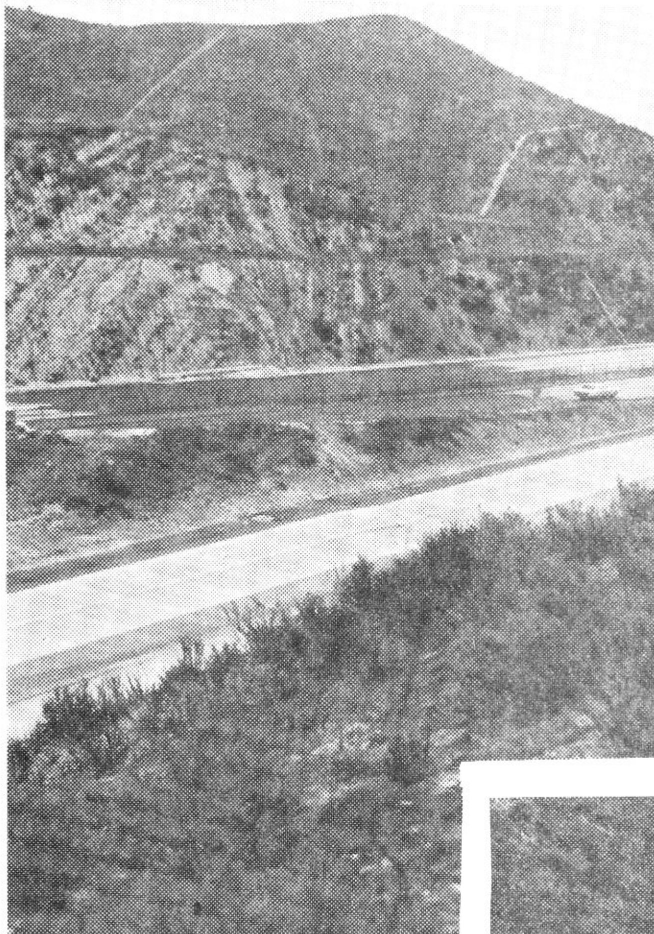


Photo 9. Looking easterly along discontinuous fault trace. Route 5 (patched) is in background, Route 5 southbound truck lane in center. Note break and disrupted profile in truck lane pavement.

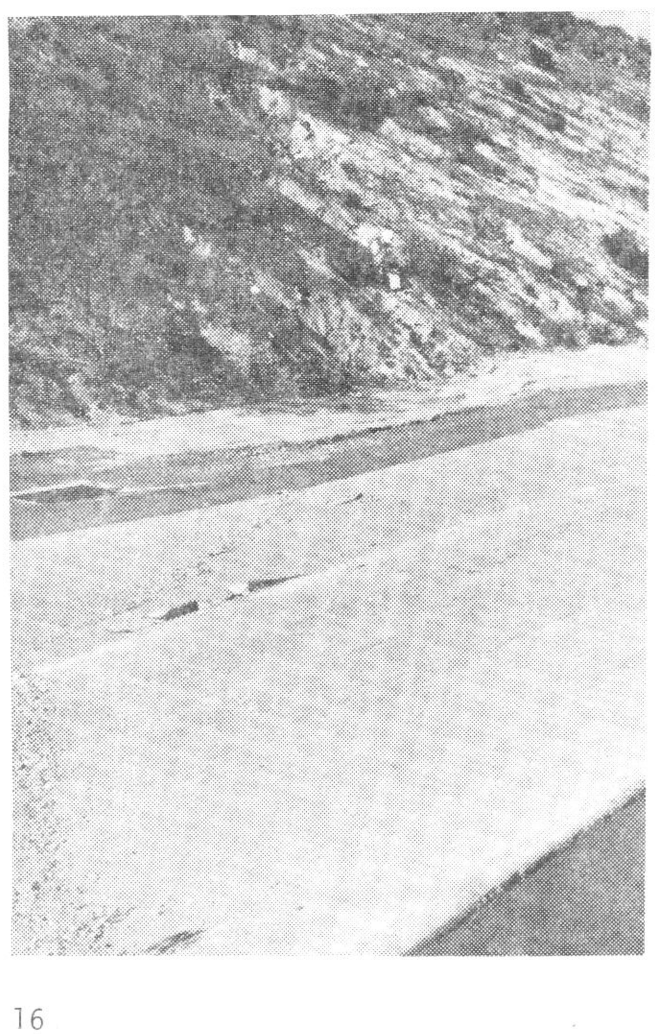


Photo 10. Looking westerly across truck lane pavement. Fault trace extended up cut slope in background.

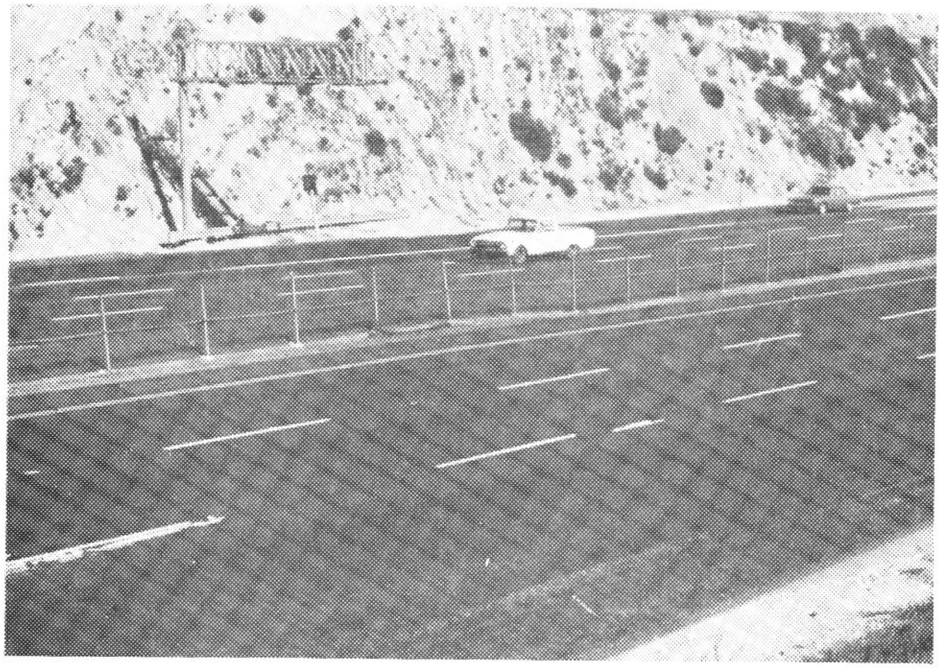


Photo 11. Looking easterly across the patched area of Route 5. A break in profile of the median fence may be noted directly in line with the vehicle near the center of photo.

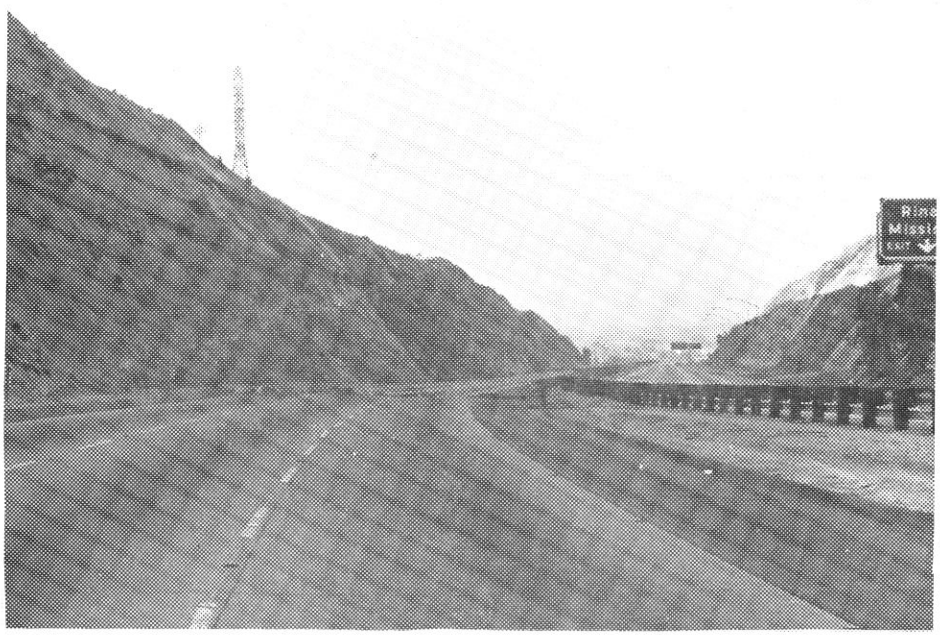


Photo 12. Looking southerly at northbound lanes of Route 405. Fault zone is believed to extend across the roadway in the area of broken pavement.

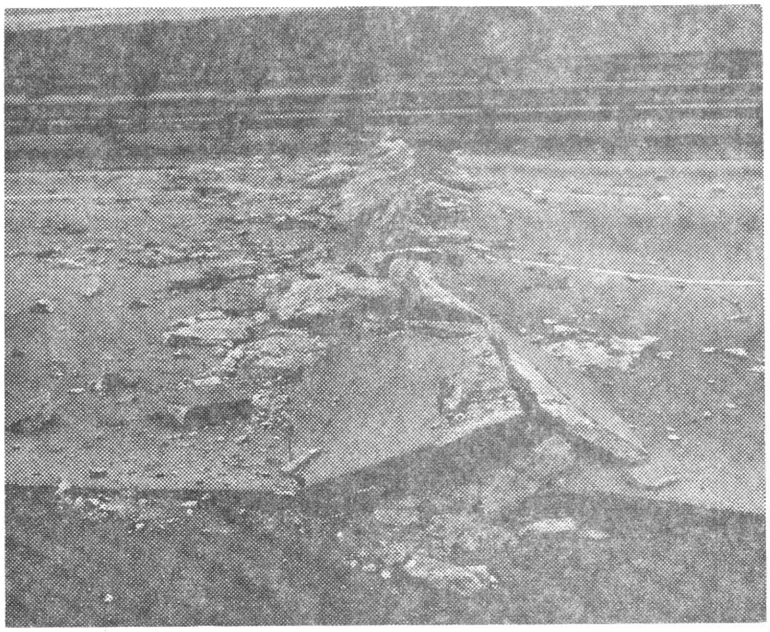


Photo 13. Looking westerly across northbound lanes of Route 405. Pavement break is on a general projection of nearby fault trace mapped by authors.

Photo 14. Looking westerly across Route 5. Note slide in cut slope.

Location 5

At this location, a Blucher Avenue fill slid toward the reservoir, as shown in the lower right center of Photo 6. Lateral movement of the fill and pavement damage are shown in Photo 15. Failure of the fill was possibly due to liquefaction of the saturated foundation material, although no supporting evidence to that effect was found.

Paved grounds of the Caltrans' maintenance yard immediately east of the slide suffered various amounts of pavement breakage. An example is shown in Photo 16 which illustrates a one-foot thrust of pavement, north over south. Another compressional feature was noted across Blucher Avenue, south of the slide where the roadway enters a cut section (Photo 6); however, details of the feature were not recorded.

Photo 15. Looking southerly along Blucher Avenue at results of fill movement to the west.

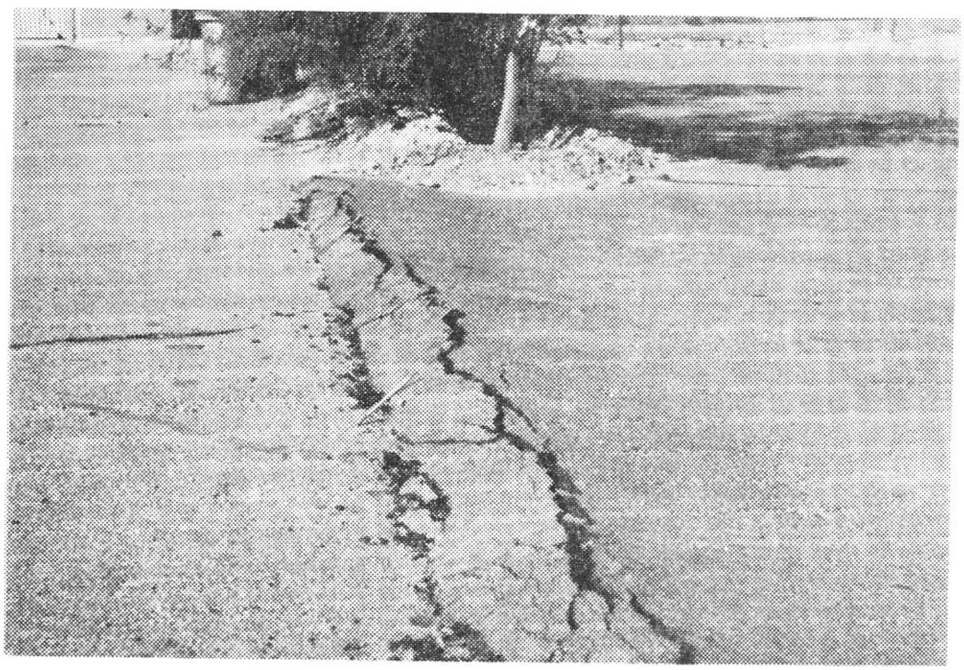


Photo 16. Looking westerly at maintenance station parking area. Note pavement breaks and thrusting of north over south.

ROUTE 5, BETWEEN THE ROUTE 5/405 SEPARATION AND ROUTE 5/210 INTERCHANGE

This 1.8 mile section of Route 5 passes through low-lying hills made up of nonmarine sediments and crosses canyons that contain shallow alluvial deposits. Groundwater is very near the surface of canyon bottoms at the southerly end of the section. Roadway and original ground profiles are shown in Figure 4.

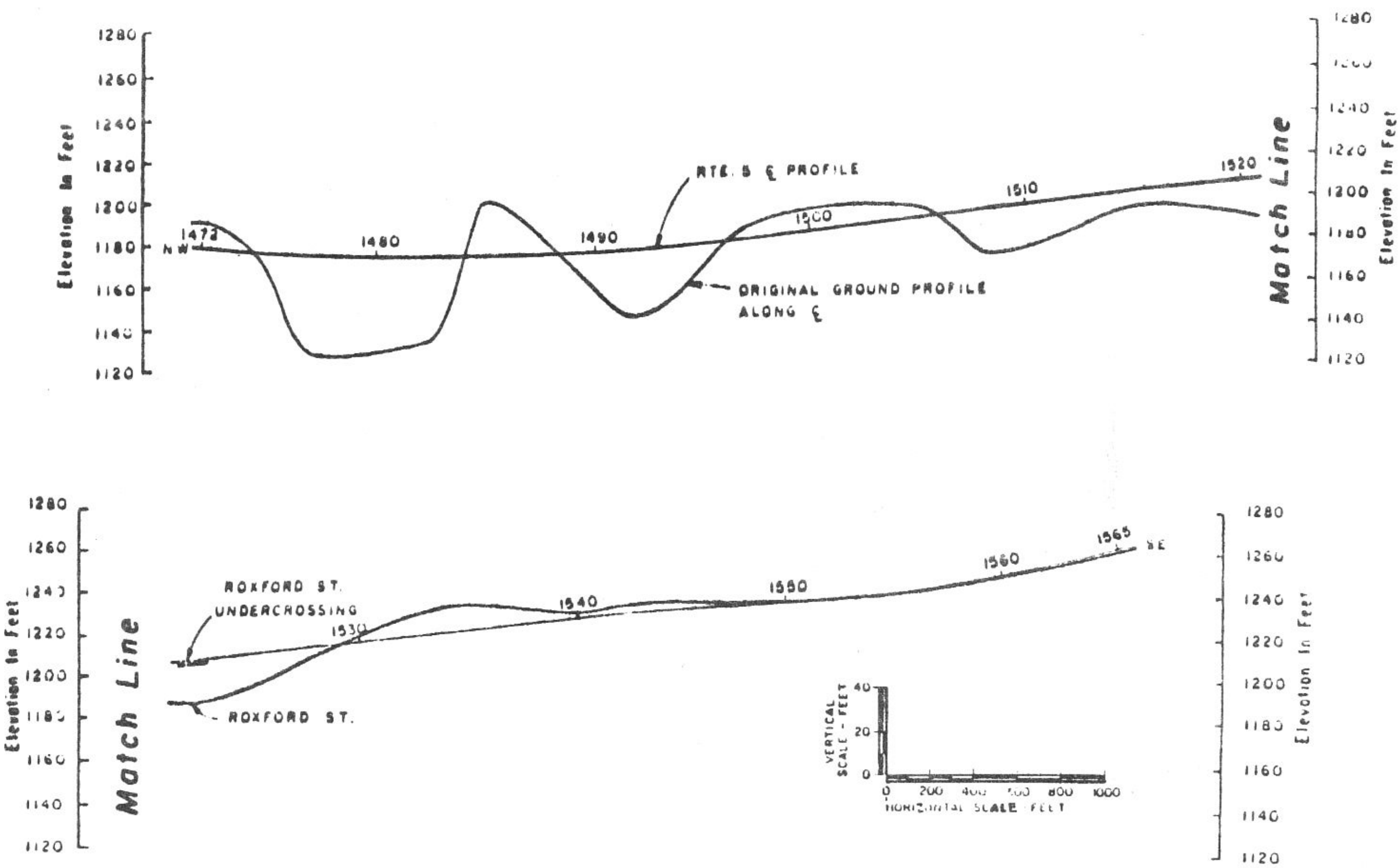


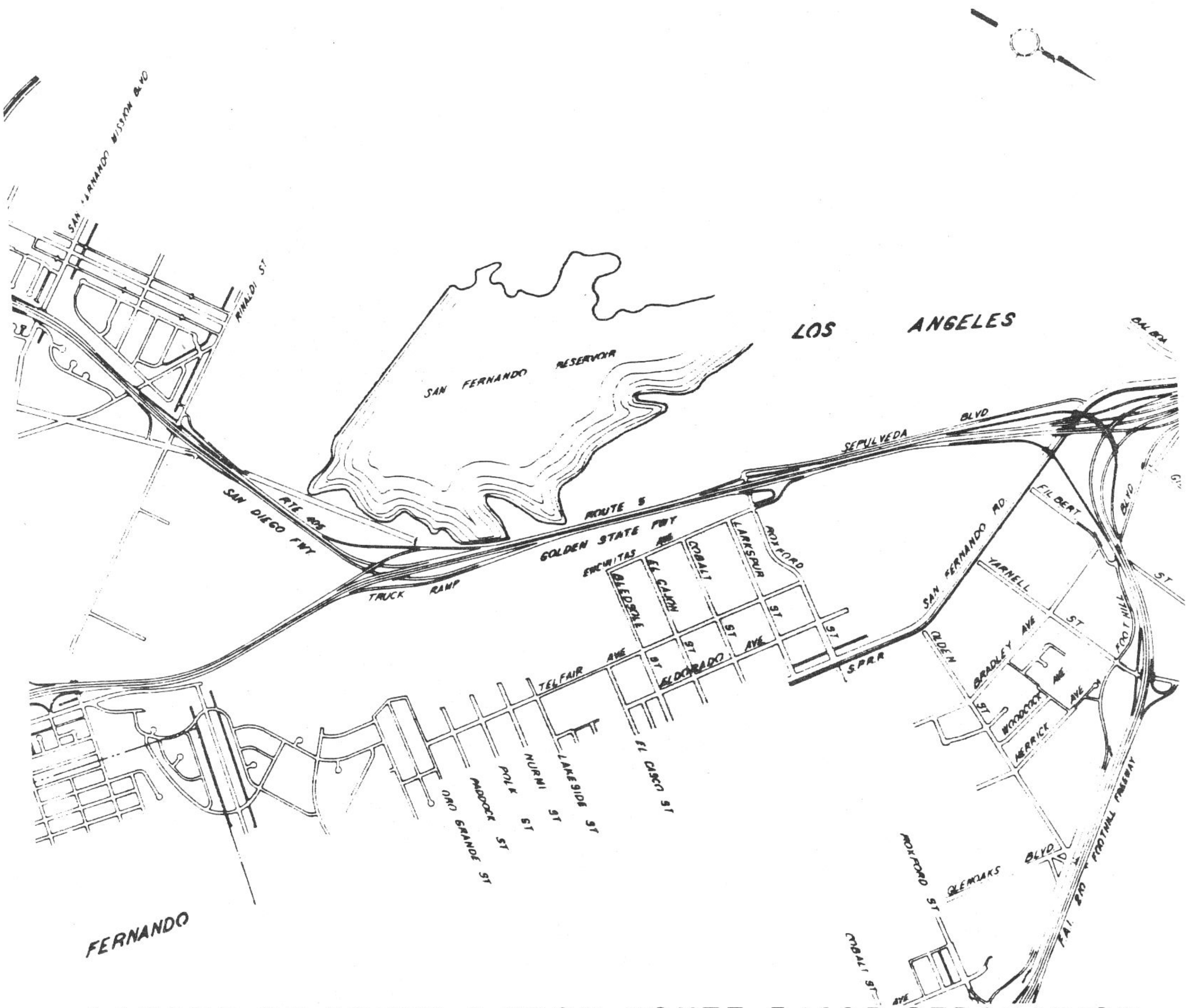
Figure 4
Original ground and Route 5 centerline profiles from Route 5/405 separation to Route 5/210 Interchange.

The following damage location numbers are shown on Figure 5.

Location 1

Immediately north of the 5/405 separation, the roadway crosses a canyon on about 45 feet of fill as shown in Photo 17. The embankment, founded on saturated, relatively weak fine-grained alluvial deposits, experienced vertical and lateral movement due to the foundation soil behavior. Cracking in slopes on both sides of the fill (Photos 18 and 19) suggest that much larger movements typical of foundation failures would have occurred had strong motion been of a longer duration. Vertical movement resulted in pavement slab displacements as shown in Photos 20 and 21.

Concrete pavement slabs were thrust together near the south cut-fill contact as shown in Photos 22 and 23, as well as being separated along the same contact shown in Photo 21. Longitudinal separation occurred between pavement slabs due to fill lateral movement as shown in Photo 24.



LAYOUT OF ROUTE 5 FROM ROUTE 5/405 SEPARATION TO ROUTE 5/210 INTERCHANGE

FIGURE 5

Photo 17. Looking northeasterly at Route 5. Note roadway embankment and pavement patches in left center of photo.

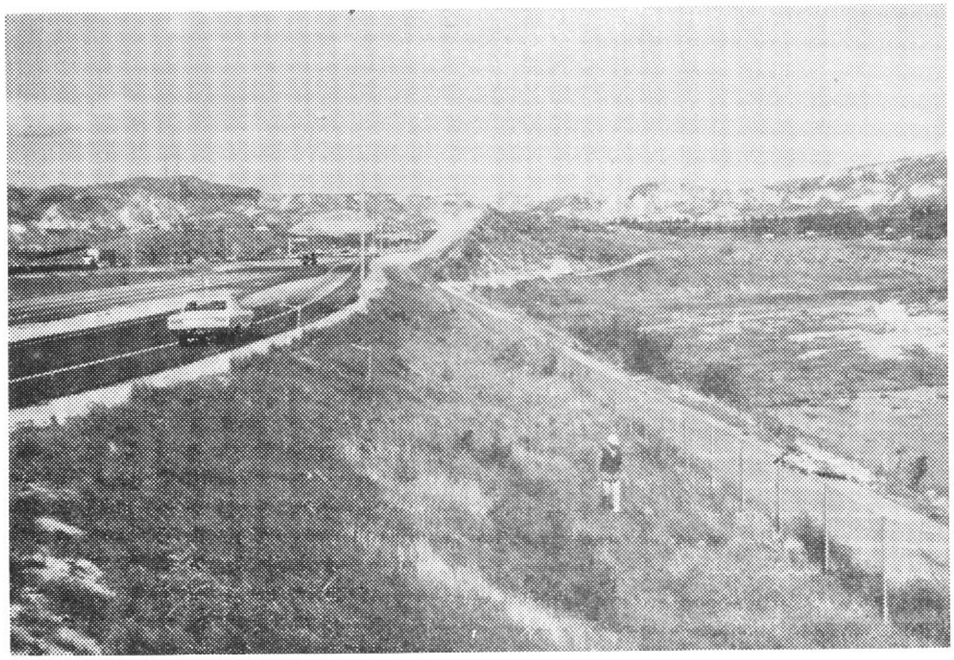


Photo 18. Looking northerly along easterly slope of Route 5 embankment.

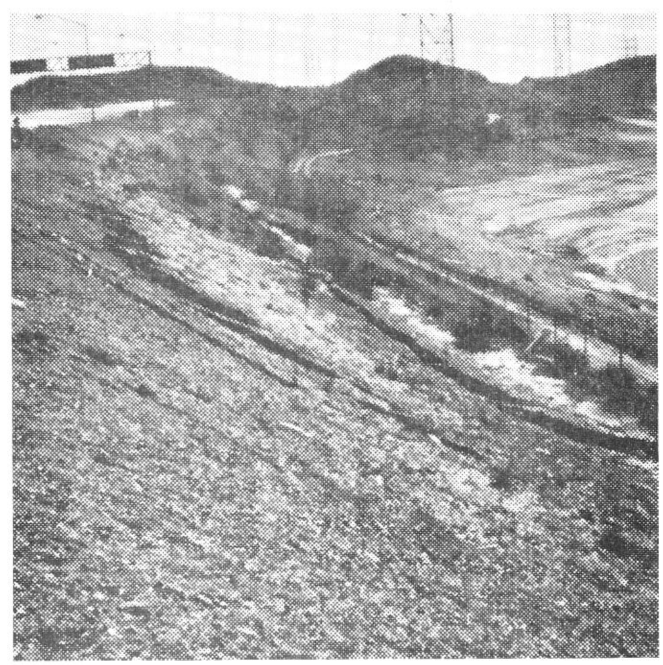


Photo 19. Looking south at westerly slope of embankment. Note cracks and lateral movement in slope.

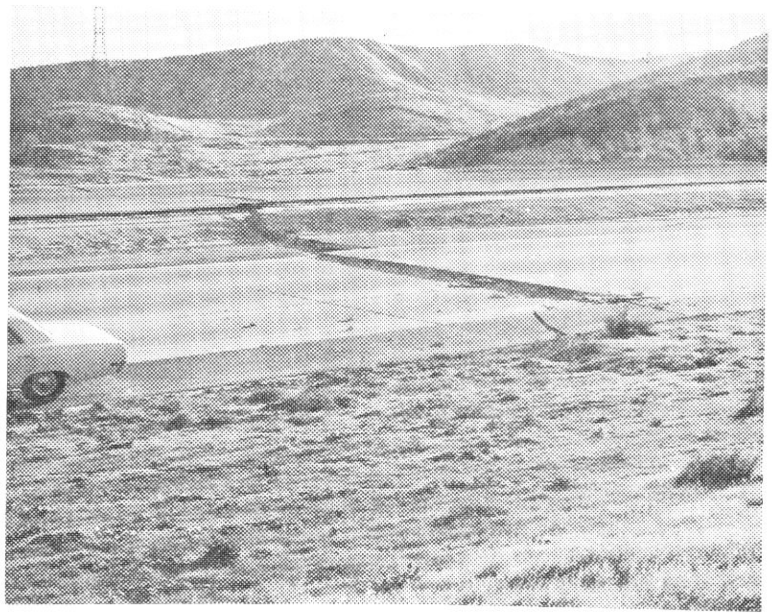


Photo 20. Vertical displacement along pavement slab joints near fill/cut contacts.

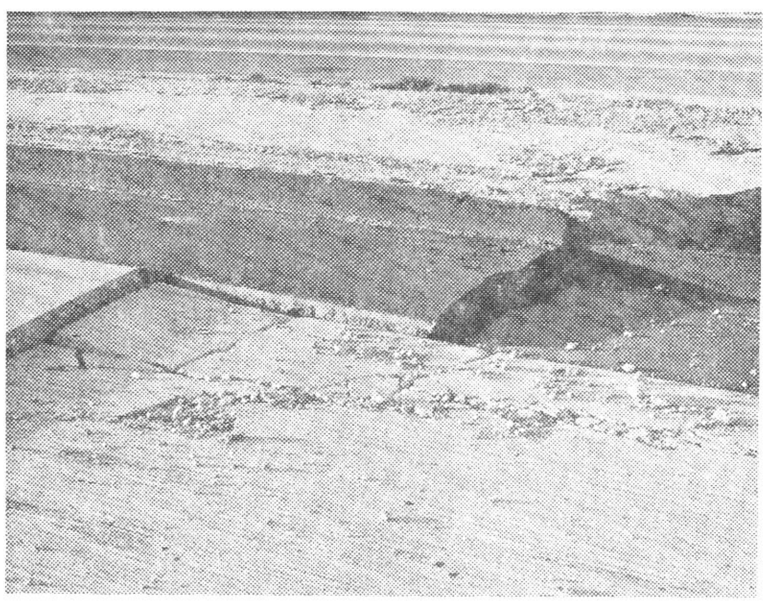


Photo 21. Vertical displacement along pavement slab joints near fill/cut contacts.

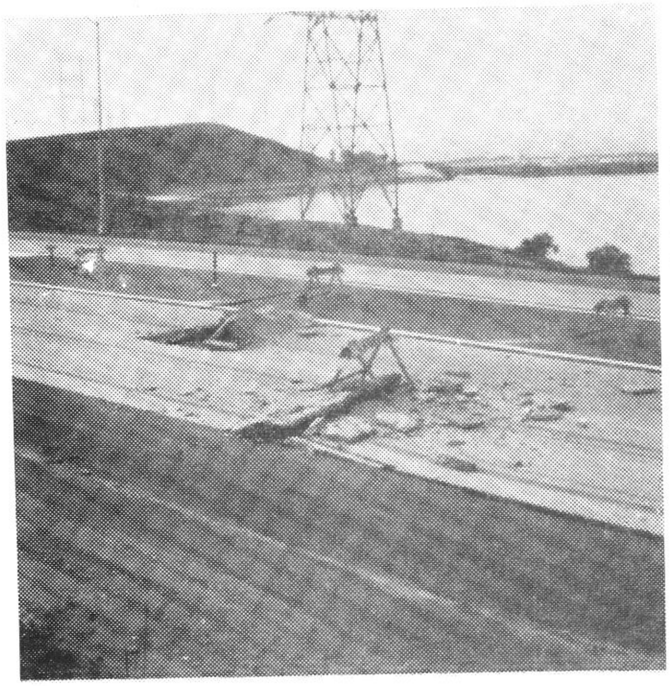
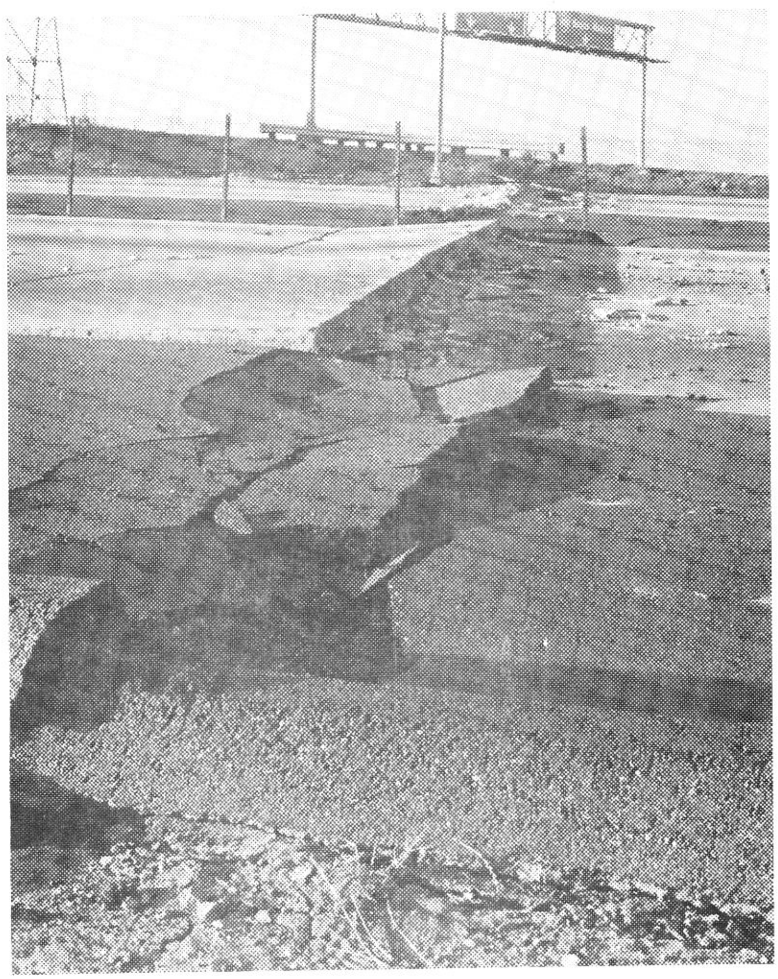


Photo 22 and 23. Breaking and overriding of pavement slabs at transverse joints due to longitudinal thrusting action.



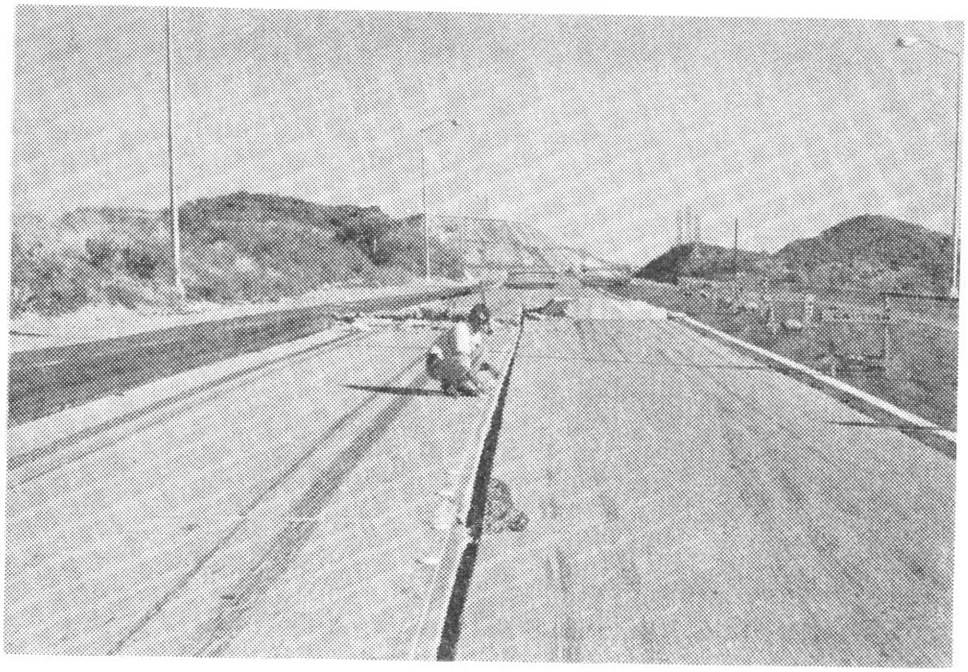


Photo 24. Slab separation along longitudinal joint due to lateral fill movement.

Location 2

The top of the cut slope (Photo 25) was distressed by slope-shattering and transverse cracks, shown in detail in Photo 26.

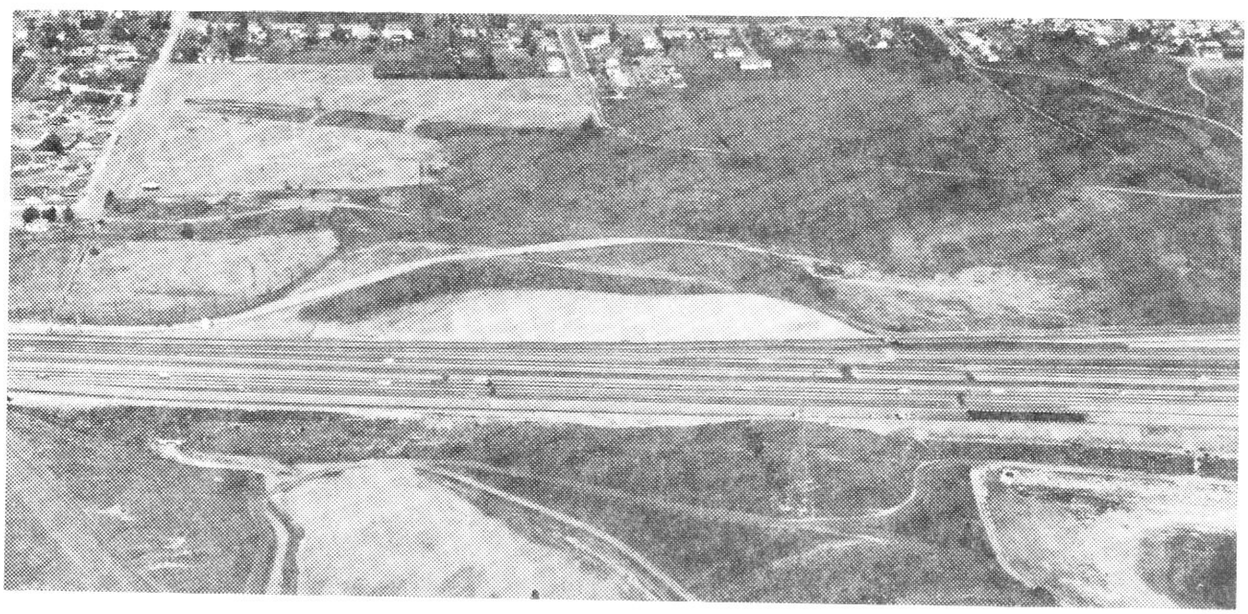


Photo 25. Looking northeasterly at Route 5. Cut slope in top center sustained cracking. Note patches in pavement.

Photo 26. Typical cracks in cut slope shown in Photo 25.

Location 3

Just north of Location 2, a 30-foot high embankment underwent enough movement to cause cracking in both slopes, and settlement near the cut-fill contacts. The riding surface across this embankment required repairs in the south-bound lanes at the south cut-fill contact. A 12-foot steel plate drainage structure through the fill was distorted at the inlet (Photo 27), and outlet (Photo 28), but remained structurally intact.

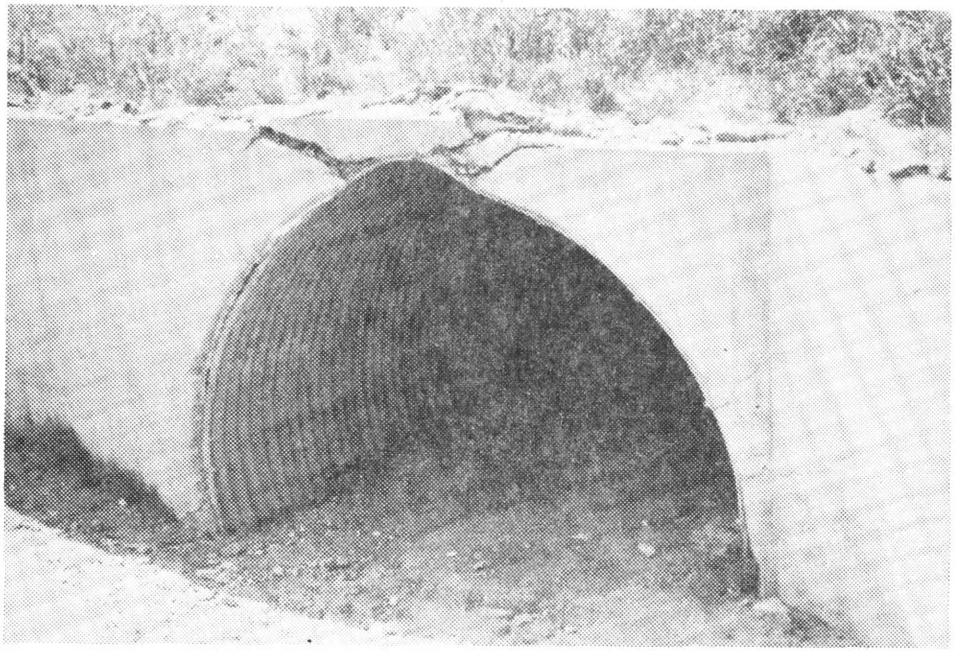


Photo 27. Cracking and distortion in drainage structure headwall.

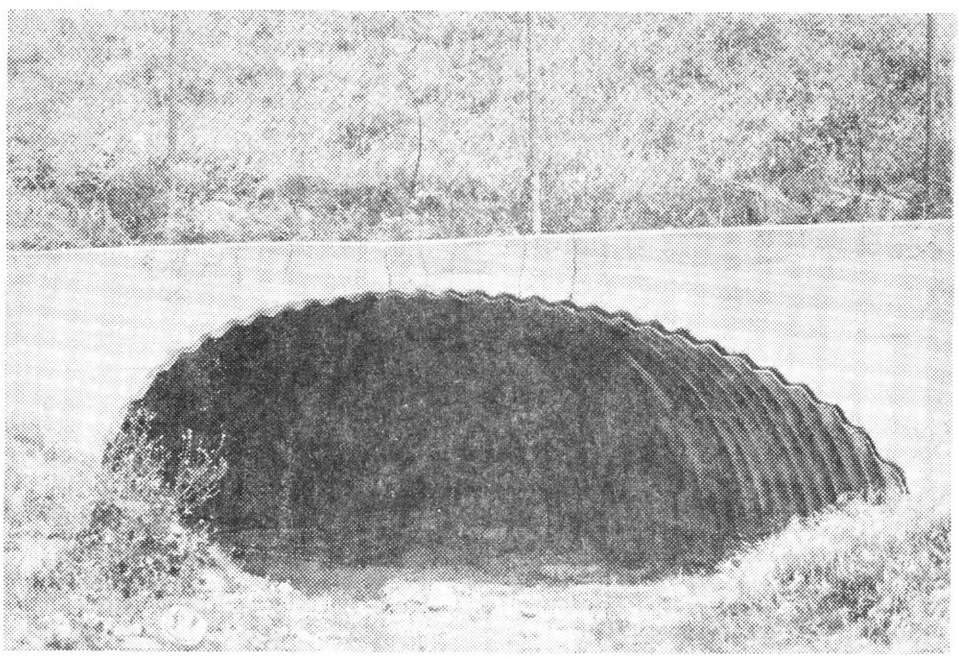


Photo 28. Cracking in drainage structure endwall. Note separation of concrete and metal at top of structure.

Location 4

Rotational movement occurred in the cut slope on the west side of the freeway shown in Photo 29. The movement, although small, was sufficient to develop cracks behind the top of cut (Photos 30 and 31) and to upthrust the south-bound roadway surface and cause slab separation, which required patching. Four hundred feet to the south, another upthrust bulge occurred in the roadway but it did not require immediate repairs. It was not obvious upon first observation that slope movements of this nature were the cause of the roadway riding surface damage.

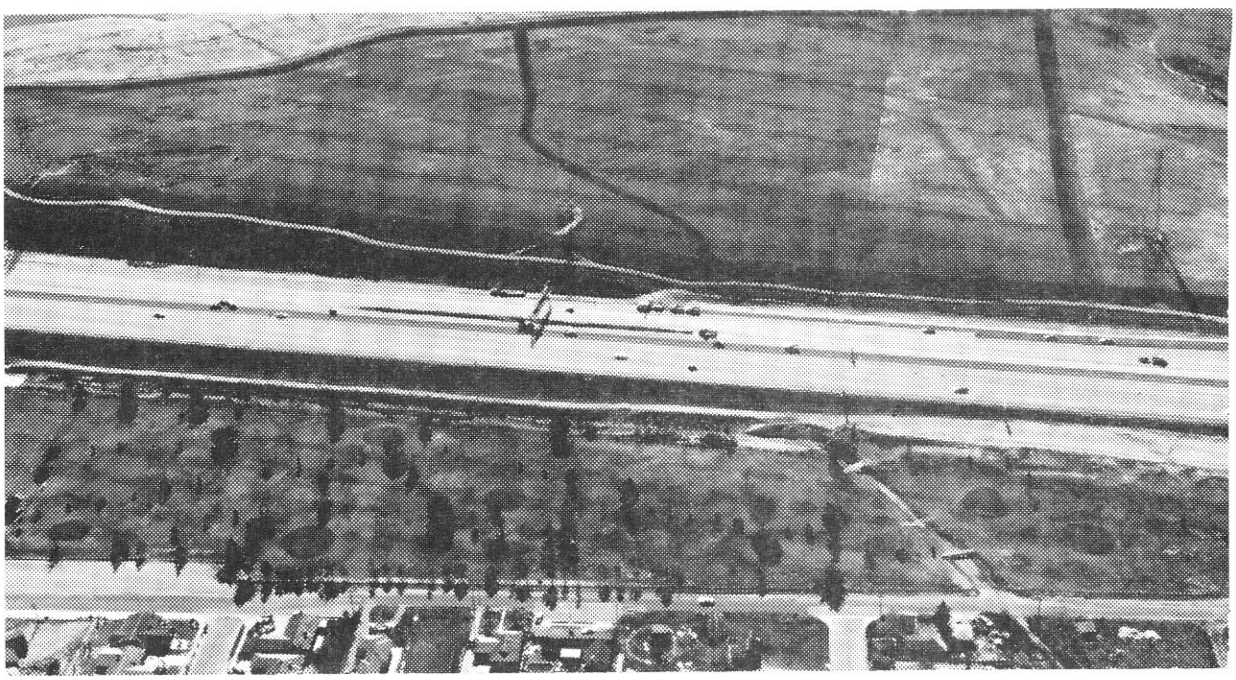


Photo 29. Looking westerly at Route 5.

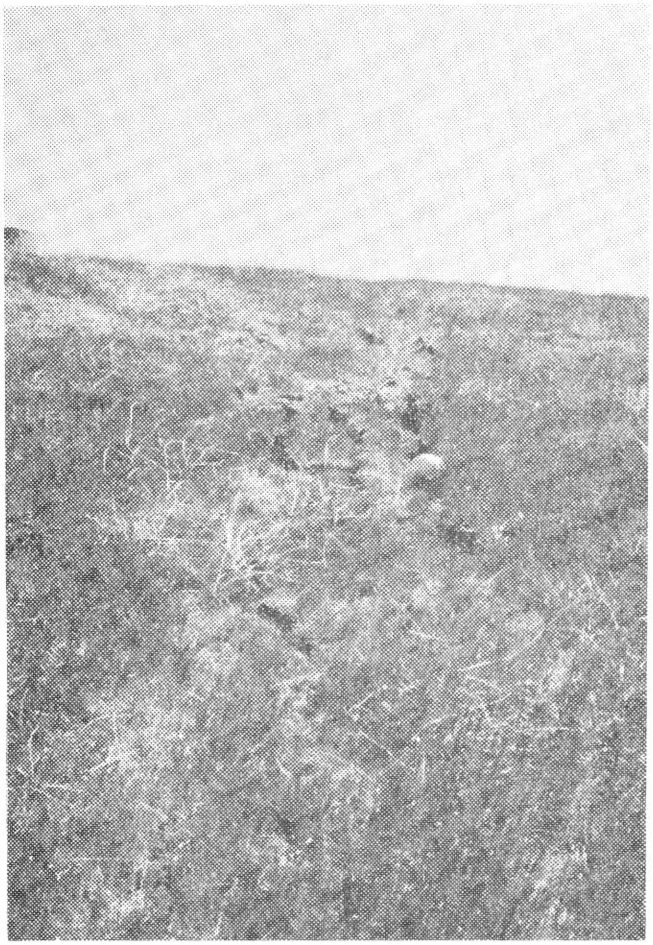


Photo 30. Cracking behind top of cut slope.

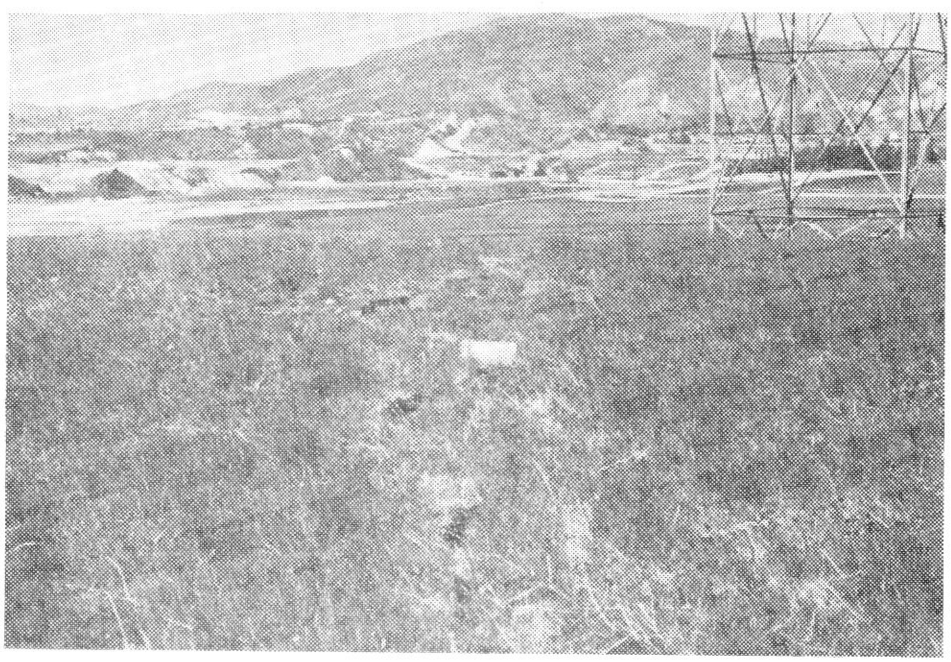


Photo 31. Cracking behind top of cut slope.

Location 5

Between the cut described for Location 4 and Roxford Street, Route 5 is carried on fill. An 84-inch concrete drainage pipe was located under most of the fill length, beginning at Roxford Street and outletting near the cut of Location 4. The outlet may be seen in Photo 29, right of center, at the bend in the open channel. This pipe was cracked badly from the headwall on the east side of the fill for a distance of about 600 feet to the south and west. Longitudinal cracking was practically continuous, mainly between the 8 and 11 o'clock and 2 and 4 o'clock positions when viewed looking westerly. Cracks up to one inch wide with lateral offsets up to one inch were noted. Also, peripheral cracks at joints had opened about 1/4-inch at several locations. The longitudinal crack pattern is shown in Photo 32.

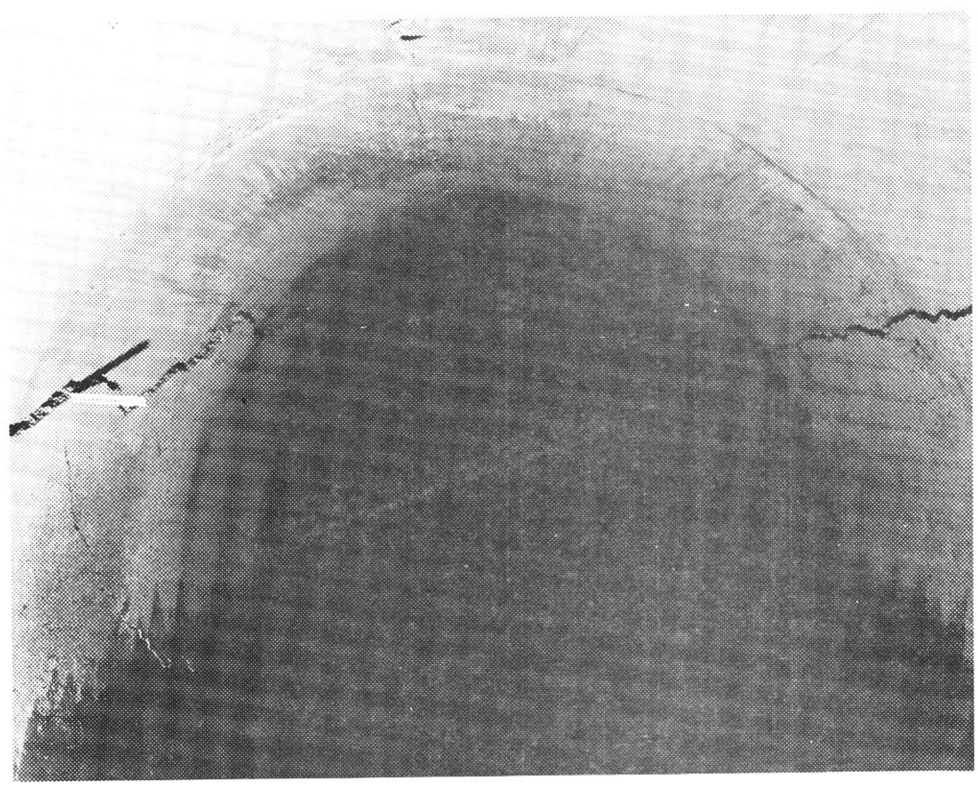


Photo 32. Typical longitudinal cracks in 84-inch concrete pipe.

Location 6

At the time of the earthquake a bridge widening project was under construction at the Roxford Street Interchange, shown in Photo 33. In addition to structural damage shown in Photos 34 and 35, settlement occurred in approach fills and structure backfill shown in Photo 36.

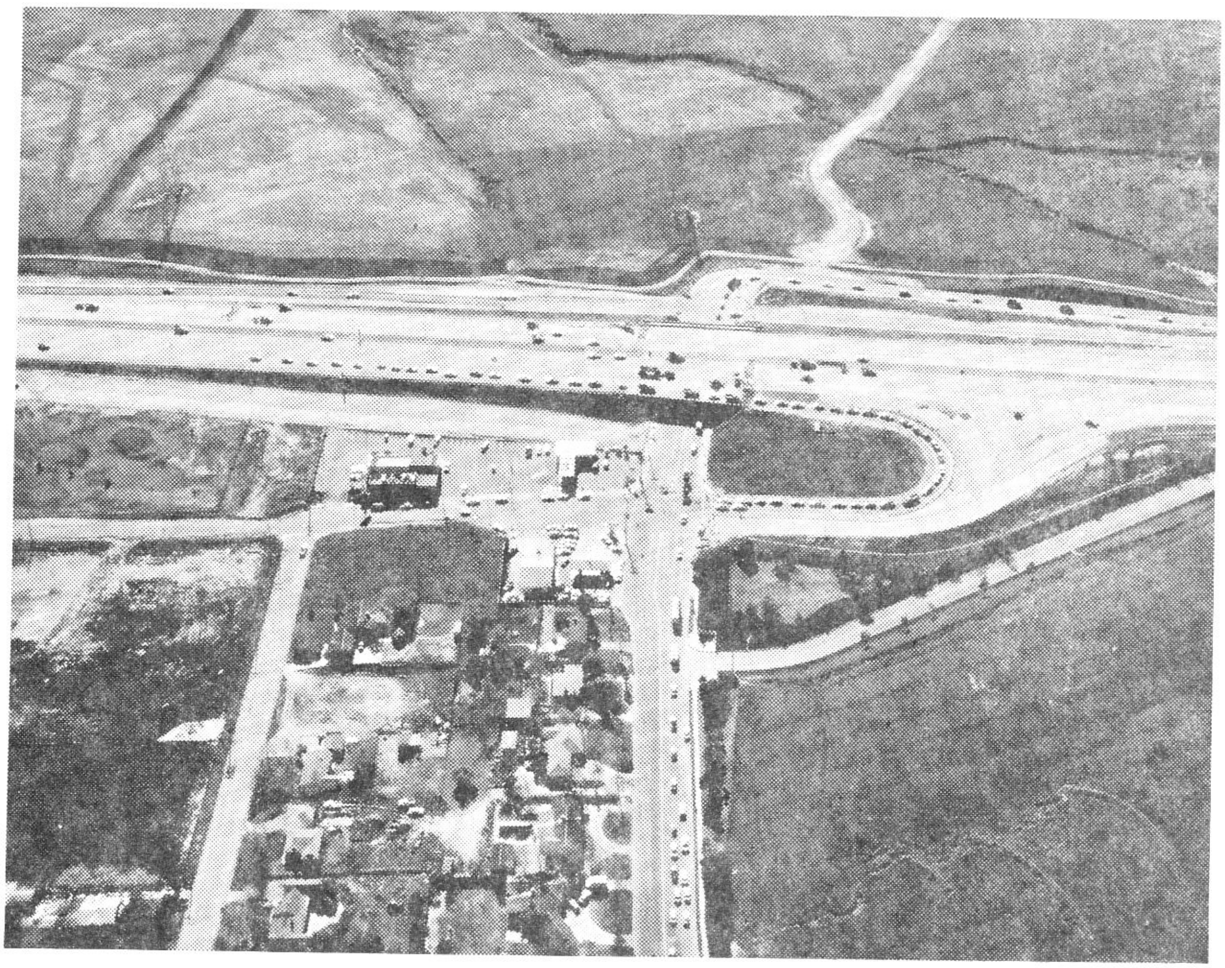
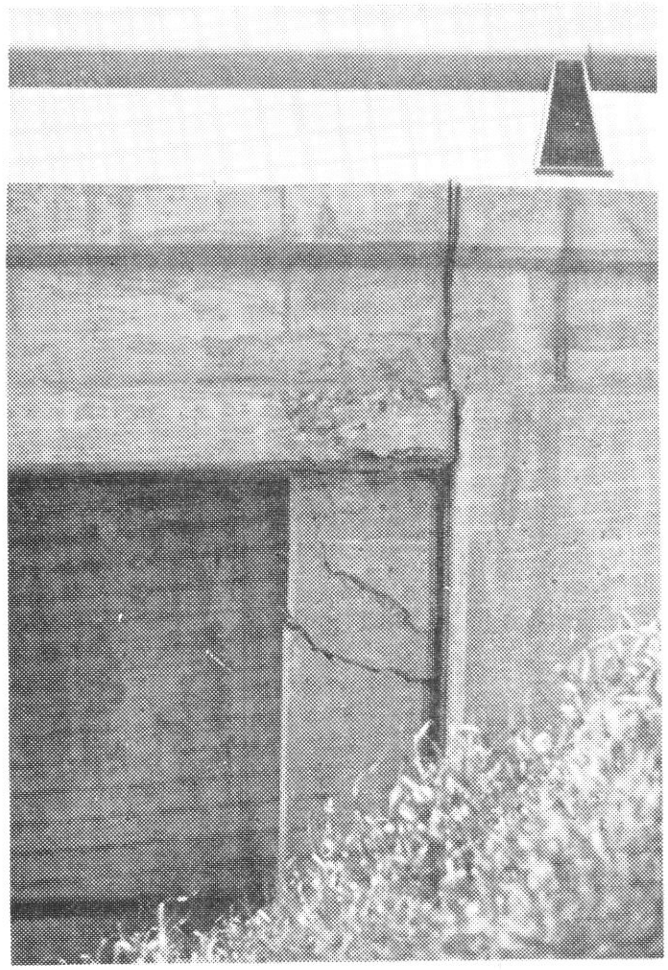


Photo 33. Looking westerly at the Route 5/Roxford Street Interchange.

Photo 34. Separation and settlement of wing-wall, cracking of abutment diaphram, Roxford Street Undercrossing.



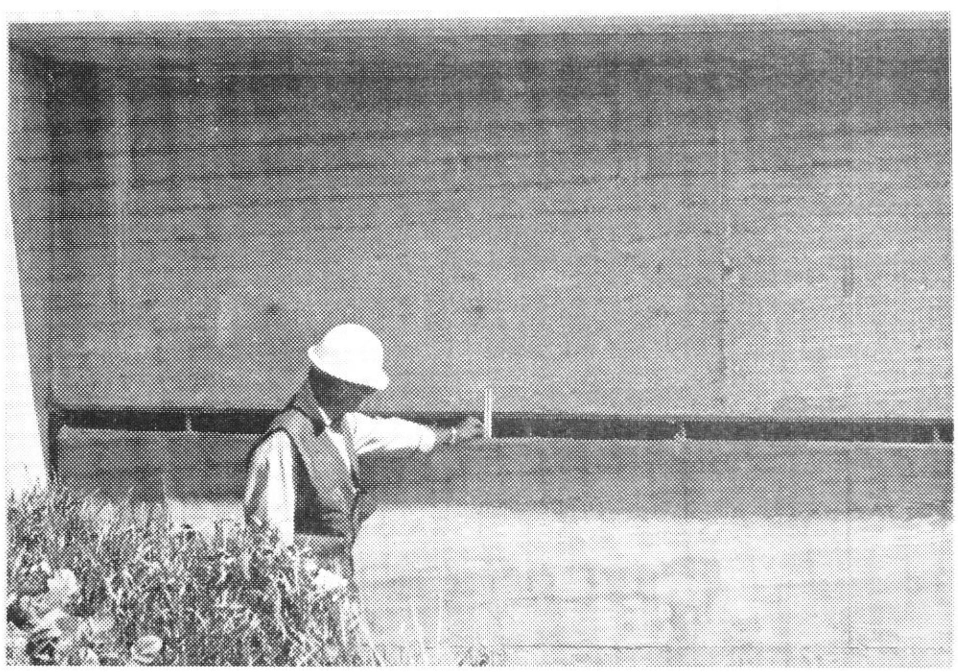


Photo 35. Settlement of concrete curtain wall from bridge girder. Note exposed rebars along separation.

Photo 36. Abutment backfill settlement, north end of Roxford Street Undercrossing.

Location 7

Roadway settlement of a few inches occurred at the cut to fill contact shown in Photo 37, right of center. A low fill crosses the drainage-way that follows the alluvium contact with a slight hill made up of bedded sediments. From this location, the roadway is essentially constructed along the natural ground profile for a 1/4-mile distance, northward.

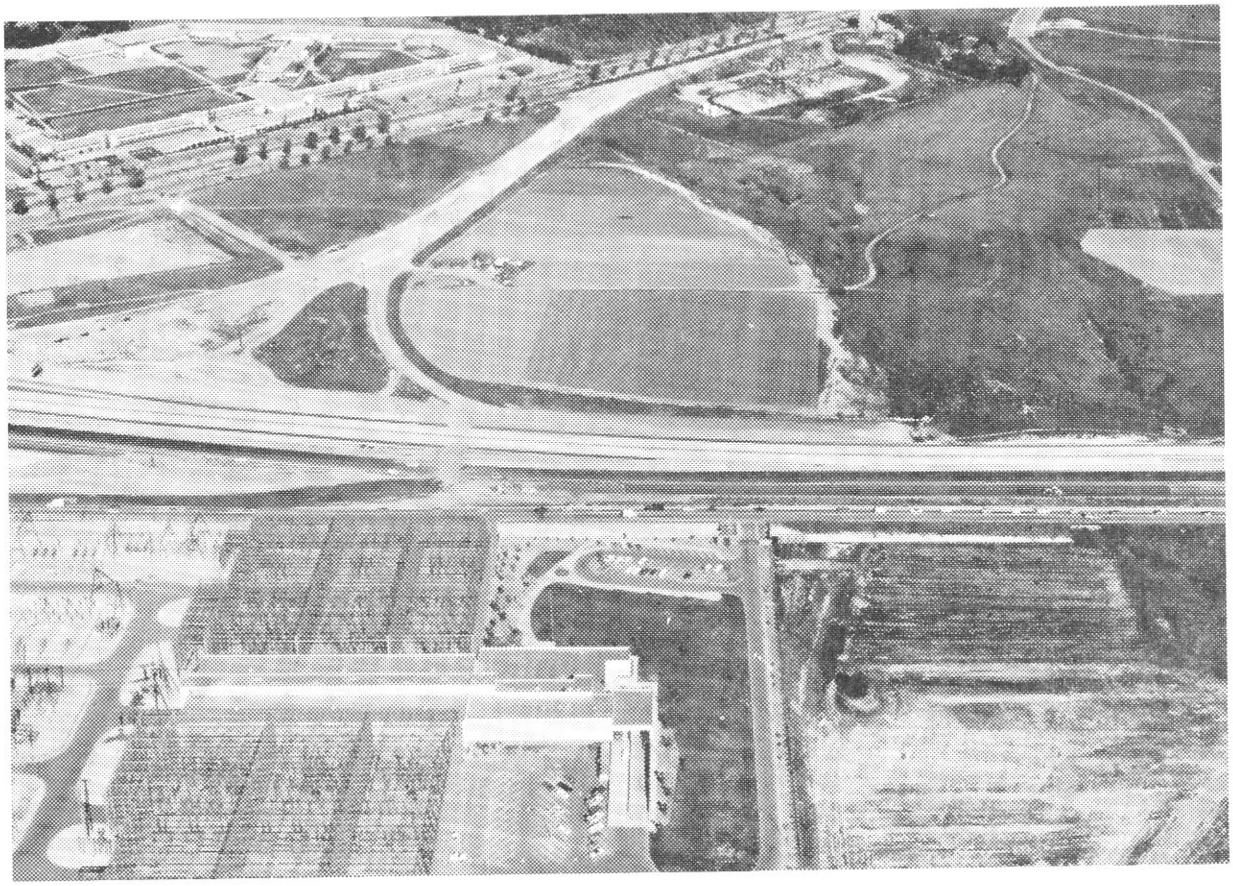


Photo 37. Looking easterly at Route 5. The Sylmar Converter Station is shown in lower left. Juvenile Hall complex is shown in middle left.

Location 8

The Juvenile Hall Landslide (Photo 37) displaced Route 5 in excess of 5 feet horizontally, transverse to the roadway (Photos 38 and 39). A compression buckle in the concrete pavement also occurred, as shown in Photo 39.

Lateral movement of the roadway along the northern flank of the slide resulted in separation, rotation, and differential heaving of pavement slabs as illustrated in Photo 40. Photos 41-43 show other features of the slide, detailed analyses of which may be found in References 7 and 9.

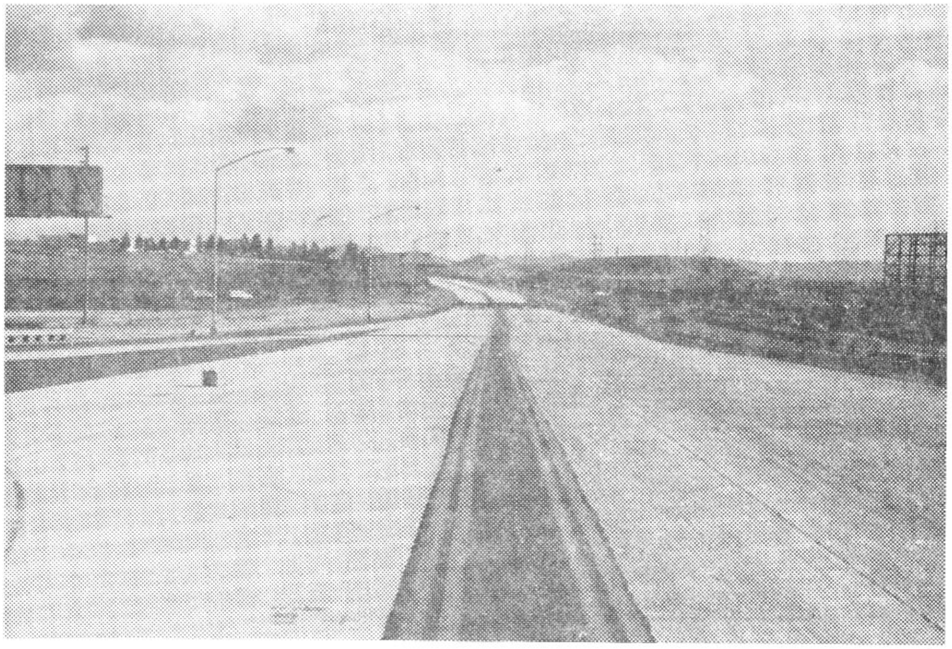


Photo 38. Looking south along Route 5. Note displacement of striping to right (west) caused by Juvenile Hall slide.

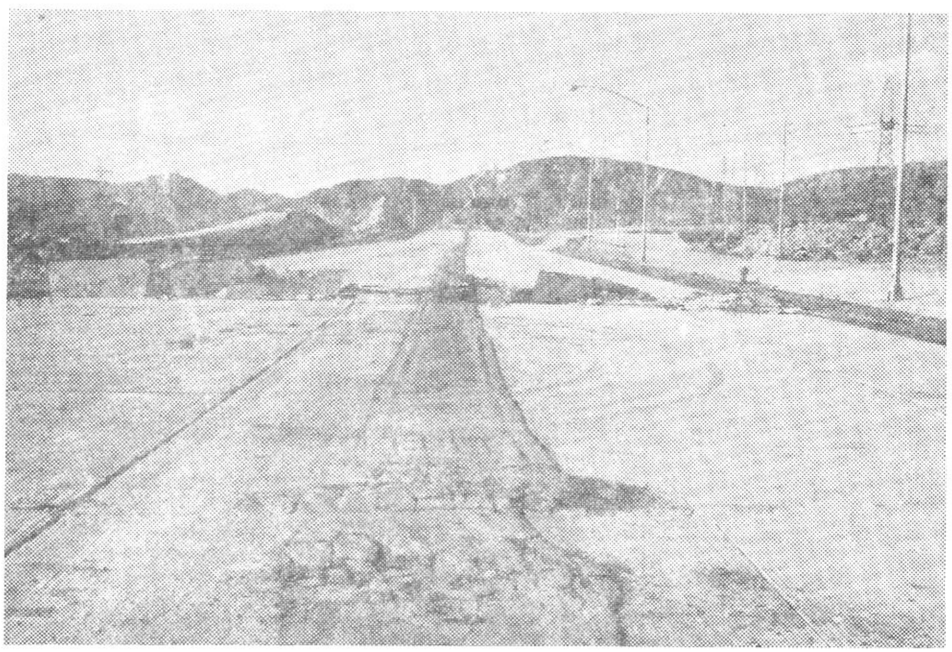


Photo 39. Looking north along Route 5. Note relative lateral slab displacement sharply delineated by striping. Northerly limit of slide displacement is shown by striping in far background. Note pavement buckle caused by extreme thrusting of slabs.

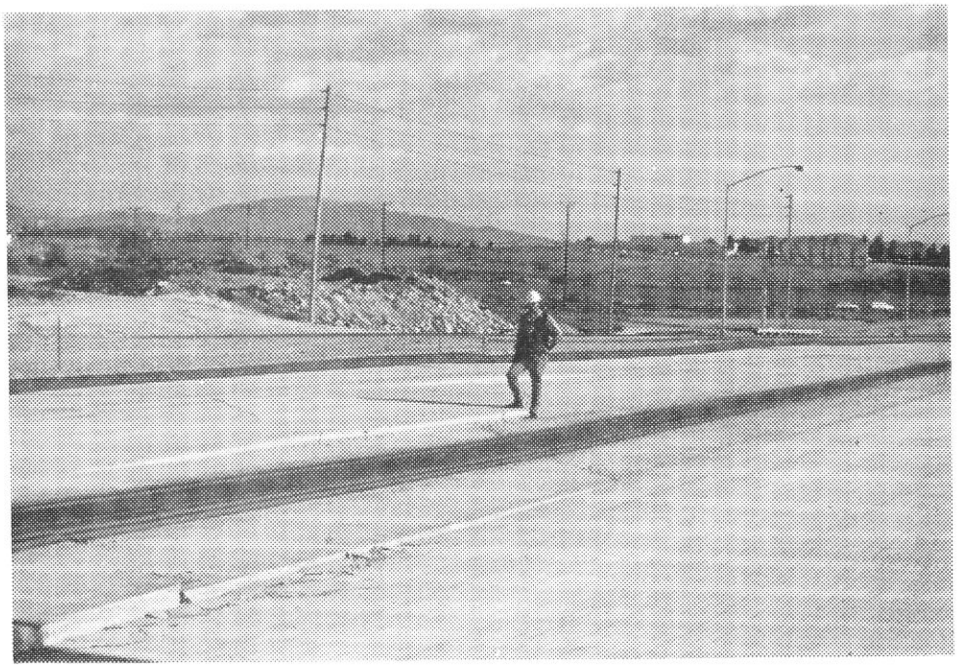


Photo 40. Looking southeasterly at Route 5. Northern flank of Juvenile Hall slide crossed pavement here, resulting in pavement slab movement.

Photo 41. Looking north along concrete lined drainage channel between Route 5 (right) and the Sylmar Converter Station (left). Lateral movement of Juvenile Hall slide (right to left) was largely absorbed by the channel.

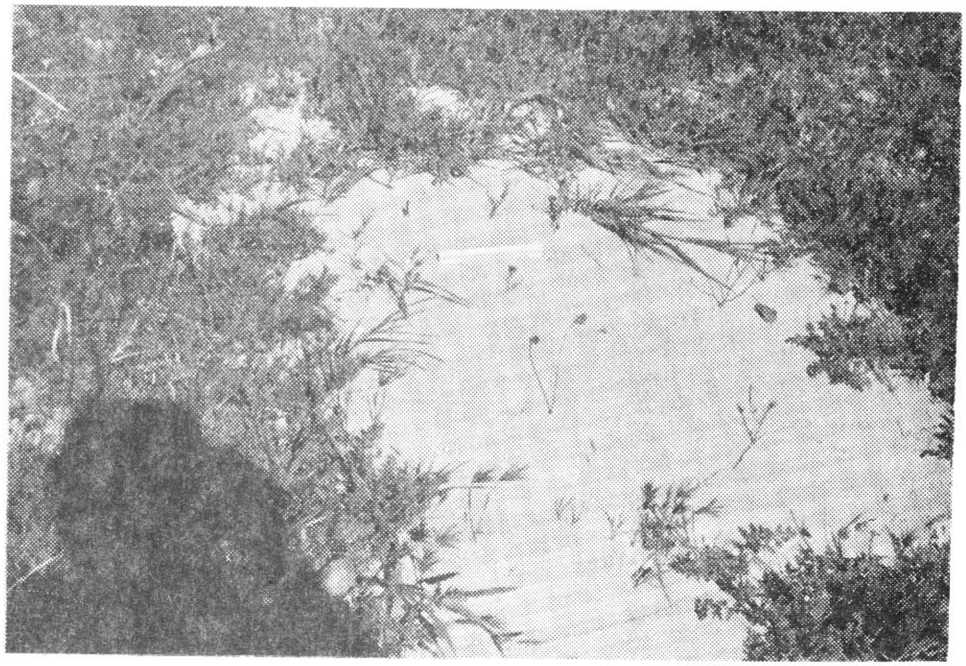


Photo 42. Liquefaction sand boil in central portion of Juvenile Hall slide.

Photo 43. Northern flank of slide, viewed from inside Juvenile Hall complex.

ROUTE 5/210 INTERCHANGE AREA

The Route 5/210 Interchange area suffered the most severe damage of the entire earthquake-damaged transportation system. In addition to collapsed bridges that blocked vehicular and railroad traffic, the roadway was distressed by tectonic uplift, differential settlement, embankment shear, spreading, and tension cracking.

Photos 44 and 45 show the interchange as it appeared the day after the earthquake. Specific damage locations to be discussed are indicated by number in Figure 6.

The terrain in the Route 5/210 area has over 100 feet of relief and is made up of sandstones of the Saugus formation. The canyons contain deposits of recent alluvium. Original ground and roadway profiles of Route 5 northbound and the A-Y Connector are shown in Figures 7 and 8.

Figure 6
Layout of the Route 5/210 Interchange

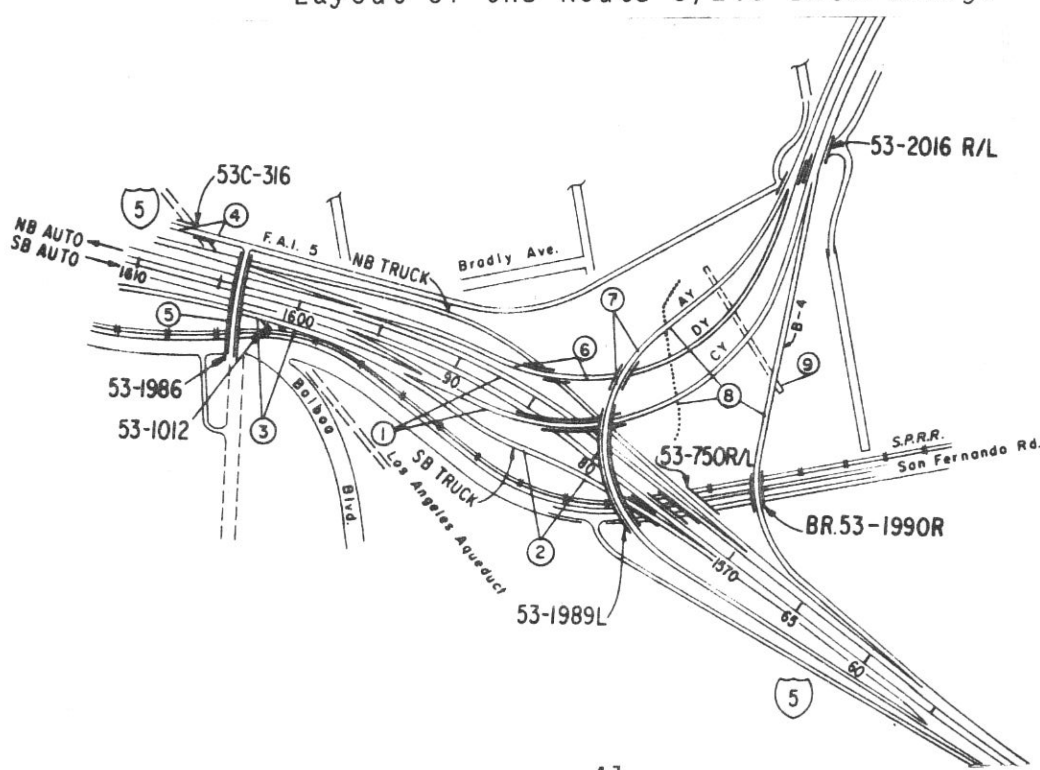


Photo 44. Looking easterly at Route 5. The Balboa Boulevard Overcrossing structure is in left center. The Los Angeles Aqueduct crosses the freeway from upper left to lower right. The Route 5/210 Interchange off the photo to the right.

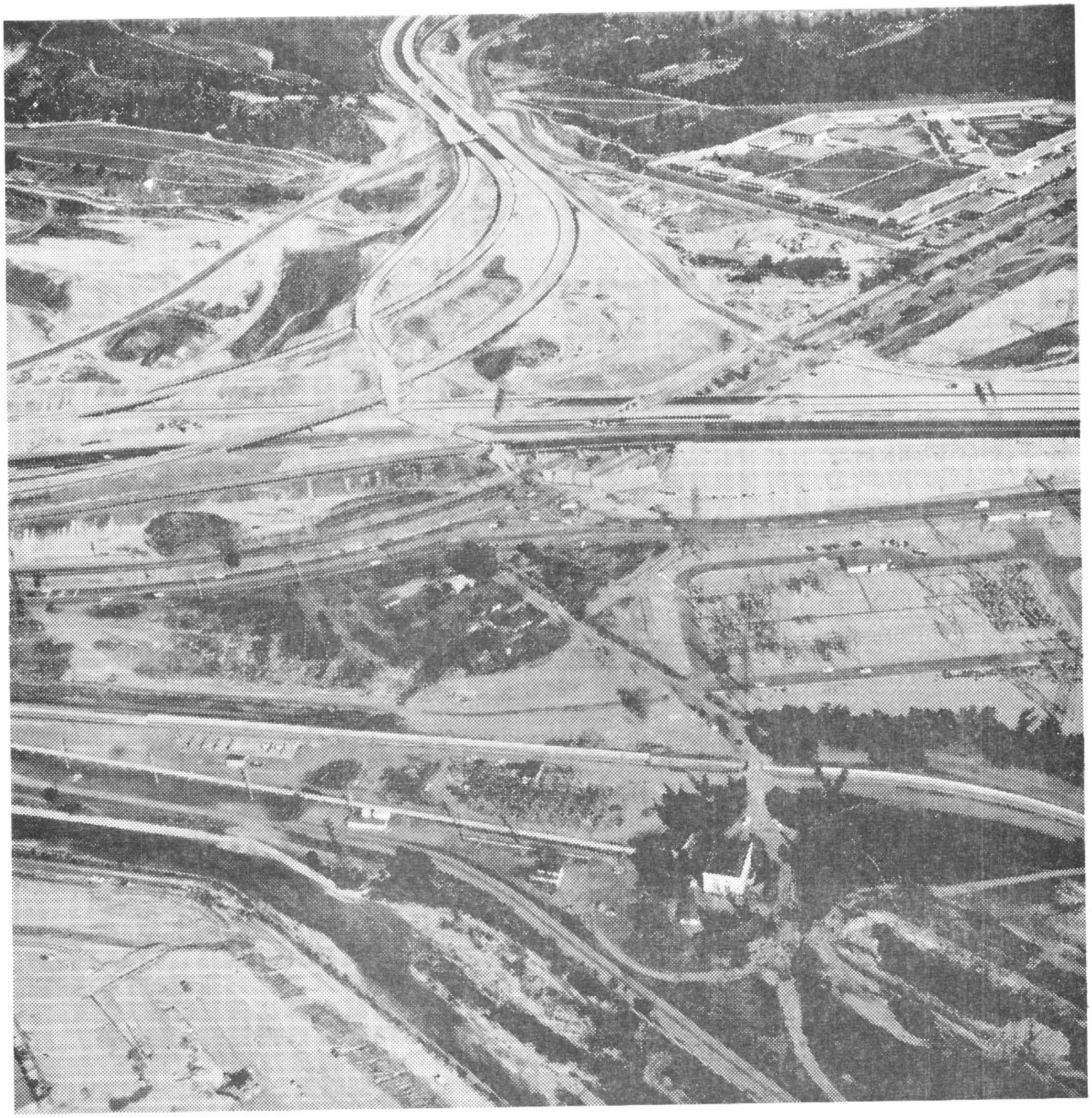
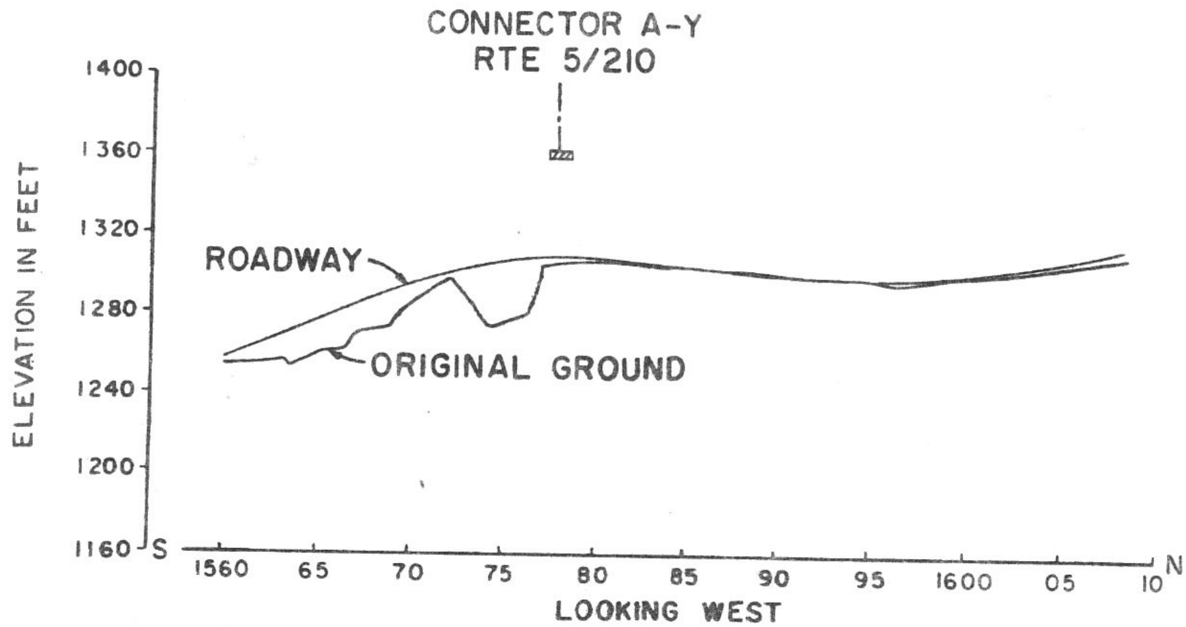
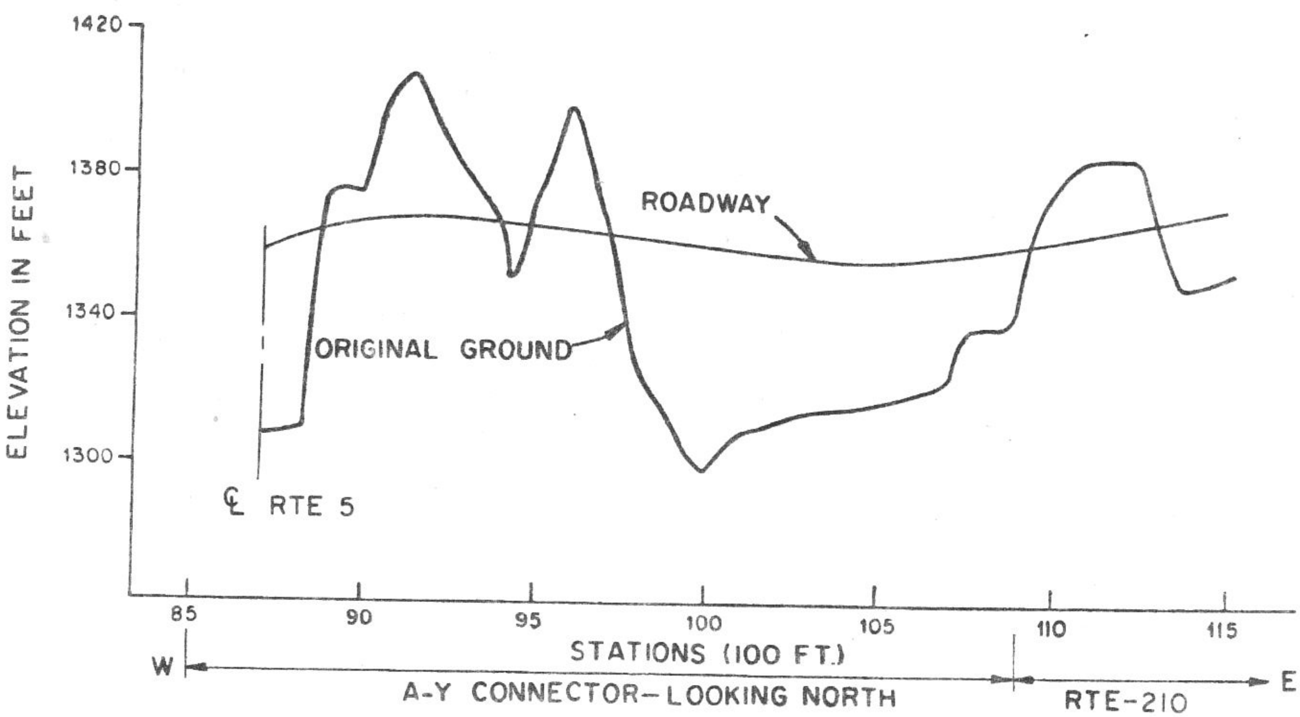


Photo 45. Looking easterly at the Route 5/210 Interchange. Route 5 is shown left to right and Route 210 at the top of the photo. Note the Juvenile Hall complex in upper right and the Sylmar Converter Station in right center.



ORIGINAL GROUND AND NORTH BOUND ROUTE 5
PROFILES IN ROUTE 5/210 INTERCHANGE AREA

FIGURE 7



ORIGINAL GROUND AND ROADWAY PROFILES
IN ROUTE 5/210 INTERCHANGE AREA

FIGURE 8

Location 1

Pavement slabs were faulted badly across the northbound roadbed of Route 5 and Connectors C-Y, D-Y, and M-Y. As may be noted from Photos 46, 47, and 48, typical faulting and slab separation occurred at both longitudinal and transverse joints. The large displacement shown in Photo 47 is the result of a large crack extending up the fill slope. An interesting feature of the pavement distress is that Route 5 at this location is essentially at original ground level (Figure 7) although the slab behavior was typical of pavement on fills.

Design and post-quake profiles of Connectors C-Y and M-Y are shown in Figures 9 and 10, respectively. The plots show that despite a regional uplift in this area the post-quake profile is lower than the design profile due to fill settlement caused by spreading and densification.

East of this location, natural slopes in the steep hills were shattered in a north-easterly trending belt approximately 500 feet in width.

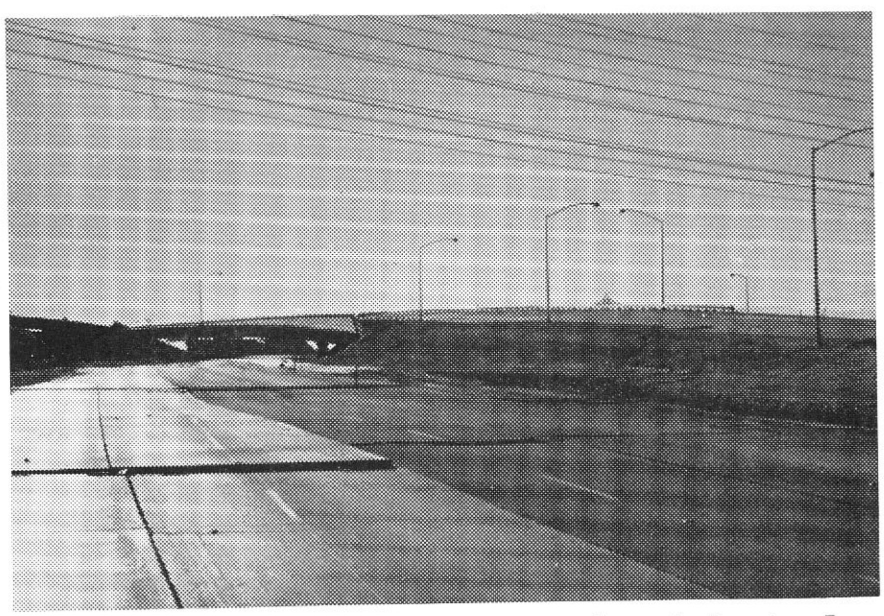


Photo 46. Looking southeasterly at Route 5 northbound truck lanes. Note slab separation, laterally and vertically, at joints. Damaged D-Y Connector Structure is in background.

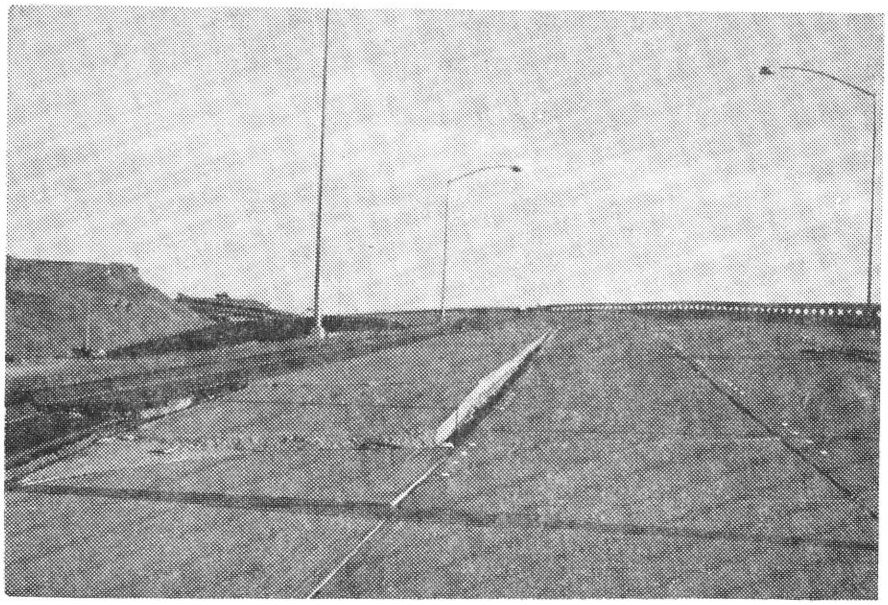


Photo 47. Looking southeasterly along Connector D-Y, at damaged fill-supported pavement.

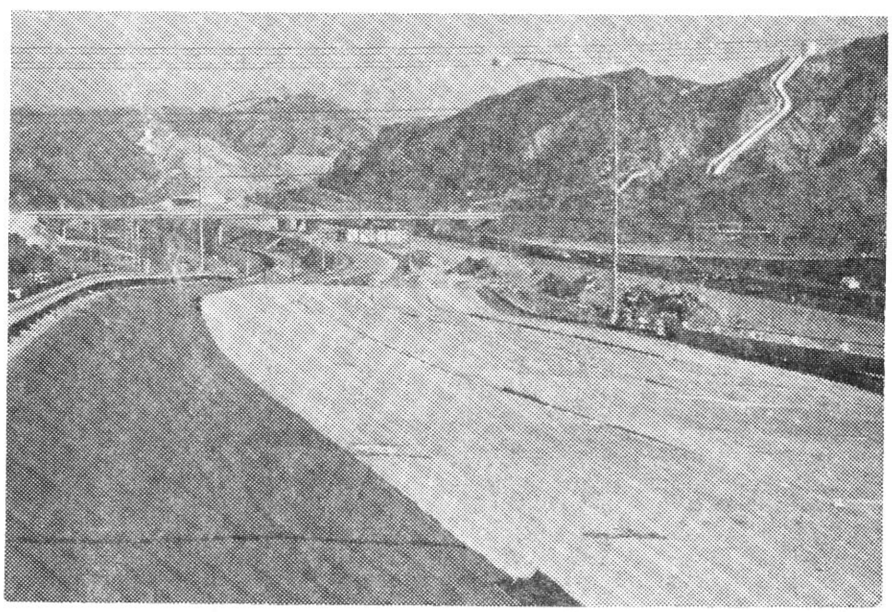
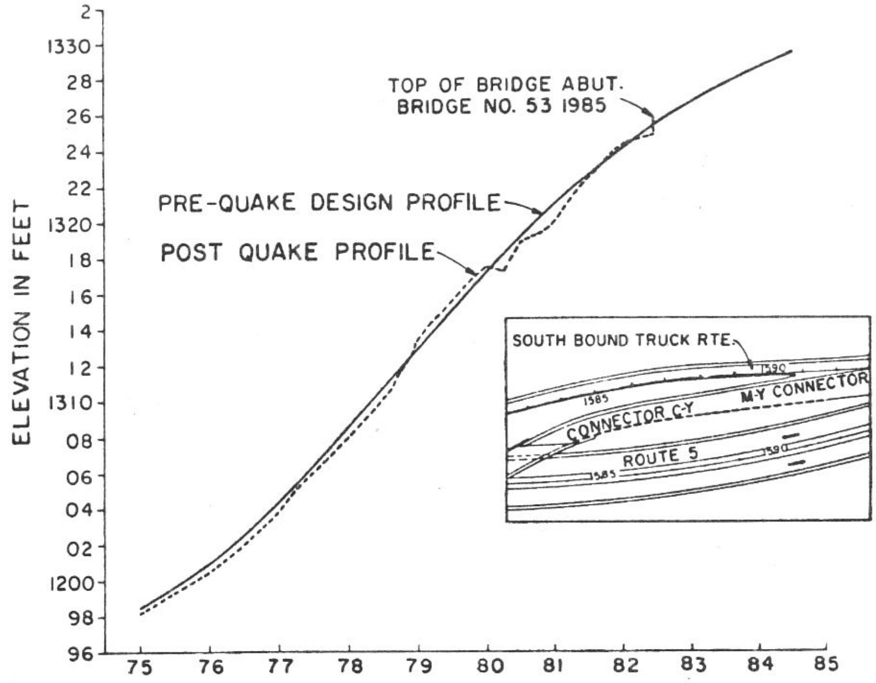
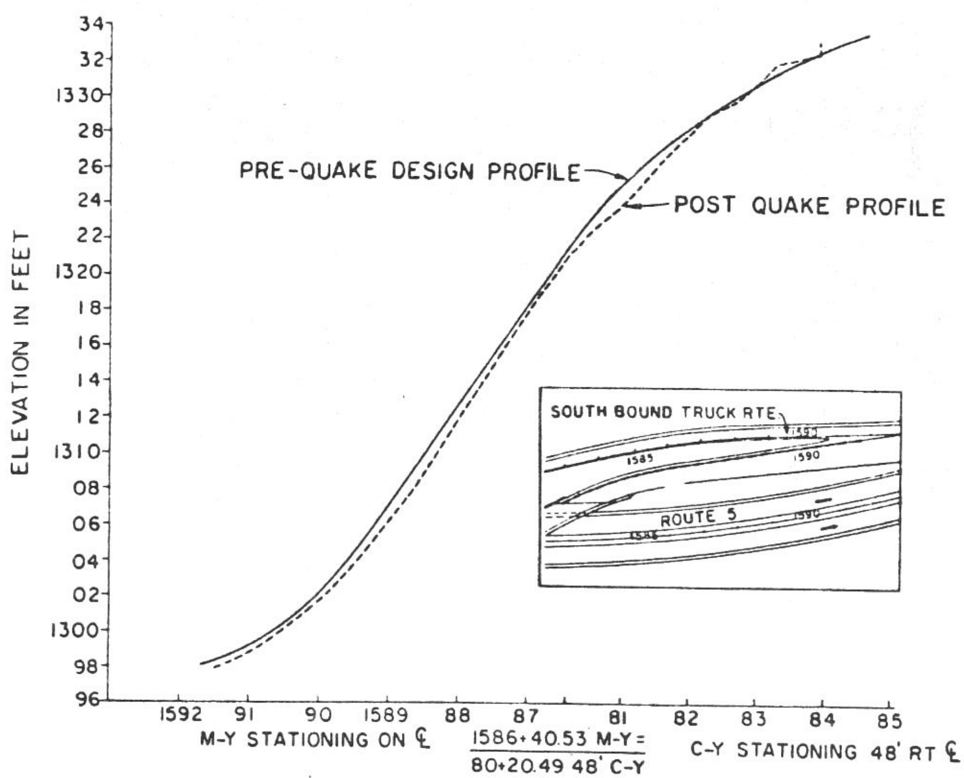


Photo 48. Looking northwesterly along Connector D-Y, at pavement slab faulting, separation, and undulations due to fill movement.



C-Y CONNECTOR PRE-QUAKE AND POST-QUAKE PROFILES

FIGURE 9



M-Y CONNECTOR PRE-QUAKE AND POST-QUAKE PROFILES

FIGURE 10

Location 2

The pavement of the Route 5 Southbound Truck Freeway was faulted and displaced laterally due to shearing action in the fill (Photo 49). The shear pattern, traced diagonally down the fill slope, outlined a complete volume of material that probably would have collapsed had strong motion been of a longer duration. The roadway profile and alignment were deformed as shown in Figures 11 and 12.

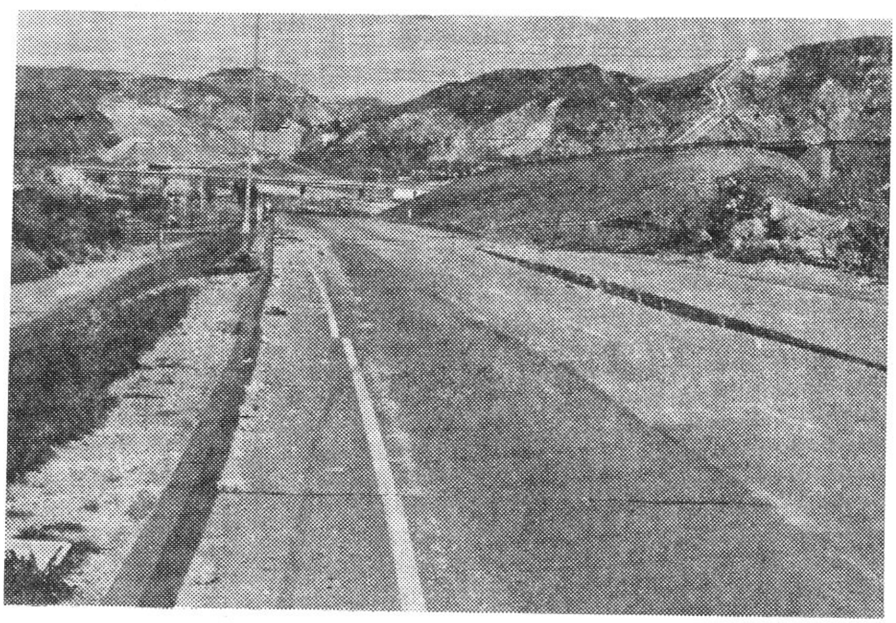
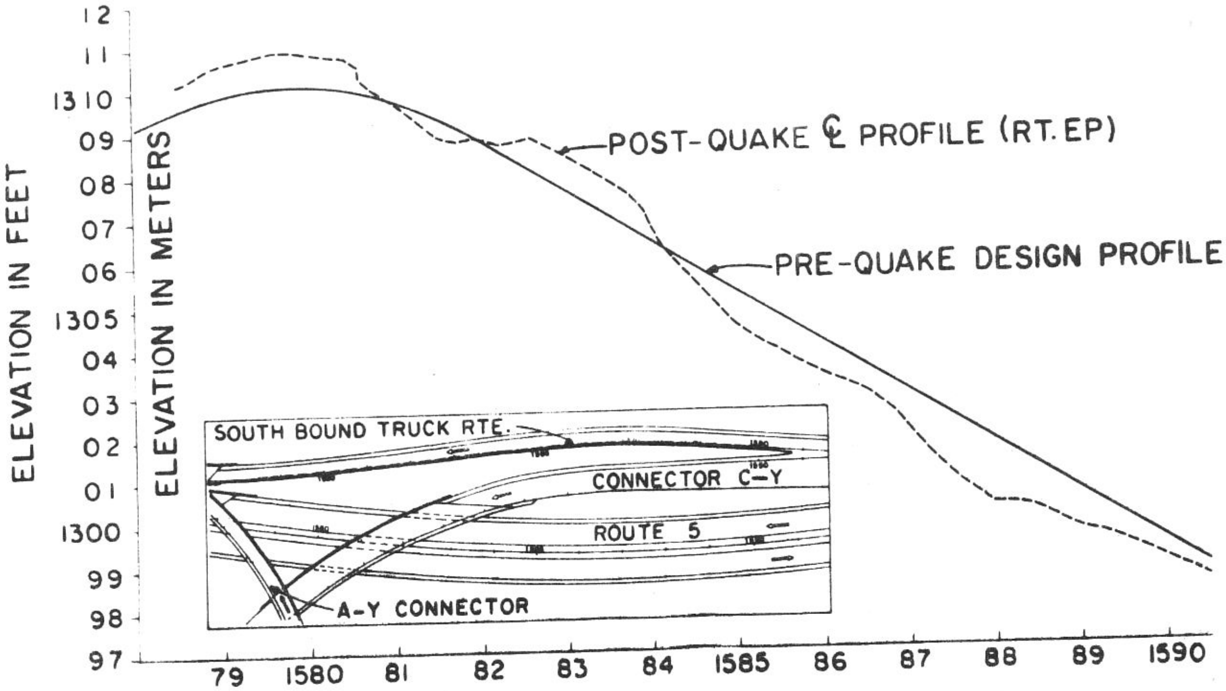


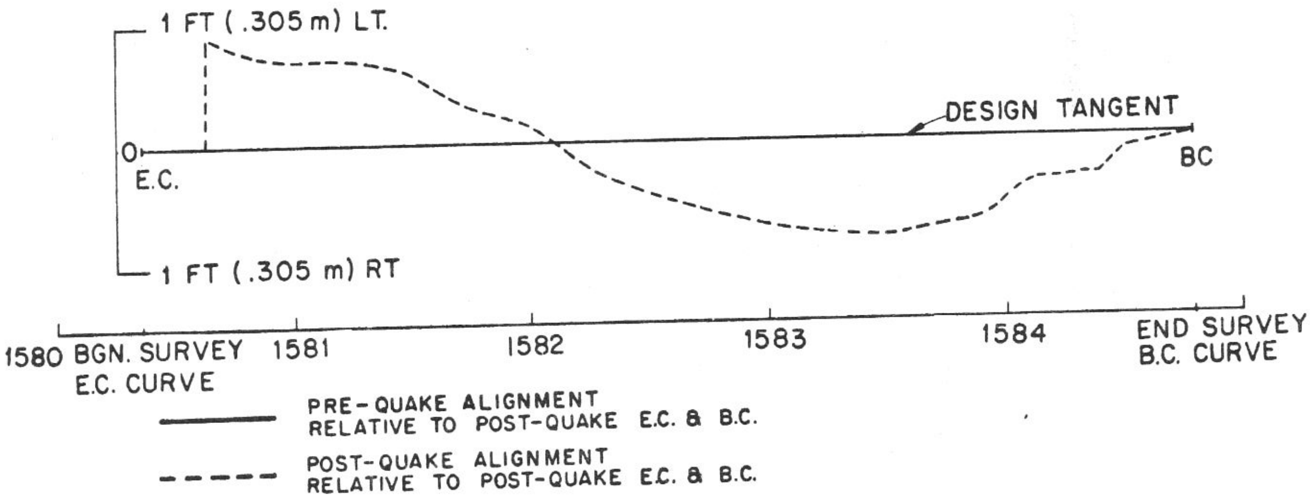
Photo 49. Looking northerly at damaged pavement of Route 5 southbound truck freeway caused by fill movement.



CENTERLINE PROFILE SB RTE 5 TRUCK FREEWAY

REFERENCE: SURVEY TLA 005K 135-136
PLAN 07-LA-5,210 SHT. 13

FIGURE 11



REFERENCE: SURVEY TLA 005 K-132
PLAN 07-LA-5,210 SHT. 13

HORIZONTAL ALIGNMENT OF A PORTION OF ROUTE 5 SOUTHBOUND TRUCK FREEWAY BEFORE AND AFTER THE QUAKE

FIGURE 12

Location 3

At this location, the Southbound Truck Freeway crosses the Los Angeles aqueduct and the penstock over separate structures. The aqueduct structure is founded on piles and covered with about six feet of fill. The penstock structure is founded on piles with the top of the structure serving as the roadway. The "rigid inclusion" effect of the aqueduct structure may be noted from Photo 50, which shows an undulating roadway profile caused by differential vertical movements. The before and after earthquake profiles are shown in Figure 13. This type of distress is typical of locations where large rigid structures cross through or below fills which, along with foundation materials, are subsequently densified by ground shocks. Bridge design borings indicate slightly compact to dense sand and sandy gravel to depths of about 40 feet below the original ground surface. From Figure 13 it appears that nearly all the "settlement" occurred due to compaction of the foundation soils. Since the regional uplift due to the earthquake in the area was approximately 0.3 foot, densification of fill material appears to have been minimal as determined by a comparison of pre-post earthquake profiles over the buried structure (Br. 53-1011), Figure 13.

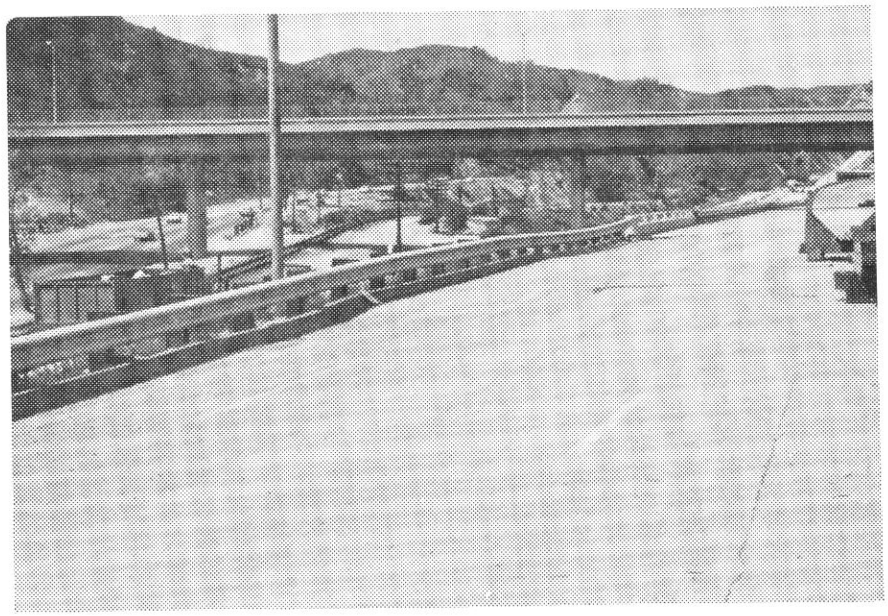
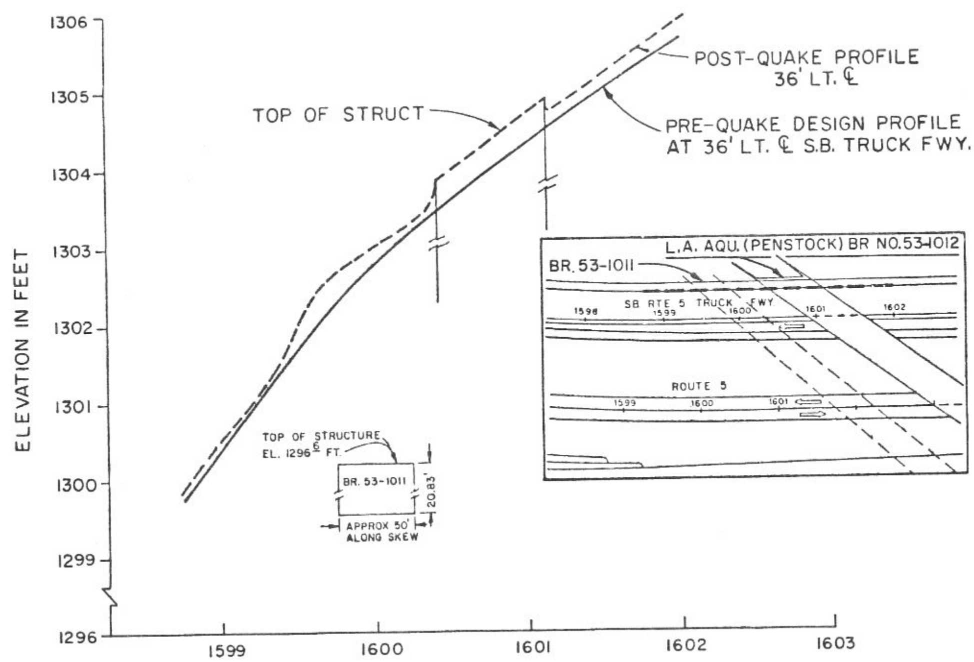


Photo 50. Looking northerly at undulating pavement of Route 5 caused by differential settlement. Balboa Boulevard Overcrossing structure is in background.



BEFORE AND AFTER QUAKE PROFILES OF PAVEMENT SHOWN IN PHOTO 50.

FIGURE 13

Location 4

Foothill Boulevard crosses the northerly end of the aqueduct structure (Bridge No. 53-1011) on about 22 feet of fill above the deck, and then, proceeding northwesterly, immediately crosses the penstock on Bridge 53c-316 (extreme left center of Photo 44). A closer view of the two structures is given in Photo 51. Shortly after the earthquake, fill material was removed from the deck of Br. 53-1011 to relieve stress on the sheared deck of the structure. Photo 52 is a view of the deck soffit, the portion that was overlain by about 22 feet of fill.

Backfill at the northwesterly abutment of Br. 53c-316 settled about 8 inches as shown in Photo 53. The settlement is clearly defined in the photograph because the pavement was asphalt concrete which does not provide a bridging effect as do the portland cement concrete slabs.

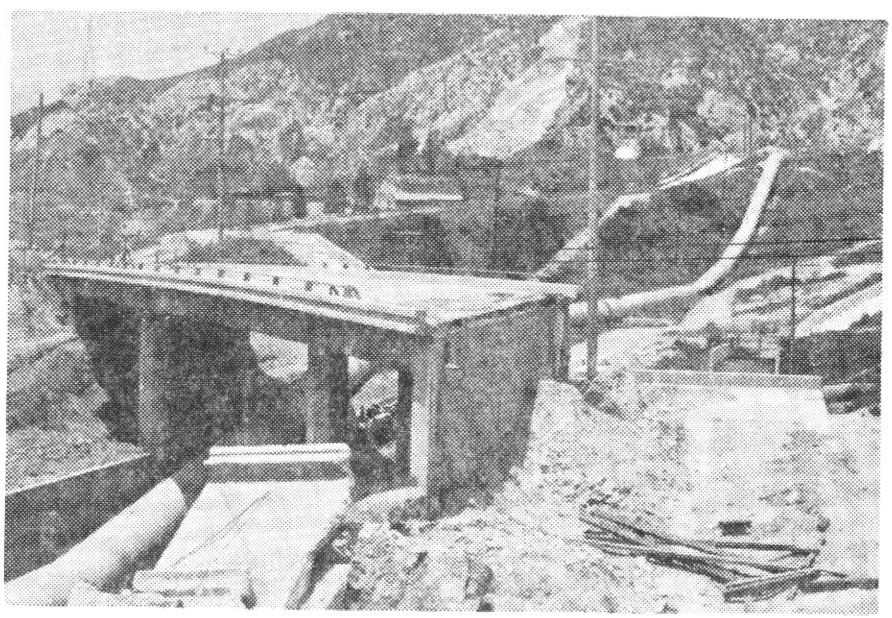
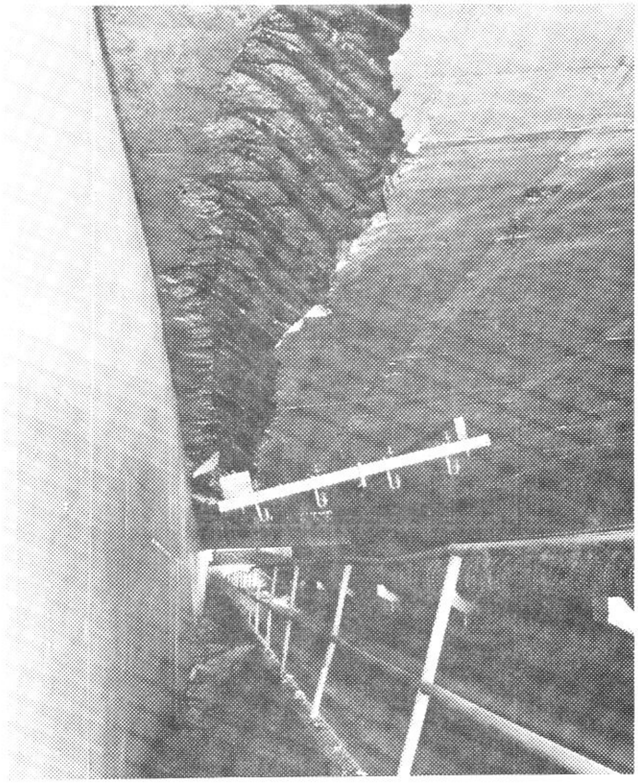


Photo 51. Looking northeasterly at Bridge 53c-316 which carries Foothill Boulevard over the penstock. Immediately to the right of the bridge, the uncovered deck of the aqueduct structure, Bridge 53-1011, is visible.

Photo 52. Southwesterly view from inside Bridge 53-1011 showing damage.



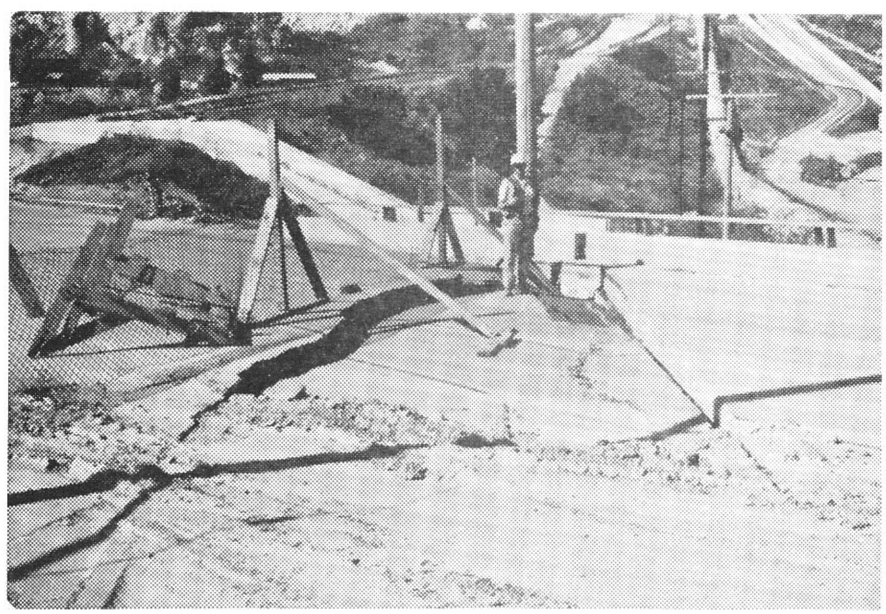


Photo 53. Settlement of abutment backfill at northerly end of Bridge 53c-316.

Location 5

Both approaches to Balboa Boulevard Overcrossing suffered backfill settlement. Photo 54 shows the easterly approach settlement of about 8 inches at the bridgedeck and wingwall joint. The bridge abutment was founded on piles while the wingwalls were on spread footings founded in the embankment. Approximately 25 feet of fill rests over original ground at the easterly bridge approach.

The westerly approach was constructed in cut section with both the bridge abutment and wingwalls founded on spread footings. Backfill at the paving notch settled several inches, while the bridge deck and wingwall remained in essentially the same place, as shown in Photo 55. The abutment failed in shear and rotation about a horizontal axis as shown in Photo 56 and tended to arch the deck as can be seen along the neat line of the concrete quardrail shown in Photo 57.

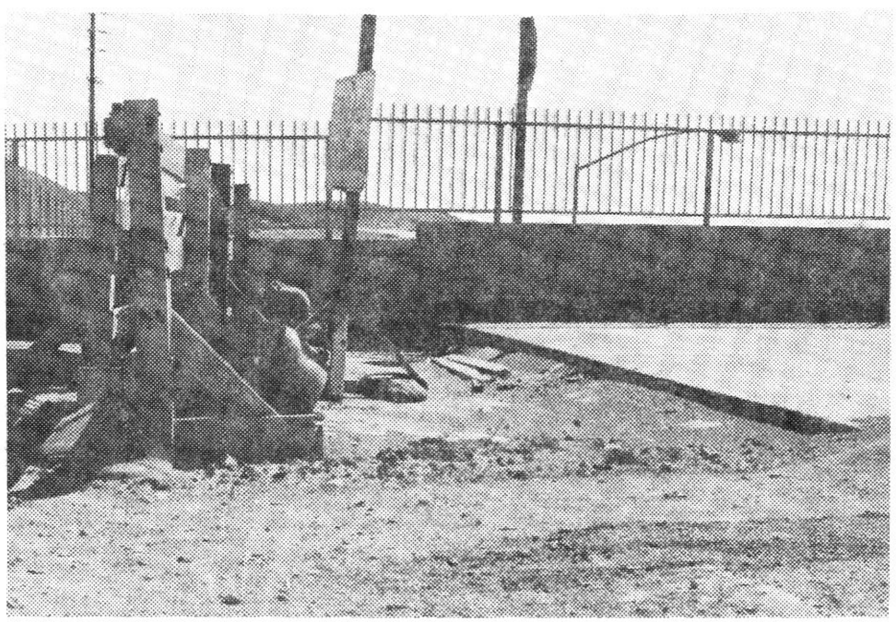


Photo 54. Approach and wingwall settlement at easterly end of Balboa Boulevard Overcrossing.

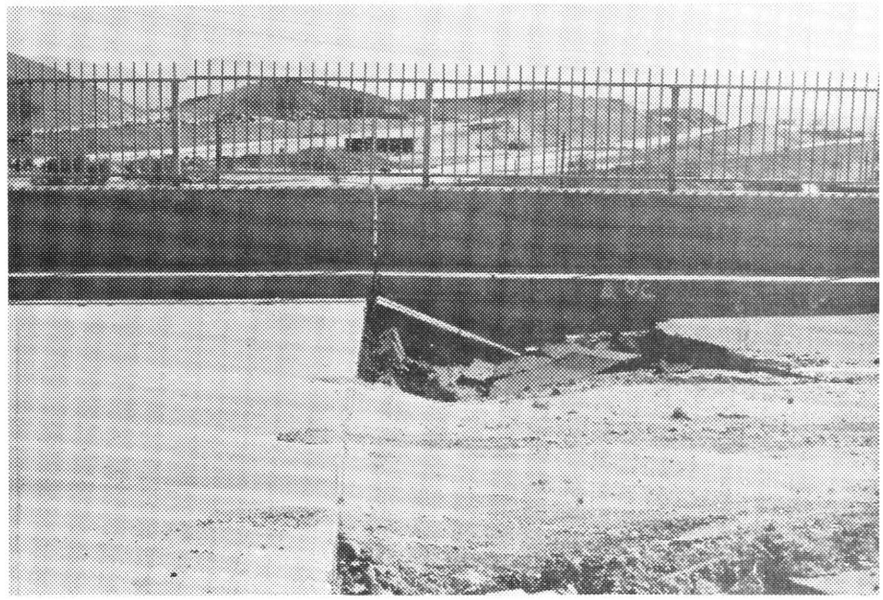
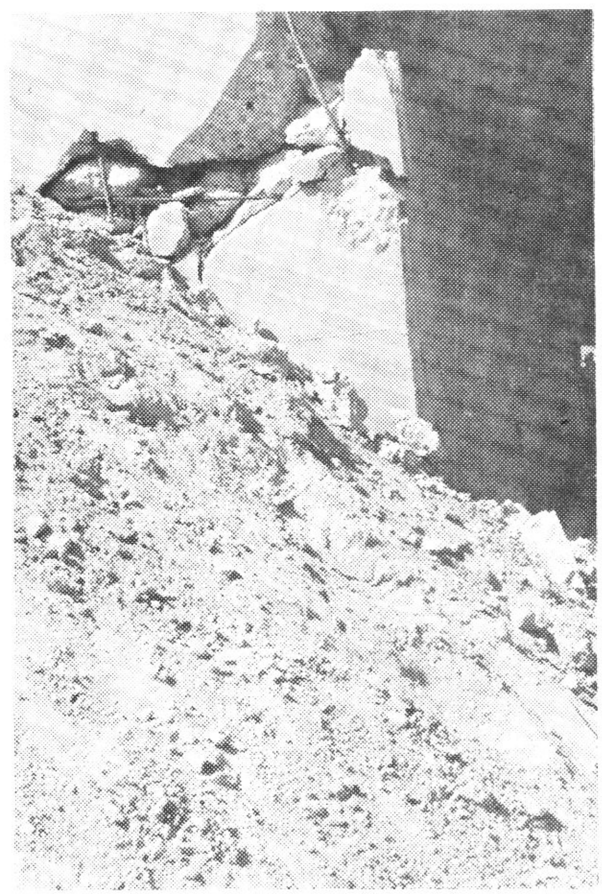


Photo 55. Westerly approach to Balboa Boulevard Overcrossing. Note abutment backfill settlement but lack of differential settlement between wingwall and abutment.

Photo 56. Damaged westerly abutment of Balboa Boulevard Overcrossing.



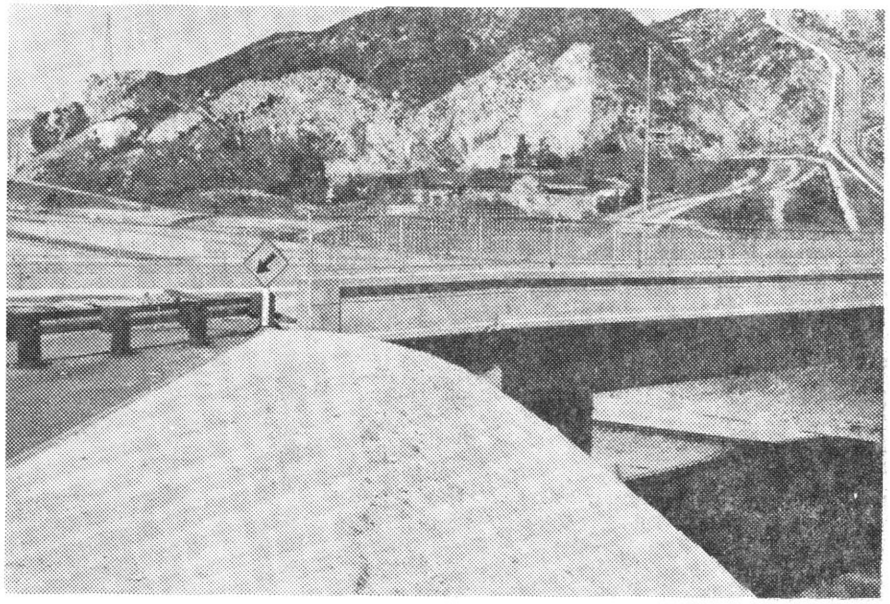
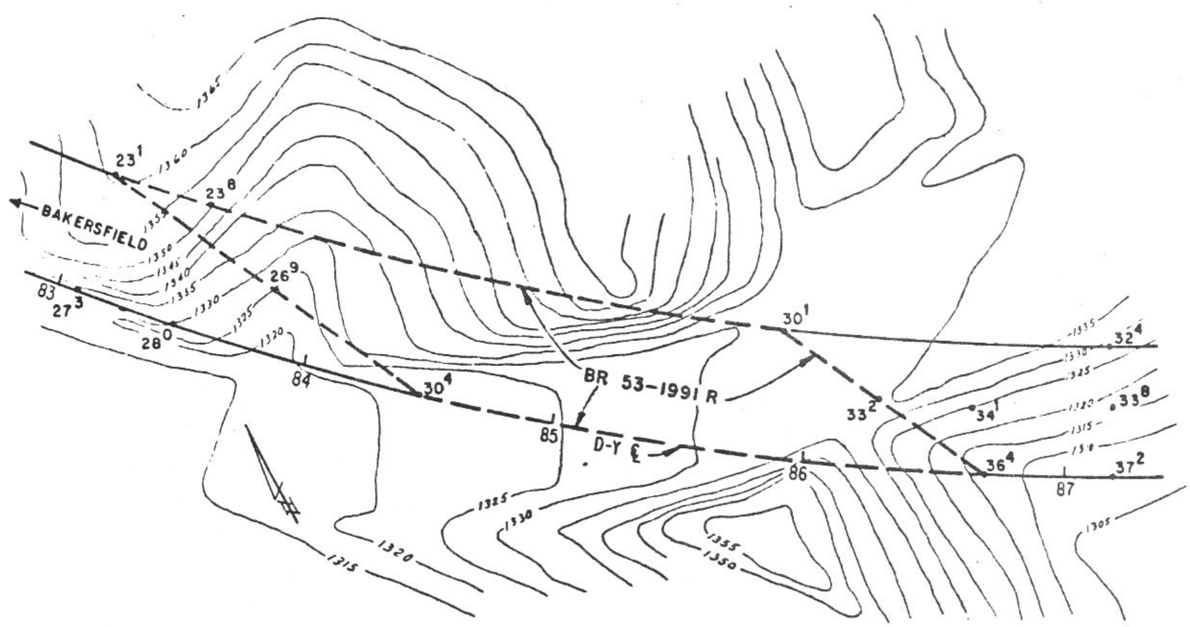


Photo 57. Looking northeasterly at westerly abutment of Balboa Boulevard Overcrossing. Note slight arch in structure caused by abutment movement.

Location 6

The approaches to this structure (Br. 53-1991R) are a combination of cut and fill. Figure 14 shows the original ground topography with the structure superimposed over it. The entire bridge is founded on spread footings in original ground that is composed of very dense sand, gravelly sand, and sandy gravel with cobbles of the Saugus formation. "Settlement" at the easterly bridge approach, Photos 58 and 59, was about two feet at its maximum. At the westerly abutment, "settlement" was several inches; with a portion of the resulting distress to pavement shown in Photo 60.

The vertical alignment of the bridge and retaining wall (wingwall) remained essentially the same, as can be seen in Photo 59. Since both were founded on spread footings in very dense original ground, and the vertical alignment remained uniform, the observed settlement is believed to be due to densification and loss of backfill material into voids created by the superstructure displacement during strong seismic shaking.



LEGEND

28³ = FINISH GRADE ELEV.

–1330 = ORIGINAL GROUND CONTOURS

20 40 FEET

5 10 15 METERS

CONTOURS SHOWING ORIGINAL GROUND TOPOGRAPHY AT BRIDGE 53-1991R ON THE D-Y CONNECTOR OF THE ROUTE 5/210 INTERCHANGE

FIGURE 14

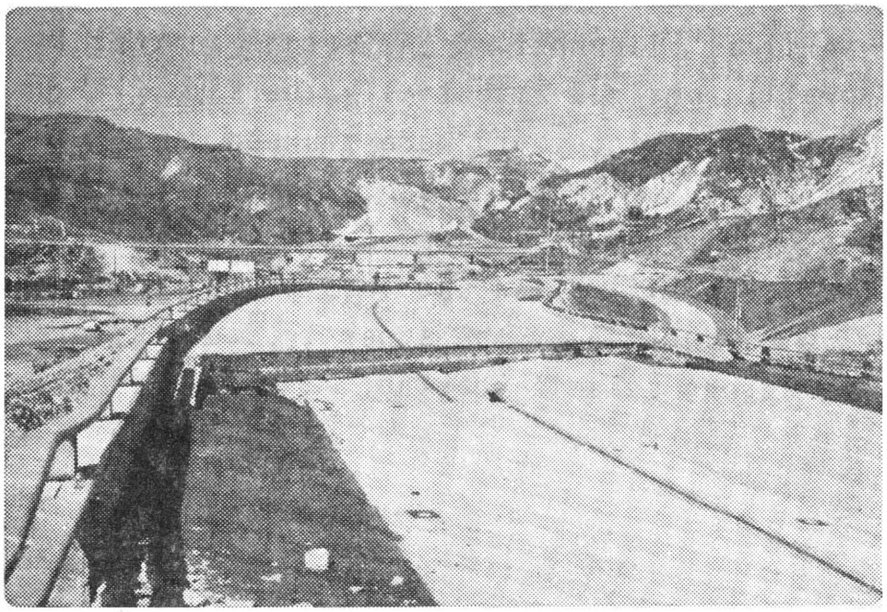


Photo 58. Settlement at the easterly approach to Bridge 53-1991R on the D-Y Connector, Route 5/210 Interchange.

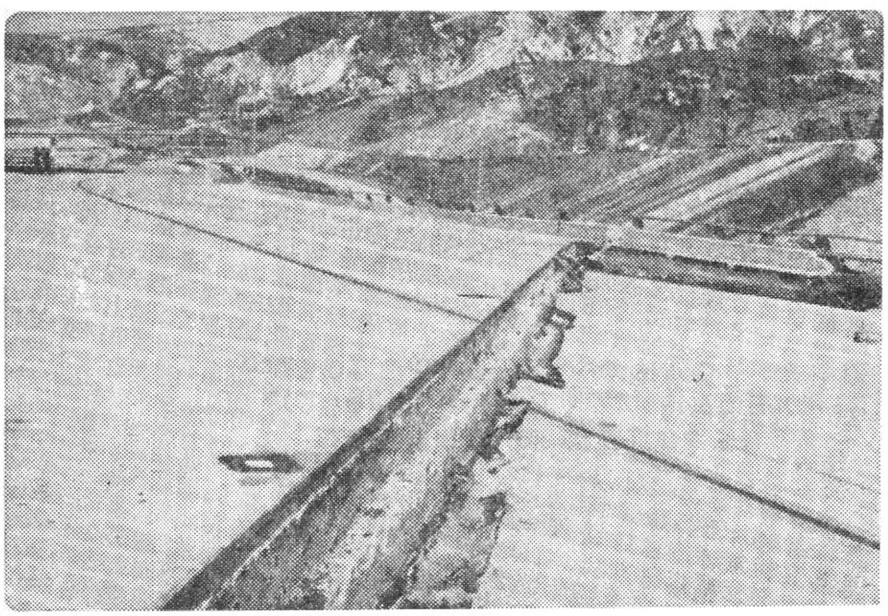
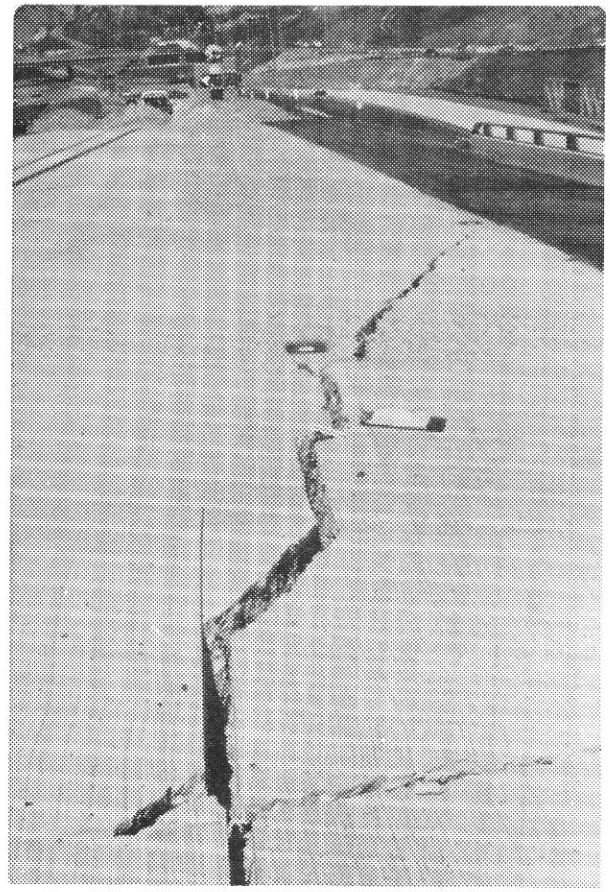


Photo 59. Settlement at the easterly approach to Bridge 53-1991R on the D-Y Connector, Route 5/210 Interchange.

Photo 60. Cracking and movement of D-Y Connector pavement immediately west of Bridge 53-1991R.



Location 7

The bridge at this location is founded on spread footings in original ground. The westerly approach is entirely within cut, while the easterly approach is made up of 12 to 30 feet of fill across a narrow steep canyon. The easterly approach pavement slabs cracked, separated, and settled a few inches at the bridge deck, as shown in Photos 61 and 62; whereas little or no pavement distress was noted at the westerly approach. Severe shaking of the bridge did occur as evidenced by the cracked wingwalls and abutment (Ref. 4, p. 207, Index 10), which would allow fill spreading and consequent pavement distress.

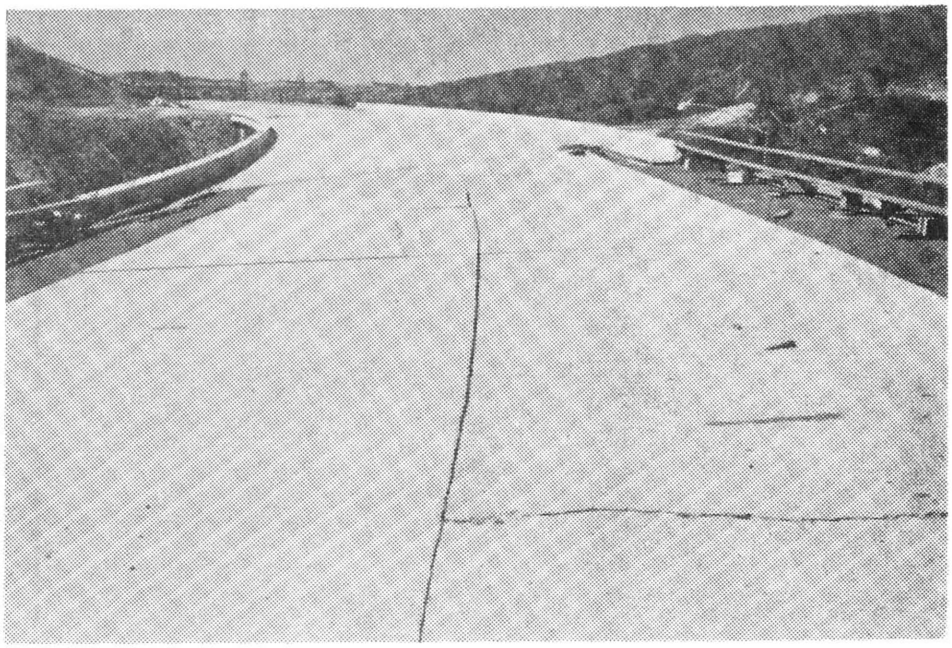


Photo 61. Pavement cracks and movement at the easterly approach to Bridge 53-1988L on the A-Y Connector, Route 5/210 Interchange.

Photo 62. Transverse pavement crack at cut/fill contact near easterly approach to Bridge 53-1988 on the A-Y Connector.

Location 8

Pavement and soil-cracking occurred in the general area of the cut-fill contact shown in plan on Figure 6. From Connector C-Y the cracking diverged away from the contact in a southerly direction. The cracking progressed from one crack across Connector A-Y, Photo 63, to a zone approximately 200 feet wide south of Connector C-Y. Individual cracks show up to one inch of horizontal separation and generally 1 to 2 inches of vertical separation with the east side down relative to the west. Photo 64 shows the pavement displacement across Connector D-Y and Photos 65 and 66 show the cracking and displacement across Connector C-Y. No cracking on either fill slope was found in this zone. However, south of the toe of fill for a distance of about 70 feet cracking was observed in original ground.

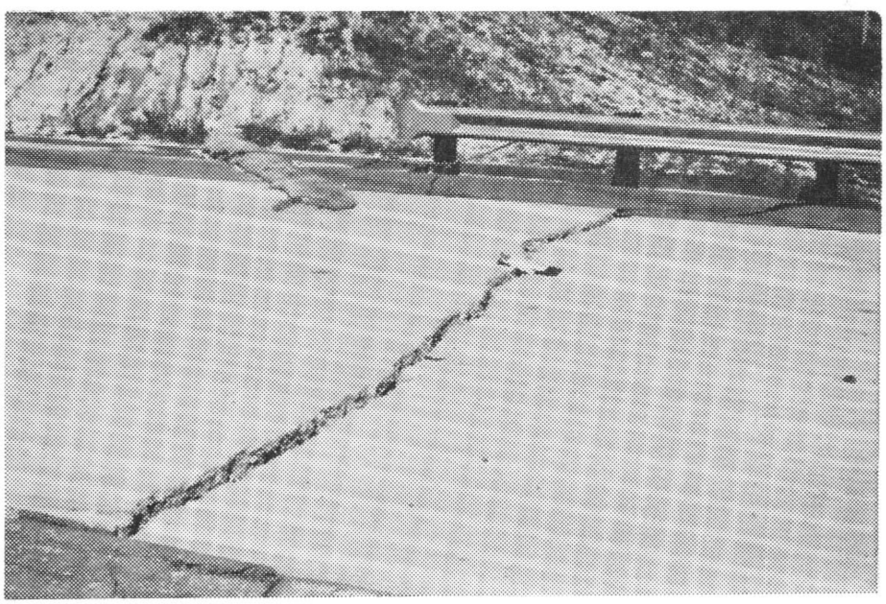


Photo 63. Looking northerly at transverse pavement crack at the cut/fill contact on A-Y Connector. Note horizontal and vertical separation along crack.

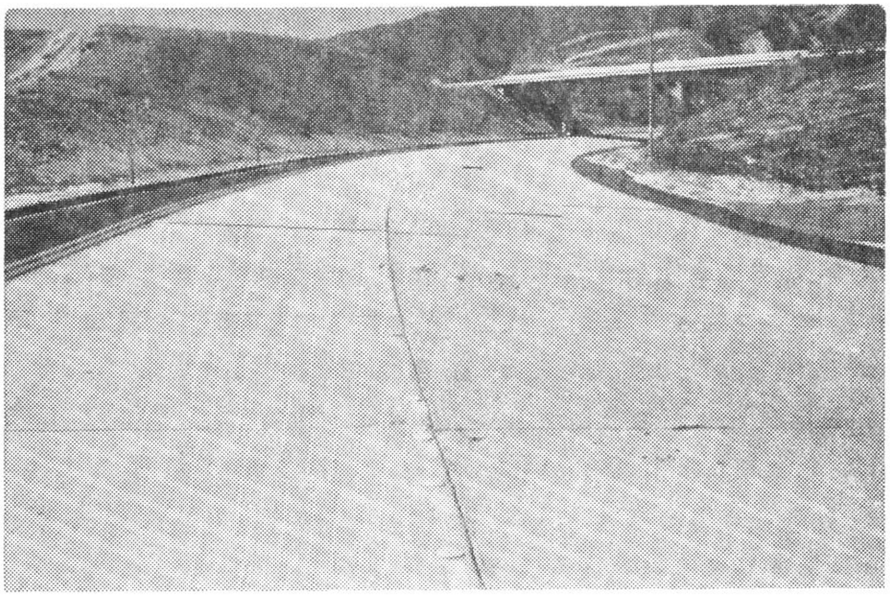


Photo 64. Looking northwesterly at pavement slab displacement through cut/fill zone on Connector D-Y.

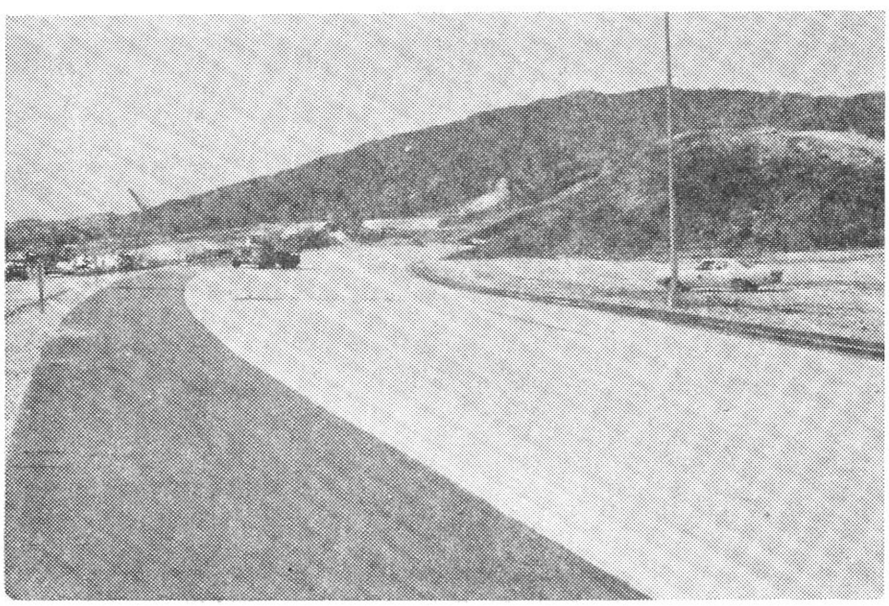


Photo 65. Looking northwesterly at pavement slab displacement through cut/fill zone on Connector C-Y.

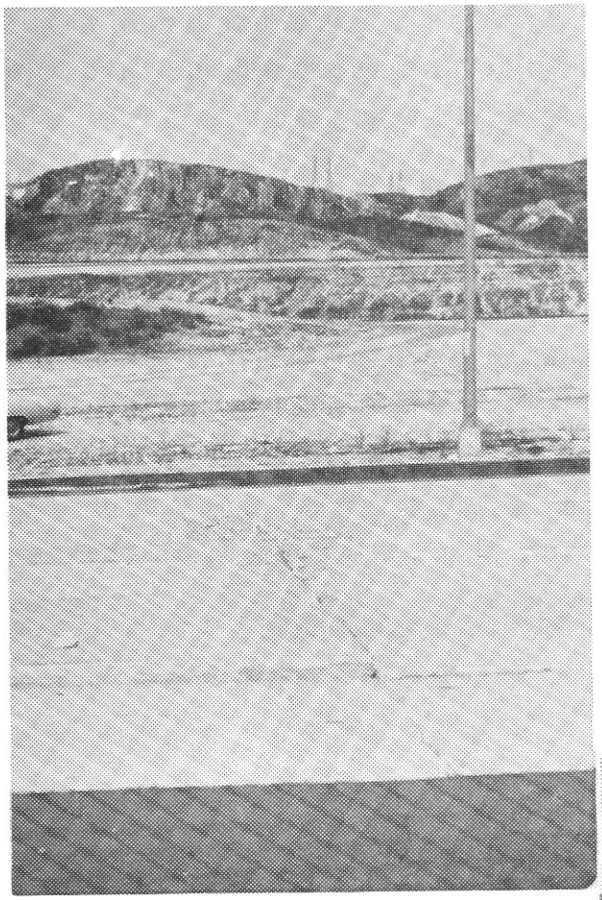


Photo 66. Transverse pavement crack on Connector C-Y. (Light standard is same as shown in Photo 65.)

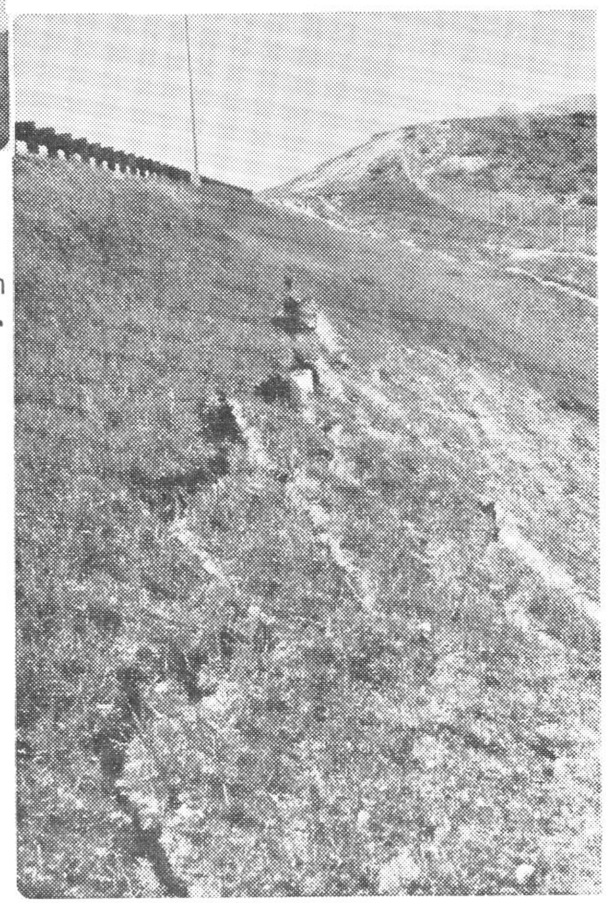


Photo 67. Cracking in north fill slope of Connector A-Y. The pavement crack shown in Photo 63 occurred at cut/fill contact in background.

On the north slope of the fill and about 300 feet east of the cut-fill contact a surficial slide occurred that is shown in Photo 67. This failure extended up into the roadway causing minor pavement distress and a slight dip in profile.

Location 9

The conduit at this location is a double reinforced concrete box 6 feet wide by 11 feet high at the outlet (south end). Fill height over the southerly portion of the box was about 65 feet. Vertical members of the box experienced cracking as shown in Photo 68 that extended into the box for a distance of about 200 feet.

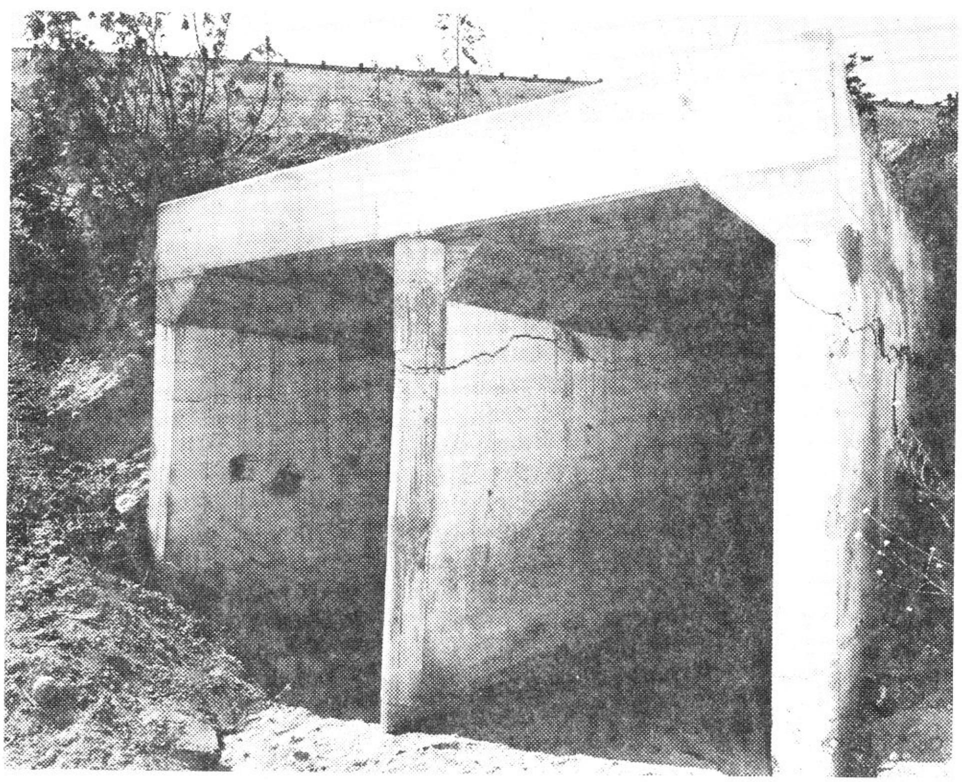
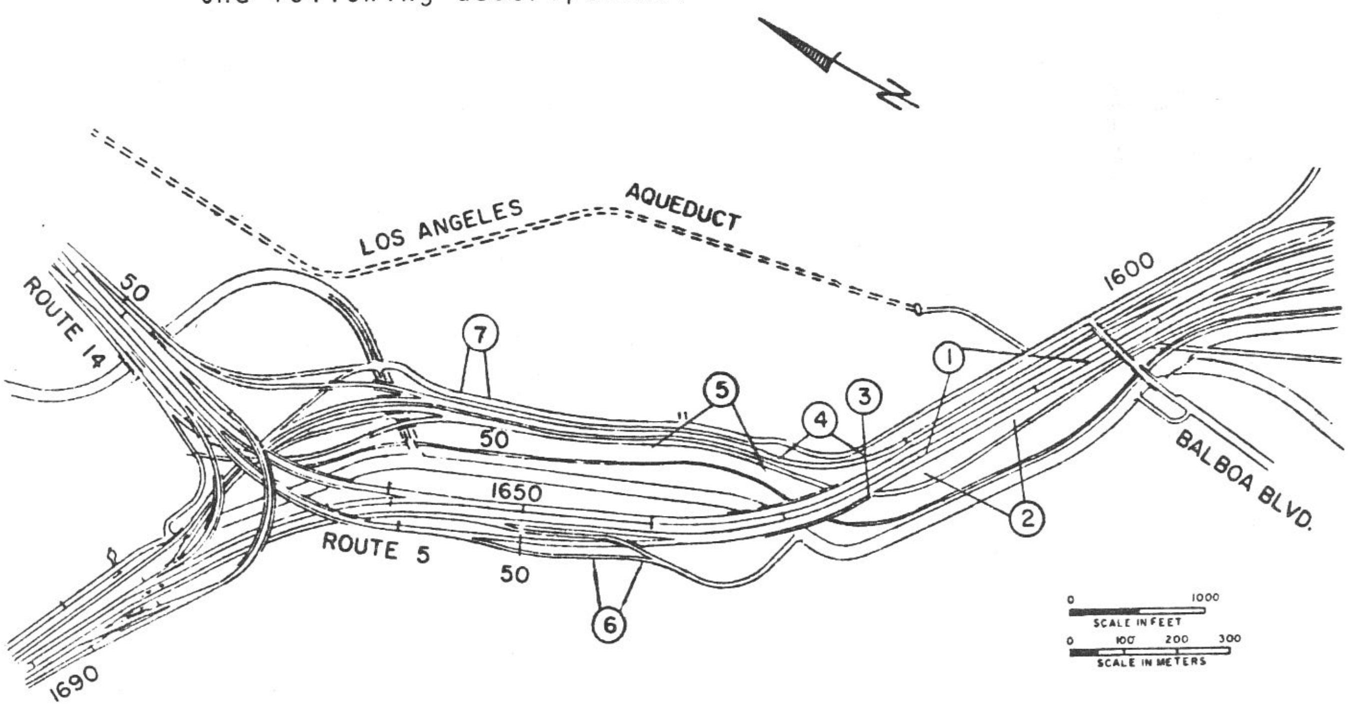


Photo 68. Cracking in reinforced concrete double box culvert. Note vertical members are out of plumb.

I-5, FROM BALBOA BLVD. THROUGH 5/14 INTERCHANGE

From Balboa Boulevard northward, Route 5 is constructed on both sides of a steep canyon with high cut and fill sections. At the time of the earthquake, widening of the system was in progress with extensive cut and fill operations nearly complete on the west side of the canyon. The east side of the canyon carried traffic along the old alignment. The realignment of Foothill Boulevard along the east side of the canyon and east of the freeway was complete and involved a large cut section. Photos 69-72 show progressive aerial views from north to south of this portion of Route 5, taken the day after the earthquake. Photo 73 is a southerly view of the Route 5/14 Interchange, also taken the day after the earthquake.

Figure 15 is a map of the area showing the locations of the following descriptions.



LAYOUT OF ROUTE 5 BETWEEN BALBOA BOULEVARD AND THE ROUTE 5/14 INTERCHANGE

FIGURE 15

Photo 69. Looking easterly at the Route 5/14 Interchange.

Photo 70. Looking easterly at Route 5, immediately south of the Route 5/14 Interchange.

Photo 71. Looking easterly at Route 5.

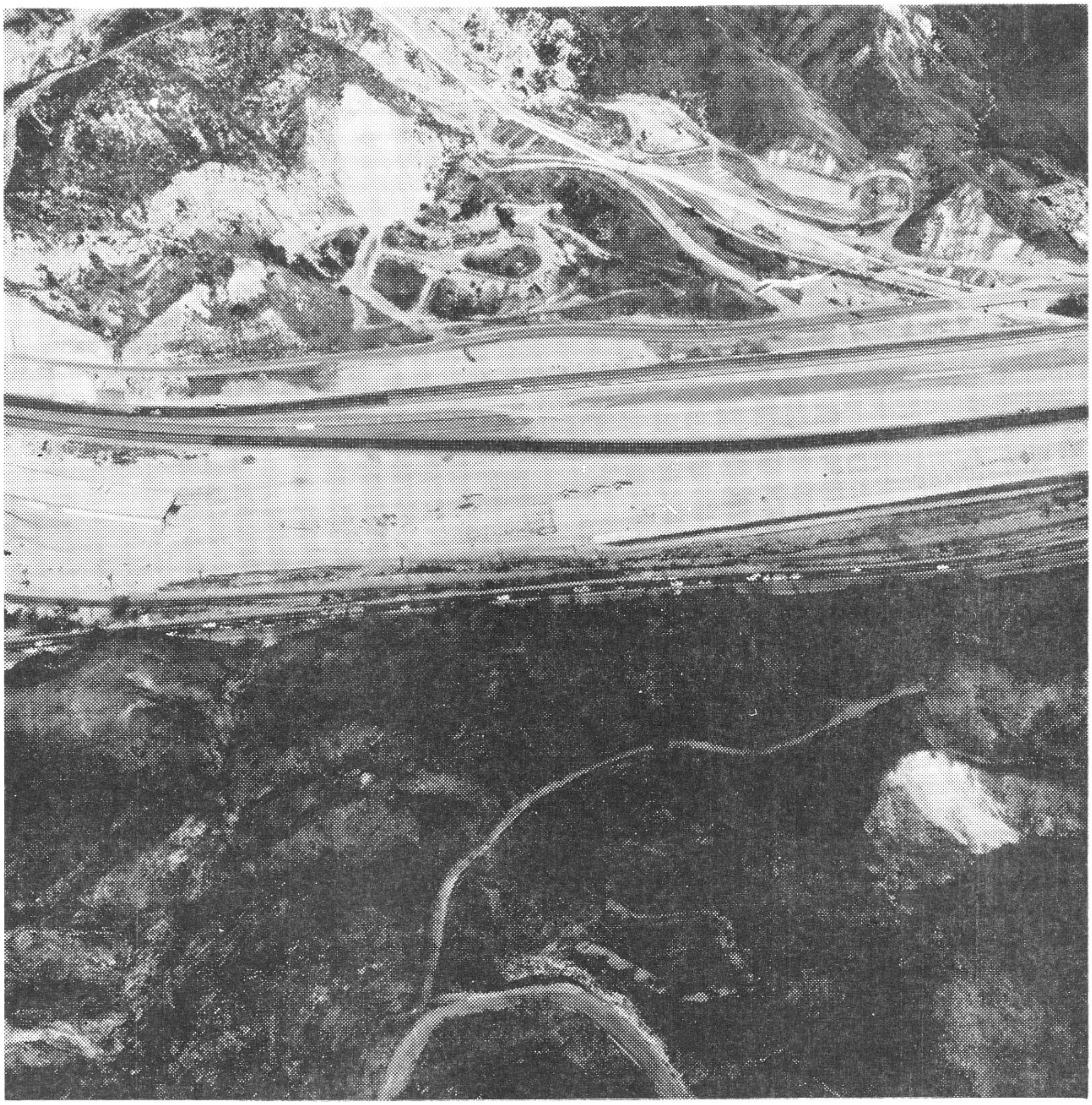


Photo 72. Looking easterly at Route 5 immediately north of Balboa Boulevard Overcrossing.

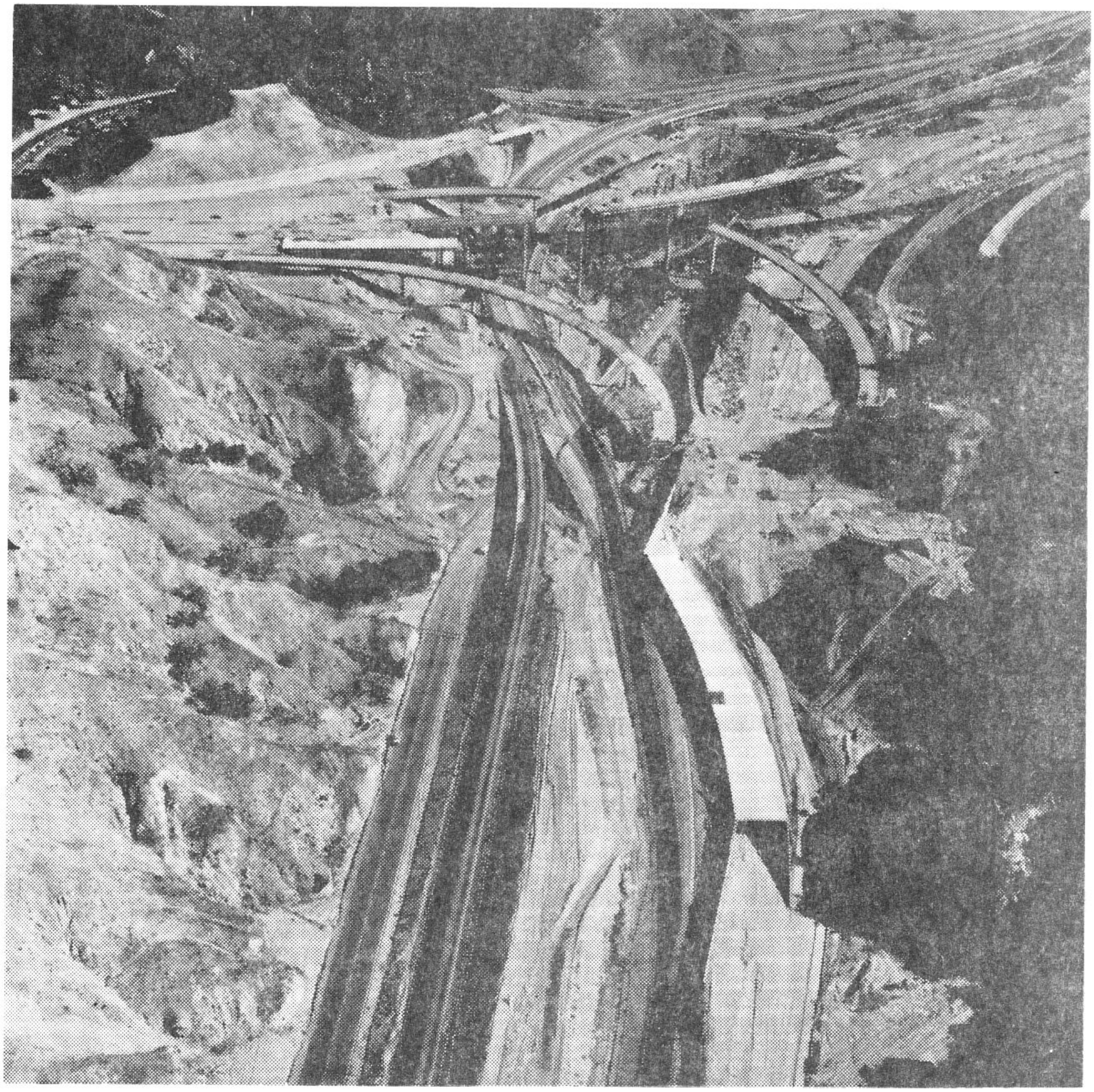


Photo 73. Looking southerly at the Route 5/14 Interchange. Route 14 is in upper left of photo.

Location 1

Very little soil or pavement distress was found in this area. A crack across the asphalt pavement of temporary Route 5 (right central portion of Photo 72) showed 1/2-inch of lateral displacement, south side moved west relative to north side. About 450 feet north of this offset, a tension crack appeared in the pavement with one inch of separation.

Location 2

A natural drainage channel at this location was routed through a double reinforced concrete box 8 feet wide by 12 feet high. The north end joined an existing reinforced concrete arch over which the railroad crossed. No damage was observed at the north end. However, at the south end, longitudinal hairline cracks occurred in the walls for at least 200 feet into the culvert.

Location 3

Cracking of both soil and concrete occurred transverse to the highway at this location. Photo 71 shows roadway concrete broken (right center above the curved reinforced concrete box structure) at what appears to be very near the cut-fill contact. Photos 74 and 75 are close-ups of this break, looking east. Soil cracking occurred through the above concrete box structure and continued westerly into the bridge abutment area (barely visible right of center in Photo 71). The hills to the east of the roadway along the trend of the cracking were shattered in a 100-ft wide zone. The above cracking appears to follow closely the upper Santa Susana thrust fault as mapped by others (Ref. 5, Vol. III, p. 213, Figure 1). East of the shattered slopes a large rockfall occurred, which is visible in Photo 71 (top right).

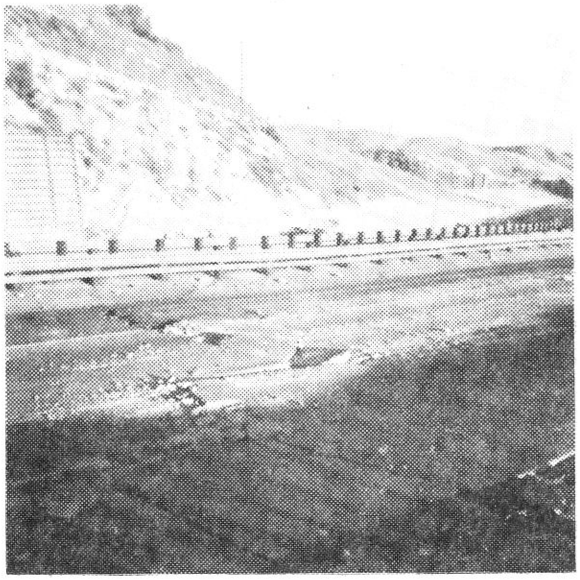


Photo 74. Looking easterly at pavement damage on Route 5 north of Balboa Boulevard.

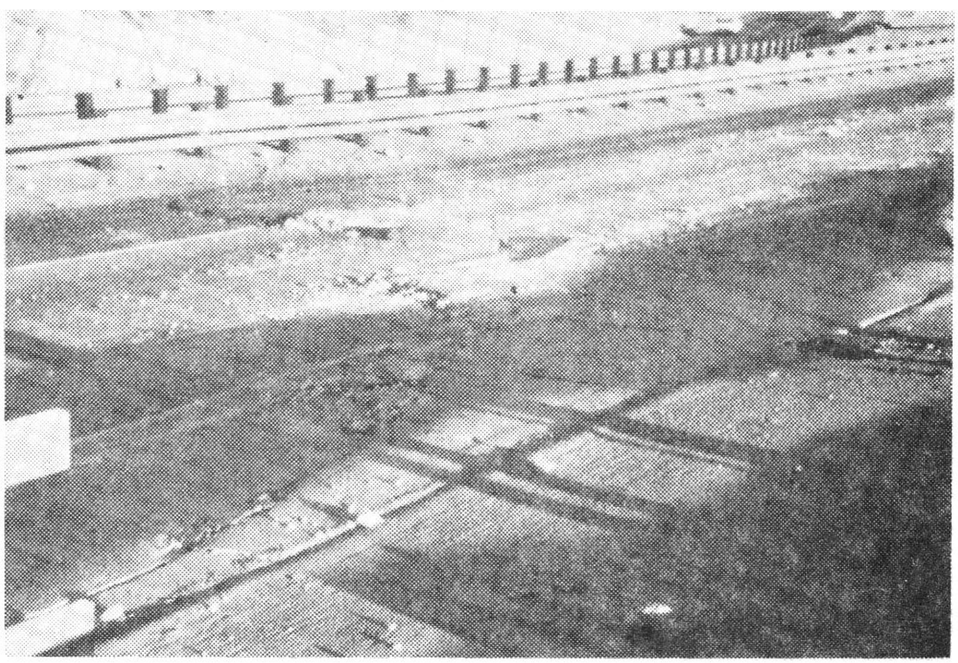


Photo 75. Closer view of pavement damage shown in Photo 74.

Location 4

Rockfalls occurred in the steep, well-indurated sandstone cut face (upper right central portion of Photo 71).

Location 5

Minor compression spalling occurred at the construction joints in a retaining wall that separates the railroad from highway embankment. The fill supported by the wall slumped and longitudinal cracking occurred in the roadway, as shown in Photo 76.

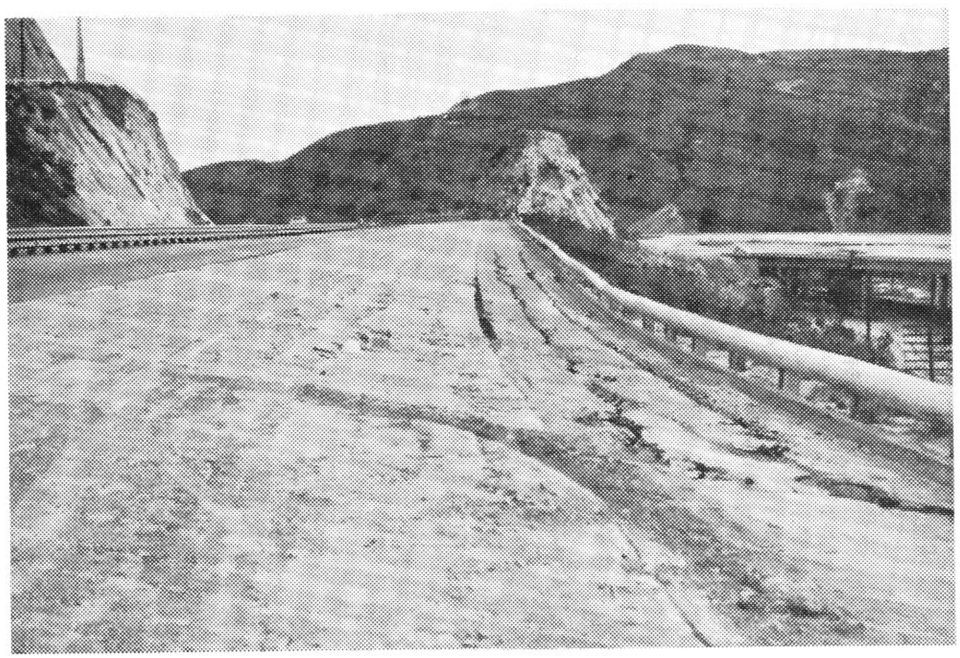


Photo 76. Longitudinal cracking in pavement due to lateral movement at top of cantilever retaining wall. Pavement has been broken mechanicall for removal.

Location 6

A large landslide occurred in a cut slope at this location (Photos 77 and 78). Had the roadway been completed, frontage road access from one approach would have been blocked by the slide. An interesting aspect of this slope is that

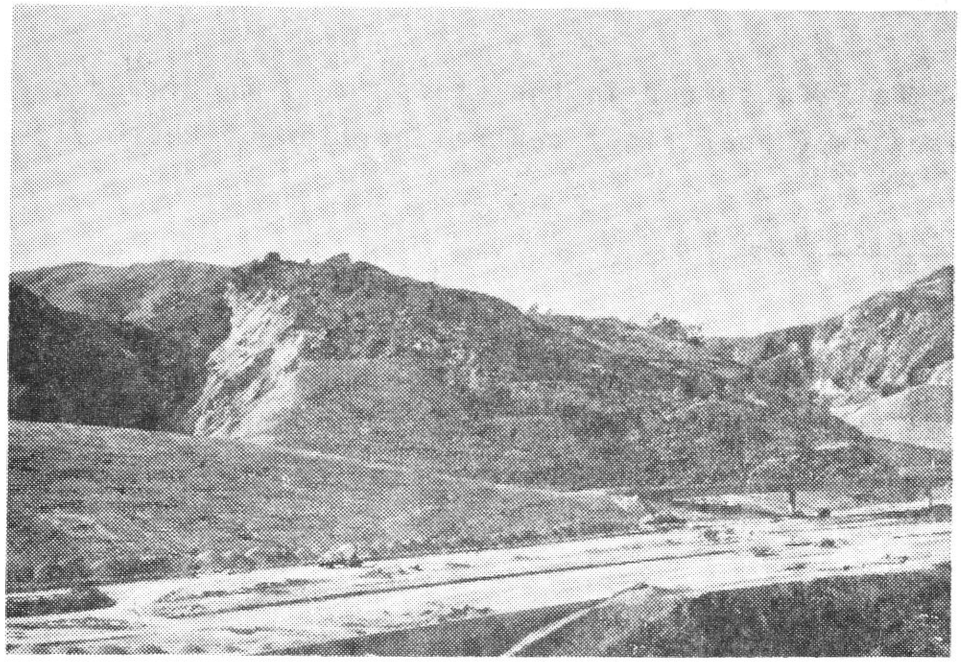


Photo 77. Looking westerly at a cut slope slide immediately south of the Route 5/14 Interchange.

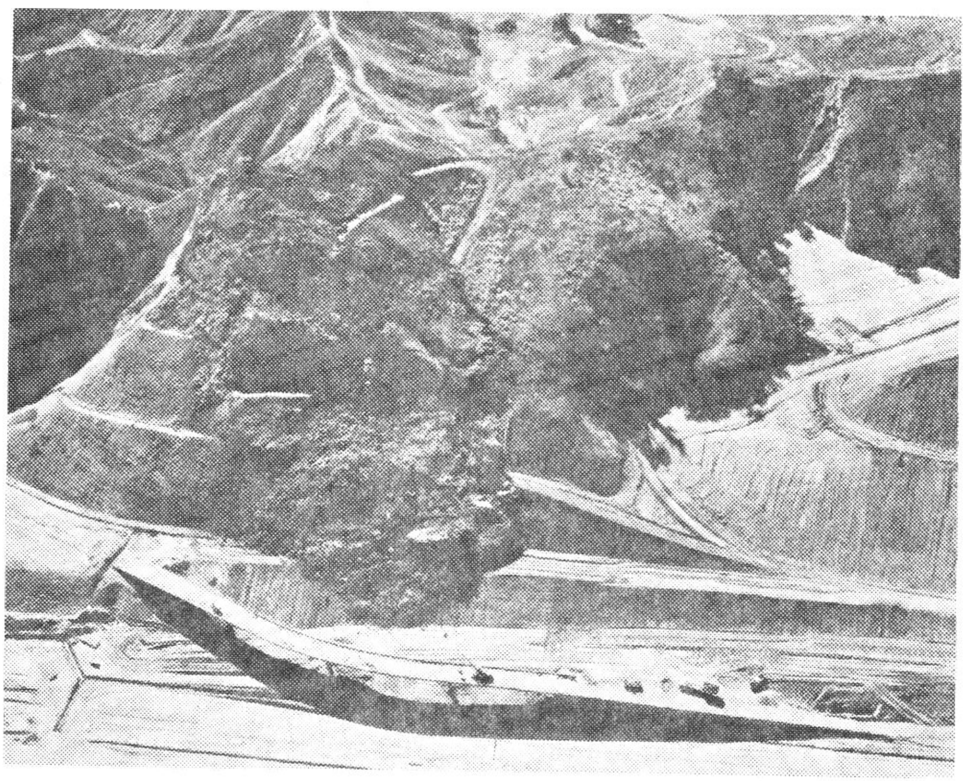


Photo 78. Aerial view of slide shown in Photo 77. Note blockage of unfinished connector by slide debris.

the cut left a nob with a rather high slenderness ratio. The bedding planes of the friable sediments dipped toward the cut face, and the top of the slide was at the apex of the cut and a very steep natural slope facing west. The top of the slide and west-facing slope may be seen in the lower right of Photo 70.

Location 7

A slump landslide that occurred in a cut slope is shown in Photos 69 (upper right) and 79. The slide, which did not encroach upon the roadway, occurred in a clayey silt with a high moisture content.

Photo 79. Looking easterly at a slide in a cut slope, immediately south of the Route 5/14 Interchange.

Embankment Settlement

Embankments in this area settled up to four inches as determined by construction stake checks, and cracking up to two inches wide occurred along what appeared to be cut to fill contacts. Since construction had not reached the paving stage at the time of the earthquake, embankment settlement and cracking were of minor consequence compared to that within paved areas.

ROUTE 5, NORTH OF THE 5/14 INTERCHANGE

Geodetic surveys conducted after the earthquake show that regional warping of the earth's crust extended at least ten miles northerly of the 5/14 Interchange (5). However, damage to Route 5 north of the 5/14 Interchange appeared to be due to seismic shaking and secondary local ground movement, as opposed to damage caused by geologic faulting or upheaval.

Damage to bridge structures included slight shifting of the superstructure and concrete spalling at the hinges of Gavin Canyon Undercrossing. Other bridge damage consisted of minor cracking and spalling of concrete and shifting of bearing seats on two steel girder bridges.

Damage to the roadway was confined to an area approximately two miles north of the interchange and appeared to be the aggravation of an existing problem area. Photo 80 shows the flank of an ancient landslide (slide on left) in the cut face. Earthquake-induced movement of the old slide occurred and resulted in uplifting of the roadway shown at the bottom of the photo. Photo 81 shows a close-up view of the uplifted AC curb of the previous photo.

Further north, a cut slope on the east side of the highway had ground breakage behind the top of cut as shown in Photos 82 and 83.

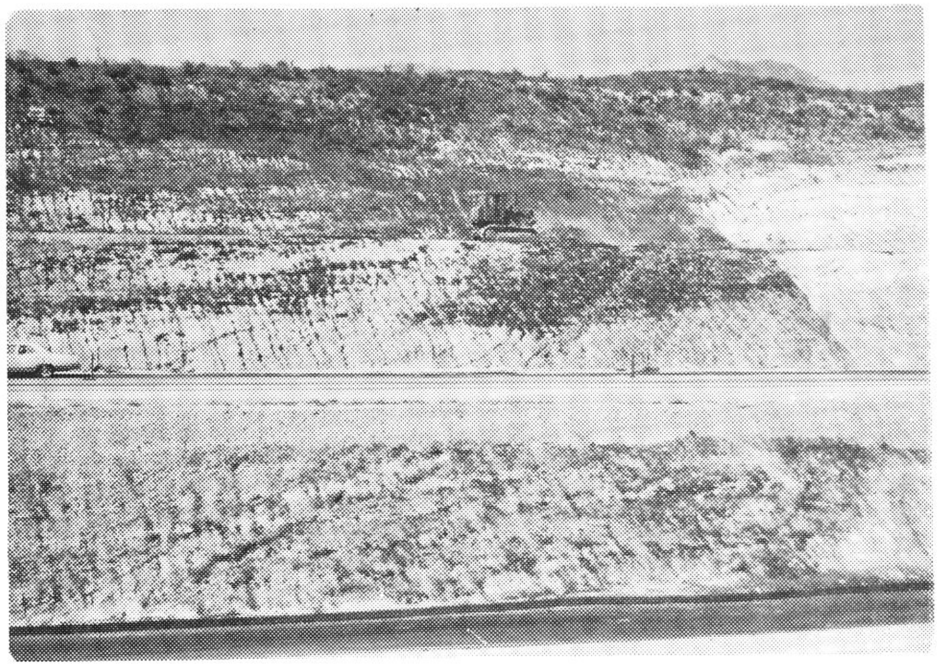
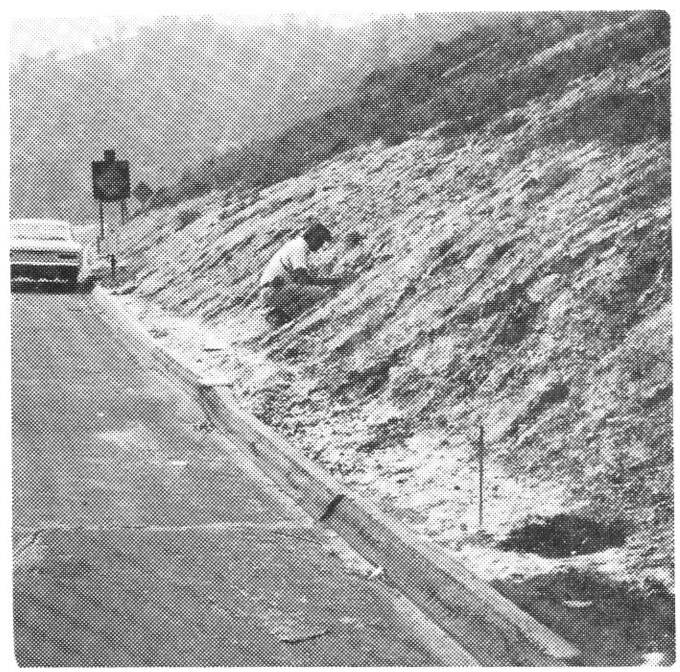


Photo 80. Looking easterly at quake-activated ancient slide in cut slope.

Photo 81. Upthrusting of pavement (Note AC dike) caused by movement of ancient slide.



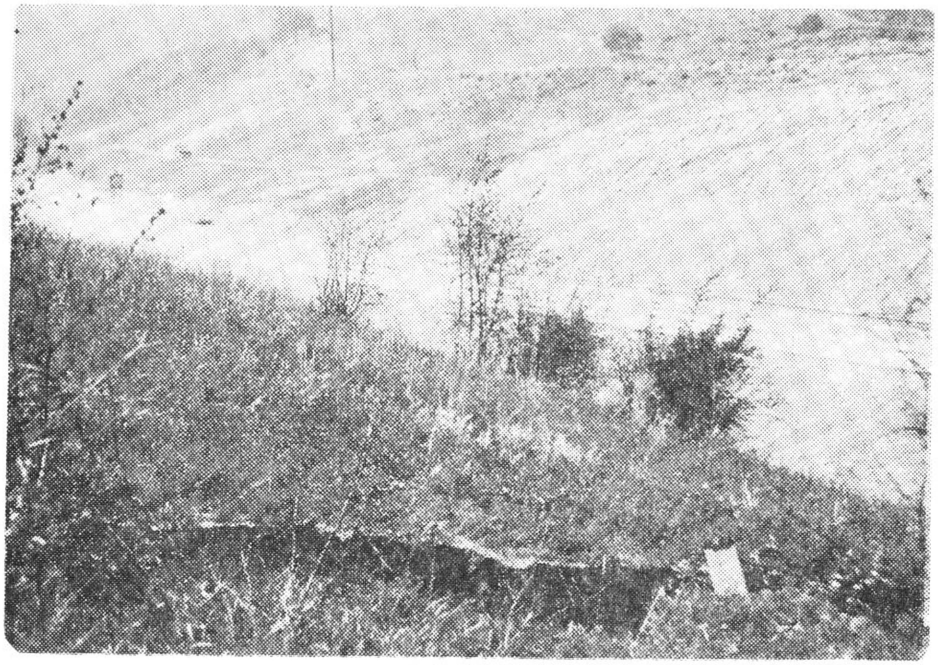


Photo 82. Ground breakage at top of cut slope on east side of Route 5.

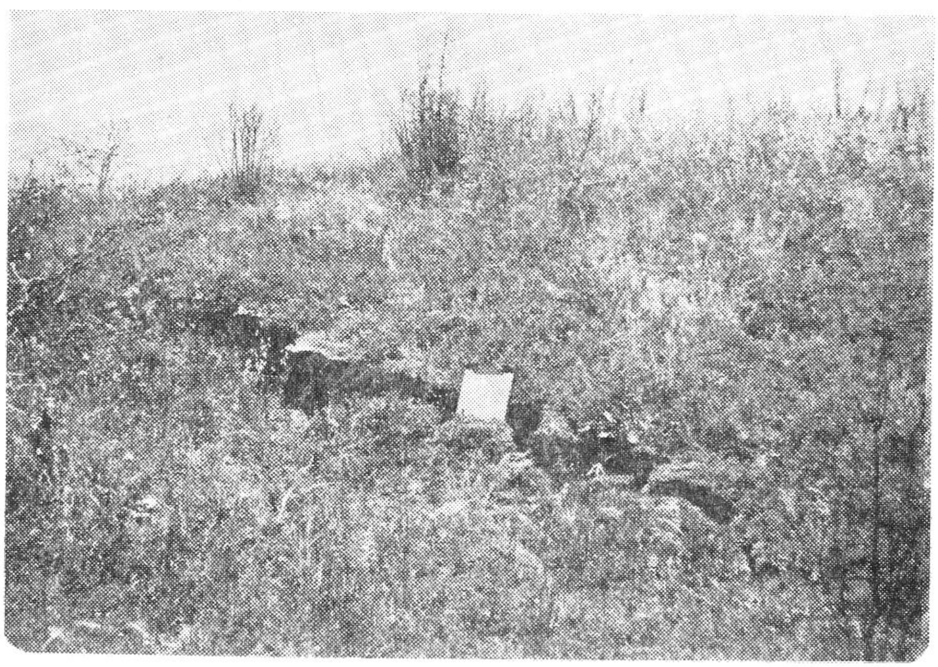
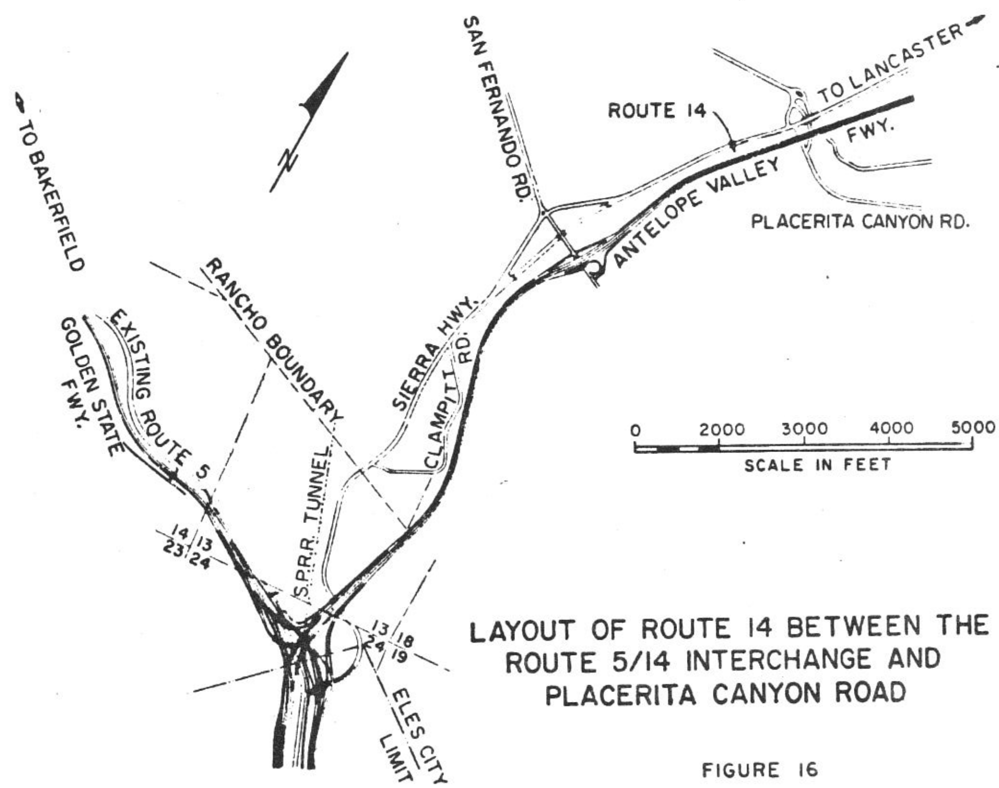


Photo 83. Ground breakage at top of cut slope on east side of Route 5.

ROUTE 14, FROM THE 5/14 INTERCHANGE THROUGH THE SANTA CLARA RIVER CROSSING

At the time of the earthquake, Route 14 was under construction from Route 5, east, through the Santa Clara River Crossing. Figure 16 shows the alignment to Placerita Canyon Road. The Santa Clara River Crossing is approximately four miles northeast of Placerita Canyon. The route is constructed in high cut and fill sections through highly faulted and folded sedimentary rock units that range from well-indurated conglomerates and sandstones to friable and unstable lake bed deposits of mudstones and claystones. Photo 84 shows the contorted lakebed deposits typical of the roadway section from 1-1/2 miles east of Placerita Canyon to the Santa Clara River. Photo 85 shows the scale of the cut and fill construction typical of Route 14.



LAYOUT OF ROUTE 14 BETWEEN THE ROUTE 5/14 INTERCHANGE AND PLACERITA CANYON ROAD

FIGURE 16

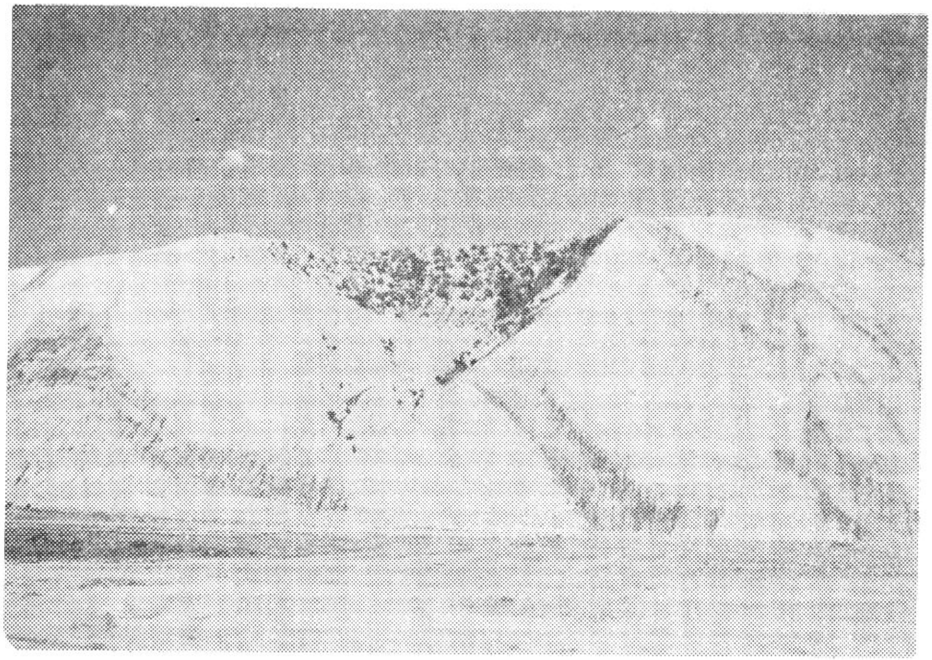


Photo 84. Looking northwesterly at typical contorted geological features east of Placerita Canyon Road.

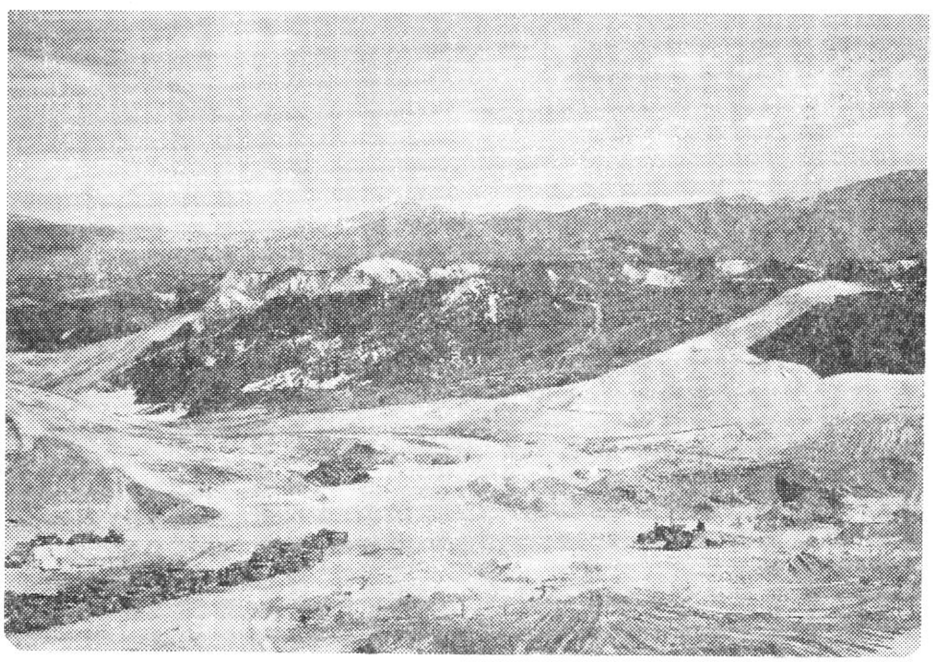


Photo 85. Looking northeasterly at typical scale of earthworks on Route 14.

At the time of the earthquake, the construction stages of the various bridges ranged from forming to complete. Backfill settlement at the abutments, observed at several locations along the route, amounted to an inconvenience only, as the roadway finished grade had not been completed.

Photo 86 shows 8 to 10 inches of backfill settlement that occurred at the west abutment of the westbound structures for the Via Princessa Undercrossing. Photo 87 shows the area to be backfilled at the west abutment of the east-bound structure. Assuming that construction practices were similar for both structures, a comparison of the two photos show the settlement was confined to the backfilled area. It is probable that the settlement resulted from spreading during seismic shaking as the wingwalls were not yet constructed. The abutments of the structures were either sheared or distressed similar to that shown in Photo 88, with the exception of the west abutment of the eastbound structure.

Backfill settlement of 6 to 8 inches at the west abutment of the westbound roadway structure of the Santa Clara River Crossing is shown in Photo 89. The settlement was confined to the backfilled area of the abutment. No displacement of the wingwalls and abutment wall (constructed as a unit) was noted. Photo 90 shows a few inches of backfill settlement at the west abutment of the eastbound structure. The abutment and wingwalls were constructed as a unit and no bridge damage was detected. Again, the settlement appears to be confined to the backfilled area.

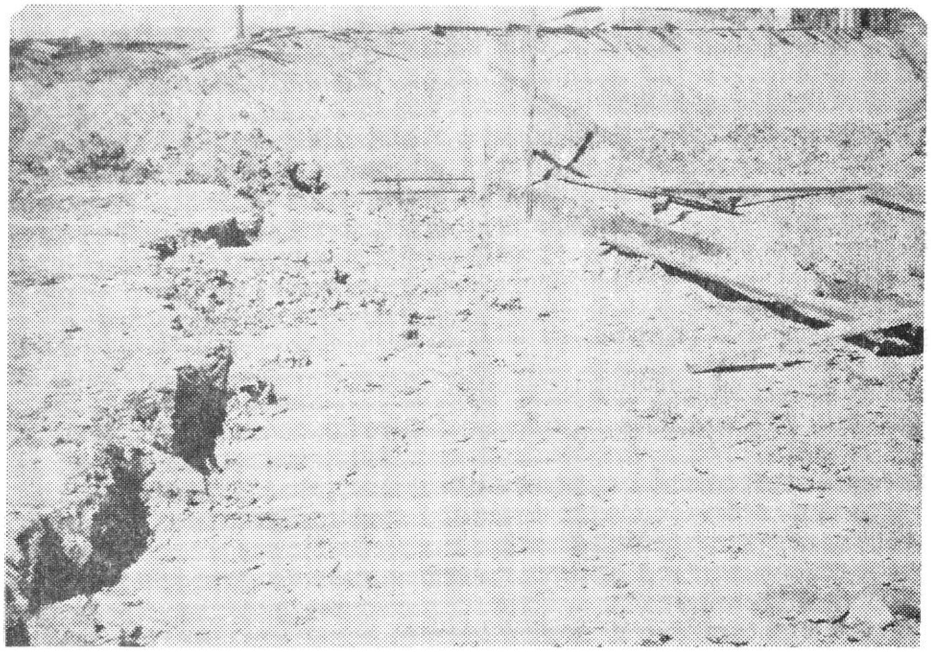


Photo 86. Abutment backfill settlement at the Via Princessa Undercrossing. Note large crack which delineates limit of backfill.

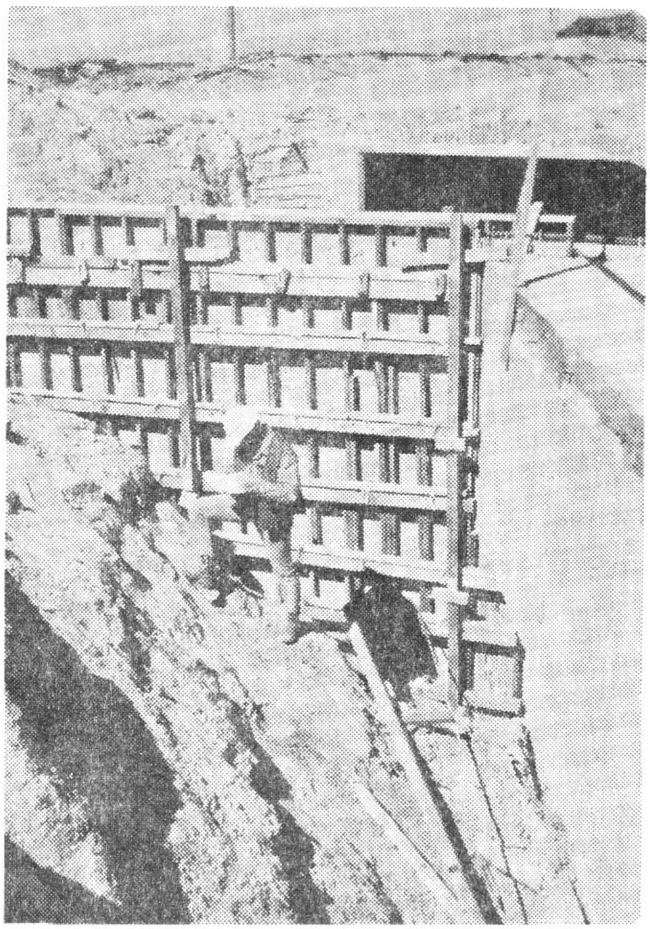
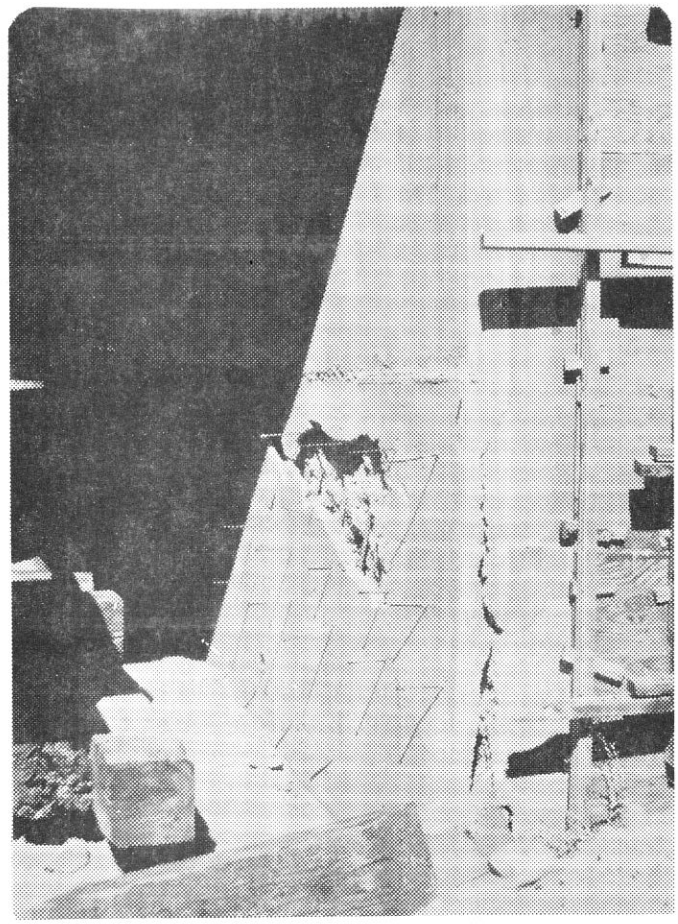


Photo 87. Abutment prior to backfilling at the Via Princessa Undercrossing.

Photo 88. Typical abutment damage through the eastern portion of Route 14.



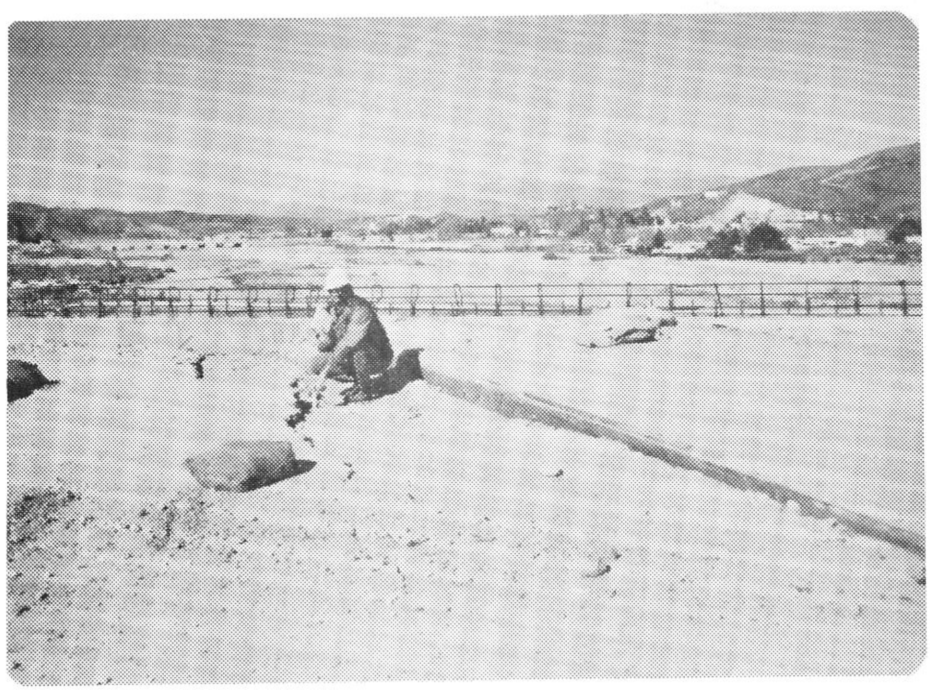


Photo 89. Abutment backfill settlement at the Santa Clara River Bridge (west abutment, westbound structure).

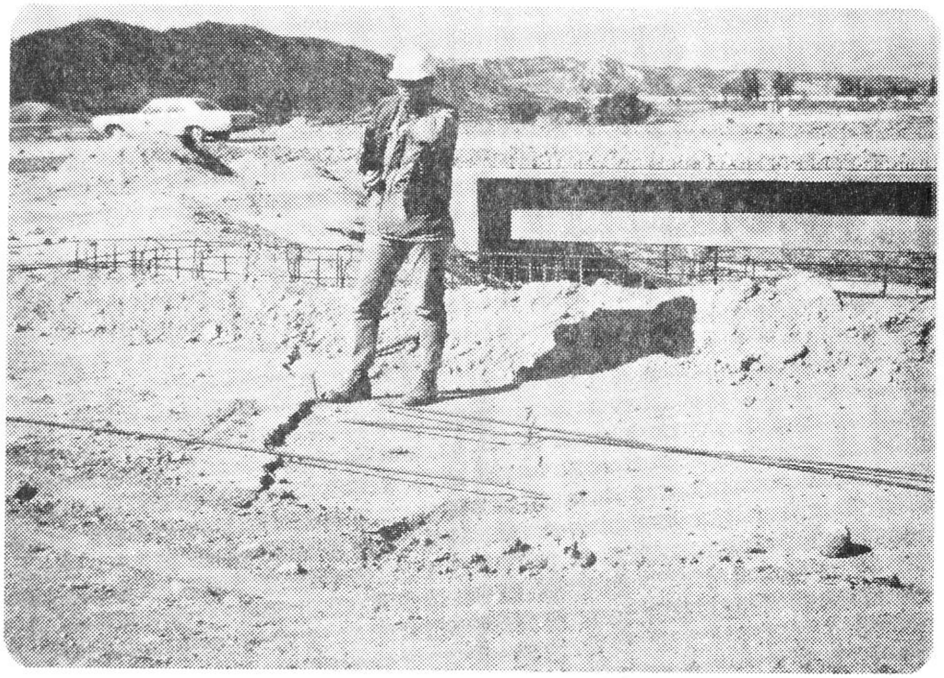


Photo 90. Abutment backfill settlement at the Santa Clara River Bridge (west abutment, eastbound structure).

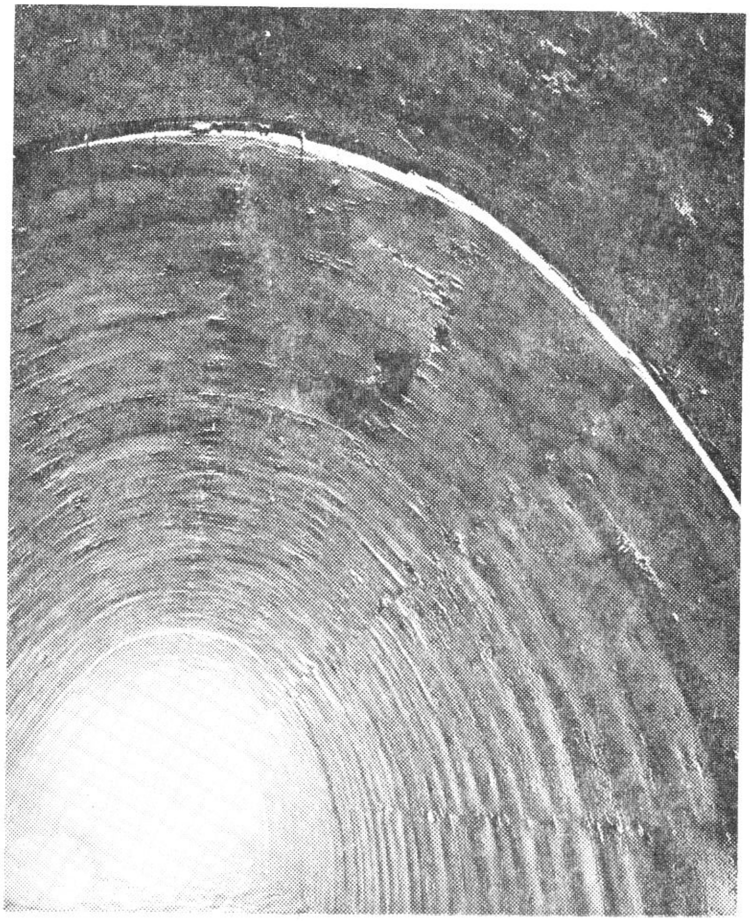


Photo 91. Minor spalling and joint separation in an 11-foot concrete arch culvert.

The above descriptions are typical of the distress encountered at other structures along Route 14. Structures west of Placerita Canyon Overcrossing to the 5/14 Interchange area appeared to be undamaged and only minor amounts of backfill settlement were noted at some structures.

An 11-foot span reinforced concrete arch culvert near the Via Princessa Overcrossing was completed, backfilled, and had roadway embankment placed over it to specified grade. Plans specified that backfill around the arch include baled straw, one bale thick, for those sections under embankment heights greater than 30 feet above the crown. Maximum fill height over the crown was about 55 feet for a 150-foot length of the culvert, and at least 30 feet of fill height for a 250-foot length of the culvert. Damage to the culvert beneath 30 feet or more of fill consisted of expansion joint separation of ±1 inch; longitudinal cracking in the upper one-half of the culvert ranged from hairline to about 1/4-inch; and one section of the top spalled, as shown in Photo 91.

Slight cracking was noted in other portions of the culvert but it appeared to be very minor. The effect of the baled straw over the culvert for the higher fill-covered areas may have been beneficial in cushioning the culvert during earthquake-induced ground movements.

A 72-inch diameter reinforced concrete pipe crossing under Route 14 just east of the Sierra Highway Undercrossing had hairline to 1/8-inch longitudinal cracks in the upper 1/3 of the pipe on the east wall for nearly the length of the pipe and some discontinuous cracking in the top. No distortion of the invert was noted and the pipe appeared to be in good condition, overall.

Other drainage conduits investigated under Route 14 included a 10-foot wide x 8-foot high double reinforced concrete box and a 14-foot span reinforced concrete arch. No visible damage to either structure was detected.

ROUTE 210 FROM ROUTE 5/210 INTERCHANGE THROUGH MACLAY STREET

At the time of the earthquake, Route 210 was open to traffic from the Route 5/210 Interchange on the west, to Maclay Street on the east. Embankment was placed from Maclay Street easterly for approximately 7000 feet as staged construction. Construction had not been started on the proposed Route 118/210 Interchange.

The completed portion of Route 210 and the surrounding topography are shown in Photo 92. Profiles of original ground, roadway design gradeline, and a post-earthquake change in elevation profile of the roadway are shown in Figure 17. As-built plans were used in the compilation of Figure 17 for the roadway and original ground profiles. Post-earthquake elevation surveys of the roadway were compared to the as-built profile and the change in elevation for each station is plotted on the figure.

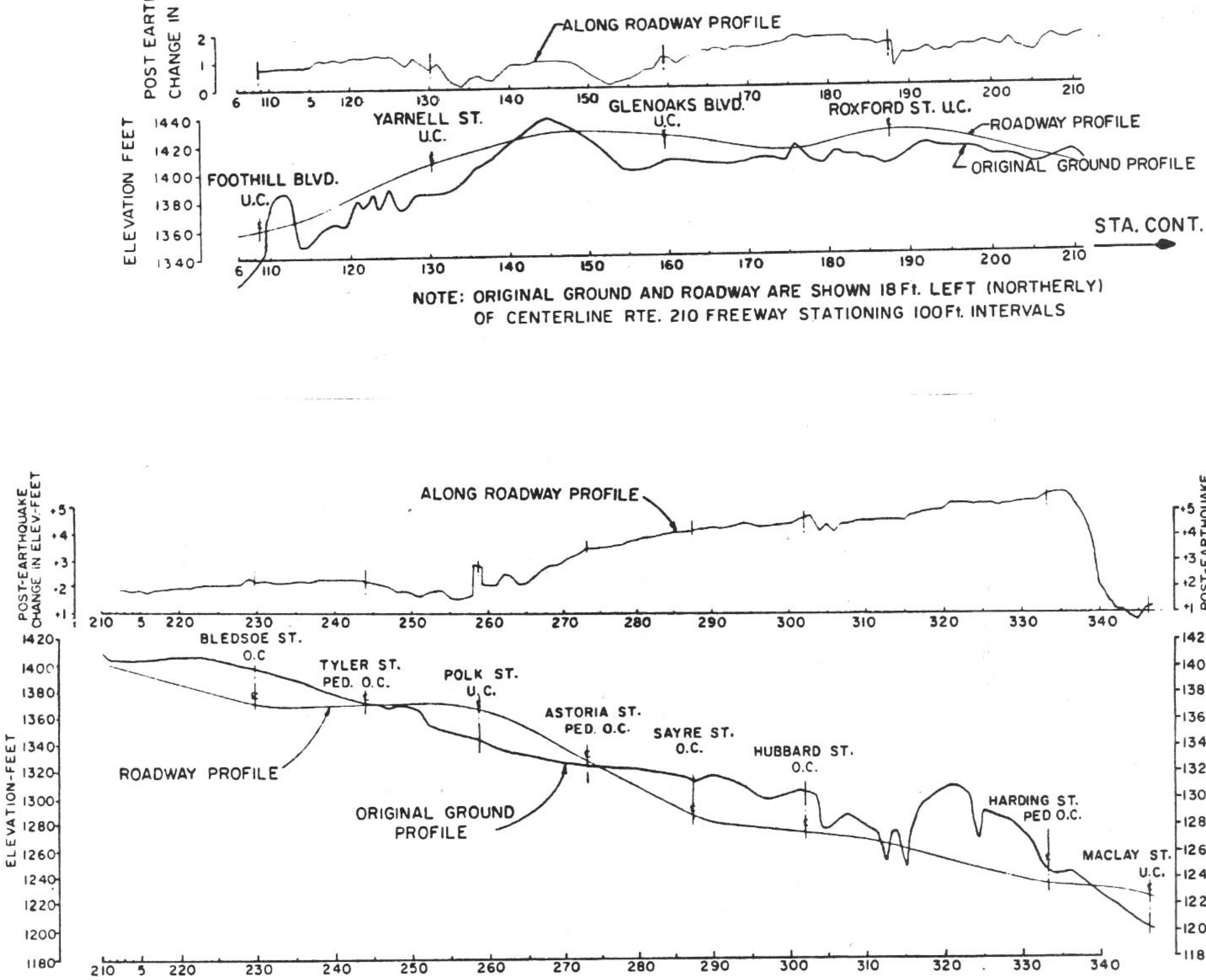
The general trend of the post-earthquake survey shows that a regional tilt of the ground surface occurred, sloping upward to the east as measured along the Route 210 alignment. Maximum bedrock uplift was 5.6 feet at the Sylmar Fault (Freeway Station 335) and the minimum bedrock uplift was about 0.75 feet at the Foothill Boulevard Undercrossing (Freeway Station 109).

Damage description location numbers are shown on Figure 18.

Location 1

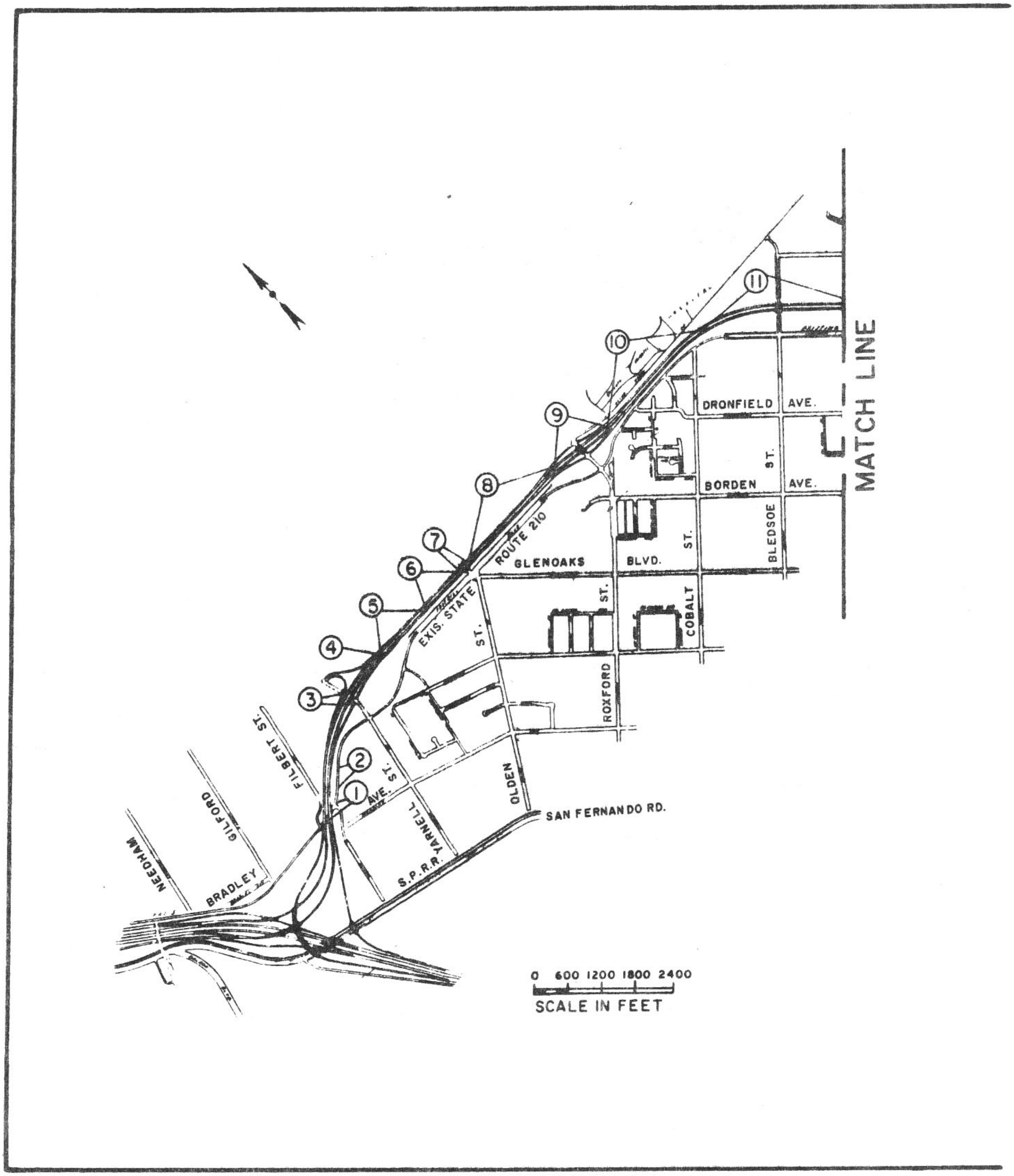
In addition to the structural damage to the Foothill Undercrossing (Photo 93), embankment distress in the form of shear cracks and settlement occurred at the approaches. Backfill settlement measuring up to one foot resulted in approach slab breakage.

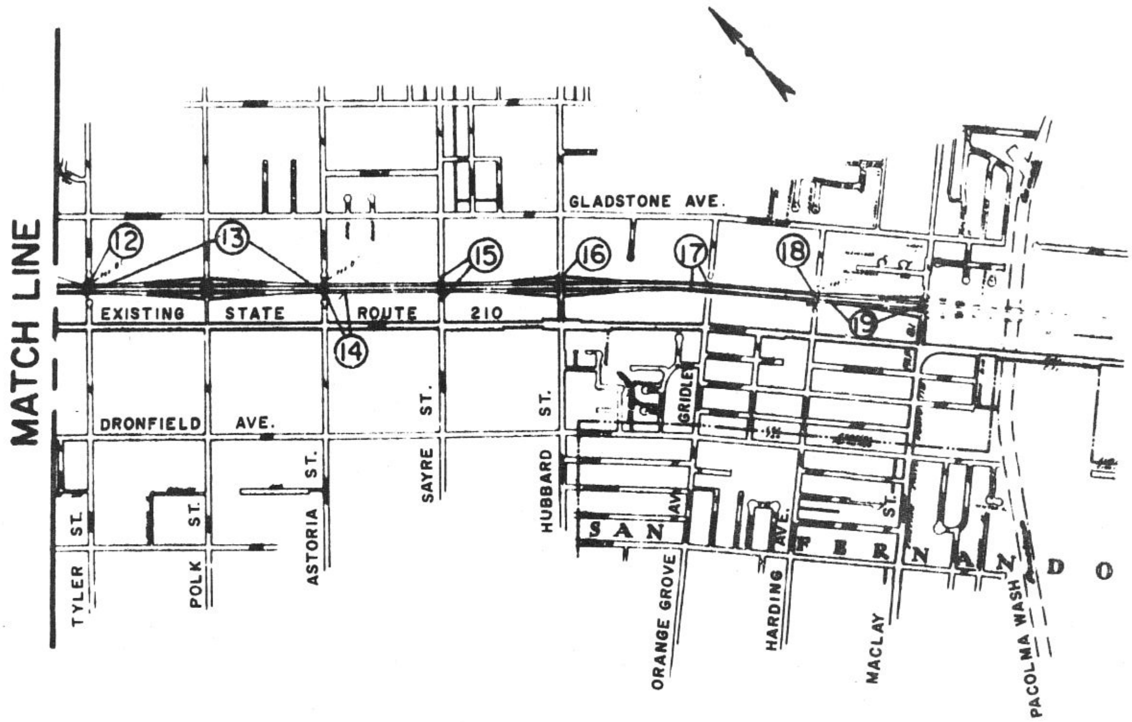
Photo 92. Looking easterly at the completed portion of Route 210. The Juvenile Hall complex is in right center of photo.



ROUTE 210 DESIGN AND ORIGINAL GROUND PROFILE AND POST QUAKE CHANGE IN DESIGN PROFILE

FIGURE 17


MATCH LINE
DRONFIELD AVE.
BORDEN AVE.
BLEDSOE ST.
ST.
ROUTE 210
EXIS. STATE
GLENOAKS BLVD.
ST.
COBALT
ROXFORD ST.
OLDEN ST.
YARNELL ST.
AVE.
FILBERT ST.
GILFORD
NEEDHAM
BRADLEY
S.P.R.R.
SAN FERNANDO RD.
0 600 1200 1800 2400
SCALE IN FEET
1
2
3
4
5
6
7
8
9
10
11



LAYOUT OF THE COMPLETED PORTION OF ROUTE 210

FIGURE 18

Photo 93. Looking northerly at the Foothill Boulevard Undercrossing.

Location 2

Pavement slab separations of 1/4 to 1/2-inch occurred along expansion joints in the outside lanes of the east-bound roadway for a distance of 650 feet east of Foothill Boulevard. Minor undulations in the roadway profile between Foothill Boulevard and Yarnell Street resulted from fill and/or foundation densification.

Location 3

Fills at the Yarnell Street Undercrossing (Photo 94) settled about three inches at the west approach (Photos 95 and 96) and about six inches at the east approach. Photo 97 shows cracking and settlement in the median at the wing wall. Again, settlement appears to be the combined result of densification of fill and underlying aluvium caused by the seismic shaking.

Both bridges are single spans with abutments supported on piles. Piles extended through the fill to about 25 feet below the original ground surface and were founded in very dense sand and sandy gravel. Inspection of the bridge log of test borings shows about a four-fold increase in energy required to penetrate the foundation material at the pile-tip elevation as opposed to the uppermost ten to fifteen feet of material. Overburden material continuously increased in density from the original ground to the pile-tip foundation material.

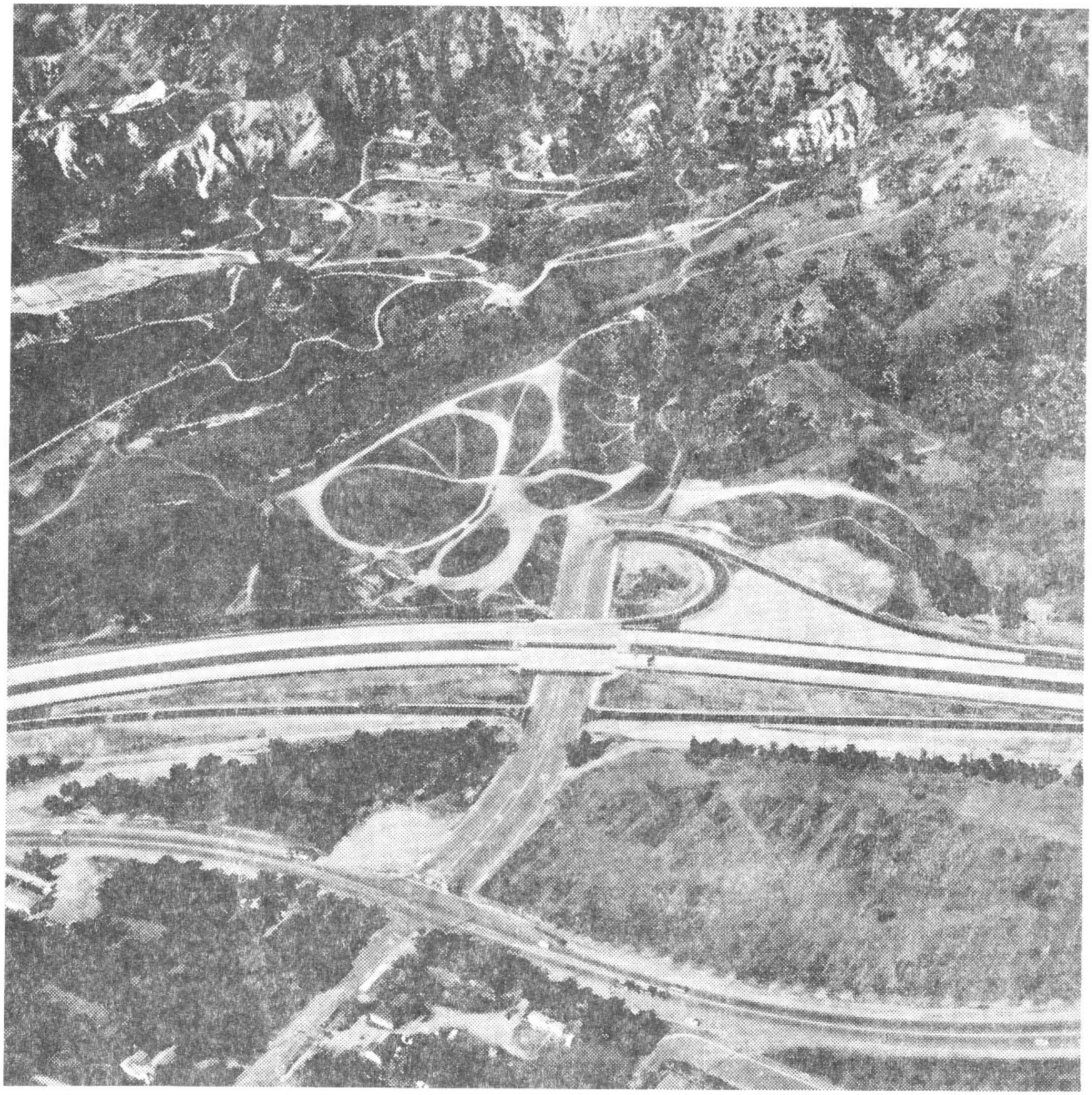


Photo 94. Looking northerly at the Yarnell Street Undercrossing.

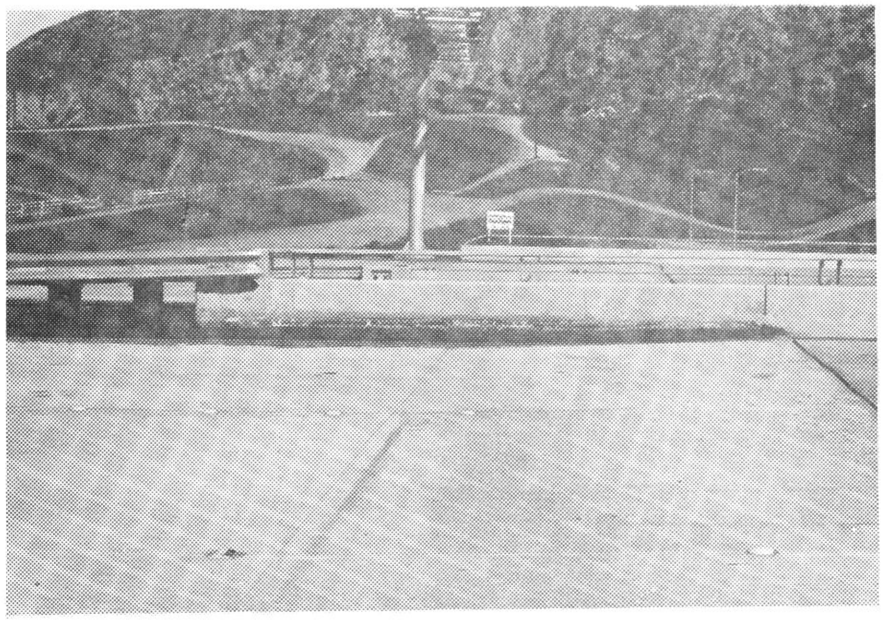


Photo 95. Fill and abutment backfill settlement at the Yarnell Street Undercrossing, eastbound roadway, west approach. Note ramp effect of approach slab to structure deck.

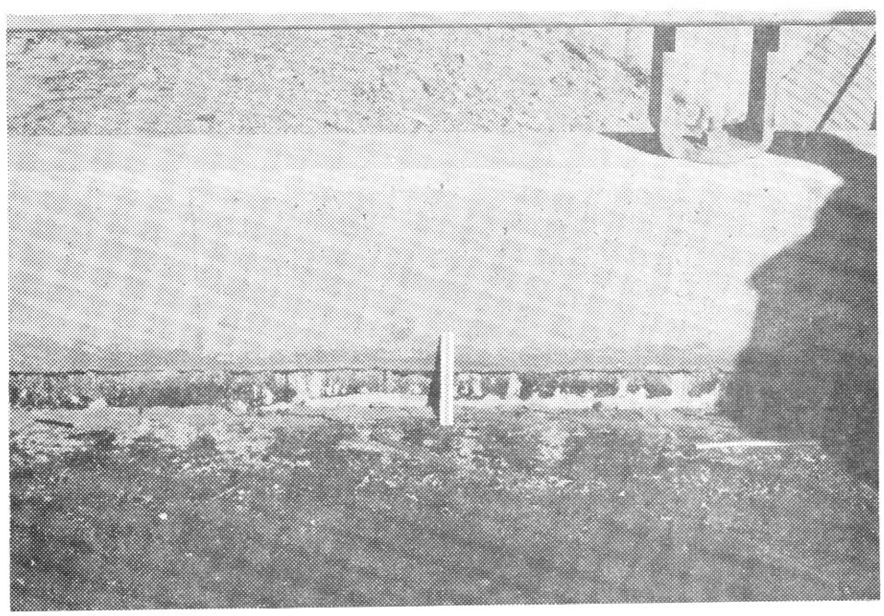


Photo 96. Same location as Photo 95. Note settlement of AC shoulder pavement relative to wingwall.

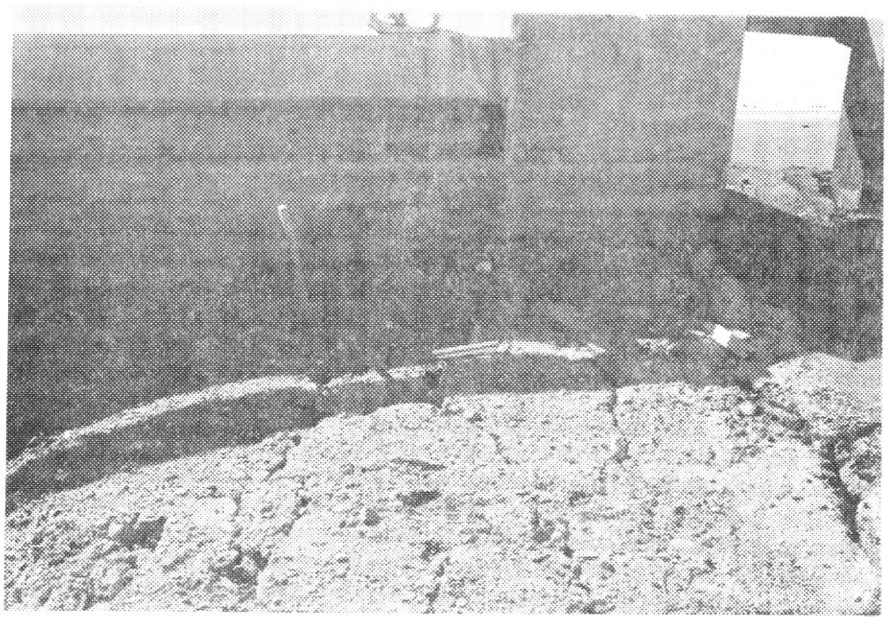


Photo 97. Fill cracking and settlement in median at Yarnell Street Undercrossing.

Location 4

Pavement slab separation of the longitudinal construction joint of 1/2-inch to 1-1/4-inches occurred in the westbound roadway for a length of 560 feet at this location. Fill height through this section varied from 12 feet at the east limit to 24 feet at the west limit as shown in Figure 17.

Location 5

Pavement undulations through this location appear to reflect the cut/fill profile and varying fill thickness.

Location 6

The asphalt concrete paved median ditch at this location had a longitudinal crack along the flowline for a distance of about 850 feet. The ditch pavement had collapsed in places for a width of up to 1/2-foot. This section of roadway is on fill that is about 20 feet high at the freeway centerline. The roadway starts into cut section about 150 feet west of the median tension crack. The portland cement concrete pavement in both roadways showed separation along the longitudinal construction joints ranging from 1/4-inch to more than one inch.

Location 7

In the Glenoaks Boulevard area (Photo 98), Route 210 is constructed on about 20 feet of fill over alluvium that is made up of gravelly sands. A comparison of the constructed profile vs. the post-earthquake profile (Figure 17) shows that slightly more than three inches of fill settlement occurred at the west bridge approaches and about 4-1/2 inches of fill settlement occurred at the east approaches. Both bridges are set on piles that are founded about 30 feet below original ground.

No structural damage to the bridges was found (4), but concrete slope paving beneath the superstructures caused minor damage to the concrete sidewalks and curbs along Glenoaks Boulevard (Photo 99).

The slope paving, which settled about 3 inches at the bridge abutment and about 2 inches at the sidewalk, transmitted forces from the abutments to the sidewalk and curbs.

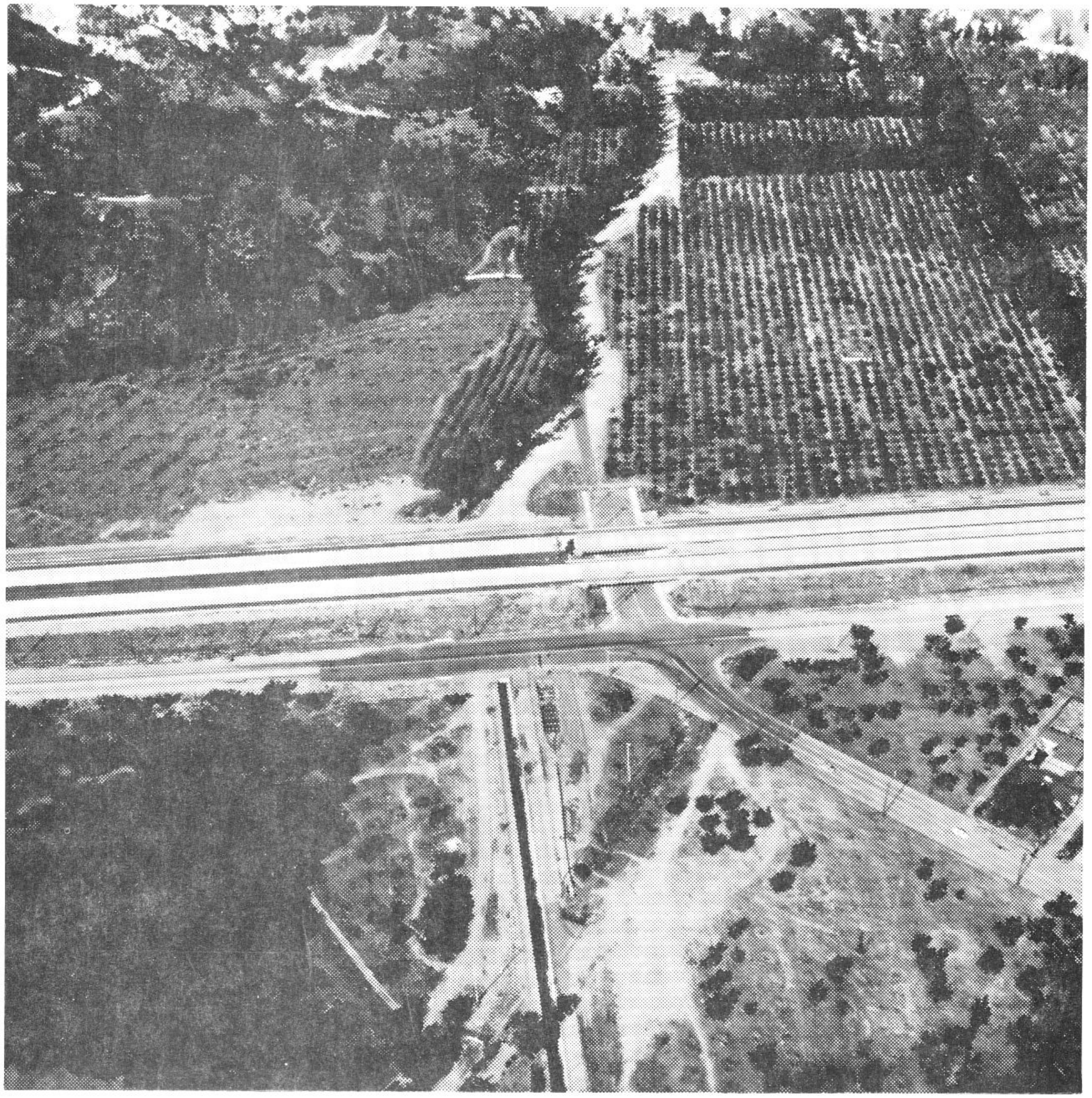


Photo 98. Looking northerly at the Glenoaks Boulevard Undercrossing.

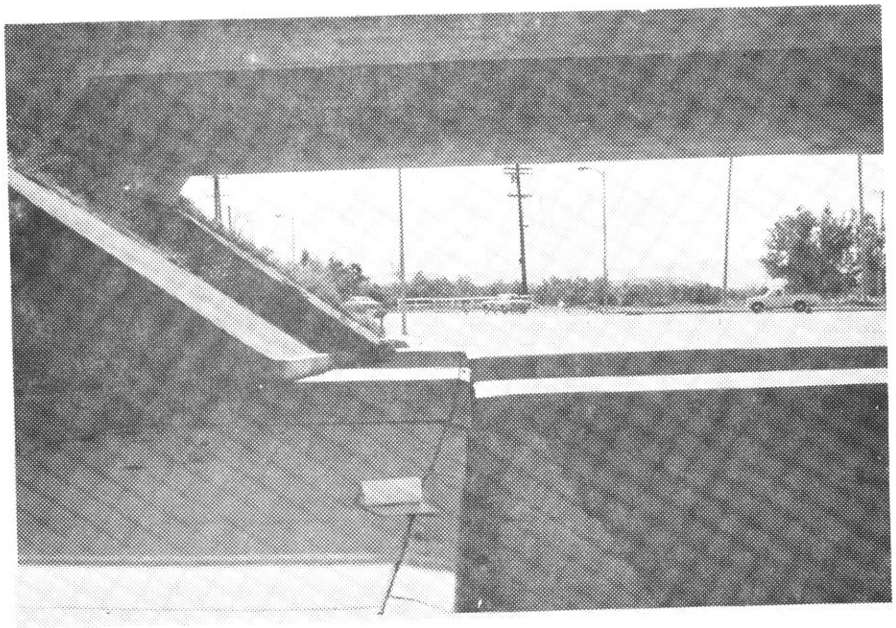


Photo 99. Looking south along Glenoaks Boulevard. Note rotated and displaced curb due to lateral force of displaced slope paving that was transmitted through sidewalk.

Photo 100. Looking west across Glenoaks Boulevard at tension cracks in AC pavement caused by differential settlement.

The asphalt concrete pavement of Glenoaks Boulevard settled differentially where it crosses beneath the freeway, resulting in transverse tension cracks as shown in Photos 100 and 101. The cracks are nearly coincident with the freeway toe of fill line. The break in pavement profile appears to be nearly over the southerly edge of a buried 11-foot wide by

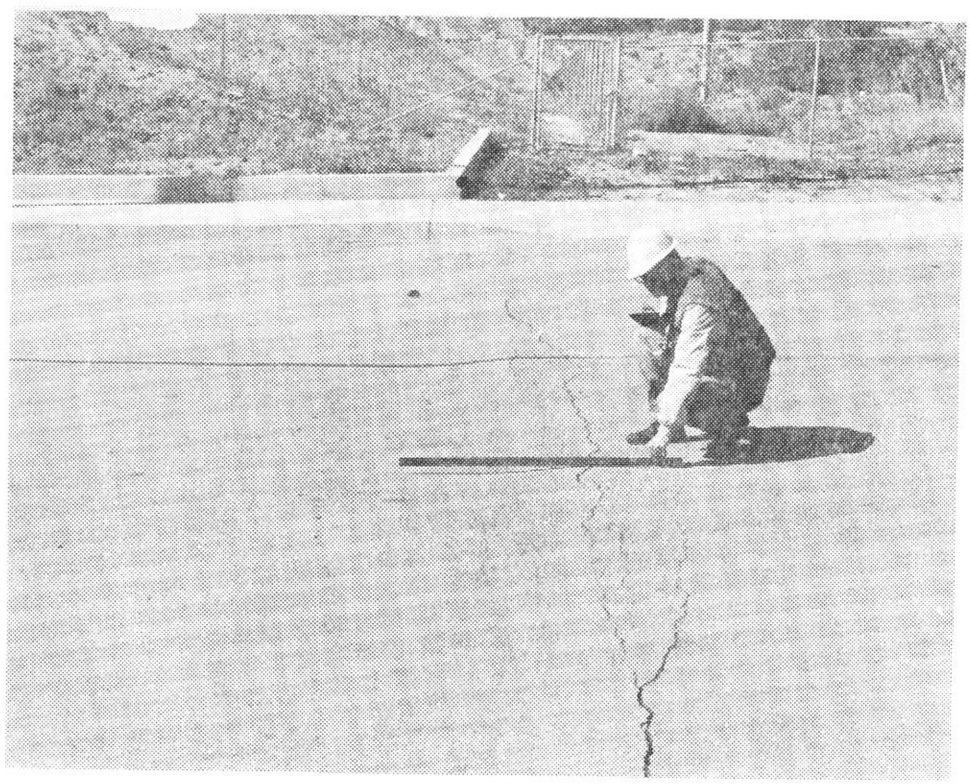


Photo 101. Looking west across Glenoaks Boulevard at tension cracks in AC pavement caused by differential settlement.

8-foot high reinforced concrete box storm drain covered by about 3 feet of earth. The storm drain begins to curve near the centerline of Glenoaks Boulevard and crosses the centerline of Route 210 (beneath fill) about 150 to the west. No break in the pavement profile could be discerned along the north edge of the box.

Location 8

An undulating roadway surface, shown in Figure 17, was the result of tectonic uplift combined with differential settlement in the roadway embankment and underlying gravelly sands.

Location 9

Seismic shaking at the I-210/Roxford Street Interchange (Photo 102) appears to have been much more intense than that to the west for a distance of about one mile along the freeway alignment. The east abutment for the eastbound structure separated from the pilings and the structure shifted about three feet to the south as shown in Photo 103. Fill spreading and settlement disrupted the roadway profile (Photos 104 and 105). Pavement slab separation and longitudinal cracks resulted from fill displacement (Photos 106, 107, 108, and 109). A compression buckle formed in asphalt concrete pavement in the eastbound off-ramp (Photos 105 and 110). Lateral shear displacement of fill is evidenced by bulging and a developed thrust offset near the toe of fill at the west approach to the eastbound bridge. Bulging occurred at the toe of fill along the eastbound off-ramp for a distance of several hundred feet.

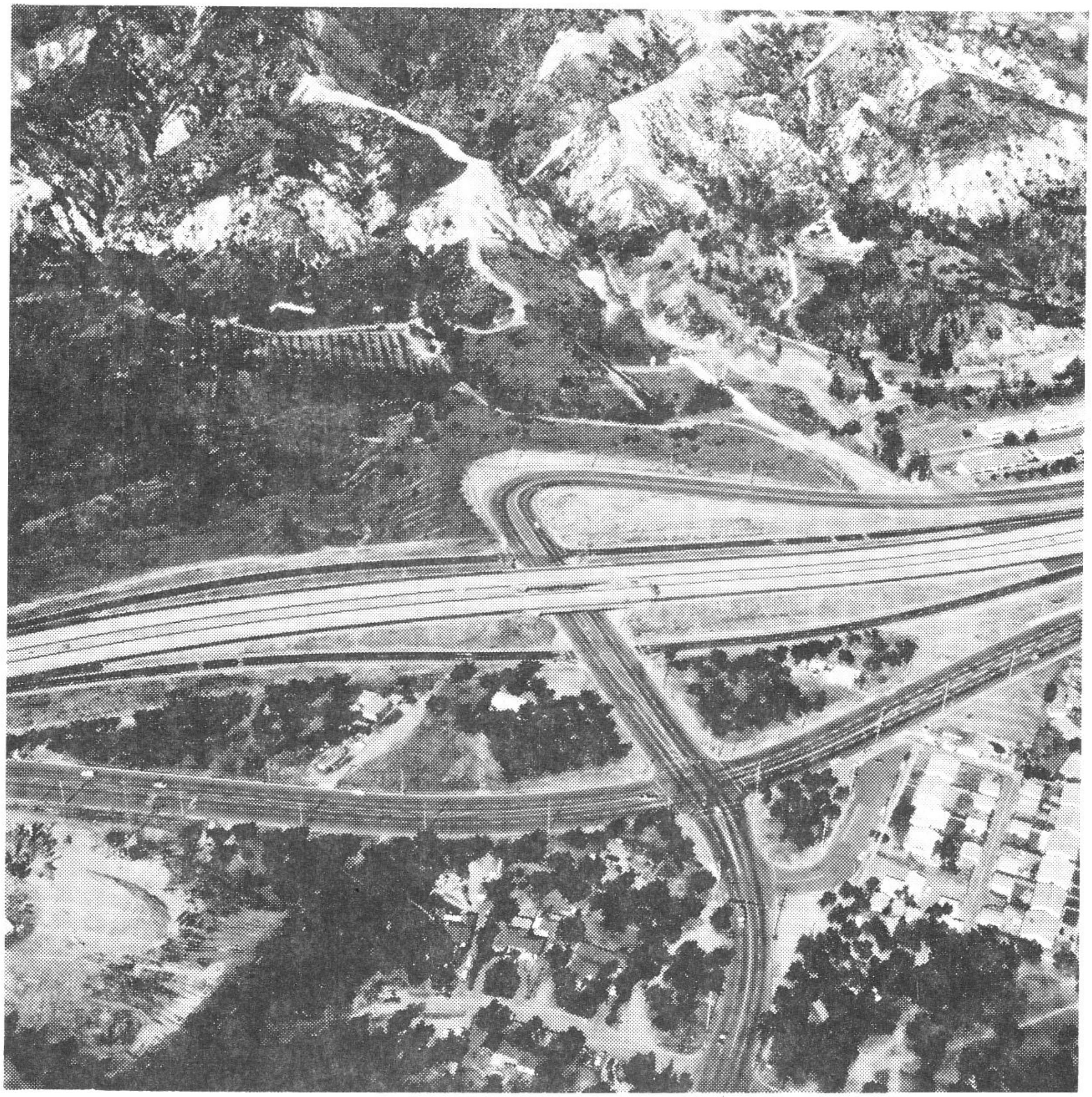


Photo 102. Looking northerly at the Roxford Street Undercrossing.

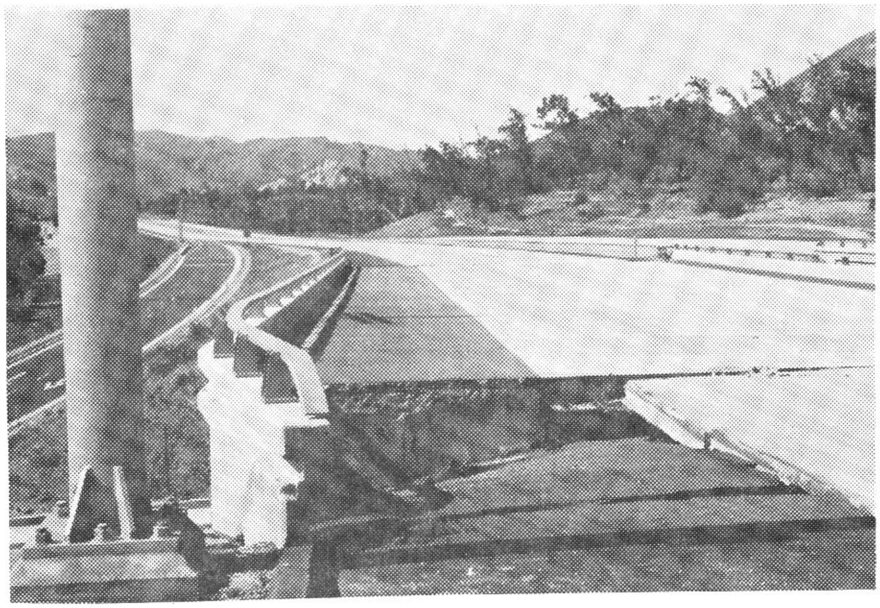


Photo 103. Looking west at the eastbound structure of the Roxford Street Undercrossing. Note rotational movement of structure.

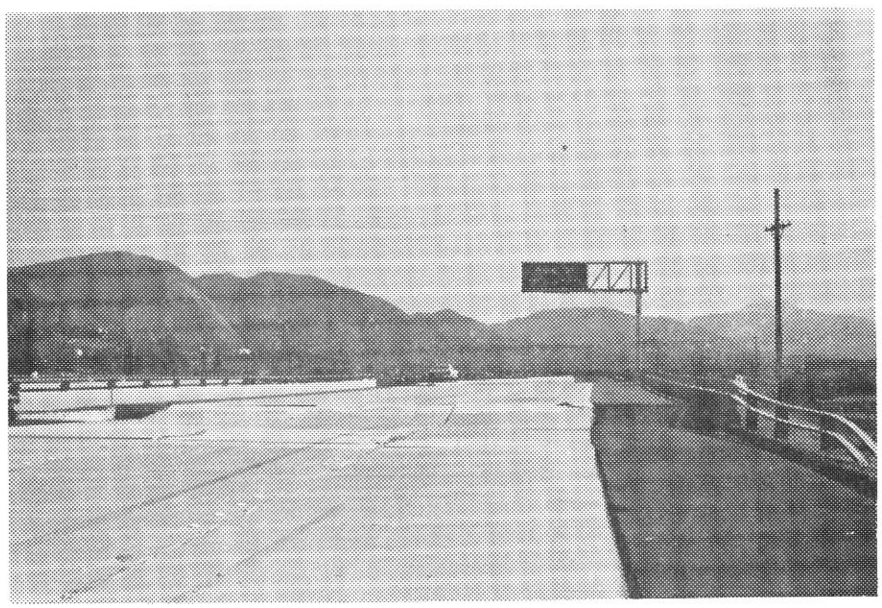


Photo 104. Looking east at the eastbound structure of the Roxford Street Undercrossing. Note settlement of pavement relative to structure deck. Note also separation of pavement slabs and the broken approach slab. The tire skid marks leave little doubt as to the whereabouts of one motorist at the time of the earthquake.

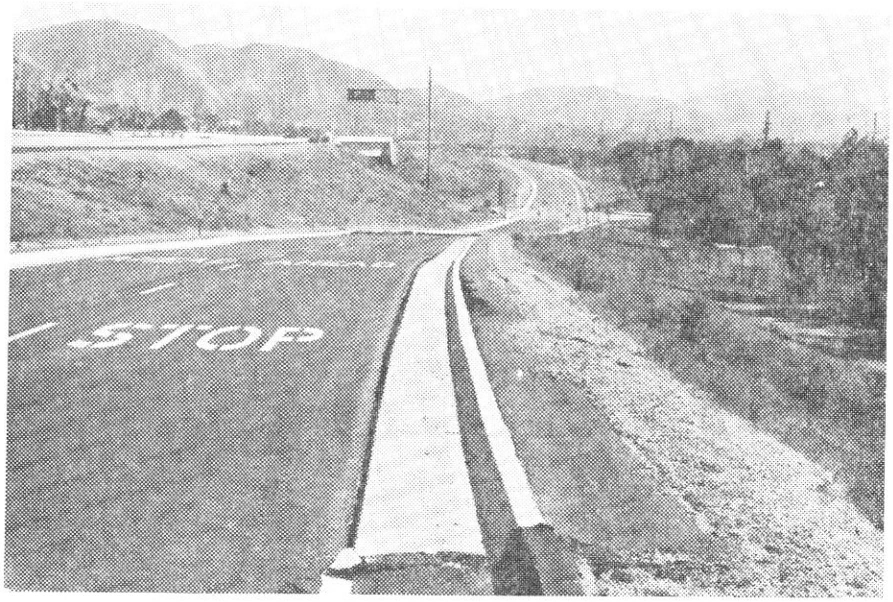


Photo 105. Eastbound off ramp at the Route 210/Roxford Street Interchange. Note disruptions in pavement profile and alignment.

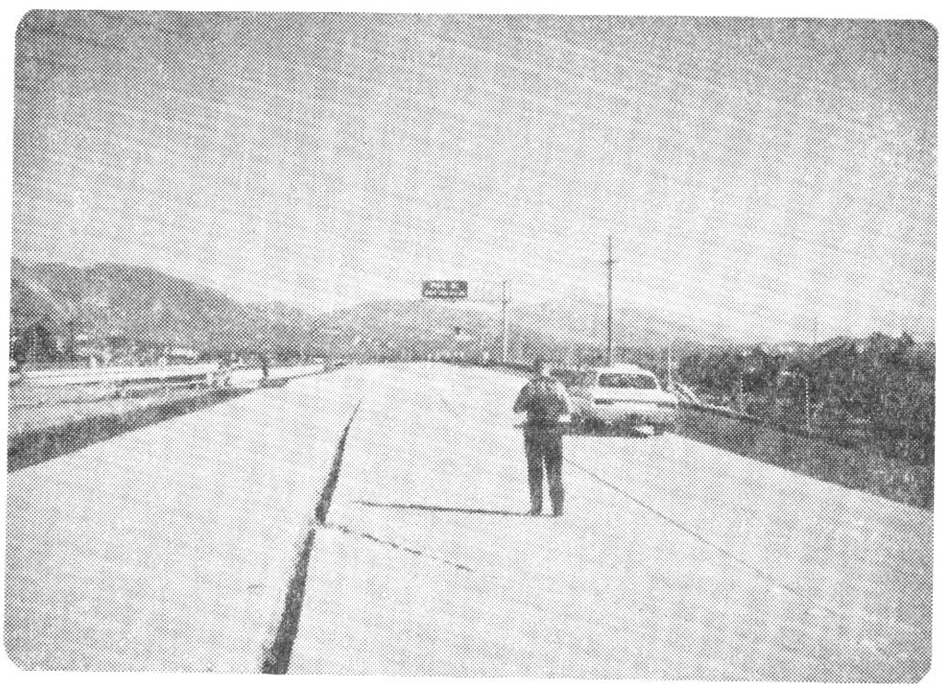


Photo 106. Looking east at Roxford Street Undercrossing. Note pavement separation at longitudinal joint and diagonal crack caused by fill slippage and differential settlement.

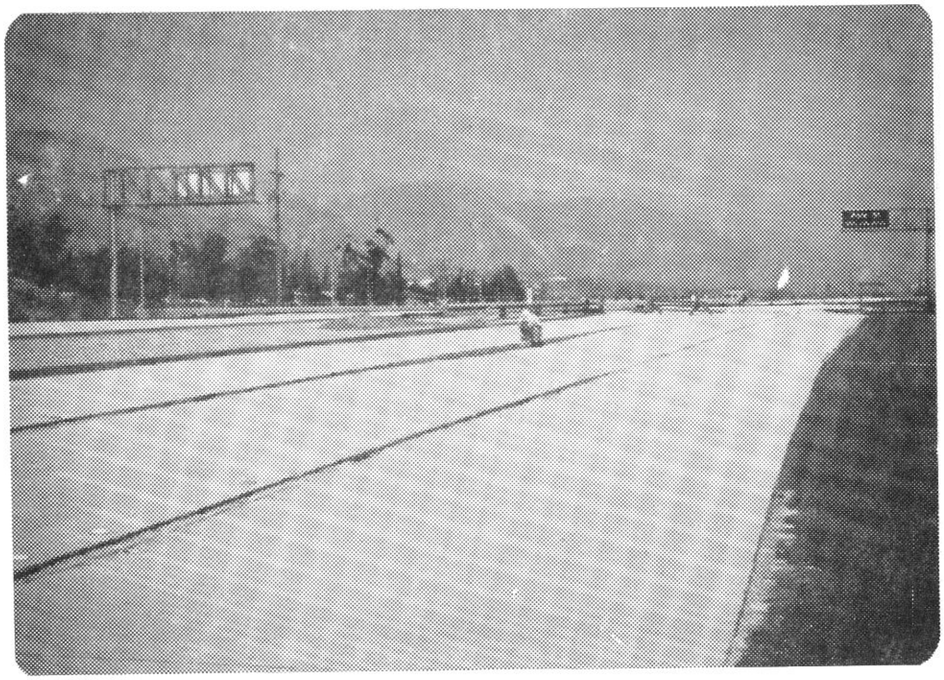


Photo 107. Same location as Photo 106. Note fualting of pavement at longitudinal joint.

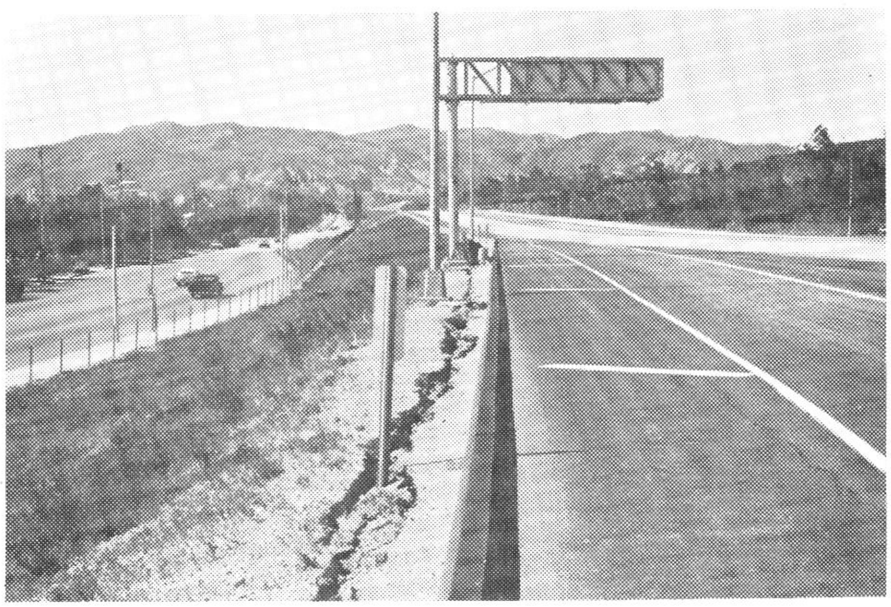


Photo 108. Looking west at eastbound off ramp to Roxford Street. Note transverse pavement cracks and wide longitudinal crack between fill hinge point and AC dike.

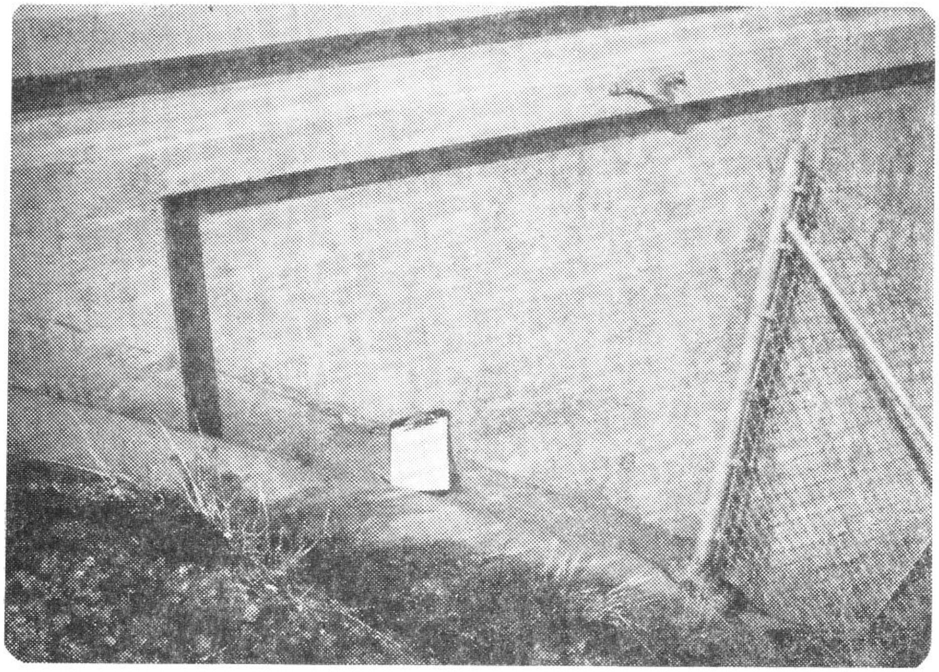


Photo 109. Wingwall at Roxford Street Undercrossing. Note fill settlement relative to structure as delineated by staining on wall.

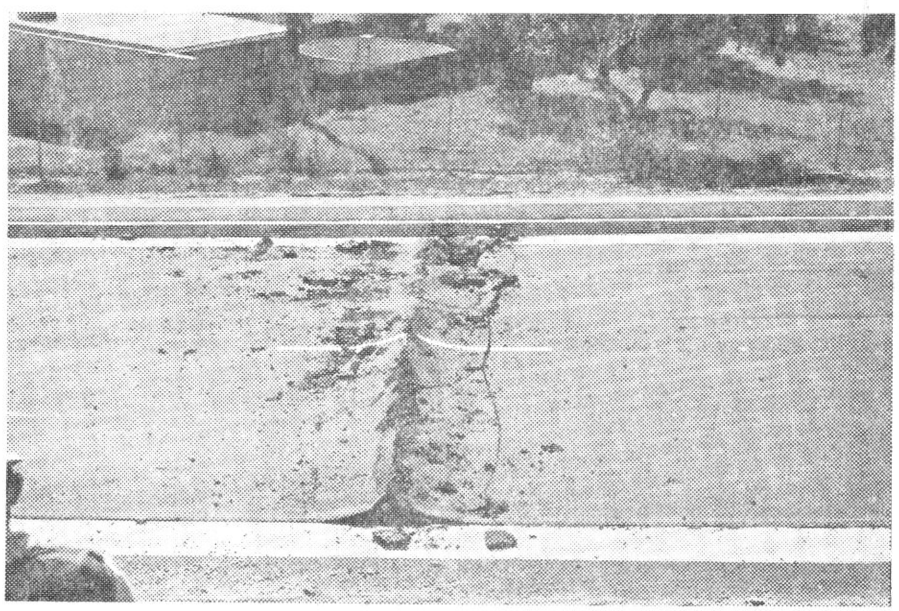


Photo 110. Compression buckle in AC pavement of eastbound off ramp to Roxford Street.

An 18-inch diameter corrugated metal pipe vertical down drain through the fill median east of the undercrossing structures was compressed as shown in Photo 111. The compression wrinkling occurred about 9 feet below the roadway surface, which is the approximate depth to original ground. The pipe empties into the top of a 7-ft wide by 6-1/2-ft high reinforced concrete box (Niansfield Street Channel), which crosses the freeway on a skew below the original ground elevation.

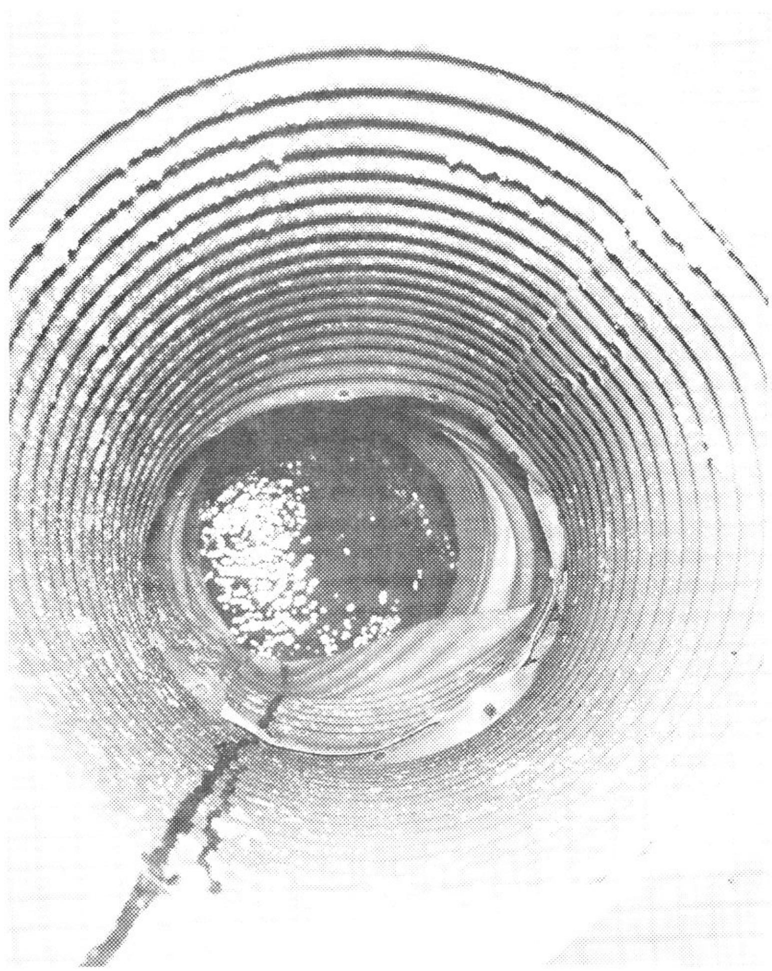


Photo 111. Compression wrinkling of vertical CMP downdrain caused by shearing forces during fill settlement.

A number of small ground-breaks extended more than 1000 feet on each side of the freeway in an east-northeast trend. They consisted of tension cracks opened up to three inches; small faults, with up to four inches of vertical displacement; and compression features in the pavement, concrete curb and gutters, and a concrete channel. Building damage in this trend was severe enough that some houses were posted "unsafe". Older buildings to the west of the Olive View Hospital were either destroyed or severely damaged. It is believed that subsurface faulting may have occurred in an east-northeast trend through the I-210/Roxford Street Interchange area. Observed field evidence for extending this trend to the southwest and northeast exists but is beyond the scope of this report.

Location 10

Progressing from west to east (left to right on Photo 112), the freeway is constructed in sidehill cut-fill to a full cut-section (right 1/3 of the photo). A perceptible bulge occurred across the freeway at the transition to full cut-section. The bulge was about 200 feet wide across the roadway with a maximum uplift of 0.4 feet relative to each side of it.

Photo 112. Looking north at that portion of Route 210 immediately east of the Roxford Street Undercrossing.

Longitudinal cracking and lateral movement of the sidehill fill resulted in pavement slab separation and faulting as shown in Photos 113, 114, and 115. Fill shear and thrusting at depth created a bulge and shattering slightly above the toe for a distance of about 500 feet as shown in Photo 116. The chainlink fence in Photos 115 and 116 was tilted towards the freeway several degrees as shown in Photo 117. No obvious distress occurred to the sidewalk, curb and gutter, or pavement of Foothill Boulevard adjacent to the above fence. Photo 118 shows typical cracking that occurred along the top of cut on the north side of the freeway.

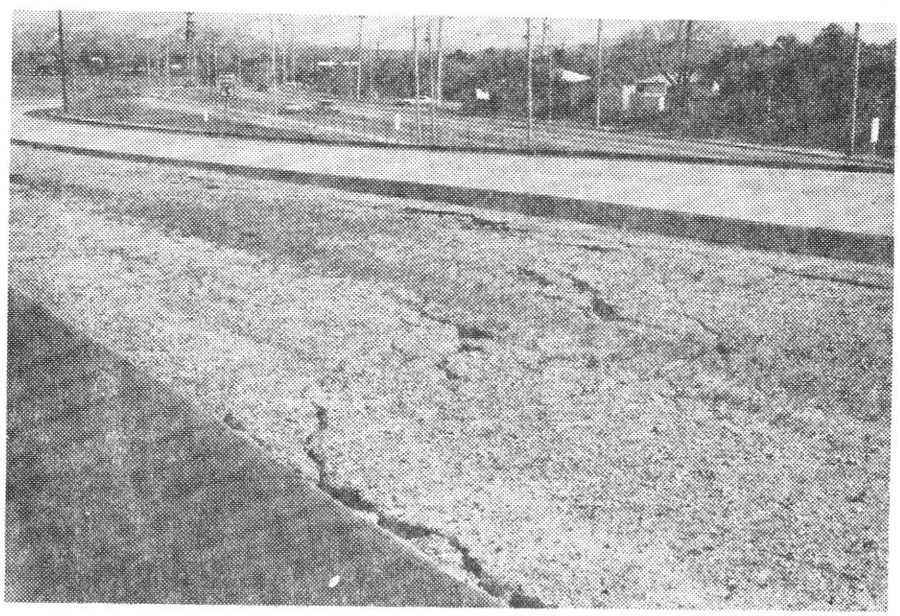


Photo 113. Sidehill fill cracking in median east of Roxford Street.

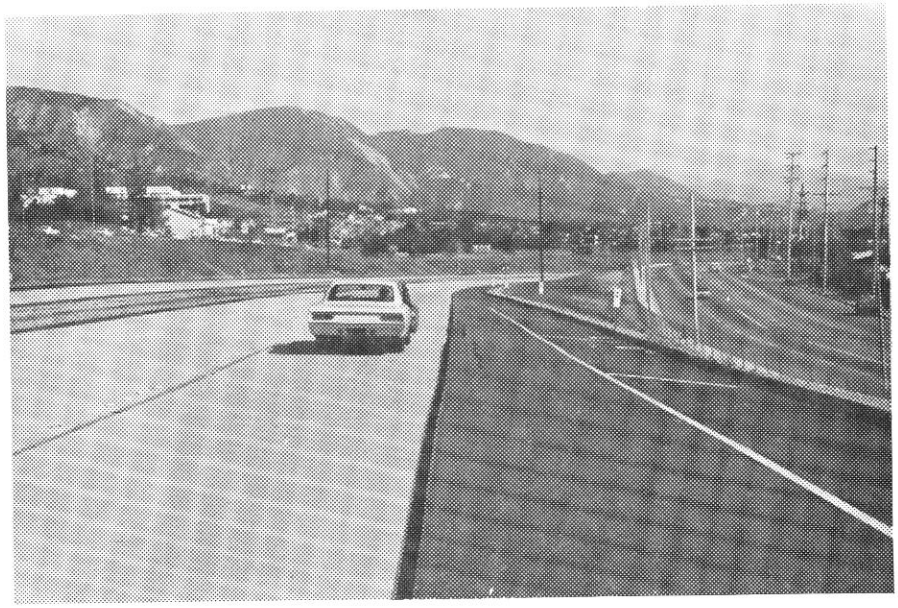


Photo 114. Slab separation and faulting in sidehill fill east of Roxford Street.

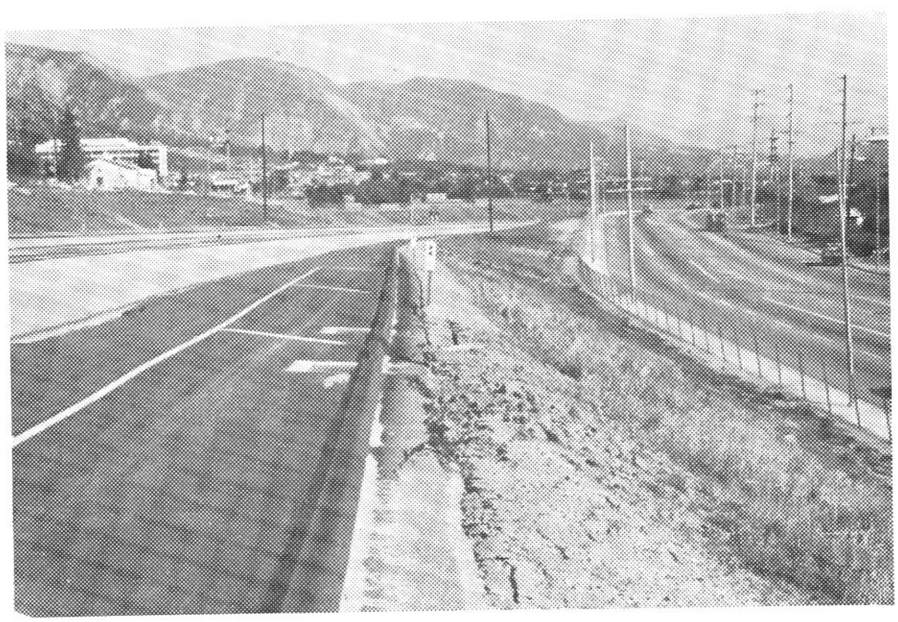


Photo 115. Shattering of outer portion of fill east of Roxford Street.

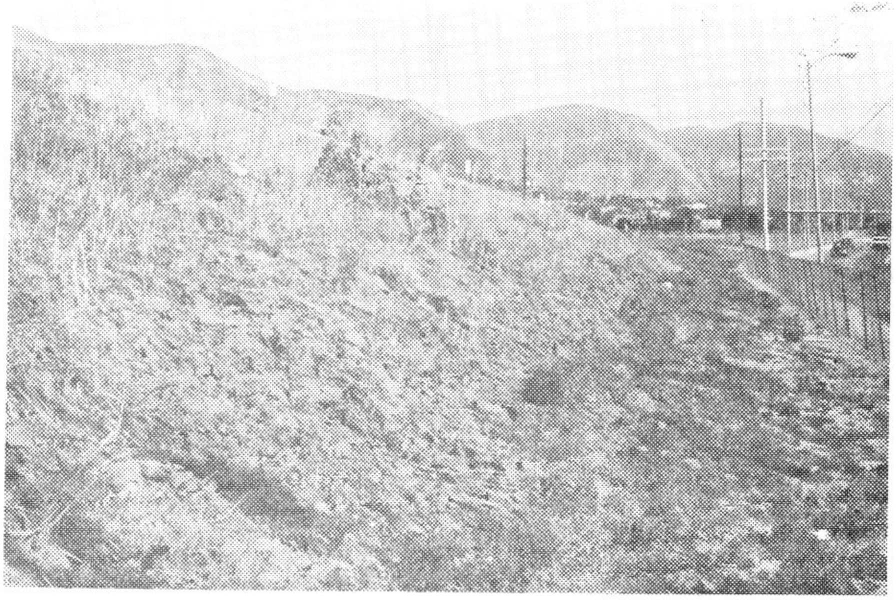


Photo 116. Slope shattering of fill shown in Photo 115.

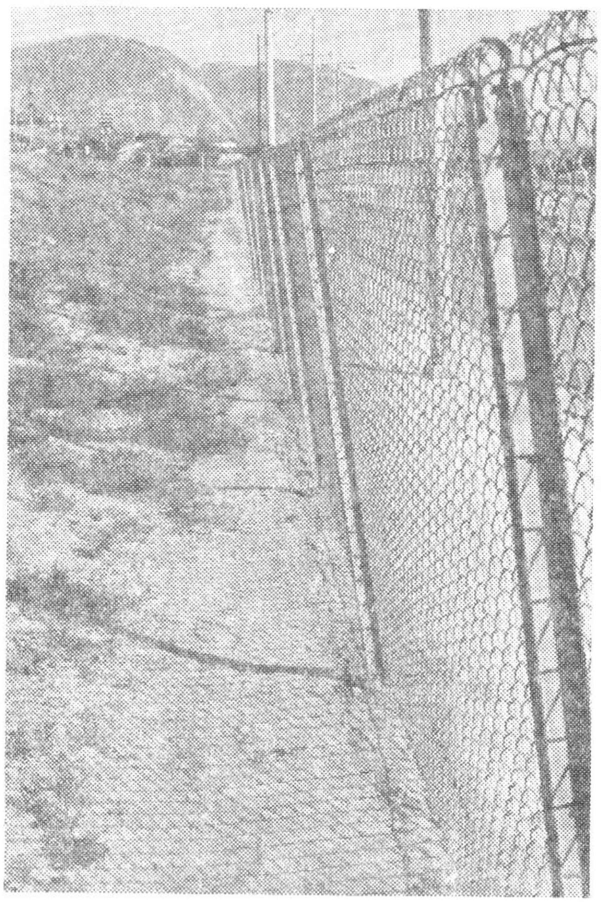


Photo 117. Toe of fill shown in Photos 115 and 116. Note rotation of fence inward, toward fill.

Photo 118. Looking west at cracks along top of cut slope.

Location 11

The freeway in this location is a full cut section with about 25 feet of vertical relief (Photo 119). The material consists of gravelly sands, uncemented and poorly consolidated in the upper several feet.

The concrete pavement and asphalt shoulders showed virtually no distress and maintained good rideability characteristics.

The cut slopes were distressed to various degrees by tension cracking and a minor amount of sliding. Photo 120 shows tension cracks that occurred at the top of cut on the northerly side of the road. The cracking was nearly continuous for the length of the cut and was generally within a 20 feet-wide zone.

Soil-cracking at top of cut on the southerly side of the road is shown in Photos 121 and 122. Cracks extended along, and parallel to, the top of cut for a distance of about 2,000 feet in a zone that was generally 20-25 feet wide, but which in some cases extended 70-80 feet in back of the cut. The cut face on the south side of the road was distressed more than that on the north side.

The Bledsoe Street Overcrossing suffered differential vertical displacements at both abutments. The north approach slab settled a small amount relative to the bridge deck, apparently due to densification of the backfill material. The south approach settled two inches relative to the abutment as may be seen in Photo 122. An interesting feature here is that both abutments settled relative to the deck over the center bent thus creating a vertical curve in the deck. Both abutments as well as the center columns were founded on spread footings.

Photo 119. Looking northerly at Route 210. Bledsoe Street Overcrossing is in center, Olive View Hospital in upper left-center.

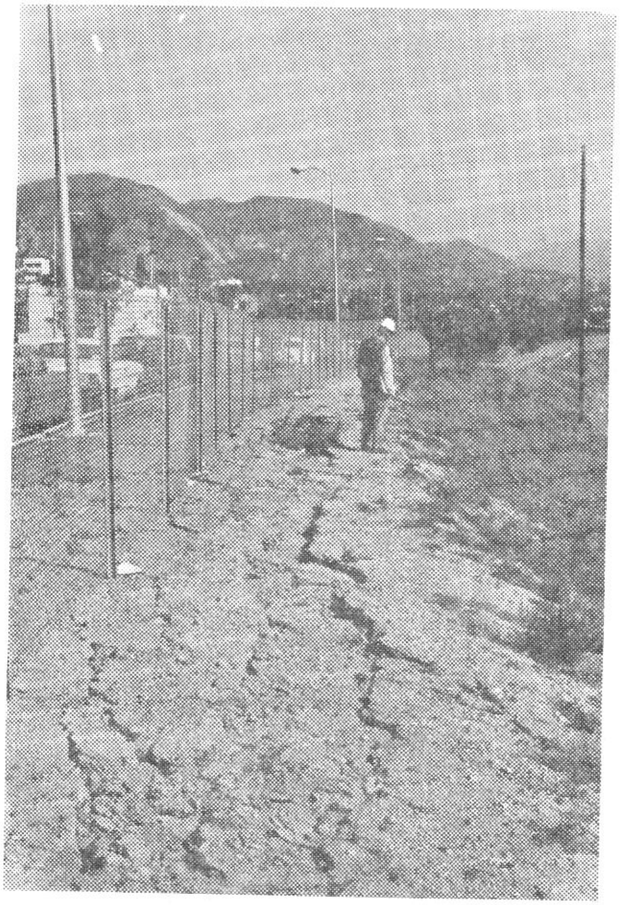


Photo 120. Looking easterly at tension cracks along top of cut slope.

Photo 121. Looking east from Bledsoe Street at cracking in cut slope. The Tyler Street Pedestrian Overcrossing structure is in background.

Photo 122. Looking west at southern end of Bledsoe Street Overcrossing. Note cracks in cut slope and settlement of wingwall relative to structure.

Location 12

The freeway beneath the Tyler Street pedestrian overcrossing is constructed in a slight cut section. No roadway damage was visible, but the overcrossing suffered structural damage as described in Reference 4.

Location 13

The Polk Street/Route 210 Interchange area is shown in Photo 123. The freeway at this location is constructed on 3000 lineal feet of embankment, with a maximum height of about 23 feet between cut sections. Subsurface drainage facilities both parallel and cross the freeway within the interchange area.

The most obvious roadway damage at this location was the abrupt break in roadway profile at the bridge approaches where differential settlement in excess of one foot occurred, as shown in Photos 124 and 125. Embankment settlement of over one foot relative to abutments, shown in Photos 126 and 127, is typical of all the bridge abutment-embankment relationships. Even with the extreme settlement, the approach slabs remained serviceable to emergency traffic at this particular location as shown in Photo 128. Both bridges were founded on piles that penetrated original ground at least 30 feet with the pile tips in dense, poorly sorted gravelly sand.

Photo 123. Looking north at the Route 210/Polk Street Interchange.

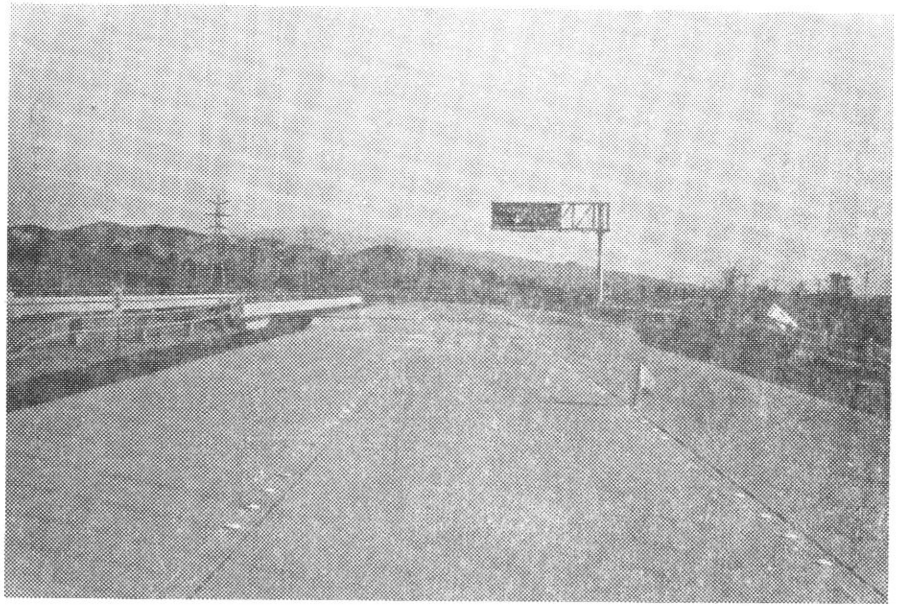


Photo 124. Looking east at eastbound structure of Polk Street Undercrossing. Note difference in profile between roadway and structure deck.

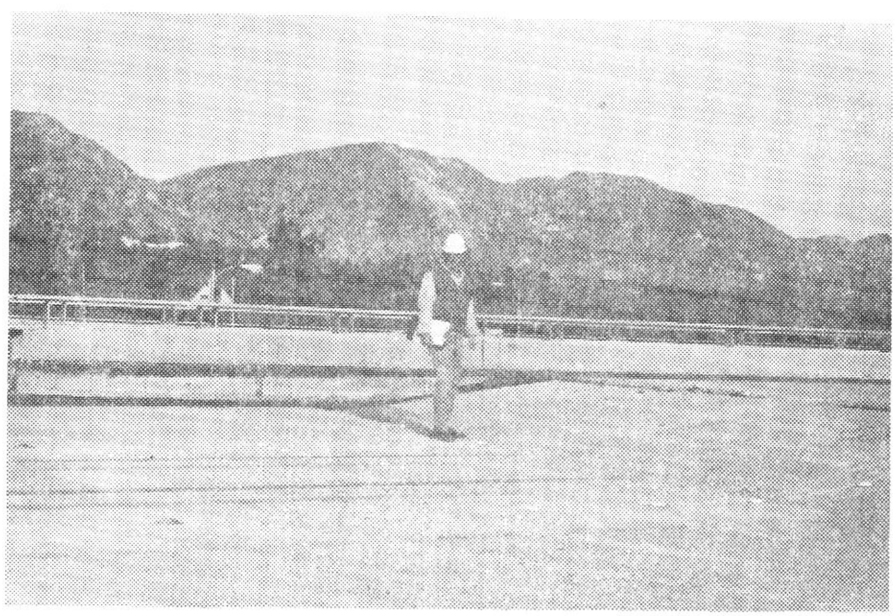


Photo 125. Fill settlement at west approach to eastbound structure at Polk Street Undercrossing. Note approach slab break and resulting ramp effect. The amount of settlement is indicated by the dark (AC) marking along the bridge rail.

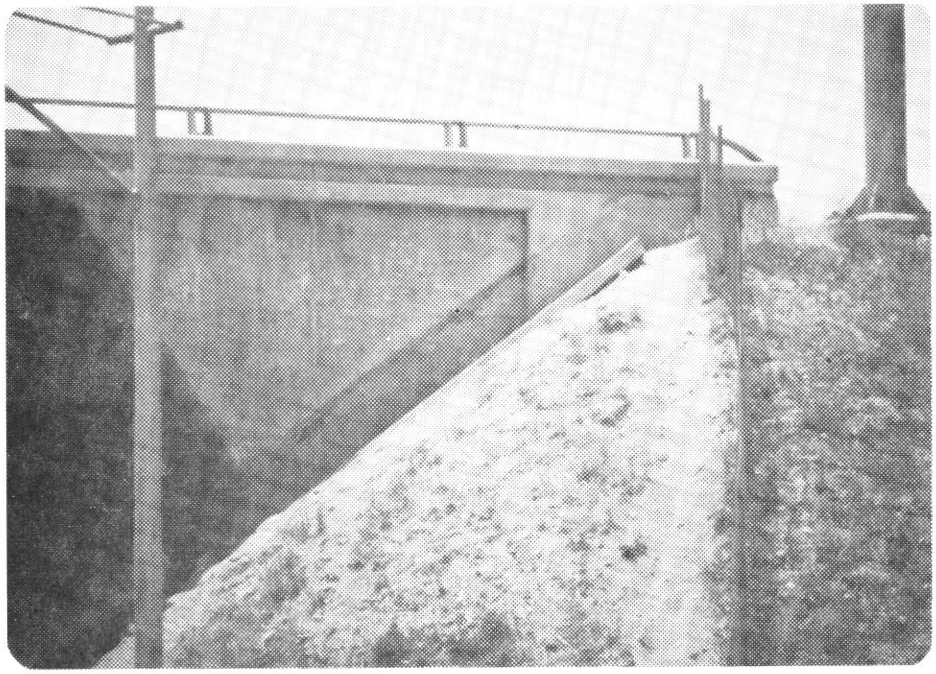


Photo 126. Fill settlement relative to abutment - south side of fill.

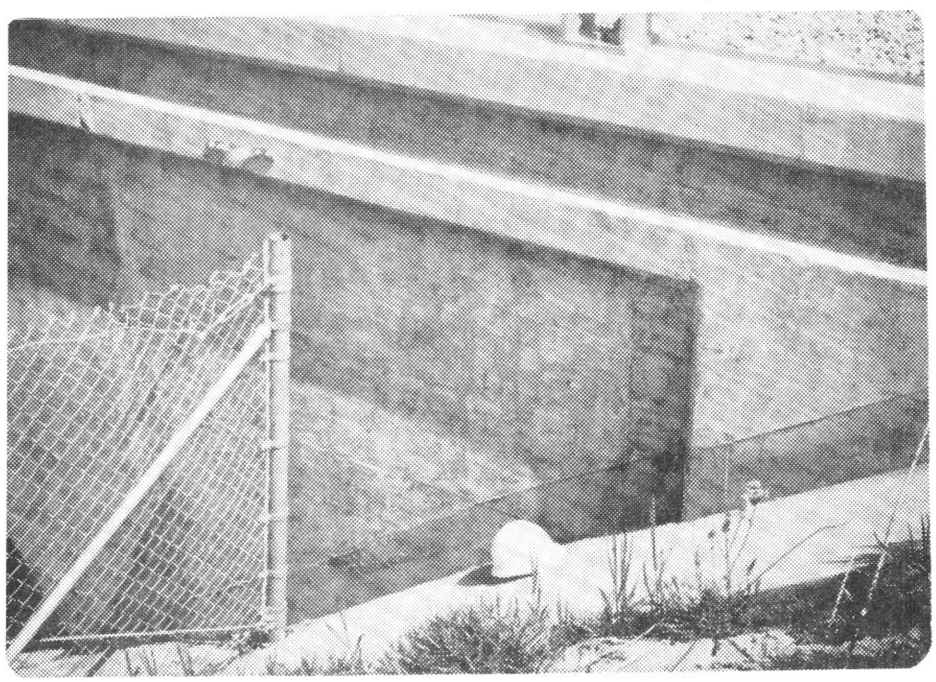


Photo 127. Fill settlement relative to abutment - median area of fill.

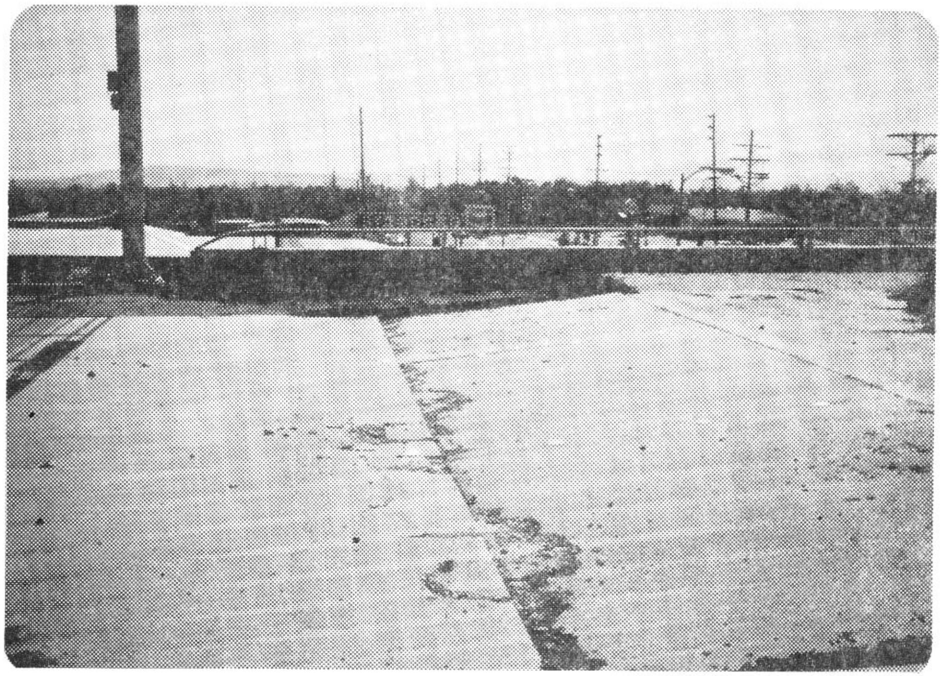


Photo 128. East approach of eastbound structure of Polk Street Undercrossing. Note approach slab spalling along transverse joint.

Embankment settlement, from a visual standpoint, appeared to be quite uniform throughout the embankment section. Roadway rideability remained good with the exception of bridge approaches. Undulations in the pavement profile detectable by eye usually resulted from underlying drainage structures. Concrete pavement slab separation along longitudinal and transverse construction joints started at the beginning of fill (Station 245±, Figure 17) and continued intermittently to Station 251± with separations in the range of 1/2-inch. Beyond Station 251 the separations were more or less continuous and more prevalent as the Polk Street UC bridges were approached, with openings in the 1 to 1-1/2-inch range. East of the bridges, slab separation was less intense and was not evident beyond Station 264±. Examples of slab separations are shown in Photos 129 and 130.

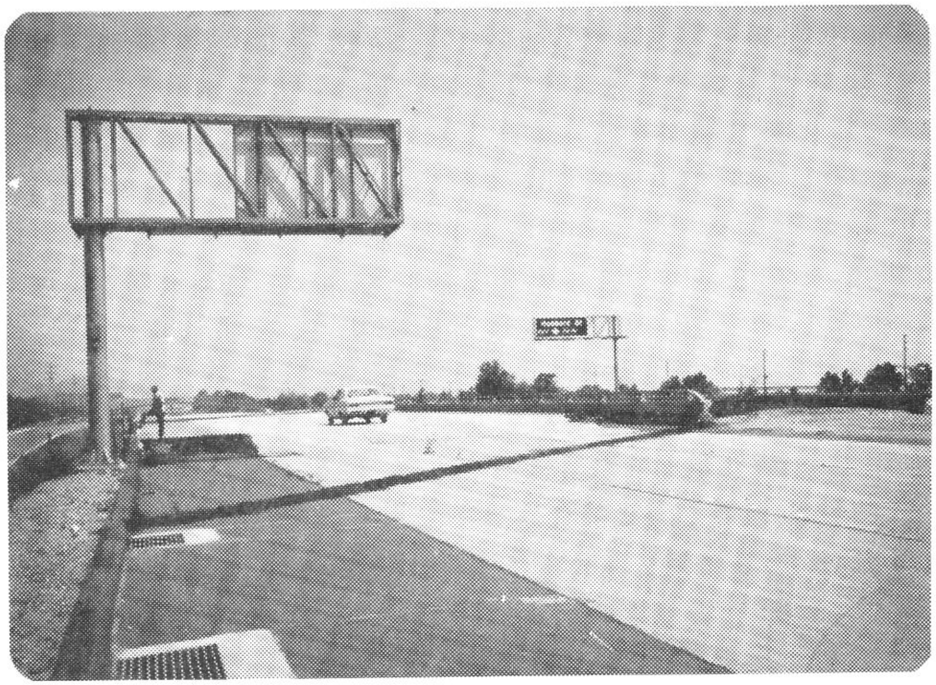


Photo 129. Westbound lanes at Polk Street Undercrossing. Note slab separation at joints.

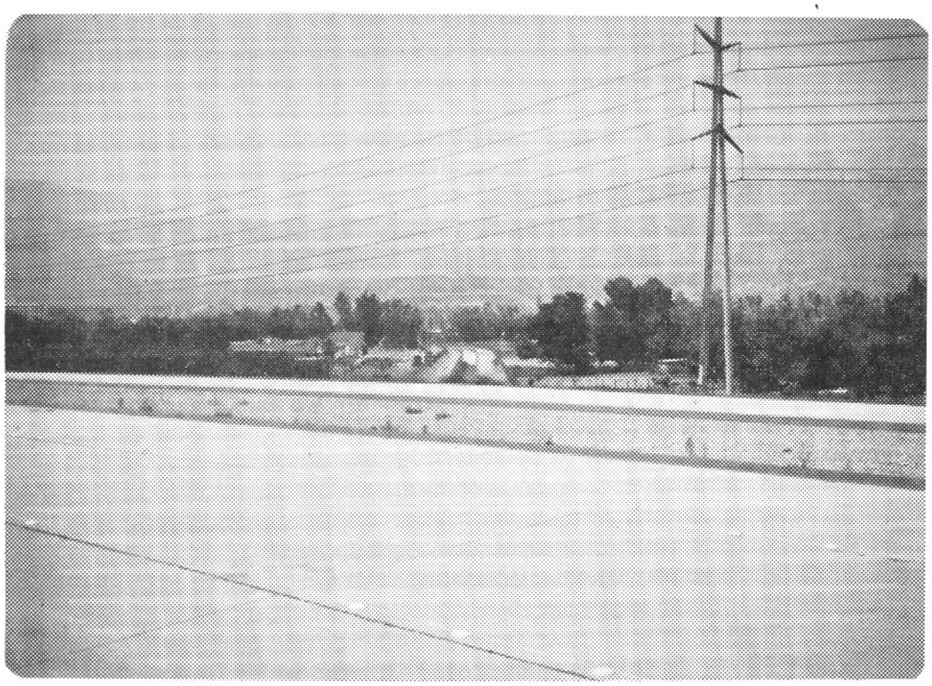


Photo 130. Eastbound lanes at Polk Street Undercrossing. Further examples of slab separation at joints.

The embankment settled more or less as a unit without any detectable fissures within its mass that would indicate shear displacements. Some heaving did occur along portions of the toe but not to the extent that would indicate fill failure. Photo 131 shows a portion of the heaved embankment toe along the westbound on-ramp. The electrical conduit shown in the photo was forced out of the fill slope as the embankment settled.

A compression buckle occurred in the westbound on-ramp asphalt concrete pavement as shown in Photo 132. It is interesting to note that the buckle did not continue into the PCC curb and gutter sections on either side of it.

Major drainage structures beneath and parallel to the freeway are reinforced concrete boxes. Feeder lines into the boxes are constructed of reinforced concrete pipe and corrugated metal pipe.

Wilson Canyon Channel crosses diagonally beneath the freeway in an 11 1/2-foot high by minimum 15-foot wide reinforced concrete box. Differential settlement of freeway embankment was observed in the roadway profile of this area, and Photo 133 shows the resulting "hump" over the RCB where it crosses the eastbound off ramp (center of photo). Another buried reinforced concrete box 7 feet high by 8 feet wide branches into Wilson Canyon at the north toe of the freeway embankment. Lateral earth pressures on the freeway embankment side of the box caused craking in the wall as shown in Photos 134 and 135. These photographs were taken in the segment of the box opposite freeway Stations 250+50 to 252±. The lateral entering the box (right side of Photo 135 is a

24-inch diameter reinforced concrete pipe that collects surface drainage from the freeway roadbed. The outlet (shown in the box) is about 8 feet below the original ground surface. Vertical offsets of one inch at the pipe joints were caused by densification and resulting settlement of alluvium materials below the freeway embankment.

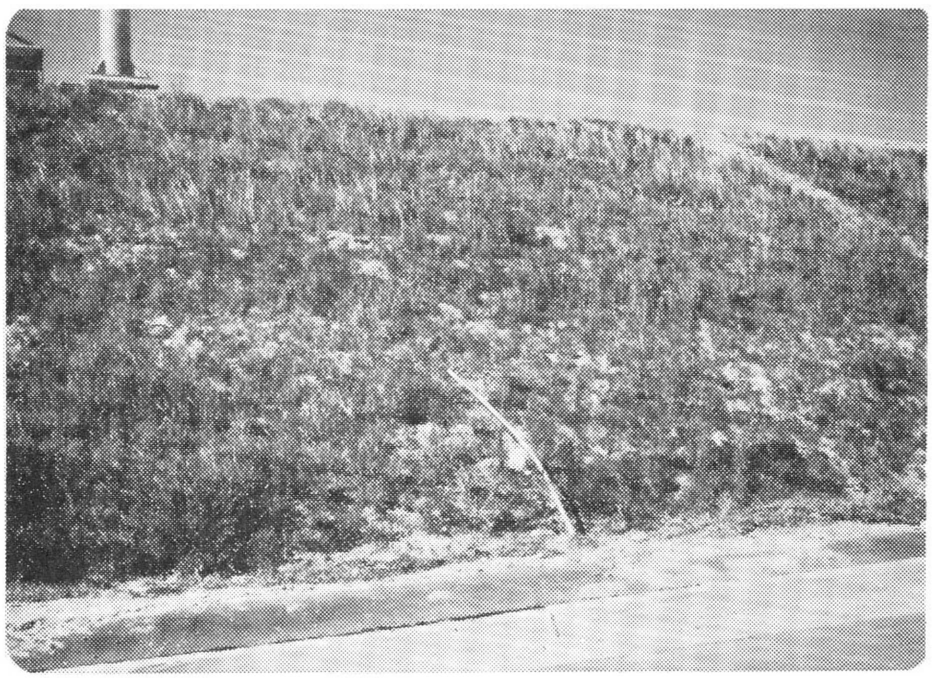


Photo 131. West approach fill to Polk Street structure. Note conduit that was forced out of slope as fill settled.

Photo 132. Compression buckle in AC ramp pavement. Note intact PCC curb and gutter on each side of pavement.

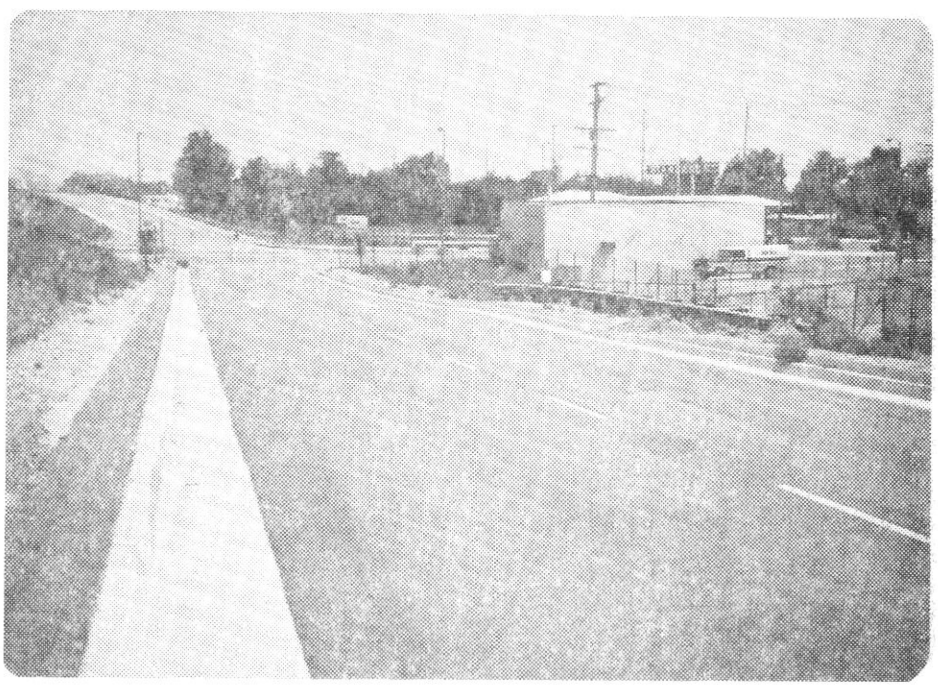
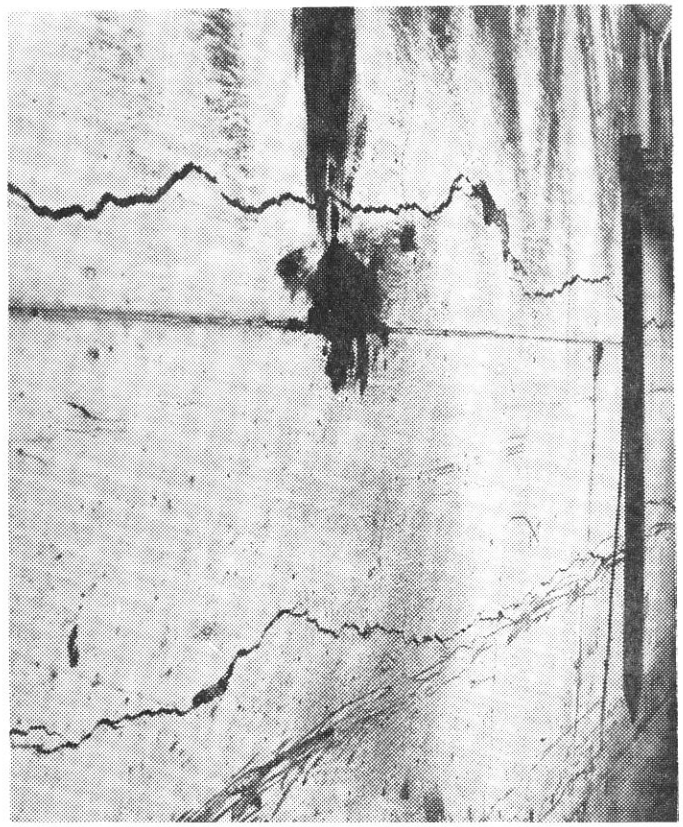


Photo 133. Eastbound off ramp to Polk Street. Note "hump" in pavement over underlying drainage structure.

Photo 134. Cracking and horizontal displacement of reinforced concrete box drainage structure below toe of fill.



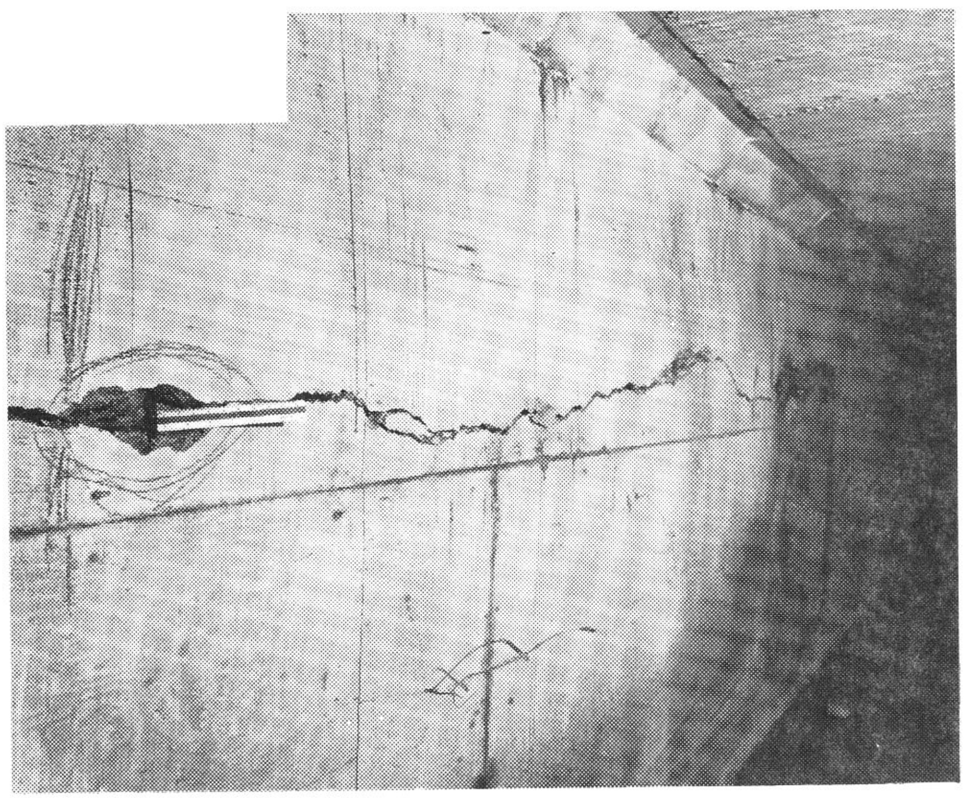


Photo 135. Crack in drainage structure of Photo 134.

Location 14

The Astoria Street pedestrian overcrossing is located within a fill-to-cut transition area.

Roadway damage consisted of compression buckles in the concrete pavement that affected all lanes of both roadbeds in a 300-foot long zone as illustrated in Photos 136-139. No evidence of tectonic rupture was observed in the buckled zone. Non-tectonic buckles observed at this location and others tended to occur at cut-to-fill transition zones or fill to essentially original ground transition zones.

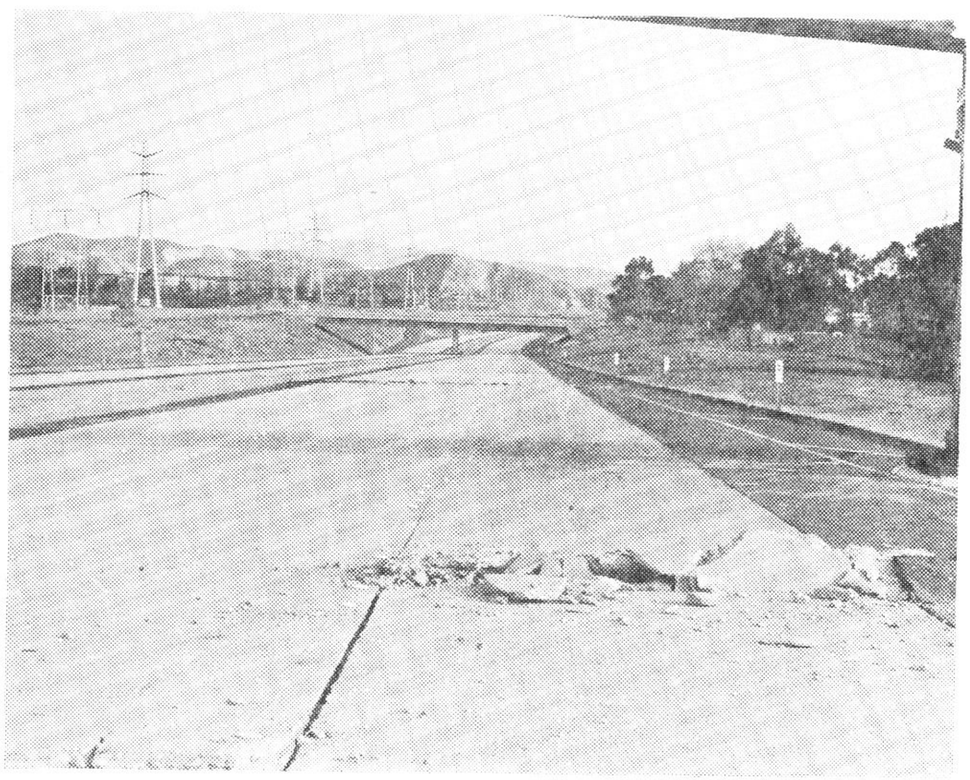


Photo 136. Looking east at compression features in pavement slabs. Note spalling occurred only in one lane.

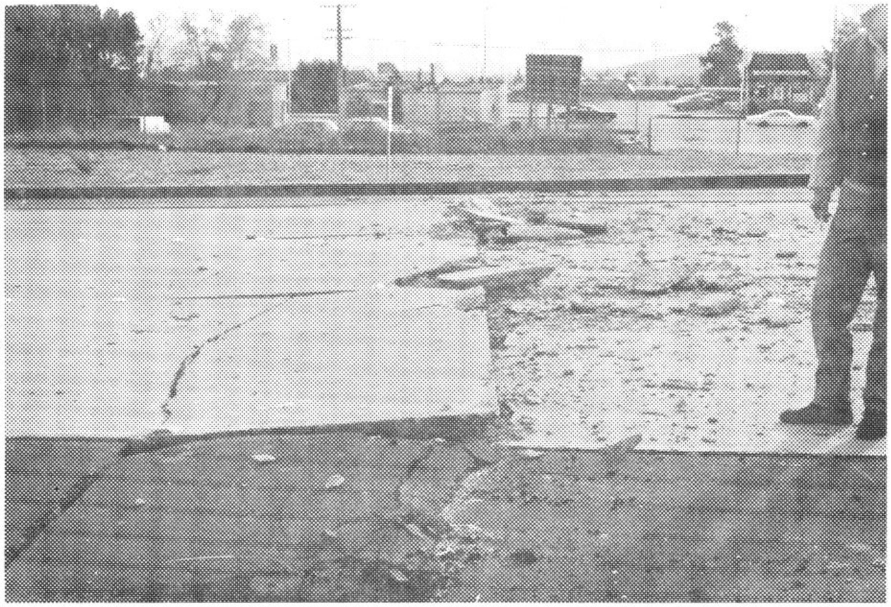


Photo 137. Compression at transverse joint in eastbound lanes. Note slab overthrusting east to west.

Photo 138. Close view of slab compression. Note lower portion of overthrusted slab sheared off on impact.

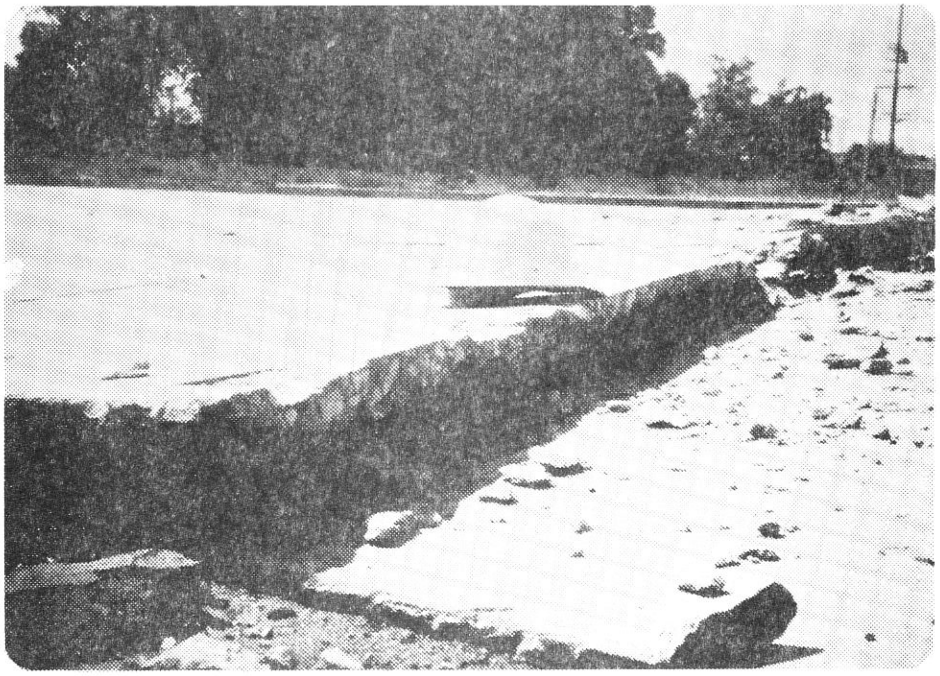


Photo 139. Slab compression and overthrusting in same general area as Photo 137.

Location 15

At this location, Sayre Street crosses over the freeway that is in about 25 feet of cut (Photo 140).

The bridge approaches and wingwalls settled about 2 inches as shown in Photo 141. Tension cracks developed behind the top of cut on the south side and down the cut slope to about mid-height. Some cracks were up to 2 feet deep. Material at this location is composed of dense, slightly silty very fine sand, with a small amount of coarse sand and fine gravel.

Photo 140. Looking northerly at Sayre Street Overcrossing.

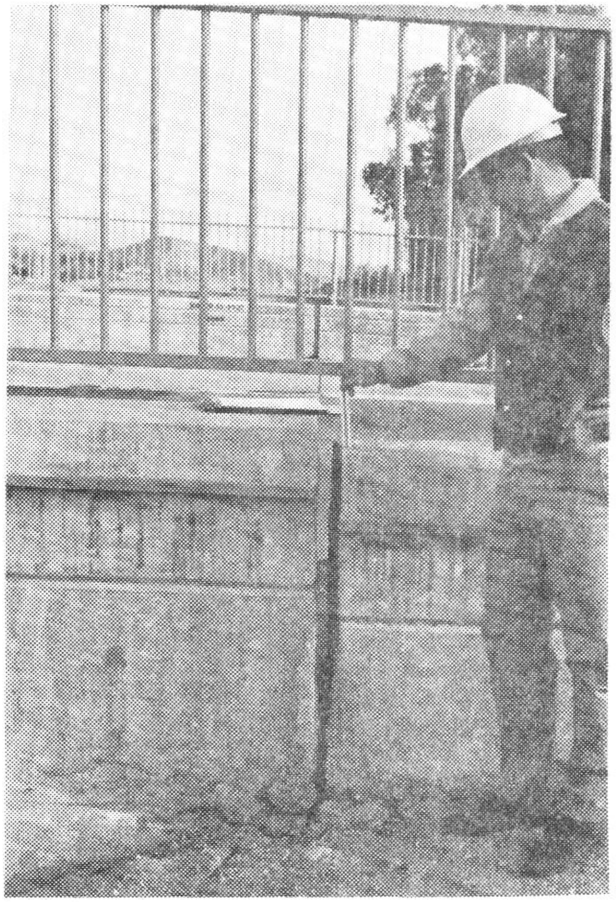


Photo 141. Settlement and separation of wingwall at the Sayre Street Overcrossing.

Location 16

The Hubbard Street Overcrossing and interchange area is shown in Photo 142. The freeway is entirely in cut section, up to 30 feet high. No visible damage occurred to the bridge structure, which is founded on spread footings. At the northwest corner of the bridge, the approach slab settled about two inches. South of the bridge, a slight amount of cracking occurred at the curb-returns of the on and off-ramps.

Photo 142. Looking northerly at the Route 210/ Hubbard Street Interchange.

The only damage observed on the main roadbed of the freeway was a pavement buckle about 4 inches high in the median asphalt concrete shoulder of the westbound roadbed, about 20 feet west of the bridge. Later profile surveys of the roadway indicate a 600-foot zone of undulations in the Pavement, beginning at the mentioned buckling and extending to Station 307±.

Both ramps to the east of the bridge suffered pavement buckles. The eastbound on-ramp buckle affected only the asphalt concrete pavement. The westbound off-ramp buckle affected the asphalt concrete pavement and the outside PCC curb and gutter section as shown in Photo 143.

Photo 143. Compression buckle in westbound off ramp to Hubbard Street. Note PCC curb and gutter near freeway was unaffected whereas other curb and gutter section buckled.

Location 17

A pavement break in the eastbound roadway (Photo 144) plus undulations in the roadway appeared to be due to faulting. Cracking in the cut slope on the north side of the freeway (Photo 145) lends support to the belief that faulting occurred here.

A formative slide in the south cut slope opposite freeway Station 321+00 caused minor up-thrusting of the pavement.

Location 18

The Harding Street pedestrian overcrossing at this location was damaged (4) but no roadway damage was observed.

Location 19

The area between Harding and Maclay Streets, shown in Photo 146, is where the Sylmar Fault trace crossed the freeway and disrupted the roadway across a 600-foot wide zone. Photo 147 is a view, looking east, that shows left lateral displacement across the fault of about 4 feet. Vertical displacement of about 4-1/2 feet occurred across the fault, with the west (foreground) up, relative to the east. Photos 148-151 are views of the faulted area.

At the Maclay Street Undercrossing, highway embankment differential settlement of about 7 inches occurred between the highway embankment and bridge deck. Breakage of curb and slope paving beneath the structures occurred along Maclay Street similar to that described for the Glenoaks Boulevard Undercrossing.

The structures are founded on piles that penetrate 17 to 20 feet below the original ground surface. Foundation materials consist of very dense, coarse, sandy, gravel with cobbles and boulders, overlain by a 7 to 12 feet thick layer of loose to dense fine sand.

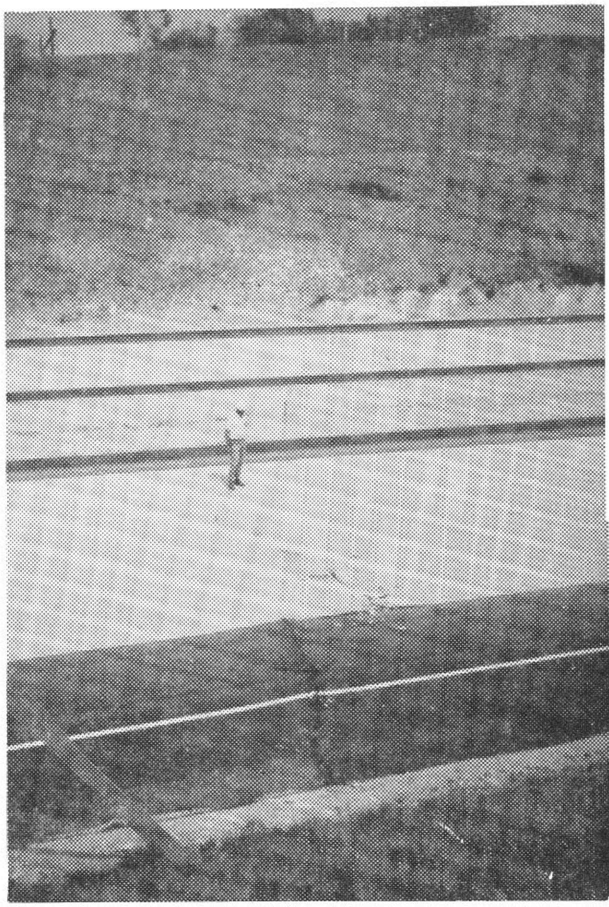


Photo 144. Looking northerly at pavement break in eastbound lanes.

Photo 145. Cracking in north cut slope opposite pavement break shown in Photo 143.

Photo 146. Looking northeasterly at end of completed portion of Route 210. Maclay Street Undercrossing is at right-center, Harding Street Pedestrian Overcrossing at left-center.

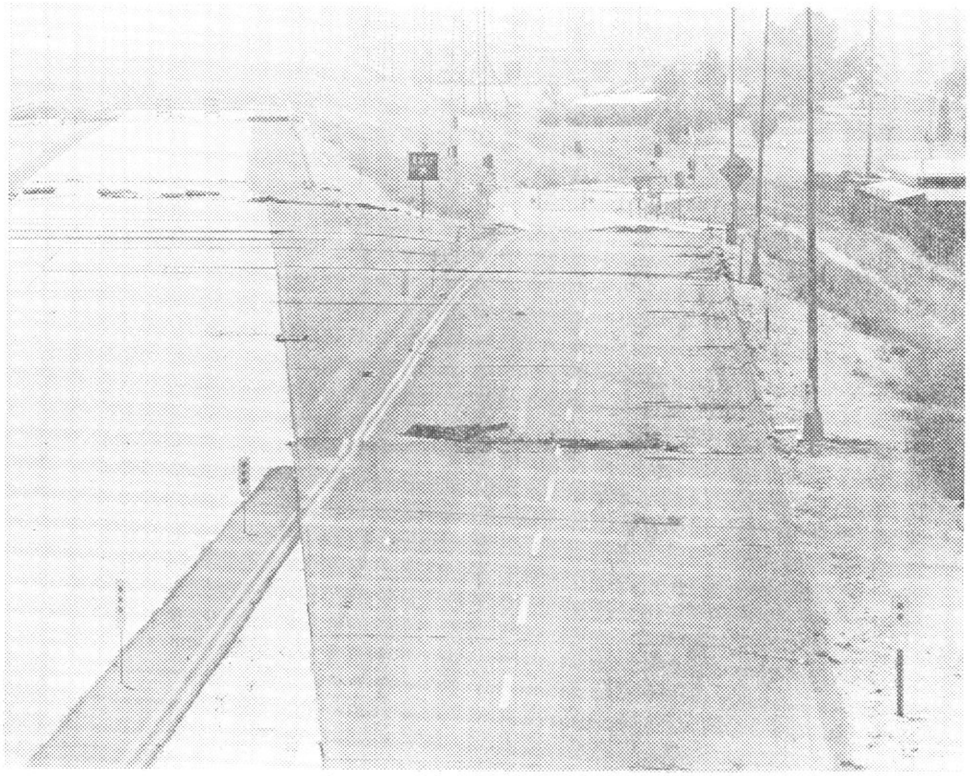


Photo 147. Looking east at fault zone crossing freeway between Harding Street and Maclay Street.

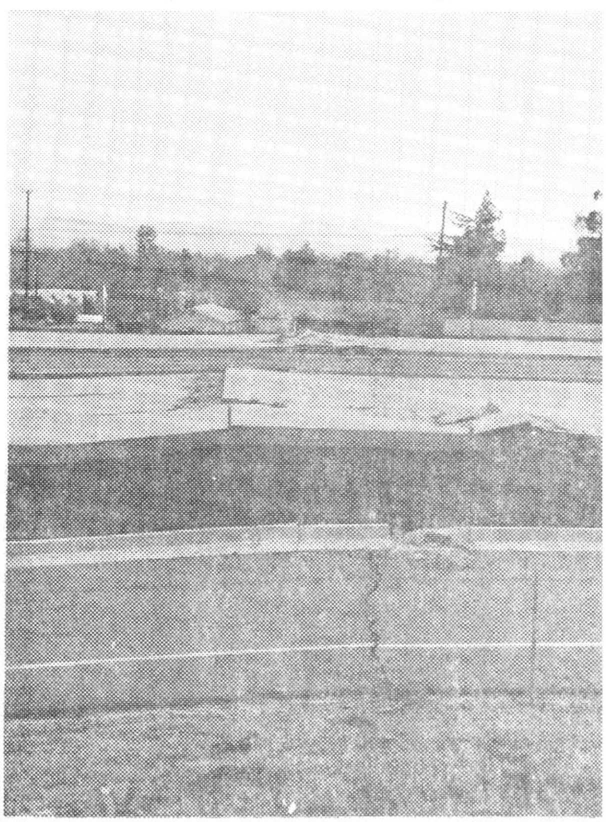


Photo 148. Looking south at fault zone crossing freeway.

Photo 149. Looking westerly at fault crossing freeway.

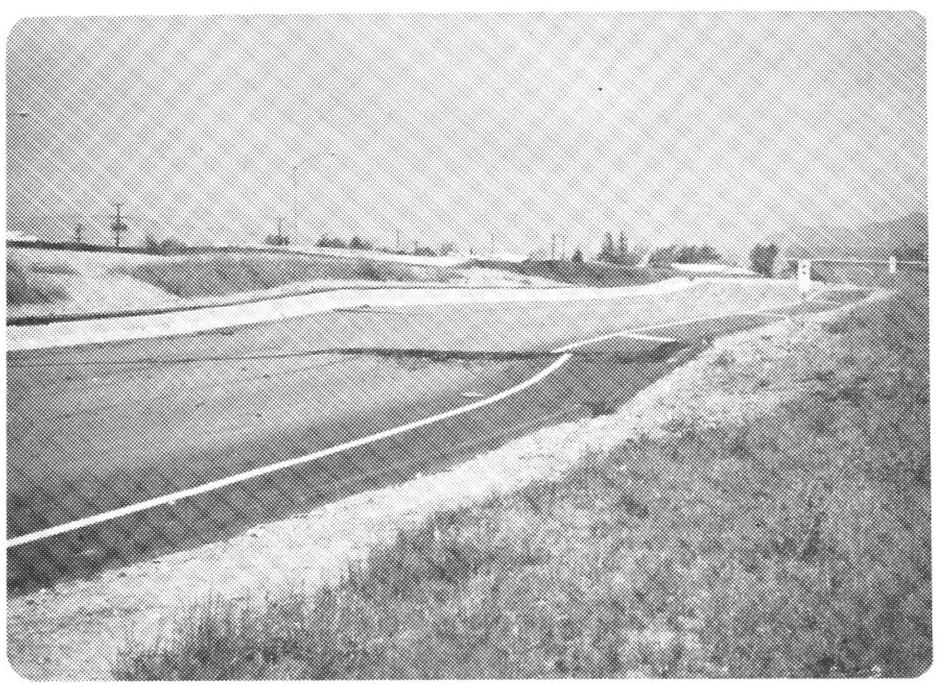


Photo 150. Westbound on-ramp from Maclay Street in fault zone.

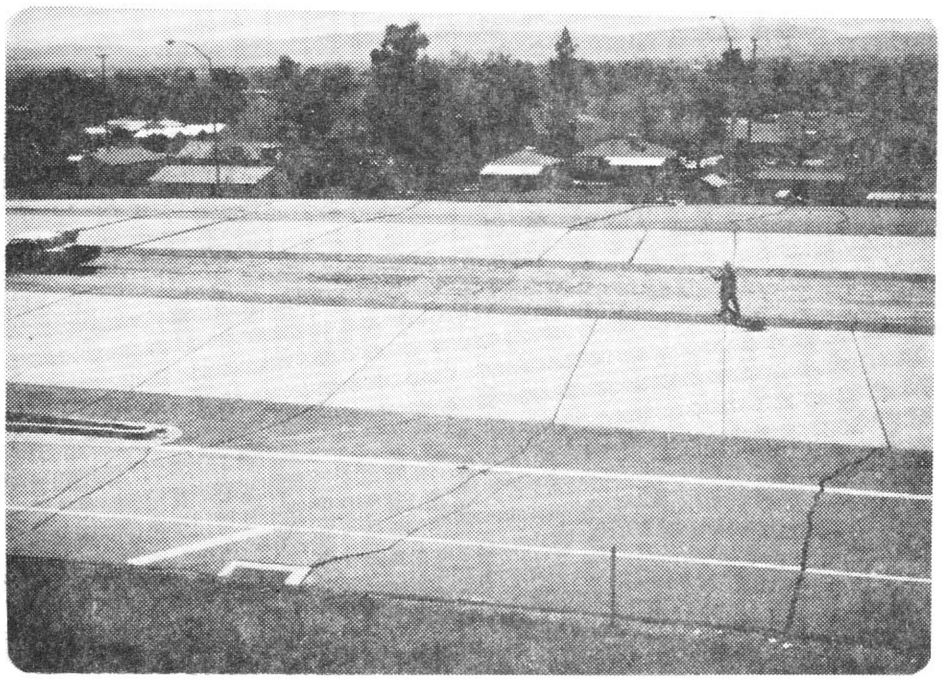


Photo 151. Looking south at cracks and slab separation through fault zone.

ROUTE 118, FROM ROUTE 210 THROUGH ROUTE 405

At the time of the earthquake Route 118 was in stage construction between Route 210 to the northeast and Route 405 to the southwest. The alignment, shown in Figure 2, is on embankment, most of which was in-place prior to the earthquake. Reinforced concrete retaining walls were in place at various locations on the route. Neither roadway pavement nor bridges were under construction or in place.

The Route 210/118 Interchange, located about one mile east of Maclay Street/Route 210 Interchange, was in the final stages of design when the earthquake occurred. Embankment was in-place along Route 210 from Maclay Street east to the 210/118 Interchange area, and in-place beyond the interchange area for another 2500(±) feet.

During the earthquake, geologic upheaval occurred within the Route 118/210 Interchange area in the form of faulting, horizontal compression of alluvium, and numerous landslides with attendant soil cracking and displacement. Photos 152-155, present an overall impression of ground disturbance in the area of the interchange.

Embankment damage generally consisted of minor to moderate cracking at or near the hinge points and at corners as shown in Photos 156 and 157. Although extremely minor, this type of cracking would have resulted in slab separation and possible faulting had the pavement been in place.

Retaining walls, such as shown in Photos 158-160, had no visible signs of damage.

Photo 152. Faulting and ground upheaval adjacent to Foothill Boulevard. Route 210 is off photo to left.

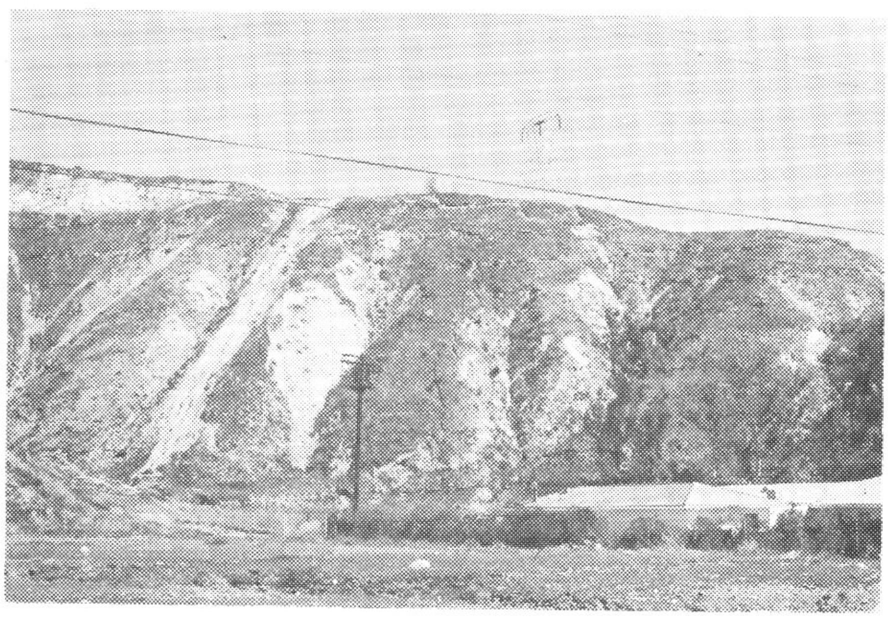


Photo 153. Looking northeasterly at planned site of Route 210/118 Interchange.

Photo 154. Cracking and displacement in slopes of hills shown in Photo 152.

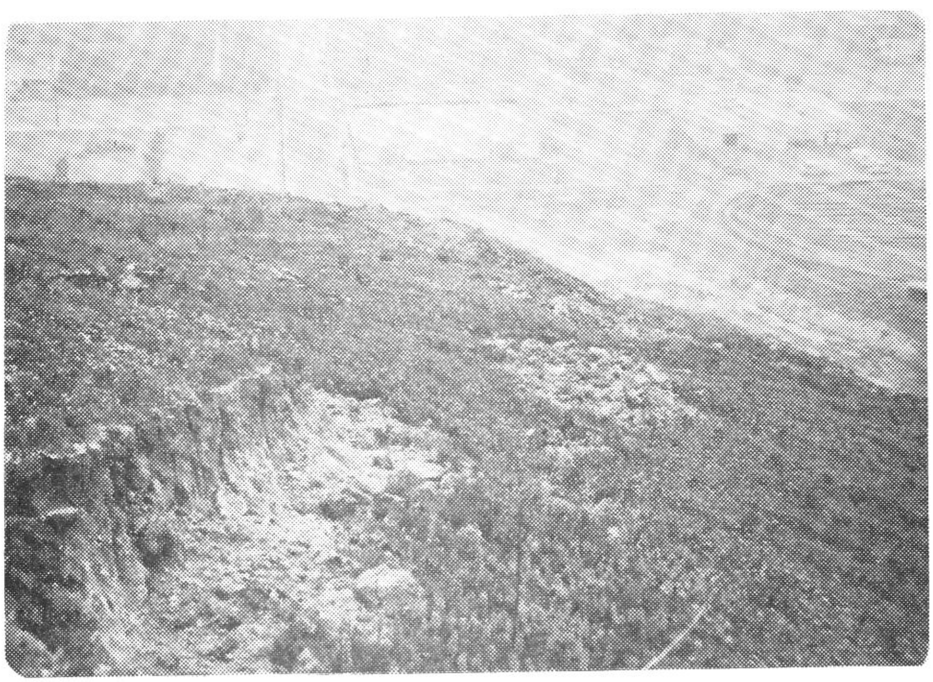


Photo 155. Cracking and displacement in slopes of hills shown in Photo 152.

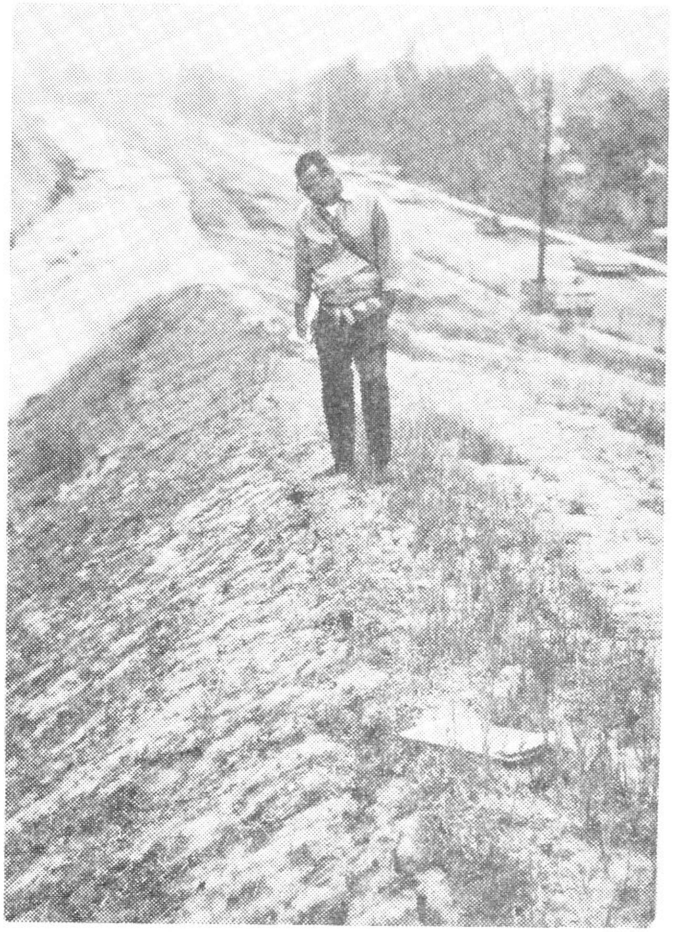
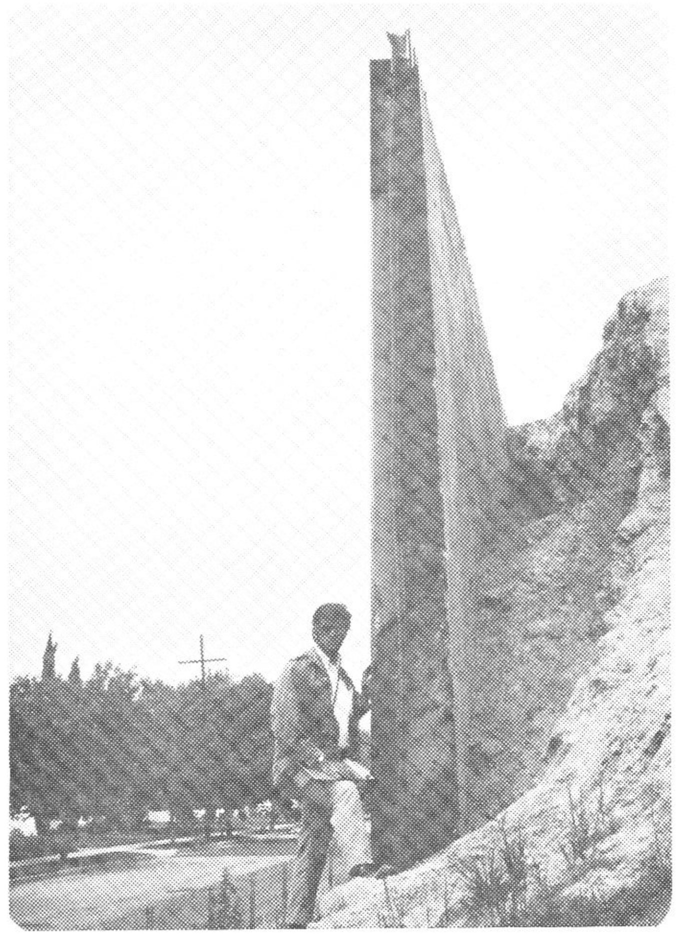


Photo 156. Cracking along hinge point of Route 118 stage construction fill.

Photo 157. Typical cracking at corner of Route 118 approach fill.

Photo 158. Completed retaining wall on Route 118 showed no evidence of damage.



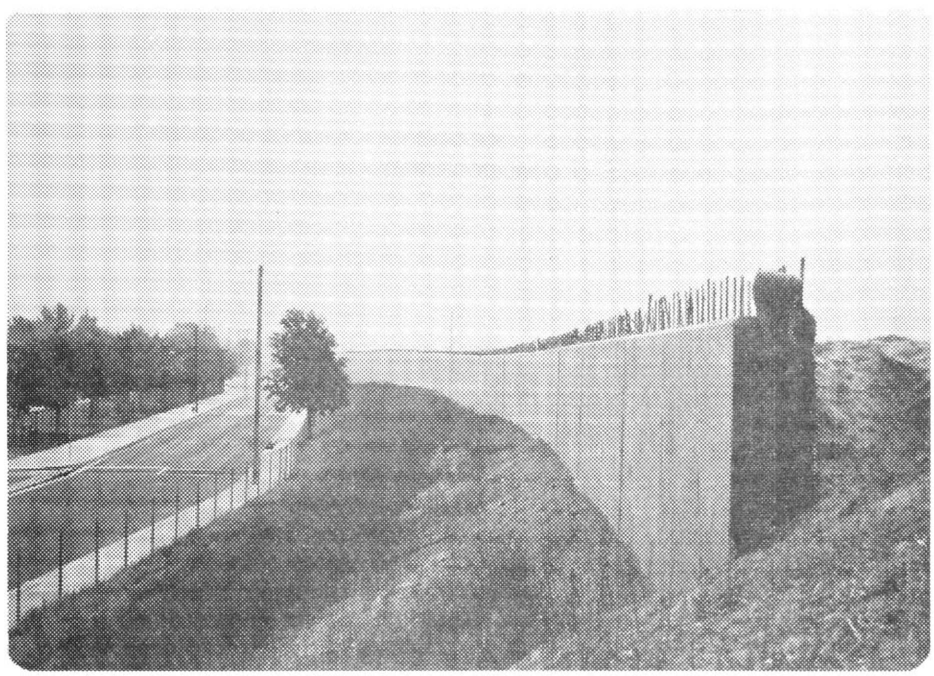


Photo 159. Completed retaining wall on Route 118 showed no evidence of damage.

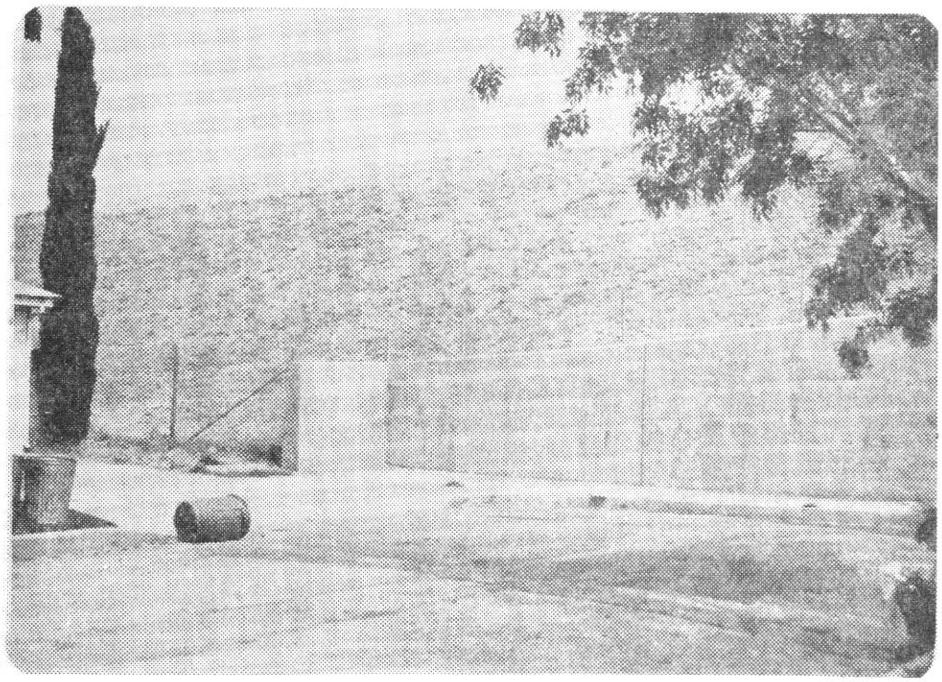


Photo 160. Retaining wall was undamaged but lateral forces transmitted through sidewalk cracked curb.

DISCUSSION OF DAMAGE

Assessments of roadway damage can be made from several points of view. For example, the extent of damage to individual roadway elements such as embankments, cut slopes, drainage structures, the pavement section, etc., can be appraised in terms of structural integrity, ability to function, and costs - as well as time - for repairs. While this approach is necessary to appreciate some of the aspects of dynamic behavior, and beneficial with respect to improving design procedures, it tends to neglect the overall effect on the facility as a whole. Since roads, and major arteries in particular, are one of the lifelines for essential activities during emergencies, damage to individual roadway elements also must be assessed in relation to what effects such damage had on the safe, functional operation of the travelled way.

This section discusses damage to roadway elements. The overall effect of the damage on facility operation will be summarized in the next section.

Cut Slopes

The behavior of depressed section cut slopes (2:1) in alluvium along Route 210 was dependent primarily on type of material. For example, slopes in dense, stiff, brittle materials (silts and silty sands containing enough non-plastic fines to have undergone desiccation to the extent that a cemented appearance was evident) fractured and shattered badly over distances of 20 to 30 feet behind top of cut, and often in the face itself. Materials of moderate density and a slightly greater moisture content experienced fewer cracks and no shattering. Often only one or two large cracks were found along the slope top. Slopes in loose materials sustained several small cracks behind the slope (where surface desiccation had occurred) in a parallel pattern and the slope moved successively in small amounts at each crack toward the unsupported surface. Sloughing or slumping in the slope face also occurred in this type of material.

No slides occurred in depressed section slopes, probably due to a combination of the following factors: the surrounding ground beyond top of slope was flat so that overall slope height was limited to depth of cut; the granular alluvium increased in density, and hence strength, with depth below original ground; and groundwater was not a factor because of its depth.

Depressed section cut slopes had no detectable effects on the travelled way. However, overcrossing structures were affected by the lateral (inward) and downward slope movements. The abutment footings moved in accordance with the slope movements and the entire structure tended to settle more or less uniformly. A uniform settlement was prevented, however, by the structure center bent whose columns were founded in the denser material below freeway grade, and a bow developed in the deck.

Slopes in small to moderate cuts through ridges and hills along portions of Routes 5, 210, and 14 experienced cracking near the top of cut and frequently in original ground beyond the cut. Generally, the cracks were fewer but larger than those observed in depressed section slopes, often up to 8 inches wide. No slides involving large movements occurred; but at some locations slides were in formative-stages and very small rotational type movements did occur. These movements, when the toe failure type, did not affect the travelled way. However, the deeper-seated base type movements affected the pavement by upthrusting action, which caused slab separation at longitudinal and transverse joints. A few diagonal cracks across raised slabs developed on Route 5 within a few days time due to traffic loads and uneven slab support. The effect on traffic lanes was slight unevenness in the roadway profile.

The only large slide movements occurred in large cuts on Route 5. One slide, which moved along a bedding plane, completely blocked a connecting ramp under construction in the Route 5/14 Interchange area. This slide, plus the reactivation of an older slide north of the interchange, suggest that for large cut slopes, weak planes such as bedding, joints and fractures, and zones of old movement are more important in determining dynamic stability than height or steepness of slope. It was noted, for example, that some very steep slopes on Route 14 performed well with no evidence of cracking or movement.

Embankments

The type of embankment damage most often observed was subsidence due to vibratory densification of fill and foundation materials. These materials consisted of nonplastic silty, gravelly sands, except at isolated locations. Subsurface soils were densified not only under fills but in areas outside the confining influence of fill weight. Greater densification did occur under the fills, however, especially at locations where the materials were in a very loose state prior to the quake. Generally, densification of foundation materials resulted in much greater amounts of fill subsidence than densification of the fill itself, because the deposits of alluvium affected by the ground vibrations were in a looser pre-quake condition than the fill and existed in substantially greater depth at most locations than the fill was thick.

The contribution of subsurface soil densification to fill subsidence can easily be detected at locations where rigid inclusions in natural ground pass under the fills. Differential subsidence results from ground shaking and a ridge or high area in the roadway surface is created over the non-yielding inclusion. As the fill on either side of the rigid element subsides, shear cracks are formed in the fill slopes, beginning at the toes and extending up the slope face. In cases of small relative movement the cracks may extend only about half way up the slope, but as total subsidence increases the cracks continue to develop and extend across the roadway. They may or may not be reflected in the pavement section but can be observed in unpaved medians and shoulders. This type of differential subsidence occurred at the Polk Street/210 Interchange, on Route 5 just north of Rinaldi Street, and on Route 5 near the Balboa Boulevard Overcrossing structure.

Some spreading of fills near structures was associated with densification within the fill due to the tendency for movement toward unsupported surfaces. This type of spreading in the fill proper was minimal, fairly uniform, and confined for the most part to the ends of approach fills under bridges. Fill spreading caused by lateral movement in foundation soils during densification was observed at a few locations but was limited in amount due to the inherent high shear strength of foundation soils. This resulted in bulging in the lower half of fill slopes and formation of longitudinal tension cracks in the upper half as the fill subsided differentially in a plane normal to centerline. Enough lateral movement near the fill toe did occur at some locations to cause minor thrusting of inside curb and gutter sections of freeway ramps. Also, as the fills subsided, the lateral component of slope paving movement sheared and rotated the curbs along the surface streets under the freeway structures.

Slipouts varying in size and amount of movement occurred in fills at several locations. Maximum movement did not exceed what normally would be considered a small amount but was sufficient to cause differential vertical displacements at longitudinal joints in pavement slabs, diagonal cracks across the pavement at the flanks, and minimal thrusting near the toe. Larger movements were prevented by the high strength properties of fill and foundation materials. The larger slipouts experienced the greatest movement and were observed to have occurred in long fills, all of which were 20-30 feet high. The smaller slipouts actually were in formative stages and involved very little movement, which was often difficult to detect. They usually occurred in fills crossing small ravines and developed because of differential response by fill and natural ground. Consequently, lateral limits of these slides generally followed the cut/fill contact.

Three fill slipouts caused by plastic type movements in fine-grained compressible foundation soils occurred within one fairly small area on Route 5 just east of the Lower Van Norman

Lake. These slipouts involved more vertical and lateral movement than those occurring in fills only. The portion of fill undergoing movement dropped a maximum of one to 1.5 feet. Lateral movement was somewhat less due to the rotational effect, but large cracks up the fill slope were formed at the flanks. These slipouts covered a larger portion of the roadway than those in fills only.

The effect of subsidence and associated cracking on future fill stability was considered negligible because of the high quality of fill materials. However, large quantities of water entering the larger cracks prior to repair work could have caused sliding and sloughing within slopes and shoulder areas where random longitudinal cracking was more prevalent. Slipouts within fills only did not present stability problems for the same reason. No additional slippage occurred at locations involving failure in foundation materials since no additional loading was imposed and the soil strength improved to near pre-quake levels through consolidation.

The major effects of embankment behavior on the travelled way were disruptions in roadway profile at cut/fill contacts and bridge approaches; and pavement slab faulting at locations of greatest fill movement. The abrupt changes in profile at overcrossing structures along Route 210 were large enough to prevent traffic use of the road.

The Pavement Section

Damage to the pavement section was found to depend almost entirely on the response of underlying materials to the ground motion and shocks generated by the quake. The most prevalent types of damage consisted of lateral and longitudinal separation between slabs at joints, differential vertical movement and resulting loss of pavement plane, transverse and diagonal cracking, and buckling at compression zones.

Separation between slabs was usually small, with no detectable loss of constructed roadway plane and it resulted from the response of fills to ground shaking. In some cases the slabs separated by moving over the cement-treated base which remained uncracked, while in other cases the entire depth of section moved due to crack development in the CTB.

Slab separation was generally accompanied by differential vertical movements over areas of various sizes. Relative vertical and horizontal movements between any two adjacent slabs were often quite small, and at some locations even difficult to observe except from certain vantage points. Nevertheless, that the paving plane was disrupted could be felt when driving over the affected slabs at freeway speeds. These differential vertical movements occurred mostly in fill sections where the underlying fill material had subsided and created a dip; but some were noted in cut sections, where the slabs had been raised due to an upthrusting action at the toe of potential slides involving the cut slope.

Slab separation accompanied by differential vertical displacements along longitudinal joints was noted at a few locations in fill sections. These were locations of well-delineated slipouts within the fill. Where sufficient movement had occurred within the slipout delineation, diagonal cracks across slabs were in evidence.

Slab cracking at locations other than joints resulted from relative movement along the contact between cut and fill. In cases where original ground was very steep, movement was practically vertical and no differential horizontal movement along the crack was noted. Separation along joints was noted in some cases but was apparently absent at others. Diagonal cracks not associated with slipout delineations resulted from differential lateral movement of underlying materials such as the San Fernando Juvenile Hall Slide. This type of cracking was generally accompanied by rotational-type slab separation at joints on either side of the diagonal shear crack.

Slab separation in the vicinity of separation structures is believed to have been partially caused by the action of the structure being transmitted through the approach slabs to the pavement slabs, especially near those structures that experienced more violent motions.

The remaining type of damage to the pavement section consisted of compression buckles. This phenomenon was observed to occur almost exclusively in at-grade sections, the one exception being in cut along the line of contact between bedded sediments and alluvium.

Faulting due to tectonic activity previously described resulted in slight to moderate observed road damage. The well-defined Sylmar Fault, for example, crossed Routes 210, 5, and 405 almost normal to roadways at locations sufficiently removed from separation structures that no damage to bridges resulted. The fault crossed Routes 5 and 405 immediately south of their junction (just east of the Lower

Van Norman Lake Dam) where both roadways are in cut and resulted in very minor breakage in the pavement section. The thrusting effect, however, did disrupt the pavement profile as the north side of the fault break moved up relative to the south side. The break in roadway profile was not serious and was feathered-in fairly easily with AC patching. The fault crossed Route 210 about 550 feet west of the Maclay Street Undercrossing at a location where the roadway was almost at grade. Upthrusting of the west side was more pronounced than at the Routes 5 and 405 fault trace and a much wider zone was affected, as shown in transverse cracks in pavement slabs and separation and vertical displacements of slabs at joints. Damage was more severe than at the Routes 5 and 405 fault trace due to the greater relative vertical movement and the longer lengths of pavement within the fault zone.

The least amount of damage to pavements occurred in cut sections, whereas the greatest damage occurred on embankments. Except for occasional compression buckles, pavements at grade performed about as well as those in cut.

Drainage Structures

The degree of damage sustained by drainage structures was largely dependent on the response of the earthworks adjacent to them. Reinforced concrete boxes and pipes sustained cracks and joint separations ranging in size from hairline to one or more inches in width, and exposing reinforcing steel in many cases. Concrete pipes and boxes crossing the roadway at (or near) normal angles to centerline sustained less cracking but more joint separation by extension. Vertical offsets were noted in pipes where complete joint separation had occurred. Concrete pipes and boxes crossing the roadway at acute skew angles (or paralleling the centerline in some cases) sustained more cracking and less joint separation. Cracking in pipes occurred primarily within the middle thirds of quadrants and rarely in the crowns, inverts, or at springlines.

Corrugated metal pipes were observed frequently to be distorted in shape and profile but no separations at joints were found. From all appearances, the pipes were structurally intact. Frequent but minor cracking in concrete headwalls was noted.

Damage to drainage structures was confined to the Route 5/210 Interchange area, the area immediately south of the Route 5/Roxford Street Interchange, Route 14, and along Route 210. These are the locations at which ground motion appeared to be more intense based on overall observations of freeway damage.

Bridge Approaches

Widespread fill settlement occurred at bridge approaches on all freeways inspected for damage. On existing Routes 5 and 405 south of the Mission Hills, the problem affected the rideability, particularly on Route 405, even after hasty AC patches were placed at approaches to relieve the abruptness of changes in profile. However, the settlement was not so great as to prevent traffic use of the facilities prior to further repair work. The effect was much worse on Route 210, which was closed to traffic for an indefinite period, due to differences in roadway profile in excess of one foot.

The large differential settlements at all undercrossing structure approaches along Route 210 resulted from a combination of the following. First, the embankments subsided substantially due to densification of fill and foundation materials. Secondly, the single span structures settled very little relative to the fills since they were founded on piles driven to dense strata. As the fills settled away from the pile-supported abutment diaphragms, voids developed through which portions of the permeable backfill flowed. The remaining backfill material, reduced to a still smaller volume by densification, subsided more than the embankments, leaving depressions behind the abutment diaphragms. Reinforced concrete approach slabs, when still supported at the bridge deck by the paving notch and at the other end by fill material, bridged the backfill depressions and resembled ramps up to and down from the structure decks.

Differential settlement between approach fills and abutment backfill occurred throughout the inspected area. Even at locations where fill settlement was minimal the abutment backfill densified enough to result in up to two inches of settlement and development of transverse cracks along fill/backfill contacts. The cracks, with vertical offsets, were especially evident in construction areas where pavement had not been placed, and in AC pavements. In areas of concrete pavement, the settlement and cracks were blocked from view. However, vertical offsets between the AC shoulders and concrete approach slabs indicated the presence of a void directly below the approach slab. The lack of slab support over the backfill area and repeated traffic loadings would eventually lead to cracking of the slabs.

It is interesting that all backfill settlement observed, whether it was abutment backfill or trench backfill over pipes, resembled that which occurs under static conditions on most engineering works over long periods of time. Such backfill settlement is probably due to a combination of factors. However, the two most important factors are believed to be backfill placement methods and differences between backfill material and the in-place material.

Retaining Structures

Performance of reinforced concrete cantilever-type retaining walls was difficult to assess since only two were located in the apparent zones of most intense ground motion. One wall in the Route 5/210 Interchange area was constructed with the base serving as a strip footing founded in embankment. The profile of this wall was disrupted due to differential displacements within the fill, but no differential lateral wall movements were noted. The second wall, located just south of the Route 5/14 Interchange, experienced enough lateral movement to permit parallel cracks to develop in the AC pavement behind the wall top. Total permanent movements were small and did not indicate that structural integrity of the wall had been affected.

The lack of large permanent lateral wall movements would suggest that design procedures for static loadings utilize a factor of safety sufficient to prevent damage by earthquake loadings of the magnitude experienced during the San Fernando event.

General Comments

The degree of damage actually sustained by the freeways appears to have resulted largely from a combination of intensity and duration of strong ground motion, geologic conditions, and type of construction. While the intensity of the San Fernando event was sufficient to cause great damage, the duration was short enough to preclude complete collapse of several embankments, cut slopes, and drainage structures. The relatively loose upper alluvial materials densified and subsided or tended to subside, thereby increasing pore pressures in areas of high groundwater. The results were large total and differential settlements, lateral movement with some slumping of the upper portions of cut slopes, and fill foundation failures. Important geologic features in the larger cuts included bedding planes and joint/fracture patterns. The cut and fill type construction contributed significantly to the degree of damage due to fill and foundation densification, cracking, and spreading. The results included pavement slab separation, cracking, faulting, and (in some cases) loss of shoulder support. Transverse cracks and differential settlement through alternating cut/fill areas occurred and resulted in an abrupt rolling profile, and loss of pavement plane transverse to the roadway.

In view of the damage sustained by the freeways, it is interesting to note that damage to the Southern Pacific Transportation Company's railroad operating facilities was considered inconsequential in comparison to that caused by previous earthquakes at other locations (9). Furthermore, damage to the Los Angeles County road system, while termed substantial, was considered relatively small when compared to damage caused by rainstorms of January and February 1969 (10).

This disparity in degree of damage may be explained by considering the previously mentioned factors that are believed to have been largely responsible for freeway damage. Through the areas of most intense ground motion, the SP tracks and county roads were essentially at grade; i.e., either no cut/fill construction or, at most, a minimum amount. However, during the Arvin-Techachapi earthquake (M=7.7) of 1952, SP facilities that included heavy cut/fill construction suffered severe damage (11) and required extensive repair and reconstruction. Highways (State Routes 140 and 178 and U.S. Routes 99 and 466) also sustained heavy damage largely as a result of (1) slides, rockfall, and slumping in cut slopes and steep natural slopes; and (2), fill settlement accompanied by transverse and longitudinal cracking (12).

EFFECTS OF THE EARTHQUAKE

The effects of the earthquake on traffic movement became apparent during the first hour after the event. Portions of the damaged freeways were systematically closed to traffic and temporary detours established. Within a few hours, emergency contracts were let, mainly for shoring of structures and preparation of detours, and restoration planning was begun.

Six construction contracts were underway at the time of the earthquake. Two of the contracts would be terminated. Not until April 1975 (4 years after the earthquake), would the last of the damage be repaired and the terminated contracts completed.

Highway Closures

The Angeles Crest Highway (State Route 2), located north and east of Los Angeles in the San Gabriel Mountains, was closed by earthquake-induced landslides. Removal of slide debris permitted it to be reopened to traffic on the morning of February 15. No restoration was required for this route. (Route 2 was not inspected by the writers until February 19, 10 days after the earthquake.)

Interstate Route 5, the Golden State Freeway, the only direct route from Los Angeles north to the San Joaquin Valley, was impassable because of pavement damage and the loss of bridges at San Fernando Road. It was closed from the Hollywood Freeway, Route 170, near Pacoima to Calgrove Boulevard in Newhall, a distance of fifteen miles.

Damage to structures had closed old Route 14, Sierra Highway, and new Route 14, Antelope Valley Freeway, at the interchange with Route 5.

Interstate Route 210, the recently completed Foothill Freeway, was closed from I-5 to the end of the freeway at Maclay Street near the City of San Fernando. This route would remain closed until the completion of reconstruction on June 8, 1973. The adjacent and parallel conventional State Highway Route 210, Foothill Boulevard, had pavement damage but was passable. It should be noted that Foothill Boulevard, which continued to carry traffic, was constructed at grade as were other surface streets in the area and provided a means of comparing cut/fill and at-grade construction.

The San Diego Freeway, Interstate Route 405, could not be used from Rinaldi Street in Mission Hills to the junction with Route 5. This closure was because of pavement damage and the fallen truck ramp bridge at the Route 5/405 Junction. Later in the day Route 405 was closed from the Ventura Freeway, Route 101, to I-5 because of the evacuation of approximately 80,000 people living below the site of the Van Norman reservoir. It was reopened in stages as far north as Rinaldi Street by midday February 11. The northbound connection to I-5 opened in the late afternoon of February 12 and the southbound connection from I-5 was back in service the afternoon of February 14. This allowed the San Diego Freeway to operate in a normal manner until the damage was repaired.

Detours and Traffic Movement

First Stage Detour

Starting the evening of February 9 to the evening of February 11, one north-south lane in each direction was open through the area of damage. Northbound I-5 traffic was taken off at Roxford Street and routed via Roxford to Sepulveda Boulevard; then northerly on Sepulveda to San Fernando Road; then northerly along San Fernando to Sierra Highway, old State Route 14; then north to San Fernando Road, State Route 126. From there the detour went northwesterly on Route 126 back to I-5 near Newhall. The southbound traffic flowed along the reverse direction of the routing.

Second Stage Detour

This detour was opened to traffic during the evening of February 11. It allowed passenger vehicles but no trucks on two lanes each direction on I-5 and one lane each direction on State Route 14. Northbound traffic was taken from I-5 at Roxford Street over to Sepulveda Boulevard; then northerly on Sepulveda to San Fernando Road; along San Fernando to Sierra Highway; right on Sierra, then a left turn up the southbound I-5 off ramp to Sierra Highway. A cross-over connected this ramp to the patched northbound lanes of I-5. The northbound lanes became unrestricted at Calgrove Boulevard.

Southbound traffic left I-5 at Calgrove Boulevard and moved along the old state highway that had been replaced by the Golden State freeway. Near Sierra Highway, traffic rejoined San Fernando Road and went south on San Fernando to Sepulveda Boulevard, then along Sepulveda and back on the I-5 freeway at the Roxford Street on-ramp.

Trucks could not use the first stage detour at all and were not permitted on the second stage until February 19. During the first ten-day period trucks were required to use State Route 126 in Ventura County and then back into Los Angeles County on State Route 101, the Ventura Freeway. Depending on the route used between SR-126 and SR-101, this movement required an additional 60 miles or more.

In general, traffic moved well on the second stage detour. Congestion did develop every Sunday afternoon in the southbound direction. Delays of 45 minutes were experienced by motorists, who, once committed to the detour, had no easy turnaround or service stations available for approximately 10 miles.

To help ease frustrations, advance warning signs, roadside radio messages, "reassurance" signing, and service patrol vehicles were added to the detour. Advance warning signs were posted ahead of the congested area warning traffic of the length of delay. Traffic then moved in to a roadside radio advisory zone where signs were posted instructing motorists to tune to 830 kH on the radio for a recorded message. Service patrol vehicles were located along the detour to provide emergency repairs, water, and gasoline to any disabled motorist. Vehicles stalled in hazardous positions were pushed to a safe area off the road. "Reassurance" signs were placed to remind the motorist that the end of the detour would be reached. Instead of exclusively formal signs, an attempt was made to reduce tensions by adding a human touch. One such sign bore the message "End congestion 2 miles ahead - hang in there." Public response to this was very good.

Third Stage Detour

By the evening of the day of the earthquake, it had been determined that a detour to provide traffic service for a three, or more, year period would be required. This detour was located in the field by Friday, February 12, and plans for it were flown to Sacramento on February 15 for approval by the California Highway Commission.

Stage three provided six traffic lanes on I-5 with direct connections to SR-14. It skirted the devastated area just to the east of most of the damage. Construction was completed and the detour opened to traffic on April 14, 1971. This detour became the temporary Interstate 5 Freeway. Southbound I-5 had been closed for 66 days, 2 hours; northbound for 66 days, 4 hours and 30 minutes.

Projects Under Construction During the Earthquake

Six major highway projects were under construction at the time of the earthquake. Three had such slight damage that repairs were made and the contracts completed without difficulty. The remaining three suffered major damage and posed administrative problems. The contracts were:

1. Interstate 5/210 (Contract 07-068314) on which work had started June 5, 1968. The projected completion date

was the fall of 1971. Approximately 80% of the work had been done. Damage was so extensive that work was suspended on March 1, 1971 and the contract formally terminated October 8, 1971. Net payment to the contractor was $10,646,264, with $86,178 of the total being paid for work done after the earthquake.

2. Interstate 5/State Route 14 (Contract 07-068324). Work started April 9, 1969. Projected completion date was March 1972. The work which was 79% complete on February 9 was suspended March 1, 1971 and the contract was terminated December 4, 1971. Net payment was $19,048,309, with $64,490 of the total paid for work after the earthquake.

3. State Routes 14, 126 (Contract 07-035624). Work started on this contract during October 1969 and was 60% complete on February 9, 1971. The contractor did not choose to terminate and completed the work on July 27, 1972. The state did not pay for the damage caused to the project by the earthquake. The contractor suffered a loss on part of the damage and had part of it paid by insurance coverage. Ordered modifications were made under contract change order. The total added cost for these changes was $173,613. No working days were added to the project because of the earthquake.

Termination of Contracts 07-068314 and 07-068324 was done under Senate Bill 682. This bill was an urgency act approved by the governor on July 7, 1971. It declared that damage to state highway facilities under construction in the vicinity of the City of San Fernando had created a severe obstacle to public traffic and was a continuing danger. In addition to ordering the Department of Public Works to make a thorough investigation, it permitted the termination of contracts by agreement with the contractor.

The contractors were paid the cost of work completed at the time of the earthquake without the inclusion of a profit provided that this cost was not in excess of what he would have been paid if the contract had been completed.

The act also required that the Department of Public Works report to the Legislature the action it was taking to preclude the necessity of future legislation to terminate construction contracts damaged or destroyed by a natural disaster. The action taken was to add a new part to Section 7, "Legal Relations and Responsibility", to the State Standard Specifications.

Emergency Contracts

A number of emergency contracts were let starting within a few hours of the earthquake. These contracts provided for the construction of temporary detours, major detours, shoring of structures, epoxy repair work, and removing bridges. The contracts were:

Type of Work	Date of Start	Date of Completion	Cost
Temporary detours, remove bridge from railroad LA5 07-288814	2/9/71	2/12/71	$77,191
Shoring, Bledsoe Street O.C. LA210 07-286124	2/10/71	2/18/71	$13,993
Shoring and cribbing Foothill Boulevard O.C. LA210 07-286114	2/10/71	2/25/71	$41,083
Detour structure repair, temporary bridge road work LA5 07-285824	2/25/71	4/30/71	$1,281,482
Detour, structure repair, road work and inspection LA5 07-285834	3/15/71	5/28/71	$233,026
Repair drainage(5) structures by epoxy injection LA5, LA210 07-287864	12/9/71	2/18/72	$103,012
Repair Drainage Structures at East Canyon Channel and at Roxford Street LA5 07-287854	12/13/71	2/4/72	$11,110
		Total	$1,760,897

Investigation, Reconstruction and Completion

Investigation of damage was made by examining the pavement visually, removing shoulders and dropped slabs to determine the length that support had been lost, and by inspecting the inside of the storm drain system when it could be entered. Borings were made at some earthquake-induced landslides to determine the limits of removal.

These investigations disclosed that, in general, underground facilities performed well during the earthquake; and that cuts, fills, and pavement were damaged in a predictable manner. This indicated that a new location for the facilities was not needed. Reconstruction was done on the existing alignment. The only modifications were those made to bridges to make them less vulnerable to earthquake damage.

Reconstruction and completion were done under the following 9 contracts at a total cost of $25.5 million.

1. LA14-Route 14/5 Separation and Overhead (07-287814) consisted of constructing portions of 2 structures and restoring existing portions of the same 2 structures. Work started on February 28, 1972, and ended September 26, 1972, at a cost of $563,294.

2. LA5, at and near Balboa Boulevard Overcrossing (07-285844). The work consisted of building one bridge, removing portions of a bridge, constructing portions of a bridge, removing and reconstructing part of a rigid frame structure, removing and reconstructing wall sections and removing a bridge. Work was started on April 11, 1972, and was completed on April 20, 1973, at a cost of $2,651,910.

3. LA210, between Yarnell Street and Maclay Street (07-286104). This project was for the repair of the damaged freeway. Eleven bridges were repaired, highway lighting installed, drainage facilities reconstructed, and new pavement placed on a 4.3 mile project. November 1, 1972 was the starting date with completion on June 8, 1973. The total cost was $1,833,485.

4. LA210, between Yarnell Street and Maclay Street (07-286134). This project was for the reconstruction by resurfacing of 4.1 miles of the old state highway. Work started December 26, 1972, and was completed on June 4, 1973, at a cost of $160,197.

5. LA5, LA405 near Van Norman Reservoir (07-287824). This project was for restoring the highway by grading and surfacing with portland cement concrete and asphalt concrete. A bridge was constructed and 5 existing bridges were repaired. Start of work was February 20, 1973, with completion on April 23, 1974, at a cost of $2,651,910.

6. LA5, LA210 (07-287834). Work consisted of constructing a freeway. Bridge work included construction of bridges, removing a bridge, removing portions of bridges, and reconstructing bridges. March 15, 1973 was the starting date. Work was completed April 22, 1975. The cost was $7,534,208. This contract completed the terminated 07-068314 contract.

7. LA5,14 (07-287844). The work as let under this contract consisted of repairing, reconstructing and completing earthquake damaged freeways and bridges that had been part of the terminated 07-068324 contract. Work started March 26, 1973 and was completed February 11, 1975. Cost for this work was $9,683,854.

8. LA5, north of 5/14 Interchange (07-288304). This work was for the restoration of the highway by repaving with portland cement concrete. Start of work was on November 1, 1973 with completion of work on February 15, 1974. Total cost was $225,861.

9. LA405, 0.2 mile north of Rinaldi Street (07-287904) was for the removal of slide material and the construction of embankments. The $198,333 project started on November 28, 1973 and was completed on February 20, 1974.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based on field observations and comparisons made during a 6-week review period shortly after the earthquake of February 9, 1971, and briefer reviews subsequent to the Pt. Mugu (M=5.7±) and Imperial Valley (M=6.4) earthquakes of February 21, 1973 and October 15, 1979, respectively (13,14).

(1) Overall cut slope performance was considered good from the standpoint of slide development, although three slides did occur in large cuts. One was a bedding plane failure and the other two (one of which had shown pre-quake sliding tendencies) developed in the upper portions of the slopes and did not collapse completely. The potential damage from large cut slope failure is very real, however, and large cuts should be evaluated for their failure potential in high earthquake risk areas. The installation of a fairly large bench at, or slightly above, mid-height of the slope to retain or slow potential sliding masses, as well as widening at grade for storing slide material should both be given consideration where soils and geologic conditions are appropriate. Slope-flattening may be of benefit and should be considered in conjunction with right-of-way restrictions, damage potential, and other pertinent factors.

(2) Embankments were found to be susceptible to several types of damage, including: shear failure, subsidence due to densification, spreading, and longitudinal and transverse cracking caused by ground motion.

Damage to fills with respect to shear failure was considered minor despite the fact that three slipouts did develop in very good material. This suggests that high fills should be evaluated for their failure potential in high earthquake risk areas. Appropriate seismic coefficients should be used in design analyses for those sites where large fills over dynamically susceptible foundations are considered. Subsidence due to densification within the fill cannot be eliminated entirely and can be reduced only by keeping fill heights as low as practical. It is very doubtful whether increased compaction beyond that presently required would be economically desirable.

Although fill-spreading was observed at some locations, it was minimal. Further reduction of spreading could only be attained by providing slopes as flat as 4:1 or 5:1. This is normally impractical.

(3) Fill damage caused by foundation soil response to the earthquake consisted of: (1) subsidence due to the more pronounced densification of looser underlying sands and gravel; and (2), slipouts initiated in the weaker, fine-grained soils supporting portions of Route 5 just north of the Route 5/405 junction. No reasonable treatment outside the scope of current procedures is available for these problems. For example, the foundation failure on Route 5 resulted in a vertical subsidence of only one foot and even less lateral movement. The lack of greater movement indicates that the foundation material had good strength characteristics prior to the quake. Prevention or reduction of densification in loose foundation soils could be achieved only by costly predensification or by injection of material to fill the voids during construction.

(4) Another type of damage associated with fill subsidence is the differential movement at abrupt cut/fill contacts. This results in transverse cracks and vertical displacements across the pavement. For shallow fills the problem was minor and no special treatment is recommended. For the higher fills the problem cannot be reduced without significantly flattening the slope contact between original ground and the new embankment. This would not be a practical solution. Emphasis should continue to be placed on requiring excavation to 6 feet into the existing hillside and recompacting the soil as now required when placing new fills.

(5) Settlement at bridge approaches is a complex problem that involves the bridge and its foundation as well as the embankment and its foundation, plus structural backfill material and the different dynamic responses of each element. The damage would be minimized if the dynamic response of the bridge, bridge foundation, embankment and foundation soils were all identical, but this is not technically feasible. Findings do indicate, however, that longer and stronger approach slabs would be of benefit. Structural backfill should consist primarily of sand with some silt being desirable to produce a well-graded material that will readily densify by presently-used methods. Gravel, especially in large quantities, when mixed with uniformly-graded sands hinders compaction by normal methods and results in a condition highly susceptible to much further densification by severe ground shocks.

(6) Because of the complex nature of California's geology with its statewide system and subsystems of faults, it is sometimes necessary to construct a freeway across a fault

that is either known (or suspected) to be active. Field observations indicate that road damage by post-construction fault movement can be minimized if the roadway profile across the fault is kept as close as possible to natural ground. The desirability of crossing a fault in cut or fill depends on the geology, soils, and geometric conditions at each site. Particular attention should be given to cut depth, especially in loose or badly fractured materials. The employment of fill sections should be kept to a minimum.

(7) Geotechnical studies for transportation projects should be expanded to include greater emphasis on seismic risk considerations. Particular attention should be placed on the location and geometrics of critical interchanges when they are planned near known active faults. This information should be available at the planning stage when there is some latitude in highway location.

(8) Consideration should be given to using variable seismic load factors in the design of earthworks. Selection of the appropriate load factor should be based on evaluation of potential seismic intensity, estimated dynamic response of the foundation material, and the consequences of failure of the facility.

(9) In retrospect, the types of damage sustained by highway earthworks and foundations were the same that have been sustained by other land transportation facilities during previous earthquakes. The unique feature of the San Fernando event with respect to freeway damage probably was the extremely large volume of traffic affected. Similar damage to highway earthworks and foundations with similar results may be expected from future earthquakes of equal or greater magnitude that occur in large metropolitan areas.

IMPLEMENTATION

Findings from this study have been implemented since very early in the project. Perhaps the first step toward implementation was the development of preliminary guidelines for determining when seismic investigations and dynamic analyses would be advisable for proposed projects. These guidelines, intended for use by District Materials personnel, were included in Section IX, Earthquake Considerations, of California Test 130, Materials Report Outline (included here as Appendix A). Subsequently, an earthquake engineering course was developed and presented to District personnel. These efforts have resulted in more detailed discussions of seismic considerations in project geotechnical reports prepared by the Districts, and in-depth studies of particular sites where potential problem situations were recognized.

Cyclic triaxial testing equipment was obtained by the Transportation Laboratory to evaluate soil behavior under dynamic loading. This equipment has been used to test sand samples obtained from below the water table by the use of a TransLab-developed sampler which employs a freezing technique for sample retention.

As experience was gained from different sites an investigational procedure was developed by TransLab that is currently used for proposed projects (except bridges) when earthquake engineering studies are necessary. The procedure, described by Hannon and Jackura (15), utilizes the statistical approach to determine a design earthquake upon which subsequent analyses are based. The Office of Structures employs a different procedure based on site maximum credible bedrock acceleration (16).

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APPENDIX A

IX. EARTHQUAKE CONSIDERATIONS

A. The following criteria will serve as guidelines for determining when seismic investigations and dynamic analyses would be advisable. The criteria are divided into two categories; expected bedrock acceleration level on a state-wide basis, and roadway geometric and soil conditions at specific sites.

B. Bedrock Accelerations

The state map shown in Figure 1 depicts by means of contours the maximum expected bedrock accelerations from earthquakes in California. This base map was prepared in 1972 by the Division of Mines and Geology for use as a guide by engineers and planners. For purposes of highway planning and design, the three zones denoted as A, B, and C have been added. Zone A includes that portion of the state for which maximum expected bedrock accelerations will equal or exceed 0.4 times the force of gravity. Zone B is that portion for which maximum expected bedrock accelerations will be equal to or greater than 0.1 g but less than 0.4 g. Zone C is that portion for which maximum expected bedrock accelerations will be less than 0.1 g. These zones are not intended to correlate with any prior zoning from building codes or technical publications.

For highway projects located within Zone A, seismic investigations and dynamic analyses should be performed in accordance with the criteria outlined in the next section.

For highway projects located within Zone C, seismic investigations and dynamic analyses would not be necessary except for extreme local soil, geologic, or groundwater conditions as determined from normal materials investigations.

For highway projects located within Zone B, seismic investigations and dynamic analyses would not normally be necessary if such action can be justified by properly considering the criteria of the following section and local soil conditions.

C. Seismic Criteria

1. Embankments and Their Foundations

Embankments 150 feet or more in height, as measured vertically between downhill toe and hinge point, should be designed based on a dynamic analysis regardless of type or quality of foundation materials. Due consideration must be given the foundation in the dynamic analysis, however, since it constitutes part of the embankment-foundation system.

Embankments less than 150 feet in height should be considered for dynamic analyses according to a relationship between embankment height and predicted settlement within the foundation. This relationship is shown in Figure 2 and is based on the premise that greater foundation settlements for any given embankment height indicate increasing susceptibility to strength loss during a seismic event.

All approach embankments that support bridges or separation structures on spread footings should be analyzed dynamically to ensure non-collapse of structure end support.

2. Cut Slopes

Cut slopes for two-lane undivided roads should be designed based on dynamic analyses when the height of slope, as measured by the vertical distance between the toe and top of cut, is equal to or greater than 20W/TI, where W is the width between roadway hinge points and TI is the Traffic Index.

Cut slopes for multilane divided roads should be designed based on dynamic analyses when the height of slope is equal to or greater than 10W/TI, where W and TI are as previously defined.

3. Special Conditions

In addition to the geometric criteria listed in 1 and 2 above, special soil and geologic conditions, when encountered, should be taken into account through dynamic analyses.

These conditions include liquefaction potential of loose cohesionless deposits with groundwater within 40 feet of ground surface; old landslides uphill from new route locations, and old landslides to be crossed by proposed roadways.

X. MATERIALS AVAILABLE

A. Local Sources. (1) Discuss, list, or suggest specific local sources; and (2) Summarize pertinent current test data for local material sources investigated.

B. Commercial sources. List available sources and haul distances.

XI. RECOMMENDED MATERIALS SPECIFICATIONS

A. Include suggested specifications for all materials used in the structural section, plus Imported Borrow, Structure Backfill, Permeable Material, and Rock Slope Protection Material. Include estimated cement factor to be used in PCC Pavement.

B. Submit justification for any deviation from Standard Specifications and Standard Special Provisions.

XII. SOIL SURVEY AND TEST SUMMARY SHEET

A. This section provides a concise summary of field sampling and materials testing, and the physical relationship to the plan and profile of the planned improvement. If sampling or testing is performed, in general, it should be summarized in this section as a matter of record. The standard soils survey sheet is satisfactory in most cases, but it is the prerogative of the Districts to determine the exact presentation of this information that is most suitable to their needs.

For example, the characteristics of the terrain will determine the appropriate treatment of the plan and profile portions. The object is to present sufficient graphical information so that someone not immediately familiar with the area will quickly grasp the essential features of the project, and the relationships of the sampling and testing to that project.

B. The following comments are relevant to items listed on the standard soils survey sheet.

1. Sieve Analysis. The 1½", ¾", ⅜", No. 4 and No. 200 sizes are normally satisfactory for routine soil samples. Include the 5 micron and 1 micron sizes where the clay and colloid content may be an important consideration. For permeability studies the complete grading analysis from 1½" through 1 micron is usually required.
2. Sand Equivalent and Plasticity Index Portion. Summarize the results of these tests if performed, and include Liquid Limit with Plasticity Index.
3. R-value Portion. Summarize pertinent moisture and density information from R-value tests. These data are helpful in evaluating the overall characteristics of the soil.
4. Plan Portion. Where the characteristics of the terrain require a plan portion, show the location of all borings and samples on the plan portion along with such stationing, topography, and other details as necessary for study of the test data and profiles.
5. Profile Portion. Show the original ground lines, centerline profiles, stationing and/or post miles, location and depth of samples and borings, and description of the soils on the profile portion. The boring records should include the field soil description, sample depth, water table, and penetration resistance. If any boring records are shown separately from the centerline profile, they should be isolated by a dividing line to show that they are not located in respect to the profile. Their vertical scale should be clearly shown, as well as a legend to define the meaning of any symbols, abbreviations, etc., as used by the District.

XIII. APPENDIX

Examples of material which may be included in this section are: copies of reports by outside consultants, seismic surveys, special correspondence or memos, maps and cross-sections, photographs, etc.

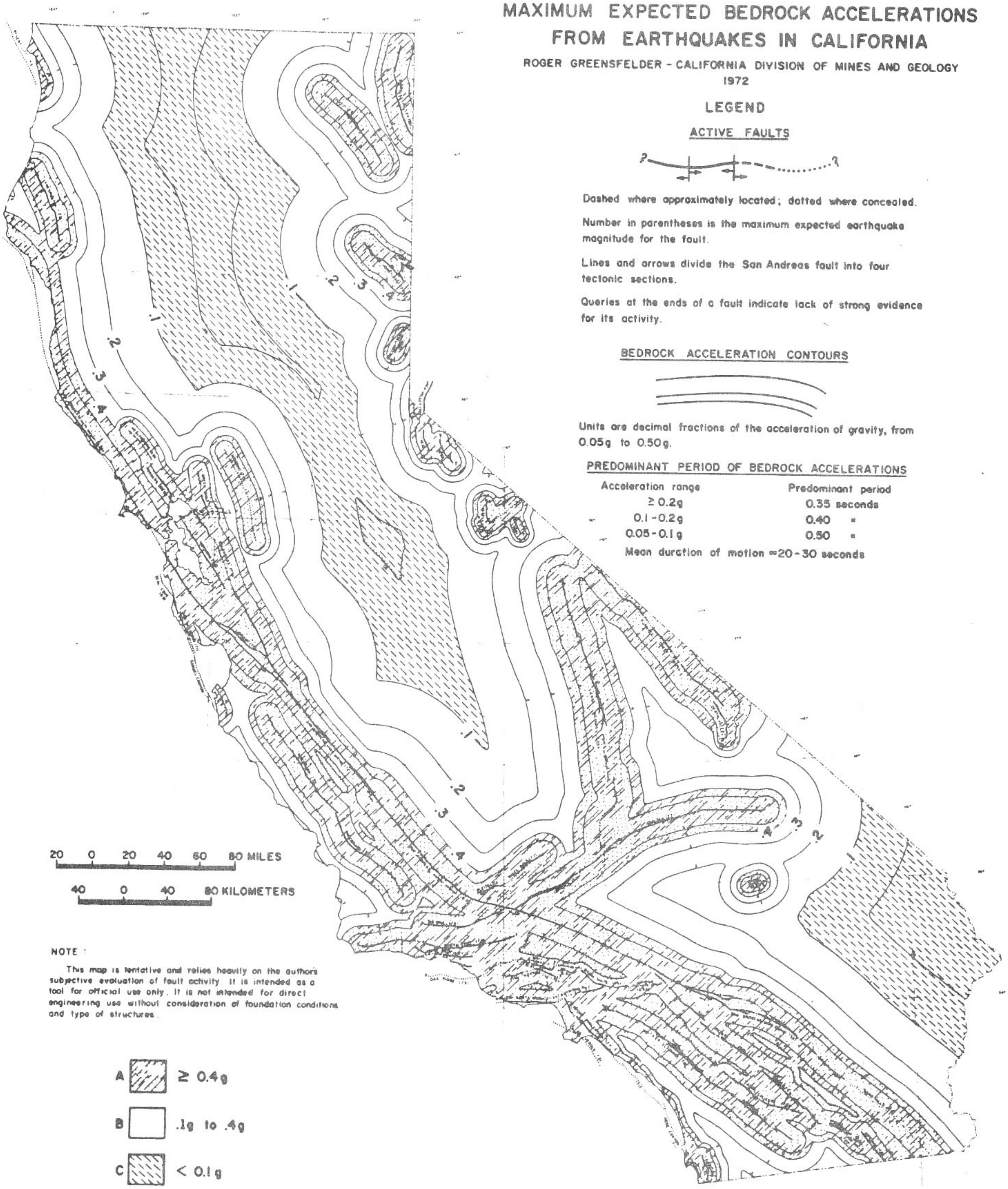
End of Text (6 pgs.) on Calif 130

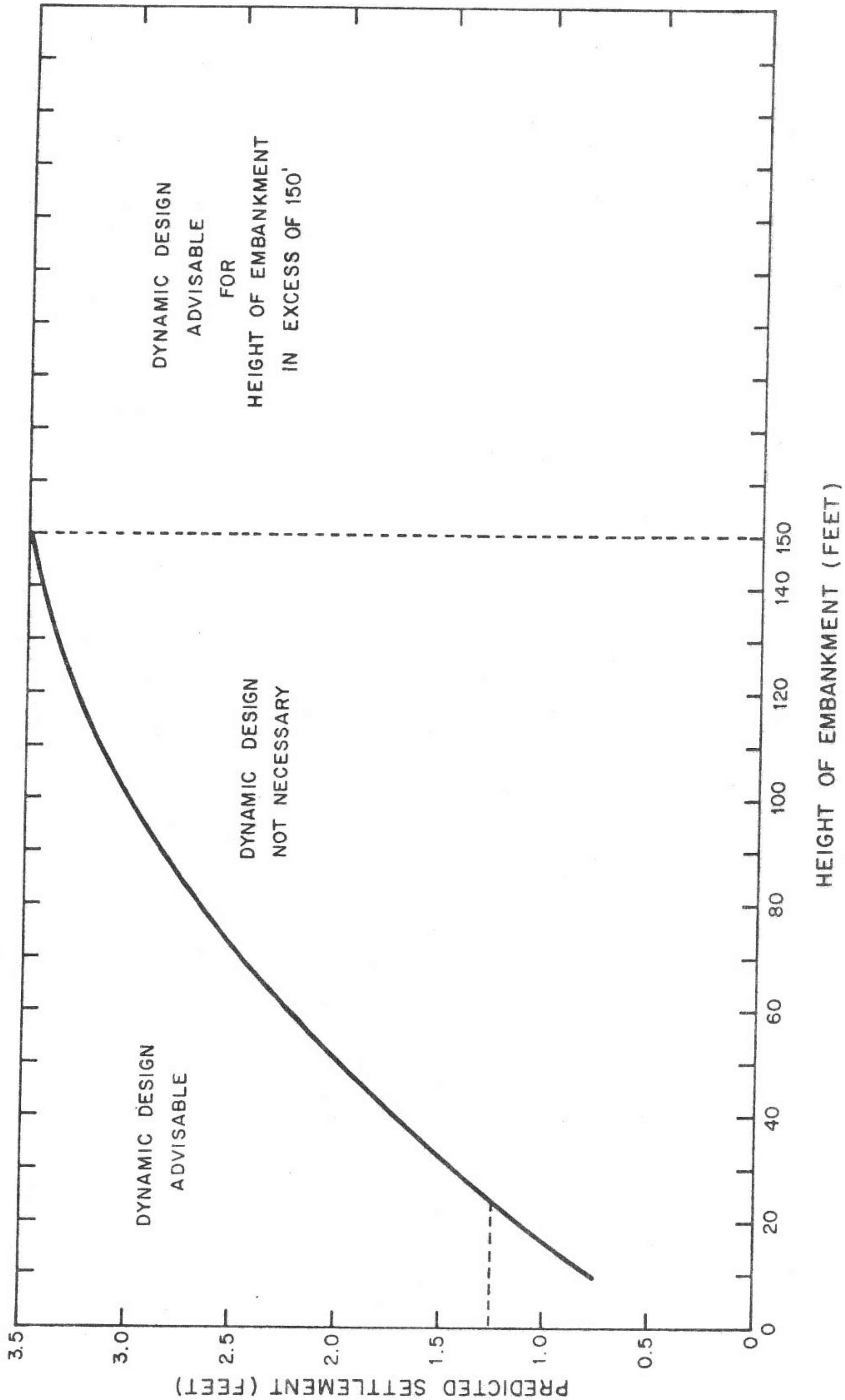
FIGURE I

Figure 1

MAXIMUM EXPECTED BEDROCK ACCELERATIONS FROM EARTHQUAKES IN CALIFORNIA

ROGER GREENSFELDER - CALIFORNIA DIVISION OF MINES AND GEOLOGY
1972





EMBANKMENT HEIGHT AND SETTLEMENT CRITERIA FOR DYNAMIC ANALYSES

FIGURE 2

77624—500 5-78 2,300

Photoelectronic composition by
CALIFORNIA OFFICE OF STATE PRINTING