ENGINEERING FEATURES

OF THE

SAN FERNANDO EARTHQUAKE OF FEBRUARY 9, 1971

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PREFACE

Because of its consequences, the San Fernando earthquake was a major earthquake from the engineering point of view, even though it was only a moderate shock in seismological terms. As a result of the many effects of the earthquake, a large number of detailed studies and reports will be forthcoming from a wide variety of sources, and the papers collected in this volume are only preliminary studies of some of the more important and interesting engineering features of the earthquake. The papers were prepared by staff and students working in earthquake engineering within the Division of Engineering and Applied Science at the California Institute of Technology.

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P. C. Jennings

TABLE OF CONTENTS

GENERAL FEATURES OF THE SAN FERNANDO EARTHQUAKE

by

George W. Housner

Introduction

A destructive earthquake struck the northern portion of the Los Angeles metropolitan area at six o'clock in the morning on February 9, 1971. This magnitude 6.6 shock was not a great earthquake, but it centered on the northern edge of a metropolitan area of over 8,000,000 inhabitants (Fig. 1.1), and this special circumstance had an important bearing on its overall effect. There was an unusual concentration of large and costly public works in the region of very strong ground shaking, and these sustained severe damage. The urban area subjected to the strongest shaking is shown in Fig. 1.2. Approximately 400,000 persons were subjected to very strong ground shaking (25%g or greater), and approximately 2,000,000 more to moderately strong shaking (15%g to 25%g). The major part of the metropolitan area was not strongly affected by the earthquake, and its resources were available to assist in counteracting the damaging effects of the shock. The overall damage has been estimated at some $500 million ($225 million private, $275 million public), but this was not so large as to constitute a severe shock to the economy of the metropolitan area. It has been reported that some 450 homes, 60 apartment buildings and 400 commercial buildings were so damaged as to be vacated; some 6000 homes, 200 apartment buildings, and 450 commercial buildings had appreciable damage; and some 20,000 structures had minor damage.

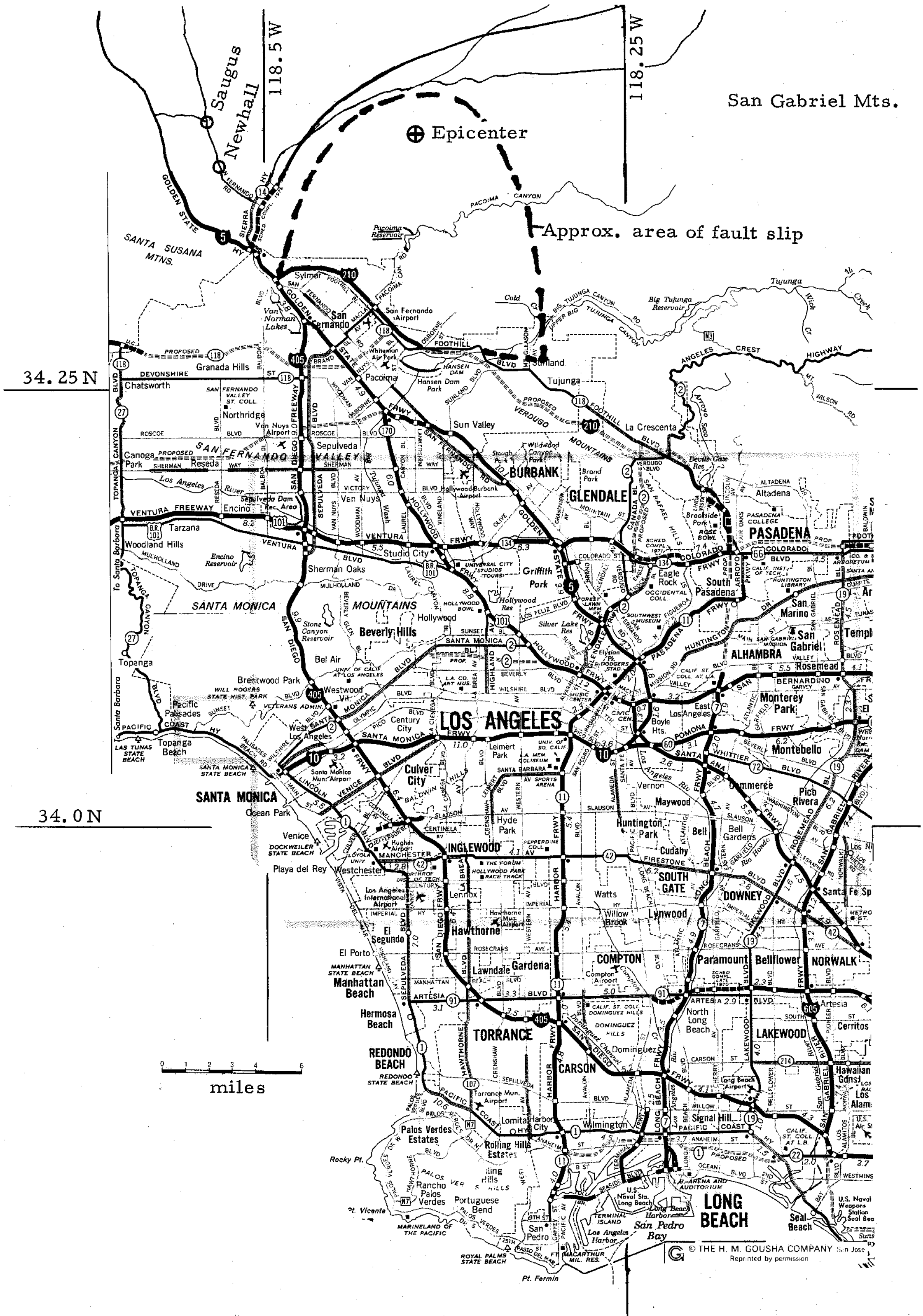

Figure 1.1 Map of western Los Angeles.

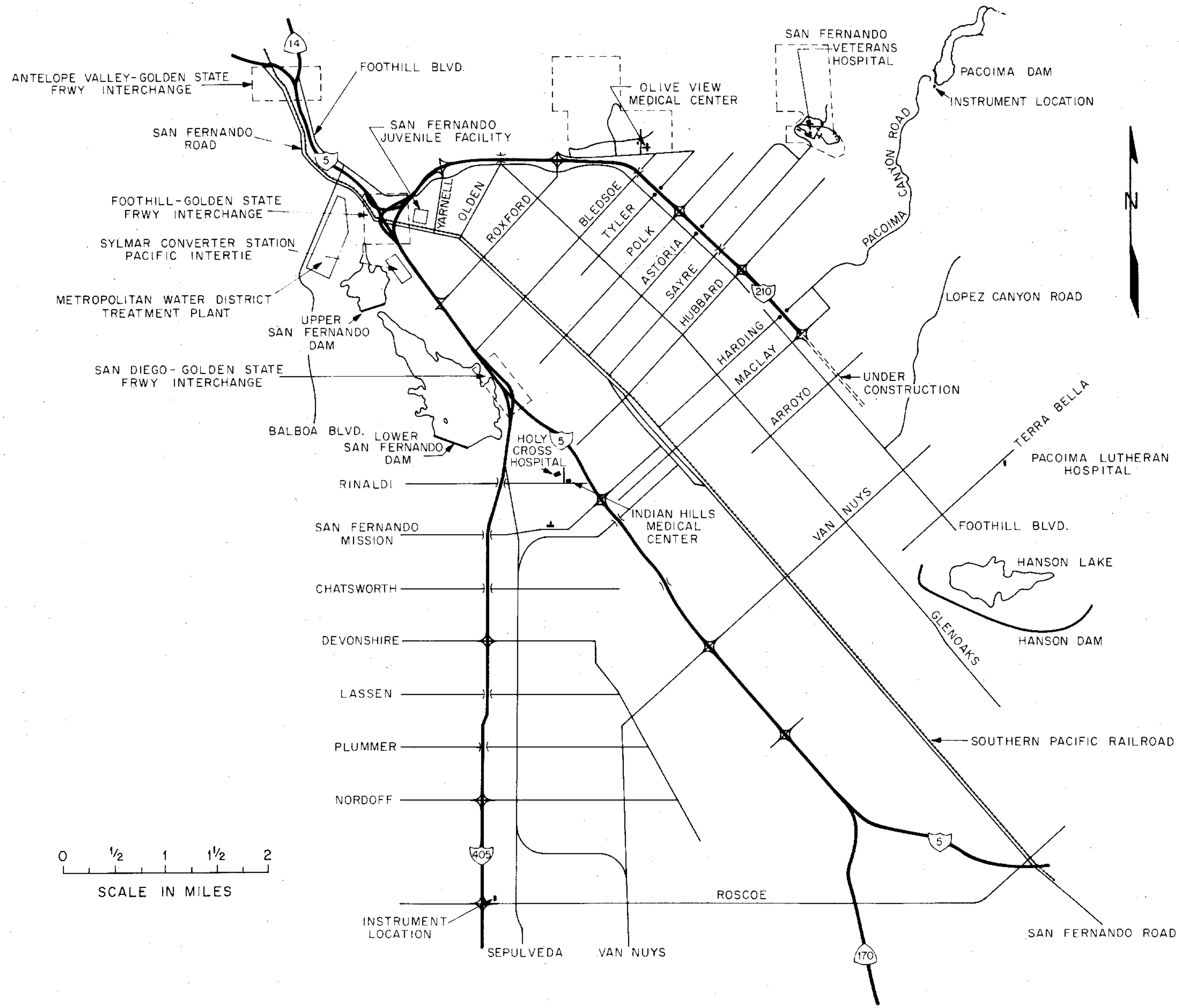

Figure 1.2 Plan of northern San Fernando Valley showing main points of interest and major streets.

Fifty-nine persons lost their lives because of damage caused by the earthquake. Forty-six were killed in the collapse of the Veterans Administration Hospital building, two were killed by a collapsing freeway overpass, four died in houses. Also, two in iron lungs died when power failed at the Olive View Hospital and it was reported that nine persons died from heart attacks brought on by the earthquake. In view of the large number of persons in the strongly-shaken region, this is a surprisingly small death toll which must be attributed to several fortunate circumstances. The duration of strong shaking was only 8 to 12 seconds, depending on distance, which permitted severely damaged buildings to survive without collapsing. Also, the San Fernando Valley is a relatively new urban area, and over 95% of the buildings and houses were constructed since 1933 when earthquake design requirements were first incorporated in the building code following the Long Beach earthquake. There were a number of pre-1933 buildings at the Veterans Administration Hospital, at the Olive View Hospital, and in the old city of San Fernando (which is completely surrounded by the newer part of the city of Los Angeles); and a relatively few additional old buildings were located in the Valley. Some of these old buildings were severely damaged and the death toll would have been much greater had the earthquake occurred several hours later when the streets were crowded and the buildings occupied. Fortunately, there was little traffic on the freeways on which a number of overpasses collapsed. Other fortunate circumstances prevented catastrophic release of the water impounded by the Van Norman dam, but 80,000 persons living below the dams were evacuated from their homes for several days as a precautionary measure. Had all of the circumstances of this earthquake been adverse, there could have been tens of thousands of casualties.

Modern buildings, designed according to the building code, performed for the most part as had been expected except for a few that collapsed wholly or in part. A number of modern buildings in the region of strong shaking were severely damaged and, had the duration of strong shaking been longer, they probably would have collapsed also. Some of the structures that should have been functional after the earthquake were not. For example, four hospitals in the northern San Fernando Valley were out of operation at the end of the earthquake just when the need for hospitals was greatest. The lesson is clear that certain structures should be designed to be stronger than the minimum requirements of the building code. These should include hospitals, schools that will be used for emergency shelters, buildings housing such things as emergency communications, fire departments and other agencies relied upon to cope with disasters, and also certain utilities that are depended upon to mitigate a disaster. These should all be designed to be functional after the earthquake. The building departments responsible for checking the plans of a structure should be given the authority to require that adequate earthquake resistance is provided by its layout and design.

In many ways, the San Fernando earthquake can be taken as an object lesson that indicates what might happen if a great earthquake were to shake the entire metropolitan area. It also provides an opportunity to learn how best to prepare for the great earthquake[1].

[1] The San Fernando Earthquake of February 9, 1971, Lessons from a Moderate Earthquake on the Fringe of a Densely Populated Region, National Academy of Sciences—National Academy of Engineering, Washington, 1971.

Geologic Features of the Earthquake

Many of the features of the magnitude 6.6 San Fernando earthquake differed from those of past earthquakes in southern California, such as the El Centro 1940 shock (magnitude 7.1) and the Long Beach 1933 shock (magnitude 6.3). This is attributed to the fact that the San Fernando earthquake was generated by thrust faulting, whereas the El Centro, and presumably the Long Beach also, were generated by strike-slip faulting in which the relative fault displacement was mainly horizontal across an essentially vertical fault plane. The fault which generated the San Fernando earthquake had not been generally recognized by geologists and did not appear on most geological maps. The area had not been studied in depth by geologists. The seismic history of southern California indicated that the general region of the earthquake had relatively low seismicity during the 20th century. From this it may be concluded that in a highly seismic region, such as California, the fact that an area has had a recent history of low seismicity does not necessarily mean that earthquakes are not likely to occur, but, on the contrary, may mean just the opposite. It is necessary to distinguish between areas that are tectonically active and those that are not.

According to the theory of continental drift (sea-floor spreading) the coast of California and the adjacent oceanic crustal plate, of which it is part, are moving in a northwesterly direction relative to the North American continent at a rate of a couple of inches per year. Intermittent strike-slip fault displacements on the San Andreas and other similarly-oriented faults provide the main accommodation to this movement. Because of other deformations, the San Andreas fault has a bend north of the location of the San Fernando earthquake, and more complex strains and displacements are required for compatibility in this vicinity. San Fernando Valley is on a portion of the earth's crust that is thrusting under the San Gabriel mountains and the recent earthquake was a sudden readjustment to these pressures. It has been inferred that the earthquake was generated by slip on a fault making an angle of approximately 45° with the horizontal, as shown in Fig. 1.3, with presumably strike-slip motion on a vertical plane beneath and along the western edge of the main slip plane[2,3]. The relative motion across the fault plane was generated by the rock above moving upward along the fault and the rock beneath moving downward along the fault. This reverse thrust faulting is shown in Fig. 1.3. It has been estimated that the relative displacement across the fault in its upper parts was approximately five or six feet in the direction shown by the arrows in Fig. 1.3, and also approximately five to six feet of horizontal left lateral component of displacement. The approximate extent of the slipped area of the fault is indicated in Fig. 1.1, but it should be noted that the ground surface over this slipped area is at a varying distance above the fault plane. For example, the epicenter at the northern end of the area was approximately 8 miles above the hypocenter on the fault plane, where the slipping began. At the southern end of the area, the fault plane comes close to the surface of the ground so that a significant portion of the energy release was at a relatively shallow depth. The center of the earthquake can be taken to be near Pacoima dam, and the central region of the earthquake

[2] San Fernando Earthquake, 9 February 1971, Report of the Seismological Laboratory, California Institute of Technology, March, 1971.

[3] The San Fernando Earthquake of February 9, 1971, U. S. Geological Survey Professional Paper 733.

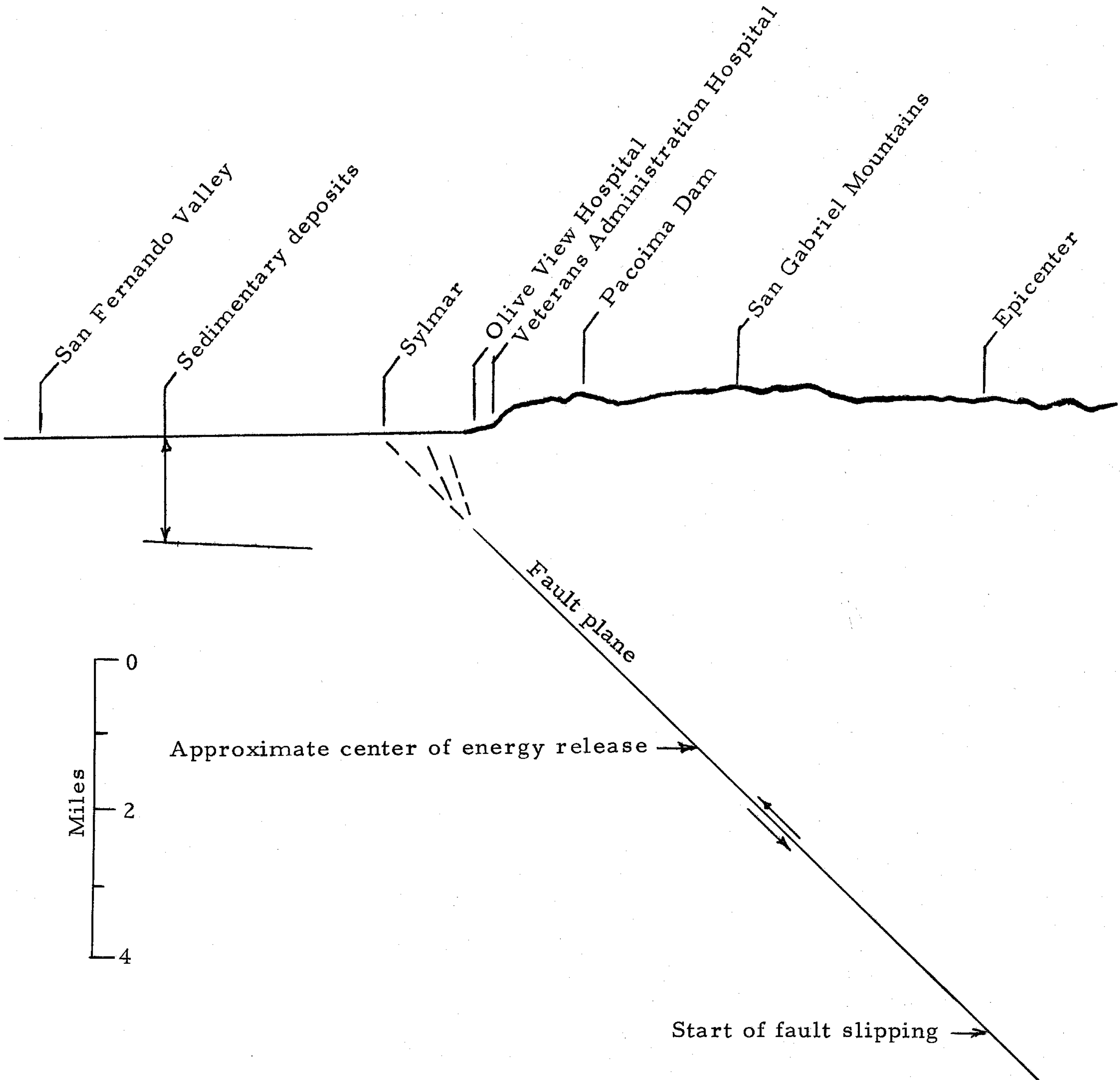

Figure 1.3 Schematic N-S cross-section showing inferred faulting.

can be taken to be the area within the dotted line in Fig. 1.1. The approximate area of fault slip was inferred from the location of epicenters of aftershocks as shown in Fig. 1.4. There have been numerous aftershocks; on March 31 one of magnitude 4.9 occurred which caused some damage in localized areas.

Surface expression of faulting has been identified as shown in Fig. 1.5. In those locations where the zones of ground displacement and deformation passed under structures the consequent displacement of the foundations caused considerable damage, and this was made worse by the ground shaking. Details of the observed displacements[2] are presented in Figs. 1.6 and 1.7 where it is seen that the vertical displacements across the fault trace range from a few inches to about six feet, and lateral displacements range from a few inches to about five feet. Significant surface traces of faulting have not been identified to the west of the Sylmar fault segment shown in Fig. 1.5. No surface traces of faulting have been identified on the Veterans Administration Hospital grounds or on the Olive View Hospital grounds. Neither have fault traces been identified around the southwest portion of the slipped area shown in Fig. 1.1 where such traces might be expected. It is presumed that the ground movements related to the fault displacement have been diffused through the upper part of the sedimentary deposits, and appear at the surface as widespread deformations of the ground. That such ground deformations did occur was evidenced by damage to rigid elements in and on the ground.

A determination of the absolute displacements of the ground surface must wait on the completion of precise surveys that start at relatively distant bench marks outside of the possible zone of influence of the faulting. It has been reported by the U. S. Geological Survey that measurements do show

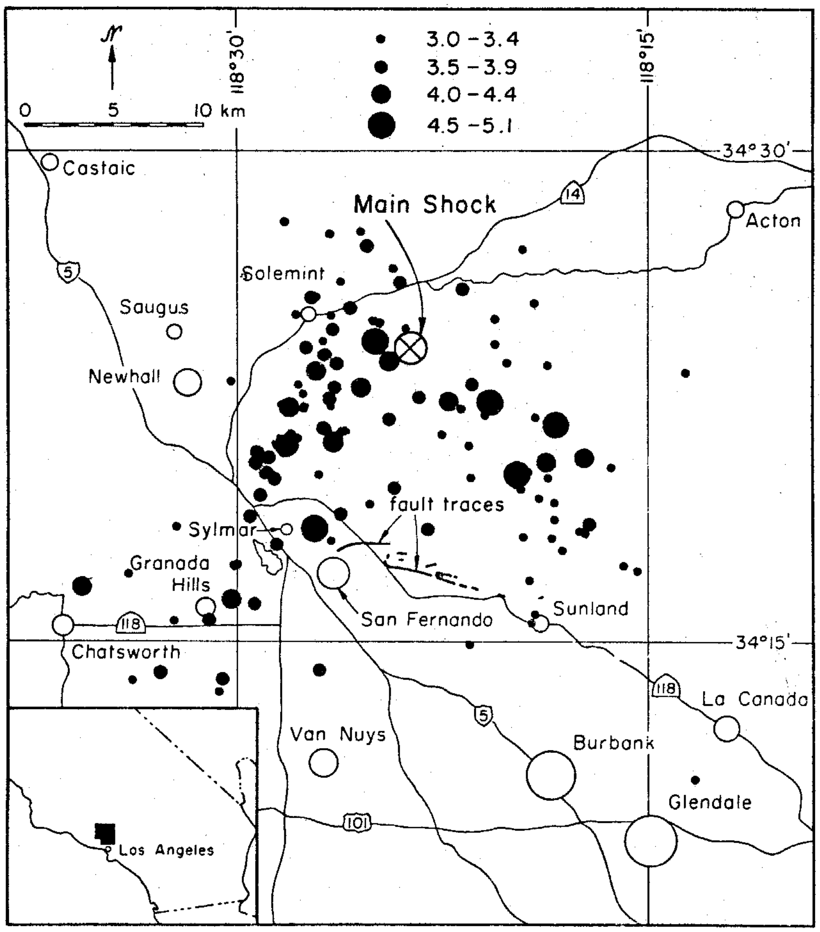

Figure 1.4 The San Fernando Earthquake of February 9, 1971. The map shows the epicenter of the magnitude 6.6 main shock and of representative aftershocks (magnitudes greater than 3) through March 1, 1971. (Report on the San Fernando Earthquake of 9 February 1971, Seismological Laboratory, California Institute of Technology).

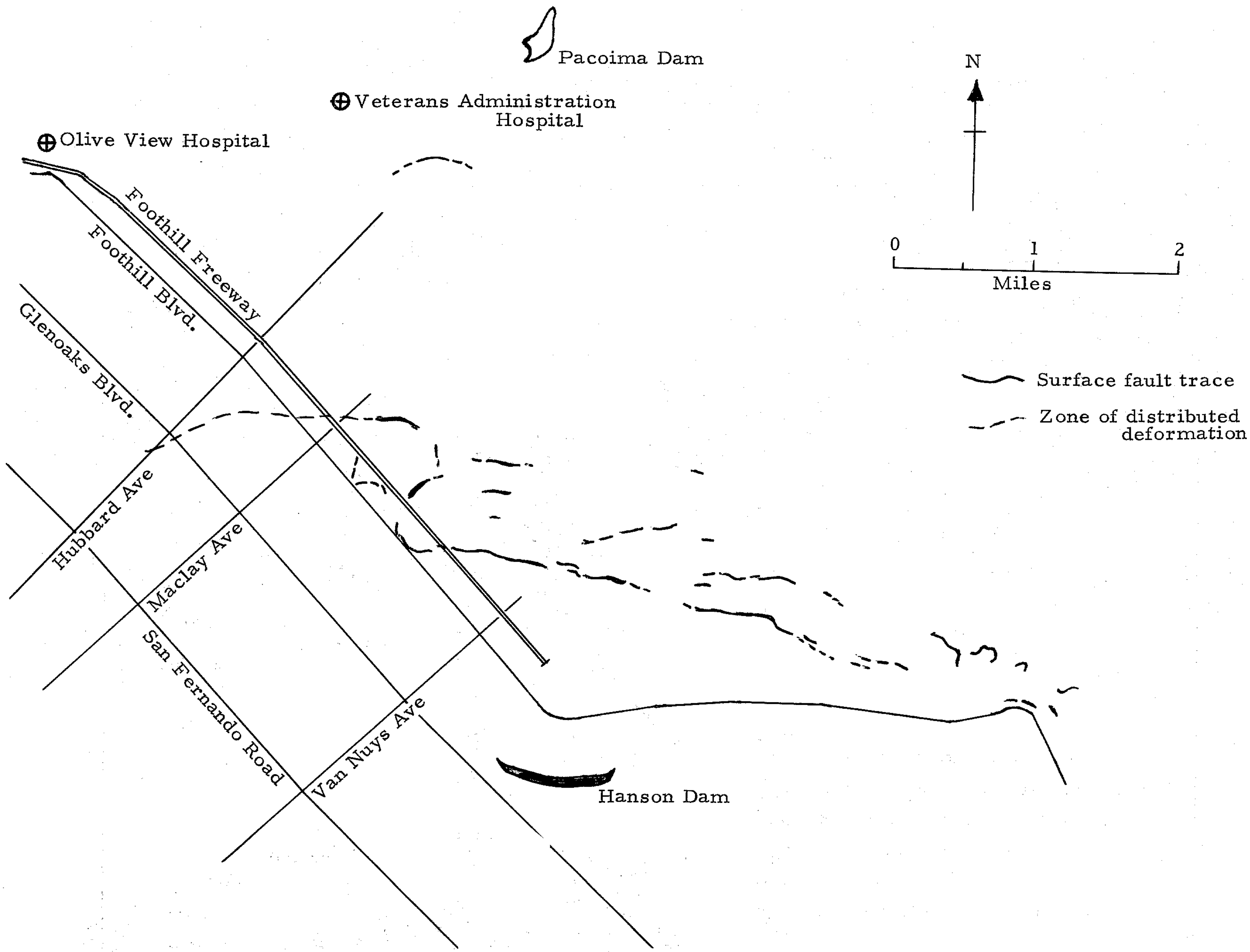

Figure 1.5 Location of surface faulting.

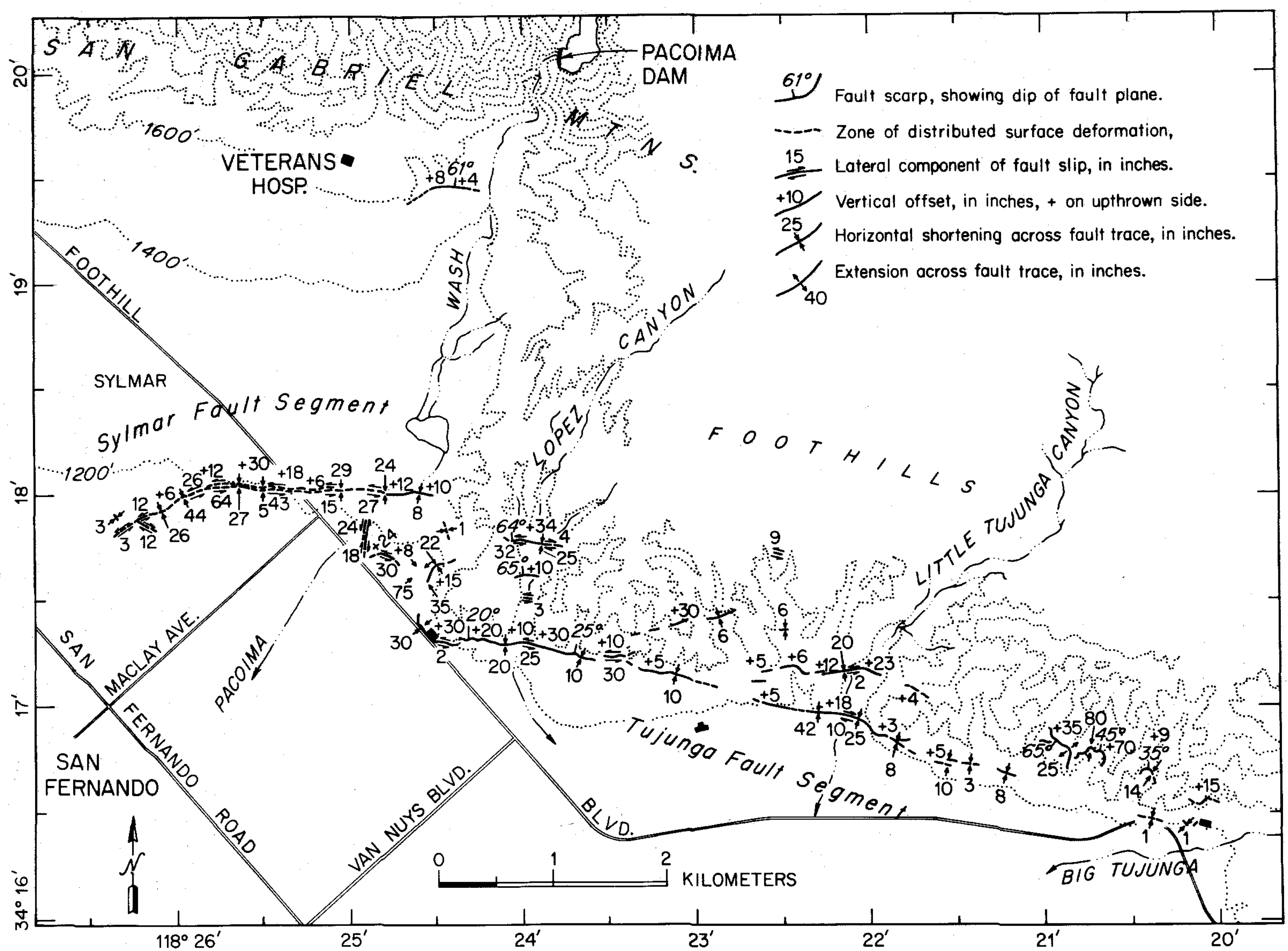

Figure 1.6 Details of surface faulting. (Report on the San Fernando Earthquake of 9 February 1971, Seismological Laboratory, California Institute of Technology).

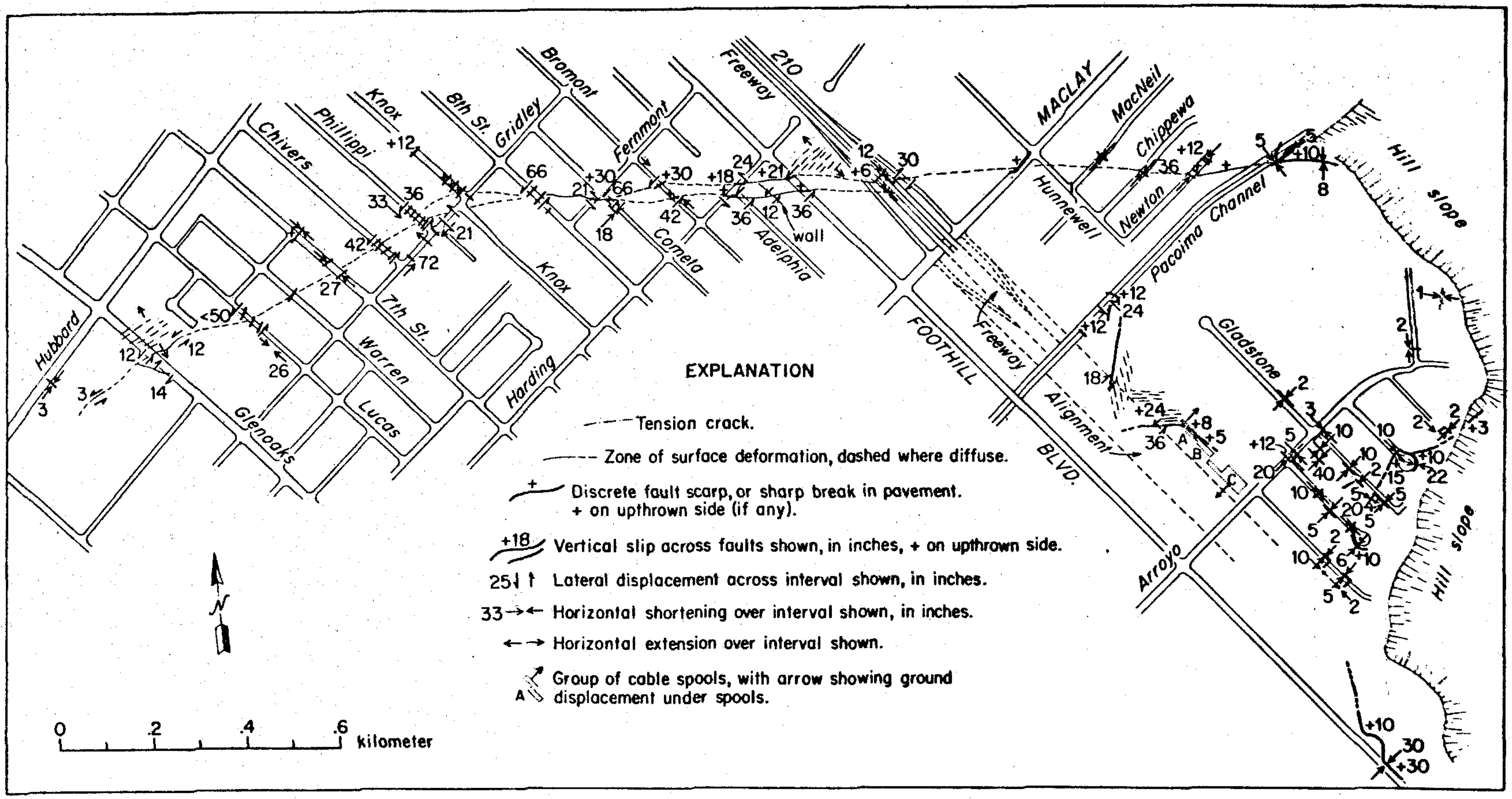

Figure 1.7 Details of Sylmar fault segment. (Report on the San Fernando Earthquake of 9 February 1971. Seismological Laboratory, California Institute of Technology).

that the ground surface south of Pacoima dam was elevated by the earthquake. The greatest amount of elevation was at a point approximately halfway between Pacoima dam and Hansen dam (see Fig. 1.1).

Movements of the Ground

The San Fernando earthquake is noteworthy for the extensive movements of soil which were much more in evidence than in any previous earthquake in southern California. These soil movements can be attributed to the deformations generated by the thrust faulting and to the poor consolidation of the surface alluvium. Some very obvious ground displacements were generated by surface faulting, and some zones of distributed strong deformation of the ground were clearly directly related to faulting (see Figs. 1.6 and 1.7). In addition, there were widespread deformations of the ground which would not have been evident had there been no urban development in the region. Buried gaslines, water mains and sewer lines suffered extensive damage because these rigid elements could not accommodate themselves to the strains in the ground. For example, a buried 16-inch diameter, 1/4-inch thick, steel gas pipe running the full length of Glenoaks Avenue and beyond it to the north, is reported to have had 53 failures. Some 300 miles of buried sewer lines were in the valley, and it has been estimated that approximately 15 miles of it will have to be replaced because of damage at numerous locations. The buried pipelines of the water distribution system are reported to have had approximately 1000 breaks.

In some instances, rigid elements on the surface showed evidences of strains in the ground beneath. For example, a 2-inch steel pipe in an open field buckled into the air because of compressive displacements at the points at which it was attached to the ground. Underground pipes and concrete water channels were damaged. Concrete curbs along streets were shattered in compression by compressive strains in the ground. In some cases, sidewalks buckled, and concrete highways suffered compression failures. Where asphalt covered the ground, its relatively high rigidity caused it to crack when the ground experienced straining. The points of buckling and cracking of these rigid elements were not always coincident with points of high strain in the ground, but more often they were the integrated evidence of ground straining that was distributed over an appreciable distance. Evidences of such ground deformations were observed from the GranadaHills area to the Tujunga area.

There were many landslides and rockslides on the steep slopes of the foothills and along the canyons of the San Gabriel mountains. There was evidence of landsliding in alluvial ground in the northern part of the valley, and also in filled ground, and there were movements attributed to a lurching of the ground. So far, it has not been possible to identify precisely the underlying causes of all of the ground movements, and there are different opinions as to the role played by fault movement at depth.

Recording of Ground and Building Motions

On the positive side, the earthquake provided a large amount of valuable data on ground and building motions that will notably increase engineering knowledge of earthquakes. Many strong-motion accelerographs in and around the metropolitan area recorded the ground shaking, and over 200 informative records of ground and building motions were obtained. Some 250 strong-motion accelerographs in southern California, maintained cooperatively by the National Ocean Survey and the California Institute of Technology, provided by far the greatest amount of strong-motion data recorded in any earthquake. The largest single owner of these instruments is the Earthquake Engineering Research Laboratory of the California Institute of Technology (25); the next largest is the National Ocean Survey (17), and smaller numbers are owned by the California State Department of Water Resources, the Los Angeles Department of Water and Power, the Los Angeles County Flood Control District, and other agencies. Owners of 60 buildings have supplied three accelerographs each as required by the city of Los Angeles building ordinance for buildings over seven stories in height[4].

It was especially fortunate that the Los Angeles County Flood Control District had installed an accelerograph on the rock ridge adjacent to its 372-ft high concrete arch dam built in 1928 in Pacoima canyon, for this location was directly above the plane of fault slip and was at the very center of the earthquake, (see Figs. 1.1 and 1.3). During construction of the dam a volume of insecure rock from the left abutment of the dam had been removed and replaced by a concrete thrust block; the accelerograph was located on the mountain ridge above where the rock had been removed. This was the first time that such a recording had been made in the central region of a destructive earthquake generated by thrust faulting, and unusually strong ground shaking

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[4]The present provision in the building code that requires strong-motion accelerographs to be installed in high-rise buildings is as follows: (1) General. Every building over six stories in height with an aggregate floor area of 60,000 square feet or more, and every building over ten stories in height regardless of floor area, shall be provided with not less than three approved recording accelerographs. (2) Location. The instruments shall be located in the basement, midportion, and near the top of the building. Each instrument shall be located in an accessible position.

was recorded with peak accelerations exceeding 1g. The mountain ridge underwent considerable relative movement of rock masses with extensive cracking of the gunite concrete surface coating. Further cracking and movements reportedly took place during the weeks following the earthquake. It is not clear to what extent the unusual location of the accelerograph influenced the recorded accelerations. The duration of very strong shaking was approximately 8 seconds. Response spectra for the Pacoima dam accelerations are shown in Figs. 1.8, 1.9 and 1.10. The shape of the S16°E spectrum is similar to that of the Parkfield, California earthquake (50%g maximum acceleration) but has ordinates about 50% larger and is stronger in high frequencies.

In general, except for the Pacoima accelerogram, the strong-motion records were similar to the usual earthquake records. The many records of ground accelerations will provide much more information about spatial variation of ground motions, influence of local geology, etc., than previous U.S. earthquakes, and analysis of these records is now underway at the California Institute of Technology. A preliminary examination of the spatial variation among the accelerograms indicates a very complicated behavior in some instances. For example, the peak acceleration at the Caltech Seismological Laboratory, on granite 17 miles southeast of the Pacoima dam, was 18%g. Three miles north of the Seismological Laboratory, at the Caltech Jet Propulsion Laboratory on firm sedimentary rock, the peak acceleration was 21%g. At the Caltech campus, three miles east of the Seismological Laboratory, on 900 feet of alluvium and sedimentary deposits, the recorded peak acceleration was 21%g in the basement of the Millikan Library building, and was 11%g in the basement of the Athenaeum building about 300 yards east of the library. These values, of course, are

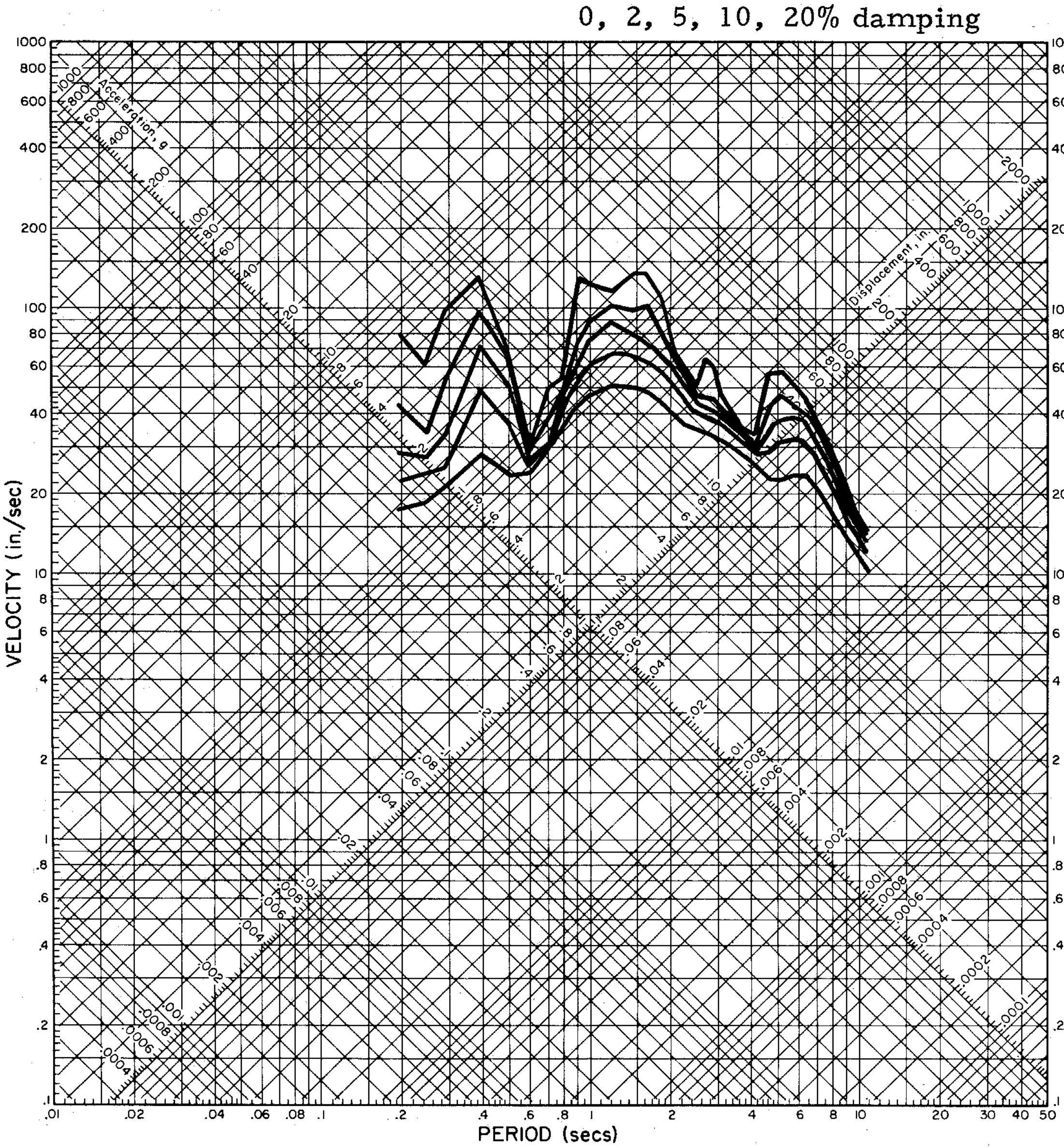

Figure 1.8. Velocity response spectrum $\left(S_d \frac{2\pi}{T} \right)$, component S16°E, Pacoima Dam accelerogram, San Fernando, California, earthquake, 9 February 1971. California Institute of Technology.

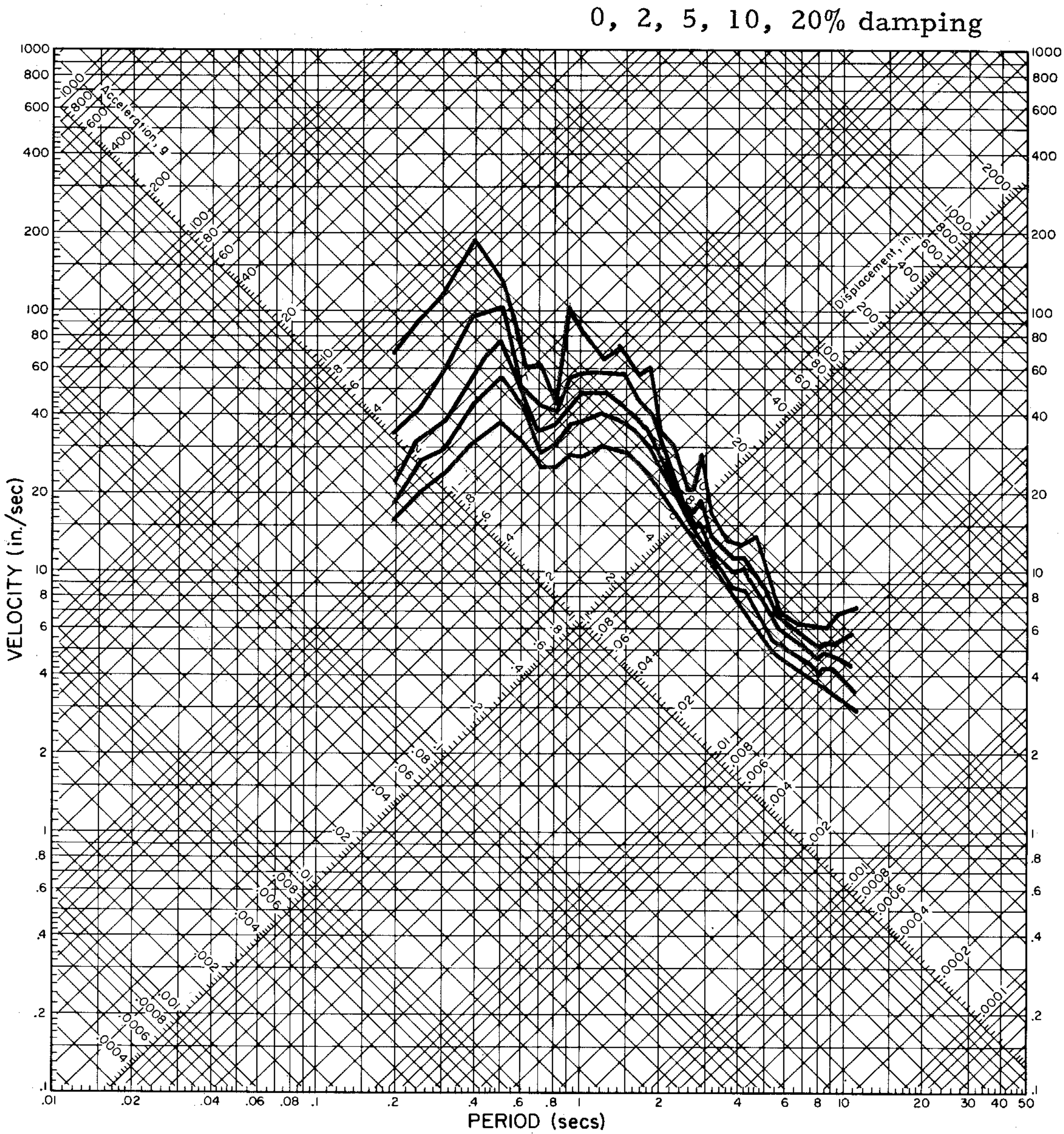

Figure 1.9. Velocity response spectrum $\left(S_d \frac{2\pi}{T}\right)$,
component S74° W, Pacoima Dam accelerogram,
San Fernando, California, earthquake, 9 February 1971.
California Institute of Technology.

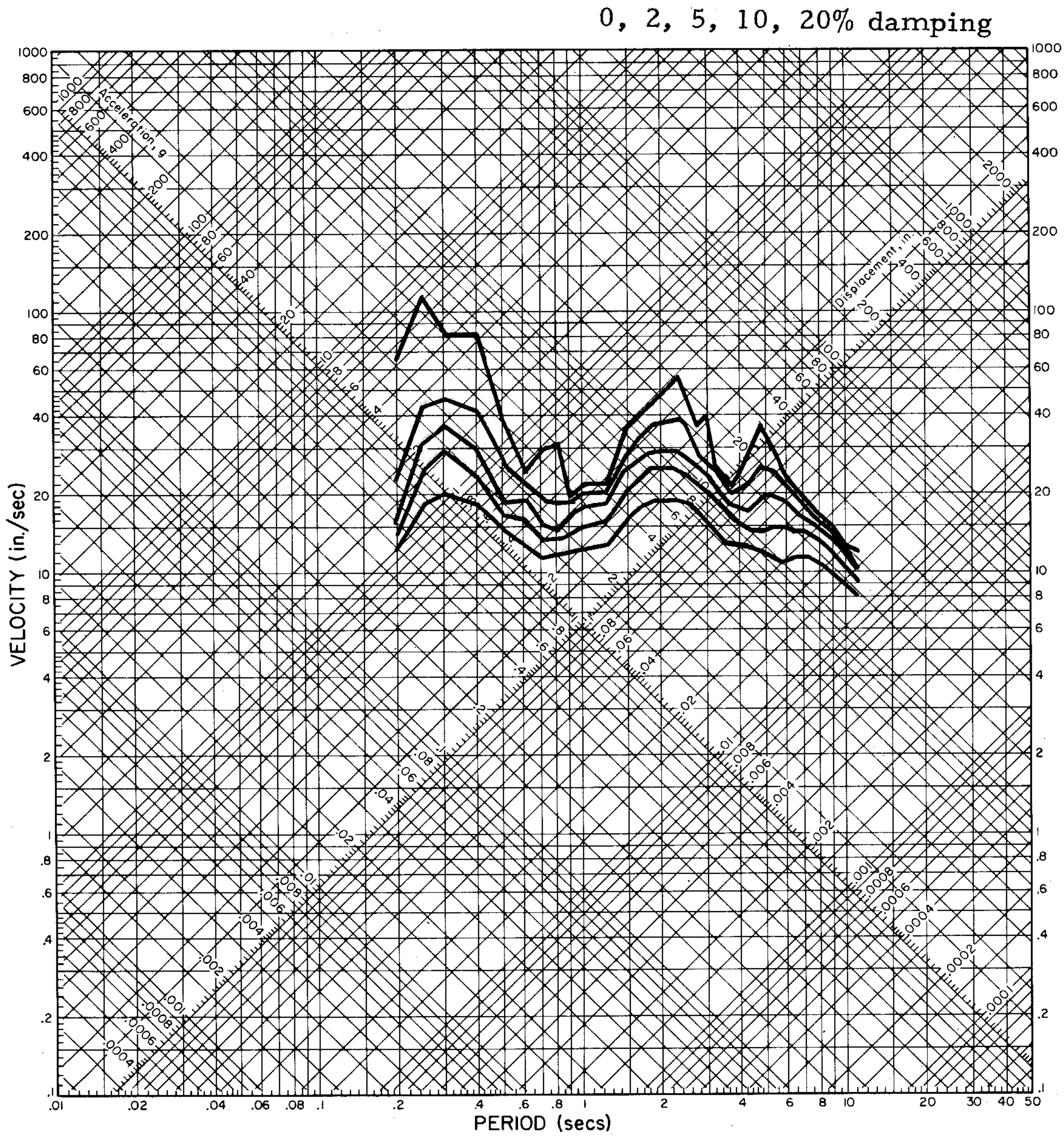

Figure 1. 10. Velocity response spectrum $\left(S_d \frac{2\pi}{T} \right)$, vertical component, Pacoima Dam accelerogram, San Fernando, California, earthquake, 9 February 1971. California Institute of Technology.

not necessarily indicative of the relative peak accelerations that will be recorded during future earthquakes. Peak accelerations recorded at various locations are given in the Appendix.

An example of the special information provided by the accelerographs installed in buildings is the recorded behavior of the Caltech Millikan Library. Before the earthquake, small amplitude vibration measurements made on the nine-story, reinforced concrete, shear-wall building showed the fundamental east-west period to be 0.66 secs. The earthquake record on the roof shows a prominent motion having a period of about 1 sec and an acceleration amplitude of 0.34g. After the earthquake, small amplitude measurements showed the period to be 0.76 secs. This anomalous behavior was not observed in the north-south component of motion. The change in period is believed to be the consequence of alteration of structural action of the large ornamental concrete grills bolted to the north and south walls.

The roof accelerograms of a number of multistory buildings had peak accelerations in the range of 25%g to 40%g. The records obtained in the high-rise buildings, for example, the 42-story Union Bank building, showed that the vibrations of the buildings were in good agreement with standard methods of calculating building response on the digital computer.

Aspects of Ground Shaking

It is generally agreed that the ground accelerations in the region of strongest shaking were essentially as great as would be expected in an earthquake of very large magnitude. If the magnitude of the San Fernando earthquake had been 8+ instead of 6.6, the east-west extent of faulting would have been some 20 times longer but a more severe ground shaking in the central region would not be expected, for a point on the surface of the ground would have been in the same proximity to the causative fault. The duration of strong shaking would have been longer, and the area affected by shaking would have been much larger for the magnitude 8+ shock; hence, the severity of damage and the extent of damage would have been greater.

Although there were many accelerographs in the Los Angeles metropolitan region, none was located in the northern San Fernando Valley and, hence, no records were obtained near the points of severe damage at the Olive View Hospital, the Veterans Administration Hospital, the Van Norman dams, the Pacific Intertie Converter Station, and the freeway overpass structures. The ground motion was recorded at Pacoima dam on top of the causative fault and the next closest record was obtained at the Holiday Inn at Roscoe Boulevard and Orion Street, approximately seven miles south of Pacoima dam. From these records estimates can be made of the intensity of ground shaking in the northern San Fernando Valley. It seems likely that the strong ground shaking in the northern part of the San Fernando Valley had peak accelerations in the approximate range of 30-50%g. Whether or not the ground shaking at specific sites had special characteristics is not possible to determine. It has been suggested that perhaps the ground motion at the Olive View Hospital and at the freeway overpass structures had unusually strong vertical components of motion. However, the Pacoima dam record and the Holiday Inn record do not bear this out for they had vertical components that were less strong than the horizontal components. The main Olive View Hospital building, the Psychiatric Day-Care building and the new power plant building all showed evidence of very strong horizontal shaking, but other buildings nearby survived with little damage. The old buildings that collapsed at the Veterans Administration

Hospital indicate that the ground shaking was strong; however, there were other old buildings at the site which, although damaged, did not collapse completely, and there were newer buildings at the site designed to resist earthquakes that survived with little structural damage. Without instrumental records, however, it is not possible to know precisely the nature of the ground shaking at these locations.

Damage at the Balboa Water Treatment Plant, just north of the Van Norman dams, was consistent with ground shaking having peak accelerations in the range of 30-50%g. The extensive damage to the Pacific Intertie Converter Station, just east of the Van Norman dams, was also consistent with ground accelerations in this range. The damage to the Van Norman dams also seems consistent with ground shaking having peak accelerations in the range of 30-50%g. Although the two old dams — the lower Van Norman dam and the upper Van Norman dam — were badly damaged, and were in the process of failing during the earthquake, the new dam at the bypass reservoir was not damaged.

It is possible that more thorough study of the earthquake will reveal additional information about the ground motion, but it is not necessary to suppose there were some highly unusual characteristics of ground shaking to account for the observed damage.

There was much interest in the effect of the shaking at the San Onofre Nuclear Power Plant; however, this installation was approximately 80 miles southwest of the center of the earthquake and the peak accelerations on the ground were only 1.5%g. Hence, the severity of shaking was not sufficiently high to be a good test of the performance of buildings and equipment.

An offshore oil drilling platform, approximately 60 miles northwest of the earthquake, vibrated with peak accelerations about 10%g when the ground beneath had a peak acceleration of about 3%g.

<u>Topography of Northern San Fernando Valley</u>

The locations of strongest ground shaking and the most significant damage were in the northern San Fernando Valley at the sites of Pacoima dam, Veterans Administration Hospital, the Olive View Hospital, the San Fernando Juvenile Facility, the freeway overpass structures, the Balboa Water Treatment Plant, the Pacific Intertie Converter Station, the Van Norman dams, the Indian Hills Medical Center building, and the Holy Cross Hospital. The topography around these locations, and the layouts of the facilities, are shown in oblique aerial photographs in Figs. 1.11 to 1.21.

An overall view of the northern San Fernando Valley and the points of most interest are visible in Fig. 1.11. This shows the Pacoima reservoir and dam, the Veterans Administration Hospital southwest of the dam, the site of the Olive View Hospital to the west of the Veterans Administration Hospital, and the Van Norman Reservoir southwest of Olive View Hospital. Figure 1.12 shows the topography of the San Gabriel mountains in the vicinity of Pacoima dam and the location of the accelerograph.

The Veterans Administration Hospital site and the surrounding terrain are shown in Fig. 1.14. At the time this photograph was taken, one month after the earthquake, the old buildings that had collapsed had been removed from its site, piece by piece, in the search for survivors and the pieces were placed in an adjacent open field. This building debris is visible in the center of the photograph.

Figure 1.11 Looking southwest over Pacoima Dam and San Fernando Valley. The Santa Monica mountains in the background separate Van Nuys from Hollywood. The southwest traveling street that lines up with the dam is Hubbard Avenue. The prominent street that crosses it is Foothill Freeway. The Veteran's Administration Hospital is just to the right of the northeast end of Hubbard Avenue. The Van Norman reservoirs are in the right background just beyond and parallel to Foothill Freeway. Olive View Hospital can be seen at the right, just north of Foothill Freeway.

Figure 1.12 Looking into the San Gabriel mountains northeast over Pacoima Dam. The caretaker's house is in the center foreground at the mouth of Pacoima Canyon. The location of the accelerograph is shown by the white arrow on the water pointing at the top of the ridge above the near abutment of the dam.

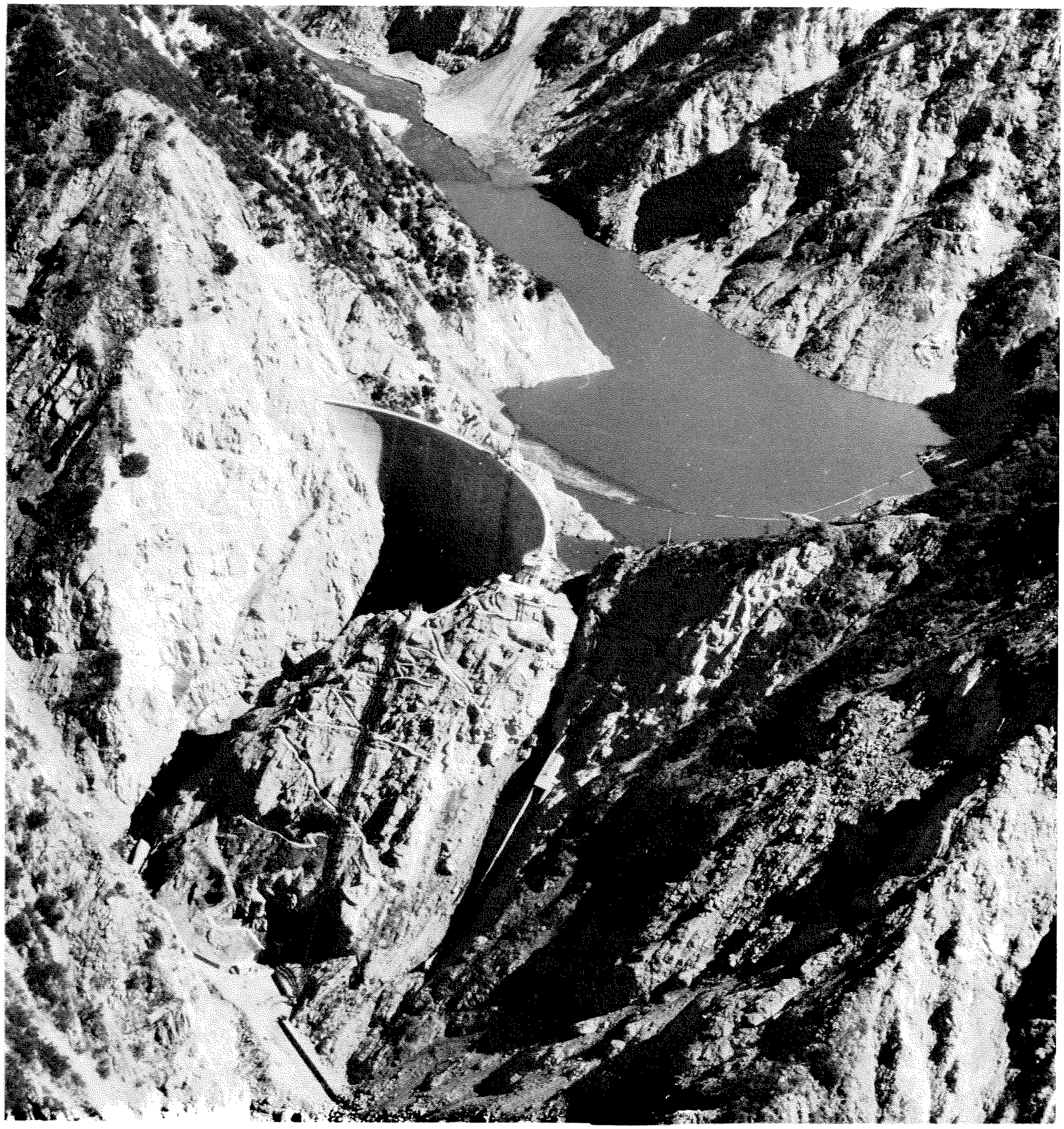

Figure 1.13 Near view of Pacoima Dam looking northeast. The small white accelerograph house is to the left of and adjacent to the larger white water tank on top of the ridge to the right and above the near abutment. This 372 ft high concrete arch dam was constructed in 1928. The dam was not damaged by the earthquake. Another view of the dam is shown in the paper by Hudson and Trifunac.

Figure 1.14 View of Veteran's Administration site looking northeast. Hubbard Avenue is on the right and Sayre Street is on the left running into the hospital grounds. In the field between Sayre Street and Hubbard Avenue, the heart-shaped area is formed by the pieces of the collapsed building which were trucked here as the remains of the building were carefully picked up piece by piece during the search for survivors. Pacoima Dam is visible in the background.

The Olive View Hospital site is shown in Fig. 1.15. The damaged new hospital buildings are visible just north of the Foothill freeway. The old hospital complex is to the left of the large new building. Many of these old pre-1933 buildings were severely damaged and a number collapsed. In the left foreground of the photograph there can be seen typical one-story wood and plaster residences which received little damage from ground shaking.

The Pacific Intertie Converter Station and the San Fernando Valley Juvenile Hall (detention home) are shown in Fig. 1.16. The Converter building, in the right foreground, is actually two buildings with a separation joint. The left-hand building houses the converters, and the right-hand building contains offices, controls, etc. The cages on each side of the converter building enclose electrical equipment, and additional equipment is in the yard to the left of the cages. The Juvenile facility, built in the form of an irregular pentagon, is northeast of the converter station. The Foothill freeway meets the Golden State freeway in the left middle of the photograph. The freeway interchange at this point was under construction at the time of the earthquake.

The lower Van Norman reservoir is shown in Fig. 1.17, a month after the earthquake, with the water level drawn down. The pass through the San Gabriel mountains is visible in the top center of the photograph. This pass is the reason why many of the facilities were located in the northern San Fernando Valley. The Owens Valley water for the Van Norman reservoirs, the Feather River water for the Balboa Treatment Plant, and electrical power transmission lines come south through the mountains at this location.

The lower Van Norman dam is shown in Fig. 1.18, and the downstream portion of San Fernando Valley is shown beyond the dam. In the foreground

Figure 1.15 Looking northeast over Olive View Hospital grounds beyond Foothill Freeway. The old hospital buildings are in the center midground of the photo and the large new hospital building is to the right. The intersection of Bledsoe Street traveling northeast) and Glenoaks Street (traveling northwest) is in the center foreground.

Figure 1.16 Looking northeast over the Pacific Intertie Converter Station in the foreground. The Golden State Freeway, under construction, is just beyond the Converter Station. The Southern Pacific Railroad tracks intersect the freeway and pass just in front of the San Fernando Juvenile Facility which is the pentagonal arrangement of buildings. In the background is Foothill Freeway, under construction.

Figure 1.17 Looking north over lower Van Norman Reservoir. The new by-pass reservoir is at the upper left of the lower Van Norman Reservoir. The upper Van Norman Reservoir is north of the lower reservoir, and the Balboa Water Treatment Plant is north of the upper reservoir. The San Diego Freeway in the foreground joins the Golden State Freeway at the upper right side of the photograph.

Figure 1.18 Looking southwest over lower Van Norman Reservoir. In the foreground, the San Diego Freeway merges into the Golden State Freeway. The lower Van Norman Dam is in the left background. The photograph was taken on March 15, 1971 when the reservoir was almost empty. The new by-pass reservoir dam is visible in the right background.

of the photograph the San Diego freeway merges into the Golden State freeway which goes north to the right-hand side of the photograph.

Figure 1.19 is a view looking southwest across the Golden State freeway and the new Bypass Reservoir which is between the upper and the lower Van Norman reservoirs.

Figure 1.20 shows the locations of the Indian Hills Medical Center building and the Holy Cross Hospital building. These new buildings were severely damaged by large vibrational stresses and strains. Analyses of their behavior during the earthquake should provide valuable information on design of buildings and on building code requirements.

Figure 1.21 shows the grounds of the Metropolitan Water District Balboa Water Treatment Plant which was under construction at the time of the earthquake. This plant will treat the water brought down from the Feather River to northern California. The site was excavated at the left side and brought up to grade with fill at the center and right side. The Golden State freeway passes the site in the background heading into the pass through the mountains.

Performance of High-Rise Buildings

Before the earthquake public concern had been expressed about the ability of new high-rise buildings in central Los Angeles to withstand earthquake ground shaking even though many of these were designed to be stronger than the building code requires. There was much relief when it was learned that none of the high-rise buildings had suffered significant structural damage. (Some of the multistory buildings in San Fernando Valley did receive structural damage). In general, the high-rise buildings vibrated as they had been expected to, and calculations of the response to recorded ground motions were in good

Figure 1.19 Looking southwest over the northern end of upper Van Norman Reservoir. In the left background is the new by-pass reservoir and earth dam which was not damaged by the earthquake. The upper Van Norman Dam and a portion of the reservoir is seen in the upper right of the photo. This dam started to fail downstream during the earthquake and the crest moved 5 ft.

Figure 1.20 In the center of the photograph Rinaldi Avenue passes under Golden State Freeway. The damaged Indian Hills Medical Center Building faces on Rinaldi, and just beyond and to the left is the Holy Cross Hospital building, also damaged. Lower Van Norman Reservoir is just outside of the upper left of the photograph.

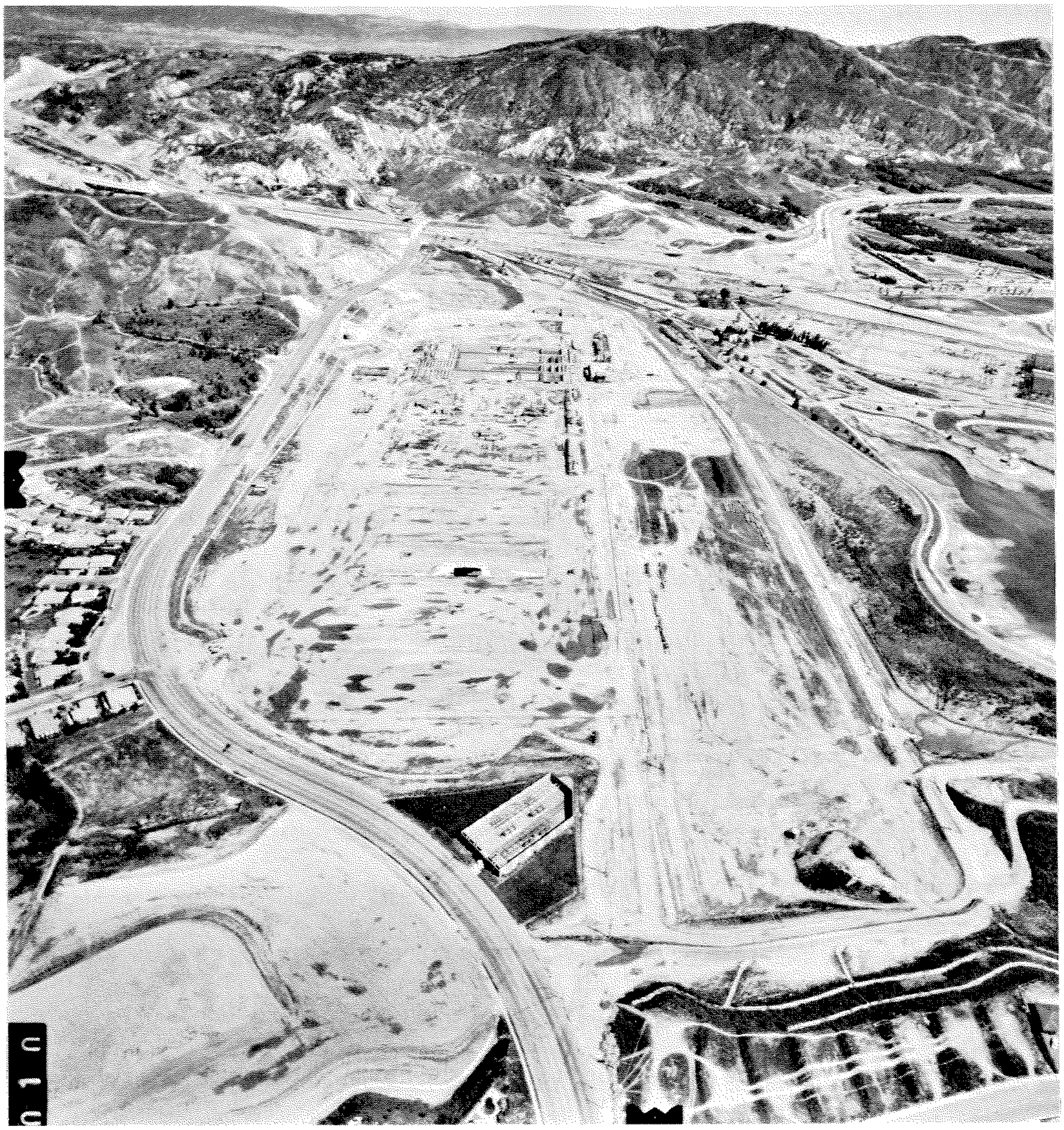

Figure 1.21 Aerial view looking N 27°W across Metropolitan Water District Balboa Water Treatment Plant site. The settling basins are in the background and the rectangular outline of the underground reservoir can be seen just this side of the contractor's work area. The small dark rectangle is a portion of the south edge of the reservoir not yet covered. Balboa Blvd. is on the left-hand side of the site, upper Van Norman Reservoir is on the right-hand side, and the Pacific Intertie Converter Station and the San Fernando Juvenile Facility are just beyond. The old San Fernando Electric Power Plant is in the small white building between the Intertie and the settling basins. In the center foreground is the Balboa Blvd. electrical switching station of the Los Angeles Department of Water and Power where cantilevered concrete walls were damaged by the earthquake.

agreement with the recorded response. Hence, this earthquake confirmed the method of analysis and design used for some of the high-rise buildings in which the response of the building to an assumed ground acceleration is calculated on the digital computer, and the design of the building is based on the computed forces and deformations. It must be noted, however, that the intensity of shaking in the region where the high-rise buildings were located was not a test of the ultimate strength of these buildings, as it might have been had the buildings been in northern San Fernando Valley. The peak ground accelerations at the locations of the taller buildings was in the range of 10-20%g. These ground accelerations produce strong vibrations and, in some cases, the amplitude of the shaking caused stresses to approach yieldpoint, but the structures were not near the failing point. The peak accelerations at the tops of many of the multistory buildings were in the range of 20%g to 40%g. It is clear that the design accelerations specified in the code are much smaller than the accelerations actually experienced by many structures. It also appears that the dynamic strengths of some structures are greater than implied by the allowable static design stresses specified in the building code. It would be desirable to revise the code to be more realistic in these aspects, and to ensure the dynamic strength necessary to withstand very strong ground shaking without collapse and without damage so severe as to be hazardous. It would also be desirable to have an occupancy factor in the code that would require greater resistance for buildings with large occupancy, and with special occupancy such as schools, theatres, etc.

The good agreement between digital computer calculation of the response of high-rise buildings with the recorded response indicates that for major structures it is feasible to base the design upon a dynamic analysis in which the response of the structure is compared to time histories of ground acceleration of an intensity likely to be experienced at the site. The expertise among engineers has now reached the point where it is practicable to make this a requirement in the building codes for major structures. Although the high-rise buildings did not suffer significant structural damage, they did vibrate with sufficiently large amplitudes so that in many cases there was appreciable architectural damage — that is, cracking of plaster walls and ceilings, damage to electrical and mechanical equipment, particularly to elevators, etc. In a number of cases, the architectural damage exceeded \$100,000. Certain of this architectural damage was of a nature that could have been avoided at little extra cost had forethought been given to the problem. It is of interest to note that even though the high-rise buildings vibrated strongly, there was relatively little breakage of window panes, which indicates good design.

In a large metropolitan area it is very difficult to obtain complete data about the number and types of structures subjected to ground shaking, but an idea of the number of multistory buildings exposed to the ground shaking can be derived from the following information. The Building Owners and Managers Association of Los Angeles reports that 207 office buildings, exceeding seven stories in height, were constructed in the metropolitan Los Angeles area in the period 1947-70 (years of start of construction). These had a total permit evaluation of \$1,291,000,000. The distribution of these buildings according to number of stories is shown in Table I. The twin-52-story buildings were under construction at the time of the earthquake. Their structural steel frames and the concrete floors were essentially complete, the exterior finish was approximately 70% complete, but little of the interior finish had been installed. No damage was sustained by these structures. The valuation of the 207 office buildings is shown in Table II.

Damage to Old Structures

Old buildings that had not been designed to resist earthquakes received much damage. Those close to the center of the earthquake were severely damaged and some collapsed, and those at a greater distance sometimes suffered appreciable damage. Some of the old buildings as far away as 30 miles lost portions of their walls, were severely cracked, and suffered other kinds of damage. One man was killed in central Los Angeles when a brick wall of a building fell on him. This re-emphasized the fact that the old buildings are a major earthquake hazard in southern California. There are over 20,000 old masonry buildings in the Los Angeles area, and it is necessary only that an earthquake come close to make then a great hazard to the occupants, to passers-by on the sidewalks, and to the occupants of adjacent buildings. The problem posed by the occupancy of these old buildings should be faced by the State and local governments, and a program for strengthening or razing these buildings be initiated.

Some 8,000 old buildings in metropolitan Los Angeles have had their parapet walls removed, or strengthened, under a special program for hazard reduction. It is clear that this program did indeed reduce the earthquake hazard of these buildings under conditions of moderate shaking, but the structures are still very hazardous under conditions of strong ground shaking.

TABLE I

Number of Office Buildings Constructed in Metropolitan Los Angeles, 1947-1970

Number of Stories	Number of Buildings
8	31
9	20
10	28
11	16
12	31
13	22
14	10
15	8
16	4
17	3
18	3
19	5
20	5
21	2
22	5
23	1
24	0
25	0
26	3
27	2
30	2
32	2
42	2
52	2
	207

TABLE II

Construction of Office Buildings
Eight Stories and Over in
Metropolitan Los Angeles, 1947-1970

Year Begun	No. of Buildings	Sq. Footage in 1000 sq. ft.	Permit Valuation in $1,000,000
1947	2	615	7.8
1948	1	504	11.9
1949	1	120	1.2
1950	3	507	5.0
1951	2	250	4.8
1952	2	320	6.5
1953	1	273	4.3
1954	1	201	2.8
1955	5	662	10.9
1956	2	612	18.2
1957	4	2,140	27.5
1958	8	1,489	32.9
1959	8	1,276	35.4
1960	6	1,060	28.2
1961	11	2,096	54.1
1962	17	3,893	96.5
1963	13	3,357	76.0
1964	11	1,707	47.5
1965	17	3,993	98.7
1966	11	3,091	85.6
1967	11	3,216	90.4
1968	20	6,088	156.7
1969	27	9,185	252.6
1970	23	4,975	135.6
	207 buildings	51,624 sq. ft.	$1,291 million

Damage to New Structures

Some new structures that had been designed to resist earthquakes were damaged in the region of strong ground shaking. Examples of these are the main Olive View Hospital building, the Psychiatric Day-Care Center and the new power plant building at Olive View, the one-story tilt-up buildings in San Fernando Industrial Park, the Pacoima Lutheran Hospital, the Indian Hills Medical Center, the Kaiser Memorial Hospital, the Holy Cross Hospital, the Union Bank Building (Sherman Oaks) and others. These buildings appear to have been designed according to the requirements of the building codes that were in force, and did not have any gross defects in their construction. It must be concluded that the ground shaking was more severe than the building code requirements premised. The earthquake demonstrated that a building designed to the minimum requirements of the building code, without any extra capacity to resist earthquakes, is susceptible to appreciable damage from strong ground shaking and is susceptible to collapse in the event of very strong ground shaking. The minimum requirements of the building code should be revised to insure that collapse or hazardous damage will not result.

The damage losses for some structures were large. The new Olive View Hospital buildings cost $27,000,000, and this facility was essentially a total loss. The Balboa Water Treatment Plant under construction at the time of the earthquake was a $45,000,000 project and it received an estimated $5-$10 million of damage. The $6.5 million San Fernando Juvenile facility was extensively damaged and is probably a total loss. An estimated $15 million, or more, of damage was sustained by the bridges and overpass structures on the Golden State, Foothill, San Diego and Antelope Valley freeways. Damage at the large Thatcher Glass Factory between Newhall and Saugus has been estimated at $10 million. A number of industrial buildings in the San Fernando Valley were also damaged but the mounting losses have not been made known.

Behavior of Dams

The Pacoima dam (concrete arch) in the center of the earthquake was not damaged by ground shaking, but one abutment showed evidences of movement and distortion during the earthquake, and it was reported that the chord distance between abutments had shortened by about one inch. There were many earth dams in the region of moderately strong to strong shaking (15%g or greater), and the new dams designed during recent years withstood the earthquake very well. Some of these showed evidence of having deformed during the earthquake, but they had no significant damage. On the other hand, the old dams behaved badly. The two old hydraulic earth-fill dams at the Van Norman reservoirs both were in the process of failing during the earthquake, and had the shaking been stronger or of longer duration, one or both of the dams almost certainly would have released the water in the reservoir. This experience emphasizes again the hazard of the old dams that have not been designed with earthquakes in mind. All old dams in California should be brought up to modern standards of safety.

The modern 200-ft high Santa Felicia earth dam, 20 miles northwest of Pacoima dam, experienced maximum crest accelerations of 20%g. The dam was undamaged except for a narrow meandering crack across the crest at the east abutment, apparently shallow. The Hansen flood control dam, an earth structure, was not damaged.

Damage to Electrical Facilities and Utilities

The new Converter Station of the Pacific Intertie was completed in 1970 at a cost of $110 million, and the damage to this facility has been estimated at $30 million. The damage to the Converter Station and to other switching stations in the area of strong shaking showed that electrical equipment needs better design to resist earthquakes. Although there was extensive and costly damage to electrical equipment, it appears that appropriate design to resist earthquakes could be required without excessive increase in cost.

Most of the northern San Fernando Valley was serviced by the General Telephone Company, and it received an estimated \$4.5 million of damage, and some ten to twenty thousand customers lost service for over a month. The pipelines of the gas distribution system received many breaks because of ground deformation, and seventeen thousand customers lost service for from four to twelve days. The water distribution system was ruptured in about 1,000 places, and many lines were plugged with sand and debris put into the system by damage at lower Van Norman dam. The gas, water and sewer systems were damaged mainly by ground deformations. A detailed study of what happened to these underground piping systems during the earthquake will provide data about the nature of the ground deformations.

Damage to Freeway Structures

There were many recently completed freeway structures and many that were still under construction in the region of strong shaking. Some of these structures collapsed, and many received damage. The standard code for bridge structures in the United States is that issued by the American Association of State Highway Officials. The 1965 edition has the following inadequate earthquake design provisions.

> "In regions where earthquakes may be anticipated, provision shall be made to accommodate lateral forces from earthquakes as follows:
>
> $$EQ = CD$$
>
> where
>
> EQ = lateral force applied horizontally in any direction at center of gravity of the weight of the structure.

D = deadload of structure

C = 0.02 for structures founded on spreadfootings on material rated as 4 tons or more/sq ft

= 0.04 for structures founded on spreadfootings on material rated as less than 4 tons/sq ft.

= 0.06 for structures founded on piles.

Live load may be neglected."

It is reported that the California State Highway Department did not follow the AASHO code, but rather based its earthquake design forces for bridge structures on the Uniform Building Code formula for the base shear coefficient. Although this procedure gives a stronger design, in most cases, than the AASHO code, it clearly was not adequate for very strong shaking.

Damage to Houses

The typical southern California house is a one-story, wood stud and plaster structure with a wood roof. The building code specifies certain requirements for earthquake resistance, such as anchor bolts fastening the wood frame to the foundation walls, bracing in the walls, reinforcing bars in the brick chimneys, and the chimneys anchored to the wood framing. Few of these houses received appreciable damage except where their foundations were disturbed by permanent ground displacements. On the other hand, a number of split-level houses, with garage under a portion of the house, did not fare so well. Some of these collapsed entirely, and some were wracked badly (see Chapter 3). Only four persons reportedly lost their lives in houses during the earthquake, and one of these was in a very old stone-wall house. It can be concluded that the typical modern, one-story house in southern California is resistant to earthquakes and is relatively safe. On the other

Accelerograph Location	Maximum Acceleration (g)
120. Los Angeles 3440 University 5th Floor	.129L .140T .083V
121. Los Angeles 3440 University Roof	.235L .260T .088V
122. Los Angeles 15107 Vanowen Bsmt	.107L .120T .116V
123. Los Angeles 15107 Vanowen 4th Floor	.227L .260T .194V
124. Los Angeles 15107 Vanowen Roof	.341L .385T .173V
125. Los Angeles 14724 Ventura 1st Floor	.356L .274T .107V
126. Los Angeles 14724 Ventura Penthouse	.315L .208T
127. Los Angeles 15433 Ventura 7th Floor	.242L .170T .153V
128. Los Angeles 15433 Ventura 13th Floor	.268L .226T .029V
129. Los Angeles 15250 Ventura Bsmt	.227L .140T .10V
130. Los Angeles 15250 Ventura 7th Floor	.261L .176T .131V
131. Los Angeles 15910 Ventura Bsmt	.130L .153T .112V
132. Los Angeles 15910 Ventura 9th Floor	.178L .129T .221V
133. Los Angeles 15910 Ventura 19th Floor	.224L .227T .210V
134. Los Angeles 945 Tiverton 8th Floor	.123L .226T .105V
135. Los Angeles 945 Tiverton 14th Floor	.143L .181T .146V
136. Los Angeles 6200 Wilshire Ground Floor	.134L .133T .038V
137. Los Angeles 6200 Wilshire 10th Floor	.280L .147T .068V
138. Los Angeles 6200 Wilshire 17th Floor	.300L .261T .074V
139. Los Angeles 5900 Wilshire Penthouse	.140L .170T .152V
140. Los Angeles 5900 Wilshire 16th Floor	.097L .121T .084V
141. Los Angeles 5900 Wilshire B Parking Lot	.066L .073T .029V
142. Los Angeles 6430 Sunset 1st Floor	.191L .143T .087V
143. Los Angeles 3345 Wilshire Bsmt	.121L .094T .069V
144. Los Angeles 3345 Wilshire 2nd Floor	.167L .113T
145. Los Angeles 3345 Wilshire 12th Floor	.206L .250T .124V

compliance with the State law that requires all school buildings to be rebuilt in conformity with the Field Act by 1975, or abandoned.

Architectural and Mechanical Damage

It is the responsibility of the engineer to design the foundations, the columns, the beams, the walls and the floor systems of a structure in such a way that it can withstand strong earthquake ground shaking without undue damage. On the other hand, it is the responsibility of the architect to design interior finish, plaster partitions and ceilings, to specify the installation of electrical and mechanical equipment and the major furnishings within the building so that they can withstand earthquake shaking without excessive damage. The design of some of these items is covered by the Los Angeles Building Code, for example, window frames must be designed so that when the building deforms the glass does not shatter, and hanging light fixtures must be designed so that they do not fall during strong shaking. However, most of the electrical and mechanical equipment in the interior finish of buildings is not governed by any requirements of the building code. There was much architectural and mechanical damage to buildings during the recent earthquake. Plaster partitions fractured, ceilings cracked and, in some cases, fell down, the balance weights of many elevators jumped out of their guides and became entangled, air conditioning equipment fell off of its mounts, etc. Much of this damage could have been avoided by giving attention to earthquake effects when the finish and equipment were designed. This improvement in resistance to earthquakes could have been achieved with little extra cost. There was little window breakage in high-rise buildings, which shows that architectural damage can indeed be controlled.

Earthquake Insurance

Earthquake insurance is carried by some industrial and commercial concerns and some have programs of self-insurance. It has been reported, for example, that in 1964 a total premium of $8 million for earthquake insurance was written in the United States. Very few houses, however, are covered by earthquake insurance. The annual rates for this coverage in Los Angeles range from about $1.50 to $3.00 per $1000 of insurance, with 2% to 10% deductible. The situation with respect to earthquake insurance seems to be not much different from that described 50 years ago by John R. Freeman[5]:

> "At present, adequate earthquake insurance is difficult to obtain in California and is unsatisfactory in coverage in many cases. Most insurance companies hesitate to write California earthquake insurance in other than relatively small amounts, and this chiefly upon preferred risks at high rates of premium, with limitations in the policies which are often unsatisfactory to the property owner.
>
> "Even at the present high rates it is said to be difficult in many cases for property owners to obtain sufficient earthquake insurance; and when one succeeds in obtaining a policy at a high rate he may find it so drafted as to protect the banker or mortgagee and the underwriter much more fully than it protects the owner of the equity in the building."

Freeman suggested that earthquake insurance should be a clause in the fire insurance policy. If done this way, the cost to a home owner could be much reduced. Some insurance companies indicate that an even broader base is needed, and that this could be provided by having a disaster clause

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[5] Earthquake Damage and Earthquake Insurance, McGraw-Hill, 1932.

in fire insurance policies throughout the United States to cover earthquakes, floods, tornadoes, etc. The insurance companies also point out that their reserves limit the amount of risk that can be assumed, and that the Government rules for the insurance business make it difficult to build up reserves for an increased writing of earthquake insurance and they are naturally wary of being exposed to excessive risk in the event of a great disaster.

The total damage done by the San Fernando earthquake to ordinary houses built since 1933 in the northern San Fernando Valley was remarkably small. It is estimated that the earthquake damage was less than 2% of the valuation of the houses that were subjected to ground shaking of 20%g or greater. This indicates that a broad-base coverage at modest rates for modern homes in southern California should be possible.

Disaster Relief Activities

The response of the metropolitan area to the effect of the earthquake was, in general, rapid and effective. The fire departments, police departments, and other City, County and State agencies responded quickly as did the U. S. Army Corps of Engineers. The only place where there was a breakdown in the response to the disaster was in the communication system, and this had a particularly unfortunate consequence at the Veterans Administration Hospital. The hospitals in the Los Angeles area had developed a special communication system (HEAR) just for disasters which even had auxilliary power supplies in case of failure of electrical power. At the Veterans Administration Hospital, however, the emergency power supply was damaged by the building that collapsed and as the telephone system was not operating after the earthquake, there was no direct means of communication for summoning aid. A man was dispatched by automobile and he reported to a traffic policeman who, in turn, reported by radio to police headquarters. However, in the process of doing this a misunderstanding arose so that police headquarters supposed that this report referred to the damage at the Olive View Hospital which had already been reported, and no aid was dispatched. It was an hour and twenty minutes after the earthquake that a reconnaissance helicopter happened to fly over the Veterans Administration Hospital and observed that a building had collapsed and, hence, aid from the police and fire departments was late in reaching the hospital. The boundary line between the City and the County of Los Angeles went through the region of the strongest shaking, but this caused no jurisdictional difficulties. It had been agreed, for example, that in an emergency, whichever fire department first saw a trouble spot would take charge, and the City fire department took charge of Veterans Administration Hospital even though it was in County territory.

The communication between the State Highway Department and the highway police was very good. Immediately after the earthquake the police informed the Highway Department of the overpass structures that had collapsed, so that they were aware, within 15 minutes of the earthquake, of the need for clearing away the fallen spans, and the work started very quickly.

The day of the earthquake, the Red Cross[6] set up emergency shelters at Granada Hills High School, San Fernando Junior High School, Chatsworth

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6 The American National Red Cross Report on the Southern California Earthquake, April, 1971.

High School, Lawrence Junior High School and Nobel Junior High School. During the next few days shelters were also set up at Francis Polytechnic High School, Pacoima Junior High School, McCrae Junior High School, and Porter Junior High School. Food was provided in the shelters and in addition two large feeding stations were set up at Santa Rosa Church and San Fernando Junior High School. Approximately 6000 meals were served each day for ten days at each of the two large feeding stations. A total of 175,000 meals were served at shelters, relief centers and feeding centers, plus 52,000 sandwiches. Eight mobile canteens dispensed 18,000 sandwiches, 7200 gallons of water, and 12,000 of milk and orange drink during February 12 to February 16. The Red Cross assisted over 11,000 families at a cost of over $1,100,000. During the emergency period, more than 3000 persons, having minor injuries, were cared for in the Red Cross shelters and relief centers. More than 1500 injured persons were admitted to San Fernando Valley hospitals. It was reported that so many premature babies were born the first few days after the earthquake that hospital facilities were strained.

The General Telephone Company system in the San Fernando Valley was put out of operation because of earthquake damage and even the larger Bell Telephone System in Los Angeles was in difficulty for a few days following the earthquake because of an overload of telephone calls. It has been reported that after the earthquake the daily number of long-distance telephone calls into and out of the metropolitan area tripled and overloaded the system, and the number of local calls was so great that the system was overloaded. Hence, for awhile it was only possible to complete a telephone call if, by chance, the call was accepted by the overloaded system.

The fire department received several hundred calls after the earthquake and of these about one-half dozen developed into significant blazes, one of which is shown in Fig. 1. 22. Sizable areas were without water after the earthquake, but large fires did not develop. This experience, however, raises the question as to what might be the result if a larger earthquake were to affect a major portion of the metropolitan area at a time when the air temperature was high, the humidity low, and strong winds were blowing.

Some of the affected areas in the northern San Fernando Valley were without water and sewers, and to alleviate conditions, temporary facilities were moved in. Temporary telephone booths were set up where the regular service had been disrupted.

In many of the homes, dishes, books, and other things on shelves fell to the floor; lamps and bookcases overturned, etc. Many concrete-block garden walls fell over. In industrial buildings, racks were overturned, parts cabinets spilled their contents on the floor, equipment toppled or moved about if not securely anchored. Many store buildings had their contents jumbled about (Fig. 1. 23). Markets suffered from the shock and had recourse to unusual sales procedures following the earthquake, as shown in Fig. 1. 24. Following the earthquake, the Federal Government announced that it would provide rehabilitation funds for damage to buildings and facilities owned by local, City and County governments. No provisions were made, however, to reimburse the individual home owner who had suffered appreciable loss. The Small Business Administration announced in Los Angeles that 7228 low-interest loans — with value of $34, 500, 000 — had been approved by April 30. Of this, 6875 loans worth $26, 200, 000 were to repair or rebuild homes

1.22 A fire in San Fernando Valley, after the earthquake. Fire damage in the Valley was small despite the fact that some regions were without water supply.

Figure 1.23 Many stores in the area had their stocks thrown to the floor. In some cases this resulted in considerable damage. Libraries as distant as Pasadena had the books strewn about and bookcases collapsed.

Figure 1.24 Parking lot sale at Von's Market on Foothill Blvd. after the earthquake.

damaged in the temblor. It was announced that the Small Business Administration loans required the repayment of the first $500 and everything above $3000, with the intervening $2500 forgiven. There was a rather widespread feeling in the community that the Federal, State or local governments should help those home owners that had had heavy earthquake damage, so that these losses would be spread over a larger base.

A rather unusual situation developed after the earthquake in that the persons severely affected by the earthquake were more or less surrounded by eight million who went about their normal day-to-day business. Heavy automobile traffic flowed through San Fernando Valley and many persons came to view the damage. After a few weeks, some of the inhabitants of the severely affected area forcefully expressed annoyance with sightseers.

Although the response to this disaster was, in general, effective, it does raise questions as to the possible effects of a great earthquake striking the metropolitan area under adverse conditions. In this event, a much larger relief effort would be needed, and careful preparation for such a disaster would be essential.

STRONG MOTION RECORDS FROM THE SAN FERNANDO EARTHQUAKE

A. Accelerogram Processing

by

D. E. Hudson

Accelerogram Collection and Reproduction. The simultaneous triggering of 272 strong-motion accelerographs in the Southern California region over distances of several hundred miles presented an unprecedented problem of record collection and data handling.

After the earthquake, an immediate decision was made in the Los Angeles office of the Seismological Field Survey (NOAA) that in the interests of minimizing the risks of losing records, only the regular experienced staff members connected with the office would pick up the photographic records from the field instruments. It was felt that the existing film supplies in the accelerographs would be adequate to record the important aftershocks, and that the time delay in collecting the records would entail no practical disadvantage, in view of the fact that preliminary reports had indicated that there had been no significant structural damage in any of the buildings housing the accelerographs.

By February 10, 1971, one day after the earthquake, the first group of accelerograms had been collected from the buildings nearest the epicentral region, and had been developed and scaled for approximate peak acceleration values. The Los Angeles Department of Building and Safety was immediately notified of the availability of these records for inspection by engineers and building owners. By this time it was also evident that there had in fact been no significant structural damage to any of the buildings housing the strong-motion accelerographs, so that no special system was necessary for the rapid communication of accelerograph readings to building owners. The large values of acceleration measured on the first records, however, made it clear that a major effort should be made to ensure a rapid and wide distribution of the results.

During a period of some two to three weeks after the earthquake, all of the photographic records were brought in to the Los Angeles office of the Seismological Field Survey where they were developed and a preliminary labeling and scaling of peak accelerations was carried out. This was done on a priority basis, with the records nearest the epicentral region being collected and processed first, because of the intense interest of all earthquake investigators. The accelerographs themselves were all checked for proper operation and left with a full film supply in readiness for the next earthquake.

The Pacoima Dam accelerograph, located virtually at the epicenter of the earthquake, could not be reached for several days after the earthquake because of large rock slides which blocked the access road, and which rendered visits to the sites on foot hazardous. The precipitous nature of the steep canyon location precluded the use of helicopters.

The records from the five accelerograph types making up the network were in the following forms: Standard U. S. Coast and Geodetic and AR-240, 12 in. wide photographic paper; RFT-250 and SMA-1, 70-mm. film; and the MO-2, 35-mm. film. The record lengths varied from a few feet to 30 feet, with several records as long as 80 feet. The total set of 227 usable records consisted of 91 paper records, 91 70-mm. records, and 45 35-mm. records.

Accelerogram Processing and Analysis Program. To render assistance to the Seismological Field Survey with immediate problems of record reproduction and dissemination, and to provide for eventual complete data analysis, the Engineering Division of the National Science Foundation and the Earthquake Engineering Research Laboratory at the California Institute of Technology developed a data handling and processing program immediately following the earthquake. The main objectives of this special NSF program in the data processing phase were:

(1) To produce accurate archival copies of all records, as well as copies suitable for digitization and data processing.

(2) To produce multiple copies of a form suitable for immediate display and distribution to all interested parties, and to arrange for such distribution.

(3) To carry out an accurate digitization of all useable records, in a form compatible with past accelerogram analysis.

(4) To prepare for all records in a standard form corrected accelerogram data, integrated velocity and displacement curves, and Fourier and response spectrum curves in various standard forms.

Record Reproduction. After preliminary labeling and checking in the Los Angeles office of the Seismological Field Survey, all originals were taken to the Jet Propulsion Laboratory in Pasadena where permanent photographic labels were made for each record, including all pertinent instrumental information. These labels were spliced on all records for future reproduction.

An accurate one-to-one film copy was then made at the Jet Propulsion Laboratory Photographic Department of the initial strong-motion portion of each record. A standard grid was photographed along with each record to check any distortions in the processing. One negative and one positive were made of each record on a stable film base. These short film copies than became the basis for reproduction of multiple copies for immediate distribution.

For distribution copies of the 12-inch paper accelerograms, the short film positive was used in a regular black-line print machine. In this way records could be reproduced economically in numbers of a dozen or so to follow demand. Part of this copying was done at the Jet Propulsion Laboratory, and part was done in the Aeronautics Department at the California Institute of Technology.

For distribution copies of the 70-mm. and 35-mm. short film records, the short negative copies were sent to the Rapid Blue Print Company in Los Angeles for full-size photographic prints. This resulted in accurate black-line-on-white copies for distribution, which were also suitable for direct Xerox copying for preliminary work. The Xerox copying was carried out in the Mechanical Engineering Department at the California Institute of Technology.

To make the records available for public inspection as soon as possible, a room was established on the campus of the California Institute of Technology in which copies of all accelerograms were posted as soon as available, along with additional information on instrument location, aftershock epicenters, etc. Wide publicity was given to the availability of this information through press and television coverage. In the weeks following the earthquake several thousand visitors availed themselves of this opportunity, including more than a hundred earthquake experts from Japan. Sets of multiple copies were also delivered to the Los Angeles Office of the Seismological Field Survey for distribution to building owners through the Los Angeles Department of Building and Safety, and to the Earthquake Engineering Research Institute - NOAA group at the University of California at Los Angeles.

To produce archival copies of the complete 12-inch paper records, the original records were taken in small lots to the Continental Graphics Company in Los Angeles, where one negative and two positives were produced of the entire length of each original record. Since many of the records were 20 feet long, and several were as long as 80 feet, this involved special processing facilities. As an additional control on the accuracy of the reproductions, technicians from the Jet Propulsion Laboratory measured and recorded a set of standard lengths on the original records and on the photographic copies.

To produce the archival full-length copies of the 35-mm. and 70-mm. records, a negative and two positives were produced by Yale Laboratories in Los Angeles. Special control of this process was required to produce suitable records for automatic image processing digitization.

In addition to the above standard records, certain extra editions were produced of several accelerograms having special interest. Three hundred copies were printed in accurate full-scale format of the Pacoima Dam accelerogram, and a similar set was produced for the basement and roof records at Millikan Library as an example of building response measurements. Considering all of the above processes, within a few weeks of the earthquake several thousand accelerograms had been distributed very widely around the world.

Along with the above record reproduction process, the Jet Propulsion Laboratory staff prepared a code number and tabulation listing for all accelerograms, and location maps for the instrument installations. In addition, the Jet Propulsion Laboratory arranged through a temporary leave of absence system for the transfer of a number of professional data processers and technicians to the campus of the California Institute of Technology for assistance with record processing and digitization. With this assistance, the accelerograph constants were checked from the calibration runs included on the accelerograms.

Digitization Program. In view of the large number of acclerograms it was immediately realized that record digitization would be a major task. Since a good deal of experience with accuracy evaluation and with the training of operators had been obtained on the Benson-Lehner 099D Datareducer*, it was decided that this "semi-automatic" method of digitization would be started immediately, but that in addition an intensive investigation of more automatic systems based on digital imaging processing techniques would be initiated.

During the past few years, several digital imaging processes have been developed for special applications. Such systems have been applied to the digitization of space photographs, and to the digitization of bubble-chamber tracks for particle physics investigations. It appeared

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*Hudson, D. E., Nigam, N. C., and Trifunac, M. D., "Analysis of Strong-Motion Accelerograph Records", Proc. 4th World Conference on Earthquake Engineering, Santiago, Chile, 1969.

Trifunac, M. D., "Low Frequency Digitization Errors and a New Method for Zero Baseline Correction of Strong-Motion Accelerograms", Report EERL 70-07, Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, 1970.

that several of these systems would be directly applicable to accelerograph digitization, but that a considerable computer programming job would be involved. In addition, it was clear that the problem of accuracy evaluation would take a different form and would require a detailed investigation. Since all of the available automatic systems required that the record be in the form of a 35-mm. or 70-mm. film strip, it was decided that the 12-inch paper records would be digitized semi-automatically with the hope that by the time they were finished, the automatic system would be in successful operation.

To assist with the semi-automatic digitization, time was made available on a second 099 Datareducer through the courtesy of the Shell Oil Company in Los Angeles. A staff of some twelve operators consisting of Jet Propulsion Laboratory technicians and part-time students from the California Institute of Technology was trained and checked for accuracy. With this staff and the two 099 Datareducers, all 90 paper accelerograms were digitized compatible with past accuracy standards within three months of the earthquake.

At the time of completion of the paper records, it was evident that the automatic digitization process would be feasible, and that the bulk of the film accelerograms could be digitized in this new way. It was also clear that some of the film records were not of a quality for automatic digitization, but would require the individual attention possible with the semi-automatic method. It was accordingly decided to carry out automatic processing for all suitable records, and to finish up the others with the 099 Datareducer.

In retrospect, it appears that the system of semi-automatic digitization used on the paper accelerograms was an effective way of dealing with the problem. The success of the approach depended on the availability of a group of experienced part-time operators from the Jet Propulsion Laboratory and from the Caltech student group. This is an essential feature since no one person can engage in the digitization for more than two or three hours without an inevitable degradation of accuracy. Given the relatively infrequent nature of strong earthquakes, such semi-automatic digitization is a feasible though laborious approach.

As a check on the fully automatic digitization process, a 35-mm. record of average quality was sent to Information International Incorporated in Los Angeles, where the digitization was carried out and supplied on magnetic tape. This tape was then processed at the Computing Center of the California Institute of Technology, and a large scale 8X plot of a selected portion of the record was plotted on the Calcomp plotter. This computer plotted accelerogram was then superimposed directly on an 8X enlargement of the original 35-mm. accelerogram. The excellent agreement obtained for this direct comparison indicated that a satisfactory accuracy had been achieved in the automatic digitization process.

The Accelerograph Network. The location of all of the accelerographs triggered by the earthquake is shown on the set of maps included as Figure 2.1. Table I is a location index for these maps. Table II is the basic list of accelerograms including the code number and pertinent information on the record.

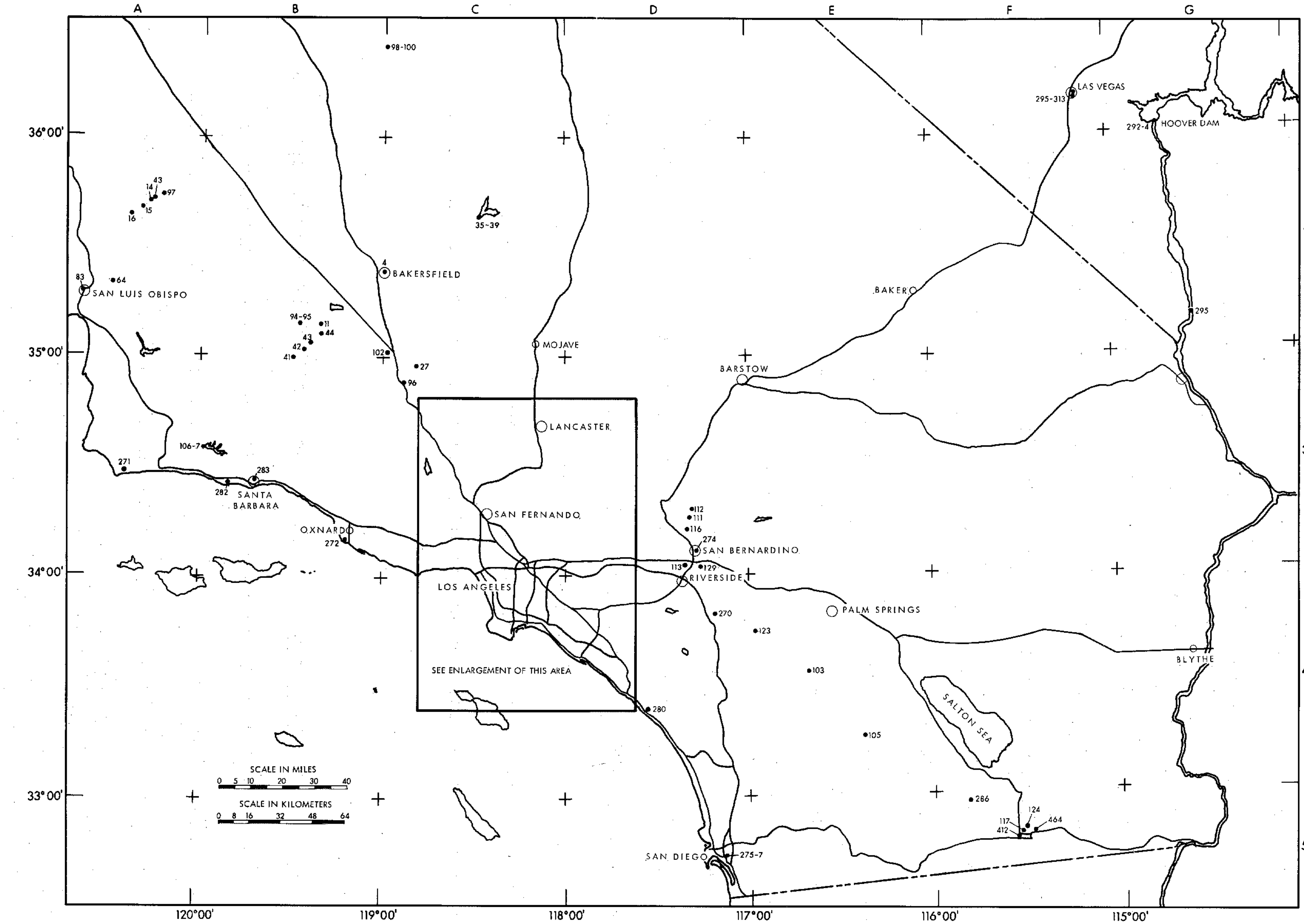

Figure 2. 1a Accelerograph location map 1

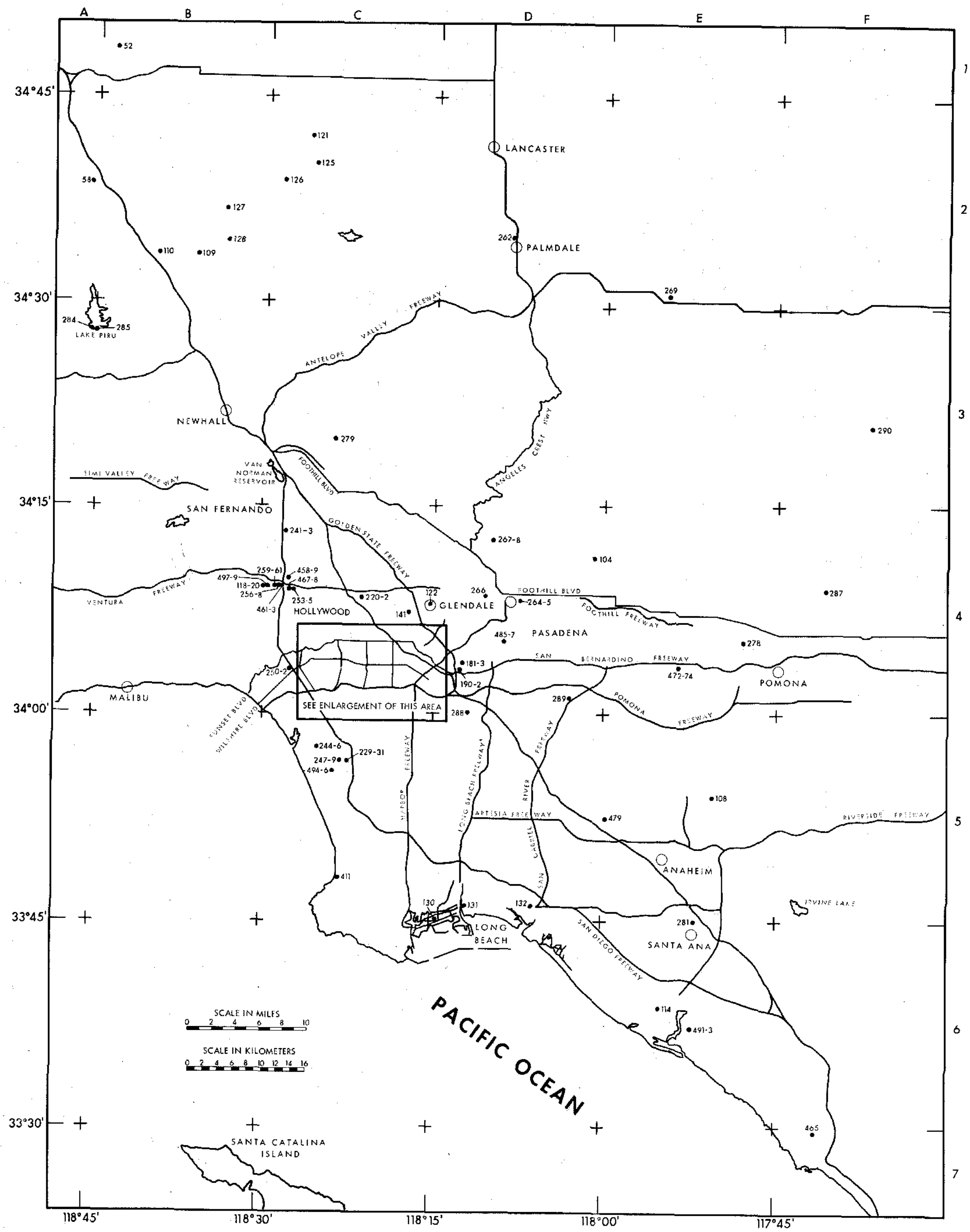

Figure 2.1b Accelerograph location map 2

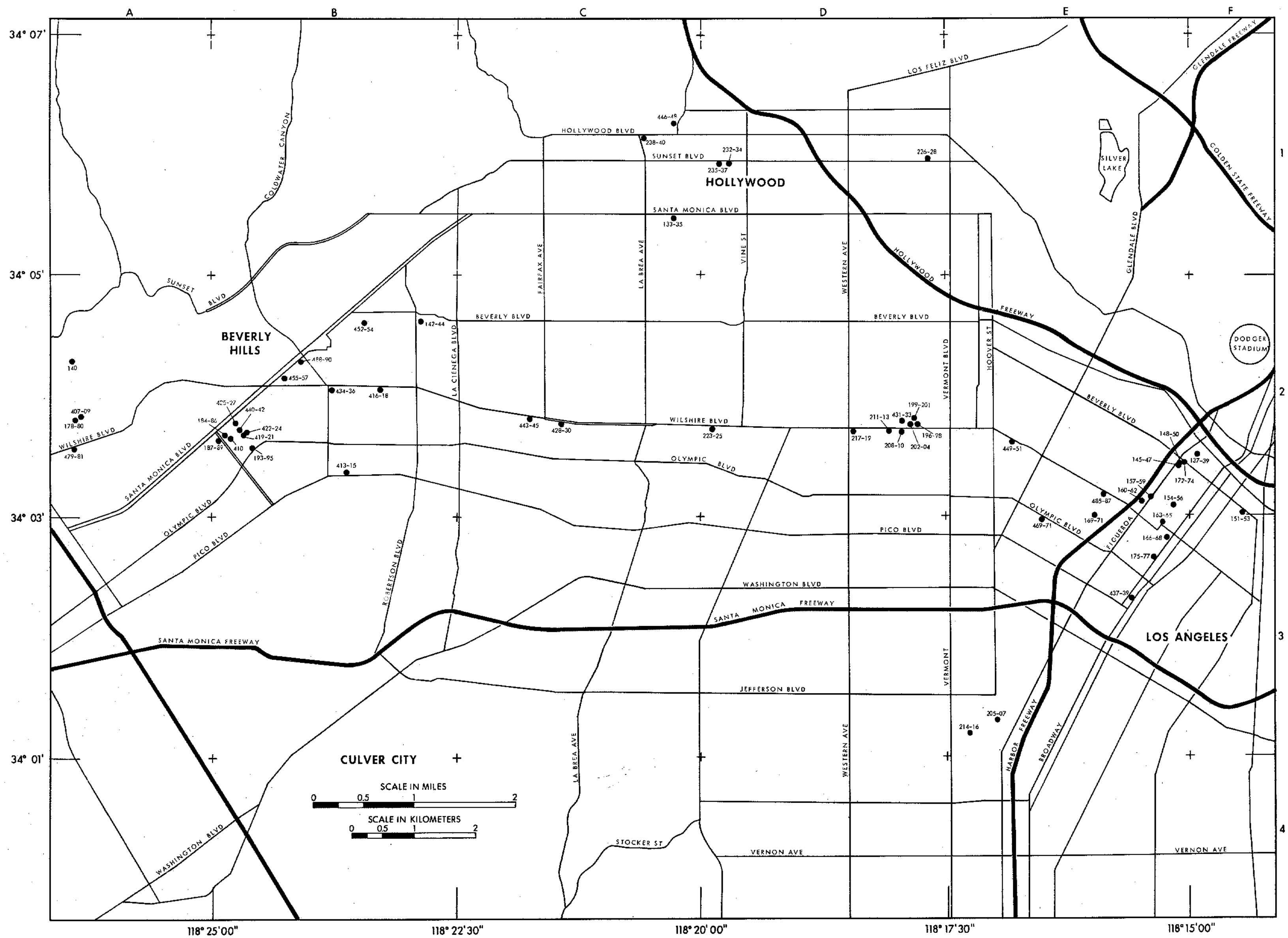

Figure 2. 1c Accelerograph location map 3

TABLE I
CENTRAL AND SOUTHERN CALIFORNIA STRONG-MOTION ACCELEROGRAPH NETWORK

Accelerograph Site	I. D. Number*	Map No.	Map Location
Alhambra			
Fremont, 900 South	482, 483, 484	2	D4
Anza	103	1	E4
Arcadia			
Santa Anita Reservoir	104	2	D4
Bakersfield	4	1	B2
Beverly Hills			
Camden Drive, 430	488, 489, 490	3	B2
Oakhurst, 435 North	452, 453, 454	2	C2
Roxbury, 450 North	455, 456, 457	3	B2
Wilshire, 9100	416, 417, 418	3	B2
Wilshire, 9450	434, 435, 436	3	B2
Borrego Springs	105	1	E4
Brea			
Carbon Canyon Dam	108	2	E5
Castaic			
Castaic	110	2	B2
Castaic Dam			
Cedar Springs			
Cedar Springs-Abutment	112	1	D3
Cedar Springs-Allen Ranch	111	1	D3
Cholame-Shandon Array	13, 14, 15, 16	1	A2
Colton	113	1	D3
Costa Mesa	114	2	E6
Devils Canyon	116	1	D3
El Centro			
El Centro	117	1	F5
El Centro Community Hospital	412	1	F5
Meadows Union School	464	1	F5
Fairmont Reservoir	121	2	C2
Fullerton			
Nutwood, 2600	476, 477, 478	2	E5
Glendale			
Broadway, 633 East	122	2	D4
Grapevine			
Tehachapi Pumping Plant	27	1	C3

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Accelerograph Site	I. D. Number*	Map No.	Map Location
Hemet	123	1	E4
Hoover Dam	292, 293, 294	1	G1
Imperial			
Imperial Valley College	124	1	F5
Isabella Dam	35, 36, 37, 38, 39	1	C2
Lake Hughes			
Array No. 1, 4	125, 126	2	C2
Array No. 9	128, 129	2	B2
Loma Linda	129	1	D3
Long Beach			
Long Beach Utility Bldg.	131	2	D5
Long Beach State College	132	2	D5
Terminal Island	130	2	D5
Los Angeles			
Airport Blvd., 9841	247, 248, 249	2	C5
Avenue of Stars, 1900	184, 185, 186	3	B2
Avenue of Stars, 1901	187, 188, 189	3	B2
Beverly Drive, 1177	413, 414, 415	3	B2
Century Blvd., 5260	229, 230, 231	2	C5
Century City Ground Station	410	3	B2
Century Park East, 1800	425, 426, 427	3	B2
" " " , 1880	440, 441, 442	3	B2
" " " , 1888	419, 420, 421	3	B2
" " " , 1888 (Ramp)	422, 423, 424	3	B2
" " " , 2080	193, 194, 195	3	B2
Figueroa, 222 South	145, 146, 147	3	E2
" , 234 "	148, 149, 150	3	E2
" , 455 "	157, 158, 159	3	E2
First, 250 East	151, 152, 153	3	F2
" , 800 West	172, 173, 174	3	E2
Fremont, 533 South	160, 161, 162	3	E2
Garland, 750 South	169, 170, 171	3	E2
Grand, 420 South	154, 155, 156	3	E2
Griffith Observatory	141	2	C4
Hilgard, 930	407, 408, 409	3	A2
Hill, 1150 South	437, 438, 439	3	E3
Hollywood Storage Bldg. (1025 N. Highland)			
Building	133, 134	3	C1
Parking Lot	135	3	C1
Hoover, 3663	214, 215, 216	3	E3
Lankershim, 3838	220, 221, 222	2	C4
L. A. Water & Power	137, 138, 139	3	F2

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Accelerograph Site	I. D. Number*	Map No.	Map Location
Los Angeles (continued)			
Lincoln Blvd., 8639	244, 245, 246	2	C5
Marengo, 1640	181, 182, 183	2	D4
Normandie, 616 South	431, 432, 433	3	D2
Olive, 646 South	166, 167, 168	3	E3
" , 808 South	175, 176, 177	3	E3
Olympic Blvd., 1625 South	469, 470, 471	3	E3
Orchid Avenue, 1760 North	446, 447, 448	3	C1
Orion, 8244	241, 242, 243	2	C4
Robertson, 120 North	142, 143, 144	3	B2
San Vincente, 11661	250, 251, 252	2	C4
Sixth Street, 611 West	163, 164, 165	3	E3
" " , 3407	199, 200, 201	3	D2
Sunset, 4867	226, 227, 228	3	D1
" , 6430	232, 233, 234	3	D1
" , 6464	235, 236, 237	3	D1
Tiverton, 945	178, 179, 180	3	A2
U. C. L. A. (Reactor Bldg.)	140	3	A2
University, 3440 (U. S. C.)	205, 206, 207	3	E3
Van Owen, 15107	458, 459, 460	2	C4
Ventura, 14724	253, 254, 255	2	C4
" , 15250	466, 467, 468	2	C4
" , 15433	256, 257, 258	2	C4
" , 15910	461, 462, 463	2	C4
" , 16055	259, 260, 261	2	C4
" , 16661	118, 119, 120	2	C4
Wilshire, 1200	485, 486, 487	3	E2
" , 2500	449, 450, 451	3	E2
" , 3345	196, 197, 198	3	D2
" , 3411	202, 203, 204	3	D2
" , 3470	208, 209, 210	3	D2
" , 3550	211, 212, 213	3	D2
" , 3710	217, 218, 219	3	D2
" , 4680	223, 224, 225	3	D2
" , 5900	428, 429, 430	3	C2
" , 6200	443, 444, 445	3	C2
" , 10880	479, 480, 481	3	A2
Zonal, 2011	190, 191, 192	2	D4
Las Vegas	296-313 (18)	1	F1
Maricopa Array	41, 42, 43, 44	1	B2
Orange			
Chapman, 4000 West	472, 473, 474	2	E5
Oso Pumping Plant (Gorman)	52	2	B1
Pacoima			
Pacoima Dam	279	2	C3

*Permanent Identification Number in Annual List of Stations Issued by Seismological Field Survey, NOA, NOAA

Accelerograph Site	I. D. Number*	Map No.	Map Location
Palmdale	262	2	D2
Palos Verdes Estates	411	2	C5
Pasadena			
Athenaeum, C.I.T.	475	2	D4
Millikan Library, C.I.T.	264, 265	2	D4
J.P.L.	267, 268	2	D4
Seis. Lab, C.I.T.	266	2	D4
Pearblossom	269	2	E2
Perris	270	1	D4
Point Conception	271	1	A3
Port Hueneme	272	1	B3
Pyramid	58	2	A2
San Bernardino	274	1	D3
San Diego			
San Diego Light & Power	277	1	D5
San Diego Gas & Electric	275, 276	1	D5
San Dimas			
Puddingstone Reservoir	278	2	E4
San Juan Capistrano	465	2	F7
Salinas Dam	64	1	A2
San Luis Obispo			
(City Recreation Bldg.)	83	1	A2
San Onofre	280	1	D4
Santa Ana	281	2	E5
Santa Barbara			
Santa Barbara (City Hall)	283	1	B3
Univ. of California	282	1	B3
Santa Felicia Dam (Piru)	284, 285	2	A3
Superstition Mountain	286	1	F5
Taft	94, 95	1	B2
Tejon			
Fort Tejon	96	1	C3
Temblor II (Cholame)	97	1	A2
Terminus	97, 98, 99	1	B1
Upland			
San Antonio Dam	287	2	F4

*Permanent Identification Number in Annual List of Stations Issued by Seismological Field Survey, NOA, NOAA

Accelerograph Site	I. D. Number*	Map No.	Map Location
Vernon	288	2	D4
Wheeler Ridge	102	1	C2
Whittier			
Whittier Narrows Dam	289	2	D4
Wrightwood	290	2	F3

* Permanent Identification Number in Annual List of Stations Issued by Seismological Field Survey, NOA, NOAA

TABLE II
NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.001	Pacoima Dam	Pacoima Dam	AR-240	179	S 74° W S = 7.55 cm/g	Down S = 7.62 cm/g	S 16° E S = 7.60 cm/g	12"	28'
71.002	Los Angeles 8244 Orion Blvd.	4th Floor	AR-240	210	North S* = 7.6 cm/g	Down S* = 7.6 cm/g	West S* = 7.6 cm/g	12"	10'
71.003	Los Angeles 1901 Avenue of the Stars	9th Floor	AR-240	283	N 460 W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 44° W S* = 7.6 cm/g	12"	8'
71.004	Wheeler Ridge	Wheeler Ridge	AR-240	112	South S* = 7.6 cm/g	Down S* = 7.6 cm/g	East S* = 7.6 cm/g	12"	5'
71.005	Los Angeles 250 E. First St.	Basement	AR-240	287	N 36° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 54° W S* = 7.6 cm/g	12"	10'
71.006	Los Angeles 1901 Avenue of the Stars	Sub-basement	AR-240	278	N 46° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 44° W S* = 7.6 cm/g	12"	7'
71.007	Castaic	Old Ridge Route	AR-240	124	N 21° E S = 8.1 cm/g	Down S = 7.9 cm/g	N 69° W S* = 7.6 cm/g	12"	14'
71.008	Los Angeles 8244 Orion Blvd.	1st Floor	AR-240	190	North S* = 7.6 cm/g	Down S* = 7.6 cm/g	West S* = 7.6 cm/g	12"	10'
71.009	Lake Hughes	Array Sta. 12	AR-240	217	N 21° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 69° W S* = 7.6 cm/g	12"	5'
71.010	Los Angeles 1640 S. Marengo St.	8th Floor	AR-240	230	N 38° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 52° W S* = 7.6 cm/g	12"	14'
71.011	Los Angeles 250 E. First St.	8th Floor	AR-240	294	N 36° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 54° W S* = 7.6 cm/g	12"	9'
71.012	Lake Hughes	Array Sta 9	AR-240	162	N 21° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 69° W S* = 7.6 cm/g	12"	5'
71.013	Los Angeles 1640 S. Marengo St.	1st Floor	AR-240	235	N 38° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 52° W S* = 7.6 cm/g	12"	15'
71.014	Los Angeles 8244 Orion Blvd.	8th Floor	AR-240	199	North S* = 7.6 cm/g	Down S* = 7.6 cm/g	West S* = 7.6 cm/g	12"	10'
71.015	Los Angeles 1640 S. Marengo St.	4th Floor	AR-240	229	N 38° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 52° W S* = 7.6 cm/g	12"	13'
71.016	Los Angeles 250 E. First St.	17th Floor	AR-240	295	N 36° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 54° W S* = 7.6 cm/g	12"	8'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.017	Los Angeles 1901 Avenue of the Stars	21st Floor	AR-240	275	N 46° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 44° W S* = 7.6 cm/g	12"	10'
71.018	Pasadena Caltech	Seismological Laboratory	RFT-250	193	South S* = 1.9 cm/g	Down S* = 1.9 cm/g	East S* = 1.9 cm/g	70 mm	16'
71.019	Pasadena Caltech	Athenaeum	SMA-1	124	East S = 1.82 cm/g	Down S = 1.95 cm/g	North S = 1.79 cm/g	70 mm	3'
71.020	Maricopa	Array Sta. 4	RFT-250	162-A	S 40° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 50° E S* = 1.9 cm/g	70 mm	3'
71.021	Maricopa	Array Sta. 3	RFT-250	194-A	S 40° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 50° E S* = 1.9 cm/g	70 mm	4'
71.022	Pasadena Caltech	Millikan Library Basement	RFT-250	198	East S* = 1.9 cm/g	Down S = 1.9 cm/g	North S* = 1.9 cm/g	70 mm	16'
71.023	Pasadena Caltech	Millikan Library 10th Floor	RFT-250	200	East S* = 1.9 cm/g	Down S* = 1.9 cm/g	North S* = 1.9 cm/g	70 mm	16'
71.024	Los Angeles 15250 Ventura Blvd.	Basement	SMA-1	185	N 11° E S = 1.95 cm/g	Down S = 1.70 cm/g	N 79° W S = 1.78 cm/g	70 mm	4'
71.025	Los Angeles 15250 Ventura Blvd.	Roof	SMA-1	183	N 11° E S = 1.92 cm/g	Down S = 1.91 cm/g	N 79° W S = 1.70 cm/g	70 mm	4'
71.026	Los Angeles 15250 Ventura Blvd.	7th Floor	SMA-1	184	N 11° E S = 1.80 cm/g	Down S = 1.71 cm/g	N 79° W S = 1.88 cm/g	70 mm	4'
71.027	Los Angeles 15107 Vanowen St.	Basement	RFT-250	267	West S = 1.87 cm/g	Down S* = 1.90 cm/g	South S = 1.91 cm/g	70 mm	82'
71.028	Los Angeles 15107 Vanowen St.	4th Floor	RFT-250	270	West S = 1.89 cm/g	Down S* = 1.90 cm/g	South S = 1.84 cm/g	70 mm	53'
71.029	Los Angeles 3838 Lankershim Blvd.	21st Floor	RFT-250	150	North S* = 1.9 cm/g	Down S* = 1.9 cm/g	West S* = 1.9 cm/g	70 mm	6'
71.030	Los Angeles 15107 Vanowen	Roof	RFT-250	254	West S = 1.96 cm/g	Down S* = 1.90 cm/g	South S = 1.97 cm/g	70 mm	86'
71.031	Pasadena Jet Propulsion Laboratory	9th Floor	RFT-250	199	S 8° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 82° E S* = 1.9 cm/g	70 mm	16'
71.032	Pasadena Jet Propulsion Laboratory	Basement	RFT-250	195	S 8° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 82° E S* = 1.9 cm/g	70 mm	16'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.033	Los Angeles 1150 S. Hill St.	10th Floor	RFT-250	276	S 53° E S = 1.88 cm/g	Down S* = 1.90 cm/g	N 37° E S = 1.81 cm/g	70 mm	9'
71.034	Los Angeles 1150 S. Hill St.	5th Floor	RFT-250	271	S 53° E S = 1.85 cm/g	Down S* = 1.90 cm/g	N 37° E S = 1.85 cm/g	70 mm	8'
71.035	Los Angeles 1150 S. Hill St.	Sub basement	RFT-250	277	S 53° E S = 1.89 cm/g	Down S* = 1.90 cm/g	N 37° E S = 1.89 cm/g	70 mm	8'
71.036	Los Angeles 3838 Lankershim Blvd.	Sub-basement	RFT-250	155	North S* = 1.9 cm/g	Down S* - 1.9 cm/g	West S* - 1.9 cm/g	70 mm	6'
71.037	Los Angeles 8639 Lincoln Ave.	6th Floor	RFT-250	170	S - W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S - E S* = 1.9 cm/g	70 mm	6'
71.038	Los Angeles 611 W. Sixth St.	Basement	RFT-250	139	N 52° W S* - 1.9 cm/g	Down S* - 1.9 cm/g	N 38° E S* = 1.9 cm/g	70 mm	5'
71.039	Los Angeles 8639 Lincoln Ave.	Basement	RFT-250	169	S - W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S - E S* - 1.9 cm/g	70 mm	5'
71.040	Los Angeles 611 W. Sixth St.	42nd Floor	RFT-250	122	N 52° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	N 38° E S* = 1.9 cm/g	70 mm	5'
71.041	Los Angeles 8639 Lincoln Ave.	12th Floor	RFT-250	168	S - W S* - 1.9 cm/g	Down S* - 1.9 cm/g	S - E S* = 1.9 cm/g	70 mm	6'
71.042	Los Angeles 3838 Lankershim Blvd.	11th Floor	RFT-250	154	North S* = 1.9 cm/g	Down S* = 1.9 cm/g	West S* - 1.9 cm/g	70 mm	5'
71.043	Los Angeles 611 W. Sixth St.	24th Floor	RFT-250	146	N 52° W S* - 1.9 cm/g	Down S* = 1.9 cm/g	N 38° E S* = 1.9 cm/g	70 mm	3'
71.044	Los Angeles 3710 Wilshire Blvd.	10th Floor	AR-240	220	West S* = 7.6 cm/g	Down S* = 7.6 cm/g	South S* = 7.6 cm/g	12"	7'
71.045	Los Angeles 3710 Wilshire Blvd.	5th Floor	AR-240	219	West S* = 7.6 cm/g	Down S* = 7.6 cm/g	South S* = 7.6 cm/g	12"	6'
71.046	Los Angeles 3710 Wilshire Blvd.	Basement	AR-240	221	West S* - 7.6 cm/g	Down S* = 7.6 cm/g	South S* - 7.6 cm/g	12"	7'
71.047	Los Angeles 4680 Wilshire Blvd.	6th Floor	AR-240	279	N 15° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 75° W S* = 7.6 cm/g	12"	11'
71.048	Los Angeles 4680 Wilshire Blvd.	Basement	AR-240	268	N 15° E S* = 7.6 cm/g	Down S* - 7.6 cm/g	N 75° W S* = 7.6 cm/g	12"	10-1/2'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.049	Los Angeles 4680 Wilshire Blvd.	3rd Floor	AR-240	284	N 15° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 75° W S* = 7.6 cm/g	12"	10'
71.050	Los Angeles 7080 Hollywood Blvd.	Basement	AR-240	269	East S* = 7.6 cm/g	Down S* = 7.6 cm/g	North S* = 7.6 cm/g	12"	5-1/2'
71.051	Los Angeles 7080 Hollywood Blvd.	6th Floor	AR-240	270	West S* = 7.6 cm/g	Down S* = 7.6 cm/g	South S* = 7.6 cm/g	12"	5-1/4'
71.052	Los Angeles 7080 Hollywood Blvd.	12th Floor	AR-240	267	East S* = 7.6 cm/g	Down S* = 7.6 cm/g	North S* = 7.6 cm/g	12"	6'
71.053	Los Angeles 4867 Sunset Blvd.	Basement	AR-240	260	S 89° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 01° E S* = 7.6 cm/g	12"	6-1/2'
71.054	Los Angeles 4867 Sunset Blvd.	2nd Floor	AR-240	286	S 01° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 89° E S* = 7.6 cm/g	12"	6'
71.055	Los Angeles 4867 Sunset Blvd.	7th Floor	AR-240	282	S 89° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 01° E S* = 7.6 cm/g	12"	5-1/2'
71.056	Los Angeles 3470 Wilshire Blvd.	Sub-basement	AR-240	248	North S* = 7.6 cm/g	Down S* = 7.6 cm/g	West S* = 7.6 cm/g	12"	9-1/2'
71.057	Los Angeles 3470 Wilshire Blvd.	5th Floor	AR-240	246	East S* = 7.6 cm/g	Down S* = 7.6 cm/g	North S* = 7.6 cm/g	12"	9'
71.058	Los Angeles 3470 Wilshire Blvd.	11th Floor	AR-240	297	East S* = 7.6 cm/g	Down S* = 7.6 cm/g	North S* = 7.6 cm/g	12"	7'
71.059	Los Angeles Water and Power Bldg.	Basement	AR-240	152	N 50° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 40° W S* = 7.6 cm/g	12"	14-1/2'
71.060	Los Angeles 445 Figueroa St.	Sub-basement	AR-240	208	N 52° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 38° W S* = 7.6 cm/g	12"	22-1/2'
71.061	Grapevine Tehachapi Pumping Plant	C.N.R. Site	AR-240	167	South S* = 7.6 cm/g	Down S* = 7.6 cm/g	East S* = 7.6 cm/g	12"	2-1/2'
71.062	Santa Felicia Dam	Outlet Works	AR-240	242	S 82° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 08° E S* = 7.6 cm/g	12"	6-1/2'
71.063	Santa Felicia Dam	Crest	AR-240	241	S 75° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 15° E S* = 7.6 cm/g	12"	7-3/4'
71.064	Palmdale Fire Sta.	Storage Room	RFT-250	189	S 60° E S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 30° W S* = 1.9 cm/g	70 mm	3'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.065	Lake Hughes Array	Station 4	RFT-250	164	S 21° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 69° E S* = 1.9 cm/g	70 mm	2-3/4'
71.066	Carbon Canyon Dam	Carbon Canyon Dam	RFT-250	131	S 40° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 50° E S* = 1.9 cm/g	70 mm	2-3/4'
71.067	Whittier Narrows Dam	Whittier Narrows Dam	RFT-250	130	S 53° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 37° E S* = 1.9 cm/g	70 mm	3-3/4'
71.068	San Antonio Dam	San Antonio Dam	RFT-250	132	N 75° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	N 15° E S* = 2.0 cm/g	70 mm	2-1/2'
71.069	Los Angeles Griffith Park Observatory	Moon Room	RFT-250	158	South S* = 1.9 cm/g	Down S* = 1.9 cm/g	West S* = 1.9 cm/g	70 mm	5-1/4'
71.070	Los Angeles 616 S. Normandie Ave.	Basement	SMA-1	117	North S = 1.61 cm/g	Down S = 1.72 cm/g	West S = 1.66 cm/g	70 mm	4-3/4'
71.071	Alhambra 900 S. Fremont Ave.	Basement	SMA-1	179	West S = 1.60 cm/g	Down S = 1.75 cm/g	South S = 1.78 cm/g	70 mm	4'
71.072	Los Angeles 1625 Olympic Blvd.	Ground Floor	SMA-1	146	N 28° E S = 1.92 cm/g	Down S = 1.70 cm/g	N 62° W S = 1.80 cm/g	70 mm	6-1/4'
71.073	Los Angeles 616 S. Normandie Ave.	Roof	SMA-1	119	North S = 1.60 cm/g	Down S = 1.82 cm/g	West S = 1.76 cm/g	70 mm	8'
71.074	Los Angeles 616 S. Normandie Ave.	8th Floor	SMA-1	118	North S = 1.80 cm/g	Down S = 1.62 cm/g	West S = 1.62 cm/g	70 mm	4-1/2'
71.075	Los Angeles 1625 Olympic Blvd.	6th Floor	SMA-1	147	N 28° E S = 1.93 cm/g	Down S = 1.72 cm/g	N 62° W S = 1.82 cm/g	70 mm	5-1/4'
71.076	Los Angeles 420 S. Grand Ave.	2nd Floor	RFT-250	125	S 37° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 53° E S* = 1.9 cm/g	70 mm	8-1/4'
71.077	Los Angeles 420 S. Grand Ave.	10th Floor	RFT-250	124	S 37° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 53° E S* = 1.9 cm/g	70 mm	4-1/2'
71.078	Los Angeles 420 S. Grand Ave.	17th Floor	RFT-250	126	S 37° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 53° E S* = 1.9 cm/g	70 mm	7-1/2'
71.079	Los Angeles 750 Garland Ave.	Ground Floor	RFT-250	153	S 30° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	N 60° W S* = 1.9 cm/g	70 mm	7'
71.080	Los Angeles 750 Garland Ave.	2nd Floor	RFT-250	152	S 30° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	N 60° W S* = 1.9 cm/g	70 mm	9-1/2'

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NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.081	Los Angeles 750 Garland Ave.	6th Floor	RFT-250	151	S 30° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	N 60° W S* = 1.9 cm/g	70 mm	9-1/4'
71.082	Alhambra 900 S. Fremont Ave.	12th Floor	SMA-1	165	West S = 1.70 cm/g	Down S = 1.75 cm/g	South S = 1.80 cm/g	70 mm	4'
71.083	Alhambra 900 S. Fremont Ave.	6th Floor	SMA-1	187	West S = 1.80 cm/g	Down S = 1.75 cm/g	South S = 1.70 cm/g	70 mm	5'
71.084	Los Angeles 1880 Century Park East	Parking 1st Level	SMA-1	121	N 54° E S = 1.92 cm/g	Down S = 1.64 cm/g	N 36° W S = 1.82 cm/g	70 mm	12-1/2'
71.085	Los Angeles 1880 Century Park East	7th Floor	SMA-1	115	N 54° E S = 1.85 cm/g	Down S = 1.74 cm/g	N 36° W S = 1.87 cm/g	70 mm	13-1/2'
71.086	Los Angeles 1880 Century Park East	Penthouse	SMA-1	111	N 54° E S = 1.85 cm/g	Down S = 1.68 cm/g	N 36° W S = 1.53 cm/g	70 mm	12'
71.087	Los Angeles 435 Oakhurst Ave.	Basement	SMA-1	109	North S = 1.66 cm/g	Down S = 1.58 cm/g	West S = 1.69 cm/g	70 mm	4-1/2'
71.088	Los Angeles 435 Oakhurst Ave.	5th Floor	SMA-1	105	North S = 1.70 cm/g	Down S = 1.70 cm/g	West S = 1.60 cm/g	70 mm	3-1/2'
71.089	Los Angeles 435 Oakhurst Ave.	Roof	SMA-1	107	North S = 1.71 cm/g	Down S = 1.56 cm/g	West S = 1.76 cm/g	70 mm	4'
71.090	Los Angeles 1625 Olympic Blvd.	10th Floor	SMA-1	145	N 28° E S = 1.82 cm/g	Down S = 1.81 cm/g	N 62° W S = 1.91 cm/g	70 mm	7-1/2'
71.091	Los Angeles 445 Figueroa St.	19th Floor	AR-240	231	N 52° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 38° W S* = 7.6 cm/g	12"	29'
71.092	Los Angeles Water and Power Bldg.	7th Floor	AR-240	150	N 50° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 40° W S* = 7.6 cm/g	12"	22'
71.093	Los Angeles Water and Power Bldg.	15th Floor	AR-240	151	N 50° W S* = 7.6 cm/g	Down S = 7.6 cm/g	S 40° W S* = 7.6 cm/g	12"	20-1/2'
71.094	Los Angeles 945 Tiverton Ave.	8th Floor	AR-240	244	N 78° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 12° W S* = 7.6 cm/g	12"	70'
71.095	Los Angeles 945 Tiverton Ave.	14th Floor	AR-240	243	N 78° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 12° W S* = 7.6 cm/g	12"	68'
71.096	Los Angeles 3407 Sixth St.	Basement	AR-240	225	South S* = 7.6 cm/g	Down S* = 7.6 cm/g	East S* = 7.6 cm/g	12"	10'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.097	Los Angeles 3407 Sixth St.	4th Floor	AR-240	223	South S* = 7.6 cm/g	Down S* = 7.6 cm/g	East S* = 7.6 cm/g	12"	11'
71.098	Los Angeles 3407 Sixth St.	Penthouse	AR-240	233	South S* = 7.6 cm/g	Down S* = 7.6 cm/g	East S* = 7.6 cm/g	12"	9'
71.099	San Onofre Southern Calif. Edison	Nuclear Power Plant	AR-240	153	N 33° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 57° W S* = 7.6 cm/g	12"	8'
71.100	Vernon	CDM Bldg.	Standard	41-A	Up S = 12.9 cm/g	S 07° W S = 13.2 cm/g	N 83° W S = 13.2 cm/g	12"	13'
71.101	Santa Ana	Orange County Engineering Bldg.	Standard	11-M	Up S = 11.5 cm/g	S 86° W Mag. = .94 S 04° E** S = 11.9 cm/g	N 04° W Mag. = .96 S 86° W*** S = 11.7 cm/g	12"	12'
71.102	Glendale 633 E. Broadway	Municipal Service Bldg.	AR-240	216	S 70° E S* = 15.2 cm/g	Down S* = 15.2 cm/g	S 20° W S* = 15.2 cm/g	12"	7'
71.103	Los Angeles 808 S. Olive St.	4th Level	AR-240	226	S 53° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 37° E S* = 7.6 cm/g	12"	17'
71.104	Los Angeles 808 S. Olive St.	8th Level	AR-240	206	S 37° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 53° E S* = 7.6 cm/g	12"	17'
71.105	Los Angeles 2011 Zonal Ave.	Basement	AR-240	296	S 28° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 62° E S* = 7.6 cm/g	12"	6'
71.106	Los Angeles 2011 Zonal Ave.	5th Floor	AR-240	292	S 28° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 62° E S* = 7.6 cm/g	12"	7'
71.107	Los Angeles 2011 Zonal Ave.	9th Floor	AR-240	302	S 28° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 62° E S* = 7.6 cm/g	12"	6'
71.108	Los Angeles 3345 Wilshire Blvd.	Basement	AR-240	300	South S* = 7.6 cm/g	Down S* = 7.6 cm/g	East S* = 7.6 cm/g	12"	7'
71.109	Los Angeles 3345 Wilshire Blvd.	2nd Floor	AR-240	298	South S = 7.6 cm/g	Down S = 7.6 cm/g	East S = 7.6 cm/g	12"	7'
71.110	Los Angeles 3345 Wilshire Blvd.	12th Floor	AR-240	299	South S* = 7.6 cm/g	Down S* = 7.6 cm/g	East S* = 7.6 cm/g	12"	7'
71.111	Los Angeles 120 N. Robertson Blvd.	Sub-basement	AR-240	239	S 02° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 88° E S* = 7.6 cm/g	12"	12'

*Nominal Sensitivity
**Identifying Number, 4.
***Identifying Number, 5.

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.112	Los Angeles 120 N. Robertson Blvd.	4th Floor	AR-240	237	S 02° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 88° E S* = 7.6 cm/g	12"	13'
71.113	Los Angeles 120 N. Robertson Blvd.	9th Floor	AR-240	238	S 02° W S* = 7.6 cm/g	Down S* 7.6 cm/g	S 88° E S* - 7.6 cm/g	12"	12'
71.114	Los Angeles 646 S. Olive Ave.	Basement	AR 240	262	S 37° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 53° E S* = 7.6 cm/g	12"	13'
71.115	Los Angeles 646 S. Olive Ave.	4th Level	AR-240	266	S 37° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 53° E S* = 7.6 cm/g	12"	13'
71.116	Los Angeles 646 S. Olive Ave.	Roof	AR-240	265	S 37° W S* = 7.6 cm/g	Down S* . 7.6 cm/g	S 53° E S* - 7.6 cm/g	12"	13'
71.117	Los Angeles 1888 Century Park East	14th Floor	RFT-250	269	N 54° E S = 1.87 cm/g	Down S = 1.90 cm/g	N 36° W S = 1.86 cm/g	70 mm	7'
71.118	Los Angeles 1888 Century Park East	21st Floor	RFT-250	268	N 54° E S = 1.87 cm/g	Down S = 1.90 cm/g	N 36° W S = 1.89 cm/g	70 mm	9'
71.119	Palos Verdes Estates	2516 Via Tejon	RFT-250	138	S 25° E S* - 3.8 cm/g	Down S* = 3.8 cm/g	N 65° E S* = 3.8 cm/g	70 mm	5'
71.120	Beverly Hills 420 N. Roxbury Dr.	5th Floor	SMA-1	153	N 50° E S = 1.81 cm/g	Down S = 1.86 cm/g	N 40° W S = 1.69 cm/g	70 mm	9'
71.121	Beverly Hills 420 N. Roxbury Dr.	1st Floor	SMA-1	151	N 50° E S = 1.83 cm/g	Down S = 1.80 cm/g	N 40° W S = 1.80 cm/g	70 mm	8'
71.122	Beverly Hills 420 N. Roxbury Dr.	10th Floor	SMA-1	152	N 50° E S = 1.66 cm/g	Down S = 1.84 cm/g	N 40° W S - 1.81 cm/g	70 mm	8'
71.123	Orange 4000 W. Chapman Ave.	Basement	RFT-250	272	West S = 1.83 cm/g	Down S = 1.77 cm/g	South S = 1.83 cm/g	70 mm	8'
71.124	Orange 4000 W. Chapman Ave.	10th Floor	RFT-250	274	West S = 1.76 cm/g	Down S = 1.84 cm/g	South S = 1.89 cm/g	70 mm	9'
71.125	Orange 4000 W. Chapman Ave.	19th Floor	RFT-250	266	West S = 1.84 cm/g	Down S = 1.90 cm/g	South S - 1.80 cm/g	70 mm	8'
71.126	Los Angeles 1800 Century Park East	Basement (P 3)	SMA-1	141	S 36° E S = 1.98 cm/g	Down S = 1.80 cm/g	N 54° E S = 1.80 cm/g	70 mm	7'
71.127	Los Angeles 1800 Century Park East	5th Floor	SMA-1	140	S 36° E S - 1.85 cm/g	Down S . 1.78 cm/g	N 54° E S = 1.78 cm/g	70 mm	8'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.128	Los Angeles 1800 Century Park East	Penthouse	SMA-1	135	S 36° E S = 1.73 cm/g	Down S = 1.82 cm/g	N 54° E S = 1.69 cm/g	70 mm	7'
71.129	Los Angeles 2500 Wilshire Blvd.	Basement	SMA-1	130	N 29° E S = 1.96 cm/g	Down S = 1.83 cm/g	N 61° W S = 2.00 cm/g	70 mm	6'
71.130	Los Angeles 2500 Wilshire Blvd.	8th Floor	SMA-1	129	N 29° E S = 2.02 cm/g	Down S = 2.00 cm/g	N 61° W S = 1.90 cm/g	70 mm	8'
71.131	Los Angeles 2500 Wilshire Blvd.	Roof	SMA-1	131	N 29° E S = 1.92 cm/g	Down S = 1.90 cm/g	N 61° W S = 2.04 cm/g	70 mm	4'
71.132	Fullerton 2600 Nutwood Ave.	Penthouse West Wing	SMA-1	110	West S = 1.76 cm/g	Down S = 1.68 cm/g	South S = 1.68 cm/g	70 mm	4'
71.133	Fullerton 2600 Nutwood Ave.	Basement	SMA-1	113	West S = 1.69 cm/g	Down S = 1.72 cm/g	South S = 1.69 cm/g	70 mm	4'
71.134	Fullerton 2600 Nutwood Ave.	Penthouse (center)	SMA-1	116	West S = 1.74 cm/g	Down S = 1.36 cm/g	South S = 1.90 cm/g	70 mm	4'
71.135	Los Angeles 15910 Ventura Blvd.	Basement	SMA-1	182	S 09° W S = 2.00 cm/g	Down S* = 1.95 cm/g	S 81° E S = 2.09 cm/g	70 mm	17'
71.136	Los Angeles 15910 Ventura Blvd.	9th Floor	SMA-1	181	S 09° W S = 1.74 cm/g	Down S* = 1.9 cm/g	S 81° E S = 1.93 cm/g	70 mm	20'
71.137	Los Angeles 15910 Ventura Blvd.	19th Floor	SMA-1	180	S 09° W S = 1.87 cm/g	Down S* = 1.95 cm/g	S 81° E S = 1.67 cm/g	70 mm	19'
71.138	Los Angeles 1888 Century Park East	Parking Ramp 9th Floor	RFT 250	275	S 36° E S = 1.84 cm/g	Down S = 1.90 cm/g	N 54° E S = 1.89 cm/g	70 mm	6'
71.139	Los Angeles 1888 Century Park East	Parking Ramp 5th Floor	RFT-250	265	S 36° E S = 1.87 cm/g	Down S = 1.90 cm/g	N 54° E S = 1.86 cm/g	70 mm	8'
71.140	San Juan Capistrano	San Juan Capistrano	RFT-250	180	N 33° E S = 1.99 cm/g	Down S = 1.9 cm/g	N 57° W S = 1.91 cm/g	70 mm	7'
71.141	Long Beach Long Beach State College	Ground Level	RFT-250	184	N 76° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 14° W S* = 1.9 cm/g	70 mm	11'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.142	Colton	Edison Company	Standard	38-A	Up S = 13.1 cm/g	East S = 14.0 cm/g	South S = 13.4 cm/g	12"	4'
71.143	Tejon	Ft. Tejon	AR-240	115	East S = 7.6 cm/g	Down S = 7.6 cm/g	North S = 7.6 cm/g	12"	2'
71.144	Pearblossom	Pumping Plant	AR-240	215	North S* = 7.6 cm/g	Down S* = 7.6 cm/g	West S* = 7.6 cm/g	12"	4'
71.145	Cedar Springs	Pumping Plant	AR-240	187	S 36° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 54° E S = 7.6 cm/g	12"	4'
71.146	Gorman	Oso Pumping Plant	AR-240	194	North S* = 7.6 cm/g	Down S* = 7.6 cm/g	West S* = 7.6 cm/g	12"	6'
71.147	Los Angeles	UCLA Reactor Laboratory	Standard	62-S	Up S = 21.0 cm/g West** Magnification 1.0	South S = 21.0 cm/g South*** Magnification 1.0	East S = 22.0 cm/g	12"	21'
71.148	Costa Mesa 666 W. 19th St.	Ground Floor	AR-240	184	South S* = 7.6 cm/g	Down S* = 7.6 cm/g	East S* = 7.6 cm/g	12"	15'
71.149	Long Beach 215 W. Broadway	Utilities Bldg.	Standard	4-M	Up S = 13.0 cm/g East** Magnification .9	North S = 13.1 cm/g South*** Magnification .9	East S = 13.2 cm/g	12"	7'
71.150	Arcadia Santa Anita	Reservoir	AR-240	240	N 03° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 87° W S* = 7.6 cm/g	12"	5'
71.151	Port Hueneme	Navy Laboratory	Standard	1-A	Up S = 19.6 cm/g West** Magnification .95	South S = 19.5 cm/g North*** Magnification .95	West S = 19.8 cm/g	12"	12'
71.152	Lake Hughes	Array Sta. 1	AR-240	132	N 21° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 69° E S* = 7.6 cm/g	12"	6'

*Nominal Sensitivity
**Identifying Number, 4
***Identifying Number, 5

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY
FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers 1	Other identifying numbers 2	Other identifying numbers 3	Record Type	Record Length
71.153	San Dimas Puddingstone	Reservoir	AR-240	178	N 55° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 35° W S* = 7.6 cm/g	12"	7'
71.154	Long Beach	Terminal Island	Standard	13-D	Up S = 14.2 cm/g	S 69° W S = 14.1 cm/g	N 21° W S = 13.3 cm/g	6"	12'
71.155	Los Angeles Hollywood Storage	P. E. Lot	Standard	1-D	Up S = 12.9 cm/g	East S = 12.1 cm/g	South S = 13.2 cm/g	6"	43'
71.156	Los Angeles Hollywood Storage	Basement	Standard	22-D	Up S = 12.8 cm/g	East S = 13.3 cm/g	South S = 12.4 cm/g	6"	11'
71.157	Wrightwood 6047 Park Dr.	Wrightwood	SMA-1	232	S 25° W S = 1.66 cm/g	Down S = 1.80 cm/g	S 65° E S = 1.79 cm/g	70 mm	3'
71.158	San Bernardino	Hall of Records	RFT-250	140	East S* = 1.9 cm/g	Down S* = 1.9 cm/g	North S* = 1.9 cm/g	70 mm	4'
71.159	Wrightwood 6074 Park Dr.	Wrightwood	RFT-250	186	S 25° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 65° E S* = 1.9 cm/g	70 mm	3'
71.160	Los Angeles 9841 Airport Blvd.	Basement	MO-2	88	North S* = 1.65 cm/g	West S* = 1.72 cm/g	Up S* = 2.35 cm/g	35 mm	7'
71.161	Los Angeles 9841 Airport Blvd.	15th Floor	MO-2	98	North S* = 1.60 cm/g	West S* = 1.60 cm/g	Up S* = 2.40 cm/g	35 mm	8'
71.162	Los Angeles 14724 Ventura Blvd.	1st Floor	MO-2	165	S 12° W S* = 1.51 cm/g	N 78° W S* = 1.53 cm/g	Up S* = 2.36 cm/g	35 mm	15'
71.163	Los Angeles 14724 Ventura Blvd.	6th Floor	MO-2	129	S 12° W S* = 1.57 cm/g	N 78° W S* = 1.57 cm/g	Up S* = 2.42 cm/g	35 mm	16'
71.164	Los Angeles 14724 Ventura Blvd.	Penthouse	MO-2	188	S 12° W S* = 1.52 cm/g	N 78° W S* = 1.53 cm/g	Up S* = 2.34 cm/g	35 mm	16'
71.165	Hollywood 1760 N. Orchid Ave.	Ground Floor	MO-2	152	South S = 1.47 cm/g	East S = 1.52 cm/g	Up S = 2.34 cm/g	35 mm	13'
71.166	Hollywood 1760 N. Orchid Ave.	12th Floor	MO-2	135	East S = 1.55 cm/g	South S = 1.54 cm/g	Up S = 2.26 cm/g	35 mm	12'
71.167	Hollywood 1760 N. Orchid Ave.	23rd Floor	MO-2	132	East S = 1.57 cm/g	South S = 1.54 cm/g	Up S* = 2.42 cm/g	35 mm	13'
71.168	Beverly Hills 9100 Wilshire Blvd.	Basement		148	East S = 1.49 cm/g	South S = 1.54 cm/g	Up S* = 2.44 cm/g	35 mm	8'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.169	Beverly Hills 9100 Wilshire Blvd.	5th Floor	MO-2	133	East S = 1.51 cm/g	South S = 1.42 cm/g	Up S* = 2.30 cm/g	35 mm	6'
71.170	Los Angeles 800 W. First St.	1st Floor	MO-2	59	N 37° E S = 1.59 cm/g	W 53° W S = 1.60 cm/g	Up S = 2.36 cm/g	35 mm	12'
71.171	Los Angeles 800 W. First St.	16th Floor	MO-2	60	N 37° E S* = 1.57 cm/g	N 53° W S* = 1.59 cm/g	Up S* = 2.39 cm/g	35 mm	13'
71.172	Los Angeles 800 W. First St.	33rd Floor	MO-2	81	N 37° E S* = 1.61 cm/g	N 53° W S* = 1.56 cm/g	Up S* = 2.36 cm/g	35 mm	12'
71.173	Los Angeles 222 Figueroa St.	20th Floor	MO-2	160	S 37° W S* = 1.55 cm/g	N 53° W S* = 1.54 cm/g	Up S* = 2.44 cm/g	35 mm	5'
71.174	Los Angeles 222 Figueroa St.	1st Floor	MO-2	110	S 37° W S* = 1.51 cm/g	N 53° W S* = 1.53 cm/g	Up S* = 2.32 cm/g	35 mm	6'
71.175	Fairmont Reservoir	Reservoir	Standard	79-M	Up S = 16.6 cm/g S 56° W** Static Mag. = 1	N 34° W S = 17.4 cm/g N 34° W*** Static Mag. = 1	N 56° E S = 16.7 cm/g	12"	7'
71.176	Santa Barbara	University of California	RFT-250	183	N 42° E S* = 1.9 cm/g	Up S* = 1.9 cm/g	S 48° E S* = 1.9 cm/g	70 mm	5'
71.177	Los Angeles 6200 Wilshire Blvd.	17th Floor	MO-2	127	N 08° E S = 1.50 cm/g	N 82° W S = 1.57 cm/g	Up S = 2.32 cm/g	35 mm	6'
71.178	Los Angeles 6200 Wilshire Blvd.	Ground Floor	MO-2	175	N 08° E S = 1.49 cm/g	N 82° W S = 1.57 cm/g	Up S = 2.33 cm/g	35 mm	7'
71.179	Los Angeles 6200 Wilshire Blvd.	10th Floor	MO-2	185	N 08° E S = 1.50 cm/g	N 82° W S = 1.49 cm/g	Up S = 2.34 cm/g	35 mm	7'
71.180	Los Angeles 3440 University Ave.	5th Floor	MO-2	89	S 61° E S* = 1.55 cm/g	N 29° E S* = 1.57 cm/g	Up S* = 2.42 cm/g	35 mm	3'
71.181	Los Angeles 3440 University Ave.	Basement	MO-2	97	S 61° E S* = 1.63 cm/g	N 29° E S* = 1.57 cm/g	Up S* = 2.42 cm/g	35 mm	4'
71.182	Los Angeles 3440 University Ave.	Roof	MO-2	105	S 61° E S* = 1.62 cm/g	N 29° E S* = 1.53 cm/g	Up S* = 2.62 cm/g	35 mm	4'
71.183	Los Angeles 1177 Beverly Dr.	Basement	MO-2	170	N 31° W S = 1.54 cm/g	N 59° E S = 1.57 cm/g	Up S = 2.38 cm/g	35 mm	9'

*Nominal Sensitivity
**Identifying Number, 4
***Identifying Number, 5

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY
FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.184	Los Angeles 5900 Wilshire Blvd.	16th Floor	MO-2	161	S 07° W S = 1.55 cm/g	N 83° W S = 1.49 cm/g	Up S = 2.37 cm/g	35 mm	3'
71.185	Los Angeles 5900 Wilshire Blvd.	"B" Parking Lot	MO-2	112	S 07° W S = 1.52 cm/g	N 83° W S = 1.50 cm/g	Up S = 2.39 cm/g	35 mm	4'
71.186	Los Angeles 5900 Wilshire Blvd.	Penthouse	MO-2	178	S 07° W S = 1.50 cm/g	N 83° W S = 1.47 cm/g	Up S = 2.30 cm/g	35 mm	6'
71.187	Los Angeles 3411 Wilshire Blvd.	5th Basement	MO-2	61	West S* = 1.64 cm/g	South S* = 1.59 cm/g	Up S* = 2.30 cm/g	35 mm	4'
71.188	Los Angeles 3550 Wilshire Blvd.	Basement	MO-2	194	West S = 1.52 cm/g	North S = 1.58 cm/g	Up S = 2.35 cm/g	35 mm	6'
71.189	Los Angeles 5260 Century Blvd.	Roof	MO-2	63	East S* = 1.57 cm/g	North S* = 1.58 cm/g	Up S* = 2.45 cm/g	35 mm	41'
71.190	Los Angeles 5260 Century Blvd.	1st Floor	MO-2	72	East S* = 1.63 cm/g	North S* = 1.61 cm/g	Up S* = 2.31 cm/g	35 mm	3'
71.191	Los Angeles 5260 Century Blvd.	4th Floor	MO-2	76	East S* = 1.63 cm/g	North S* = 1.61 cm/g	Up S* = 2.31 cm/g	35 mm	4'
71.192	Los Angeles 6464 Sunset Blvd.	Basement	MO-2	123	East S* = 1.52 cm/g	South S* = 1.49 cm/g	Up S* = 2.40 cm/g	35 mm	15'
71.193	Los Angeles 6464 Sunset Blvd.	12th Floor	MO-2	116	East S* = 1.51 cm/g	South S* = 1.57 cm/g	Up S* = 2.44 cm/g	35 mm	16'
71.194	Los Angeles 6430 Sunset Blvd.	1st Floor	MO-2	56	South S* = 1.57 cm/g	East S* = 1.54 cm/g	Up S* = 2.43 cm/g	35 mm	14'
71.195	Los Angeles 930 Hilgard Ave.	15th Floor	MO-2	158	N 76° W S = 1.54 cm/g	N 14° E S = 1.57 cm/g	Up S = 2.28 cm/g	35 mm	18'
71.196	Los Angeles 1900 Avenue of the Stars	Basement	MO-2	121	N 44° E S* = 1.58 cm/g	S 44° E S* = 1.58 cm/g	Up S* = 2.30 cm/g	35 mm	5'
71.197	Los Angeles 1900 Avenue of the Stars	29th Floor	MO-2	197	N 44° E S* = 1.54 cm/g	S 44° E S* = 1.54 cm/g	Up S* = 2.37 cm/g	35 mm	5'
71.198	Los Angeles 234 Figueroa St.	Basement	MO-2	54	S 53° E S* = 1.61 cm/g	N 37° E S* = 1.62 cm/g	Up S* = 2.46 cm/g	35 mm	3'
71.199	Los Angeles 234 Figueroa St.	Roof	MO-2	62	S 53° E S* = 1.65 cm/g	N 37° E S* = 1.62 cm/g	Up S* = 2.36 cm/g	35 mm	3'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address		Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.200	Los Angeles 533 S. Fremont Ave.	Basement	MO-2	73	N 30° W S* = 1.58 cm/g	S 60° W S* = 1.56 cm/g	Up S* = 2.40 cm/g	35 mm	4'
71.201	Los Angeles 533 S. Fremont Ave.	6th Floor	MO-2	93	N 30° W S* = 1.57 cm/g	S 60° W S* = 1.63 cm/g	Up S* = 2.44 cm/g	35 mm	4'
71.202	Los Angeles 11661 San Vicente Blvd.	5th Floor	MO-2	172	S 55° W S* = 1.53 cm/g	N 35° W S* = 1.58 cm/g	Up S* = 2.38 cm/g	35 mm	13'
71.203	Los Angeles 11661 San Vicente Blvd.	11th Floor	MO-2	198	S 55° W S* = 1.55 cm/g	N 35° W S* = 1.55 cm/g	Up S* = 2.41 cm/g	35 mm	13'
71.204	Los Angeles 15433 Ventura Blvd.	7th Floor	MO-2	104	N 12° E S* = 1.57 cm/g	N 78° W S* = 1.53 cm/g	Up S* = 2.48 cm/g	35 mm	8'
71.205	Los Angeles 15433 Ventura Blvd.	13th Floor	MO-2	86	N 12° E S* = 1.60 cm/g	N 78° W S* = 1.63 cm/g	Up S* = 2.37 cm/g	35 mm	19'
71.206	Los Angeles 2080 Century Park East	Roof	MO-2	199	N 40° W S* = 1.55 cm/g	N 50° E S* = 1.54 cm/g	Up S* = 2.43 cm/g	35 mm	9'
71.207	Los Angeles 3550 Wilshire Blvd.	Roof	MO-2	193	West S = 1.54 cm/g	North S = 1.49 cm/g	Up S = 2.40 cm/g	35 mm	12'
71.208	Los Angeles 3550 Wilshire Blvd.	11th Floor	MO-2	186	West S = 1.51 cm/g	North S = 1.58 cm/g	Up S = 2.32 cm/g	35 mm	7'
71.209	Los Angeles 3411 Wilshire Blvd.	Penthouse	MO-2	32	West S = 1.65 cm/g	South S = 1.54 cm/g	Up S = 2.47 cm/g	35 mm	3'
71.210	Los Angeles 3411 Wilshire Blvd.	13th Floor	MO-2	68	West S = 1.61 cm/g	South S = 1.59 cm/g	Up S = 2.48 cm/g	35 mm	4'
71.211	Los Angeles 533 S. Fremont Ave.	11th Floor	MO-2	77	N 30° W S = 1.57 cm/g	S 60° W S = 1.58 cm/g	Up S = 2.50 cm/g	35 mm	7'
71.212	Los Angeles 222 Figueroa St.	12 Floor	MO-2	192	S 37° W S = 1.48 cm/g	N 53° W S = 1.52 cm/g	Up S = 2.36 cm/g	35 mm	6'
71.213	Los Angeles 930 Hilgard Ave.	8th Floor	MO-2	200	N 76° W S = 1.50 cm/g	N 14° E S = 1.55 cm/g	Up S = 2.38 cm/g	35 mm	13'
71.214	Los Angeles 930 Hilgard Ave.	Basement	MO-2	166	N 76° W S = 1.51 cm/g	N 14° E S = 1.53 cm/g	Up S = 2.34 cm/g	35 mm	23'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.215	Los Angeles 9100 Wilshire Blvd.	Roof Level	MO-2	107	East S = 1.58 cm/g	South S = 1.56 cm/g	Up S = 2.39 cm/g	35 mm	7'
71.216	Los Angeles 1900 Avenue of the Stars	16th Floor	MO-2	144	N 44° E S = 1.52 cm/g	S = 46° E S = 1.53 cm/g	Up S = 2.31 cm/g	35 mm	8'
71.217	Maricopa	Array Sta. 1	RFT-250	191	S 40° W S = 1.9 cm/g	Down S = 1.9 cm/g	S 50° E S = 1.9 cm/g	70 mm	3'
71.218	Maricopa	Array Sta. 2	RFT-250	192	S 40° W S = 1.9 cm/g	Down S = 1.9 cm/g	S 50° E S = 1.9 cm/g	70 mm	3'
71.219	Isabella Dam	Gallery	RFT-250	108	N 14° E S = 1.9 cm/g	Down S = 1.9 cm/g	N 76° W S = 1.9 cm/g	70 mm	3'
71.220	Isabella Dam	Auxiliary Abutment	RFT-250	112	N 14° E S = 1.9 cm/g	Down S = 1.9 cm/g	N 76° W S = 1.9 cm/g	70 mm	3'
71.221	Isabella Dam	Auxiliary Crest	RFT-250	110	N 14° E S = 1.9 cm/g	Down S = 1.9 cm/g	N 76° W S = 1.9 cm/g	70 mm	3'
71.222	Isabella Dam	Control Tower	RFT-250	111	N 14° E S = 1.9 cm/g	Down S = 1.9 cm/g	N 76° W S = 1.9 cm/g	70 mm	3'
71.223	Isabella Dam	Crest	RFT-250	109	N 14° E S = 1.9 cm/g	Down S = 1.9 cm/g	N 76° W S = 1.9 cm/g	70 mm	3'
71.224	Terminus Dam	Control Tower	RFT-250	105	S 81° E S = 1.9 cm/g	Down S = 1.9 cm/g	N 9° E S = 1.9 cm/g	70 mm	3'
71.225	Terminus Dam	Crest	RFT-250	106	S 61° E S = 1.9 cm/g	Down S = 1.9 cm/g	N 9° E S = 1.9 cm/g	70 mm	2'
71.226	Terminus Dam	North Abutment	RFT-250	107	S 81° E S = 1.9 cm/g	Down S = 1.9 cm/g	N 9° E S = 1.9 cm/g	70 mm	3'
71.227	Loma Linda Loma Linda University	Medical Center Basement	RFT-250	187	East S* = 1.9 cm/g	Down S* = 1.9 cm/g	North S* = 1.9 cm/g	70 mm	'
71.228	San Diego Gas and Electric Bldg.	Basement	RFT-250	209	East S* = 1.9 cm/g	Down S* = 1.9 cm/g	North S* = 1.9 cm/g	70 mm	5'
71.229	Anza Anza Post Office	Storage Room	RFT-250	181	N 45° E S* = 1.9 cm/g	Down S* = 1.9 cm/g	N 45° W S* = 1.9 cm/g	70 mm	4'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers			Record	
					1	2	3	Type	Length
71.230	Borrego Springs Fire Dept.	Shop	RFT-250	157	S 45° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 45° E S* = 1.9 cm/g	70 mm	3'
71.231	El Centro	Community Hospital	RFT-250	182	S 52° W S* = 1.97 cm/g	Down S* = 1.9 cm/g	S 38° E S* = 1.97 cm/g	70 mm	6'
71.232 After-shock	Sand Canyon	Los Angeles County Fire Sta.	SMA-1	168	N 45° E S = 1.79 cm/g	Down S = 1.91 cm/g	N 45° W S = 1.87 cm/g	70 mm	3'
71.233 After-shock	Los Angeles Van Norman Resv.	Meter House	SMA-1	214	North S = 1.85 cm/g	Down S = 1.85 cm/g	West S = 1.95 cm/g	70 mm	5'
71.234	Hemet Fire Station	Hose Storage Room	RFT-250	159	S 45° W S* = 1.9 cm/g	Down S* = 1.9 cm/g	S 45° E S* = 1.9 cm/g	70 mm	5'
71.235	Bakersfield	Harvey Auditorium	C&GS DM	18	LDM	West	MAG = 1	6"	3'
71.236	Bakersfield	Harvey Auditorium	C&GS DM	18	RDM	South	MAG = 1	6"	3'
71.237	Bakersfield	Harvey Auditorium	C&GS DM	29	Up S = 13.3 cm/g	South S = 13.4 cm/g	West S = 13.0 cm/g	6"	4'
71.238	Taft Lincoln School	Tunnel	C&GS	6	Up S = 18.0 cm/g	N 21° E S = 19.9 cm/g	S 69° E S = 20.2 cm/g	6"	7'
71.239	San Diego Light and Power Co.	Service Building	C&GS	5D	Up S = 20.3 cm/g	East S = 19.5 cm/g	South S = 20.1 cm/g	6"	8'
71.240	Cholame-Shandon	Array Sta. 2	AR-240	133	N 51° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 39° W S* = 7.6 cm/g	12"	4'
71.241	Cholame-Shandon	Array Sta. 8	AR-240	166	N 51° E S* = 7.6 cm/g	Down S* = 7.6 cm/g	N 39° W S* = 7.6 cm/g	12"	4'
71.242	Taft Lincoln School	Roof	AR-240	271	S 21° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 69° E S* = 7.6 cm/g	12"	6'
71.243	Buena Vista Taft	CWR site	AR-240	107	South S* = 7.6 cm/g	Down S* = 7.6 cm/g	East S* = 7.6 cm/g	12"	6'

*Nominal Sensitivity

NATIONAL OCEAN SURVEY NOAA: SEISMOLOGICAL FIELD SURVEY
FOR EARTHQUAKE ON FEB. 9, 1971 (Contd)

Log No.	Address	Location	Machine type	Serial No.	Other identifying numbers 1	Other identifying numbers 2	Other identifying numbers 3	Record Type	Record Length
71.244	Hoover Dam	Intake Tower	C&GS	B-2	Up S = 19.1 cm/g S 45° W** MAG = 1.1	N 45° W S = 20.8 cm/g N 45° W*** MAG = 1.1	N 45° E S = 20.9 cm/g	12"	6'
71.245	Hoover Dam	Oil House	C&GS	B-3	Up S = 19.3 cm/g S 45° W** MAG = 1.0	N 45° W S = 19.9 cm/g	N 45° W MAG = 1.0 N 45° E*** S = 20.2 cm/g	12"	8'
71.246	Hoover Dam	1215 Gallery	C&GS	B-1	Up S = 19.7 cm/g	S 45° W MAG = 1.0 S 45° E** S = 19.4 cm/g	S 45° E MAG = 1.0 S 45° W*** S = 19.3 cm/g	12"	7'
71.247	Los Angeles 808 S. Olive Ave.	Basement	AR-240	198	S 37° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 53° E S* = 7.6 cm/g	12"	18'
71.248	Cedar Springs	Pump House	AR-240	187	S 36° W S* = 7.6 cm/g	Down S* = 7.6 cm/g	S 54° E S* = 7.6 cm/g	12"	3'
71.249 After- shock	Los Angeles 8244 Orion	8th Floor	AR-240	199	North S* = 7.6 cm/g	Down S* = 7.6 cm/g	West S* = 7.6 cm/g	12"	5'
71.250 After- shock	Los Angeles 8244 Orion Blvd.	4th Floor	AR-240	210	North S* = 7.6 cm/g	Down S* = 7.6 cm/g	West S* = 7.6 cm/g	12"	5'
71.251 After- shock	Los Angeles 8244 Orion Blvd.	1st Floor	AR-240	190	North S* = 7.6 cm/g	Down S* = 7.6 cm/g	West S* = 7.6 cm/g	12"	6'

*Nominal Sensitivity.
**Identifying Number, 4.
***Identifying Number, 5.

<u>Sample Accelerograms.</u> Figures 2.2a, b illustrate sets of three building accelerograms obtained on AR-240 accelerographs recording on 12-inch paper. The record at 8244 Orion is of interest as being the building closest (ten miles) to the epicenter. Figures 2.2c, d are typical records at AR-240 special stations. Figure 2.2e is a sample of a set of three building accelerograms recorded on 35-mm film by MO-2 type accelerographs. Figure 2.2f is a set of three building accelerograms recorded on 70-mm film by SMA-1 accelerographs, and Figure 2.2g is from a SMA-1 ground station.

The records of Figure 2.3 are from a telephone line interconnected accelerograph system joining the Caltech campus, the Seismological Laboratory, and the Jet Propulsion Laboratory, over a distance of some six miles. These five RFT-250 accelerographs are so arranged that the first one to trigger from earthquake ground motion will simultaneously start the other instruments in the network*. For the San Fernando earthquake, the direction of earthquake wave travel from the epicenter is such that the Seismological Laboratory and the Jet Propulsion Laboratory accelerographs should trigger about the same time, some one to two seconds before the initial earthquake waves would reach the Millikan Library accelerograph on the Caltech campus. A comparison of the records of Figure 2.3 shows that the Millikan Library recording was started in advance of the main earthquake ground motions as indicated at the other stations. A comparison of the Millikan Library record with that obtained at another building on the campus, the Athenaeum, which was not on the interconnected network,

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*Keightley, W.O., "A Strong-Motion Accelerograph Array with Telephone Line Interconnections," Report No. EERL 70-05, Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, 1970.

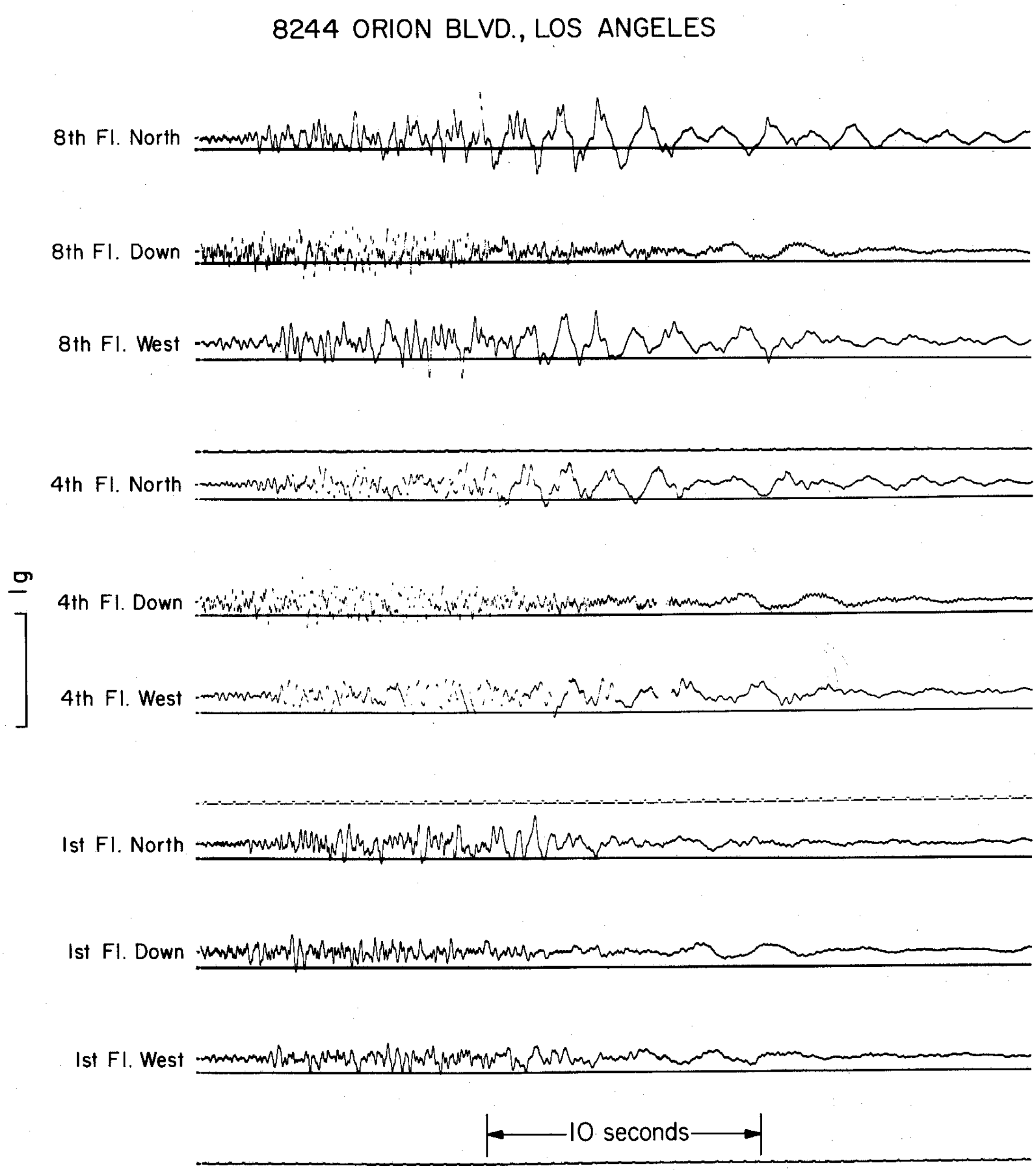

Figure 2.2a AR-240 Accelerograph record from Holiday Inn, 8244 Orion

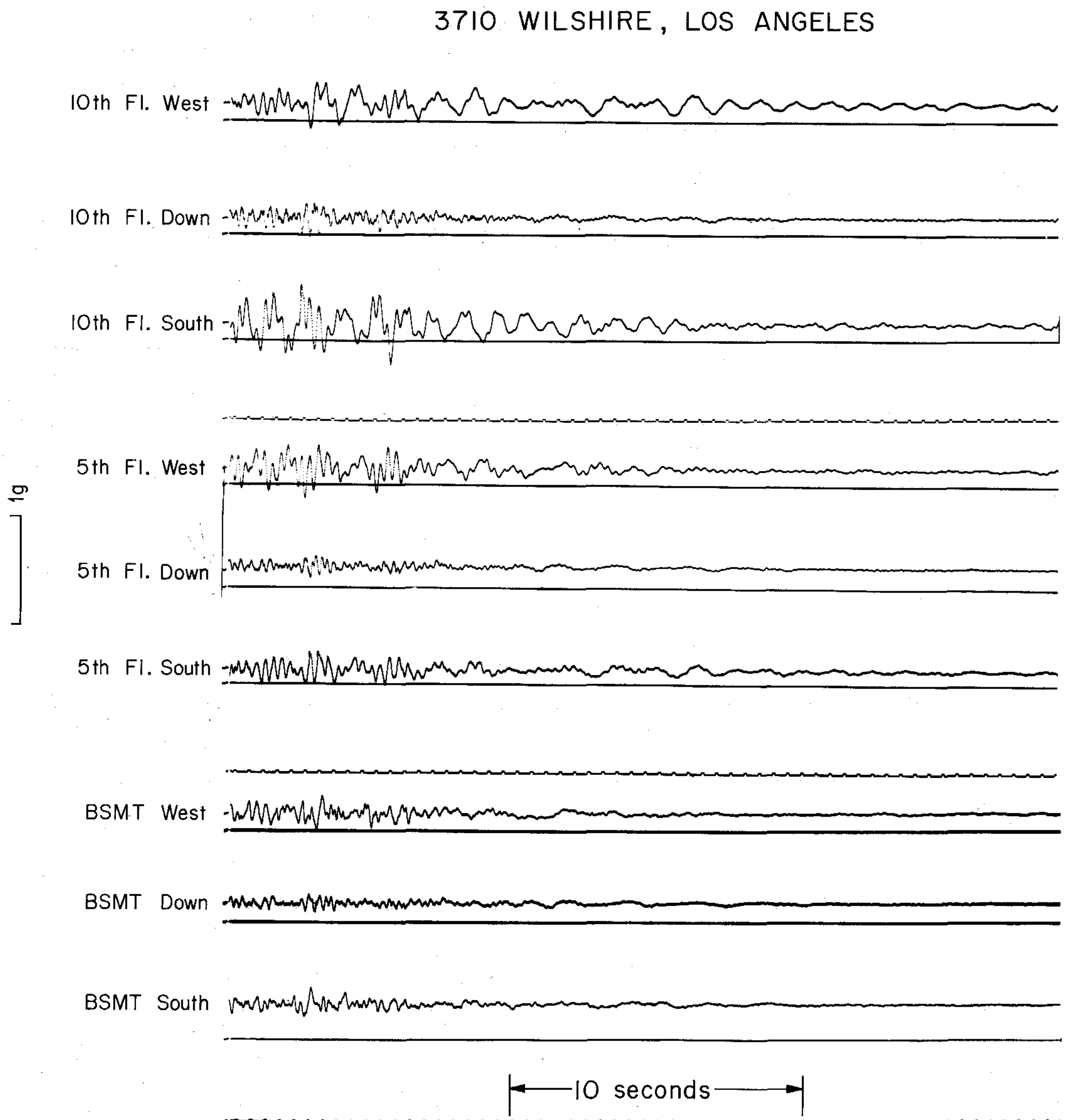

Figure 2.2b AR-240 Accelerograph record from 3710 Wilshire Blvd., Los Angeles

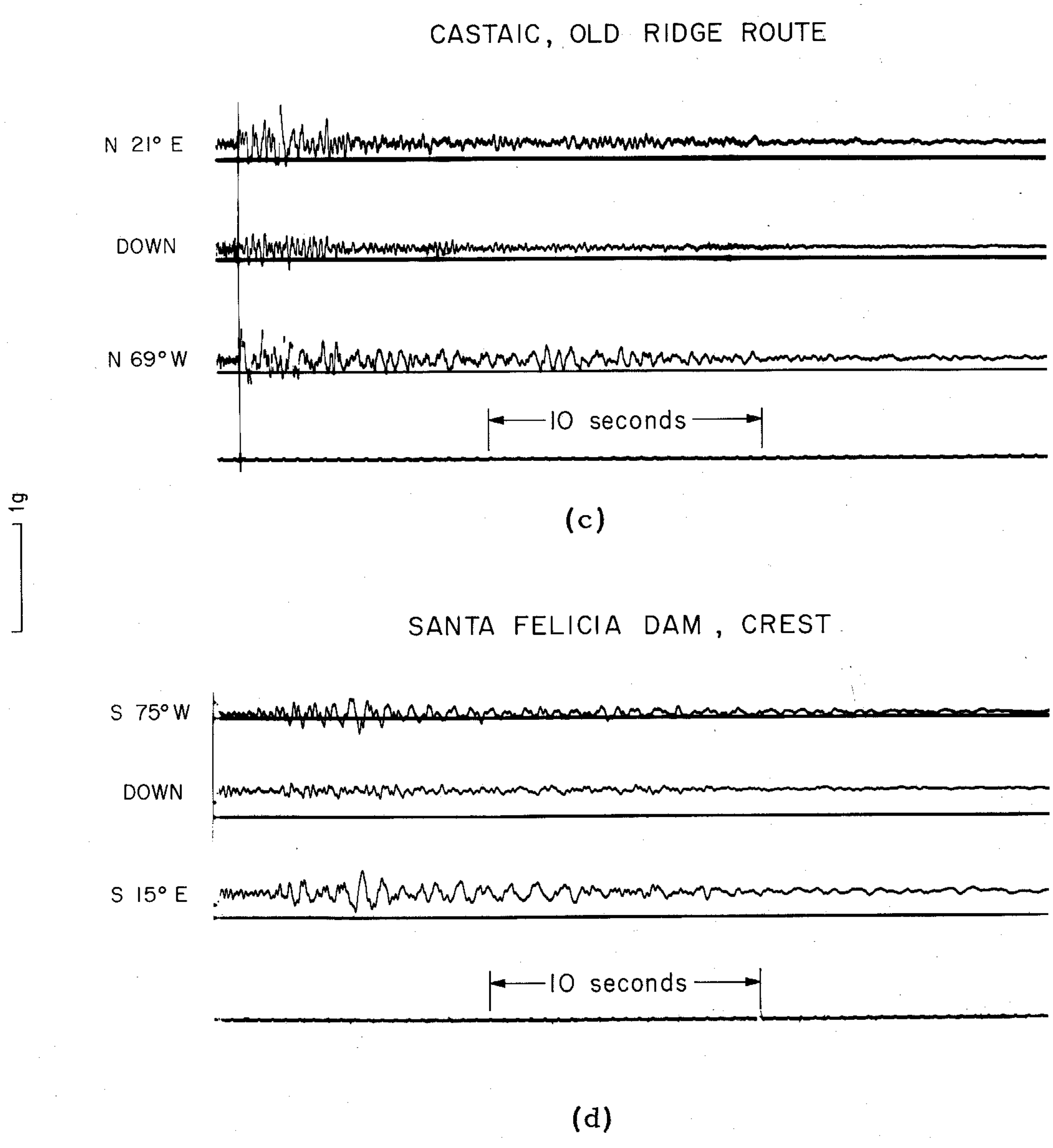

Figure 2.2c, d AR-240 Accelerograph records from special sites.

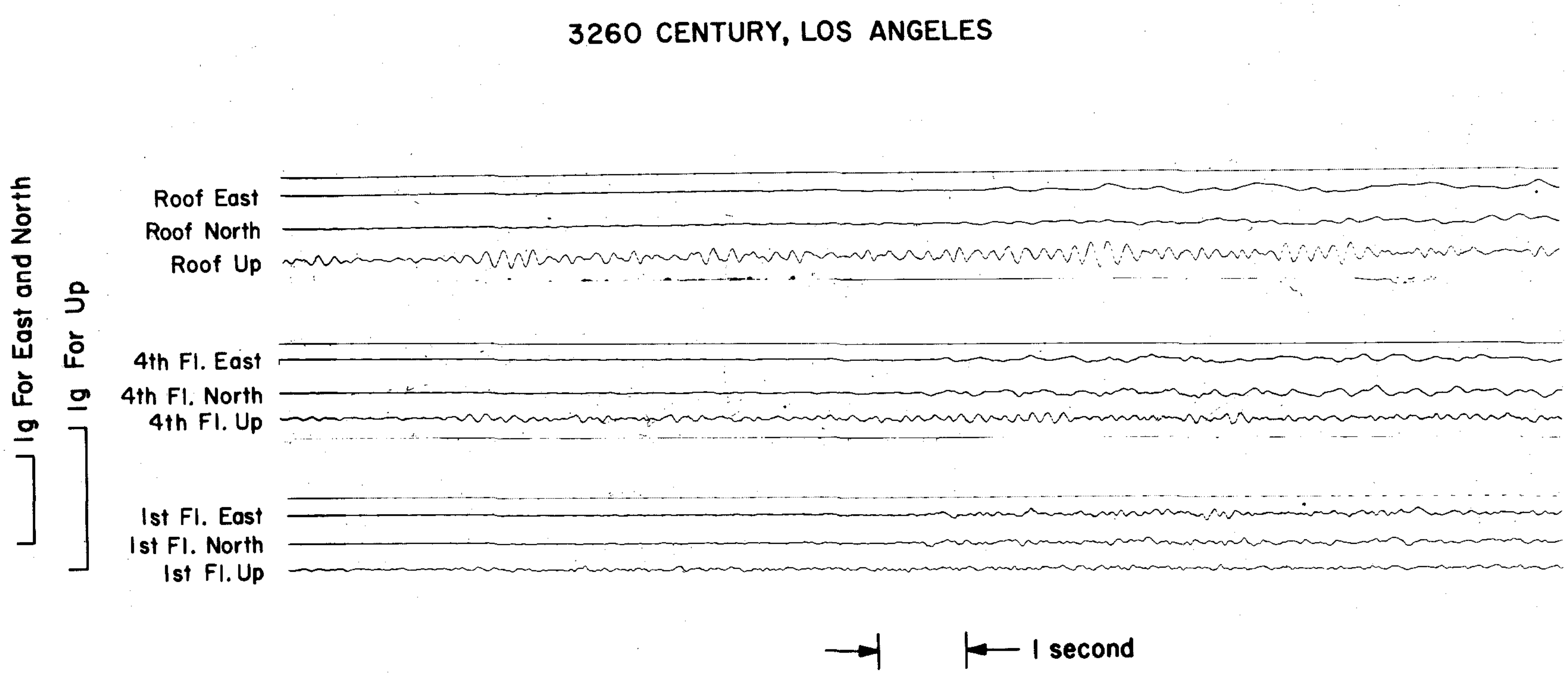

Figure 2.2e MO-2 Accelerograph record from 3260 Century Blvd., Los Angeles

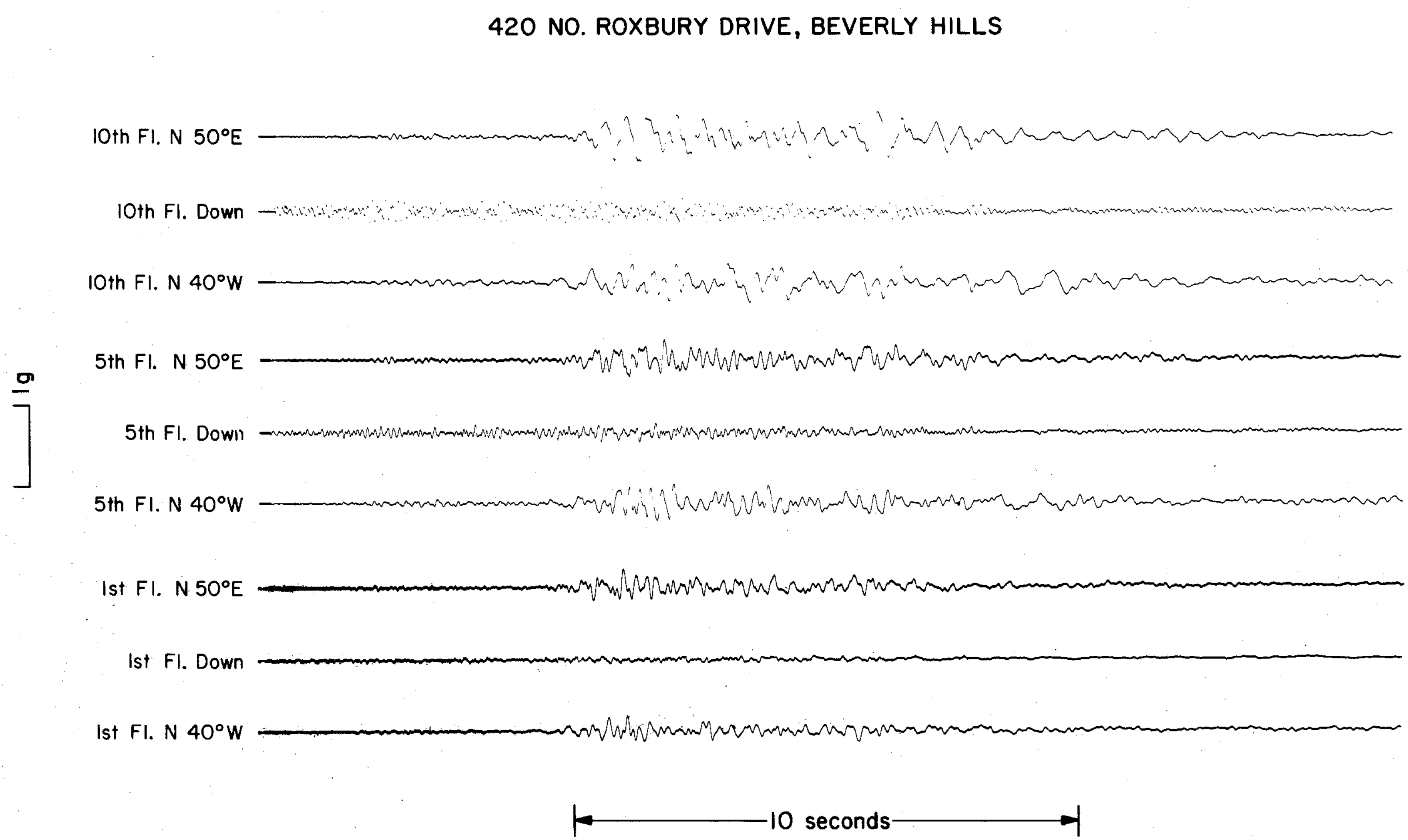

Figure 2.2f SMA-1 Accelerograph record from 420 N. Roxbury Drive, Beverly Hills.

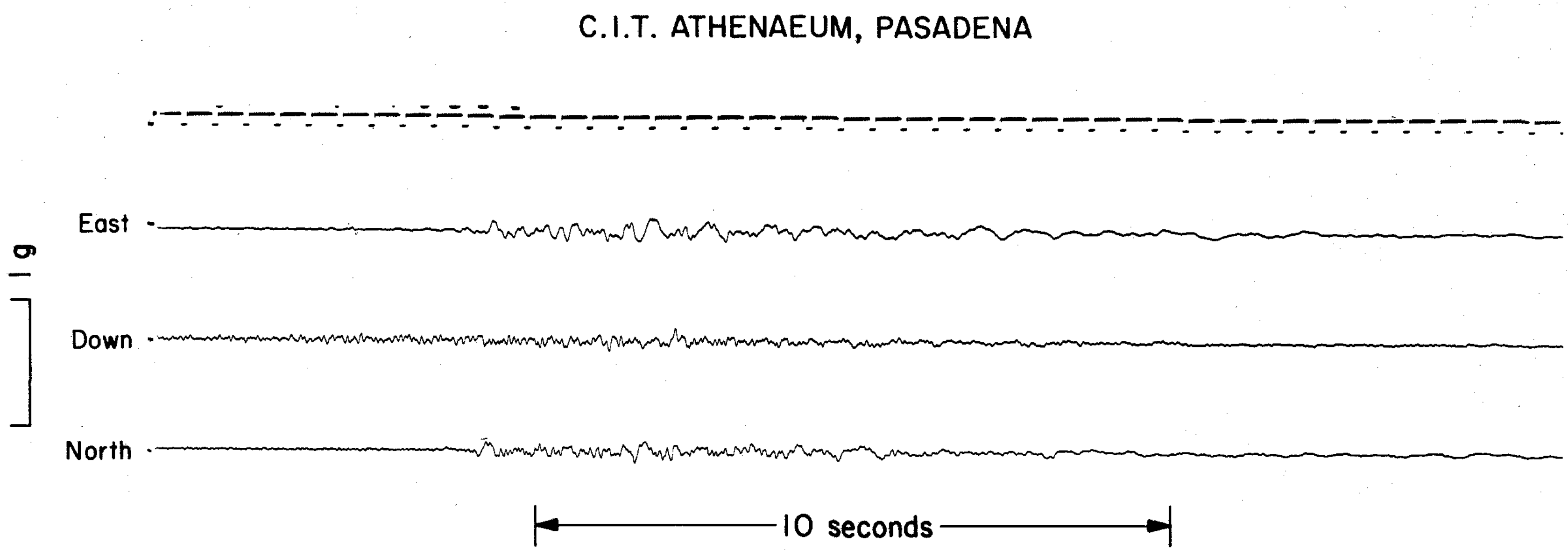

Figure 2.2g SMA-1 Accelerograph record from the Athenaeum, California Institute of Technology, Pasadena.

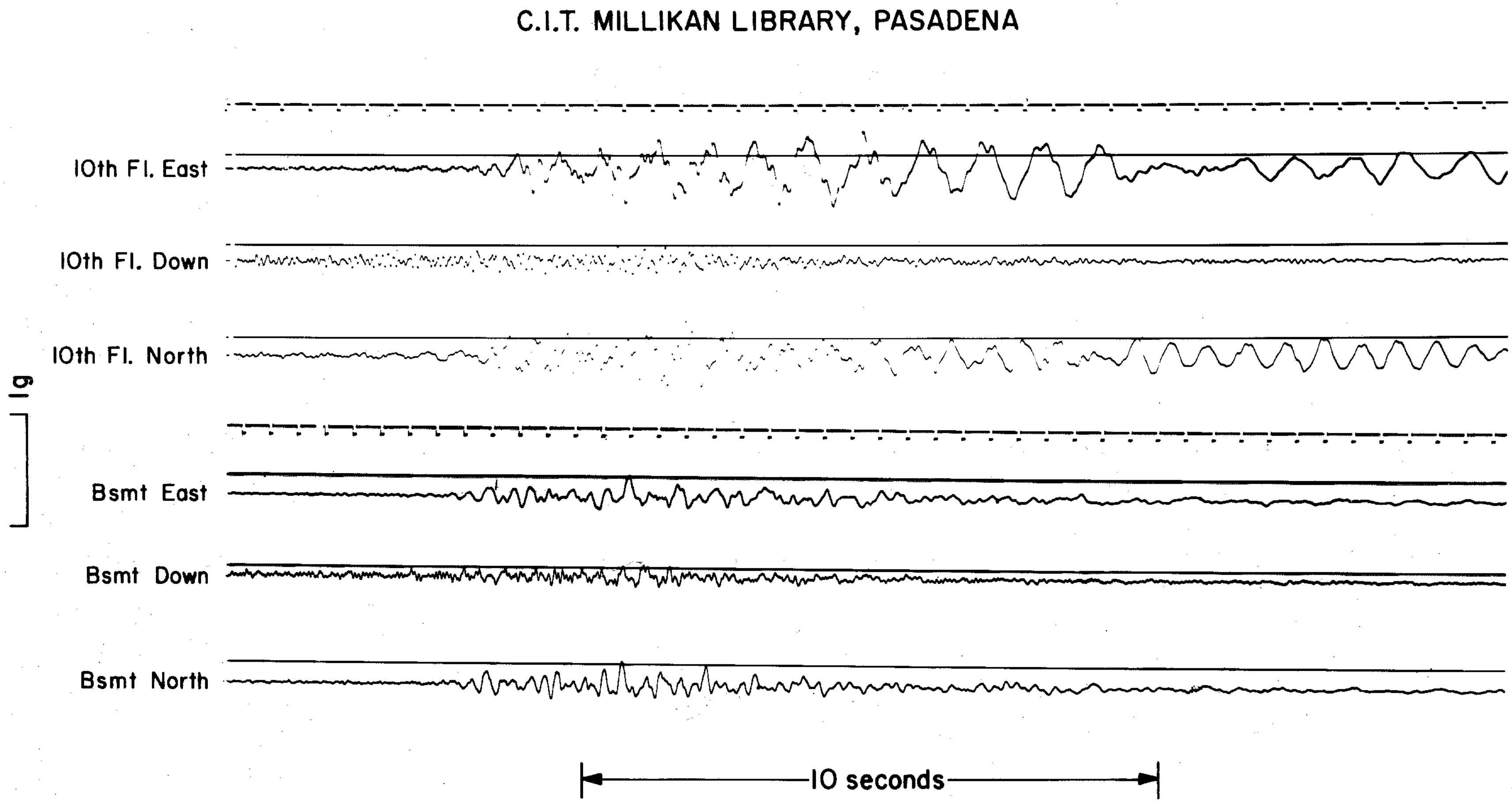

Figure 2.3a Accelerograph record from the interconnected instrument system, Millikan Library station on the C.I.T. campus, Pasadena.

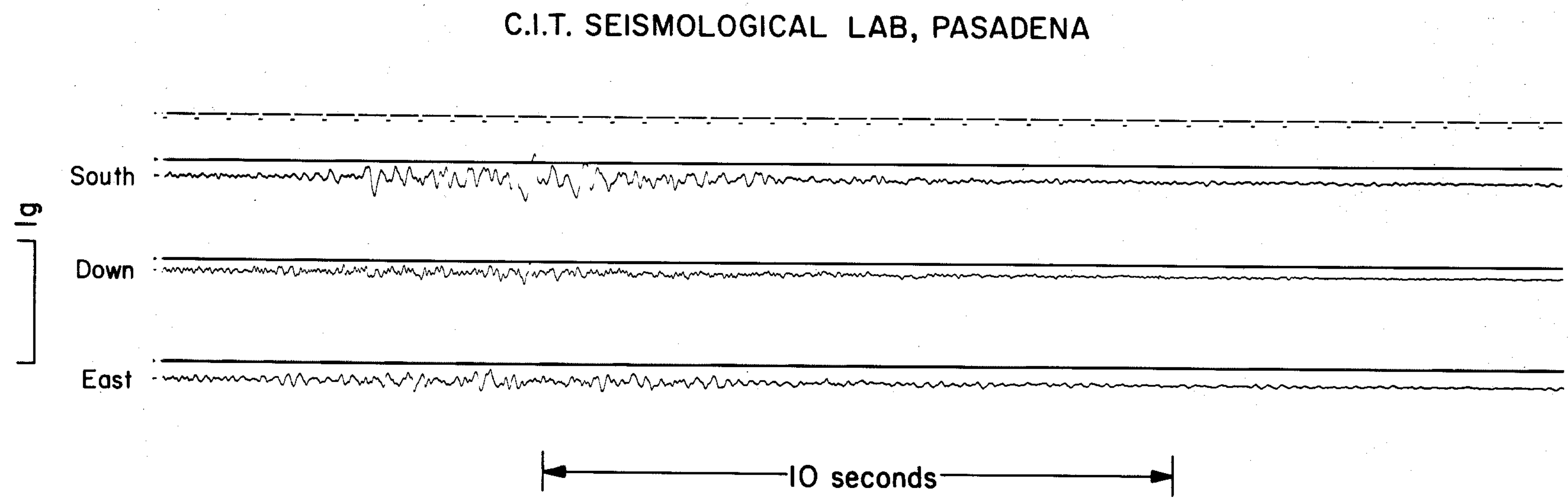

Figure 2. 3b Accelerograph record from the interconnected instrument system station at the C. I. T. Seismological Laboratory, Pasadena.

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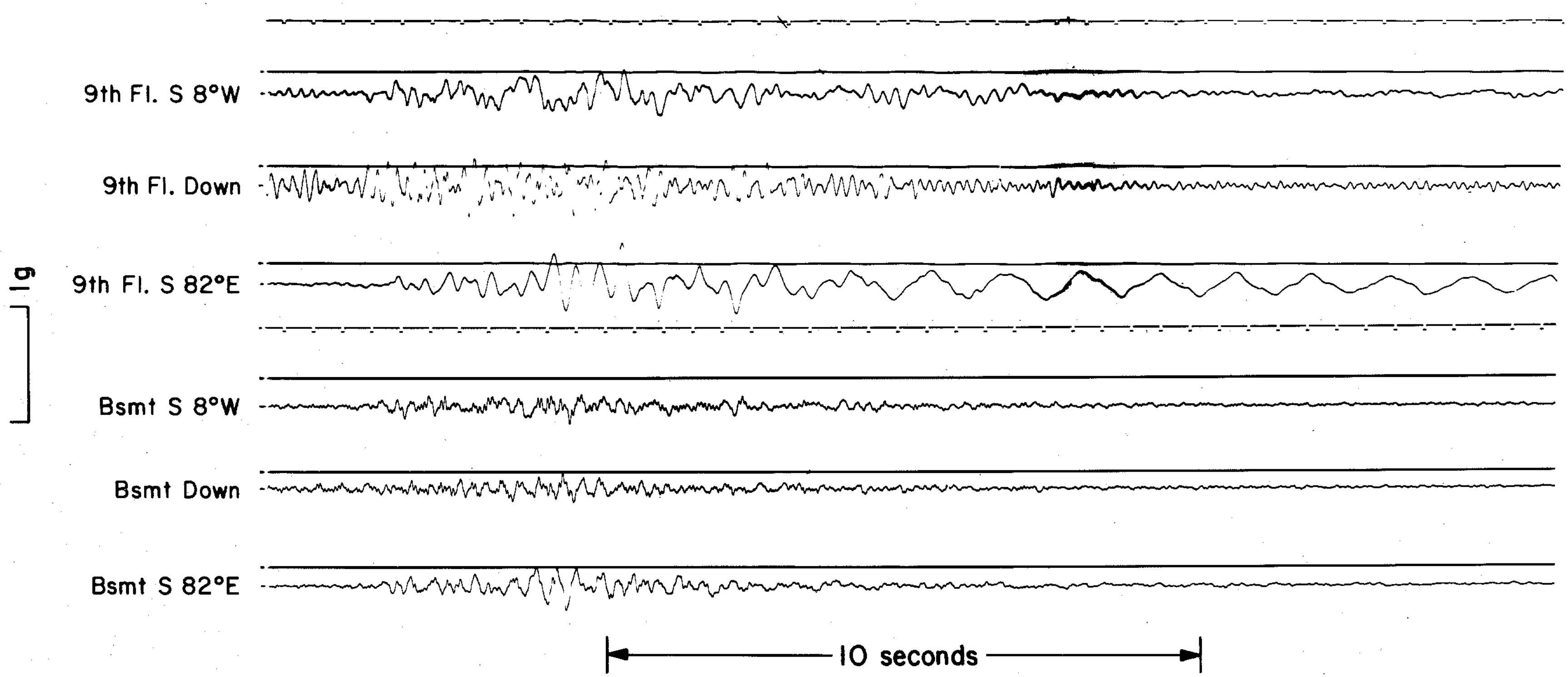

Figure 2. 3c Accelerograph record from the interconnected instrument system station at the Jet Propulsion Laboratory, Pasadena.

indicates that the self-triggered Athenaeum accelerograph started before the distance-triggered Library instrument. This is because the SMA-1 accelerograph in the Athenaeum is triggered by a vertical starter, and the arrival of the vertical waves preceded the arrival of the horizontal waves which triggered the RFT-250 accelerographs in the network by an amount more than enough to make up for the gain in triggering time in the inter-connected network. In this case the vertical starter was evidently a simpler way to ensure an early start than the interconnected network.

It is also evident that the additional complexity of the interconnection system has not reduced the reliability of the system, which operated correctly in all respects. For this particular earthquake, the main ground motion was preceded by smaller shaking which served to start the accelerograph in good time to record a relatively complete picture of significant ground motion. The telephone interconnected system, or an accelerograph with a memory, was therefore not required. This is probably also likely to be true for most damaging earthquakes, although the possibility certainly exists that in some cases some information might be lost for an abrupt beginning.

Millikan Library Response. The accelerogram of Figure 2.3a taken on the roof of the nine-story Millikan Library on the Caltech campus is of unusual interest because of the complete dynamic investigations which had been made on this building before the earthquake.*

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*Kuroiwa, J. H., "Vibration Test of a Multistory Building," Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, 1967.

Blandford, R. R., McLamore, V. R., and Aunon, J., "Structural Analysis of Millikan Library from Ambient Vibrations," Report No. 616-0268-2107, Teledyne Earth Sciences, February, 1968.

In Figure 2.4a is shown the record of a man-excited low-level vibration which indicates that the fundamental EW lateral natural period of vibration at small amplitudes of motion was 0.66 seconds before the earthquake. This value was also confirmed by forced vibration resonance tests at a considerably higher force level, and by ambient vibration tests involving wind and microtremor excitations. The earthquake accelerogram of Figure 2.4b shows that during the earthquake vibrations, which reached a peak acceleration of 0.37 g, the period of fundamental mode was 1.01 seconds. This considerable lengthening of the period is to be attributed to the much higher levels of the building motion and to the nonlinear character of the structure.

The accelerogram of Figure 2.4c shows that for the small building motions excited by a small earthquake aftershock, the fundamental period was 0.76 seconds. A low-level wind-excited test was then run to check the final post-earthquake state of the building, and the period was found to be 0.77 seconds, as in Figure 2.4d It is evident that after the earthquake the small-amplitude motion had been permanently lengthened by some 15 percent. There was apparently no significant structural damage to the building, and the period change is probably to be attributed to alterations in the attachment of pre-cast concrete window panel sections which made up the N-S faces of the building.

Such permanent period changes have been noted in past earthquakes*, and have been observed in numerous buildings during the San Fernando earthquake, as will be reported in more detail in the course of investigations now being conducted.

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*Esteva, L., y Nieto, J. A., "El Temblor de Lima, Peru, October 17, 1966," Vol. XXXVII, Revista Ingenieria, Mexico, 1967.

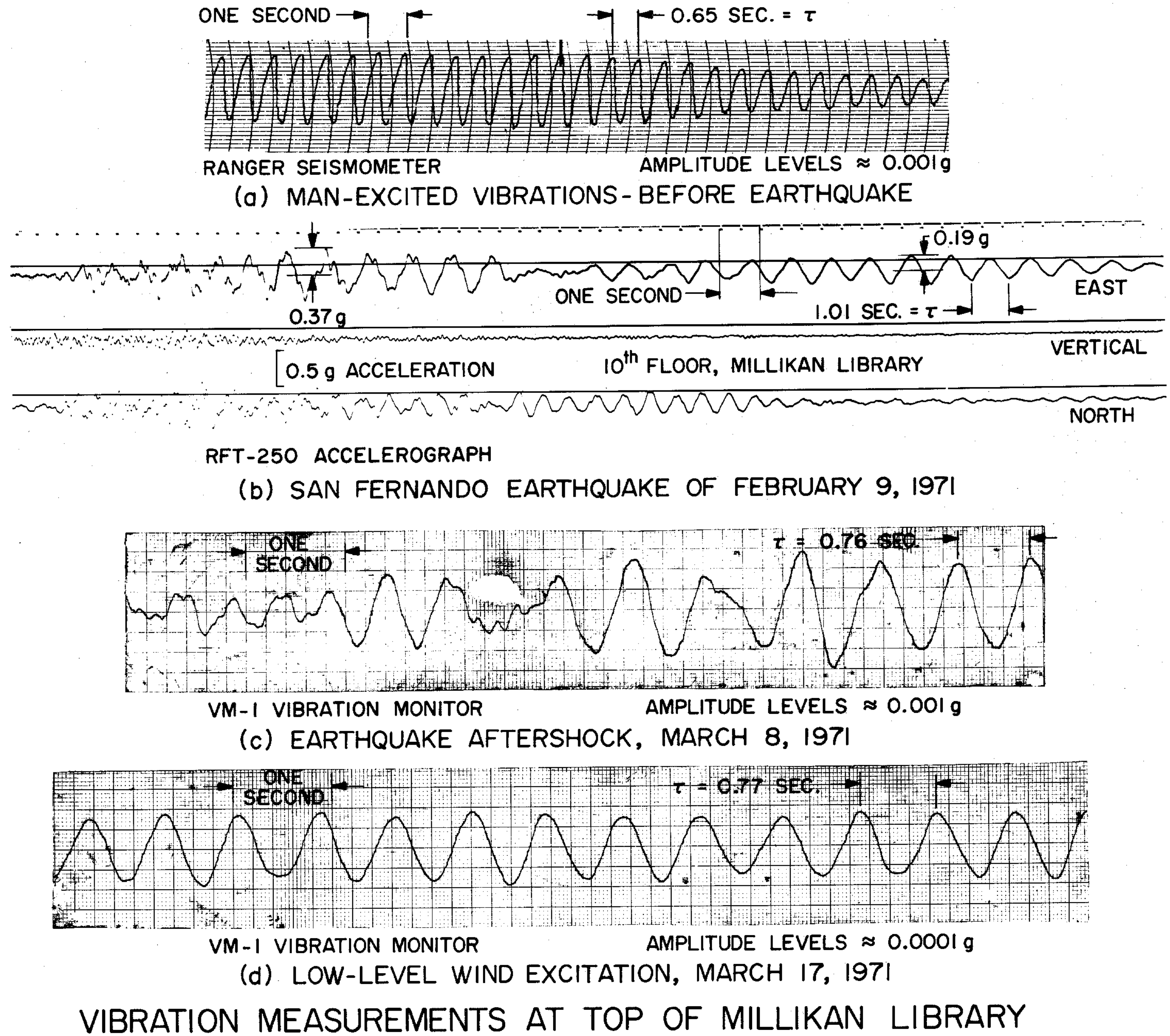

Figure 2.4 Instrument records from Millikan Library.

The Pacoima Dam Accelerogram. Because of the very special interest and importance of the Pacoima Dam record, which was obtained virtually on top of the earthquake, a special effort was made for a prompt analysis of the record and dissemination of the results. Since the site conditions under which the record was obtained were unusual, a special investigation was made of the situation there, and of the condition of the accelerograph after the earthquake. These investigations, and the analysis of the accelerogram, have been detailed in a paper to appear in a future issue of the Bulletin of the Seismological Society of America. To make this information available at an earlier date, the paper is being pre-printed in the present report in the following section.

As mentioned in the following paper, the Pacoima Dam record is unique in that a small permanent tilt of the horizontal starter pendulum kept the accelerograph running to record a whole initial sequence of aftershocks in exact time scale. Because of the special interest of this complete record, all 360 seconds of the accelerogram are reproduced to scale in Figure 2.5.

Acknowledgements. It will be recognized that the above data handling and processing program involved the cooperation of a large number of organizations and individuals, many of whom made major contributions of time under difficult circumstances. We particularly appreciate the fine cooperation we have always received from the Seismological Field Survey (NOAA) including W. K. Cloud, chief; W. R. Maley, head of the Los Angeles office; and E. C. Etheridge, of the Los Angeles staff. From the San Francisco office of the Seismological Field Survey, C. F. Knudson, B. J. Morrill, and their staff of technicians rendered

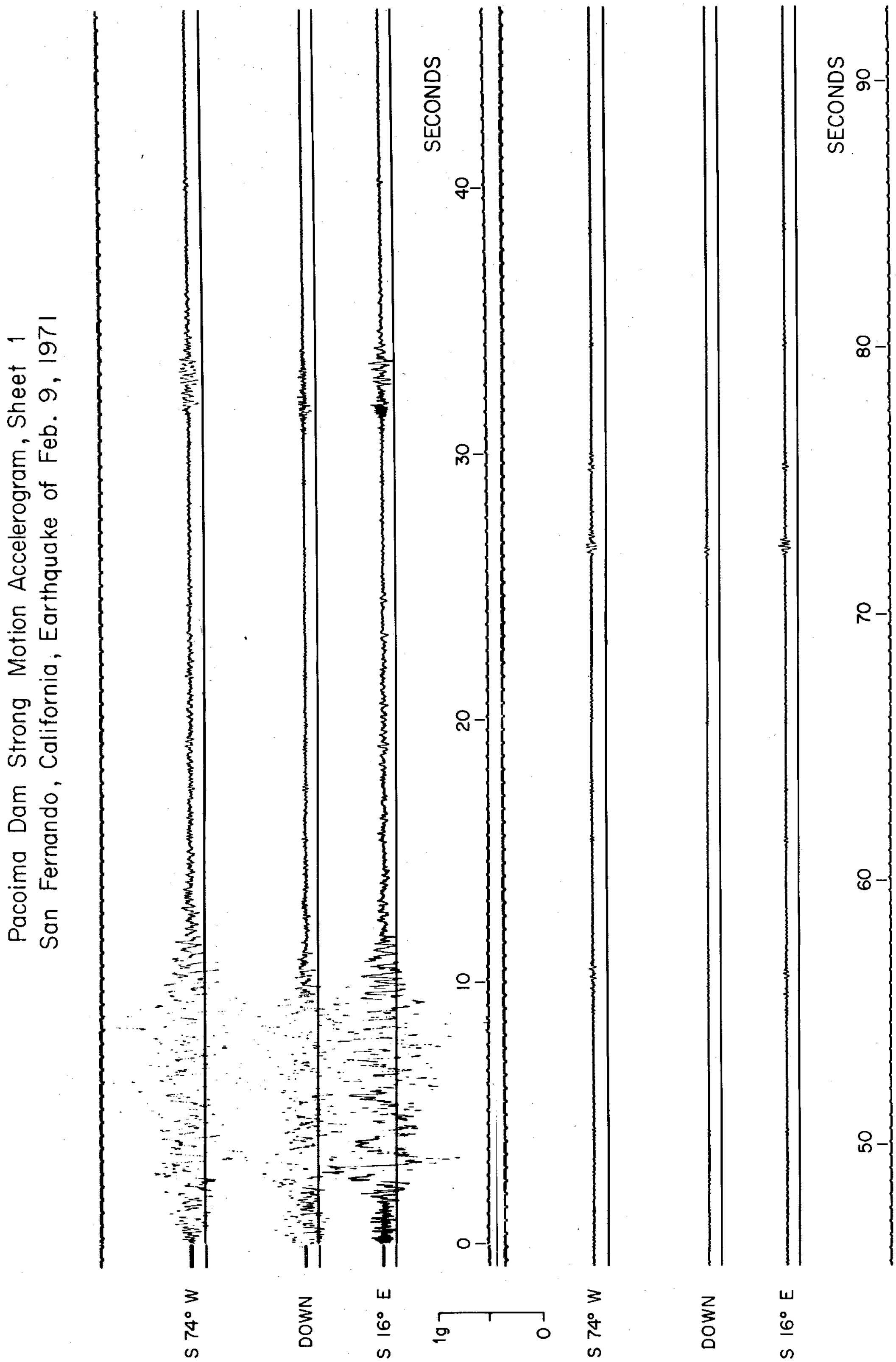

Figure 2.5 Pacoima Dam accelerograph record.

Pacoima Dam Strong Motion Accelerogram, Sheet 2
San Fernando, California, Earthquake of Feb. 9, 1971

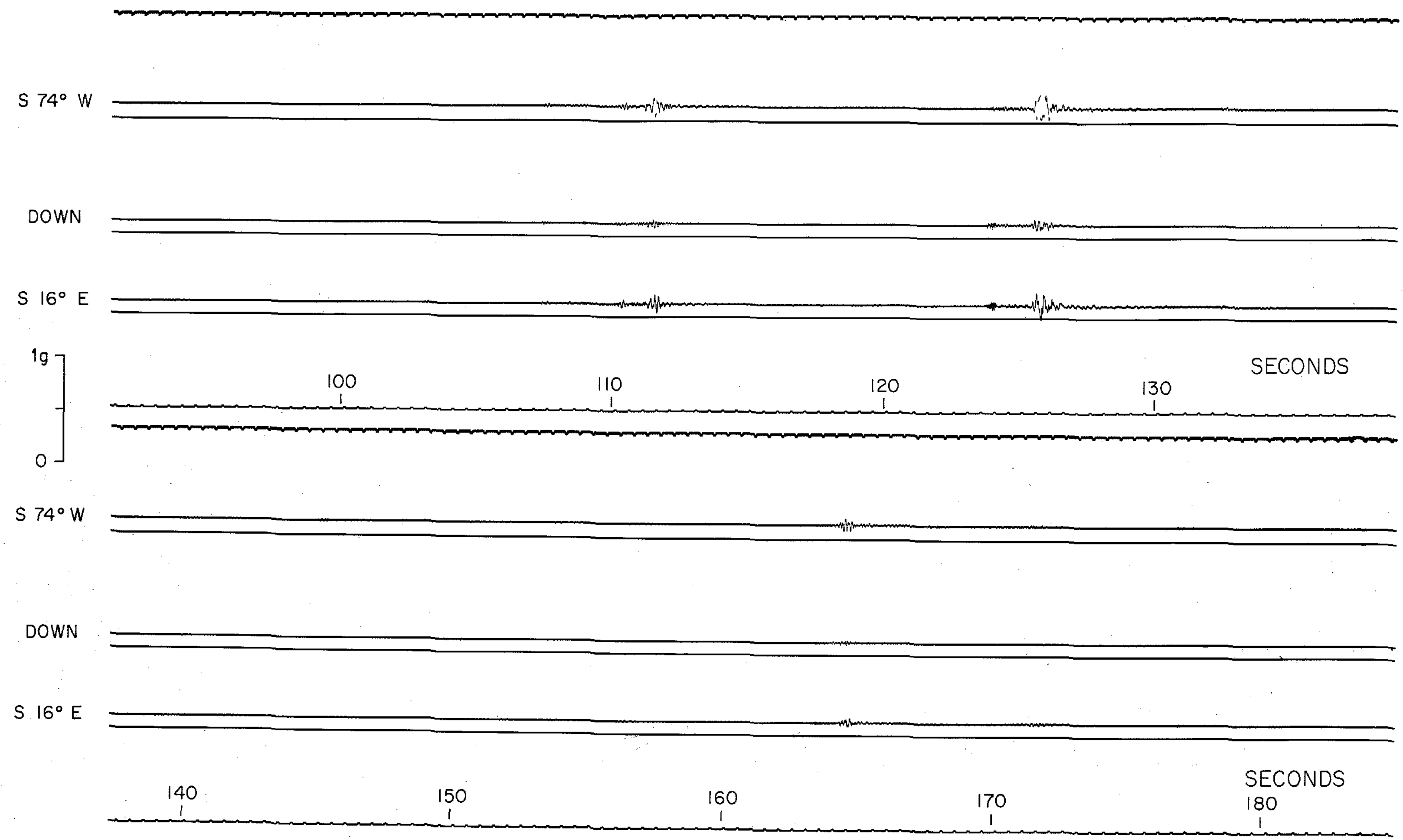

Figure 2.5 (cont'd)

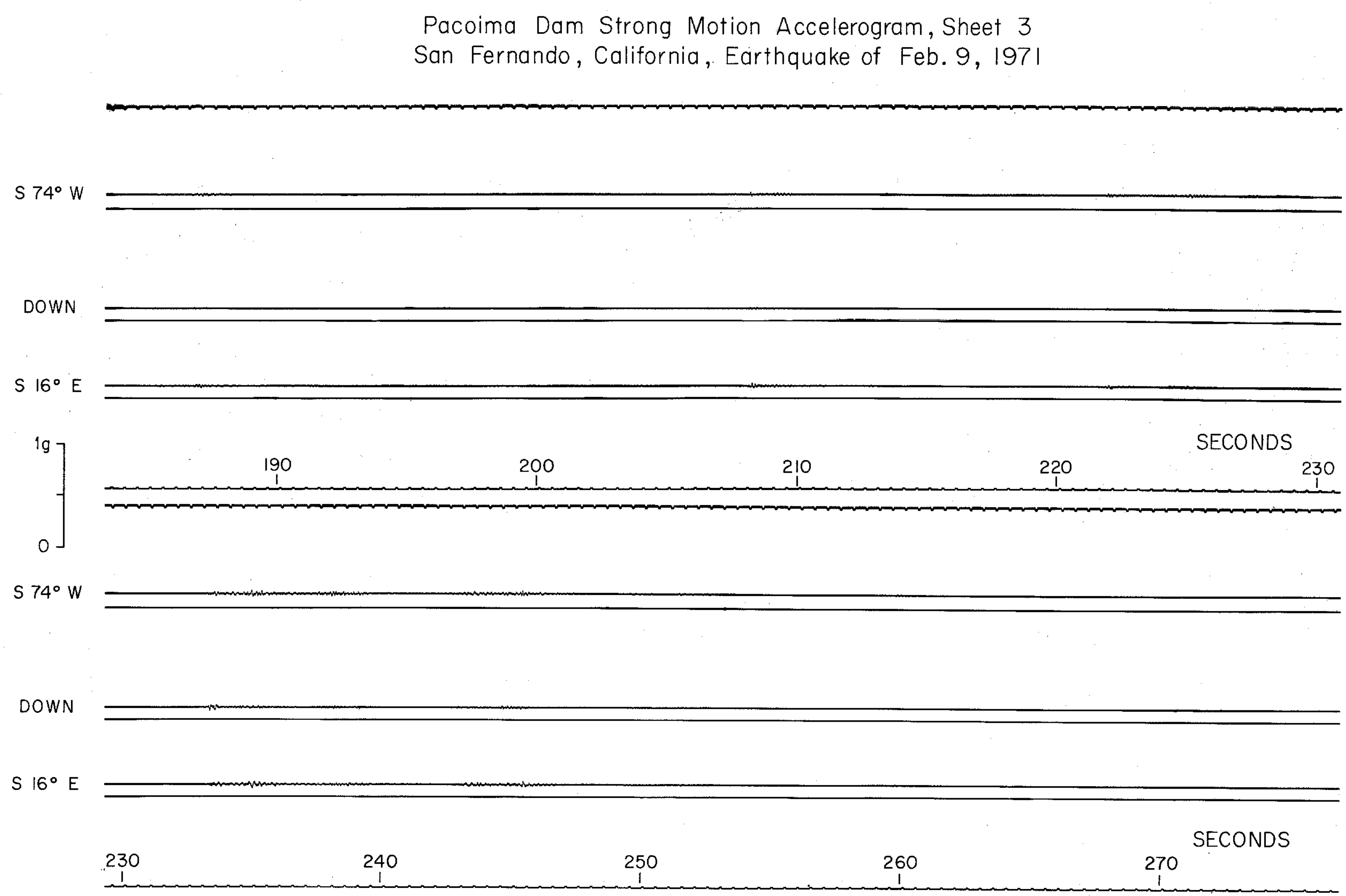

Figure 2. 5 (cont'd)

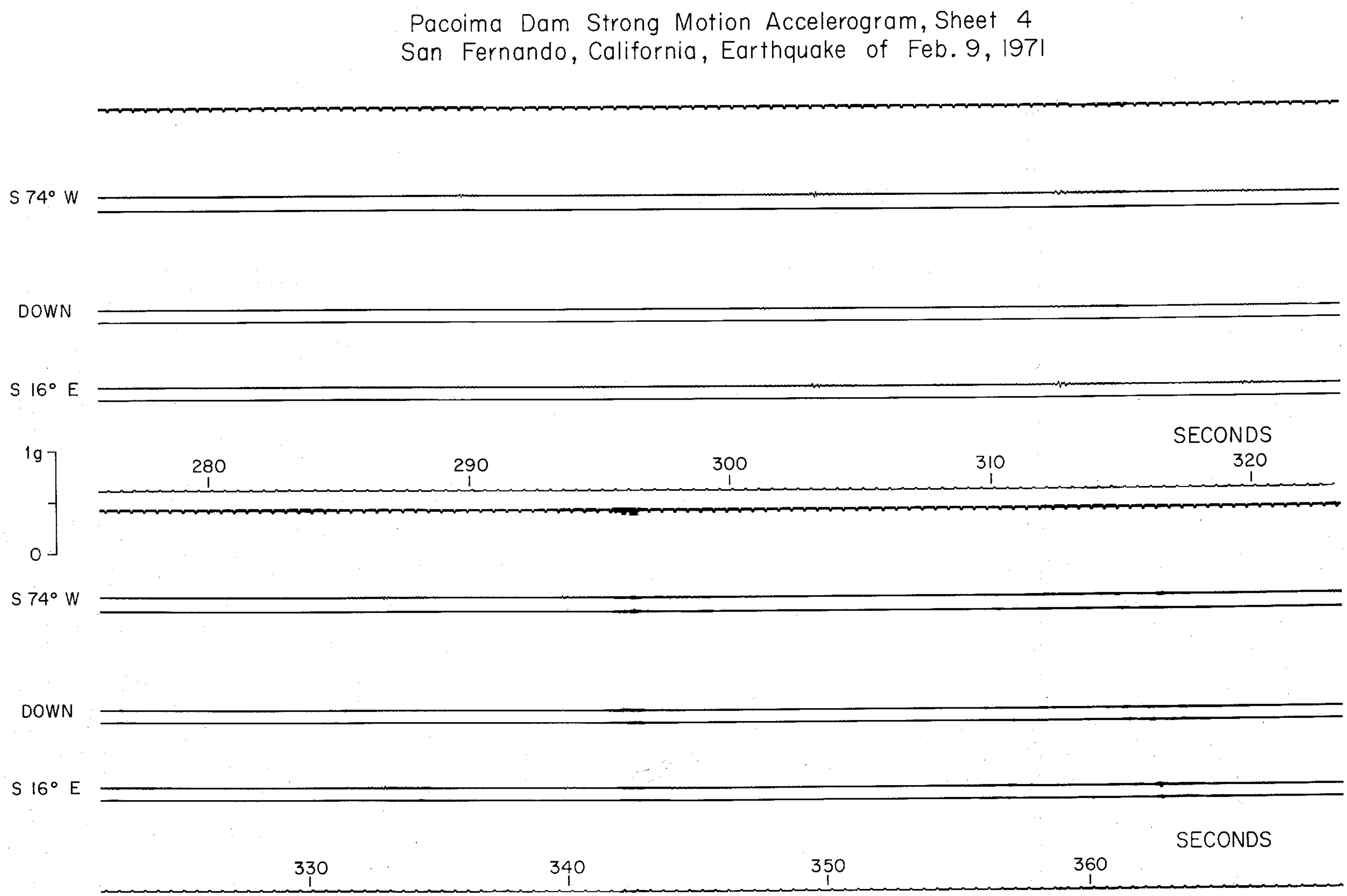

Figure 2. 5 (cont'd)

great assistance. Virgilio Perez of the San Francisco office carried out the exacting job of preliminary checking and scaling of the records, and assisted with record digitization. A key member of the Caltech staff was Richard J. Dielman, who has been assisting the Seismological Field Survey with field installation and servicing, and who rendered great service after the earthquake in the recovery of records, in accelerograph site investigations, and in the handling of the records for processing. We were very fortunate in being able to borrow from the Lamont-Doherty Geological Observatory of Columbia University the services of M. D. Trifunac, whose past experience in accelerograph data processing proved invaluable to us. A. G. Brady, with the assistance of A. Vijayaraghavan, in a remarkably short time trained a very effective team of digitizing operators, and carried out the complicated programming details necessary for computer processing of the accelerograms. At the Jet Propulsion Laboratory we received very quick cooperation from W. H. Pickering, director, and M. E. Alper, who was our main coordinating link with J.P.L. R. B. Ford and R. D. Windmiller of the photographic processing laboratory were of great assistance in the reproduction of the records and M. R. Trubert offered very useful advice in laying the groundwork for the automatic digitizing process. The aid of J. D. Patterson in preparing basic maps is also appreciated.

The program was made possible and its prompt implementation after the earthquake was assured by the special efforts of C. C. Thiel, C. A. Babendreier, and M. P. Gaus of the Engineering Mechanics Department, Engineering Division, National Science Foundation.

B. Analysis of the Pacoima Dam Accelerogram

by

M. D. Trifunac and D. E. Hudson

Abstract

Integrated ground velocities and displacements calculated from the accelerogram recorded at the Pacoima dam site indicate that the strong ground motion was predominantly in the vertical and NS direction, in general agreement with the mechanism of faulting as inferred from aftershock studies, and with fault displacements observed in the field. High frequency peak accelerations of 1.25g were recorded in two horizontal directions, these being the highest ground accelerations so far recorded for earthquakes. Response spectrum curves calculated from the accelerograms do not show unusual features, and the numerical values are consistent with past experience. The high frequency, high amplitude impulsive ground motion associated with the highest peak accelerations did not contribute significantly to the overall response spectrum values. The presence of the high frequency motions in the recorded accelerograms is presumably the consequence of the proximity of the recording site to the fault dislocation.

* * * * * * * * * *

Introduction. The strong earthquake ground motion of the San Fernando, California, earthquake of February 9, 1971 was recorded on over 200 accelerographs of the Southern California strong-motion network. These accelerographs were located at various ground sites, buildings and dams.

By number and quality of the records the instrumental coverage of this earthquake is by far the most extensive and complete in the history of strong-motion seismology.

Although this 6.6 magnitude earthquake is not large from the seismological point of view, it was associated with very severe ground motions and must be ranked as a major event from the standpoint of damage and general engineering implications.

The instrumentally determined focus of the main shock, at a depth of about 13 km, represented the region in which faulting was initiated. The fracture then propagated up and to the south, past and under the Pacoima Dam accelerograph site, and intercepted the surface in the Sylmar-San Fernando area (Figure 2.6).The Veterans Hospital which collapsed killing 44 people, and the Olive View Hospital which was severely damaged with 3 deaths, were located about 3 km to the north of the surface faulting (Figure 2.6).

The local severity of shaking in the Sylmar-San Fernando area appears to have been as strong as would be expected for the largest shocks in California, although longer fault breaks would result in greater durations of ground shaking. The idealized empirical relation between magnitude and fault length (Housner, 1970) would predict the fault length for this earthquake to be about 15 km, which is in excellent agreement with the observed surface faulting (Figure 2.6) and the overall dislocation size outlined by the distribution of aftershocks (Division of Geological and Planetary Sciences, California Institute of Technology, 1971 - Figure 2.7. For an earthquake of magnitude 8 or greater, the surface faulting might extend for several hundreds of kilometers and as a result the strong ground motion would last several times longer than the motion recorded during the San Fernando earthquake.

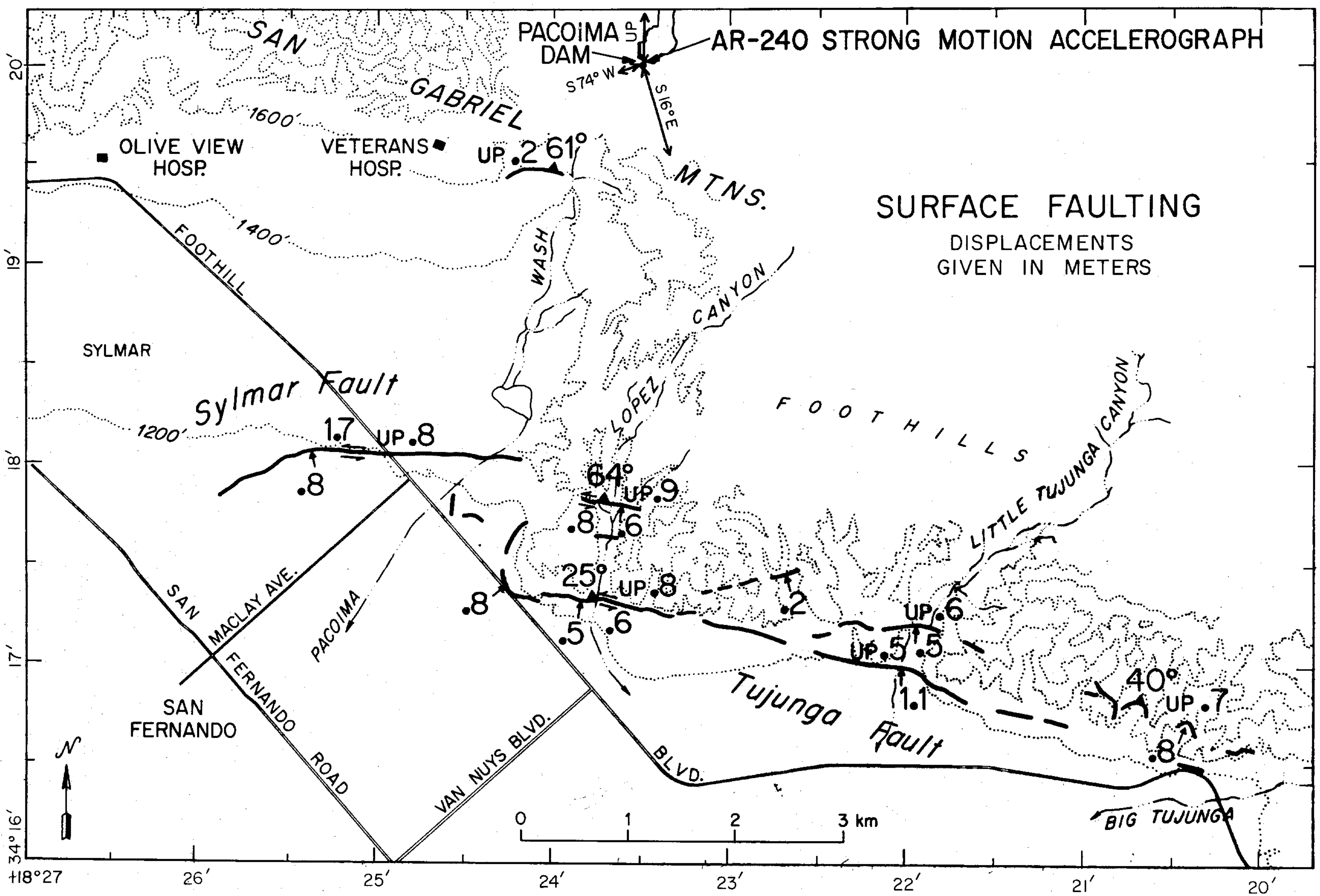

Figure 2.6 Surface faulting in the Sylmar-San Fernando-Tujunga area (reproduced by permission from the Division of Geological and Planetary Sciences, California Institute of Technology, and Kamb et al, 1971), and its relation to the recorded strong ground motion at the Pacoima Dam.

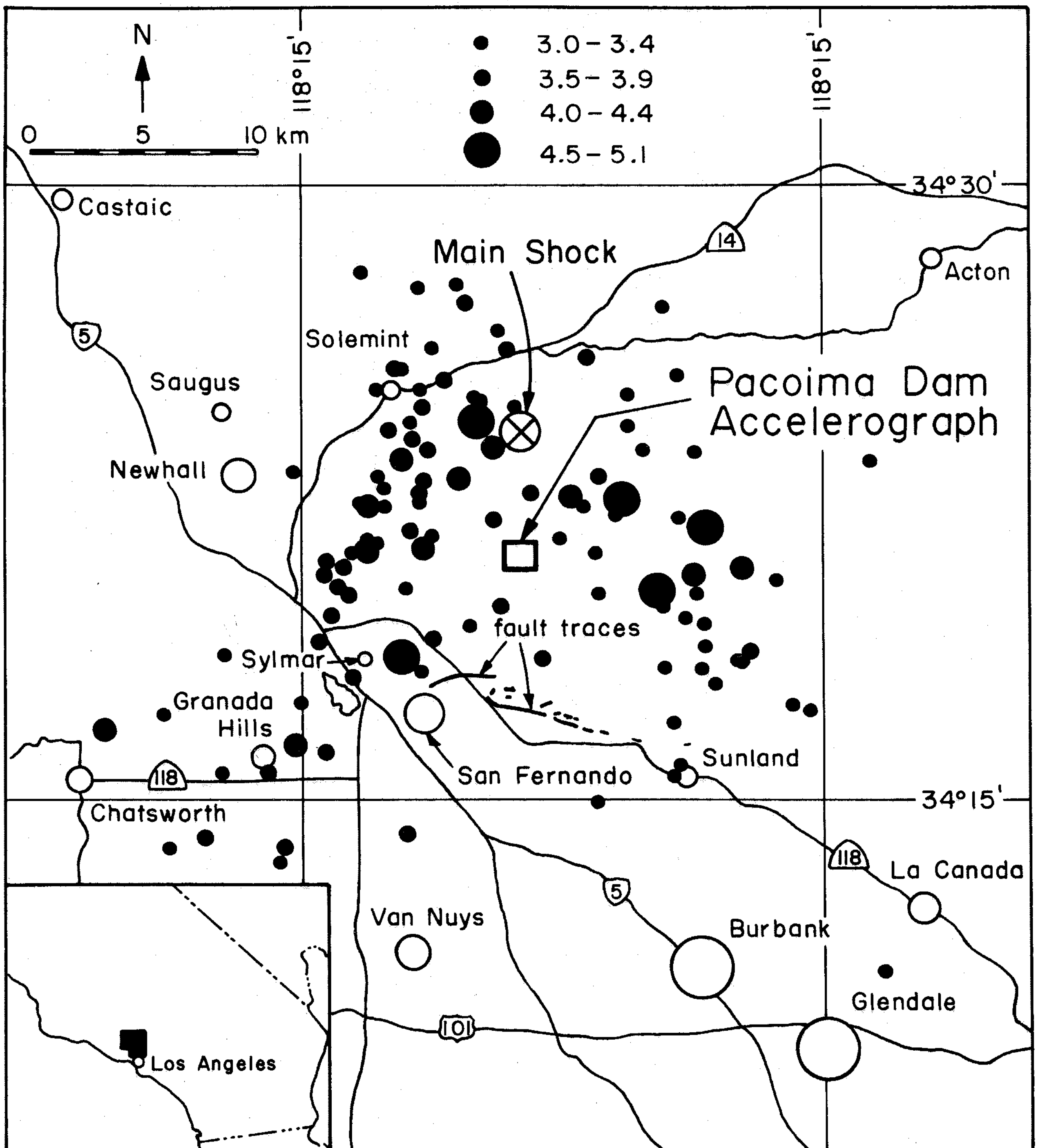

Figure 2.7 Map of the epicenters of the main shock and representative aftershocks of the San Fernando earthquake of magnitude 3.0 and greater, through 23 February 1971 (reproduced by permission from the Division of Geological and Planetary Sciences, California Institute of Technology, 1971).

The Accelerograph Site. Because of the importance of the Pacoima Dam record, which was obtained virtually at the center of the event, and because of the rather special nature of the site, it is believed that a relatively detailed description of the site and of the instrument installation is justified.

Figure 2.8 shows an oblique aerial photograph looking south over the Pacoima Dam site with the San Fernando Valley in the distance. The location of the AR-240 accelerograph at Pacoima Dam is shown, and the locations of the two heavily damaged hospitals are indicated in the background. Figure 2.9 shows a close up view of the dam, with the location of the accelerograph on a rocky spine adjacent to the dam abutment indicated. Figure 2.10 shows a view of the circular instrument house seen from below on the dam, and Figure 2.11 is a view of the dam and the instrument house from above looking north. In both of these figures extensive cracking of the gneissic granite-diorite rock will be noted. Many of the cracks penetrate through the smooth gunite coating into the rock below.

It is not known, however, the extent to which the surface fractures of the gunite coating relect the conditions of the major rock mass below. Relatively small cracks of the dimensions of the gunite fractures would be associated with higher frequencies than those involved in the approximately 10 cps motions observed on the record (Figures 2.16, 2.17 and 2.18). The extent to which presently unknown details of the ridge structure may influence the recorded motions is for the time being a matter for speculation only.

As can be seen in Figure 2.10, about 5 meters to the west of the accelerograph a small rock slide occurred (about 5-10 m^3) during the

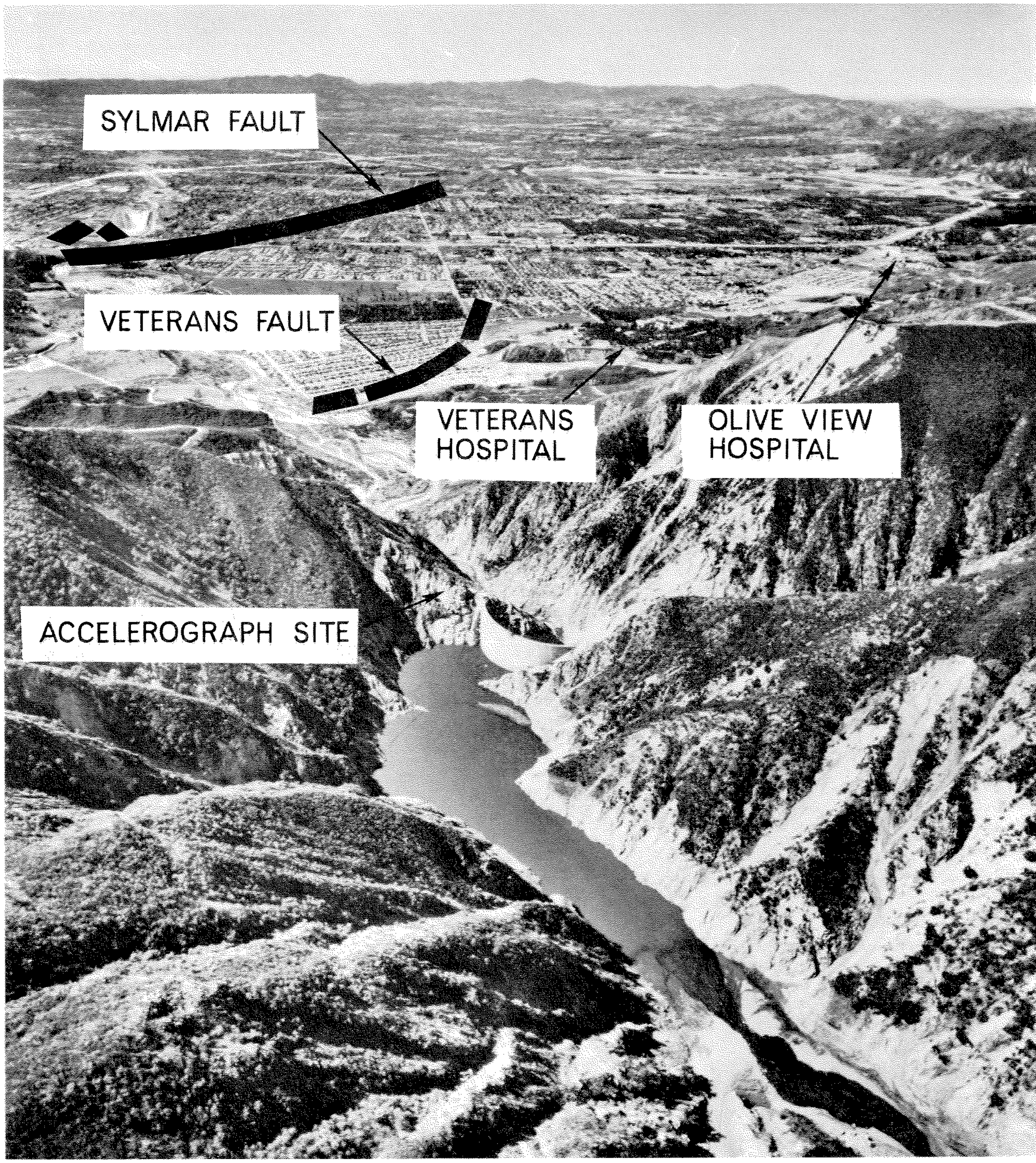

Figure 2.8 Oblique aerial view looking southwest over the Pacoima Dam site.

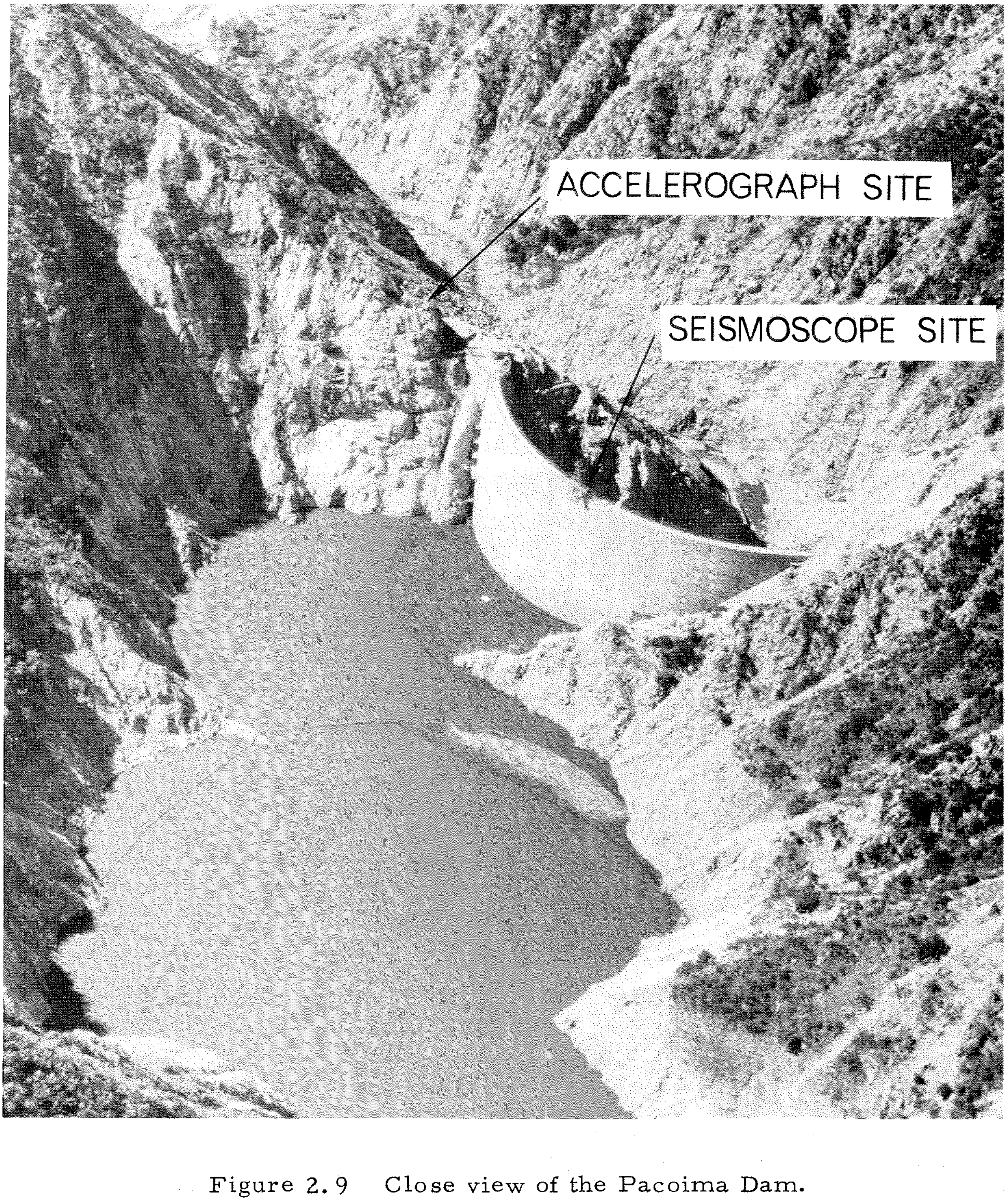

Figure 2.9 Close view of the Pacoima Dam.

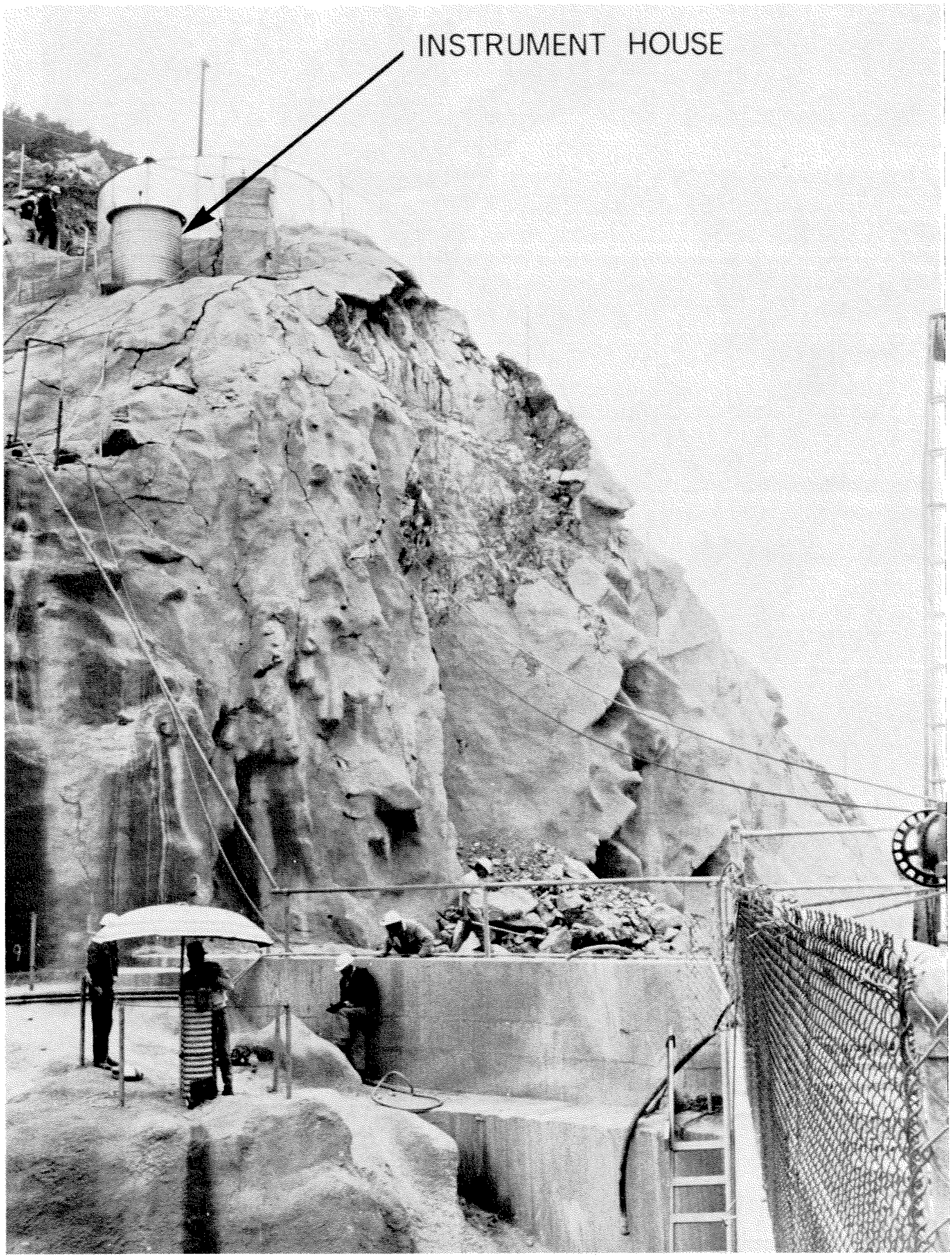

Figure 2.10 Strong-motion AR-240 accelerograph site and the small rock slide.

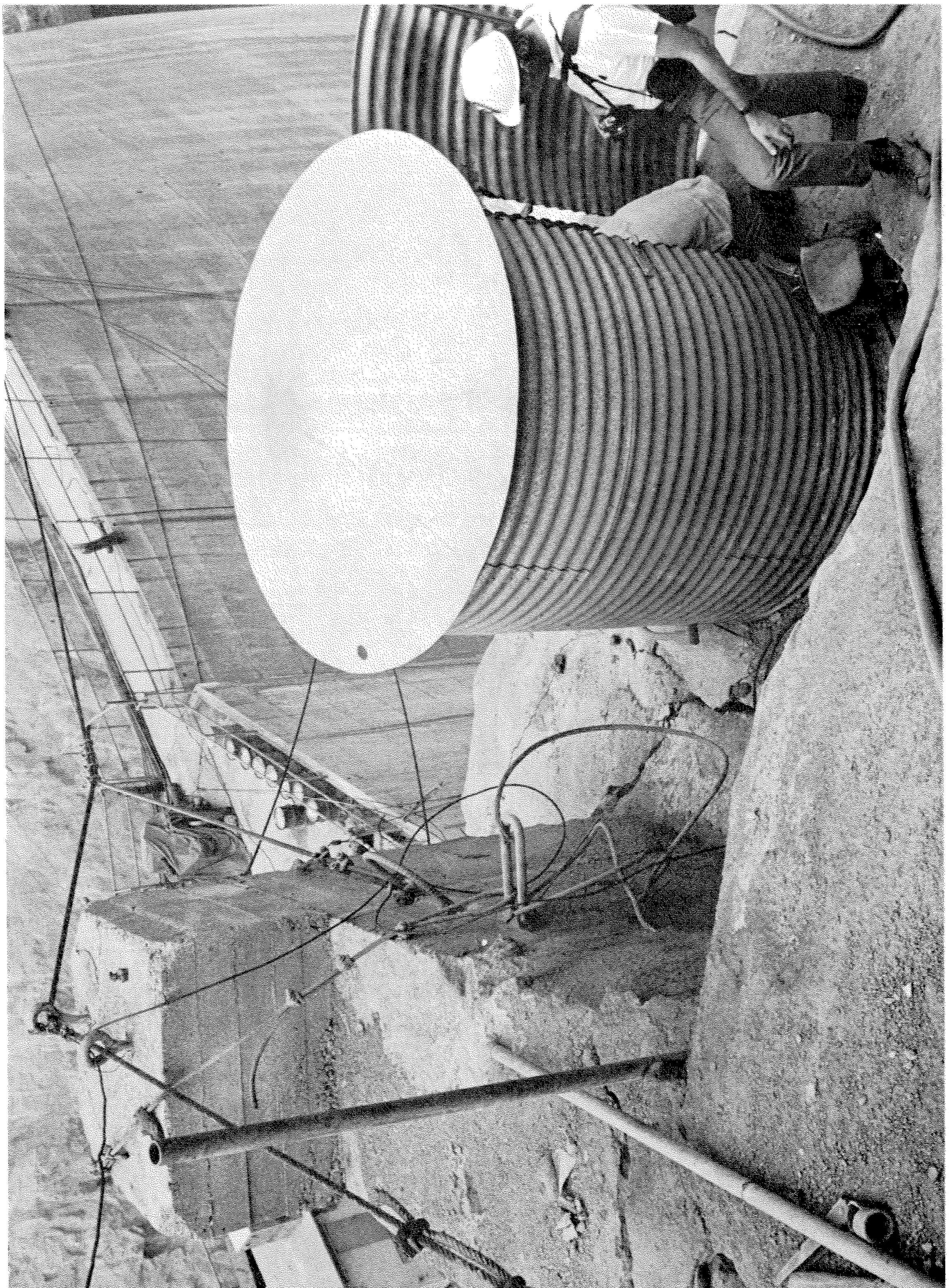

Figure 2.11 View of the dam and the instrument site.

earthquake, which fortunately was not large enough to disturb the accelerograph foundation. As will be seen in Figure 2.12, one of the cracks penetrated into the foundation of the instrument house, although as will be noted the instrument pier itself, which was separated by an inch or so from the foundation of the circular house, was not cracked. As of two months after the earthquake changes in the configuration of these foundation cracks indicate that long term motion of some kind is continuing. After the earthquake, the instrument mounting pier was still solidly attached to the foundation rock, and the mounting bolts attaching the accelerograph to the pier were tight and undisturbed. The only sign of disturbance was a small permanent tilt of the instrument during the earthquake, which could be estimated to be of the order of 0.5° in the NW direction. The amount of this permanent tilt could be estimated with fair accuracy from the adjustments required after the earthquake to re-level the accelerograph (Dielman, personal communication). A view of the accelerograph mounted on its concrete pedestal within the circular house after the earthquake is shown in Figure 2.13.

A plan view of the dam, and the abutment area, with the instrument locations indicated, is shown in Figure 2.14. Also shown is the location of the Wilmot Seismoscope on the crest of the dam. During the first few seconds of earthquake motion, the motion of the crest of the dam was so severe that the seismoscope glass record plate was dislodged from its retaining ring, so that no useable seismoscope record was obtained.

As a further indication of the setting of the site within the aftershock region, Figure 2.7 may be referred to. The site is approximately 8 km south of the instrumentally determined epicenter and is nearly in the center and above the tentative fault dislocation surface striking N 72° W and dropping about 45° towards the north (Kamb, et al, 1971).

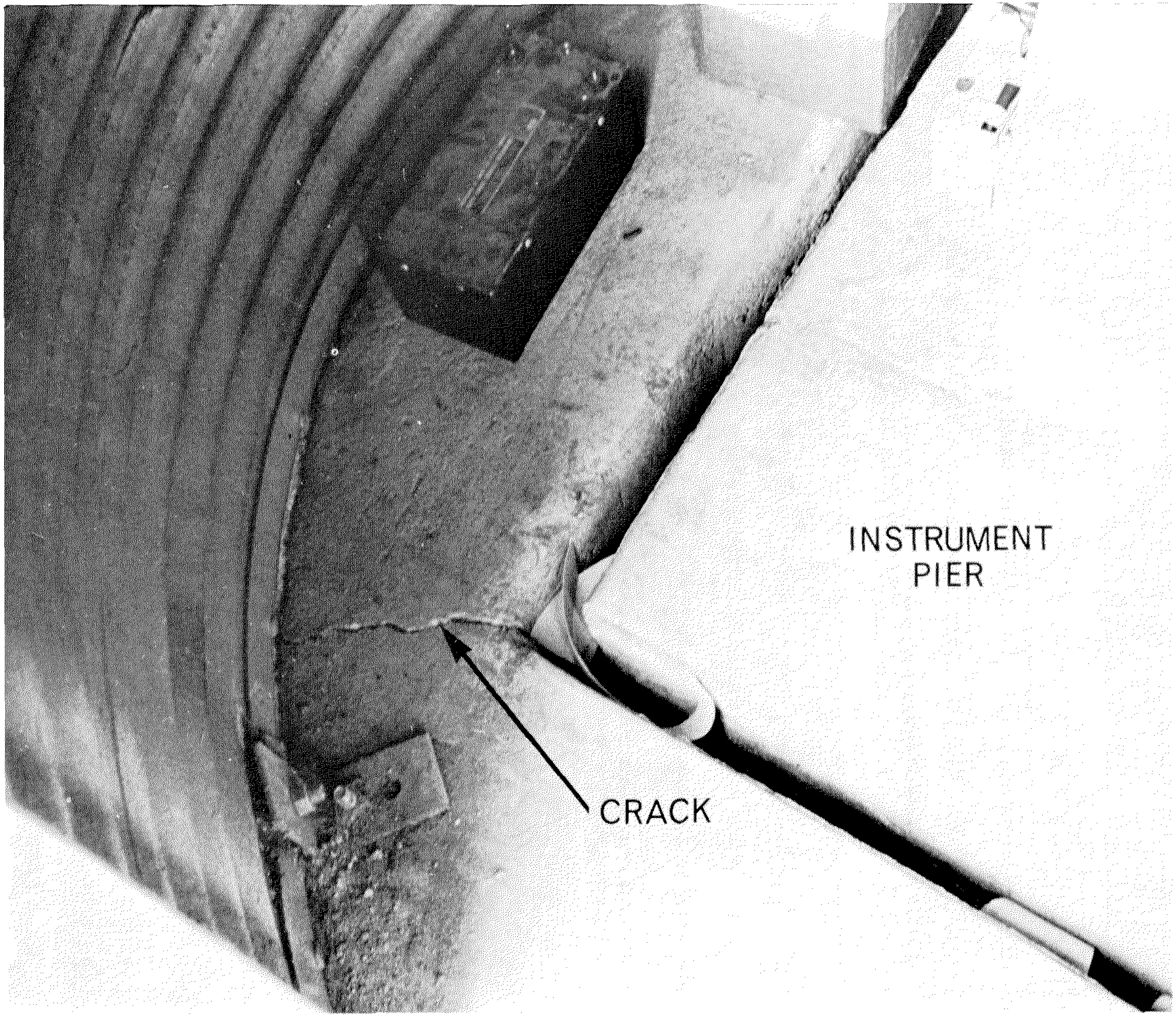

Figure 2.12 Cracks in the foundation of the instrument house.

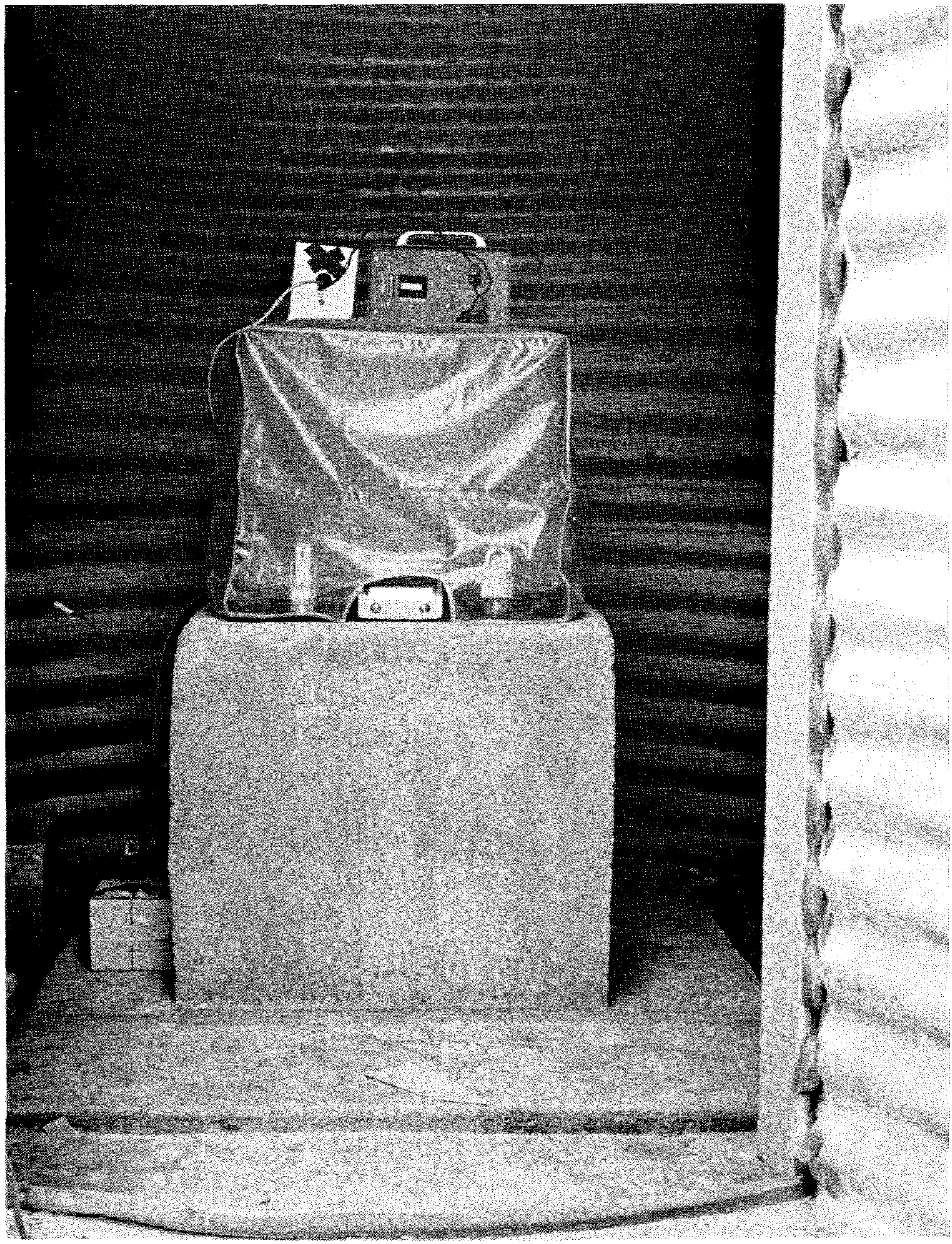

Figure 2. 13 AR-240 accelerograph after the earthquake.

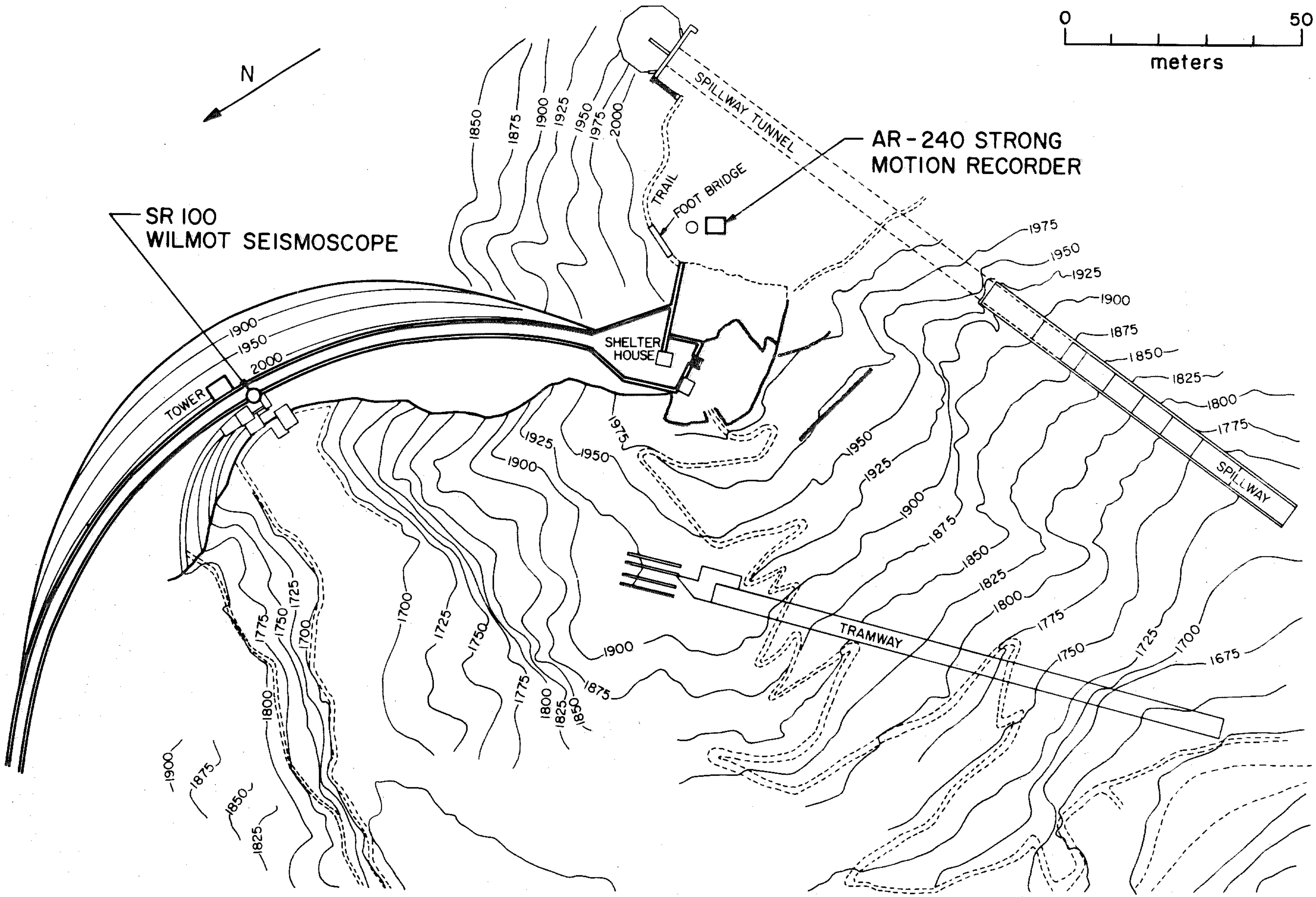

Figure 2.14 Pacoima Dam site with AR-240 strong-motion accelerograph and SR 100 Wilmot Seismoscope.

Accelerograph Performance. The AR-240 strong-motion accelerograph at the Pacoima Dam site records two horizontal and one vertical components of acceleration on 12 inch wide photographic paper. The accelerograph transducers have natural frequencies of about 19 cps and damping of approximately 60% critical (Hudson, 1970). The instrument is one of several owned by the Los Angeles County Flood Control District and is a part of the Southern California strong-motion accelerograph network maintained by the Seismological Field Survey of the NOAA National Ocean Survey.

As mentioned above, after the earthquake the accelerograph foundation remained tilted through a small angle which was estimated to be about 0.5 degrees. This small angle was sufficient to actuate the starting pendulum and the instrument recorded continuously for some six minutes until it ran out of paper.* During this interval at least 30 aftershocks were recorded. In one sense, the small permanent tilt of the foundation can be considered to be a fortunate occurrence, since it permitted this recording of the beginning of the aftershock sequence, and indicated the exact sequence of aftershock events in the epicentral region. These details of the aftershock sequence will be of importance in investigating the mechanism of energy release.

In order to check the instrument performance, tilt, free vibration and damping tests were performed after the earthquake. The tilt test showed that the sensitivity of the accelerograph had not changed significantly. The alignment of the transducer axes relative to the

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*See Figure 2.5

instrument base was also checked during the tilt test (Trifunac and Hudson, 1970). It was found that the two horizontal transducers were well within a 1° alignment. The vertical transducer sensitivity vector was about 5 degrees from the vertical in the longitudinal direction.

Judging from the point of view of the accuracy of the typical strong-motion accelerograph (Trifunac and Hudson, 1970) it may be concluded that the AR-240 accelerograph at the Pacoima Dam site performed essentially to specifications and that the recorded acceleration traces may be adopted as representative of the actual motion of the instrument foundation. The peak acceleration values remained on scale on the photographic paper, and there is no evidence of appreciable nonlinear response at the maximum amplitudes involved.

After the earthquake the instrument base was tilted in approximately the NW direction, relative to its position prior to the earthquake. Since the preliminary calculations of the ground displacement indicated a significant shift in the accelerograph baseline, clearly a consequence of such tilt, tests were conducted to ascertain likely limits for such displacements. The accelerograph was tilted in the NW direction through an angle which just closed the starter pendulum gap, and in this way a lower bound estimate of the acceleration baseline shift could be determined. The results of this test are given in Table 1.

Table 1

Direction of baseline shift for tilt in NW direction	Baseline shift in cm/sec^2
N 74 E	13.3
Down	7.7
N 16 W	2.6

The integration of the digitized accelerograms (Figure 2.15) including the first aftershock (about 42 seconds) indicates that the tilt must have occurred within the first 10 to 15 seconds of the strong motion. This can be concluded from the behavior of the integrated velocity curves. If a straight line fitted to the velocity curves (Trifunac, 1970), over the interval between 12 and 42 seconds, is extrapolated back to the zero time, the resulting displacement curves indicate the following permanent displacements after the earthquake:

Table 2

Permanent displacement in direction	Permanent displacement amplitude in meters
N 74 E	1.0
Up	1.3
S 16 E	1.7

It might be tempting to interpret these results in terms of the observed surface fault displacements (Figure 2.6, Kamb et al., 1971). However, if it is assumed that the tilting indeed took place during the first 10 seconds, the lower bound for the "permanent displacements" obtained only from the tilt would be (based on data of Table 1):

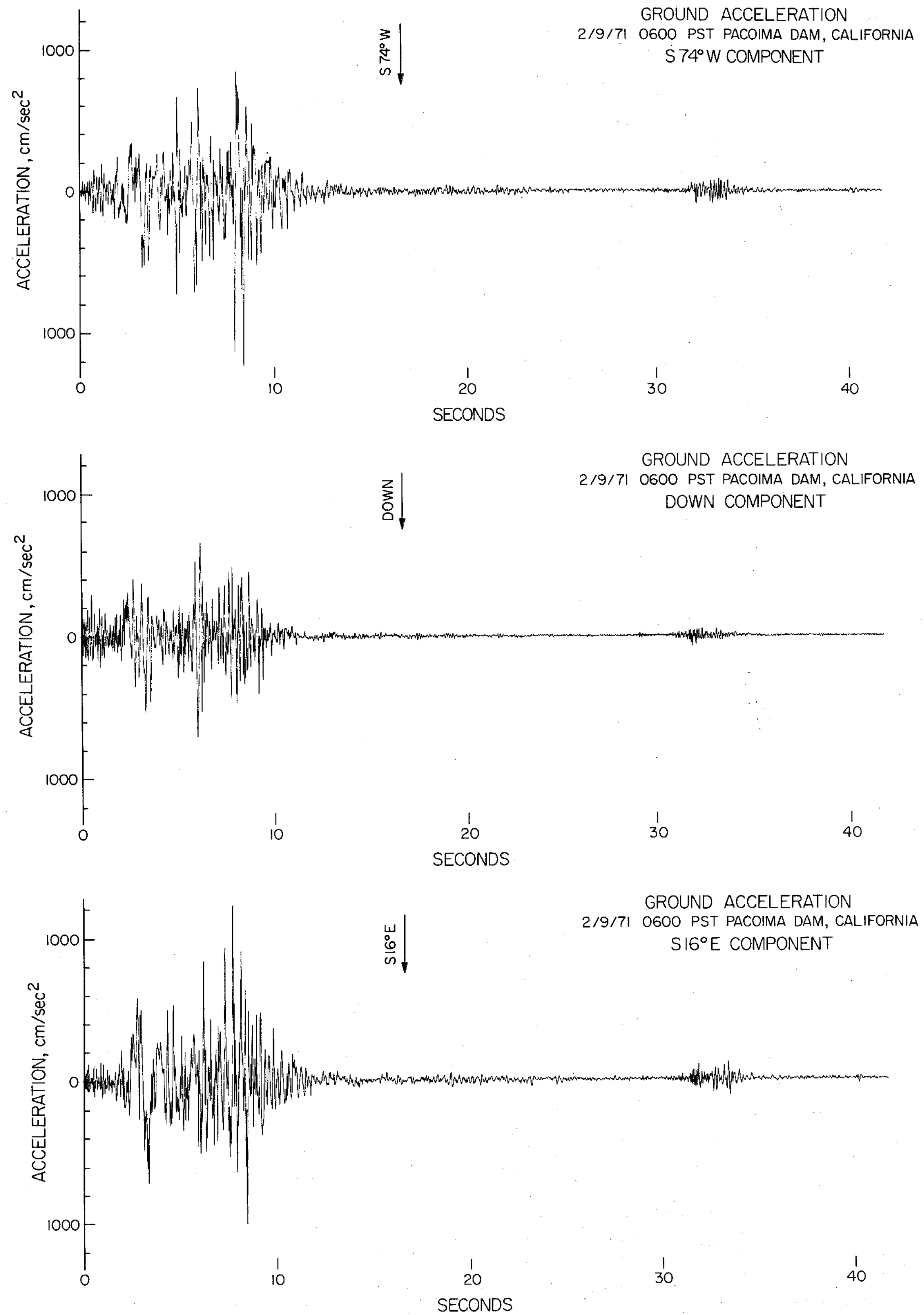

Figure 2.15 Plot of digitized accelerograms recorded at the Pacoima Dam.

Table 3

Permanent displacement in direction	Lower bounds on permanent displacement amplitudes caused by tilt in meters
S 74 W	4
Up	3
S 16 E	1

For this calculation it was assumed that the acceleration zero baseline is determined by its fixed position after the tilt is completed, 10 seconds after the instrument has triggered, and that the tilt occurred uniformly over the 10 second interval. Comparing the amplitudes given in Table 2 and Table 3 it may be concluded that the tilt was large enough to prevent any estimation of the permanent displacements associated with the earthquake.

Data Processing and Ground Motion Calculations. Figure 2.15 is a plot of the first 42 seconds of the digitized accelerograms including the first aftershock. The strong motion representing the main energy release lasted for about 7 seconds and the first aftershock was recorded about 29 seconds after the instrument was triggered. The first 15 seconds of the acceleration were chosen for the analysis of ground motion.

The AR-240 accelerogram was digitized at Caltech on a Benson Lehner 099D data reducer and processed by the standard methods developed in recent years for strong-motion accelerogram analysis (Hudson, et al, 1969). The quality of the original record was excellent.* The trace was clear and continuous with the exception of one 1.25 g peak on the S 16° E

*See Figure 8.6.

component at 7.6 seconds. At this point the trace was lost above the 1 g level and had to be extrapolated. Because of the excellent photographic quality of the trace, this extrapolation could be carried out with confidence. The error in the peak is believed to be less than 0.1 g, which would not appreciably influence any calculations based on the accelerogram. This digitization of the Pacoima Dam accelerogram is believed to be as accurate as may be achieved by presently available techniques and equipment.

The baseline correction was performed by high-pass filtering the uncorrected data above the frequency 0.07 cps. This means that all periods longer than approximately 16 seconds have been removed from the record, and hence that no information on permanent displacements is to be expected from the analysis. This filtering procedure, and the least square fitting of a straight line to the ground velocity, which gives an estimate of the initial velocity, constitute a new method recently proposed for standard baseline correction of the strong-motion accelerograms (Trifunac, 1970). The resulting acceleration, velocity and displacement curves are shown for the three recorded components in Figures 2.16, 2.17 and 2.18. As may be seen in these figures, the maximum acceleration is 1.25 g for both horizontal components and 0.70 g for the vertical component, and the peak velocity is 115 cm/sec.

As already mentioned the tilting of the instrument base must have taken place during the first 15 seconds of the strong motion. Thus the displacements in Figures 2.16, 2.17 and 2.18 may contain an unknown contribution from the tilting of the instrument in addition to the actual ground motion. Nevertheless, the computed ground motion indicates that the biggest displacements were vertical and in the North-South direction, in general agreement with observed surface faulting (Figure 2.6).

In terms of acceleration amplitude the ground motions recorded at the Pacoima Dam site are the largest so far measured during any earthquake.

SAN FERNANDO EARTHQUAKE
2/9/71 06:00 PST PACOIMA DAM, CALIFORNIA
S 74° W COMPONENT

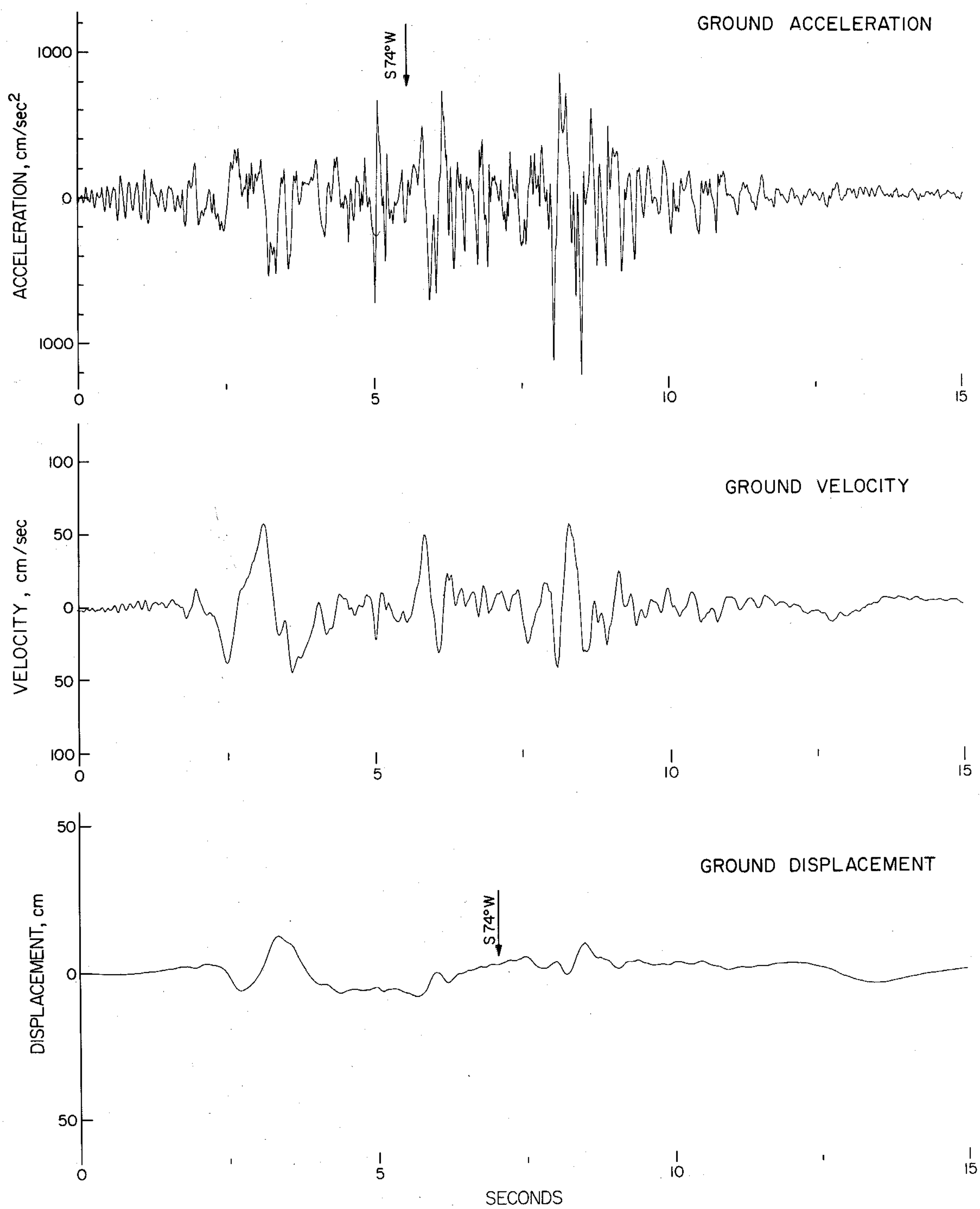

Figure 2.16. S 74° W motion, Pacoima Dam.

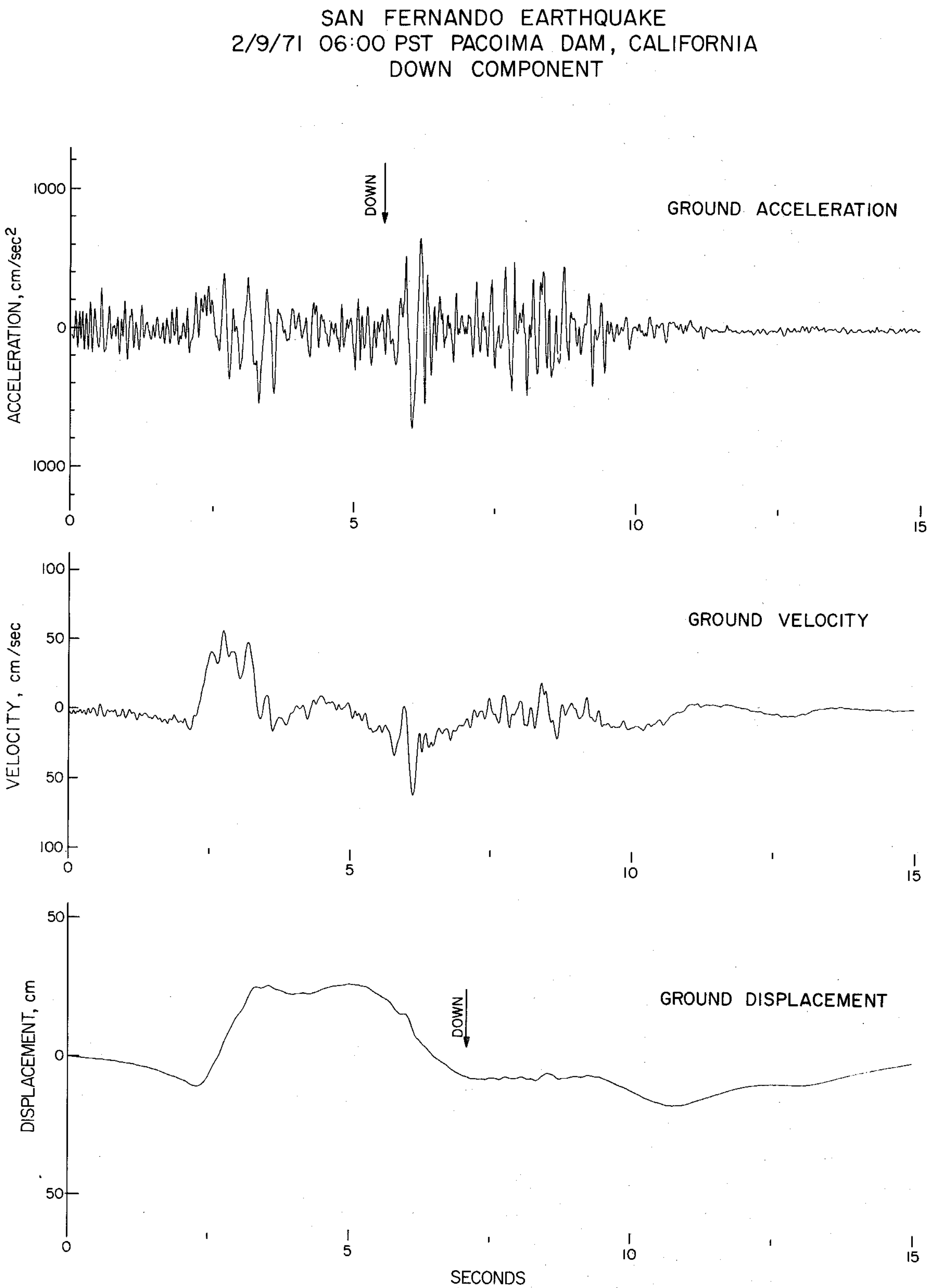

Figure 2.17. Down motion, Pacoima Dam.

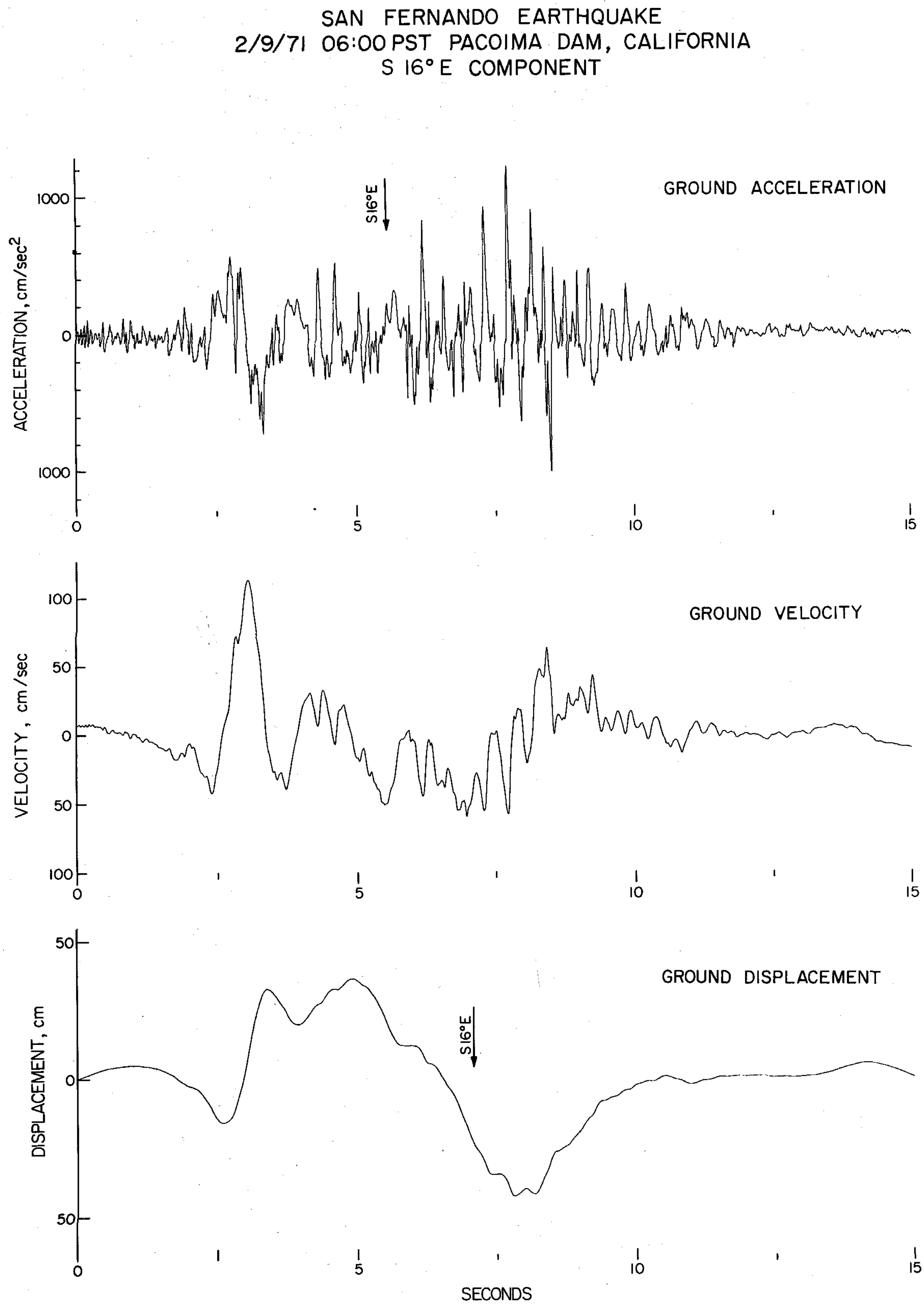

Figure 2. 18. S 16° E motion, Pacoima Dam.

The relatively short duration of the severe shaking is a consequence of the short fault rupture.

Response Spectra. The computed relative velocity and $S_d \frac{2\pi}{T}$ response spectra (S_d = displacement spectrum; T = period) are shown in Figures 2.19 and 2.20. For each acceleration component the response spectrum curves were calculated for 0, 2, 5, 10 and 20 percent of critical damping. As expected, the relative velocity and $S_d \frac{2\pi}{T}$ spectra are very similar for short periods while the $S_d \frac{2\pi}{T}$ spectrum falls off more rapidly for longer periods. It may be recalled that the zero damped relative velocity response spectrum is an approximate representative of the Fourier amplitude spectrum of the accelerogram.

The spectrum curves for the horizontal S 16 E and S 74 W components show peaks at about 0.4 and 1.4 second periods, while the spectra for the vertical component indicates predominant periods near 0.3 and 2 seconds. The short duration of the strong motion is reflected in the nature of the response spectra of Figure 2.19, which shows a relatively flat character for periods longer than 5 or 6 seconds. In the period range 0.5 to about 3 seconds the spectral amplitudes are similar to those calculated for the El Centro 1940 accelerogram (Alford, et al, 1951). The high frequency spectral amplitudes in the Pacoima Dam record are not incompatible with past experience. Similar high frequency characteristics can be noted on records from the Parkfield, California, earthquake if June 27, 1966 (Housner and Trifunac, 1967) and for the Koyna, India, earthquake of December 10, 1967 (Gupta, et al, 1971). Direct comparisons with these other earthquakes are difficult because of significant differences in the location of the accelerographs with respect to the pattern of faulting,

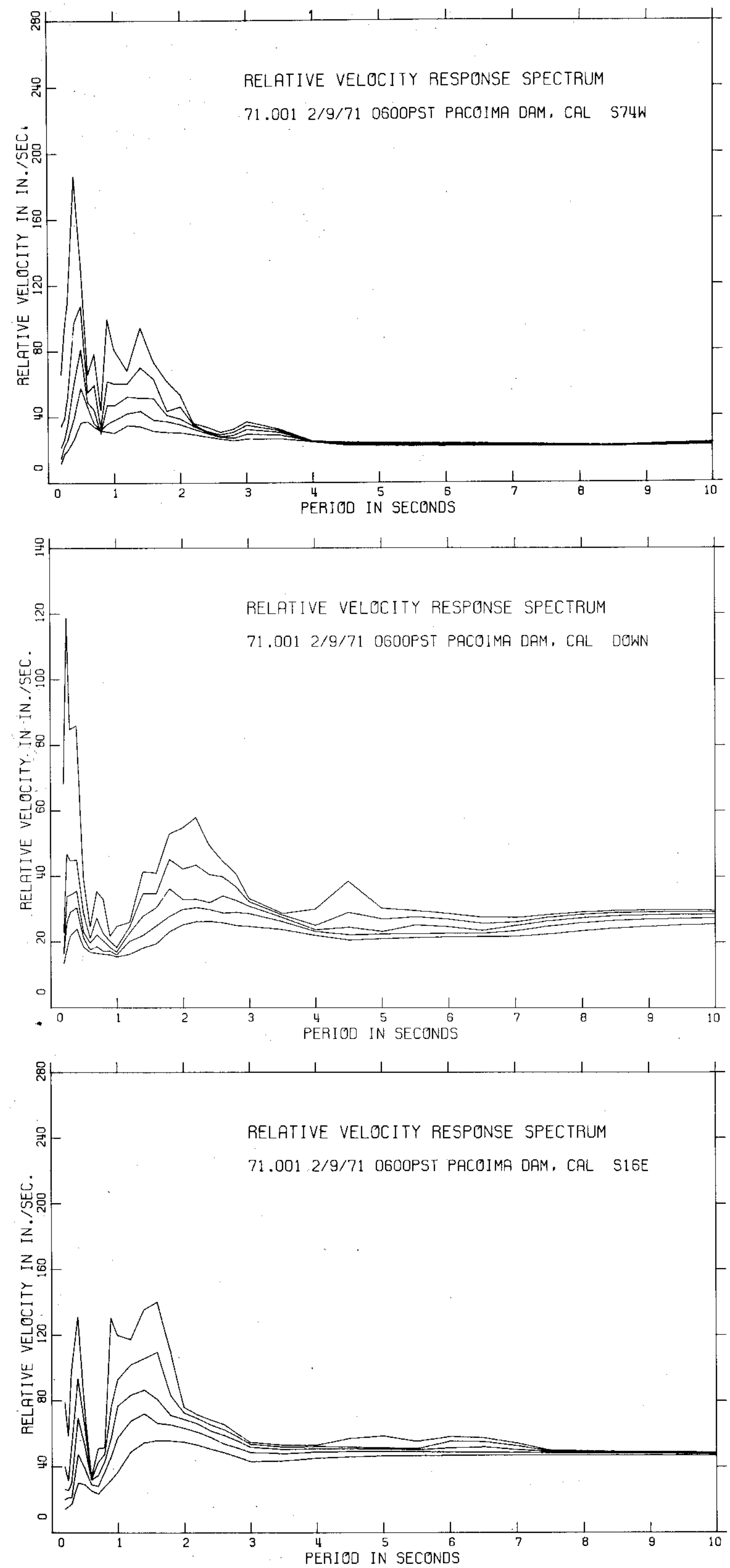

Figure 2.19 Relative velocity response spectra, Pacoima Dam. The curves are for 0, 2, 5, 10, and 20 percent damping.

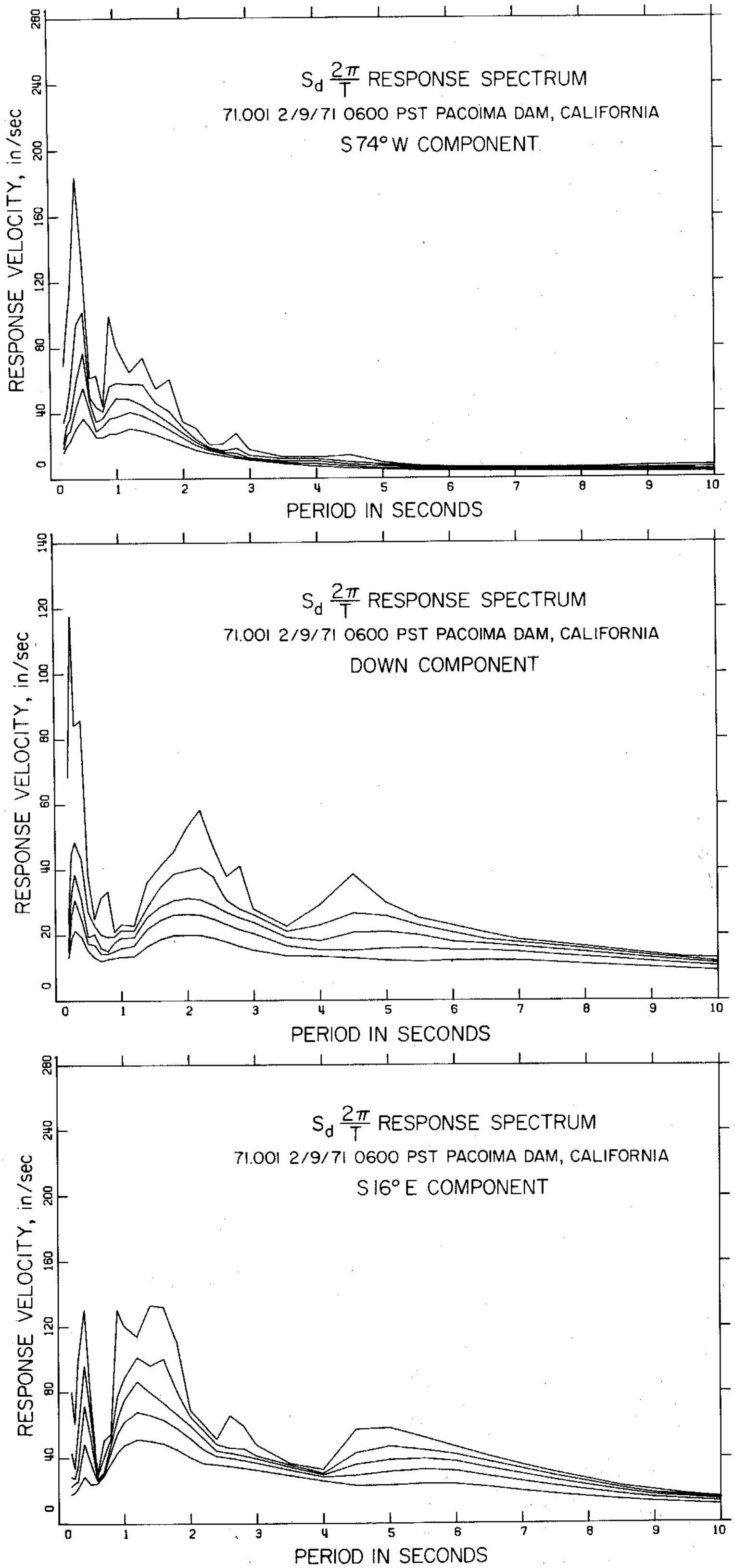

Figure 2.20 $S_d\frac{2\pi}{T}$ response spectra, Pacoima Dam. The curves are for 0, 2, 5, 10, and 20 percent damping.

and in the sizes of the events. Table 4 gives an approximate idea of such comparisons.

Table 4

Earthquake	Magnitude	Distance* km	Peak Accel. g	Peak Velocity in/sec	Approx S_V for T>3 sec in/sec
San Fernando, 1971	6.6[1]	5	1.25	45	50
El Centro, 1940	6.4[2]	10[2]	0.33	17	30
Parkfield, 1966	5.5[3]	0.2[5]	0.50	28	30
Koyna, India, 1967	6-6.3[4]	5	0.63[6]	9	15

*Estimated distance from accelerograph to portion of fault surface associated with maximum energy release.

1. Division of Geological and Planetary Sciences, California Institute of Technology (1971).
2. M. D. Trifunac and J. N. Brune (1970).
3. G. W. Housner and M. D. Trifunac (1967).
4. H. G. Gupta, B. K. Rastogi and H. Narain (1971).
5. K. Aki (1968).
6. J. Krishna, et al., 1969

The strong earthquake ground motion recorded during the Parkfield, California, 1966 earthquake (Housner and Trifunac, 1967) may be considered as a typical example of a short impulsive type ground motion. On the other hand the motion recorded at El Centro during the Imperial Valley, California, 1940 earthquake (Trifunac and Brune, 1970) is an example of the relatively long shaking produced by multiple events successively occurring along a fault about 40 miles long. The ground acceleration,. velocity and displacement curves plotted in Figures 2.16, 2.17 and 2.18 show that from the engineering point of view, the duration of the energy release during the San Fernando earthquake is somewhere between that of Parkfield and El Centro.

The Engineering Significance of the Pacoima Results. One of the important facts about strong earthquake ground motion is that large ground acceleration amplitudes in themselves do not necessarily indicate severe damage to structures. It is also clear that high spectral accelerations do not always tell the whole story. The response spectrum curves alone cannot give a complete picture of the effects of the time duration of the acceleration history. These facts have been clearly demonstrated by the spectra calculated for the Parkfield earthquake (Housner and Trifunac, 1967) and the El Centro earthquake (Alford, et al., 1951). Thus the high spectral amplitudes in Figures 2.19 and 2.20 do not necessarily mean that this motion was very destructive for structures of all types. Pacoima Dam, for example, apparently suffered no significant damage.

The San Fernando earthquake with strong motion lasting about 7 seconds now becomes an excellent example of a strong ground acceleration of short to moderately long duration. If the shaking had continued for another few seconds much greater damage would have resulted, and many buildings and bridges so far only partially damaged would have collapsed. It is mainly this effect of the duration of shaking on structural damage, that calls for detailed investigations of the pattern of earthquake energy release in time.

Acknowledgments

In addition to the acknowledgments of Part A, we are indebted to the Los Angeles County Flood Control District for their forward-looking program of instrumentation and in particular to Mr. E. J. Zielbauer of that organization for cooperation with instrument siting, site visits after the earthquake, and for providing maps and information on the site and the dam.

We wish also to thank Professors C. Allen and B. Kamb of the California Institute of Technology for permission to reproduce their data in Figures 2.6 and 2.7.

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EARTHQUAKE DAMAGE TO BUILDINGS

by

G. A. Frazier, J. H. Wood and G. W. Housner

There is more to be learned about the design and construction of earthquake-resistant buildings as a result of the recent San Fernando Valley earthquake than from any previous earthquake. The many strong-motion accelerographs near the earthquake and throughout the Los Angeles Basin provide essential information on the frequency, amplitude and duration of ground shaking at the various building sites, and those in buildings show how the structures vibrated. Also, a wide variety of building types are located in the metropolitan area and these experienced ground shaking ranging from weak to very strong. Some modern buildings survived strong shaking without damage, whereas others were severely damaged and some even collapsed; hence, much can be learned from those full-scale experiments. Several multistoried, reinforced concrete buildings designed by modern techniques were damaged severely, notably the new Olive View Hospital buildings, the Indian Hills Medical Center, and the Holy Cross Hospital all in northern San Fernando Valley. These structures were located in the region of intense ground shaking, and were subjected to accelerations well above those for which they were designed. In the southern San Fernando Valley, a number of ten- to twenty-story concrete frame buildings experienced ground shaking about as intense as the maximum they could resist without severe damage. The structural integrity of these buildings was not significantly impaired; however, in some instances extensive cracking was found in concrete members. Fortunately, there were very few deaths caused by structural failures in buildings designed to withstand earthquake loadings according to the building codes. Nearly all of the deaths attributed to structural failures occurred in old buildings constructed before earthquake design provisions were made mandatory.

Throughout the San Fernando Valley, structures and structural components made of wood or steel generally received less severe damage, when overstressed, than those constructed of concrete, masonry block, or brick. However, there were many examples of well-designed concrete and masonry construction in the region of strongest ground shaking that survived the earthquake without significant damage. Modern wood and plaster one-story, residential houses performed well from the standpoint of safety for the occupants and resistance to damage from ground shaking.

Much architectural and mechanical damage occurred which was not related to the structural frame of the building and most of the damage could have been avoided with proper planning. Elevator cables jammed, air conditioning units shifted from their mountings, water heater tanks overturned, and plaster partitions cracked. Items more hazardous to building occupants were collapsing bookshelves, toppling furniture and falling glass from broken light fixtures and broken windows. The cost of these often unnecessary losses was high in many instances. For example, Richard Purcell, building coordinator for San Fernando Valley State College, reports that damage of this type cost $245,000 at the college, located about 10 miles southwest of Pacoima dam. Only very minor cracking of structural members was found in the various buildings on this campus, which included an eight-story dormitory and an eight-story office tower.

A description of some of the buildings in the area of strong ground shaking and a brief explanation of the resulting damage follows.

Olive View Hospital

A wealth of information about the behavior of structures during strong ground shaking is provided by the building complex at Olive View Hospital pictured in Fig. 3.1 and located on the map in Fig. 1.2. The county hospital complex, at the site of the Olive View Hospital, is located three miles west of Pacoima dam at the north edge of an alluvial fan at the base of the San Gabriel mountains. The soil at the site is firmly compacted and the footings of the new hospital building were designed for a bearing pressure of 6,000 psf. The zone of surface faulting is to the south of the hospital; therefore, it may be inferred that the hospital is located directly above the zone of subsurface faulting. The nearest instrument for measuring ground motions was stationed three miles away at Pacoima dam which was essentially at the center of the earthquake. This instrument, which was founded on a steep rock ridge directly over the subsurface zone of extended faulting, recorded horizontal accelerations over 0.7g, with a few isolated peaks of horizontal motion exceeding 1.0g. The strong phase of shaking lasted approximately 8 secs. Another instrument seven miles south at the Holiday Inn (Fig. 1.2) recorded a peak acceleration of 0.28g. It is apparent from the level of damage at the Olive View site that peak accelerations exceeded the measured value of 0.28g recorded further south; it appears likely that peak accelerations on the order of twice this value may have occurred at the Olive View Hospital. Unfortunately, just how intense the ground shaking was at various points in northern Sylmar cannot be accurately assessed.

The type of construction of the damaged buildings on the hospital grounds ranged from old wood and masonry buildings to modern

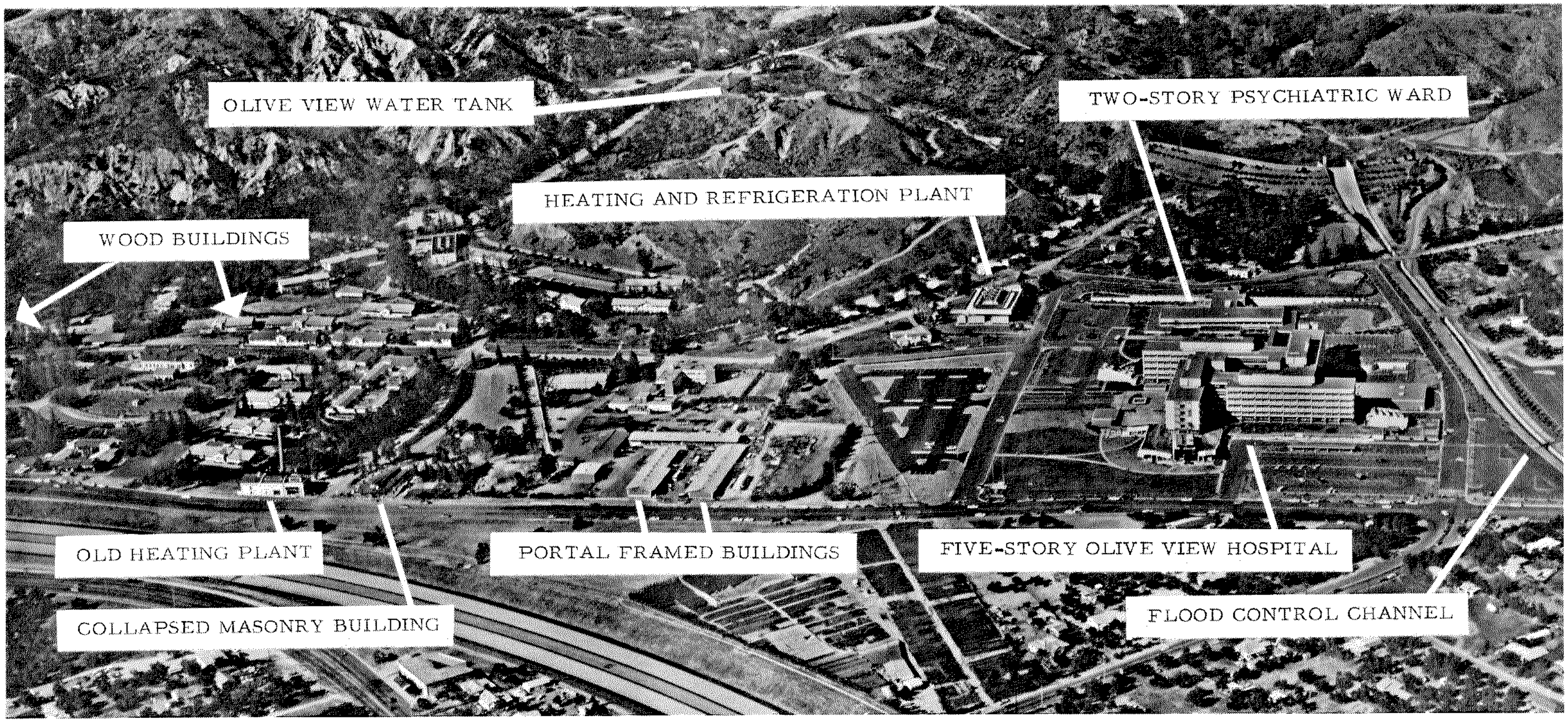


Figure 3.1 Olive View Hospital site, looking north.

reinforced concrete structures. Two masonry buildings, constructed prior to the enactment of earthquake design requirements in 1933, collapsed; an old concrete heating plant building, which did not collapse, was so damaged as to be a total loss; and yet a two-story unreinforced brick building of 1925-30 vintage sustained only moderate damage to one wall and it continued in use, (Figs. 3.2 - 3.5). In contrast to the generally extensive damage to the older masonry construction, a modern laboratory building and two brick single-story buildings with rigid portal frames of reinforced concrete appear to have suffered no structural damage (Figs. 3.6 and 3.7).

About 30 fairly large one- and two-story old wood structures are included in the complex of buildings at Olive View Hospital. Nearly all of these wooden buildings were structurally damaged by the earthquake; some were completely destroyed (Fig. 3.8). Extensive ground settlement occurred in some of the filled portions of the terraced landscape, however this did not appear to be a significant factor in the building damage. Most of the wood buildings shifted off the supporting members that elevate the floor about two feet above the foundation at ground surface (Fig. 3.9). A similar weakness was observed in many of the damaged residential houses along Almetz and Aldergrove Streets to the east of the hospital grounds.

The most significant feature at the hospital site,and in some ways perhaps the most significant building feature of the San Fernando earthquake, was the extensive damage inflicted on a $27.5 million complex of modern reinforced concrete structures, designed in 1964 and completed in 1970. Figure 3.10 shows two buildings that were severely damaged by the earthquake. The smaller structure is the two-story Psychiatric Day-Care Center whose

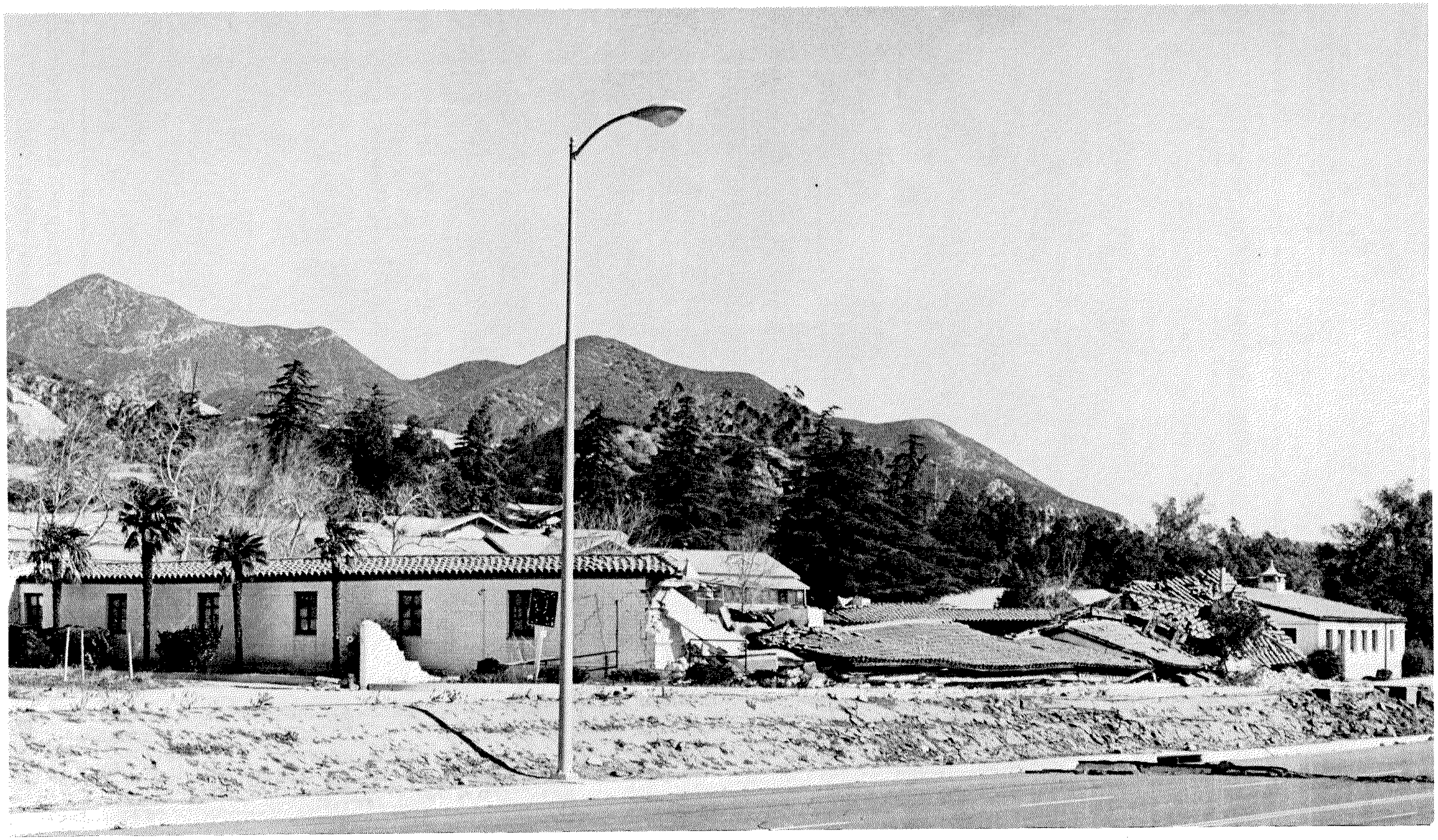


Figure 3.2 Single-story unreinforced brick building at the Olive View Hospital constructed prior to 1933, the year lateral load requirements were first placed in the building code. Ralph Samuels photo.

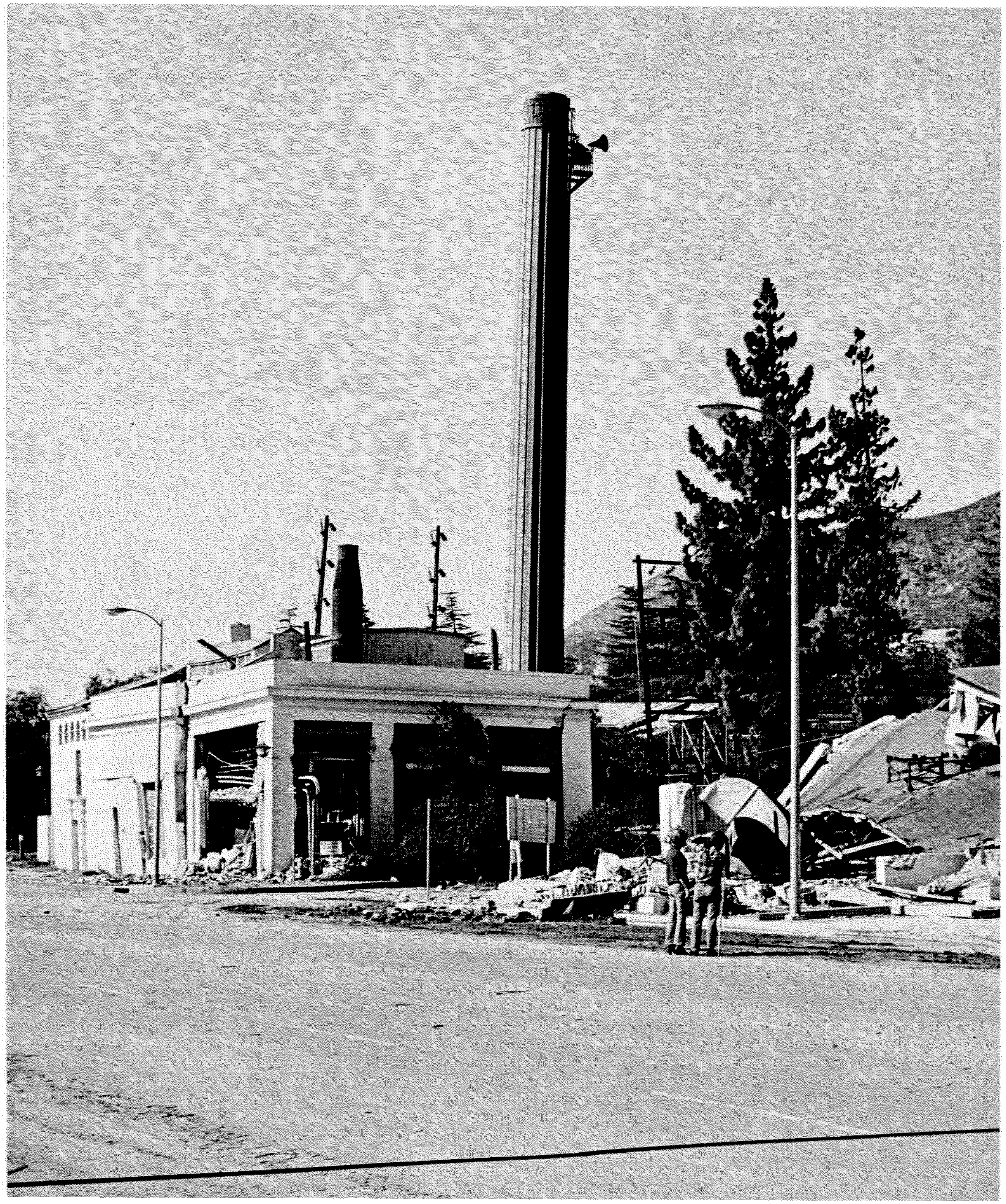


Figure 3.3 The old heating plant was severely damaged and the masonry building to the right collapsed at the Olive View Hospital. The concrete stack, which is leaning slightly, was removed shortly after the quake. Ralph Samuels photo.

Figure 3.4 This old masonry building at Olive View Hospital, adjacent to the heating plant of the previous figure, was not designed to withstand earthquake loadings.

Figure 3.5 Unreinforced brick building constructed prior to 1933, Olive View Hospital. This building withstood the strong ground shaking surprisingly well; a few bricks were dislodged on the far side of the building. The building, just southwest of the new power plant buildings, remained in use after the earthquake.

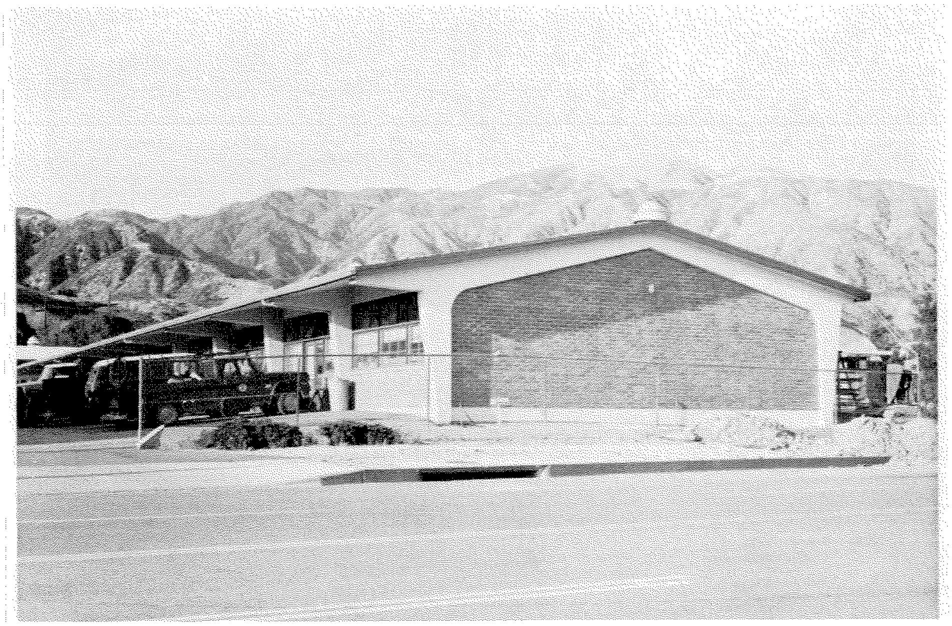


Figure 3.6 Reinforced concrete portal framed building with no visible damage. Olive View Hospital.

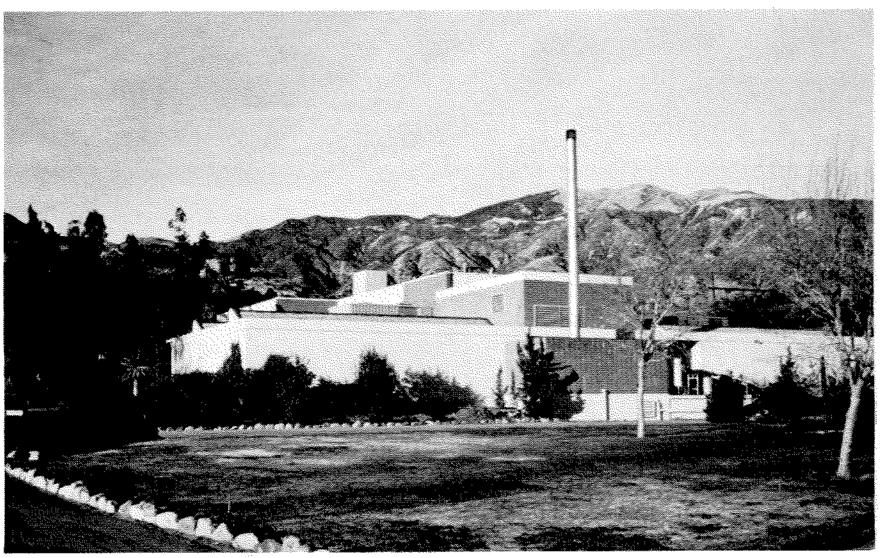


Figure 3.7 New laboratory building at Olive View Hospital received only minor damage.

Figure 3.8 One of the more severely damaged old wooden buildings at the Olive View Hospital.

Figure 3.9 Old wooden building moved to the right and nearly fell from its concrete support props, Olive View Hospital.

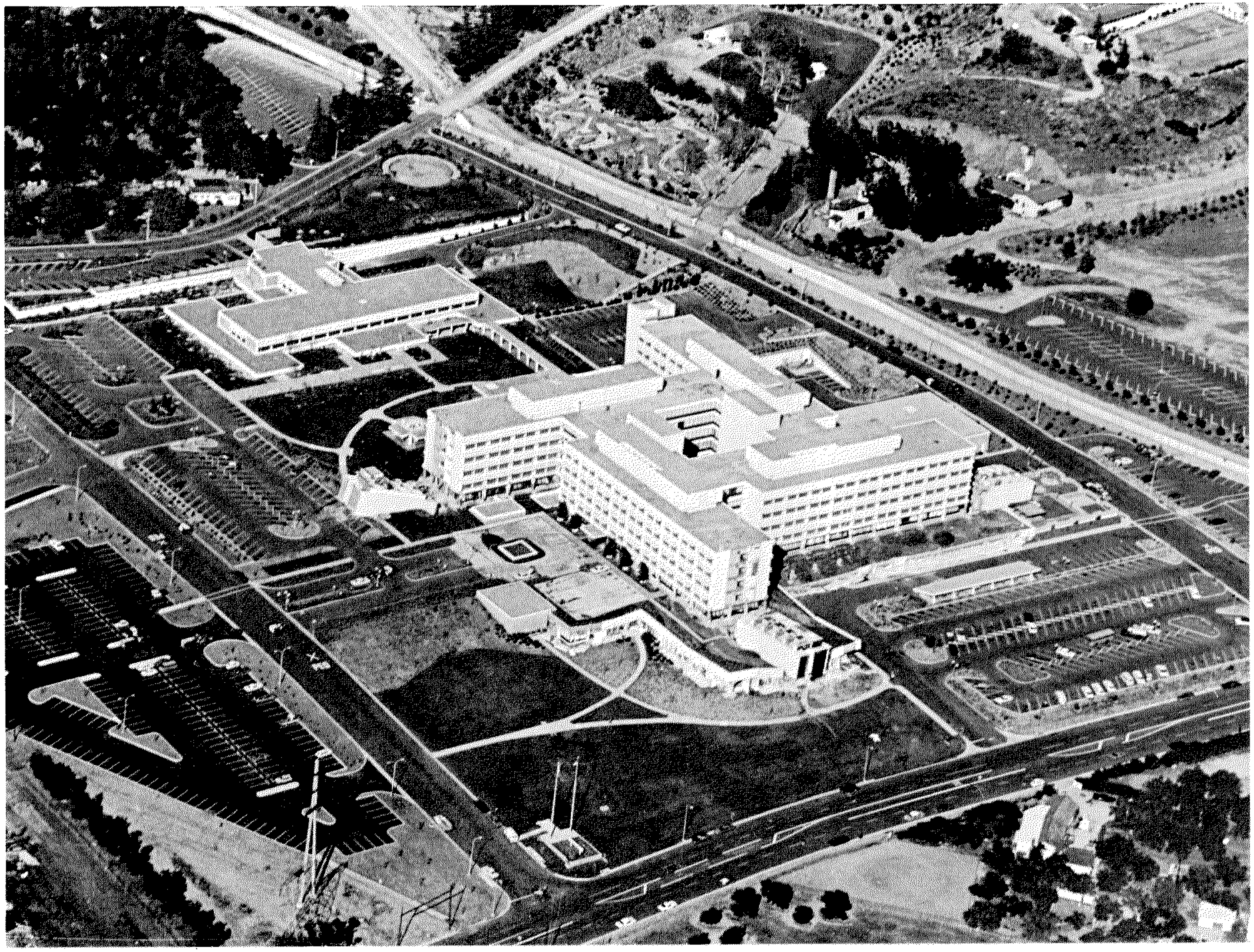


Figure 3.10 Aerial view of the Olive View Hospital, looking north. The collapsed two-story Psychiatric Day Care Center is in the upper left. Ralph Samuels photo.

first story collapsed (Fig. 3.11). Fortunately the first floor was not occupied at the early hour of the earthquake. The second floor came down essentially intact, riding the failing first-floor columns to the ground and coming to rest about one-half a column length to the south and east (Fig. 3.12). Figure 3.13 shows a failed column with the reinforcing steel visible. Lightweight concrete (2500 psi) was used in the structural columns and beams as well as in the ornamental concrete, and this shattered during collapse.

The sequence of failure is thought to have been as follows. The large shear forces between the upper level and the ground caused concrete to spall and break in the overstressed, tied, reinforced concrete columns of the first story, thereby reducing both the lateral and the vertical strength of the columns. This column deterioration probably progressed for a few cycles of response until the concrete cores were badly fractured. At this stage the columns could not support the structure, which then collapsed. Although the upper portion of the building appears nearly undamaged, other than a slight bow in the middle and a few broken windows, it did in fact receive serious structural damage, presumably during the collapse. The north wall of the building backed up against an earth embankment with retaining wall so that it could not have collapsed in a northerly direction (Fig. 3.12).

The five-story-plus-basement hospital,* seen in Fig. 3.10, was occupied the morning of the earthquake by nearly 600 patients and personnel. Two patients and one employee reportedly died from injuries and two patients in respirators died from interruption of electric power; however, under different circumstances the death toll might have been much higher. For example, it is doubtful if the already badly damaged columns could have stood up for another five seconds of strong ground shaking. As seen in subsequent figures,

*The new buildings at Olive View appeared to have been designed and constructed in essential agreement with the earthquake provisions of the 1964 building code.

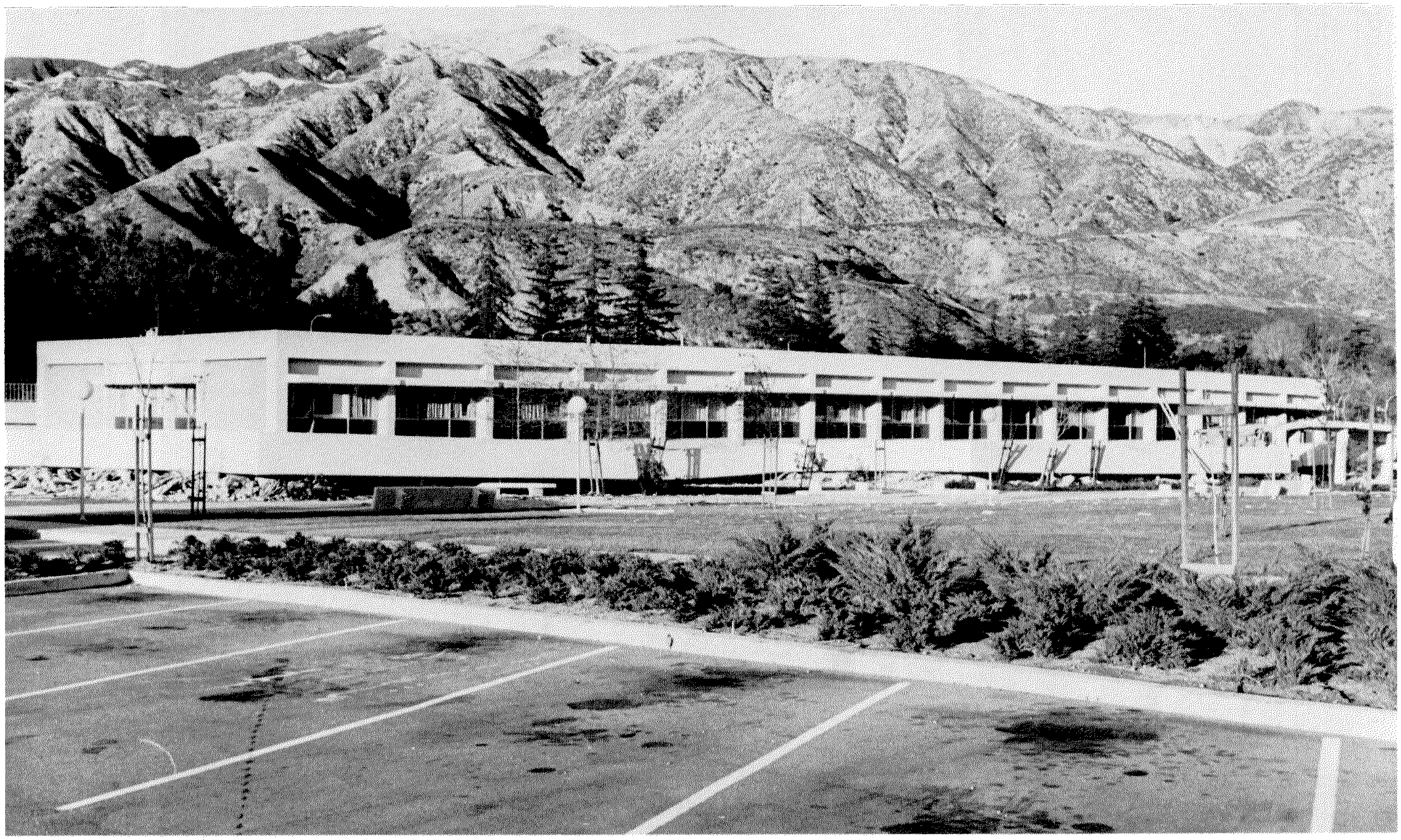


Figure 3.11 Two-story Psychiatric Day Care Center just north of the main building at Olive View Hospital collapsed towards the south.

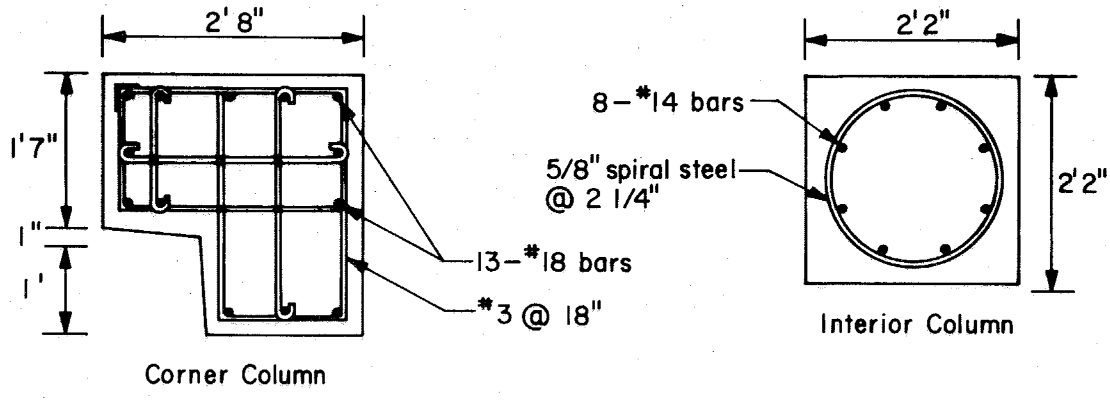
Figure 3.12 North entrance of the Psychiatric Day Care Center, Olive View Hospital. The second story fell down and to the south.

Figure 3.13 Destroyed column in Psychiatric Day Care Center, Olive View Hospital. The lightweight concrete shattered completely.

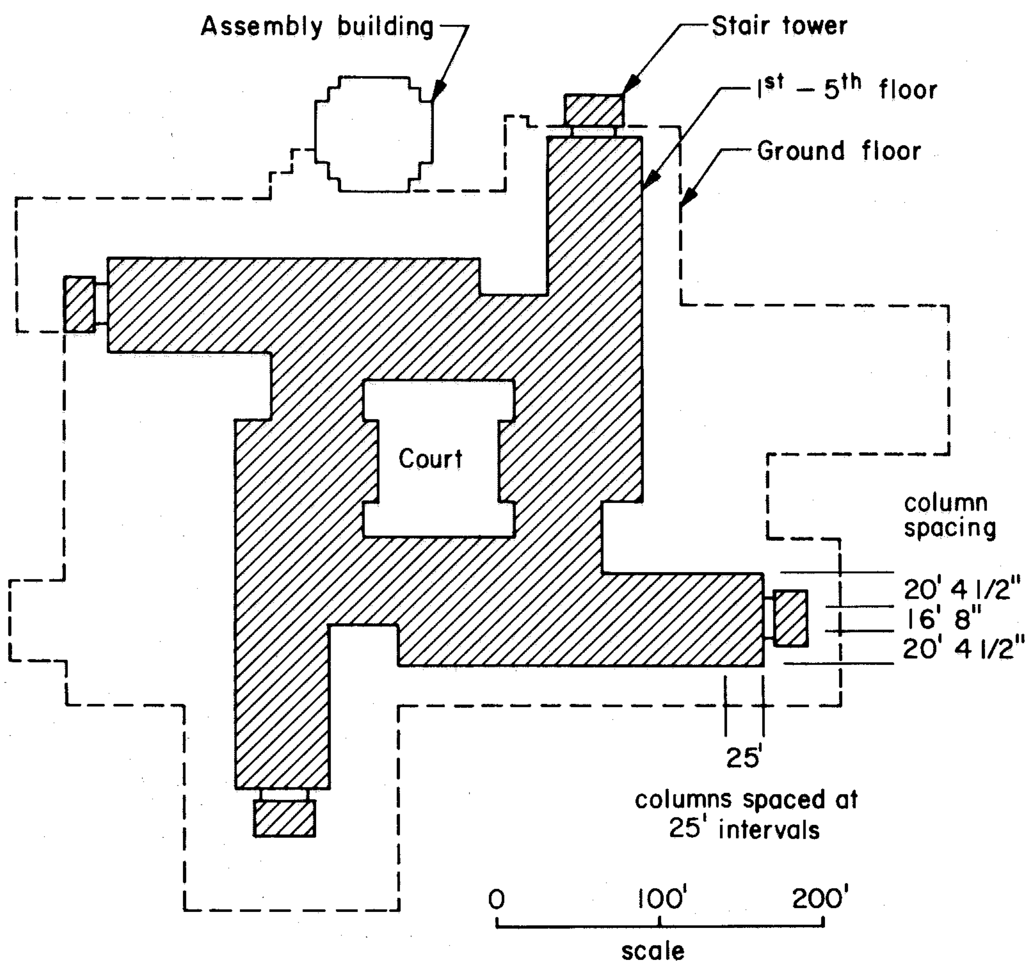
the entire structure from the second floor up came to rest offset nearly two feet to the north and one foot to the east. Also, had a fire occurred, which is quite possible under the circumstances, a disaster could have ensued as only two small stairs were usable for evacuation and these had been damaged and distorted by the building deformation.

A plan view of the hospital is given in Fig. 3.14 showing the symmetry of the building about an open courtyard in the center. The building is composed of four rectangular structures that form the walls of the courtyard and extend outward about 100 feet from the central hub of the building, with a stairwell tower at the end of each wing. The concrete floor slabs were continuous throughout the four wings and remained so during the earthquake. General views of the exterior of the damaged building are shown in Figs. 3.15 - 3.21. No surface faulting has been identified on the site and the foundation of the building was not disrupted.

The upper portion of the wings are strengthened in the short dimension by shear walls running the full width of the wing at one end (Figs. 3.16 - 3.19) and by shear walls about half the width of the wing at the other end (Fig. 3.21). The fact that these shear walls extend down only to the second floor level appears to be a significant factor in the gross behavior of the structure. Shear forces at first story and basement levels are carried by the reinforced concrete frame. The shear walls made the upper four stories behave essentially as a massive, rigid box supported on flexible columns. On the south and east sides of the building the basement floor was on the level of the streets (Fig. 3.10) and, hence, it appeared to be a six-story building without basement. On the north and west sides the basement story backed up against a retaining wall with a separation of a few inches. Hence, when vibrating to the south, the flexible frame was two stories high, but when vibrating to the



Typical First Floor Column Details



Plan View of Olive View Medical Center

Figure 3. 14 Plan view and typical column details, Olive View Hospital.

Figure 3. 15 The main entrance of the Olive View Hospital on the west side of the building.

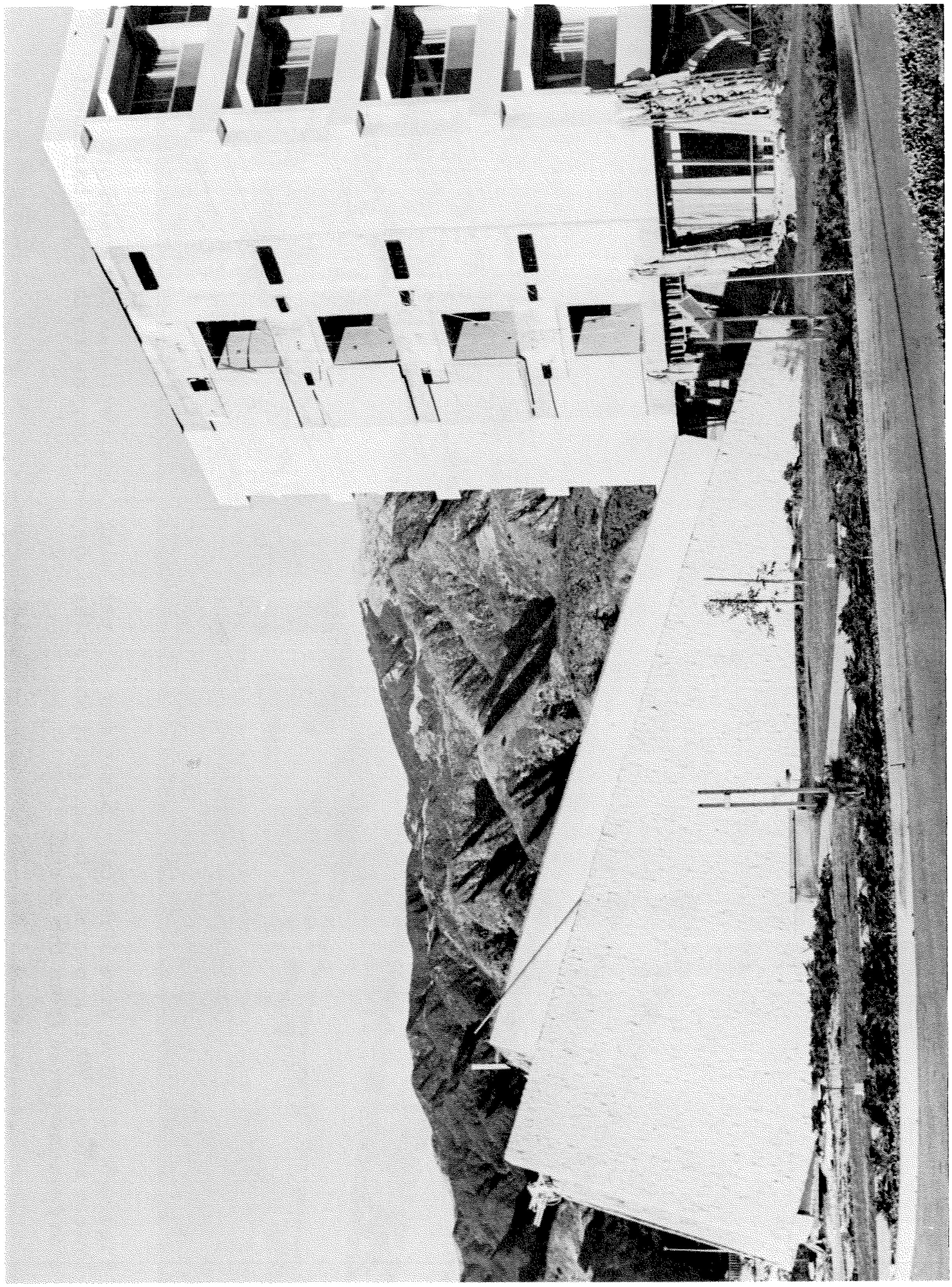


Figure 3.16 Collapsed stairwell at west end of Olive View Hospital building.

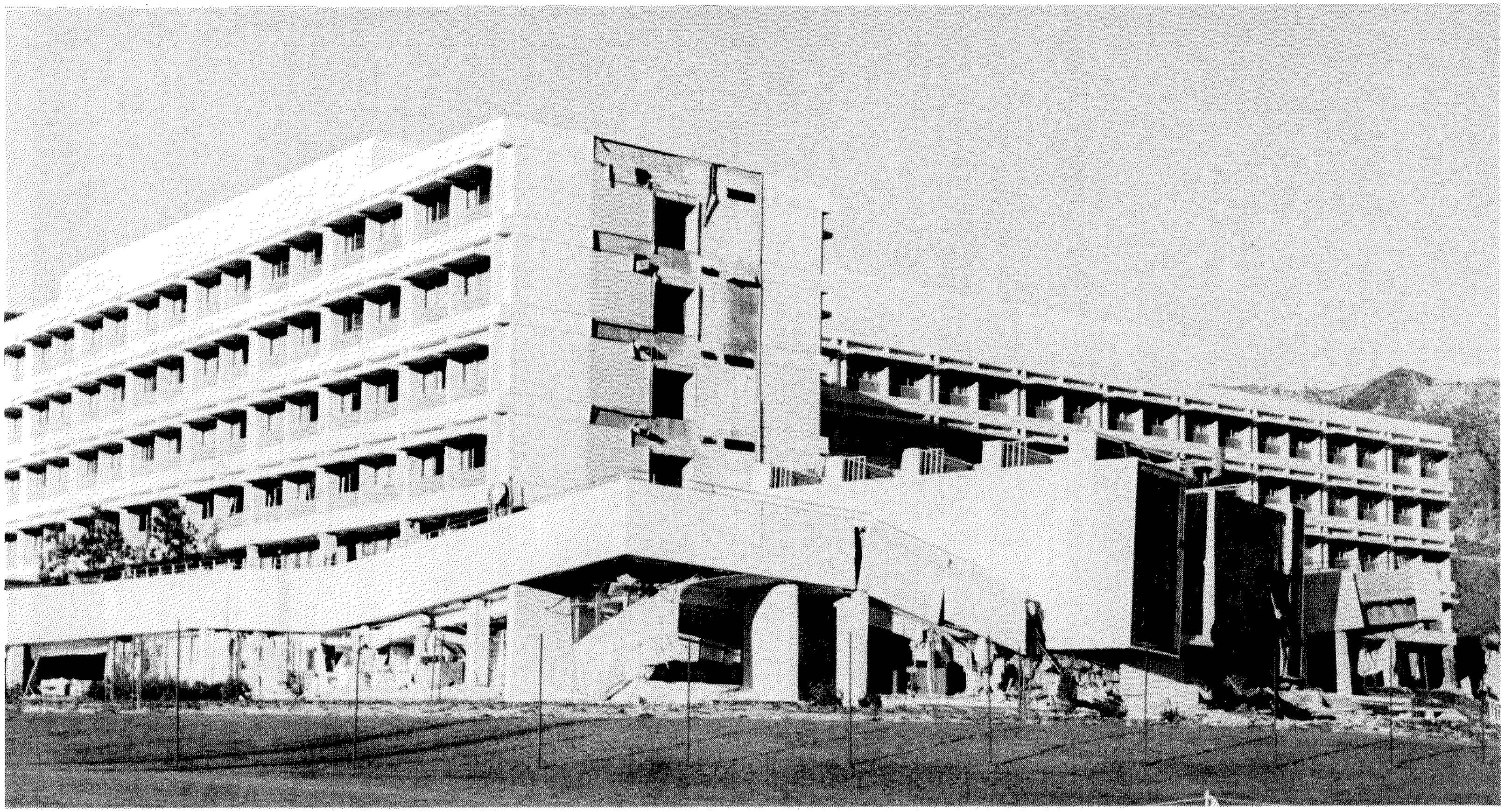


Figure 3.17 Olive View Hospital. South end of west wing. Stairwell has collapsed onto lowest story. What appears to be the first story when viewed from the south is the basement story when viewed from the north.

Figure 3.18 Olive View Hospital, looking north at the east end of the main building. Overturned stairwell at right, and collapsed canopy in left foreground.

Figure 3. 19 Olive View Hospital; west end of north wing. The collapsed stairwell is at the left. The shattered L-shaped corner column did not have a spirally wound core as did the other columns in the photo.

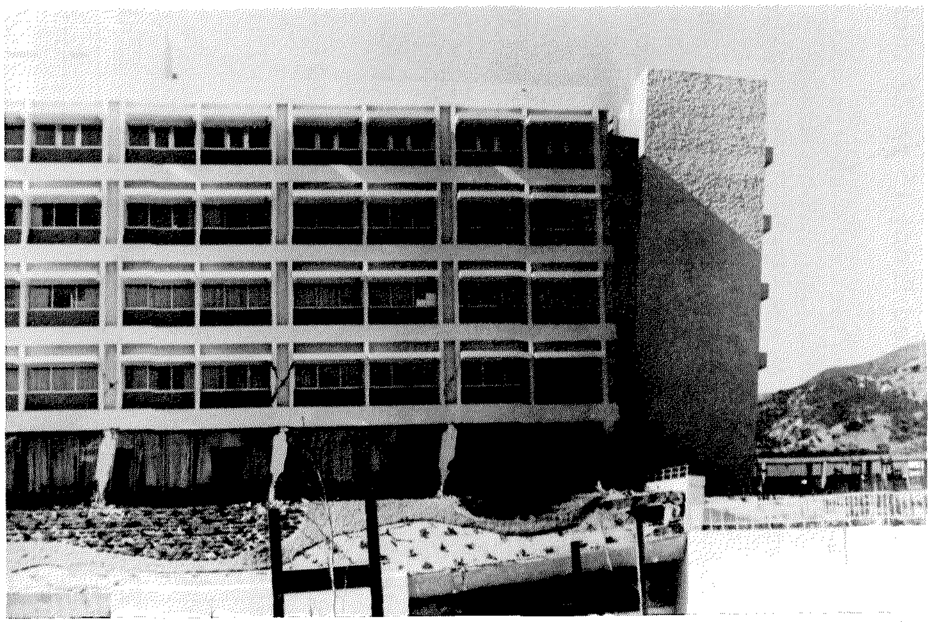


Figure 3.20 Looking west at the leaning north stair tower, Olive View Hospital. This stair tower was supported on footings, whereas the other three stair towers, which fell, were supported from below on the basement frame. The garden in the foreground was originally horizontal.

Figure 3.21 Shear wall at the north end of the southwest wing, Olive View Hospital.

north with large amplitude, there was effectively only one story of columns acting to resist horizontal shear. As seen in Fig. 3.14, the basement story extended beyond the upper portion to the west and north. As a consequence of this design, damage was mainly concentrated in the first story columns. While some shear cracking in the upper story columns was visible, indicating strong vibrations, particularly away from the end walls (Figs. 3.15 and 3.20), essentially all of the permanent lateral displacement of the four wings resulted from distortions in the first floor columns (Fig. 3.15), and to a lesser extent by damage to the basement columns, some of which showed displacement of from two to six inches. The two elevator shafts, visible from the courtyard (Fig. 3.22) moved to the north along with the upper floors by sliding along a separation joint at the first floor level (Fig. 3.23). The horizontal displacement at the base of the elevator shaft varied from 15 to 22 inches. Probably as a result of the lateral stiffness of the elevator shafts and interior stairwells, there were no visible distortions of the main building in the upper level floors facing the courtyard. Further evidence of the stiffness of the upper stories is provided by the fact that not a single upper-story window facing the courtyard was broken.

Three of the four stairwell towers toppled outward, away from the main buildings (Figs. 3.10 , 3.16, 3.17 and 3.18), and fell partly into the basement story. These towers were supported by beams and basement-story tied concrete columns. The remaining tower, on the north, was supported directly on footings (Fig. 3.20) and is leaning about 10° from the vertical. The fact that there was a separation between the stairwell towers and the main building is clearly visible in the figures. The towers were designed as separate structures, presumably because their rigidities were significantly different from that of the main buildings. Damage at the second floor level indicates

Figure 3.22 Looking southwest into the courtyard enclosed by the main building at the Olive View Hospital.

Figure 3.23 Exterior wall of elevator shaft moved about 16 inches to the north at the separation joint.

that perhaps there was not a complete separation at this level.

The hospital failure is further complicated by the presence of a basement floor which is considerably larger in plan than the building overhead (Fig. 3.14). Judging from the damage found in ground floor columns directly below the four wings it appears that some portions of the first floor moved horizontally about two to six inches. In some columns localized spalling of concrete at the joint between the ground level column and the first floor beams was observed while in others no damage was found. Many of the basement columns supporting the towers failed completely in shear.

There is extensive damage to the terrace structure supported by basement level columns along the periphery of the building (Figs. 3.18 and 3.20). It is conceivable that the impact of the stair towers on the terraces contributed to the terrace failures at points well removed from the actual points of impact. This conjecture would help to explain the remarkable contrast in the extent of damage at the first floor level in the terraces (major failure) as compared with the first floor level directly beneath the four wings (minor failure).

The general views of the hospital building show distortions in the first story columns, which were of two designs. The twelve corner columns were L-shaped with six ties (No. 3's at 18 in.) spaced over the story height as seen in Fig. 3.24; these columns were completely shattered. The 152 other columns had spiral steel ties (5/8 in. at 2-1/4 in. spacing) and although they lost much of the concrete covering, they retained load-carrying capacity (Figs. 3.25, 3.26 and 3.27). This ductile behavior resulted in spectacular effects in the interior of the building as seen in Fig. 3.28. While the major building damage was restricted to the first floor columns, there were shear failures in some of the upper story columns (Figs. 3.29 and 3.30). Also, many of the connections were damaged (Figs. 3.16, 3.19, 3.29 and 3.31). The ability of the spirally-reinforced columns to remain load carrying after such large overstraining demonstrates the advantage of reinforcing concrete for ductile behavior.

Figure 3.24 Shattered L-shaped corner column, Olive View Hospital. The ties are No. 3's at 18 in on centers.

Figure 3.25 Spirally reinforced column at the Olive View Hospital. The untied reinforcing steel outside the core of columns spalled away on most of the columns of this type.

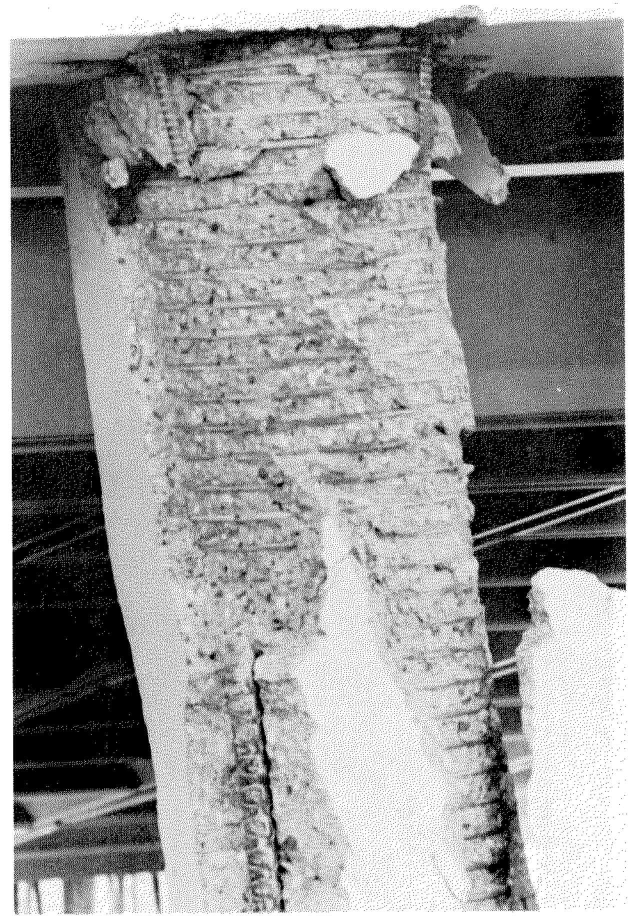


Figure 3.26 Details of the core of a spirally reinforced column at the Olive View Hospital. The spiral reinforcing was very effective in confining the concrete in the columns.

Figure 3.27 Damaged column at Olive View Hospital. This square concrete column had a circular, spirally wound core, and had four large bars outside the core at the corners. Those with tied outside bars performed somewhat better than those without ties shown in Figure 3.25.

Figure 3.28 Cafeteria in Olive View Hospital.

Figure 3.29 Shear failure in a second story column of the west wing, Olive View Hospital.

Figure 3.30 Above main entrance of Olive View Hospital. View showing cracks in second and third story of west wall of south wing.

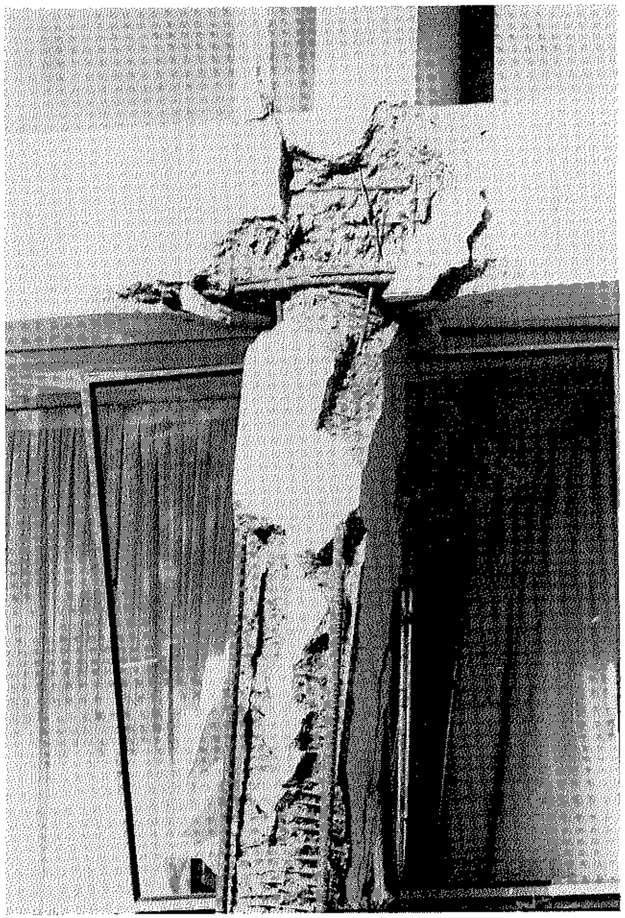


Figure 3. 31 Connection damage between second floor beam and spiral column, Olive View Hospital

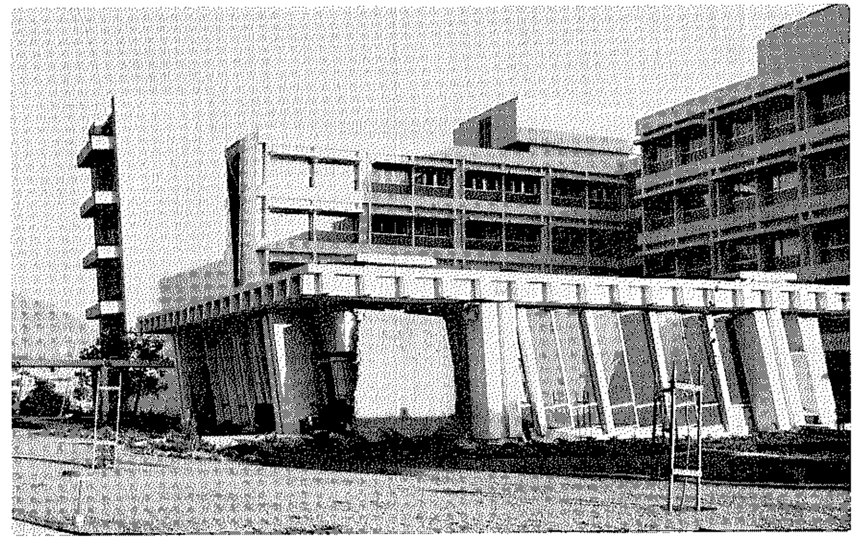


Figure 3. 32 Damaged assembly hall at Olive View Hospital. Failure occurred at the column-roof connection.

Adjacent to the hospital on the north is a small one-story assembly hall that was severely damaged (Fig. 3.32). The auditorium is composed of a waffle-grid lightweight concrete ropf supported by eight lighweight, reinforced concrete columns. The first floor and the basement floor of the hospital join with those of the auditorium (Fig. 3.14). It is not known to what extent the presence of the basement contributed to the failure; however, the distortions pictured in Fig. 3.32 appear to be due to insufficient horizontal strength above the first-floor level. Failures occurred at the tops and bottoms of the columns.

Other lightweight concrete construction in the area also suffered failure at the connections between structural members. Note, for example, the independent, cantilevered canopy around the exhaust pavilion pictured in Figs. 3.33 and 3.34, also to the north of the hospital. The nature of the cracking resembles torsional failure that could have resulted from strong horizontal ground shaking, but vertical motions also could have contributed to the failure. The lightweight concrete sidewalk canopy running between the hospital and the psychiatric ward also suffered damage at the connections. Some of this damage may have resulted from the displacements of the buildings at either end. Portions of the canopy are visible in Figs. 3.10 and 3.32.

To the south of the Olive View Hospital, an ambulance parking pergola collapsed on the emergency vehicles underneath (Figs. 3.10, 3.18 and 3.35). This collapse was a combination of column failure and foundation failure. The six supporting columns along the south edge rotated with relatively little column damage, while the six supporting columns along the north edge sheared off with the lower segments remaining nearly vertical.

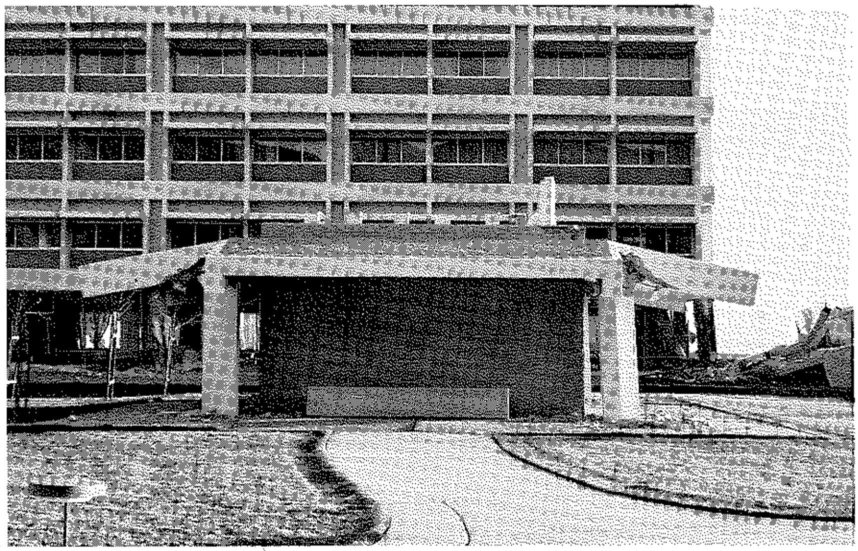


Figure 3. 33 Exhaust pavilion to the north of the main building of the Olive View Hospital. The overhangs were originally horizontal. The cantilevered canopy was supported on four columns and is independent of the exhaust structure.

Figure 3. 34 Beam and connection failure in the free-standing canopy of the exhaust pavilion, Olive View Hospital. The failure pattern suggests torsional failure of the beams from lateral motion of the structure. Note the scars on the brick facing of the separate, interior structure.

Figure 3.35 Concrete canopy collapsed onto ambulances at Olive View Hospital, just south of main building. The columns were originally vertical.

Pictured in Figs. 3.36 - 3.39 is the new central heating and refrigeration plant, about 150 yards northwest of the main hospital building. This steel-framed building is about 20 feet high with reinforced concrete columns on the exterior. Brick filler walls on the east, south and north walls resisted the horizontal forces. Some damage to the structural frame was found in the connection at the top of one of the reinforced concrete columns (Fig. 3.37), however, the building appears to be structurally sound. Figure 3.39 shows the north wall with the exterior metal lath and plaster cladding shaken loose and the tensile steel cross bracing (1/2" x 4") stretched out of shape. One of the easterly pairs of steel cross braces broke at the weld, and its companion was almost fractured through at the end of the earthquake. One end of a large boiler that was originally about three feet from the westerly panel of the wall slid into the wall and was responsible for some of the damage to the plaster wall and the stretching of the bars. Marks on the floor and damage to the facilities indicate that the equipment inside, which was not fastened down, slid during the earthquake (Fig. 3.40). The fact that all of the equipment tended to move in a generally northerly direction with respect to the building is consistent with the fault movement and indicates that in addition to transient shaking, the ground also had a permanent southward displacement during the earthquake, i.e., the building displaced with respect to the equipment.

Ground cracking was observed at the Olive View site in the parking lots of the main structure and in the adjacent fields. Also there were disruptions of the ground immediately adjacent to the main building which resulted from relative movement between the ground and the large basement of the structure. As of this writing, no ground displacements have been found that are clearly tied to faulting at the site, and it is concluded that faulting is not responsible for the damage to the facility. Furthermore, the dimensions

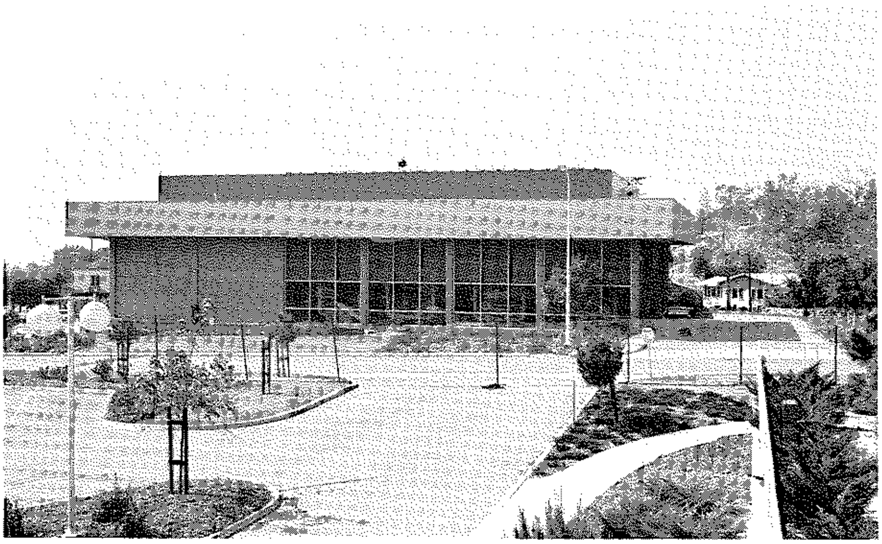


Figure 3. 36 East wall of central heating and refrigeration plant, 150 yards northwest of the main buildings of Olive View Hospital.

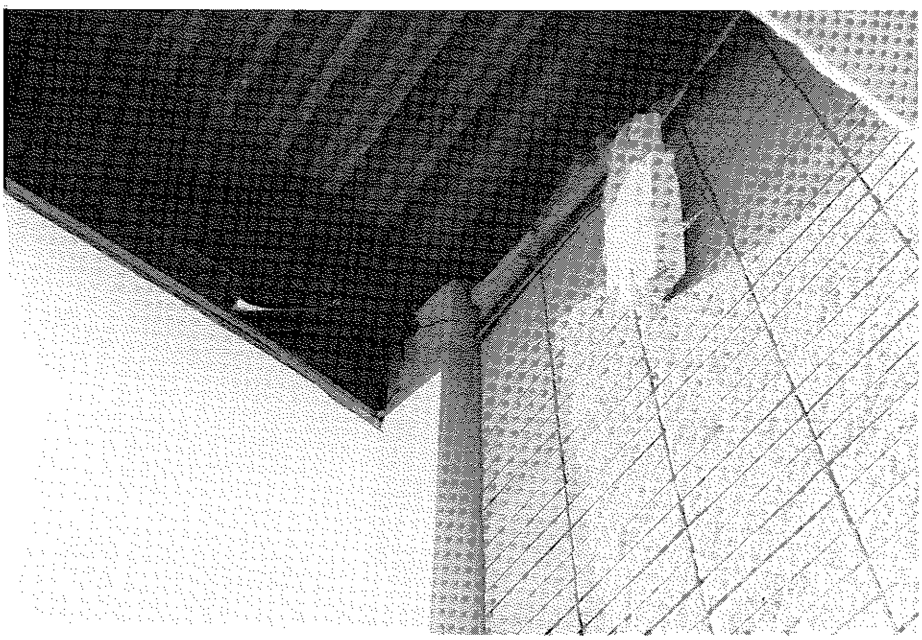


Figure 3. 37 Damage at the connection between the reinforced columns and the steel beams, central heating and refrigeration plant, Olive View Hospital.

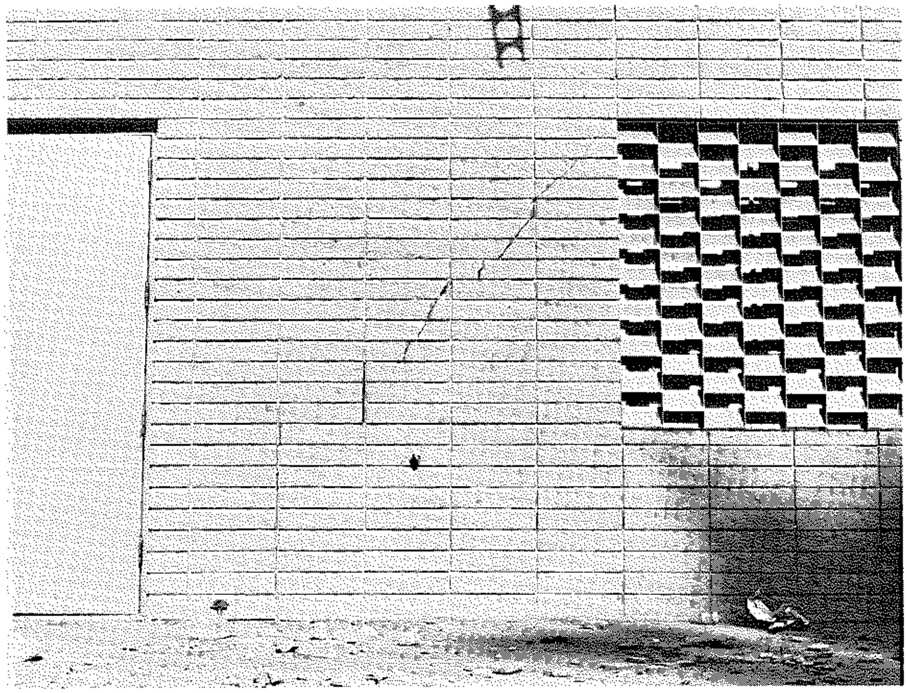


Figure 3.38 Crack in west wall of power plant building at Olive View Hospital. A similar crack, though smaller, occurred in the east wall.

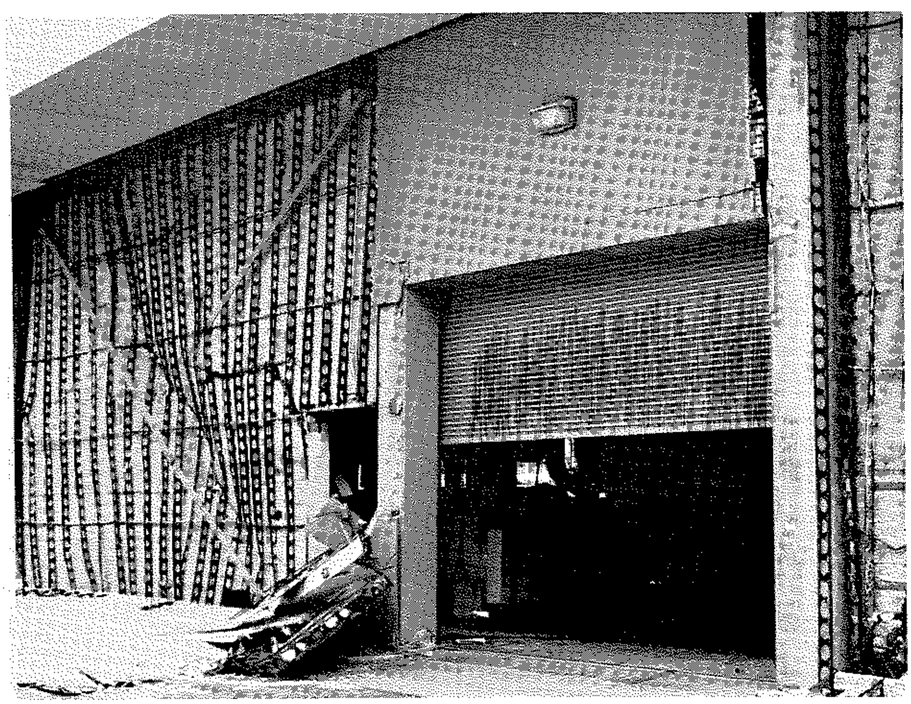


Figure 3.39 The two outside bays of the north wall of the power plant building at Olive View Hospital had steel strap crossbracing (1/2" x 4"). One strap failed at the weld and its companion had almost failed. The The two straps in the west panel were badly deformed.

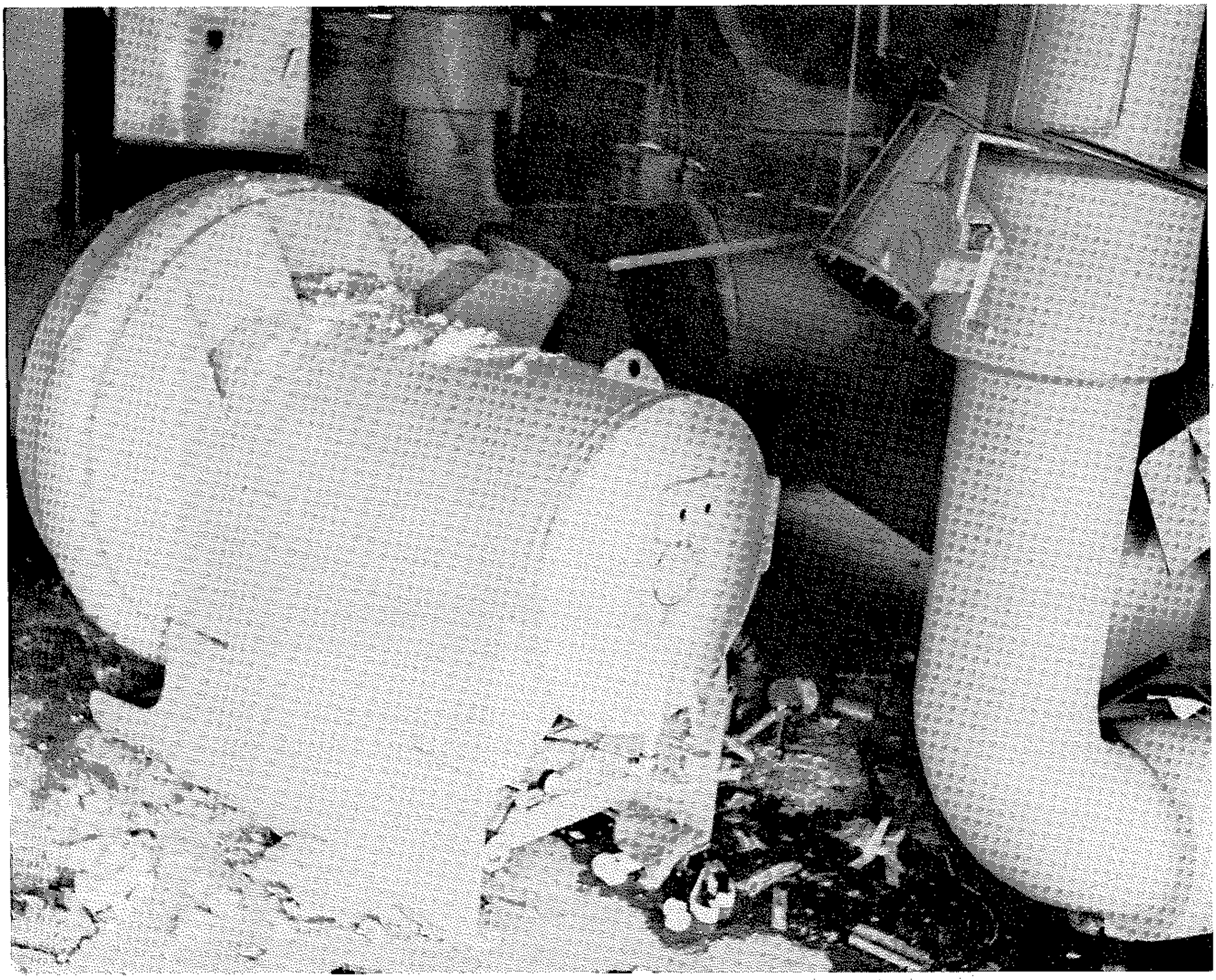


Figure 3.40 Equipment that was not bolted to the floor in the Olive View Central Heating and Refrigeration Plant moved about.

of the ground displacements are on the order of an inch near the building, much smaller than the structural displacements. From the available evidence it is concluded that the damage to the main buildings at Olive View Hospital was caused by strong ground motion.

San Fernando Veterans Hospital*

The San Fernando Veterans Hospital complex was in the area of strongest shaking (Fig. 1.2) and it was here that the greatest concentration of fatalities occurred when two major structures collapsed killing 46 people, most of whom were patients. The complex consists of a number of buildings constructed in 1925, with major additions added in 1938 and 1949. The site is shown in the aerial photograph in Fig. 3.41 which was taken the day of the earthquake. Many of the buildings to be discussed below are identified in Fig. 3.41.

The older buildings at the Veterans Hospital were built before earthquake-resistant design practices were enforced. They have concrete frames and concrete floors, and hollow-tile exterior and interior walls, a type of construction known to be hazardous in the event of a strong earthquake. Although the buildings met existing standards at the time of construction, they possessed little earthquake resistance and would not be acceptable by modern standards. As seen in Fig. 3.41 and subsequent figures, most of the older buildings either collapsed or suffered severe damage. By contrast, the two newer buildings on either side of the major collapse, as well as an additional wing added to one of the destroyed buildings, were built in 1938, 1944 and 1950, respectively,

*"Report on Investigation of Earthquake Damage to Structures at the Veterans Administration Hospital, 13000 Sayre Street, San Fernando, California," Brandow and Johnston Associates, March 12, 1971.

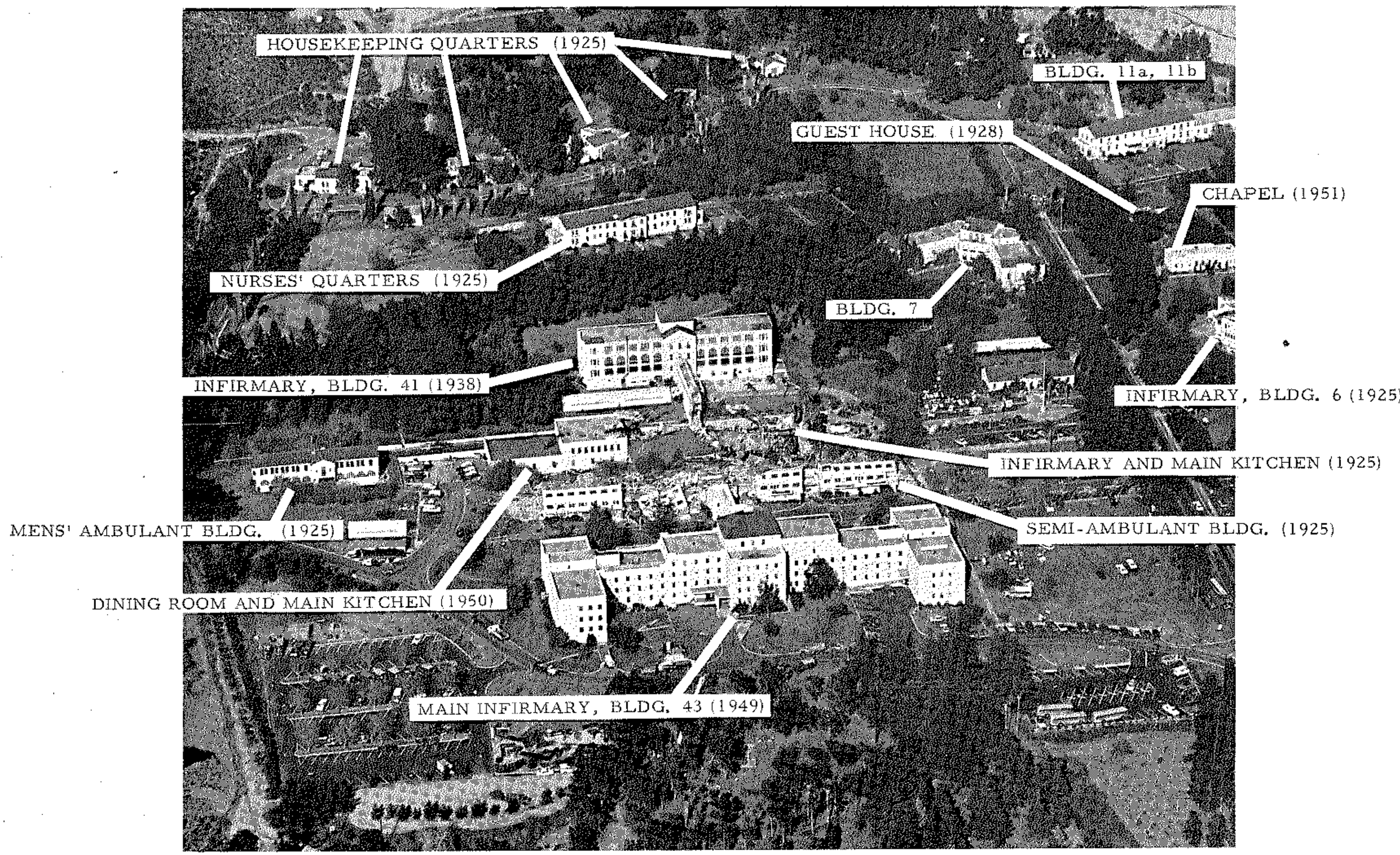


Figure 3.41 Aerial view, looking northeast, of the San Fernando Veterans Hospital taken the day of the earthquake. Ralph Samuels photo.

after the inclusion of earthquake-resistant design measures in the building code. These reinforced concrete structures survived the earthquake with no serious structural damage, although interior furnishings and medical equipment were greatly disturbed by the vibrations.*

Two adjacent three-story buildings, the Semi-Ambulant Building (constructed in 1925) and the Infirmary and main kitchen (1925) suffered major collapse as shown in Figs. 3.41 - 3.44. Closer views of the structural elements are given in Figs. 3.45 and 3.46. All of these photographs, with the exception of Fig. 3.41 as noted above, were taken the afternoon of the day following the earthquake. In the rescue operations, the rubble proved to be difficult to remove despite round-the-clock efforts of rescue workers. It was four days after the earthquake that the last victim was recovered and the last survivor was rescued 58 hours after the building collapsed.

The newer Infirmary Building, Building 41 (1938) is partly visible in Fig. 3.42, and portions of the Infirmary and the Dining Room and kitchen addition, Building 26 (1950) can be seen in Fig. 3.43. A photograph of the central portion of the main Infirmary Building, Building 43 (1944) looking east is given in Fig. 3.47. These three buildings, all of which border the collapsed structures (Fig. 3.41) survived the earthquake with at most minor cracking. The structures had all been designed to resist earthquakes and were of the concrete shear-wall type; they were actually stronger than required by the building code.

The Mens' Ambulant Building, Building 3 (1925), is a smaller two-story structure, located to the northwest of the main building of the hospital (Fig. 3.41). As can be seen in Figs. 3.48 - 3.51, the building suffered severe structural damage at the beam-to-column connections and at the arches where the columns failed in shear.

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*The four code-designed buildings at this site survived without significant structural damage despite being just one mile from Pacoima Dam.

Figure 3.42 Rescue operations at the Veterans Hospital, looking north.

Figure 3.43 Rescue operations and debris removal at the Veterans Hospital. Opposite view from Figure 3.42.

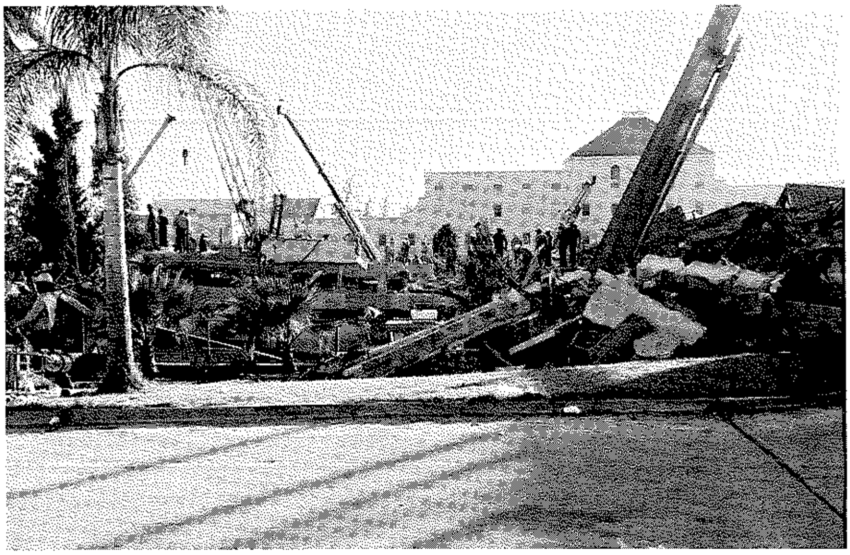


Figure 3.44 Collapsed building at the Veterans Administration Hospital. Building in background was not damaged significantly.

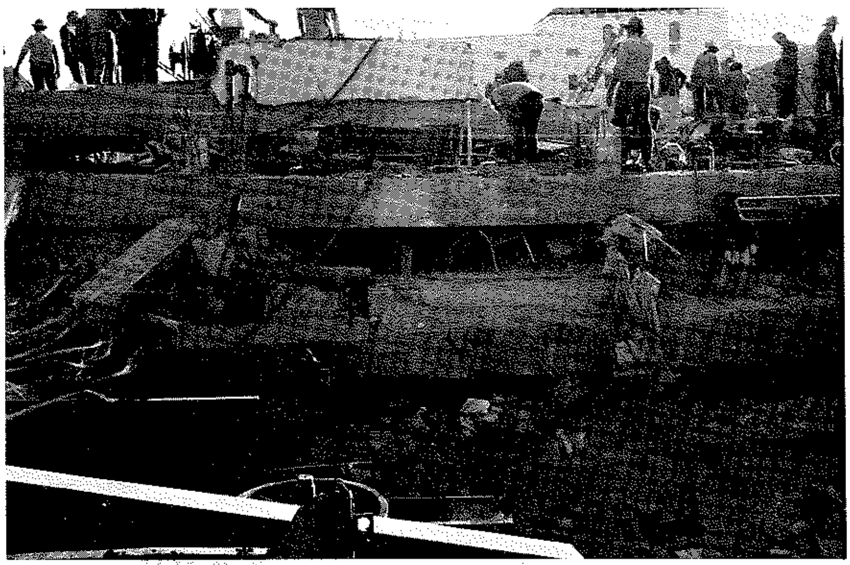


Figure 3.45 Details of collapsed structure, Veterans Hospital. Building in background was not damaged significantly.

Figure 3.46 The collapsed Semi-Ambulent Building at the San Fernando Veterans Hospital. View is looking to the northeast shortly after the earthquake. L. A. Times photo.

Figure 3.47 The Main Infirmary Building, a concrete frame and shear-wall building at the Veterans Administration Hospital, was not seriously damaged. The building was constructed in 1949, after earthquake provisions were a part of the building code.

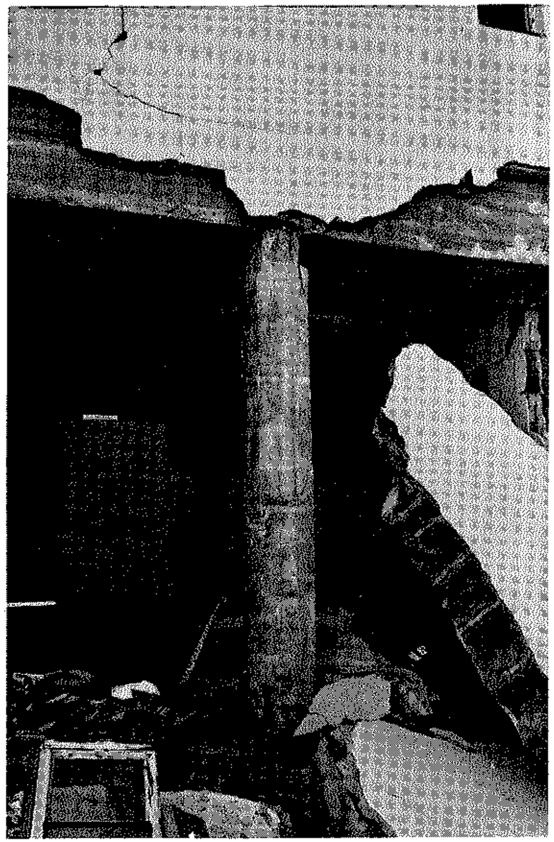


Figure 3.48 Damage to southeast end of the Mens' Ambulant Building; typical old construction. Note column-to-beam connections and portion of collapsed tile filler-wall. Veterans Hospital.

Figure 3.49 Eastern corner of the Men's Ambulant Building, Veterans Hospital, showing column damage.

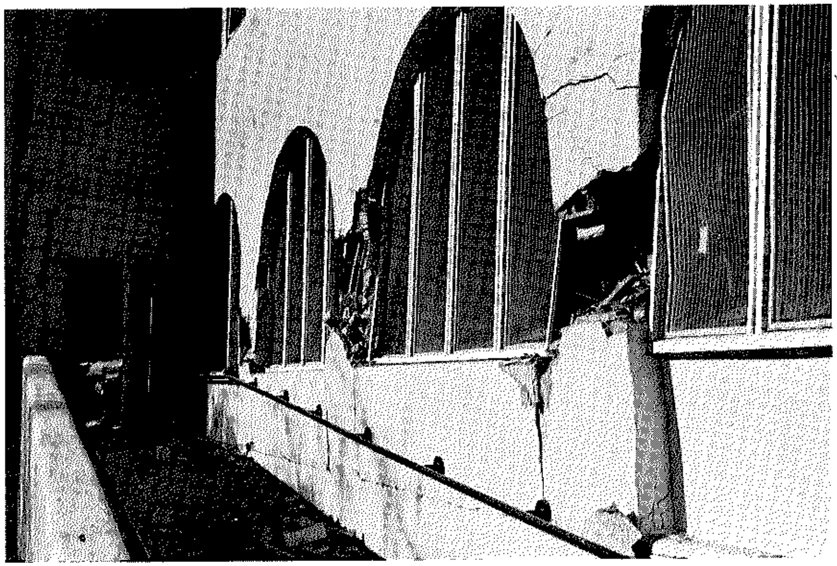


Figure 3.50 Shear failure in columns in southwest side of Men's Ambulant Building, Veterans Hospital.

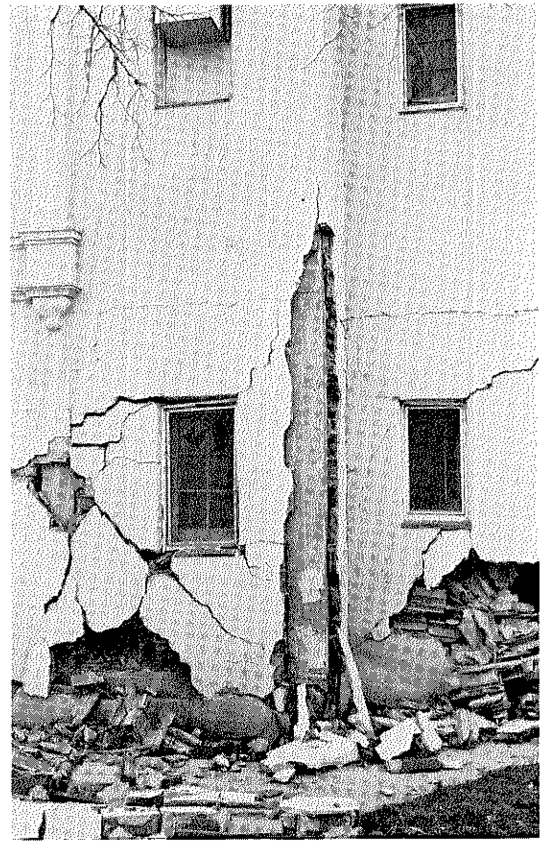


Figure 3.51 Northeast side of Men's Ambulant Building, Veterans Hospital. Note base of column.

The two neighboring Infirmary Buildings, Buildings 5 and 6 (1925), one of which is seen in the far right of Fig. 3.41, are similar in construction to the Mens' Ambulant Building. As is shown in Figs. 3.52 - 3.55, these buildings also received extensive structural damage.

Many other buildings at the site were severely damaged or collapsed. For example, the Nurses' quarters (1925) shown in Fig. 3.56 and the Engineering Office and Shop Building (1925) shown in Fig. 3.57. Wood frame and tile construction also suffered severely as seen in Fig. 3.58 which shows the Guest House, constructed in 1928. The Chapel, of newer construction (1951), survived without damage as is seen in Fig. 3.59.

The Housekeeping quarters, Buildings 13, 14, 15, 16, 17 and 18 (1925) suffered varying degrees of damage including large fractures and partial collapse in one instance. The Nursing Home Care Unit, Building 11a(1925) and the Addition, Building 11b (1929) received severe damage to the walls although the frames are essentially intact. The Recreation Supply and Fiscal Building, Building 7 (1925) received similar damage.

The strong ground motion at the site caused some heavy mechanical and electrical equipment to move several inches as shown by the foundation marks visible in Figs. 3.60 and 3.61. Similar movement of medical and office equipment was noted in the interior of buildings.

There was evidence of soil movement at the site, but no evidence of faulting. The most noticeable ground movements were those associated with a slide on the edge of the wash to the east of the main building complex as shown in Fig. 3.62. Also, there was downslope movement of the soil to the north of the Infirmary (Building 41) as indicated by Fig. 3.63. The sidewalk put in compression by the movement shown in Fig. 3.63 buckled at its other end, some 50 ft away where it abuts an entrance to Building 41.

Figure 3.52 Infirmary Building (Bldg. 5), Veterans Hospital, looking south.

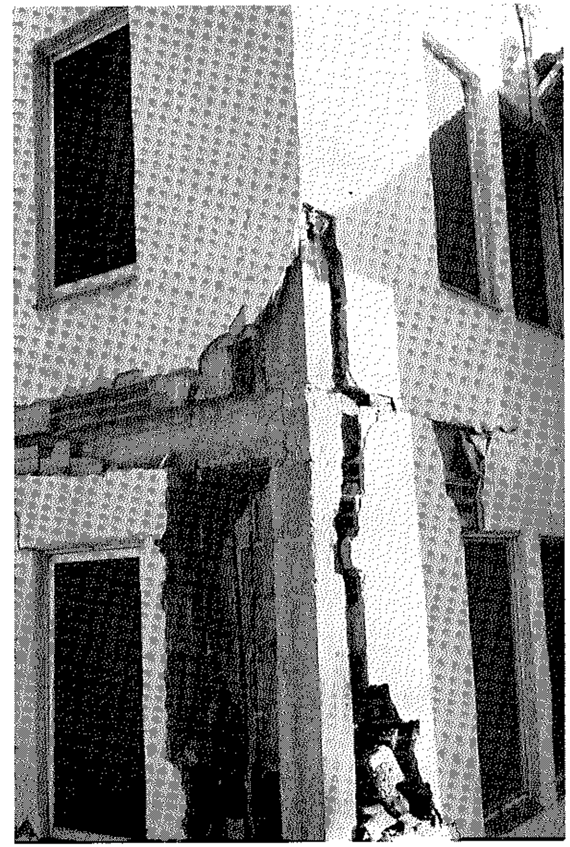


Figure 3.53 Beam-to-column connection, Infirmary Building (Bldg. 5), Veterans Hospital.

Figure 3.54 Failure of column at base, Infirmary Building (Bldg. 6), Veterans Hospital.

Figure 3.55 Shear failure of column, Infirmary Building (Bldg. 6), Veterans Hospital.

Figure 3.56 Nurses' Quarters (Bldg. 12) at the Veterans Hospital, as seen from the southwest. The gable wall fell in front of the entrance.

Figure 3.57 Collapsed Engineering and Shop Building at the Veterans Hospital.

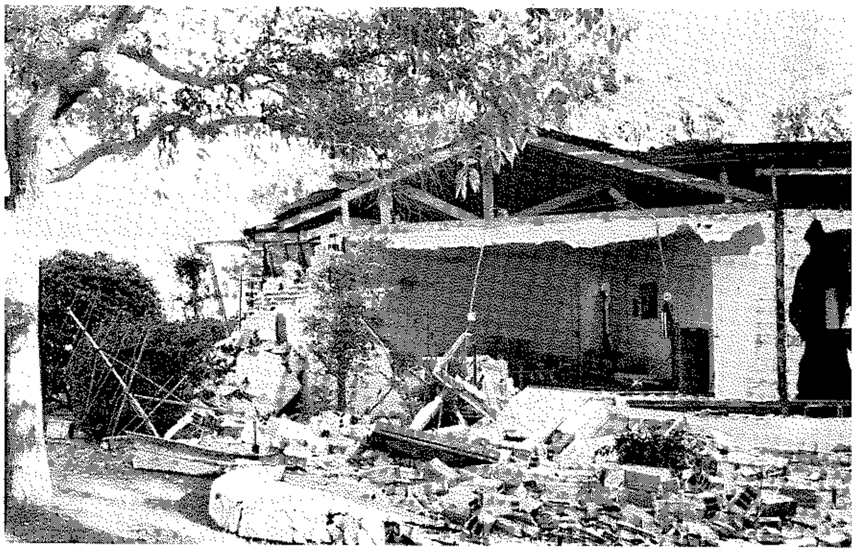


Figure 3.58 Partly collapsed wood frame and tile Guest House, Veterans Hospital.

Figure 3.59 The chapel at the Veterans Hospital, built in 1951, suffered no apparent structural damage.

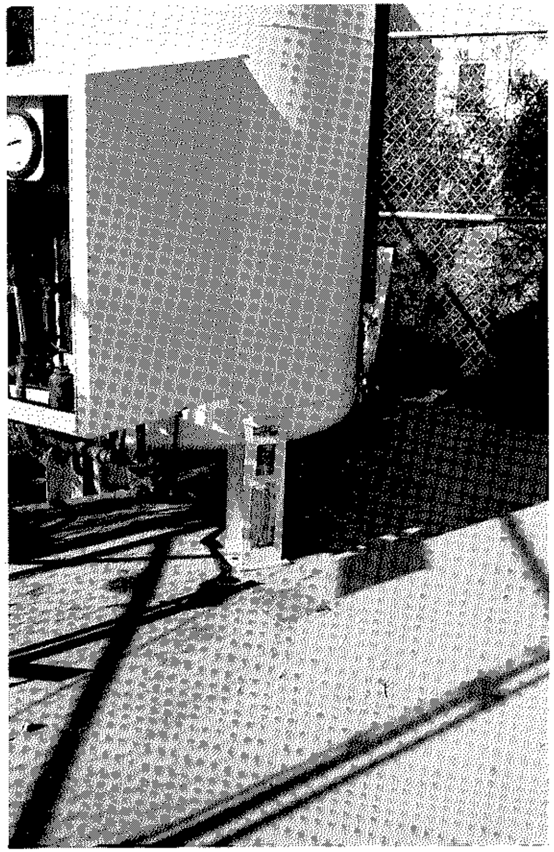


Figure 3.60 Marks on foundation show movement of storage tank, Veterans Hospital.

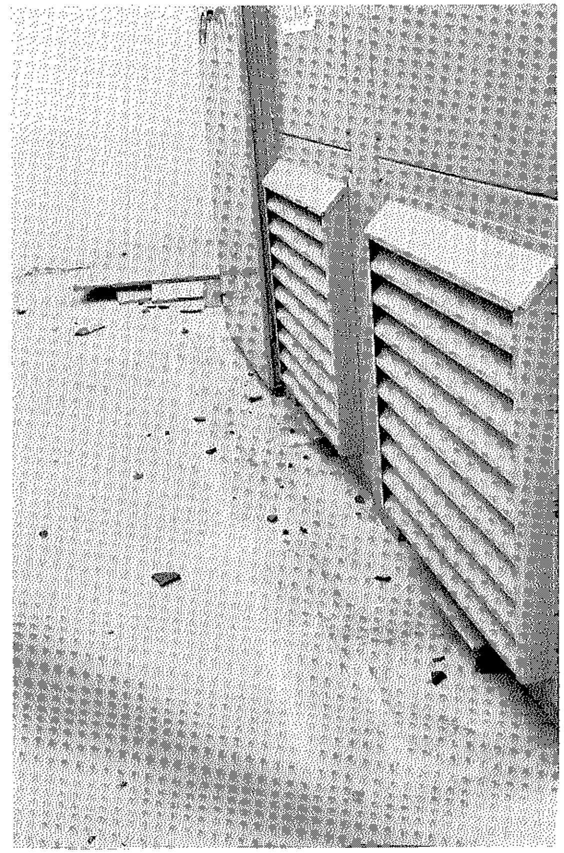


Figure 3.61 Electrical equipment was displaced several inches by earthquake motions, Veterans Hospital.

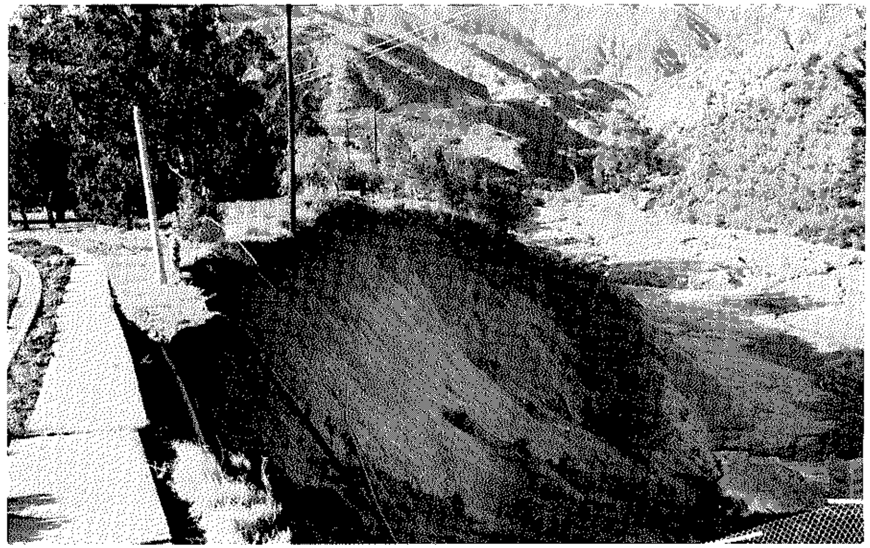


Figure 3.62 Landslide at eastern edge of site, Veterans Hospital.

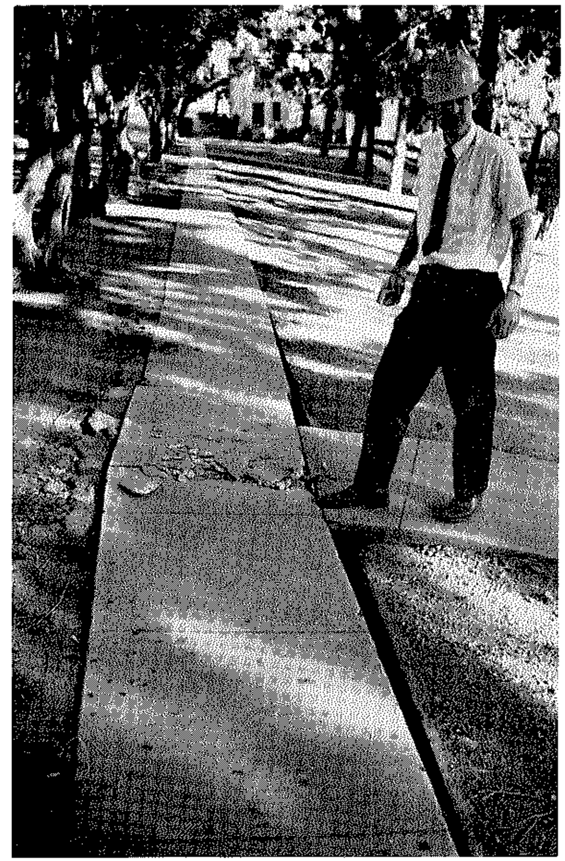


Figure 3.63 Sidewalk failure from ground movements near Infirmary Building (Bldg. 41), Veterans Hospital.

The damage and collapse of the San Fernando Veterans Hospital was obviously a result, primarily, of the age of the structure, with 1933, the year of inclusion of earthquake provisions in the building code, serving as the dividing line. For the older buildings, the effects of the earthquake seem to be a clear result of inadequate resistance and intense ground shaking. The loss of life in the collapse of the major structures underlines the great earthquake hazard posed by older buildings. For buildings serving as Veterans hospitals, the responsibility for identification of hazardous buildings and their rehabilitation or removal clearly lies with the Veterans Administration. At this writing, it is understood that the necessary program is in its beginning stages.

Holy Cross Hospital

Holy Cross Hospital is a modern seven-story reinforced concrete building (Figs. 3.64, 3.65 and 3.66) located next to the six-story Indian Hills Medical Center building on Rinaldi Street, three miles south of Olive View Hospital (Fig. 1.2). The Holy Cross Hospital was the more severely damaged of the two buildings, and it may not be economical to restore the building to its former condition. At the time of the earthquake, about 170 patients and 100 personnel occupied the hospital; no deaths were reported.

The hospital building is 184 ft long, 88 ft wide, and 88 ft high with nine equally-spaced columns in the long direction. The west end of the building contains two 33 ft-6 in. shear walls separated by 21 ft-6 in. (Fig. 3.69). Two columns are symmetrically positioned in the space between the two shear walls. The other end of the building (Fig. 3.67) has much the same structural makeup except for the presence of an exterior stairwell constructed alongside one of the shear walls. The exterior shape of the building is further complicated

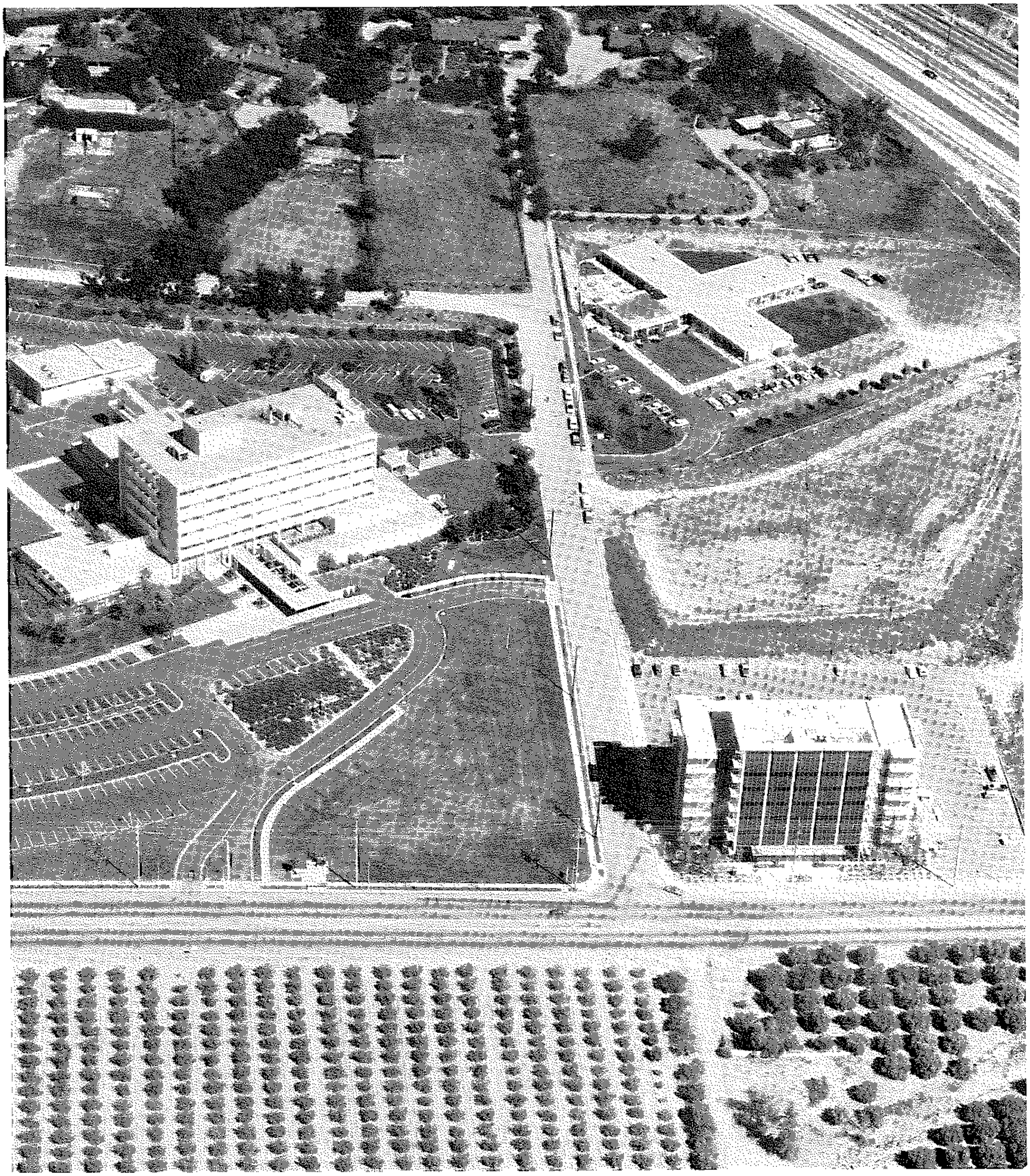


Figure 3.64 Aerial view looking north across Rinaldi Street. Pictured are Indian Hills Medical Center (lower right), Holy Cross Hospital (left), and Holy Cross Continuing Care Facility (upper right).

Figure 3.65 Holy Cross Hospital looking northwest. A break in the end wall at the fourth floor construction joint is visible at the right.

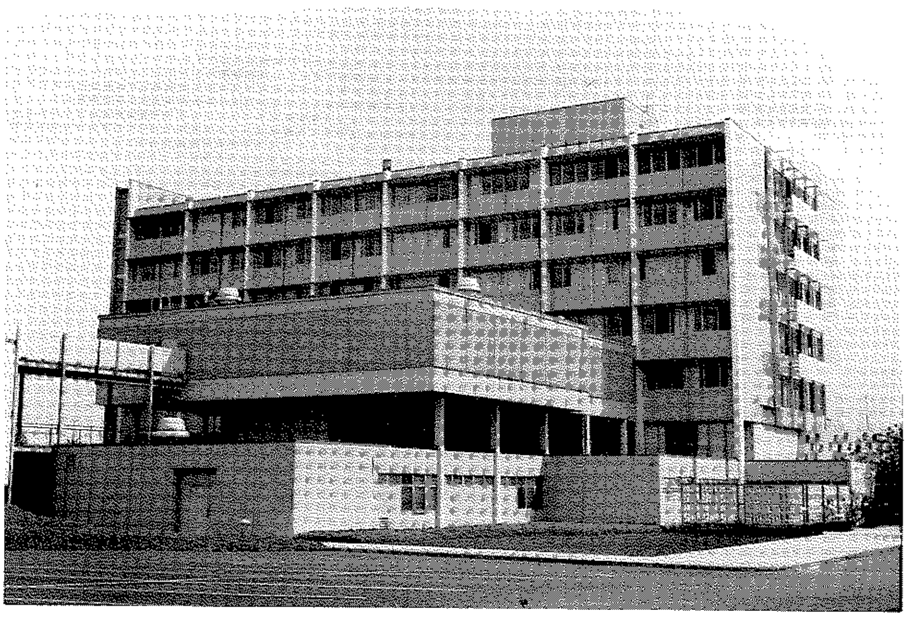


Figure 3.66 Holy Cross Hospital looking southeast.

Figure 3.67 Holy Cross Hospital showing fractured end wall at the fourth floor construction joint.

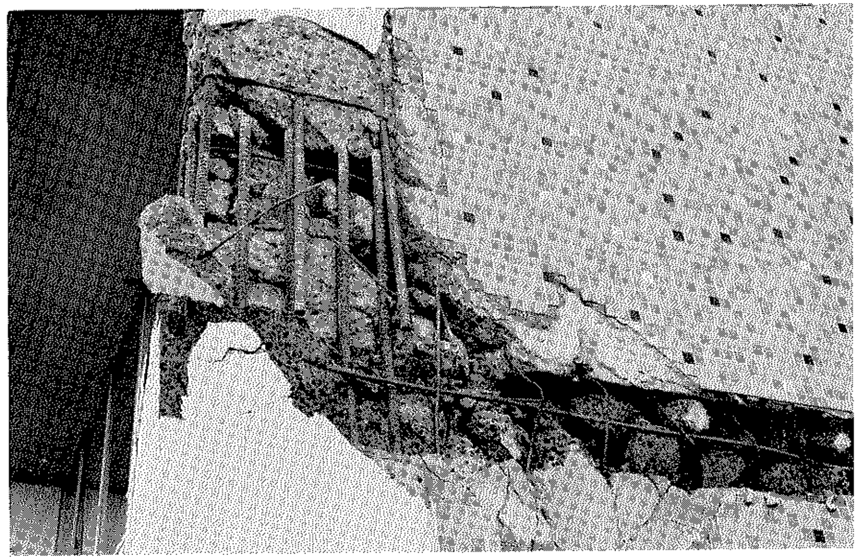


Figure 3.68 Closeup view of the end wall fracture at the fourth floor construction joint. The distorted vertical reinforcing steel in the wall indicates large movements have taken place. The vertical reinforcing steel in the corner column is lapped at the fourth floor level, and a reduced number of bars are continued above this level.

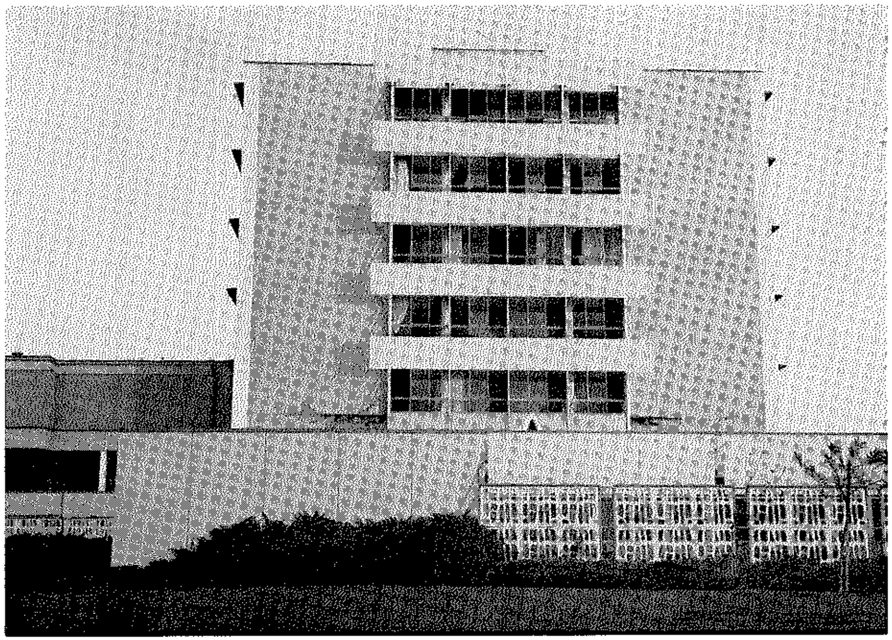


Figure 3.69 End walls at Holy Cross Hospital showing breaks at the third floor construction joints. Opposite view from Figure 3.67.

Figure 3.70 The third floor column between the end walls shown in Figure 3.69 was destroyed. Holy Cross Hospital.

at lower levels by one-, two- and three-story structural appendages non-symmetrically located around the building. While the reinforced concrete frame has some horizontal load-carrying capacity, it appears that lateral forces are carried primarily by the walls, exterior shear walls in the narrow dimension and reinforced elevator and stairwell walls in the long dimension. The foundations are cast-in-place concrete piles.

The nature of the wall cracking indicates that failures occurred in both horizontal directions with somewhat greater displacements in the narrow dimension of the building. The most spectacular failures developed in the end walls at the third and fourth floor levels where the lightweight concrete floor slabs intersected the end walls (Figs. 3.67, 3.68 and 3.69). At the northeast end of the building, the end with the exterior stairwell, this failure was so extensive in the construction joint at the fourth floor level that the wall and the corner column lost much of their vertical load-carrying capacity (Fig. 3.68). Judging from the buckled vertical reinforcing steel exposed in the wall, an inch or two of vertical settlement took place at the break. The vertical steel in the corner column was spliced above the fourth floor. This factor appears to have contributed to the large amount of concrete that spalled along the column. The lightweight concrete floor slab adjacent to the end wall damage was cracked and broken.

At the other end of the building (southwest end), both end walls were fractured at the construction joint where the lightweight concrete floor slab from the third floor intersects the end walls (Fig. 3.69). The breakage is greatest at points away from the corner columns. The two third floor end columns (positioned between the two end walls) apparently failed in shear; the most severely damaged of the two is pictured in Fig. 3.70. Excessive vertical loads in this column, a consequence of the adjacent end wall failure, probably contributed to the column failure. After failure the column lost essentially all of its vertical load-carrying capacity, and consequently, it was compressed about two inches. The third floor at this end of the building was badly fractured with one inch of vertical offset at some points. The continuation of the two end columns on the fourth story contained only minor concrete spalling near the structural connections at the floor and ceiling.

Other damage in the building varies considerably from floor to floor. No cracking was found in the basement structure nor in the underground utility tunnel walls. On the first floor one wall oriented in the long direction of the building was shattered over a 10-foot square area with cracks opened to 1/2 inch and large pieces of concrete protruding nearly an inch. In the same part of the building, a heavy steel autoclave was bulging two to three inches from its concrete housing. These features suggest that large relative displacements occurred at the first floor level.

All of the walls investigated on the second and third floor levels (end walls, elevator walls, and the stairwell walls) contained cracks generally angling at 45° to the vertical. At some points small chunks of concrete were dislodged, exposing the reinforcing steel (Fig. 3.71). Noticeably less wall cracking occurred on the fourth story, and essentially no wall cracking occurred in the fifth through the seventh stories. In fact, with the exception of slight cracking in the external columns, there was no evidence of structural damage above the fourth story. As pictured in Fig. 3.72, strong shaking at the seventh floor caused shelves and their contents to topple. This would have been a very dangerous place to be during the earthquake.

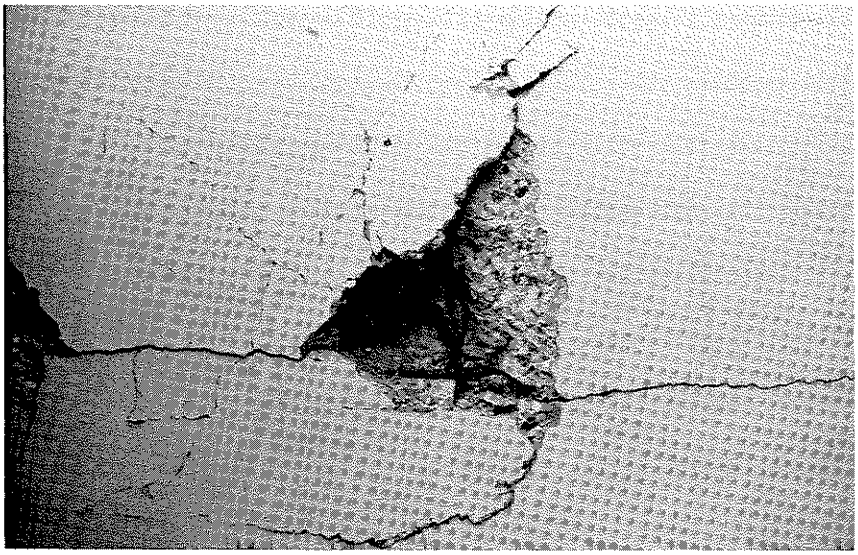


Figure 3.71 Most of the interior concrete walls in the first, second, and third stories of Holy Cross Hospital were cracked. At some points, where concrete spalled, the reinforcing steel was visible.

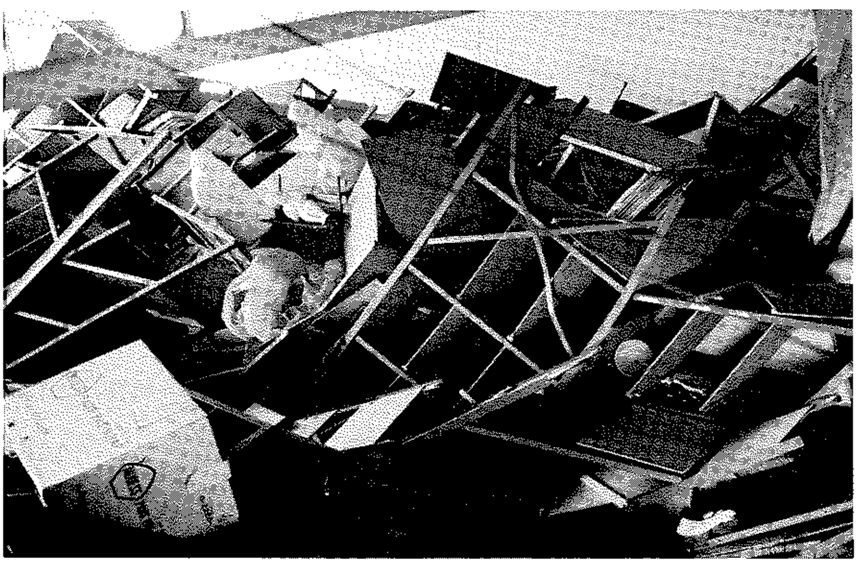


Figure 3.72 The storage shelves on the seventh floor of the Holy Cross Hospital were not properly secured. Potential hazards of this type can be avoided.

Some of the columns at the ends of the building and columns in a three level corridor appendage to the building failed in shear with cracks angling at 45° to the vertical (Figs. 3.70 and 3.73). It was the excessive deformations in the narrow direction of the building that caused these shear failures. Deformations in the long direction of the building resulted in column damage of a different type. The exterior columns contained longitudinal and angled meandering cracks away from the floor connections (Fig. 3.74). This type of cracking is often associated with a torsional failure. One possible explanation is that the in-plane stiffness provided by the exterior wall panels caused torsional deformations in the columns during longitudinal motions of the building. The torsional loading would be induced in the columns by the nonsymmetric (one-sided) connection between the columns and the panels. This conjecture is supported to some degree by the presence of small amounts of concrete spalling at the connections between the columns and the panels, which indicates a large transfer of load at these points.

Indian Hills Medical Center

The $1.8 million Indian Hills Medical Center is a new six-story office building located on a site adjacent to the Holy Cross Hospital on Rinaldi Street between the Golden State and San Diego freeways (Figs. 3.64 and 1.2). Two views of the building are shown in Figs. 3.77 and 3.78. Sections of the building had been occupied for about one year, but some of the interior finishing in the upper stories was incomplete at the time of the earthquake.

The site is three miles south of Olive View Hospital. It is estimated that the peak ground acceleration at this site was in the range of 30%g to 40%g. There is no evidence of permanent ground displacements near the building and the damage to the structure appears to be entirely due to strong ground shaking.

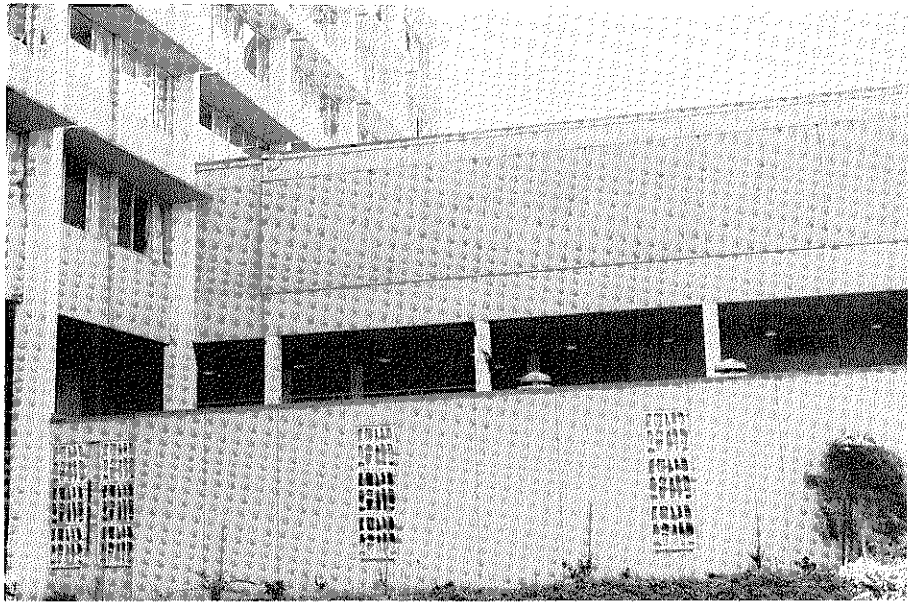


Figure 3.73 Three-story corridor annex to the Holy Cross Hospital. Column damage is present in both the annex and the hospital building.

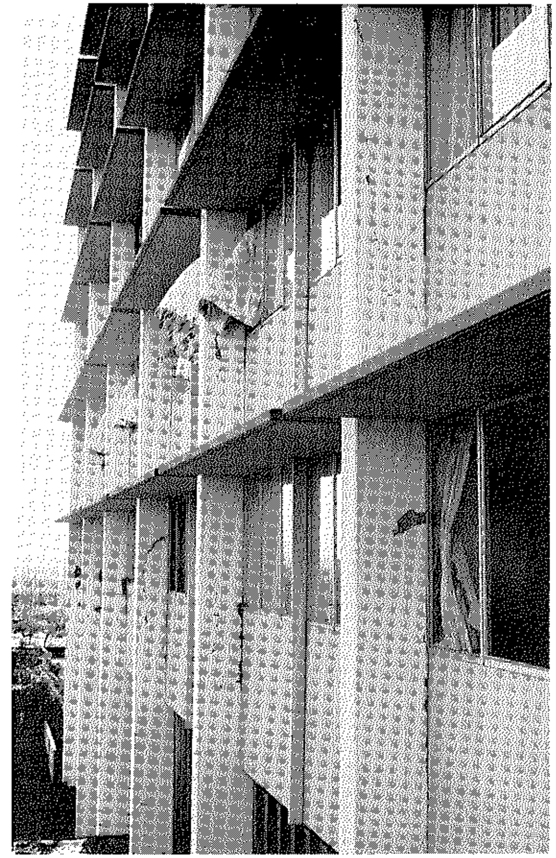


Figure 3.74 Exterior columns of Holy Cross Hospital with meandering cracks at points away from the floor connections.

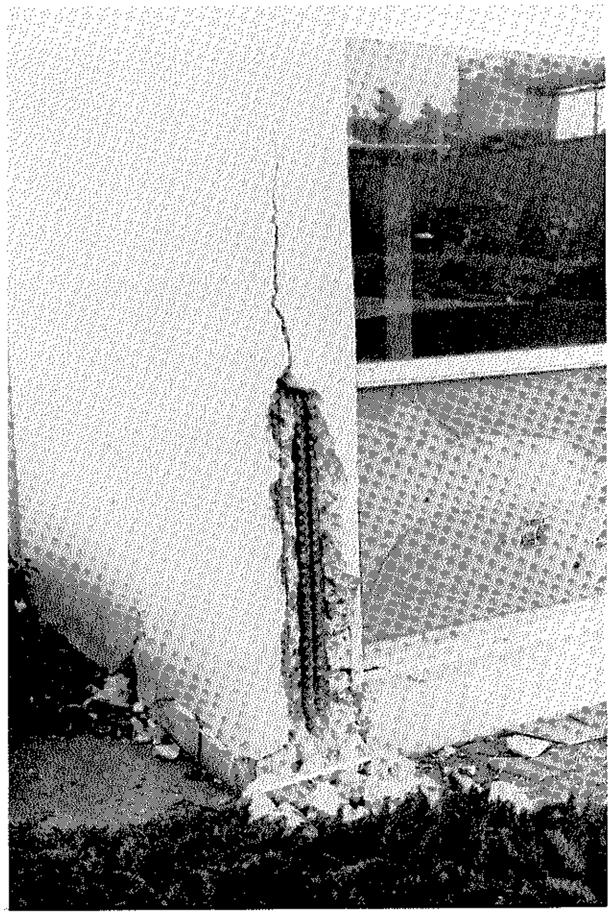


Figure 3.75 Fractured shear wall. Holy Cross Hospital.

Figure 3.76 Fractured spandrel beams on the north side. Holy Cross Hospital.

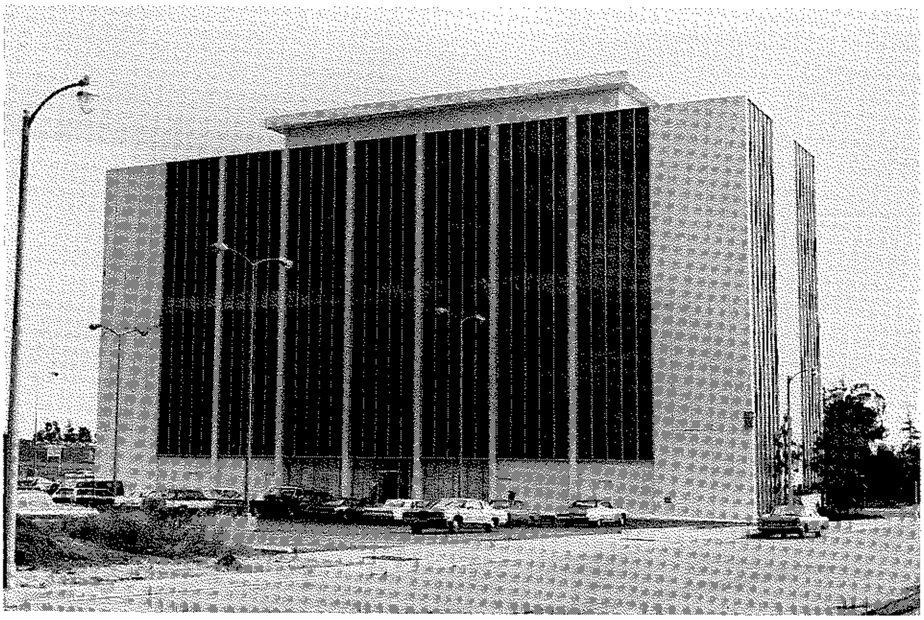


Figure 3.77 North elevation of the Indian Hills Medical Center as seen the day after the earthquake.

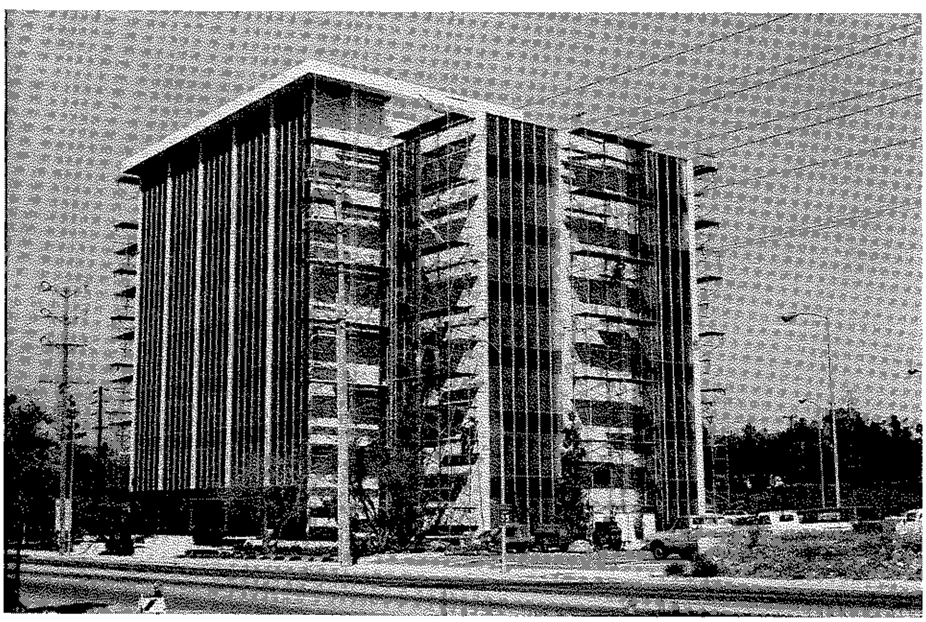
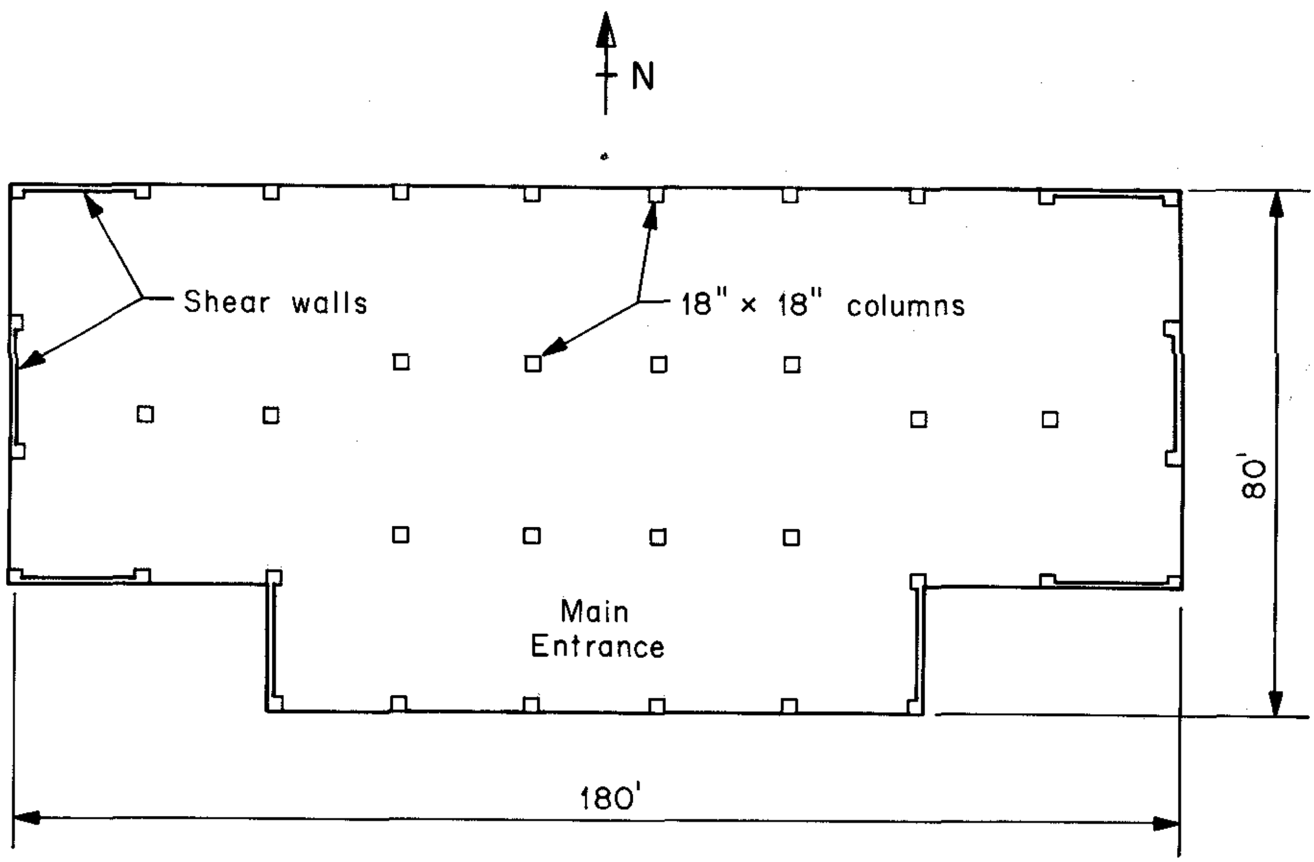


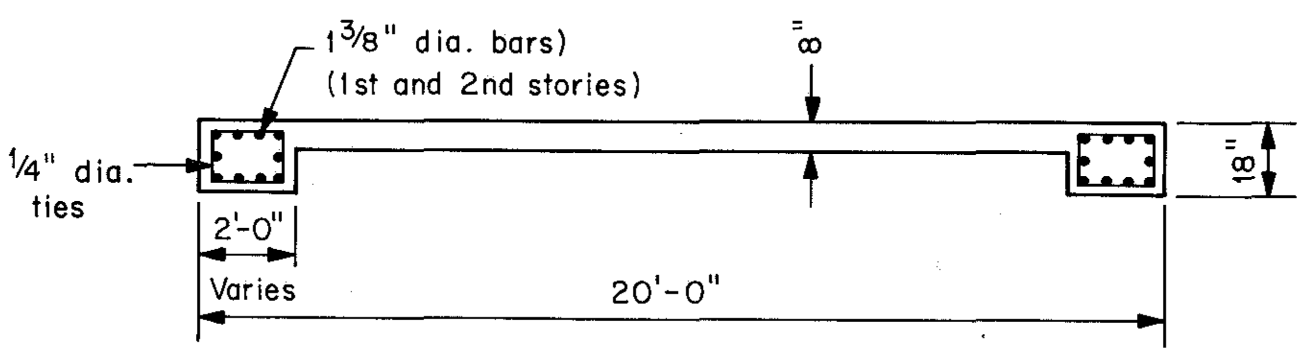
Figure 3.78 Southeast corner of the Indian Hills Medical Center. Repair operations six weeks after the earthquake.

Lateral loads are resisted by eight external shear walls (vertical cantilever beams) arranged to provide equal and symmetrical resistance along both major axes of the building. A reinforced concrete frame carries the vertical loads and provides additional lateral load resistance of greater flexibility. Approximate dimensions and the arrangement of the main structural members are shown in Fig. 3.79. The shear walls and columns are constructed of normal-weight concrete. The floor slabs and beams are made of lightweight concrete. At each floor level the lightweight slabs were poured across the entire cross-section of the walls. The foundations are concrete piles.

Major structural damage was confined to the shear walls which exhibited two distinct types of overstressing. One damage type was characterized by horizontal cracking and partial failure along the construction joints between the lightweight concrete floor slabs and the walls. The horizontal cracking was visible on all the walls and at most floor levels, although the cracks were most pronounced at the second, third and fourth floor levels. (Possible damage at the first floor level was partially obscured from view by backfill and pavements). Permanent displacements in excess of 1/4 inch were visible along the horizontal cracks above the floor slab. Typical horizontal cracking and spalling of the concrete around the main vertical reinforcement in the wall is shown in Figs. 3.80, 3.81 and 3.82. During repair operations concrete was removed from the spalled areas (Fig. 3.83) revealing some of the reinforcement details given in Fig. 3.79. It appears that the main vertical wall reinforcement was lapped above each floor level. Because of the weakness of the construction joint formed against the lightweight



PLAN



TYPICAL CROSS-SECTION OF SHEAR WALLS

Figure 3.79 Indian Hills Medical Center, approximate dimensions.

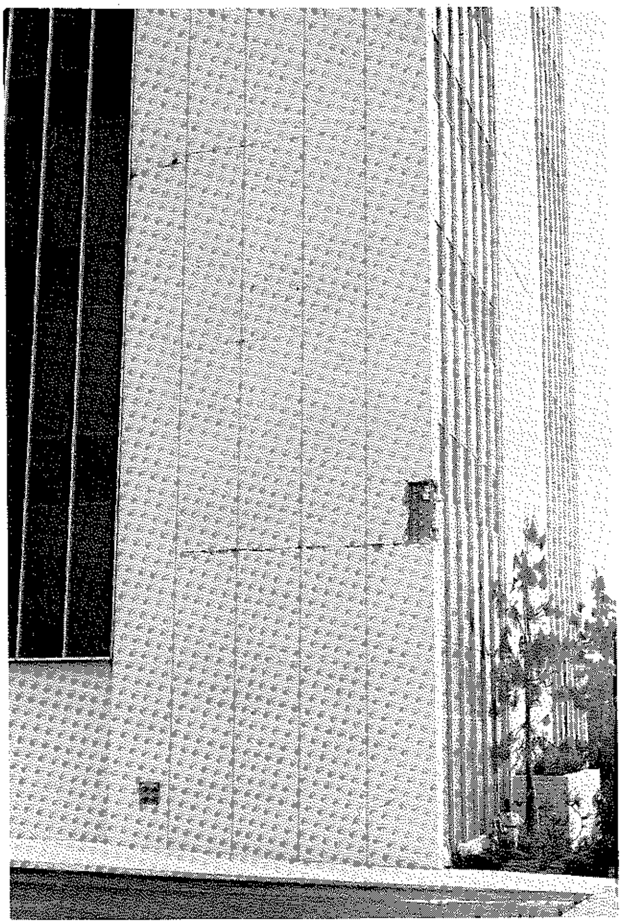


Figure 3.80 Cracking and spalling on western wall, north elevation of Indian Hills Medical Center.

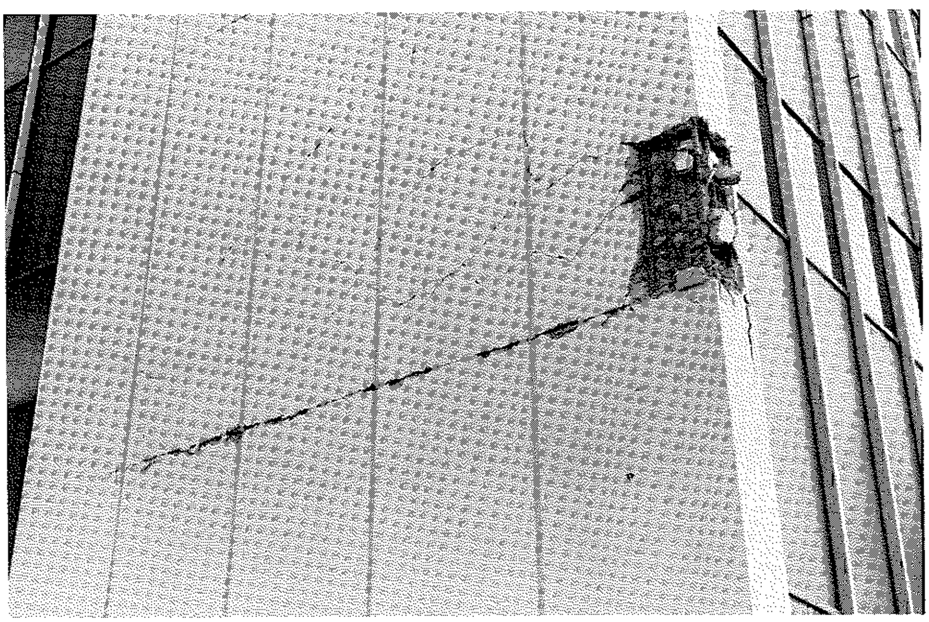


Figure 3.81 Closer view of spalled area shown in Figure 3.80. Indian Hills Medical Center.

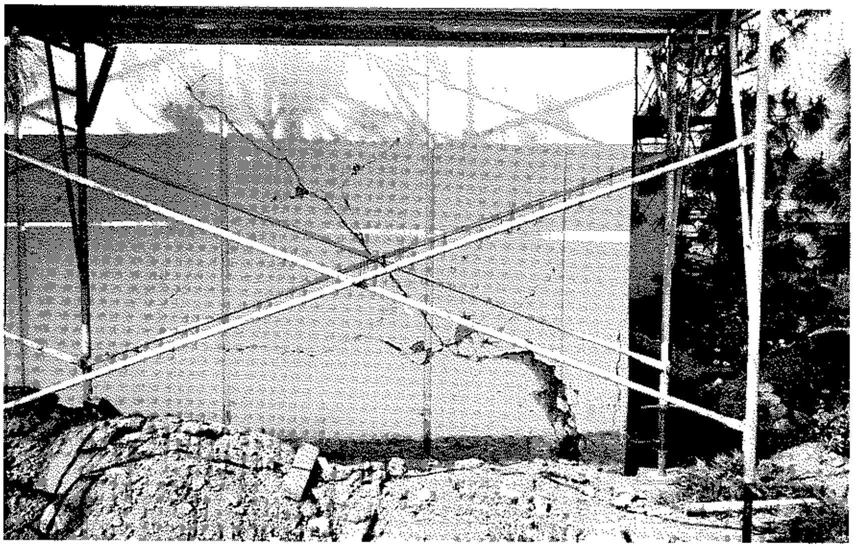


Figure 3.82 Cracking and spalling at the base of the eastern wall, south elevation of Indian Hills Medical Center.

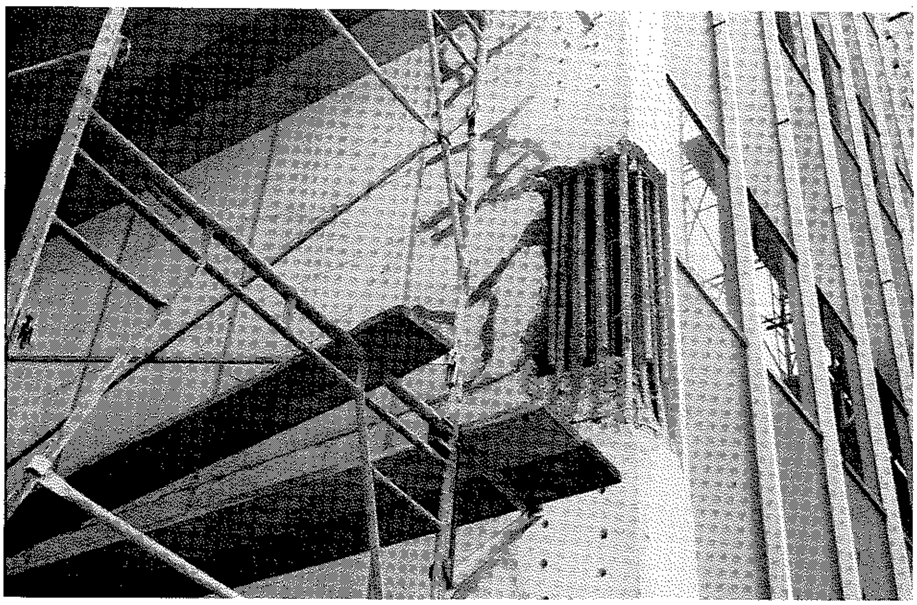


Figure 3.83 Vertical wall reinforcement exposed during repair operations. Same view as shown in Figure 3.81. Indian Hills Medical Center.

concrete slab, the wall shear at the joints was apparently resisted primarily by dowel action of the vertical steel in the wall. The spalling of the concrete was probably an indication of the onset of dowel and bond failures.

The second type of wall damage was the formation of 45° diagonal tension cracking over the full height of all the walls. The cracking was most pronounced in the lower stories, and some cracks were opened to a width of more than 1/16 inch. Significant crack patterns were visible in the upper stories which probably indicated a reduced design strength had been used in the upper wall sections. The pattern of cracking is illustrated in Figs. 3.80 and 3.81. No evidence of flexural cracking or failure was detected.

The reinforced concrete frame suffered moderate damage in the upper stories as can be seen in illustrations Figs. 3.84 and 3.85. Reinforcement had been exposed by spalling at the beam-column connections and the floor slabs were cracked. Slab cracking was pronounced in regions close to some of the shear walls. In the lower stories the interior finish prevented full inspection of the frame, but there was no evidence of damage to the external columns below the fourth floor level. The external glass curtain walls were not damaged.

Repair operations commenced on the building a few days after the earthquake. Cracks in some of the walls and frame were filled with epoxy and spalled areas of concrete were removed and recast. The epoxy treatment was then abandoned because of the difficulty in getting good penetration and the walls were instead strengthened by the addition of new reinforced gunite concrete walls on the exterior of the existing walls. Holes drilled for the purpose of making a connection to the new walls can be seen in Fig. 3.83. The extent of cracking in the shear walls is shown in Figs. 3.86 and 3.87. This more or less uniform distribution of cracks in all of the shear walls was very striking and it indicates that the design provided almost an optimum

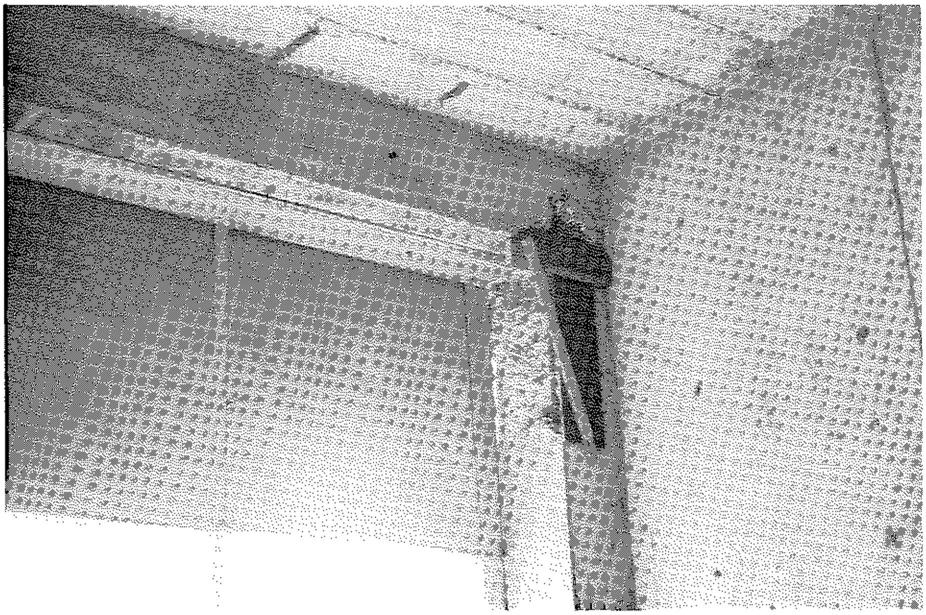


Figure 3.84 Spalling at connection between lightweight concrete beam and normal concrete column at the fourth story, Indian Hills Medical Center.

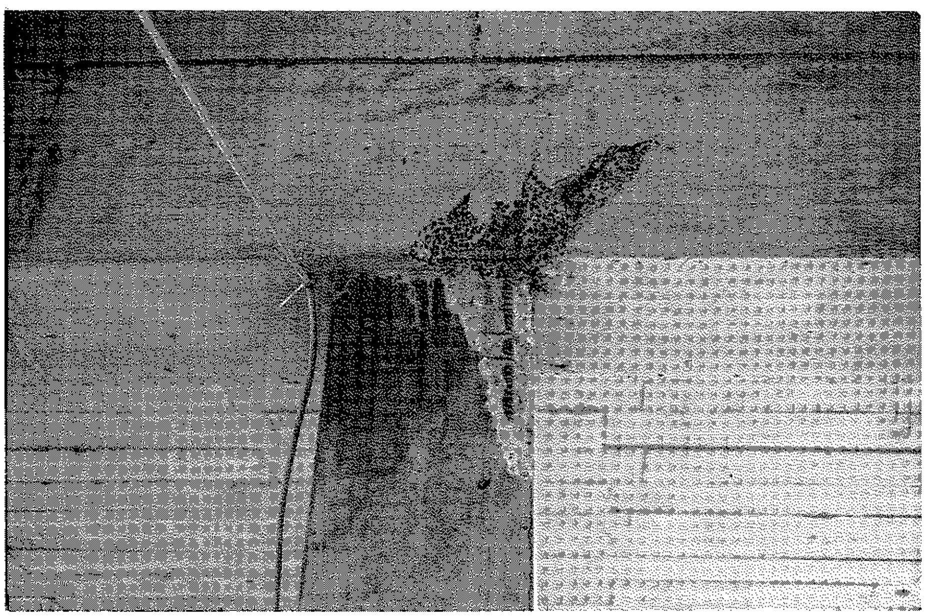


Figure 3.85 Spalling at beam-wall connection at the fifth story, Indian Hills Medical Center.

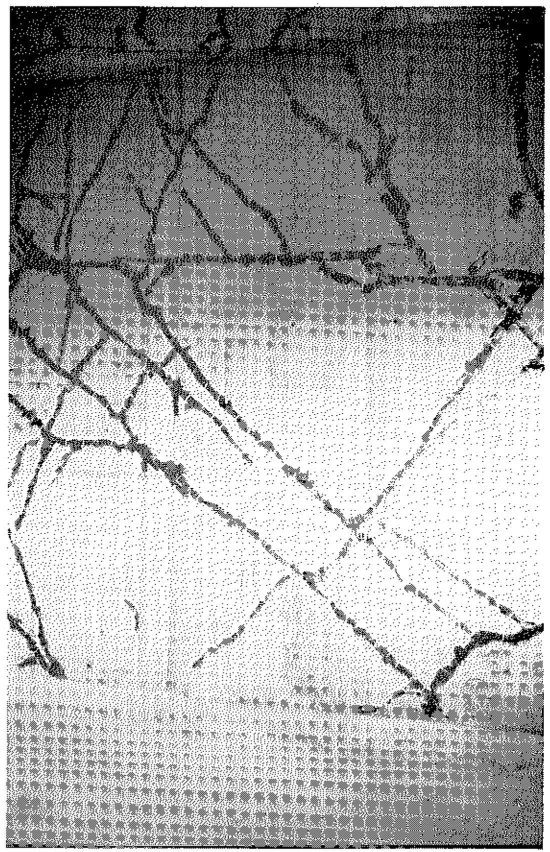


Figure 3.86 Interior view of shear wall with cracks made visible by the epoxy. Indian Hills Medical Center.

Figure 3.87 Exterior shear wall being repaired with epoxy. The epoxy makes the cracks visible and shows how badly cracked the walls were. Indian Hills Medical Center.

distribution of resistance. The Indian Hills Medical Center building demonstrates that the minimum building code requirements for this type of structure lead to extensive damage when the ground motion has a peak acceleration in the range of 30%g to 40%g. The damage to this building and to the Holy Cross Hospital building would, no doubt, have become much more severe if the duration of the strong shaking had been significantly longer, and partial collapse might well have ensued. These buildings provide important data on the adequacy of the minimum requirements of the building code.

The San Fernando Juvenile Facility

The San Fernando Juvenile Facility is located just east of the intersection of the Southern Pacific Railroad tracks and the Golden State Freeway. This new $6.5 million facility has a plan shape in the form of an irregular pentagon surrounding an interior court (Figs. 3.88 and 3.89). Most of the facility is constructed of hollow concrete block with reinforcing bars grouted into the cells of the blocks, and with reinforced concrete floors and roofs. Some of the structures had reinforced concrete columns and beams.

Damage to the facility was caused by gross permanent movements of the ground from landslides as well as by shaking of the ground. The most severe damage was caused by differential ground displacements as seen in Figs. 3.90 - 3.97. Cracks and deformation of the asphalt slab in the parking lot at the front of the facility made clear that the ground beneath had undergone appreciable permanent displacements. Within the courtyard there was also much evidence of ground displacement which, in numerous instances, caused severe damage to the structures.

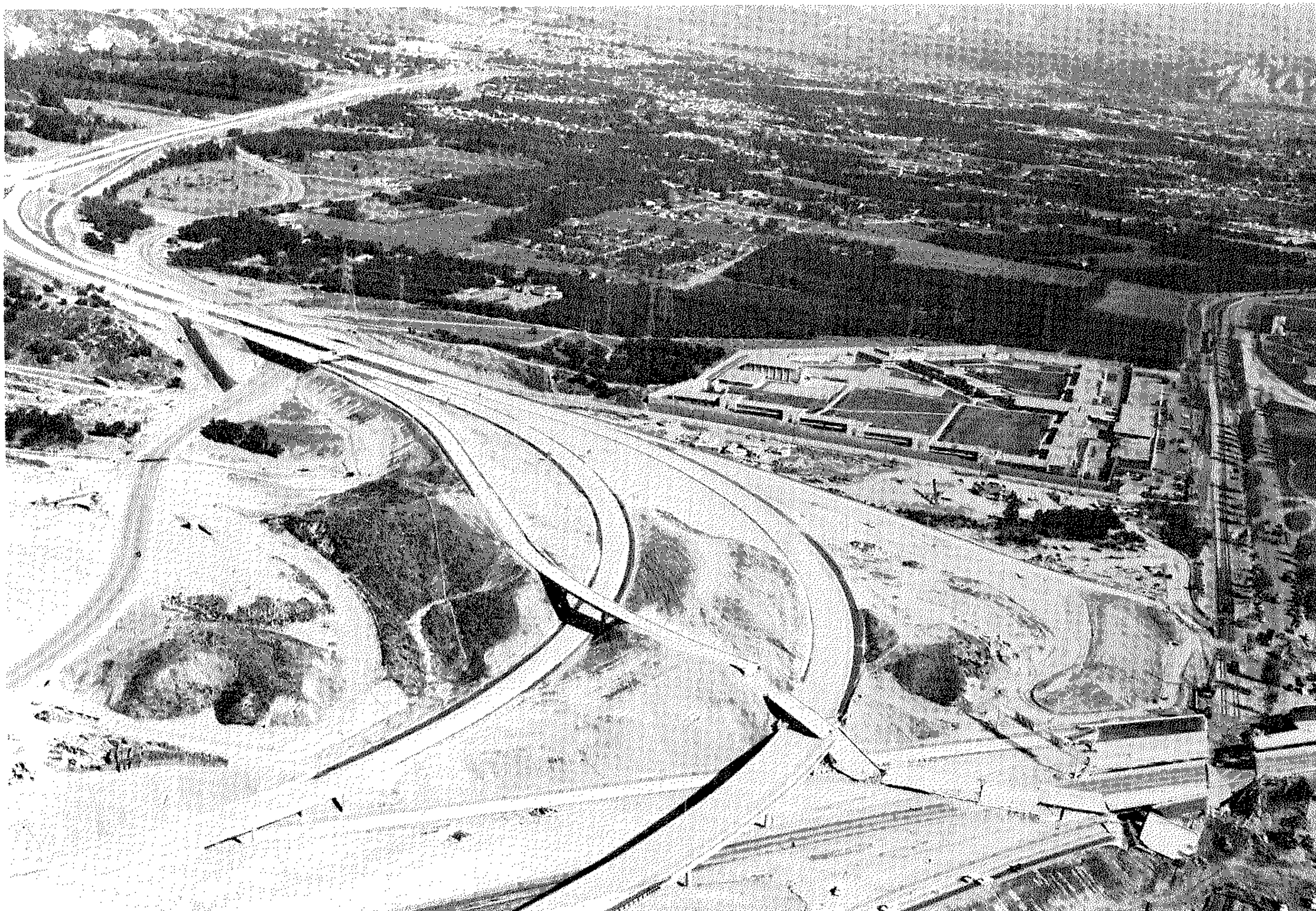


Figure 3.88 View looking east over the San Fernando Juvenile Facility and the Golden State-Foothill freeway interchange. Olive View Hospital is in the left background.

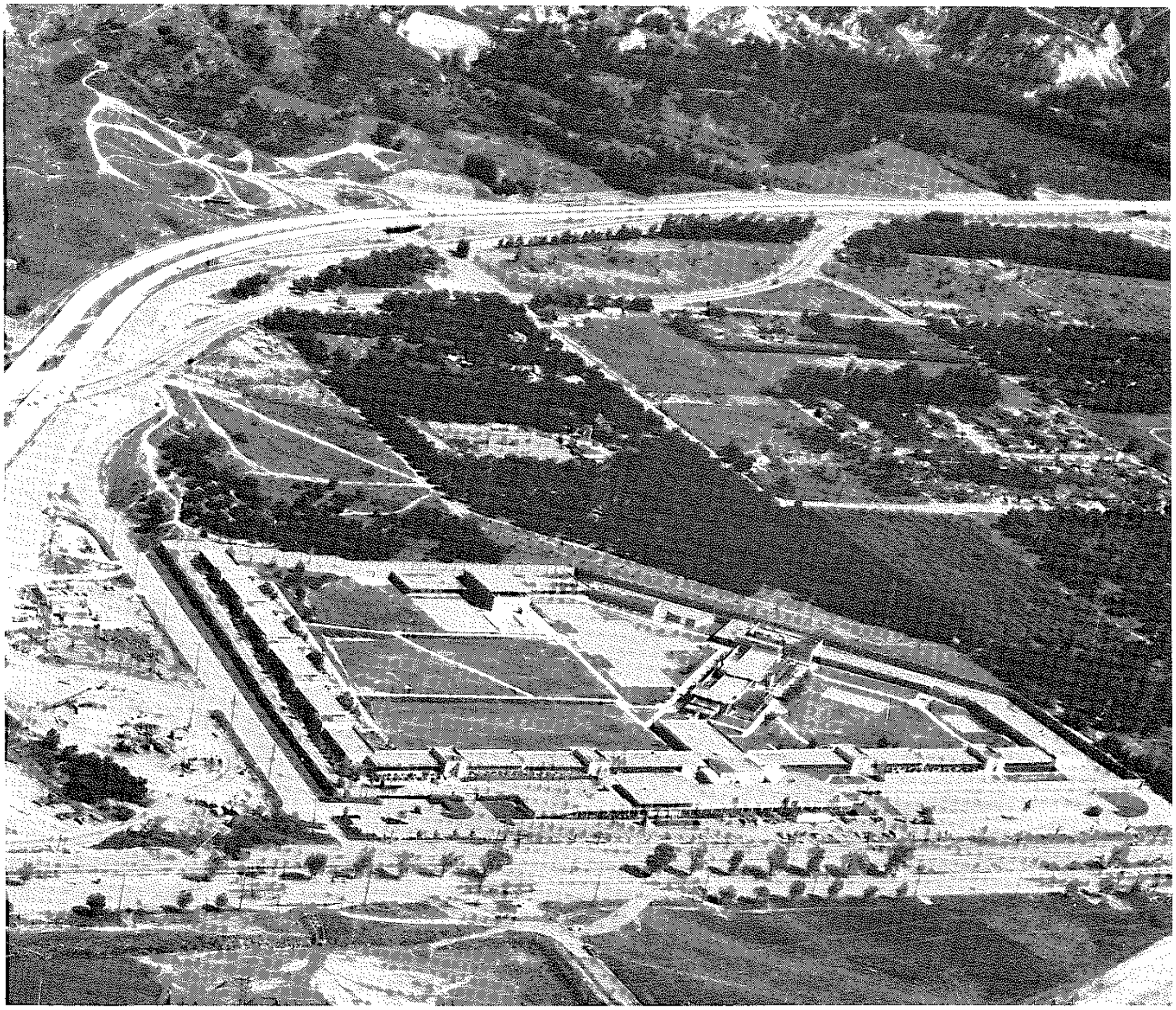


Figure 3.89 San Fernando Juvenile Facility. The collapsed wing is in the foreground; the taller gymnasium structure is at the rear. The Southern Pacific Railroad line passes just in front of the facility. Foothill Blvd. and Foothill Freeway are in the background.

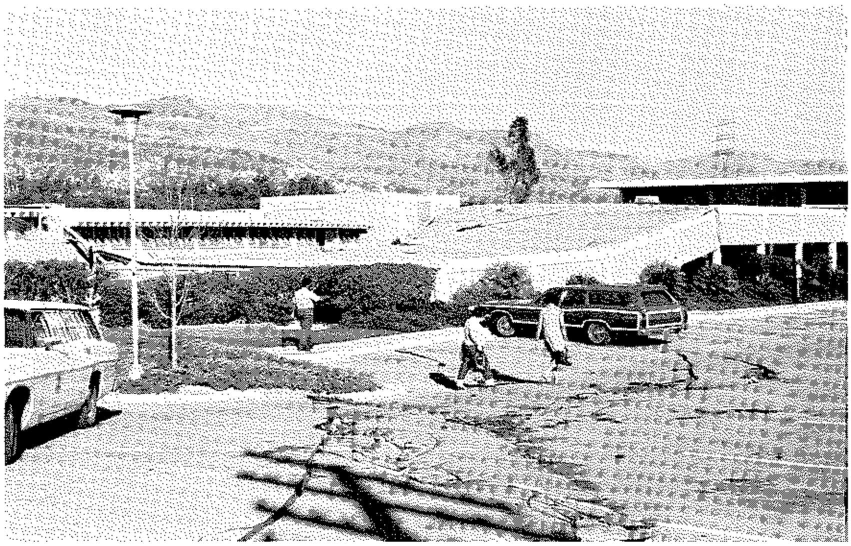


Figure 3. 90 View of cracking and change in elevation of asphalt parking area; collapsed wing of concrete frame Administration Building in the background. San Fernando Juvenile Facility.

Figure 3. 91 Evidences of ground movement in central parking area of the San Fernando Juvenile Facility.

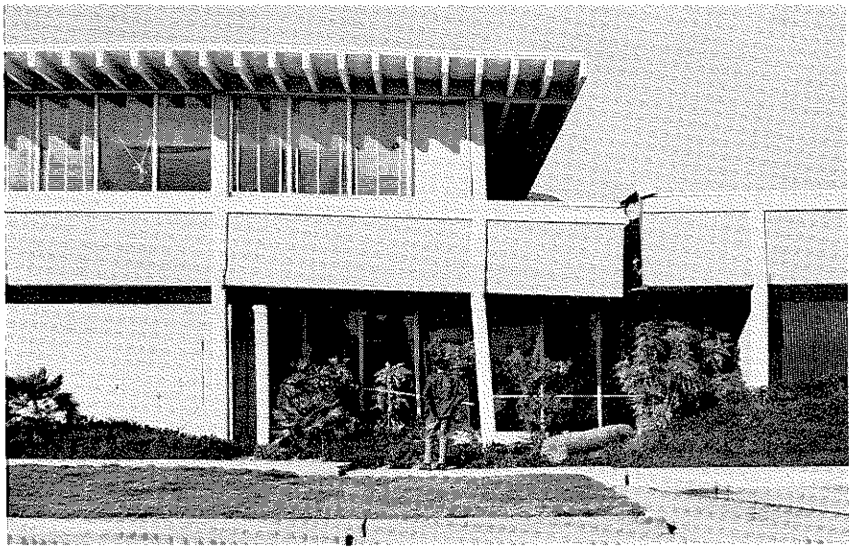


Figure 3.92 View showing ground displacements, reinforced concrete column failure, and opening of separation joint. San Fernando Juvenile Facility.

Figure 3.93 Interior courtyard looking west past collapsed wing of main building. Numerous cracks are visible in the asphalt parking area. San Fernando Juvenile Facility.

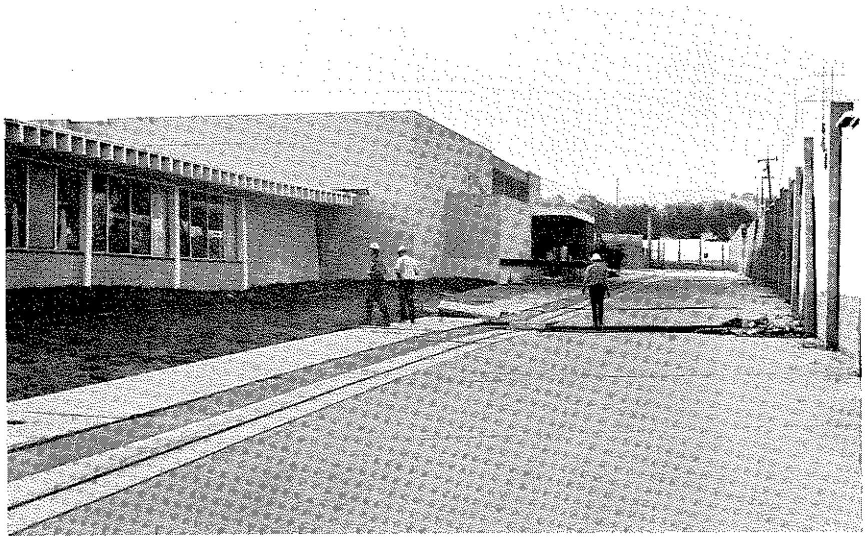


Figure 3.94 Ground displacement and structural damage near northern boundary of San Fernando Juvenile Facility.

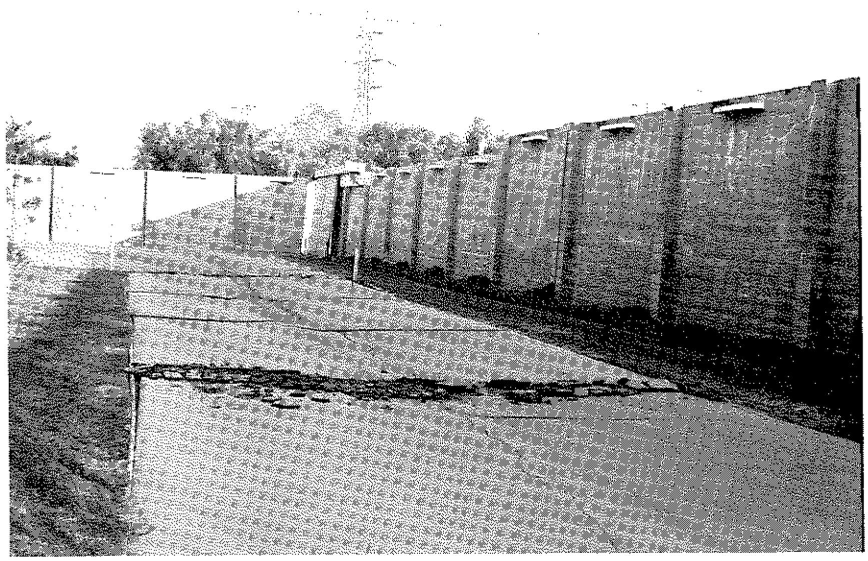


Figure 3.95 Ground displacement near northeast corner of San Fernando Juvenile Facility. The walls are concrete block construction.

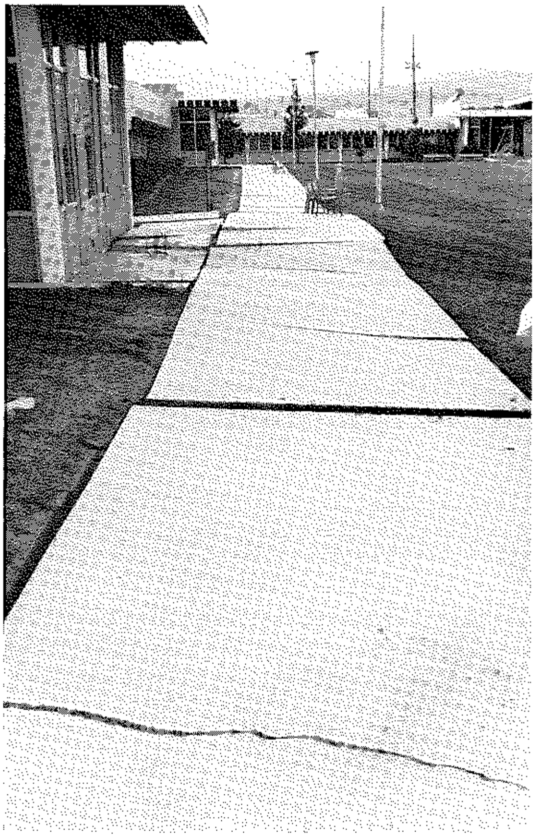


Figure 3.96 Sidewalk along western end of San Fernando Juvenile Facility.

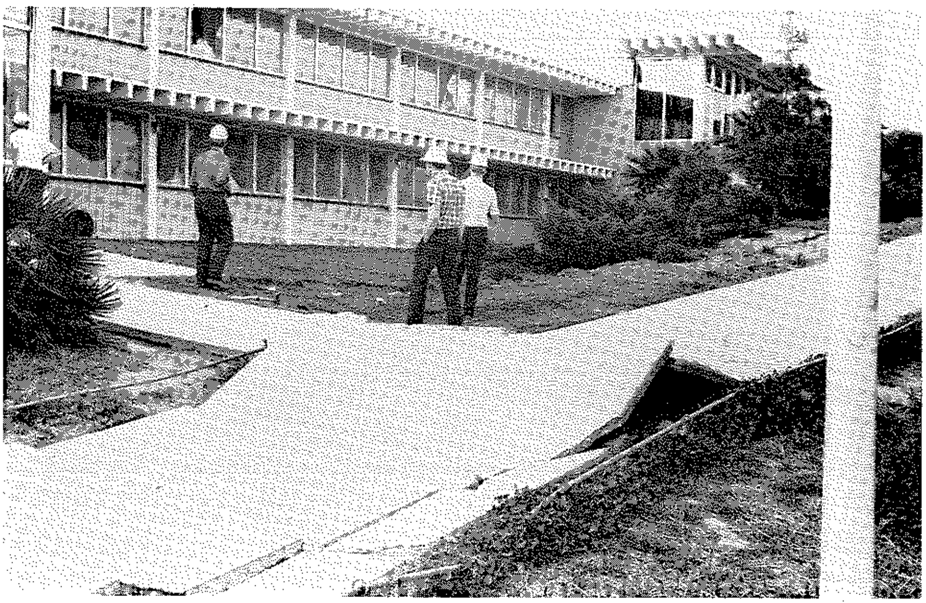


Figure 3.97 Buckled sidewalk at San Fernando Juvenile Facility.

The evidences of ground movement within and outside of the Juvenile Facility have been attributed to a sliding and subsidence of the ground. The Southern Pacific Railroad tracks just south of the facility were badly distorted by permanent ground displacement. There was further evidence of ground movement over an area extending in a southwesterly direction through and past the Pacific Intertie to the upper Van Norman Reservoir. It has been suggested by some that this elongated area of gross ground displacement may be a reflection of faulting in the underlying rock. There is evidence of damage due to ground shaking so that even if there had been no permanent ground movement, the facility would still have received damage, but it would not have been as severe.

Structural damage from permanent ground movements and from ground shaking is illustrated by Figs. 3.98 - 3.105. The school gymnasium, designed and checked according to the Field Act, suffered damage to the roof. The cross-bracing rods between the steel roof trusses failed and the window glass and framing fell (Figs. 3.102, 3.103 and 3.104). The concrete block steam plant building was severely damaged (Fig. 3.105); the peculiar nature of the damage appears to be the consequence of having only vertical reinforcing bars.

The Juvenile Facility was so severely damaged that it probably will be razed.

Pacoima Memorial Lutheran Hospital

The Pacoima Lutheran Hospital located on Eldridge Avenue (Fig. 1.2), was built in 1959. The location is just north of Hansen dam and about one-half mile from the main trace of surface faulting.

The hospital consists of a three-story unit (with a basement level), approximately 10 years old, containing 110 beds and a new two-story mental

Figure 3.98 Damage to reinforced concrete block wall at the San Fernando Juvenile Facility. See also Figure 3.99.

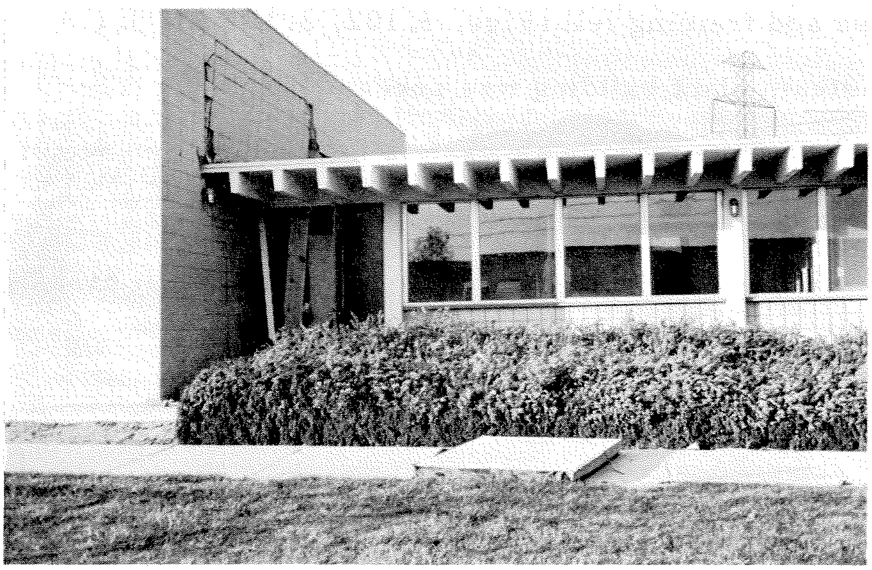


Figure 3.99 Damage to concrete block wall caused by pounding of adjacent roof. San Fernando Juvenile Facility.

Figure 3.100 Fallen debris from concrete-block wall. San Fernando Juvenile Facility.

Figure 3.101 Damage to concrete-block wall. San Fernando Juvenile Facility.

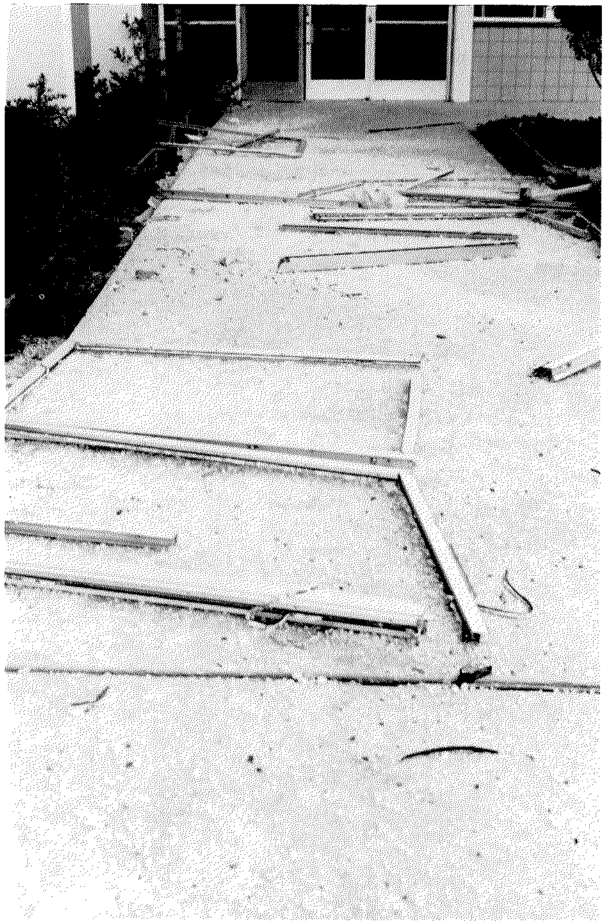


Figure 3. 102 Window frames and shattered safety glass from gymnasium skylight (see following photograph). San Fernando Juvenile Facility.

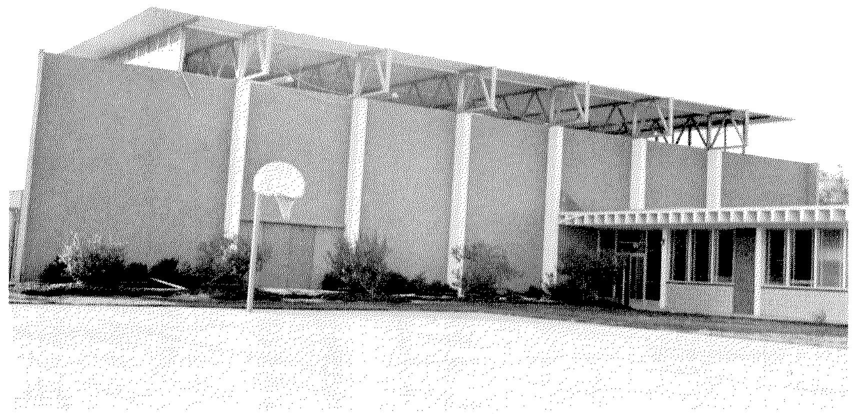


Figure 3. 103 Gymnasium building. Cross-bracing rods between trusses were broken and safety glass window panes were shattered. (See preceding photo). San Fernando Juvenile Facility.

Figure 3.104 Steel roof trusses in gymnasium building broke the cross-bracing rods over the walls. Shattered safety glass covered the floor. San Fernando Juvenile Facility.

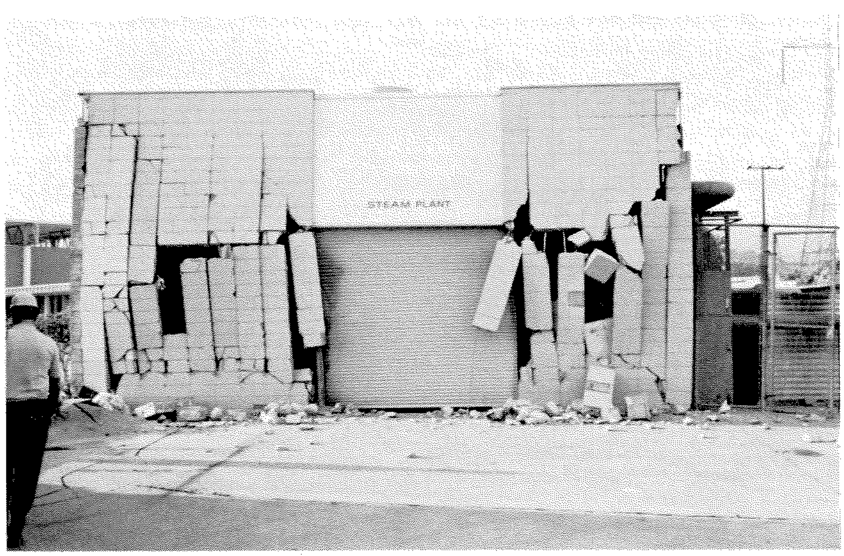


Figure 3.105 Walls of steam plant building at San Fernando Juvenile Facility were shattered.

health unit housing 28 beds and administrative offices. A single-story appendage is located at the northern end of the three-story unit. The three-story unit is constructed of reinforced concrete; the floor and roof are concrete pan joists and the foundations are supported by 40-ft concrete piles. The two-story unit is of reinforced masonry.

The three-story building received severe structural damage and the patients were evacuated shortly after the main shock. The extent of the damage may be serious enough to require demolition of the building. The two-story wing received only minor damage of a nonstructural nature, and is still in operation. The interior finish of a two-story steel frame medical center building adjacent to the hospital wings sustained heavy damage. Damage to the exterior of this building appeared to be minor and there was no indication of structural damage.

The major lateral load resisting elements of the three-story wing can be seen in Figs. 3.106 and 3.107. In the short direction, lateral loads are resisted by the shear walls at the ends of the structure. The building is rigidly connected at the eastern end to an elevator and stairwell tower which resists most of the lateral load in the long direction. The shear walls and the elevator tower were extensively damaged by 45° cracking. Illustrations, Figs. 3.108 - 3.111 show the extent of some of the structural damage.

The three-story wing is founded on a basement level and backfill has been placed along the northern face of the wing. The backfill settled 9 in., damaging the connecting hallway from the two-story building. The extent of the settlement can be seen in Fig. 3.108.

The internal plaster lining of the three-story wing was extensively damaged. Details of the internal framework of the building are not known, but the nature of the internal damage suggests that the connections between the external concrete walls and the interior frame failed.

Figure 3.106 Three-story wing. Pacoima Lutheran Hospital, looking east.

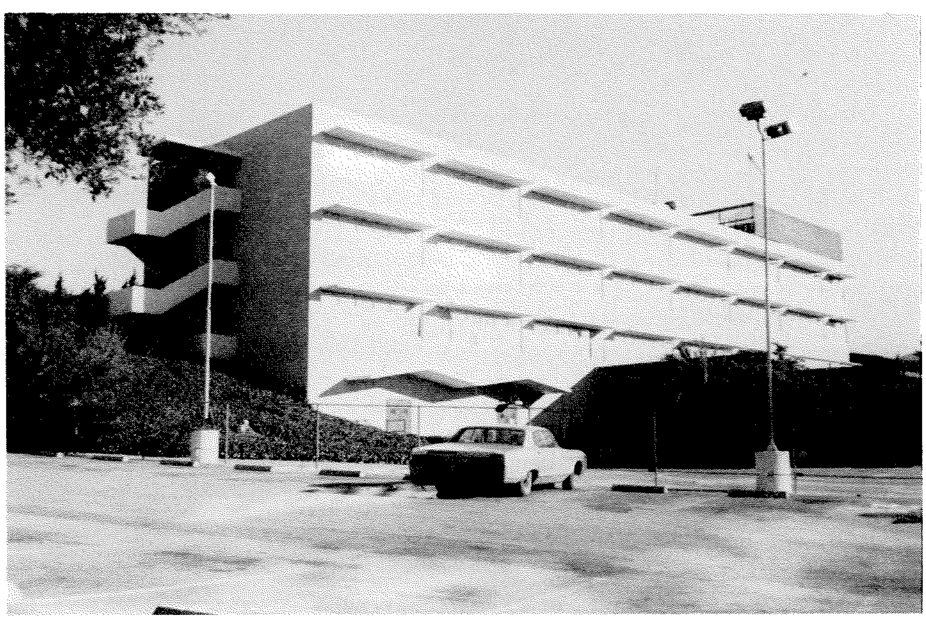


Figure 3.107 Three-story wing, Pacoima Lutheran Hospital, looking north.

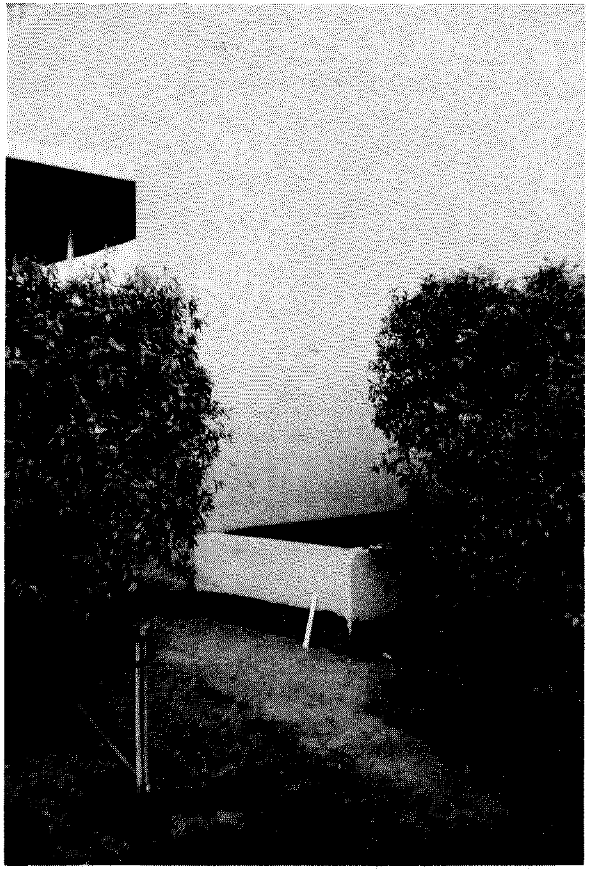


Figure 3.108 Cracking in west shear wall and settlement of backfill. Three-story wing of the Pacoima Lutheran Hospital.

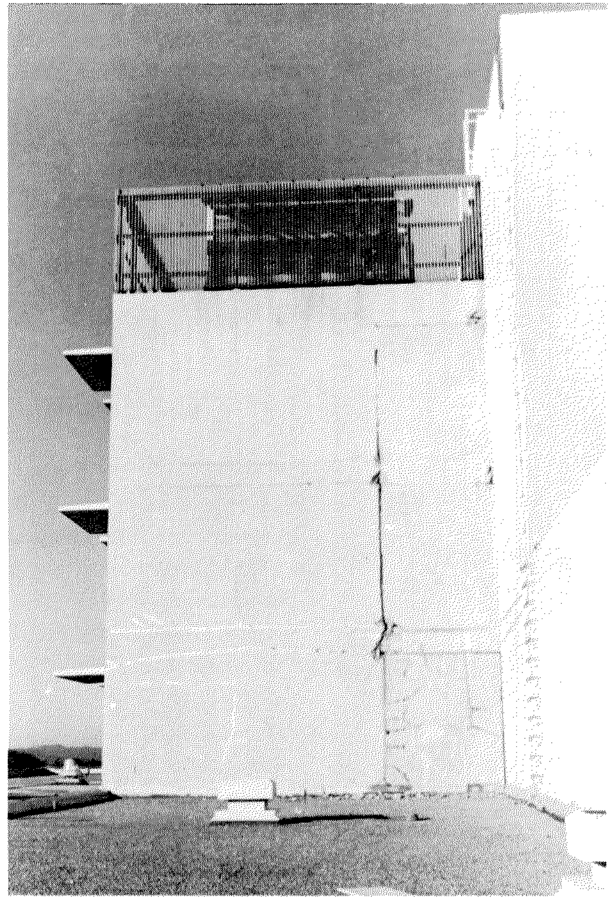


Figure 3.109 Cracking and spalling in the east shear wall of the three-story wing, Pacoima Lutheran Hospital.

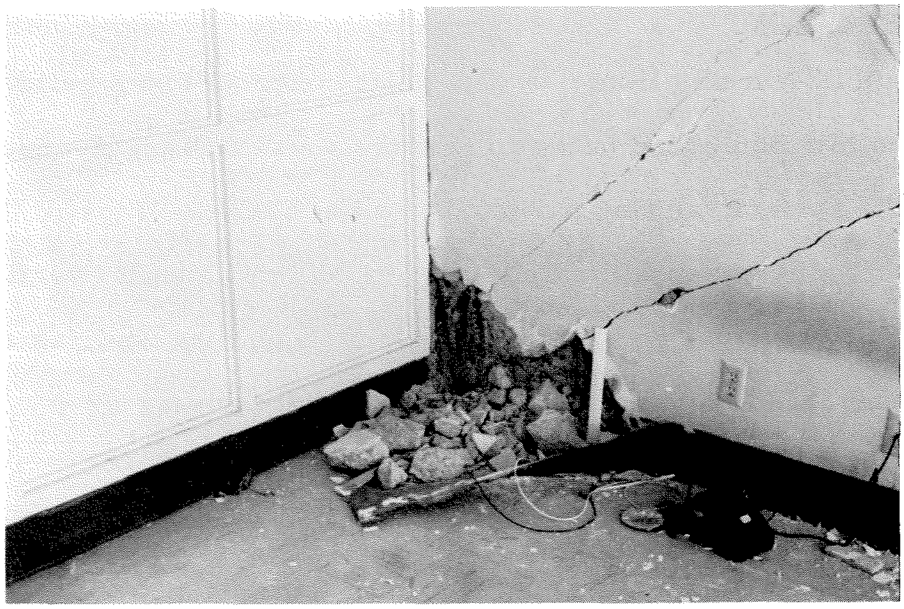


Figure 3.110 Interior view of the eastern shear wall of the three-story wing, Pacoima Lutheran Hospital.

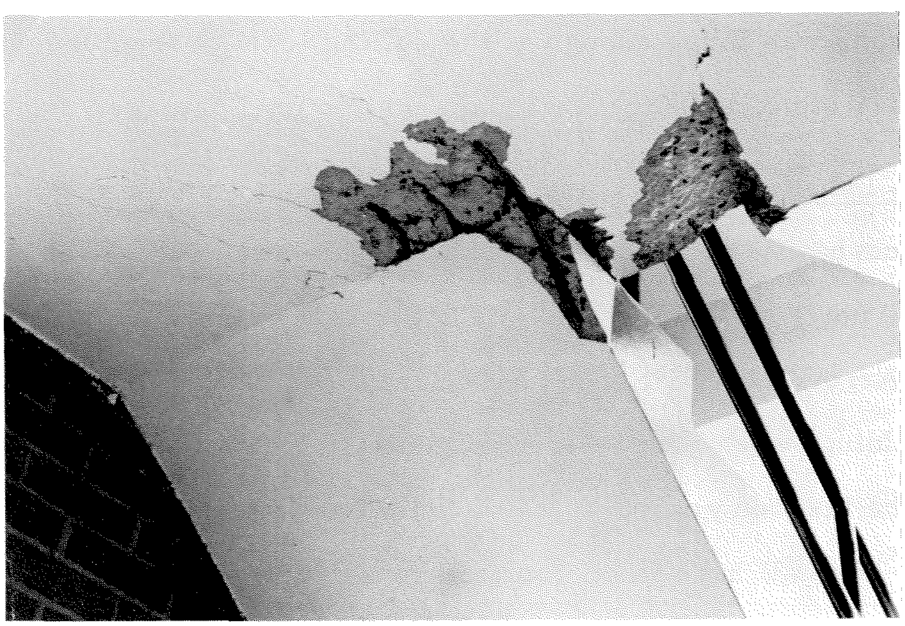


Figure 3.111 Stair well of three-story wing, Pacoima Lutheran Hospital.

Foothill Nursing Home

The Foothill Nursing Home, a one-story concrete block building, was located on Foothill Boulevard between MacKay and Van Nuys Avenues (Fig. 1. 2). A branch of the surface faulting passed under the building (Figs. 1. 5 and 1. 6) and subjected it to approximately 20 inches of differential vertical displacement. The structure was severely distorted and cracked, as shown in Figs. 3. 112 and 3. 113, and presumably must be abandoned.

Bendix Building

The largest steel-frame structure in the area of strongest shaking was the Bendix building off Roxford Avenue in Sylmar. The building can be seen in Fig. 8. 1; it is the large, white, squarish structure south of the eastern edge of the San Fernando Juvenile Facility. It is a high-bay, one-story commercial structure used for light manufacturing. The structural frame of the building was undamaged, but displacement of the frame at the interior columns was large enough at the ceiling level to fracture water pipes and cause other nonstructural and architectural damage.

San Fernando Industrial Park

Over one-half of a group of about 30 single-story industrial buildings located on the northern side of Foothill Boulevard in the vicinity of the intersection with Arroyo Street (Fig. 1. 2) suffered serious structural damage. All of the buildings are of modern construction and are thought to have been built within the last ten years. Typical buildings in the group are shown in Fig. 3. 114. They all have wood roofs and most roofs have plywood sheathing. Approximately 18 buildings were of tilt-up wall slab construction, and of these, 11 received serious structural damage. Six of the twelve concrete block buildings were damaged and one of the two brick buildings suffered a

Figure 3.112 Ground displacements caused by faulting, Foothill Nursing Home.

Figure 3.113 Structural damage to block walls, Foothill Nursing Home.

Figure 3.114 San Fernando Industrial Park. Omneco and Cosmic Plastic Buildings in the foreground. Permanent ground displacements have forced these buildings together causing serious structural damage.

Figure 3.115 Permanent ground displacements show in the sidewalk and parking lot. Donaldson Building.

serious failure. However, as discussed below, the degree of damage appeared to be more related to large permanent ground displacements rather than to the type of construction.

The construction of these buildings is a highly competitive undertaking with strong pressure to keep the cost low and, hence, the tendency is to just satisfy the requirements of the building code. It appears that buildings of this type can withstand moderately strong ground shaking without experiencing much damage, and it is probably not justified to spend an appreciable amount of money to increase their earthquake resistance just to protect the capital investment. However, the damage that occurred would have been extremely hazardous in many cases had the buildings been occupied, and an increase in the earthquake resistance of these structures seems justified by safety considerations alone. It appears that a significant increase in earthquake resistance can be obtained, at little extra cost, by improving the method of connecting the tilt-up panels together, and by improving the connection between the roof and the top of the walls.

The Industrial Park was located in the region of strong ground motion and all of the buildings were within 1/4-mile of a surface expression of the main fault trace (Figs. 1.5 and 1.6). A number of buildings received damage that was obviously related to permanent ground displacements and, in particular, the buildings closest to the foot of the ridge, which extends southward from the San Gabriel mountains to Foothill Boulevard, were most affected by the faulting. The eastern corner of the Donaldson building shown in Figs. 3.115 and 3.116 was directly over a fault trace and suffered extensive damage from a vertical differential displacement of approximately one foot. Two buildings, the Cosmic Plastics building and the Omneco

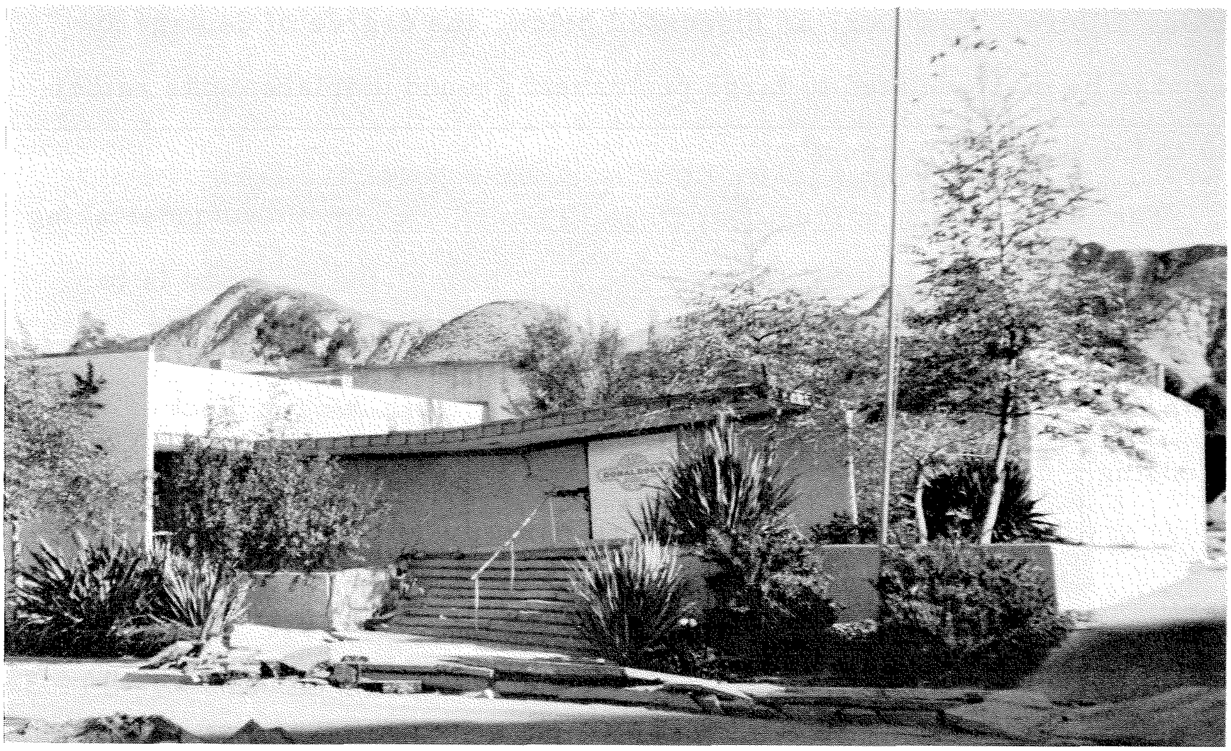


Figure 3. 116 Structural damage resulting from permanent ground displacements. Donaldson Building.

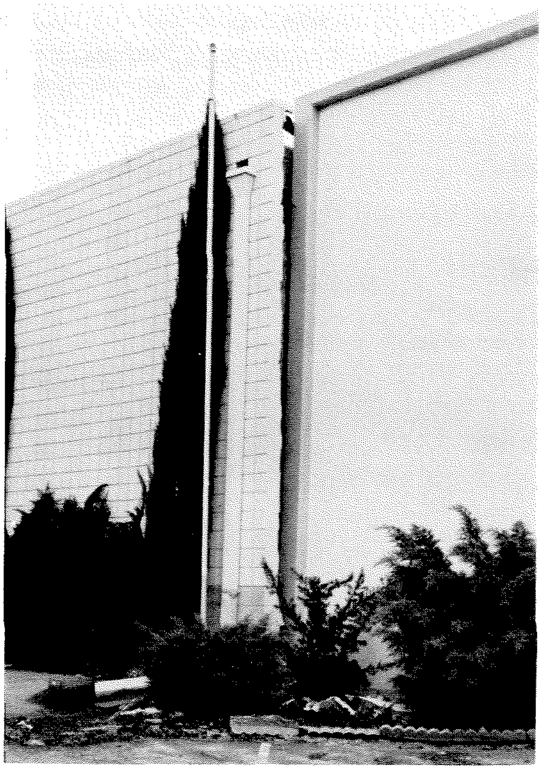


Figure 3. 117 Permanent ground displacements caused a collision between the Omneco and Cosmic Plastics buildings.

building, which were built adjoining one another but with a separation gap, were thrust together and were overlapping by approximately one foot after the earthquake. Figure 3.117 shows a view of the overlapping front walls of the two buildings. The 8-inch concrete block walls of the Cosmic Plastic building, and the tilt-up slab walls of the Omneco building, were displaced and extensively cracked by the collision.

Tilt-up construction is a common method of building single-story industrial buildings in southern California. A typical procedure is to cast reinforced concrete slabs on the floor slab of the building, then lift the slabs to a vertical position and form a connection between the slabs with a cast-in-place concrete column. Commonly, the slabs have dimensions of about 20 ft x 20 ft x 6 in., and the columns are of the order of 12 in. x 12 in.

The Vector Electronics building was one of the most extensively damaged tilt-up buildings in the area. Sections of the roof structure adjacent to the exterior walls collapsed into the building and two of the tilt-up slabs on the southwest wall fell outward from the building. Ground surface cracking and displacements were not particularly pronounced in the vicinity of this building and it appears that most of the damage resulted from strong ground shaking. The building has plan dimensions of about 220 ft x 160 ft., and is essentially a box structure enclosed by a flat timber roof and four tilt-up exterior walls. Although of large dimensions compared with the other tilt-up buildings, the construction details are typical. A view of the collapsed section of wall is shown in Fig. 3.118. The photograph was taken two weeks after the earthquake, and timber shoring had been used to realign and support the damaged sections of wall. Details of the walls and connections are shown in Figs. 3.119, 3.120 and 3.121. The wall slabs are 19 ft wide and have a maximum thickness of 8 inches. They are connected to the columns with

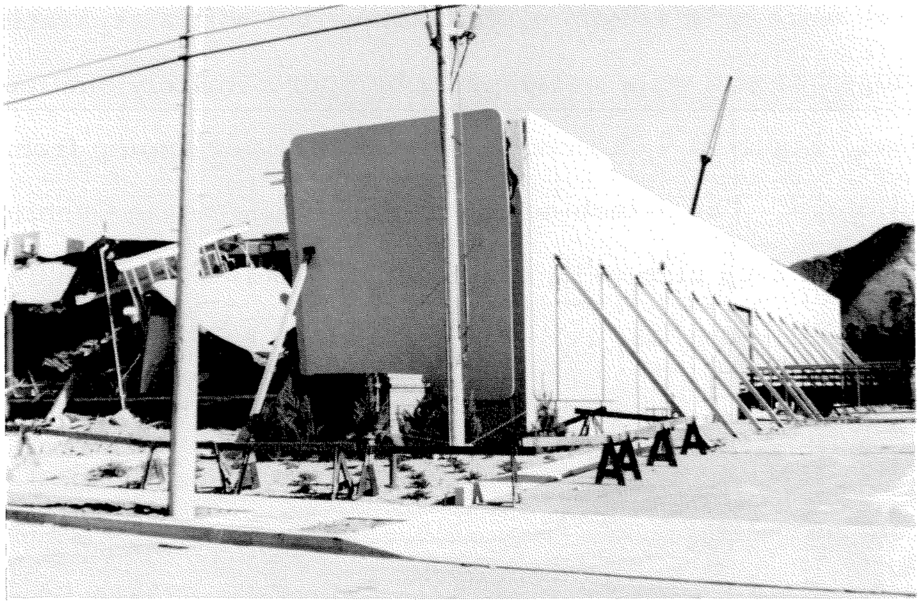


Figure 3.118 Collapsed section of tilt-up wall. Vector Electronics Building.

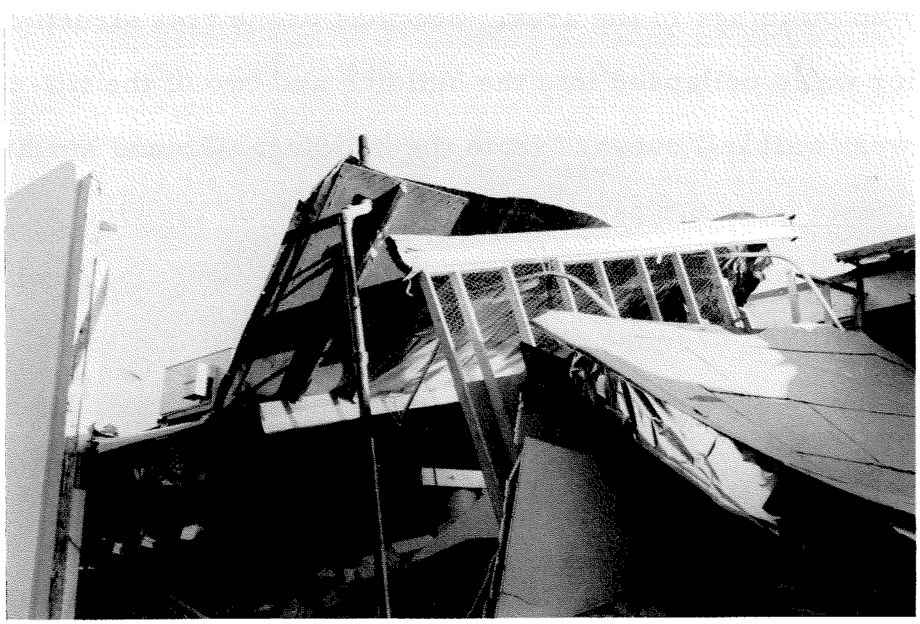


Figure 3.119 A section of collapsed roof showing the 1/2 in plywood, 11 in x 3-1/2 in purlins and steel connectors. Vector Electronics Building.

Figure 3.120 Collapsed tilt-up slabs showing the 12 in x 12 in column. Vector Electronics Building.

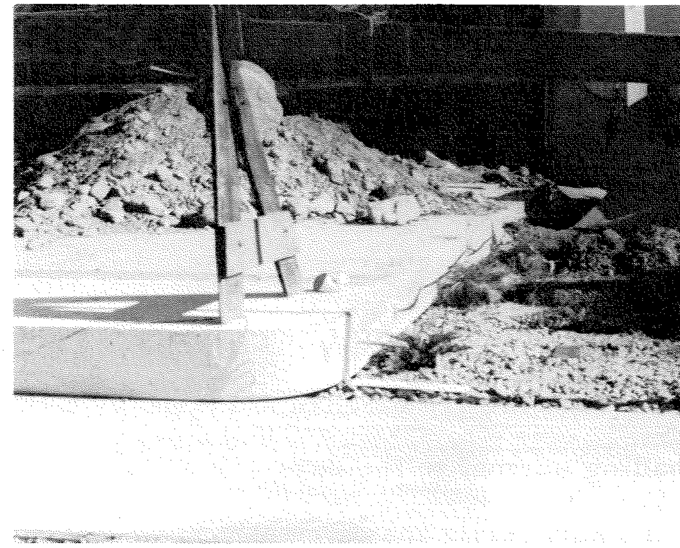


Figure 3.121 Four in long, 1/2 in diameter connecting bars in collapsed slab. Vector Electronics Building.

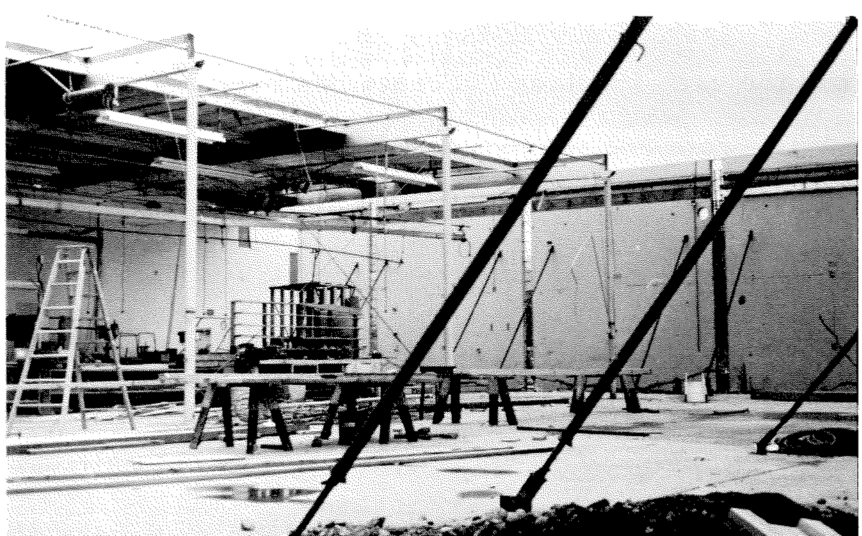


Figure 3.122 An interior view of the Vector Electronics Building. Part of the roof has been removed to carry out repairs.

1/2-in diameter bars extending 4 in into the column and placed at 1-ft 9-in centers. The columns are 12 in x 12 in and are constructed of lightweight concrete. An interior view in Fig. 3.122 taken eight weeks after the earthquake shows the extent of the repair operations and details of the roof construction. Laminated timber beams, which span approximately 80 ft, support a 1/2-in thick plywood roof cladding.

The failure mechanism appears to have been initiated by excessive loading normal to the exterior walls. The roof structure and interior supports were apparently insufficiently rigid to allow all of the shear loads to be transmitted to the walls aligned with the forces. The exterior walls were effective in resisting parallel shearing loads, but offered very little resistance to forces normal to the wall. The wall-to-floor slab and the roof-to-wall connections had no appreciable moment resistance. Repeated loadings perpendicular to the planes of the walls are thought to have loosened the roof-to-wall and wall-to-column connections leading to eventual failure.

A section of the northern wall of the Adrian's building is shown in Fig. 3.123 and details of the connections can be seen in Fig. 3.124. Repair is underway, and the cast-in-place concrete has been removed from the column reinforcement. The roof and wall structures of this building remained intact but the connections were loosened by the severe shaking. Presumably, the repair operation is completed by realigning the slabs, then recasting the columns.

The Gibralter Plastic Products building was of a design different from most of the tilt-up buildings in the area, and illustrated a different type of failure. The tilt-up slabs in the front face of the building were divided by window openings which extended the full width of the slabs. Horizontal loads

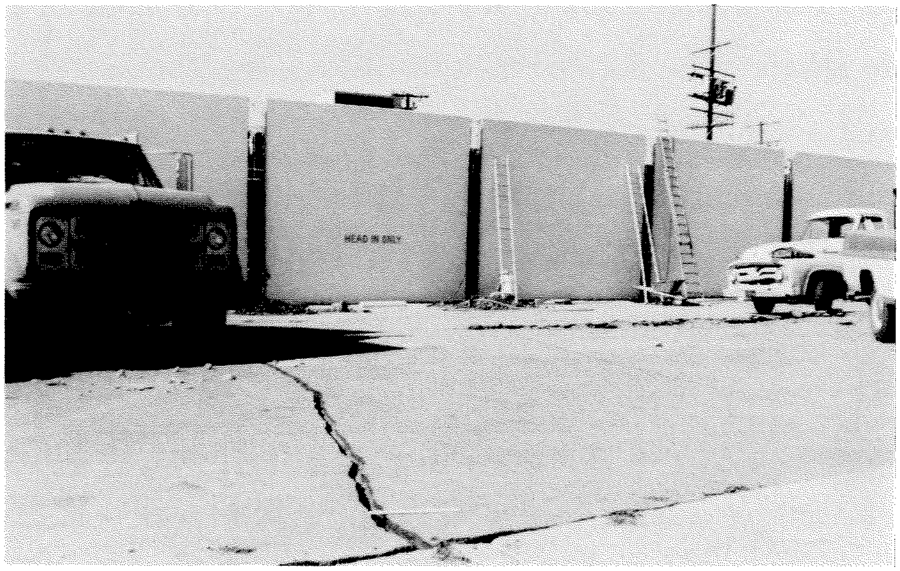


Figure 3.123 Repair work on tilt-up slab wall, two weeks after the earthquake. Adrian's Building.

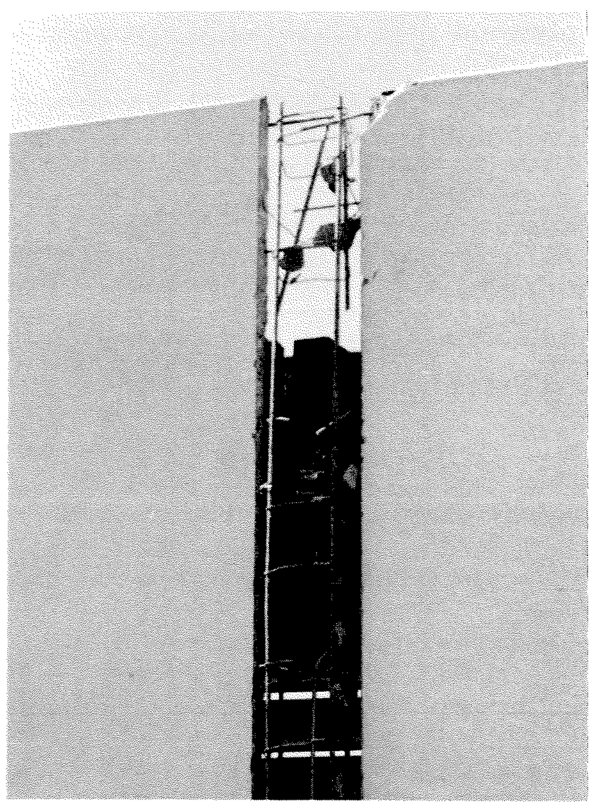


Figure 3.124 Wall and beam connection details. Adrian's Building.

transmitted to this wall were resisted by the 12 in x 12 in column sections which failed in shear. The building and the damaged columns are shown in Figs. 3.125 and 3.126. The structure has been repaired by casting concrete in the two end slab window openings, thereby creating more shear resistance.

Most of the concrete block buildings that received extensive damage were subjected to large permanent ground displacements. Figure 3.127 shows a collapsed concrete block wall in the M & L Machine and Stamping Company building. Sections of the roof and wall collapsed at two ends of the building. This building did not appear to have been subjected to large permanent ground displacements and the collapse of the walls and roof appeared to be due to failure of the roof-to-wall connection.

The roof-to-wall connections also failed at two end walls in the brick-constructed Bell Metrics building (Fig. 3.128) with complete collapse of one of the walls. The roof-to-wall connection in this building was formed by nailing the 1/2-in thick plywood roof cladding to a timber ledger bolted to the top section of the wall. The roof purlins were connected to the ledger with steel connectors. The walls were constructed of two layers of bricks separated by a 3-in thick mortar layer. Vertical reinforcement in the collapsed wall consisted of 5/8-in diameter bars placed at 4 ft centers.

Industrial Buildings - San Fernando Road, Bledsoe Street and Bradley Avenue

Approximately 20 industrial buildings in the vicinity of the Bledsoe Street and San Fernando Road (Fig. 1.2) intersection received significant structural damage. Most of the buildings were single-story structures with tilt-up walls and relatively large plan dimensions. Failure of the wall-to-roof connection with partial collapse of the roof was a common form of damage. The extent of the roof collapses can be seen in the aerial view (Fig. 3.129).

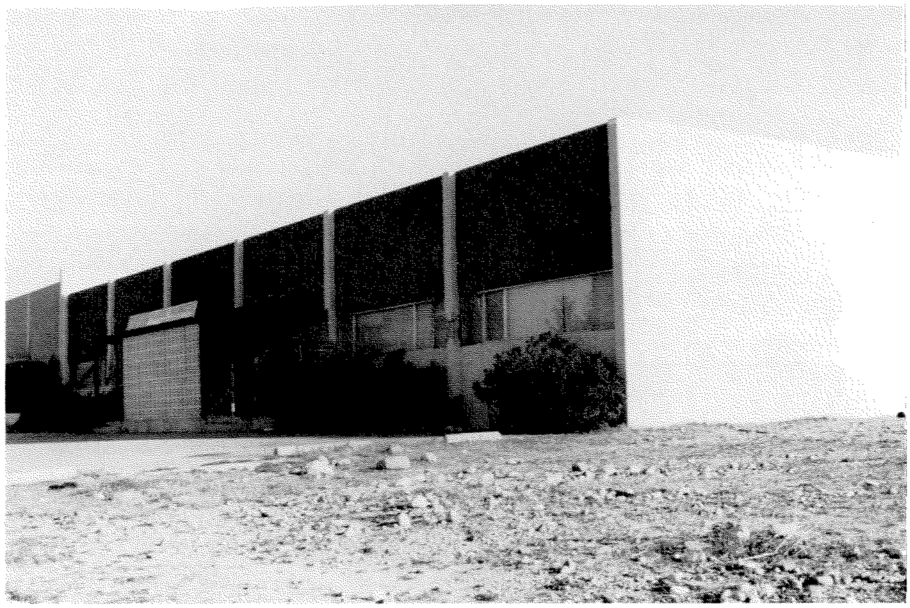


Figure 3.125 Front elevation (southwest wall) of the Gibralter Building.

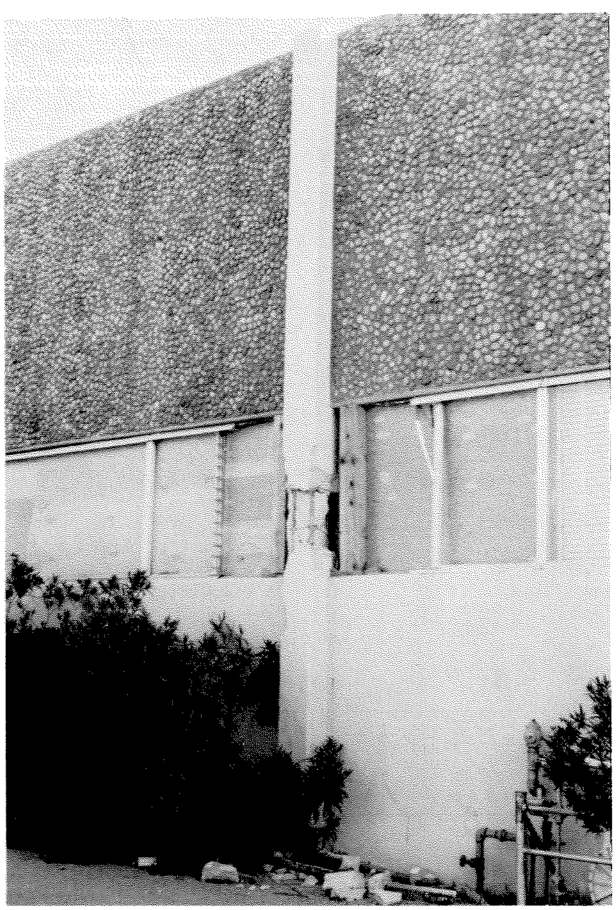


Figure 3.126 Failure of 12-in x 12 in column, Gibralter Plastic Products Building.

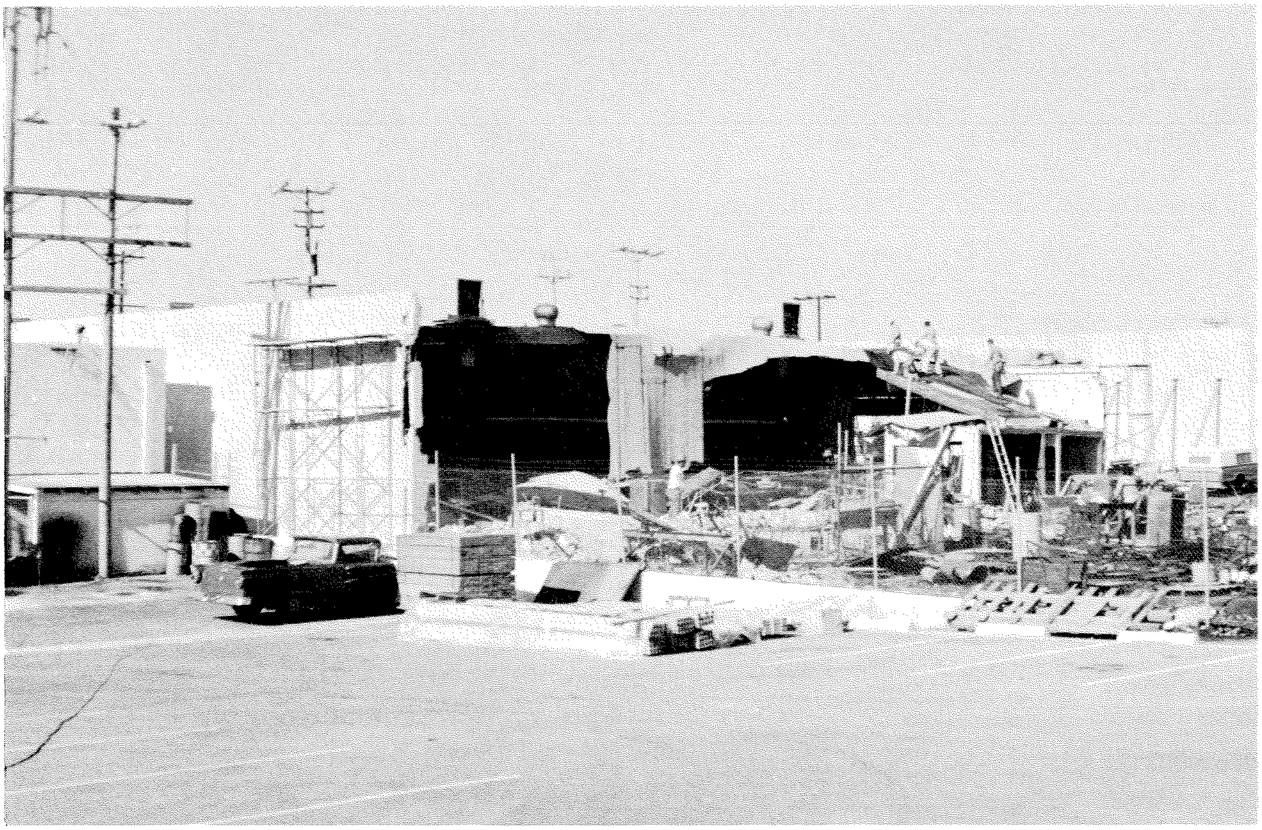


Figure 3. 127 Collapse of concrete block wall (northeast wall). M & L Machine and Stamping Company Building.

Figure 3. 128 Collapse of brick wall (southeast wall). Bell Metrics Building.

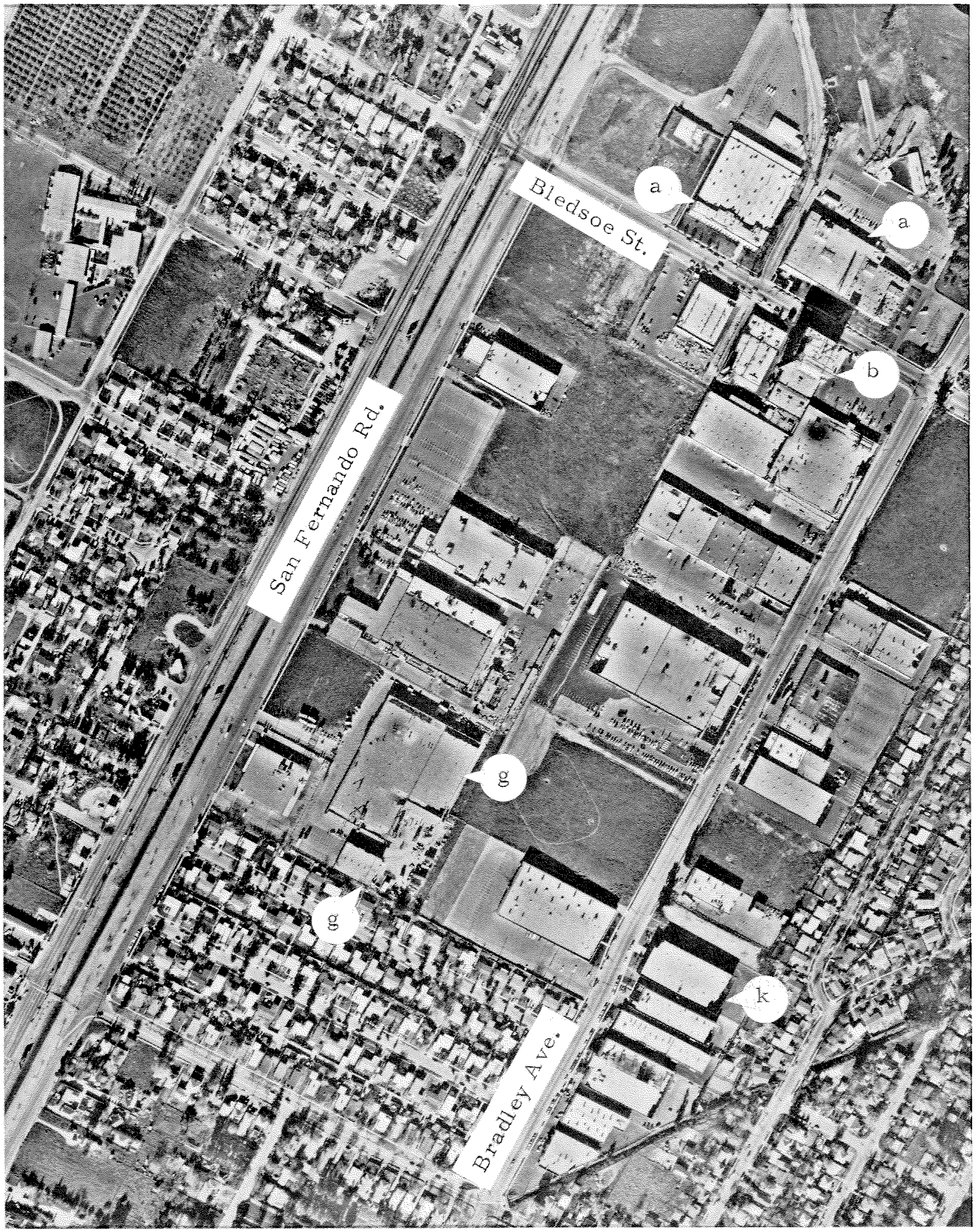


Figure 3.129 Roof collapses on tilt-up factory buildings. The identification letters refer to the list of buildings given in the text.

The buildings in this area were west of the surface traces of faulting. There was evidence of soil and pavement cracking in some parts of the area but it does not appear that any of the buildings were subjected to large permanent ground displacements.

The following list contains most of the buildings that received significant damage. Several of these structures are discussed in more detail below and some are identified in Fig. 3.129.

(a)	International Telephone and Telegraph (three buildings) Bledsoe Street and San Fernando Road	Tilt-up
(b)	Nethercutt (four-story building) Bledsoe Street	Shear wall
(c)	Bauman Weitz Inc. Bradley Avenue	Tilt-up
(d)	Augerscope Inc. San Fernando Road	Tilt-up
(e)	Sierracin Corp. San Fernando Road	Tilt-up
(f)	Lance Industries Bradley Avenue	Tilt-up
(g)	Sawyer Cabinet Inc. San Fernando Road	Tilt-up
(h)	Valley Todeco Inc. Bradley Avenue	Tilt-up
(i)	Un-named Building 12840 Bradley Avenue	Tilt-up
(j)	All Phase Color Corp. Bradley Avenue	Tilt-up
(k)	New Building 12880 Bradley Avenue	Tilt-up
(l)	New Building 12950 Bradley Avenue	Tilt-up

(m)	Dynasciences Bradley Avenue	Tilt-up
(n)	Atlas Coverall and Uniform Supply Bradley Avenue	Concrete block

International Telephone and Telegraph Buildings: The two buildings on the northernmost side of Bledsoe Street received quite extensive structural damage. Damage to a smaller building on San Fernando Road was relatively minor and appeared to be restricted to a loosening of some of the tilt-up wall joints.

A view of the eastern building on Bledsoe Street is shown in Fig. 3.130. The building has approximate plan dimensions of 360 ft x 180 ft and is a combination of a single-story factory section and a two-story office section. Three of the exterior walls are of tilt-up slab construction. The front wall of the building is constructed of precast concrete walls and slabs which appear to have been connected together by cast-in-place columns. Extensive cracking of the columns and loosening of the precast panels at the front of the building occurred during the earthquake. A repaired section of the building is shown in Fig. 3.131. Parts of the concrete columns were removed during repairs and epoxy was pumped into the cracks and joints. The repaired sections show as darker areas in the photograph. Permission to inspect the interior of this building was not obtainable but it is believed that the building was reoccupied approximately one month after the earthquake and that interior damage was not extensive.

The western building on Bledsoe Street is of similar construction to the eastern building described above and has approximate plan dimensions of 240 ft x 270 ft. A two-story office section approximately 40 ft wide forms

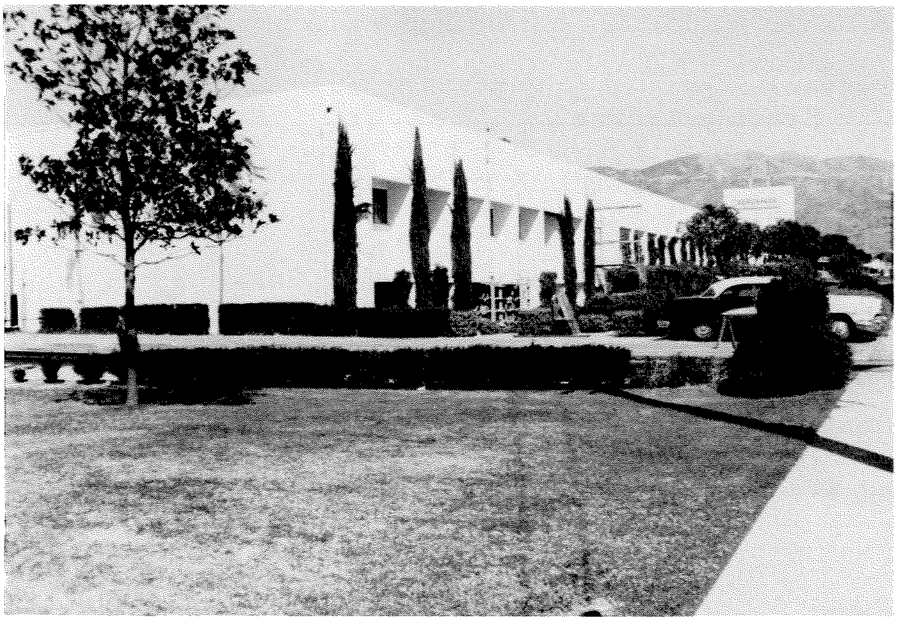


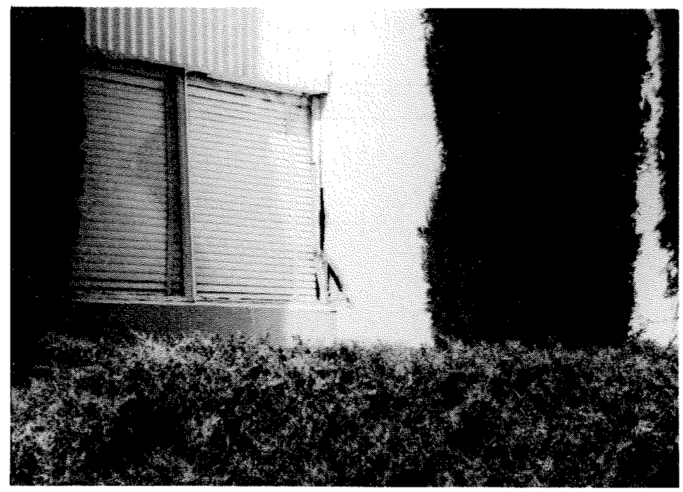
Figure 3.130 International Telephone and Telegraph eastern building, Aerospace Division, Bledsoe Street.

Figure 3.131 Repaired section of building shown in Figure 3.130. Sections of the columns have been recast and cracks filled with epoxy.

the front of the building and the remaining area is a single-story factory. The columns and walls in the front face of the building received extensive cracking and most of the glass in this face was broken. Typical damage is shown in Figs. 3.132 and 3.133. Details of the internal damage are not known but the building was unoccupied six weeks after the earthquake and it is believed that quite extensive repairs are necessary.

In both buildings described above the front walls have considerably less strength than the other exterior walls. In the eastern building lateral forces in the direction along the wall are resisted by 3 ft x 1 ft columns at about 10 ft centers acting in their weak direction. In the western building forces along the direction of the front wall are resisted by the combined action of similar 3 ft x 1 ft columns and a wall approximately 10 ft wide (Fig. 3.133). The torsional imbalance of the buildings and the loading of the exterior walls normal to their plane are presumably the major factors contributing to the observed damage.

Nethercutt Museum: The Nethercutt 4-story building is located alongside a group of single-story tilt-up buildings owned by the same firm on Bledsoe Street. The tilt-up walls in one of the single-story buildings adjoining the 4-story structure received damage which probably resulted from pounding between the two structures. The other single-story buildings appeared undamaged but the 4-story structure, which was under construction at the time of the earthquake, received a moderate amount of structural damage. A view of the structure, which is to be used as a private antique car museum, is shown in Fig. 3.134. The exterior structure consists of 8-in thick normal



(b)

(a)

Figure 3.132 Loosening and cracking of precast concrete members, front wall of International Telephone and Telegraph western building, Bledsoe Street.

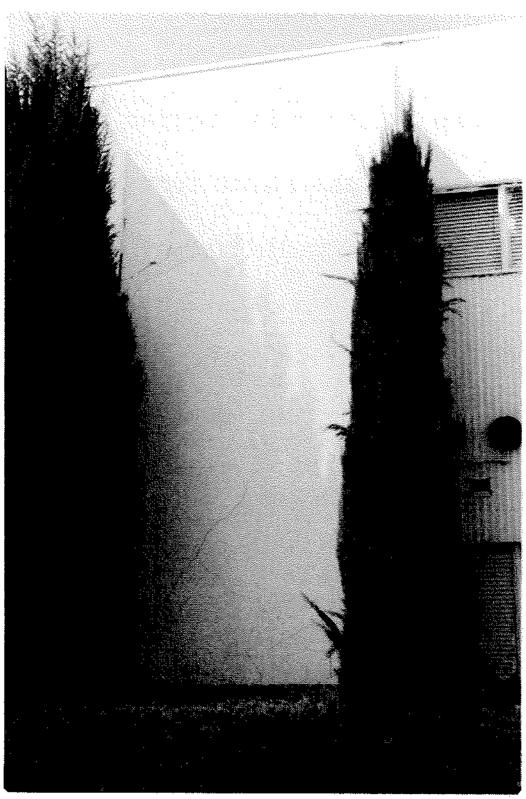


Figure 3.133 Cracked wall, front face of International Telephone and Telegraph western building, Bledsoe Street.

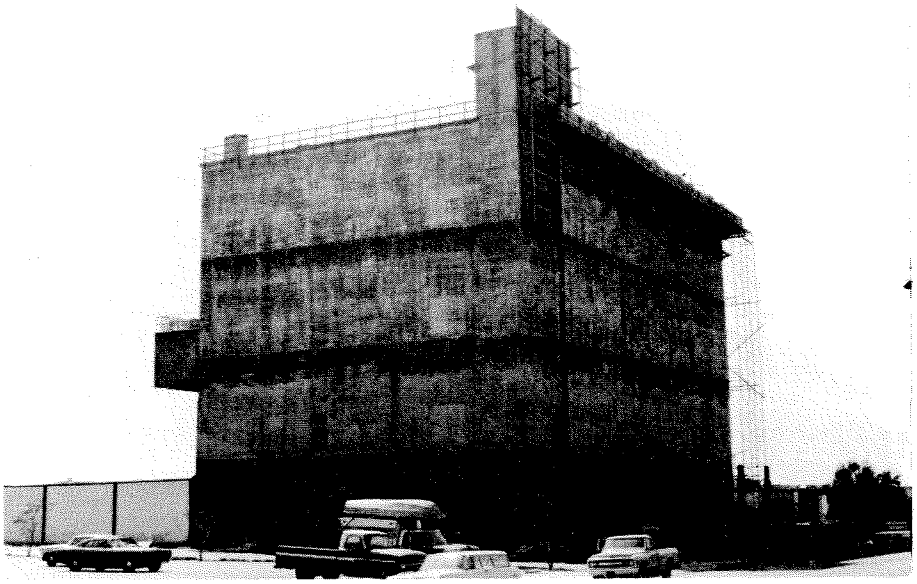


Figure 3. 134 Nethercutt Museum, Bledsoe Street.

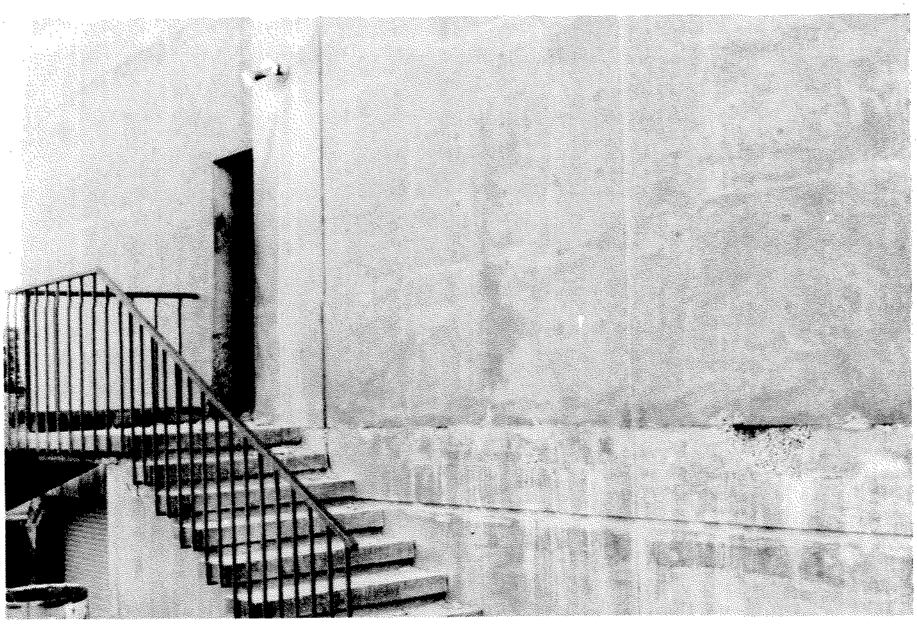


Figure 3. 135 Cracking and spalling on the construction joint above the first floor slab. Lightweight concrete was used for the slab and beam construction. Nethercutt Museum.

weight concrete walls constructed without window openings. In the interior of the building a reinforced concrete frame supports lightweight concrete floor slabs. The exterior walls were complete at the time of the earthquake but the internal structure was under construction.

The exterior wall structure was damaged by both movement on construction joints and 45° cracking. At the first floor level the walls were divided by the casting of a lightweight concrete floor slab and beam of 2 ft total thickness through the wall section. Figure 3.135 shows the spalling of the slab concrete which occurred at the construction joint between the walls and the top surface of the lightweight concrete slab. The external walls of the basement received fairly extensive 45° diagonal cracking shown by a section of wall repaired by epoxy after the earthquake (Fig. 3.136). A 1/4-in wide crack which formed at a door opening in the basement wall is shown in Fig. 3.137. Cracks that were observed in the first and second story exterior walls were particularly pronounced at the corners of the building.

Sawyer Cabinet, Inc.: Figure 3.138 shows an aerial view taken three days after the earthquake of the two Sawyer buildings located on San Fernando Road. Both of the buildings have tilt-up concrete wall slabs and timber roofs, and their construction is typical of many of the damaged buildings in this area. Sections of the roof on both buildings collapsed or partly collapsed; however, damage to the smaller building was relatively minor.

The structural layout and plan dimensions of the large building are also shown in Fig. 3.138. After initial construction the building was extended and the internal tilt-up wall was originally an exterior wall. The dimensions of the tilt-up concrete slabs are 19 ft x 19 ft x 6 in. A 12-in x 12 in reinforced, cast-in-place concrete column is used to make the vertical connection between

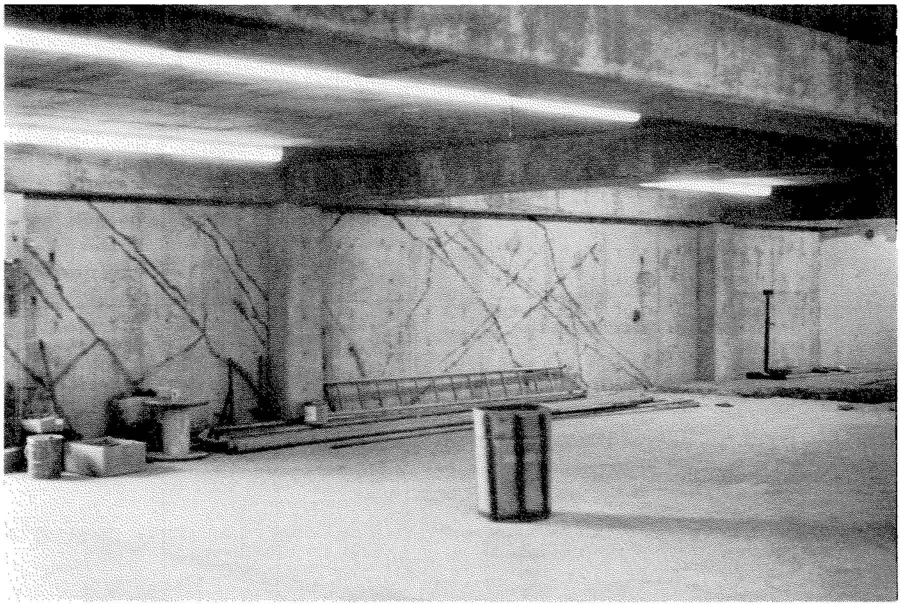


Figure 3.136 Repaired crack in a basement wall. Nethercutt Museum.

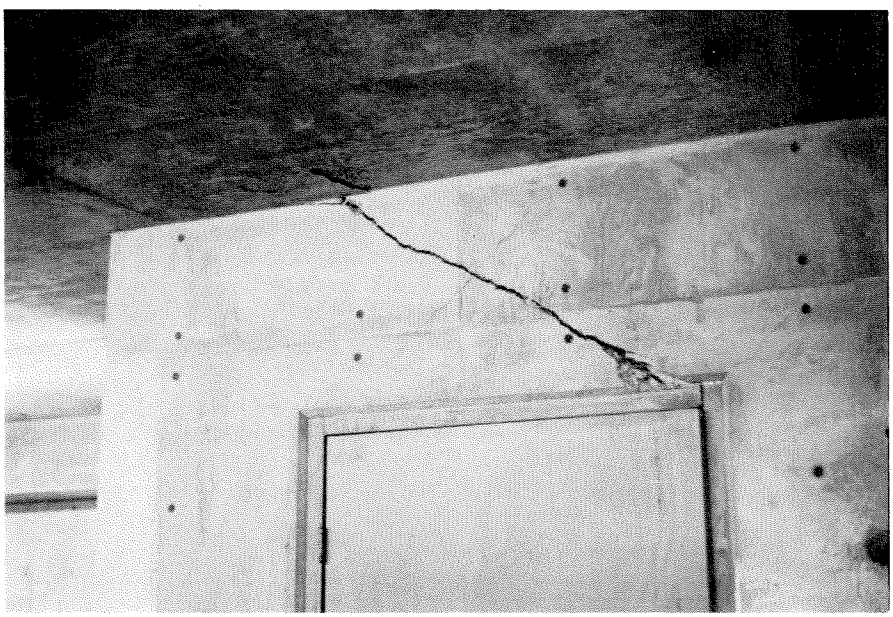
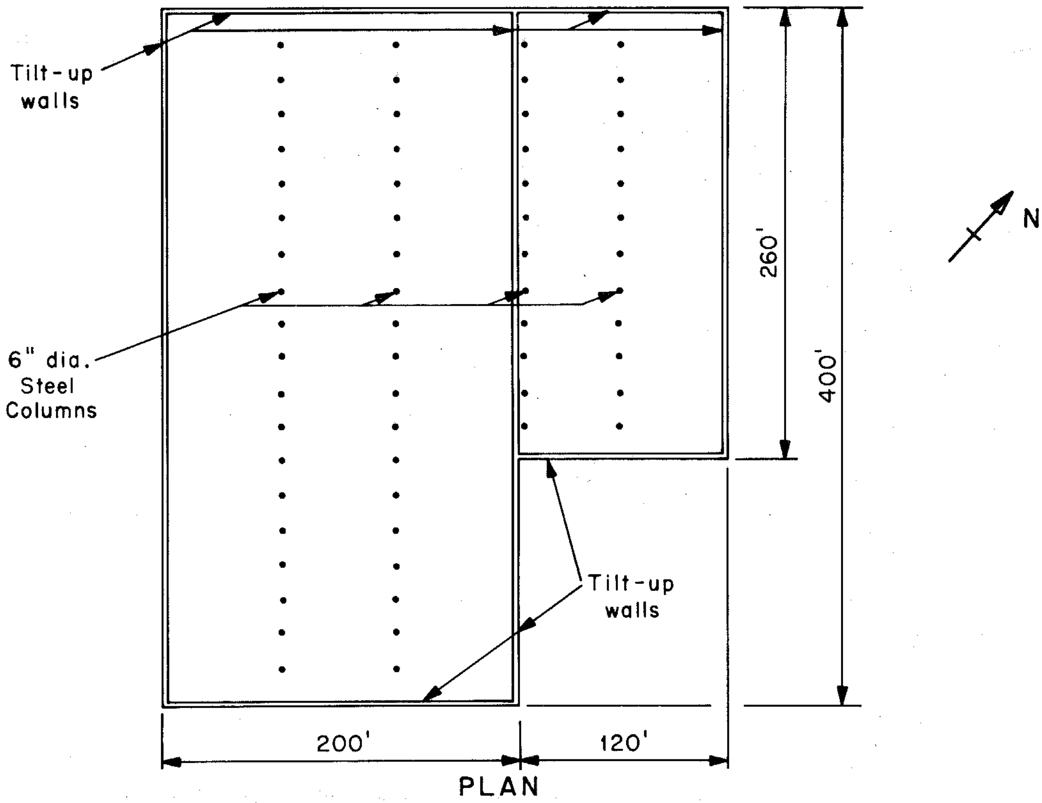


Figure 3.137 Wide cracks over door in basement. Nethercutt Museum.



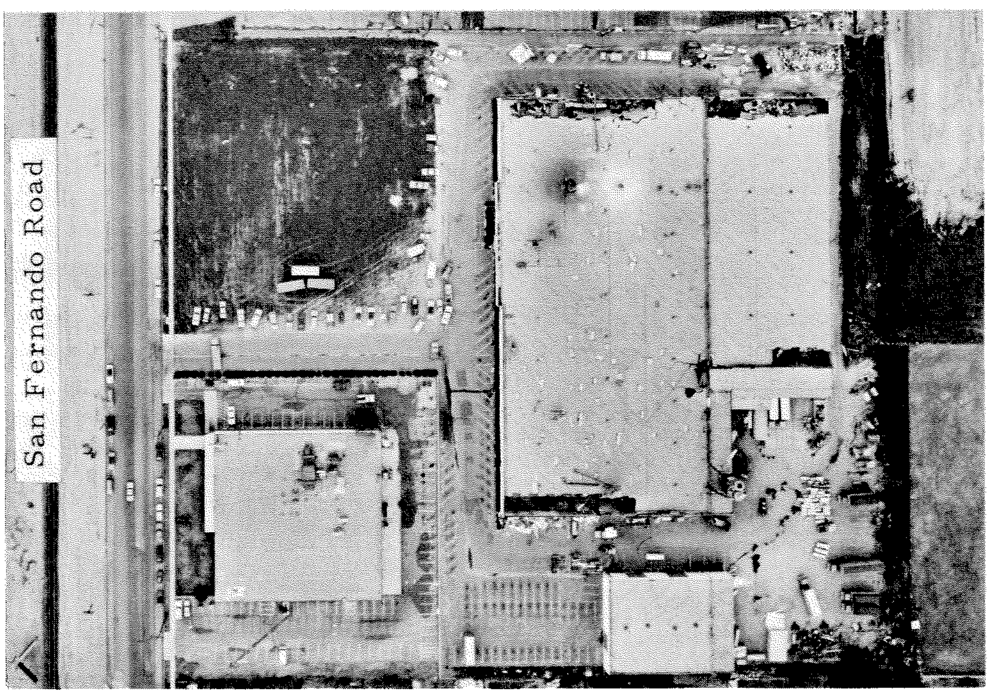


Figure 3.138 Plan dimension and aerial view of roof collapse of Sawyer Cabinet, Inc. Aerial view taken three days after earthquake.

the slabs. Reinforcing bars extend from the base of the wall slabs into the concrete floor slab of the building but the connection detail used is not capable of providing appreciable moment resistance. A section of the external walls which were being repaired six weeks after the earthquake is shown in Fig. 3.139.

Details of the timber roof structure can be seen in Fig. 3.140. The main roof beams are of glue-laminated construction and 70 ft sections are spliced together to span between the walls. The beams are also supported by 6-in diameter steel columns in the interior of the building; this reduces their effective span (for vertical loading) to approximately 70 ft. The roof cladding is 1/2 in thick plywood.

Details of the method of connecting the roof beams and purlins to the tilt-up walls are shown in Figs. 3.141 and 3.142. In both types of connection the plywood cladding is nailed to a timber ledger which is bolted to the top of the walls. Neither of the two connections has appreciable moment resistance.

The pattern of failure of the connections and collapse of the roof of this building appears similar to that described previously for the industrial buildings in the San Fernando Industrial Park. Lack of rigidity in the long-span roof structure and the internal supports has resulted in appreciable horizontal load being transmitted to the external walls in a direction normal to their plane. The roof connection to the end walls (NW, SE) probably failed because of the repeated loading normal to the wall. At this connection the nails pulled through the plywood cladding and the purlins separated from the steel connectors. Sections of the roof adjacent to both end walls collapsed

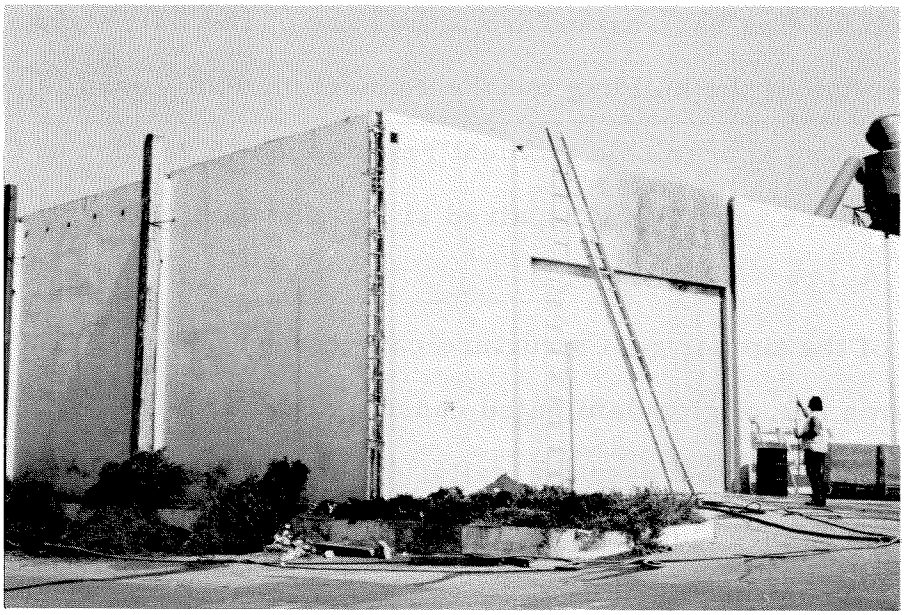


Figure 3.139 Repair of cracked tilt-up walls. A sand-blasting operation prior to application of gunite. Sawyer Cabinet Inc. building.

Figure 3.140 Interior view showing the roof structure and 6 in diameter steel supports. Sawyer Cabinet Inc. Building.

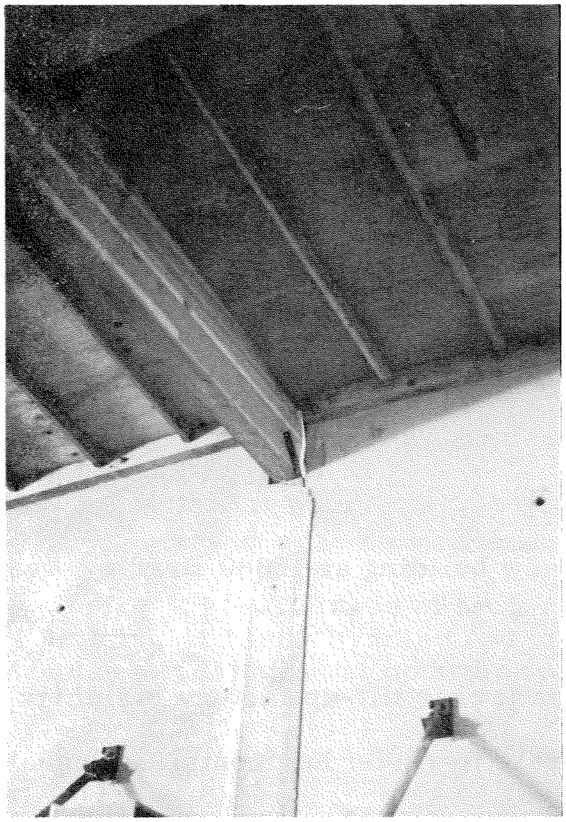


Figure 3.141 Detail of roof beam connection to tilt-up walls. The bracing at the middle of the walls was added after the earthquake. Sawyer Cabinet Inc. building.

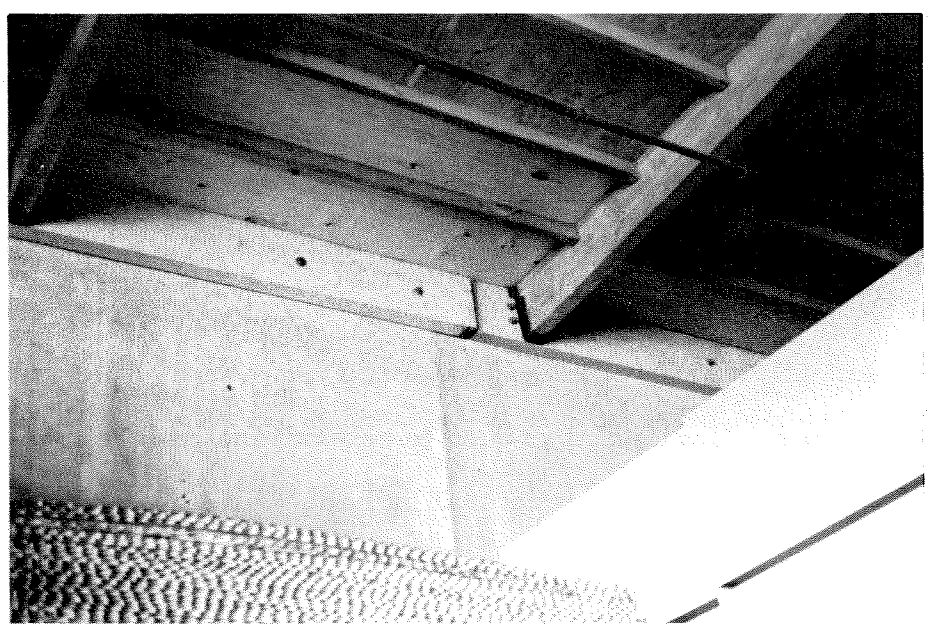


Figure 3.142 Detail of roof purlin connection to tilt-up walls. The steel connector is nailed to the timber ledger and the 11 in x 3-1/2 in purlin. Sawyer Cabinet Inc. building.

leaving the building as virtually a mechanism in the NE-SW direction. Movement of the building in this direction loosened the beam connections and damaged the interior fittings in an office section at the front of the building. Some of the wall slabs received quite extensive cracking and were being repaired by spraying with gunite at the time of inspection.

Other Tilt-Up Buildings: A number of tilt-up buildings experienced damage similar to that described for the Sawyer Cabinet building. Commonly, the roofs collapsed because of the failure of the roof-to-wall connection. Details of a typical failure which occurred in a new building at 12880 Bradley Avenue are shown in Figs. 3.143 to 3.146.

It is evident that in many of the larger tilt-up factory buildings the structural connections were inadequate to resist horizontal earthquake loads, and the plywood roof sheathing had inadequate strength. To achieve the desired resistance, more rigidity in the roof structure and internal vertical supports should be provided. If it is not possible to provide a sufficiently rigid roof or interior bracing, then a moment-resisting connection between the wall and roof should be provided. In addition, the connection between the tilt-up slabs should be made stronger.

Tall Buildings in the Southern San Fernando Valley

Several 8- to 21-story buildings are located in the southern part of the San Fernando Valley along the Ventura freeway (Fig. 1.1) about 16 miles south of the epicenter. Horizontal ground accelerations exceeded 20%g in much of this area, yet no major structural damage was reported for the tall buildings. While this fact is encouraging, it should be realized that the duration of the ground motion was not as long as would be expected in a great

Figure 3.143 Collapse of a corner section in a new building at 12880 Bradley Avenue. The roof-to-wall connections failed at both the front and rear walls.

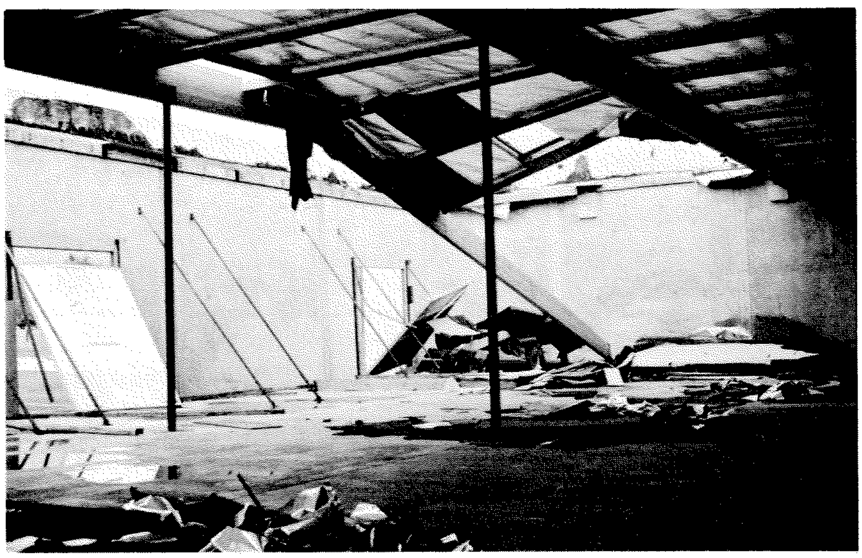


Figure 3.144 Collapse of a section of roof at the rear wall of the building shown in Figure 3.143. Laminated timber roof beams are supported at midspan by 6 in diameter steel columns.

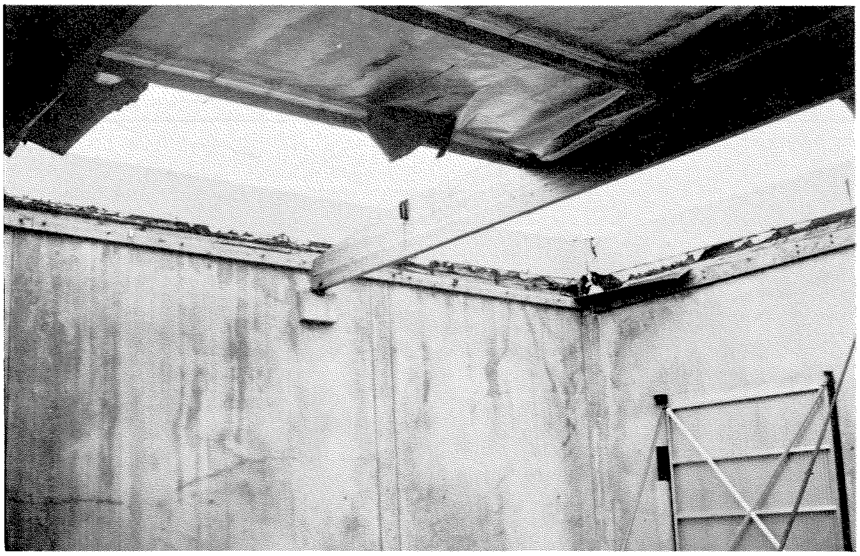


Figure 3.145 Roof-structure details in the building at 12880 Bradley Ave. The plywood roof cladding is nailed to the timber ledger bolted to tops of the walls.

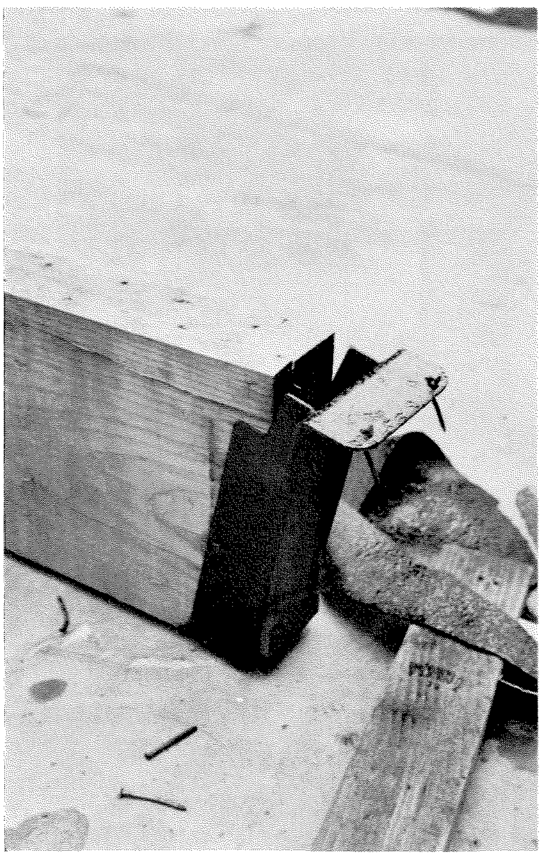


Figure 3.146 Steel connector used to fasten the 11 in x 3-1/2" purlin to the main beams and to the timber ledger on the walls. 12880 Bradley Ave.

earthquake. If ground motions of the same intensity had lasted for 20 seconds, it is expected that the damage would have been significantly greater, and more widespread because of the generally increased response of the fundamental modes of the structures and the aggravating effects of repeated vibrations on the damage that occurred in the first 10 seconds of motion (the approximate duration of the San Fernando earthquake at this location). Thus, these same buildings would likely suffer more damage from a large earthquake on the San Andreas fault.

Some of the more prominent buildings along the Ventura freeway at the southern boundary of the San Fernando Valley are listed below, proceeding in a generally easterly direction. Where available, information on the response of the structures to the earthquake is included.

1. Barclay's Bank Building (10 stories), 18321 Ventura Boulevard, Tarzana.

2. Ventura-Petit Building (9 stories), 16661 Ventura Boulevard, Encino.

3. United California Bank Building (14 stories), 16633 Ventura Boulevard, Encino.

4. Encino Medical Center (8 stories), 16260 Ventura Boulevard, Encino.

5. Lincoln Bank Building (12-story reinforced concrete frame), 16255 Ventura Boulevard, Encino.

6. Ventura-Woodley Building (12 stories), 16055 Ventura Boulevard, Encino. This building contained accelerograph recorders, but no record was obtained.

7. K-B Valley Center (17 stories), 15910 Ventura Voulevard, Encino (Fig. 3.147). This steel-framed building was just completed and was 40% occupied. Earthquake accelerations were recorded in the basement, at the 9th floor, and at the top floor. The peak horizontal acceleration recorded in the basement was 15%g; the peak in the upper stories of the building was 23%g. The vertical accelerations were slightly less. The building reportedly suffered no structural damage and only minor architectural damage.

8. Union Bank Building (13 Stories), 15233 Ventura Boulevard, Sherman Oaks (Fig. 3.148). Earthquake motions were recorded across the street in the Bank of California building with measured peak horizontal accelerations reaching 23%g in the basement. As the result of this strong ground shaking the 13-story Union Bank Building suffered cracks in the lower levels of the reinforced concrete frame, primarily in the connections at the second floor level (Fig. 3.149). The four corner columns were most severely damaged. Minor cracks were also observed in the stairwells at the lower levels, and cracks running nearly vertical were found in an interior sub-basement wall, apparently serving as a shear wall for lateral forces acting in the narrow dimension of the building (Fig. 3.150).

Rather extensive damage was caused to a single-level parking structure adjacent to the Union Bank Building. The reinforced concrete structure has no walls; consequently, the forces that resulted from horizontal ground shaking were carried by the supporting columns, many of which were severely cracked (Fig. 3.151 and 3.152). It appears that some of the column cracking is explained by horizontal shifting of the overhead beams on the columns. In these cases, the column cracks appear to follow the connecting steel which ties the beam to the column (Figs. 3.152 and 3.153). The parking structure is being repaired.

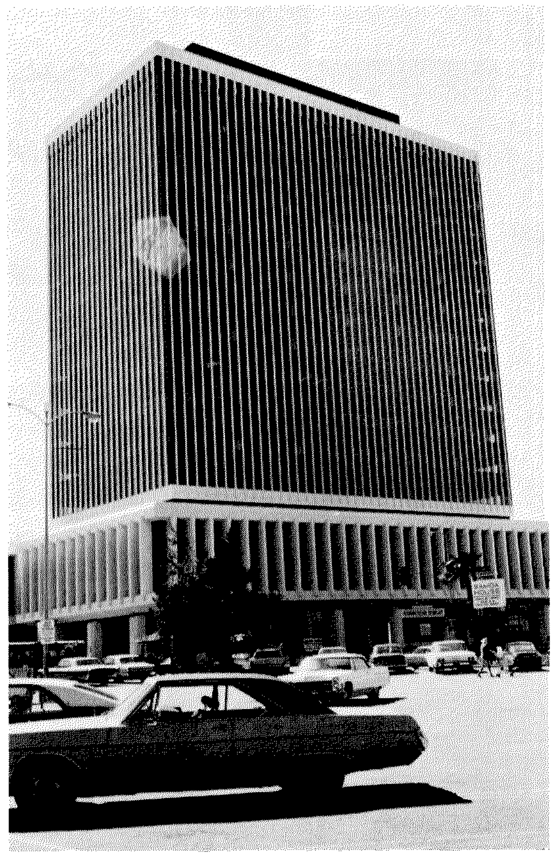


Figure 3.147 K-B Valley Center, 17-story steel-framed building, 15910 Ventura Boulevard, Encino.

Figure 3.148 Thirteen-story reinforced concrete frame, Union Bank Building, 15233 Ventura Boulevard, Sherman Oaks.

Figure 3.149 The cracks in the reinforced concrete frame were filled with epoxy, Union Bank building, Sherman Oaks.

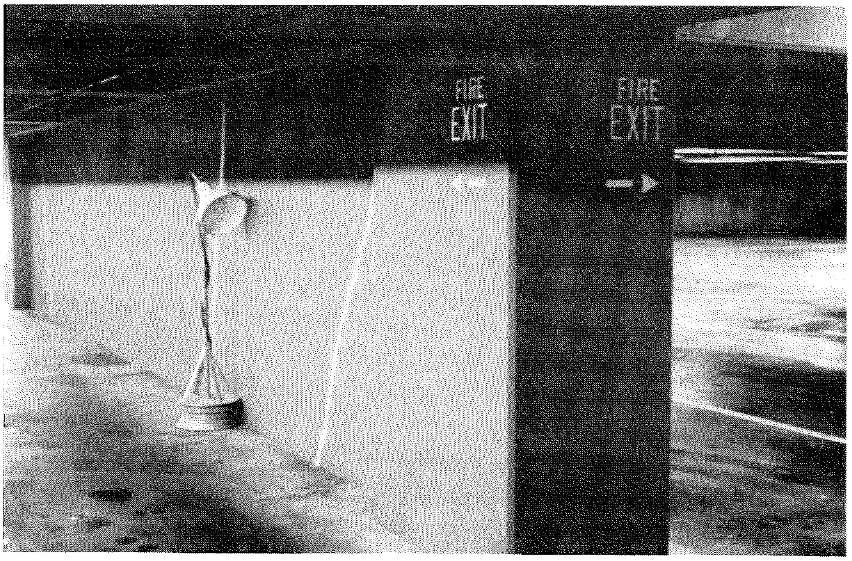


Figure 3.150 A light colored epoxy was used to fill cracks in this sub-basement interior wall aligned with the short dimension of the Union Bank building, Sherman Oaks.

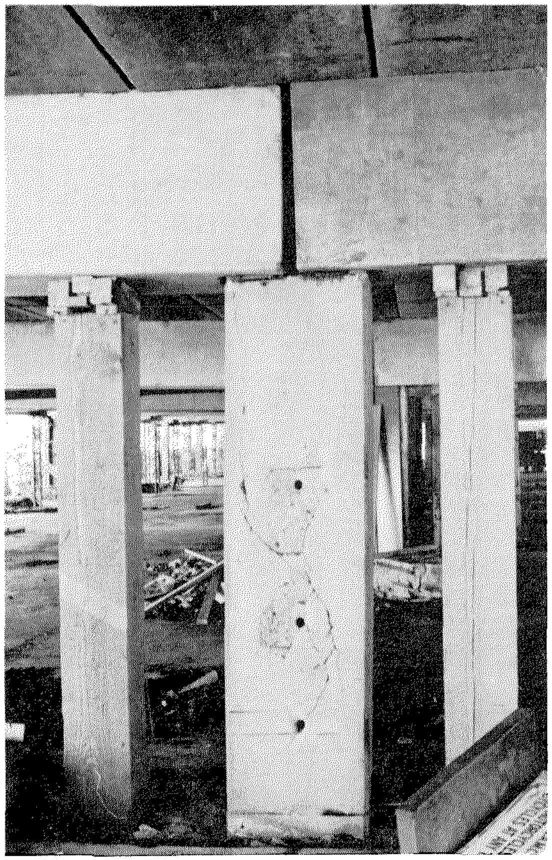


Figure 3.151 Cracked column and displaced beams in the parking structure adjacent to the Union Bank, Sherman Oaks.

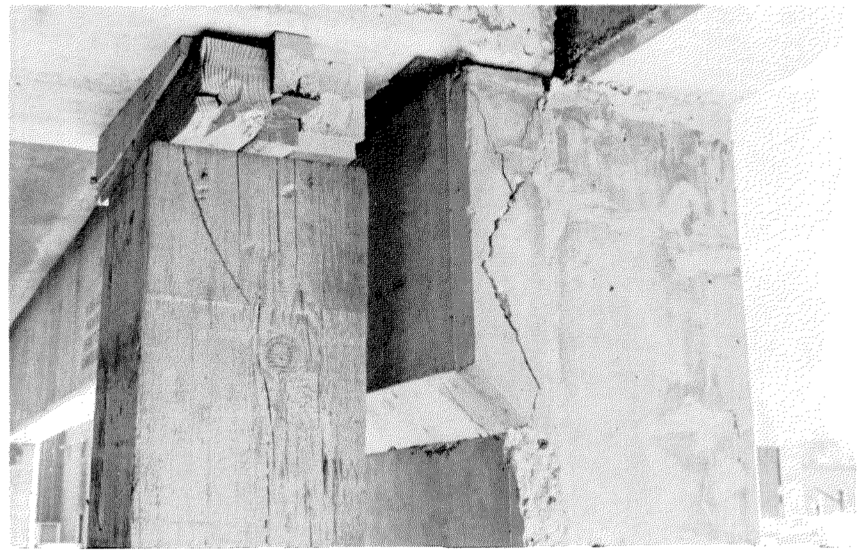


Figure 3.152 The cracks in the exterior column of the Union Bank parking structure were probably caused by horizontal movements of the overhead beam.

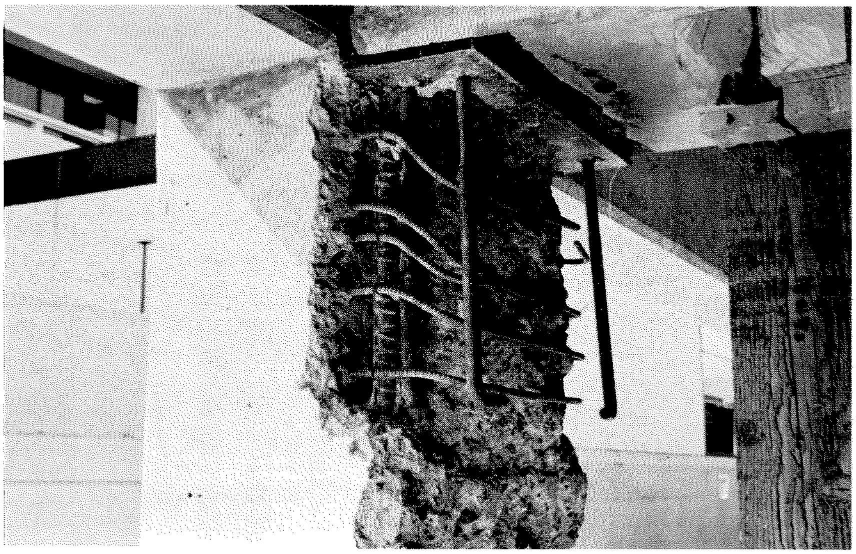


Figure 3. 153 Concrete removed from the exterior column of the Union Bank parking structure to reveal the reinforcing steel.

Figure 3. 154 Twelve-story reinforced concrete frame; the Bank of California, 15250 Ventura Boulevard, Sherman Oaks.

9. The Bank of California (12 stories), 15250 Ventura Boulevard, Sherman Oaks (Fig. 3.154). Strong-motion accelerographs are located on the basement, 7th floor, and the roof of the building. The instruments recorded horizontal peak accelerations of 23%g in the basement and 28%g on the roof. At the three recording stations in the building vertical accelerations were only about half as great as the corresponding horizontal accelerations.

The reinforced concrete frame of the 12-story building developed cracks over most of the height of the building as the result of the strong ground shaking. The cracks, shown in Figure 3.155, are being filled with epoxy to effect repair. More severe damage to the exterior frame is visible from inside the building. Many of the connections between the lightweight concrete beams and the normal weight concrete columns suffered cracks and some concrete spalling. An example of this damage, visible on an unfinished wall at the fourth floor level, is pictured in Fig. 3.156. Repair work on these damaged connections involved the emplacement of epoxy in the cracks and the regions of spalled concrete. Mortar was placed over the epoxy in regions where the concrete spalled to a depth of 2 or 3 inches.

The parking building for the Bank of California had no visible damage, probably because of the horizontal strength provided by the exterior walls of the concrete structure.

10. Certified Life Tower (14 stories), 14724 Ventura Boulevard, Sherman Oaks.

11. Lincoln Savings and Loan (8 stories), 13701 Riverside Drive, Sherman Oaks. No damage was observed on the exterior of this reinforced concrete building shown in Fig. 3.157.

Figure 3.155 Epoxy was used to repair cracks in the exterior frame of the Bank of California.

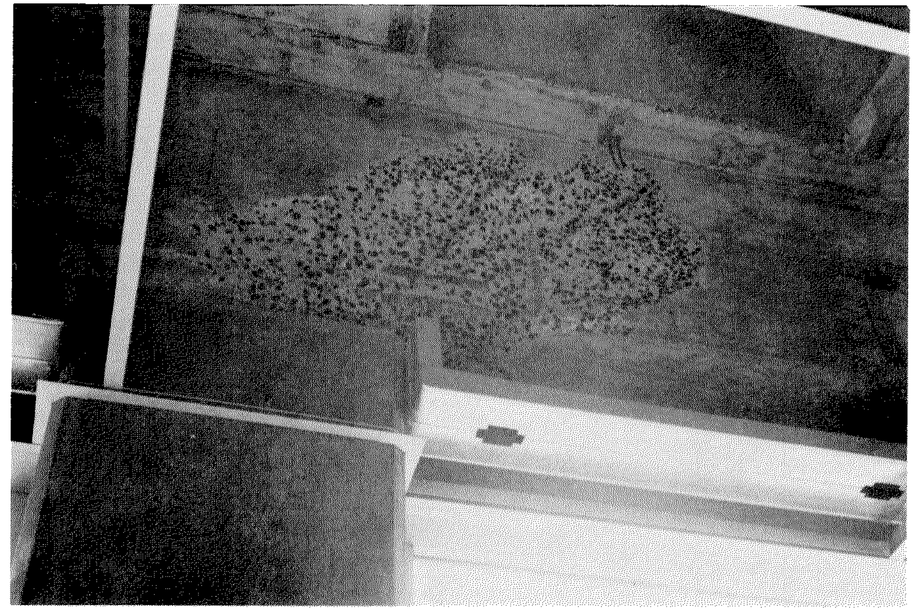


Figure 3.156 The concrete cracked and spalled in the region of the beam-column connection with no visible damage to the reinforcing steel, fourth floor level, Bank of California.

Figure 3. 157 Eight-story Lincoln Savings building, 13701 Riverside Drive, Sherman Oaks.

Figure 3. 158 Panorama Towers building on Van Nuys Blvd. in Van Nuys. Example of a multi-story building in the southern San Fernando Valley.

12. North Hollywood Federal Savings (6 stories), 4455 Lankershim Boulevard, North Hollywood. Minor cracking was found in the elevator walls at one end of the reinforced concrete building.

13. MCA Building (14 stories), 3900 Lankershim Boulevard, Universal City. No damage was observed on the exterior of this steel-framed building.

14. Sheraton Universal Hotel (21 stories), 3838 Lankershim Boulevard, Universal City. It is thought that this building has a steel frame with no structural damage. Some minor damage was found in nonstructural components and a large tile mural was destroyed by spalling. Cracks reportedly were developing in the plaster walls on the lower levels of the building many days after the main earthquake.

The Holiday Inn

The seven-story Holiday Inn (Figs. 3.159 through 3.162), located at the corner of Roscoe Boulevard and Orion Street was instrumented with three strong-motion accelerographs as required by the Los Angeles City Building Code. The building is of particular importance because it was the nearest instrumented building to the center of the earthquake (Fig. 1.2).

The accelerograph on the ground floor level (there is no basement) recorded a peak horizontal acceleration of 27%g. The accelerograph in the penthouse on the roof recorded a peak acceleration of 40%g. Strong vertical motions, 17%g on the first floor and 22%g on the roof, also were recorded.

The building is of reinforced concrete construction. The two end walls are concrete shear walls and the two side walls are reinforced concrete frames composed of beams and columns. From the recorded accelerations it is clear that the building underwent severe deformation during the earthquake.

Figure 3.159 North elevation of Holiday Inn. This building is located at the intersection of Roscoe Blvd. and Orion St. six miles south of Balboa Blvd. Water Treatment Plant. Motion recorded on the ground floor had a peak acceleration of 27%g and motion recorded on the penthouse floor had a peak acceleration of 40%g.

Figure 3.160 North elevation of building on March 21st. The second, third and fourth floors suffered severe non-structural damage estimated to be approximately $250,000. An aftershock seven weeks after the main earthquake damaged the repairs that had already been made.

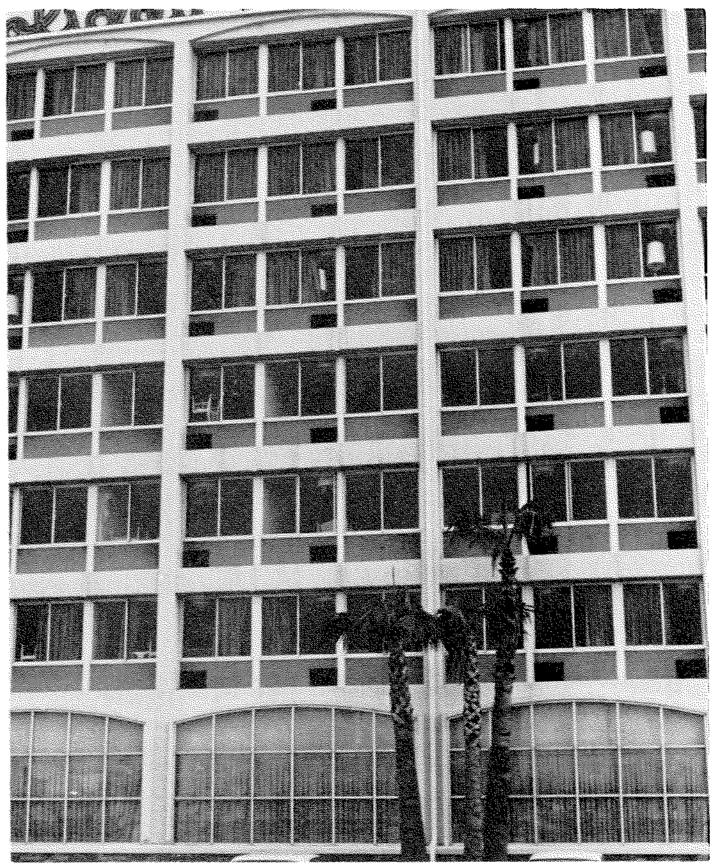


Figure 3. 161 West elevation of Holiday Inn.

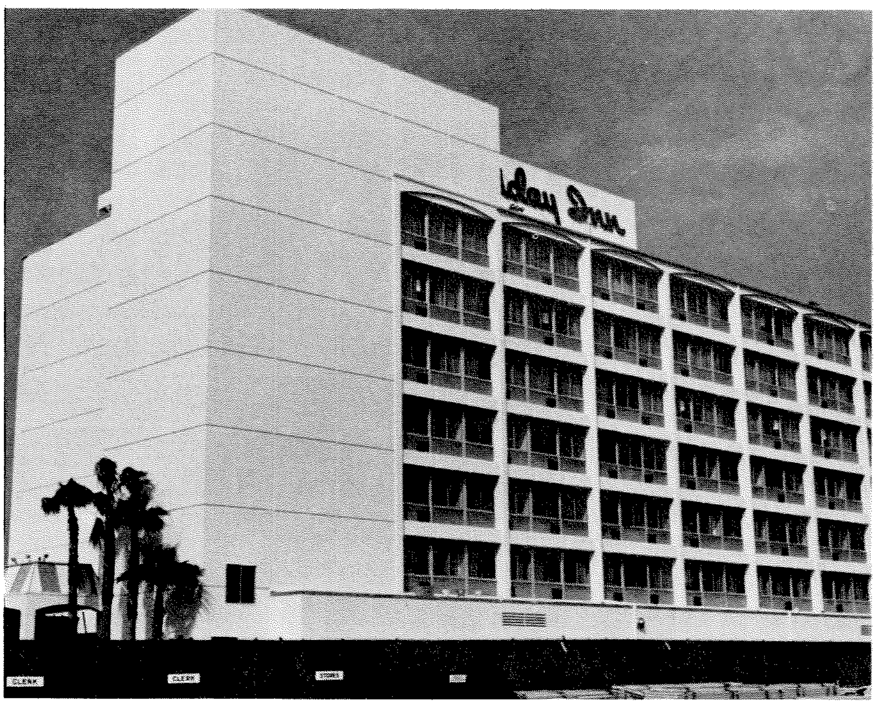


Figure 3. 162 South elevation of Holiday Inn. The concrete frame underwent some cracking which indicated strains beyond the yield point.

These deformations were sufficiently great to cause extensive damage to the interior plaster walls, to the plumbing fixtures, etc. on the second, third and fourth floors. The upper three floors were not damaged severely. The nonstructural damage has been estimated at approximately $250,000. Repairs were begun shortly after the earthquake and additional damage was done by a large aftershock. The structural frame received some cracks, indicating strains beyond the elastic limit; the cracks were repaired with epoxy cement.

Millikan Library

The Millikan Library Building on the campus of the California Institute of Technology (Figs. 3.163 and 3.164) is a reinforced concrete shear-wall structure, nine stories high, plus an enclosed roof area which is used for air-conditioning equipment. The east and west exterior walls of the building act as two shear walls. In the other direction, the north and south walls of the building are formed of relatively flexible beam and columns with large, precast, concrete, ornamental panels bolted in place at each floor level and at each bay, the lateral resistance being provided by the north and south walls of the elevator shaft. The dynamic properties of the building were investigated before the earthquake, and the natural period of vibration in the east-west direction was determined to be 0.66 seconds. After the earthquake the period was measured to be 0.76 seconds. During the earthquake the accelerograph on the roof showed the building to be vibrating in the east-west direction with a period of one second, and a peak acceleration of 35%g. The accelerations in the upper floors of the building caused many of the bookshelves to collapse, and threw many of the books to the floor. With the exception of hairline cracking in the plaster around some window panels, no damage to the building itself was observed. The different periods of

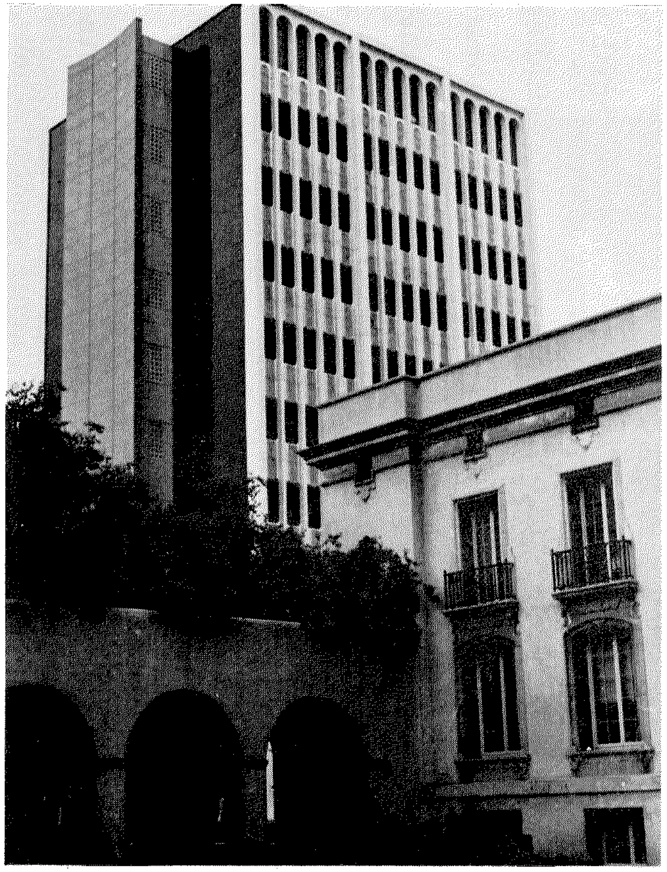


Figure 3.163 North elevation of the Millikan Library building on the Caltech campus, a 9-story, plus enclosed roof, concrete shear-wall structure. The north and south faces of the building have precast concrete panels bolted to the floor beams.

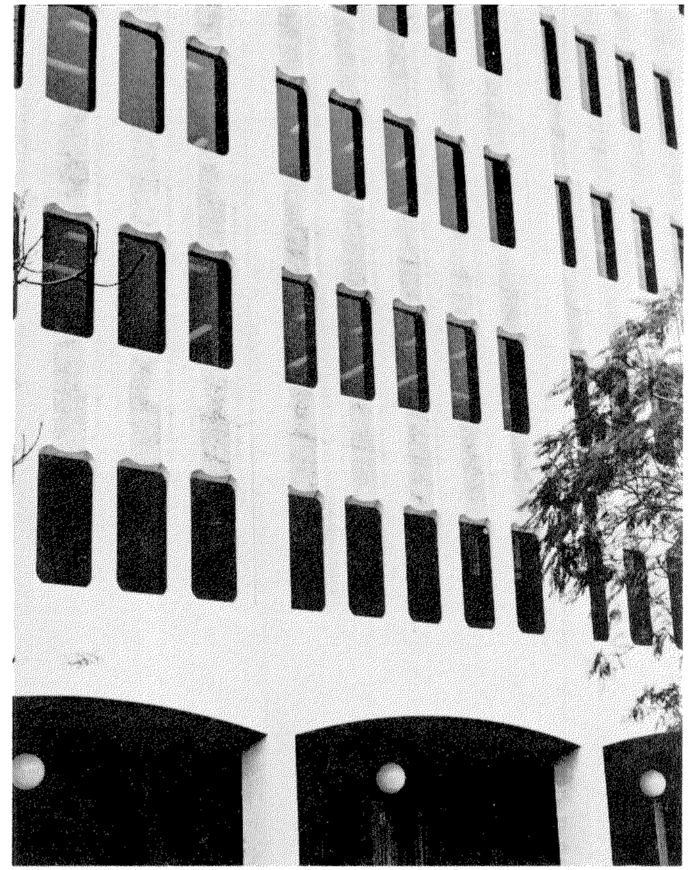


Figure 3.164 Each precast concrete panel extends from column to column, and from floor to floor. The concrete walls of the elevator shaft form the resisting elements for horizontal forces in the east-west elevation.

vibration before, during, and after the earthquake are attributed to the action of the concrete panels bolted on the east and west faces of the building. No change in natural period of vibration was observed in the north-south direction.

Schools

The Los Angeles School System has a student population of about 640,000 and has 10,000 school buildings of which 6,000 could be classified as major buildings. Figure 3.165 shows the locations of the schools in northwestern Los Angeles; each has several buildings. Approximately 25 school sites were in the region of very strong ground shaking. At the time of the 1933 Long Beach earthquake there were approximately 800 school buildings, none of which had been specifically designed to resist earthquakes. Since 1933, the school population has greatly increased and many new school buildings have been built to meet the requirements of the Field Act, a state law governing the earthquake design of school buildings. The State Office of Architecture and Construction checks the design of school buildings and supervises construction. A program of rebuilding the old schools to make them meet the requirements of the Field Act has been underway. By 1971, all but 160 of the old schools, such as shown in Fig. 3.166, had been rebuilt. Of these, 50 were wood frame buildings and 110 were either brick-bearing wall buildings or concrete buildings. Most schools in the San Fernando Valley were post-1933, and these experienced little damage from shaking. Some wood frame buildings were shifted on their foundations, and some cracking was experienced. Some damage was experienced by the school buildings at the San Fernando Juvenile Facility where a concrete block shear wall was damaged by excessive shear force and the cross-bracing rods between the

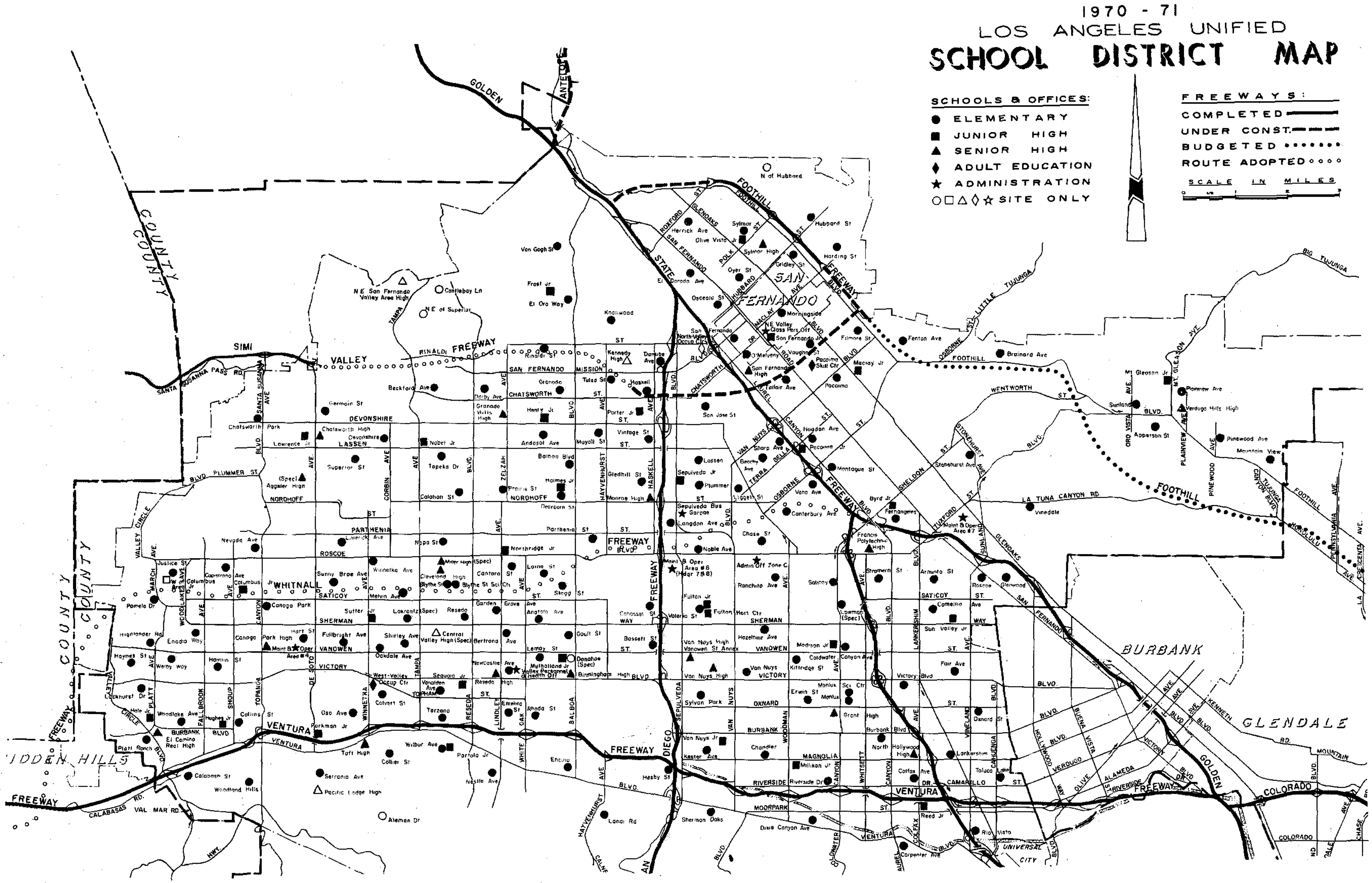


Figure 3.165 Location of schools in San Fernando Valley. Several of the school buildings in Sylmar were damaged by permanent differential foundation displacements but this damage would not have been hazardous to occupants.

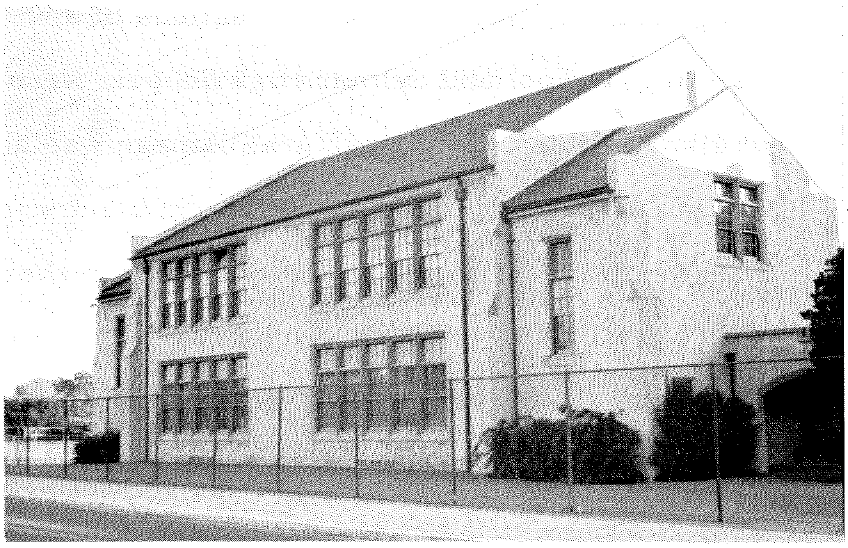


Figure 3.166 Morningside School at Maclay and Fifth Streets in San Fernando city. This well-constructed 1928 brick wall structure was severely cracked. It had brick corridor walls and crosswalls, concrete corridor slabs, and wood roof and floors. The gable walls had been anchored to the roof framing in recent years.

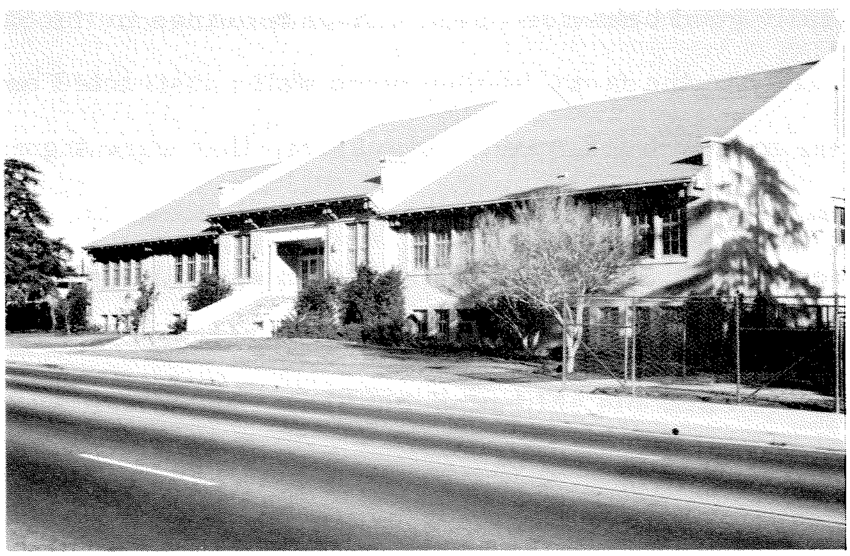


Figure 3.167 Morningside School at Maclay and Fifth Streets in San Fernando city. This 1915 building had been strengthened recently to make it satisfy the requirements of the building code for earthquake resistant design.

steel trusses over the gymnasium failed. The foundations of several school buildings, such as Van Gogh School and Sylmar High School, were disrupted by permanent ground displacements, but structural damage was not such as to constitute an undue hazard to occupants. However, in some cases, light fixtures and ceilings supposedly designed to withstand earthquake shaking fell and these would have been hazardous. It can be concluded that the newer school buildings in the San Fernando Valley, one or two stories in height, whose design and construction were supervised by the California State Office of Architecture and Construction, for the most part performed well during the earthquake. However, additional attention should be given to avoiding hazards arising from nonstructural damage such as falling light fixtures and ceilings, toppling equipment, etc.; the structural damage from shaking that a few of the schools received should also be investigated.

Some school buildings in adjacent communities, such as La Crescenta and Glendale, received some damage that was not of a major nature.

The generally good behavior of the school buildings in the San Fernando Valley is due mainly to the fact that they were well constructed one- and two-story structures, mostly wood frame and plaster, that were significantly stronger than the minimum requirements of the building code. The code governing the design of school buildings specifies essentially the same earthquake forces as the Uniform Building Code.

Of the pre-1933 schools, Morningside School at Maclay Avenue and Fifth Street in San Fernando (Fig. 3.167) was closest to the center of the earthquake. One of the two structures at this site was a good quality, two-story brick bearing wall building (1928) which was very badly cracked by the ground shaking. Although it would not have injured occupants during the earthquake, it did appear afterwards to be in a most dangerous condition, and it has been demolished. The main Morningside building was a two-story concrete-wall structure erected in 1915. The reinforcing bars were only in the exterior faces of the walls; the floors and roof were wood; the interior partitions were wood lath and plaster. A few years ago this structure was rebuilt to conform with the Field Act at approximately 60% of its replacement cost. Chases were made on the interior faces of the walls and reinforcing bars were gunited into place; floor and roof were anchored to the walls; some wall openings were filled in, horizontal diaphragms were provided, etc. The reconstructed building was designed for a 15%g acceleration force, and it came through the earthquake with no structural damage except some overstraining where the gable walls were anchored to the roof framing. Some plaster fell off of the 1915 interior partitions. This was a good test of the program for rebuilding old schools to meet the Field Act requirements, and it showed that the rebuilding did indeed make the structure safe during strong earthquake ground shaking.

Of the 110 old school buildings that had not been strengthened to conform with the Field Act, some did receive damage, notably Los Angeles High School, even though they were 20 miles or more from the center of the earthquake. This made it clear that these buildings would indeed have been very hazardous had the earthquake centered farther south, in which case a large number of the 110 old buildings would have been in the region of very strong shaking. Since the earthquake, most of the old buildings have been closed. A few were judged to be in a good condition and not unduly hazardous to the occupants, and these will be used for a limited period of time. A recent state law in California requires that all school buildings must meet the requirements of the Field Act by 1975, and if they do not, they must be closed.

Thus, the problem of the old, unsafe school buildings which first came to public attention in 1933 will finally be solved after 42 years.

Old Buildings

Earthquake-resistant design provisions were put into the Los Angeles Building Code in 1933 following the Long Beach earthquake. In the metropolitan Los Angeles area, there are many thousands of masonry bearing wall buildings that were erected before the Long Beach earthquake and were not designed to withstand earthquake forces. One of these, the Veterans Administration Hospital building in Sylmar, collapsed and killed 46 of the occupants. Most of the San Fernando Valley was built up since 1933, so most of the buildings were designed to resist earthquakes. The city of San Fernando, the Olive View Sanatorium and the Veterans Hospital, however, are older, and they did have a number of pre-1933 buildings that were damaged as was seen in earlier figures and in Figs. 3.168 through 3.174. Damage to old buildings occurred also in Pasadena (Fig. 3.175). Several old buildings in central Los Angeles, some 20 miles from the center of the earthquake, also had walls collapse, and one man was killed when the front wall of one of these buildings fell on him. It is clear that if the earthquake had centered some 20 miles farther south, in the old part of town, many old buildings would have collapsed, and if the shock had come during working hours, there would have been a large loss of life. The earthquake hazard to the occupants of old buildings should be recognized by the city governments in California and programs should be initiated to strengthen or to remove such structures.

Figure 3.168 Complete collapse of old brick wall building that was not designed to resist earthquakes. A service building at Olive View Hospital. Ralph Samuels photo.

Figure 3.169
Collapsed old
building at
Olive View
Hospital.

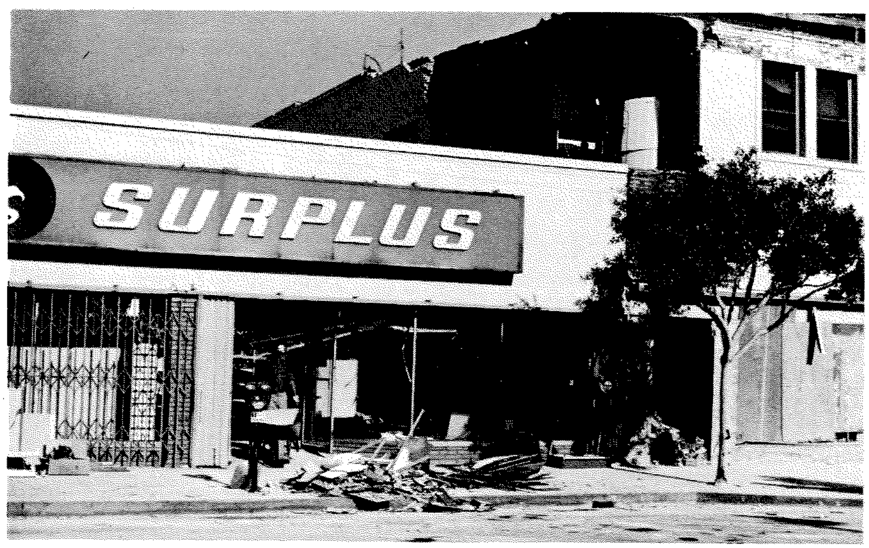


Figure 3.170 Wall of old building collapsed onto roof of adjacent building in San Fernando city.

Figure 3.171 Brick walls of old building in San Fernando city collapsed leaving floors supported on interior wood and plaster partitions.

Figure 3.172 This building was hazardous not only to its occupants but also to the occupants of adjacent buildings and to passersby on the street.

Figure 3. 173 This old building in San Fernando city was badly damaged by the earthquake. Bricks from the upper walls have fallen onto the sidewalk. Ralph Samuels photo.

Figure 3.174 Old San Fernando Mission located just south of Holy Cross Hospital and Indian Hills Medical Center, and three miles north of Holiday Inn. The mission building was first constructed around 1800 and has been rebuilt and repaired since then. Its walls are adobe brick (unburnt clay). There was sufficient cracking of the walls for the city to declare it unsafe for occupancy.

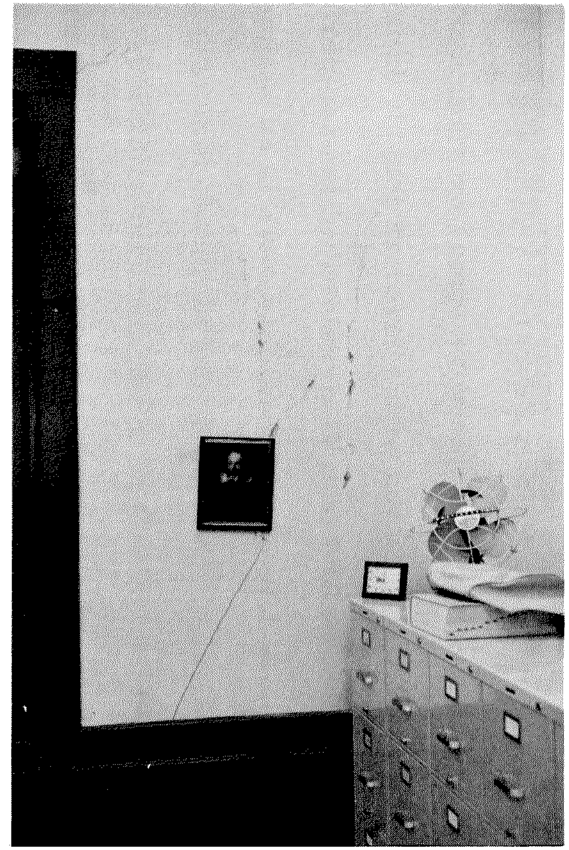


Figure 3.175 Hollow tile filler walls were cracked in old buildings as far away as Pasadena. Throop Hall, California Institute of Technology.

Residences

The large majority of houses in the San Fernando Valley are modern one-story structures and have a wooden frame with stucco exterior walls, consisting of a paper and wire mesh coated with a layer of roughly finished plaster about 3/4 of an inch thick. Overall, the residential houses performed well from the standpoint of the safety of the occupants. While an estimated 6,000 homes had damage, 450 of which were declared unsafe for occupancy, there were only two lives lost as a result of injury in residential houses. Considering the many thousands of occupied houses that experienced strong ground shaking and destructive ground displacements, this is indeed a small toll, markedly less than would be expected based on experience with many foreign earthquakes where large sections of residential dwellings experienced strong ground shaking with disastrous results. The San Fernando earthquake has convincingly shown again the high degree of earthquake resistance of the typical wood frame house.

The severe damage to residential dwellings was particularly extensive in two regions. One region follows a zone of permanent surface distortion starting in the vicinity of the intersection of Hubbard Street and Glenoaks Boulevard and runs easterly across Foothill freeway just northwest of the Maclay crossing and on into the foothills to the east (Figs. 1.2, 1.5, 1.6 and 1.7). Many of the houses within a block of the large surface displacements suffered extensive damage; structures were shifted from their foundations with horizontal and vertical cracks running completely through walls and in some cases completely through houses. However, no completely collapsed houses were found in this area. Houses more than one block away from the large surface displacements suffered only a few isolated structural failures visible from the street. It appears that ground displacement and ground deformation were the major causes of damage to the single-story residential houses in this region.

Two- and three-story apartment dwellings in the Sylmar-San Fernando area did not endure the earthquake as well as the smaller single-unit residential houses. Many of the apartment buildings had large open-area garages at the ground level and probably had a lateral strength only slightly greater than that for the one-story houses, but with considerably more structure above the ground level to be supported. Figures 3.176 - 3.179 show the extensive damage to two such apartment buildings located on Foothill Boulevard, near Maclay Street, directly on the surface faulting. Many of the wood framed apartment houses, which were larger and taller than the single unit houses, suffered extensive damage at locations well away from the surface faulting. One rather extreme example of an older dwelling is pictured in Fig. 3.180.

The other region where the residential houses suffered extensive damage was along the northern few blocks of Sylmar, at the base of the San Gabriel Mountains, between the Olive View Hospital and the Veterans Administration Hospital (Fig. 3.181). No surface breaks were observed in this region, yet the damage was very severe to the split-level houses along Almetz and Aldergrove Streets. Numerous houses were shifted from their foundations, some were damaged beyond repair (Fig. 3.182), chimneys fell, and nearly all of the concrete block walls toppled. Evidence indicates essentially all of the residential damage in this area was due to horizontal ground shaking. One common mechanism of failure was found in most of the houses in this area and is similar to that found in the old timber buildings at Olive View Hospital. The wood cribbing, on which the living quarters were supported, had insufficient lateral bracing (Fig. 3.183). During the earthquake the living quarters

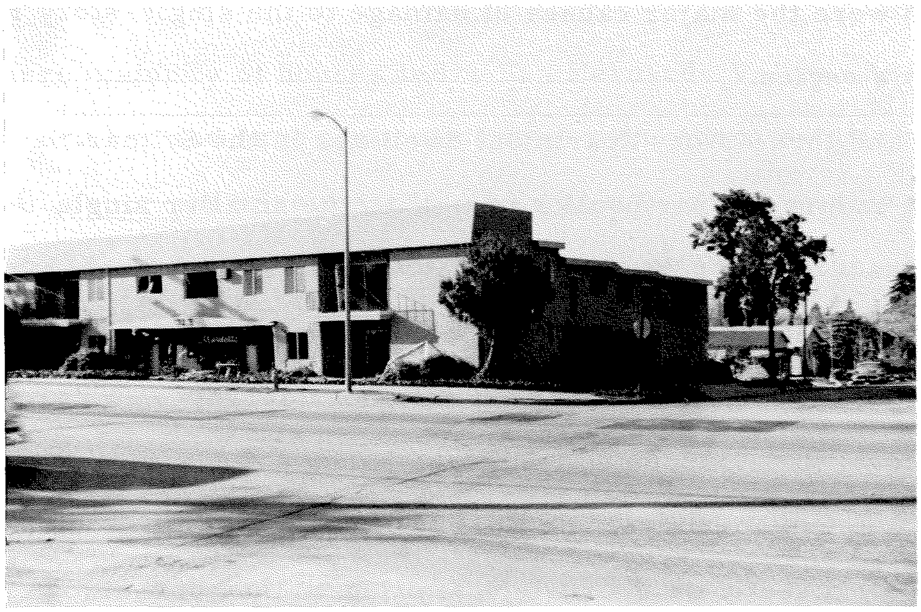


Figure 3. 176 Mardette Apartment Buildings. Northwest building. Foothill Boulevard, San Fernando.

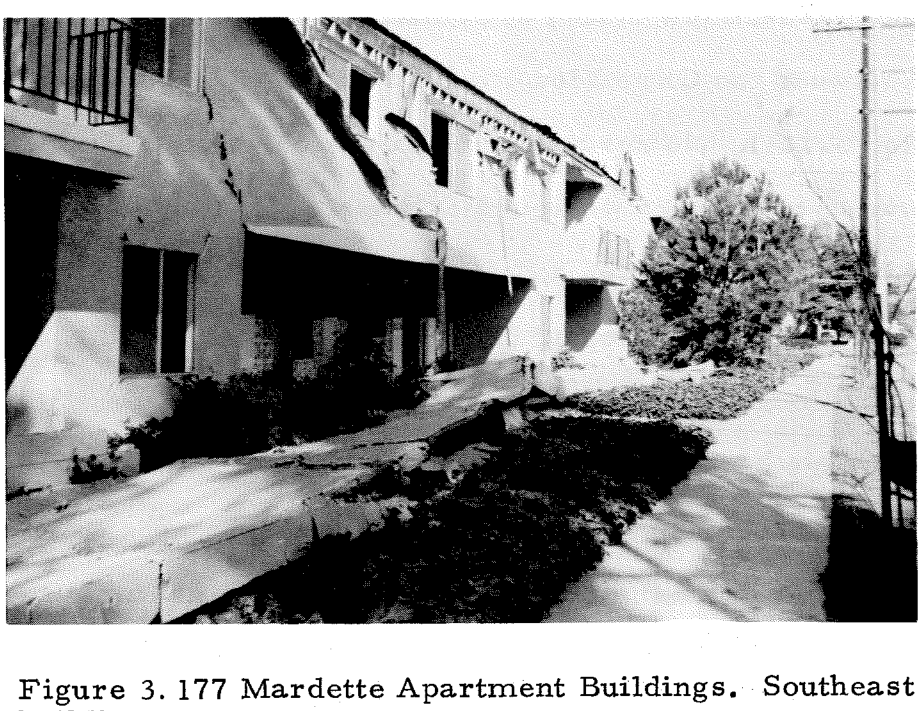


Figure 3. 177 Mardette Apartment Buildings. Southeast building.

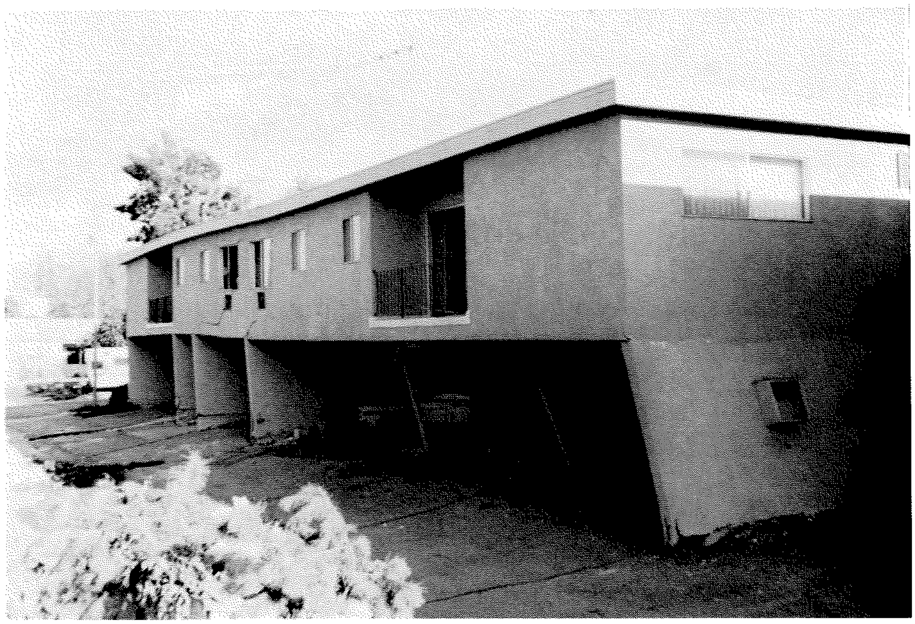


Figure 3.178 Rear wall northwest building. Mardette Apartments.

Figure 3.179 Wall displacement, rear of northwest building, Mardette Apartments.

Figure 3.180 Two-story wood apartment building on the corner of Fourth Street and Brand Boulevard, San Fernando.

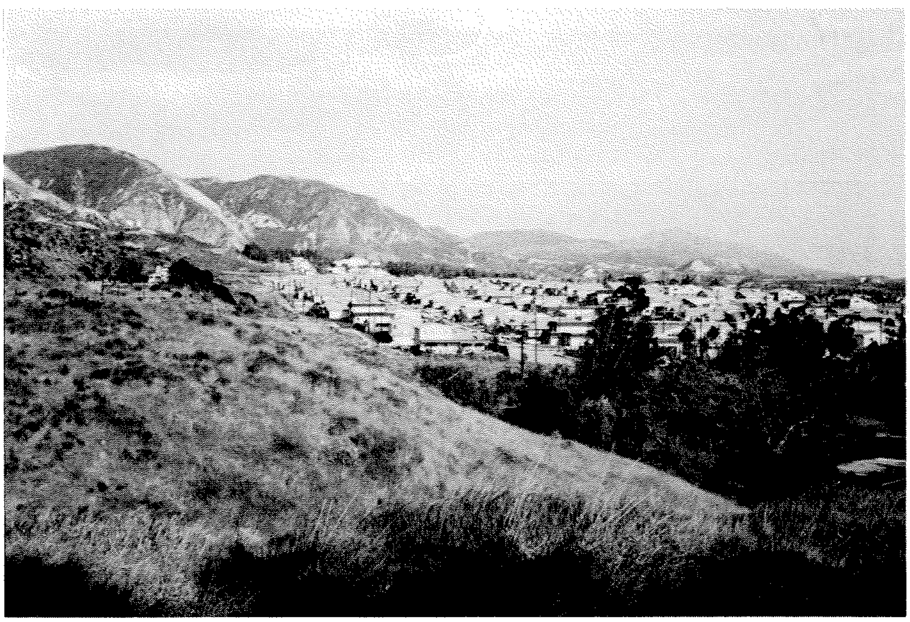


Figure 3.181 Looking east toward the Veterans Administration Hospital at the northern few blocks of Sylmar (Almetz and Aldergrove Streets) where extensive residential damage occurred.

Figure 3. 182 Two of the most severely damaged split-level houses on Almetz Street.

Figure 3. 183 A house on Aldergrove Street that shows horizontal movement on its wood supports.

of several of the houses moved sideways enough so that they fell off their supports and in so doing, split off from the garage portion of the house (Fig. 3.184). Additional bracing in the crawlspace under these houses could have prevented many total losses.

Another equally important type of failure was found in the many two-story and split-level houses. Because the Los Angeles Building Code does not delineate between the earthquake requirements of one- and two-story residential houses,* the two-story houses were more vulnerable to horizontal ground shaking than the one-story houses. This problem was particularly acute in the split-level houses with a bedroom over the garage (Figs. 3.182 and 3.184). The large garage area with a nearly open wall for the door has insufficient lateral strength to support the horizontal inertial loads of the overhead structure. As a result, there were many distorted houses with garage doors that would not close. As pictured in Figs. 3.185 and 3.186, two such houses on Fenton Street suffered partial collapse. The garage walls failed, and the overhead bedrooms collapsed to ground level. Many of the heavily damaged houses in this area contained both weaknesses described above: insufficient lateral bracing in the garage and insufficient lateral bracing in the crawl space under the house.

This area of the San Fernando Valley may have experienced more intense ground shaking than areas a few blocks farther to the south, but this cannot be stated with confidence because relative damage is not a reliable measure of the relative strength of ground motion. This residential area was subjected to very strong ground shaking but it may have been more vulnerable

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*The code earthquake requirements for residences consist of specifications for bracing, anchor bolts, etc. The ordinary house is not designed by an engineer.

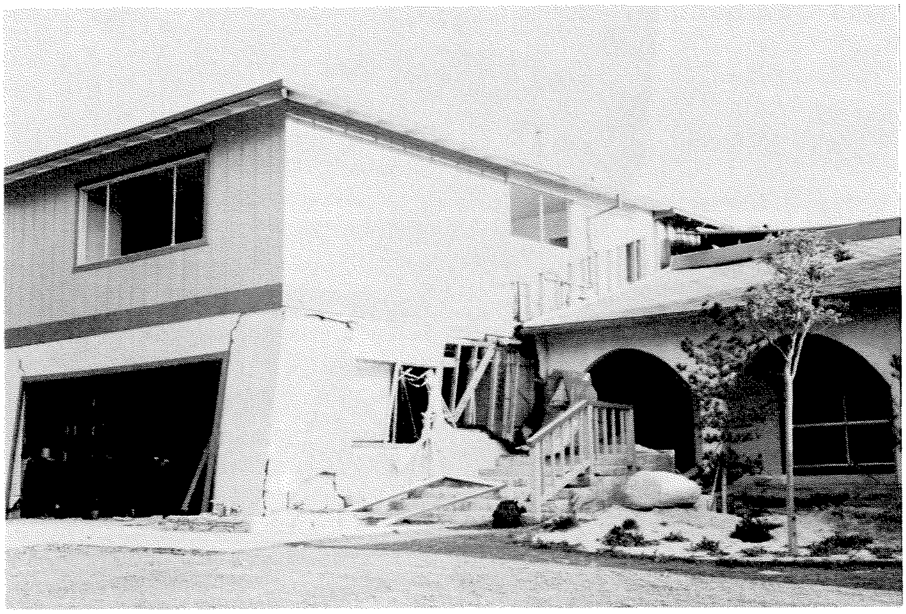


Figure 3.184 A split-level house on Almetz Street; the living quarters fell from the underlying supports.

Figure 3.185 Garage walls collapsed from under the overhead bedroom of this split-level house on the corner of Fenton and Tyler.

Figure 3.186 Garage walls that supported an overhead bedroom collapsed in this split-level home on Fenton Avenue between Polk and Tyler. Ralph Samuels photo.

to earthquake damage because of construction details noted above.

Concrete block walls were a common feature in this region of northern Sylmar. The large majority of the walls had vertical reinforcing bars at widely spaced intervals (10 to 12 ft) and often these were not bonded to the wall by concrete or mortar. Most of these walls failed (Fig. 3.187). Brick chimneys in the area also suffered widespread failures (Fig. 3.188). Four 1/2-in vertical reinforcing bars were present in most of the chimneys, but sometimes they were not grouted in place. It was observed that the chimneys were generally not tied to the house except at the roof level.

Similar damage to chimneys and walls was observed in Granada Hills residential area northwest of upper Van Norman reservoir.

Just east of the Veterans Administration Hospital a number of wood-frame houses in various stages of construction provided some interesting information pertaining to the lateral strength of residential houses. The frame and roof of the houses were essentially complete at the time of the earthquake. From the response of the houses, it appears that the wire mesh and paper backing placed on the exterior walls in preparation for a coat of stucco provided a significant amount of lateral strength. These houses with the wire-paper lateral support fared much better than those houses with just a frame and a roof (Figs. 3.189 and 3.190). In fact, nearly all of those without the seemingly small reinforcement provided by the wire and paper collapsed. On the other hand, one house in the development with a newly completed stucco exterior suffered no visible damage. Another interesting feature was the fact that although the houses were of similar design, there was no apparent consistency in the direction of collapse.

Semi-permanent housetrailers in the San Fernando Valley were not immune to earthquake-induced damage. Many such trailers were shifted horizontally on their supporting jacks, and in some instances the jacks punctured through the trailer floor. Considerable damage was inflicted in

Figure 3. 187 Concrete block walls fell in the back yards of the houses along Almetz Street.

Figure 3. 188 Typical chimney with four half inch vertical reinforcing bars, structurally independent of the adjacent house, Aldergrove Street.

Figure 3. 189 Heavily damaged and collapsed residential houses under construction, east of Veterans Administration Hospital.

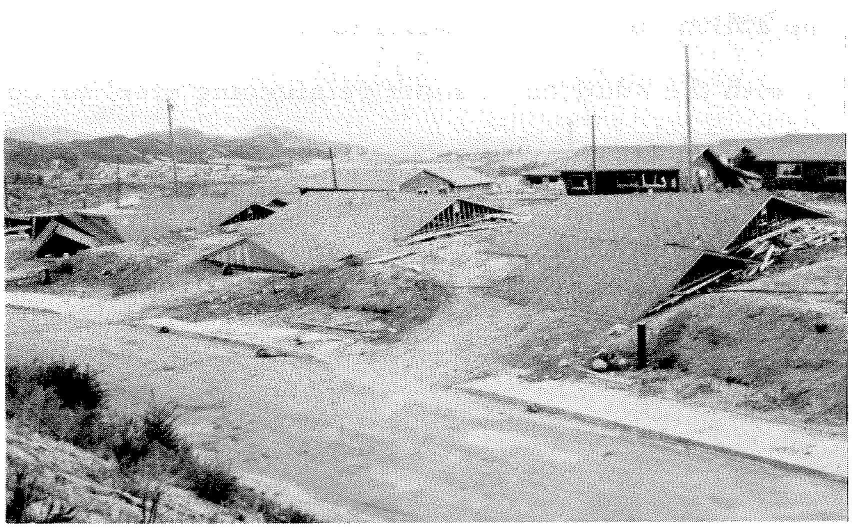


Figure 3. 190 Heavily damaged and collapsed residential houses under construction, east of Veterans Administration Hospital.

a trailer park at the mouth of Lopez Canyon, immediately adjacent to a major surface fault displacement (Figs. 1.2 and 1.6). The typical trailer support has little lateral resistance and the amount of damage could have been reduced markedly by modest improvements in the design of the supports.

Masonry components such as stone facing, chimneys, and concrete block walls failed over a much wider region than the structural elements of houses. Many broken and collapsed concrete block walls were observed up to four miles south of the southern edge of the San Gabriel Mountains and up to eight miles southwest of Olive View Hospital. The walls that failed at the greater distances from the intense ground shaking had obvious deficiencies; in some cases the hollow-core concrete blocks were laid without any supporting columns or reinforcing steel or the steel was not bonded to the wall. Others, with widely spaced concrete-filled block columns suffered damage between the columns. The ability of the block walls to withstand ground shaking was improved greatly with the addition of some reinforcing steel or concrete filler in the hollow cores of the walls. There were numerous examples of concrete block walls that performed well even under intense ground shaking. No damage was noted around the exterior of a concrete block church with some surrounding concrete block walls located on the corner of Polk and Fenton Streets (just one block from two split-level houses with collapsed garages). Block construction was used extensively at the San Fernando Valley Juvenile Hall, including a 12- to 14-foot high block fence (Fig. 3.95) that survived the earthquake well except in area of permanent ground displacement.

<u>Summary</u>

In the San Fernando earthquake, damage and loss of life was much greater in old buildings than in post-1933 buildings that had been designed to resist earthquake forces. It is clear that the greatest earthquake hazard in southern California comes from the many old buildings and, hence, it is important to strengthen these or to phase them out of existence.

In comparison to the old buildings, the newer buildings that had been designed to resist earthquake forces behaved well, with damage held within acceptable limits, except in the northern San Fernando Valley where the ground shaking was in the range 30 to 50%g peak acceleration. In this region of very strong shaking some new buildings were severely damaged and a few collapsed. On the other hand, in the same area, some code-designed buildings survived with no significant structural damage. Improvements should be made in the building code to eliminate the possibility of structures sustaining collapse or damage so severe as to be hazardous, even when subjected to very strong ground motions. The changes in the building code should be aimed at achieving a consistent degree of earthquake resistance for structures of different types which is adequate for survival of strong shaking without excessive damage.

Some specific improvements that should be made in the building code are as follows:

1) Concrete members should be designed for greater ductility with closer spacing of tie bars, where large bending moments and strains may be incurred.

2) Careful attention should be given to the location of reinforcing bar splices for these can have an influence on damage.

3) The design of shear walls should be reviewed and revised to improve the ability to survive strong ground shaking without severe cracking and local failure.

4) Methods of connecting wood roofs to masonry or concrete walls of one-story industrial and commercial buildings should be improved and the rigidity of the wood roofs should be increased.

5) Lightweight concrete, when used for columns and beams, appears to shatter badly when overstrained and, therefore, special reinforcement should be provided. When lightweight concrete is used for floors it should not be run through highly stressed shear walls for this leads to zones of weakness.

6) The interconnection of tilt-up wall panels should be improved.

7) Buildings that survived without structural damage often had appreciable architectural damage and costly damage to mechanical and electrical equipment. In many cases considerable expense was incurred in repairing damage to plaster walls, ceilings, light fixtures, windowpanes, elevators, air conditioning equipment, emergency power supplies, etc. Much of this could have been avoided at little extra cost by giving consideration to earthquake forces and building deformations when the building was designed.

It appears that most of the needed strength can be designed into future buildings with little additional cost of construction, particularly if the structural layout of the building is made with a view toward improving earthquake resistance.

SAN FERNANDO EARTHQUAKE, 9 FEBRUARY 1971 PRELIMINARY SOIL ENGINEERING REPORT

by

R. F. SCOTT

General Geological and Soil Conditions

The area of the San Fernando Valley most affected by the earthquake is a region of high topographic relief with elevations ranging from 1000 ft in the valley floor to 4000 ft in the San Gabriel Mountains, four to five miles to the north. (See Figs. 1.2 and 4.16). Its geology is diverse and complicated, including rocks whose ages extend from the present back to Precambrian, north of the San Gabriel fault. The following description of the geology of the area is based mainly on the work of Oakeshott (1958). The region is extensively divided by faults, generally with trends varying from southeast-northwest to east-west. The faults, which are of the thrust type, dip to the north at angles between 30° and 60° to the horizontal; they represent the response of the rocks to the north-south compressional forces in the crust of this part of California. The rocks of the valley floor have been over-ridden by southward movement of the older rocks of the San Gabriel Mountains.

In succession, proceeding from south to north, the underlying rocks consist of a folded and faulted sequence of rocks of upper Miocene and Pliocene ages of the Modelo, Repetto and Pico formations which form the Mission Hills. A local unconformity exists between these formations and the Saugus formation of Lower Pleistocene age to the north. The Saugus formation forms a syncline with an approximately northeast-southwest axis in the west end of the valley, north of the Mission Hills. The axis orientation becomes east-west to the east and the rocks become more tightly folded in this direction, with the section becoming thinner. Just south of the contact between the Pleistocene rocks and the diorite gneiss and granodiorite of Pre-Cretaceous and Lower Cretaceous ages respectively, the folding develops into a syncline followed by an anticline with nearly vertical beds in a horizontal distance of about 3000 ft. The contact between the two rock types occurs at the base of the San Gabriel foothills less than a mile north of the east-west section of Foothill Boulevard near the Olive View and Veteran's Hospitals, and continues east and southeast through the spur of the foothills to the north of the Hansen Flood Control Basin. (Figure 1.2). This foothill section is then composed from the south of rocks of Pliocene age which meet unconformably with the Pleistocene rocks of the Saugus Formation. Landslide areas are present along the limbs of the syncline and anticline.

The Modelo and Pico formations consist of marine sandstone, siltstone, and conglomerate, while the Saugus is a continental sandstone and conglomerate. Along the canyons in the hills and foothills of the area are found Recent terrace deposits of sands, silts and gravels, and the seasonal flows from the canyons have deposited Recent alluvial sand, silt, and gravel over the San Fernando valley floor. Locally, the alluvium may reach depths of up to several hundred feet.

In contrast to other parts of the Los Angeles area, the water table in the San Fernando and Sylmar areas is close to ground surface, being encountered at depths up to a few tens of feet. The water table is very close to the surface in the vicinity of the upper and lower Van Norman Lakes. Groundwater in the valley generally drains to the southeast and east.

Finer-grained alluvial soils occur in the vicinity of the Van Norman Lakes and grade into coarser materials towards the mountains. The combination of finer-grained materials and high water tables gives rise to generally lower soil density and poorer foundation conditions around the lakes.

At the northern shore of the upper Van Norman Lake the upper 15 to 30 ft of the soil consists of loose soft silt, sand and clayey-silt mixtures, with occasional gravel. Lenses of finer-grained materials are common. At greater depths the soil is sandier and becomes considerably more dense.

As a consequence of the topographical relief and the varied soil and rock conditions, post-earthquake examinations of the area most affected revealed a variety of soil and rock faulting, cracking, and fissuring, as well as vertical and horizontal ground displacements of up to several feet (apart from the lower San Fernando dam whose movements were substantially greater). It is not easy to separate the surface evidence of merely local movements of slumping and sliding from those related to tectonic motions of the underlying geological structure, and it is likely that in time the picture will change from that presented here. At present, well-defined east-west zones of faulting have been mapped (Kamb et al., 1971) (a) along the base of the foothills north of Hansen Basin, (b) through San Fernando a few blocks north of the airport, and (c), north of the Veteran's Administration Hospital. However, pressure ridges and overthrusts run north-south along the base of the west side of the foothill spur north of Hansen Basin, and are accompanied by parallel cracks to the east behind the top of the slope. In addition, northeast-southwest en echelon cracks forming an east-west zone of fracture were observed to run through both natural rock and filled ground to the west of the Juvenile Facility and in the vicinity of the intersection of the Golden State and Foothill freeways. Extensive ground fracturing and displacements were observed around the margins of the upper Van Norman Lake, and in the Metropolitan Water District (MWD) Jensen Treatment Plant. A zone of fracturing and ground displacement runs from north of the Juvenile Facility southwest through the San Fernando Road, the Golden State freeway, and the Sylmar Converter Station. Ground fractures were observed in the vicinity of the Olive View Hospital, and in many other locations in San Fernando and Sylmar.

Slope failures and incipient slope failures occurred in a number of areas. The largest occurred in cuts on both sides of the Golden State freeway near its intersection with California State Route 14, and on the hill northwest of the Jensen Treatment Plant. Both upper and lower San Fernando dams experienced slides. Rock slides occurred in many canyons and were especially noticeable in Pacoima Canyon. Knowledge of the precise magnitudes and directions of ground movements will await the results of detailed surveys.

Effects of Earthquake

(a) Dam Failures; Slope Stability.

Three earth dams impound water in the Van Norman Lakes. The lower or more southerly San Fernando dam measures approximately 2000 ft along the crest and is about 140 ft high. The upper San Fernando dam has a length of 1200 ft along the axis and is about 60 ft high. A smaller dam finished in 1970 retains water in a bypass reservoir adjacent to the lower Van Norman Lake. It has a crest length of about 600 ft and a maximum height of 90 ft. The lower dam was initially built of hydraulic fill between 1912 and 1915 and was subsequently raised and modified in 1929/1930 and 1940 by dry fill and rolled fill additions. The upper slopes have a gradient of 2-1/2 horizontal to 1 vertical. The upper dam was built later in 1919-1921 also of hydraulic fill, and has a concrete-faced upstream slope of 2-1/2 to 1, and the same downstream slope interrupted by a berm 100 ft wide. The slopes of the bypass dam which was placed in service in 1970 are

2-1/2 to 1 upstream and 3 to 1 downstream; they are protected by a layer of asphaltic concrete. Cross sections of the three dams are shown in the attached Figure 4.1, and a photograph, Figure 6.39, shows the lower and bypass dams a few months before the earthquake. The lower San Fernando dam suffered minor damage in an earthquake of 30 August 1930.

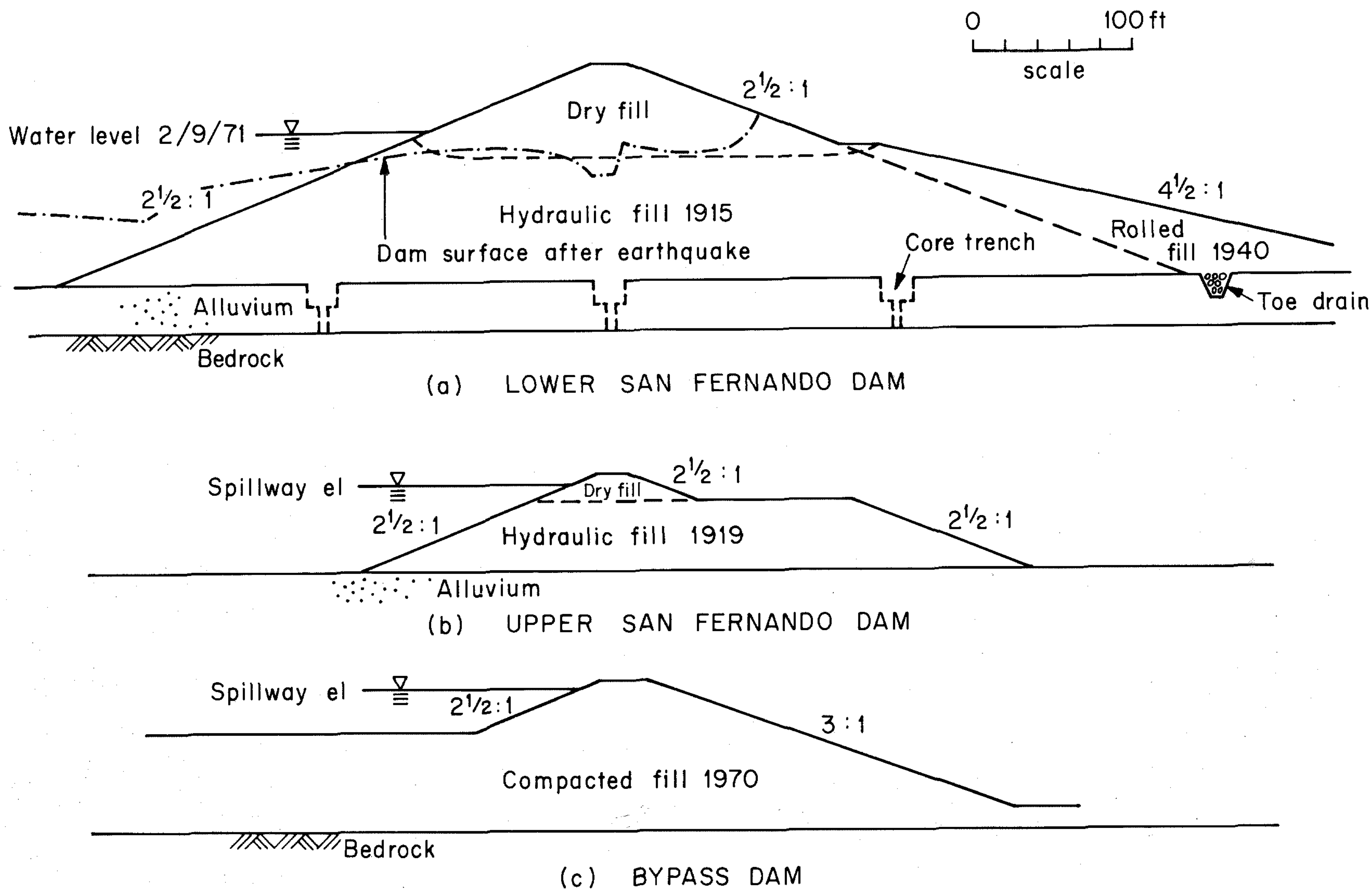
Although the capacity of the lower Van Norman Lake is 20,000 acre-feet, it was only slightly more than half full at the time of the earthquake. The capacity of the upper lake is 1800 acre-feet, and it was full and spilling over the spillway at the time of the earthquake. This is a normal operating condition. The bypass reservoir has a capacity of 275 acre-feet, and it was also, as is normal, full and spilling over its spillway at the time of the earthquake.

Two seismoscopes were installed at the lower San Fernando dam, one on the east abutment and one on the crest. Good records of the earthquake were recovered from both instruments although the peak values are off-scale. Estimation of the peak values gives the following numbers on the approximate relative velocity response spectrum at a period of 0.75 secs and 10% damping.

Seismoscope	Component	Spectrum value ft/sec
Abutment	NS	2.1
	EW	2.0
Crest	NS	1.6
	EW	1.6

These may be compared with a value of about 1.1 ft/sec for the average of the two horizontal components of the 1940 El Centro record and an average of about 2.8 ft/sec from the two horizontal components of the Pacoima Dam record in the 1971 San Fernando earthquake, both at the same period and damping. Although the NS and EW components of the record on the crest are equal, it is of interest that the spectrum value obtained from the record on the crest is slightly greater for the component resolved parallel to the dam axis (1.7 ft/sec) than for the component perpendicular to the axis (1.2 ft/sec).

The lower dam suffered a massive slope failure along almost the entire upstream surface as a consequence of the earthquake, (Figs. 4.2,



TYPICAL PROFILES OF SAN FERNANDO DAMS

Figure 4.1 Cross section of three San Fernando earth dams.

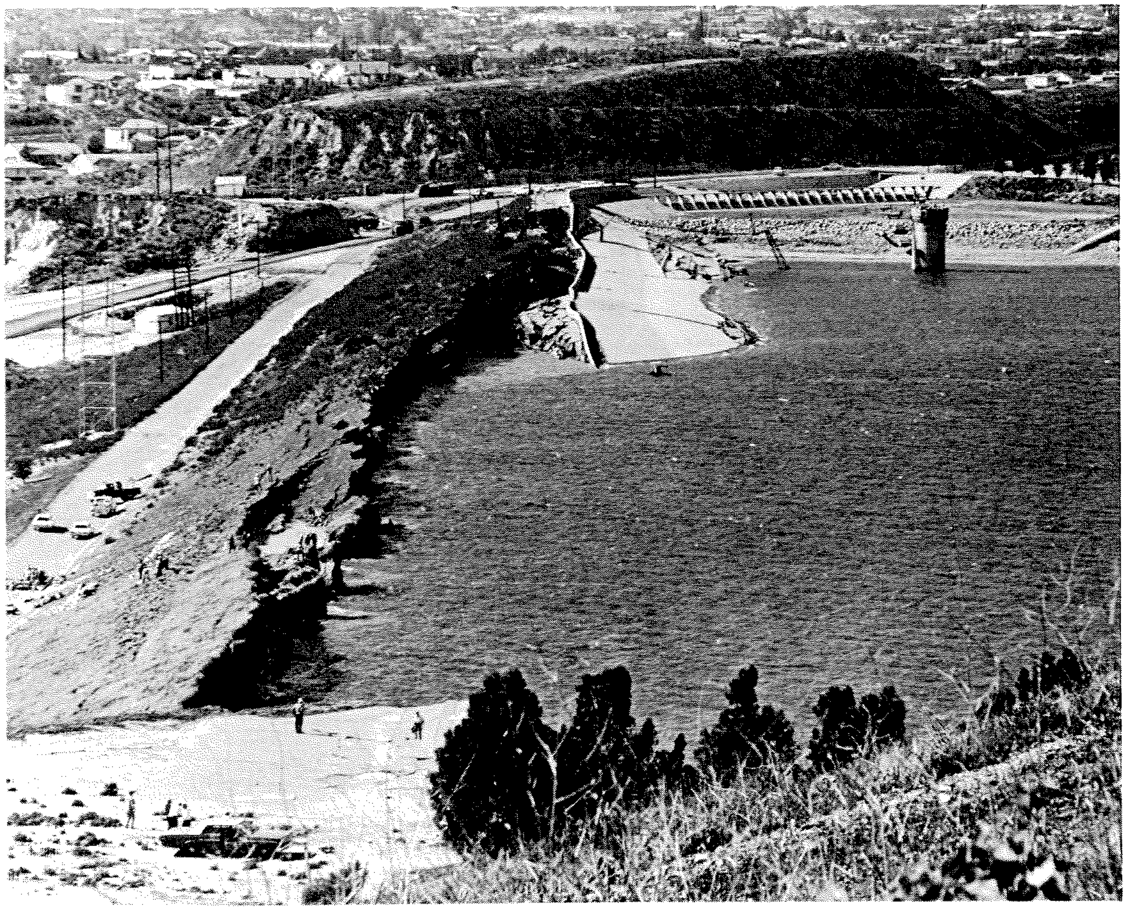


Figure 4.2 Oblique view of lower San Fernando Dam, 9 February 1971, looking west. The upstream face of the dam had a concrete slab facing to prevent erosion of the dam surface.

4.3 and 4.4) although the reservoir water, which was at a low level, was prevented from catastrophic release by a fortuitous freeboard of a few feet at the remaining earth fill section. One of the two outlet towers toppled over. The sliding soil in the slope failure may have contributed to the tower failure. The upper dam underwent a partial failure of the downstream slope, (Fig. 4.5) so that the crest settled about 3 ft and moved about 5 ft laterally downstream. Although the upper lake was full, no water escaped. In the case of the upper dam, the failure apparently took place along a curved surface of shearing through the soil, since pressure ridges are visible at the toe of the slide, where the sliding mass encountered passive resistance (Fig. 4.6). With the drawdown of the reservoir the complete surface of the slide mass at the lower dam can be seen (Fig. 4.4).

It was reported by the Los Angeles Department of Water and Power that an examination of the abutments of the lower dam showed no sign of damage due to faulting or geological structural movements. Seepage as measured in the drainage systems downstream of each dam increased after the earthquake. In the lower dam it soon returned to normal, but part of the drain system in the upper dam continued to show increased flows, indicating the possibility of some damage. Pore pressures as indicated by observation wells at both dams increased slightly after the earthquake, but returned to normal in a few days.

The surface of the compacted-fill bypass dam is reported to have settled 0.4 ft. Cracks are apparent in the asphalt surfacing of the dam, some open and some overlapped. Excavation through the asphalt at these cracks and into the fill has revealed that there are no cracks or rupture surfaces in the fill below the cracks. The dam appears otherwise undamaged and it seems likely that the surface layer of asphalt fractured

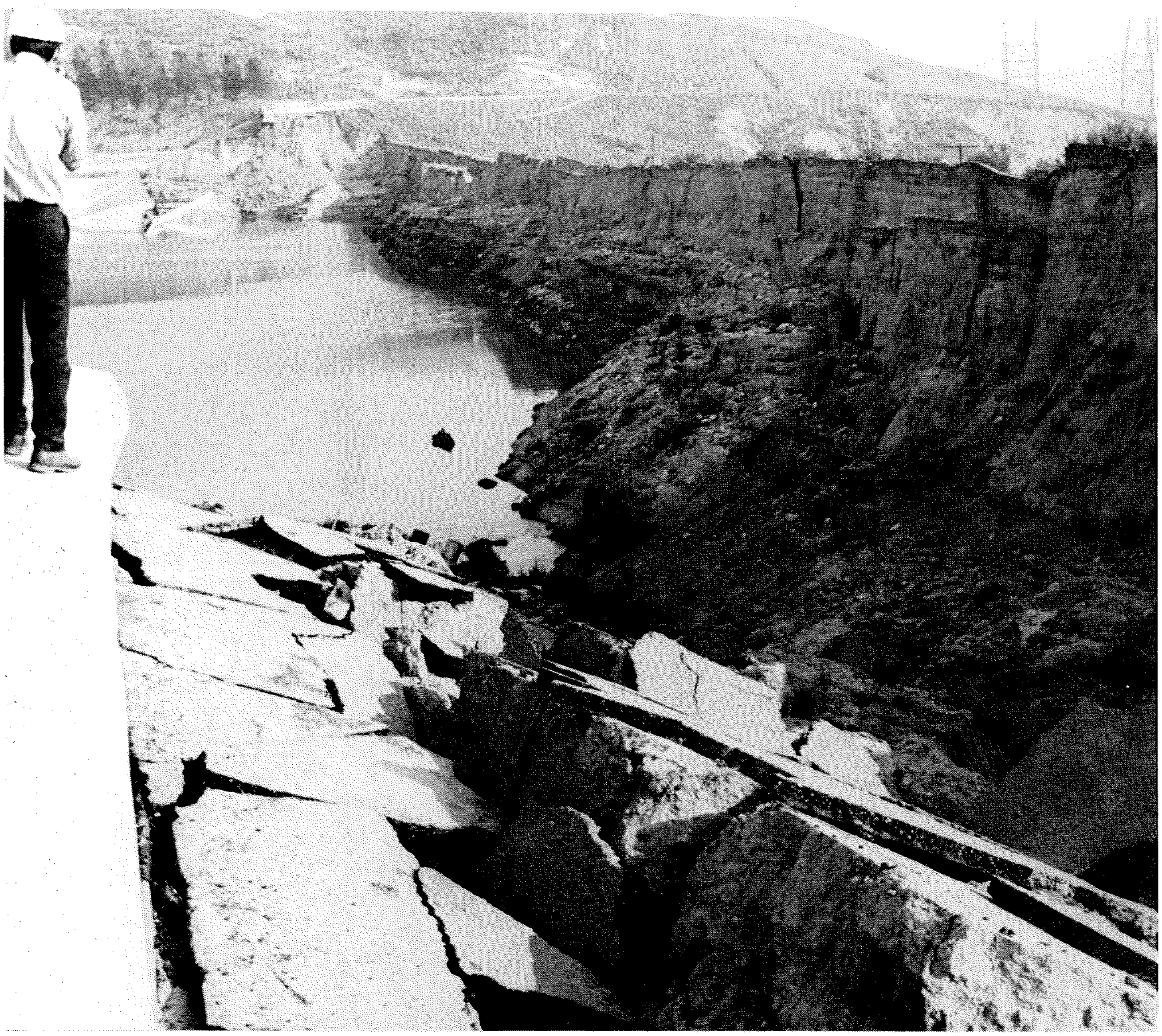


Figure 4. 3 Ground view of crest of lower San Fernando Dam, 21 February 1971, looking east.

Figure 4.4 Vertical aerial view of lower San Fernando Dam, March, 1971, showing dam after the reservoir had been drained.

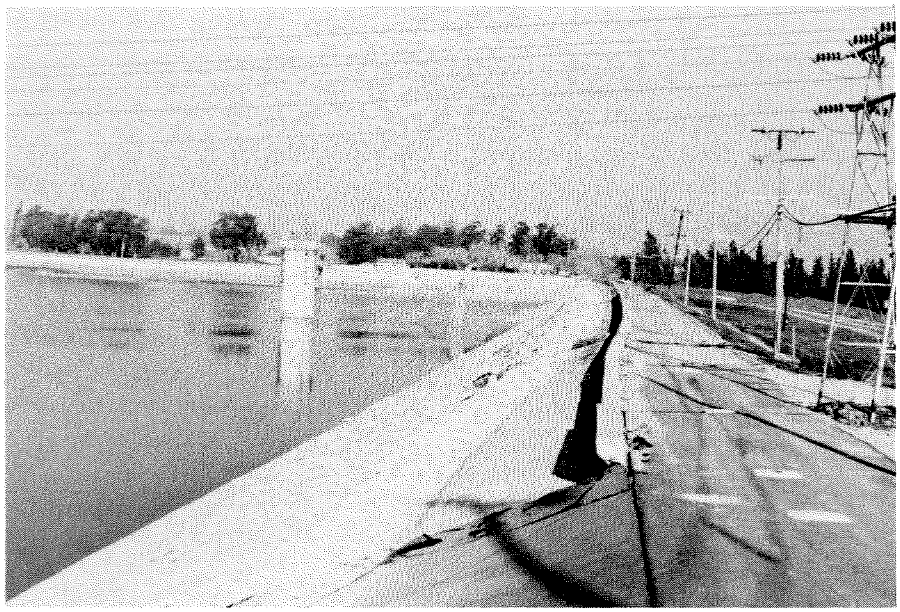


Figure 4. 5 Downstream displacement of upper San Fernando Dam, looking east. Picture taken 21 February 1971.

Figure 4. 6 Displacement at downstream toe of upper San Fernando Dam. Dam is to left; thrusting has crushed concrete box in foreground. Pressure ridge crosses side road behind box and passes to right of bushes in middle distance. 21 February 1971.

as a result of the dynamic movements of the dam during the earthquake. It will be of interest to calculate the displacements when it is possible to estimate the nature of the generating vibrations in the base rock.

Although ground displacements and cracks due to underlying geological structural motions and faulting have occurred in a number of locations in the area affected by the earthquake, such movements do not appear at present to have contributed to the soil failures in the upper and lower Van Norman dams. Instead, these failures seem to have been related primarily to the shaking which the dams experienced. Since the strong ground motions lasted only a few seconds, whereas the displacements of the failed section of the lower San Fernando dam (Fig. 4. 4) must have taken at least several minutes to develop, it appears that the earthquake triggered the failure, which continued to develop after the strong ground motion ceased. This would imply a change in the material properties of the dam, generated by the earthquake. The appearance of the failure in Fig. 4. 4 suggests that liquefaction may have played a part in the movements.

A variety of types of slope failure, slumping, and ground displacements associated with surface relief has occurred throughout the area affected by the earthquake. Numerous rockfalls occurred in the San Gabriel mountains and were most conspicuous in Pacoima Canyon (Fig. 4. 7) where they blocked access to Pacoima Dam and the adjacent strong-motion accelerograph for a number of days.

Slumping took place extensively around the perimeter of the upper Van Norman Lake and along the inlet channel to the lake (Figs. 4. 8 and 4. 9). The displacements appear to be due to movement of the relatively soft saturated sediments toward the lake, and are particularly noticeable around the peninsula supporting the solar observatory at the north end of the lake.

Figure 4.7 Oblique aerial photograph of 15 March 1971, showing numerous rockfalls in Pacoima Canyon.

Figure 4.8 Aerial photograph of Jensen Treatment Plant, looking north, on 11 February 1971. Slope movement has occurred on hill to west. Upper Van Norman Reservoir is to the right.

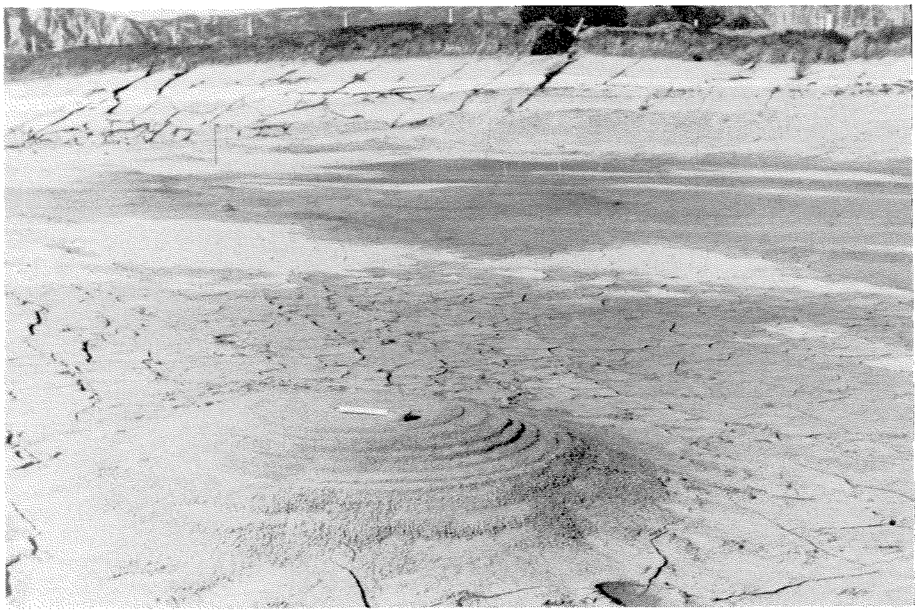


Figure 4. 9 Slumping around perimeter of upper Van Norman Lake, as seen on 5 March 1971. Sand boil in foreground has been differentially eroded by falling lake level.

Figure 4. 10 Incipient slope failure adjacent to Interchange of Foothill and Golden State freeways, 11 February 1971.

At the MWD Jensen Water Treatment Plant, substantial ground movements took place toward the lake, (Fig. 4.8) causing extensive damage to a number of structures at the plant which was nearing completion. The ground movements appear to be related to the location of fill at the site, since a line separating cut from fill in the area also appears to divide the heavily-damaged portions of the plant from those in which less damage is apparent. A large incipient slope failure is also apparent in the cut-back hill to the west of Balboa Boulevard adjoining the treatment plant. A number of slope movements have also occurred in the vicinity of the freeway interchange nearby (Figs. 4.10 and 4.11).

Ground surface displacements which have been attributed to geological faulting have occurred along the east-west trending base of the San Gabriel foothills, just north of Foothill Boulevard in San Fernando. However, similar thrust displacements also occur along the north-south base of the foothills west of Lopez Canyon. In the foothills above these thrusts and between them and Lopez Canyon, cracks and ruptures appear in the ground, with a north-south orientation. Since both the base thrusts and the apex cracks run at right angles to the direction of the recognized faulting, it seems possible that these movements have developed as a consequence of slumping on a massive scale on the north-south trending portion of the foothills in this area.

Ground Displacements; Liquefaction

Many surface cracks and some ground displacements are apparent in areas where they cannot readily be explained on the basis of topographic relief or geological faulting. The most conspicuous and damaging of these occurs in a zone of vertical and lateral displacements about 900 ft in width extending from a location in the orange groves northeast of Sylmar Juvenile

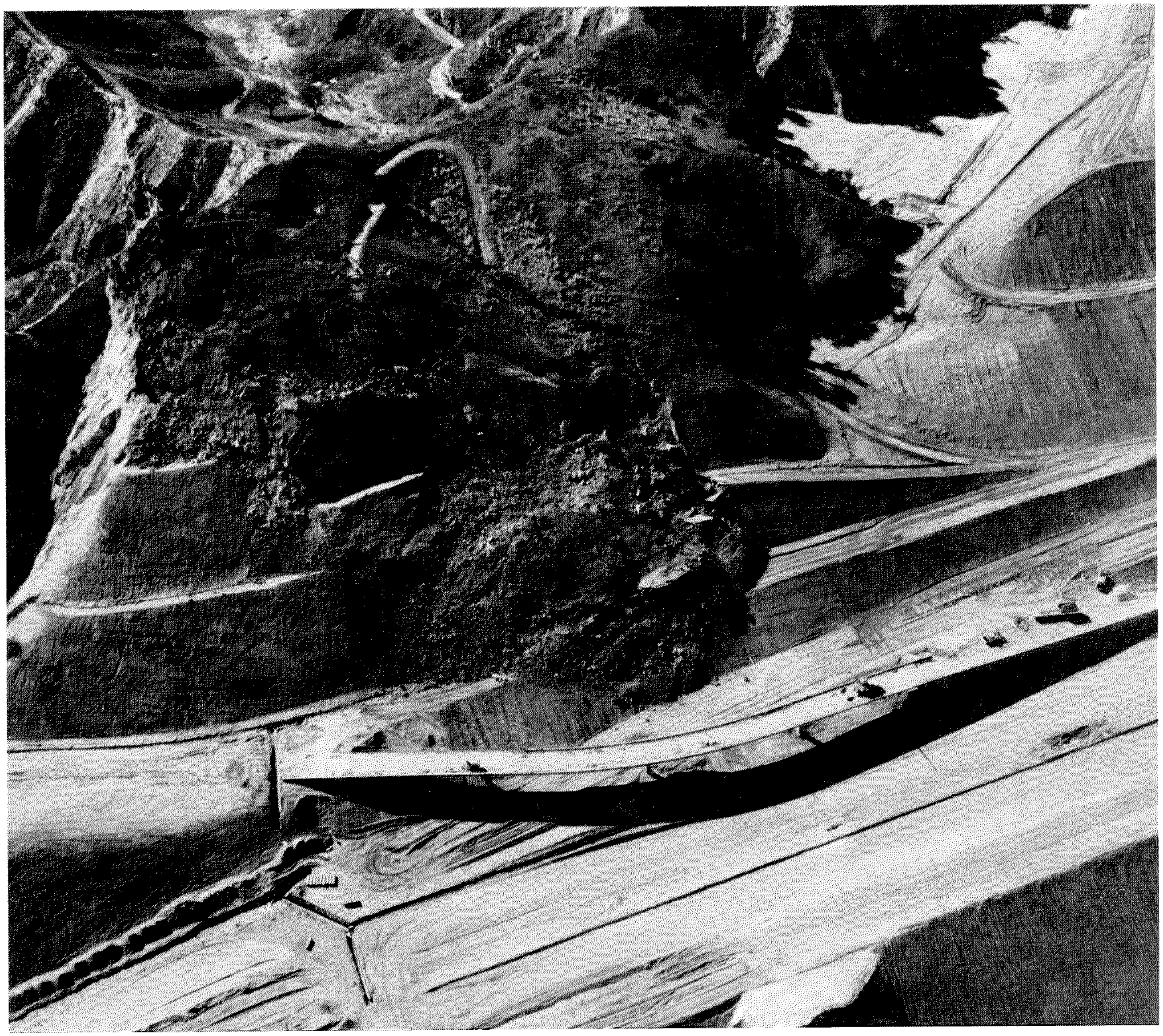


Figure 4.11 Slope failure on the west side of the Golden State freeway (Interstate 5), near the interchange with the Foothill freeway.

Hall to the upper Van Norman Lake, a distance of about 4000 ft to the southwest (Fig. 4. 12). This zone passes through the Juvenile Facility, across the Southern Pacific Railroad tracks, San Fernando Road, and Golden State freeway, and into the Sylmar Northwest Intertie Converter Station (Fig. 4. 13). Settlements and displacements have also occurred between the converter station and the margin of the upper Van Norman Lake, but in this area it is difficult to determine whether they are due to the apparently subsidence-related mechanism in the rest of the zone, or to the slumps and slope failures which occurred around the reservoir perimeter. The maximum difference in elevation from the northeast end of this displacement zone to the Sylmar Converter Station is about 50 ft. There is a further drop of about 20 ft from the converter station to the lake level at the time of the earthquake. Along the zone of displacement, different amounts of movements have occurred ranging from a few inches to perhaps three feet. Horizontal displacements towards the lake have occurred (Fig. 4. 14) but appear to be less near the lake than at, for example, the railroad tracks (Fig. 4. 15). No horizontal displacement is visible along the fence at the southwest boundary of the converter station. Considerable damage was caused at the Juvenile Facility by the ground movements (Figs. 3. 89 - 3. 97).

A number of explanations of these movements may be put forward; they may have developed as a result of (1) underlying geological faulting, (2) a landslide toward the lake, (3) settlement in a region underlain by low density or poorly compacted soil. Although the axis of axis of a syncline in the Pleistocene underlying rocks runs northeast-southwest through the most northerly portion of upper Van Norman Lake, no faults have been mapped in this specific area, where the rock is overlain by up to one hundred

Figure 4. 12 Aerial photograph of ground displacement extending from Juvenile Facility to Sylmar Converter Station.

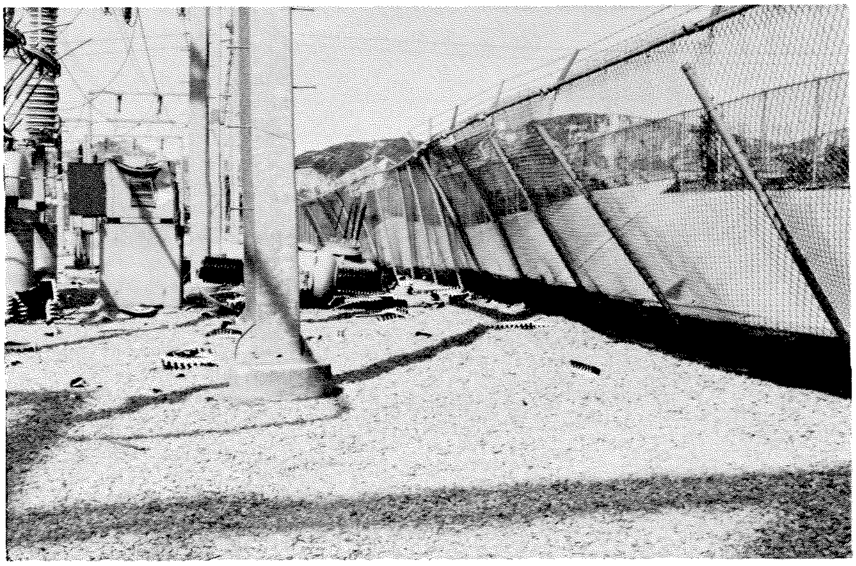


Figure 4. 13 Ground displacements along fence and drainage channel at northeast boundary of Sylmar Converter Station. The settlement of the station in foreground with respect to the concrete lining of the channel is apparent.

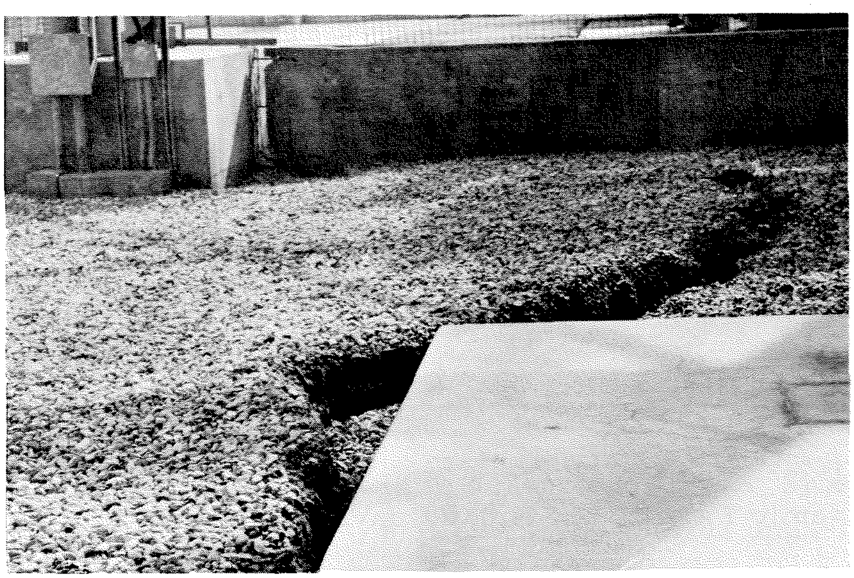


Figure 4. 14 Horizontal displacements inside Sylmar Converter Station evident in ground crack and in separation of wall from column footing in background.

Figure 4.15 The Southern Pacific Railroad tracks just south of the San Fernando Juvenile Facility at the east end show evidence of substantial ground movement.

feet of alluvial deposits. If the movements were due to a slide, or incipient slide in the soil, it would be expected that the greatest horizontal movements would occur at the unrestrained face along the lake margin and diminish with distance to the northeast. As described above, however, the actual displacements are at variance with this model. At present, therefore, it seems most likely that the movements arose through vibration-induced compaction of the sediments underlying the area. The densification of the material would cause both vertical displacements and displacements downslope in the direction of the lake. The amount of movement at any location would depend on the thickness and degree of looseness of the underlying layer of low initial compaction.

A comparison of recent and older maps of the area (Figs. 4.16a and 4.16b) shows that two former natural drainage channels emerged from the foothills to the north and debouched into the area now occupied by the upper Van Norman Lake. A view of the area as it appeared in 1929 is shown in Fig. 4.17. The lower course of the more southerly of these channels follows fairly closely the region of cracking and ground displacement under discussion, between the lake and the small debris basin southwest of the Juvenile Facility. However, there appear to be no topographic features through the facility and to the northeast which would explain the northerly extension of the zone of displacement as observed. A more detailed evaluation of the cause or causes must await the results of precise aerial surveys which will reveal the nature of the movements.

Since the groundwater is fairly close to the surface in this region, the underlying sediments are saturated, and it might be expected that vigorous shaking of saturated loose sediments would give rise to soil liquefaction phenomena. No large slides or gross movements along the

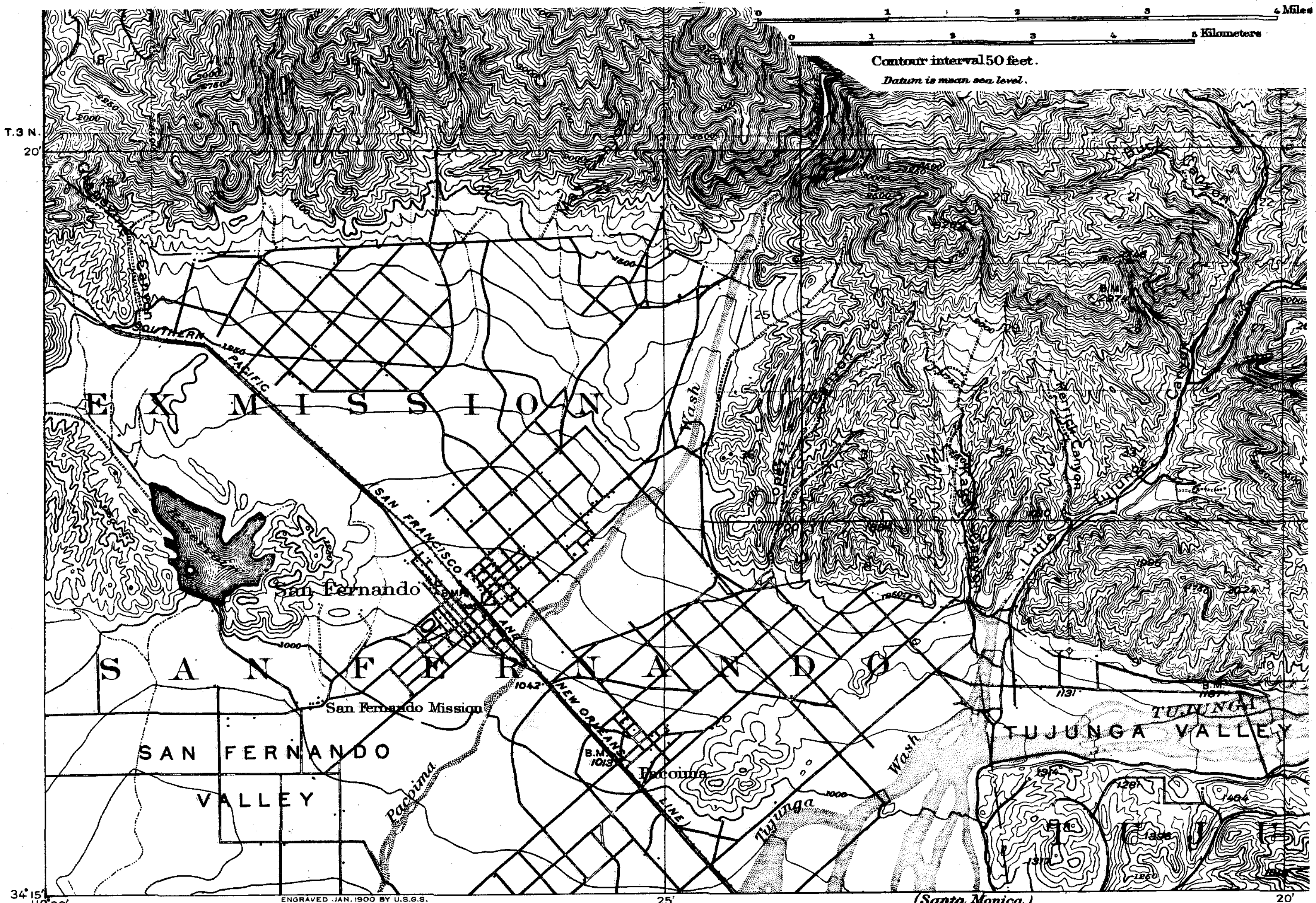


Figure 4. 16a USGS topographic map of the San Fernando area, edition of 1900; reprinted 1929.

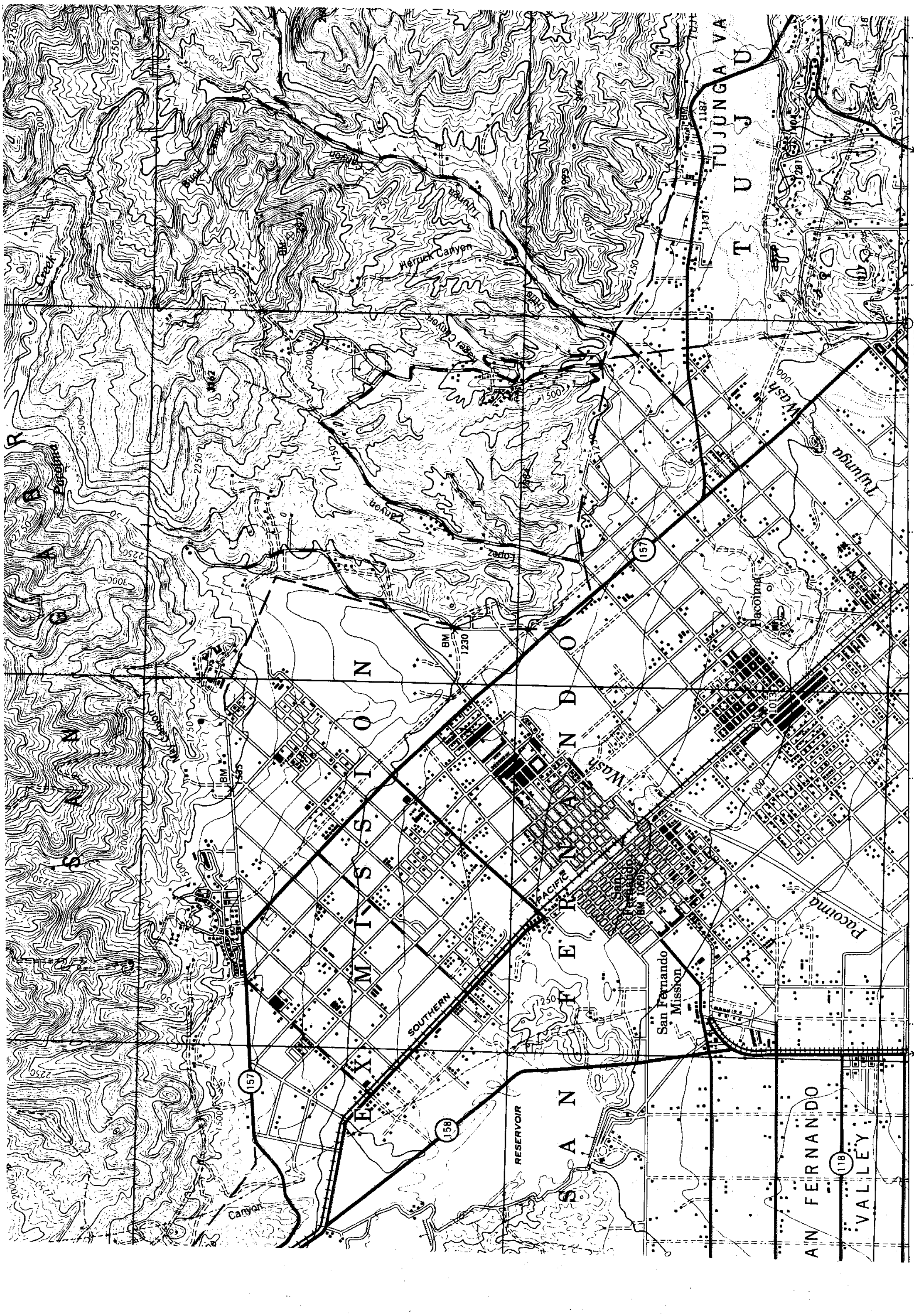


Figure 4.16b USGS topographic map of the San Fernando area issued in edition of 1945.

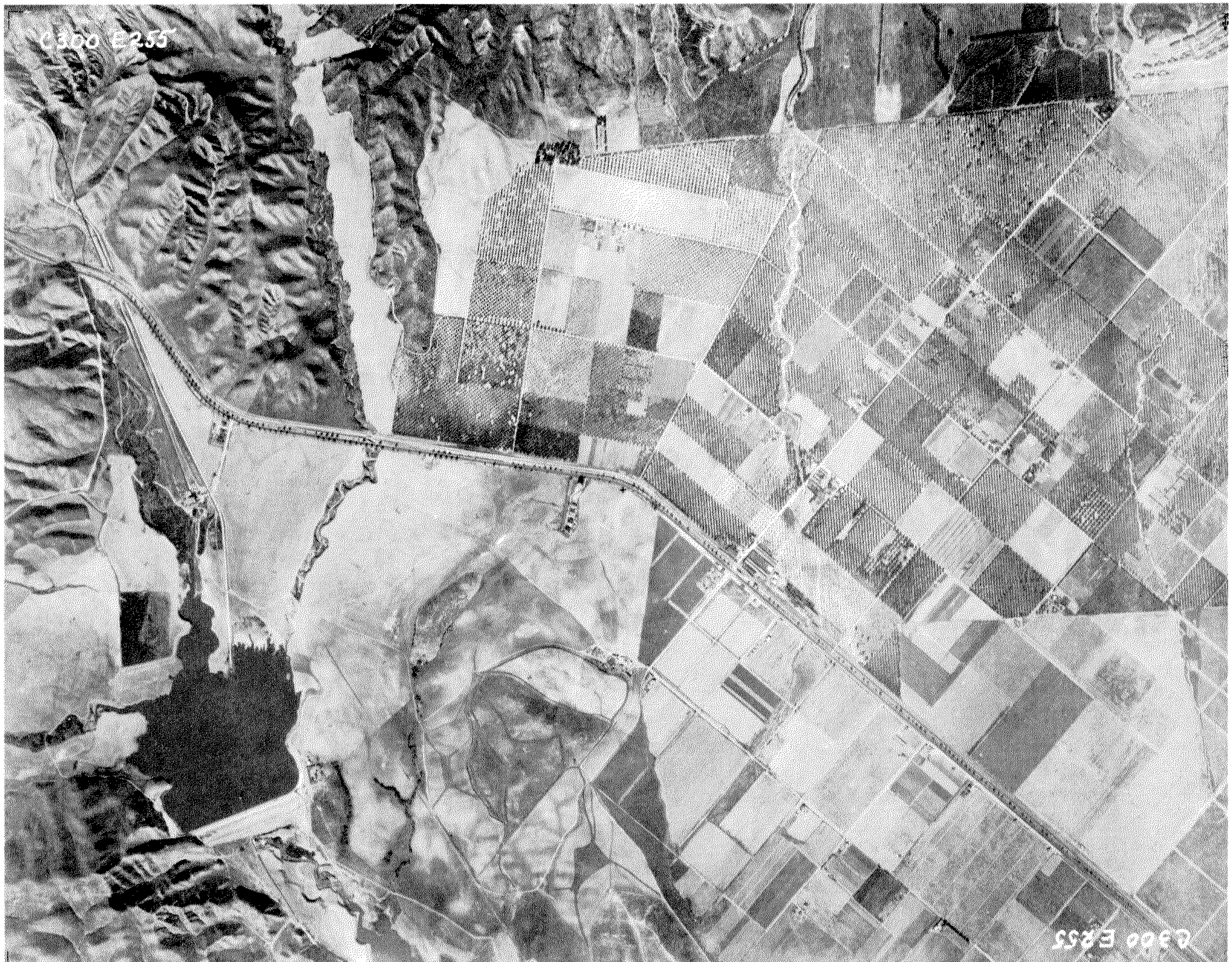


Figure 4. 17 Vertical aerial photograph of San Fernando in 1929.

various slopes have occurred in such a manner as to suggest that relatively widespread or long-lived liquefaction processes developed. However, a number of sandboils are apparent in some areas and indicate that local liquefaction did occur (Fig. 4.18).

These boils have become apparent in two clearly-bounded zones at the north end of the upper Van Norman Lake since the lake level was lowered after the earthquake (Fig. 4.19). All of the boils occurred under water in the lake. One zone about 200 ft wide runs from the north edge of the observatory peninsula southwest to the west margin of the lake. Its projection onshore at the peninsula delineates an area of extensive slumping of the slopes. The other zone, 150 ft wide, runs from the shore of the lake west of the Sylmar converter station into the lake and along the southeast shoreline (Fig. 4.19). Damage to adjacent slopes is apparent, but it should be noted that the slopes around the lake have also fractured due to slumping elsewhere. Both of these areas of sand boils have been investigated in a preliminary fashion. The sandboils in the more northerly zone are the larger of those in the two regions, ranging from a foot to several feet in diameter and up to about 6 or 9 inches in thickness. The soil composing the core of the boil consists of fine to medium sands and silts. A cross section through the deposit in which they appear indicates that they probably developed as a result of liquefaction of a fine to medium sand layer, whose upper surface is at a depth of 4 to 6 ft below a finer-grained clayey silt upper layer (Fig. 4.9). Some of the boils are elongate, whereas some others appear in rows, indicating that they have formed along the path of fissures in the surface material.

In the more easterly zone of boils, the sand cones range from a few inches to a foot or two in diameter and are spaced a few feet apart on the

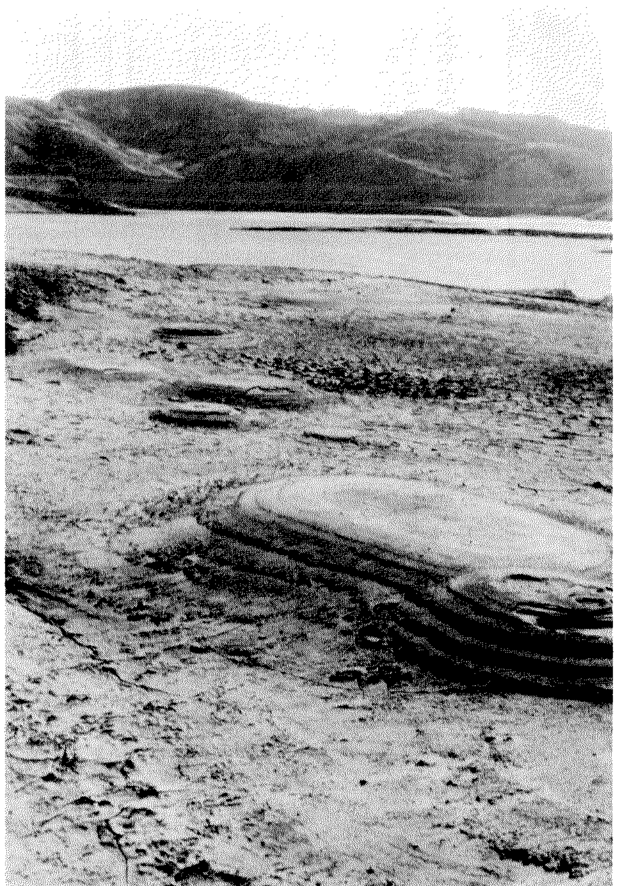


Figure 4.18a Sand boils in upper Van Norman Lake revealed by lowering of lake level.

Figure 4. 18b. Cross section through small boil. Pencil marks level of top layer from which sand was brought up to surface to form boil. Pictures taken 5 March 1971.

Figure 4. 19 Aerial photograph of upper Van Norman Lake showing zones in which sand boils are encountered.

lake bed. Cross-sectioning of one typical boil 18 inches in diameter indicated that the upper surface of the liquefying layer lay about 18 inches below the soil surface (Fig. 4.18b). In this area, the lake bed surface is fractured and fissured in polygonal desiccation patterns caused by both the present and past lowering of the water level.

Laboratory studies (Scott and Zuckerman, 1971) have shown that the presence of a finer-grained nonliquefying upper layer above the sand liquefaction layer is an essential prerequite for the formation of sand boils during vibrations. After earthquake-induced liquefaction of the lower sand layer, cavities develop at the base of the fine-grained stratum and work their way to the surface where a mixture of water and sand is discharged in a fountain. The resulting sand boil comprises material drawn from the interface between the two soils. The thinner the upper, less permeable layer, the more closely-spaced and smaller are the sand boils for a given thickness of liquefying layer. The size and spacing of the sand boils in both zones in the upper Van Norman Lake are consistent with the laboratory studies and the presence of a liquefying layer a few feet below the ground surface. From the amount of material associated with the sandboils it further appears that the thickness of the liquefied layer was small, of the order of a few feet.

The boils developed in these zones because of the presence of the underlying loose sand layer, probably a depositional feature along the path of a former natural drainage channel. In the more westerly of the two zones, the boils may also have been associated with shearing-induced liquefaction developed by the slumping and sliding of the lakeshore. The region of deformation and ground displacement through the Sylmar Converter Station may also be an expression of the behavior of the same former drainage channel.

Two adjacent, but isolated sand boils are apparent in the level ground one or two hundred feet south of the toe of the upper San Fernando dam. A similar feature occurred in the soil deposits just north of Lopez dam to the east, and two or three sand cones apparently developed during the earthquake and are visible at the toe of the Jensen Treatment Plant fill slope west of upper Van Norman Lake. Boils were reported in the debris basin south of the Juvenile Facility but were not seen by the writer or his colleagues.

Footings and Foundations

To the present, no clear indications of footing or foundation failures resulting from earthquake-caused dynamic loads have been found. Where the freeway overpasses collapsed, the footings remained in place, although the column reinforcing bar bond failed and the reinforcement was pulled out of the footings (Fig. 4.20). Lateral movements of some bridge abutments on piles at right angles to the line of the roadway caused failures in the piles below the pile cap (Fig. 4.21). The role of the footings in the failure of portions of the Olive View Hospital awaits removal of the collapsed structure and excavation at the site. Although a number of modern reinforced concrete retaining walls up to 12 ft high show permanent displacement and some damage (Fig. 4.22) none is known to have collapsed under the dynamic loadings. Ground fissures parallel to and behind the walls give indications of vibrational movements of the wall and backfill in a number of cases.

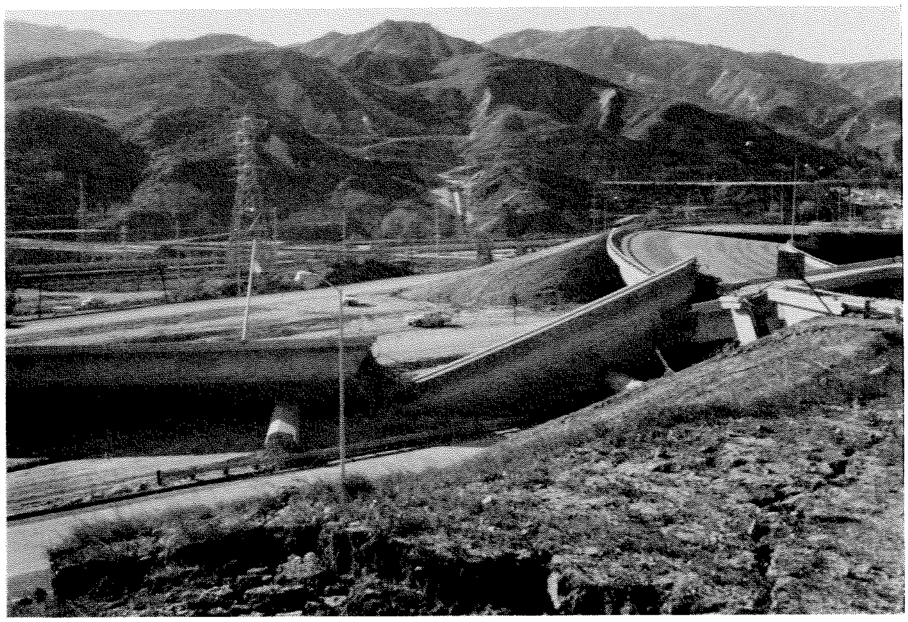


Figure 4.20 Overturned freeway overpass at intersection of Foothill and Golden State freeways. Overpass column reinforcing has pulled out of foundation. Partial slope failure seen in embankment fill in foreground.

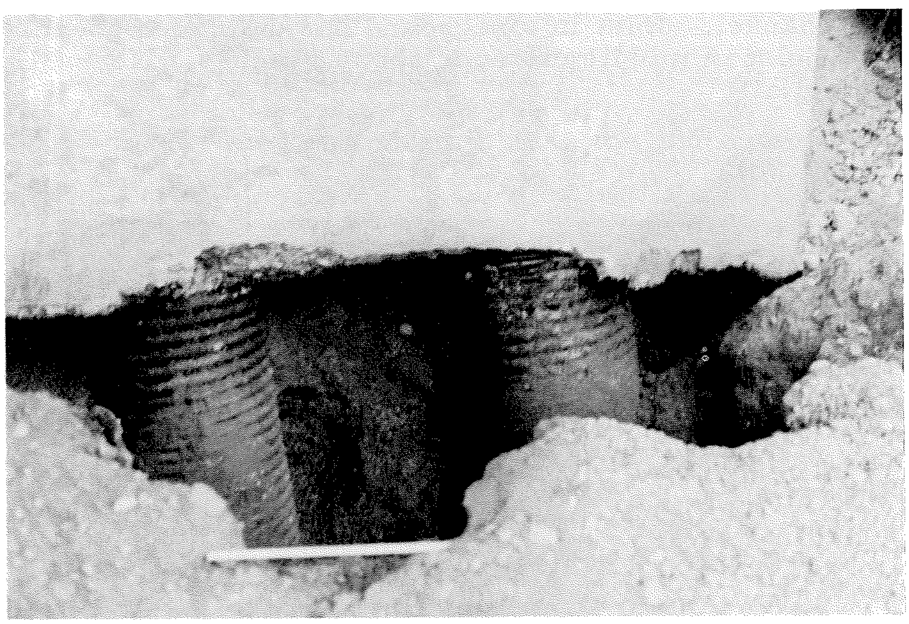


Figure 4.21 Lateral movement of Roxford Street bridge on abutments has failed tops of piles. (Foothill freeway).

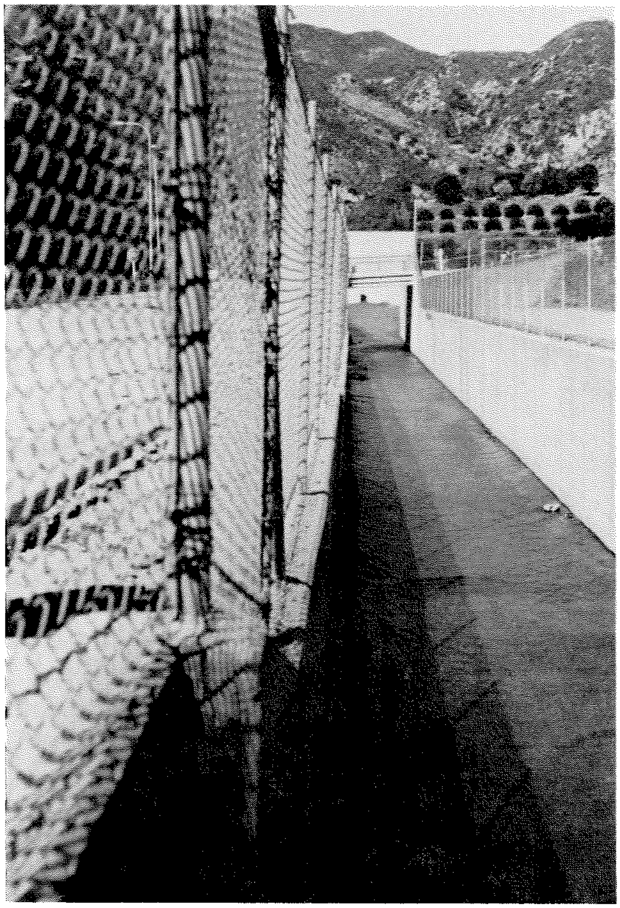


Figure 4.22 Motion of retaining wall of flood control channel towards channel. Joint in foreground has displaced about 4 inches; joint in middle distance about 2 inches. Near Olive View Hospital.

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EARTHQUAKE EFFECTS ON UTILITIES

by W. D. Iwan

Sylmar Converter Station

The $110 million Sylmar Converter Station of the Pacific Northwest-California Direct Current Intertie was completely disabled by the earthquake. The facility was designed to handle 1440 megawatts of power, and was operating at approximately 570 megawatts at the time of the temblor. The station is located just west of Interstate Highway 5 near the heavily damaged interchange with Interstate Highway 210. The aerial photograph (Figure 5.1) shows the precise location of the site and its relation to surrounding features.

The facility consists of a combined reinforced concrete and steel frame building (Figure 5.2) housing the d-c rectifying equipment along with service and administrative personnel, a large attached birdcage area (Figure 5.3) and an open yard. The building structure showed signs of damage principally in the area near the junction between service and equipment sections. One basement wall of the service section appeared to be pushed in slightly, and another basement wall of the service section was fractured at a point where it was connected to a perpendicular wall of the equipment section. The two sections of the building are supported on common concrete columns where they come together at the basement level. These columns showed clear evidence of structural damage as shown in Figures 5.4 and 5.5. In addition, there was evidence of pounding and some cracking where the floor slabs of the two separate sections met (Figure 5.6). A survey has indicated that the exterior columns of the building (along with the exterior walls and adjacent portions of the

Figure 5.1 Aerial photo showing Sylmar Converter Station (near center of photo), Sylmar Switching Station (right center of photo), a portion of the Foothill-Golden State freeway interchange (lower right corner), Juvenile Facility (just visible at lower edge of photo), Olive Switching Station (lower left corner) and a portion of the upper Van Norman Reservoir (upper edge of photo). The center line of the photograph from bottom to top points in a south by southwesterly direction.

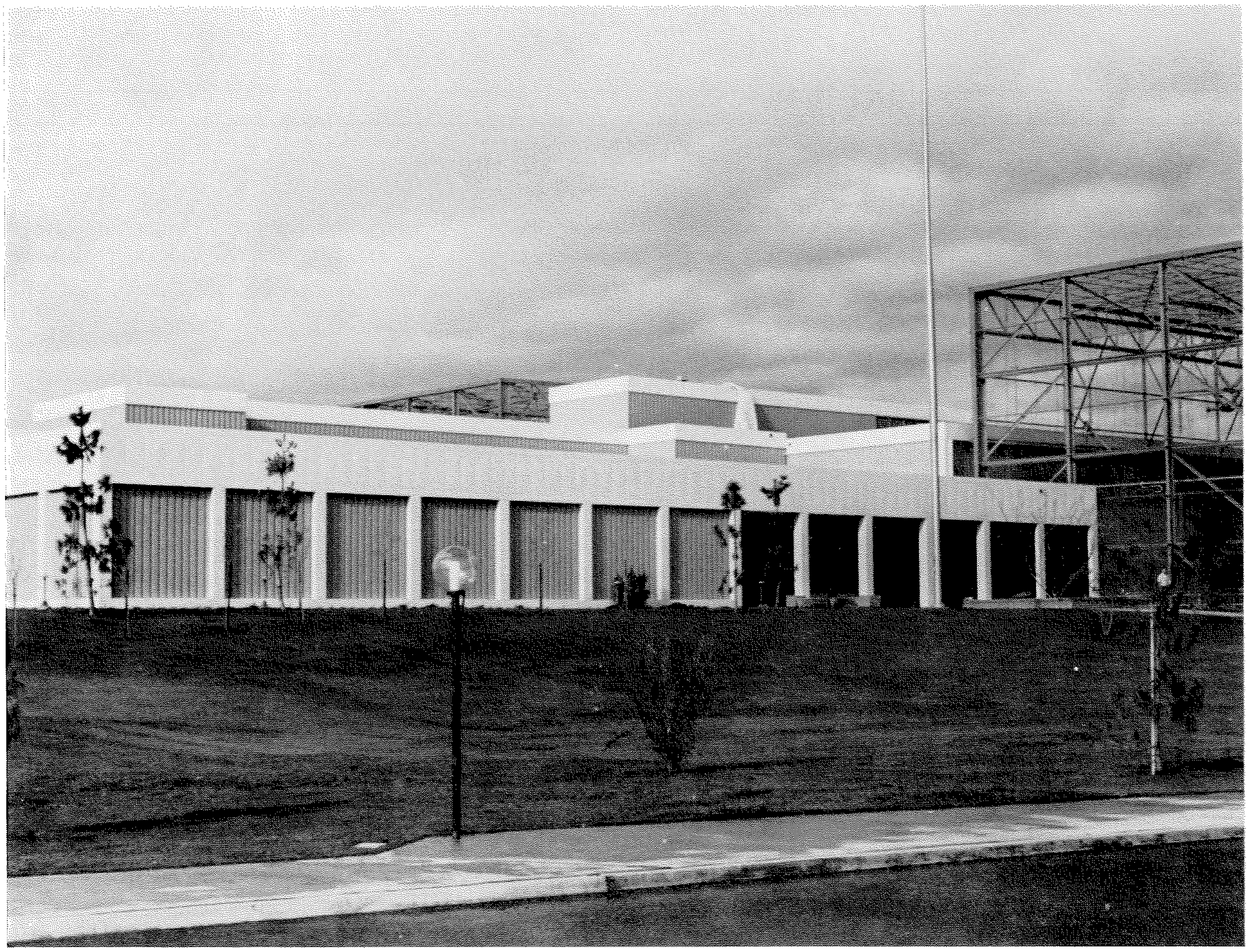


Figure 5. 2 Sylmar Converter Station Building. The service portion of the building is on the near side and the "valve hall" portion is behind and to the right.

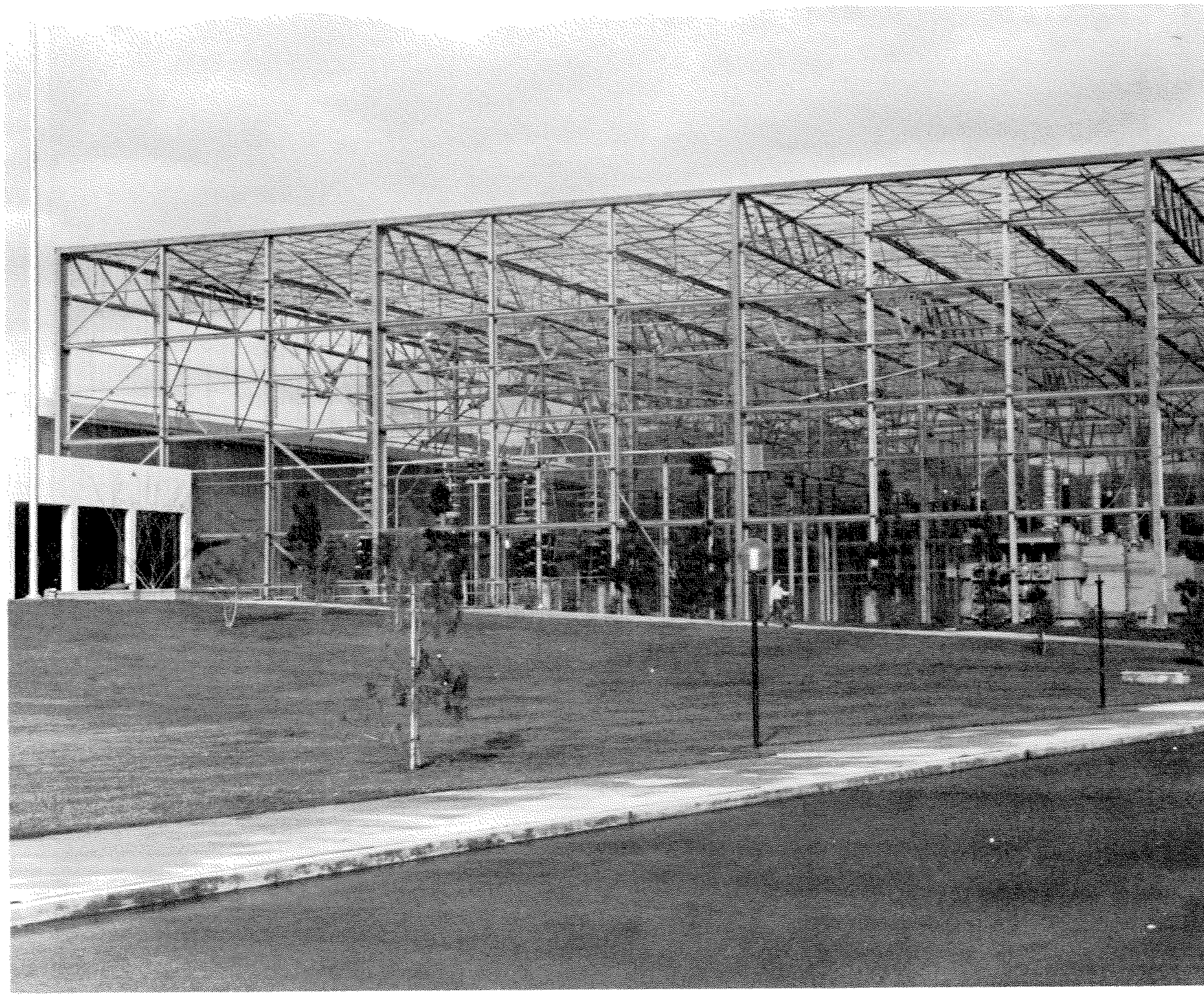


Figure 5. 3 "Birdcage" on east side of Sylmar Converter Station. The crossbracing of the cage was damaged and some of the equipment suspended from the roof of the cage was damaged.

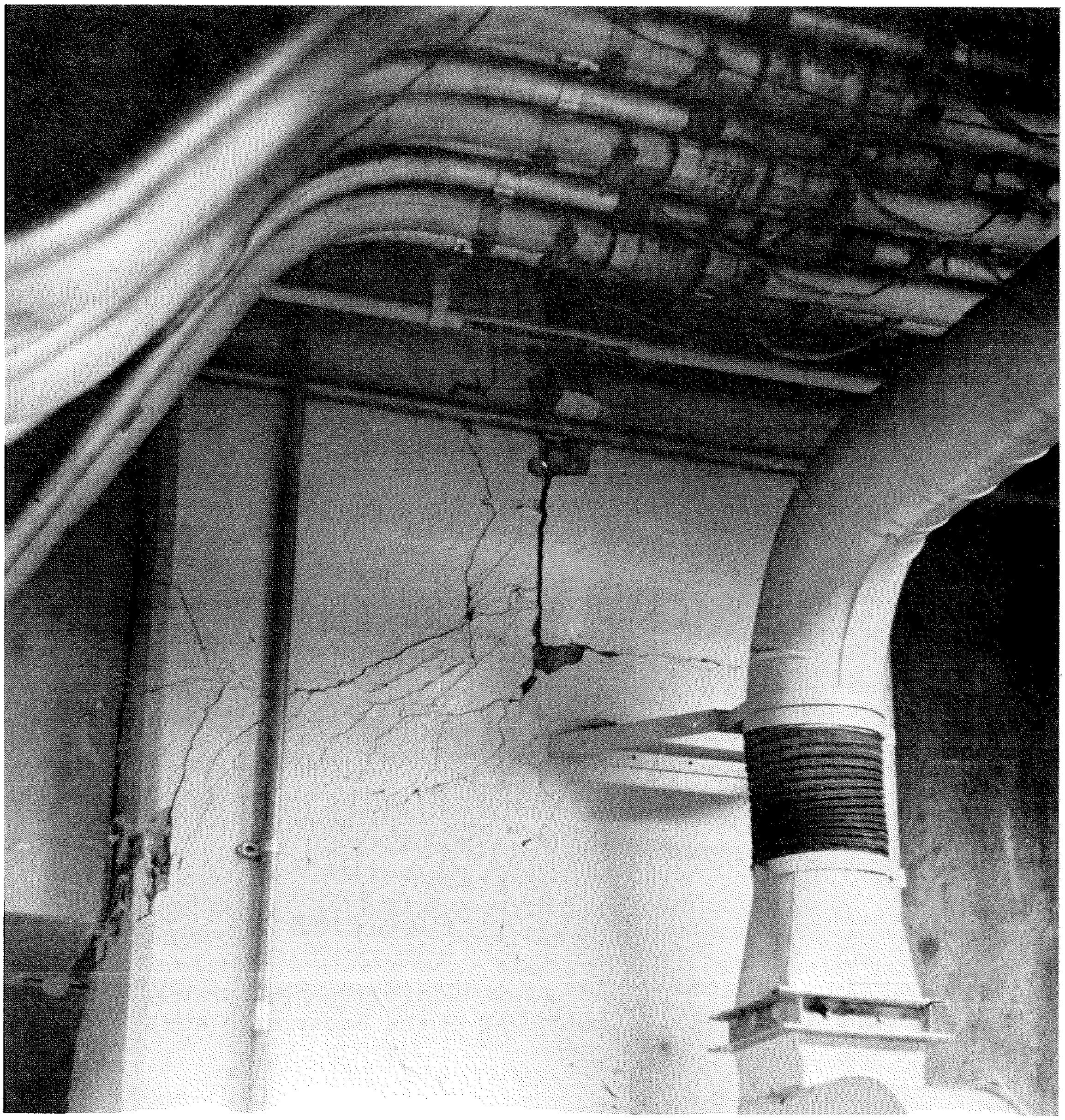


Figure 5.4 Basement support column at juncture of service section and the valve section, Sylmar Converter Station. The two sections of the building tended to move independently, fracturing the common support column.

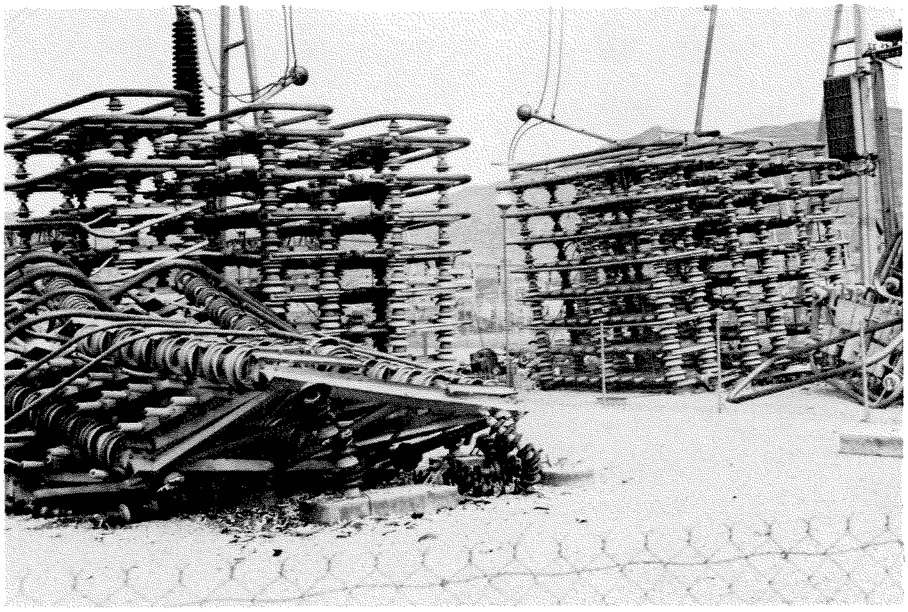


Figure 5. 15 Damaged capacitor stacks in yard north of service and equipment building, Sylmar Converter Station

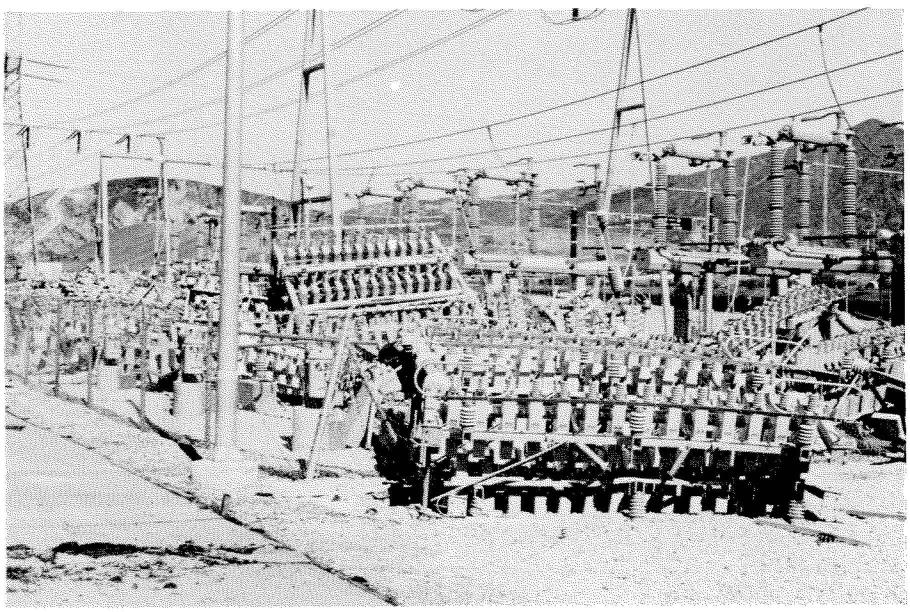


Figure 5. 16 Damaged capacitor stacks in yard north of service and equipment building, Sylmar Converter Station.

basement floor slab) are depressed by some 3-4 inches with respect to the center columns of the structure. The effects of this relative movement are readily apparent in the numerous cracks in the basement floor slab and its very noticeable slope in certain areas.

The equipment section contains the mercury-arc rectifiers or valves used in a-c/d-c conversion. Forty-two valves are arranged in two parallel rows. Figure 5.7 shows a view of one section of the valve hall prior to the earthquake. Each valve assembly has three main structural elements. The valve support structure consists of a horizontal platform which rests upon six porcelain columns. The columns range in height to a maximum of 11 feet and have a maximum diameter of 14 inches. The valve itself rests upon rails attached to the platform and weighs approximately 14,500 pounds. The valves are bolted to the support platforms except when they are removed for service. Each valve has six tubes or anodes which project from its top. Above each valve is a current divider. This structure weighs approximately 8,100 pounds, and is suspended from the ceiling by means of four insulator strings. Each string has a rated strength of approximately 15,000 pounds. There are no structural connections between the valves and the current dividers.

There were no reported cases of failure of the valve supporting structures. The major damage was caused by failure of the current divider suspension systems. In most instances two or more insulator strings failed on each unit. This allowed the current divider to either drop onto the anodes or to bang into them as it swung back and forth on its remaining supports (see Figures 5.8 - 5.12). All of the valves were rendered inoperable and release of mercury from damaged anode housings caused a severe contamination problem.

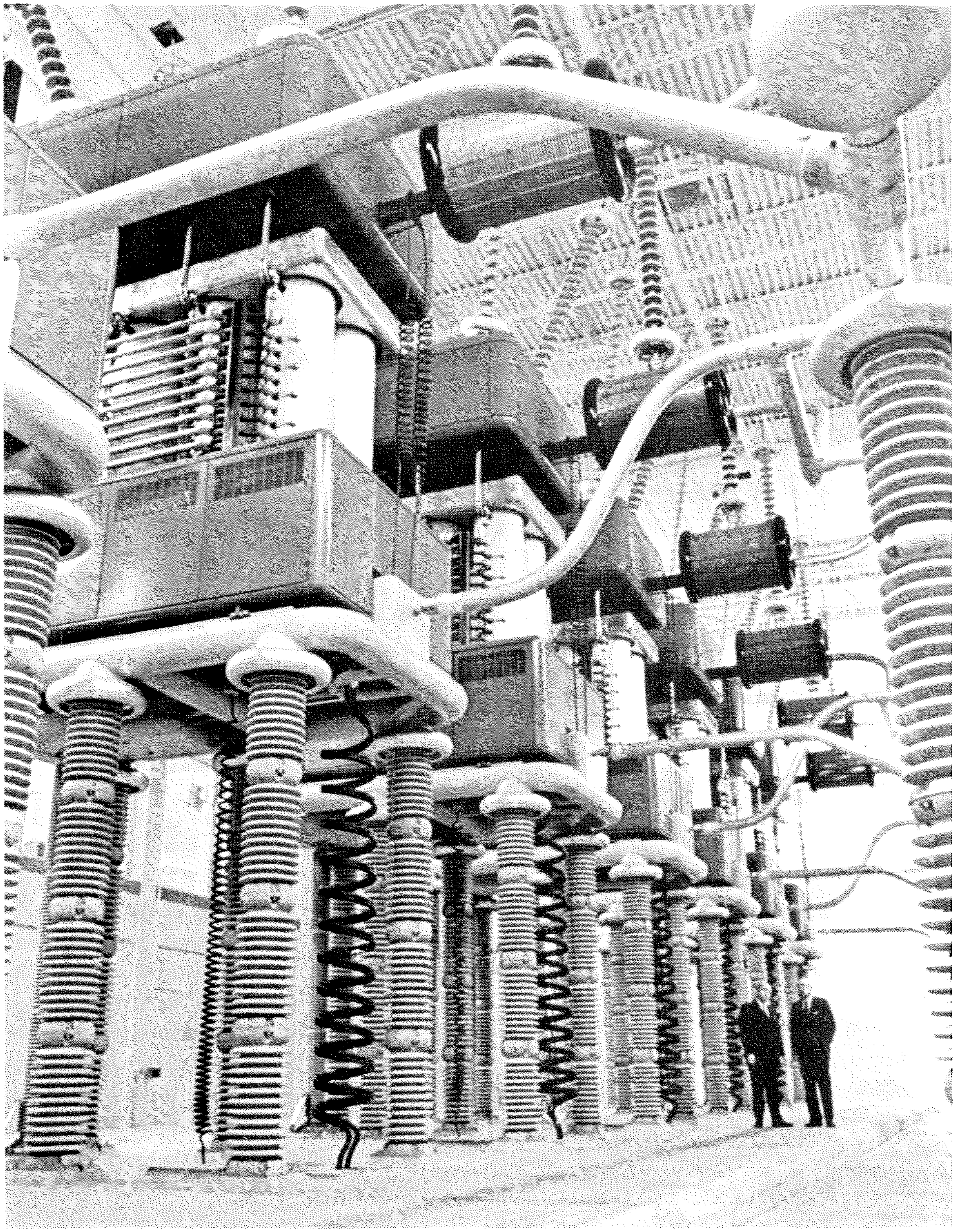


Figure 5.7 View of a portion of the "valve hall" of the Sylmar Converter Station before the earthquake.

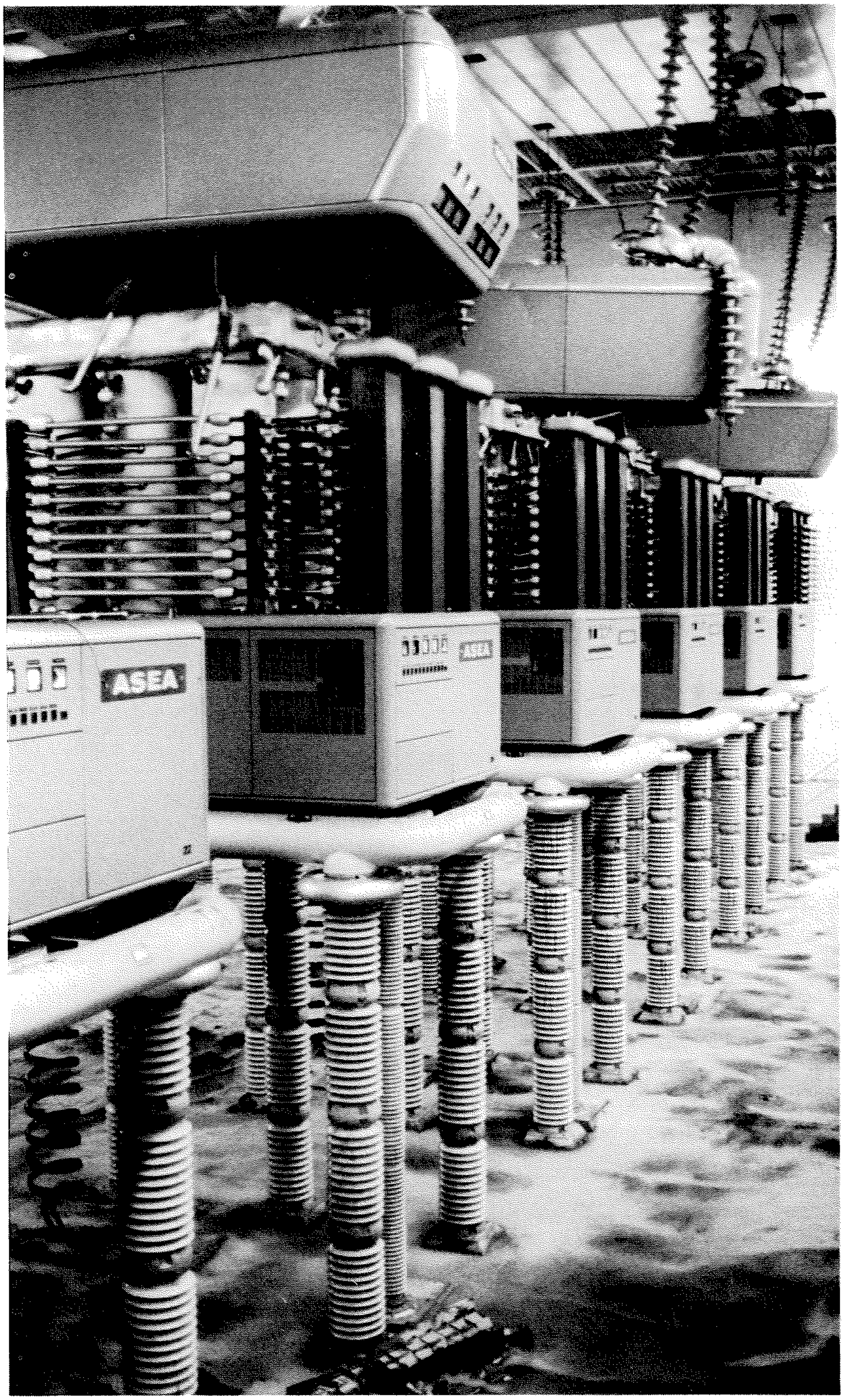


Figure 5.8 View of the "valve hall" of the Sylmar Converter Station after the earthquake. Porcelain supports were not damaged by the ground motion.

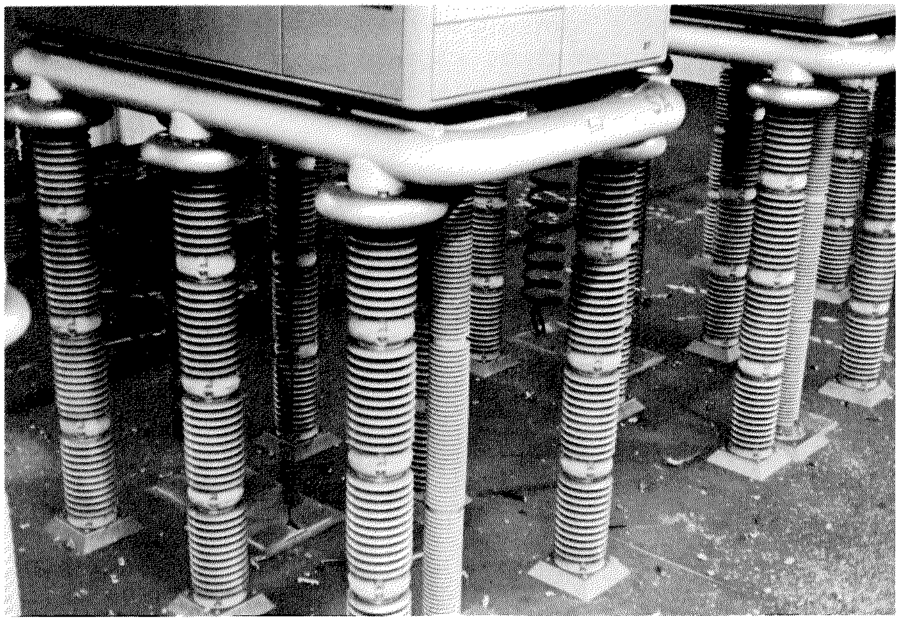


Figure 5.9 Support structure for valve assembly, Sylmar Converter Station.

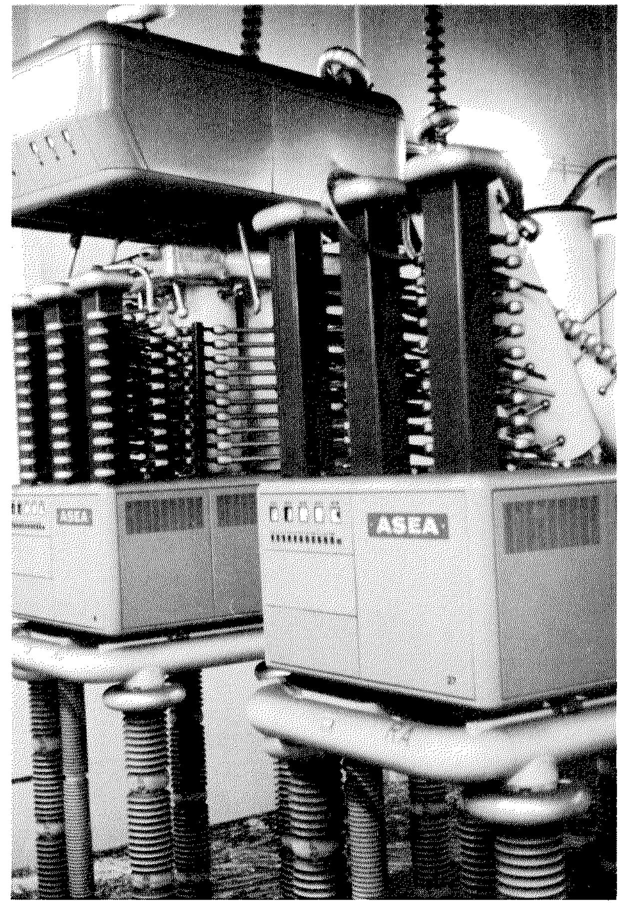


Figure 5.10 Valve assembly and current divider, Sylmar Converter Station. Note damage to anodes caused by current divider in near unit.

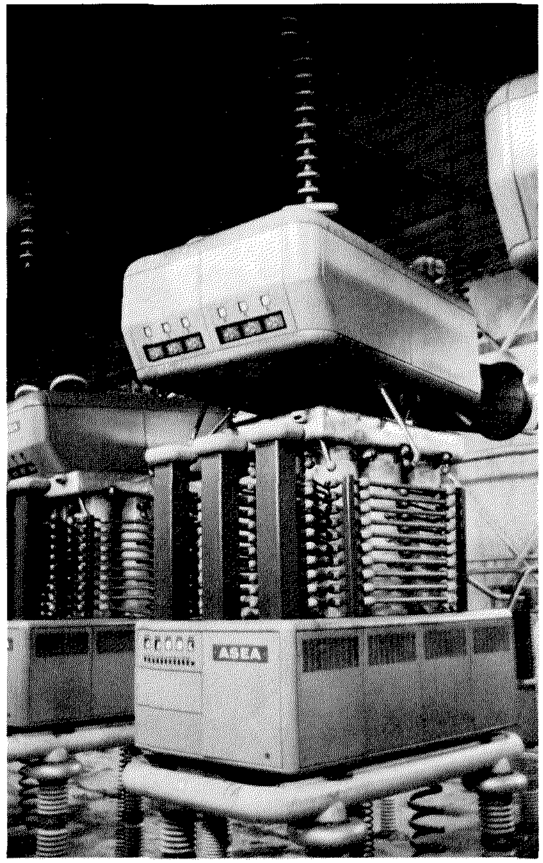


Figure 5.11 View of valve assembly, showing failure of current divider suspension, Sylmar Converter Station.

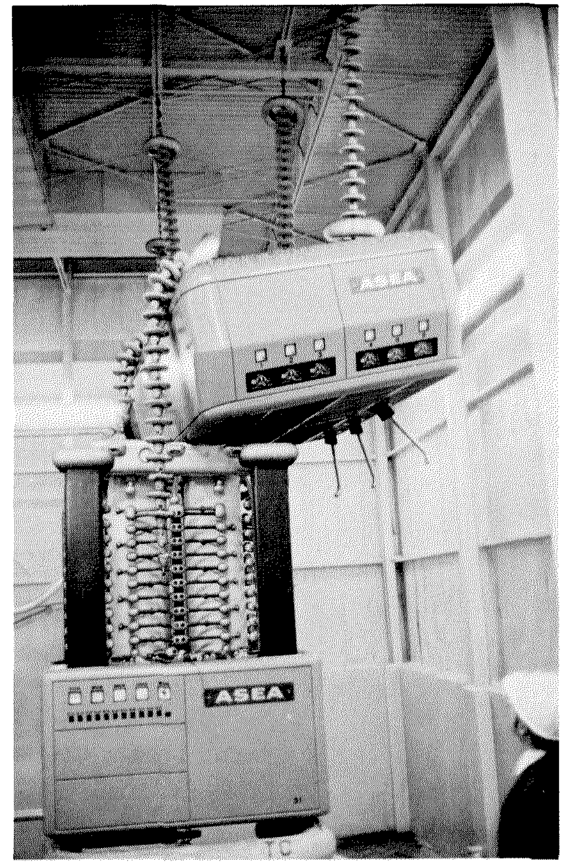


Figure 5.12 View of valve assembly, showing failure of current divider suspension, Sylmar Converter Station.

The large wire mesh birdcage adjacent to the main building showed definite signs of overstraining. Several diagonal members in the supporting structure were bent, some bolts were pulled loose, and a number of attachment plates were sheared. RC filters and suspended busswork in the birdcage were heavily damaged. Most of these components relied upon percelain for their structural strength. Two auxiliary power transformers in this area slipped partially off their footings, but did not topple (Figure 5.25). They were not bolted down.

In the yard there was major damage to nearly every type of electrical equipment. The primary air-blast circuit breakers failed, but the backup breakers did not. In addition, several large capacitor arrays collapsed as shown in Figures 5.15 - 5.17. These arrays were all supported by porcelain structural members.

Among those units which relied upon more conventional material for strength the most striking failures were those associated with the large a-c harmonic filter reactors (Figures 5.19 - 5.21). These oil-filled units are approximately 20 feet high, 7 feet in diameter and weigh 7,000 lbs. Each unit rested on a concrete footing and was attached to the footing at four points. In general, the attachment was through a steel holddown plate which was secured to the footing by four one-half inch Nelson stud anchors (Figures 5.22 and 5.23). However, in one case, the unit was bolted directly to the footing by means of four expansion-type bolts (Figure 5.20). The anchors failed on four of six units, and the units fell in a northerly to westerly direction.

There were clear signs of relative movement between footings and the surrounding soil in some areas of the yard (Figure 5.27), indicating that soil-structure interaction may have been an important factor in

Figure 5. 13 Air-blast circuit breakers, Sylmar Switching Station.

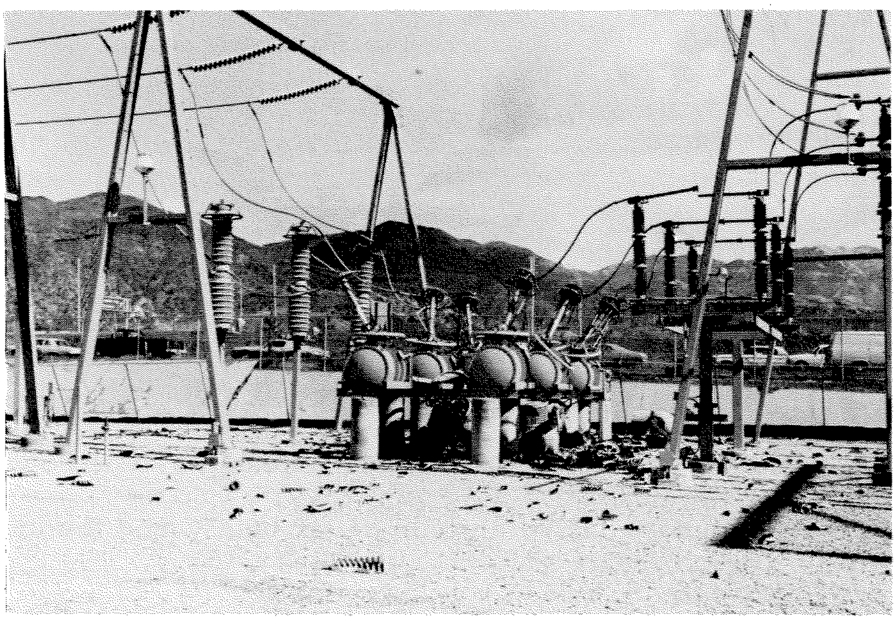


Figure 5.14 Air-blast circuit breakers, Sylmar Converter Station.

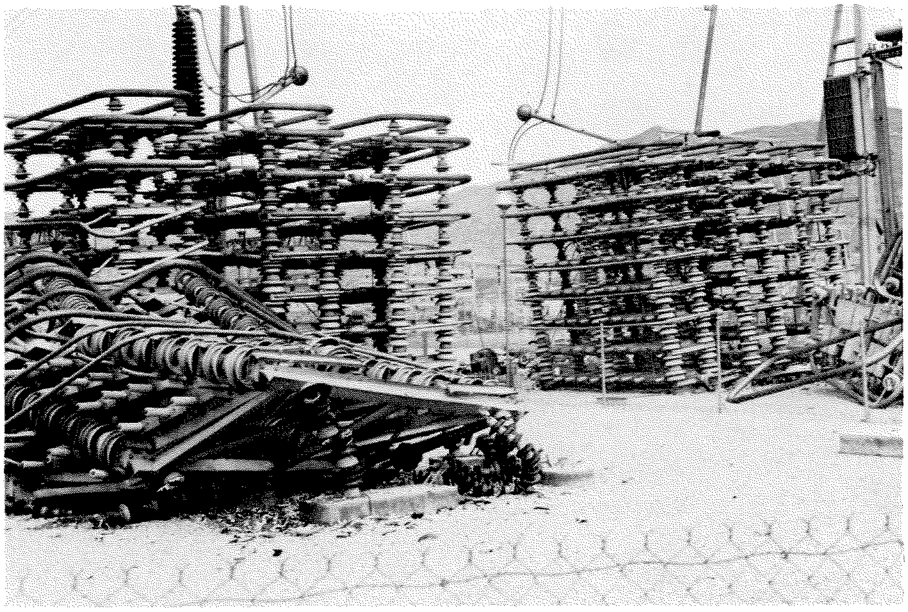


Figure 5. 15 Damaged capacitor stacks in yard north of service and equipment building, Sylmar Converter Station

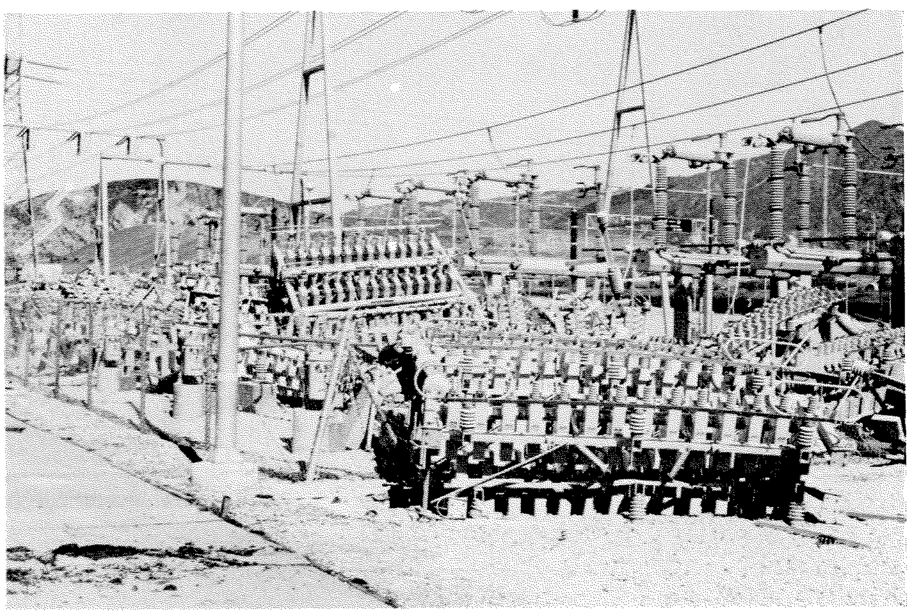


Figure 5. 16 Damaged capacitor stacks in yard north of service and equipment building, Sylmar Converter Station.

Figure 5.17 Detailed view of fallen capacitor stacks, Sylmar Converter Station.

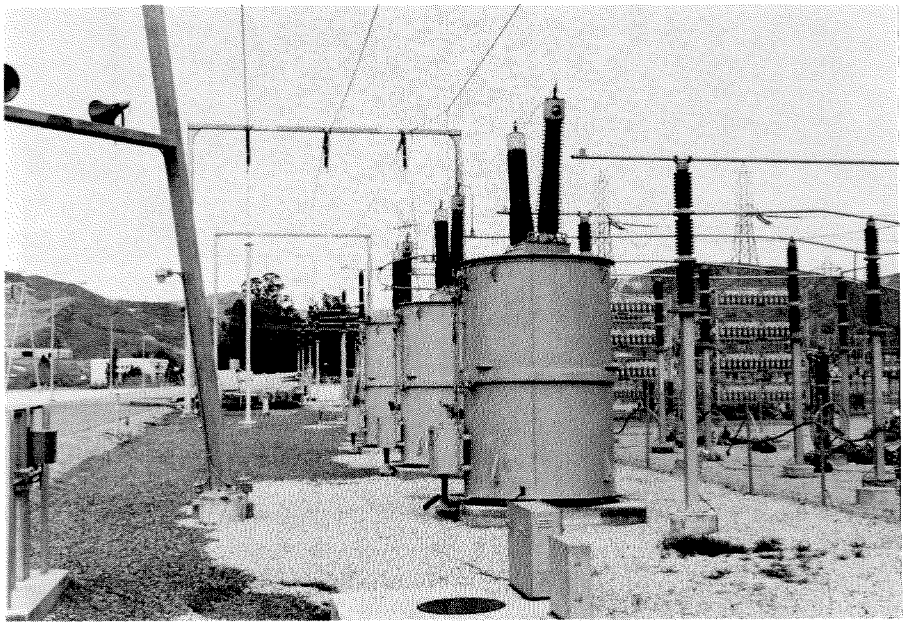


Figure 5.18 General view of yard showing fallen reactor in background, Sylmar Converter Station.

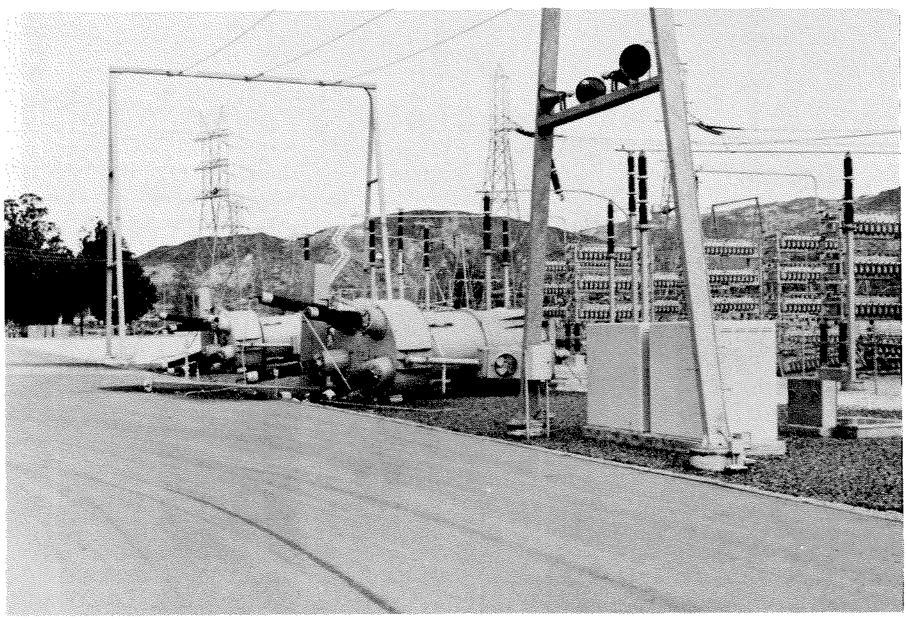


Figure 5.19 Fallen a-c harmonic filter reactors, Sylmar Converter Station.

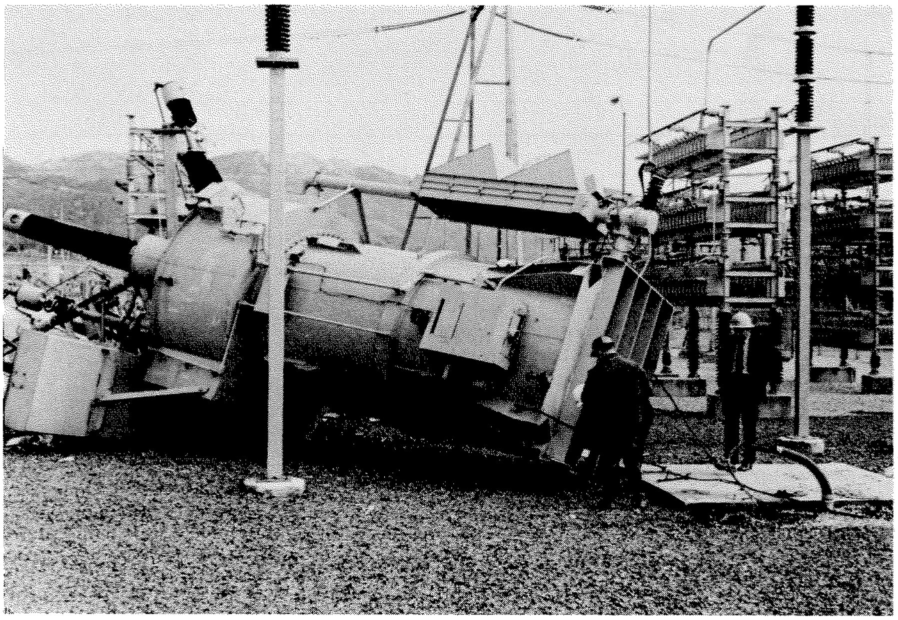


Figure 5. 20 a-c harmonic filter reactor, Sylmar Converter Station.

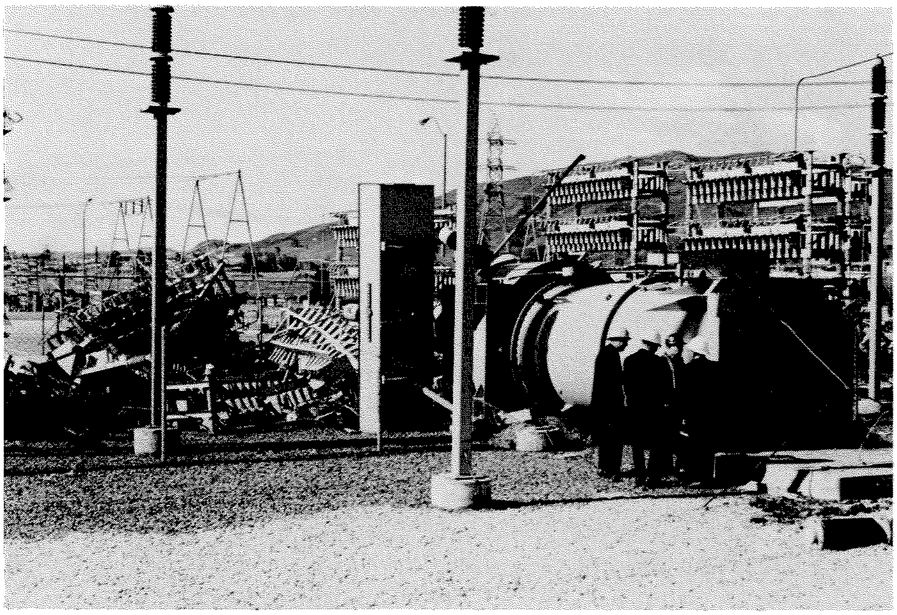


Figure 5.21 a-c harmonic filter reactor, Sylmar Converter Station.

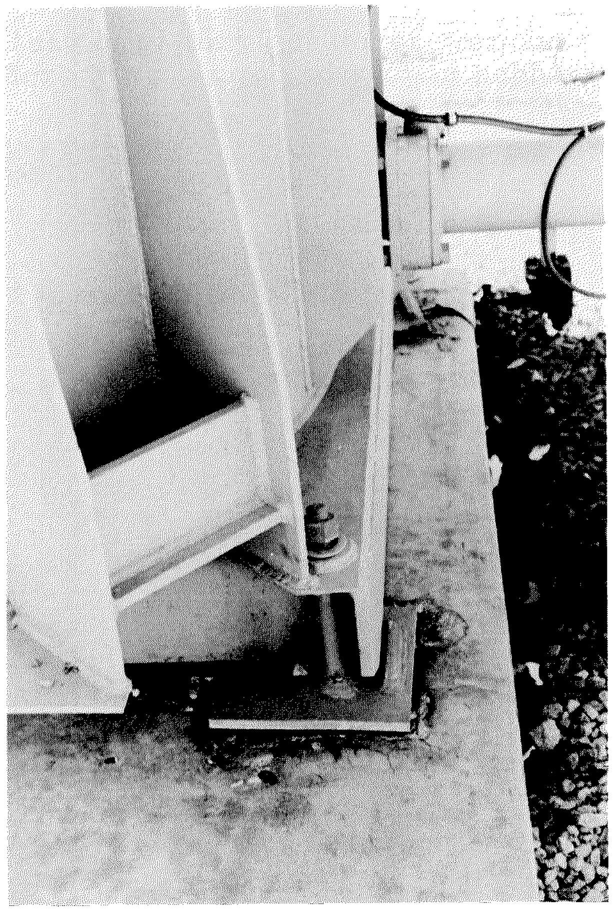


Figure 5.22 Details of hold-down plate for a-c harmonic filter reactor, Sylmar Converter Station.

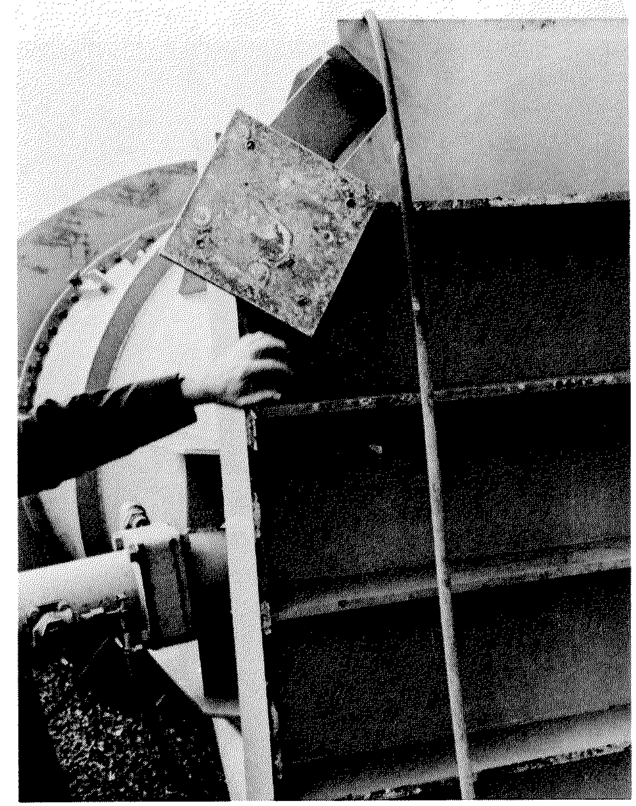


Figure 5.23 Details of hold-down plate for a-c harmonic filter reactor, Sylmar Converter Station.

Figure 5.24 Failed equipment support in yard of Sylmar Converter Station.

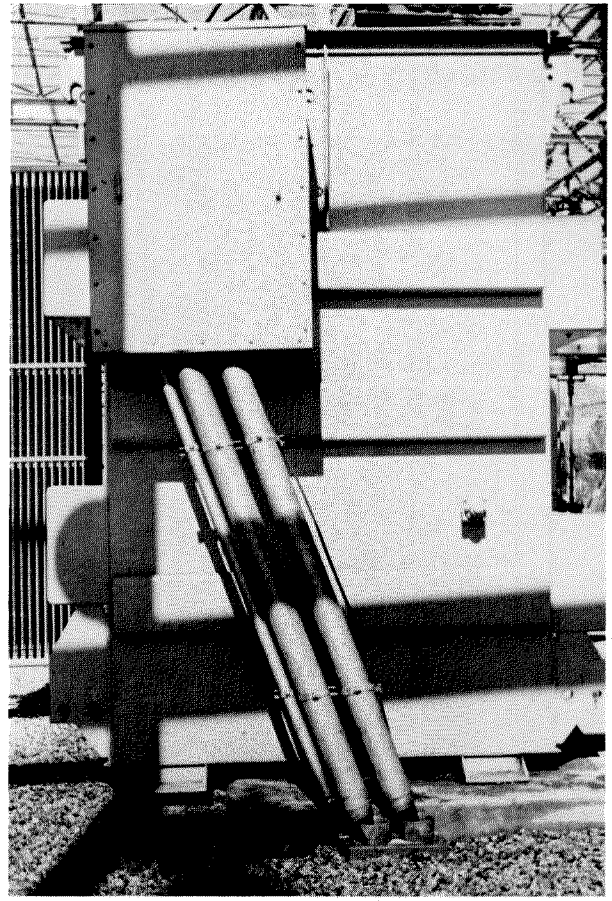


Figure 5.25 Auxiliary power transformer, Sylmar Converter Station. Tranformer was not bolted to footing.

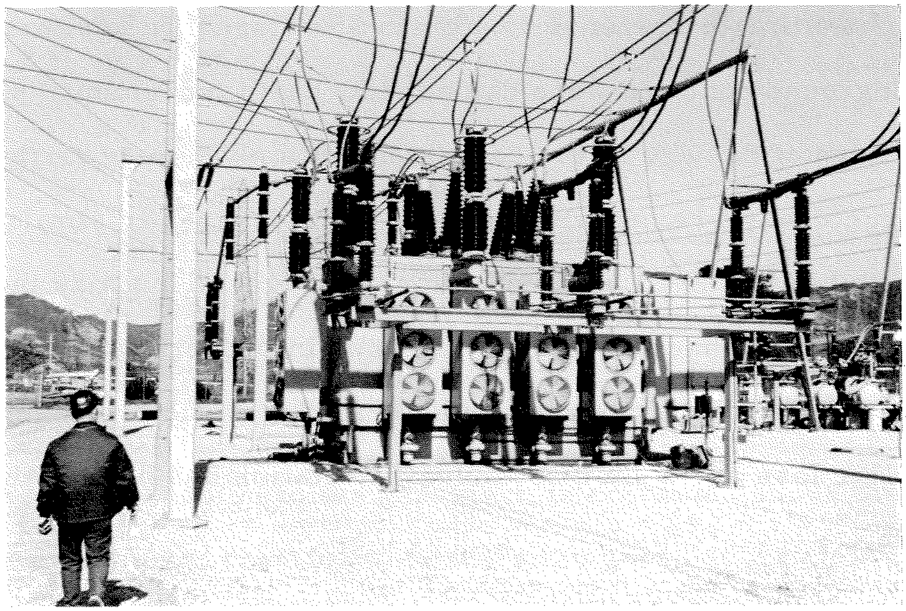


Figure 5.26 View of relatively undamaged equipment at the Sylmar Converter Station.

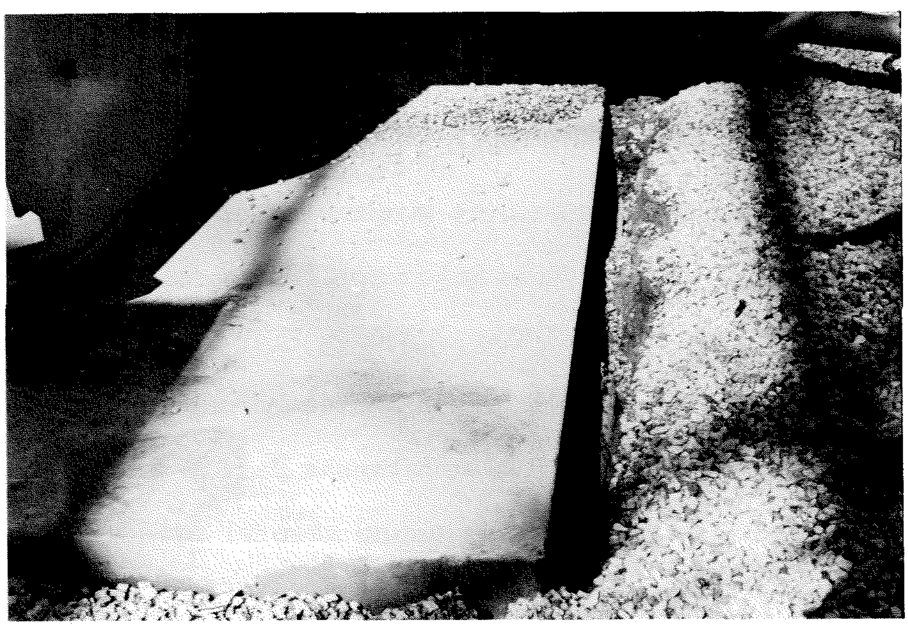


Figure 5.27 Valve transformer footing, showing evidence of footing movement. Sylmar Converter Station.

determining the ultimate response of a given structure. In addition, several underground conduits were damaged by earth movement.

The pattern of damage at the converter station was noteworthy in that some structures were heavily damaged while apparently similar adjacent structures received relatively little damage. This may indicate that local variations in anchor strength, porcelain properties and soil-structure interaction were important in determining structural survival.

The dollar loss at the Sylmar Converter Station has been estimated at $28 million.

Sylmar and Olive Switching Stations

The Sylmar Switching Station is located immediately north of the Sylmar Converter Station (Figure 5. 1). The air circuit breakers at this location were all knocked out by the earthquake (Figure 5. 13). In addition, several buss-potential transformers fell from their ten-foot high steel support posts. It has been reported that material deformation at the base of the posts indicates severe north-south rocking during the temblor. Two large oil-filled transformers at the site moved on their footings but did not fall.

The Olive Switching Station, also shown in Figure 5. 1, is located on the south side of San Fernando Road immediately east of the intersection of Interstate Highways 5 and 210. This station is part of the transmission system handling power from Owens Gorge and several aqueduct plants. The damage to equipment at this facility was similar to that observed at nearby a-c facilities. All of the air-blast circuit breakers were disabled, and several oil circuit breakers broke their anchors. There were several different types of anchors involved. Six large transformers were supported on tracks above the grade. Their only constraint was wedges used to prevent sliding of the units on the tracks. All of these transformers fell from their tracks.

Vincent Substation

The Vincent Substation is the northernmost Southern California Edison Company connection to two 500 KV intertie lines. There are two banks of transformers at the station, and each bank is rated at 1,000,000 KVA. The transformers are of the oil-filled type with a sudden-pressure-change sensing relay coupled to the gas which is entrained above the oil. The relays on one bank of transformers were triggered at the time of the earthquake presumably due to sloshing of the oil in the transformer housing.

The major physical damage at the site was to one 500 KV power circuit breaker. One complete three-phase circuit breaker consists of nine canisters weighing approximately 3500 pounds each. These canisters rest atop hollow gas-filled porcelain columns which are approximately 18 inches in diameter, and 15 feet high. The three columns of one phase of the circuit breaker are mounted on a support structure which in turn rest on six legs with a separate footing for each pair of legs. Some additional horizontal strength is provided by two steeply inclined diagonal porcelain struts which are attached to each canister. Three modules failed. The supporting columns were destroyed and the canisters were left suspended from attached powerlines.

Residential Electrical Service

Electrical service failed briefly in many areas of the Los Angeles Basin immediately after the earthquake. The Los Angeles Department of Water and Power estimated that 100,000 homes were without power for some time on Tuesday, 9 February. Interruption of service was in many instances due to sway and subsequent touching of powerlines. In most cases, the momentary contact of the lines tripped automatic line-load relays which were later reset, and no permanent damage was incurred. However, in some cases the touching lines were severed by the resulting electrical discharge. There were several instances of small residential transformers falling from utility poles.

Repair was hampered in some of the hardest hit areas, such as Newhall, due to the restricted access resulting from highway closures.

Natural Gas

The Southern California Gas Company is the major supplier of natural gas to the area of extensive earthquake damage. They indicate that there was damage to four primary gas feeder lines in the 12-inch to 26-inch diameter class. These lines deliver gas from the San Joaquin Valley. The damage occurred between Newhall and San Fernando. Heaviest damage to the local distribution system occurred within a 10-20 square mile section of the Sylmar-San Fernando area. This system consists principally of a network of 2-4 inch welded steel lines. Approximately 450 breaks were reported. Service to some 17,000 customers was interrupted.

The Los Angeles Times indicated that there were reports of some 456 fires within the first 8 hours after the earthquake but the Southern California Gas Company has said that very few fires were caused by leaking gas lines.

Some of the most spectacular gas line failures occurred along Glen Oaks Boulevard near its intersection with Hubbard Street. Here, escaping gas from ruptured 16-inch gas lines created numerous craters in the roadway (Figures 5.28 - 5.31). These craters were of varying

Figure 5.28 Aerial photo of area near intersection of Hubbard Street and Glenoaks Boulevard showing craters from ruptured gas main.

Figure 5.29 Shattered curb on northeast side of Glenoaks Boulevard about one block southeast of Hubbard Street. This approximately 18" of displacement of curb reflects the compressive deformation of the soil along this street where numerous gas line failures occurred.

Figure 5. 30 Craters from ruptured gas main on Glenoaks Boulevard. Fourteen such craters were formed.

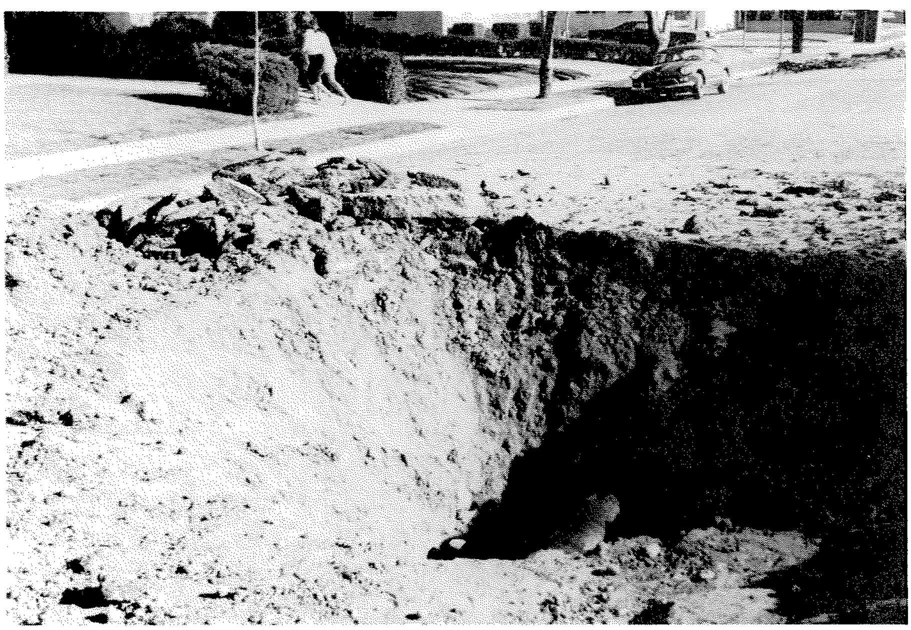


Figure 5. 31 Crater from ruptured gas main on Glenoaks Boulevard. The 16 in diameter line has already been repaired.

size, with some as large as 10 feet in diameter and 8 feet deep. In some cases the gas was ignited causing damage to surrounding structures (Figure 5.33). Several of the failures appeared to result from axial compression of the gas line, as indicated in Figure 5.32). In other cases the lines were simply pulled apart (Figure 5.34).

Water mains were ruptured in the immediate area of the gas line failures. This may have been due to earth movement, or may have resulted from gas line eruptions. The water line breaks made repairs to the gas lines more difficult, and probably led to greater contamination of the repaired lines. Some ten days after the earthquake, abnormally low gas pressures were reported in the Sunland-Tujunga area, and service to some 10,000 homes was discontinued while clogged filters at regulator stations were cleaned.

There were also reports of ruptured gas lines in Highland Park and in the Glendale-La Canada area. In Eagle Rock, some 23 miles from the epicenter, a ruptured gas line blew a crater in a freeway overpass bridge.

The major transmission lines from out-of-state, 30-inch diameter and above, did not suffer any apparent damage.

Total damage to the gas utility system has been estimated at approximately $2 million.

Water Distribution Systems

Main Los Angeles Department of Water and Power water feeder lines into the Mission Hills, Granada Hills, Porter Ranch and Sylmar areas broke during the earthquake cutting off water to scores of homes. The 48-inch Granada Trunk Line had 19 breaks and the 54-inch Susana Trunk Line had 3 breaks. Several older steel-rivited lines pulled apart, and some lines were offset by as much as 2 feet. Many small surface

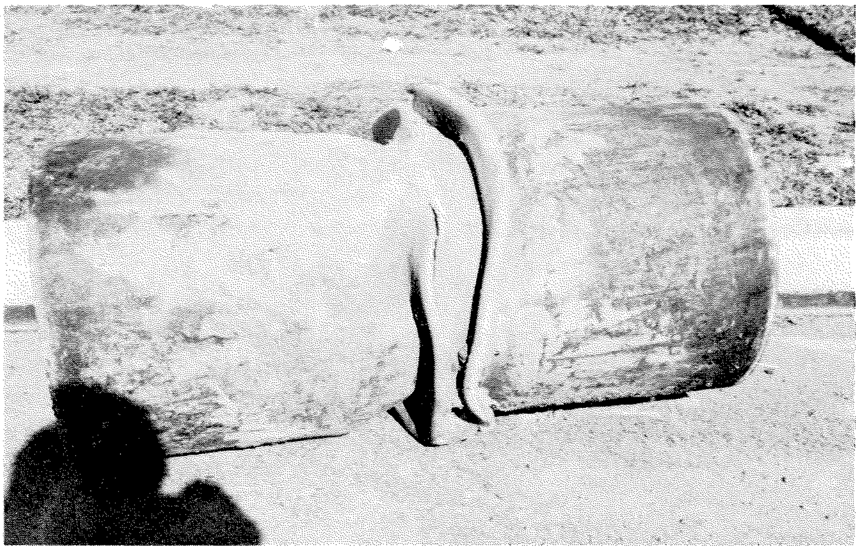


Figure 5.32 Section of 16 in ruptured gas line from Glenoaks Boulevard.

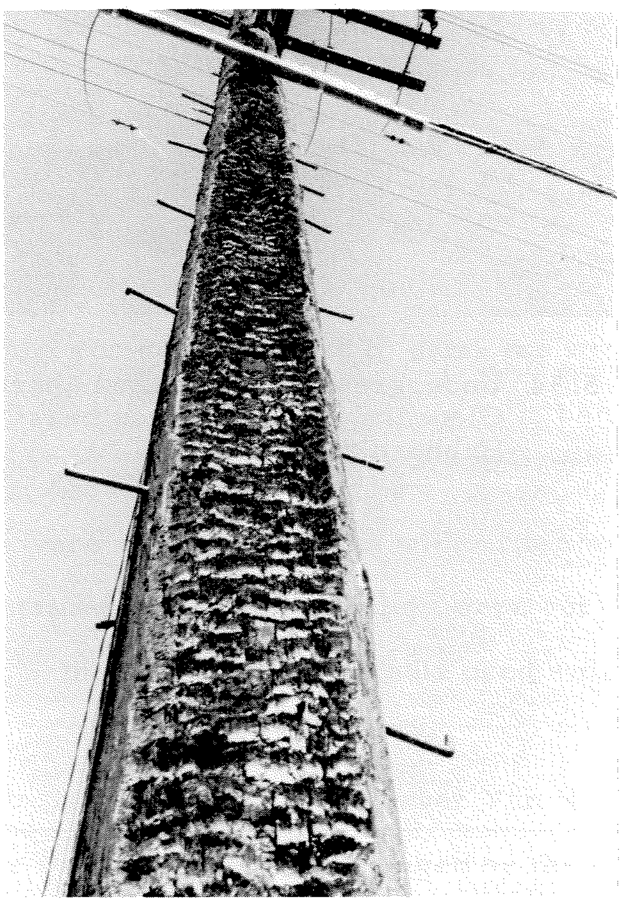


Figure 5.33 Utility pole damaged by fire following rupture of gas main on Glenoaks Boulevard.

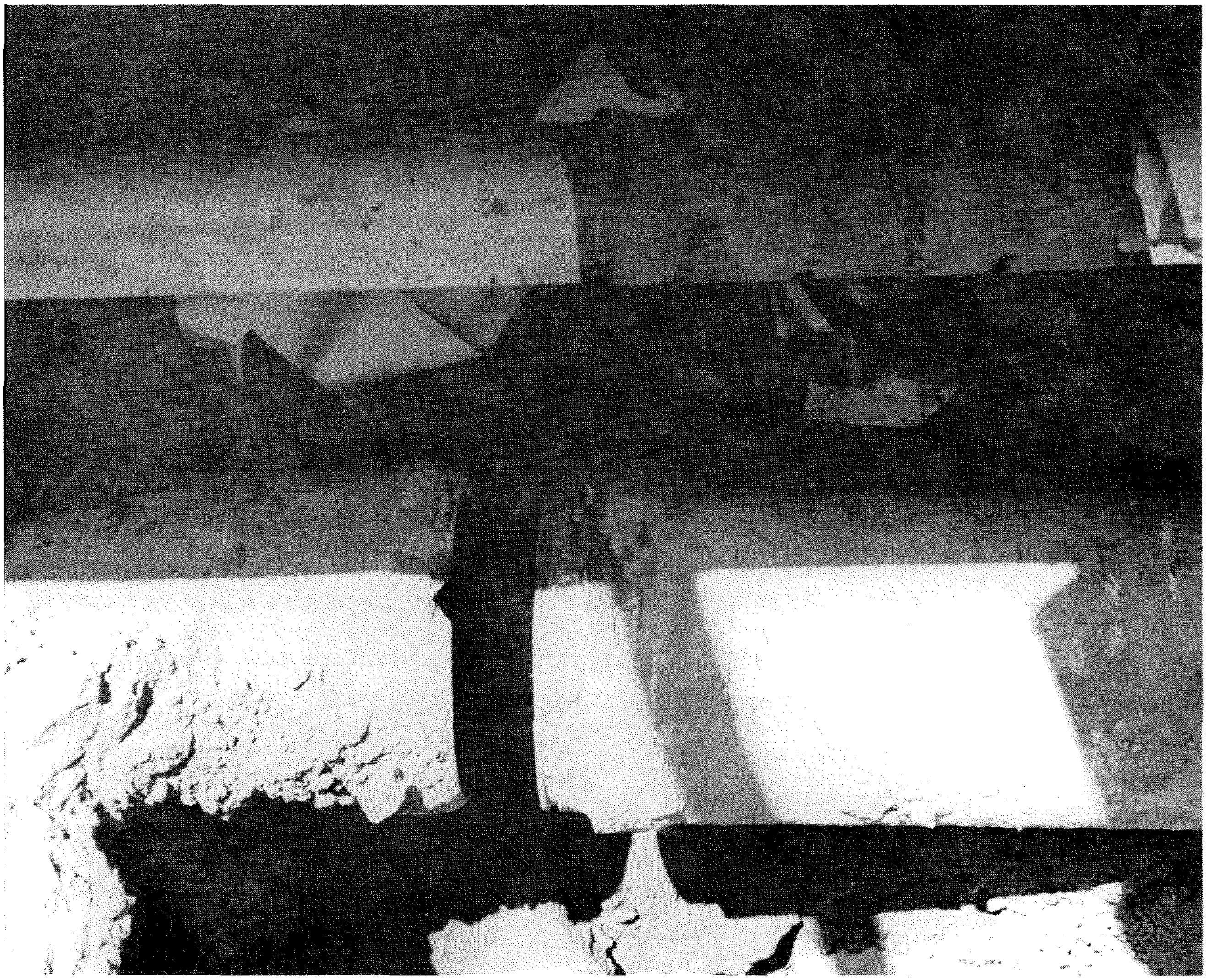


Figure 5.34. Underground pipes pulled apart on Glenoaks Street just southeast of Hubbard Street.

laterals to residences were broken at the feeder lines, further compounding the repair problem. Several fire hydrants were also broken off as a result of earth movement (Figure 5.35). The city-owned water system in the City of San Fernando was shut down for a time, with damage reported to local distribution equipment. Broken water lines were reported as far away as 16 miles from the epicenter in La Canada, but most of the damage was restricted to the areas listed above.

Both Los Angeles aqueducts were damaged in the area immediately north of the Cascades. One aqueduct was put back into service within two days after the earthquake and the other was back in service by early April. A gravity feeder into the Maclay Reservoir at Maclay and Gladstone Avenues was rather heavily damaged and the 5.3 million gallon reservoir itself suffered extensive damage. The walls of the reservoir were cracked and the roof partially collapsed.

Shortly after the earthquake, water in the area surrounding the Van Norman Reservoirs became brackish, and although the Los Angeles Department of Water and Power announced that the water was potable in most instances, many residents refused to drink tap water. It is believed that a possible source of the discoloration may have been the disturbance of settlements in the reservoirs of the area.

Water trucks were sent to the areas without tap water, and residents filled their own containers. Chemical toilets were set up in affected residential areas. Some residents reportedly used swimming-pool water to flush toilets.

The 99-inch bypass line around the lower Van Norman Reservoir completed last year was essentially undamaged by the earthquake. This line is now being used to transfer water from the upper Van Norman

Figure 5.35 Fire hydrant broken by ground movements, Sylmar area.

Reservoir to the major trunk lines at the outlet of the disabled lower Van Norman Reservoir.

Total damages to the water system, including damage to the Van Norman Reservoirs, is estimated at $48 million.

Telephone Service

Disabling damage to telephone equipment and facilities appeared to be confined to those populated areas closest to the epicenter. However telephone facilities all over the Los Angeles area felt the impact of abnormally heavy communication loads. American Telephone and Telegraph reported that there were 3.5 million long-distance calls into and out of the Los Angeles area on Tuesday, and the volume of calls handled on both Tuesday and Wednesday was 200% of normal. Local switching stations were also taxed by the many calls to fire and police departments.

The most extensive damage to facilities was in the Sylmar and Newhall areas. The General Telephone central switching facility in Sylmar was put out of commission affecting between 10 and 20 thousand customers. This facility is located at the intersection of Polk Street and Borden Avenue in Sylmar (Figure 5.36). There was cracking of wall panels, cracking of joinery between columns and walls, diaphragm slippage in the roof, cracking of floor slab and joinery separation, cracking in the column seats, and cracking in the cable vaults. Most of the 12 ft-high equipment bays toppled as illustrated by Figure 5.37. The major cabling was attached at the top of each bay and was either broken or pulled loose when the bays fell. The bays are normally anchored to the concrete floor and have support members near their top, but the details of these features are not known at the present time. The loss at this facility was estimated at $4.5 million. The company estimated that full service would not be

Figure 5. 36 General Telephone switching facility at Polk Street and Borden Avenue.

Figure 5.37 Interior of damaged General Telephone switching facility at Polk Street and Borden Avenue.

restored until May 1971. Mobile units were moved in to restore partial service in the hard-hit Sylmar area. Message centers were set up where local residents could be notified of incoming calls.

The Pacific Telephone facility in Newhall reported flooding in basement areas and equipment bays were twisted but the facility was quickly put back into commission. There were reports of light damage to other switching facilities, but this did not affect service. Some equipment bays tipped and had to be jacked back into position. There were also reports of conduit failures due to earth movement at various locations, but no reports of associated cable breakage or interruption of service.

EARTHQUAKE DAMAGE TO FREEWAY STRUCTURES

by P. C. Jennings and J. H. Wood

Introduction

One of the most notable features of the earthquake was the damage sustained by freeway structures. This damage, confined mainly to the epicentral area (Figure 1.2) included vibration failures of overpass structures, embankment and abutment failures, failures from ground movements, faulting damage to pavements, and slumping of cuts and fills. There were 42 bridge structures that received significant damage, (counting twin freeway bridges as single structures), and five structures collapsed. The total damage to the freeway system has been estimated to be about 15 million dollars.

The most spectacular damage occurred to overpass structures at three major interchanges: the Golden State freeway (Interstate 5) and the Antelope Valley freeway (California 14); the Golden State freeway and the Foothill freeway (Interstate 210); and the Golden State freeway and the San Diego freeway (Interstate 405). As seen in Figure 1.2, these three interchanges are all in the region of strong shaking. Earthquake damage to bridges, primarily on the Foothill, Antelope Valley and Golden State freeways, was the other major contributor to damage costs.

Major Interchanges

The interchange between the Golden State freeway and the Antelope Valley freeway was under construction at the time of the earthquake as shown in Figure 6.1, taken before the earthquake. A post-earthquake view from approximately the same location is given in Figure 6.2, and Figure 6.3 shows the interchange from the opposite direction. As is seen

Figure 6.1 Pre-earthquake view of the Golden State-Antelope Valley freeway interchange, looking west. Photographed by Ralph Samuels

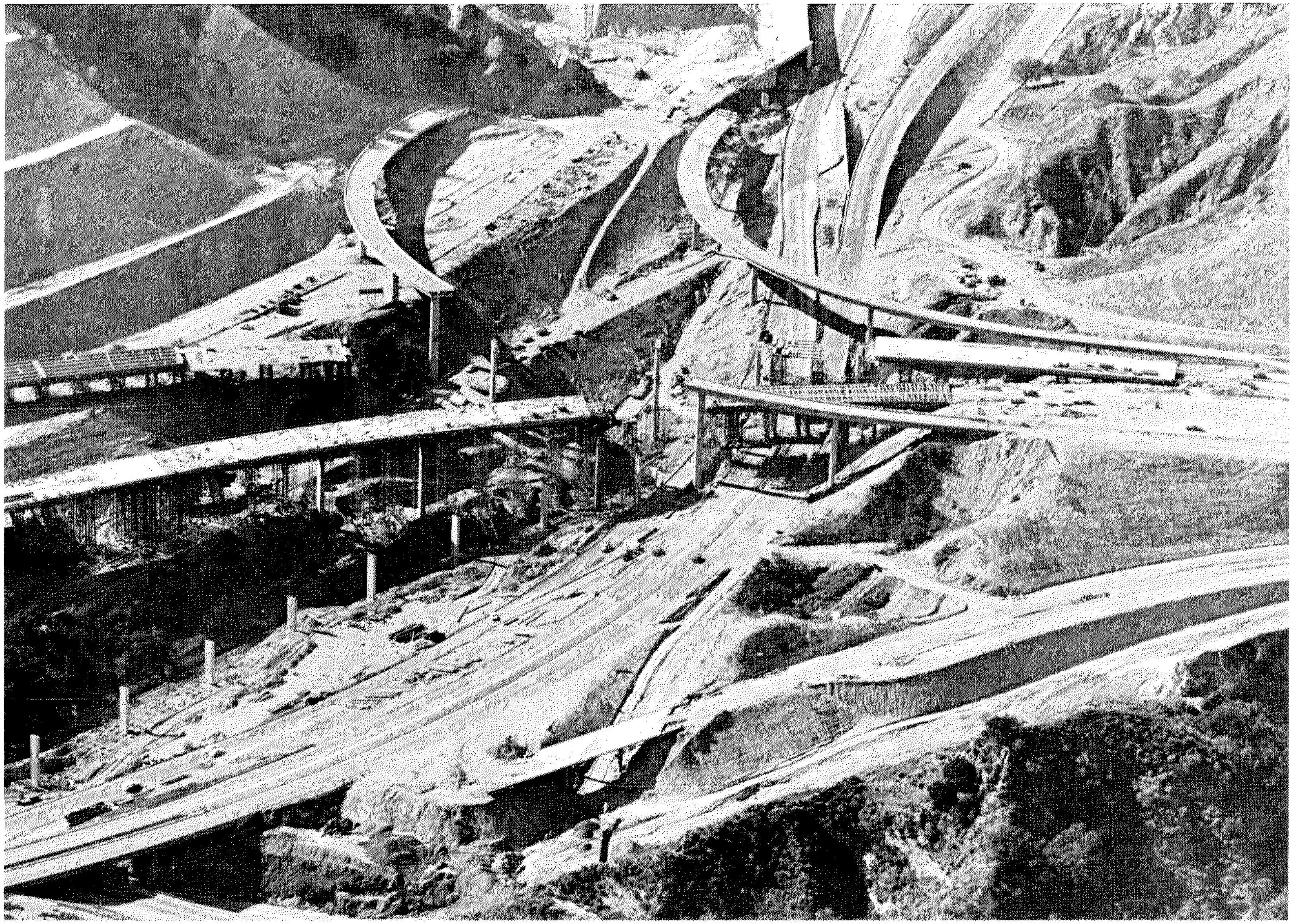


Figure 6.2 Post-earthquake view, looking west, of the Golden State-Antelope Valley freeway interchange. The center two spans of the highest overpass have collapsed. Photographed by Ralph Samuels

Figure 6.3 Post-earthquake view, looking east, of the Golden State-Antelope Valley freeway interchange.

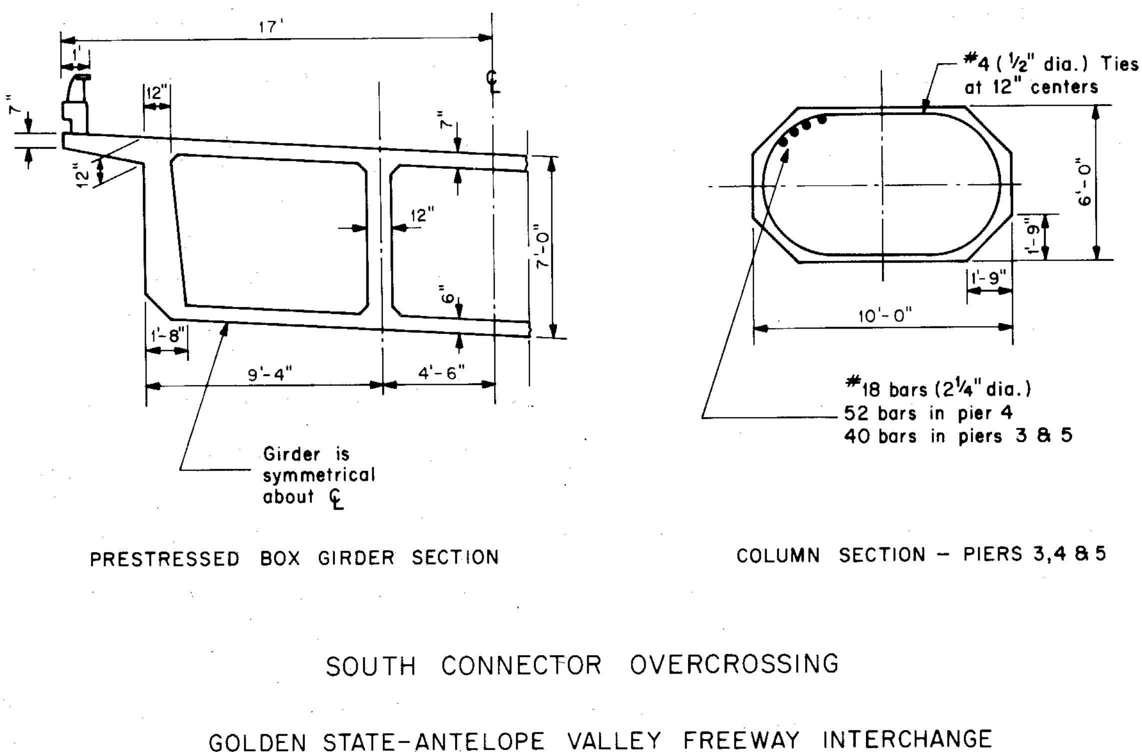


Figure 6.4 Details of collapsed structure, Golden State-Antelope Valley freeway interchange.

in the figures, the central section of the curved, nine-span South Connector Overcrossing collapsed. This bridge was structurally complete at the time of the earthquake. The collapsed section consisted of a two-span, prestressed, post-tensioned box girder supported by a central column and by reinforced concrete box sections at the ends. The central supporting column collapsed with the superstructure and minor damage was done to the spans at either end of the collapsed section. Figures 6.4 and 6.5 give approximate dimensions for the structure.

The 14-in seat which supported the western end of the collapsed section is shown in Figure 6.6 and closer views of the seat on the fallen section are given in Figures 6.7 and 6.8. The fractured 1-1/2 in diameter steel bolt shown in the figures is one of three at each joint in the superstructure used to equalize the longitudinal deflections expected at the joints from creep and temperature effects. The pier in the center of Figure 6.6 apparently was damaged slightly by the collapse of the nearby structure. This pier is for an incomplete overpass as can be seen by examination of Figure 6.3.

The collapsed column is shown in Figure 6.9 in which it is seen that failures occurred at the ends, and the central portion of the column suffered little damage. A crane crushed by the impact of the falling bridge can be seen under the top end of the pier. Some construction details at the head of the fallen column are visible in Figure 6.10.

The remaining portion of the overcrossing suffered only moderate damage as shown in Figures 6.11, 6.12 and 6.13. Figure 6.11 is a view of the western section of the overpass at column 6 (see Figures 6.5, 6.2 and 6.3). There is evidence of movement at the joint, but the column appears undamaged. Figure 6.12 shows the next joint toward the abutment

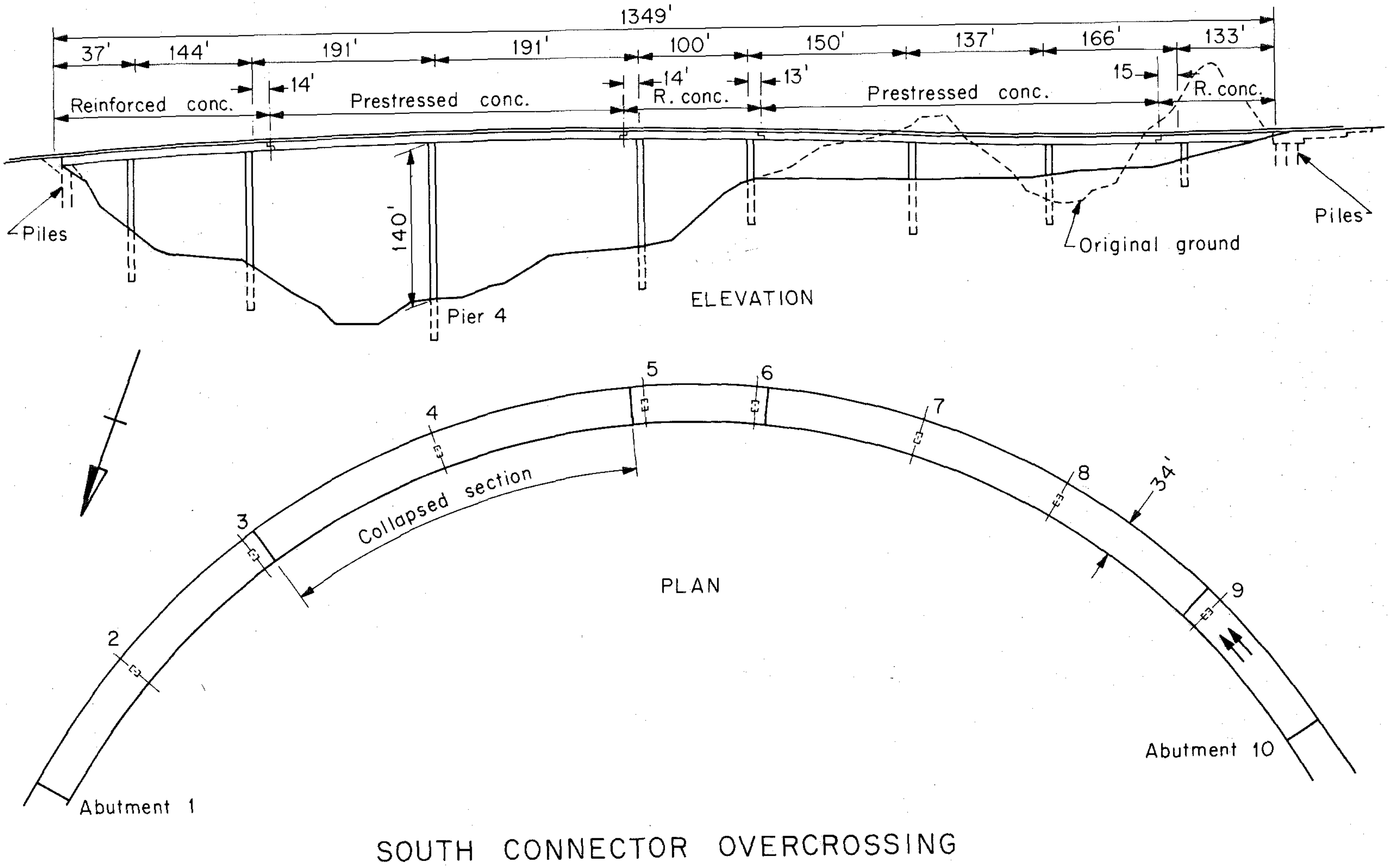


Figure 6.5 Plan and elevation view of the South Connector overcrossing, Golden State-Antelope Valley freeway interchange.

Figure 6.6 Western support for the collapsed overcrossing spans, Golden State-Antelope Valley freeway interchange.

Figure 6.7 Seating detail of the western end of collapsed overcrossing, Golden State-Antelope Valley freeway interchange.

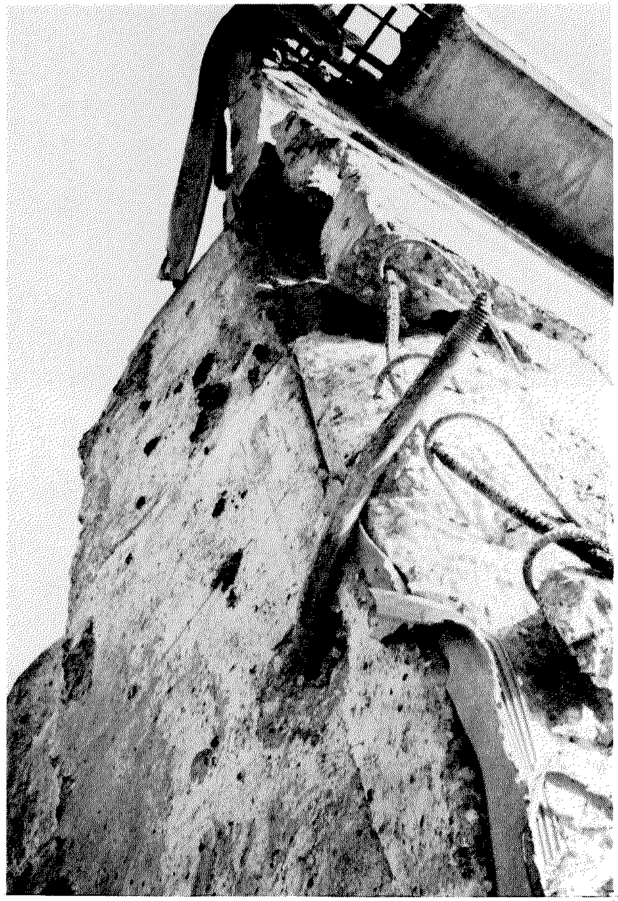


Figure 6.8 Closeup view of western end of collapsed overcrossing, Golden State-Antelope Valley freeway interchange. The bolt is one of three used to control longitudinal deformation at the joint.

Figure 6.9 Central pier of collapsed overcrossing, Golden State-Antelope Valley freeway interchange. View is looking west from the eastern portion of the remaining structure.

Figure 6. 10 Top of the central supporting column of the collapsed overcrossing at the Golden State - Antelope Valley freeway interchange.

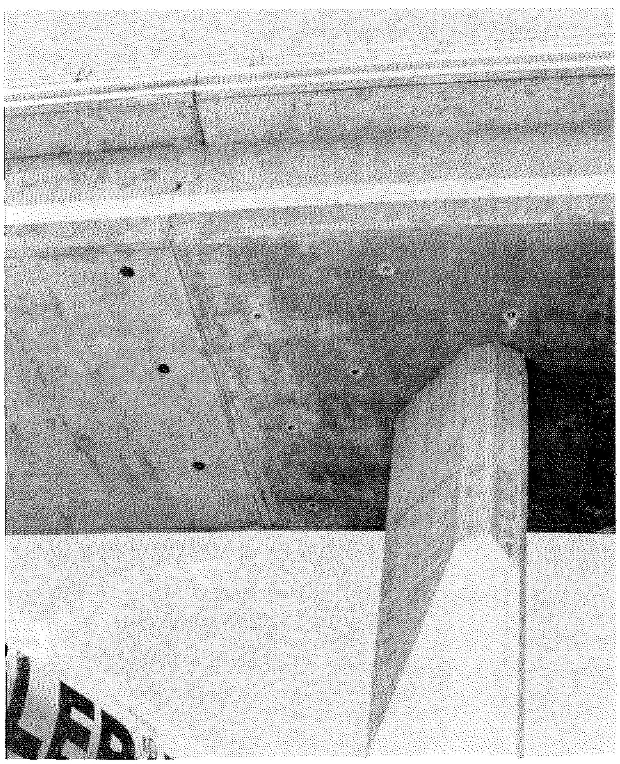


Figure 6. 11 Joint and column on the remaining western portion of the collapsed overcrossing, Golden State-Antelope Valley freeway interchange.

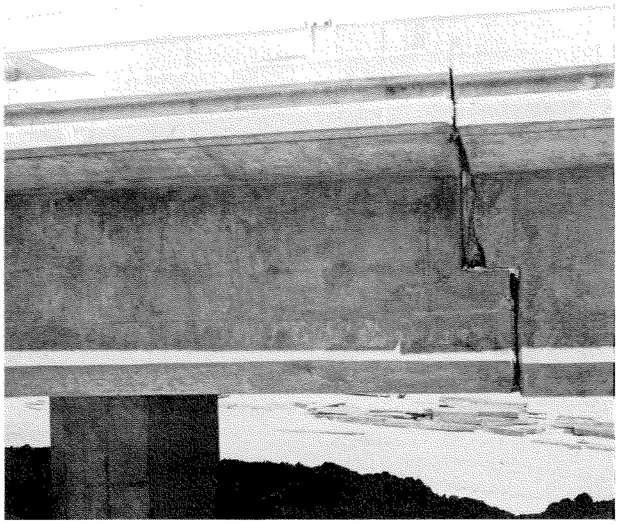


Figure 6.12 Damaged joint on the remaining western section of the collapsed overcrossing, Golden State-Antelope Valley freeway interchange.

Figure 6.13 Damaged column on the remaining western section of the collapsed overcrossing, Golden State-Antelope Valley interchange.

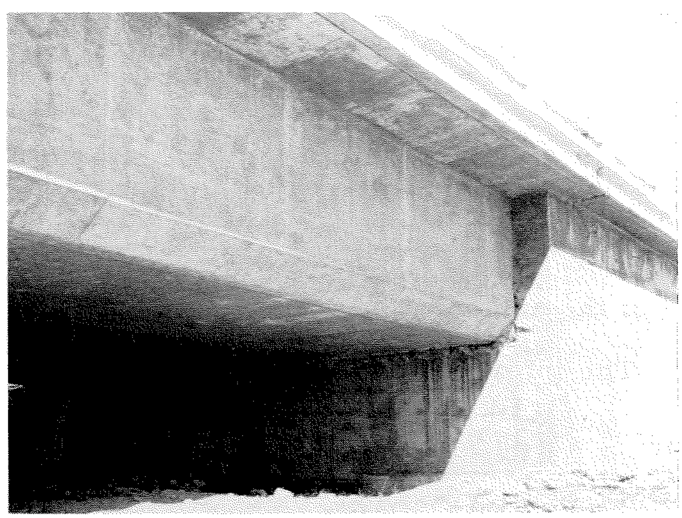


Figure 6.14 Abutment of the remaining eastern section of the collapsed over-crossing, Golden State-Antelope Valley freeway interchange.

on the western section of the overpass and again there is evidence of some movement. Column 9, the column nearest the western abutment, is shown in Figure 6.13. It suffered shear cracking and the surrounding soil shows signs of rotation at the base of the pier. The abutment of the eastern section was damaged also, as can be seen in Figure 6.14.

There are a variety of possible ways that the bridge structure might have failed and it is hoped that more detailed studies will show how the collapse actually occurred. There are two points that do seem clear at this time, however. First, the evidence strongly indicates that it was a vibration failure, and permanent ground displacements (none have been noted) are not thought to have played a significant role in the collapse. Second, the small length of seating at the ends of the fallen section, the lack of effective ties to neighboring sections, and the general configuration of the inverted-pendulum structure are indicative of inadequate attention to the effects of strong earthquake motion.

The rest of the interchange, including the portions under construction, escaped major damage as is shown, for example, in Figure 6.15. Some damage occurred when the falsework under a prestressed box girder settled as seen in Figure 6.16. The girder was not prestressed at the time of the earthquake.

The interchange between the Golden State and Foothill freeways (Figure 1.7) is a large complex of overcrossings and bridges as can be seen in the aerial photographs (Figures 6.17, 6.18 and 6.19). The interchange was in the final stages of construction at the time of the earthquake and the major structures were complete, but not all were open to traffic. Many collapses and failures occurred in the interchange complex and it was here that two men were killed when a collapsing overpass crushed

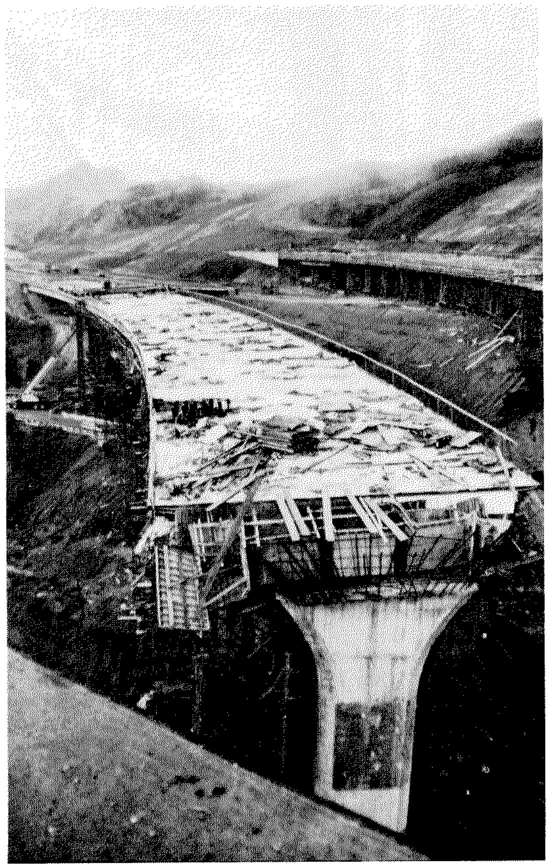


Figure 6. 15 Bridge under construction at the time of the earthquake, Golden State-Antelope Valley freeway interchange.

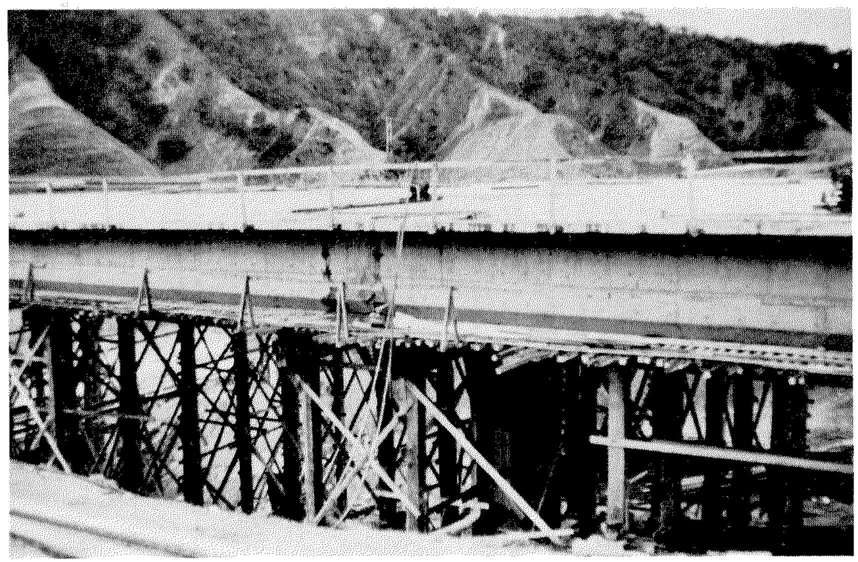


Figure 6. 16 Damage caused by settlement of falsework, Golden State-Antelope Valley freeway interchange.

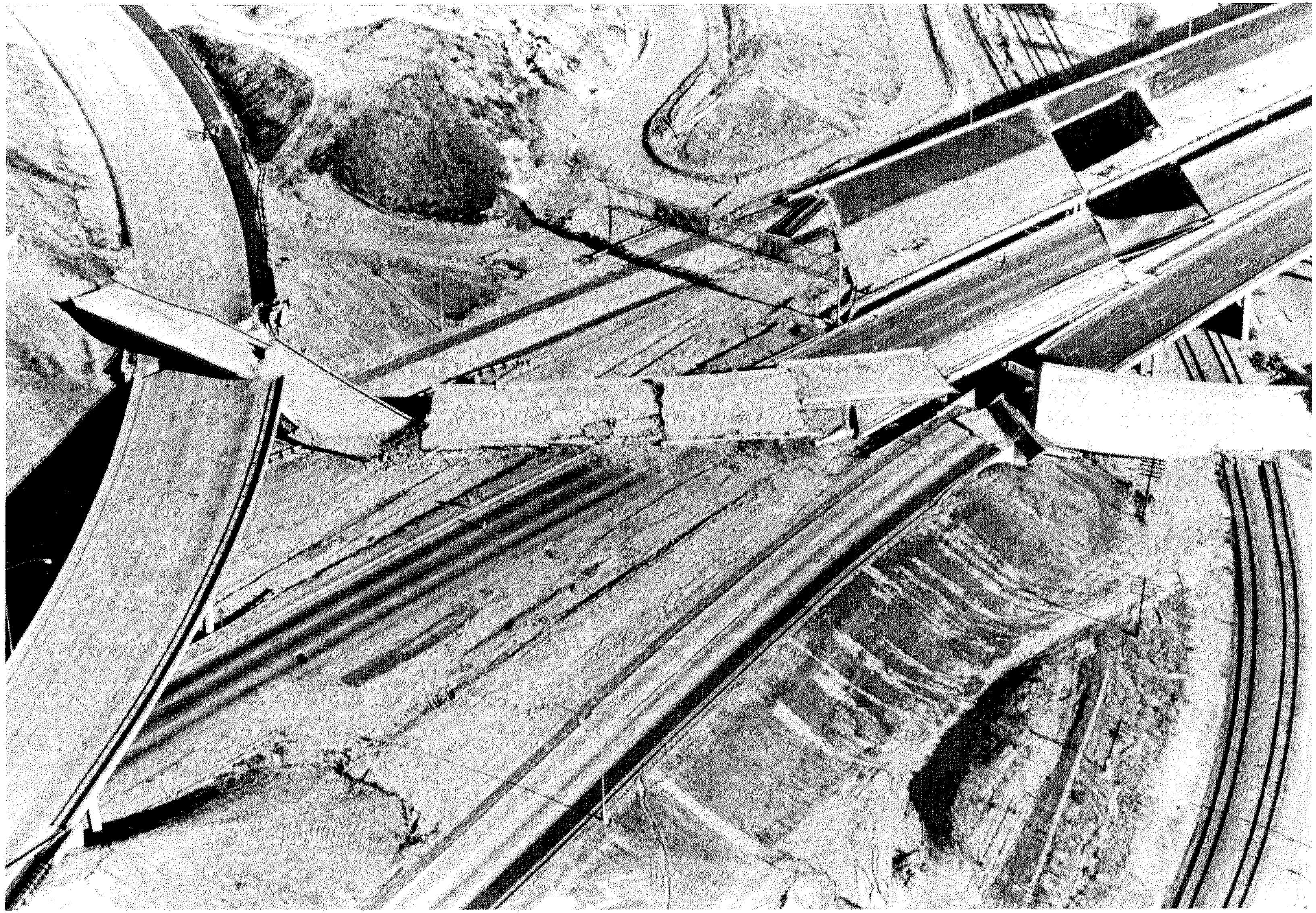


Figure 6.17 Earthquake damage to Golden State-Foothill freeway interchange. L.A. Times photograph. Taken the day of the earthquake.

Figure 6.18 Vertical aerial photograph of the Golden State-Foothill freeway interchange, taken February 12, 1971.

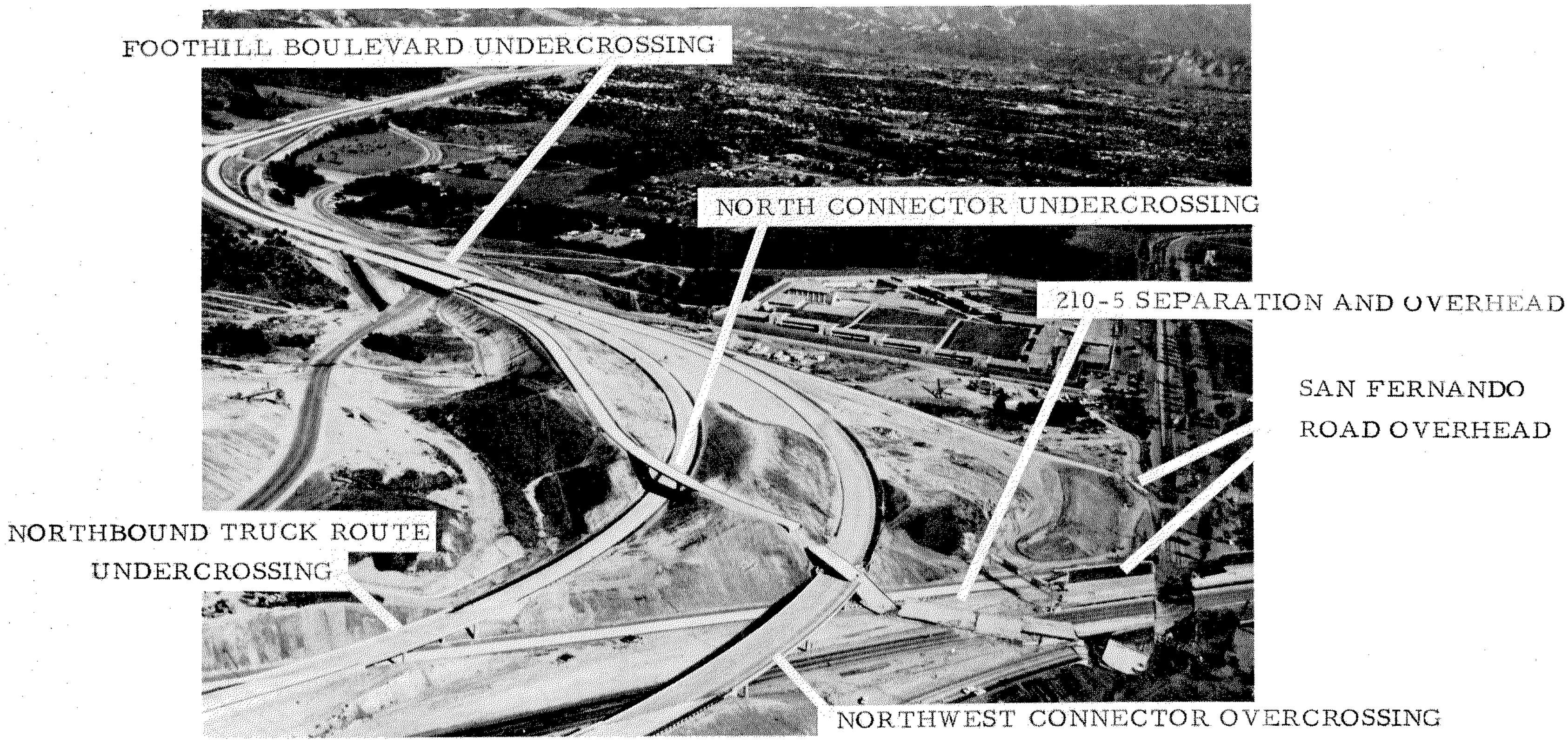


Figure 6.19 Aerial photograph of the Golden State-Foothill freeway interchange, looking southeast.

their pickup truck. Some of the major structures in the interchange are identified in Figure 6.19 for purposes of the discussion which follows.

The most obvious damage is to the Separation and Overhead structure which brings westbound traffic from the Foothill freeway into the southbound lanes of the Golden State freeway. The overpass structure was completed, but the filled section of the off-ramp down to the southbound Golden State was not paved. General views of the collapsed reinforced concrete box structure are shown in Figures 6.20 and 6.21. These photographs, taken on February 11, are from the east side of the Golden State freeway, looking southwest and northwest, respectively. Figure 6.20 is taken from a more southerly position than Figure 6.21. The illustrations of the fallen overpass show that it fell generally to the north, with failures of the supporting columns occurring at both top and bottom. It is not known whether the failure of the column bases was a cause or result of the collapse, but the failure of the reinforcing steel in bond indicates that the full moment capacity of the columns at their bases was not reached.

Some of the construction details and dimensions of the fallen overpass are presented in Figure 6.22 and a closer view of one of the fallen sections is shown in Figure 6.23, which shows the seating detail, including a shear key. It is seen that there is no positive connection between adjacent sections, and the 14-in seat may have been too short to maintain effective contact between the two sections of the bridge during the vibrations and ground movements caused by the earthquake.

There were obvious signs of permanent ground movement in the area of the interchange, the nearby Juvenile Hall facility, the Sylmar Converter Station and the Metropolitan Water District Treatment Plant (Figure 1.2). At this writing the cause of the ground movements is not

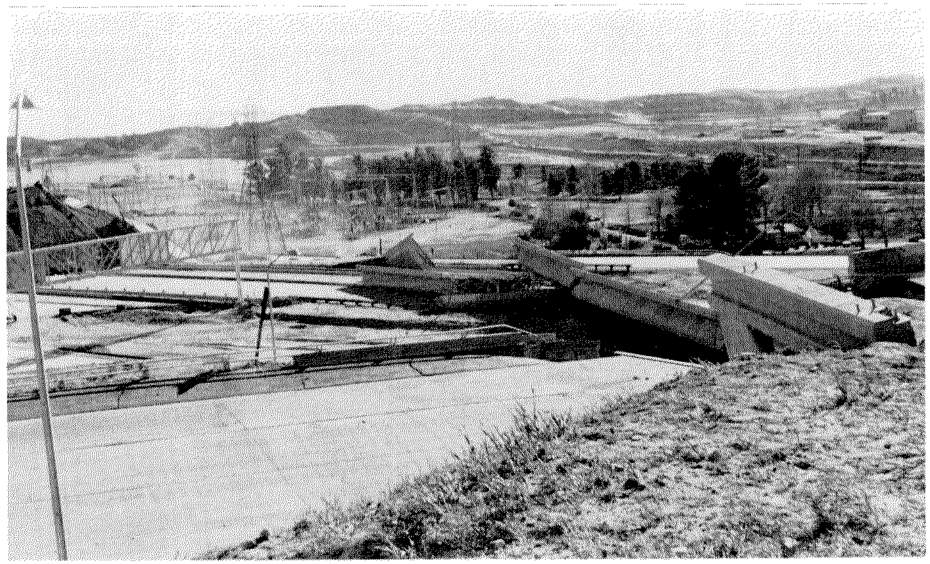


Figure 6.20 Southern portion of collapsed separation and overhead structure, Golden State-Foothill freeway interchange.

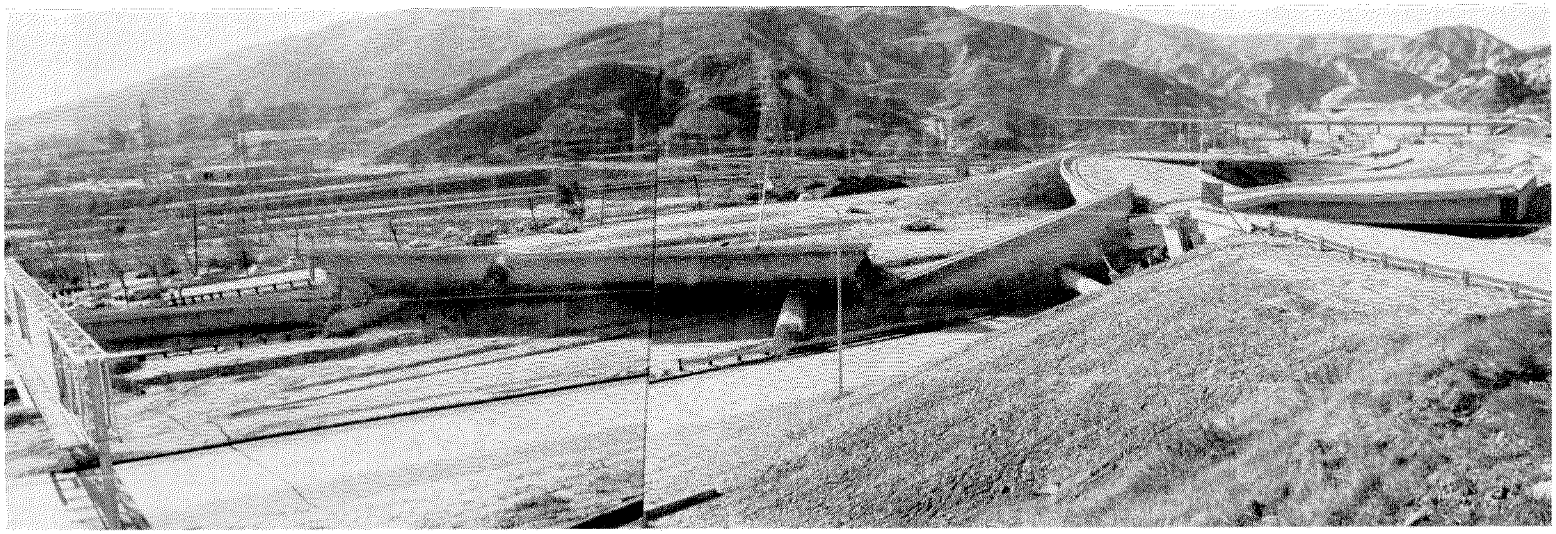
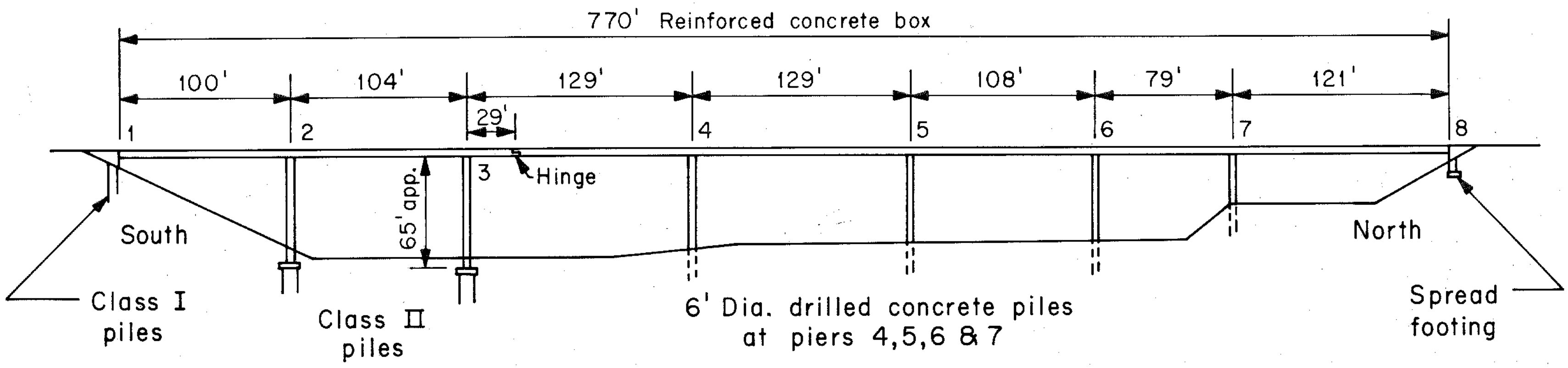
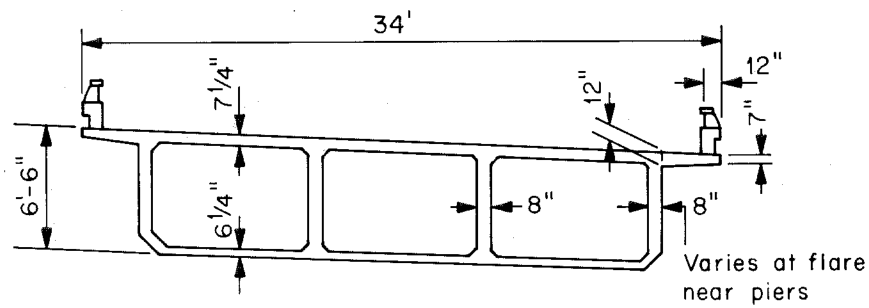


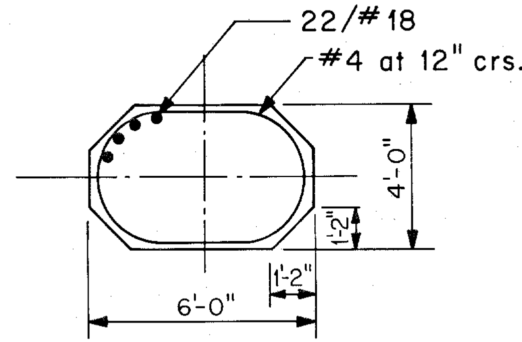
Figure 6.21 Northern and central portions of collapsed separation and overhead structure, Golden State-Foothill freeway interchange.



ELEVATION



R.C. BOX GIRDER SECTION



COLUMN SECTION

SEPARATION AND OVERHEAD STRUCTURE

Figure 6.22 Construction details of the separation and overhead structure, Golden State-Foothill freeway interchange.

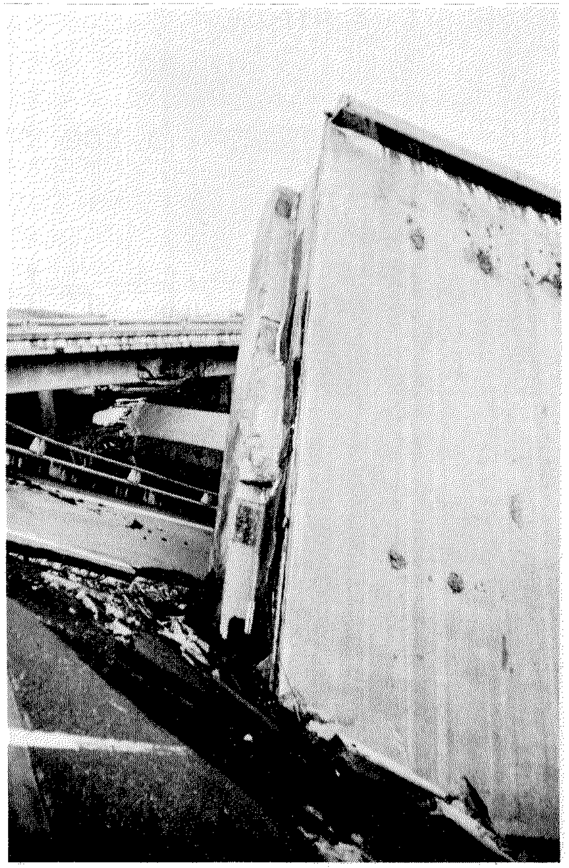


Figure 6.23 Seating details of fallen separation and overhead structure, Golden State-Foothill freeway interchange.

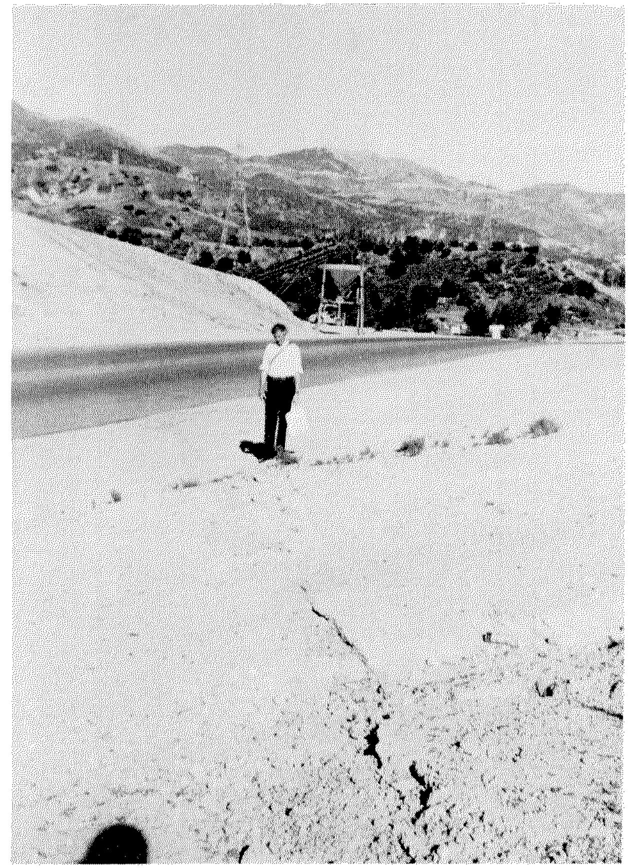


Figure 6.24 Looking northeast along a through-going crack in the area of the Golden State-Foothill freeway interchange.

resolved; both tectonic movement and slumping over a large area have been advanced as the basic cause of the observed displacements. Figure 6.24 shows an example of the permanent ground displacements observed in the area of the interchange. The crack, which showed left-lateral movement of about two inches, and perhaps some thrusting of the right (northeast) side, could be traced for about 100 yards. It was not obviously related to any features of the construction in areas of cut and fill nor to the local topography on a section of the crack which traversed a section of natural ground. The degree to which the permanent ground movements contributed to the failure of the overpass and the other failures in the area will have to await more detailed study.

As can be seen in Figures 6.17, 6.18 and 6.19, the collapsed overpass fell onto the northeast abutment of the Northwest Connector overcrossing which feeds southbound traffic on the Golden State freeway to eastbound traffic on the Foothill freeway. As a result of the impact, and probably augmented by vibration and soil movements, the Northwest Connector overcrossing suffered severe damage. The reinforced concrete-box bridge, viewed from the north, is illustrated in Figures 6.25, 6.26 and 6.27. Figure 6.28 is a view of the abutment nearest the column shown in the foreground of Figure 6.27.

North of the Northwest Connector overcrossing is a similar, but smaller, overcrossing serving the transition road from the westbound Foothill freeway to the northbound Golden State freeway. This bridge, termed the Northbound Truck Route undercrossing, can be seen in the left foreground of Figure 6.19. Two views of the southern side of the bridge are given in Figures 6.29 and 6.30. These figures show column damage and evidence of substantial abutment movement. Figures 6.31 and 6.32

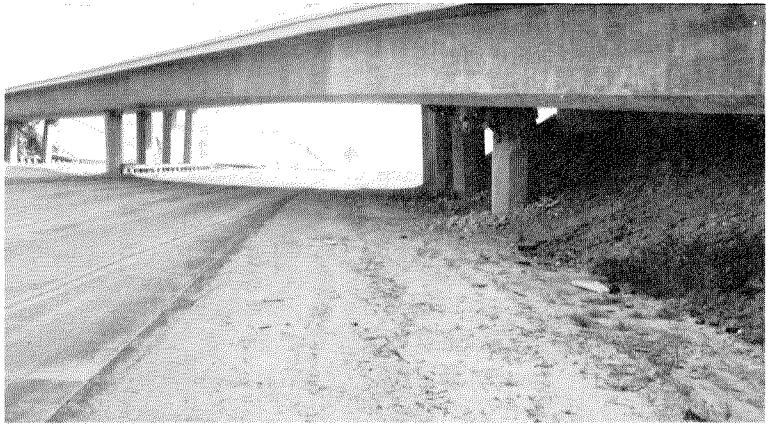


Figure 6.25 Northwest connector overcrossing, Golden State-Foothill freeway interchange.

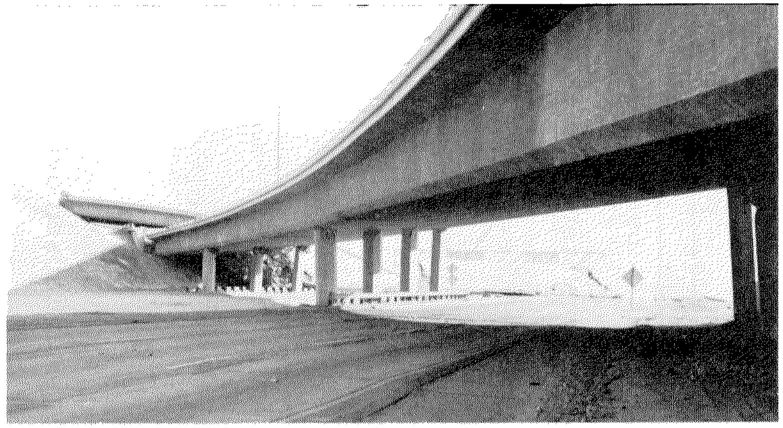


Figure 6.26 Northwest connector overcrossing, Golden State-Foothill freeway interchange.

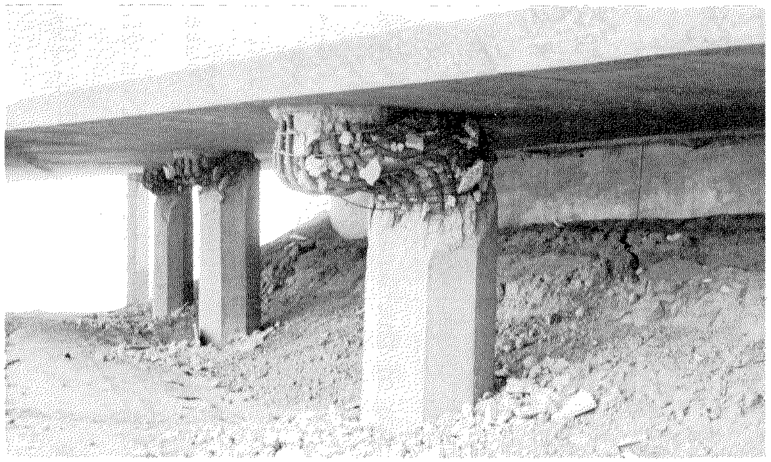


Figure 6.27 Column failures, northwest connector overcrossing, Golden State-Foothill freeway interchange.

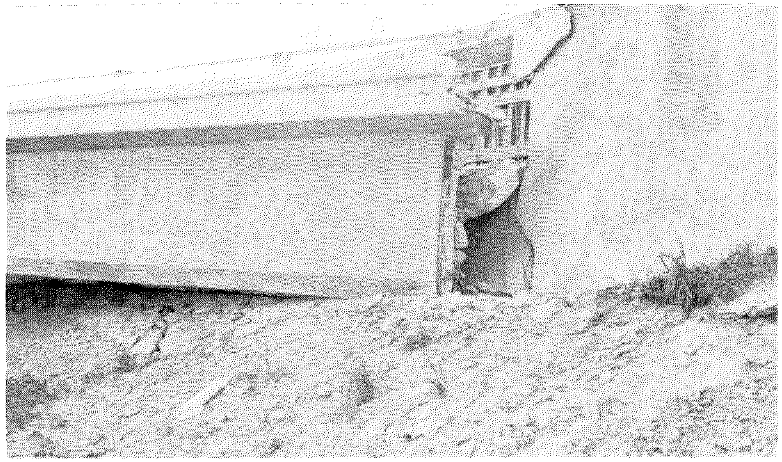


Figure 6.28 Northwest abutment of the northwest connector overcrossing, Golden State-Foothill freeway interchange.

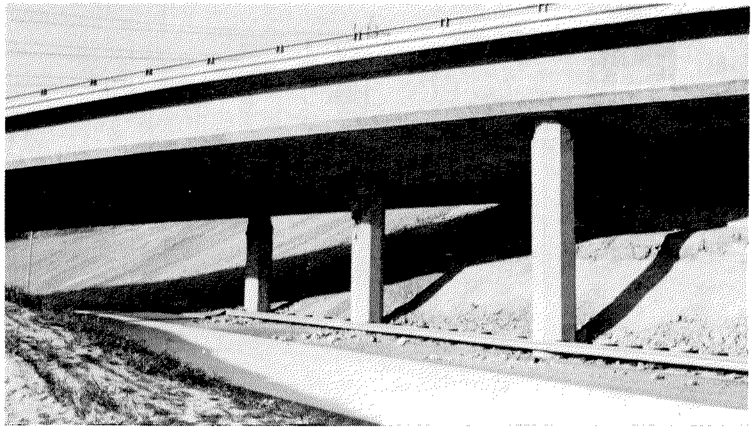


Figure 6.29 Northbound Truck Route undercrossing, Golden State-Foothill freeway interchange.

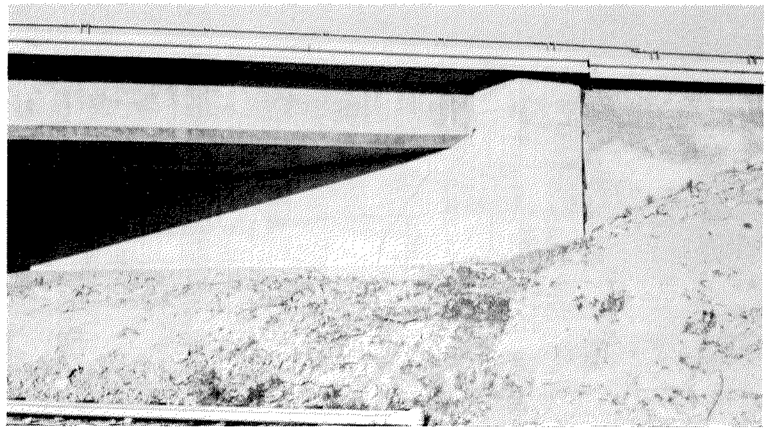


Figure 6.30 Eastern abutment, northbound Truck Route undercrossing, Golden State-Foothill freeway interchange.

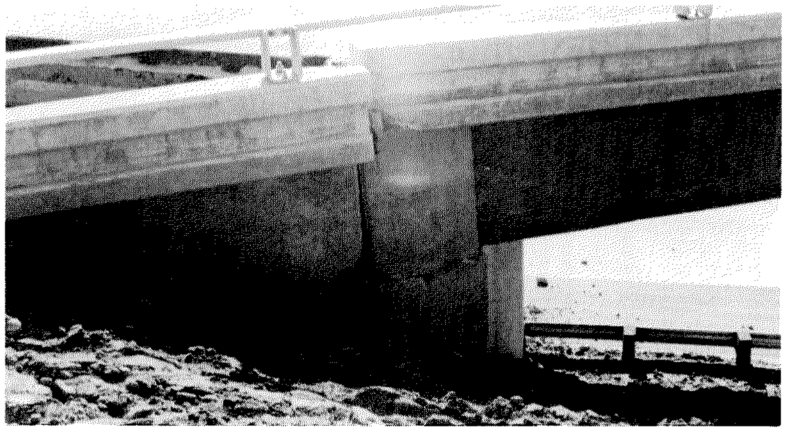


Figure 6.31 Eastern abutment, northbound Truck Route undercrossing, Golden State-Foothill freeway interchange.

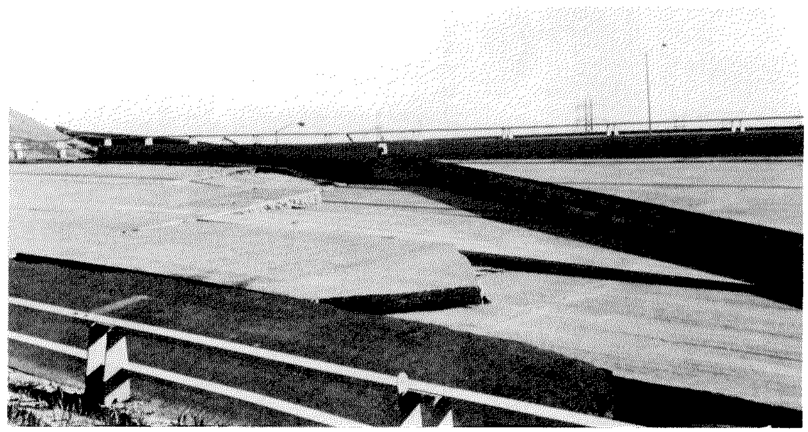


Figure 6.32 Backfill slumping at the eastern end of the Truck Route undercrossing, Golden State-Foothill freeway interchange.

are taken from near the east abutment and show the extent of column and abutment damage; the failure of the backfill at the abutment was especially severe. It is interesting that this bridge, the Northwest Connector over-crossing, and other skew bridges in the area, were left with a permanent, rotational set in the sense of increasing skewness.

Collapse occurred also at the San Fernando Road Overhead which serves as an overpass for the Golden State freeway where it crosses San Fernando Road and the Southern Pacific Railroad, (Figures 6.17, 6.18 and 6.19). As is seen most clearly in Figure 6.17, two sections fell onto the tracks and another section was destroyed by the fallen Separation and Overhead structure. A closer view of the structures looking to the southwest is given in Figure 6.33, and Figure 6.34 is a view looking northwest, parallel to the railroad tracks. This figure shows the damage to the columns and the separation at the abutment. A closer view of the abutment and the damaged columns is given in Figure 6.35 which shows that this bridge also has a permanent set in the direction of increasing skewness. The bridges are seven-span structures with the central spans over the railroad constructed of both steel and precast, prestressed concrete girders. The other spans at either side of the rail crossing are of reinforced concrete box construction. Some details of the joints of the overhead which fell can be seen in Figure 6.36. It seems probable that the seating details for the steel girders could not accommodate the relative movement of the bridge structures during the earthquake. In this regard, it can be seen in Figure 6.17 that the spans which did not fall at the railroad crossing all had only a single joint.

Because the fallen sections blocked an important rail line, they were rapidly removed, as can be seen by comparing Figure 6.18 (February 12)

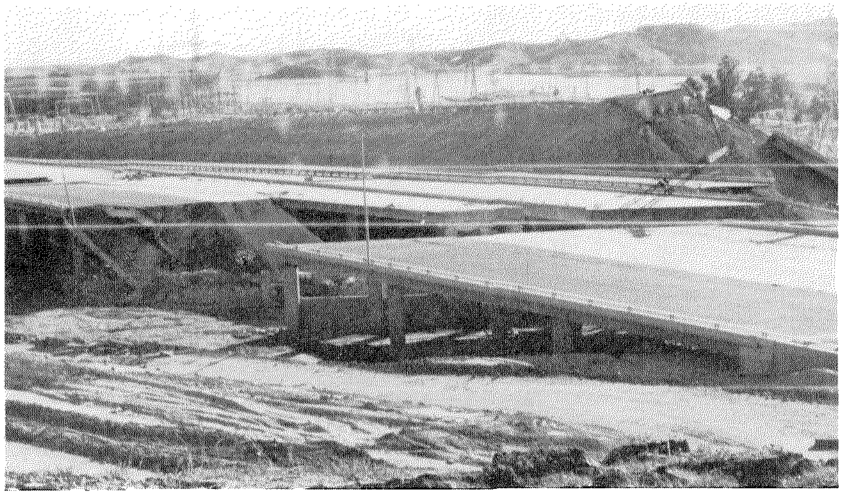


Figure 6.33 San Fernando Road overhead, looking southwest, Golden State-Foothill freeway interchange.

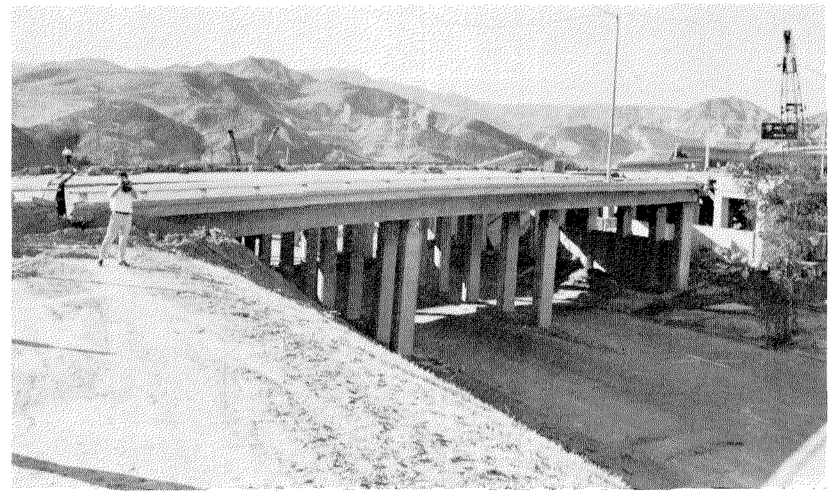


Figure 6.34 San Fernando Road overhead, looking northwest. Golden State-Foothill freeway interchange.

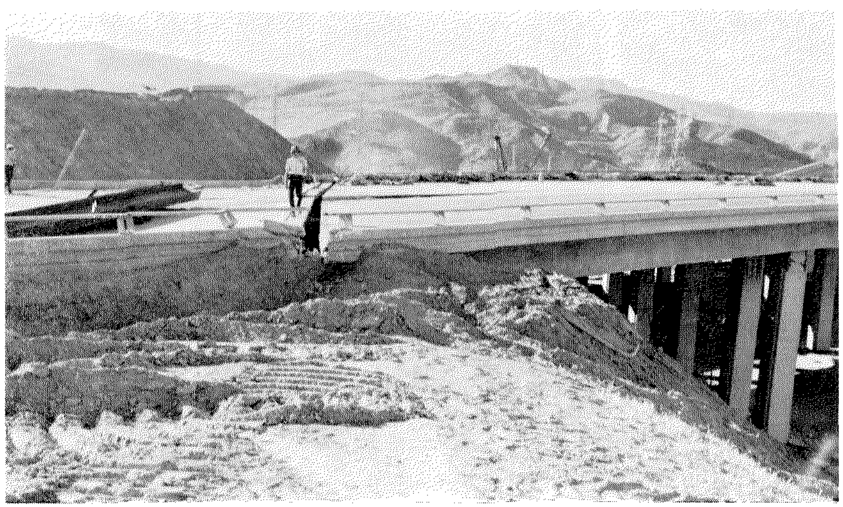


Figure 6.35 Southeast abutment, San Fernando Road overhead, Golden State-Foothill freeway interchange.

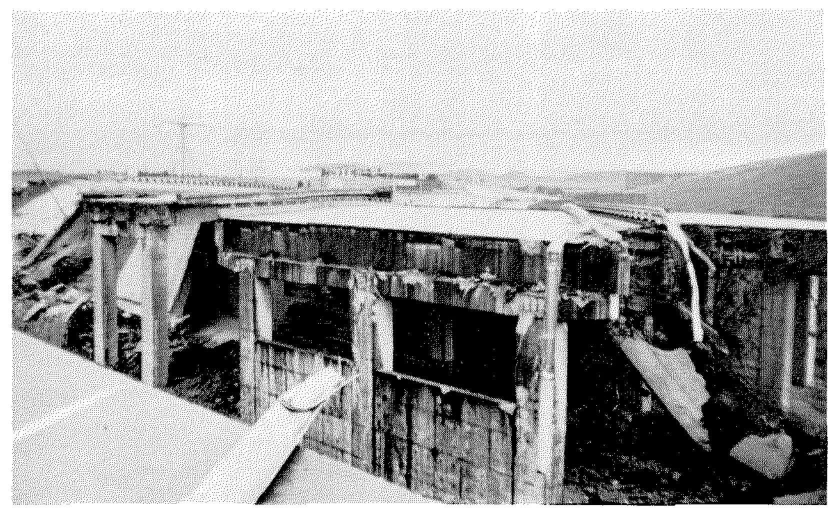


Figure 6.36 San Fernando Road overhead, Golden State-Foothill freeway interchange. The view shows the southwestern seat of the collapsed steel girder bridge section.

and Figure 6.17 (February 9).

Farther southeast along the railroad line the transition road from the northbound Golden State freeway to the eastbound Foothill freeway also crosses both the railroad and San Fernando Road as can be seen in Figure 6.37. The two-span, prestressed concrete box bridge suffered severe damage at the base of the supporting column and was quickly demolished and removed as is evident from Figures 6.18 and 6.19.

At the east end of the fallen Separation and Overhead is a 184 ft long, 3-span, reinforced concrete box bridge, the North Connector undercrossing, which crosses the transition road from the eastbound Foothill freeway to the northbound Golden State freeway (Figure 6.19). This structure, shown in Figure 6.38, showed no serious damage although there was some settling of the abutment backfill.

The interchange between the Golden State and San Diego freeways (Figure 1.7) is shown in its pre-earthquake state, looking west, in Figure 6.39. During the earthquake the Southbound Truck Ramp collapsed on its two supporting columns as is seen in Figure 6.40a. The collapsed bridge was quickly removed to open the San Diego freeway to traffic as is shown in the vertical aerial photo, Figure 6.40b, taken February 12.

Bridges and Overpasses

The newly-constructed Foothill freeway is in the zone of strongest shaking (Figures 1.1, 1.2 and 1.4) and most of the bridges, overpasses and footbridges on this freeway were damaged to some extent. The description which follows begins at the junction of the Foothill and Golden State freeways, taking the bridges in order from that interchange to the last completed structure at Maclay Street (Figure 1.2).

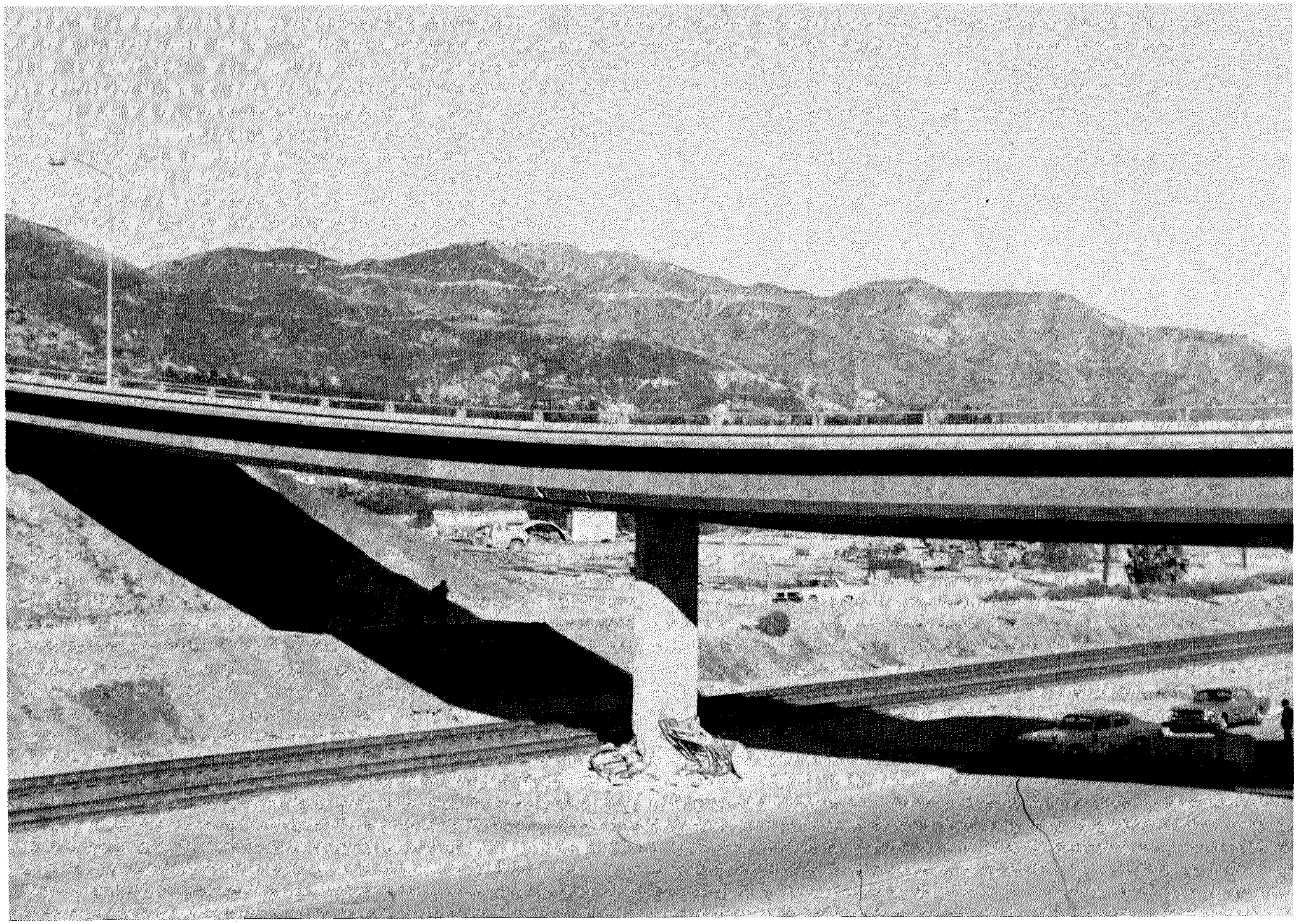


Figure 6.37a San Fernando Road overhead transition bridge showing damage to the base of the single column. The top of the column was reported to be undamaged.

Figure 6. 37b. Aerial view of the San Fernando Road overhead.

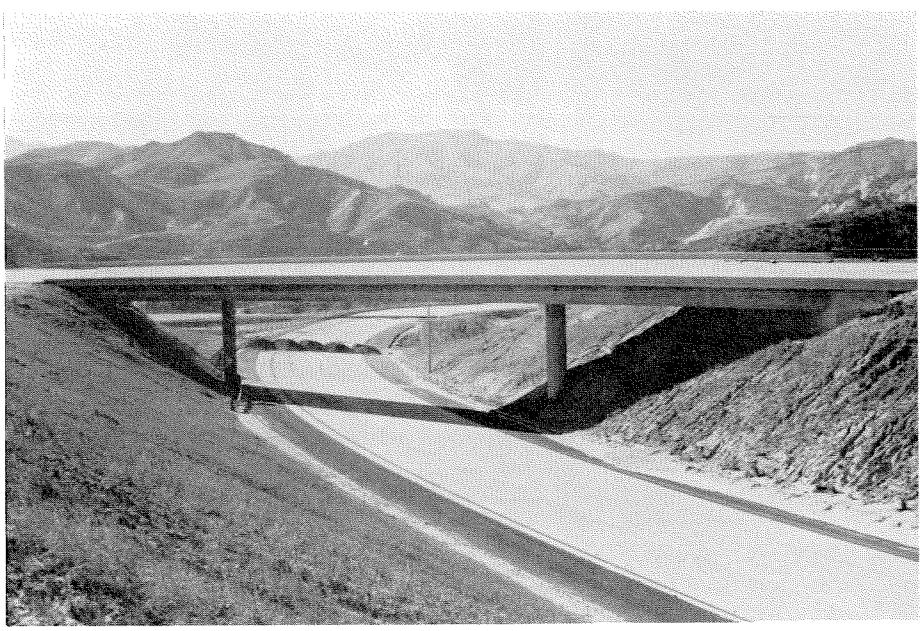


Figure 6. 38 North connector undercrossing, Golden State-Foothill freeway interchange. Bridge suffered only minor damage at abutment backfill.

Figure 6.39 Pre-earthquake view of the Golden State-San Diego freeway interchange and lower Van Norman Lake. North is to the right. Photographed by Ralph Samuels.

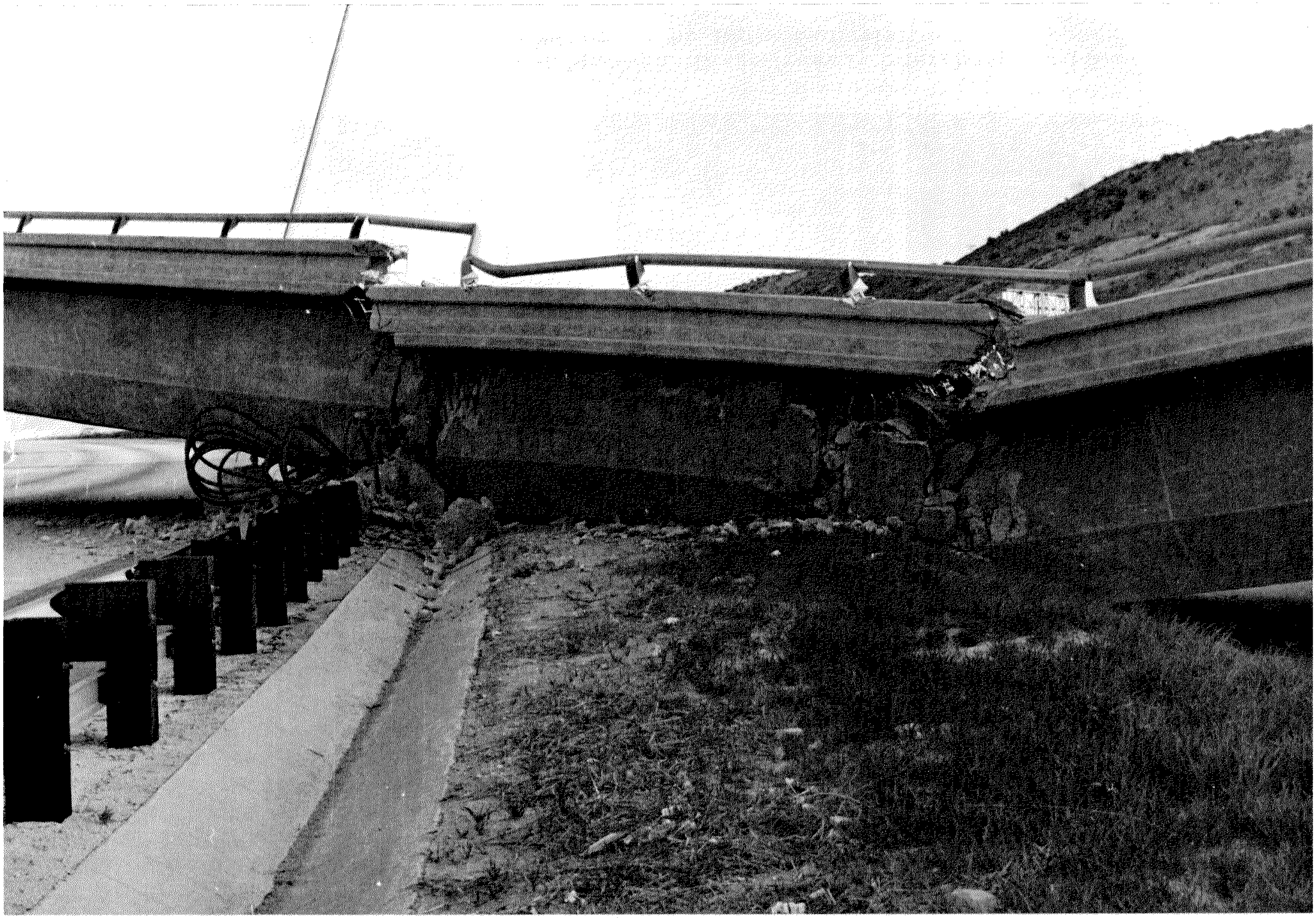


Figure 6. 40a Collapsed Southbound Truck Ramp at the Golden State-San Diego freeway interchange. looking northwest. The reinforcing steel of the central columns is visible in the left center of the photograph. California Division of Highways photograph.

Figure 6.40b Vertical aerial photograph of the Golden State-San Diego freeway interchange, taken February 12, 1971. North is to the right.

The Foothill Boulevard undercrossing of the Foothill freeway is at the eastern edge of the Foothill and Golden State freeway interchange as can be seen in the left background of Figure 6.19. The opposing freeway lanes are on separate reinforced concrete box bridges, both of which were damaged, with the heaviest damage occurring at the southeastern bridge. The general layout of the bridges and some representative dimensions are included in Figure 6.41.

A view looking east underneath the bridges is given in Figure 6.42. The damage to the central columns in the southeastern bridge, to the concrete median surrounding the central column bases in the northwestern bridge and to the tops of the columns on the left abutment of the northwestern bridge can be seen in this figure. The failure of three of the four central columns of the southeastern bridge is seen more clearly in Figure 6.43 which was taken looking west. The column nearest the camera is the shortest of the four (see Figure 6.41). Closer views of the four columns in the order in which they appear in Figure 6.43 are given in Figure 6.44. Figure 6.45 shows details of the column construction, including the 2-1/4 in diameter reinforcing steel and the 1/2-in diameter ties, spaced at 12 inches.

On the northwestern bridge, the stiffening action of the retaining walls appeared to have forced a failure into the top of the columns as seen in Figures 6.46. The abutments also were damaged as is clear from Figures 6.47 and 6.48; the latter figure shows the southeastern bridge permanently offset in the direction of increasing skewness by about three inches.

The failure of the bridges is obviously complex and explanations are not offered here. It is clear, however, that the extent of the failures in the central columns was aggravated by the inadequate ties. Only one of the lapped ties on Column 1 (Figure 6.44) was found to have yielded and

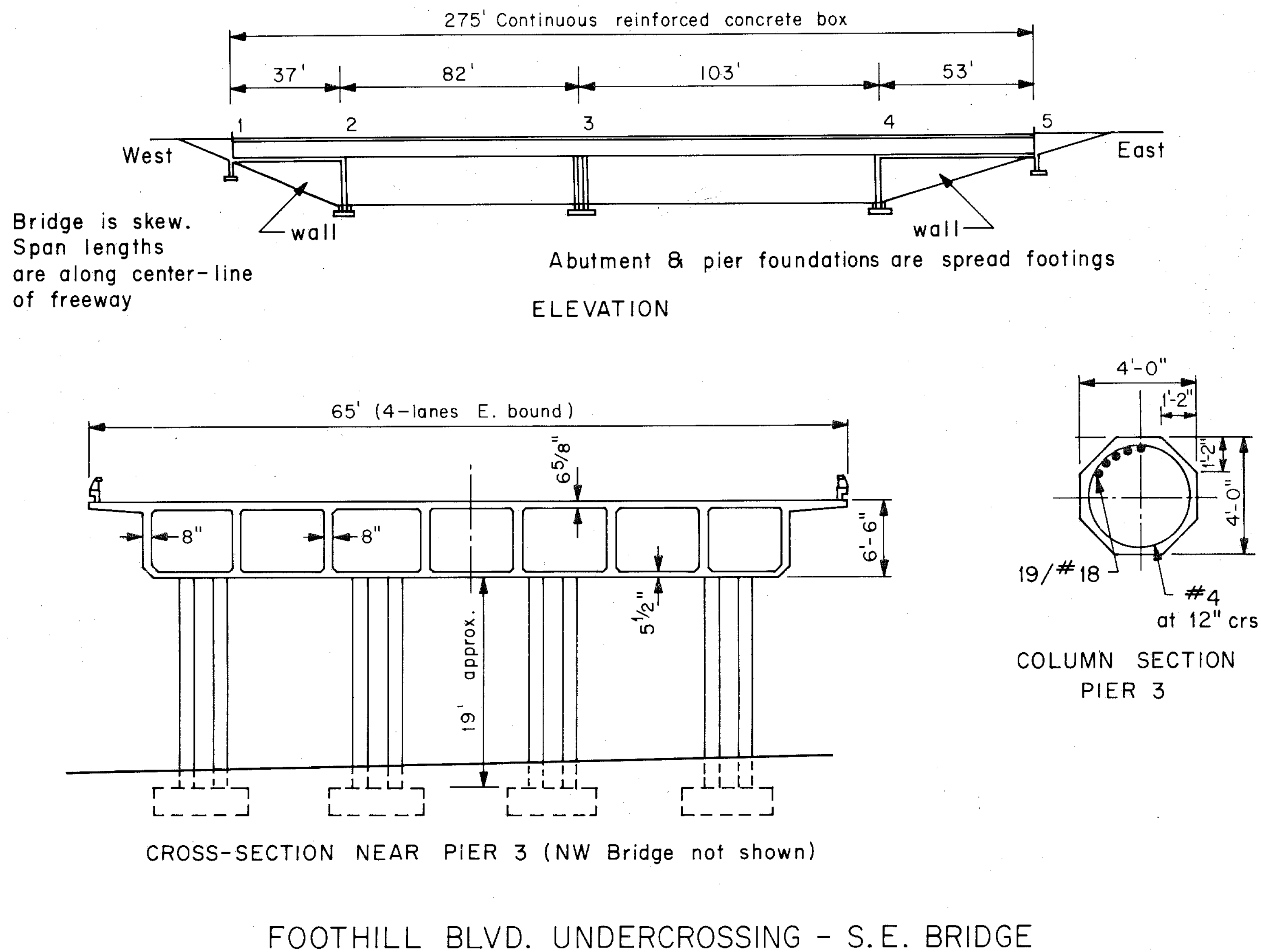


Figure 6.41 Construction details of the Foothill Boulevard undercrossing, Foothill freeway.

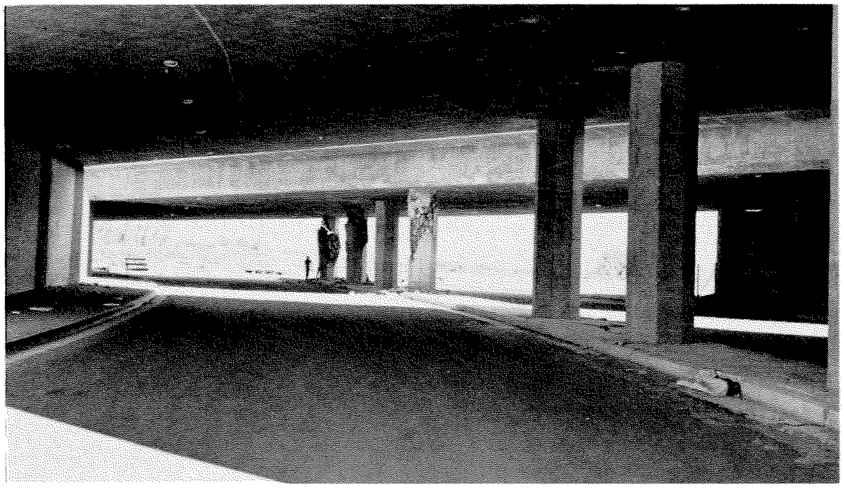


Figure 6.42 Looking east under the Foothill Boulevard undercrossing, Foothill freeway.

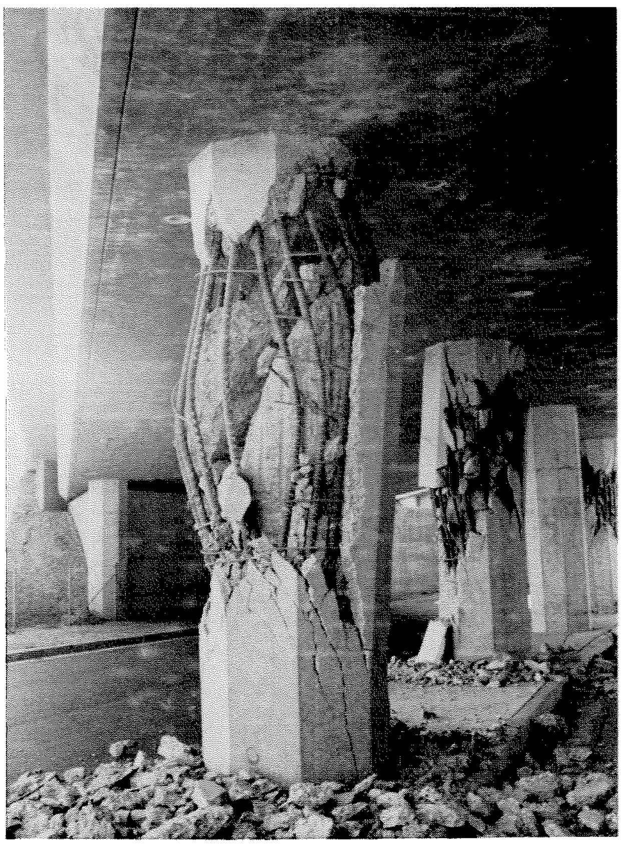
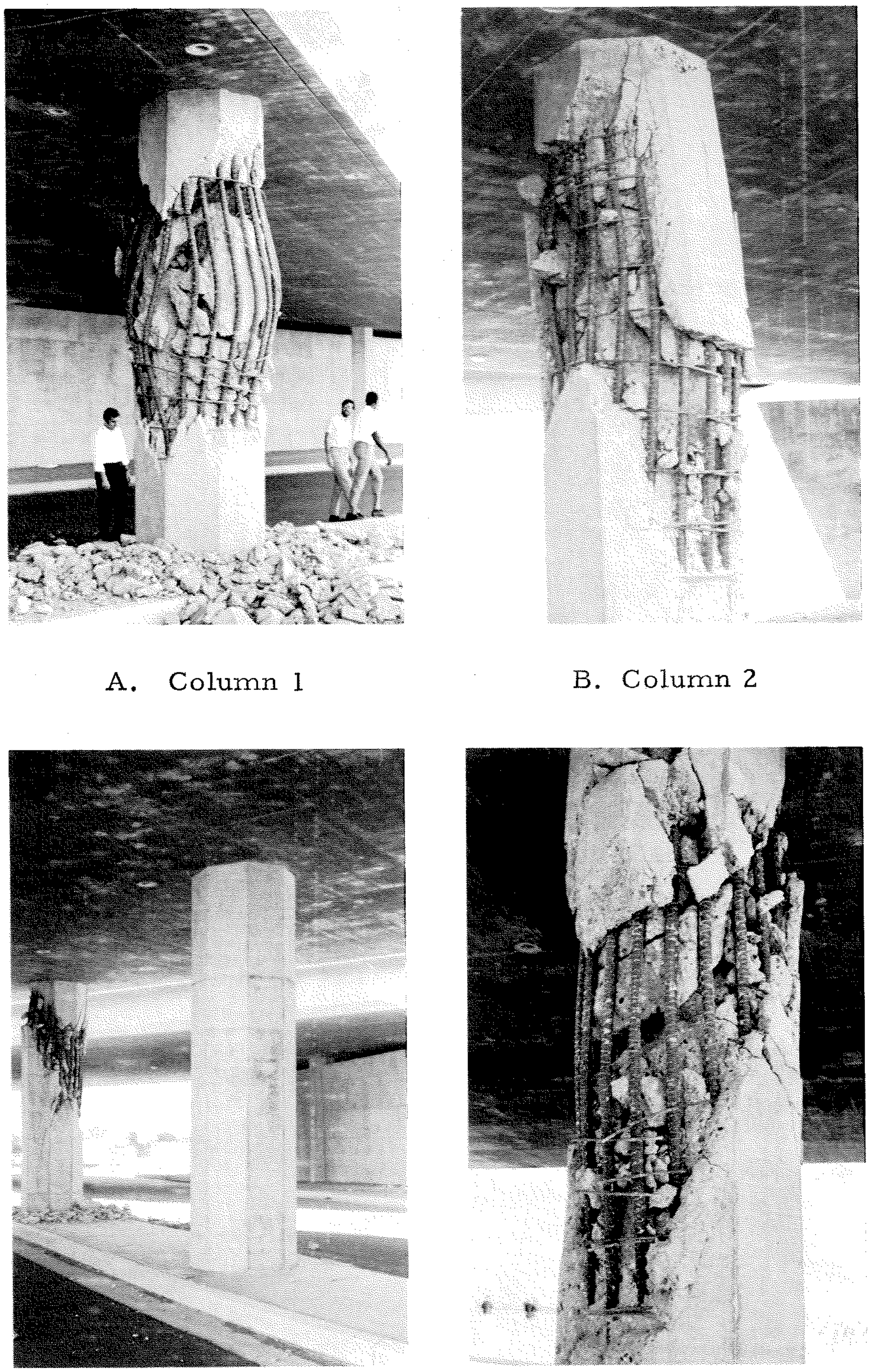


Figure 6.43 Failed columns of the southeastern bridge, Foothill Boulevard undercrossing, Foothill freeway.



A. Column 1

B. Column 2

C. Column 3

D. Column 4

Figure 6.44 Column damage at the southeastern bridge, Foothill Boulevard undercrossing, Foothill freeway.

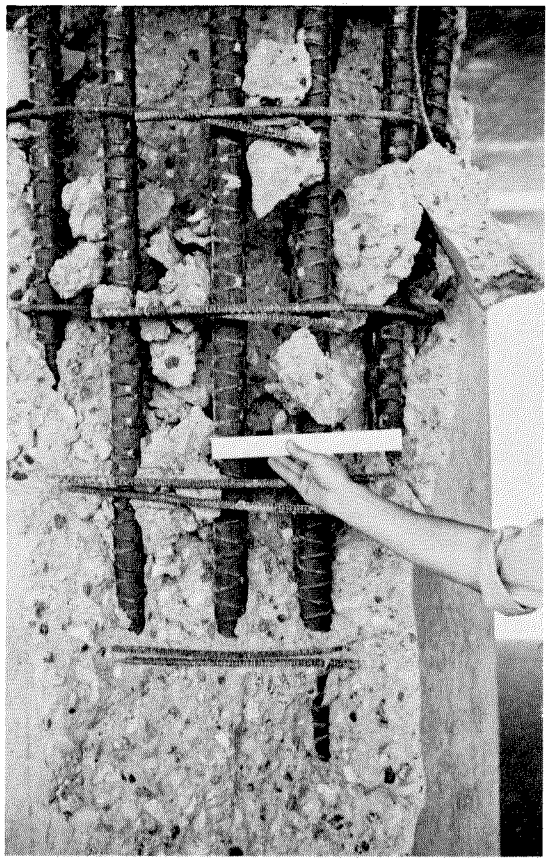


Figure 6.45 Details of column construction, Foothill Boulevard undercrossing, Foothill freeway.

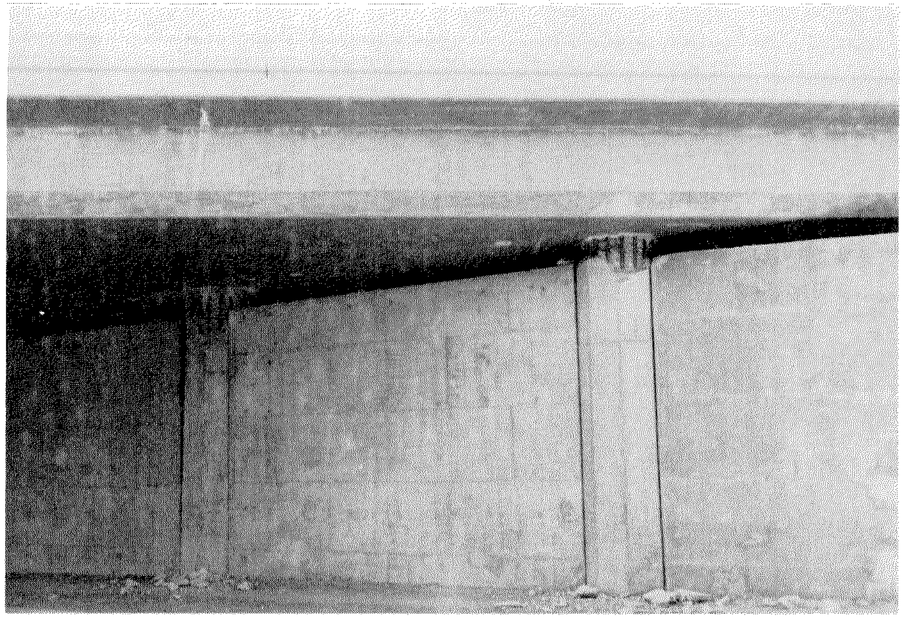


Figure 6.46 Column damage at tops of retaining walls, Foothill Boulevard undercrossing, Foothill freeway.

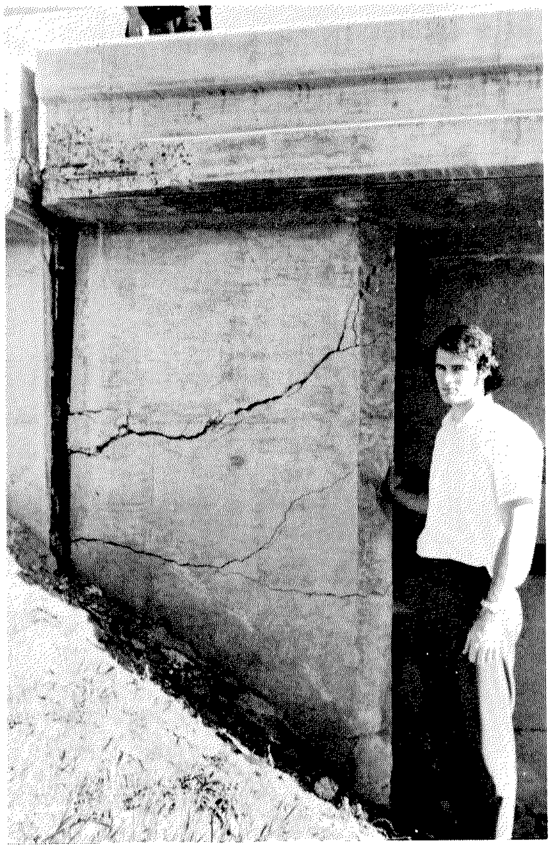


Figure 6. 47 Northernmost abutment of the northwestern bridge, Foothill Boulevard undercrossing, Foothill freeway.

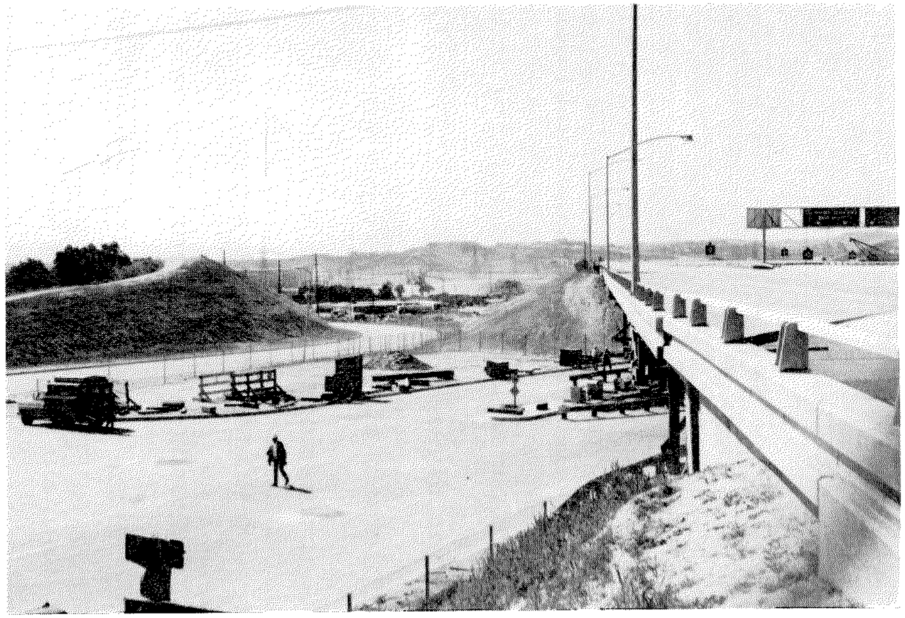


Figure 6. 48 Looking southwest along the Foothill freeway at the Foothill Boulevard undercrossing. Note bridge offset.

fractured. The other ties became ineffective when the concrete shell outside the reinforcing cage cracked and spalled.

The Yarnell Street undercrossing (Figures 6.49 and 6.50) and the Glenoaks Boulevard undercrossing (Figures 6.51 and 6.52) are each twin, single-span bridges and are of similar, prestressed concrete box design. They are 148 ft and 132 ft long, respectively. There was no significant damage to the bridge structures but the concrete aprons fractured and settled near the abutments as is seen in Figures 6.50 and 6.52. There was approximately two inches of settlement of the freeway pavement at the abutments of the bridges at both undercrossings.

The Roxford Street undercrossing (Figure 6.53) is similar to the Yarnell and Glenoaks crossings but the bridges are longer, spanning 151 ft. The bridges suffered serious damage to the retaining walls, piling, concrete aprons and adjacent pavement, but the main prestressed box girders of the bridge were not damaged. The southern and northern sides of the eastern abutment of the southern bridge are shown in Figures 6.54a and 6.54b respectively. These figures show the retaining wall failure resulting from the permanent movement of the bridge, about 2-1/2 ft at this abutment. The adjacent abutment of the northern bridge moved in the same direction about nine inches. A view of the retaining wall from the direction opposite Figure 6.54b is shown in Figure 6.55. The piles under this abutment appeared to have failed in bending at the pile caps. The opposite abutments of the bridges are illustrated by Figure 6.56, in which it is seen that the apron on the southern bridge has been removed as part of the work to expose the piles. The abutment of the southern bridge (Figure 6.56) moved about two inches to the south; the western abutment of the northern bridge has not moved significantly. The bridges, initially skew by 12°, have been made

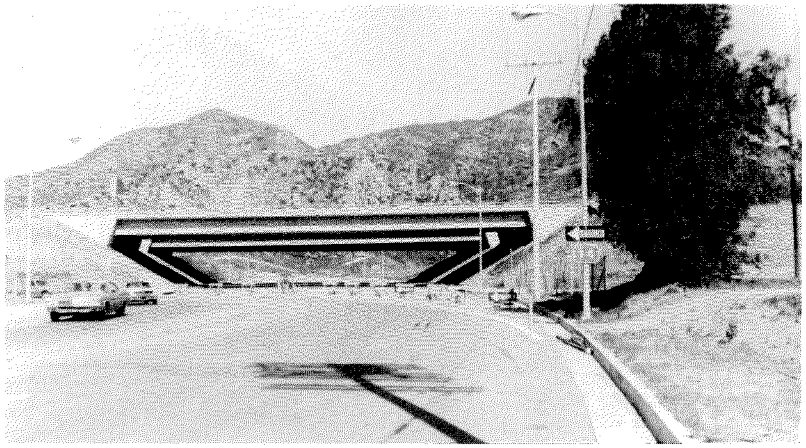


Figure 6.49 Yarnell Street undercrossing, Foothill freeway.

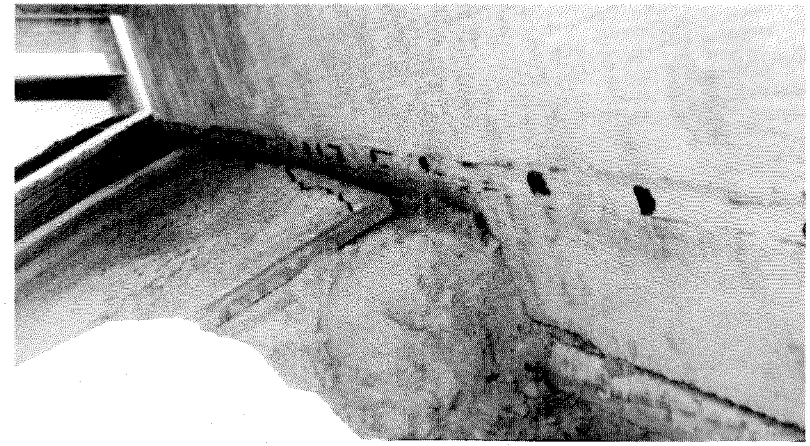


Figure 6.50 Movement of concrete apron, Yarnell Street undercrossing, Foothill freeway.

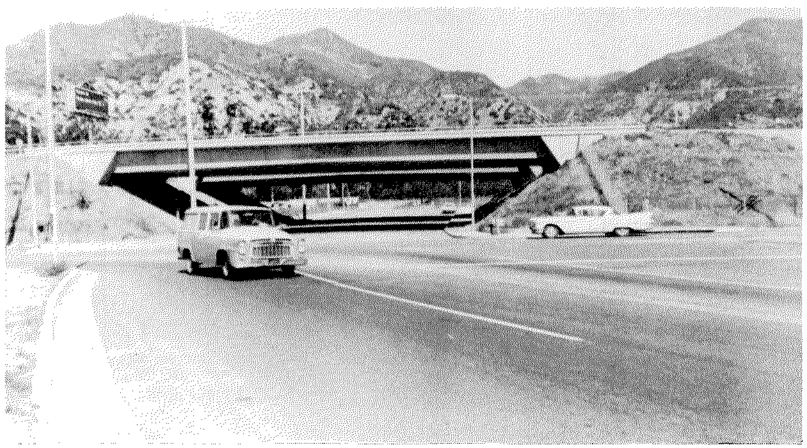


Figure 6.51 Glenoaks Boulevard undercrossing, Foothill freeway.

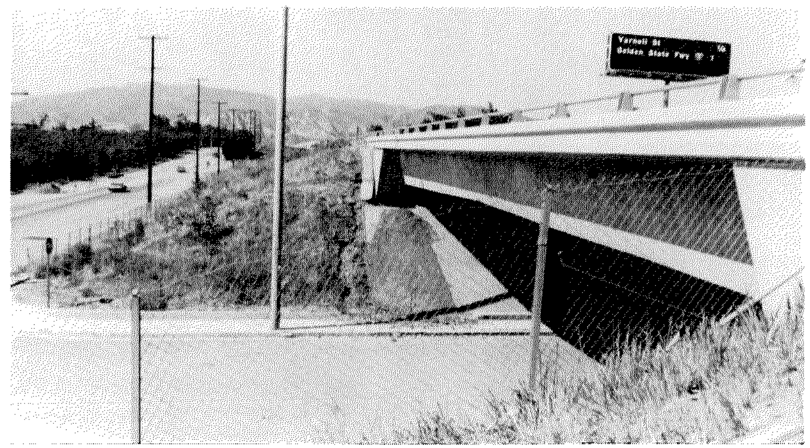


Figure 6.52 The concrete apron moved approximately 9 in at the Glenoaks Boulevard undercrossing, Foothill freeway.

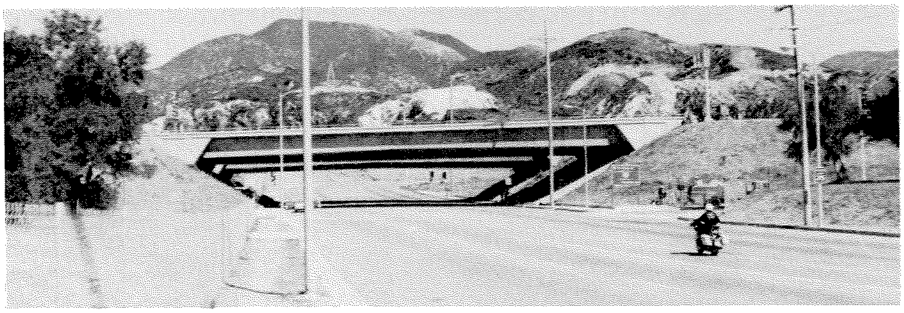


Figure 6.53 Roxford Street undercrossing, Foothill freeway.

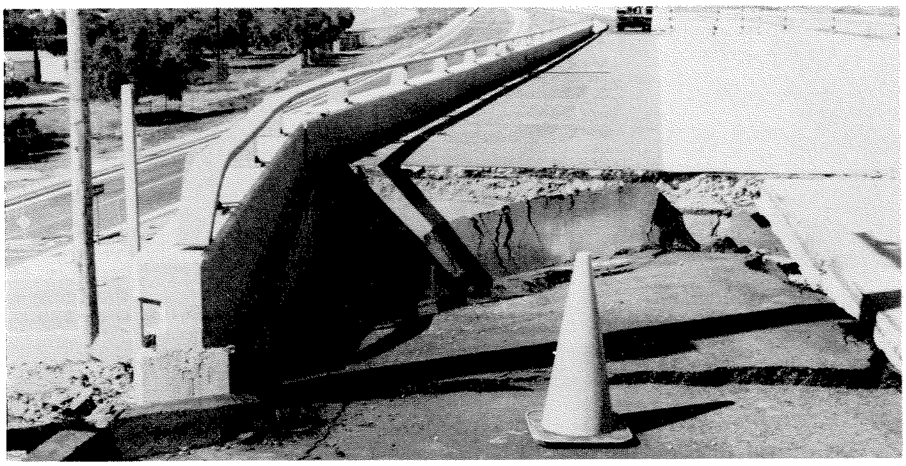


Figure 6.54a Eastern abutment of the southern bridge of the Roxford Street undercrossing, Foothill freeway.

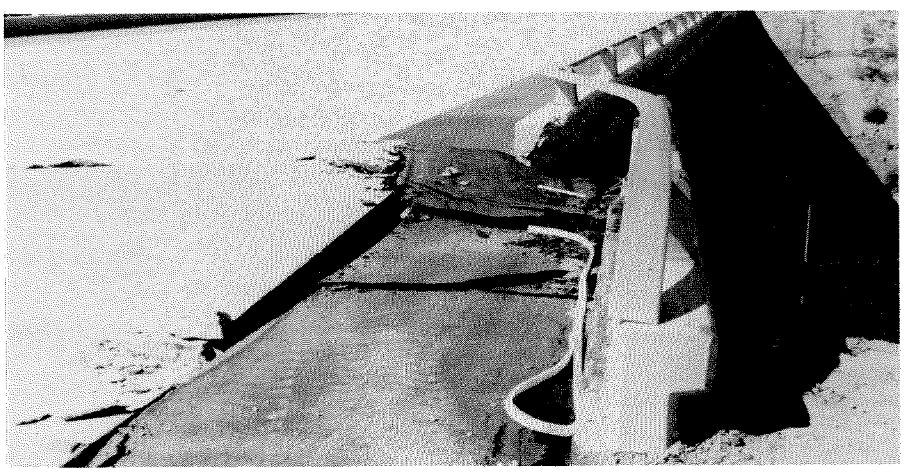


Figure 6.54b Eastern abutment of the southern bridge of the Roxford Street undercrossing, Foothill freeway.

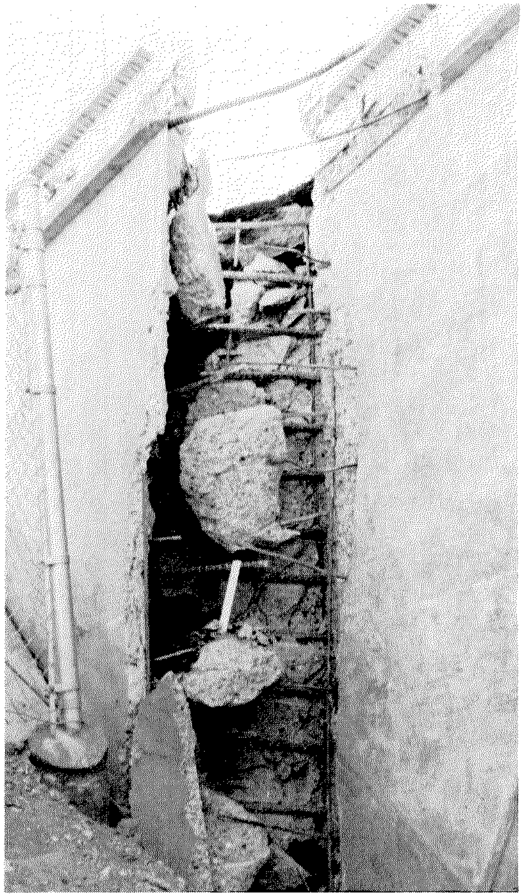


Figure 6.55 Eastern abutment of the southern bridge of the Roxford Street undercrossing, Foothill freeway.

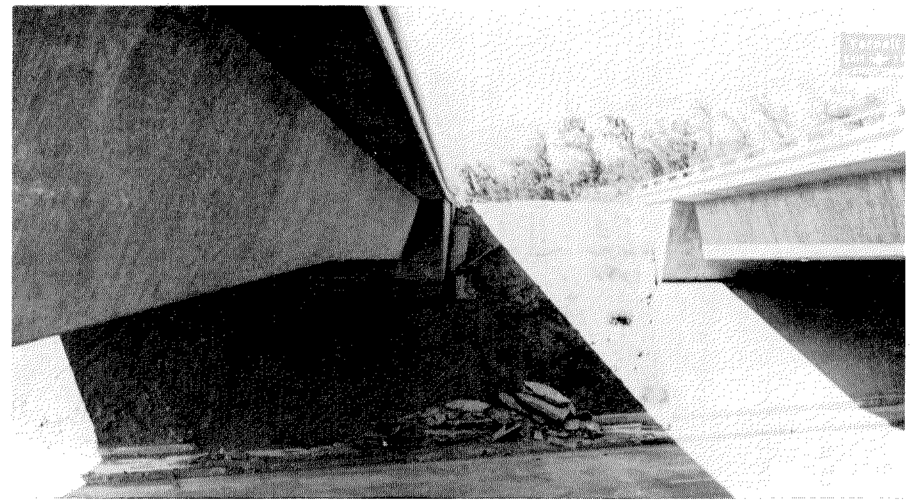


Figure 6.56 Western abutments of the Roxford Street undercrossing, Foothill freeway.

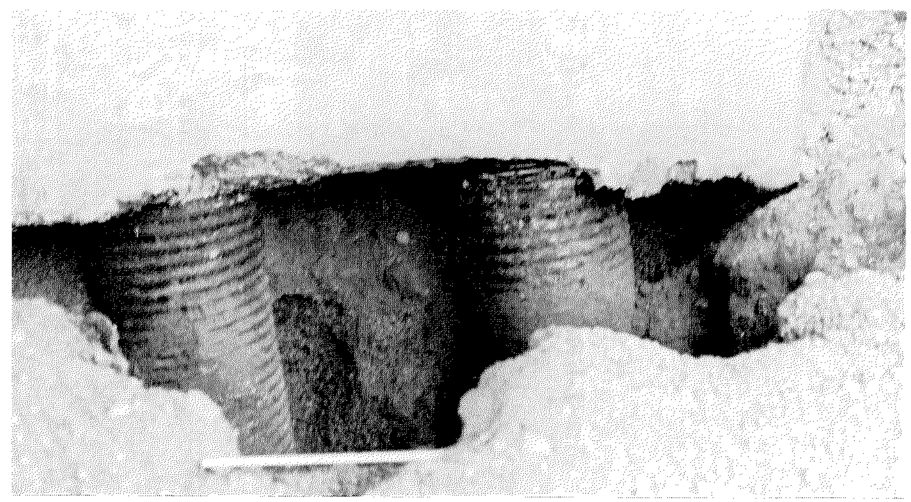


Figure 6.57 Piling damage at the western abutment of the southern bridge, Roxford undercrossing of the Foothill freeway.

slightly more skew by the permanent displacements. Two of the exposed piles of the west abutment of the southern bridge are seen in Figure 6.57. The piles, which are concrete cast in 15-in diameter corrugated steel pipe (Class I) show evidence of hinging at the top and have sheared through the soil.

The Bledsoe Street overpass (Figure 6.58) is a four-lane, two-span, reinforced concrete structure with twin central columns. The two spans are 101 ft and 107 ft, with the northern span being the longer. The shaking caused the abutments of the bridge to settle approximately two inches with respect to the pavement at either end as shown in Figure 6.59 and 6.60. This settlement is believed to be the cause of the cracking observed in the deck structure over the columns. The cracks reached 1/32-in width in the sidewalk. The abutments have spread-footing foundations and are in original ground. The bridge, which is about 1/4 mile from the Olive View Hospital (Figure 1.2) also suffered vibrational damage as is evident at the top of the two columns, shown in Figures 6.61 and 6.62. The major motion of the bridge implied by the damage is longitudinal — the more flexible direction of the central columns.

The Tyler Street footbridge shown in Figures 6.63 and 6.64 was damaged heavily by the earthquake. The primary motion indicated by the damage was longitudinal, in the more flexible direction of most of the columns. All of these columns, including the tallest and presumably most flexible (Figure 6.65) showed spalling at the top, indicating overstress in bending. An idea of the extent of longitudinal motion can be seen in the damage to the northeast abutment shown in Figures 6.66 and 6.67. The bridge seating was not tied to the abutment structure and the sidewalk damage in Figure 6.67 was caused by pounding at the abutment joint.

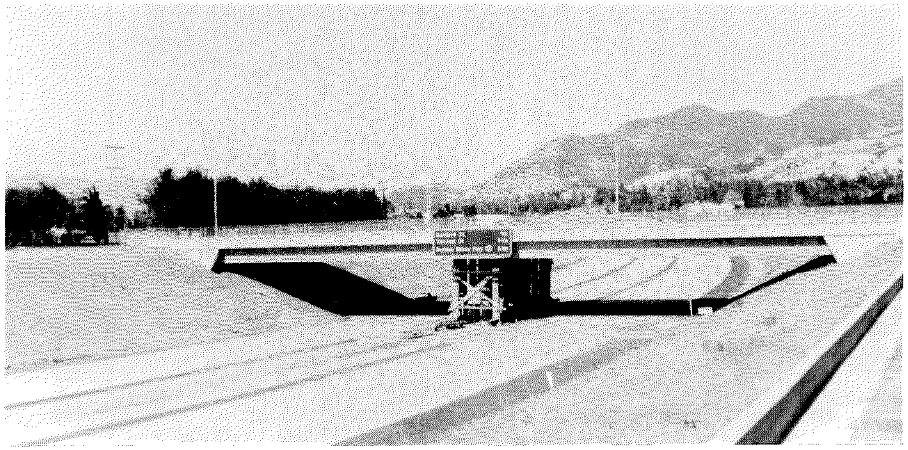


Figure 6. 58 Bledsoe Street overpass, Foothill freeway.

Figure 6. 59 Settlement of southwestern abutment of the Bledsoe Street overpass, Foothill freeway.

Figure 6. 60 Closeup view of abutment settlement shown in Figure 6. 59, Foothill freeway.

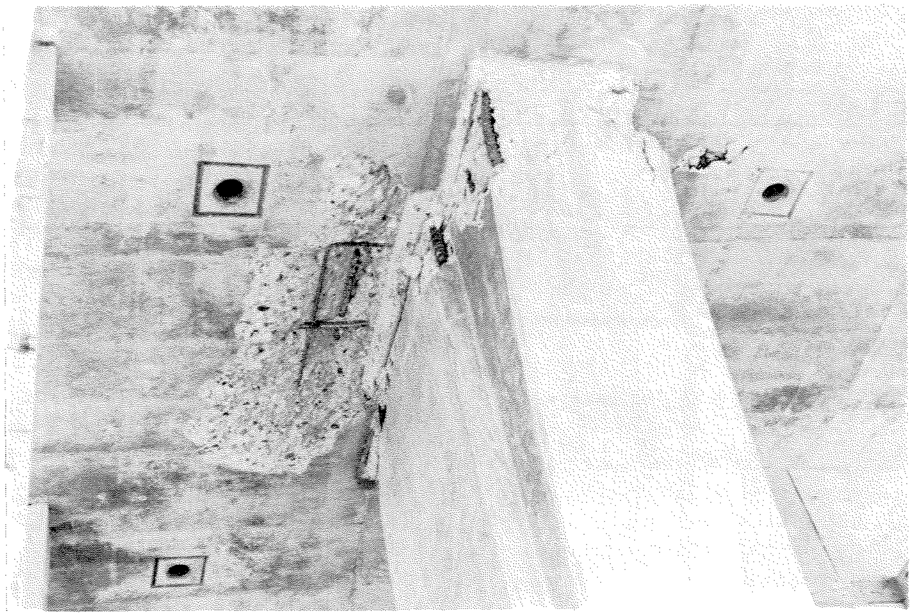


Figure 6.61 Column damage at the Bledsoe Street overpass, Foothill freeway.

Figure 6.62 Column damage at the Bledsoe Street overpass, Foothill freeway.

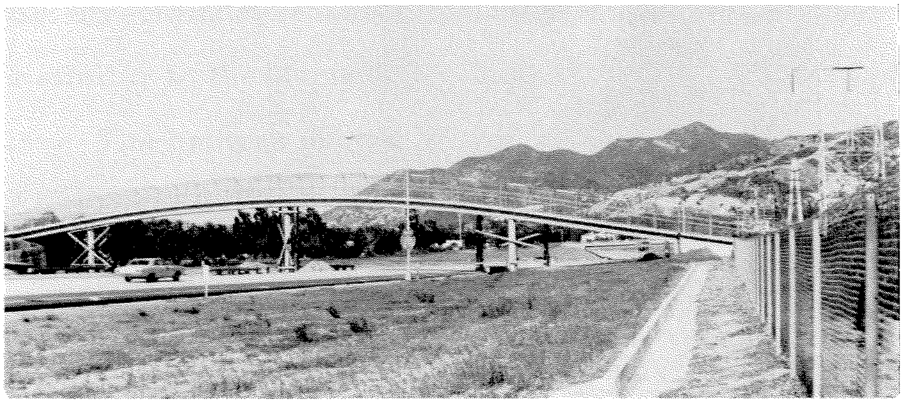


Figure 6.63 Tyler Street footbridge, Foothill freeway.

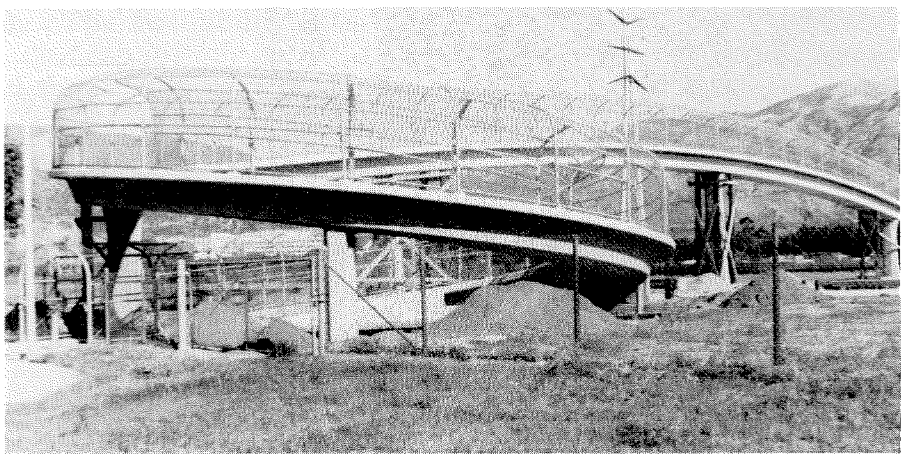


Figure 6.64 Tyler Street footbridge, Foothill freeway.

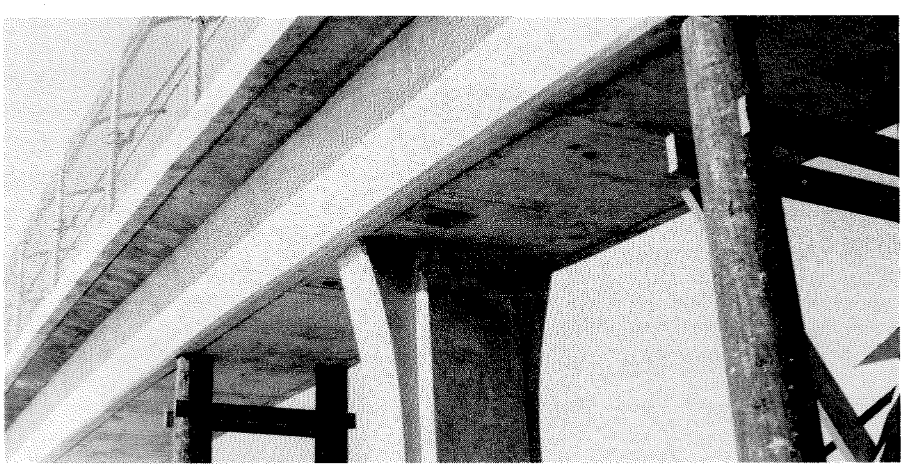


Figure 6.65 Damage to the tallest column of the Tyler Street footbridge, Foothill freeway.

(5) A number of freeway overpasses and bridges collapsed causing two deaths and major disruptions of traffic. In a great earthquake many deaths and injuries could result from overpass and bridge collapses, and the interruptions of transportation could greatly magnify the disastrous effects of the earthquake. It is obviously important that freeway and highway bridges be designed for adequate safety against collapse, though economic considerations would dictate that some damage is acceptable. Present standard requirements for earthquake design of highway bridges are inadequate and should be revised in conformity with the current state of knowledge in earthquake engineering. Furthermore, important existing structures that could collapse in the event of similar earthquake motions should be identified and strengthened so that collapse will not occur.

(6) Many buildings in the San Fernando Valley, and some in Los Angeles received costly architectural damage even though experiencing no structural damage. Plaster cracking was a common occurrence, light fixtures and ceilings fell, air conditioning equipment on isolation mounts was disturbed, equipment that was not bolted to the floor moved about, elevator weights and cables became entangled, bookcases and partitions fell over, and furniture moved about and tumbled. Much of this damage was the consequence of oversight and could have been avoided by simple and inexpensive means. In the region of strong shaking, architectural damage was often severe. For example, even if the Olive View Hospital building had survived without structural damage, it would not have been functional because of extensive architectural damage and disruption of equipment. The possibility of architectural damage should be recognized by the architect and engineer when a building is designed and appropriate preventative measures should

The Polk Street undercrossing consists of twin, 146 ft single-span bridges similar to the Yarnell, Glenoaks and Roxford bridges discussed earlier. The bridges showed minor damage, including downslope movement of the concrete apron and about a six-inch settlement of the abutment backfill. There was no damage to the abutment and no obvious damage to the 15-in diameter piles, but the piles did show some signs of lateral movement.

The Astoria Street footbridge closely resembles the structure at Tyler Street (Figures 6.63 and 6.64) and suffered similar, but less extensive damage. The two abutments (Figures 6.68 and 6.69) were heavily damaged and there was minor spalling and cracking at the tops of the shorter columns.

The overpass at Sayre Street (Figure 6.70) is of construction similar to the Bledsoe Street bridge, but is narrower, carrying only two lanes of traffic. The columns and abutments have spread-footing foundations on original ground. There was evidence of longitudinal movement of the bridge and the backfilling behind the abutment has settled approximately three inches with respect to the bridge (Figure 6.71) with a consequent failure of the abutment approach slabs. Fine cracking was observed in the sidewalks over the central pier which probably indicates settlement of the abutments.

The Hubbard Street overcrossing, not illustrated, is similar in construction to the bridges at Sayre and Bledsoe Streets and carries four lanes of traffic. The columns and abutments have spread footings on original ground. Damage to the sidewalk paving and curbs at both abutments indicated longitudinal movement of the bridge. Although there was no significant relative displacement between the bridge approaches and the abutment,

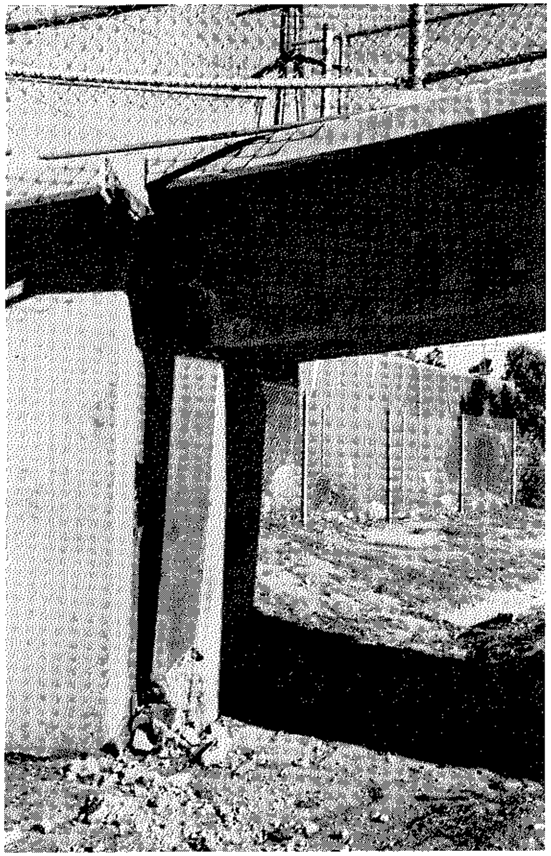


Figure 6.68 Northeastern abutment of the Astoria Street footbridge, Foothill freeway.

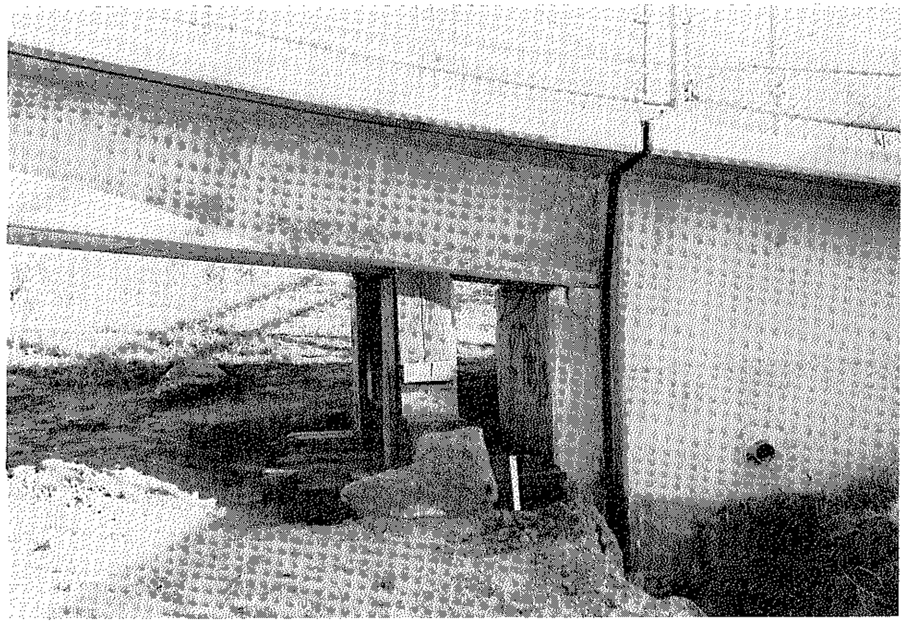


Figure 6.69 Southwestern abutment of the Astoria Street footbridge, Foothill freeway.

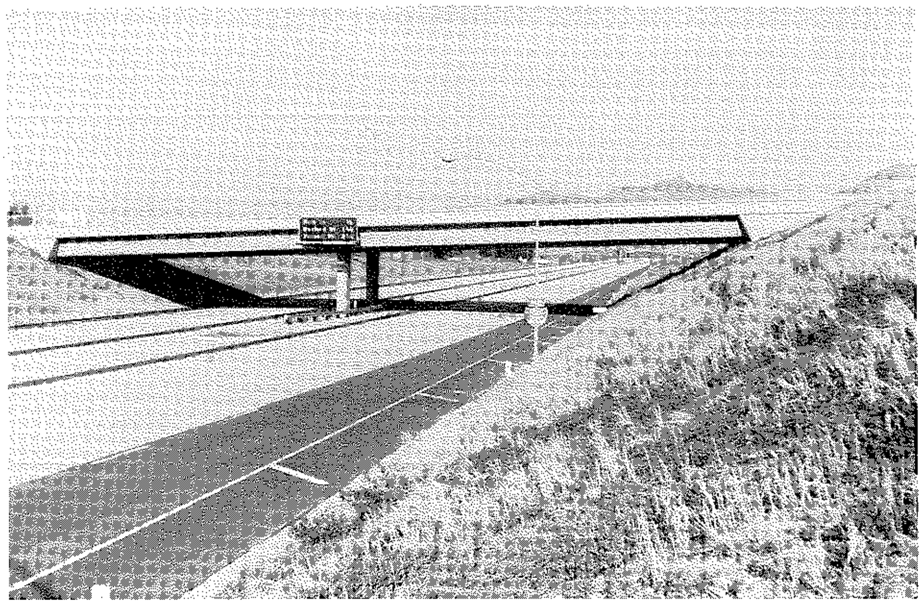


Figure 6.70 Sayre Street overpass, Foothill freeway.

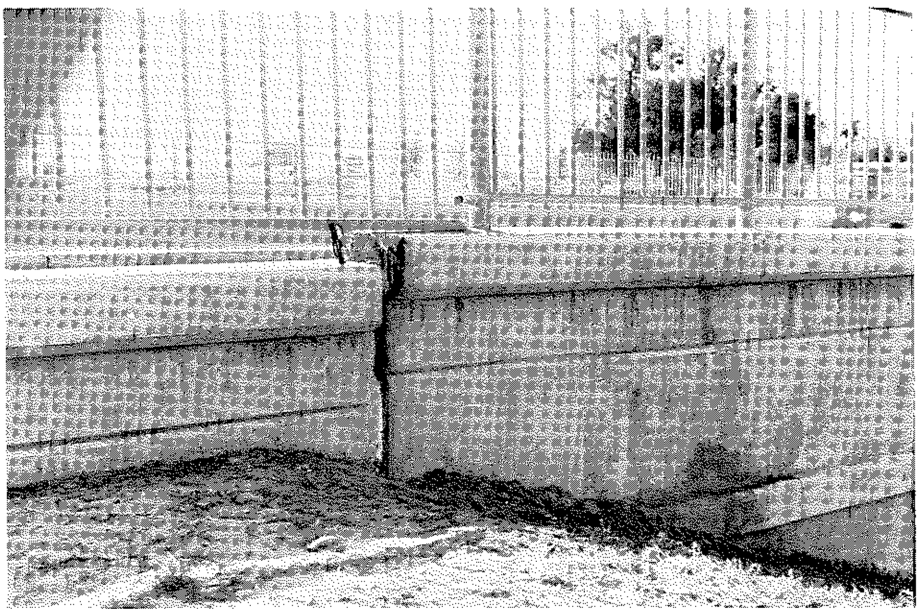


Figure 6.71 Southwest abutment of the Sayre Street overpass, Foothill freeway.

the cracking in the sidewalk over the central pier suggests that the abutments have settled. The crack patterns and widths (about 1/32 in) are similar to those found on the Bledsoe Street overcrossing.

The Harding Street footbridge is about one mile southeast of Hubbard Street. The footbridge is different in design from those discussed above and is shown in Figures 6.72 and 6.73. Damage to the northeast abutment of the bridge is seen in Figure 6.74. The bridge experienced major longitudinal and lateral movements as evidenced by the sidewalk fracturing at the northeast end (Figure 6.75) and the soil movement at the base of the columns (Figure 6.76).

The easternmost completed structure on this section of the freeway is the 145 ft long Maclay Street undercrossing. The bridges are similar in construction to those at Roxford and the other single-span, twin structures described above. The bridges suffered minor damage, including downslope movement of the concrete aprons (Figure 6.77) and about one foot settlement of the abutment backfill (Figure 6.78). There was no obvious damage to the 15-in piles (exposed for inspection) although the bridges showed evidence of movement.

Surface faulting crossed the freeway near Maclay Street and the pavement was fractured at this point, showing permanent vertical and horizontal displacements (Figure 6.79) as well as shortening, as is seen in Figure 6.80. Although spectacular, such pavement damage is relatively minor and is inexpensive to repair.

Bridge and pavement damage occurred also on the Golden State, San Diego and Antelope Valley freeways, as well as at other scattered locations in the epicentral area. These other locations were not examined in as much detail as the Foothill freeway and only examples of earthquake

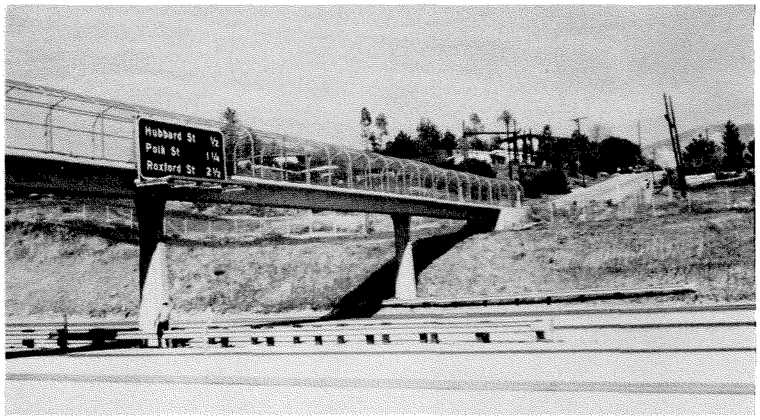


Figure 6.72 Harding Street footbridge, Foothill freeway.

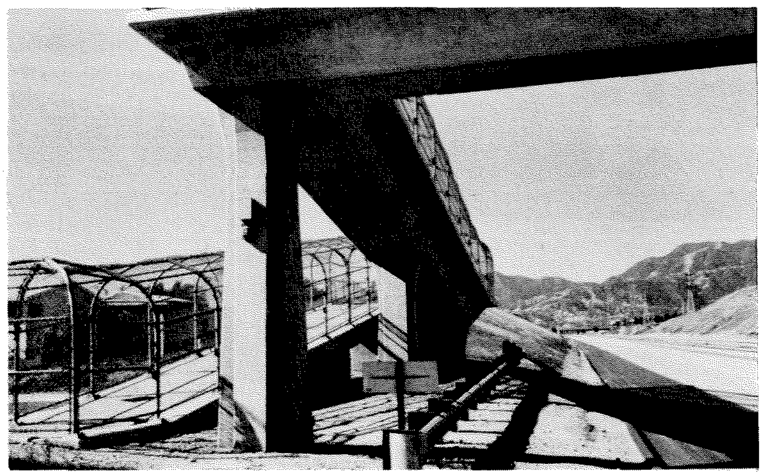


Figure 6.73 Harding Street footbridge, Foothill freeway.

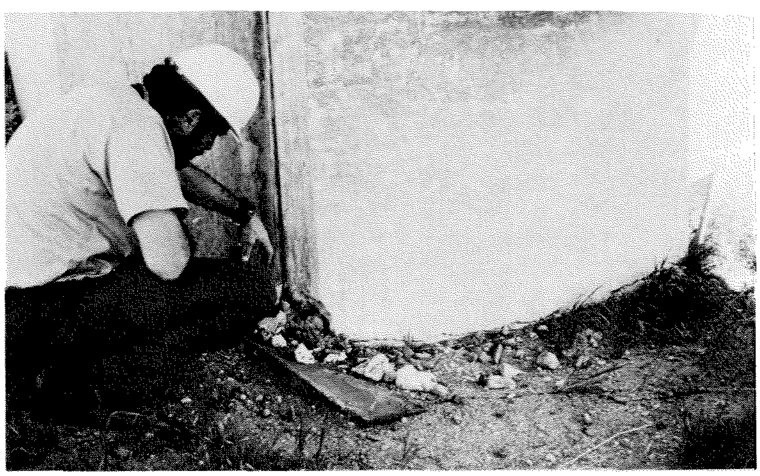


Figure 6.74 Northeast abutment of the Harding Street footbridge, Foothill freeway.

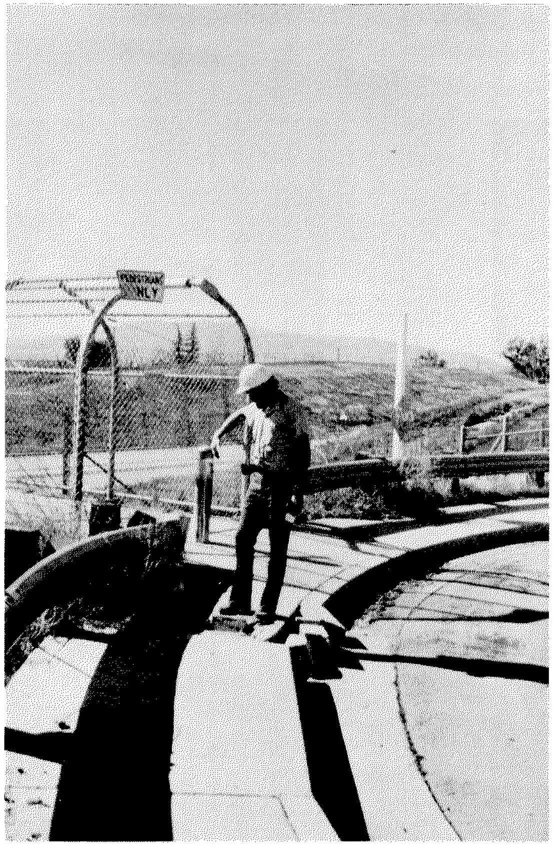


Figure 6.75 Sidewalk damage caused by abutment pounding, Harding Street footbridge, Foothill freeway.

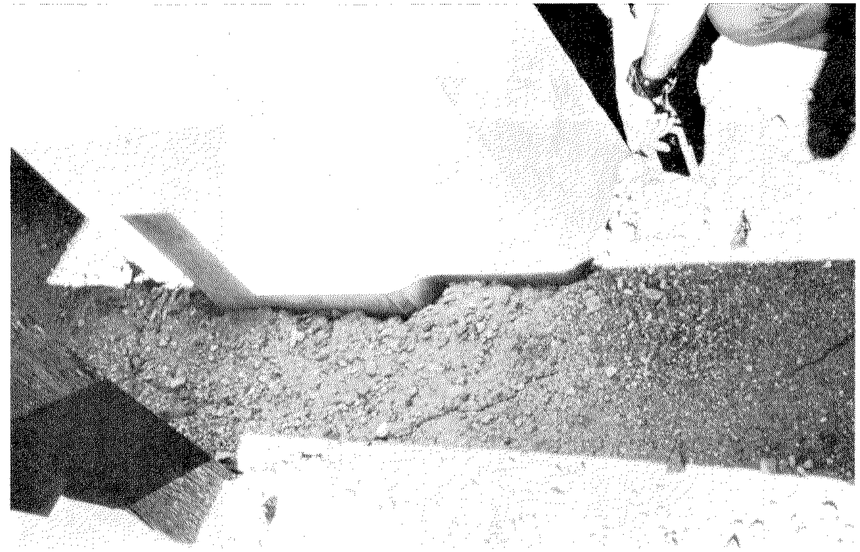


Figure 6.76 Soil movements at the base of central column, Harding Street footbridge, Foothill freeway.

Figure 6.77 Northwestern abutment of the Maclay Street undercrossing, Foothill freeway. Note the movement of the concrete apron and the settlement of the embankment.

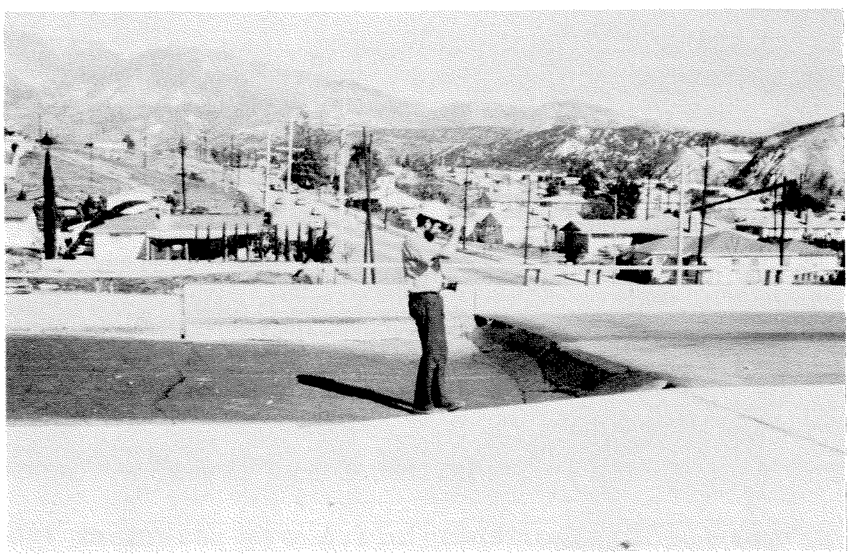


Figure 6.78 Settlement of abutment backfill, Maclay Street undercrossing, Foothill freeway.

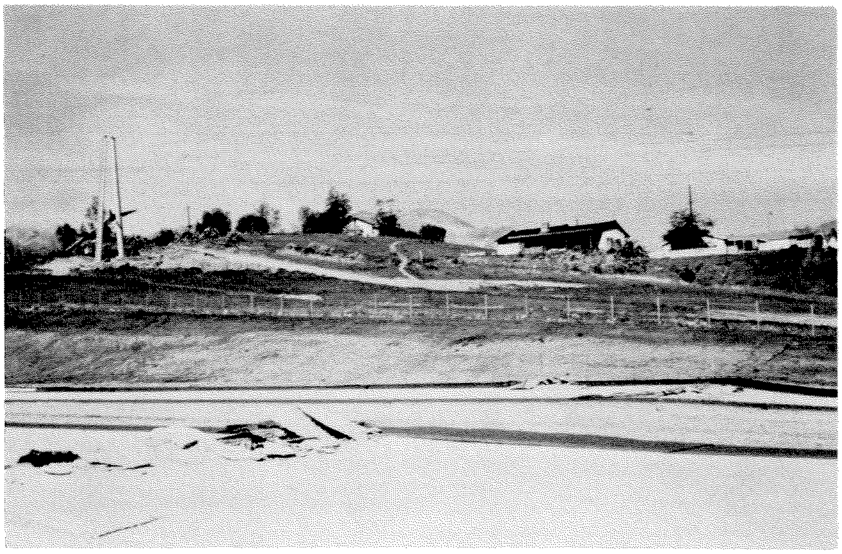


Figure 6.79 Pavement damage from surface faulting. Foothill freeway northwest of Maclay Street.

Figure 6.80 Shortening of freeway pavement caused by faulting. Foothill freeway near Maclay Street.

damage are given in the discussion which follows. In particular there were several damaged bridges on the Golden State freeway which are not discussed.

The San Diego freeway bridges suffered backfill settlement on most structures as far south as Roscoe Boulevard (Figure 1.2) as shown by pavement displacements and repairs using asphaltic concrete. The freeway was open to traffic shortly after the earthquake and it is thought that no major bridge damage occurred.

On the Golden State freeway there was major damage to the Roxford Street undercrossing as is seen in Figures 6.81 and 6.82. Some of the concrete removal prior to repair of the northern abutment is illustrated by Figure 6.83. Also, there was pavement damage from ground movements north of Roxford Street near the upper Van Norman Lake (Figure 6.84).

Farther north, the 674 ft long, seven-span Balboa Street overcrossing (Figures 1.2 and 6.85) suffered damage to the abutments as shown in Figures 6.86 and 6.87. The shoring seen in Figure 6.85 is supporting a damaged hinge joint. In addition, foundation settlements are thought to have occurred under some of the columns.

A portion of the new Golden State freeway north of Balboa was not completed at the time of the earthquake, although the bridge structures were finished. The second bridge north of the Balboa overcrossing, the west Sylmar Overhead, shown in Figure 6.88, suffered major damage. Some of the abutment failures are illustrated by Figures 6.89 and 6.90. Spalling of a joint on the eastern face of the structure is shown in Figure 6.91.

Just south of the interchange with the Antelope Valley freeway, an older skew bridge on the truck lanes of the Golden State freeway, the Sierra Highway undercrossing, experienced abutment damage as is seen

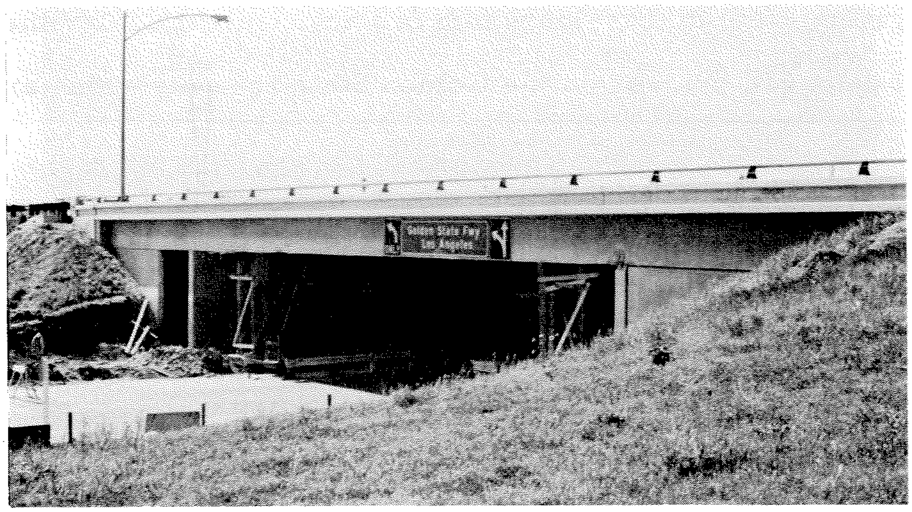


Figure 6.81 Roxford Street undercrossing, looking south, Golden State freeway.

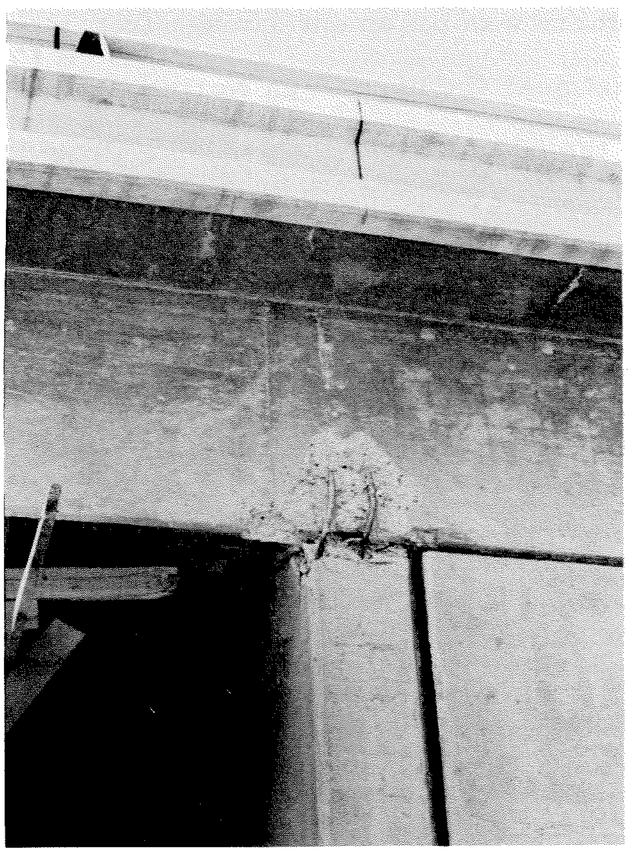


Figure 6.82 Damage to northern pier wall of the Roxford Street undercrossing, Golden State freeway.

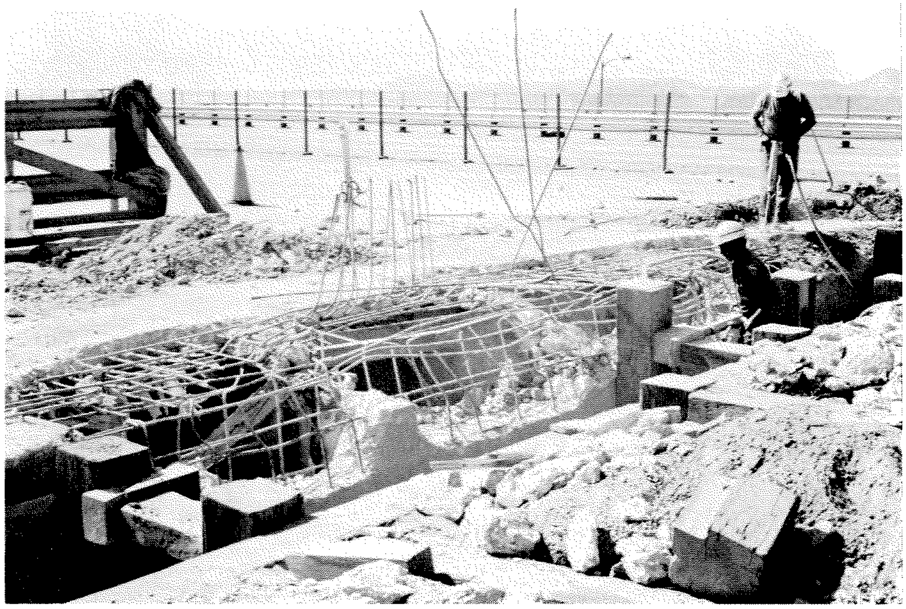


Figure 6.83 Concrete removal prior to repair of northern abutment of northbound lanes of Roxford Street undercrossing, Golden State freeway.

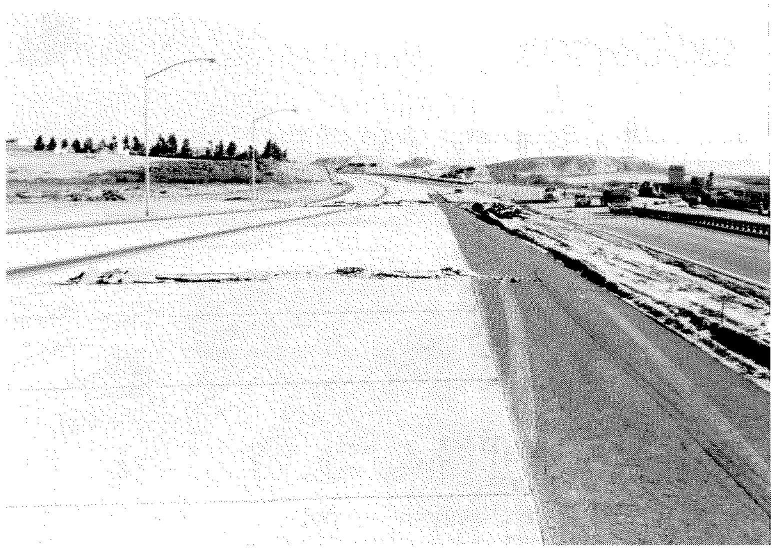


Figure 6.84 Pavement damage from ground movements. Looking south on the Golden State freeway north of the Roxford undercrossing.

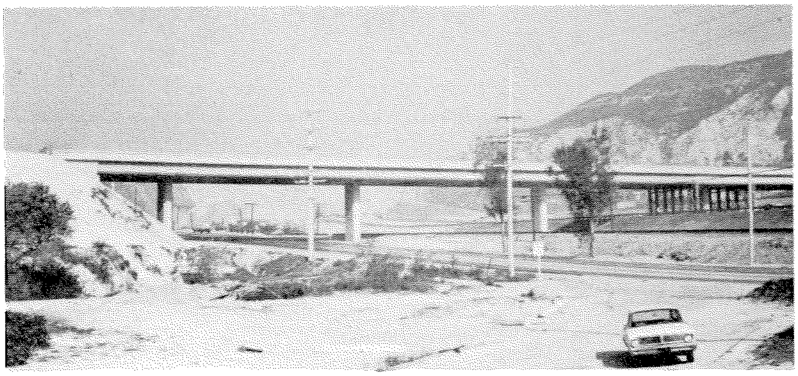


Figure 6.85 Balboa Street overcrossing, Golden State freeway

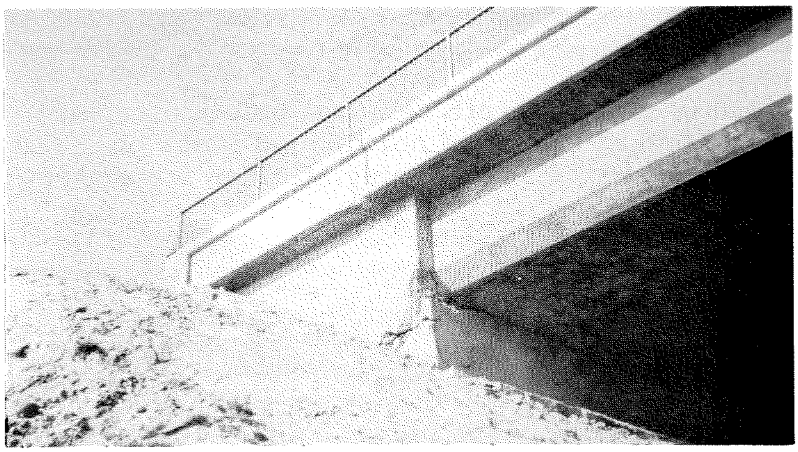


Figure 6.86 Southern face of the western abutment, Balboa Street overcrossing, Golden State freeway.

Figure 6.87 Northern face of the western abutment, Balboa Street overcrossing, Golden State freeway.

Figure 6.88 Looking southwest at the West Sylmar Overhead, north of Balboa Street on the Golden State freeway.

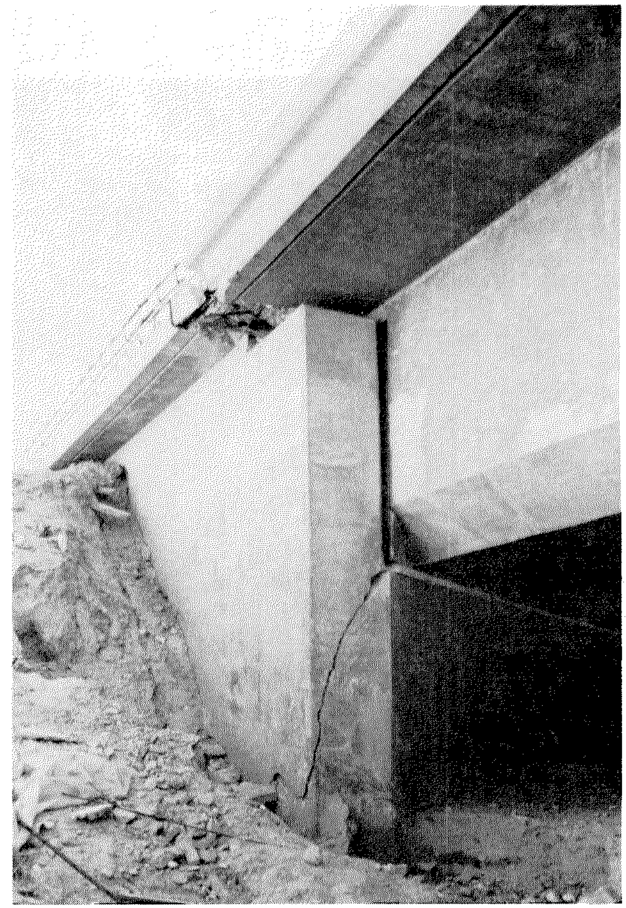


Figure 6.89 Eastern face of the southern abutment of the West Sylmar Overhead, Golden State freeway.

Figure 6.90 Damage to the northern abutment of the West Sylmar Overhead, Golden State freeway.

Figure 6.91 Spalling on the eastern face of the West Sylmar Overhead, Golden State freeway.

looking south in Figure 6.92, and looking north in Figure 6.93.

The first bridge structure on the Golden State freeway north of the Antelope Valley interchange, the Gavin Canyon undercrossing, (not shown), showed minor damage to the hinge joints of the superstructure. Farther north on the Golden State freeway two steel girder bridges suffered minor damage resulting from shearing of the keepers on the girder bearings.

The Antelope Valley freeway (California 14) runs in a northeasterly direction from its interchange with the Golden State freeway (Figure 1.2). Approximately 30 structures on the freeway, comprising 12 overpasses, undercrossings or interchanges lie within ten miles of the epicenter of the main shock. Most of these structures are located north and east of the epicenter, where the ground motion appears to have been less intense than in the Sylmar area. A section of the freeway is incomplete and several structures were under construction at the time of the earthquake.

In general, damage to the structures on the Antelope Valley freeway was relatively minor. Although a large number of structures showed evidence of small movements, only five of the approximately 30 bridge structures suffered significant damage.

Figure 6.94 shows the Sierra Highway undercrossing structures which will carry the new freeway across existing Highway 14, one-quarter mile east of the Golden State-Antelope Valley freeway interchange. These bridges are visible in the right of Figure 6.1. The structures are continuous, reinforced concrete box-girders supported on concrete columns. Damage to the south face of the east abutment is shown in Figure 6.95. The superstructure was cast with the abutment, and the damage shown has resulted from pounding of the abutment against the adjoining retaining wall.

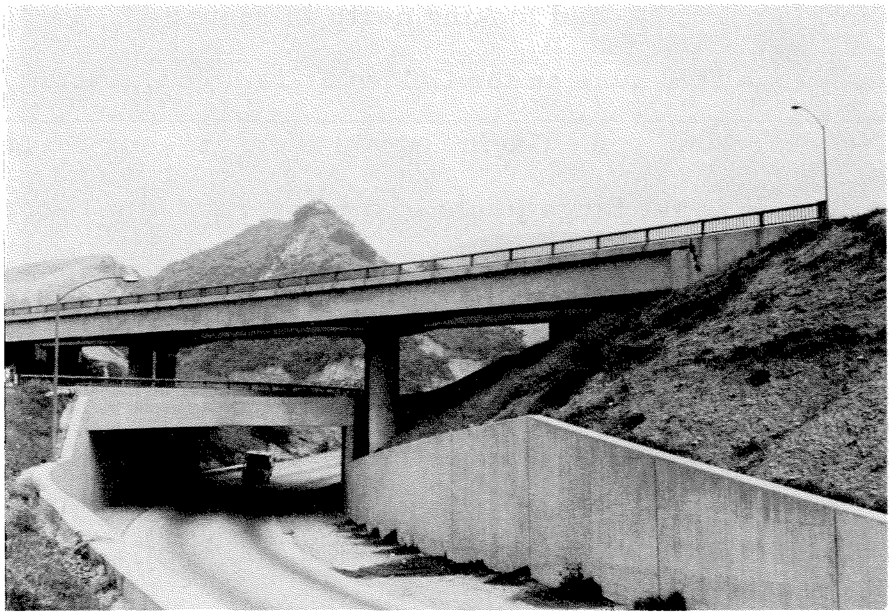


Figure 6.92 The Sierra Highway undercrossing on the Golden State freeway just south of the interchange with the Antelope Valley freeway. View looking south.

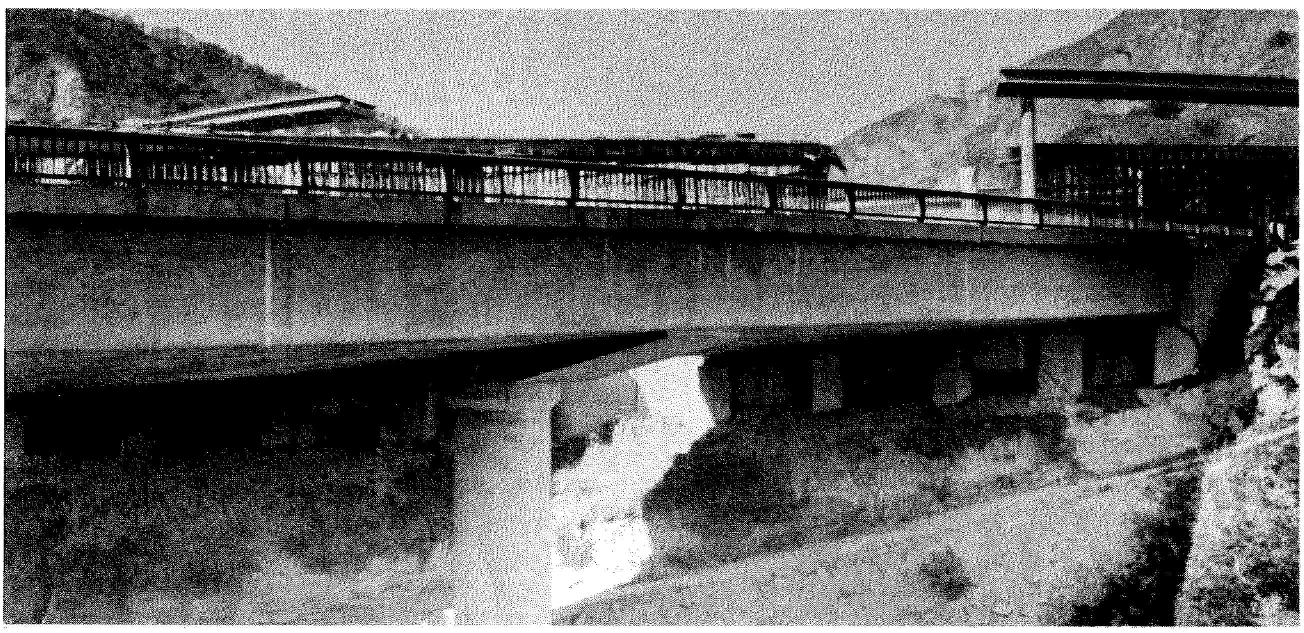


Figure 6.93 The older Sierra Highway undercrossing on the Golden State freeway, looking north. The Antelope Valley interchange is in the background.

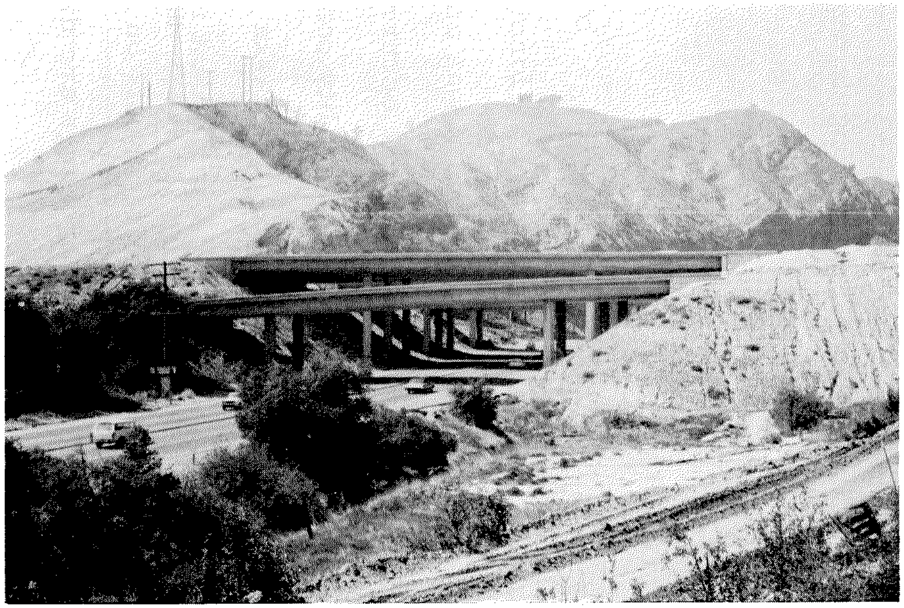


Figure 6.94 Sierra Highway undercrossing on the Antelope Valley freeway near the Golden State - Antelope Valley freeway interchange.

Figure 6.95 Damaged southern face of the eastern abutment of the Sierra Highway undercrossing, Antelope Valley freeway.

The bridges shown in Figure 6.96 comprise the Via Princessa undercrossing, located six miles northeast of the interchange with the Golden State freeway. The 156 ft long, single span, prestressed concrete box-girder bridges were structurally complete at the time of the earthquake, but backfilling of the abutments was not finished. The abutment walls of both bridges have been damaged by longitudinal movements as is seen in Figure 6.97. The failure is thought to be from the combined effects of shear and bending in the wall.

Two, ten-span concrete box bridges which carry the freeway across the Santa Clara River near Solemint (seven miles northeast of the interchange with the Golden State freeway) are shown in Figure 6.98. Both bridges showed evidence of movement and received minor damage from pounding at the joints of the superstructure. Damage to one of the joints, which are about 30° skew, is shown in Figure 6.99. The pounding damage in Figure 6.99 is typical of that observed on many other bridges in the epicentral area.

Discussion and Conclusions

The presentation above had concentrated on earthquake damage, but it is important to realize that, in general, the freeway structures performed very well in the area of moderately strong ground shaking, and it was only in the area of strongest shaking that serious damage and collapse occurred. This feature is thought to be primarily a consequence of good quality construction and inherent lateral strength. Also, in contrast to the Alaska earthquake and recent Japanese earthquakes, there were no significant roadway embankment failures in the freeway system. (The slides in cuts on the Golden State freeway north of the Foothill freeway interchange are discussed in chapter four).

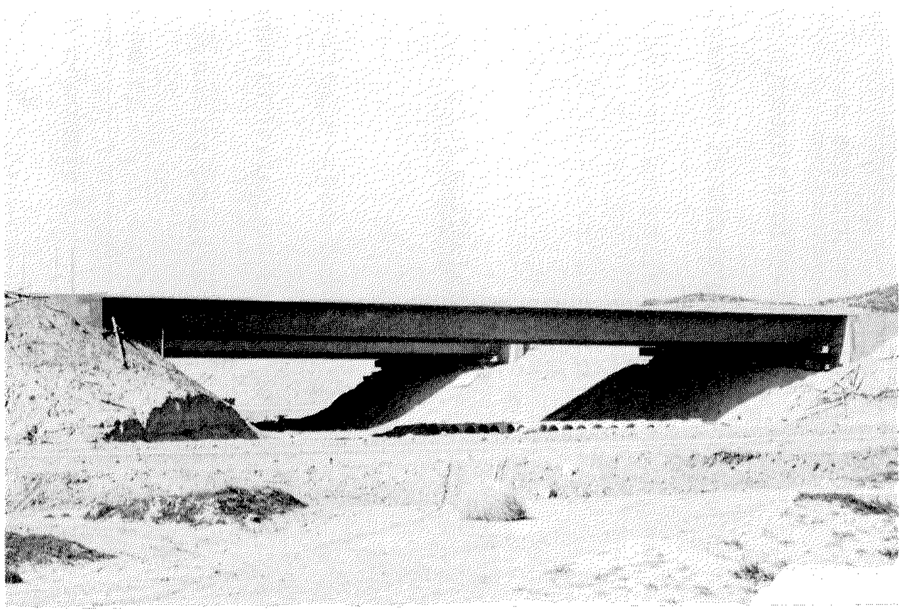


Figure 6.96 Via Princessa undercrossing southwest of Solemint, Antelope Valley freeway.

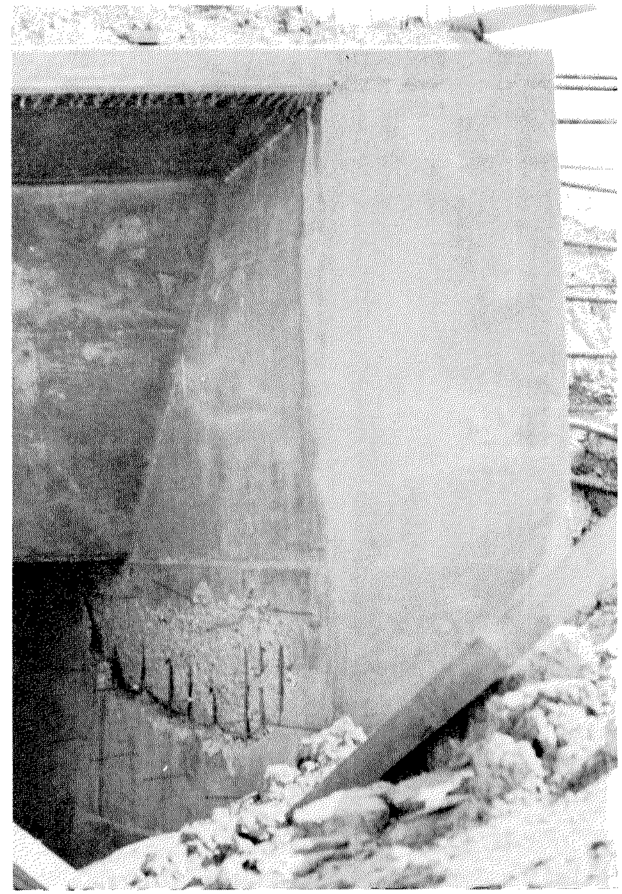


Figure 6.97 East abutment of the westbound lanes of the Via Princessa undercrossing, Antelope Valley freeway.

Figure 6.98 Santa Clara River bridges, Antelope Valley freeway.

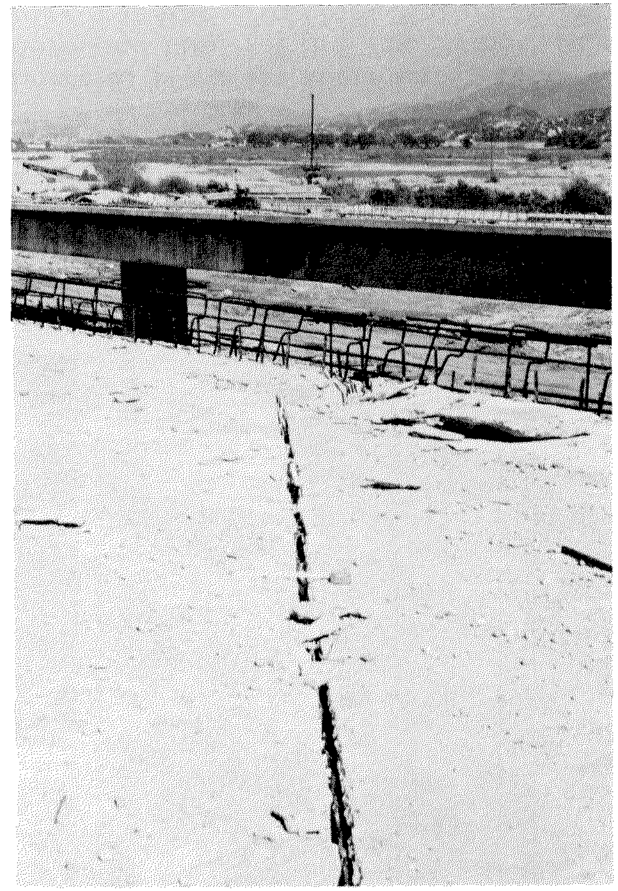


Figure 6.99 Pounding damage at a girder joint of one of the Santa Clara River bridges, Antelope Valley freeway.

The performance of the freeway structures in the area of strongest shaking, particularly the collapses of the overhead structures, points out that improvements must be made in the earthquake-resistant design of freeway structures. In looking at the freeway system, the overall impression is that the possible earthquake response of the structures has not had much influence on the design. This is probably a reflection of code requirements which are adapted from the SEAOC code and typically give a lateral force varying between the limits of two per cent and ten per cent of the weight of the structure. This minimal lateral strength requirement could be met by most structures without modifications to the design. These design requirements may be an improvement over the standard AASHO code, but they are inadequate for modern freeway structures, especially inverted-pendulum structures like the high overpasses. These structures should be designed with an approach that considers the dynamics of earthquake response. Because there exist numerous high overpasses in other portions of the freeway system that are presumably no more earthquake resistant than those which collapsed in the San Fernando earthquake, it is recommended that these structures be re-examined to determine their earthquake resistance. If necessary,modifications should be made, at least sufficient to prevent collapse in the event of very strong shaking.

The single-span bridges, for example those forming the Roxford Street undercrossing on the Foothill freeway, also should be examined from the dynamic point of view. The abutment piles are concrete, cast in 15-in diameter steel corrugated pipes and although such piles are presumably more than adequate for vertical loads, they are quite flexible for horizontal motions. Thus, from the dynamic point of view, the bridges consist of heavy masses, (the box girders), supported by 15-in diameter columns at their ends; in effect, they are single-degree of freedom oscillators. The structures are very flexible under horizontal loads, and the restraining effects of the soil around the piles and the freeway pavement provide the primary lateral and longitudinal stiffness. In the case of the Roxford Street bridge the restraints were insufficient. Other bridges, such as the Bledsoe Street overcrossing, are similar dynamic systems although the central columns increase significantly the lateral stiffness.

One feature of the earthquake damage to the bridges that needs more research to clarify the mechanics is the tendency of skew bridges to suffer permanent displacements in the sense of increasing skewness. This damage pattern was noted also in the Alaska earthquake of 1964.

It is well known that successful earthquake-resistant building design often depends on details, and this seems equally true for the bridges. Two details which this earthquake showed to be in need of re-examination are the joints in the box girders of bridges and overpasses, and the ties confining the column steel. Because collapse should not occur even in the strongest shaking, the seats in the box-girder joints should be longer and some positive connection between sections of the bridges may have to be provided.

It is considered that improvement of the column tie detail would provide a significant increase in earthquake resistance. Most of the damaged columns contained 1/2-in lateral ties at 12-in centers and, in general, the column failures observed were severe with considerable loss of concrete from the central region of the colum. Because a number of columns, particularly those supporting skew bridges, were subjected to unusual loading combinations, it is suggested that the area of tie steel be significantly increased over the full height of the columns. This would provide additional shear resistance as well as providing ductility for vertical and horizontal loads. The ties on the damaged columns had 18-in laps and with the spalling of the outer concrete they became ineffective. A more positive anchorage of ties, such as a standard 135° bend around the vertical steel, with an extension into the column center, is required.

Most freeway structures possess a relatively high amount of lateral resistance as shown by the limited area of serious damage, and it therefore seems possible to achieve the necessary improvement in seismic resistance with only a modest increase in cost.

EARTHQUAKE EFFECTS ON SPECIAL STRUCTURES

by G. W. Housner, P. C. Jennings and A. G. Brady

There were many special structures and facilities in the area affected by the earthquake which do not fall within the categories of the previous chapters of this report. Therefore, a few of the more interesting and informative of these structures are grouped together in the following pages.

Balboa Water Treatment Plant

The Balboa Water Treatment Plant, (also known as the Jensen Treatment Plant) located just west of the upper Van Norman Reservoir, was under construction at the time of the earthquake (Figs. 7.1, 7.2). This $25 million facility will receive water brought down from the Feather River by the California State Water Project. The water will be treated at this plant before being distributed by the Metropolitan Water District. The plant consists of a group of settling basins (Figs. 7.3, 7.4, 7.5) with an adjacent control building at the north end of the site, and a large underground reservoir at the south end of the site. An aerial view of the site is shown in Fig. 1.21. The original site was on terrain sloping down to the east. The western part of the site (sedimentary rock) was excavated to the present ground level, and the central and eastern parts of the site were covered by filled ground which sloped downward at the eastern edge to the level of the ground at the upper Van Norman Reservoir. After the earthquake there was evidence that the filled ground had settled and had moved towards the east. The sturdy, concrete, control building, which was just to the east of the settling basins, settled about four inches and moved to the east approximately

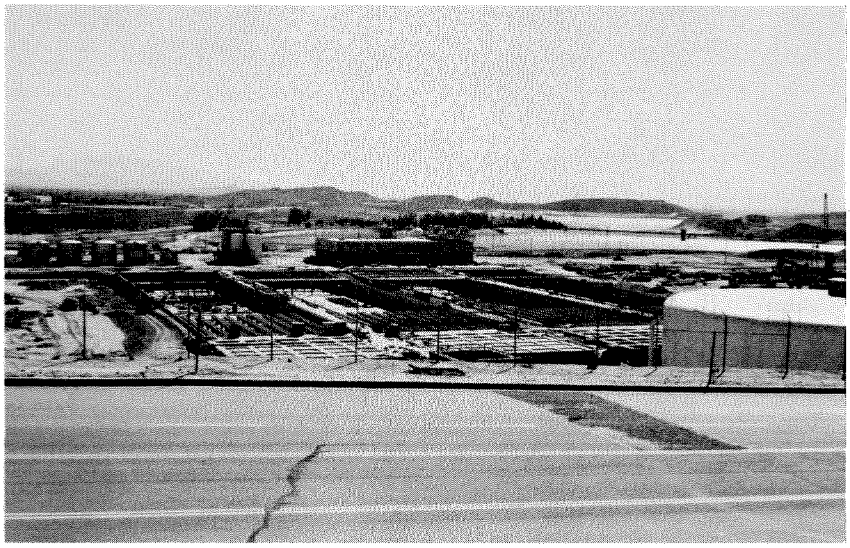


Figure 7.1 View looking east across Balboa Blvd. and the MWD settling basins. The two-story concrete control building is in the background; it suffered no cracking during the earthquake but did settle several inches and moved east several inches.

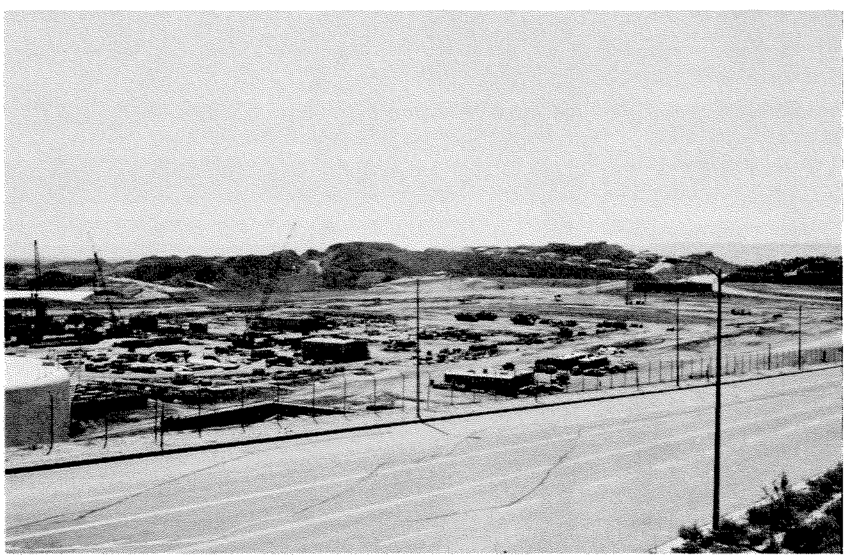


Figure 7.2 View looking southeast across Balboa Blvd. with the underground reservoir site in the background. Cracks in the Balboa Blvd. asphalt surface developed during the earthquake.

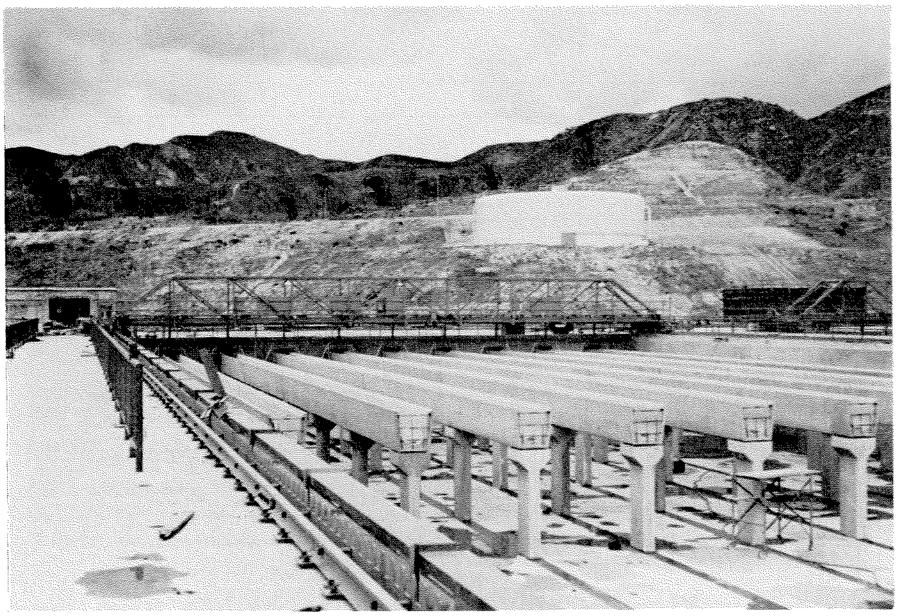
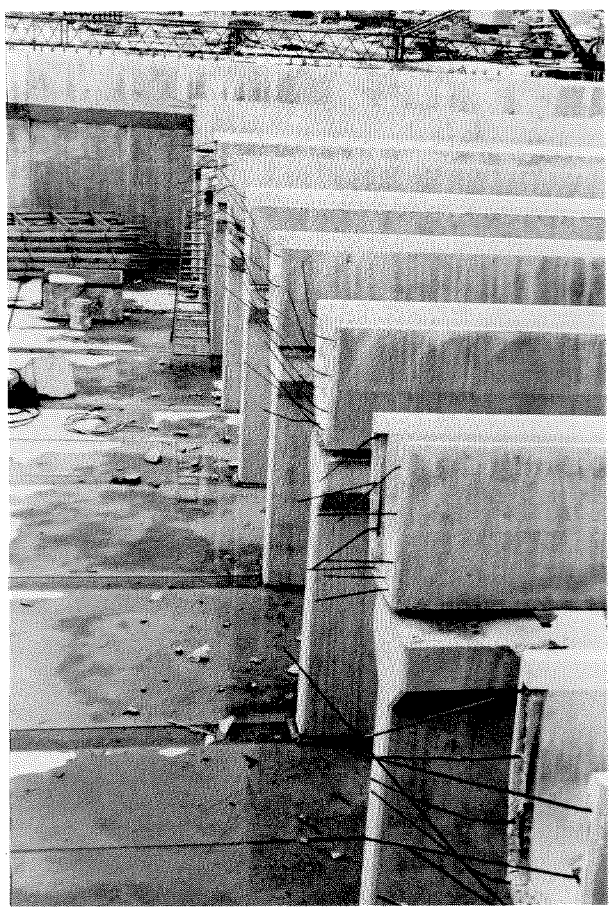


Figure 7.3 Looking westward across the incompleted settling basins, with the large MWD water tank in the background.

Figure 7.4 Incompleted channels in settling basins. The channels had not yet been anchored down and were displaced during the earthquake.



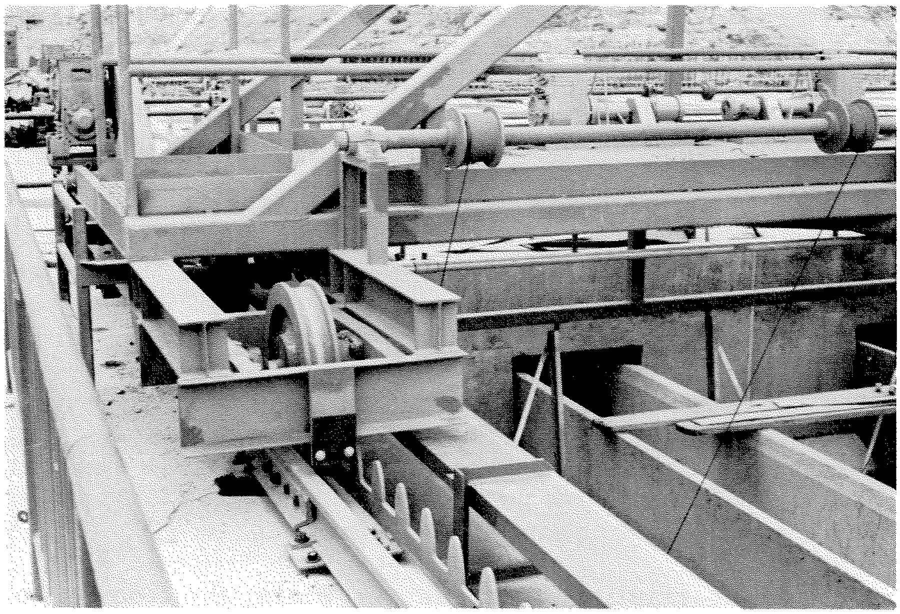


Figure 7.5 Wheel of traveling crane over settling basins jumped off the track.

Figure 7.6 Cracks in access road to large water tank. In right-hand background is the control building which settled several inches; to the left is the steel frame chemical building which was damaged by excessive vibrations (cross-bracing failed).

the same amount without cracking. However, underground facilities in this area were damaged by the ground movement. The underground reservoir, which was 500 ft by 520 ft in plan dimension, and 37.5 ft in vertical dimension, had a 14-in thick concrete roof slab supported on concrete columns 20 ft on centers, and had a 16-in thick floor slab (Figs. 7.7, 7.8). The reinforced concrete walls, roof slab and floor slab of the reservoir were severely damaged during the earthquake (Fig. 7.9). A large section of the west wall was pushed in several feet at the bottom, failing along a construction joint (Fig. 7.10). The roof slab was damaged along existing construction joints; the floor slab showed evidence of compression failures along construction joints, and many of the columns were damaged both at the top and at the bottom (Figs. 7-11, 7.12). Preliminary measurements made after the earthquake showed that some of the columns were several inches out of plumb, and there was evidence of small differences in elevation across the roof slab damage. The movement of the filled ground, which had about a 50% relative density, appeared to be the consequence of a consolidation and a sliding produced by the ground shaking.

There was no strong-motion accelerograph in the vicinity of the Balboa Water Treatment Plant site, so that the intensity of ground shaking can only be estimated. The nearest accelerograph in the valley was that at the Holiday Inn at the corner of Roscoe Boulevard and Orion Street, approximately six miles to the south. This instrument recorded a peak ground acceleration value of 28%g, hence, it is estimated that the ground shaking at the Balboa Water Treatment Plant's site was in the range of 30%-50%g maximum acceleration.

In the northwest portion of the site there was a large steel wash-water tank that had a diameter of 100 ft and was 30 ft high (Figs. 7.14, 7.15). It

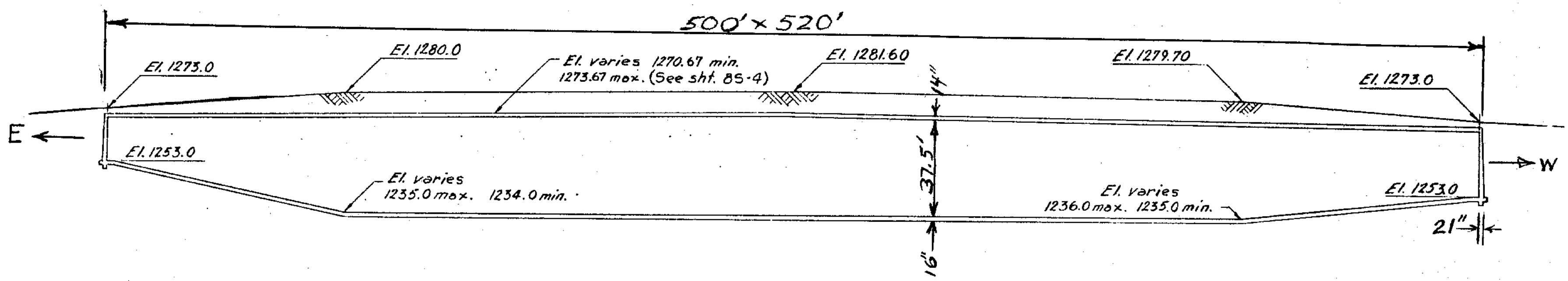


Figure 7.7 Schematic section through underground reservoir. At the time of the earthquake there was approximately 4 to 5 ft of fill ground on the northern half of the reservoir and sloping to about zero at the south edge. The base of the reservoir is thought to be resting on several feet of alluvium over the sedimentary base rock.

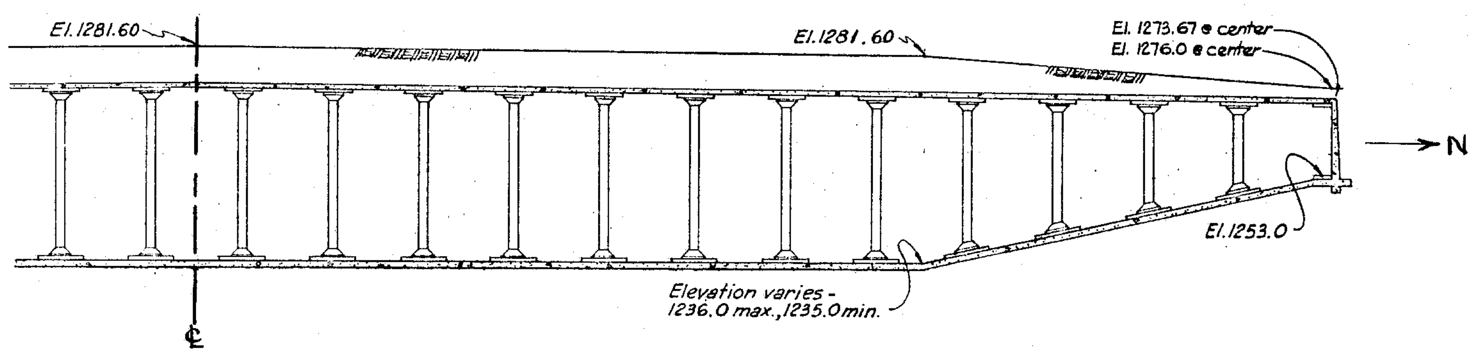


Figure 7.8 Section through reservoir. Spirally reinforced, 24" diameter concrete columns at 20' on centers both ways.

Figure 7.9 Interior view of reservoir showing damage to roof slab, and what appears to be compression failure in floor slab along a construction joint. (MWD photograph).

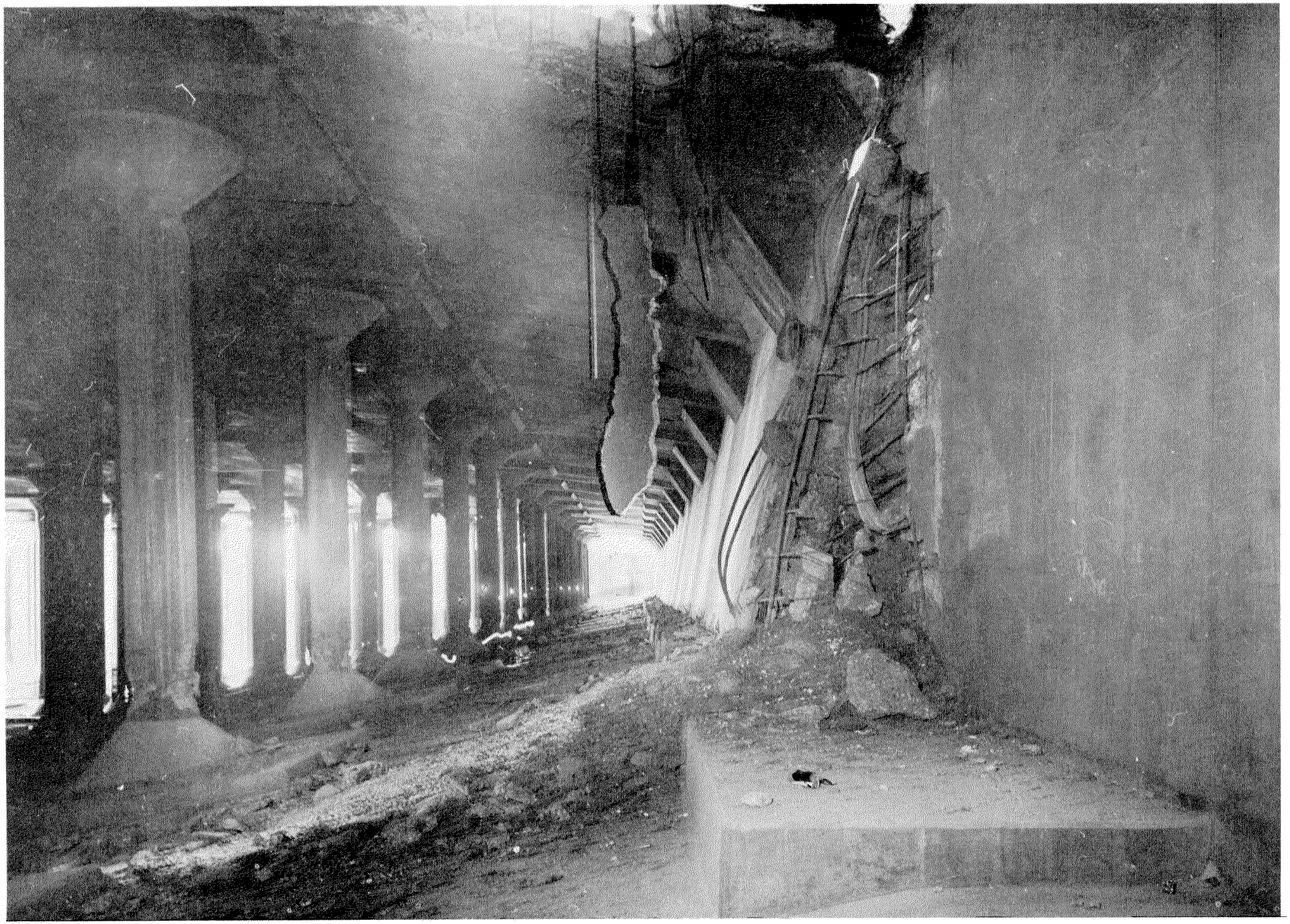


Figure 7.10 Interior view of reservoir showing failure of west wall at bottom pour joint. (MWD photograph)

Figure 7.11 Interior view of reservoir showing typical column failures and a difference in elevation across roof slab damage along the construction joint. (MWD photograph).

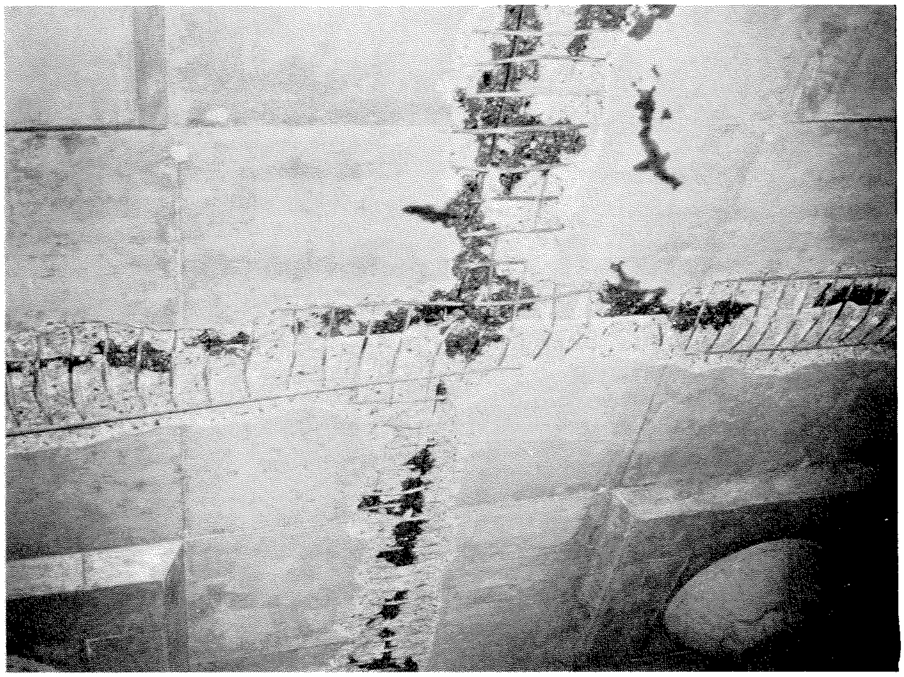


Figure 7. 12 Typical damage to roof slab along construction joints. (MWD photograph).

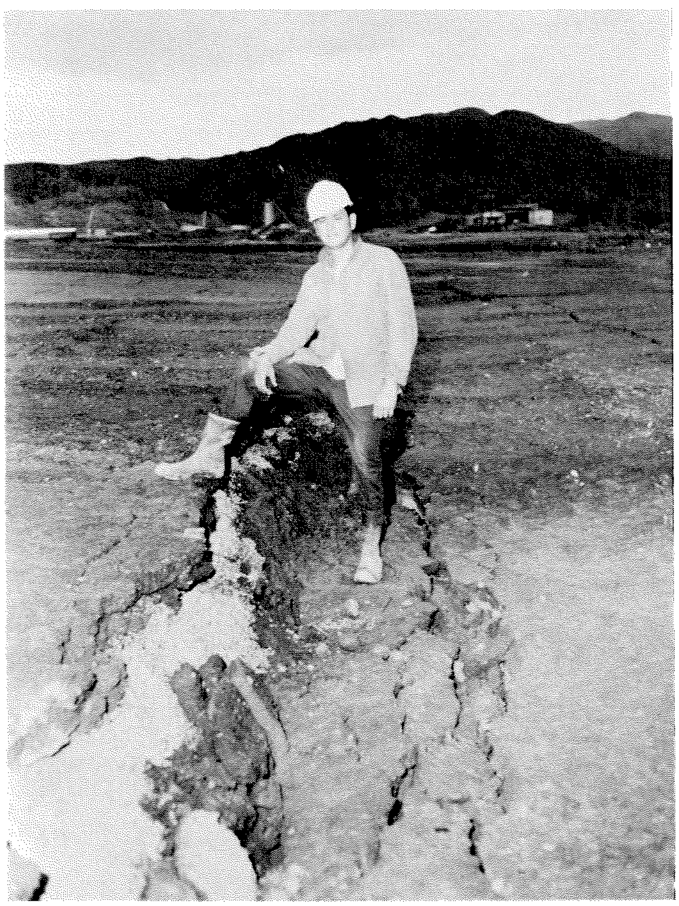


Figure 7. 13 Looking north along eastern edge of reservoir, showing settlement of filled ground.

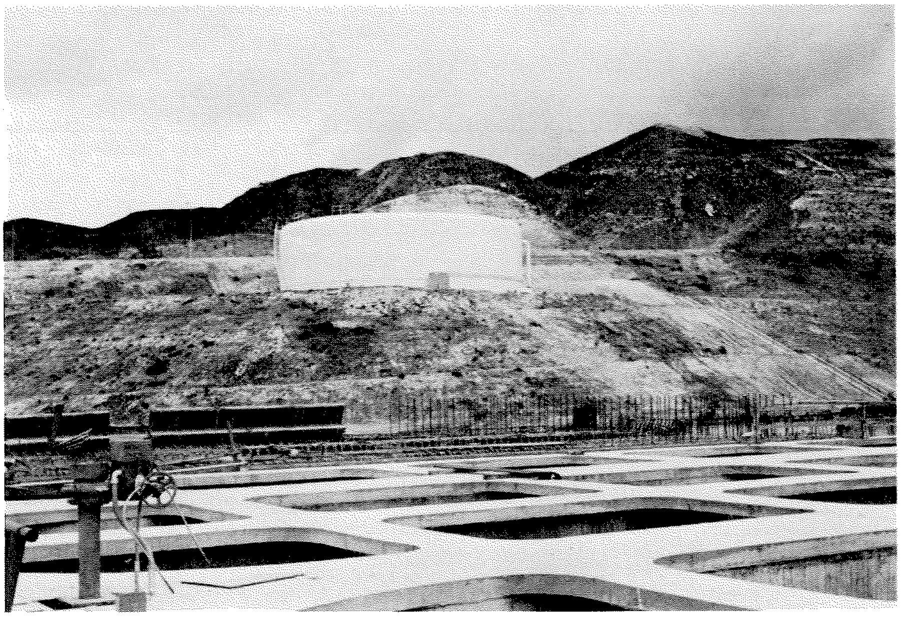


Figure 7.14 The 100 ft diameter, 30 ft high water tank above the settling basins was reported to have been about one half full at the time of the earthquake.

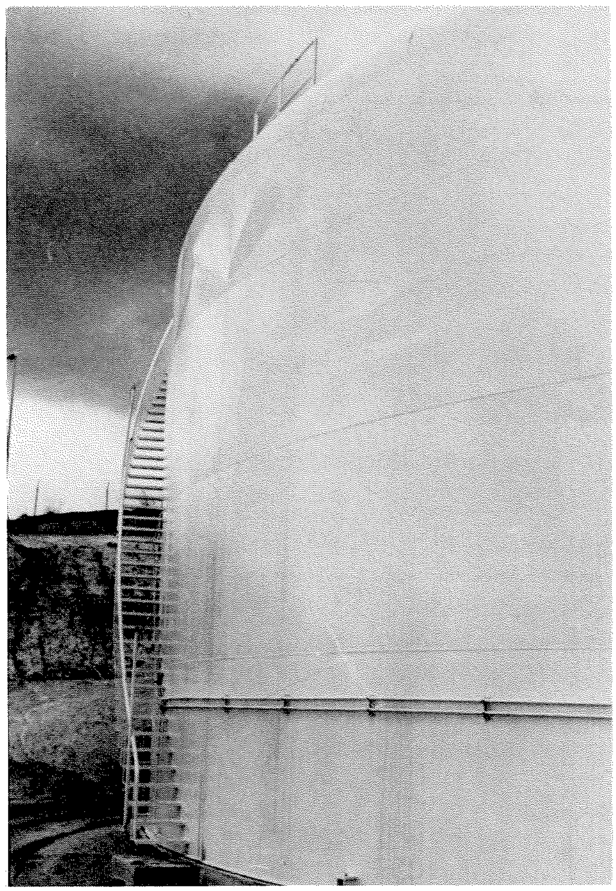


Figure 7.15 Buckle along top of tank at southern side.

was reported that at the time of the earthquake it was approximately 1/2 to 3/4 full. After the earthquake, the tank showed signs of having rocked on its foundation. Some of the anchor bolts failed in tension and others apparently failed in bond and were pulled up out of their anchorage. The amount of pullout varied from a couple of inches on the north side to 14 inches on the south side (Figs. 7.18 - 7-21). At the north side some heavy mechanical equipment was connected to the tank, and this apparently restrained its motion. It also appears from this evidence of rocking that the ground motion that affected the tank was stronger in the north-south direction that it was in the east-west direction. The upper part of the south wall of the tank was buckled; this presumably resulted from the unusual state of stress that existed in the tank at the time it was tilting up on its toe. The anchor bolts were of modest size and were spaced at considerable distance apart, so that they did not provide sufficient strength to hold the tank against uplift.

A cast-iron water hydrant was found to be shattered after the earthquake. It is supposed that this unusual damage resulted from a high, impulsive water pressure in the mains at the time of the earthquake (Fig. 7.22).

To the east of the northern part of the site was the old San Fernando Power Plant which generated electrical power from the Owens Valley water whose pipeline came down the hills northwest of the site (Fig. 7.23). The building and connected piping was severely damaged by movements of the soil, apparently a large landslide extended from the site of the Balboa Water Treatment Plant to the east past the San Fernando Power Plant.

Storage Tanks

In addition to the Metropolitan Water Department storage tank

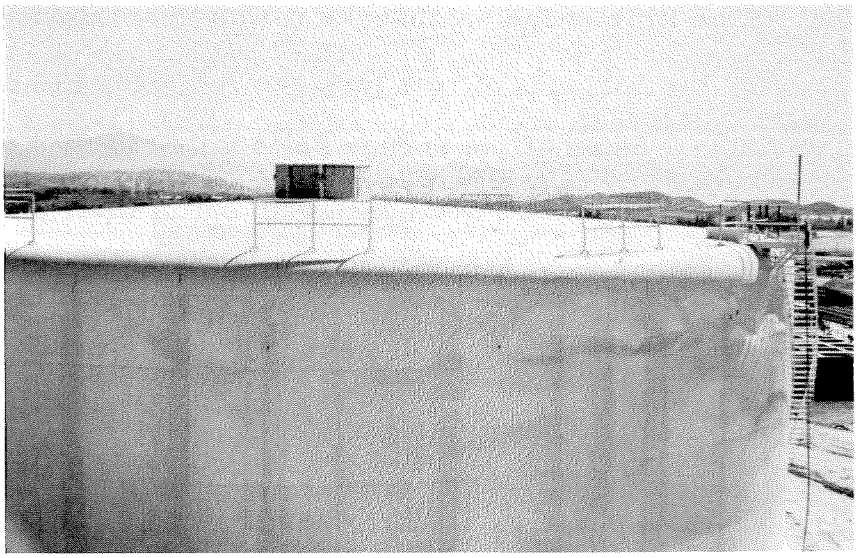


Figure 7. 16 Buckle at top of tank, looking east.

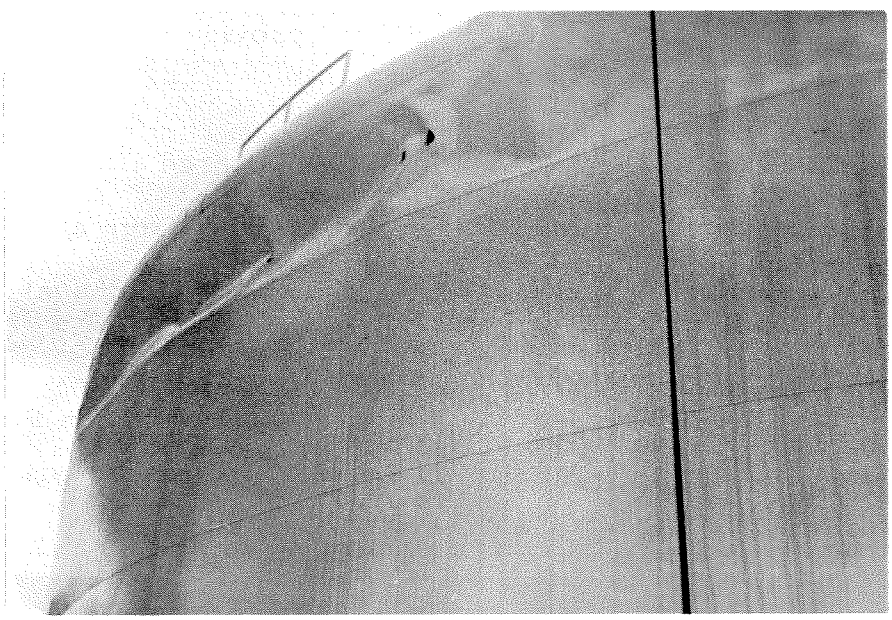


Figure 7. 17 Buckle at top of tank, photographed from ladder.

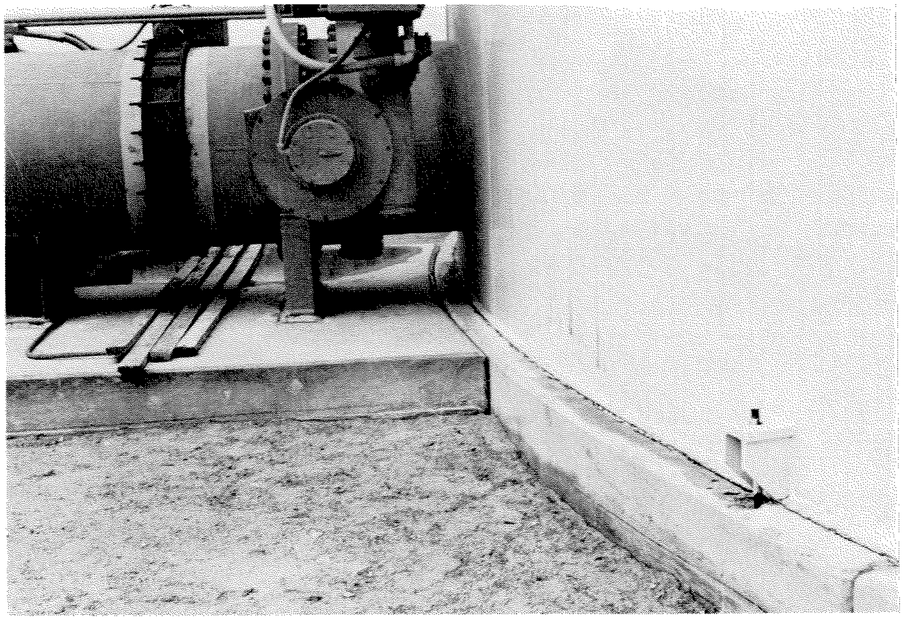


Figure 7.18 Equipment connection at north side of tank.

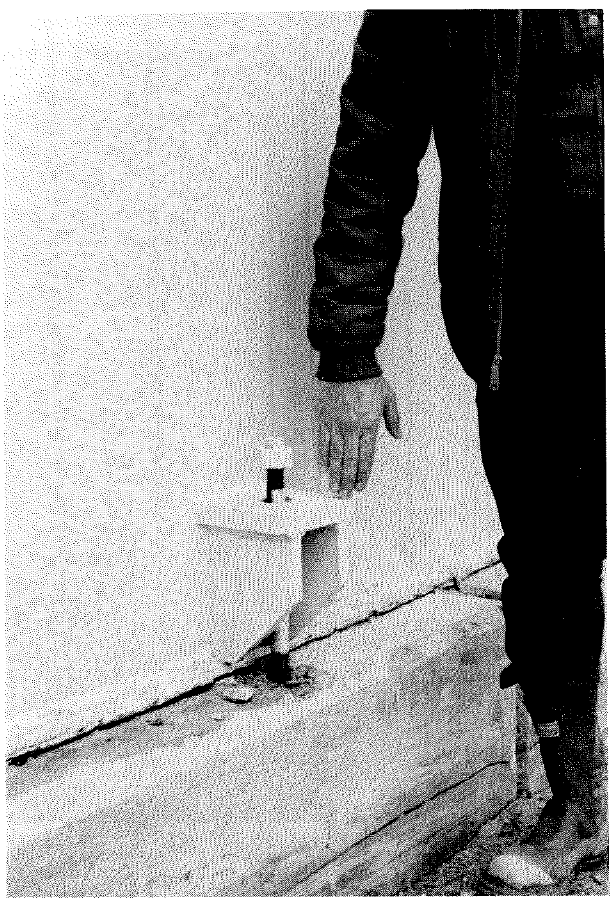


Figure 7.19 Anchor bolt pulled out; also shown in Figure 7.18.

Figure 7.20 Anchor bolt pulled up on west side of tank.

Figure 7.21 Anchor bolt pulled up 14" at south side of tank.

Figure 7.22 Cast iron water hydrant east of the settling basins was found shattered after the earthquake. MWD treatment plant.

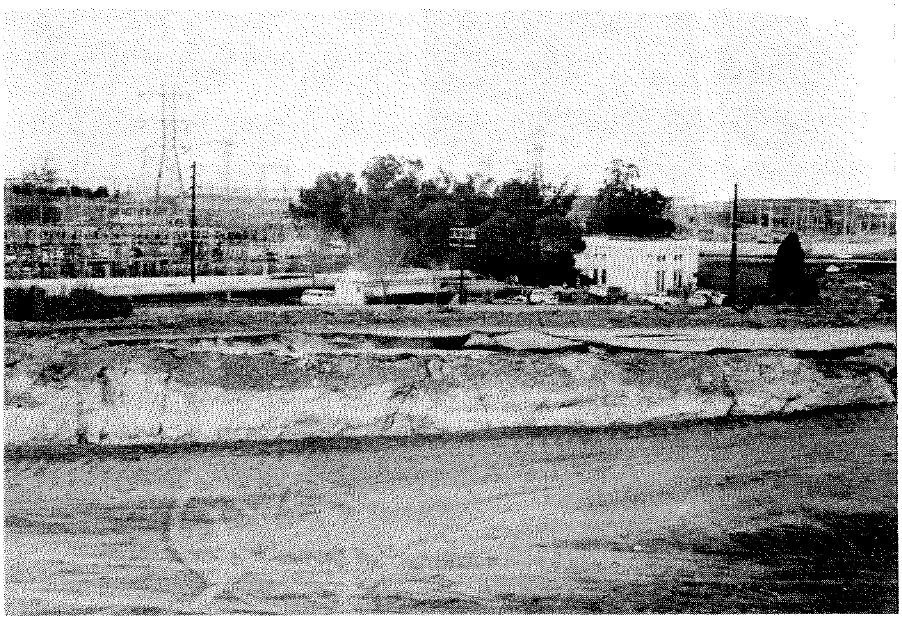


Figure 7.23 Old San Fernando power plant east of the settling basins was damaged by gross soil movements in an easterly direction.

Figure 7.24 Balboa Blvd. switchyard is surrounded by decorative cantilevered walls, presumably designed according to the requirements of the building code.

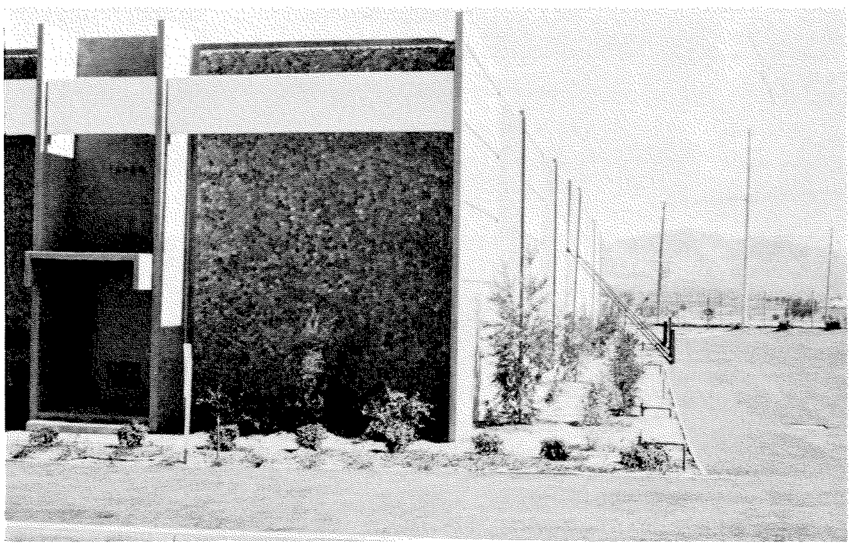


Figure 7.25 Walls of Balboa Blvd. switchyard are braced following the earthquake.

discussed above, there are several water storage tanks in the foothills of the epicentral area. The hilltop storage tank at the Olive View Hospital (Fig. 1.2) suffered damage in the form of an axisymmetric outward bulge of the shell close to ground level almost all the way around the circumference as seen in Figs. 7.26 and 7.27. The bulge covers a height of about 20 inches, and an amplitude of about 8 inches. The outlet pipe and connections broke and the floor plate broke away from the walls at one place, and the water emptied.

An older riveted steel water tank in Lopez Canyon (Fig. 1.2) suffered similar damage. The tank, reportedly about half full at the time of the earthquake, fractured at the outlet and lost its contents.

Two water tanks at the Veterans Hospital site experienced minor damage, including stretching of anchor bolts on one tank, but the structures are essentially intact.

Pergolas and Other Simple Structures

A roof shelter supported on a number of columns is a particularly simple structure, whose basic vibrational behavior during ground shaking is similar to that of a single degree of freedom oscillator. The mass of the oscillator is embodied in the roof standard and the columns provide the resistance to lateral motion. Such simple structures are important because they offer a means to learn about the character of the ground motion at sites where instrumental records are lacking.

The reinforced concrete pergola at the San Fernando Juvenile Facility shown in Fig. 7.28 rotated on its foundation during the earthquake and was left with a slight permanent set. A gas station pergola damaged by the earthquake is shown in Fig. 7.29. The structure, located at the corner of Glenoaks Boulevard and Roxford Street, is supported by two

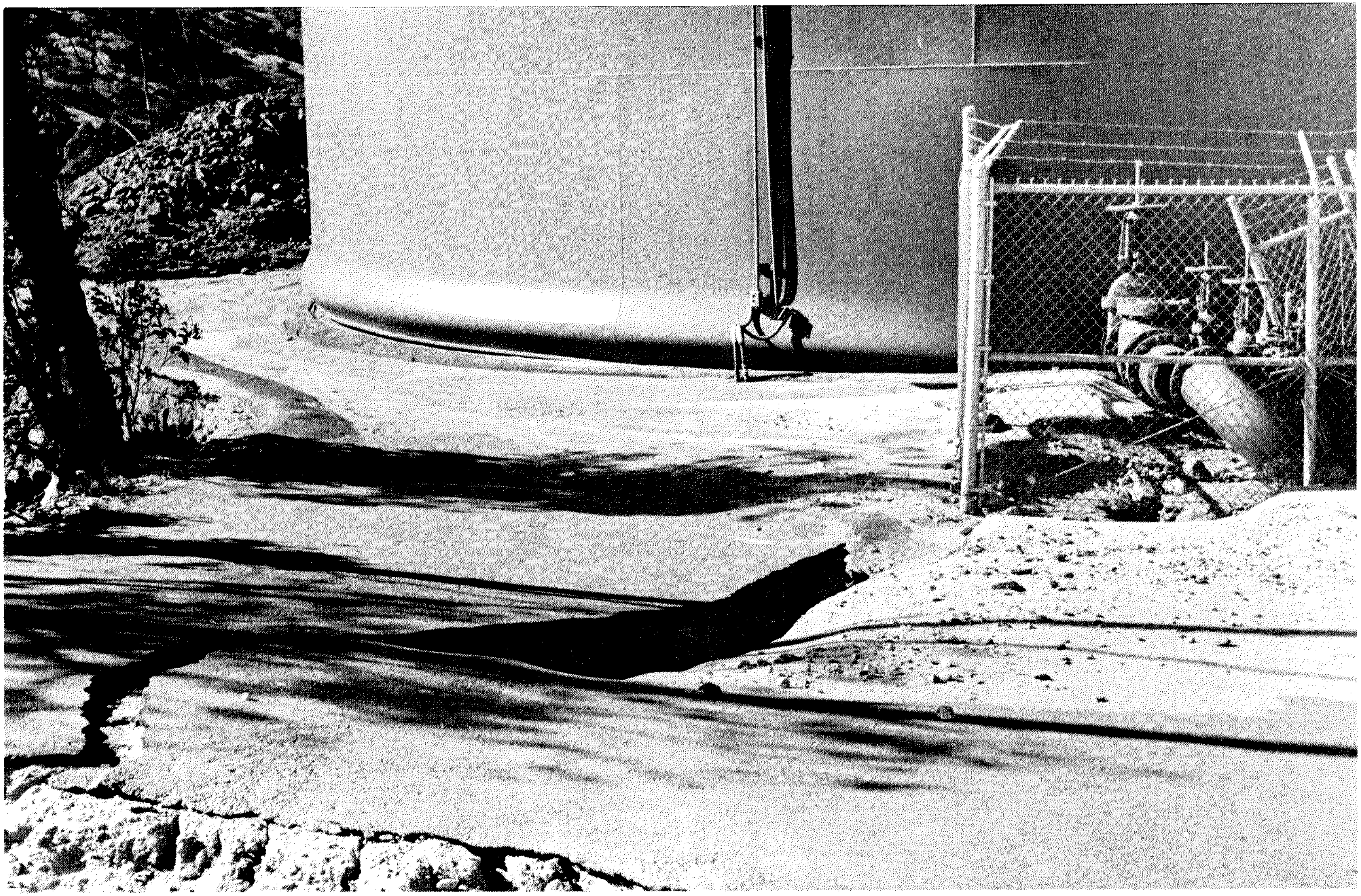


Figure 7.26 Water tank at Olive View Hospital. The vibrational damage caused failures of the tank base and adjoining piping, and the water was lost.

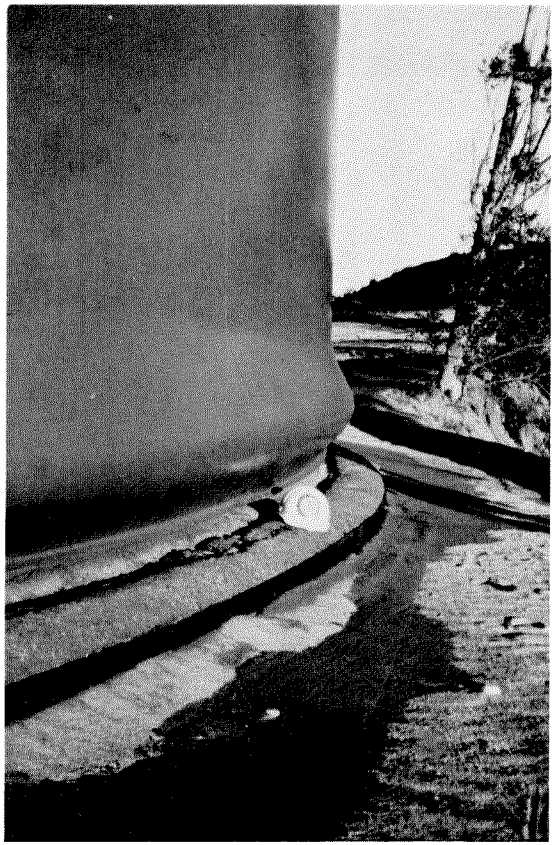


Figure 7.27 Olive View water tank showing damage to base of tank.

Figure 7.28 Reinforced concrete pergola at the San Fernando Juvenile Facility. The pergola was about 5° out-of-plumb after the earthquake.

Figure 7.29 Pergola at gasoline station at the corner of Glenoaks Boulevard and Roxton Avenue yielded at base. The supporting members are steel pipe columns.

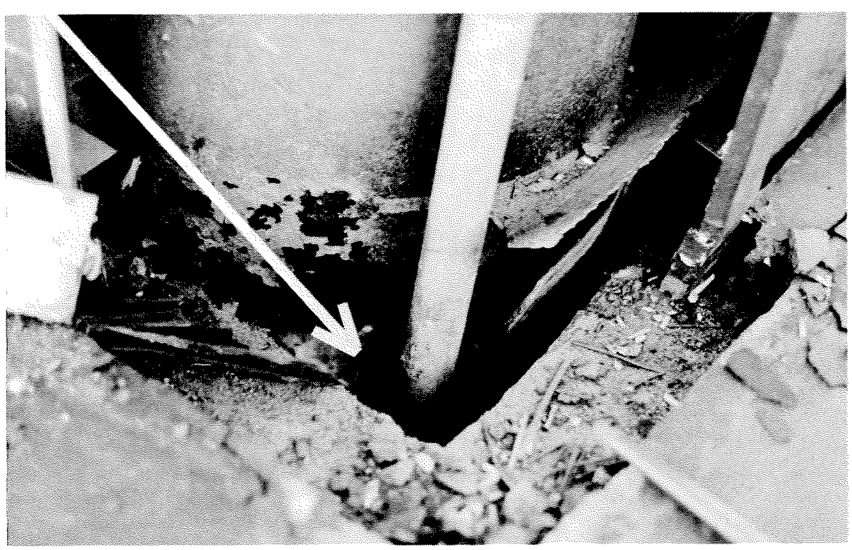


Figure 7.30 Yielded base of pipe column supporting pergola seen in Figure 7.29. A circumferential buckle is partly visible at the base of the columns.

steel pipe columns as can be seen in the closeup view of the base shown in Fig. 7.30. The pipe column developed a plastic hinge at the base with a resultant bulge strikingly similar to the one on the Olive View water tank. The bulge is partly visible in the center of Fig. 7.30. A similar pergola at the same gas station received lesser damage of the same kind, but no other damaged gas station pergolas were observed.

There were a number of small benches at Olive View Hospital which were overturned or upset during the earthquake, as can be seen in Fig. 7.31. The benches are of precast concrete construction, fastened to the concrete of the sidewalk by the detail shown in Fig. 7.32, which is the former base of the bench in the foreground of Fig. 7.31. The anchor bolts were loose in the holes in the bench legs. There was another overturned bench away from the area of sidewalk movement and ground slumping, on the other side of the covered walkway seen in Fig. 7.31. This bench is illustrated in Figs. 7.33, 7.34 and 7.35.

The benches have a high center of gravity and it is thought the primary reason for the overturning was the horizontal shaking, but research is needed to clarify the mechanics of failure.

With a few exceptions, light standards and powerline transmission towers successfully withstood the earthquake, even in the area of strongest shaking. One light standard at Olive View, which lost a bulb, can be seen in 7.33; other undamaged light standards are seen in Figs. 7.31 and 7.33. Only one of the many light standards was thrown down at the San Fernando Juvenile Facility (Fig. 7.36).

Throughout the epicentral area the familiar lattice-type transmission towers were being replaced by modern, tubular, steel towers. The old and new towers are shown together in Fig. 7.37. No significant vibrational

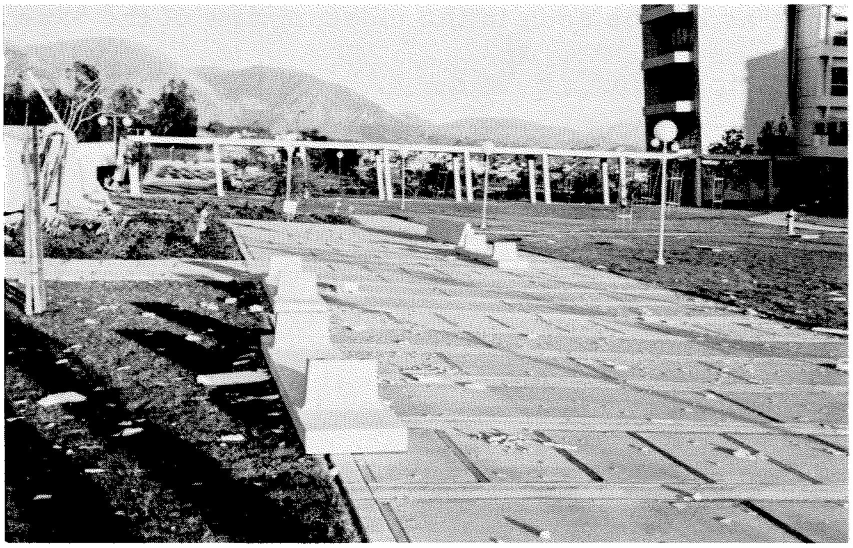


Figure 7. 31 Overturned benches on the grounds of the Olive View Hospital. The cantilevered light standards did not fail, except for one which lost a bulb.

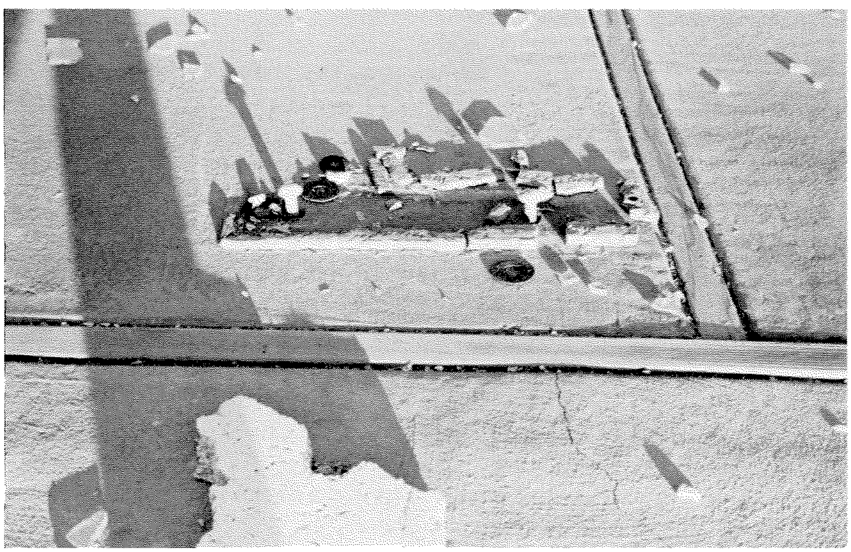


Figure 7. 32 Details of mounts for overturned concrete benches, Olive View Hospital.

Figure 7.33 Overturned bench and damaged light standard at Olive View Hospital.

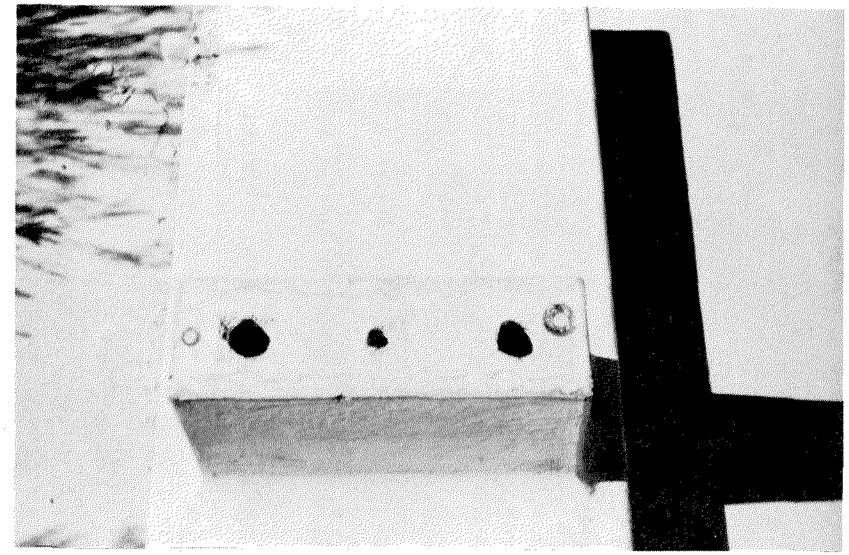


Figure 7.34 Base of overturned bench at Olive View Hospital.

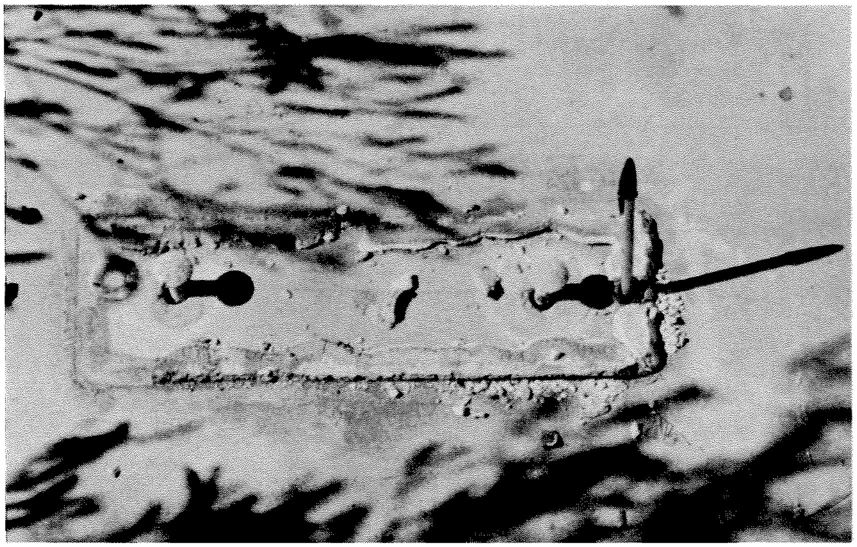


Figure 7.35 Mounting details of overturned bench shown in Figures 7.33 and 7.34.

Figure 7.36 Collapsed light standard at the San Fernando Juvenile Facility.

Figure 7.37 Light standard and new and old transmission towers near Olive View Hospital. The three structures were not damaged.

7.38 View of construction joint at midheight of an electrical transmission tower. The clips are used to secure the joint until it can be welded.

damage was observed on any of the lattice towers, although spreading of the footings was reported at several towers. Also, none of the modern towers which were completely erected at the time of the earthquake was damaged. The towers have a construction joint at about midheight, the details of which are seen in Fig. 7.38. There were two towers, under construction at the time of the earthquake, which had the upper portions fastened by clips, but the joints had not yet been welded. These two towers failed at the construction joint as shown in Figs. 7.39 and 7.40. The top of the second tower (Fig. 7.40) pierced a pickup truck which had been parked at the base of the tower.

Another type of special structure that survived the earthquake very well were the freeway signs. A typical structure of this type is the Polk Street exit sign shown in Figs. 7.41 and 7.42. This sign received no structural damage although it was in the area of strongest shaking; Olive View Hospital is visible in the background of Fig. 7.41. There was evidence of minor spalling around the base of the signpost, as is seen in Fig. 7.42, but there was no indication of yielding in the steel. From the performance in this earthquake, it is concluded that the typical freeway support is easily capable of resisting very strong shaking.

The elevated walkways at the upper and lower Van Norman reservoirs were severely damaged during the earthquake as is clear from Figs. 7.43 and 7.44. These structures are particularly susceptible to collapse if their supports suffer large displacements. In instances where the walkways are critical for safety, provisions for substantial displacements of the supports should be incorporated in the design.

One of the two railroad lines shown in Fig. 7.45 was repaired shortly after the earthquake and, as is seen in the figure, some soil motions

Figure 7.39 Two electrical transmission towers not yet welded at the construction joint failed during the earthquake. The completed towers were not damaged.

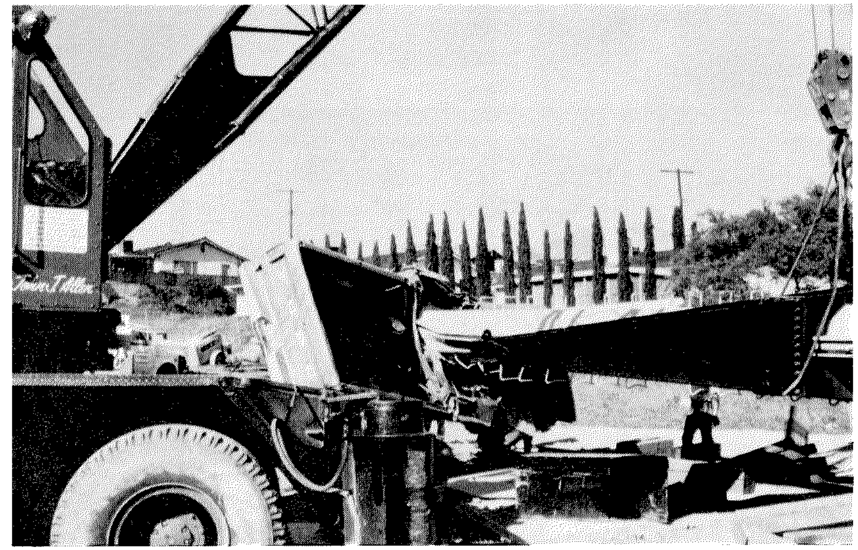


Figure 7.40 An electrical transmission tower under construction failed at an incompleted joint near the base of the tower.

Figure 7.41 A cantilevered, eccentric freeway sign near Polk Street on the Foothill freeway was undamaged by the earthquake. Olive View Hospital is in the left background.

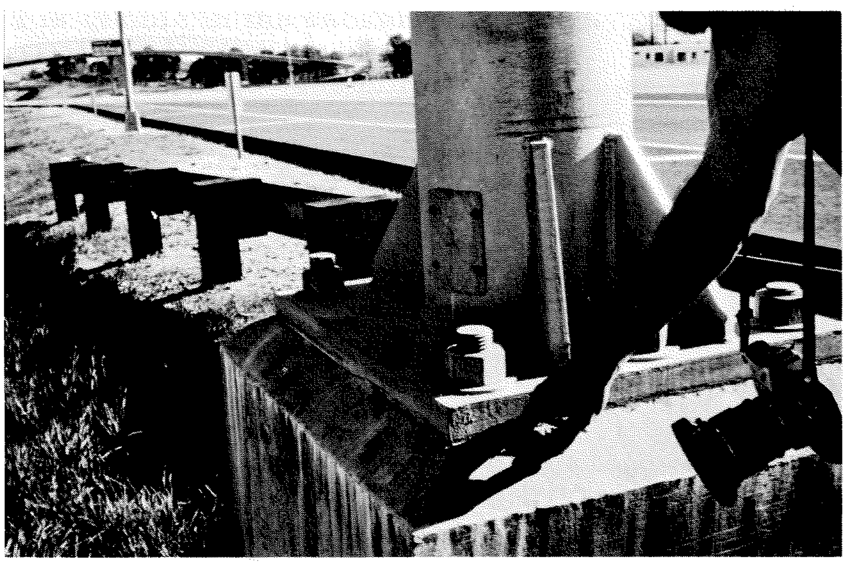


Figure 7.42 Base of the Polk Street exit sign shown above. Spalled concrete indicates small movements at the base of the undamaged structure.

Figure 7.43 Collapsed elevated walkway at upper Van Norman Reservoir.

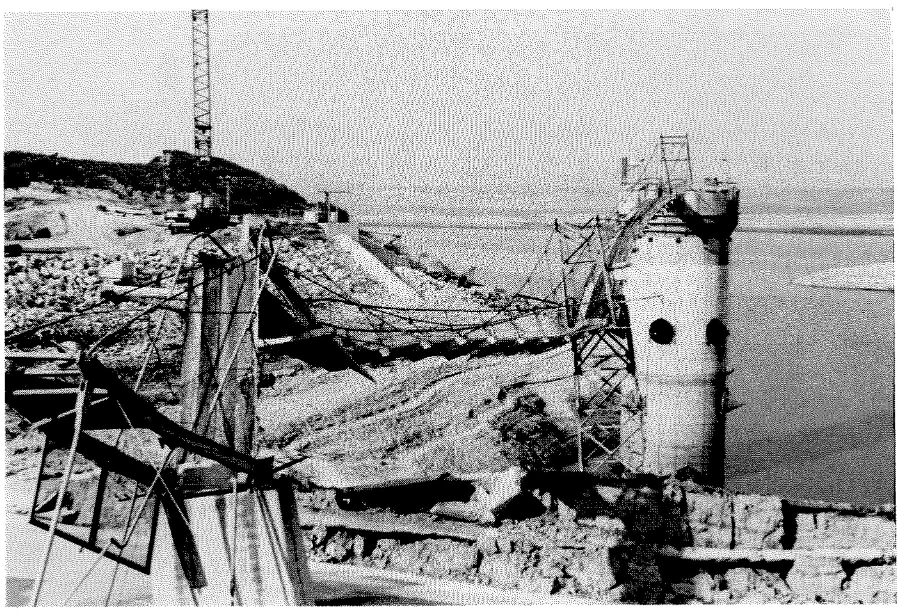


Figure 7.44 Damaged elevated walkway at the lower Van Norman Reservoir.

Figure 7. 45 Damaged and repaired railway lines near San Fernando Juvenile Hall. The repaired line shows post-earthquake movements to the left, toward upper Van Norman Reservoir.

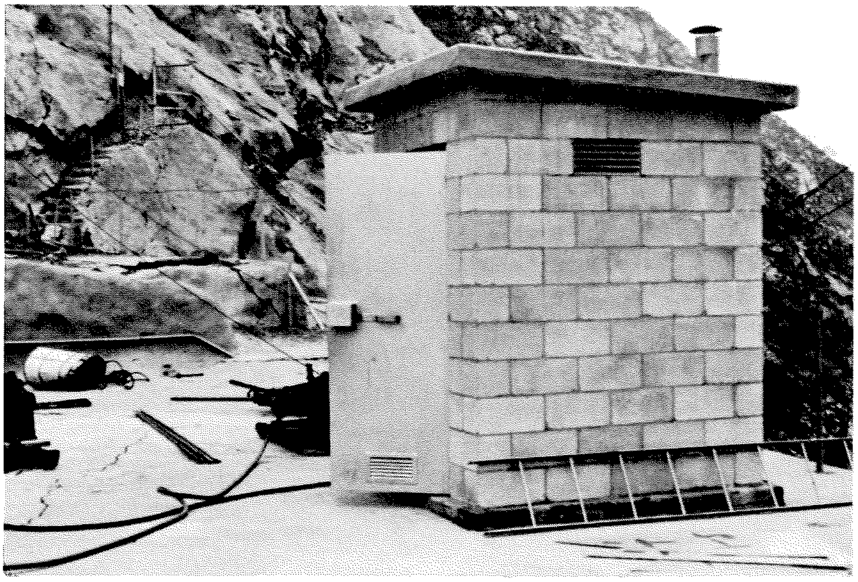


Figure 7. 46 A small concrete block structure on the abutment thrust block at Pacoima Dam was undamaged by the earthquake.

continued after the earthquake causing damage to the replaced line. The location is near the San Fernando Juvenile Facility which is visible in the right of the figure. The soil movements at this site are discussed in Chapter 4 of this report.

Figure 7.46 shows a small concrete-block structure which is on the thrust block of the southeast abutment of Pacoima Dam. Along with the dam itself, it is the closest structure to the site on which the Pacoima accelerogram was recorded. The block structure was not damaged even though it was subjected to about ten seconds of very intense shaking. Such a small rigid structure would respond in essentially a static manner and therefore must have sustained essentially the acceleration of the Pacoima record, shown in Chapter 2. The successful performance of the building indicates convincingly the feasibility of earthquake-resistant design.

Thatcher Glass Factory

The Thatcher Glass Factory, a large facility for the manufacture of bottles, located between the towns of Saugus and Newhall, sustained earthquake damage. Relatively little damage was sustained north of Pacoima Dam as compared to that sustained south of the dam. It is thought that the following circumstances account for this. North of Pacoima Dam the causative fault was at a great depth (Fig. 1.3) and, therefore, there were no manifestations of surface faulting and associated ground disruption. In addition, there were few large and massive structures in the Saugus-Newhall area which is mainly a new residential district. The Thatcher Glass Factory is the only large industrial plant in the area and it did receive damage from ground shaking, as seen in Figs. 7.45 - 7.50. There was evidence of overstraining in the steel frames of some

Figure 7.47 Thatcher Glass Factory between Newhall and Saugus. This large cluster of storage bins had its first-story supports damaged. The bins are supported on steel columns with I-beam bracing. The bracing was overloaded and underwent permanent deforations.

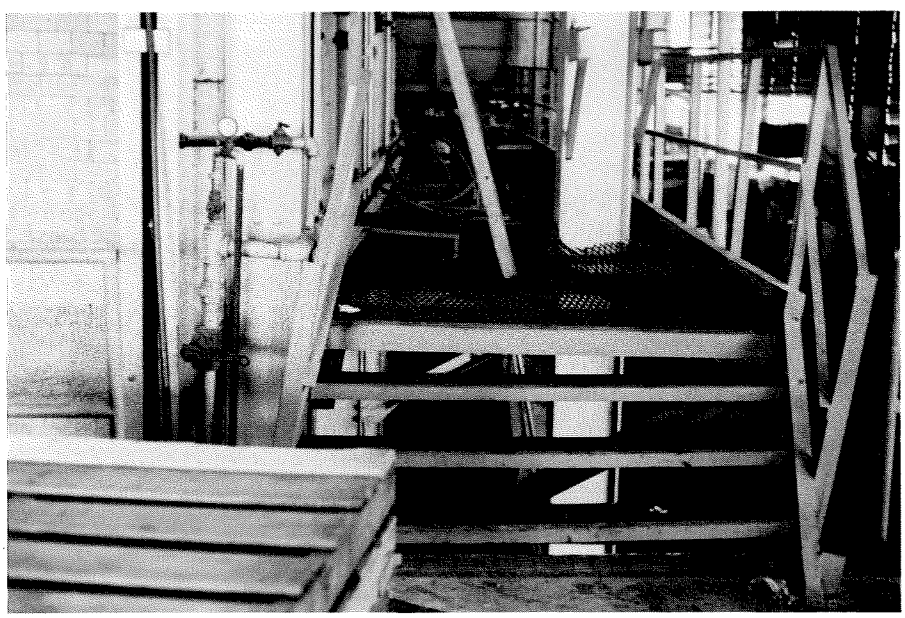


Figure 7.48 Thatcher Glass Factory. Massive glass furnace underwent a permanent horizontal displacement of about six inches.

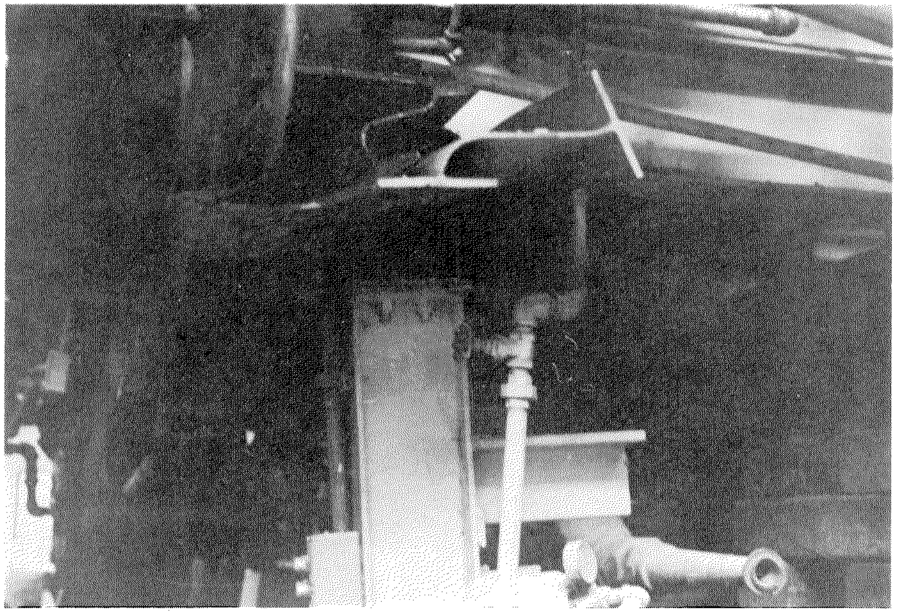


Figure 7.49 Thatcher Glass Factory. Crumpled I-beam support under one of the furnaces.

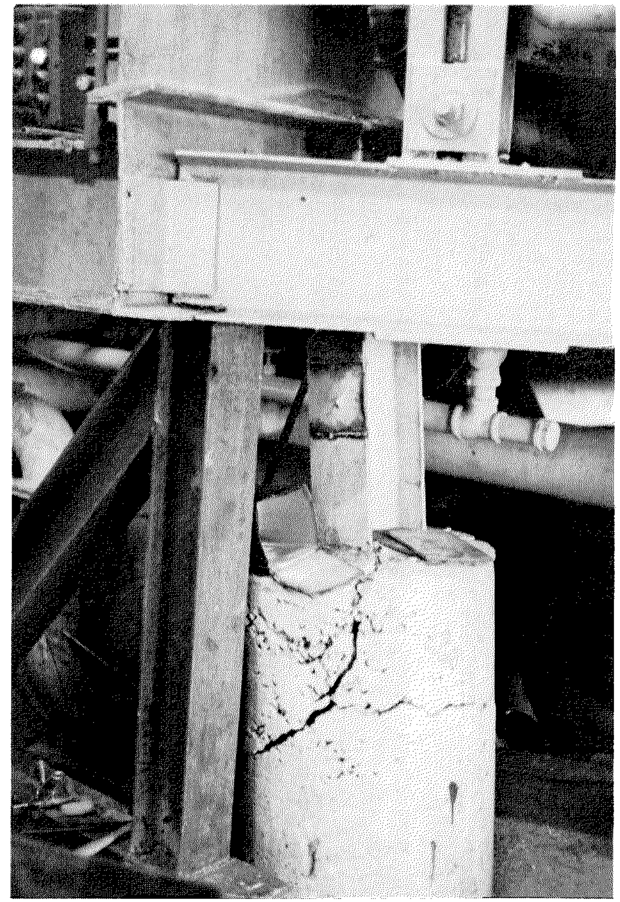


Figure 7.50 Thatcher Glass Factory. Deformed supports under a glass furnace.

of the standard one-story steel frame industrial buildings with corrugated-iron roof and siding. One of the large elevated cluster of storage bins, heavily loaded at the time of the earthquake, had its first-story steel bracing deformed by excessive horizontal shear force. Several of the large, massive glass furnaces were damaged by overstraining of the supports, and had to be rebuilt. A preliminary, uncorroborated estimate of total damage to the plant was $10 million.

Miscellaneous Damage.

The shaking damaged a wide variety of miscellaneous structures and equipment throughout the epicentral area. Such damage included sliding of large pieces of equipment in industrial shops and stores, collapse of supports for trailer houses, etc. Some examples of miscellaneous earthquake damage are included in Figs. 7.51 - 7.54.

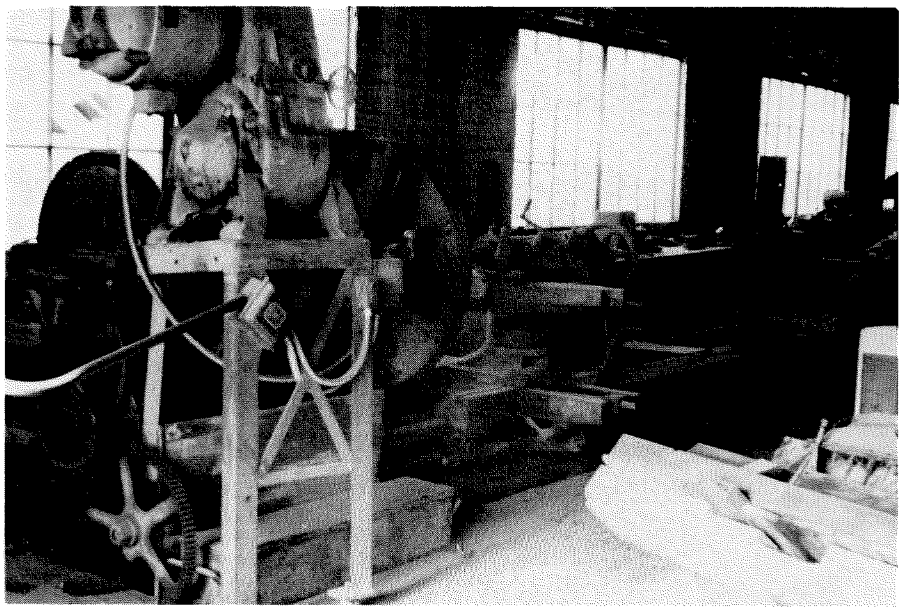


Figure 7.51 Heavy lathe at Brown's contracting yard on Glenoaks Avenue southeast of Hubbard Avenue. This heavy metal working lathe moved about 9 ft, relative to the floor, in a northerly direction, leaving a straight skid mark.

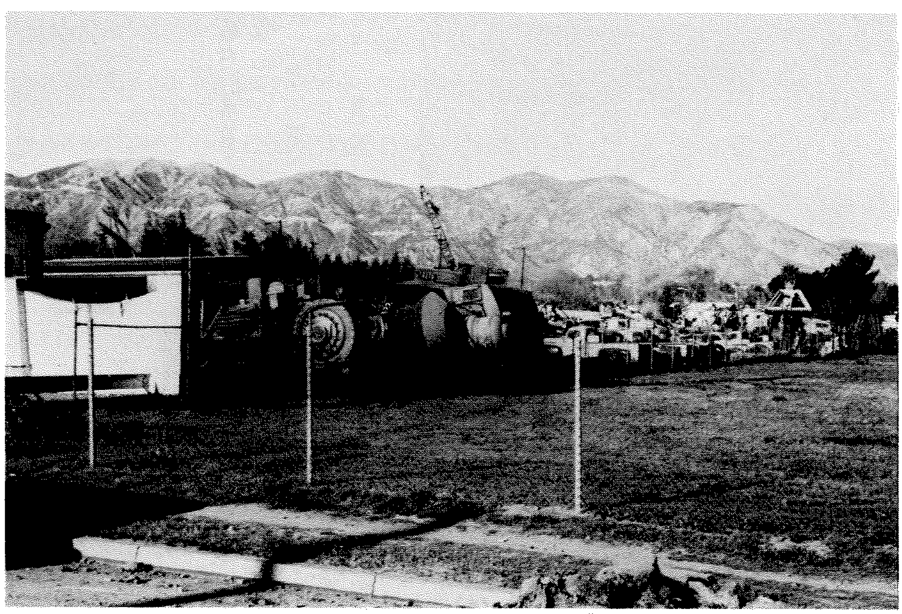


Figure 7.52 Locomotive of 1880 vintage at Brown's contracting yard overturned during the earthquake. The locomotive had been standing on a section of rail and ties. This shows that the overturning of a locomotive does not necessarily indicate extremely severe ground motion.

Figure 7.53 The supports of this steel water pipe, parallel to and just below upper Van Norman Dam had its supports deformed. The damage indicated a relative compression of the ground parallel to the pipe. The steel supports of the pipe underwent as much as several inches of deformation but the pipe was not damaged.

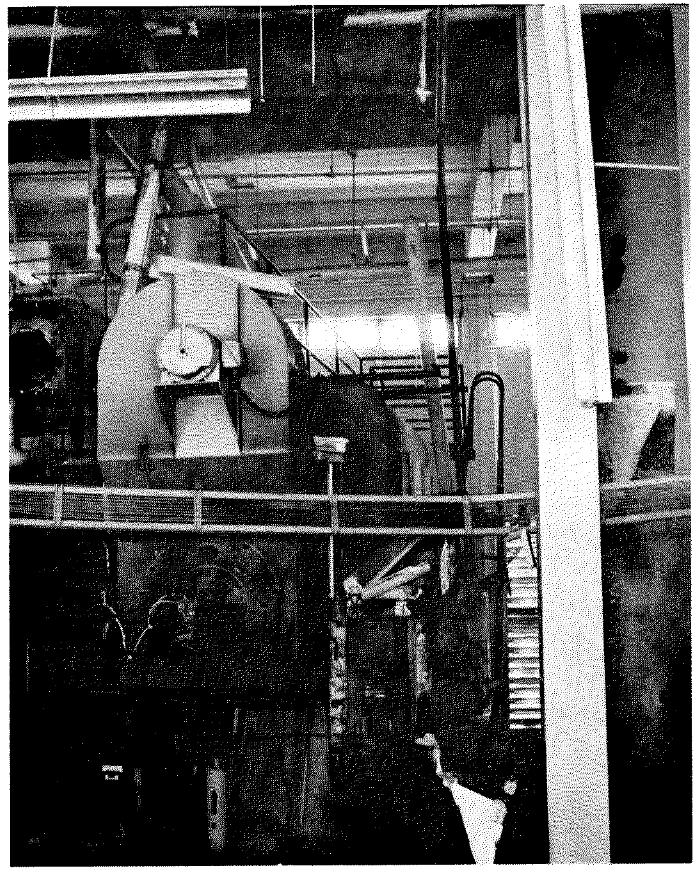


Figure 7.54 View, looking west, of one of the boilers in the new power building at Olive View Hospital. The western end of the boiler had not been anchored to the floor and it moved about four feet, relative to the floor, in a northerly direction. Equipment in this building that had been anchored to the floor did not break loose.

CONCLUSIONS AND RECOMMENDATIONS

by P. C. Jennings and G. W. Housner

Introduction

The San Fernando earthquake was unusually informative in terms of engineering in that a large metropolitan area, containing a wide variety of structures, was affected by ground shaking that ranged from low intensity to extremely high intensity. Had this event been a controlled experiment with all the necessary provisions made to record the desired data, it would have provided answers to most of the unsolved problems of earthquake engineering. Although many instrumental records were obtained, much of the desired information about ground shaking and building response was not recorded. For this reason, many of the conclusions that might have been drawn from this earthquake are not readily ascertainable, but will require more study and research. Those conclusions that have been drawn regarding specific features of earthquake damage, or ground and building motions, have been included in the preceding chapters of this report, and are not repeated here. Instead, this section is devoted to presenting general conclusions and recommendations relating to engineering practice and research, and to public safety.

Factors Limiting the Disaster

The San Fernando earthquake struck an unusual concentration of major structures and facilities, and the resulting damage and monetary loss established the earthquake as a major one from the engineering point of view. The location of the earthquake was particularly unfortunate from this viewpoint; if the epicentral area had occurred 10 miles to the west, north or east, the damage to important facilities would have been much less. On the other hand, there are other areas on the perimeter of the Los Angeles urban area that also have large concentrations of important facilities, such as the Los Angeles Harbor, to the south. Had the earthquake occurred offshore, the change in ground elevation would have generated a tsunami that might have been very destructive.

Severe as the loss of life and damage were, the earthquake cannot be said to have caused a disaster because the normal functions of the metropolitan area were not seriously disrupted, and relative normalcy was established in a few hours throughout the majority of the Los Angeles area, in a few days in most of the San Fernando Valley outside the area of very strong shaking, and in a few weeks in all areas except narrow bands where large, permanent ground movements occurred or where, like Olive View Hospital, decisions on the final course of action had not yet been made.

There were several factors which prevented this earthquake from becoming the sort of disaster that often accompanies earthquakes in many parts of the world. Foremost among these factors is simple good fortune. The survival of the San Fernando Dams* and the timing of the earthquake can only be attributed to good luck.

Another factor which mitigated the disaster potential is that the typical one-story, wood-frame house is particularly suited to survive the strongest shaking and severe ground disruptions without seriously endangering the occupants. The houses may be extremely cracked and damaged, and may even be a total economic loss as were about 450 homes

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* The upper and lower San Fernando Dams retained the water in the upper and lower Van Norman Reservoirs.

Figure 8.1 Aerial view of the Van Norman Lakes, May 19, 1971. The retention of water by the two older dams was the most fortunate feature of the earthquake.

in this earthquake, but the wood-frame house is seldom a threat to life and limb.

The fortunate timing of the earthquake at 6 a.m. is best appreciated by considering what would have happened had the earthquake occurred three hours later when the freeways would have carried heavy traffic at high speeds, when the lower floor of the Psychiatric Day Care Center at Olive View would have been occupied, when the downtown streets of San Fernando, Los Angeles and other cities would have been busy with shoppers, and when hundreds of thousands of office workers in buildings, both short and tall, would have been subjected to accelerations of the order of 20-40 percent of gravity. The timing of the San Fernando earthquake outside of school hours was not such an important factor in the low number of casualties as was the occurrence of the Long Beach earthquake after school hours. With the exception of falling ceilings and light fixtures in a few school buildings, it appears that the earthquake-resistant buildings resulting from the enforcement of the Field Act would have prevented a disaster from occurring at the schools even had they been occupied.

Another favorable factor was the lack of major landslides in densely populated areas. Such slides killed approximately 20, 000 people in Peru in 1970. Fortunately, the possibility for slides of this type in the Los Angeles area seems small.

In addition, in this earthquake the region of severe damage was limited to a small area at the edge of a large metropolitan center. As a consequence, the undamaged service facilities of greater Los Angeles were able to be focused on rescue, restoration and repair over a relatively small area. Overall, the fire, medical, police and utility services were able to treat their problems efficiently and effectively. The most

Figure 8.2 A market at the corner of Glenoaks Boulevard and Hubbard Avenue. Had the earthquake occurred during business hours the casualties would have been much greater.

serious exception to this generally satisfactory situation was an approximately ten square mile area that was without water, so that fires in this region had to be controlled with water from tanker trucks.

All of the effects mentioned above are minor, however, compared to the narrow escape from disaster presented by the two San Fernando dams. Both of these earth dams suffered massive soil failures and had the water been at the maximum restricted height behind the lower dam, or had the shaking lasted significantly longer, it seems certain that a catastrophic flow of water would have rushed through a populated section of the valley.

Intensity of Ground Shaking

The very strong ground shaking in the northern San Fernando Valley has attracted the attention of both engineers and laymen and caused apprehension about what the ground shaking might have been had the San Fernando earthquake been a significantly larger shock, for example, magnitude 8+. From the present understanding of the relation of faulting to earthquakes it appears that if the San Fernando earthquake had been larger, the main geologic difference would have been in areal extent of faulting, which would have been much greater in a larger earthquake, primarily because of a greater length of faulting. From the engineering point of view, the principal difference would be an increase in the area subjected to very strong shaking, with the consequently larger areas of possible major damage, for there would be many more sites in the same close relation to the fault as there were in the San Fernando earthquake. It is not believed that the intensity of shaking would necessarily be significantly greater if the San Fernando earthquake had been a magnitude 8+ shock because intense shaking requires a nearby release of strain energy and the sites in the northern San Fernando Valley would have been in the same proximity to the causative fault even if the length and width of faulting had extended to greater distances. The duration of strong motion would be longer, however, reflecting the contributions of larger fault displacement and more distant sources of energy to the shaking, and the added duration of motion could increase the severity of damage to structures.

Damage from Surface Faulting

A number of structures were damaged by the ground displacements of the Sylmar Fault Segment and the Tujunga Fault Segment. These were mainly residences that were wracked by differential foundation movements. No major structure was damaged severely by fault displacements. The area affected by surface fault displacements was less than one-half of one percent of the area affected by strong shaking, hence, the number of structures disturbed by faulting would naturally be very small compared to the number affected by strong shaking. As in other U.S. earthquakes, all the deaths during the earthquake were due to ground shaking and none were due to surface faulting. It is concluded for ordinary structures that the amount of damage and the hazard to life and limb from the surface faulting are very small compared to the damage and hazard from ground shaking. From the economic point of view, it is well to avoid building directly on known active faults, but it is more important to design structures so they can resist ground shaking without hazard to the occupants and without excessive damage.

Lessons from the Earthquake

Many of the more technical conclusions from the earthquake must await the results of detailed studies, but a number of general conclusions are readily apparent and these are included below. Many of these conclusions were brought into focus by this earthquake, but some are, unfortunately, the unheeded lessons from previous earthquakes.

(1) Many old, weak buildings in the region of strong and moderately strong shaking suffered severe damage, and the major loss of life occurred in two old buildings designed and built before the adoption of earthquake-resistant provisions in the building codes. From the experience in this earthquake it appears that ground accelerations with an amplitude of about 15%g mark the threshhold of serious damage for most poorer, old pre-1933 buildings, and accelerations of 30%g or greater are associated with very hazardous damage and collapse of most of the older structures. There are many thousands of such old buildings in California and this earthquake re-emphasizes the need for programs to identify such hazardous buildings and to render them safe, or to raze them over a reasonable period of time. A successful effort of this type has been underway for some time in the city of Long Beach, resulting in the strengthening or razing of about 100 old buildings; and in the city of Los Angeles, especially hazardous parapet walls on over 5000 buildings have been removed or strengthened. A much more extensive program to eliminate the major hazards of old buildings is needed, with participation by all communities in California. The program should include procedures to post buildings identified as hazardous prior to their strengthening or removal.

(2) An important consequence of the earthquake was that four hospitals in the San Fernando area were damaged so severely by shaking estimated to be in the 30-50%g range that they were no longer operational just when they could be needed most, and the patients had to be evacuated. Furthermore, the majority of the fatalities occurred in two old hospital buildings which

collapsed. It was an ironic feature of the earthquake that the most hazardous place to be, in general, was in a hospital.

To limit the disaster potential of earthquakes, certain critical structures and facilities should be designed so that they remain functional after undergoing severe ground shaking, in particular, damage should not be so severe as to require evacuation, although disruption of some of the functions for a few hours is probably unavoidable. Included in this category are hospitals, schools and other buildings designated for emergency relief shelters, high occupancy buildings, communication facilities, and buildings housing police and fire departments and other agencies needed to cope with the effects of the earthquake. To preserve the essential function of the structure will in most cases require nearly elastic response of the structural frame and careful detailing of nonstructural systems. The desired level of protection is considerably in excess of that provided by the building code, hence, special code provisions and special analyses will be required.

(3) This earthquake has furnished the first comprehensive, practical test of the earthquake provisions of the building codes. In general, modern structures designed according to the minimum requirements of the codes received only architectural damage in areas where the accelerations were 20%g or less. There was minor to appreciable structural damage in buildings subjected to shaking in the 20-30%g range, and the damage to buildings of minimum design varied from appreciable to collapse in the area of very strong shaking, (30-50%g). If the shaking had been longer in duration, as it would have been in a larger earthquake, the damage would have been more severe and more modern structures might have collapsed.

This earthquake confirmed what has been frequently stated in the past, namely, that buildings designed under the provisions of the building code can

Figure 8.3 Old buildings at the Veterans Hospital. The greatest hazard to human life in California earthquakes comes from the older buildings, built before earthquake-resistant design provisions were incorporated in the building codes.

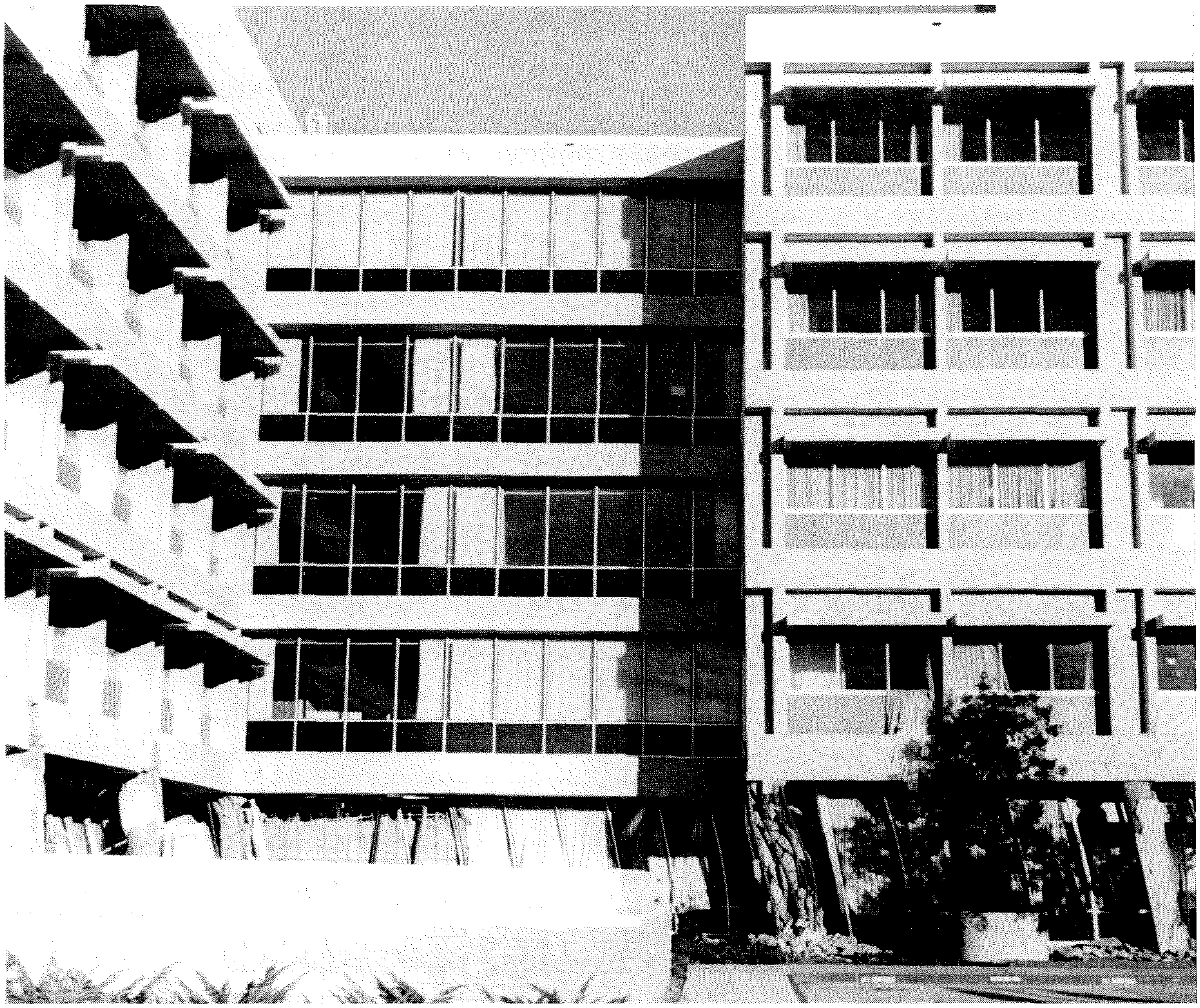

Figure 8.4 West elevation of the Olive View Hospital. The failure of this major new building and the collapse of the neighboring Psychiatric Day-Care Center show the need for improvement of seismic provisions in modern building codes.

differ markedly in earthquake resistance, depending on such things as architectural layout, structural type, engineering judgment, etc.

The earthquake showed that most modern structures can withstand significantly stronger shaking than can most older structures.* As a result, the modern buildings pose much less of a public hazard, as a rule, than the older buildings because in any earthquake the area of very strong shaking is much less than the area of moderately strong shaking. Therefore, the relative exposure to potentially damaging motion is much less for modern buildings. It is clear, however, from this earthquake, and the Alaskan earthquake of 1964, that existing building codes cannot be relied upon to provide consistently an adequate margin of safety against collapse. In view of the increasing variety of structures being built, it appears doubtful that any regulations that do not require some type of dynamic analysis of major structures will be capable of ensuring a consistently adequate level of protection.

(4) The nearly catastrophic failure of the two San Fernando dams endangered the lives of tens of thousands of people. Risks of this magnitude are clearly unacceptable and it is imperative that existing dams be brought up to modern safety standards. Such structures, in all parts of the country, should be examined thoroughly and strengthened or replaced where necessary to reduce the hazards to acceptable levels. The successful performance of a new earth-fill dam at the Van Norman site shows that modern earth-fill construction can withstand strong ground shaking estimated to have been in the 30-50%g range.

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* A few older structures of high quality did survive strong shaking without hazardous damage.

(5) A number of freeway overpasses and bridges collapsed causing two deaths and major disruptions of traffic. In a great earthquake many deaths and injuries could result from overpass and bridge collapses, and the interruptions of transportation could greatly magnify the disastrous effects of the earthquake. It is obviously important that freeway and highway bridges be designed for adequate safety against collapse, though economic considerations would dictate that some damage is acceptable. Present standard requirements for earthquake design of highway bridges are inadequate and should be revised in conformity with the current state of knowledge in earthquake engineering. Furthermore, important existing structures that could collapse in the event of similar earthquake motions should be identified and strengthened so that collapse will not occur.

(6) Many buildings in the San Fernando Valley, and some in Los Angeles received costly architectural damage even though experiencing no structural damage. Plaster cracking was a common occurrence, light fixtures and ceilings fell, air conditioning equipment on isolation mounts was disturbed, equipment that was not bolted to the floor moved about, elevator weights and cables became entangled, bookcases and partitions fell over, and furniture moved about and tumbled. Much of this damage was the consequence of oversight and could have been avoided by simple and inexpensive means. In the region of strong shaking, architectural damage was often severe. For example, even if the Olive View Hospital building had survived without structural damage, it would not have been functional because of extensive architectural damage and disruption of equipment. The possibility of architectural damage should be recognized by the architect and engineer when a building is designed and appropriate preventative measures should

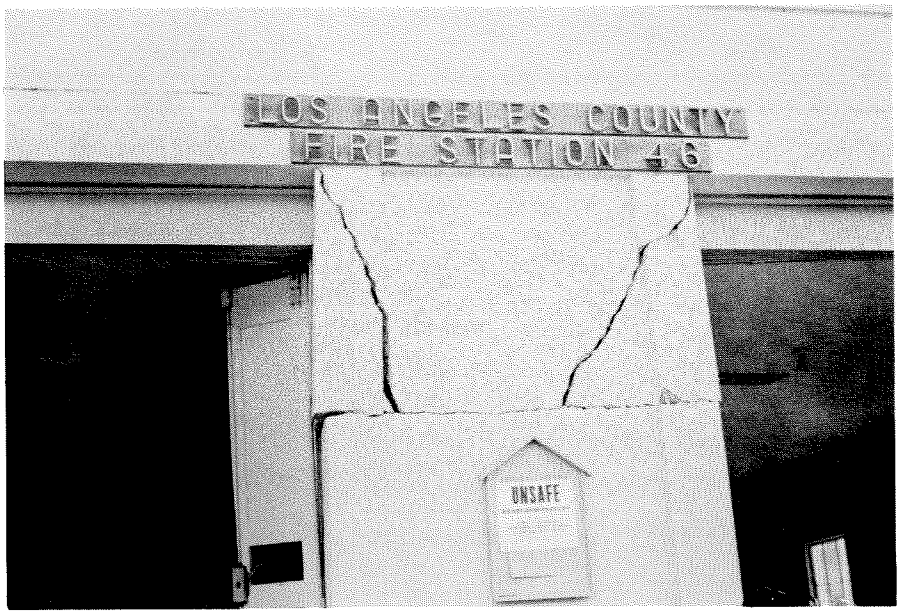
be taken to keep such damage within acceptable limits. When a building is being planned, collaboration between the architect and the engineer can often achieve a structural layout that will minimize structural damage as well as architectural damage.

(7) It is significant and encouraging that school buildings in the region of strong shaking, designed and constructed under the Field Act of the California State Legislature, did not suffer structural damage that would have been dangerous to the occupants had the schools been in session. These were mostly one- and two-story wood-frame and plaster structures. At a few schools there were fallen light fixtures and ceilings that could have been hazardous. The successfull performance of the schools demonstrates that one- and two-story school buildings can indeed be made safe by practicable code requirements even when subjected to very strong shaking ($>$ 20%g) combined with appreciable ground deformations beneath the structures. On the other hand, older school buildings that did not meet the requirements of the Field Act suffered potentially hazardous damage in regions where the ground shaking was 15%g or less, as in Los Angeles. The lesson is clear that such hazardous school buildings must be strengthened or eliminated.

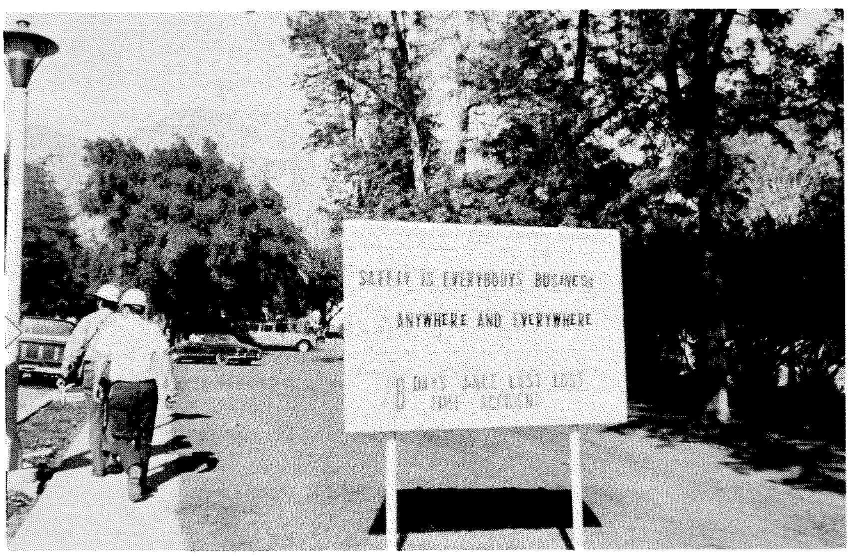
The lateral force requirements for the school buildings are essentially those of the building code and the successful performance of the one- and two-story school buildings reflects the fact that these structures actually possess, for other reasons, lateral resistance substantially in excess of the minimum code requirements.

(8) None of the tall buildings in Los Angeles was seriously damaged by the earthquake, but it should be emphasized that this earthquake was too far away from downtown Los Angeles to be a test of the ultimate strength of these structures; the ground shaking in downtown Los Angeles was on the

Figure 8.5 Tall buildings in downtown Los Angeles responded as expected to the strong shaking and did not receive structural damage from the earthquake. The shaking was not severe enough in downtown Los Angeles, however, to test the ultimate strength of these structures.



Los Angeles County Fire Station Number 46.



Veterans Administration Hospital, February 10, 1971.

Figure 8.6 Certain critical facilities should be designed to remain functional after a major earthquake.

order of 10-20%g, about half as strong as it was in the northern San Fernando Valley. Tall buildings, like other buildings, can be made to resist the strongest shaking without collapse, but this does not occur without special analysis and design, for existing codes are not as applicable to the newer tall buildings as they are to older, more familiar construction. Unless the special attention devoted to the design of recent tall buildings is continued in the design of others, tall buildings, too, can be a hazard in the event of strong shaking. Where comparisons have been made, the response of the tall buildings agrees very well with the calculated response, indicating that present techniques are capable of predicting the earthquake response of structures with satisfactory accuracy.

(9) The extensive and costly damage to the Sylmar Converter Station and several switching stations shows that these important electric power facilities must be designed to withstand ground shaking significantly stronger than has been considered in the past and the design must be based on the characteristics of dynamic response of equipment and structures. The problems posed by the materials and configurations used in these facilities differ substantially from the problems of earthquake-resistant design of buildings, so special techniques will have to be developed.

(10) From the engineering point of view, this earthquake was easily the best recorded shock in history. The records include the strongest ground motion yet recorded, and much valuable information on building response to strong shaking was obtained. Considering the success of the southern California strong-motion network during the San Fernando earthquake and the value of the data obtained, the present inadequate network in the remainder of the United States should be greatly expanded, with emphasis on urban areas in seismic regions and on sites of important

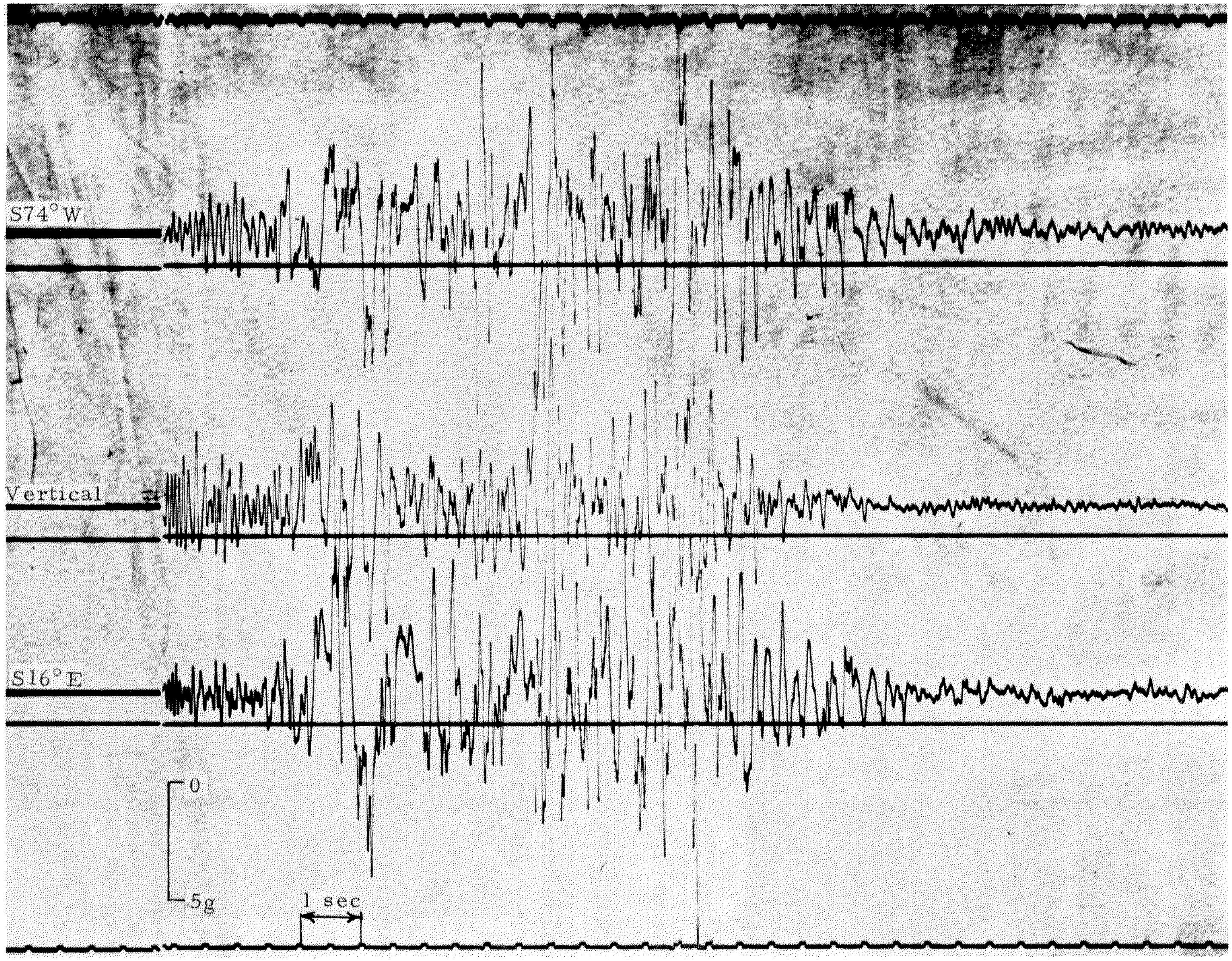


Figure 8.7 A photograph of the original record obtained at Pacoima Dam in the earthquake of February 9, 1971. The strong motion lasted about 8 secs and included the most intense accelerations yet recorded, with a few peaks of the horizontal motion exceeding 1g.

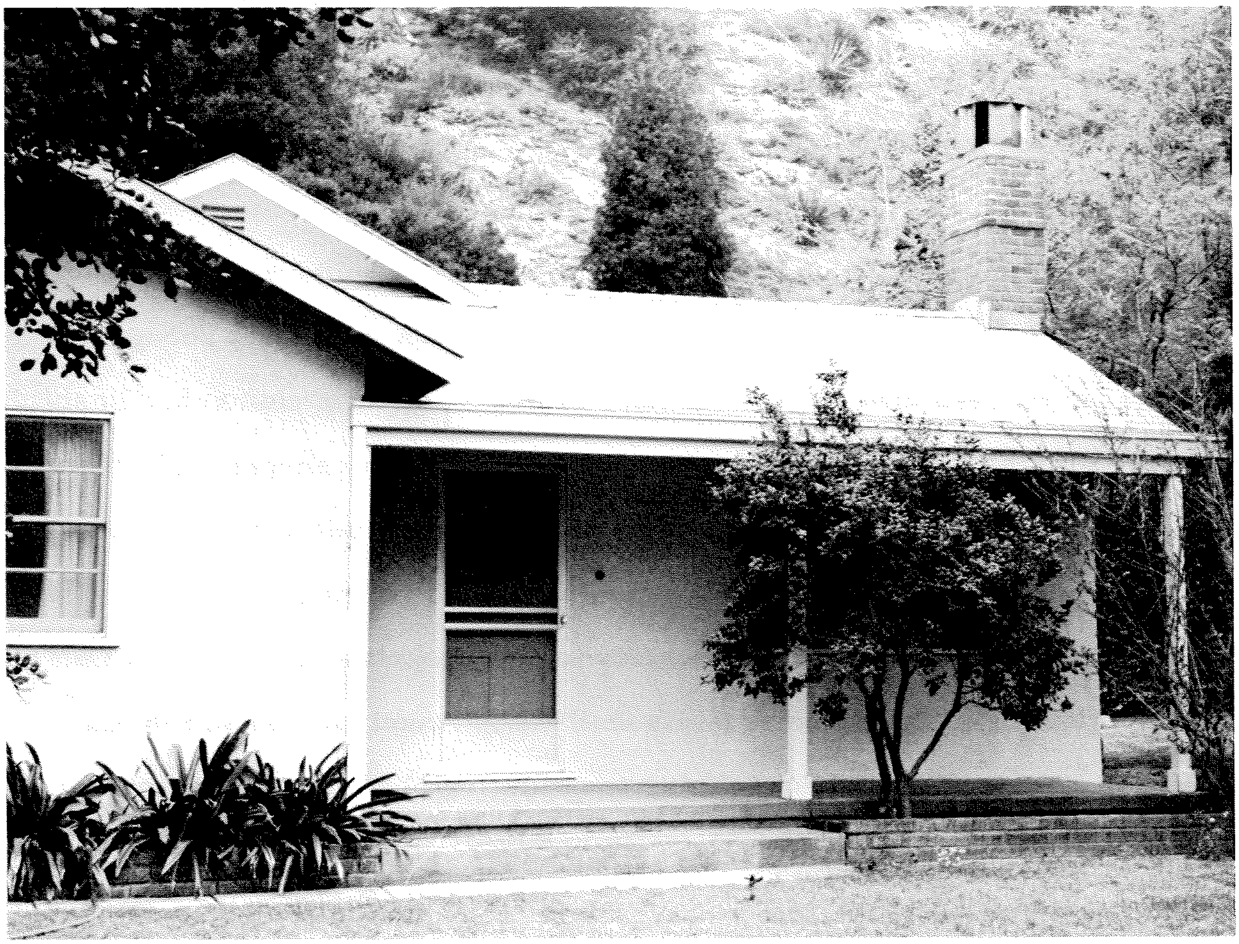


Figure 8.8 The Pacoima Dam caretaker's house at the mouth of Pacoima Canyon. This older structure, with a brick chimney, was not damaged by the shaking even though it was within one-half mile of Pacoima Dam and within one mile of the Veterans Administration Hospital. The degree to which the ground shaking here might have differed from that at these other sites is not known.

structures such as dams and nuclear power plants.

One of the major problems that still needs to be resolved is the degree to which the intensity of the ground motion can vary from point to point in the central region of a major earthquake.

(11) The geological information on the earthquake obtained so far indicates that the fault rupture took place on a fault not generally recognized before the earthquake and one not on most geologic maps of the area. Post-earthquake studies indicate that the fault could have been recognized on many portions of its length had sufficiently detailed geologic studies been made. Even if the fault had been recognized prior to the earthquake, however, it is hard to believe that it would have been widely recognized as being more hazardous than the many existing faults in the Los Angeles area which did not slip during the earthquake. In fact, the relative obscurity of the fault before the earthquake suggests the reverse is more likely.

Nor does the problem of relative hazard from moderate earthquakes seem resolvable by simply identifying all the minor active faults, especially when it is recalled that the existing fault pattern is a consequence of the adjustment to previous deformations. Because of the bend in the San Andreas fault and other tectonic features, the previous deformations of the earth's crust in the Los Angeles area may be significantly different from those now being imposed, and it seems too much to hope that knowledge of the existing fracture pattern will allow practically useful predictions of the fault motions needed to accommodate tectonic movements associated with moderate earthquakes.

From a practical viewpoint, it must be concluded that earthquakes up to about the magnitude of the San Fernando shock can occur in most of southern California, either on thrust faults or on strike-slip faults, and their occurrence cannot be predicted in either space or time to any satisfactory degree. This is not to say that geological and seismological information is not useful in assessing seismic hazards, but rather to point out its limitations. It should be realized also that the strong shaking occurs over a relatively wide area, and even if the causative fault could be identified and avoided, the hazard from shaking cannot realistically be avoided.

(12) Even given the occurrence of the earthquake on the fault, it seems well beyond the present state of the knowledge to determine with the necessary precision the location of the surface faulting in the deeper alluvium and the areas over which damaging, permanent ground displacements would occur. Therefore, many of the people who lost their houses or businesses in the Sylmar-San Fernando region because of ground displacements could have had no warning of the special hazards to which they were exposed by potential ground deformations. In the present state of knowledge, the victims of the next similar earthquake are very probably in the same position. Such hazards should be covered by earthquake insurance, with coverage of all who live in seismic areas. Some practicable form of earthquake or disaster insurance should be developed, with Federal support if necessary.

(13) Large, destructive fires did not develop after the earthquake even though minor fires occurred and sizable areas were without water. It is not clear what the disaster potential of earthquake-caused fires is, particularly in the event of an earthquake of large magnitude striking the metropolitan Los Angeles area. In view of the disastrous fires caused by the 1906 San Francisco earthquake and the 1923 Tokyo earthquake, and the

seasonal hazard in southern California from brush and timber fires, special studies should be made to evaluate the disaster potential of earthquake-caused fires in Los Angeles and, where necessary, steps should be taken to minimize the fire hazard.

(14) The San Fernando earthquake again demonstrated that the most practical approach to the problem of safety in earthquakes is earthquake-resistant design. Structures can be designed to withstand safely the most severe earthquake shaking, though this cannot be done without an increase in cost. For many structures this increase in cost is a modest one; for others it may represent a significant increase in overall investment. Once essential function and safety of life and limb have been assured, the problem of earthquake-resistant design becomes an economic problem; the intitial cost must be balanced against the possible cost of repair to earthquake damage over the expected lifetime of a structure.

Improvement of the Building Codes

The foregoing discussion emphasized that the earthquake provisions of building codes should be improved in the light of recent knowledge, particularly that derived from the San Fernando earthquake, if they are to fulfill their main purposes of protecting the public from excessive hazard during strong earthquakes and preventing an economic disaster from widespread, severe damage. In particular, it was pointed out above that existing building codes cannot be consistently relied upon to provide protection to the occupants in the event of very strong ground shaking.

The building code specifies both the level of lateral forces to be considered in the design and the stress levels at which the forces are to be resisted by the structural elements. It has been known for many years that the levels of prescribed forces are less than actually occur in earthquakes by a factor ranging from two to four, or more, and code-prescribed design stresses also underestimate what structures can do during an earthquake. The measured building responses in the San Fernando earthquake have provided dramatic confirmation of this fact. For example, the seven-story Holiday Inn frame building, about seven miles from the active region of the earthquake, had a peak acceleration at the top of 40%g and a base acceleration of 28%g, whereas the building code provisions are equivalent to a roof acceleration of about 15%g. The only structural damage reported was the cracking of some concrete members of the building frame that was repaired with epoxy cement. This building response, which is elastic or very nearly so as far as the structural frame is concerned, emphasizes the fact that buildings designed according to the code possess, in general, a level of elastic resistance that substantially exceeds that indicated by the lateral loads specified by the code. Furthermore, in the case of the Holiday Inn, the inelastic ductility of the structure provides a margin of safety against motions even greater than it experienced in the San Fernando earthquake.

Until recent years, it was generally believed by engineers that successful earthquake performance of structures was ensured by the conservatism in the stresses allowed by the codes, (even with the 33% increase for earthquake loading), by the ductility of structural elements, by the bolstering effects of many elements whose resistance, being difficult to calculate reliably, is not considered in the design, and by the conservative nature of accepted practice in structural engineering. Although these factors do continue to make most modern buildings earthquake resistant as noted above, the San Fernando and Alaskan earthquakes have made it obvious that not all modern buildings have the resistance required to ensure safety in the event of intense earthquake motions.

The earthquake force provisions of the code have not changed substantially in the last twenty years or more. In this time, however, the knowledge of the resistance of materials and the methods of calculating stresses and structural response have improved steadily. The increased knowledge of material behavior and the refinement of calculation techniques have tended to reduce the conservatism in structural design; e.g., allowable stresses are higher, columns have become smaller and spans have become longer. Also, more daring and innovative structural configurations have been made possible.

However, these advances have not been accompanied by a corresponding refinement in the assessment of the earthquake forces, which are greater than specified by the code, and as a result, the balance that may have existed between these two features of the building code has gradually been tipped in the wrong direction for some applications. An illustration of this is given in the San Fernando earthquake by the performance of the newer (1938, 1949, 1950) buildings at the Veterans Hospital and the main buildings (1965) at Olive View Hospital. Both of these building groups were in the zone of strongest shaking, and were nominally designed to resist essentially the same lateral forces. The Veterans Administration buildings survived the earthquake successfully, those at Olive View did not. The most substantial difference in the two complexes is the amount of conservatism in the design of the box-like buildings at the VA site compared to that in the design of the frames and shear walls at Olive View. Other differences, such as material quality and details of aseismic design are thought to weigh in favor of Olive View.

The present level of earthquake protection provided by the codes in recent years is perhaps best exemplified by the performance of the Indian Hills Medical Center and the nearby Holy Cross Hospital. Both of these modern structures were severely damaged by the earthquake, with the Indian Hills structure being repairable and the Holy Cross Hospital requiring at least partial demolition. Both buildings are judged to have been on the verge of much more serious damage, and had the shaking been longer, some partial collapses would probably have occurred. The earthquake subjected both buildings to very strong shaking estimated to be in the 30-40%g range. From the performance of the structures and the assessment of the ground motion, it is concluded that the present level of resistance provided by the code for this type of structure is close to, but on the unconservative side of, the desired safe level.

In our judgment the equivalent static force method of design set forth by the building code is too unrealistic to be consistently effective. By underestimating both the dynamic forces of earthquake response and the dynamic resistance of buildings to different degrees, the code leaves open the possibility for unsafe structures to be built. In the present state of affairs, as materials and methods of analysis improve, the situation for buildings designed just to code standards deteriorates rather than improves, and the designer who does not understand the true level of earthquake response is given a false sense of security by the increasing refinement of his structural calculations. Many engineers, of course, recognize this difficulty with the code and perform dynamic studies of the earthquake response of proposed structures or in other ways provide resistance significantly in excess of code requirements. Such extra precautions are not standard practice, however, and it is obviously undesirable to have large discrepancies between code values of earthquake forces and actual measured values.

We conclude that the building code should be revised to reflect more realistically the actual accelerations, deformations and strengths of structures subjected to strong ground shaking. This is particularly important for major structures. The revisions should require first an increase to realistic levels of the simplified loadings used by the code to simulate earthquake response of structures. For example, both equivalent static loading for relatively rigid structures as well as dynamic loadings for more flexible structures could be specified by smoothed acceleration spectra based on recorded ground motions. Secondly, the allowable stresses for earthquake response should be based on more realistic estimates of actual properties of materials under dynamic loadings than is now the case. This would, in general, involve increases in the allowable stresses for earthquake response of some of the more familiar construction materials. Of the two recommmended increases, the first in the forces and the second in the allowable stresses,the first should be the larger in order to increase the earthquake resistance of minimum structures. If realistic estimates of the earthquake input and the material properties are embodied in the code, the deformations which are calculated for the structure will be more realistic estimates of those which might actually occur in strong shaking and, as a result, separation joints, mechanical and electrical equipment, and architectural components of the building could be designed on a more rational basis. Even if the stress levels indicated by elastic analysis exceed the yield limits of the structural members, the indicated deflections are a useful guide to the actual deformations.

The present code places heavy emphasis on the role of ductility in preventing collapse even in the event of very strong shaking. This is a good feature of the code and should be retained.

There were no instances where it appeared to the writers that vertical accelerations had been major contributors to the damage sustained by building structures, and it is not recommended at this time that vertical acceleration be included in normal seismic design procedures for typical buildings. There are special structures, however, which could be more sensitive to vertical accelerations and vertical motions should be considered in such cases.

Although it is probably expedient to retain the equivalent static method of specifying forces for typical one- or two-story structures, it seems prudent to require sufficiently high static loadings in the code for multistory structures that it is to the economic advantage of building owners to have realistic dynamic analyses made in the design of most multistory structures. *Such dynamic analyses should be required for all major structures.* Some of the tall, steel-frame buildings in Los Angeles had been designed with the aid of dynamic studies using realistic earthquake inputs and digital computer analyses. After the earthquake, dynamic analyses were again made using the San Fernando earthquake motion recorded in the basements of the buildings and good agreement was obtained with the motions recorded in the upper parts of the buildings. The computed forces in the members were also consistent with those used in the design. This is a confirmation of the reliability of dynamic analyses.

To achieve the necessary extra resistance required for special structures such as hospitals, fire stations, etc., as noted above, it is recommended that an importance or occupancy factor be included in the building codes as is the practice in some countries of the world. Such a factor should be applied to the stress or deformation levels at which earthquake motions are to be resisted rather than to the earthquake motions themselves. This approach is suggested because it is thought preferable to first determine the level of earthquake excitation and then to specify which structures should respond with nonhazardous damage and which should be able to withstand the shaking without loss of essential function, i.e., without significant damage.

In addition to the level of earthquake resistance of the structural frame, there are other features of the codes and standard practices that stand in need of improvement. Foremost among these are the codes and practices for architectural and nonstructural elements of buildings. As has been the case in many other earthquakes, ceilings, light fixtures, elevators, mechanical equipment and partitions often failed and would have caused many injuries had the earthquake occurred during working hours.

Although the typical one-story, wood-frame house again demonstrated its earthquake resistance, there was enough vibratory damage and collapse to newer, two-story and split-level wood-frame construction to indicate that present building codes and practice are not always sufficient to provide earthquake resistance comparable to that for single-story construction. Standard building practices and the applicable codes should be studied and the necessary improvements made. This is a logical duty of city building departments as they have the responsibility for issuing building permits for new construction and for condemning unsafe structures, as was done after the San Fernando earthquake.

Closure

Although a disaster to many, the San Fernando earthquake has provided a unique opportunity to learn about the effects of damaging earthquakes. In effect, the earthquake was a large, uncontrolled experiment which it is the responsibility of engineers in practice and research to study and understnad. Profitable research on the San Fernando earthquake can be done for a number of years and responsible government agencies should insure that funding at appropriate levels is provided. The earthquake has also given another warning of the disaster potential of a great earthquake, or a moderate earthquake occurring near the center of a large city, at an unfortunate time of day. Such earthquakes seem inevitable eventually, and the San Fernando earthquake can help to clarify what must be done to minimize future earthquake disasters.

APPENDIX I - TYPICAL PEAK ACCELERATIONS IN MULTI-STORY BUILDINGS

No.	Building	Date	No. Stories	Ground, Basement or 1st Floor L	T	V	Intermediate Level L	T	V	Roof or Top Floor L	T	V
				Accelerations in Fractions of g								
Reinforced Concrete												
1	15107 Vanowen	1970	7	0.11	0.11	0.11	0.24	0.23	0.20	0.36	0.40	0.17
2	8244 Orion	1967	7	0.25	0.15	0.19	0.21	0.25	0.24	0.40	0.34	0.26
3	1640 Marengo	1966	7	0.14	0.15	0.09	0.15	0.27	0.13	0.25	0.44	0.15
4	4680 Wilshire	1967	7	0.14	0.10	0.09	0.23	0.19	0.12	0.24	0.30	0.15
5	646 Olive*	1967	7	0.22	0.26	0.09	0.25	0.25	0.13	0.39	0.48	0.26
6	4687 Sunset	1966	8	0.20	0.18	0.15	0.31	0.24	0.15	0.45	0.47	0.22
7	2011 Zonal	1966	9	0.08	0.08	0.07	0.20	0.17	0.10	0.23	0.21	0.11
8	433 Oakhurst	1970	10	0.09	0.06	0.03	0.14	0.14	0.04	0.27	0.27	0.10
9	120 Robertson	1966	10	0.10	0.10	0.04	0.18	0.19	0.10	0.33	0.28	0.12
10	420 Roxbury	1969	10	0.21	0.17	0.05	0.21	0.24	0.11	0.30	0.22	0.14
11	7080 Hollywood	1966	11	0.11	0.11	0.08	0.21	0.13	0.16	0.21	0.13	0.22
12	3710 Wilshire*	1966	11	0.17	0.16	0.09	0.29	0.17	0.11	0.22	0.38	0.17
13	3470 Wilshire	1966	12	0.15	0.12	0.06	0.21	0.22	0.11	0.22	0.25	0.15
14	8639 Lincoln	1969	12	0.04	0.03	0.03	0.09	0.08	0.09	0.13	0.13	0.06
15	15250 Ventura	1971	12	0.17	0.24	0.11	0.25	0.28	0.16	0.18	0.30	0.18
16	2500 Wilshire	1969	13	0.11	0.13	0.06	0.14	0.16	0.07	0.20	0.20	0.15
17	6200 Wilshire	1970	16	0.12	0.13	0.03	0.29	0.17	0.05	0.28	0.26	0.08
18	4000 Chapman	1970	19	0.02	0.02	0.02	0.05	0.04	0.03	0.06	0.06	0.04
Steel Frame												
1	5260 Century*	1968	7	0.06	0.05	0.02	0.05	0.07	0.04	0.07	0.05	0.09
2	3407 Sixth	1966	8	0.17	0.20	0.06	0.22	0.22	0.10	0.28	0.22	0.27
3	1150 Hill	1970	10	0.12	0.09	0.05	0.10	0.12	0.09	0.12	0.12	0.15
4	900 Fremont	1971	12	0.14	0.14	0.07	0.14	0.15	0.12	0.18	0.15	0.17
5	L.A. Water and Power	1969	15	0.15	0.20	0.08	0.19	0.14	0.09	0.17	0.13	0.16
6	250 First	1967	15	0.10	0.14	0.06	0.21	0.17	0.09	0.17	0.18	0.20
7	1800 Century Park East	1970	16	0.08	0.11	0.08	0.23	0.25	0.16	0.28	0.28	0.33
8	800 First	1969	33	0.09	0.14	0.06	0.13	0.19	0.17	0.17	0.27	0.25

* Shear Wall

APPENDIX II

MAXIMUM ACCELERATIONS RECORDED DURING THE SAN FERNANDO EARTHQUAKE*

The two horizontal components are marked L and T, and the vertical component is marked V.

	Accelerograph Location	Maximum Acceleration (g)		Accelerograph Location	Maximum Acceleration (g)
1.	Alhambra 900 S. Freemont Bsmt.	.127L .108T .093V	10.	Carbon Canyon Dam	.073L .068T .042V
2.	Alhambra 900 S. Fremont 6th Floor	.145L .142T .114V	11.	Castaic Old Ridge Route	.388L .316T .178V
3.	Alhambra 900 S. Fremont 12th Floor	.172L .151T .177V	12.	Cedar Springs Pump Plant	.015L .018T .008V
4.	Arcadia Santa Anita Reservoir	.179L .236T .068V	13.	Costa Mesa 666 W 19th St. Ground Floor	.023L .035T .008V
5.	Beverly Hills 420 N. Roxbury Dr. 1st Floor	.196L .172T .037V	14.	Colton Edison Co.	.035L .044T .026V
6.	Beverly Hills 420 N. Roxbury Dr. 5th Floor	.215L .212T .102V	15.	Fairmont Station Fairmont Reservoir	.167L .153T .081V
7.	Beverly Hills 420 N. Roxbury Dr. 10th Floor	.297L .215T .119V	16.	Fullerton 2600 Nutwood Ave. Bsmt	.035L .035T .017V
8.	Beverly Hills 9100 Wilshire Blvd. Bsmt.	.161L .123T .037V	17.	Fullerton 2600 Nutwood Ave. Penthouse (west wing)	.108L .131T .041V
9.	Beverly Hills 9100 Wilshire Blvd. 5th Floor	.132L .147T .078V	18.	Fullerton 2600 Nutwood Ave. Penthouse(center)	.092L .147T .022V

*These data were taken from the report "Preliminary Approximate Maximum Accelerations, San Fernando, California, Earthquake of 9 February 1971," National Ocean Survey — NOAA, Seismological Field Survey, 13 March 1971.

Accelerograph Location	Maximum Acceleration (g)	Accelerograph Location	Maximum Acceleration (g)
19. Gorman OSO Pumping plant	.092L .094T .035V	32. Los Angeles 9841 Airport Blvd. Bsmt	.030L .029T .012V
20. Grapevine Tehachapi Pumping Plant	.046L .066T .020V	33. Los Angeles 9841 Airport Blvd. 15th Floor	.100L .093T .050V
21. Hollywood 1760 N. Orchid Ground Floor	.163L .131T .072V	34. Los Angeles 1900 Ave. of Stars 29th Floor	.149L .117T .354V
22. Hollywood 1760 N. Orchid 12th Floor	.141L .078T .141V	35. Los Angeles 1900 Ave. of Stars Bsmt	.083L .095T .057V
23. Hollywood 1760 N. Orchid 23rd Floor	.197L .110T .194V	36. Los Angeles 1901 Ave. of Stars Bsmt	.118L .171T .066V
24. Long Beach Terminal Is.	.031L .027T .019V	37. Los Angeles 1901 Ave. of Stars 9th Floor	.184L .105T .145V
25. Long Beach Utilities Bldg.	.029L .022T .022V	38. Los Angeles 1901 Ave. of Stars 21st Floor	.142L .066T .092V
26. Lake Hughes Array No. 7 Fire Station No. 78	.167L .121T .115V	39. Los Angeles 1800 Century Park East - Bsmt	.080L .100T .066V
27. Lake Hughes Array Station 4	.158L .194T .163V	40. Los Angeles 1800 Century Park East - 5th Floor	.210L .220T .157V
28. Lake Hughes-Warm Springs 9	.152L .158T .118V	41. Los Angeles 1800 Century Park East - Penthouse	.283L .284T .307V
29. Lake Hughes Array No. 12	.371L .276T .178V	42. Los Angeles 1177 Beverly Dr. Bsmt	.123L .114T .067V
30. Maricopa No. 3	.010L .010T .005V	43. Los Angeles Century City Ground	No record
31. Maricopa No. 4	.010L .010T .015V	44. Los Angeles 1880 Century Park East - 1st Parking level	.109L .126T .067V

Accelerograph Location	Maximum Acceleration (g)
45. Los Angeles 1880 Century Park East - 7th Floor	.097L .144T .120V
46. Los Angeles 1880 Century Park East - Penthouse	.102L .124T .274V
47. Los Angeles 1888 Century Park East - 14th Floor	.144L .059T .189V
48. Los Angeles 1888 Century Park East - 21st Floor	.149L .084T .352V
49. Los Angeles 1888 Century Park East - 5th Floor Parking Ramp	.176L .123T .094V
50. Los Angeles 1888 Century Park East - 9th Floor Parking Ramp	.375L .312T .110V
51. Los Angeles 2080 Century Park East - Roof	.180L .357T .234V
52. Los Angeles 5260 Century 1st Floor	.055L .056T .017V
53. Los Angeles 5260 Century 4th Floor	.042L .070T .036V
54. Los Angeles 5260 Century Roof	.089L .056T .081V
55. Los Angeles 234 Figueroa Bsmt	.173L .197T .057V
56. Los Angeles 234 Figueroa Roof	.435L .500T .168V

Accelerograph Location	Maximum Acceleration(g)
57. Los Angeles 222 Figueroa 1st Floor (See No.193)	.119L .150T .043V
58. Los Angeles 445 Figueroa Sub-bsmt	.144L .134T .060V
59. Los Angeles 445 Figueroa 19th Floor	.210L .130T .115V
60. Los Angeles 250 E. First Bsmt	.092L .132T .04V
61. Los Angeles 250 E First 8th Floor	.21L .17T .07V
62. Los Angeles 250 E. First Roof	.158L .184T .21V
63. Los Angeles 800 W. First 1st Floor	.094L .143T .059V
64. Los Angeles 800 W. First 16th Floor	.121L .182T .158V
65. Los Angeles 800 W. First 33rd Floor	.186L .294T .224V
66. Los Angeles 533 S. Fremont Bsmt	.246L .224T .083V
67. Los Angeles 533 S. Fremont 6th Floor	.343L .306T .164V
68. Los Angeles 750 Garland 2nd Floor	.221L .158T .100V
69. Los Angeles 750 Garland 6th Floor	.305L .232T .147V

Accelerograph Location	Maximum Acceleration (g)
70. Los Angeles Griffith Obsv. Moon Room	.184L .163T .121V
71. Los Angeles 420 S. Grand 2nd Floor	.121L .168T .068V
72. Los Angeles 420 S. Grand 17th Floor	.226L .321T .226V
73. Los Angeles 1150 S. Hill St. Sub-bsmt	.121L .090T .047V
74. Los Angeles 1150 S. Hill St. 5th Floor	.108L .102T .089V
75. Los Angeles 1150 S. Hill St. 10th Floor	.143L .110T .152V
76. Los Angeles 930 Hilgard 15th Floor	.149L .162T .197V
77. Los Angeles Hollywood Storage Bsmt	.154L .113T .061V
78. Los Angeles Hollywood Storage Penthouse	Malfunctioned
79. Los Angeles Hollywood Storage P. E. Lot	.218L .192T .121V
80. Los Angeles 646 S. Olive St. Bsmt	.221L .254T .081V
81. Los Angeles 646 S. Olive St. 4th Level	.253L .256T .121V
82. Los Angeles 646 S. Olive St. Roof	.380L .483T .263V
83. Los Angeles 3838 Lankershim Bsmt	.175L .127T .087V
84. Los Angeles 3838 Lankershim 21st Floor	.105L .212T .226V
85. Los Angeles 8639 Lincoln Bsmt	.037L .037T .042V
86. Los Angeles 8639 Lincoln 6th Floor	.095L .100T .052V
87. Los Angeles 8639 Lincoln 12th Floor	.121L .121T .058V
88. Los Angeles 1640 Marengo Ground	.079L .138T .142V
89. Los Angeles 1640 Marengo 4th Floor	.118L .264T .197V
90. Los Angeles 1640 Marengo 8th Floor	.132L .435T .237V
91. Los Angeles 7080 Hollywood Blvd. Bsmt	.109L .098T .063V
92. Los Angeles 7080 Hollywood Blvd. 6th Floor	.193L .123T .156V
93. Los Angeles 7980 Hollywood Blvd. 12th Floor	.213L .121T .224V
94. Los Angeles 616 S. Normandie Bsmt	.105L .114T .046V
95. Los Angeles 616 S. Normandie 8th Floor	.216L .142T .105V

Accelerograph Location	Maximum Acceleration (g)
96. Los Angeles 616 S. Normandie Roof	.306L .227T .198V
97. Los Angeles 435 Oakhurst Bsmt	.060L .088T .038V
98. Los Angeles 435 Oakhurst 5th Floor	.129L .137T .047V
99. Los Angeles 435 Oakhurst Roof	.240L .250T .102V
100. Los Angeles 808 S. Olive St. 4th Level	.263L .162T .191V
101. Los Angeles 808 S. Olive St. 8th Level	.250L .435T .237V
102. Los Angeles 1625 Olympic Ground Floor	.140L .266T .158V
103. Los Angeles 1625 Olympic 6th Floor	.181L .225T .139V
104. Los Angeles 1625 Olympic 10th Floor	.230L .277T .226V
105. Los Angeles 8244 Orion Ground	.276L .145T .171V
106. Los Angeles 8244 Orion 8th Floor - Roof	.388L .310T .224V
107. Los Angeles 611 W. 6th St. 42nd Floor	.110L .180T .110V
108. Los Angeles 611 W. 6th St. Bsmt	.097L .110T .065V
109. Los Angeles 11661 San Vicente 5th Floor	.091L .082T .113V
110. Los Angeles 11661 San Vicente Roof	.103L .093T .157V
111. Los Angeles 6464 Sunset Blvd. Bsmt	.125L .107T .075V
112. Los Angeles 6464 Sunset Blvd. 12th Floor	.271L .235T .287V
113. Los Angeles 4867 Sunset Blvd. Bsmt	.167L .170T .134V
114. Los Angeles 4867 Sunset Blvd. 2nd Floor	.296L .222T .131V
115. Los Angeles 4867 Sunset Blvd. 7th Floor	.45L .459T .196V
116. Los Angeles Water & Power Bsmt	.138L .202T .080V
117. Los Angeles Water & Power 7th Floor	.171L .131T .097V
118. Los Angeles Water & Power 15th Floor	.164L .118T .171V
119. Los Angeles 3440 University Bsmt	.080L .064T .052V

Accelerograph Location	Maximum Acceleration (g)	Accelerograph Location	Maximum Acceleration (g)
120. Los Angeles 3440 University 5th Floor	.129L .140T .083V	133. Los Angeles 15910 Ventura 19th Floor	.224L .227T .210V
121. Los Angeles 3440 University Roof	.235L .260T .088V	134. Los Angeles 945 Tiverton 8th Floor	.123L .226T .105V
122. Los Angeles 15107 Vanowen Bsmt	.107L .120T .116V	135. Los Angeles 945 Tiverton 14th Floor	.143L .181T .146V
123. Los Angeles 15107 Vanowen 4th Floor	.227L .260T .194V	136. Los Angeles 6200 Wilshire Ground Floor	.134L .133T .038V
124. Los Angeles 15107 Vanowen Roof	.341L .385T .173V	137. Los Angeles 6200 Wilshire 10th Floor	.280L .147T .068V
125. Los Angeles 14724 Ventura 1st Floor	.356L .274T .107V	138. Los Angeles 6200 Wilshire 17th Floor	.300L .261T .074V
126. Los Angeles 14724 Ventura Penthouse	.315L .208T	139. Los Angeles 5900 Wilshire Penthouse	.140L .170T .152V
127. Los Angeles 15433 Ventura 7th Floor	.242L .170T .153V	140. Los Angeles 5900 Wilshire 16th Floor	.097L .121T .084V
128. Los Angeles 15433 Ventura 13th Floor	.268L .226T .029V	141. Los Angeles 5900 Wilshire B Parking Lot	.066L .073T .029V
129. Los Angeles 15250 Ventura Bsmt	.227L .140T .10V	142. Los Angeles 6430 Sunset 1st Floor	.191L .143T .087V
130. Los Angeles 15250 Ventura 7th Floor	.261L .176T .131V	143. Los Angeles 3345 Wilshire Bsmt	.121L .094T .069V
131. Los Angeles 15910 Ventura Bsmt	.130L .153T .112V	144. Los Angeles 3345 Wilshire 2nd Floor	.167L .113T
132. Los Angeles 15910 Ventura 9th Floor	.178L .129T .221V	145. Los Angeles 3345 Wilshire 12th Floor	.206L .250T .124V

Accelerograph Location	Maximum Acceleration (g)	Accelerograph Location	Maximum Acceleration (g)
146. Los Angeles 120 N. Robertson Bsmt	.090L .093T .031V	159. Los Angeles 3710 Wilshire Bsmt	.167L .159T .081V
147. Los Angeles 120 N. Robertson 4th Floor	.177L .179T	160. Los Angeles 3710 Wilshire 5th Floor	.272L .161T .097V
148. Los Angeles 120 N. Robertson 9th Floor	.327L .276T .118V	161. Los Angeles 3710 Wilshire 10th Floor	.225L .368T .171V
149. Glendale 633 E. Broadway Municipal Service Bldg.	.275L .233T .142V	162. Los Angeles 2011 Zonal Bsmt	.079L .068T .060V
150. Los Angeles 3470 Wilshire Sub-bsmt	.147L .118T .052V	163. Los Angeles 2011 Zonal 5th Floor	.158L .178T .081V
151. Los Angeles 3470 Wilshire 5th Floor	.243L .210T .104V	164. Los Angeles 2011 Zonal 9th Floor	.200L .209T .115V
152. Los Angeles 3470 Wilshire 11th Floor	.222L .226T .156V	165. Orange 4000 W. Chapman Ave. Bsmt	.021L .021T .011V
153. Los Angeles UCLA Reactor Lb.	.104L .091T .074V	166. Orange 4000 W. Chapman Ave. 10th Floor	.051L .042T .021V
154. Los Angeles 3411 Wilshire 5th Bsmt	.140L .113T .065V	167. Orange 4000 W. Chapman Ave. 19th Floor	.081L .061T .042V
155. Los Angeles 3550 Wilshire Bsmt	.125L .177T .059V	168. Los Angeles 3407 Sixth Street Bsmt	.169L .188T .064V
156. Los Angeles 4680 Wilshire Bsmt	.121L .089T .081V	170. Los Angeles 3407 Sixth St. 4th Floor	.208L .210T .101V
157. Los Angeles 4680 Wilshire 3rd Floor	.221L .181T .130V	171. Los Angeles 3407 Sixth St. Penthouse	.288L .208T .260V
158. Los Angeles 4680 Wilshire 6th Floor	.243L .302T .160V	172. Los Angeles 2500 Wilshire Bsmt	.097L .100T .043V

Accelerograph Location	Maximum Acceleration (g)	Accelerograph Location	Maximum Acceleration (g)
173. Los Angeles 2500 Wilshire 8th Floor	.127L .157T .070V	185. Pasadena Millikan Library 10th Floor	.337L .332T .137V
174. Los Angeles 2500 Wilshire Roof	.203L .191T .139V	186. Pasadena JPL Bsmt	.165L .212T .132V
175. San Dimas Puddingstone Reservoir	.088L .052T .048V	187. Pasadena JPL 9th Floor Roof	.212L .379T .265V
176. San Juan Capistrano	.035L .026T .021V	188. Port Hueneme Navy Lab	.030L .025T .010V
177. San Bernardino Hall of Records	.047L .042T .016V	189. Pearblossom Pearblossom Pumping Plant	.104L .148T .063V
178. San Onofre SCE Nuclear Power Plant	.010L .018T .010V	190. Santa Ana Orange County Engineering Bldg.	.030L .030T .019V
179. Vernon CDM Building	.088L .113T .047V	191. Santa Barbara Univ. of Calif.	.016L .011T .011V
180. Wheeler Ridge	.021L .026T .013V	192. Santa Felicia Dam Crest	.200L .170T .060V
181. Palmdale Fire Station Storage Room	.110L .131T .084V	193. Los Angeles 222 Figueroa 20th Floor	.309L .395T .086V
182. Palos Verdes Estates 2516 Via Tejon	.039L .021T .010V	194. Pasadena Caltech Seism. Lab Bsmt	.18L .08T .07V
183. Pasadena Caltech Athenaeum	.100L .113T .102V	195. Pacoima Dam Abutment	1.25L (approx) 1.25T (approx) 0.70V
184. Pasadena Millikan Library Bsmt	.184L .216T .116V		

APPENDIX III

Los Angeles County 1968 Building Code Earthquake Regulations

SEC. 2314 — EARTHQUAKE REGULATIONS

(a) General. Every building or structure and every portion thereof shall be designed and constructed to resist stresses produced by lateral forces as provided in this Section. Stresses shall be calculated as the effect of a force applied horizontally at each floor or roof level above the foundation. The force shall be assumed to come from any horizontal direction.

The provisions of this Section apply to the structure as a unit and also to all parts thereof, including the structural frame or walls, floor and roof systems, and other structural features.

(b) Definitions. The following definitions apply only to the provisions of this Section.

SPACE FRAME is a three-dimensional structural system composed of interconnected members, other than ➤ bearing walls, laterally supported so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems. ◄

➤ **SPACE FRAME-MOMENT RESISTING** is a vertical load carrying space frame in which the members and joints are capable of resisting design lateral forces by bending moments. ◄

➤ **SPACE FRAME-DUCTILE MOMENT RESISTING** is a space frame-moment resisting complying with the requirements for a ductile moment resisting space frame as given in Section 2314 (j). ◄

➤ **LATERAL FORCE RESISTING SYSTEM** is that part of the structural system to which the lateral forces prescribed in Section 2314 (d) 1 are assigned. ◄

SPACE FRAME — VERTICAL LOAD-CARRYING is a space frame designed to carry all vertical loads.

BOX SYSTEM is a structural system without a complete vertical load-carrying space frame. In this system the required lateral forces are resisted by shear walls as hereinafter defined.

SHEAR WALL is a wall designed to resist lateral forces parallel to the wall. Braced frames subjected primarily to axial stresses shall be considered as shear walls for the purpose of this definition.

(c) **Symbols and Notations.** The following symbols and notations apply only to the provisions of this Section.

C = Numerical coefficient for base shear as specified in Section 2314 (d) 1.

C_p = Numerical coefficient as specified in Section 2314 (d) 2 and as set forth in Table No. 23-I.

D = The dimension of the building in feet in a direction parallel to the applied forces.

D_s = The plan dimension of the vertical lateral force resisting system in feet.

F_i, F_n, F_x = Lateral forces applied to a level "i," "n," or "x," respectively.

F_p = Lateral forces on the part of the structure and in the direction under consideration.

F_t = That portion of "V" considered concentrated at the top of the structure, at the level "n." The remaining portion of the total base shear "V" shall be distributed over the height of the structure including level "n" according to Formula (14-5).

H = **The height of the main portion of the building in feet above the base.**

h_i, h_n, h_x = Height in feet above the base to level "i," "n," or "x," respectively.

K = Numerical coefficient for base moment as specified in Section 2314 (h).

J_x = Numerical coefficient for overturning moment at level "x."

J = Numerical coefficient as set forth in Table No. 23-H.

Level i = Level of the structure referred to by the subscript "i."

Level n = That level which is uppermost in the main portion of the structure.

Level x = That level which is under design consideration.

M = Overturning moment at the base of the building or structure.

M_x = The overturning moment at level "x."

N = Total number of stories above exterior grade.

T = Fundamental period of vibration of the building or structure in seconds in the direction under consideration.

V = Total lateral load or shear at the base.

$$V = F_t + \sum_{i=1}^{n} F_i$$

where $i = 1$ designates first level above the base.

W = Total dead load including partitions using the actual weight of the partitions or the partition loading specified in Section 2302 (b)

$$W = \sum_{i=1}^{n} w_i$$

EXCEPTION: "W" shall be equal to the total dead load plus 25 per cent of the floor live load in storage and warehouse occupancies.

w_i, w_x = That portion of "W" which is located at or is assigned to level "i" or "x" respectively.

W_p = The weight of a part or portion of a structure.

Z = Numerical coefficient equal to one.

(d) Minimum Earthquake Forces for Structures: 1. Total lateral force and distribution of lateral force. Every structure shall be designed and constructed to withstand minimum total lateral seismic forces assumed to act nonconcurrently in the direction of each of the main axes of the structures in accordance with the following formula (For forces on parts or portions of buildings and for forces on structures other than buildings, see paragraph 2 of this Subsection):

$$V = ZKCW \qquad (14\text{-}1)$$

The value of "K" shall be not less than that set forth in Table No. 23-H. The value of "C" shall be determined in accordance with the following formula:

$$C = \frac{0.05}{\sqrt[3]{T}} \qquad (14\text{-}2)$$

Except as provided in Table No. 23-I, the maximum value of "C" need not exceed 0.10. For all one- and two-story buildings the value of "C" shall be considered as 0.10.

"T" is the fundamental period of vibration of the structure in seconds in the direction under consideration. Properly substantiated technical data for establishing the period "T" may be submitted. In the absence of such data, the value of "T" for buildings shall be determined by the following formula:

$$T = \frac{0.05 h_n}{\sqrt{D}} \qquad (14\text{-}3)$$

EXCEPTION: In all buildings in which the lateral resisting system consists of a moment-resisting space frame which resists 100 per cent of the required lateral forces and which frame is not enclosed by or adjoined by more rigid elements which would tend to prevent the frame from resisting lateral forces:

$$T = 0.10N \qquad (14\text{-}3A)$$

TABLE NO. 23-H — HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES[1]

TYPE OR ARRANGEMENT OF RESISTING ELEMENTS	VALUE[2] OF K
All building framing systems except as hereinafter classified	1.00
Buildings with a box system as specified in Section 2314 (b)	1.33
Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls using the following design criteria: (1) The frames and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames (2) The shear walls acting independently of the ductile moment resisting portions of the space frame shall resist the total required lateral forces (3) The ductile moment resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force	0.80
Buildings with a ductile moment-resisting space frame designed in accordance with the following criteria: (1) The ductile moment-resisting space frame shall have the capacity to resist the total required lateral force (2) If major rigid elements are included in addition to the ductile moment-resisting space frame, the total required lateral force shall be distributed to all resisting elements in accordance with their relative rigidities considering the interaction of the frames and rigid elements	0.67
Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building[3, 4]	3.00[5]
Structures other than buildings and other than those set forth in Table No. 23-I	2.00

[1]Where wind load as specified in Section 2307 would produce higher stresses, this load shall be used in lieu of the loads resulting from earthquake forces.

[2]Footnote No. 2 is deleted.

[3]The minimum value of "KC" shall be 0.12 and the maximum value of "KC" need not exceed 0.25.

[4]For overturning, the factor "J" as specified in Section 2314 (h) shall be 1.00.

[5]The tower shall be designed for an accidental torsion of five per cent as specified in Section 2314 (g). Elevated tanks which are supported by buildings or do not conform to type or arrangement of supporting elements as described above shall be designed in accordance with Section 2314 (d) 2 using "C_p" = .2.

The total lateral force "V" shall be distributed in the height of the structure in the following manner:

$$F_t = .004V \left(\frac{h_n}{D_s} \right)^2 \qquad (14\text{-}4)$$

F_t need not exceed 0.15 "V" and may be considered as 0 for values $\left(\frac{h_n}{D_s} \right)$ of 3 or less, and

$$F_x = \frac{(V - F_t)\, w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (14\text{-}5)$$

EXCEPTION: One-and two-story buildings shall have uniform distribution.

At each level designated as "x," the force "F_x" shall be applied over the area of the building in accordance with the mass distribution on that level.

2. **Lateral force on parts or portions of buildings or other structures.** Parts or portions of buildings or structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = ZC_p W_p \qquad (14\text{-}6)$$

The values of "C_p" are set forth in Table No. 23-I. The distribution of these forces shall be according to the gravity loads pertaining thereto.

3. **Pile foundations.** Individual pile or caisson footings of every building or structure shall be interconnected by ties each of which can carry by tension and compression a horizontal force equal to 10 per cent of the larger pile cap loading unless it can be demonstrated that equivalent restraint can be provided by other approved methods.

EXCEPTION: Ties may be omitted for belled footings having a height not exceeding six feet nor twice the diameter of the bell and for piles supporting one-story buildings of lightweight Type IV-N construction.

(e) **Distribution of Horizontal Shear.** Total shear in any horizontal plane shall be distributed to the various elements of the lateral force resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm.

Rigid elements that are assumed not to be part of the lateral force resisting system may be incorporated into buildings provided that their effect on the action of the system is considered and provided for in the design.

(f) **Drift.** Lateral deflections or drift of a story relative to its adjacent stories shall be considered in accordance with accepted engineering practice.

(g) **Horizontal Torsional Moments.** Provisions shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the center of mass and the center of rigidity. Negative torsional shears shall be neglected. Where the vertical resisting elements depend on diaphragm action for shear distribution at any level, the shear-resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than five per cent of the maximum building dimension at that level.

TABLE NO. 23-I — HORIZONTAL FORCE FACTOR "C_p" FOR PARTS OF PORTIONS OF BUILDINGS OR OTHER STRUCTURES

PART OR PORTION OF BUILDINGS	DIRECTION OF FORCE	VALUE OF C_p
Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions over 10 feet in height, masonry or concrete fences over six feet in height[1]	Normal to flat surface	0.20
Cantilever parapet and other cantilever walls, except retaining walls	Normal to flat surface	1.00
Exterior and interior ornamentations and appendages	Any direction	1.00
When connected to or a part of a building: towers, tanks, towers and tanks plus contents, chimneys, smokestacks, and penthouses	Any direction	0.20[2]
When resting on the ground, tank plus effective mass of its contents	Any direction	0.10
Floors and roofs acting as diaphragms[3]	Any direction	0.10
Connections for exterior panels or for elements complying with Section 2314 (k) 5	Any direction	2.00
Connections for prefabricated structural elements other than walls, with force applied at center of gravity of assembly[4]	Any horizontal direction	0.30

[1]See also Section 2312 (b) for minimum load on deflection criteria for interior partitions.

[2]When "h_n/D" of any building is equal to or greater than five to one increase value by 50 per cent.

[3]Floors and roofs acting as diaphragms shall be designed for a minimum value of "C_p" of 10 per cent applied to loads tributary from that story unless a greater value of "C_p" is required by the basic seismic formula $V = ZKCW$.

[4]The "W_p" shall be equal to the total load plus 25 per cent of the floor live load in storage and warehouse occupancies.

(h) **Overturning.** Every building or structure shall be designed to resist the overturning effects caused by the wind forces and related requirements specified in Section 2308, or the earthquake forces specified in this Section, whichever governs.

EXCEPTION: The axial loads from earthquake forces on vertical elements and footings in every building or structure may be modified in accordance with the following provisions:

1. The overturning moment, "M," at the base of the building or structure shall be determined in accordance with the following formula:

$$M = J\left(F_t h_n + \sum_{i=1}^{n} F_i h_i\right) \qquad (14\text{-}7)$$

WHERE:

$$J = \frac{0.5}{\sqrt[3]{T^2}} \qquad (14\text{-}8)$$

The value of "J" need not be more than 1.00.

2. The overturning moment, "M_x", at any level designated as "x" shall be determined in accordance with the following:

$$M_x = J_x\left[F_t(h_n - h_x) + \sum_{i=x}^{n} F_i (h_i - h_x)\right] \qquad (14\text{-}9)$$

WHERE:

$$J_x = J + (1 - J)\left(\frac{h_x}{h_n}\right)^3 \qquad (14\text{-}10)$$

At any level the incremental changes of the design overturning moment, in the story under consideration, shall be distributed to the various resisting elements in the same proportion as the distribution of the shears in the resisting system. Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided.

Where a vertical resisting element is discontinuous, the overturning moment carried by the lowest story of that element shall be carried down as loads to the foundation.

(i) **Set-backs.** Buildings having set-backs wherein the plan dimension of the tower in each direction is at least 75 per cent of the corresponding plan dimension of the lower part may be considered as a uniform building without set-backs for the purpose of determining seismic forces.

For other conditions of set-backs the tower shall be designed as a separate building using the larger of the seismic coefficients at the base of the tower determined by considering the tower as either a separate building for its own height or as part of the over-all structure. The resulting total shear from the tower shall be applied at the top of the lower part of the building which shall be otherwise considered separately for its own height.

(j) **Structural Systems. 1. Design Requirements.** Buildings designed with a horizontal force factor "*K*" of 0.67 or 0.80 shall have a ductile moment-resisting space frame. Buildings more than one hundred and sixty feet in height shall have a ductile moment-resisting space frame capable of resisting not less than 25 per cent of the required seismic load for the structure as a whole.

Moment-resisting space frames and ductile moment-resisting space frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces, where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load-resisting ability of the space frame.

2. **Construction.** The necessary ductility for a ductile moment-resisting space frame shall be provided by a frame of structural steel conforming to ASTM A-7, A-36 or A-441 with moment-resisting connections, or by a reinforced concrete frame complying with Section 2632 of this Code.

Shear walls in buildings exceeding one hundred and sixty feet in height shall be composed of axially loaded bracing members of ASTM A-7, A-36 or A-441 structural steel; or reinforced concrete bracing members or walls conforming with the requirements of Section 2632 of this Code.

(k) **Design Requirements. 1. Building separations.** All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflection from seismic action or wind forces. Structural separations of at least one-inch, plus one-half inch for each ten feet of height above twenty feet are considered adequate to meet the requirements of this paragraph.

2. **Minor alterations.** Minor structural alterations may be made in existing buildings and other structures, but the resistance to lateral forces shall be not less than that before such alterations were made, unless the building as altered meets the requirements of this Section of the Code.

3. **Reinforced masonry or concrete.** All elements within the structure which are of masonry or concrete and which resist seismic forces or movement shall be reinforced so as to qualify as reinforced masonry or concrete under the provisions of Chapters 24 and 26. Principal reinforcement in masonry shall be spaced two feet maximum on center in buildings using a ductile moment-resisting space frame.

4. **Combined vertical and horizontal forces.** In computing the effect of seismic force in combination with vertical loads, gravity load stresses induced in members by dead load plus design live load, except roof live load, shall be considered.

5. **Exterior elements.** Precast, nonbearing, non-shear wall panels or other elements which are attached to, or enclose the exterior, shall accommodate movements of the structure resulting from lateral forces or temperature changes. The concrete panels or other elements shall be supported by means of poured-in-place concrete or by mechanical fasteners in accordance with the following provisions:

A. Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused by wind or seismic forces; or one-fourth inch whichever is greater.

B. Connections shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failures at or near welds. Inserts in concrete shall be attached to, or hooked around reinforcing steel, or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

C. Connections to permit movement in the plane of the panel for story drift may be properly designed sliding connections using slotted or oversize holes or may be connections which permit movement by bending of steel.

6. **Minor rigid elements.** Minor rigid elements within or attached to a structure may be assumed to be expendable and not part of the lateral force resisting system.