BUILDING SCIENCE SERIES 40

# Engineering Aspects of the 1971 San Fernando Earthquake

UNITED STATES DEPARTMENT OF COMMERCE PUBLICATION

U.S. DEPARTMENT OF COMMERCE
National Bureau of Standards

## The Building Science Series

The Building Science Series disseminates technical information developed at the National Bureau of Standards on building materials, components, systems and whole structures. The Series presents research results, test methods and performance criteria related to the structural and environmental functions and the durability and safety characteristics of building elements and systems.

These publications, similar in style and content to the NBS Building Materials and Structures Reports (1938–59), are directed toward the manufacturing, design, construction and research segments of the building industry, standards organizations and officials responsible for building codes.

The material for this Series originates principally in the Building Research Division of the NBS Institute for Applied Technology. The publications are divided into three general groups: Building Systems and Processes; Health, Safety and Comfort; and Structures and Materials. Listed below are other publications in the category of—

## Structures and Materials

- Interrelations Between Cement and Concrete Properties: Part 1, Materials and Techniques, Water Requirements and Trace Elements. (C13.29/2:2) 35 cents
- Weather Resistance of Porcelain Enamels: Effect of Exposure Site and Other Variables After Seven Years. (C13.29/2:4) 20 cents
- Interrelations Between Cement and Concrete Properties: Part 2, Sulfate Expansion, Heat of Hydration, and Autoclave Expansion. (C13.29/2:5) 35 cents
- Some Properties of the Calcium Aluminoferrite Hydrates. (C13.29/2:6) 20 cents
- Organic Coatings, Properties, Selection, and Use (C13.29/2.7) $2.50
- Interrelations Between Cement and Concrete Properties: Part 3, Compressive Strengths of Portland Cement Test Mortars and Steam-Cured Mortars. (C13.29/2:8) 55 cents
- Thermal-Shock Resistance for Built-Up Membranes (C13.29/2:9) 20 cents
- Shrinkage and Creep in Prestressed Concrete. (C13.29/2:13) 15 cents
- Experimental Determination of Eccentricity of Floor Loads Applied to a Bearing Wall. (C13.29/2:14) 15 cents
- Interrelations Between Cement and Concrete Properties: Part 4, Shrinkage of Hardened Portland Cement Pastes. (C13.29/2:15) 75 cents
- Causes of Variation in Chemical Analyses and Physical Tests of Portland Cement. (C13.29/2:17) 40 cents
- A Study of the Variables Involved in the Saturating of Roofing Felts. (C13.29/2:19) 30 cents
- Proceedings of a Seminar on the Durability of Insulating Glass. (C13.29/2:20) 75 cents
- Hail Resistance of Roofing Products. (C13.29/2:23) 25 cents
- Natural Weathering of Mineral Stabilized Asphalt Coatings on Organic Felt. (C13.29/2:24) 30 cents
- Structural Performance Test of a Building System. ((C13.29/2:25) $1.25
- Exploratory Studies of Early Strength Development in Portland Cement Pastes and Mortars. (C13.29/2:28) 25 cents
- 1964 Exposure Test of Porcelain Enamels on Aluminum—Three Year Inspection. (C13.29/2:29) 25 cents
- Flexural Behavior of Prestressed Concrete Composite Tee—Beams (C13.29/2:31) 25 cents
- Compressive Strength of Slender Concrete Masonry Walls (C13.29/2:33) 40 cents
- Strength of Masonry Walls Under Compressive and Transverse Loads (C13.29/2:34) 70 cents

Send orders (use Superintendent of Documents Catalog Nos.) with remittance to:
Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402.
Remittance from foreign countries should include an additional one-fourth of the purchase price for postage.

---

[See mailing list announcement on last page.]

14

UNITED STATES DEPARTMENT OF COMMERCE • Maurice H. Stans, *Secretary*

NATIONAL BUREAU OF STANDARDS • Lewis M. Branscomb, *Director*

# Engineering Aspects of the 1971 San Fernando Earthquake

H. S. Lew, E. V. Leyendecker, and R. D. Dikkers

Building Research Division
Institute for Applied Technology
National Bureau of Standards
Washington, D.C. 20234

Building Science Series 40

Nat. Bur. Stand. (U.S.), Bldg. Sci. Ser. 40, 419 pages, (Dec. 1971)
CODEN: BSSNB

Issued December 1971

For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402
(Order by SD Catalog No. C 13.29/40), Price $3
Stock Number 0303-0940

Library of Congress Catalog Card Number: 70-186531

# PREFACE

On February 9, 1971, shortly after an earthquake struck the San Fernando, California area, the National Bureau of Standards was requested by the Office of Emergency Preparedness to send a team of engineers to the disaster area for the purposes of making observations and preparing reports relative to structural damages. A team of structural engineers from the Building Research Division, Institute for Applied Technology, National Bureau of Standards, was dispatched immediately to the disaster area. The NBS team, comprised of R. D. Dikkers, H. S. Lew, E. V. Leyendecker, and E. O. Pfrang, arrived on the scene on February 10. Professors J. E. Breen and L. C. Reese of the University of Texas at Austin joined the NBS team on February 11.

This report presents the observations of the NBS on-site inspection team (most photographs in this report were taken by the team). The material presented herein is intended to serve as (1) a documentation of damage resulting from the earthquake and (2) as a source document for further studies, research, and recommendations. This is particularly important as necessary remedial work and restoration have resulted in the removal of evidence that is essential for studies and evaluations.

The authors gratefully acknowledge those organizations and individuals who assisted in providing necessary information during and after the survey for the preparation of this report. In particular, the following individuals made direct contribution to the report and critically reviewed it:

Dr. John E. Breen, Professor of Civil Engineering
The University of Texas at Austin

Dr. William B. Joyner, U. S. Geological Survey
U. S. Department of the Interior

Dr. Lymon C. Reese, Professor of Civil Engineering
The University of Texas at Austin

Mr. Leon Stein, Supervising Structural Engineer
Office of Architecture & Construction
Department of General Services
State of California

Appreciation is also extended to the following individuals who provided valuable assistance to the NBS on-site survey team:

Office of Emergency Preparedness, Executive Office of The President

George Grace, Assistant Director for Field Operations, Washington, D. C.

Ralph D. Burns, Regional Director, Region 7, Santa Rosa, California

Terrence S. Mead, Disaster Assistant Coordinator, Santa Rosa, California

Department of Health, Education, and Welfare

Fred Zimmerman, Assistant Regional Director for Intergovernmental Relations, San Francisco, California

State of California

Michael J. Colby, Regional Manager, Region I and VI Office of Emergency Services, Los Angeles, California

Clifford Chaffin, Structural Engineer, Office of Architecture and Construction, Los Angeles, California

County of Los Angeles, California - Department of County Engineer

Coleman W. Jenkins, Superintendent of Buildings

John F. Lewis, Principal Structural Engineer

The authors also wish to express their appreciation to the numerous staff members of the National Bureau of Standards who participated in the preparation of this report. In particular, the staff of the Photographic Services Section who handled all photographic work and the staff of the Scientific and Professional Liaison Section who typed the manuscript, prepared graphic work, and provided other assistance in completing the report.

This study was funded by the Office of Emergency Preparedness (Executive Office of the President), the Office of Civil Defense (U. S. Department of Defense), and the National Bureau of Standards (U. S. Department of Commerce).

# SI Conversion Units

In recognition of the position of the USA as a signatory to the General Conference of Weights and Measures, which gave official status to the metric SI system of units in 1960, the authors assist readers interested in making use of the coherent system of SI units by giving conversion factors applicable to U. S. units used in this paper.

## Length

1 in = 0.0254* meter
1 ft = 0.3048* meter
1 mile = 1609.35 meter

## Area

1 $in^2$ = 6.4516* x $10^{-4}$ $meter^2$

1 $ft^2$ = 0.09290 $meter^2$

## Force

1 lb (lbf) = 4.448 newton
1 kip = 4448 newton

## Pressure, Stress

1 psi = 6895 newton/$meter^2$

1 ksi = 6.895 x $10^6$ newton/$meter^2$

## Mass/Volume

1 lb/$ft^3$ (lbm/$ft^3$) = 16.02 kilogram/$meter^3$

## Moment

1 kip-in = 113.0 newton-meter

---

* Exactly

TABLE OF CONTENTS

# ENGINEERING ASPECTS OF THE 1971 SAN FERNANDO EARTHQUAKE

Hai Sang Lew, Edgar V. Leyendecker, and Robert D. Dikkers*

*Immediately following the San Fernando, California earthquake (February 9, 1971), a four-man team from the Building Research Division of the National Bureau of Standards' Institute for Applied Technology, surveyed the damage to buildings and other structures. This report is based primarily on the data gathered during the survey but includes some data provided by other agencies and individuals. Based on study of these data and observations made during the survey, recommendations are made pertaining to the improvement of building and other structural design and construction practices.*

*Key Words: Bridge; building; codes; dams; earthquakes; earthquake damage; foundation geology; highways; housing; hospital; mobile home; seismic; standards; structural engineering.*

## CHAPTER 1 INTRODUCTION

### 1.1 Seismological Events

At 6:00:41.6 a.m. (Pacific Standard Time), on February 9, 1971, an earthquake with a magnitude of 6.6 on the Richter scale occurred, with its epicenter located about 14.5 km (9 miles) east of Newhall-Saugus, California, at 34° 24.0' N and 118° 23.7' W (fig. 1.1). The initial shock originated approximately 13.0 km (8 miles) below the earth's surface and faulting then propagated southward and upward, reaching the ground surface in the north San Fernando Valley area of Los

---

*Dr. Lew, Dr. Leyendecker, and Mr. Dikkers are Structural Research Engineers in the Building Research Division of the National Bureau of Standards' Institute for Applied Technology.

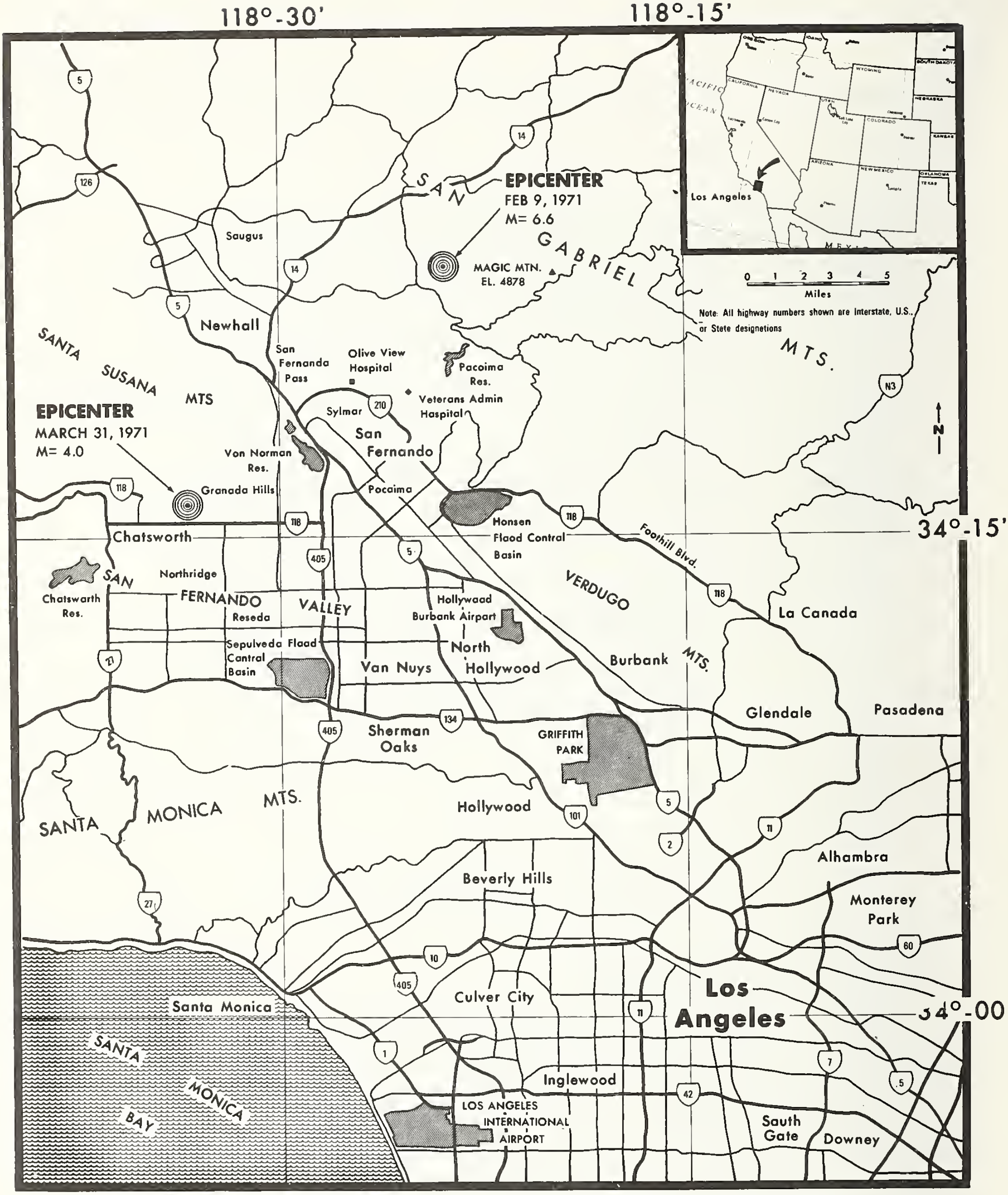



*Figure 1.1 Los Angeles, San Fernando Valley, and Newhall.*

Angeles. Figure 1.2 is an idealized cross section which illustrates the mechanics and orientation of the fault associated with the San Fernando earthquake. A fault of this type having an inclined fault plane is known as a dip-slip or thrust fault. The faulting of this earthquake induced the north block to move southward and upward about 1.5m (about 4.5 feet) and also westward about 2m (about 6 feet).

Allen et al [1.1]* report that no foreshocks exceeding magnitude 1.5 had occurred in the San Fernando Valley area in the preceeding 8 days and no shock exceeding magnitude 2.5 had been identified in the area during the preceding 4 months. The most identifiable event within the area was a shock of magnitude 2.6 that occurred north of Sylmar (fig. 1.1) on September 20, 1970. In the years prior to 1971, the San Fernando Valley had been an area of moderate seismic activity. The only earthquake occurring within this area which had as much energy release as the 1971 earthquake was the so-called Pico Canyon earthquake of 1893. Pico Canyon is located about 5 km (3 miles) west of Newhall. According to Townley and Allen [1.2], the Pico Canyon earthquake had a magnitude of approximately 6 on the Richter scale.

Following the initial shock, more than 200 aftershocks having a magnitude of 3.0 or greater occurred through March 1, 1971, including the one that followed immediately after the main shock with a magnitude of 5.1. The aftershocks for which epicentral locations have been determined are shown in figure 1.3. The greatest concentration of aftershocks lies roughly in the shape of an inverted U. It appears from this figure that the larger shocks are concentrated around the periphery of this inverted U, while smaller shocks occurred in the interior.

The most damaging aftershock since February 9 occurred at 6:52 a.m. (Pacific Standard Time) on March 31, 1971 in the Granada Hills area (fig. 1.1) and was felt over much of Los Angeles and in eastern Ventura County. This aftershock registered 4.0 on the Richter scale.

## 1.2 Earthquake Measures and Felt Area

### EARTHQUAKE MEASURES

There are two terms which are commonly used to describe the size of an earthquake. These are "intensity" and "magnitude." These two terms are often confused, and it is

*Numbers in brackets indicate literature references listed at the end of each chapter.

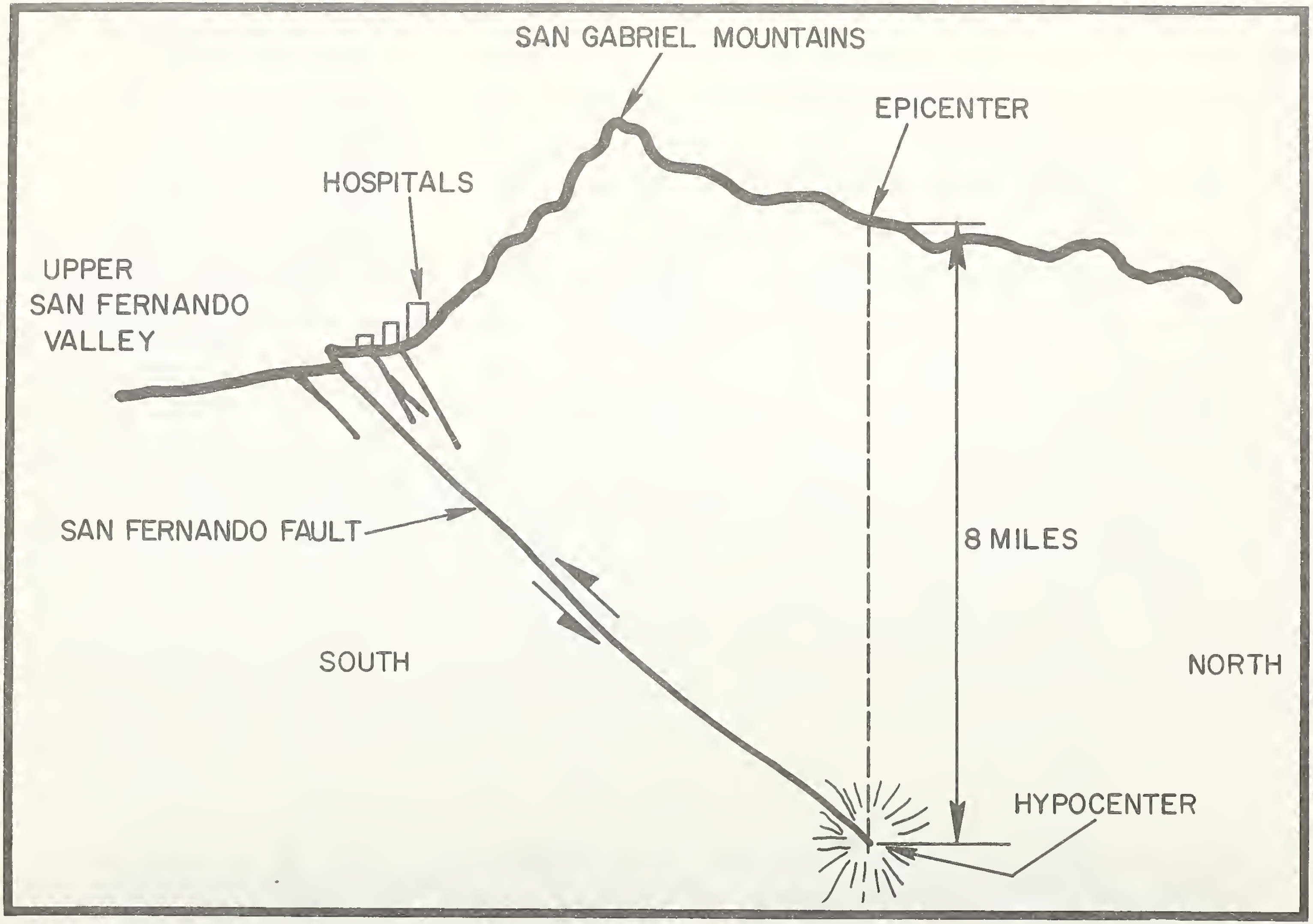



*Figure 1.2 Idealized cross section through the San Gabriel Mountains and the Upper San Fernando Valley.*

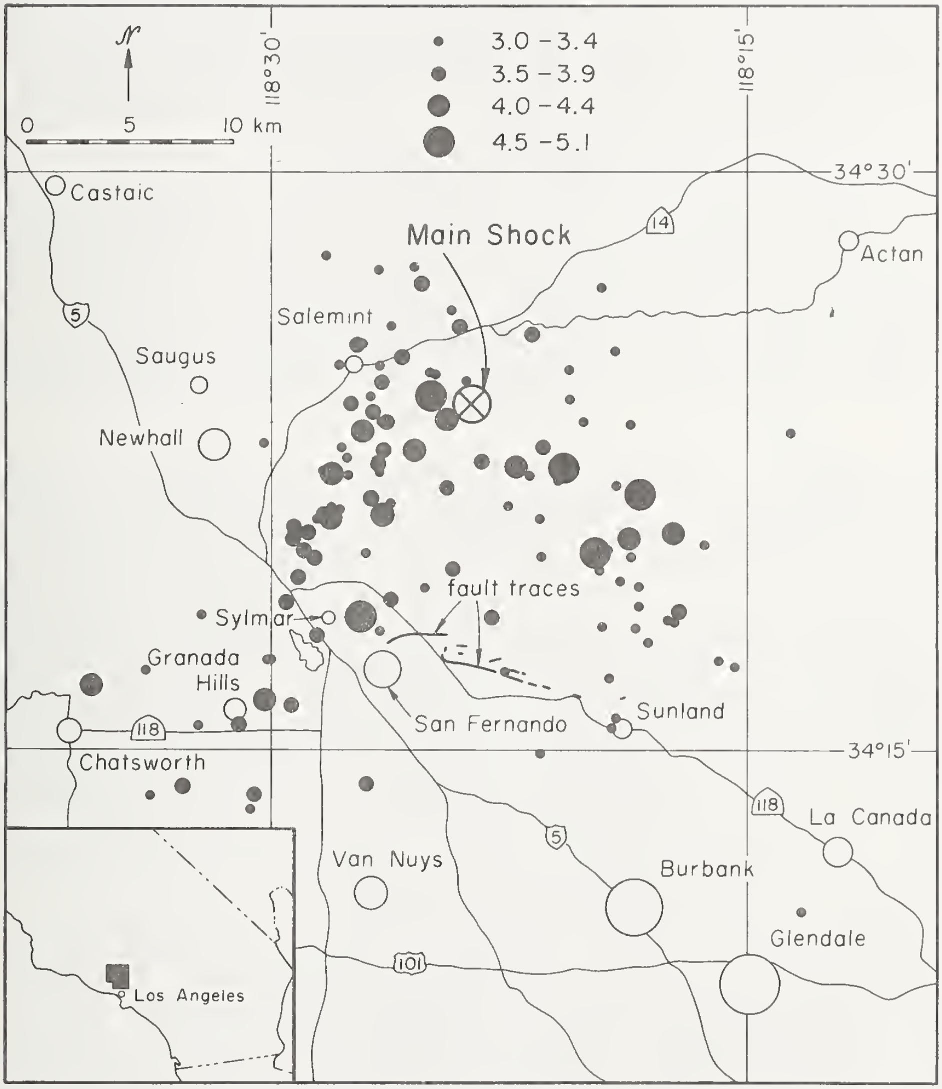



*Figure 1.3 Location of the epicenter of the main shock (magnitude 6.6) and of representative aftershocks (magnitudes greater than 3) through March 1, 1971. Approximate traces of some of the faulting activated during this earthquake are also shown. Prepared by the Seismological Laboratory, California Institute of Technology.*

important to understand the difference.

INTENSITY is an indication of an earthquake's apparent severity at a specific location, as determined by observers. It is a measure of the effects of an earthquake determined through interviews with persons in the quake-stricken area, damage surveys, and studies of earth movement. Based on this measure of intensity the seismic risk map of the United States, such as the one given in the current Uniform Building Code [1.3], was determined.

The Modified Mercalli Intensity used in the United States grades observed effects into twelve classes ranging from I to XII. Description of this scale is given in table 1.1 [1.4]. The older Rossi-Forel Intensity scale has ten categories of observed effects, and is still used in Europe. Still other intensity scales are in use in Japan and the U.S.S.R.

The potential severity of the earthquake at a particular location can also be determined from the records of a strong-motion seismometer (accelerometer) mounted on a rigid foundation. The readings from such instruments indicate the amplitude and frequency of earth accelerations at that specific site and can be integrated to determine both velocities and displacement. Such information can be used by engineers in determining the degree of motion to which structures have been subjected and to make determinations of the forces which are exerted on structures. The measure of potential severity of the earthquake at the recording station is usually expressed as a fraction of the acceleration due to gravity. A large number of such strong-motion recorders were present in the Los Angeles area.

MAGNITUDE, on the other hand, expresses the total amount of energy released by an earthquake as determined by measuring the amplitudes produced on standardized recording instruments. Thus, it is a measure of the absolute size of an earthquake and does not consider the effect at any specific location.

The Richter scale, which gives the numerical value of the magnitude, was defined by Richter in 1935 as logarithms (base 10) of the amplitude in microns of the trace written by a seismograph at a distance of 100 km from the epicenter.

Observations at distances other than 100 km can be corrected to convert them to the standard distance. Because the Richter Scale is expressed in logarithms of base 10, a unit increase in the scale is equivalent to a ten fold increase in the real trace magnitude. For example, an earthquake of magnitude 8 represents a seismograph amplitude 10 times greater than that of a magnitude 7 earthquake, 100

*Table 1.1 Modified Mercalli Intensity Scale of 1930*
*(Abridged and rewritten)*

I. Not felt.

II. Felt by persons at rest, on upper floors, or favorably placed.

III. Felt indoors. Hanging objects swing. Vibration like passing of light trucks. May not be recognized as an earthquake.

IV. Hanging objects swing. Vibration like passing of heavy trucks or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. Wooden walls and frames creak.

V. Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move.

VI. Felt by all. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, etc., off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken visibly, or heard to rustle.

VII. Difficult to stand. Noticed by drivers of automobiles. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices, also unbraced parapets and architectural ornaments. Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.

VIII. Steering of automobiles affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.

*Table 1.1 Continued*

IX. General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. General damage to foundations. Frame structures, if not bolted, shifted off foundations. Frames racked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated area sand and mud ejected, earthquake fountains, sand craters.

X. Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.

XI. Rails bent greatly. Underground pipelines completely out of service.

XII. Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.

QUALITY OF MASONRY
(BRICK OR OTHER)

Masonry A. Good workmanship, mortar, and design; reinforced especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.

Masonry B. Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.

Masonry C. Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

Masonry D. Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

times greater than that of a magnitude 6 earthquake, and so on. There is no upper limit to the Richter scale. The largest magnitude ever recorded is 8.9.

The relationship between the magnitude of an earthquake and the energy which it releases, as given by Richter [1.4], is

$$\log_{10} E = 11.4 + 1.5\ M,$$

where M is the Richter magnitude and E is the energy in ergs which is defined as the unit of work equal to a force of 1 dyne acting through a distance of 1 cm in the centimeter-gram-second system. A dyne is the force required to accelerate a free-standing gram mass 1 cm per second. A difference of one degree in magnitude corresponds to a factor of $10^{1.5}$ or 31.6 in the amount of energy released. Thus, an earthquake of magnitude 8 represents an energy release approximately 32 times greater than that of a magnitude 7 earthquake and almost 1000 times greater than that of a magnitude 6 earthquake. The energy released by an atomic bomb of the Hiroshima type, for example, is given by the Atomic Energy Commission as $8 \times 10^{20}$ ergs, which is equivalent to the energy released in an earthquake of magnitude 6.3 [1.5].

## MAGNITUDE-INTENSITY RELATIONS

Magnitude and maximum intensity of an earthquake are interdependent to some degree, but there is no close correlation between them. For example, an earthquake might have a low magnitude but because of shallow focus, poor soil condition, or poor building construction practice, it might cause a great deal of damage; thus, it would have a very high intensity. The March 31, 1971, aftershock which occurred in the Granada Hills area is a typical example. While this aftershock registered only 4.0 on the Richter scale, it caused significant damage to residential structures. On the other hand, an earthquake of very large magnitude might have a great focal depth, or might occur in an area where there is very little man-made structure to damage. It would not, therefore, have a high apparent intensity.

Within a relatively small geographic region, such as within the State of California, an approximate relationship between magnitude and intensity such as the one shown in table 1.2 has been suggested. In this table magnitude is compared with the related energy released, felt area, and felt distance [1.6]. These relationships were developed for use with ordinary ground conditions associated with metropolitan centers in California and were greatly influenced by the past history and future probability of strike-slip

*Table 1.2 Approximate Relationship Between Magnitude and Intensity*

| *Magnitude* | *Energy-Released (Ergs)* | *Felt Area (square miles)* | *Distance Felt (st. miles)* | *Intensity (Maximum Expected Modified Mercalli)* |
|---|---|---|---|---|
| *3.0 - 3.9* | $9.5 \times 10^{15} - 4.0 \times 10^{17}$ | *750* | *15* | *II - III* |
| *4.0 - 4.9* | $6.0 \times 10^{17} - 8.8 \times 10^{18}$ | *3,000* | *30* | *IV - V* |
| *5.0 - 5.9* | $9.5 \times 10^{18} - 4.0 \times 10^{20}$ | *15,000* | *70* | *VI - VII* |
| *6.0 - 6.9* | $6.0 \times 10^{20} - 8.8 \times 10^{21}$ | *50,000* | *125* | *VII - VIII* |
| *7.0 - 7.9* | $9.5 \times 10^{22} - 4.0 \times 10^{23}$ | *200,000* | *250* | *IX - X* |
| *8.0 - 8.9* | $6.0 \times 10^{23} - 8.8 \times 10^{24}$ | *800,000* | *450* | *XI - XII* |

faulting. The table does not seem as applicable to an earthquake of the San Fernando type, which was predominantly a thrust faulting. For the San Fernando earthquake's Richter Magnitude of 6.6, table 1.2 would indicate an expected Modified Mercalli (M.M.) Scale Intensity of VII or VIII, which is quite low when compared to the actual damage. Qualified observers have indicated that the actual damage in the vicinity of severely damaged hospital structures would be closer to a M.M. Scale XI. From table 1.2, this intensity would correspond to a Magnitude 8 quake, indicating that the intensity of this particular earthquake was far in excess of what would have ordinarily been expected.

By using the relationship between the amount of TNT and associated energy produced by it (one thousand tons of TNT equal to $4.2 \times 10^{19}$ ergs) [1.7], the Richter scale magnitude versus the energy equivalent in tons of TNT is plotted in figure 1.4 with several earthquakes indicated on it. It is seen that the magnitude of the San Fernando earthquake was moderate compared with others.

## FELT AREA

The February 9, 1971, earthquake was perceptible over approximately 80,000 square miles of California, Nevada, and Arizona. To establish isoseismals, the National Ocean Survey (NOS) of the National Oceanic and Atmospheric Administration (NOAA) conducted a mail survey [1.8]. As of March 4, 1971, 1860 reports had been received from a distribution of 2068

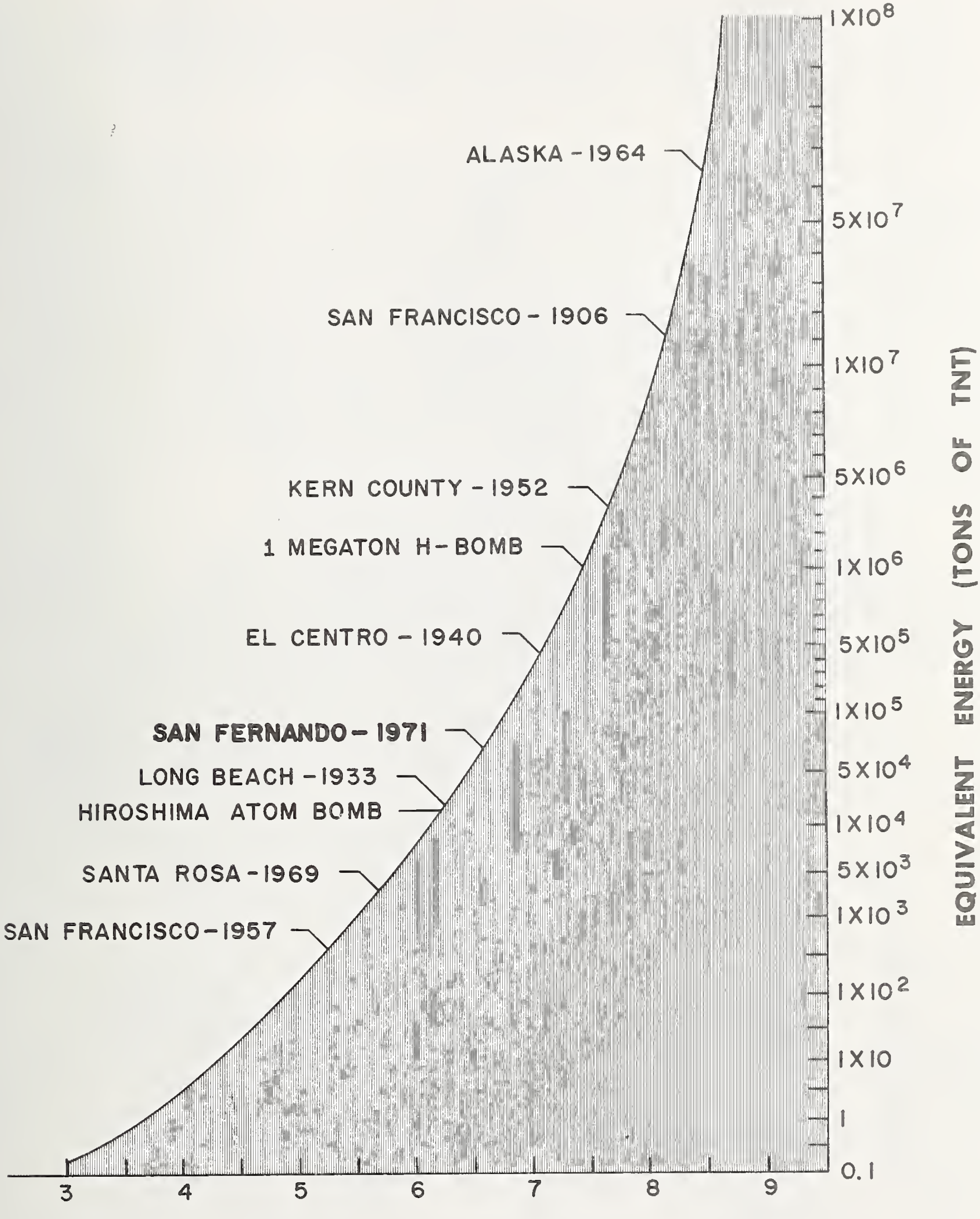



*Figure 1.4* *Comparison of Richter Scale Magnitude versus Equivalent Energy of TNT.*

cards. Of these 784 were distributed to postmasters and 1284 to various other persons, including several hundred structural engineers. Figure 1.5 shows a preliminary isoseismal map based on the mail survey, field survey, and interviews [1.8].

Based on the catastrophic damage to the San Fernando Veterans Administration Hospital, the Olive View Medical Center complex and the Holy Cross Hospital, a maximum intensity of VIII-XI has been assigned tentatively to a small area in the foothills of the northeastern corner of the San Fernando Valley. Other severe damages include collapse of freeway separation and overhead, major damage to the Van Norman Reservoir dams, severe damage to the San Fernando Juvenile Hall, the Sylmar Convertor Station, and the Pacoima Memorial Lutheran Hospital. Severe damage to structures in the San Fernando Mall and partial collapse of residences in the City of San Fernando are also included in this zone.

Zone VII extends to the Newhall-Saugus area to the north and to downtown Los Angeles to the south. Within this zone, many houses lost chimneys at roof lines and many unreinforced masonry walls collapsed. Also observed in this zone were cracks in masonry shear walls and cracks in lower-story reinforced concrete columns in highrise buildings.

Outside of the intensity VII zone, only slight-to-moderate damage was reported, principally cracked or fallen plaster and broken windows.

## 1.3 Damage and Casualty Statistics

### DAMAGE STATISTICS

The most severe damage to building structures and utility systems occurred in the upper San Fernando Valley area, which encompasses the northern end of the City of Los Angeles, Sylmar, Pacoima, Granada Hills, and the City of San Fernando. This area lies five to ten miles south of the epicenter. The reason why this area suffered more damage than the immediate epicenter area can be visualized from figure 1.2. While the rupturing initiated at some 8 miles below the epicenter, it surfaced at the upper San Fernando area. As a result, the actual ground displacement was greater in this area than it was at the epicentral area. Many damaged structures were either directly on the fault line or situated near it.

Property damage, both private and public, in the

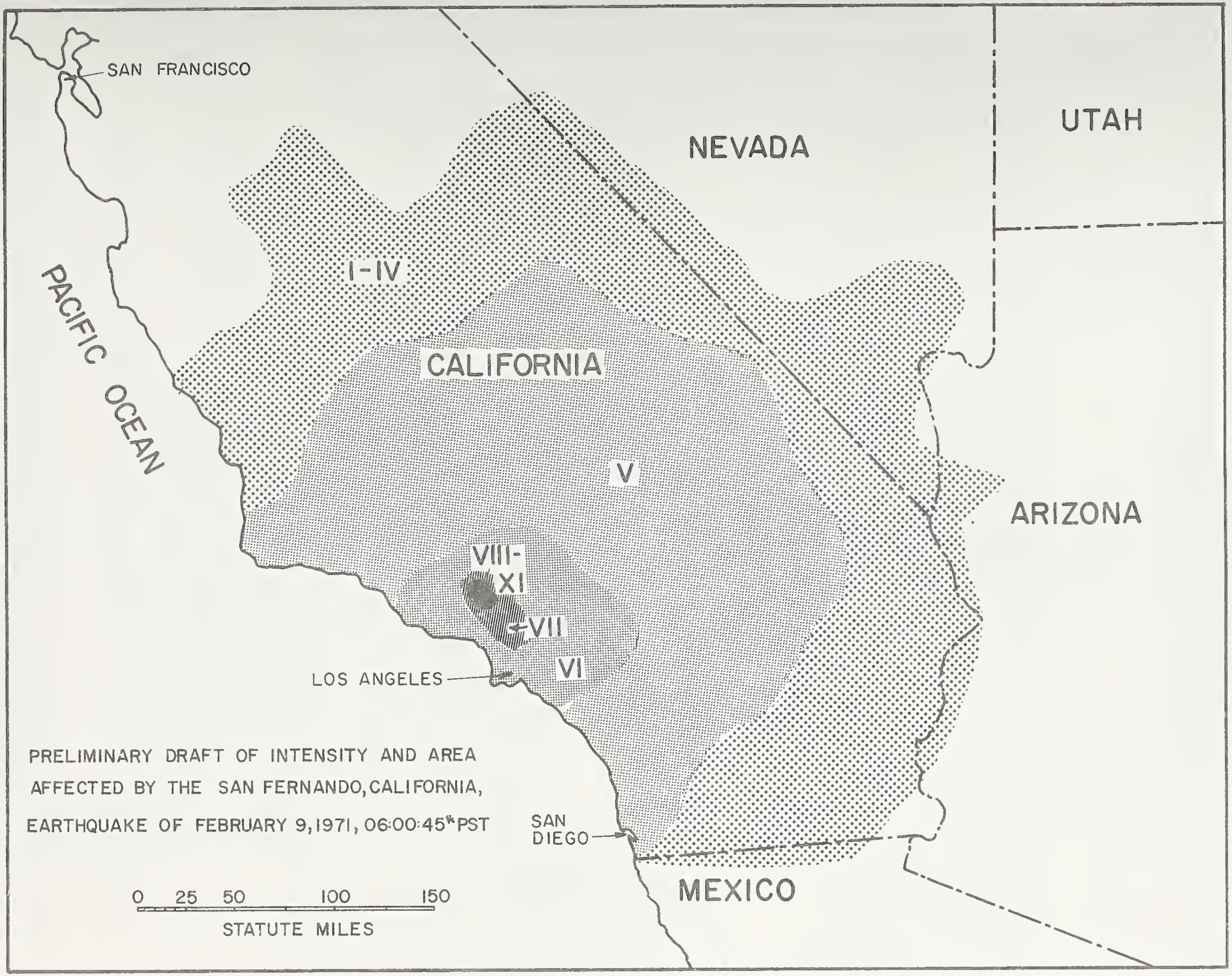



*Figure 1.5 Preliminary draft of intensity and area affected by the San Fernando, California, Earthquake of February 9, 1971.*

metropolitan Los Angeles area had been estimated at $436,600,000 as of March 5, 1971. A partial breakdown of this estimate related to private and major public property is listed in table 1.3. A breakdown of building damages is listed in table 1.4.

CASUALTY STATISTICS

The San Fernando earthquake claimed the lives of 64 persons. Of these, 46 persons lost their lives at the San Fernando Veterans Administration Hospital. Excluding this concentrated loss of lives, and considering the severity of structural damage, the number of fatalities from this earthquake was remarkably low. Such a low fatality rate was, as in several previous earthquakes, due mainly to the time of day of its occurrence. Had the earthquake occurred a few hours later, the loss of life would probably have been many times greater. A list of the number of injuries treated and the number of deaths is given in table 1.5.

## 1.4 Disaster Relief Act of 1970

The San Fernando earthquake is the first disaster to fall under the coverage of the Disaster Relief Act of 1970 (PL 91-606). This Act gives the President broad powers to supplement the efforts and resources of State and local governments in alleviating suffering, hardship, and damage caused by "major disasters." The Director, Office of Emergency Preparedness, Executive Office of the President, has also been given certain authorities and functions under this Act, including, but not limited to the following:

- The authority to direct Federal agencies to provide assistance in "major disasters."
- The authority to coordinate the activities of federal agencies in providing disaster assistance, and to direct any Federal agency to utilize its available personnel, equipment, supplies, facilities, and other resources in accordance with the authority contained in Public Law 91-606.
- The authority to appoint a Federal Coordinating Officer.
- The authority to execute for the Federal Government a joint Federal-State Disaster Assistance Agreement with the Governor of a State, for providing supplemental Federal disaster assistance.

Table 1.3 Summary of Property Damage Estimates

(As of March 5, 1971)

| | |
|---|---|
| <u>Private Property</u>: | |
| City of Los Angeles | |
| Building Damage | $160,000,000 |
| Property and Furnishing | 50,000,000 |
| County of Los Angeles | |
| Building Damage | 6,790,000 |
| Mobile Home Damage | 1,120,000 |
| Property and Furnishing | No Data |
| <u>Public Property</u>: | |
| Olive View Medical Center | $ 35,800,000 |
| San Fernando Juvenile Hall | 6,000,000 |
| Karl Holten Boys Camp | 2,500,000 |
| L.A. County Structures | 8,000,000 |
| L.A. County Equipment, Furniture, etc. | 52,000,000 |
| Sylmar Convertor Station (Los Angeles Water and Power only) | 30,000,000 |
| Sewage, Water, Electric Distribution | 20,000,000 |
| Van Norman Reservoir Dams | 34,000,000 |
| Pacoima Dam | 1,500,000 |
| Debris Basin, Disposal Area, Storm Drains | 800,000 |
| Streets, Roads and Bridges | 5,000,000 |
| Highway Structures | 15,000,000 |
| <u>Others</u>: | 8,090,000 |
| TOTAL | $436,600,000 |

*Table 1.4 Summary of Building Damage*
*(As of March 5, 1971)*

*City of Los Angeles:*

| | |
|---|---|
| *Vacated as Unsafe* | |
| *Dwellings . . . . . . . . . . . . . . . . . .* | *415 units* |
| *Apartments. . . . . . . . . . . . . . . . . .* | *54* |
| *Commercial and Industrial . . . . . . . . .* | *382* |
| *Major Damage (Structural Damage) . . . . . . .* | *5,405* |
| *Minor Damage . . . . . . . . . . . . . . . . . .* | *17,950* |
| | *24,206 units* |

*County of Los Angeles:*

| | |
|---|---|
| *Vacated as Unsafe* | |
| *Dwellings . . . . . . . . . . . . . . . . . .* | *61 units* |
| *Apartments . . . . . . . . . . . . . . . . .* | *7* |
| *Sheds, Carports, etc. . . . . . . . . . . . .* | *6* |
| *Assembly. . . . . . . . . . . . . . . . . . .* | *2* |
| *Commercial & Industrial . . . . . . . . . .* | *21* |
| *Damaged* | |
| *Dwellings . . . . . . . . . . . . . . . . . .* | *2,068 units* |
| *Apartments . . . . . . . . . . . . . . . . .* | *67* |
| *Mobile Homes. . . . . . . . . . . . . . . . .* | *1,707* |
| *Sheds, Carports, etc. . . . . . . . . . . . .* | *24* |
| *Assembly. . . . . . . . . . . . . . . . . . .* | *41* |
| *Commercial & Industrial . . . . . . . . . .* | *123* |
| *TOTAL* | *4,128 units* |

*City of San Fernando:*

| | |
|---|---|
| *Vacated as Unsafe* | |
| *Dwellings . . . . . . . . . . . . . . . . . .* | *270 units* |
| *Commercial and Industrial . . . . . . . . .* | *163* |
| *Damaged* | |
| *Dwellings . . . . . . . . . . . . . . . . . .* | *1,520 units** |
| *Commercial and Industrial . . . . . . . . .* | *No Data* |
| *TOTAL* | *2,953 units* |

**About 30 percent of the total dwelling units in the City of San Fernando*

- The authority to prescribe such rules and regulations as may be necessary and proper to implement Public Law 91-606.

*Table 1.5 Injury and Life Loss*

*(As of March 5, 1971)*

| | |
|---|---|
| *City of Los Angeles:* | |
| *Dead* | *13* |
| *Injuries* | *889* |
| *County of Los Angeles:* | |
| *Dead (46 at San Fernando VA Hospital)* | *51* |
| *Injuries* | *None Reported* |
| *Total Dead* | *64* |
| *Total Injuries* | *889* |

Under PL 91-606 the Federal Government reimburses to State and local governments the cost arising from repair, restoration, reconstruction and replacement of the damaged public facilities. This law does not directly involve relief to individuals and families. Thus, the earthquake damage to facilities such as hospitals, schools, dams, reservoirs, power distribution systems, sewers, storm drains, highways and streets are covered by this law.

For reference, several sections related to the restoration of damaged structures, of the Disaster Relief Act of 1970 are excerpted below:

*PUBLIC LAW 91-606*

*91st CONGRESS, S. 1970*

*December 31, 1970*

*AN ACT*

*To revise and expand Federal programs for relief from the effects of major disasters, and for other purposes.*

*Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That this Act may be cited as the "Disaster Relief Act of 1970"*

*MINIMUM STANDARDS FOR RESIDENTIAL STRUCTURE RESTORATION*

*SEC. 243 No loan or grant made by any relief organization operating under the supervision of the Director, for repair, restoration, reconstruction, or replacement of any residential structure located in a major disaster area shall be made unless such structure will be repaired, restored, reconstructed, or replaced in accordance with applicable standards of safety, decency, and sanitation and in conformity with applicable building codes and specifications.*

*FEDERAL FACILITIES*

*SEC. 251 The President may authorize any Federal agency to repair, reconstruct, restore, or replace any facility owned by the United States and under the jurisdiction of such agency which is damaged or destroyed by any major disaster if he determines that such repair, reconstruction, restoration, replacement is of such importance and urgency that it cannot reasonably be deferred pending the enactment of specific authorizing legislation or the making of an appropriation for such purposes. In order to carry out the provisions of this section, such repair, reconstruction, restoration, or replacement may be begun not withstanding a lack or an insufficiency of funds appropriated for such purpose, where such lack or insufficiency can be remedied by the transfer, in accordance with law, of funds appropriated to that agency for another purpose.*

*STATE AND LOCAL GOVERNMENT FACILITIES*

*SEC. 252 (a) The President is authorized to make contributions to State and local* *governments to repair, restore, reconstruct, or replace public facilities belonging to such State or local governments which were damaged or destroyed by a major disaster, except that the Federal contribution therefor shall not exceed 100 per centum of the net cost of repairing, restoring, reconstructing, or replacing any such facility on the basis of the design of such facility as it existed immediately prior to such disaster and in conformity with applicable codes, specifications, and standards.*

*(b) In the case of any such public facilities which were in the process of construction when damaged or destroyed by a major disaster, the Federal contribution shall not exceed 50 per centum of the net costs of restoring such facilities substantially to their prior to such disaster condition and of completing construction not performed prior to the major disaster to the extent the increase of such cost is attributable to changed conditions resulting from a major disaster.*

*(c) For the purposes of this section "public facility" includes any flood control, navigation, irrigation, reclamation, public power, sewage treatment and collection, water supply and distribution, watershed development, or airport facility, any non-Federal-aid street, road, or highway, and any other public building, structure, or system, other than one used exclusively for recreation purposes.*

## 1.5 References

1.1 Allen, C. R. et al, "Main Shock and Larger Aftershocks of the San Fernando Earthquake, February 9 through March 1, 1971" Geological Survey Professional Paper 733, U. S. Government Printing Office, Washington, D. C.

1.2 Townley, S. D. and Allen, M. W., "Descriptive Catalog of Earthquakes of the Pacific Coast of the United States, 1769 to 1928," Seismological Society of America Bulletin, v. 29.

1.3 "Uniform Building Code," 1970 Edition, vol. I, International Conference of Building Officials, Pasadena, California.

1.4 Richter, C. F., "Elementary Seismology," W. H. Freeman and Company, 1958.

1.5 Berg, G. N., "The Skopje, Yugoslavia Earthquake, July 26, 1963," American Iron and Steel Institute.

1.6 "Earthquakes," National Oceanic Atmospheric Administration, U. S. Government Printing Office, Washington, D. C.

1.7 "The Effects of Nuclear Weapons," U. S. Department of Defense, U. S. Government Printing Office, Washington, D. C.

1.8 Scott, N. H., "Preliminary Report of Felt Area and Intensity," Geological Survey Professional Paper 733, U. S. Government Printing Office, Washington, D. C.

## 2.1 Seismic History of California

The San Fernando earthquake is only one of a number of destructive earthquakes that have occurred in California in the present century (see table 2.1). In terms of magnitude

*Table 2.1 Property Damage and Loss of Life from Major California Earthquakes in the Present Century*

| Earthquake | Magnitude | Estimated Property Damage ($Millions) Contemporary dollars | 1971 dollars* | Estimated Loss of Life |
|---|---|---|---|---|
| San Francisco April 18, 1906 | 8.3 | 480 | 4550 | 700 |
| Santa Barbara June 29, 1925 | 6.3 | 8 | 42 | 13 |
| Long Beach March 10, 1933 | 6.3 | 40 | 196 | 120 |
| Imperial Valley May 18, 1940 | 7.1 | 6 | 28 | 9 |
| Kern County July 21, 1952 | 7.7 | 60 | 136 | 12 |
| San Fernando February 9, 1971 | 6.6 | 440 | 440 | 64 |

*Based on Engineering News-Record Building Cost Index.

it is exceeded by the San Francisco earthquake (1906), the Kern County earthquake (1952), and the Imperial Valley earthquake (1940). In terms of property damage measured in constant dollars it is exceeded only by the San Francisco earthquake, and in terms of loss of life it is exceeded by the San Francisco and Long Beach (1933) earthquakes. Sources for the following account of the seismic history of California include Richter [2.1], Wood and Heck [2.2], and Tocher [2.3].

The San Francisco earthquake of 1906 was the greatest earthquake disaster in the history of the United States. It resulted from movement on the San Andreas fault, a northwest trending break in the earth's crust along which the coastal block to the southwest moves northwestward relative to the continental block. At the time of the earthquake the fault broke over a length of at least 190 miles and the horizontal displacement reached a maximum of 21 feet (in Marin County). Shaking accompanying the earthquake caused extensive damage in San Francisco and other towns near the fault and caused a fire in San Francisco that destroyed a large part of the city.

Of the other twentieth century California earthquakes listed in table 2.1, the Imperial Valley earthquake of 1940 occurred along the Imperial fault, which is a branch of the San Andreas fault. The rest of the earthquakes were associated with other faults, the White Wolf fault for the Kern County earthquake of 1952, the Inglewood fault for the Long Beach earthquake of 1933, and a fault offshore for the Santa Barbara earthquake of 1925.

There were several major earthquakes in California during the nineteenth century, but they are difficult to evaluate because of the lack of instrumental data, the fragmentary nature of old reports, and the sparse settlement of the region. An earthquake near Santa Barbara in 1812 damaged missions over a wide area in southern California and produced a tsunami (tidal wave) in the Santa Barbara channel. At least two of the nineteenth century earthquakes are generally considered comparable in magnitude and destructive potential to the San Francisco earthquake of 1906. They are the Fort Tejon earthquake of 1857 and the Owens Valley earthquake of 1872. The 1857 earthquake was accompanied by a break along the San Andreas fault in southern California that passed within 25 miles of the city of Los Angeles. The 1872 earthquake occurred in eastern California near the Nevada border in an area that is still relatively sparsely populated. Other major nineteenth century earthquakes occurred in 1838 along the San Andreas fault near San Francisco and in 1836 and 1868 along the Hayward fault, a branch of the San Andreas on the east side of San Francisco Bay.

In analyzing the earthquake risk in California, the San Andreas fault deserves first attention. A recent analysis by C. R. Allen [2.4] suggests that the segments of the fault that broke in 1906 and 1857 (fig. 2.1) are characterized by relatively infrequent great earthquakes (magnitude larger than 8) and a comparative absence of smaller shocks and fault creep. The other segments of the fault, in contrast, show abundant small and moderate earthquakes accompanied by fault creep without earthquakes. Allen suggests that this pattern may be stable and typical of the past and probable future behavior of the fault system. In this concept, the 1857 and 1906 breaks are "locked" segments of the fault where strain accumulates to

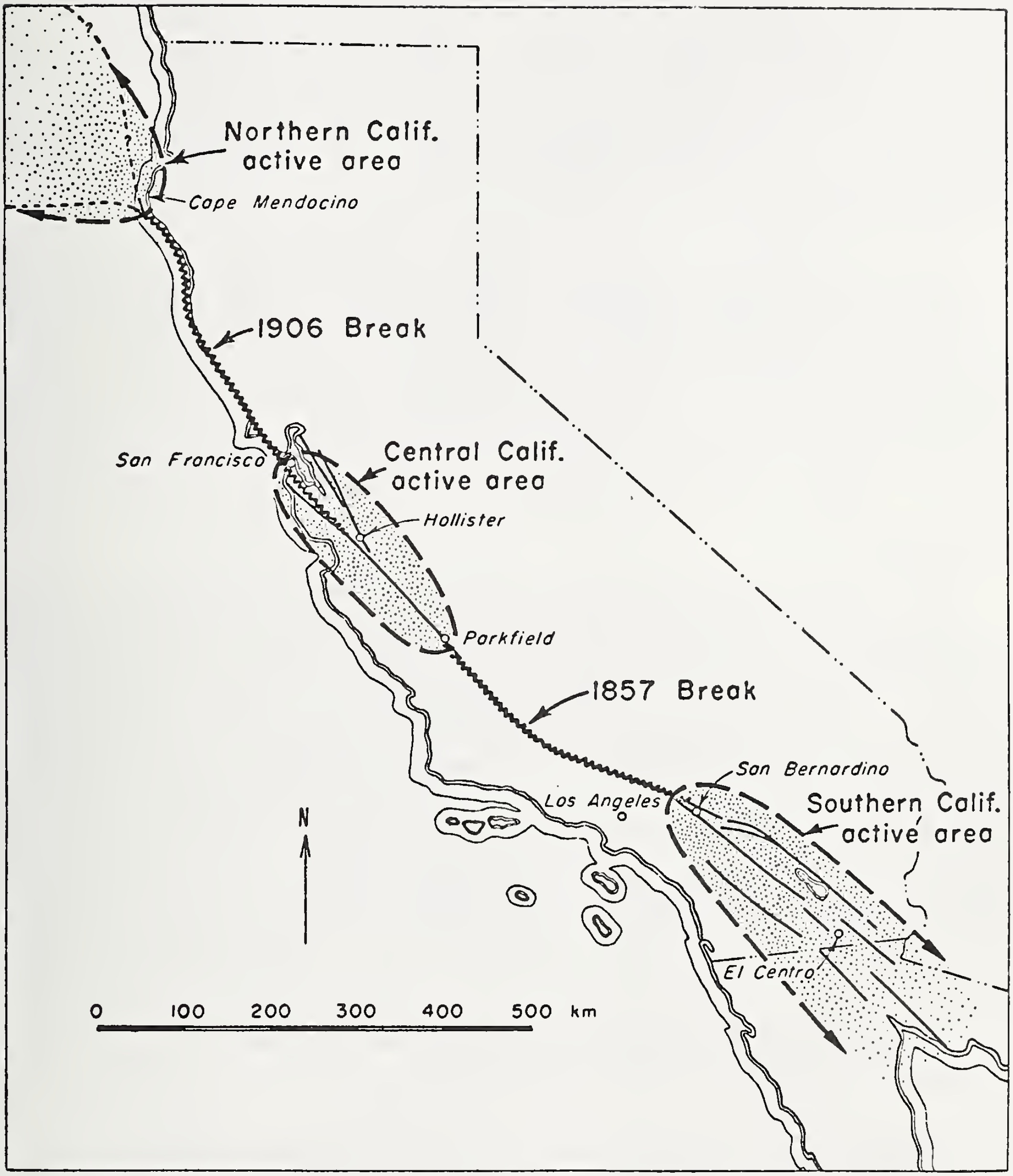



*Figure 2.1 Areas of contrasting seismic behavior along the San Andreas fault zone in California [2.4].*

a relatively high level before being released in a great earthquake, whereas strain along the other segments of the fault is released more often in the form of moderate earthquakes and fault creep. Both the major metropolitan centers of California, Los Angeles and San Francisco, lie near segments of the fault that broke during historic great earthquakes. If the foregoing analysis is correct, those segments are where great earthquakes should be expected in the future.

R. E. Wallace [2.5] estimated the possible recurrence intervals for great earthquakes along the San Andreas fault by relating geologic data on long-term offset rates with displacements and lengths of breaks recorded for historic earthquakes, fault creep rates, and behavior of different segments of the fault. He arrived at an estimated recurrence interval of 100 years with an uncertainty of at least a factor of two. In other words, the recurrence interval for great earthquakes on the San Andreas may lie between 50 and 200 years.

Although a magnitude 8+ earthquake on the San Andreas fault seems to represent the most destructive of probable future shocks in California, the twentieth century earthquake history of the state of California teaches the lesson that severe earthquakes with substantial damage and extensive loss of life must be expected from other faults. The San Fernando earthquake and three of the other five earthquakes listed in table 2.1 (Long Beach, Kern County, and Santa Barbara) occurred on faults that are not included in the San Andreas system. The San Fernando and Kern County earthquakes occurred along faults not generally recognized as active before the earthquakes. This points up the need for the development of improved methods of characterizing the potential activity of faults and the application of these methods in geologic field studies to assess the activity of faults in California and other seismically active areas.

## 2.2 Crustal Deformation in the San Fernando Earthquake

The crustal movements responsible for the San Fernando earthquake, like the other earthquakes in California's history, are part of a pattern of deformation that has been going on for tens of millions of years. An understanding of this pattern is important for a proper evaluation of the earthquake hazard in this region and a consideration of measures to reduce the hazard. The following description of the geologic setting of the San Fernando earthquake is summarized from a report by Wentworth and Yerkes [2.6].

The pattern of crustal deformation in coastal California is dominated by the San Andreas fault system. In southern California, the relative displacement of the two blocks on opposite sides of the fault has amounted to about 130 miles in the past 30 million years. North of Los Angeles near the center of the map in figure 2.2, the San Andreas makes a sharp bend from a southeasterly to a more easterly trend. In the general area to the south of this bend in the San Andreas lie the San Gabriel Mountains and further to the south and southwest the San Fernando Valley and the Los Angeles Basin. As the crustal block to the southwest of the San Andreas fault moves northwestward, the area to the south of the bend is compressed against the bend. In response to these compressive forces the San Gabriel Mountains have been thrust up and southward relative to the Los Angeles Basin and the San Fernando Valley, along north-dipping faults. As a result of movements along these faults, mainly in the past 5 to 10 million years, the San Gabriel Mountain block has been elevated relative to the Los Angeles Basin by some 12,000 feet.

The San Fernando earthquake represents a single episode in this continuing process of crustal deformation. The faulting associated with the earthquake produced a zone of surface ruptures that trends more or less eastward across the northern part of the San Fernando Valley (fig. 2.3). The ruptures in this zone, now named the San Fernando fault zone, were studied in detail by the U. S. Geological Survey [2.7 and 2.8]. Measurements near the center of the zone indicate that the block to the north had a relative movement of about 6 feet to the west and 4.5 feet vertically upward. Horizontal shortening in a north-south direction across the zone amounted to about 2 feet. Where the orientation could be reliably determined, the surface ruptures dipped to the north at angles of about 55° to 65°

Preliminary locations of aftershocks determined by Wesson, Lee, and Gibbs [2.9] and a fault plane solution by the same authors for the main shock are consistent with a principal plane of slippage dipping to the north under the San Gabriel Mountains, with a dip somewhat flatter than indicated by the ruptures at the surface. Almost all of the aftershock epicenters lie to the north of the zone of surface breakage; and, in a north-south cross-section, the aftershock foci fall into a wedge-shaped zone, the lower boundary of which dips 30° to 40° north. This lower boundary corresponds approximately to the fault-plane solution for the main shock and can tentatively be identified with the principal plane of slippage.

A substantial amount of building damage was directly related to ground rupture along the San Fernando fault zone [2.10]. The breakage of gas and water pipelines and extensive

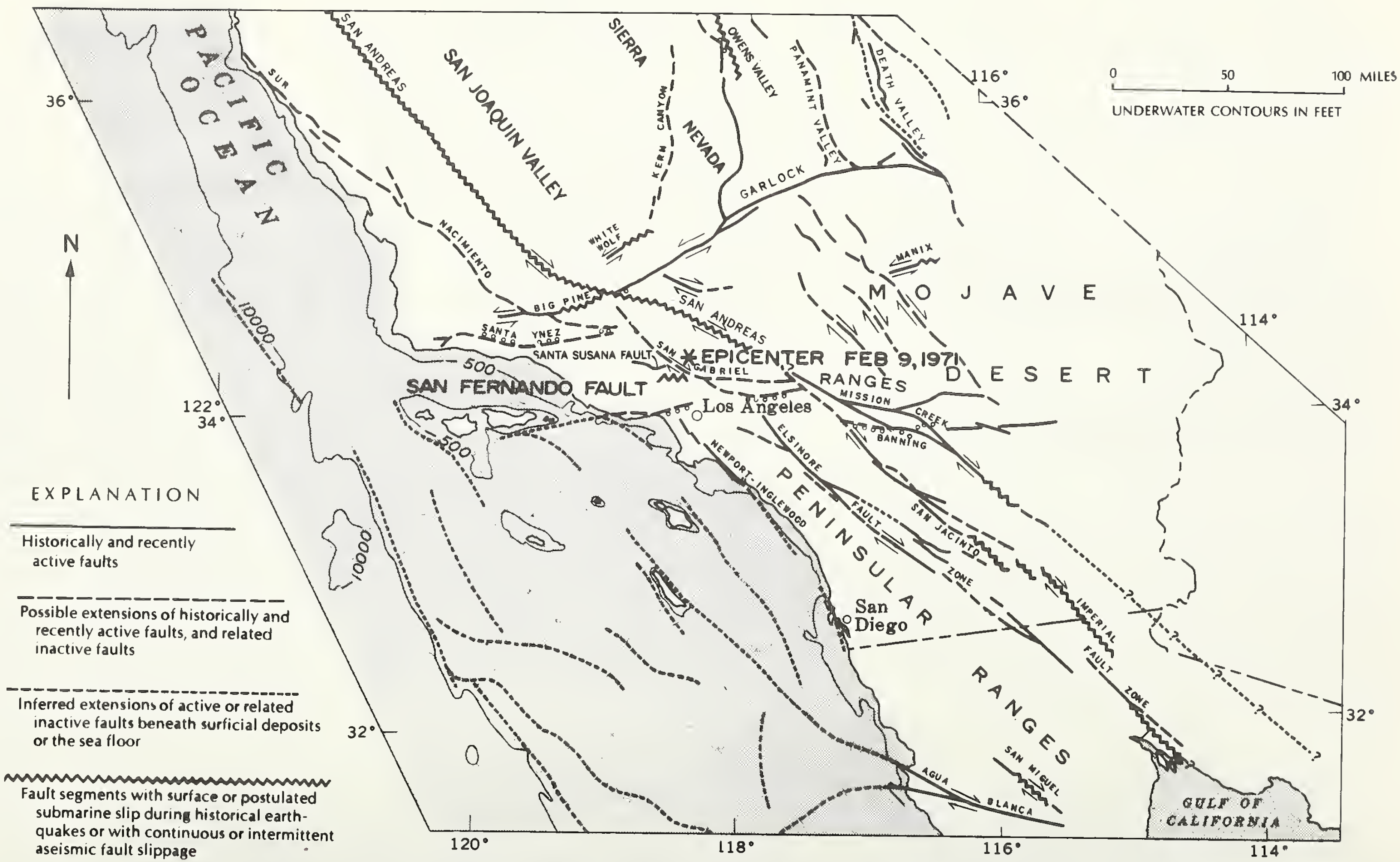



*Figure 2.2 Major historically and recently active faults of the Southern California Region [2.6].*

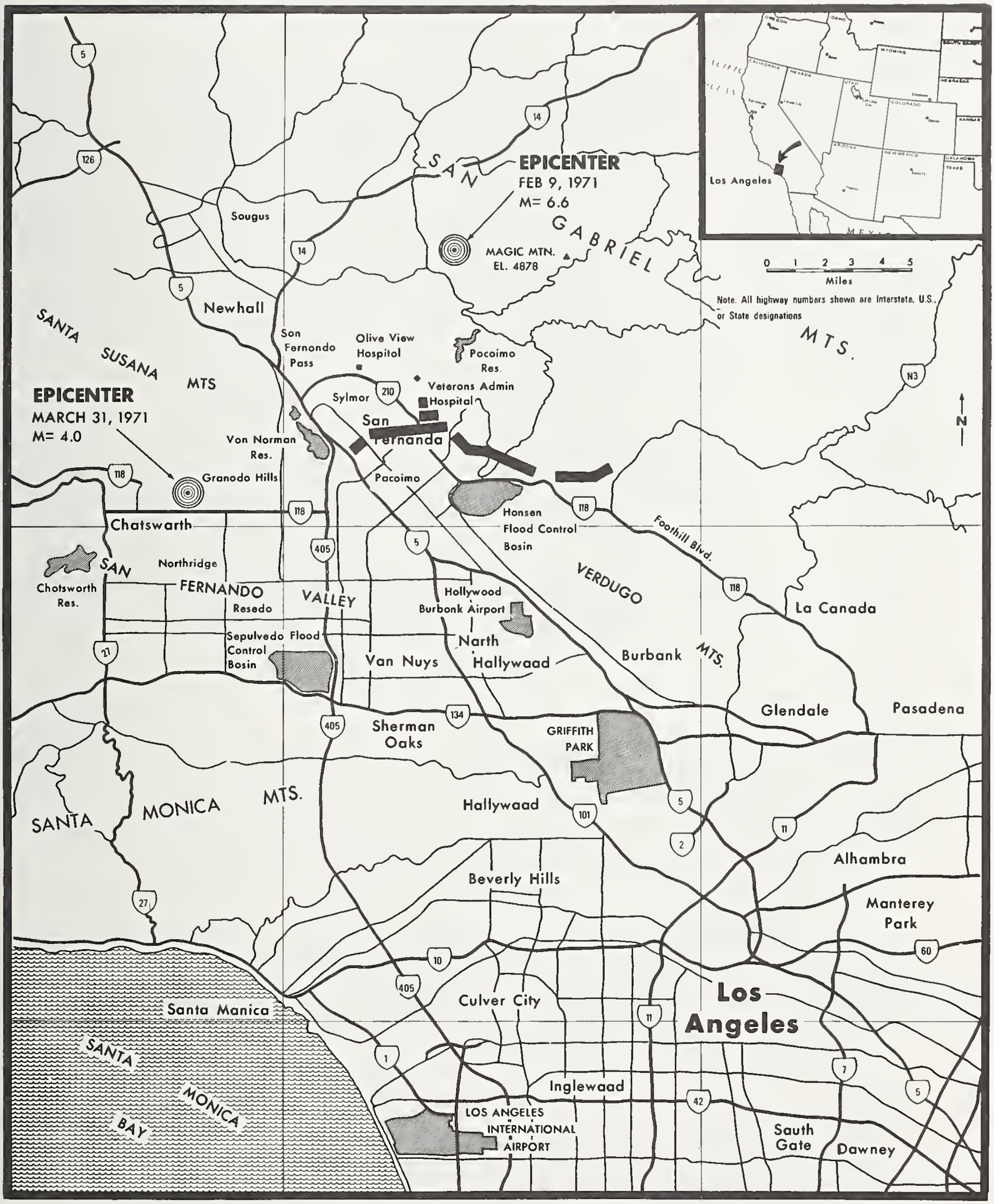



*Figure 2.3 The San Fernando fault zone. Heavy black lines show the approximate location of the zone of surface faulting.*

damage to pavements were also the direct result of rupture along the zone. The spectacular damage at the Olive View and Veterans Administration hospitals, on the other hand, appears to be due to ground shaking rather than surface ground breakage.

The San Fernando fault zone was not recognized to be active before the earthquake of February 9. Similarly the Kern County earthquake of 1952, the largest earthquake in California since 1906, occurred on the White Wolf fault, which had not been recognized as active before the earthquake. In view of the severe damage and loss of life caused by the San Fernando earthquake, it becomes important to consider whether it would have been possible to recognize the San Fernando fault zone as active before the earthquake. The 200-year historical record is too short to provide an adequate basis for evaluating fault activity. Geologic methods [2.11] have been developed for assessing the potential activity of a fault zone. These methods include a consideration of the effects of faulting on alluvial deposits and on the flow of ground water in alluvium. From their study of the San Fernando area after the earthquake, Wentworth and Yerkes [2.6] concluded that an application of these methods before the earthquake would have suggested that the fault zone should be considered active, at least for land uses requiring high safety factors.

## 2.3 References


2.1 Richter, C. F., "Elementary Seismology", W. H. Freeman and Co., 1958.

2.2 Wood, H. O., and Heck, N. H., "Stronger Earthquakes of California and Western Nevada", Part II of Earthquake History of the United States, revised ed. (through 1963) by R. A. Eppley, U. S. Coast and Geodetic Survey No. 41-1, 1966.

2.3 Tocher, Don, "Seismic History of the San Francisco Region" California Division of Mines and Geology, Special Report 57, 1959.

2.4 Allen, C. R., "The Tectonic Environments of Seismically Active and Inactive Areas along the San Andreas Fault System"; in Dickinson, W. R. and Grantz, Arthur, ed., Proceedings of the Conference on Geologic Problems of San Andreas Fault System, Stanford University, published in the Geological Sciences, Vol. 11, 1968.

2.5 Wallace, R. E., "Earthquake Recurrence Intervals on the San Andreas Fault", Geological Society American Bulletin, V. 81, 1970.

2.6 Wentworth, C. M., and Yerkes, R. F., "Geologic Setting and Activity of Faults in the San Fernando Area, California"; in The San Fernando, California, Earthquake of February 9, 1971, a preliminary report: U. S. Geological Survey Professional Paper 733, 1971.

2.7 Burford, R. O., Castle, R. O., Church, J. P., Kinoshita, W. T., Kirby, S. H., Ruthven, R. T., and Savage, J. C., "Preliminary Measurements of Tectonic Movement"; in the San Fernando, California, earthquake of February 9, 1971, a preliminary report: U. S. Geological Survey Professional Paper 733, 1971.

2.8 "U. S. Geological Survey, "Surface Faulting"; in The San Fernando, California, Earthquake of February 9, 1971, a preliminary report: U. S. Geological Survey Professional Paper 733, 1971.

2.9 Wesson, R. L., Lee, W. H. K., and Gibbs, J. F., "Aftershocks of the Earthquake"; in The San Fernando, California, earthquake of February 9, 1971, a preliminary report: U. S. Geological Survey Professional Paper 733, 1971.

2.10 Youd, T. L., and Olsen, H. W., "Damage to Constructed Works Associated with Soil Movements and Foundation Failures"; in The San Fernando, California, Earthquake of February 9, 1971, a preliminary report: U. S. Geological Survey Professional Paper 733, 1971.

2.11 Wentworth, C. W., Ziony, J. I., and Buchanan, J. M., "Preliminary Geologic Environmental Map of the Greater Los Angeles, California", U. S. Atomic Energy Commission, TID-25363, map scale 1:250,000, 1970.

# CHAPTER 3 SEISMIC WAVES AND STRONG-MOTION SEISMOGRAPHS

## 3.1 Seismic Waves

Ground motion is excited by the propagation of waves which emanate from the hypocenter (source) of an earthquake. There are four basic seismic waves: two preliminary "body" waves which travel through the earth and two which travel only at the surface. The total wave propagation pattern is shown schematically in figure 3.1. Combinations, reflections, and diffractions produce countless number of waves of other types. In addition, a large earthquake generates inelastic waves.

The two body waves are the primary (P) wave and the secondary (S) wave. The P wave travels about twice as fast as the S wave, and is the first instrumental indication that an earthquake has occurred.

The P wave is longitudinal, like a sound wave, and propagates through both liquids and solids. It travels about 4 miles per second or nearly 15,000 miles per hour. As the compressional P wave passes through the earth's crust, an object embedded in the ground or on the surface is subjected to a series of sharp pushes and pulls parallel to the wave path-motion similar to that which the passengers feel when a long train gets under way [3.1].

The S wave is transverse, like a light or radio wave, and travels barely more than half as fast as the P wave. As this wave travels, it displaces objects at right angles to the direction of wave travel. There is hardly any vertical motion associated with the S wave, as the vertical component is damped by the opposing force of gravity, however, side-to-side shaking in the horizontal plane can be extremely destructive.

Surface waves, Love and Rayleigh, are of much greater length and period, such as, 30 seconds or more versus less than one second for P waves. The Love wave produces lateral shear in the horizontal plane, and the Rayleigh wave induces a retrograde, elliptical motion, similar to wind-driven ocean waves. The speed of the Love wave is about 2.5 miles per second and the Rayleigh wave is about 10 percent slower [3.1].

Because the waves generated by an earthquake travel at different speeds, the seismic waves arrive at a given point at different times. A recording station located at a great distance could record a single event for days as reflected, combined, and resonated waves propagate through the earth's

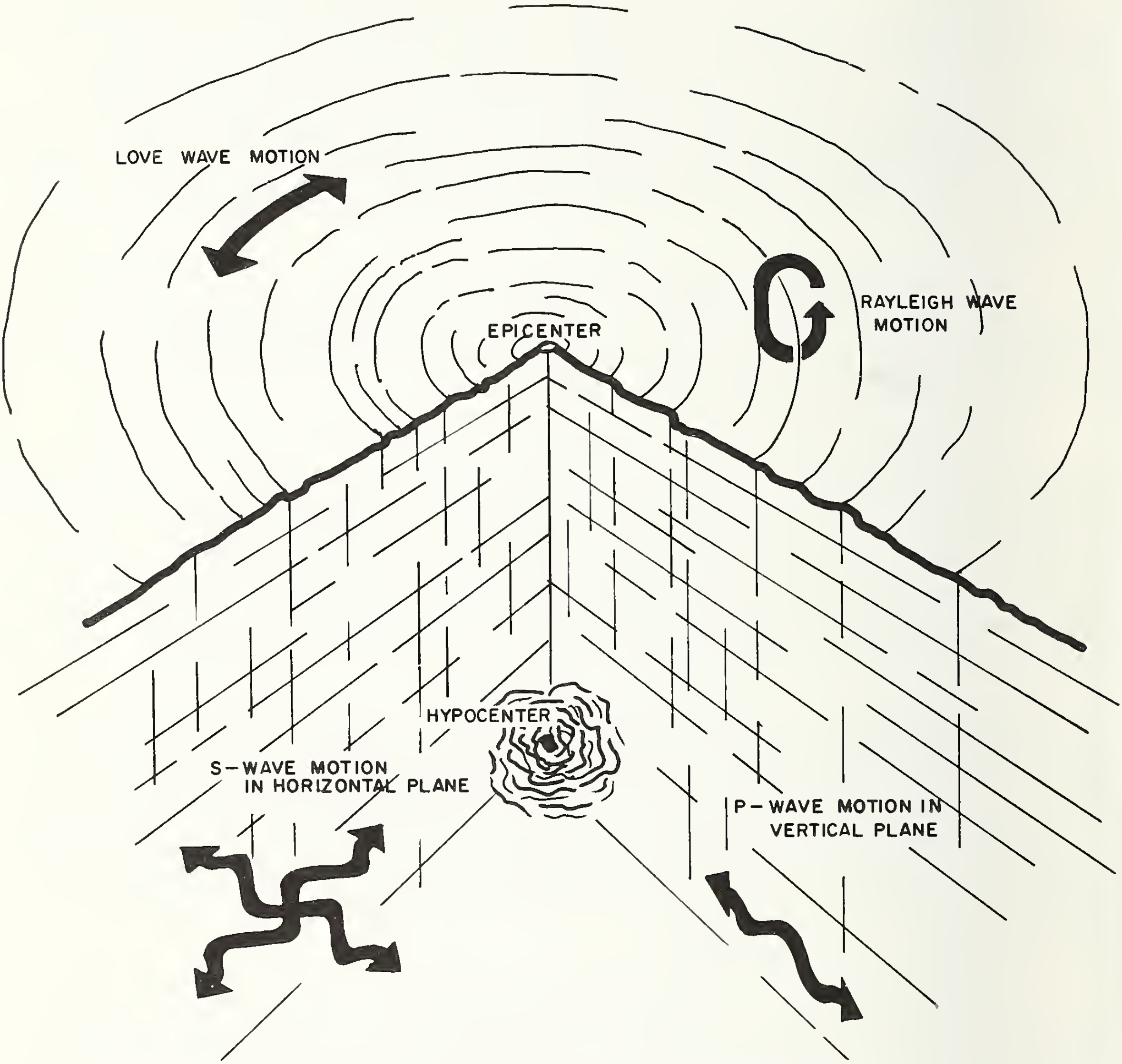



*Figure 3.1 Elastic earthquake generated waves.*

mantle and core.

The first indication of an earthquake is signaled by the arrival of the compressional (P) waves. This will be followed by the shear (S) waves and then the "ground roll" caused by the surface waves. When compared with the body waves (P and S), the surface waves usually have the stronger vibrations and probably cause most earthquake damage.

The vibrations induced by earthquakes are detected, recorded, and measured by seismographs. In the region of strong ground motions near a high-energy source, "strong-motion" seismographs are used. These seismographs are usually installed in tall and large buildings, in some dams and bridges, and at nuclear reactor sites. They are triggered by the earthquake-generated vibration above a given threshold amplitude, so that the onset of the earthquake motion is usually lost. However, the lost portion of the record has little value to earthquake engineering which is primarily concerned with the earthquake-resistance design of structures.

## 3.2 Strong-Motion Records

The San Fernando earthquake provided a large number of strong-motion acceleration records. These will undoubtly supply valuable data on ground and building motions. More than 250 strong-motion seismographs were in southern California at the time of the earthquake, of which more than 200 were in the Los Angeles area. These seismographs were installed primarily in large and tall buildings as a direct result of the inclusion of provisions in the building codes of the City of Los Angeles and of other adjoining municipalities requiring the installation of seismographs at three different levels in these buildings (see Chapter 5 for the Los Angeles area building code requirements). Most of the strong-motion seismographs are privately owned but are maintained by the Seismological Field Survey unit of National Oceanic Atmospheric Administration's National Ocean Survey as part of a cooperative network [3.2].

A typical strong-motion seismograph measures on paper or film the duration and amplitude of three acceleration components, one vertical and two mutually perpendicular horizontal components. In general, the seismograph at the basement level of a building provides a reference record of actual ground motion at a specific site and the two seismographs at the upper stories indicate response of the building. Both velocities and displacements can be obtained by integration of the acceleration data. They then can be used to compare actual performance with the performance predicted by design. In a damaged building, information obtained from such comparison could serve as an aid in locating likely overstressed members in a building for repair work.

The large number of records obtained in this earthquake will supply, for the first time, a wealth of data on the character of the ground motion and the response of structures to strong ground motion. It is hoped that study of these data will answer some of the fundamental questions in earthquake engineering, such as how much the local geology affects ground motion, and what level of energy induced by ground acceleration dissipates in buildings.

## 3.3 Strong-Motion Seismograph Data

Strong-motion seismographs located in southern California outside the Los Angeles area are shown in figure 3.2. The locations of these seismographs and their recorded accelerations are listed in table 3.1 at the end of this chapter. The table gives for each location the maximum acceleration of the vertical component (V) and the two mutually perpendicular horizontal components (H1 and H2), expressed as a fraction of gravity along with approximate periods corresponding to these acceleration components. The Old Ridge Route station at Castaic (location 6), some 15 miles away from the epicenter, shows the high value of 0.388g (gravity) in the horizontal direction and 0.178g in the vertical direction. Similarly, one station (No. 12) at Lake Hughes (location 7), some 10 miles east of Castaic, showed peak acceleration values of 0.371g in the horizontal and 0.178g in the vertical directions.

The locations of seismographs in the Los Angeles area are shown in figure 3.3 and recorded accelerations are listed in table 3.2 at the end of this chapter. Note in figure 3.3 that the nearest seismographs to the City of San Fernando are locations No. 28 and No. 29. The maximum accelerations at location No. 28, a Holiday Inn, at the ground floor were 0.276g in the horizontal direction and 0.171g in the vertical direction.

Preliminary analysis of the acceleration record obtained at Pacoima Dam (location 29) 5 miles south of the epicenter, indicates that both horizontal components exceeded 1.25g, the largest motion ever recorded from an earthquake. The vertical component had a single peak amplitude of 0.7g. The acceleration records indicate that during the first 12 seconds, the ground at the dam was subjected to almost continuous accelerations ranging from 0.5g to 0.7g. The highest acceleration level ever recorded previously in the United States was 0.5g, noted during the Parkfield, California earthquake in 1966 [3.3]. The unexpected magnitude of the ground

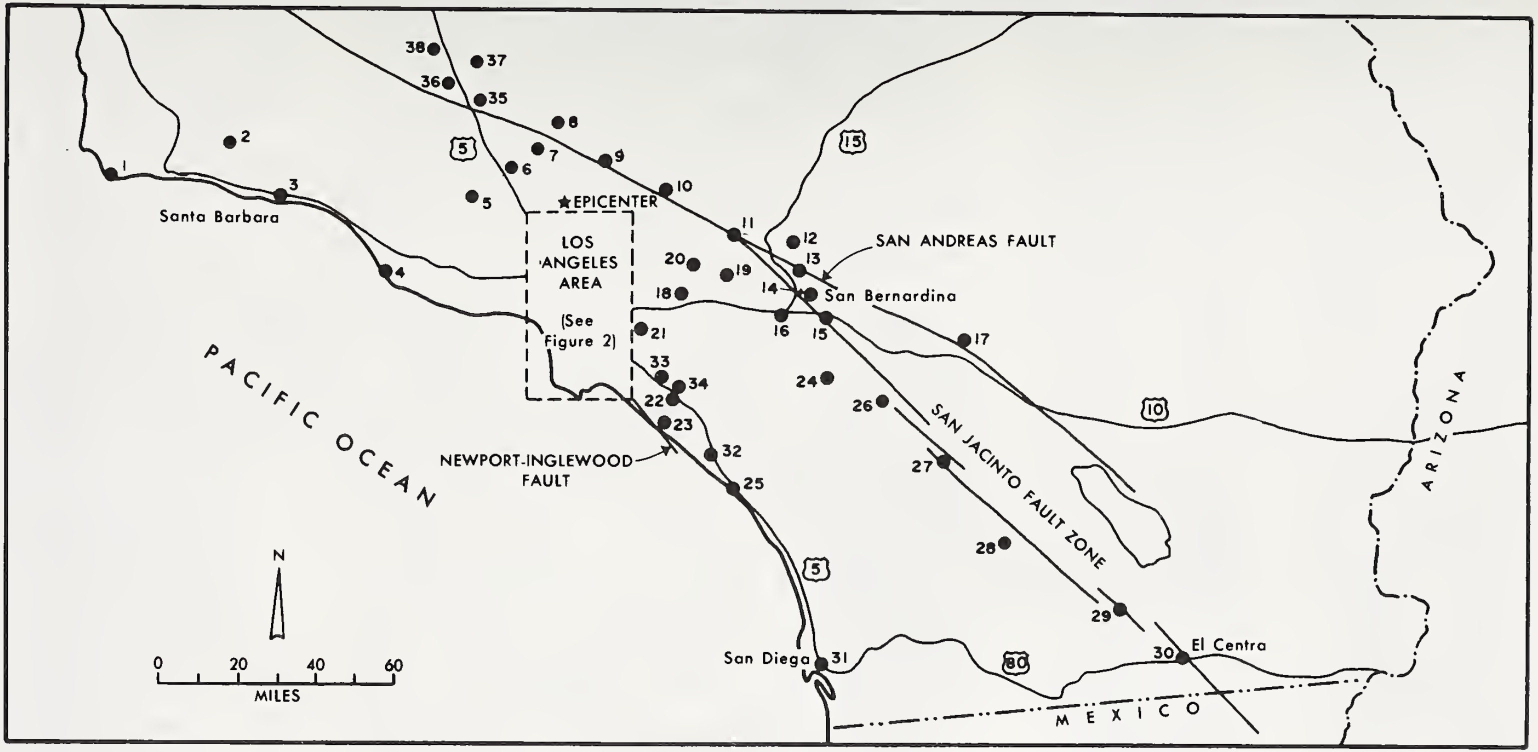



Figure 3.2 Location of strong-motion seismographs in Southern California [3.2].

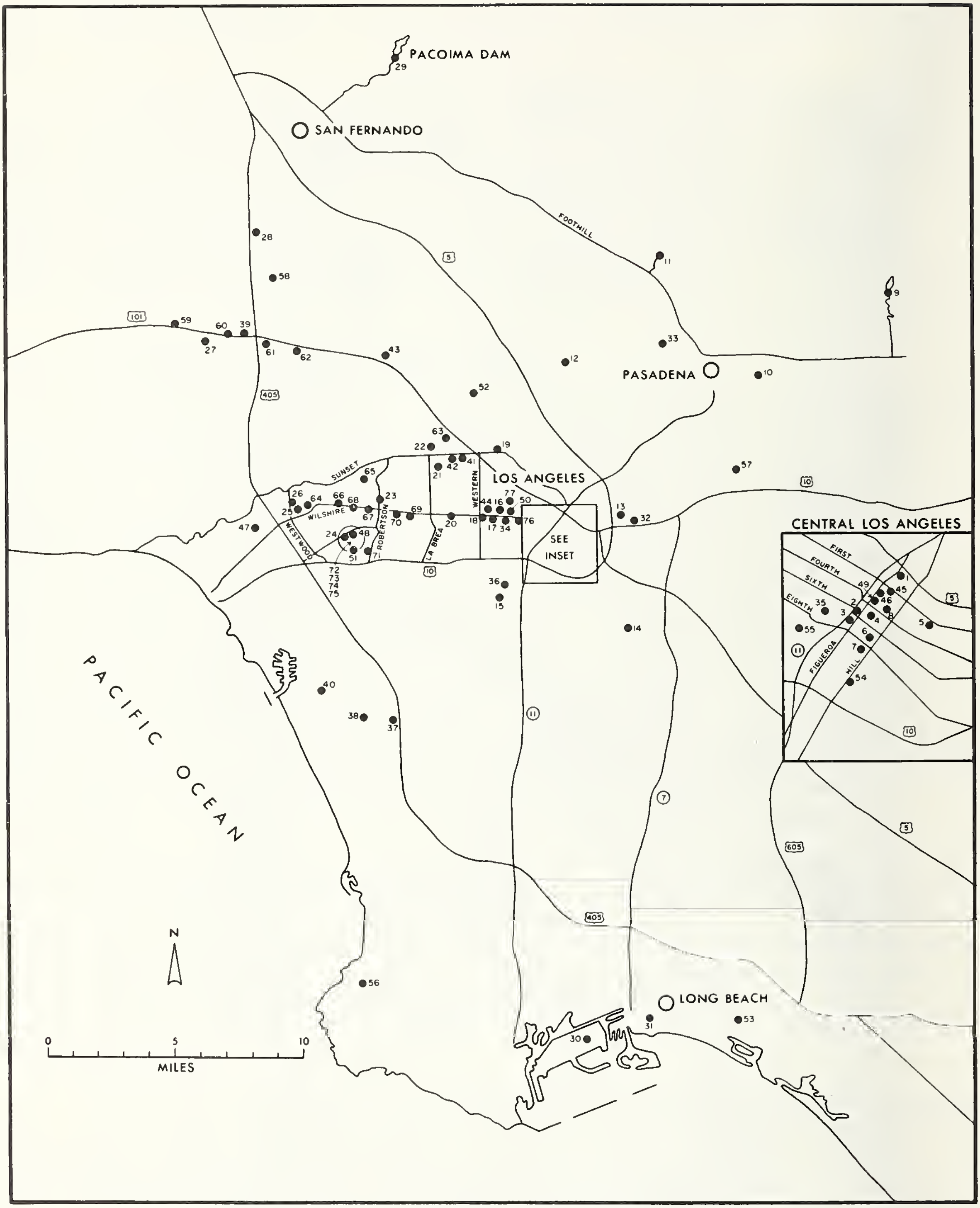



*Figure 3.3 Location of strong-motion seismographs in the Los Angeles Area [3.2].*

acceleration at the Pacoima Dam will change the thinking of many engineers and seismologists who previously thought 0.5g to be a practical maximum that could be transmitted by an earthquake. It is clearly seen in figure 3.4 that a proposed empirical relationship between the maximum acceleration and the epicentral distance based on measurements made in previous U. S. earthquakes [3.4] is not applicable to the San Fernando earthquake.

The ground level accelerations in the Los Angeles area, measured much farther from the epicenter than the Pacoima Dam, ranged from 0.10g to 0.23g in the horizontal direction and from 0.03g to 0.17g in the vertical direction. While larger values (greater than 0.2g) of acceleration were noted at stations near or in the San Fernando Valley, a few locations in downtown Los Angeles also experienced more than 0.2g.

The production of copies of the accelerograms and digitization of the records were carried out at the California Institute of Technology. The accelerograms reconstructed from the digitized records from the Pacoima Dam, the calculated relative velocity response spectra, and ground displacements from recorded accelerograms are shown in figures 3.5, 3.6, and 3.7. Maximum response was in the period of 0.3 - 0.4 second in the east-west direction, 0.4 and 1.6 second in the north-south direction and 0.2 - 0.4 second in the vertical direction. It should, however, be pointed out that high ground accelerations by themselves do not reflect damaging characteristics of an earthquake. Undamaged Pacoima Dam clearly points out this fact. As more digitized information on buildings become available, the response of each structure to the earthquake as affected by its natural period can be studied to assist in explaining structural damages sustained by many buildings in the San Fernando Valley area. The acceleration records as obtained from a number of field stations are presented in figures 3.8 through 3.18.

### 3.4 Summary

The San Fernando earthquake provided an opportunity, for the first time, to collect a wealth of acceleration records. This is a direct result of the deployment of strong-motion seismographs in buildings and other large structures by means of including provisions in building codes requiring the installation of seismographs. The acceleration data will undoubtedly serve as the single best source of scientific information for earthquake engineering research. The knowledge gained from study of the data will aid greatly in efforts to reduce hazards from future strong earthquakes.

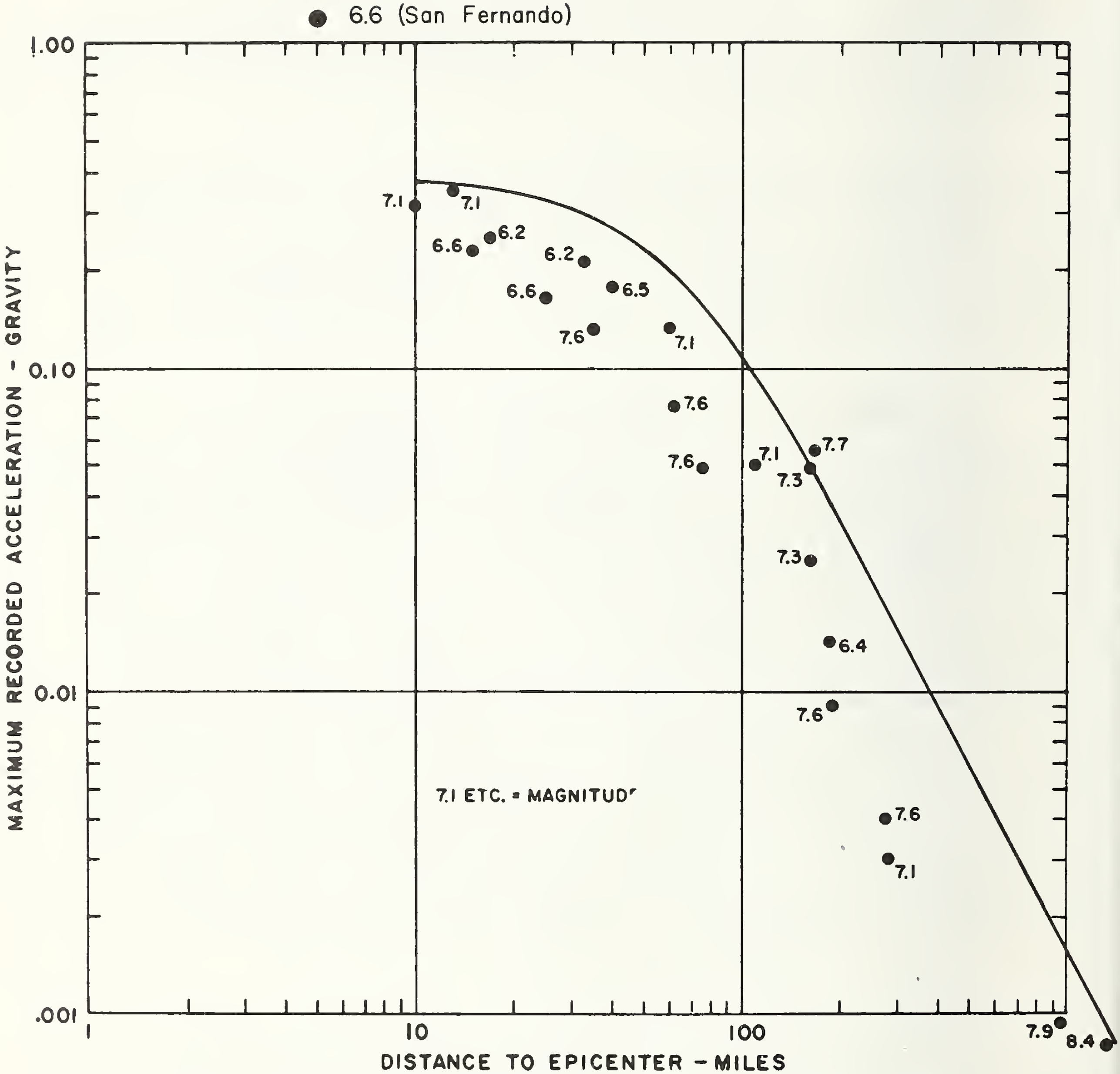
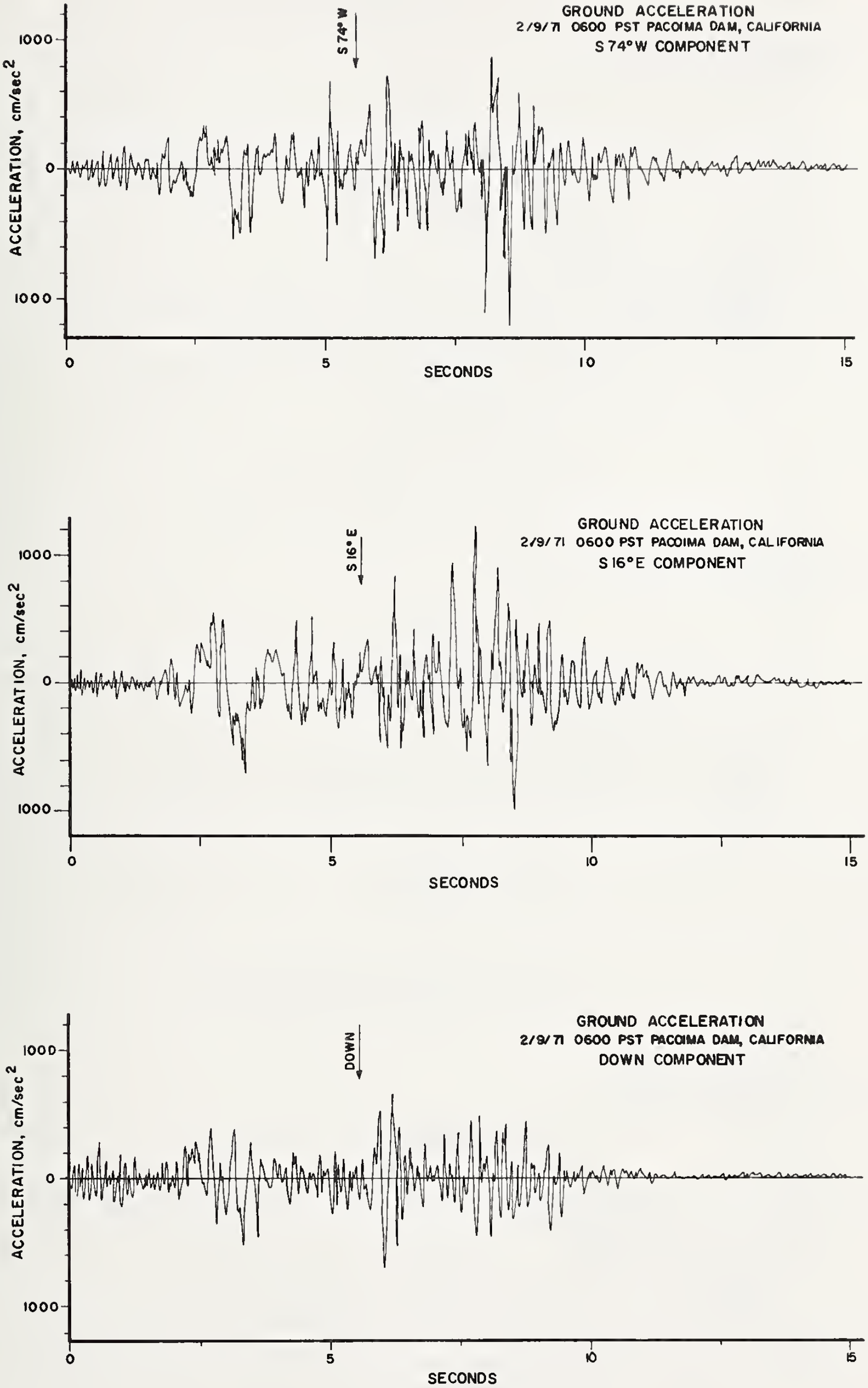
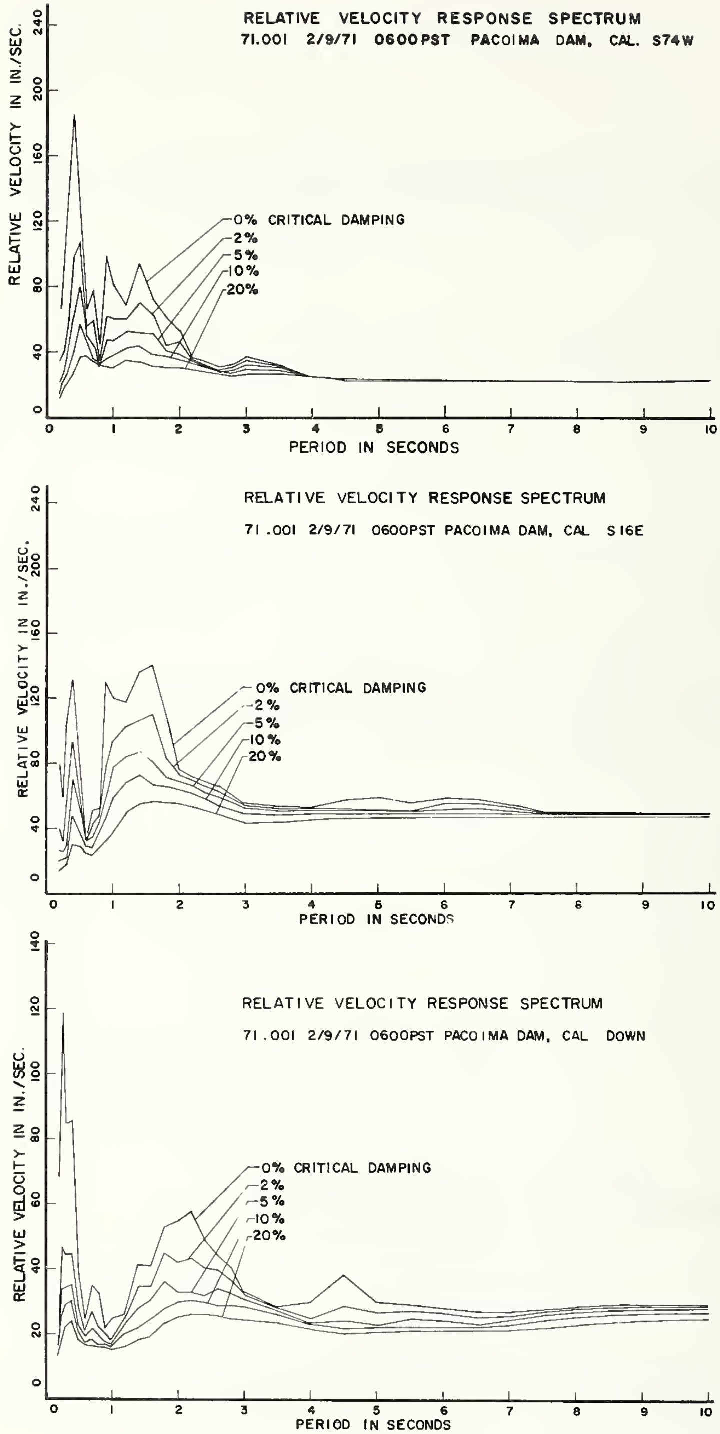



*Figure 3.4 Single-component accelerations recorded from 1933 through 1964 by strong-motion seismographs [3.4].*



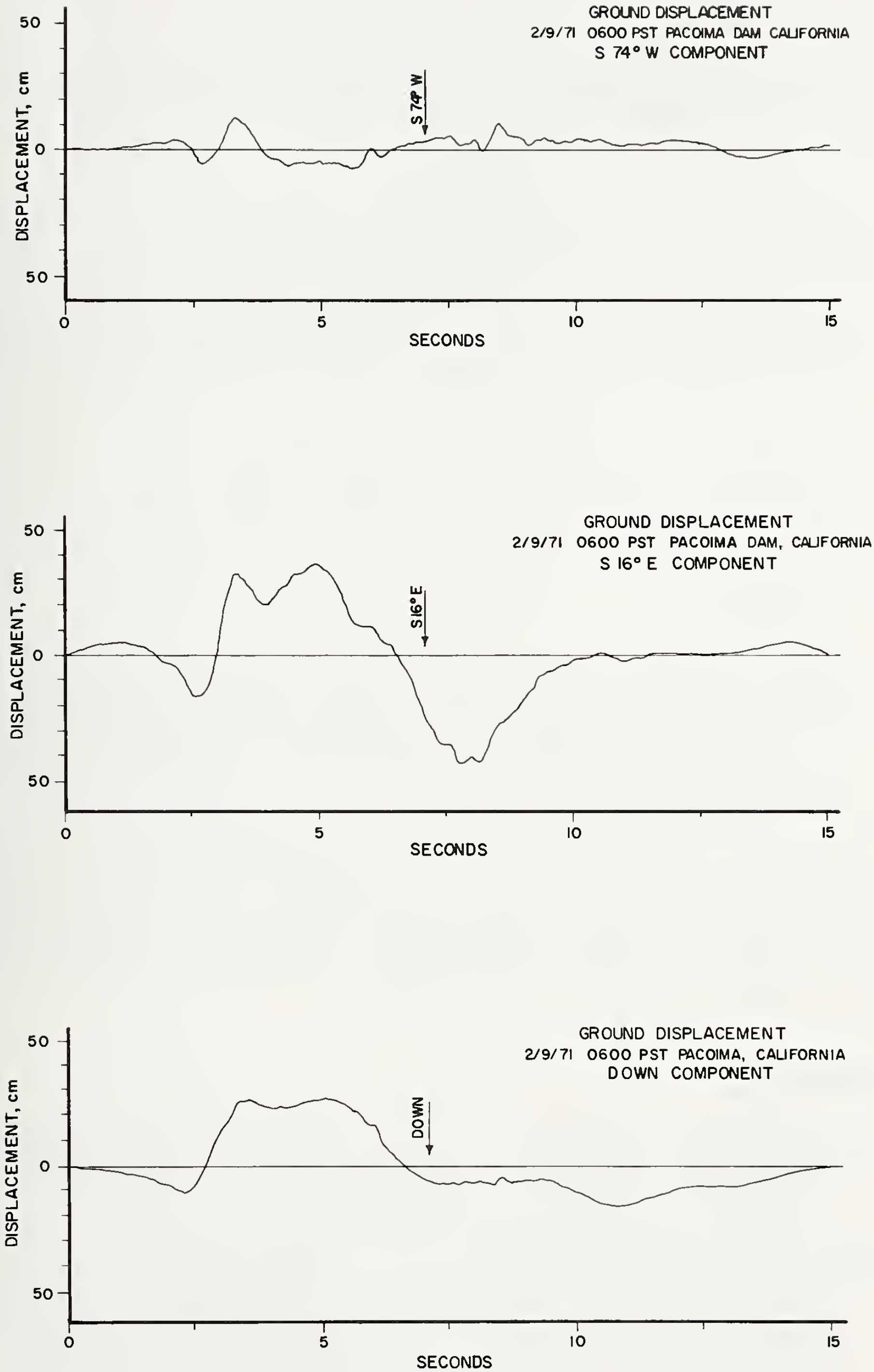

NOAA STRONG-MOTION ACCELEOGRAPH NETWORK AND
THE EARTHQUAKE ENGINEERING RESEARCH LABORATORY
CALIFORNIA INSTITUTE OF TECHNOLOGY

*Figure 3.5 Ground accelerations at Pacoima Dam.*




NOAA STRONG-MOTION ACCELEOGRAPH NETWORK AND
THE EARTHQUAKE ENGINEERING RESEARCH LABORATORY
CALIFORNIA INSTITUTE OF TECHNOLOGY

*Figure 3.6 Relative velocity response spectrum at Pacoima Dam.*




**NOAA STRONG-MOTION ACCELEOGRAPH NETWORK AND THE EARTHQUAKE ENGINEERING RESEARCH LABORATORY CALIFORNIA INSTITUTE OF TECHNOLOGY**

*Figure 3.7 Ground displacements at Pacoima Dam.*

The horizontal ground acceleration at Pacoima Dam was predominantly in a range of 0.5g - 0.7g with a peak in excess of 1.25g. The vertical acceleration had a single peak amplitude of 0.7g. Validation of these unexpected large values may prove to be difficult because the foundation which supported the seismograph cracked, and might have caused some distortions in acceleration recordings at this site. However, a post-earthquake calibration of the seismograph showed that the seismograph performed to specifications. In addition, physical evidence at the Kagel Canyon Fire Station some 2 and 3/4-miles south-southeast from Pacoima Dam indicates that vertical accelerations may have exceeded 1.0g. Morril [3.5] has reported that "A 20-ton fire truck moved 6 to 8 feet fore and apt, 2 to 3 feet sideways without leaving visible skid marks on the garage floor." This movement occurred in spite of the fact that the truck was in gear and the brakes were set. Rubber marks were found on the garage door frame three feet above the ground which appear to have been made by the right rear tire of the fire truck. The metal fender which extended a few inches beyond the tire was not damaged.

Ground accelerations in the downtown Los Angeles areas were, in most cases, less than 0.2g in the horizontal direction and 0.1g in the vertical direction. However, larger values of acceleration were recorded at the top floor of highrise buildings.

In general the systems of accelographs performed well and obtained much useful data. However, there were cases where data were not obtained due to a lack of maintenance of equipment, such as dead batteries, traces too faint to read, or lack of paper in the instrument.

## 3.5 References


3.1 "Earthquakes," National Ocean Survey, National Oceanic Atmospheric Administration, U. S. Government Printing Office, Washington, D. C., 1969.

3.2 Maley, R. P. and Cloud, W. K., "Preliminary Strong-Motion Results from the San Fernando Earthquake of February 9, 1971," Geological Survey Professional Paper 733, U. S. Government Printing Office, Washington, D. C., 1971.

3.3 "Earthquake Information Bulletin, March-April 1971," National Ocean Survey, National Oceanic Atmospheric Administration, U. S. Government Printing Office, Washington, D. C., 1971.

3.4 Cloud, W. K., "Strong-Motion and Building-Period Measurements: The Prince William Sound, Alaska, Earthquake of 1964 and Aftershocks," Vol. II, Part A, National Oceanic Atmospheric Administration, U. S. Government Printing Office, Washington, D. C. 1967.

3.5 Morril, B. J., "Evidence of Record Vertical Accelerations at Kagel Canyon during the Earthquake," Geological Survey Professional Paper 733, U. S. Government Printing Office, Washington, D. C., 1971.

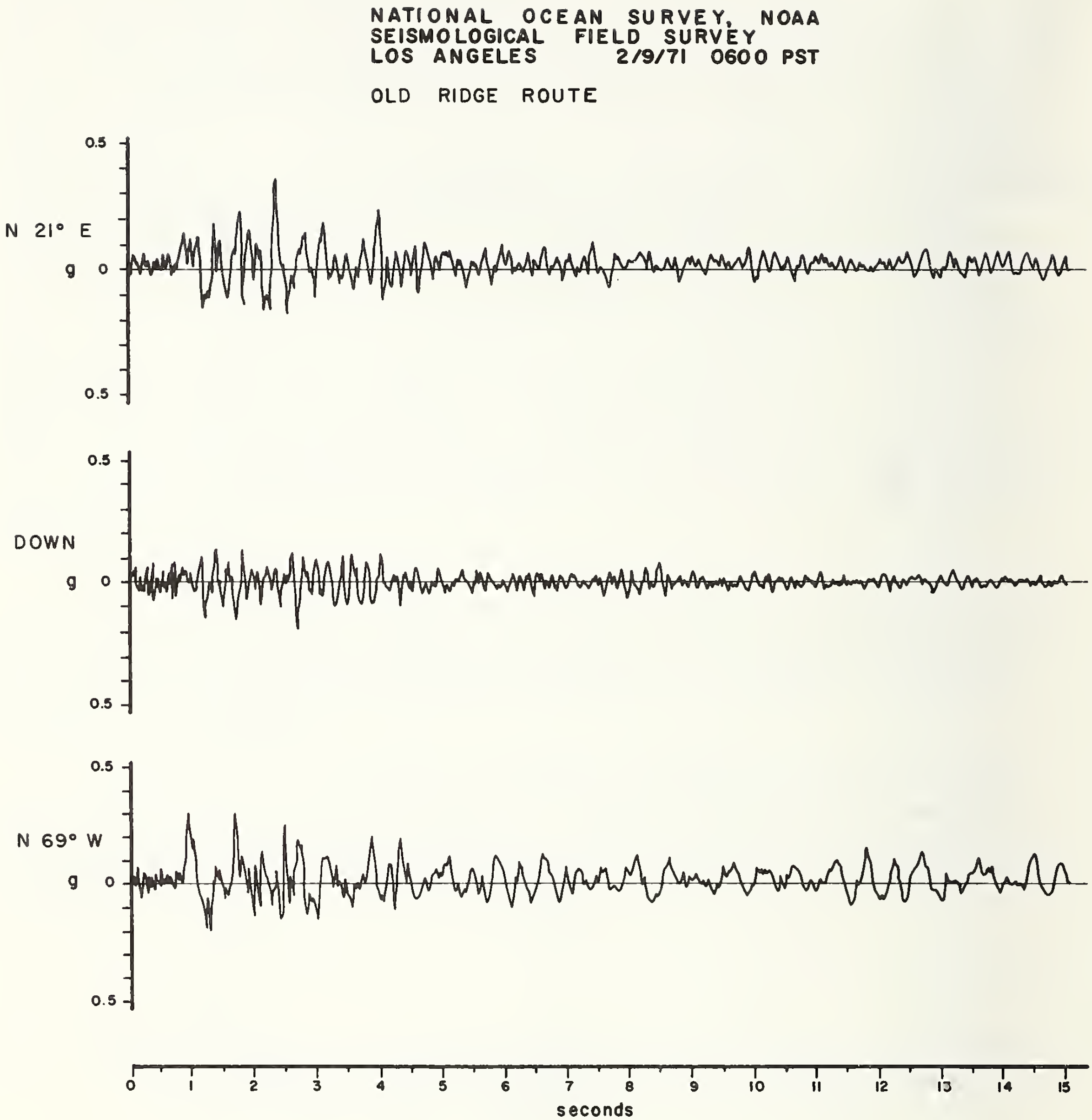



*Figure 3.8 Seismograph record, Castaic, California.*

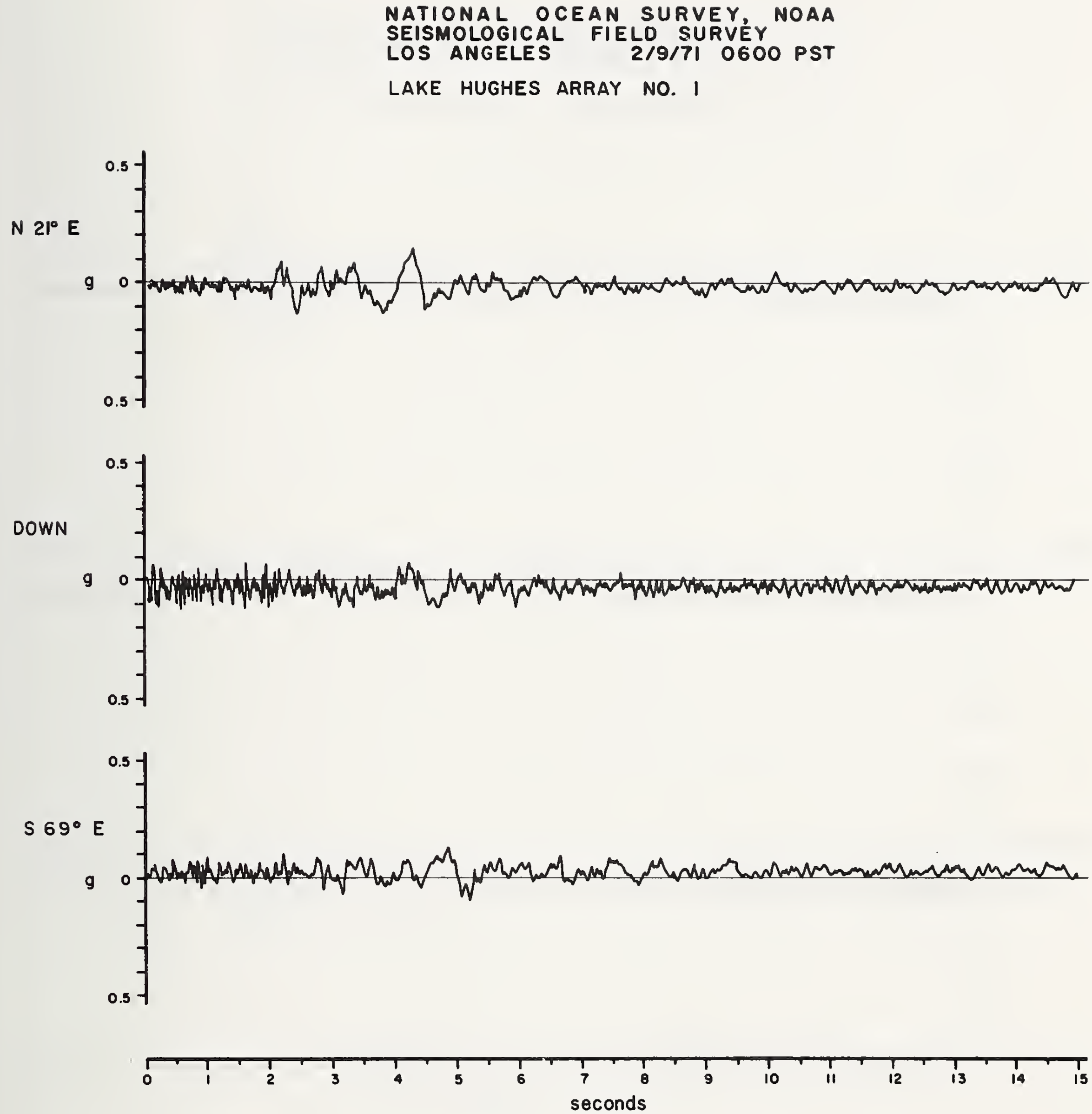



*Figure 3.9a Seismograph record, Lake Hughes, California, Station 1.*

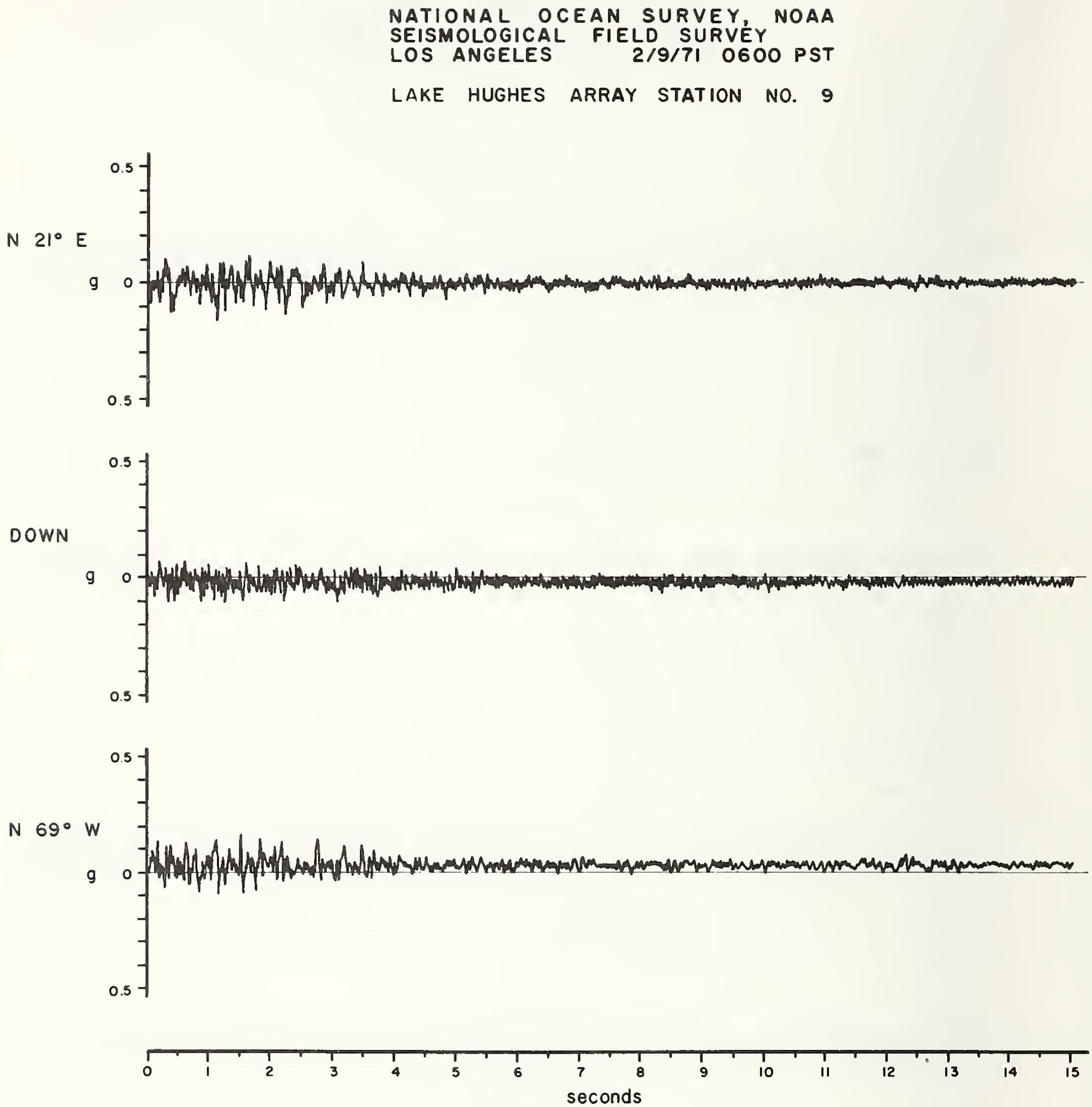



*Figure 3.9b Seismograph record, Lake Hughes, California, Station 9.*

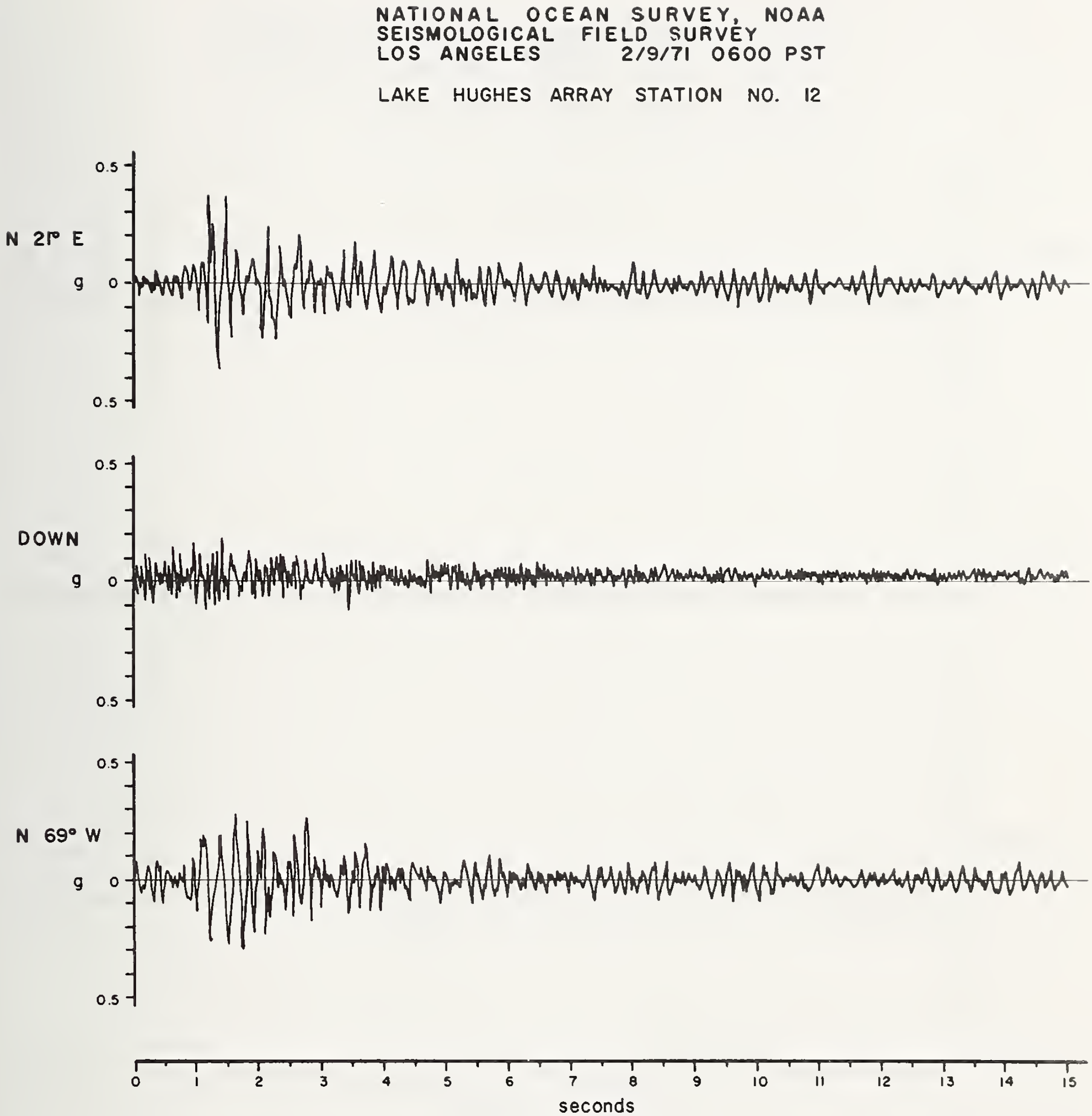



*Figure 3.9c Seismograph record, Lake Hughes, California, Station 12.*

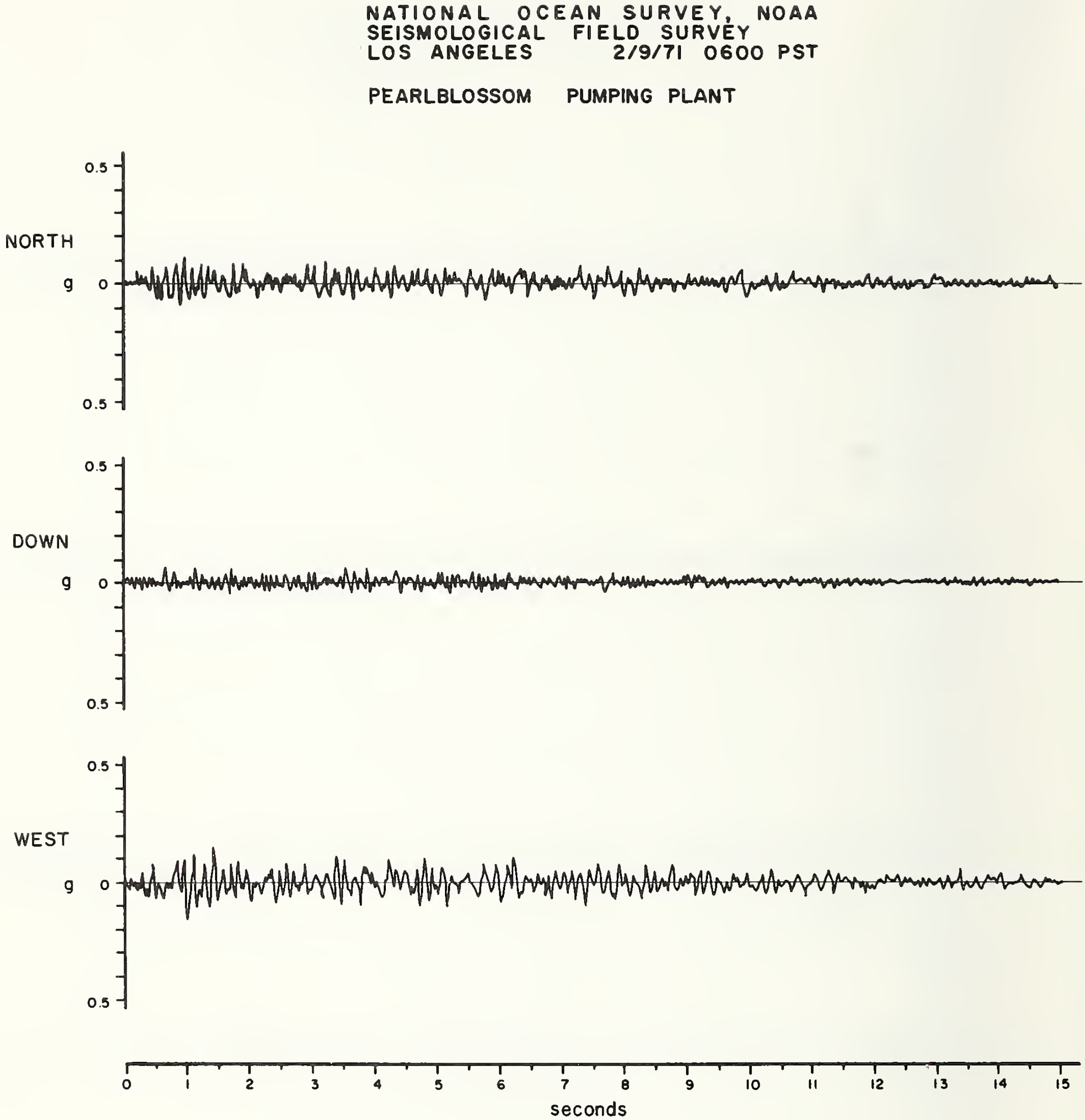



*Figure 3.10 Seismograph record, Pearlblossom, California.*

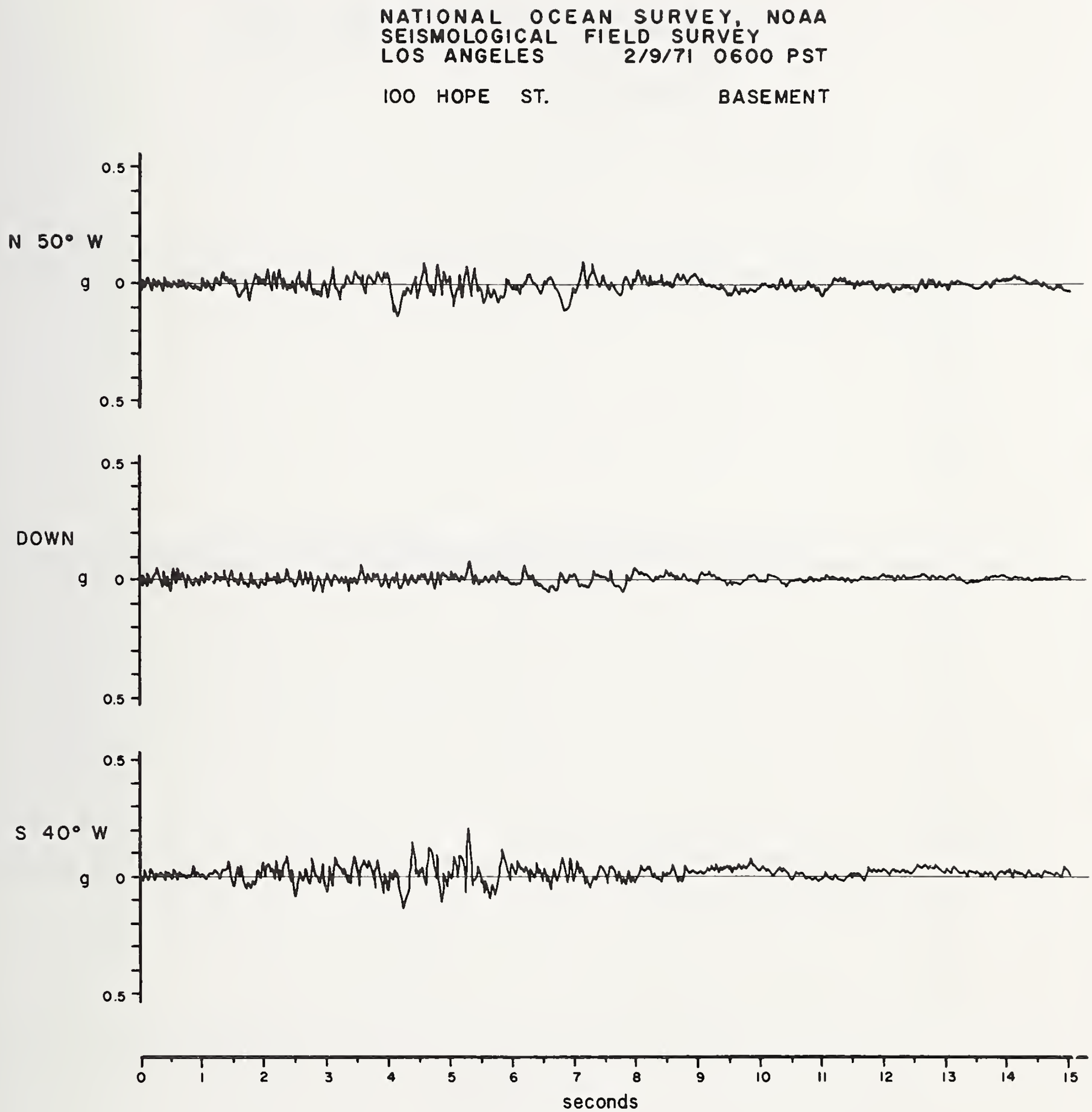



*Figure 3.11a Seismograph record, 100 Hope Street, Los Angeles, basement.*

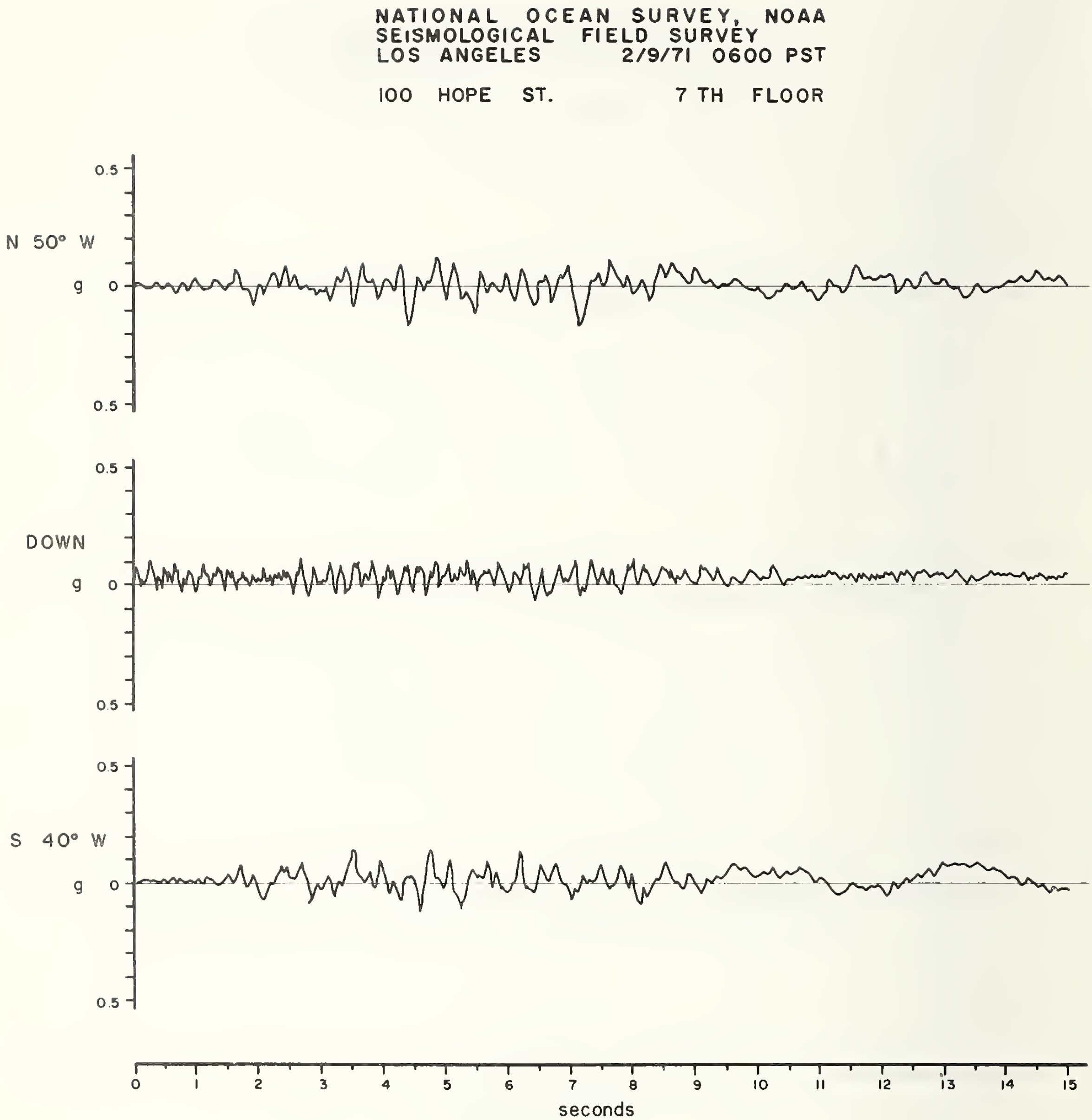



*Figure 3.11b Seismograph record, 100 Hope Street, Los Angeles, 7th floor.*

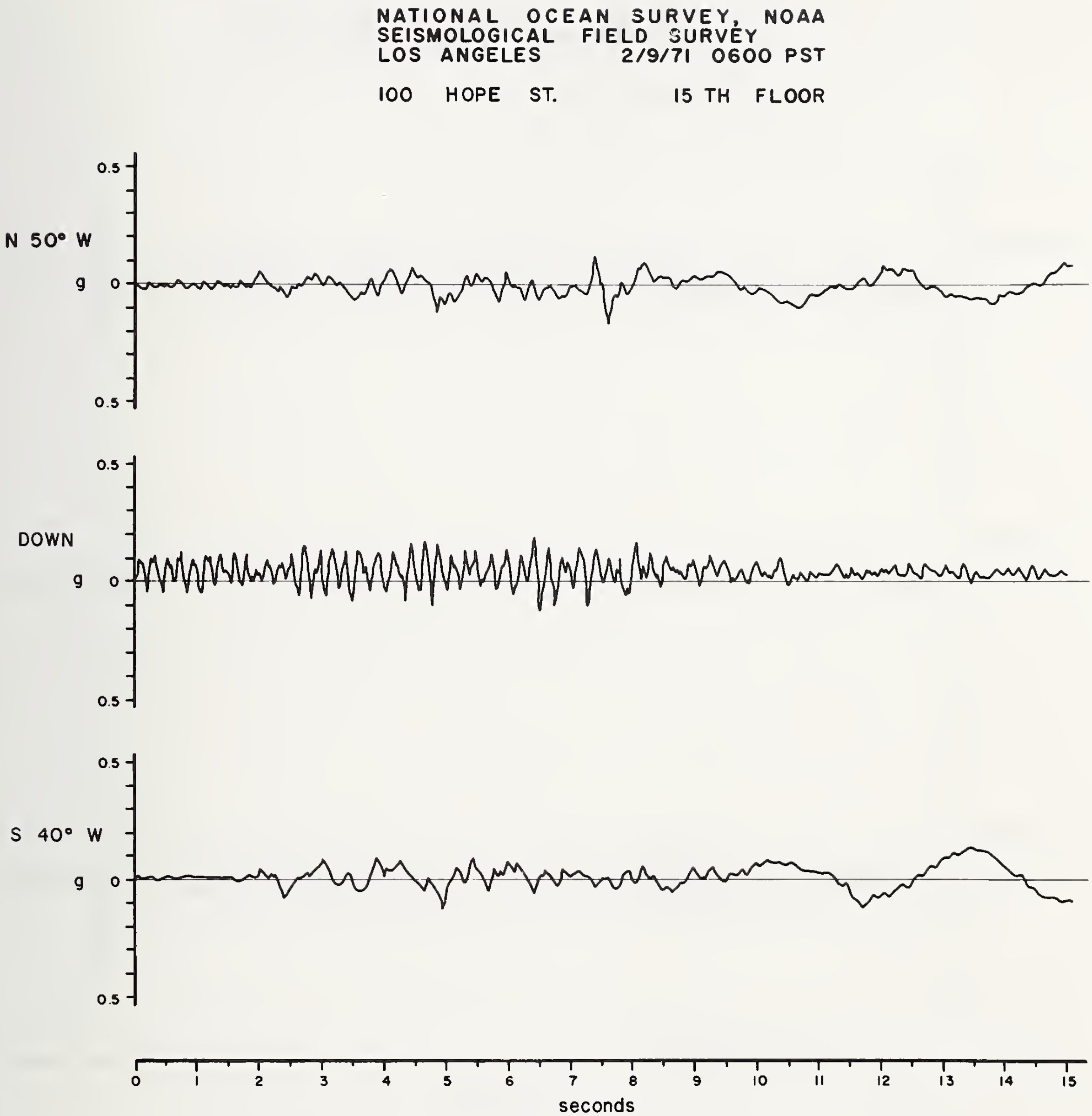



*Figure 3.11c Seismograph record, 100 Hope Street, Los Angeles, 15th floor.*

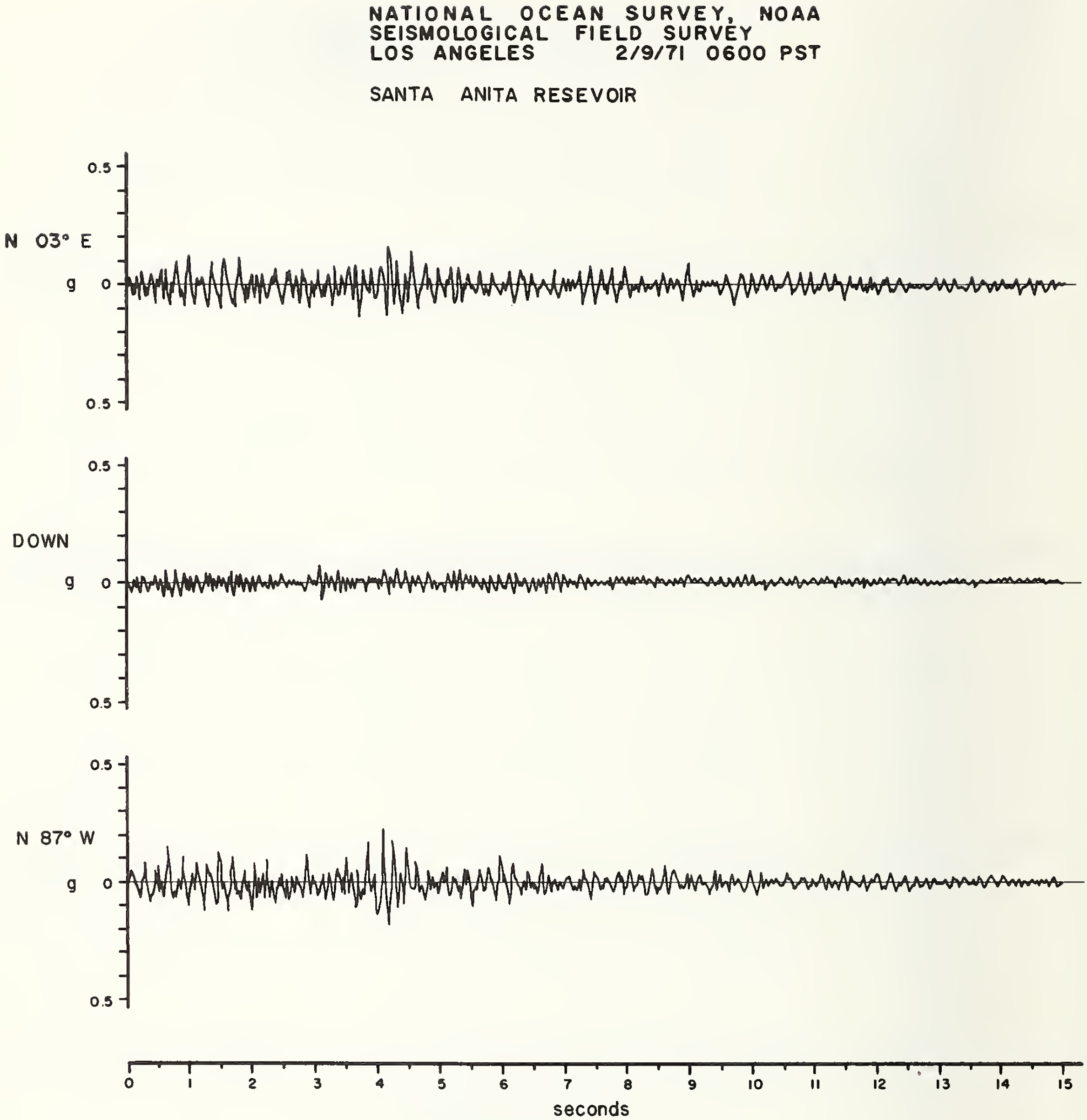



*Figure 3.12 Seismograph record, Santa Anita Reservoir.*

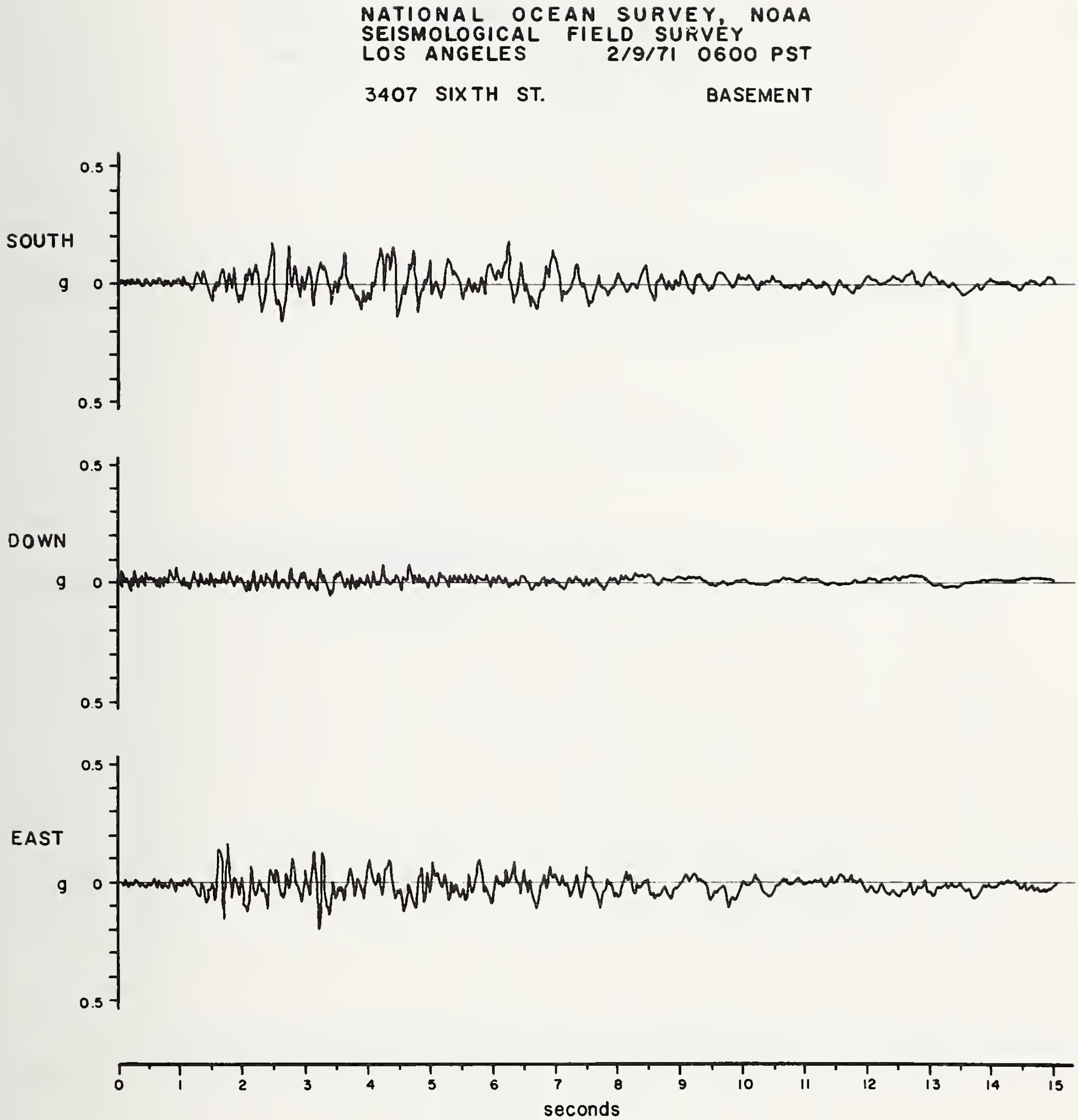



*Figure 3.13a Seismograph record, 3407 Sixth Street, Los Angeles, basement.*

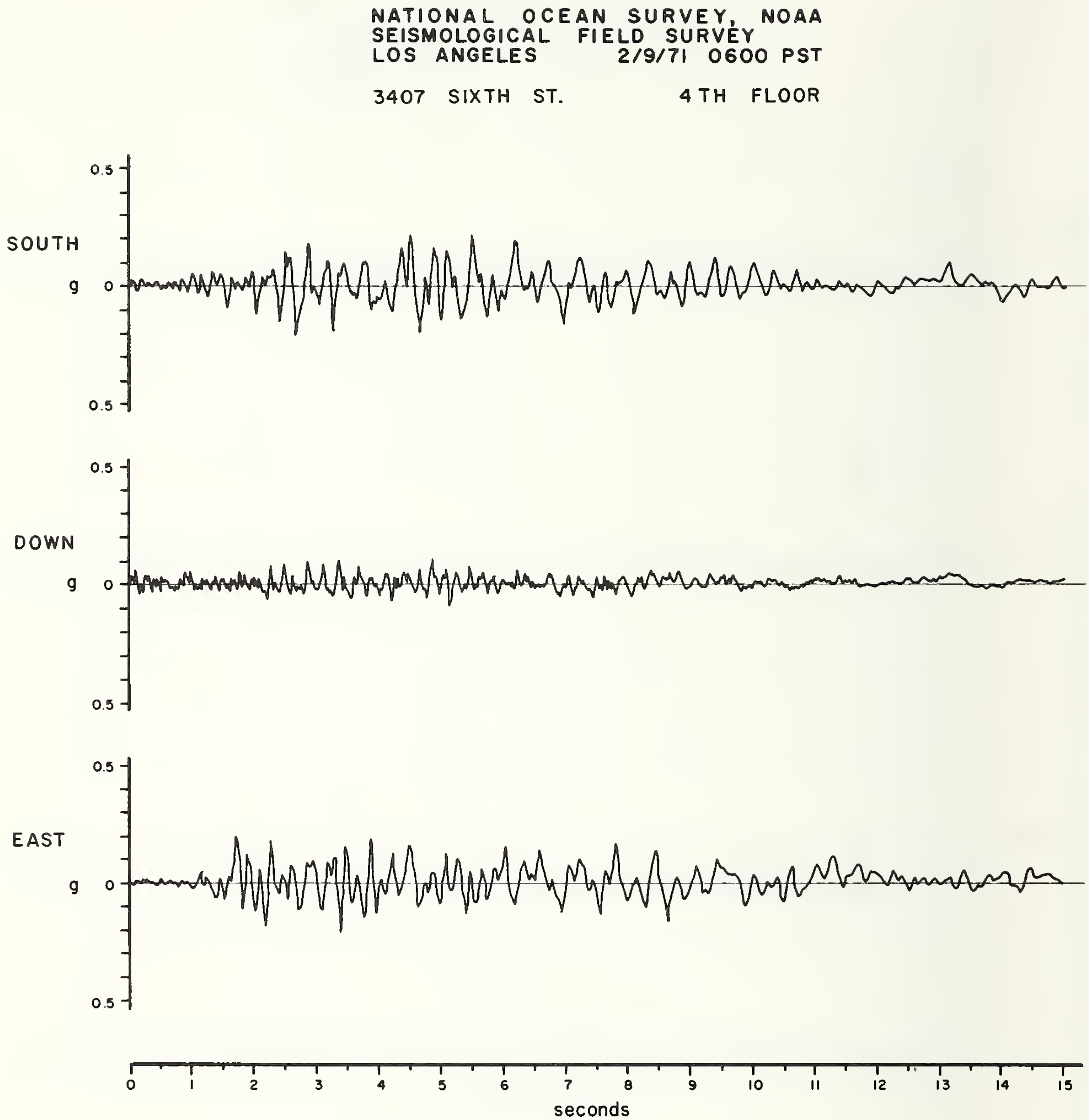



*Figure 3.13b Seismograph record, 3407 Sixth Street, Los Angeles, 4th floor.*

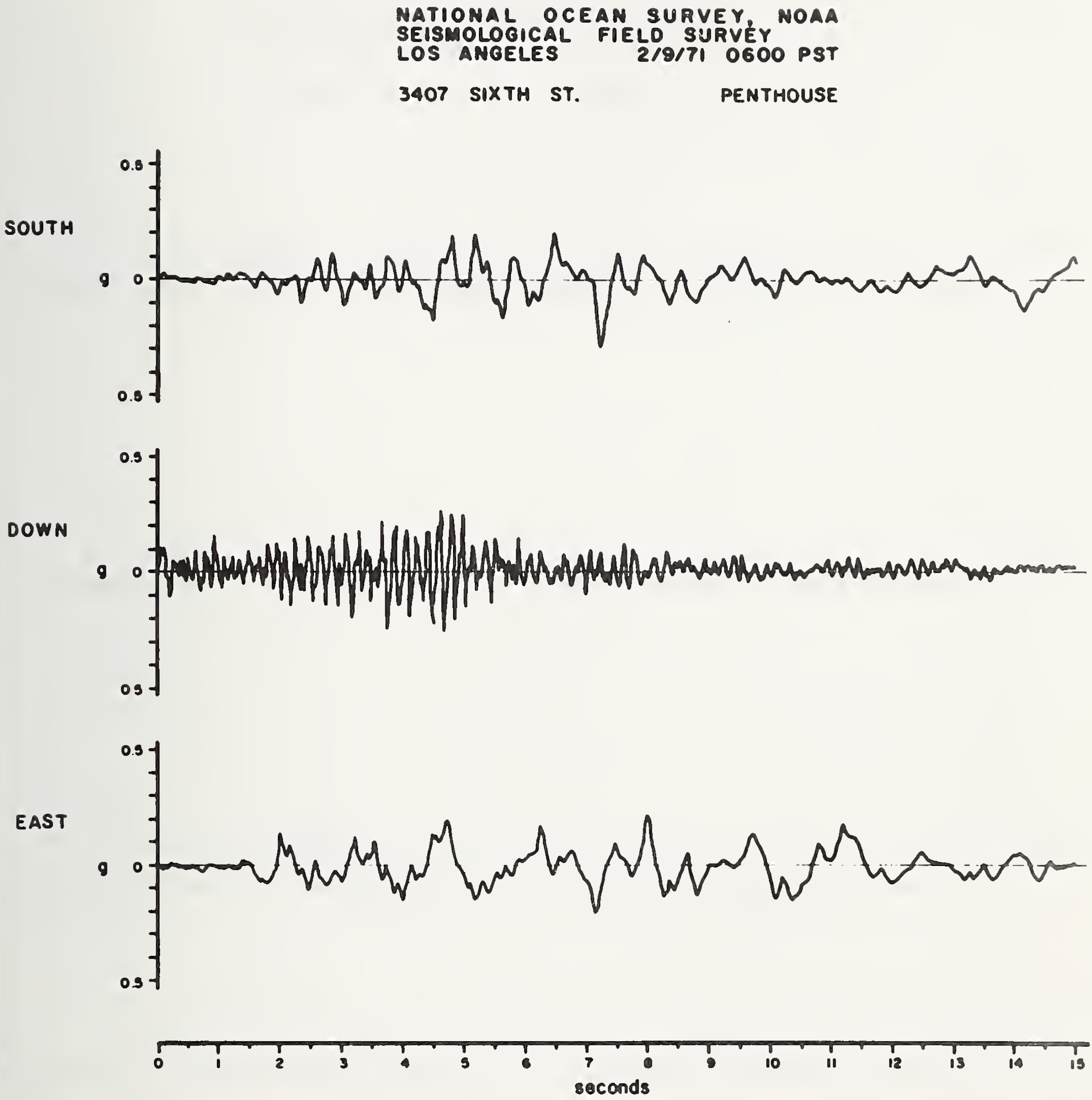



*Figure 3.13c Seismograph record, 3407 Sixth Street, Los Angeles, penthouse.*

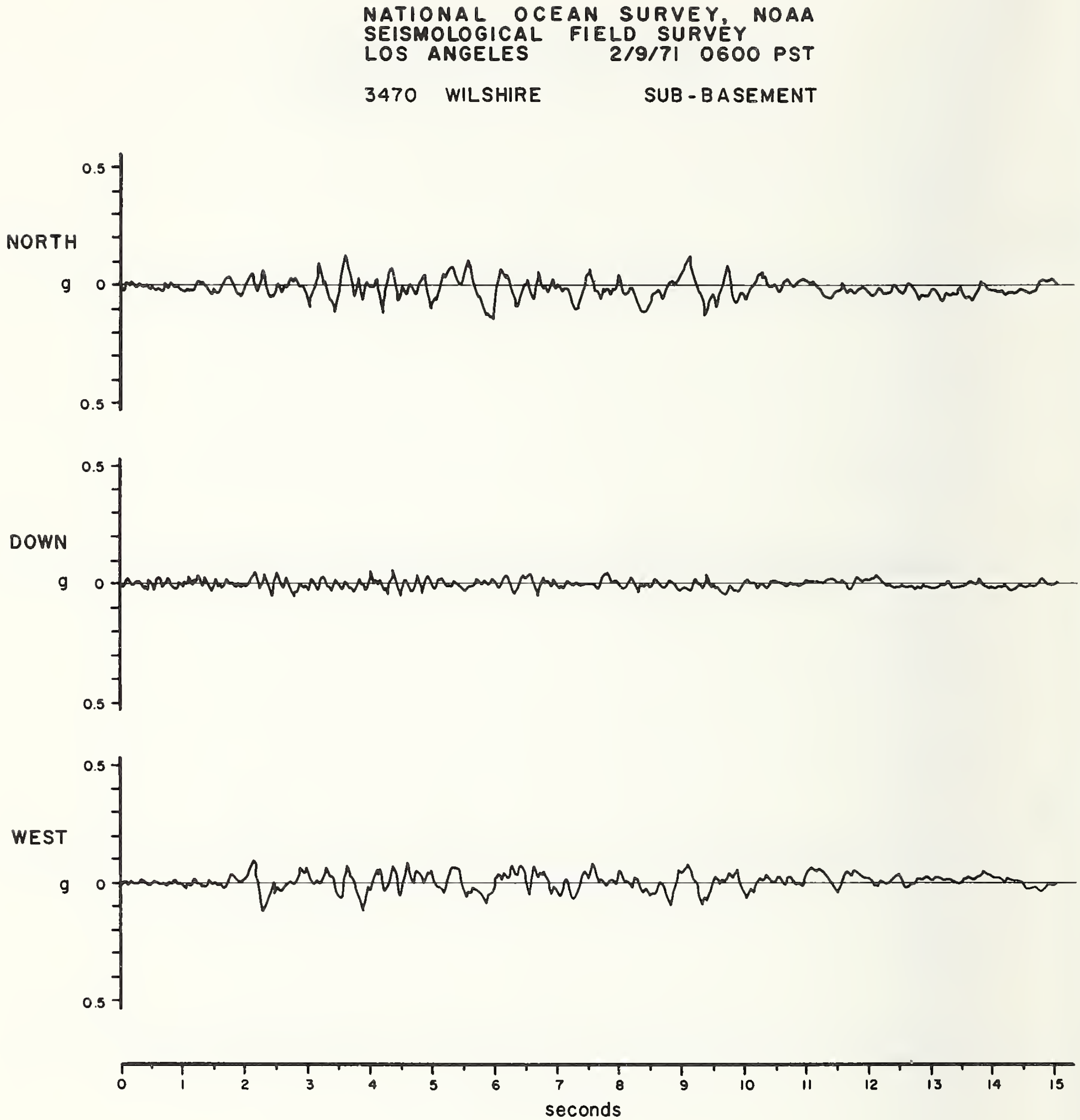



*Figure 3.14a Seismograph record, 3470 Wilshire, Los Angeles, sub-basement.*

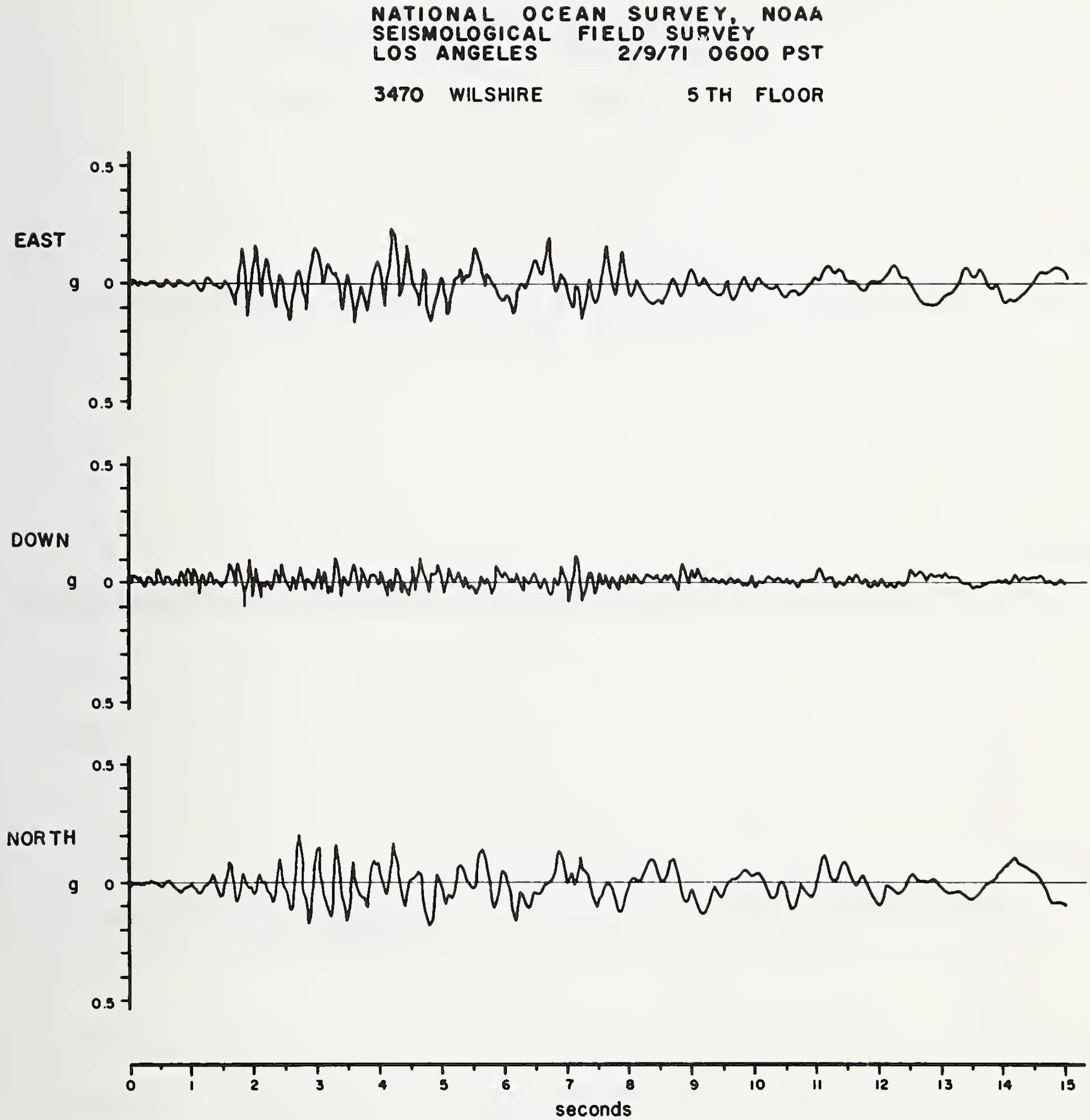



*Figure 3.14b Seismograph record, 3470 Wilshire, Los Angeles, 5th floor.*

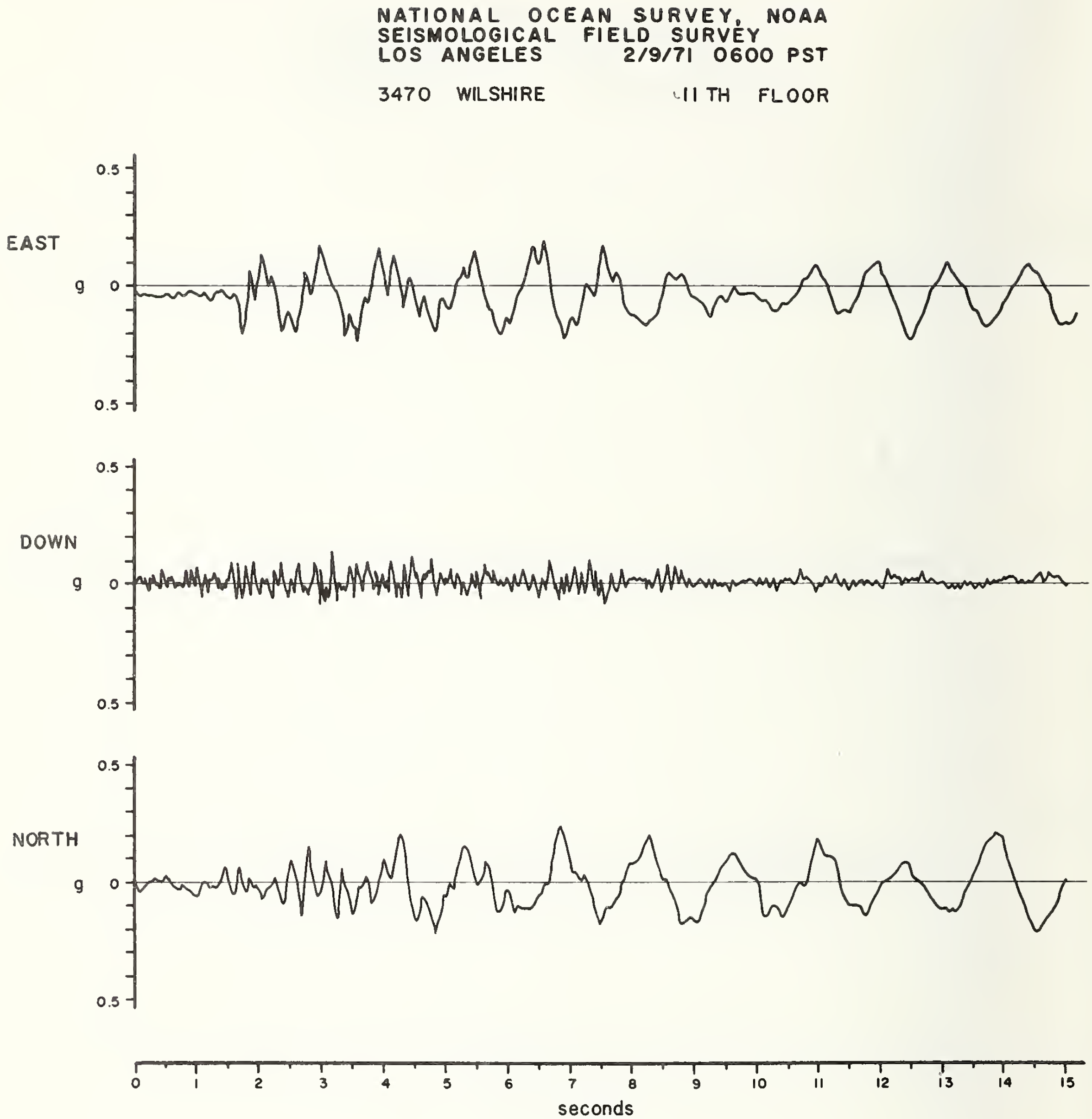



*Figure 3.14c Seismograph record, 3470 Wilshire, Los Angeles, 11th floor.*

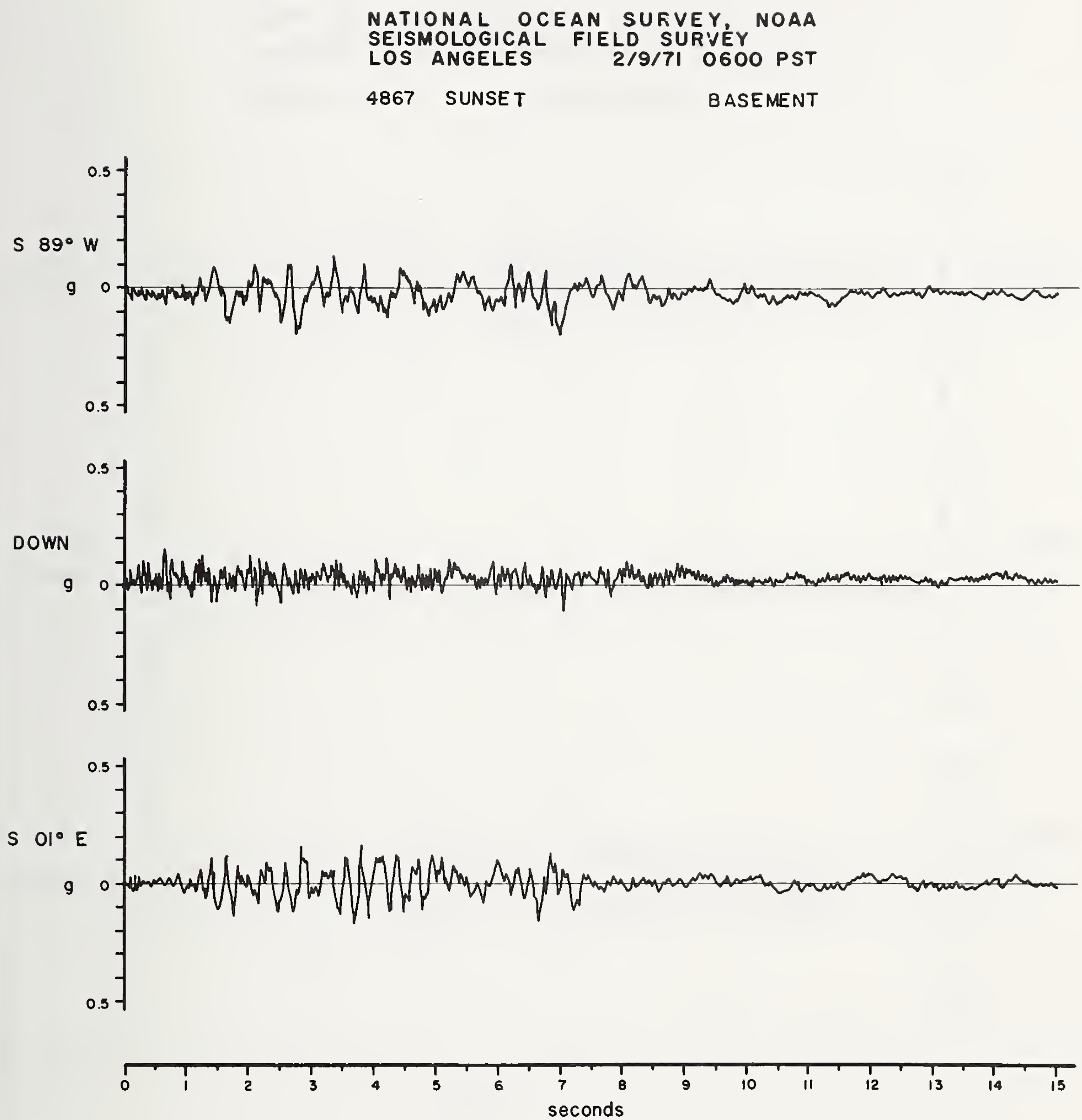



*Figure 3.15a Seismograph record, 4867 Sunset, Los Angeles, basement.*

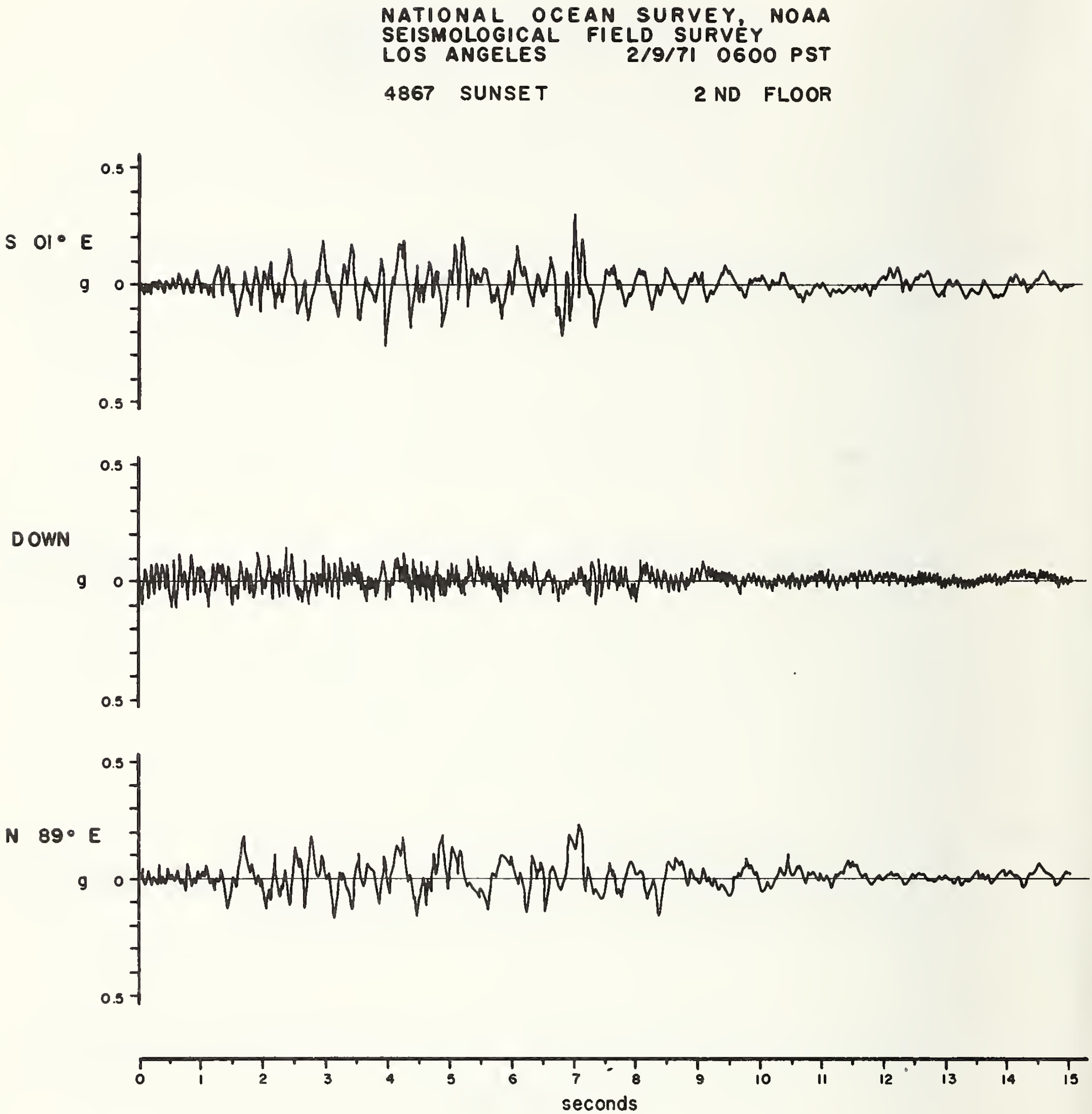



*Figure 3.15b Seismograph record, 4867 Sunset, Los Angeles, 2nd floor.*

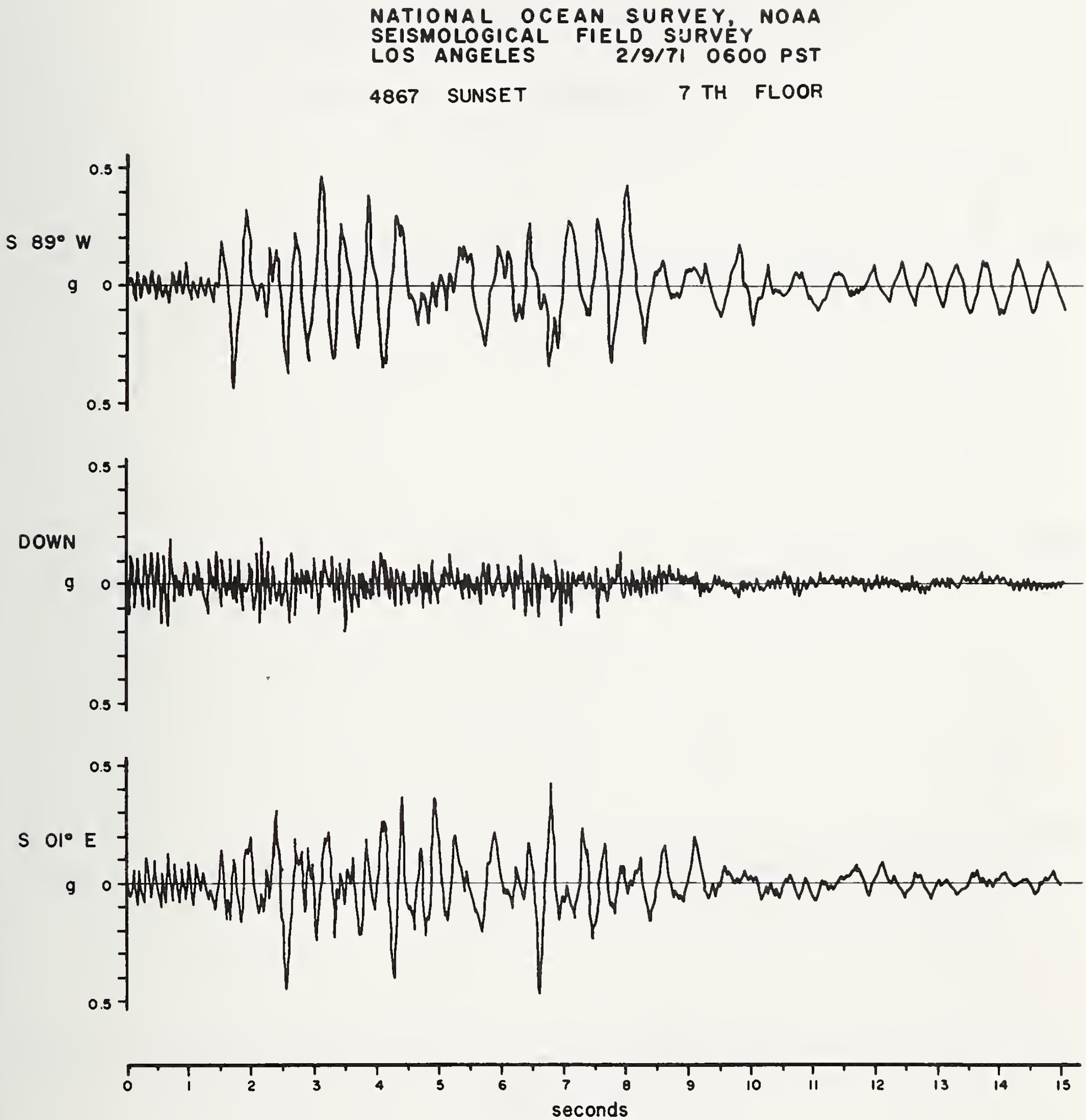



*Figure 3.15c Seismograph record, 4867 Sunset, Los Angeles, 7th floor.*

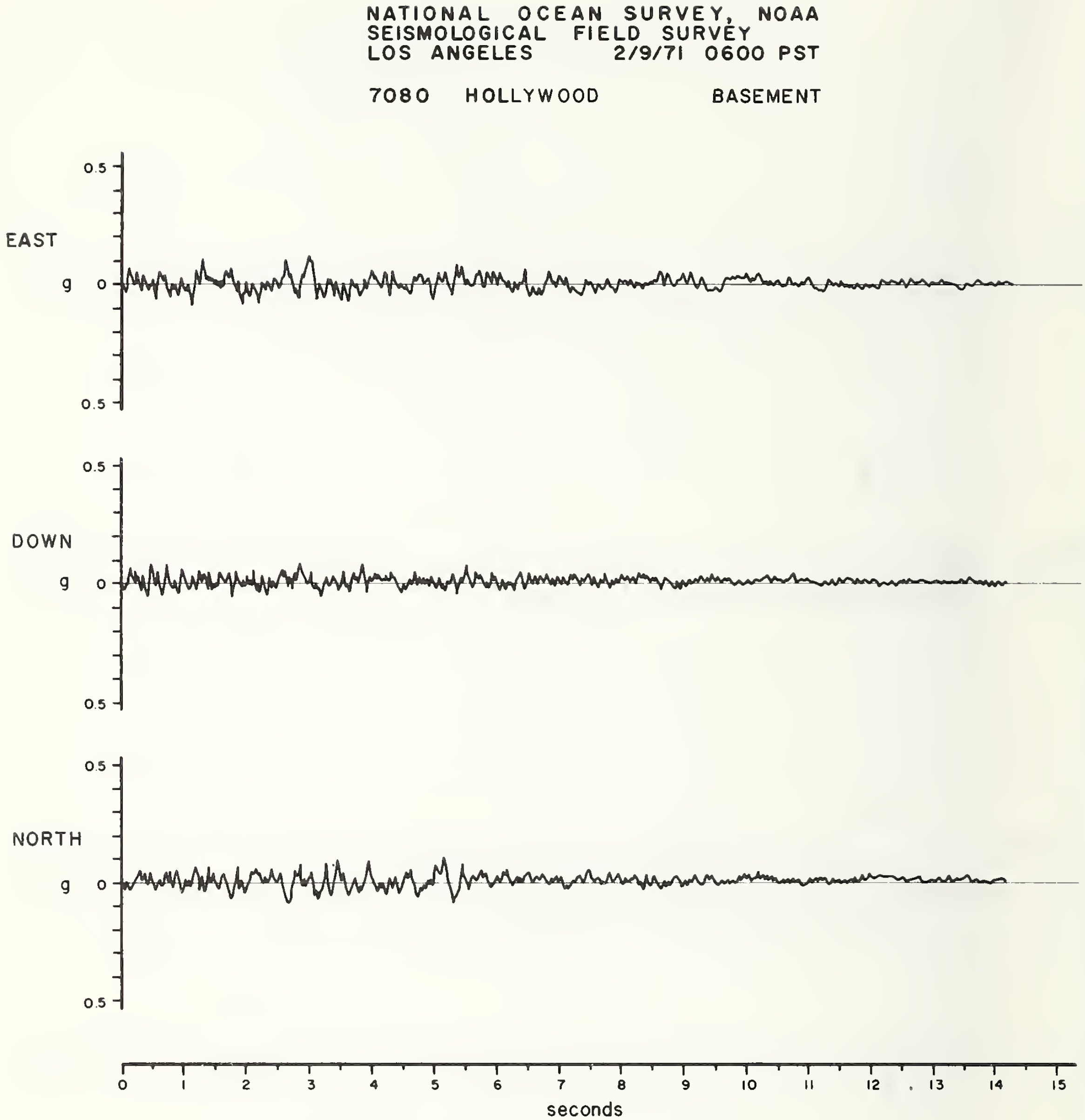



*Figure 3.16a Seismograph record, 7080 Hollywood, Los Angeles, basement.*

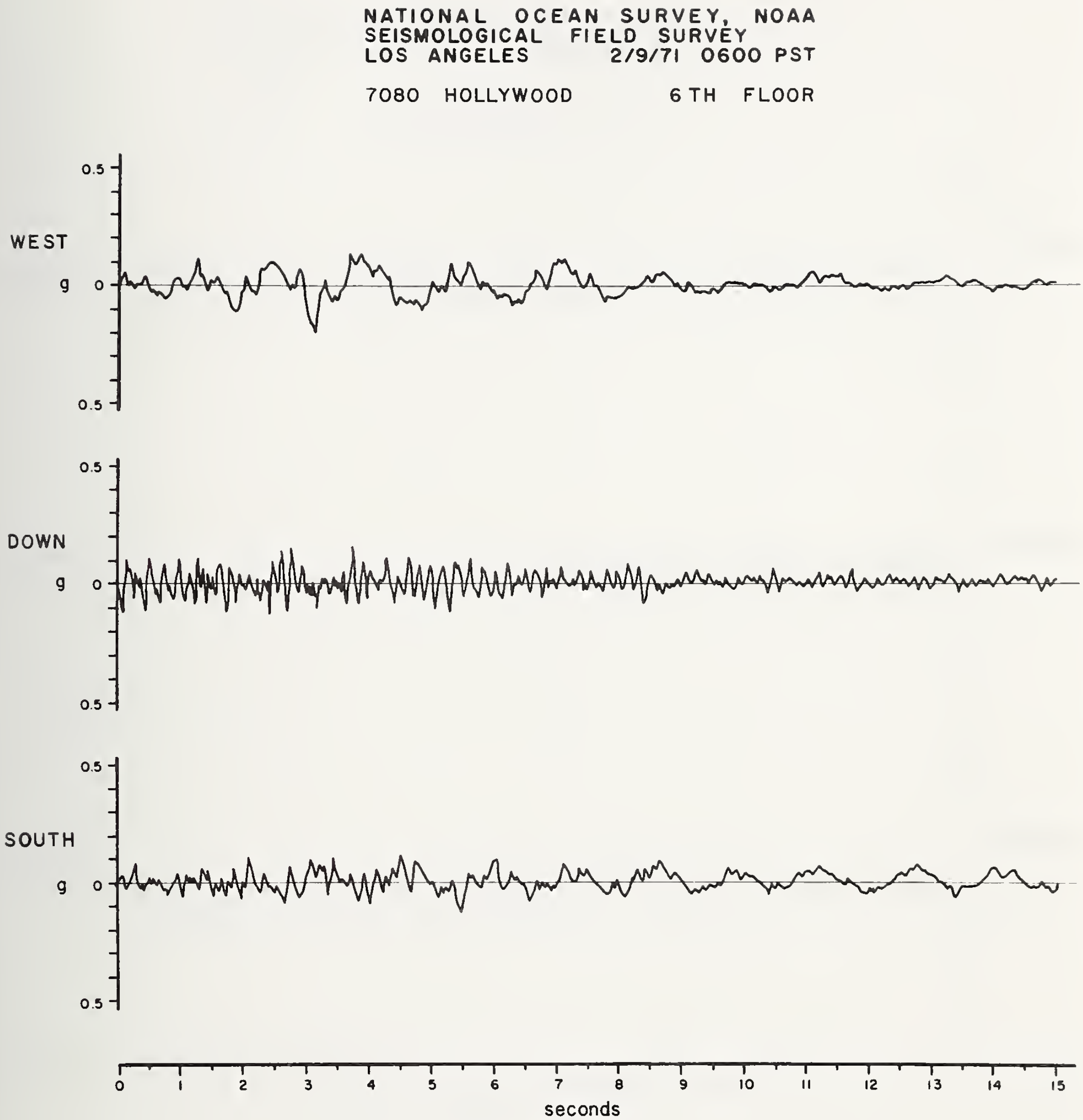



*Figure 3.16b Seismograph record, 7080 Hollywood, Los Angeles, 6th floor.*

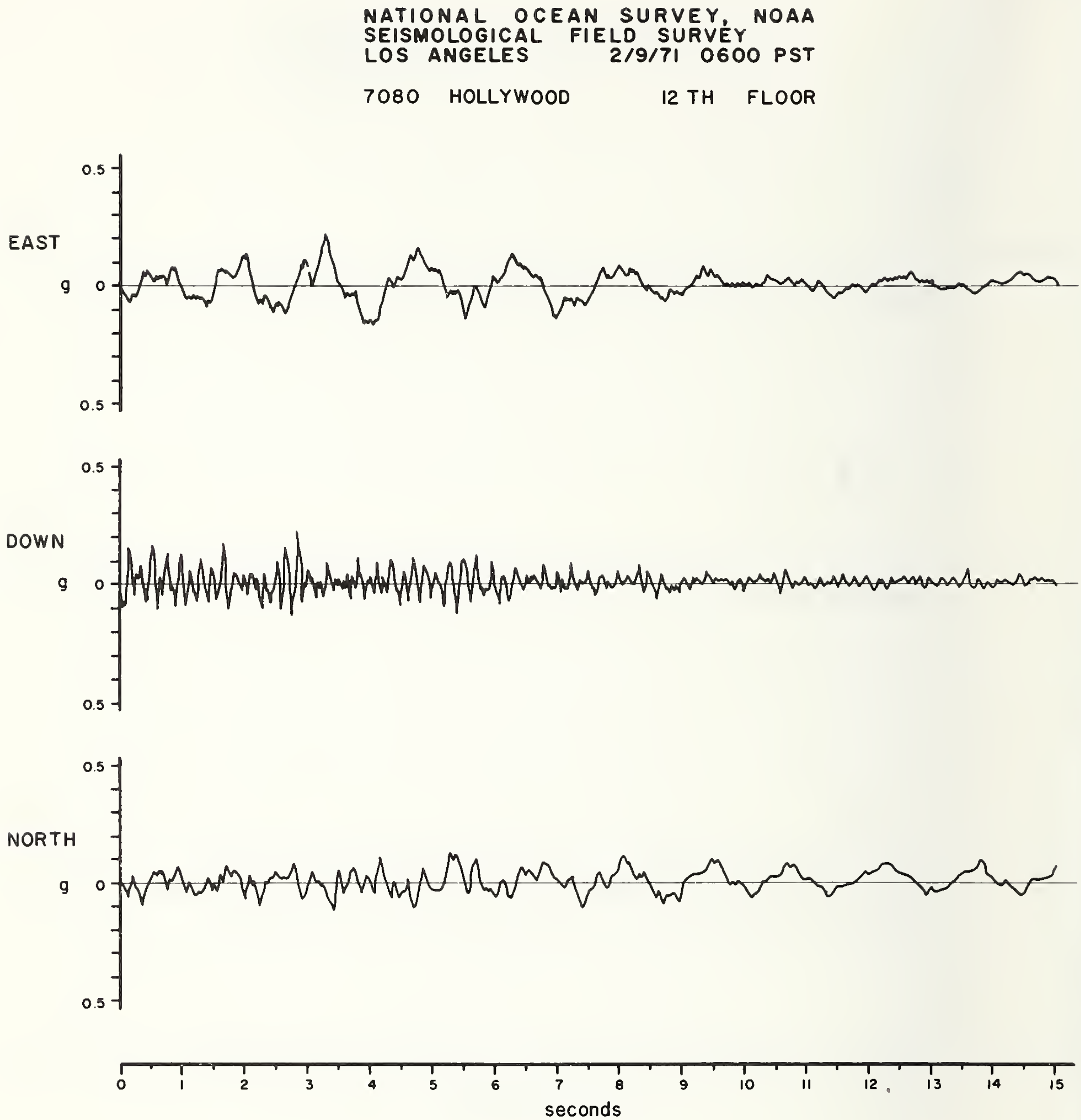



*Figure 3.16c Seismograph record, 7080 Hollywood, Los Angeles, 12th floor.*

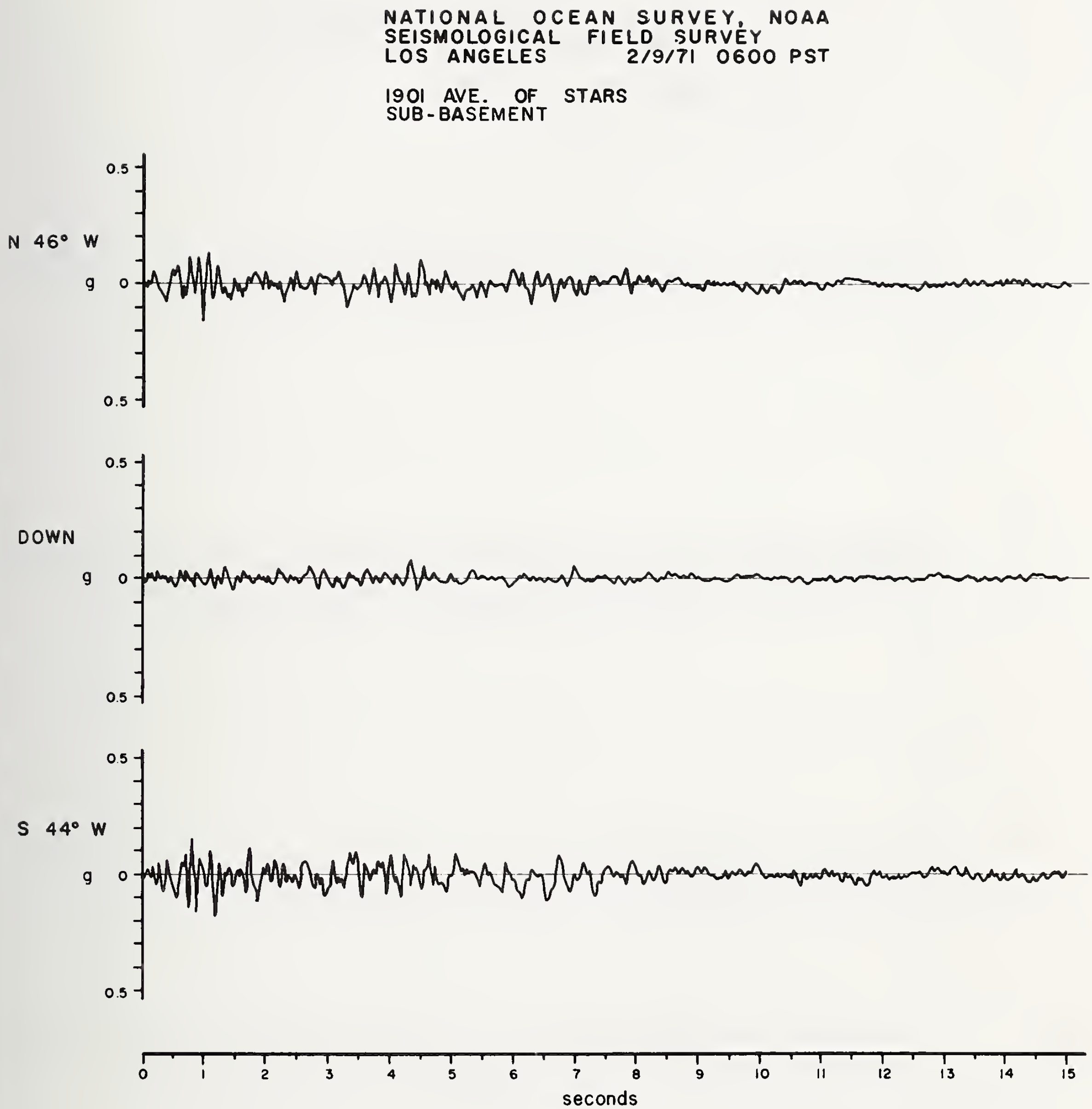



*Figure 3.17a Seismograph record, 1901 Avenue of Stars, Los Angeles, sub-basement.*

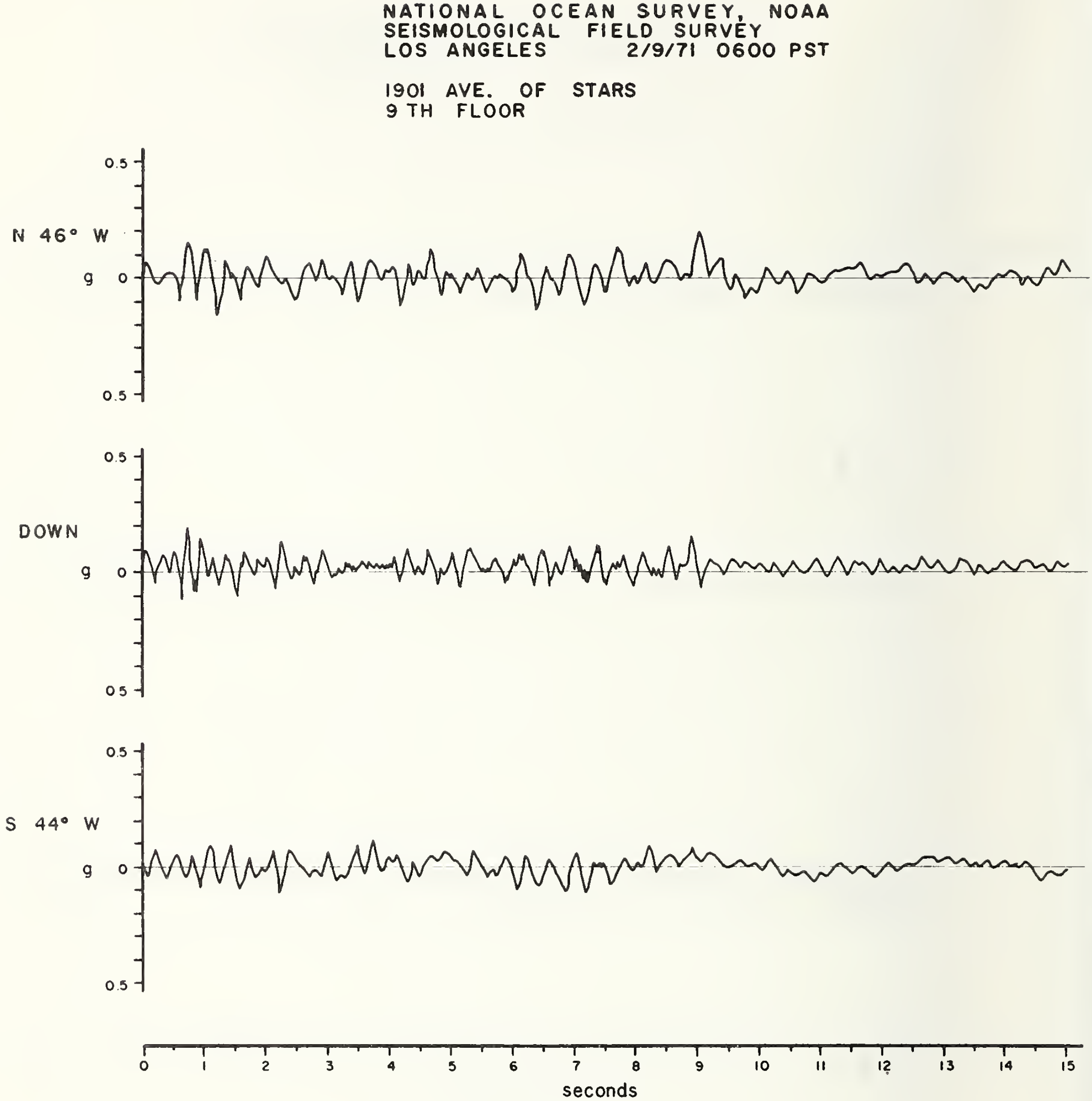



*Figure 3.17b Seismograph record, 1901 Avenue of Stars, Los Angeles, 9th floor.*

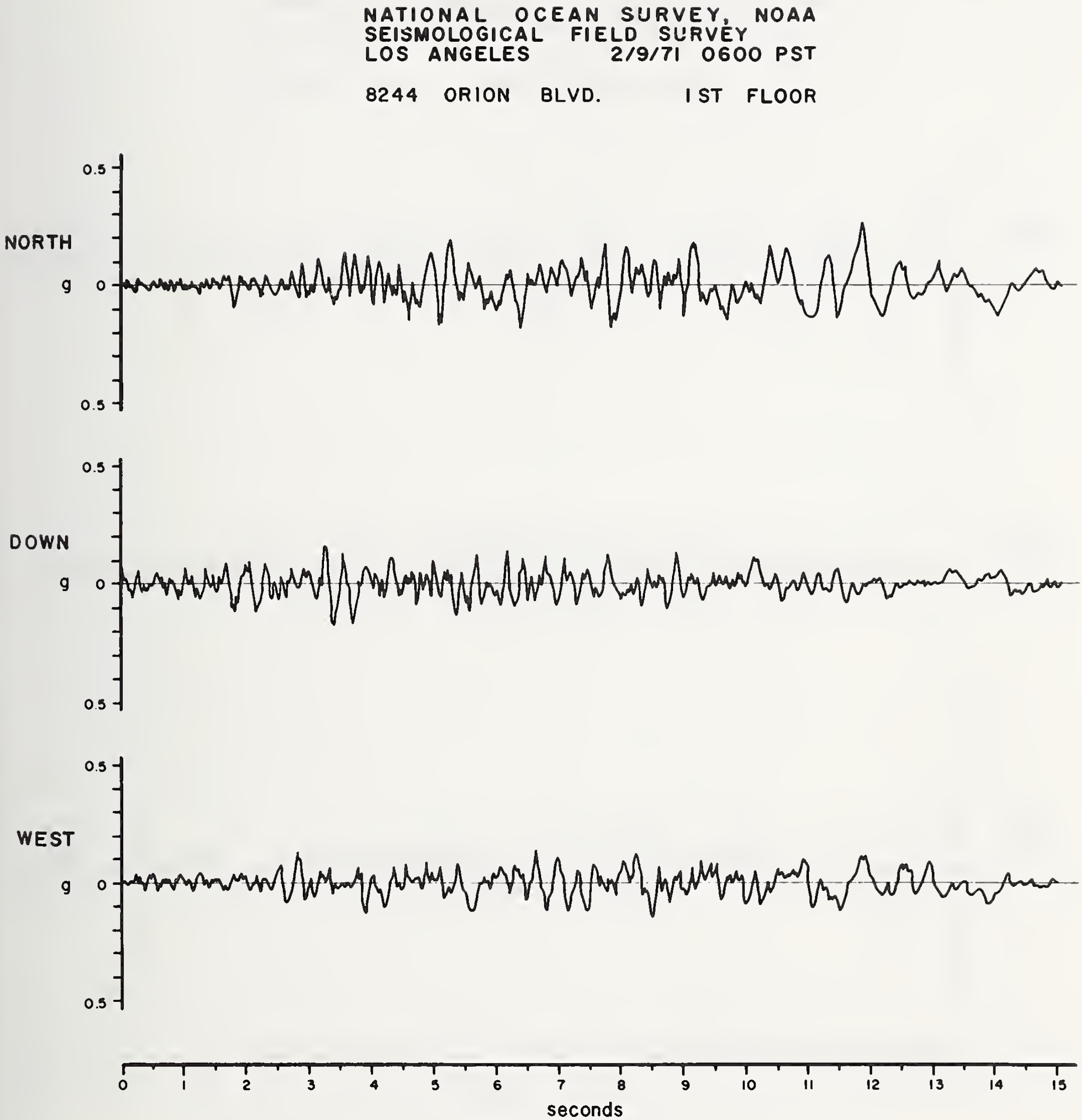



*Figure 3.18a Seismograph record, 8244 Orion Boulevard, Los Angeles, 1st floor.*

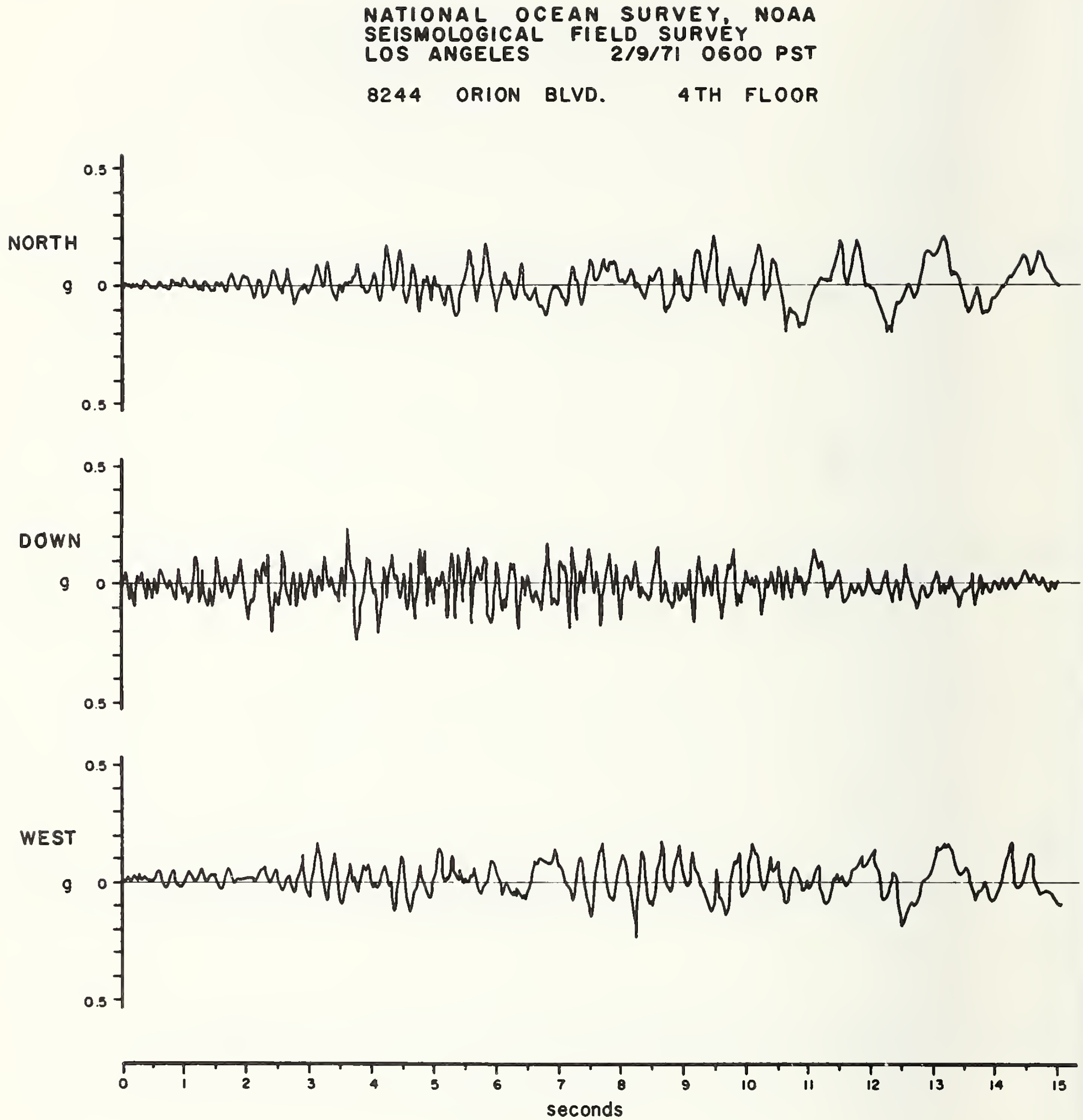



*Figure 3.18b Seismograph record, 8244 Orion Boulevard, Los Angeles, 4th floor.*

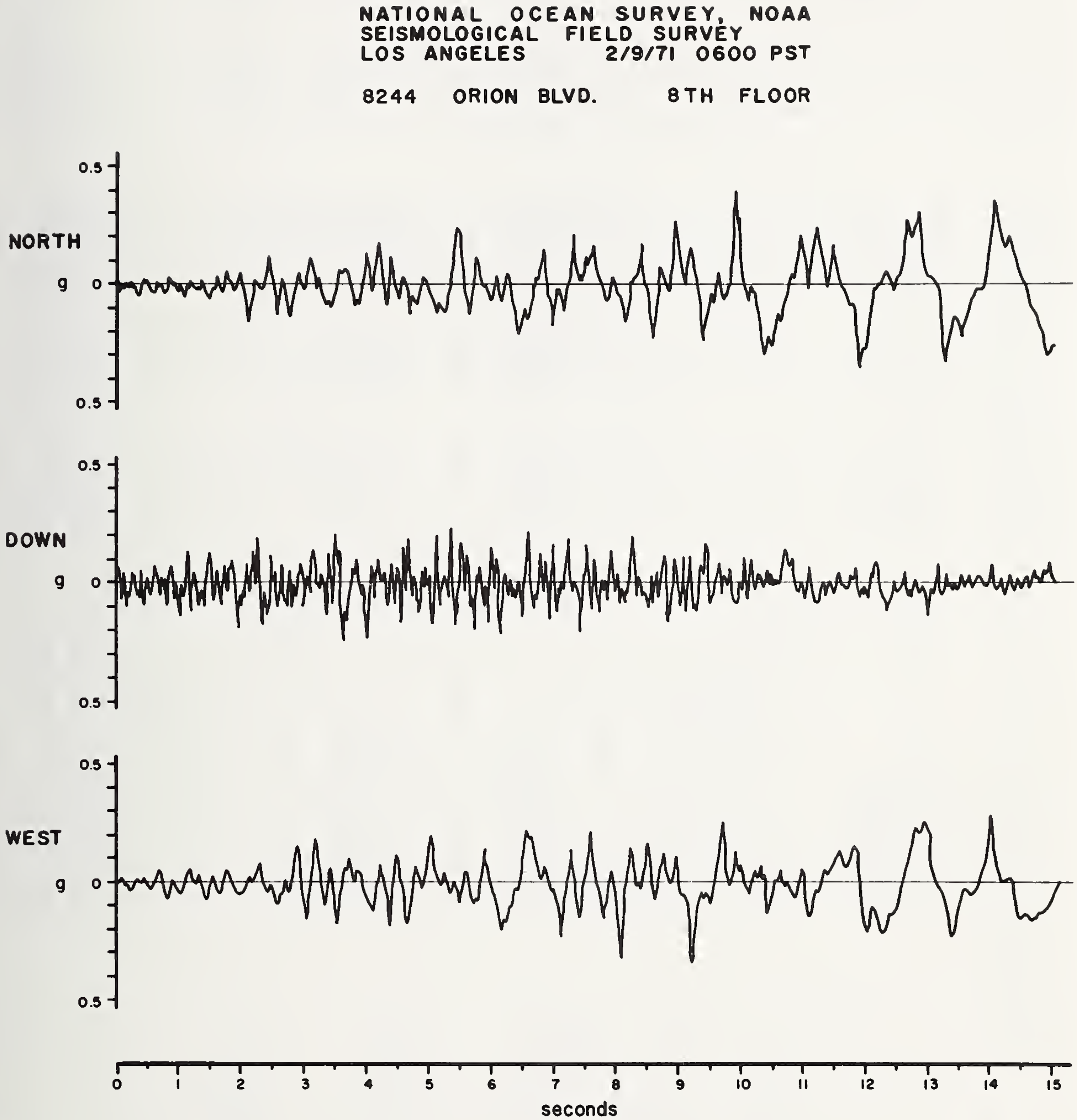



*Figure 3.18c Seismograph record, 8244 Orion Boulevard, Los Angeles, 8th floor.*

Table 3.1 Location of Strong-Motion Seismographs in Southern California with Peak Accelerations and Approximate Periods (Readings obtained from National Oceanic and Atmospheric Administration)

| Location | Station | Max. Acceleration Fraction of Gravity V | H1 | H2 | Approx. Period Second V | H1 | H2 |
|---|---|---|---|---|---|---|---|
| 1. Point Concepcion | -- | -- | -- | -- | -- | -- | -- |
| 2. Cachuma Dam | -- | -- | -- | -- | -- | -- | -- |
| 3. Santa Barbara | Univ. of Calif. | 0.011 | 0.016 | 0.011 | 0.30 | 0.26 | 0.20 |
| 4. Port Hueneme | Navy Laboratory | 0.010 | 0.030 | 0.025 | 0.30 | 0.30 | 0.72 |
| 5. Santa Felicia Dam | Outlet Works | 0.092 | 0.237 | 0.230 | 0.11 | 0.11 | 0.10 |
| | Dam Crest | 0.072 | 0.180 | 0.217 | 0.44 | 0.64 | 0.58 |
| 6. Castaic | Old Ridge Route | 0.178 | 0.388 | 0.316 | 0.20 | 0.30 | 0.20 |
| 7. Lake Hughes Array | Fire Station #78 | 0.115 | 0.167 | 0.121 | 0.08 | 0.67 | 0.54 |
| | Station #4 | 0.163 | 0.158 | 0.194 | 0.18 | 0.19 | 0.18 |
| | Springs #9 | 0.118 | 0.152 | 0.158 | 0.05 | 0.15 | 0.10 |
| | No. 12 | 0.178 | 0.371 | 0.276 | 0.08 | 0.20 | 0.20 |
| 8. Fairmont Reservoir | -- | 0.081 | 0.167 | 0.153 | 0.36 | 0.23 | 0.20 |
| 9. Palmdale | Fire Station | 0.084 | 0.110 | 0.131 | 0.10 | 0.18 | 0.18 |
| 10. Pearblossom | Pumping Plant | 0.063 | 0.104 | 0.148 | 0.13 | 0.13 | 0.14 |
| 11. Wrightwood | 6074 Park Drive | 0.016 | 0.042 | 0.036 | 0.14 | 0.27 | 0.33 |
| | 6074 Park Drive | 0.022 | 0.054 | 0.044 | 0.10 | 0.35 | 0.23 |
| 12. Cedar Springs | Pump Plant | 0.008 | 0.015 | 0.018 | 0.18 | 0.80 | 0.20 |

Table 3.1 (Cont'd)

| Location | Station | Max. Acceleration Fraction of Gravity V | H1 | H2 | Approx. Period Second V | H1 | H2 |
|---|---|---|---|---|---|---|---|
| 13. Devils Canyon | -- | -- | -- | -- | -- | -- | -- |
| 14. San Bernardino | Hall of Records | 0.016 | 0.047 | 0.042 | 0.16 | 0.21 | 0.36 |
| 15. Loma Linda | Univ. Med. Center | Too Faint to Read | | | -- | -- | -- |
| 16. Colton | Edison Company | 0.026 | 0.035 | 0.044 | 0.24 | 0.17 | 0.30 |
| 17. Desert Hot Springs | -- | -- | -- | -- | -- | -- | -- |
| 18. Puddingstone Dam | -- | 0.048 | 0.088 | 0.052 | 0.09 | 0.34 | 0.13 |
| 19. San Antonio Dam | -- | 0.031 | 0.079 | 0.058 | 0.18 | 0.26 | 0.32 |
| 20. Carbon Canyon Dam | -- | 0.042 | 0.073 | 0.068 | 0.16 | 0.29 | 0.18 |
| 21. Whittier Narrows | -- | 0.052 | 0.105 | 0.100 | 0.08 | 0.20 | 0.22 |
| 22. Santa Ana | Orange County Eng. Building | 0.019 | 0.030 | 0.030 | 0.18 | 0.16 | 0.26 |
| 23. Costa Mesa | 666 W. 19th Street | 0.008 | 0.023 | 0.035 | 0.23 | 0.28 | 0.46 |
| 24. Perris | -- | -- | -- | -- | -- | -- | -- |
| 25. San Onofre | SCE Nuclear Power Plant | 0.010 | 0.010 | 0.018 | 0.18 | 0.17 | 0.36 |
| 26. Hemet | -- | -- | -- | -- | -- | -- | -- |
| 27. Anza | -- | -- | -- | -- | -- | -- | -- |

Table 3.1 (Cont'd)

| Location | Station | Max. Acceleration Fraction of Gravity V | H1 | H2 | Approx. Period Second V | H1 | H2 |
|---|---|---|---|---|---|---|---|
| 28. Borrego Springs | -- | -- | -- | -- | -- | -- | -- |
| 29. Superstition Mountain | -- | -- | -- | -- | -- | -- | -- |
| 30. El Centro-Imperial | -- | -- | -- | -- | -- | -- | -- |
| 31. San Diego | -- | -- | -- | -- | -- | -- | -- |
| 32. San Juan Capistrano | -- | 0.021 | 0.035 | 0.026 | 0.60 | 0.24 | 0.33 |
| 33. Fullerton | 2600 Nutwood Ave. | | | | | | |
| | Basement | 0.017 | 0.035 | 0.035 | 0.19 | 0.26 | 0.38 |
| | Penthouse (W. Wing) | 0.041 | 0.108 | 0.131 | 0.15 | 0.70 | 0.62 |
| | Penthouse (Center) | 0.022 | 0.092 | 0.147 | 0.17 | 0.69 | 0.72 |
| 34. Orange | 4000 W. Chapman Ave. | -- | -- | -- | -- | -- | -- |
| | Basement | 0.011 | 0.021 | 0.021 | 0.26 | 0.36 | 0.29 |
| | 10th Floor | 0.021 | 0.051 | 0.042 | 0.22 | 0.26 | 0.29 |
| | 19th Floor | 0.042 | 0.081 | 0.061 | 0.22 | 1.10 | 0.41 |
| 35. Gorman | OSO Pumping Plant | 0.035 | 0.092 | 0.094 | 0.13 | 0.20 | 0.26 |
| 36. Fort Tejon | -- | 0.015 | 0.023 | 0.026 | 0.12 | 0.17 | 0.12 |
| 37. Grapevine | Tehachapi Pumping Plant | 0.020 | 0.046 | 0.066 | No Time Marks | | |
| 38. Wheeler Ridge | -- | 0.013 | 0.021 | 0.026 | 0.10 | 0.10 | 0.20 |

Table 3.2 Location of Strong-Motion Seismographs in the Los Angeles Area with Peak Accelerations and Approximate Periods
(Readings obtained from National Oceanic and Atmospheric Administration)

| Location | | Station | Max. Acceleration Fraction of Gravity | | | Approx. Period Second | | |
|---|---|---|---|---|---|---|---|---|
| | | | V | H1 | H2 | V | H1 | H2 |
| 1. | 111 North Hope | Basement | 0.080 | 0.138 | 0.202 | 0.24 | 0.76 | 0.15 |
| | | 7th Floor | 0.097 | 0.171 | 0.131 | 0.22 | 0.50 | 0.55 |
| | | 15th Floor | 0.171 | 0.164 | 0.118 | 0.23 | 0.50 | 3.30 |
| 2. | 445 S. Figueroa | Sub-basement | 0.060 | 0.144 | 0.134 | 0.17 | 0.40 | 0.32 |
| | | 19th Floor | 0.115 | 0.210 | 0.130 | 0.31 | 0.44 | 0.29 |
| | | 39th Floor | Out of Paper | | | -- | -- | -- |
| 3. | 533 S. Fremont | Basement | 0.083 | 0.246 | 0.224 | 0.30 | 0.38 | 0.26 |
| | | 6th Floor | 0.164 | 0.343 | 0.306 | 0.16 | 0.18 | 0.65 |
| | | Roof | Too Faint to Read | | | -- | -- | -- |
| 4. | 611 West Sixth | Basement | 0.065 | 0.097 | 0.110 | 0.18 | 0.22 | 0.30 |
| | | 24th Floor | No Data | | | -- | -- | -- |
| | | 42nd Floor | 0.110 | 0.110 | 0.180 | 0.21 | 0.42 | 0.87 |
| 5. | 250 East First | Basement | 0.040 | 0.092 | 0.132 | 0.10 | 0.10 | -- |
| | | 8th Floor | 0.070 | 0.210 | 0.170 | 0.06 | 0.40 | 0.90 |
| | | Roof | 0.210 | 0.158 | 0.184 | 0.20 | 0.60 | 0.60 |
| 6. | 646 South Olive | Basement | 0.081 | 0.221 | 0.254 | 0.13 | 0.22 | 0.23 |
| | | 4th Floor | 0.121 | 0.253 | 0.256 | 0.10 | 0.43 | 0.46 |
| | | Roof | 0.263 | 0.380 | 0.483 | 0.10 | 0.50 | 0.16 |
| 7. | 808 South Olive | Street Level | Cannot Recover Record | | | -- | -- | -- |
| | | 4th Floor | 0.191 | 0.263 | 0.162 | 0.24 | 0.43 | 0.16 |
| | | 8th Floor | 0.237 | 0.250 | 0.435 | 0.20 | 0.38 | 0.29 |

Table 3.2 (Cont'd)

| Location | Station | Max. Acceleration Fraction of Gravity V | H1 | H2 | Approx. Period Second V | H1 | H2 |
|---|---|---|---|---|---|---|---|
| 8. 420 South Grand | 2nd Floor | 0.068 | 0.121 | 0.168 | 0.13 | 0.16 | 0.38 |
| | 10th Floor | | Blank | | -- | -- | -- |
| | 17th Floor | 0.226 | 0.226 | 0.321 | 0.17 | 0.73 | 0.92 |
| 9. Santa Anita Dam | -- | 0.068 | 0.179 | 0.236 | 0.10 | 0.12 | 0.18 |
| 10. Cal. Tech. Millikan Library | Basement | 0.116 | 0.184 | 0.216 | 0.38 | 0.42 | 0.30 |
| | 10th Floor | 0.137 | 0.337 | 0.332 | 0.32 | 1.00 | 0.60 |
| Athenaeum | -- | 0.103 | 0.113 | 0.100 | 0.24 | 0.42 | 0.38 |
| 11. Jet Propulsion Lab. | Basement | 0.132 | 0.165 | 0.212 | 0.30 | 0.35 | 0.33 |
| Pasadena | 9th Floor | 0.265 | 0.213 | 0.379 | 0.17 | 0.31 | 0.35 |
| 12. 633 E. Broadway | -- | 0.142 | 0.275 | 0.233 | 0.24 | 0.73 | 0.46 |
| Glendale | -- | -- | -- | -- | -- | -- | -- |
| 13. 1640 Marengo | Ground | 0.079 | 0.142 | 0.138 | 0.20 | 0.20 | 0.20 |
| | 4th Floor | 0.118 | 0.197 | 0.264 | 0.08 | 0.20 | 0.20 |
| | 8th Floor | 0.132 | 0.237 | 0.435 | 0.10 | 0.20 | 0.20 |
| 14. 4814 Loma Vista | -- | -- | -- | -- | -- | -- | -- |
| Vernon | -- | -- | -- | -- | -- | -- | -- |
| 15. 3663 S. Hoover | -- | -- | -- | -- | -- | -- | -- |

Table 3.2 (Cont'd)

| Location | Station | Max. Acceleration Fraction of Gravity V | H1 | H2 | Approx. Period Second V | H1 | H2 |
|---|---|---|---|---|---|---|---|
| 16. 3407 W. Sixth | Basement | 0.064 | 0.169 | 0.188 | 0.14 | 0.23 | 0.13 |
| | 4th Floor | 0.101 | 0.208 | 0.210 | 0.19 | 0.47 | 0.21 |
| | Penthouse | 0.260 | 0.288 | 0.206 | 0.19 | 0.67 | 0.63 |
| 17. 2470 Wilshire | Sub-Basement | 0.052 | 0.147 | 0.118 | 0.17 | 0.29 | 0.32 |
| | 5th Floor | 0.104 | 0.243 | 0.210 | 0.14 | 0.26 | 0.29 |
| | 11th Floor | 0.156 | 0.222 | 0.226 | 0.14 | 0.86 | 1.14 |
| 18. 3710 Wilshire | Basement | 0.081 | 0.167 | 0.159 | 0.23 | 0.41 | 0.48 |
| | 5th Floor | 0.097 | 0.272 | 0.161 | 0.21 | 0.28 | 0.28 |
| 19. 4867 Sunset | Basement | 0.134 | 0.167 | 0.170 | 0.13 | 0.60 | 0.23 |
| | 2nd Floor | 0.131 | 0.296 | 0.222 | 0.09 | 0.18 | 0.52 |
| | 7th Floor | 0.196 | 0.450 | 0.459 | 0.09 | 0.42 | 0.37 |
| 20. 4680 Wilshire | Basement | 0.081 | 0.121 | 0.089 | 0.38 | 0.44 | 0.60 |
| | 3rd Floor | 0.130 | 0.221 | 0.181 | 0.13 | 0.16 | 0.12 |
| | 6th Floor | 0.160 | 0.243 | 0.302 | 0.12 | 0.40 | 0.62 |
| 21. 1025 N. Highland | No Record | -- | -- | -- | -- | -- | -- |
| 22. 7080 Hollywood | Basement | 0.063 | 0.109 | 0.098 | 0.18 | 0.26 | 0.26 |
| | 6th Floor | 0.156 | 0.193 | 0.123 | 0.18 | 0.60 | 0.46 |
| | 12th Floor | 0.224 | 0.213 | 0.121 | 0.17 | 0.42 | 0.42 |
| 23. 120 N. Robertson | Basement | 0.031 | 0.090 | 0.093 | 0.16 | 0.40 | 0.58 |
| | 4th Floor | -- | 0.177 | 0.179 | -- | 0.26 | 0.22 |
| | 9th Floor | 0.118 | 0.327 | 0.276 | 0.22 | 0.40 | 0.60 |
| 24. 1901 Ave. of Stars | Basement | 0.066 | 0.118 | 0.171 | 0.19 | 0.15 | 0.14 |
| | 9th Floor | 0.145 | 0.184 | 0.105 | 0.30 | 0.30 | 0.30 |
| | 21st Floor | 0.092 | 0.142 | 0.066 | -- | -- | -- |

Table 3.2 (Cont'd)

| Location | Station | Max. Acceleration Fraction of Gravity V | H1 | H2 | Approx. Period Second V | H1 | H2 |
|---|---|---|---|---|---|---|---|
| 25. 945 Tiverton | Sub-Basement | No Record | | | -- | -- | -- |
| | 8th Floor | 0.105 | 0.123 | 0.226 | 0.12 | 0.48 | 0.19 |
| | 14th Floor | 0.146 | 0.143 | 0.181 | 0.10 | 0.68 | 0.39 |
| 26. UCLA Eng. Building | Reactor Lab. | 0.074 | 0.104 | 0.091 | 0.18 | 0.17 | 0.14 |
| 27. 16055 Ventura | Basement | No Record | | | -- | -- | -- |
| | 1st Parking Level | No Record | | | -- | -- | -- |
| | Roof | No Record | | | -- | -- | -- |
| 28. 8244 Orion | Ground | 0.171 | 0.276 | 0.145 | 0.60 | 0.30 | 0.30 |
| | 4th Floor | 0.230 | 0.178 | 0.244 | 0.30 | -- | 0.30 |
| | 8th Floor (Roof) | 0.224 | 0.388 | 0.310 | 0.07 | 0.30 | 0.30 |
| 29. Pacoima Dam | -- | 0.71 | 1.25+ | 1.25+ | -- | -- | -- |
| 30. Terminal Island | -- | 0.019 | 0.031 | 0.027 | 0.46 | 0.23 | 0.44 |
| Long Beach | -- | -- | -- | -- | -- | -- | -- |
| 31. 215 W. Broadway | -- | -- | -- | -- | -- | -- | -- |
| Long Beach | -- | -- | -- | -- | -- | -- | -- |
| 32. 2011 Zonal | Basement | 0.060 | 0.079 | 0.068 | 0.060 | 0.079 | 0.068 |
| | 5th Floor | 0.081 | 0.158 | 0.178 | 0.081 | 0.158 | 0.178 |
| | 9th Floor | 0.115 | 0.200 | 0.209 | 0.115 | 0.200 | 0.209 |
| 33. Pasadena Seismol. Lab. | -- | -- | -- | -- | -- | -- | -- |

Table 3.2 (Cont'd)

| Location | Station | Max. Acceleration Fraction of Gravity V | H1 | H2 | Approx. Period Second V | H1 | H2 |
|---|---|---|---|---|---|---|---|
| 34. 3345 Wilshire | Basement | 0.069 | 0.121 | 0.094 | 0.10 | 0.50 | 0.40 |
| | 2nd Floor | -- | 0.167 | 0.113 | -- | 0.20 | -- |
| | 12th Floor | 0.124 | 0.206 | 0.250 | 0.14 | 1.04 | 0.95 |
| 35. 750 Garland | Ground Floor | -- | -- | -- | -- | -- | -- |
| | 2nd Floor | 0.100 | 0.221 | 0.158 | 0.13 | 0.32 | 0.34 |
| | 6th Floor | 0.147 | 0.305 | 0.232 | 0.24 | 0.81 | 1.02 |
| 36. 3440 University | Basement | 0.052 | 0.080 | 0.064 | 0.31 | 0.62 | 0.32 |
| | 5th Floor | 0.083 | 0.129 | 0.140 | 0.21 | 0.80 | 0.23 |
| | Roof | 0.088 | 0.235 | 0.260 | 0.19 | 0.19 | 0.97 |
| 37. 5260 Century | 1st Floor | 0.017 | 0.055 | 0.056 | 0.23 | 0.17 | 0.48 |
| | 4th Floor | 0.036 | 0.042 | 0.070 | 0.12 | 1.32 | 0.44 |
| | Roof | 0.081 | 0.089 | 0.056 | 0.18 | 1.55 | 1.82 |
| 38. 9841 Airport Blvd. | Basement | 0.012 | 0.030 | 0.029 | 0.24 | 0.40 | 0.27 |
| | 7th Floor | -- | -- | -- | -- | -- | -- |
| | 15th Floor | 0.050 | 0.100 | 0.093 | 0.22 | 1.57 | 1.40 |
| 39. 15433 Ventura | Basement | -- | -- | -- | -- | -- | -- |
| | 7th Floor | 0.153 | 0.242 | 0.170 | -- | -- | -- |
| | 13th Floor | 0.029 | 0.268 | 0.226 | -- | -- | -- |
| 40. 8639 Lincoln | Basement | 0.042 | 0.037 | 0.037 | 0.28 | 0.80 | 0.42 |
| | 6th Floor | 0.052 | 0.095 | 0.100 | 0.27 | 0.62 | 0.82 |
| | 12th Floor | 0.058 | 0.121 | 0.121 | 0.27 | 0.62 | 0.80 |
| 41. 6430 Sunset | 1st Floor | 0.087 | 0.191 | 0.143 | 0.10 | 0.22 | 0.21 |
| | 7th Floor | -- | -- | -- | -- | -- | -- |
| | 15th Floor | -- | -- | -- | -- | -- | -- |

Table 3.2 (Cont'd)

| Location | Station | Max. Acceleration Fraction of Gravity V | H1 | H2 | Approx. Period Second V | H1 | H2 |
|---|---|---|---|---|---|---|---|
| 42. 6464 Sunset | Basement | 0.075 | 0.125 | 0.107 | 0.17 | 0.26 | 0.45 |
| | 6th Floor | -- | -- | -- | -- | -- | -- |
| | 12th Floor | 0.287 | 0.271 | 0.235 | 0.23 | 2.10 | 0.73 |
| 43. 3838 Lankershim | Basement | 0.087 | 0.175 | 0.127 | 0.22 | 0.21 | 0.26 |
| | 11th Floor | -- | -- | -- | -- | -- | -- |
| | 21st Floor | 0.226 | 0.105 | 0.226 | 0.23 | 0.40 | 0.28 |
| 44. 3411 Wilshire | 5th Floor | 0.065 | 0.140 | 0.113 | 0.14 | 0.15 | 0.19 |
| | 13th Floor (Penthouse) | -- | -- | -- | -- | -- | -- |
| 45. 800 W. First | 1st Floor | 0.059 | 0.094 | 0.143 | -- | -- | -- |
| | 16th Floor | 0.158 | 0.121 | 0.182 | 0.18 | 0.48 | 1.16 |
| | 33rd Floor | 0.224 | 0.186 | 0.294 | 0.16 | 2.60 | 0.60 |
| 46. 222 Figueroa | 1st Floor | 0.043 | 0.119 | 0.150 | 0.18 | 0.39 | 0.48 |
| | 12th Floor | -- | -- | -- | -- | -- | -- |
| | 20th Floor | 0.086 | 0.309 | 0.395 | 0.27 | 0.42 | 1.12 |
| 47. 11661 San Vicente | G-2 Level | -- | -- | -- | -- | -- | -- |
| | 5th Floor | 0.113 | 0.091 | 0.082 | -- | -- | -- |
| | Roof | 0.157 | 0.103 | 0.093 | -- | -- | -- |
| 48. 1900 Ave. of Stars | Basement | 0.057 | 0.083 | 0.095 | 0.19 | 0.22 | 0.27 |
| | 16th Floor | -- | -- | -- | -- | -- | -- |
| | 29th Floor | 0.354 | 0.149 | 0.117 | 0.30 | 4.60 | 3.90 |
| 49. 234 Figueroa | Basement | 0.057 | 0.173 | 0.197 | 0.50 | 0.24 | 0.26 |
| | 12th Floor | -- | -- | -- | -- | -- | -- |
| | Roof | 0.168 | 0.435 | 0.500 | 0.16 | 0.70 | 0.30 |

Table 3.2 (Cont'd)

| Location | Station | Max. Acceleration Fraction of Gravity V | H1 | H2 | Approx. Period Second V | H1 | H2 |
|---|---|---|---|---|---|---|---|
| 50. 3550 Wilshire | Basement | 0.059 | 0.125 | 0.177 | 0.12 | 0.23 | 0.42 |
| | 11th Floor | | Blank | | -- | -- | -- |
| | 21st Floor | | Blank | | -- | -- | -- |
| 51. 2080 Century Park E. | Basement | | Blank | | -- | -- | -- |
| | 10th Floor | | Blank | | -- | -- | -- |
| | Roof | 0.234 | 0.180 | 0.357 | 0.19 | 0.32 | 0.22 |
| 52. Griffith Obsv. | Moon Room | 0.121 | 0.184 | 0.163 | 0.19 | 0.38 | 0.30 |
| 53. Long Beach State College | Ground Level | 0.015 | 0.042 | 0.021 | 0.26 | 0.50 | 0.22 |
| 54. 1150 S. Hill | Sub-Basement | 0.047 | 0.121 | 0.090 | 0.18 | 0.37 | 0.43 |
| | 5th Floor | 0.089 | 0.108 | 0.102 | 0.21 | 0.81 | 0.40 |
| | 10th Floor | 0.152 | 0.143 | 0.110 | 0.21 | 2.10 | 2.50 |
| 55. 1625 N. Olympic | Ground | 0.158 | 0.140 | 0.266 | 0.13 | 0.29 | 0.40 |
| | 6th Floor | 0.139 | 0.181 | 0.225 | 0.17 | 0.40 | 0.60 |
| | 10th Floor | 0.226 | 0.230 | 0.277 | 0.13 | 1.28 | 1.20 |
| 56. Palcs Verdes Estate | 2516 Via Tejon | 0.010 | 0.039 | 0.021 | 0.20 | 0.35 | 0.16 |
| 57. 900 S. Fremont Alhambra | Basement | 0.093 | 0.127 | 0.108 | 0.19 | 0.34 | 0.18 |
| | 6th Floor | 0.114 | 0.145 | 0.142 | 0.15 | 0.48 | 0.42 |
| | 12th Floor | 0.177 | 0.172 | 0.152 | 0.30 | 2.04 | 2.18 |
| 58. 15107 Vanower | Basement | 0.116 | 0.107 | 0.120 | 0.36 | 0.30 | 0.29 |
| | 4th Floor | 0.194 | 0.227 | 0.260 | 0.16 | 0.34 | 0.89 |
| | Roof | 0.173 | 0.341 | 0.385 | 0.34 | 0.42 | 0.97 |
| 59. 16661 Ventura | -- | -- | -- | -- | -- | -- | -- |

Table 3.2 (Cont'd)

| Location | Station | Max. Acceleration Fraction of Gravity V | H1 | H2 | Approx. Period Second V | H1 | H2 |
|---|---|---|---|---|---|---|---|
| 60. 15910 Ventura | Basement | 0.012 | 0.130 | 0.153 | 0.30 | 0.36 | 0.36 |
| | 9th Floor | 0.221 | 0.178 | 0.129 | 0.21 | 1.35 | 0.75 |
| | 19th Floor | 0.210 | 0.224 | 0.227 | 0.22 | 0.92 | 1.60 |
| 61. 15250 Ventura | 1st Floor | 0.100 | 0.227 | 0.140 | 0.07 | 0.25 | 0.30 |
| | 7th Floor | 0.175 | 0.250 | 0.212 | 0.10 | 0.20 | 0.25 |
| | 13th Floor (Roof) | 0.131 | 0.261 | 0.176 | 0.10 | 0.25 | 0.35 |
| 62. 14724 Ventura | 1st Floor | 0.093 | 0.258 | 0.189 | 0.15 | 0.32 | 0.27 |
| | 6th Floor | 0.107 | 0.356 | 0.274 | 0.11 | 0.22 | 0.22 |
| | Penthouse | -- | 0.315 | 0.208 | -- | 0.27 | 0.19 |
| 63. 1760 N. Orchid | Ground Floor | 0.072 | 0.163 | 0.131 | 0.22 | 0.20 | 0.16 |
| | 12th Floor | 0.141 | 0.141 | 0.078 | 0.27 | 0.56 | 0.24 |
| | 23rd Floor | 0.194 | 0.197 | 0.110 | 0.23 | 0.36 | 0.18 |
| 64. 930 Hilgard | Basement | -- | -- | -- | -- | -- | -- |
| | 8th Floor | -- | -- | -- | -- | -- | -- |
| | 15th Floor | 0.162 | 0.149 | 0.197 | 0.13 | 0.20 | 0.91 |
| 65. 435 N. Oakhurst Beverly Hills | Basement | 0.038 | 0.060 | 0.088 | 0.48 | 0.36 | 0.23 |
| | 5th Floor | 0.047 | 0.129 | 0.137 | 0.12 | 0.54 | 0.54 |
| | Roof | 0.102 | 0.240 | 0.250 | 0.20 | 0.58 | 0.43 |
| 66. 420 N. Roxbury Beverly Hills | 1st Floor | 0.037 | 0.196 | 0.172 | 0.22 | 0.13 | 0.18 |
| | 5th Floor | 0.102 | 0.215 | 0.212 | 0.18 | 0.17 | 0.23 |
| | 10th Floor | 0.119 | 0.297 | 0.215 | 0.10 | 0.50 | 0.40 |
| 67. 9100 Wilshire | Basement | 0.037 | 0.161 | 0.123 | 0.22 | 0.28 | 0.19 |
| | 5th Floor | 0.078 | 0.132 | 0.147 | -- | -- | -- |
| | Roof | -- | -- | -- | -- | -- | -- |

Table 3.2 (Cont'd)

| Location | Station | Max. Acceleration Fraction of Gravity | | | Approx. Period Second | | |
|---|---|---|---|---|---|---|---|
| | | V | H1 | H2 | V | H1 | H2 |
| 68. 9450 Wilshire | Basement | -- | -- | -- | -- | -- | -- |
| | 4th Floor | -- | -- | -- | -- | -- | -- |
| | Penthouse | -- | -- | -- | -- | -- | -- |
| 69. 5900 Wilshire | B Parking Lot | 0.029 | 0.066 | 0.073 | 0.66 | 0.16 | 0.37 |
| | 16th Floor | 0.084 | 0.097 | 0.121 | 0.32 | 0.27 | 0.21 |
| | Penthouse | 0.152 | 0.140 | 0.170 | 0.40 | 4.60 | 2.20 |
| 70. 6200 Wilshire | Ground Floor | 0.038 | 0.134 | 0.133 | 0.20 | 0.30 | 0.61 |
| | 10th Floor | 0.068 | 0.280 | 0.147 | 0.33 | 0.39 | 0.32 |
| | 17th Floor | 0.074 | 0.300 | 0.261 | 0.39 | 0.54 | 1.63 |
| 71. 1177 Beverly Drive | Basement | 0.067 | 0.123 | 0.114 | 0.12 | 0.45 | 0.37 |
| | 3rd Floor | -- | -- | -- | -- | -- | -- |
| | Penthouse | -- | -- | -- | -- | -- | -- |
| 72. 1880 Century Park E. | 1st Parking Level | 0.067 | 0.109 | 0.126 | 0.13 | 0.31 | 0.31 |
| | 7th Floor | 0.120 | 0.097 | 0.444 | 0.20 | 0.34 | -- |
| | Penthouse | 0.274 | 0.102 | 0.124 | 0.28 | 0.42 | 0.40 |
| 73. 1800 Century Park E. | Basement | 0.066 | 0.080 | 0.100 | 0.29 | 0.26 | 0.41 |
| | 5th Floor | 0.157 | 0.210 | 0.220 | 0.21 | 0.32 | 0.42 |
| | Penthouse | 0.307 | 0.283 | 0.284 | 0.21 | 0.32 | 0.38 |
| 74. Century City Ground | -- | -- | -- | -- | -- | -- | -- |
| 75. 1888 Century Park E. | Basement | -- | -- | -- | -- | -- | -- |
| | 14th Floor | 0.189 | 0.144 | 0.159 | 0.22 | 0.50 | 0.30 |
| | 21st Floor | 0.352 | 0.149 | 0.084 | 0.24 | 1.50 | 0.54 |
| | 5th Fl. Park. Ramp | 0.094 | 0.176 | 0.123 | 0.18 | 0.34 | 0.26 |
| | 9th Fl. Park. Ramp | 0.110 | 0.375 | 0.312 | 0.17 | 0.28 | 0.28 |

Table 3.2 (Cont'd)

| Location | Station | Max. Acceleration Fraction of Gravity | | | Approx. Period Second | | |
|---|---|---|---|---|---|---|---|
| | | V | H1 | H2 | V | H1 | H2 |
| 76. 2500 Wilshire | Basement | 0.043 | 0.097 | 0.100 | 0.13 | 0.18 | 0.13 |
| | 8th Floor | 0.070 | 0.127 | 0.157 | 0.22 | 0.24 | 0.52 |
| | Roof | 0.139 | 0.203 | 0.191 | 0.26 | 0.82 | 1.20 |
| 77. 616 S. Normandie | Basement | 0.046 | 0.105 | 0.114 | 0.21 | 0.60 | 0.20 |
| | 8th Floor | 0.105 | 0.216 | 0.142 | 0.20 | 0.34 | 0.39 |
| | Roof | 0.198 | 0.306 | 0.227 | 0.12 | 1.29 | 0.38 |

# CHAPTER 4 GEOLOGY AND SOIL CONDITIONS

## 4.1 Geologic Setting

The Los Angeles area is both surrounded by and interlaced with numerous geological faults. Some of the larger well-defined faults are shown on figure 4.1 in relation to Los Angeles and neighboring cities and geographical landmarks. The San Andreas fault which has been a dominent influence in California is seen to be across the mountain range from the San Fernando Valley and displayed no activity throughout this earthquake. Major damage was fairly well confined to the northern portion of the San Fernando Valley, in the Sylmar-Pacoima-San Fernando area. The predominantly east-west oriented mountain ranges are referred to by geologists as the Transverse Ranges.

The following description of geologic setting is abstracted from material presented by Wentworth and Yerkes [4.1].

> "The faulting associated with the San Fernando earthquake of February 9, 1971, occurred in the Transverse Ranges structural province, a region noted for its strong and relatively young tectonic deformation.
>
> The Transverse Ranges structural province trends eastward across southern California and constitutes a region of late Cenozoic north-south shortening that lies athwart the north-westerly regional trend of the dominant San Andreas fault system.
>
> The Transverse Ranges province consists of numerous east-trending mountain ranges and valleys typified by late Cenozoic structural deformation and strike slip, reverse, and thrust faults of similar trend [4.2 and 4.3]. The most spectacular development of the structure of this province is in the Ventura Basin, where an immense Cenozoic syncline that includes a thickness of 1,500 m of marine Pleistocene sediments has both limbs overturned and overridden by thrust [4.2]. Very young deformation in the Transverse Ranges is not restricted to the Ventura Basin, however, and late Quaternary faulting is known to occur from near Point Conception eastward to the San Andreas fault.
>
> The February 9, 1971, faulting broke the ground

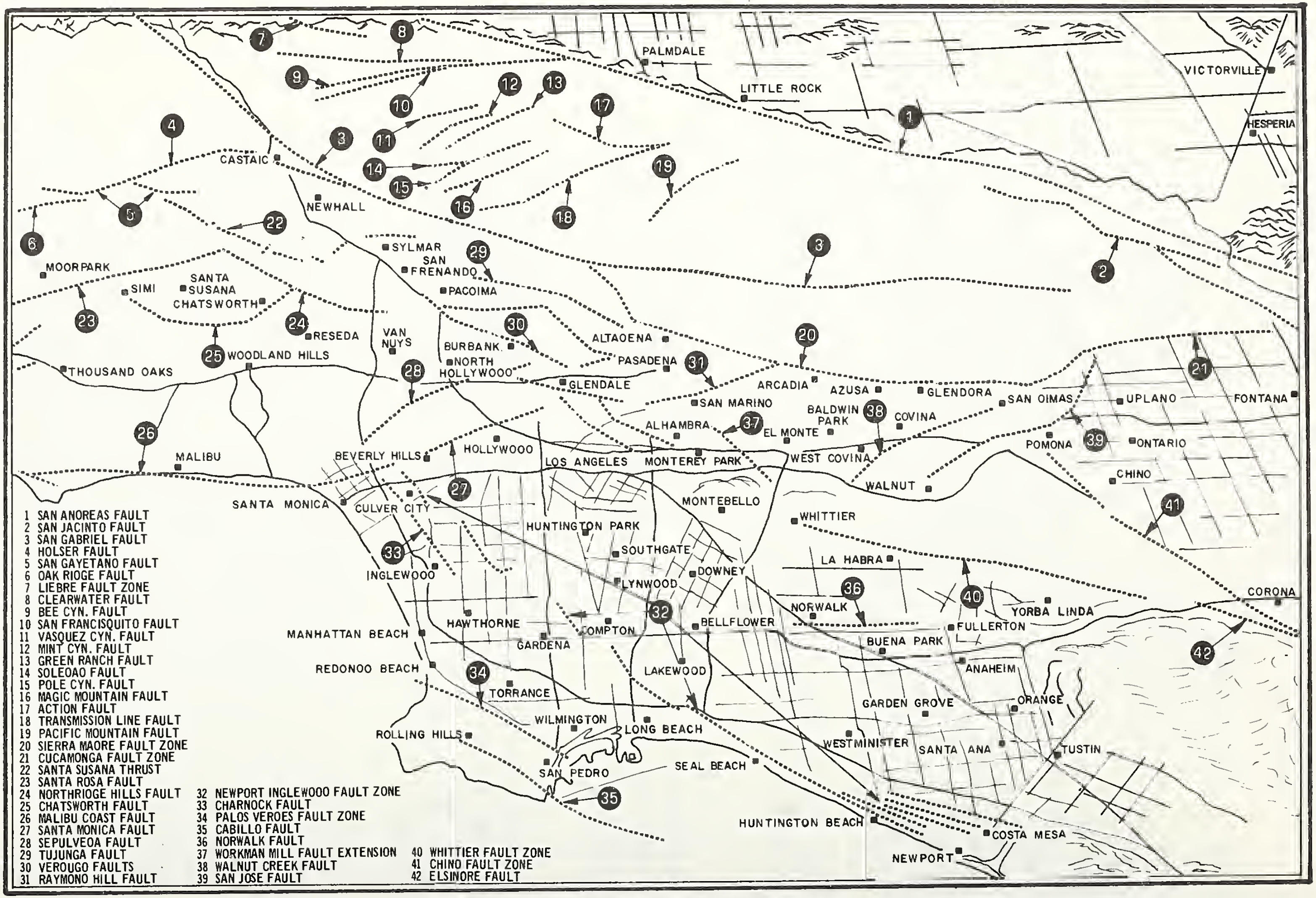


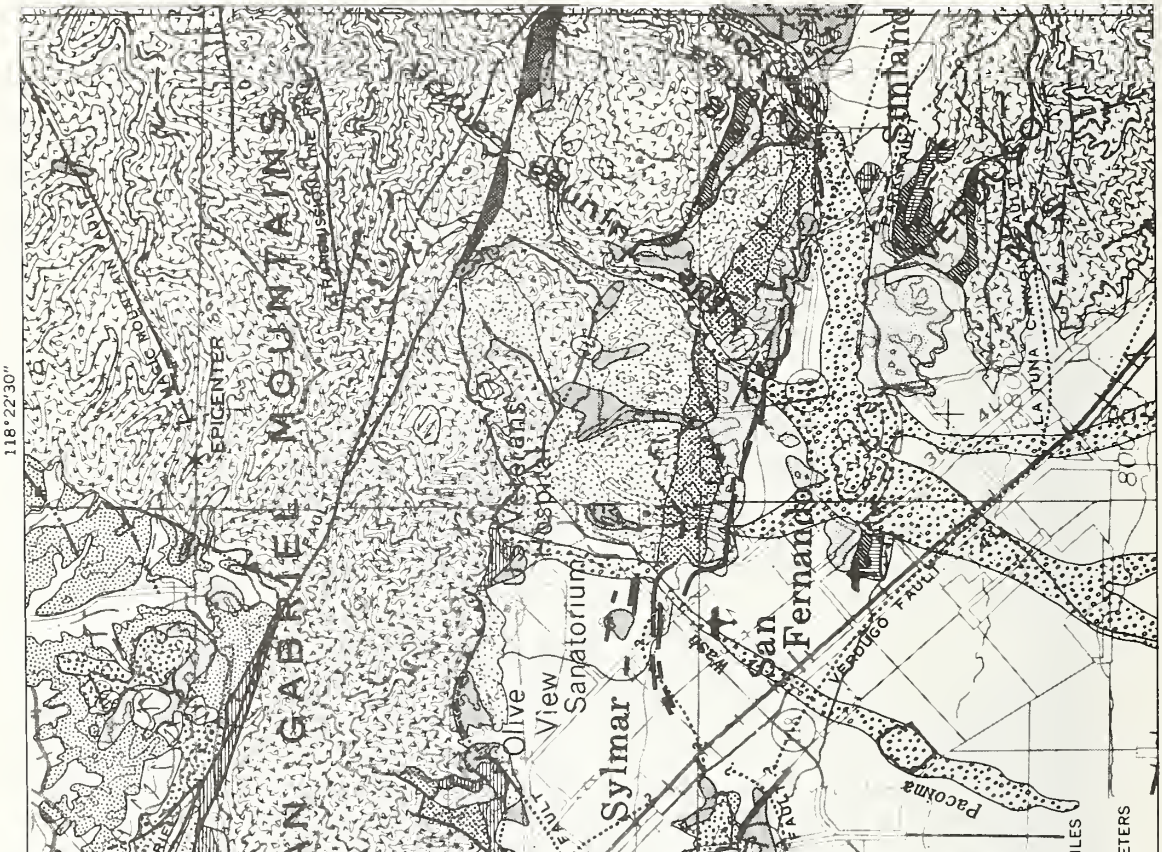

*Figure 4.1 Location of known faults in the Los Angeles area.*

surface near the north margin of the San Fernando Valley opposite the "great bend" in the San Andreas fault south of the San Joaquin Valley. The geometry of the crustal blocks in this region is such that the Transverse Ranges structural block southwest of the San Andreas fault is constrained and compressed against the bend as the block moves northwestward along the fault. During the past 5 to 10 million years this compressive deformation has thrust the mountain blocks up over adjacent valleys to the south. The movement has occurred along north-dipping reverse or thrust-fault systems such as the Sierra Madre and Santa Susana, which trend west and northwest along the southern margins of the mountain blocks. The most impressive product of this uplift is the bold southern front of the San Gabriel Mountains, which stands some 1,500 m above the San Fernando and Los Angeles areas to the south. Additional horizontal shortening and uplift of one of the blocks occurred along the ruptures that accompanied the San Fernando earthquake.

In the central Transverse Ranges the crystalline basement rocks are overlain with great unconformity by Cretaceous and younger sedimentary rocks many thousands of meters thick. The San Fernando Valley itself (fig. 4.2) is an asymmetric synclinorium developed chiefly in Miocene and younger rocks that have been deformed by late Cenozoic folding and faulting, especially at the northern margin by thrusting along the Santa Susana fault and its eastward equivalents. Based on a gravity-density model, Corbato [4.4] estimates that approximately 4,500 m of sedimentary rocks are present in the central part of the valley.

None of the late Cenozoic sedimentary rocks are particularly hard, and the Saugus Formation especially is only partially indurated and difficult to distinguish from alluvium in drill holes.

Precambrian to Cretaceous crystalline rocks of the basement complex are exposed in the San Gabriel Mountains in the northeast part of the map area and in the Verdugo Mountains in the southeast corner. These dense metamorphic and igneous rocks constitute the terrane on which the sedimentary sequence was deposited. Cretaceous and lower Tertiary sedimentary strata are only exposed locally along the Santa Susana and San Gabriel faults, although to the west and south of figure 4.2 they crop out extensively.



118°30'
118°22'30"
34° 22' 30"
34° 15'
SAN GABRIEL MOUNTAINS
EPICENTER
SAN GABRIEL FAULT
Olive View Sanatorium
Sylmar
San Fernando
Pacoima Wash
Granada Hills
San Fernando Valley
SANTA SUSANA MTS
SANTA SUSANA FAULT
GRANADA HILLS FAULT
NORTHRIDGE HILLS FAULT
MISSION HILLS FAULT
OLIVE VIEW FAULT
VERDUGO FAULT
LA TUNA CANYON FAULT
Sunland
99
2 1 0 2 4 MILES
2 0 2 4 6 KILOMETERS

# EXPLANATION

UNCONFORMITY

Qc | Qa

**Alluvial deposits**

Qc, deposits in active stream channels: unconsolidated gravel, sand, silt, and clay
Qa, unconsolidated gravel, sand, silt, and clay; commonly overlain by soil

Qt

**Stream terrace deposits**

Slightly consolidated gravel, sand, silt and clay

*Upper Pleistocene and Holocene* — QUATERNARY

UNCONFORMITY

QTs | Qs | Tsr

**Saugus Formation**

QTs, north and east part of map: slightly to moderately consolidated coarse-grained sandstone, conglomerate, and mudstone
Qs, southwest part of map: slightly consolidated conglomerate and coarse-grained sandstone
Tsr, Sunshine Ranch Member, of Pliocene age, in west part of map; siltstone with interbedded sandstone and conglomerate; locally interfingers with upper part of Pico Formation

*Pliocene and lower Pleistocene*

Tp

**Pico Formation**

Marine; soft siltstone, mudstone, and fine-grained sandstone with interbedded coarse-grained sandstone and resistant pebble conglomerate; upper part interfingers with Saugus Formation and lower part with Towsley Formation

*Pliocene*

Ttp

**Towsley Formation**

Soft siltstone and mudstone with interbedded moderately indurated, sandstone and conglomerate; basal conglomerate in west center of map is well indurated and contains boulders and blocks of locally-derived basement rocks; upper part interfingers with Pico Formation in northwest part of map

*Upper Miocene and lower Pliocene*

Tm

**Modelo Formation**

Siltstone, soft mudstone, and siliceous shale, minor interbedded sandstone

*Upper Miocene* — TERTIARY

UNCONFORMITY

Tmc

**Mint Canyon Formation**

Present only north of San Gabriel fault: continental, soft, poorly to well-consolidated sediments; upper part conglomerate and coarse-grained sandstone, mudstone, and tuff; lower part poorly sorted conglomerate with interbedded coarse-grained sandstone, siltstone, and mudstone; coarse basal conglomerate consists of angular fragments of underlying basement rocks

*Upper Miocene*

UNCONFORMITY

Tt

**Topanga Formation**

Sandstone and cobble-boulder conglomerate, poorly sorted, massive, commonly well indurated

UNCONFORMITY

Tv

**Volcanic rocks**

Basaltic flows, tuff breccias, and flow breccias, hard and dense to soft, vesicular, and amygdaloidal

*Middle Miocene* — TERTIARY

Tk

**Undifferentiated lower Tertiary and Upper Cretaceous rocks**

Chiefly sandstone, siltstone, and pebble conglomerate commonly referred to Martinez and Chico Formations: commonly well indurated and resistant. Crops out in Simi Hills in west part of map and slivers along San Gabriel fault

*Upper Cretaceous and Paleocene* — CRETACEOUS AND TERTIARY

UNCONFORMITY

bc

**Basement complex**

Igneous and meta-igneous rocks, commonly dense and crystalline. North of San Gabriel fault, chiefly gabbro, anorthosite, and granodiorite; south of San Gabriel fault, chiefly granodiorite and diorite gneiss

PRECAMBRIAN TO MESOZOIC

Contact

High-angle fault ...?... Thrust fault ...?...

**Faults**

Dashed where approximately located or inferred; dotted where concealed; queried where doubtful

Tectonic ruptures associated with earthquake of February 9, 1971

Landslide

Only selected large landslides shown; arrows indicate general direction of movement

Geology compiled by R.F. Yerkes and C.M. Wentworth, 1971

*Figure 4.2 Generalized geologic map of the San Fernando area, Los Angeles County, California (Van Norman Lakes are called San Fernando Lakes on this map base).*

Within the map area south of the San Gabriel fault, the oldest sedimentary rocks overlying the basement are the middle Miocene Topanga Formation, which includes about 300 m of conglomerate, marine sandstone, and volcanic rocks.

As much as 1,500 m of marine upper Miocene Modelo Formation is exposed along the foothills of the San Gabriel and Verdugo Mountains, in the Mission Hills, and in the Santa Susana Mountains. The Modelo is overlain by, and locally interfingers with, the Towsley Formation of late Miocene-early Pliocene age. The Towsley ranges in thickness from zero where it is overlapped by the Pico Formation just south of the San Gabriel fault zone to about 750 m in the east and west central parts of the map area.

The Pico grades upward and laterally into the late Pliocene-early Pleistocene Saugus Formation, a shallow marine to brackish-water unit that rims the north side of the San Fernando Valley. The Saugus Formation thickens westward from essentially zero at Big Tujunga Canyon to about 1,950 m at Lopez Canyon east of Pacoima Wash. An exploratory well drilled north of the Olive View fault and about 1 1/2 km west of Olive View Sanitorium penetrated a minimum stratigraphic thickness of 1,200 m of Saugus Formation, and the California State Water Rights Board report [4.5] states that the formation exceeds 1,800 m in thickness in the syncline near Sylmar.

The "middle Pleistoce" Pacoima Formation of Oakeshott [4.6] has been combined with the Saugus Formation on the map (fig. 4.2). This deformed fanglomerate-breccia unit is exposed north of Sylmar, where it unconformably overlies the Saugus and is unconformably truncated by stream-terrace and alluvial deposits.

Uplifted erosional remnants of stream-terrace gravel and sand are widely distributed in the San Fernando area, but only a few are shown on the generalized map. No direct evidence for the age of these deposits is available, although a late Pleistocene age is inferred on the basis of their stratigraphic position, lack of deformation, and unconformable relations with the underlying Saugus Formation. Oakeshott [4.6] was able to distinguish five stages of terrace formation along Pacoima Wash and two in the Lopez-Kagel

Canyon area 6 1/2 km northeast of San Fernando. Some of the late Quaternary history of the mountain blocks can be obtained from such geomorphic features.

The relatively flat surface of San Fernando Valley is underlain by unconsolidated sand, gravel, and finer sediment ranging in thickness from zero at the valley edges to greater than 200 m in the east-central part of the valley near the south margin of figure 4.2. In the Sylmar area, alluvium is perhaps 6 m thick, except along Pacoima Wash [4.5].

Sediment west of Van Norman Lakes and along the southern part of the valley has been supplied by streams draining areas of sedimentary rocks, with the result that alluvium in these areas consists predominantly of clay. In contrast, larger streams at the north margin of the valley east of the Lakes drain crystalline rocks of high relief and have produced deposits of coarser materials with abundant gravel.

## 4.2 Type of Soil Movement and Introductory Remarks

Soil movements which occur during and subsequent to earthquakes may be placed in two categories, primary movements associated with the faulting itself and secondary movements which are triggered by the stress waves generated by the earthquake. This section presents some brief comments about the second category of soil movements.

The most common soil movement of the second category is the landslide. Soil which is on a slope is stressed internally by gravity forces. There is a component of the gravity force, which may be called a driving force, that is directed downslope. The driving force is put into equilibrium by an equal and opposite force, the resisting force, which is derived from the mobilization of the shear strength of the soil. The accelerations of the surface soils due to an earthquake can cause increases in the driving force and under some conditions can cause decreases in the resisting force. When the driving force becomes greater than the maximum resisting force, instability will result and a landslide will occur. Some massive landslides, both of natural and man-made slopes, have been associated with earthquakes in the past. A recent example is the Turnagain slide which occurred as a result of the earthquake which struck the city of Anchorage, Alaska, on March 27, 1964. The slide zone was about 8,000 feet in length along the coastline and extended inwardly approximately 600 feet at one end and 1,200 feet at the other end of the slide zone [4.7].

The analyses of slopes to predict earthquake effects is a complex problem. Present state-of-the-art is such that a simple model is normally selected to represent the stability problem. A horizontal seismic force is assumed to be acting through the center of gravity of the potential sliding mass and the analysis is performed using principles of static equilibrium. The seismic force contributes to the driving force, possibly leading to instability. Most of the current design procedures presently in use do not take vertical acceleration into account.

The shear strength of the soil can be affected by the stresses induced by earthquakes. Normally, this effect is small or insignificant. However, there are two instances in which the effect can be catastrophic, disturbance of quick clays and liquefaction.

Quick clays, or clays which undergo a great loss of shear strength due to disturbance, exist in some parts of the world. These clays are rare in the United States, but in other parts of the world massive slides have been triggered by such low-level stresses as those induced by a passing railway train [4.8].

Liquefaction accounts for some spectacular soil behavior during earthquakes. If a loose deposit of saturated sand is subjected to earthquake stresses, the vibration induces a tendency toward densification of the sand stratum, reducing the intergranular pressures to zero, and converting the sand mass into a liquid. Liquefaction was present at some of the slides reported at Anchorage and caused some spectacular failures at Niigata [4.9].

Another type of soil movement which can be associated with earthquakes is the densification and consequent fracturing of the soil due merely to the passage of the stress waves through the soil. There are many deposits of loose material which can undergo sizable changes in volume due to vibrations.

## 4.3 Character of Soil at a Few Sites Affected by the San Fernando Earthquake

The surface soils in the San Fernando Valley are alluvial as a result of erosion of the mountains surrounding the valley. The character of the surface soils will be described for a few sites.

## OLIVE VIEW HOSPITAL [4.10 and 4.11]

The hospital site is on the alluvial fan that has been formed at the mouth of Wilson Canyon, which flows southerly out of the San Gabriel mountains. In former times, water flowed freely across the fan when it rained. The soils at the site consist of sands and gravels, with a layer of very large boulders at a depth of about eight feet. The sand consists of subangular to subrounded grains of quartz, feldspar, and rock fragments, with abundant biotite. The granular materials were firm to dense. For the purpose of foundation design the angle of internal friction which was recommended was 38 degrees and the recommended density was 113 pounds per cubic foot. Because of the nature of the material, considerable difficulty was encountered in putting down borings. The deepest boring penetrated to about 40 feet below the ground surface. No ground water was encountered when the test holes were drilled.

## SAN FERNANDO JUVENILE HALL [4.12]

The soil conditions at the site are extremely variable; the natural soils vary from hard, dense conglomerate beneath the westerly portion of the site to moderately soft silt and silty sand beneath the southeasterly portion of the site. The borings were carried to a depth of 30 feet at some locations. Ground water was encountered in several of the borings ranging in depths from 12 to 30 feet beneath the ground surface.

## HOLY CROSS HOSPITAL [4.13]

The soils at the site consist of non-uniform alluvial deposits of sand, silt, and clay in various portions. The upper soils are primarily fine-grained silts while the deeper strata are generally more sandy with up to 20 per cent gravel. The upper soils were found to be low in density, with low shearing resistance, and they became compressible upon saturation. The deeper soils are more dense, have higher shearing resistance, and are more uniform and less compressible. Borings at the site were carried down to a depth of 50 feet below the ground surface. Ground water was encountered at the site at approximately 25 feet below the present ground surface.

## SOUTH CONNECTOR OVER-CROSSING (Palmdale Interchange-see fig. 7.2)

Of primary interest from the standpoint of the foundation is Pier 4 (fig. 7.3). The foundation consisted of a 10 foot by 6 foot rectangular section which was placed to a penetration of approximately 45 feet below the ground surface. The pier was placed on the side of a steep valley (a slope of approximately 40 per cent).

The soils at the site are an alluvial deposit and are essentially granular in nature. At the ground surface the soils are a compact, brown, fine to very fine sand with some gravel. At a few feet below the ground surface, the soil becomes more dense and contains more gravel. Occasional thin layers of sandstone and gravel are encountered. Soil borings at the site show that very dense, dark gray, silty, fine to very fine sandstone was encountered at depth in some of the holes. It is presumed that the bottom several feet of the foundation penetrated into this sandstone. No information is available on the depth of the water table at the site.

If good construction techniques were employed, the pier should have an extremely high capacity to sustain vertical loads. Insufficient data are available to estimate the lateral deflection of the pier when subjected to horizontal loads at the ground surface.

## SEPARATION AND OVERHEAD, ROUTE 210/5 (fig. 7.11)

A description of the foundations at the site is given in table 4.1. It can be noted that a variety of foundation types were used.

The soils at the site are alluvial deposits and consist mostly of coarse-grained material. The soils at the ground surface are generally rather loose and become more dense with depth. Penetrometer values are given, presumably values which correlate with those from the standard penetration tests, in the order of 1 to 10 blows per foot in the top 15 to 20 feet and increasing to perhaps 100 blows per foot at a depth of 30 feet or less.

A typical boring log at the site is given in table 4.2.

No information is available on the location of the water table at the site.

## NORTHWEST CONNECTOR OVERCROSSING, ROUTE 210/5 (fig. 7.11)

The bents of this structure were erected on six foot outside diameter concrete piles which were cast in drilled holes which penetrated from 20 to 30 feet below ground level. The soil is an alluvial deposit consisting

*Table 4.1 Foundations for Separation and Overhead, Interstate 210/5*

| Foundation | Station | Elevation of Ground Surface (estimated), feet | Description of Foundation |
|---|---|---|---|
| Abutment 1 | 82+17 | 1271 | Class 1 concrete piles; tip elevation of piles 1232; piles have 45 ton bearing |
| Pier 2 | 83+17 | 1273 | Footing 17 ft by 19 ft; bottom elevation 1266; supported by concrete piles; tip elevation of piles 1235; piles have 45 ton bearing |
| Pier 3 | 84+21 | 1276 | Footing 17 ft by 19 ft; bottom elevation 1267; supported by concrete piles; tip elevation of piles 1242; piles have 45 ton bearing |
| Pier 4 | 85+50 | 1285 | Six-foot outside diameter pile cast in a drilled hole; tip elevation of pile 1262 |
| Pier 5 | 86+80 | 1309 | Six-foot outside diameter pile cast in a drilled hole; tip elevation of pile 1262 |
| Pier 6 | 87+88 | 1309 | Six-foot outside diameter pile cast in a drilled hole; tip elevation of pile 1274 |
| Pier 7 | 88+67 | 1343 | Six-foot outside diameter pile cast in a Drilled hole; tip elevation of pile 1273 |
| Abutment 8 | 89+88 | 1375 | Class 1 concrete piles; tip elevation of piles 1353; piles have 45 ton bearing |

principally of granular materials. The soil conditions appear to be very similar to those described at the previous two locations. A typical boring log is given in table 4.3.

## 4.4 Soil Movements Associated with San Fernando Earthquake

There is no evidence that there was any significant damage to major buildings associated with liquefaction as a result of the earthquake. The soil at the building sites described above would not be subject to liquefaction. However, there is considerable evidence of densification and subsidence of soils which resulted from the vibrations caused by the earthquakes. There are numerous evidences of ground cracking and ground distortions. Some of these ground movements are probably associated with faulting, but many others are secondary movements due to densification of the soil because of the vibrations.

There was much evidence of landslides in the hills surrounding the valley. Soil could be seen sliding from mountains several days after the quake. Morton reported that "approximately 1,000 landslides, ranging in length from about 15 to more than 300 meters were mapped from the aerial photographs" [4.14]. None of the landslides in the hills appears to have caused major inconvenience or financial losses. However, it is possible that the numerous areas where the soil has been weakened by the occurrence of the earthquake will be highly susceptible to further sliding in periods of heavy rainfall.

A major landslide of a slope which is substantially natural occurred in the vicinity of the Van Norman Reservoir [4.15]. In addition, there was extensive slumping in the alluvium around the margins of both the Lower and Upper Van Norman Reservoirs. Some detail of these movements will be given in the remainder of this section.

Also described in this section are serious movements of the man-made slopes at the Upper and Lower San Fernando Dams (which form the Van Norman Reservoirs).

### LANDSLIDES NEAR THE VAN NORMAN LAKES (RESERVOIRS)

Youd [4.15] reports that "extensive slumping occurred in the alluvium around the margins of both the Lower and Upper Van Norman Lakes. In some places a nearly continuous line of scarps formed. The scarp on the west shore of the lower lake is more than 1 kilometer long." In connection with these soil movements is the extremely interesting fact that sand boils commonly formed below the scarps. Youd goes

Table 4.2 Boring Log at Separation and Overhead, Interstate 210/5

| Depth (Feet) | Description |
|---|---|
| 0 - 6 | Loose to slightly compact, red-brown silty sand with scattered cobbles |
| 6 - 9 | Compact, red-brown clayey sand and sandy, silty clay |
| 9 - 17 | Dense to very dense, red-brown and brown, very fine sand, and silty, very fine sand with some sandy silt |
| 17 - 38 | Dense to very dense, yellow and tan, very fine sand |
| 38 - 50 | Dense to very dense, poorly sorted sand with scattered gravel layers |

Table 4.3 Boring Log at Northwest Connector Overcrossing, Interstate 210/5

| Depth (Feet) | Description |
|---|---|
| 0 - 10 | Slightly compact to compact, brown and yellow-brown sand and clayey sand with scattered gravel |
| 10 - 23 | Dense, dark brown and grey, gravelly, clayey sand and clayey silt |
| 23 - 34 | Slightly compact to compact, brown, very fine to find sand with silt and clay binder |
| 34 - 50 | Very dense, yellow-brown sand and clayey sand with scattered gravel. |

on to say:

> "A recently filled and graded area west of the upper lake was broken up by numerous fissures, which generally tended to parallel the shoreline and the contour of the previous surface. This area is considered here because of the likelihood that failure in the alluvium below the fill is responsible for the sliding. Extensional and vertical displacements in the fill indicate movement toward the reservoir either in the form of block glides or in slightly rotated slumps. Sand boils erupted in the southern part of this fracture zone some 180 meters from the lake and several meters above the water level at the time of the shock.
>
> Many fissures formed in the area west of that described in the previous paragraph. Of those observed, the most notable were in a schoolyard in the mouth of Bee Canyon, nearly one kilometer from the upper lake. These fissures were characterized by vertical displacements of as much as 50 millimeters and continuous lengths of perhaps 100 meters. They seemingly formed the head of a landslide which had moved down the very mild slope (about 1.5° or 2.5 per cent) eastward toward the upper lake".

In an excellent study of the soil movements near the Van Norman Lakes, Youd describes a major slide in the vicinity of the lakes, called the Juvenile Hall slide. Located on the slide are much of "the San Fernando Valley Juvenile Hall, trunk lines of the Southern Pacific Railroad, San Fernando Boulevard, Interstate Highway 5, the Sylmar electrical converter station, and several pipelines and canals". The slide extended for a distance of 1.2 kilometers from northeast of the juvenile hall to its toe at the shoreline of the Upper Van Norman Lake. The slope of the soil surface averages about 1.5 degrees with the maximum slope of about 3 degrees. It is estimated that the total downslope movements of the slide ranged from 0.5 to 1.5 meters.

Youd also reported that:

> "Numerous sand boils erupted on the lake floor, presumably near the toe of the slide. These were exposed after the water level was lowered. Sand boils were also found at the following locations: Behind the converter station near the base of the graded fill, in a field south of Juvenile Hall,

and along the north edge of the railroad grade in front of the Juvenile Hall Administration Building".

## FAILURES AT UPPER AND LOWER SAN FERNANDO DAMS

Both the Upper and Lower San Fernando Dams were severely damaged by the earthquake. Most of the news reports about these failures centered around the Lower San Fernando Dam where the failure was such that there was a critical danger that the dam would be breached with consequent severe flooding.

The Upper San Fernando Dam was built in 1919-1921 by hydraulic fill methods [4.16]. During the earthquake this dam subsided about 0.9 meters and shifted downstream about 1.5 meters [4.16]. Youd and Olsen reported that "extensive cracking and slumping disrupted the upstream concrete lining of this dam, and the downstream slope of the embankment also was cracked. The concrete lining around the Upper Van Norman Lake was cracked extensively near the high-water line" [4.16].

The Lower San Fernando Dam was constructed in 1913-1915 and it was modified in 1930 and 1940. The construction procedure used in 1913 was a hydraulic fill. The modifications were made using rolled-fill construction. The new construction added a downstream berm and the dam was raised. There was an eight-inch concrete rip-rap liner on the upstream face of the dam. It is estimated that about three million cubic yards of material are in the dam. The dam impounded the lower Van Norman Lake which contained 11,000 acre-feet of water at the time of the earthquake.

The earthquake triggered a massive slide along about two-thirds of the length of the dam, as shown in figure 4.3. The material slid into the reservoir. An immediate decision was made to evacuate about 79,000 people living south of the Lower Van Norman Lake. A 12-square mile area was evacuated. The water level had been lowered by 13 feet by the afternoon of the third day following the quake, and the people living in that part of the San Fernando valley were allowed to move back into their homes. The reservoir contained about 7,000 acre-feet of water at the time the people moved back.

With regard to the failure of the dams as a result of the earthquake, there is a possibility that the failure was due to liquefaction of the material which was placed by hydraulic methods. In the hydraulic fill technique, a pipeline is used to transport soil-laden water to the dam site. By appropriate placement of parallel pipelines, the

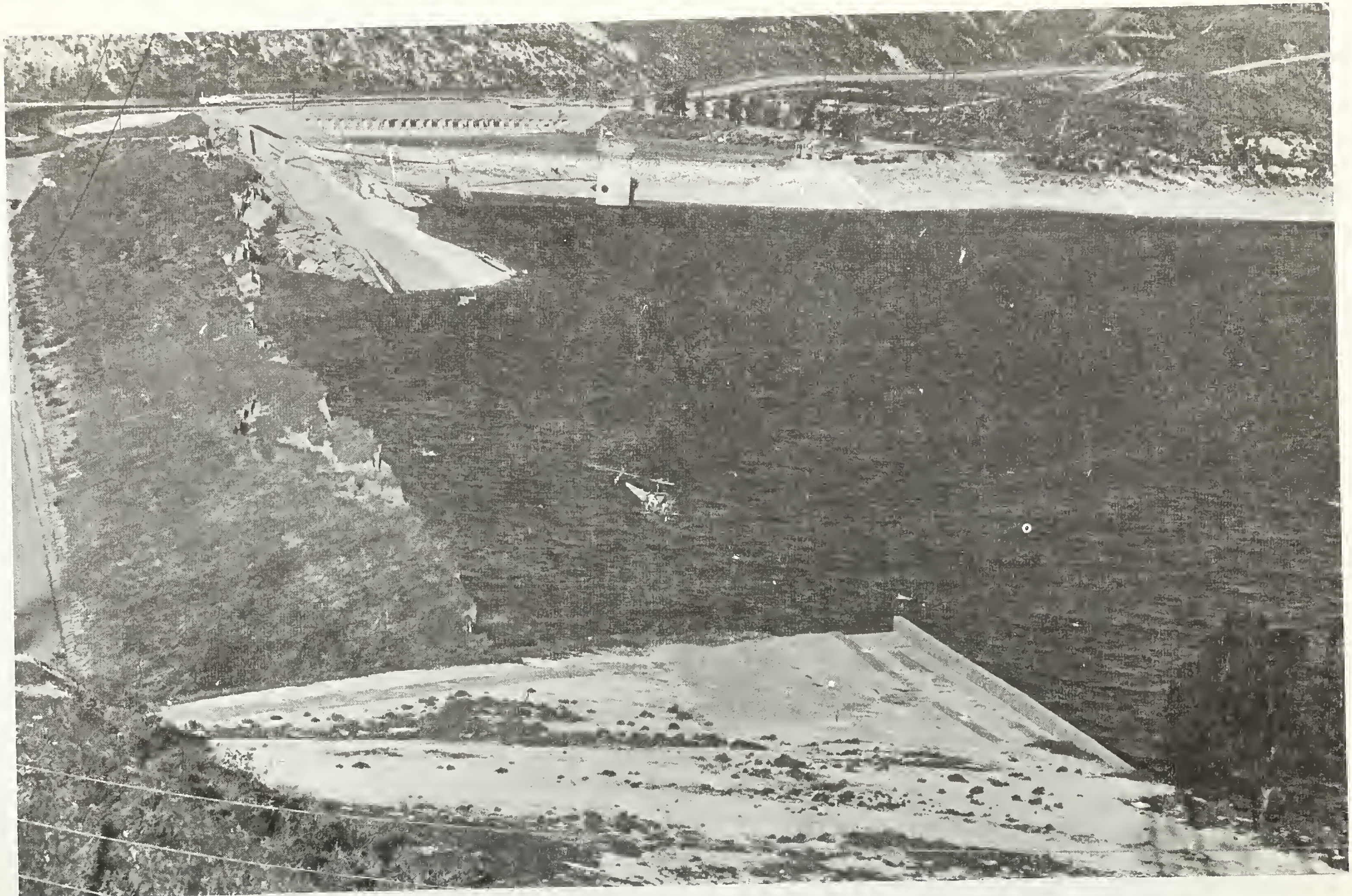


Figure 4.3 Lower San Fernando Dam immediately after quake.

mixture is allowed to flow into the fill area, the soil is deposited and the water returns to the reservoir. The technique is adjusted so that the coarser-grained materials are deposited at the upstream and downstream faces with fine-grained materials being deposited at the core of the dam. The reason that hydraulic techniques have been abandoned is that the deposited material is loose and is subject to densification, hence liquefaction, due to cyclic stresses such as those from earthquakes.

Aerial views of the Lower Van Norman Dam taken after several days of pumping to lower the level are given in figures 4.4, 4.5, 4.6 and 4.7. Figure 4.4 is a view of the Lower San Fernando Dam taken from downstream and clearly indicates the degree of collapse of the main fill over a great portion of the width of the dam. Water level was almost at the height of the lowered crest on the right side of the dam as viewed in this photo. Figure 4.5 is a similar photo taken from upstream and shows both the collapsed concrete liner and the large number of residences in the downstream flood plain. Figure 4.6 is taken from upstream and shows a relatively close-up view of the failure near the east abutment. Original height was at the level of the roadway on the crest. Figure 4.7 shows the degree of failure of the earth dam and the concrete liner.

## 4.5 Comments about Soil Movements

The San Fernando earthquake can be judged from the standpoint of the loss of lives and the property damage. However, the disaster would have been multiplied many times had the water from the Lower Van Norman Reservoir plunged down the San Fernando Valley. This occurrence probably could not have been prevented had the weakened dam been overtopped by the water or had piping been initiated due to seepage through the weakened dam. Either of these occurrences, of course, was a distinct possibility, if not probability.

It may be significant to note that hydraulic fill dams are completely out of vogue in the United States and around the world. The reason is that the material which is pumped and placed by the hydraulic methods is in a loose condition and cannot be compacted and densified and given the appropriate strength with the use of the hydraulic methods. While some strength increase of a hydraulic fill can be expected with the passage of time, there is no assurance that complete stability can be achieved.

No information is at hand regarding the seismic coefficients which were employed in the design of the Lower

*Figure 4.4 Lower San Fernando Dam after lowering of water level.*

*Figure 4.5 Lower San Fernando Dam and endangered residential area.*

*Figure 4.6 East abutment of Lower San Fernando Dam.*

*Figure 4.7 Collapsed liner of Lower San Fernando Dam.*

San Fernando Dam. Seed and Martin state that typical United States practice is to use a seismic coefficient of 0.10 to 0.15 [4.17]. The Bureau of Reclamation states, "In areas not subjected to extreme earthquake conditions a horizontal acceleration of 0.10 of gravity and a vertical acceleration of 0.05 of gravity are generally used" [4.18].

In the analytical methods currently in use for the stability analyses of earth dams, the effects of vertical accelerations are not normally taken into account. The vertical accelerations reported for the San Fernando earthquake are considerable and serious consideration must be given in the future to taking the vertical accelerations into account in analytical procedures.

In many populated regions subject to earthquakes, reservoirs have been built above the centers of population. Therefore serious consideration should be given to a careful study of current analytical methods and to reanalyses of some earth structures. Such a study is highly recommended in view of the potential disaster that could have occurred in the San Fernando valley.

## 4.6 References


4.1 Wentworth, C. M. and Yerkes, R. F., "Geologic Setting and Activity of Faults in the San Fernando Area, California," Geological Survey Professional Paper 733, U. S. Government Printing Office, Washington, D. C., 1971.

4.2 Bailey, T. L. and Jahns, R. H., Geology of the Transverse Range Province, southern California, in Jahns, R. H., ed., Geology of southern California: California Division of Mines and Geology Bulletin 170, 1954.

4.3 Jennings, C. W., and Strand, R. G., Geologic Map of California, Olaf P. Jenkins edition, Los Angeles sheet: California Division of Mines and Geology, scale 1:250,000., 1969.

4.4 Corbato, C. E., Bouguer Gravity Anomalies of the San Fernando Valley, California: California Univ. Pubs. Geol. Sci., v. 46, no. 1, 1968.

4.5 California State Water Rights Board, The City of Los Angeles vs. City of San Fernando: California State Water Rights Board, Report of Referee, No. 650079, v. I; v. II, 1962.

4.6 Oakeshott, G. B., Geology and Mineral Deposits of San Fernando Quadrangle, Los Angeles County, California: California Division of Mines and Geology Bull. 172, map scale 1:62,500, 1958.

4.7 "Anchorage Area Soil Studies, Alaska," Shannon and Wilson, Inc., Seattle, Washington, A Report to U. S. Army Engineer District, Anchorage, Alaska, 28 August 1964.

4.8 "The Landslide at Surte on the Gota River," Jakobson, B., Proceedings, Royal Swedish Geotechnical Institute, Vol. 5, p. 87, 1952.

4.9 "The Niigata Earthquake and Resulting Damage to Reinforced Concrete Buildings," International Institute of Seismology and Earthquake Engineering, Tokyo, Japan, February 1965.

4.10 "Foundation Investigation, Olive View Hospital, Olive View, California," L. T. Evans, Incorporated.

4.11 "Foundation Investigation, Central Heating and Air Conditioning Plant, Olive View Hospital, Olive View, California," Converse Foundation Engineers, Los Angeles, California, May 20, 1965.

4.12 "Report of Foundation Investigation, Proposed San Fernando Branch Juvenile Hall, San Fernando Road and Yarnell Street, Los Angeles, California," LeRoy Crandall & Associates, Los Angeles, California, September 27, 1961.

4.13 "Foundation Investigation, Proposed Holy Cross Hospital, Mission Hills Road and Indian Hills Road, San Fernando Valley, Los Angeles, California," Converse Foundation Engineering Company, Pasadena, California, July 30, 1958.

4.14 Morton, Douglas M., "Seismically Triggered Landslides in the Area Above San Fernando Valley," Geological Survey Professional Paper 733, U. S. Government Printing Office, Washington, D. C., 1971.

4.15 Youd, T. L., "Landsliding in the Vicinity of the Van Norman Lakes," The San Fernando, California Earthquake on February 9, 1971, Geological Survey Professional Paper 733, U. S. Government Printing Office, Washington, D. C., 1971.

4.16 Youd, T. L., and H. W. Olsen, "Damage to Constructed Works, Associated with Soil Movements and Foundation Failures," Geological Survey Professional Paper 733, U. S. Government Printing Office, Washington, 1971.

4.17 Seed, H. Bolton and Martin, Geoffrey R., "The Seismic Coefficient in Earth Dam Design," Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 92, No. SM3, May 1966.

4.18 United States Department of the Interior, Bureau of Reclamation, Design of Small Dams, U. S. Government Printing Office, Washington, 1965.

## 5.1 Building Code Seismic Force Requirements

### INTRODUCTION

In the United States, the requirements most used for the design of buildings to resist earthquake forces are based on provisions contained in the Uniform Building Code (UBC) published by the International Conference of Building Officials. The UBC provisions are closely patterned after the SEAOC (Structural Engineers Association of California) recommendations. In order to compare the structural designs of the various buildings that were subjected to the February 9, 1971 earthquake, it is necessary to know the provisions of the building codes under which they were designed. As more research data are developed and more accurate design methods become available, the building codes are revised to incorporate this new information. The Uniform Building Code is thoroughly revised every three years. Therefore the specific requirements of the particular code in force at time of design and construction become important in evaluating the building performance and relating that performance to the latest lateral force requirements.

Current provisions of the Uniform Building Code are based on the concept of an equivalent static lateral force V which represents the base shear induced by the ground motion. The general code expression is $V = ZKCW$, where Z is a coefficient expressing the degree of probable intensity of an earthquake in various geographical regions (figure 5.1), K is a coefficient expressing the probable performance of various types of framing and bracing systems, C is the basic coefficient for lateral force, and W is the total dead load and, in some cases, a portion of the live load. There are no requirements for consideration of vertical forces due to seismic action.

### HISTORY OF CALIFORNIA EARTHQUAKE REQUIREMENTS [5.1 and 5.2]

After the 1906 earthquake, San Francisco was rebuilt under a code which specified a 30 psf wind force to effect both wind and earthquake resistance. In the years that followed, leading structural engineers used the concept of lateral earthquake forces proportional to masses, but this simple Newtonian concept did not find its way into codes until 1927 in the Uniform Building Code and more extensively in 1933 following the Long Beach earthquake.

The Long Beach earthquake destroyed many buildings in

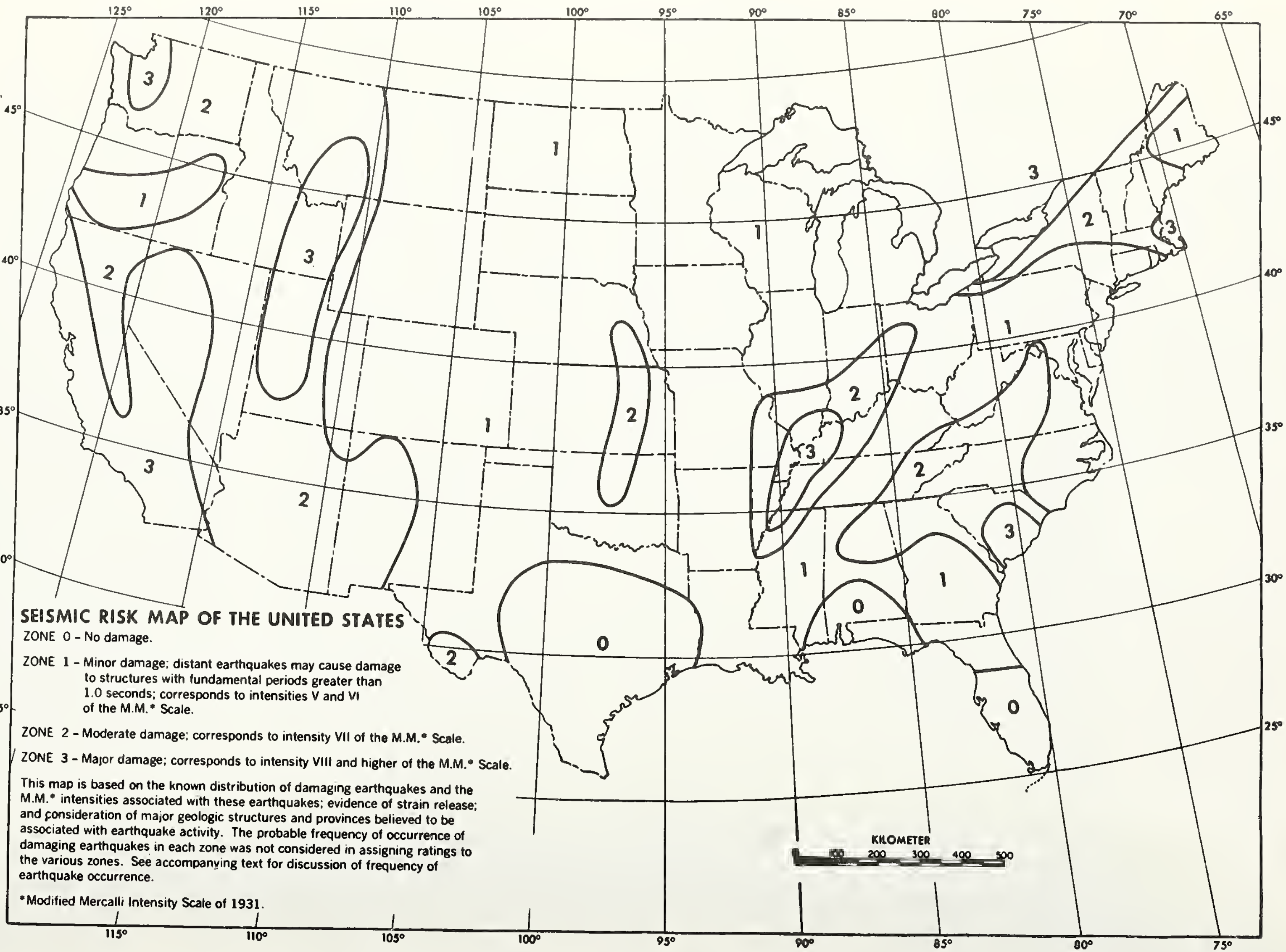



Figure 5.1 Seismic risk map of the United States (after S. T. Algermissen, 1969).

that area of California including many public school buildings. Realizing that much of the loss and damage could be avoided if the buildings had been properly constructed, the State Legislature adopted the Field Act (see chapter 6 for discussion).

In a study of code provisions, Steinburgge, Manning, and Degenkolb [5.1] described three "eras" of building code requirements for earthquake resistant design. Portions of their description are presented below, along with the description of a fourth "era".

> "The first era encompasses the earliest provisions (1927) through the 1946 edition of the Uniform Building Code. The essential features consisted of a constant "C" value, usually 8 percent gravity (g) for all buildings in Zone 3, although on very poor soil this coefficient would be doubled. The weight factor "W" included full dead load plus half of the vertical design live load. In comparing the magnitude of the "C" factor, it is important to consider the weight on which it is based and the allowable material stresses to be used in the design. Otherwise a direct comparison of "C" is meaningless."
>
> "This early code seemed reasonable for low buildings of only a few stories in height. In taller structures, however, the earthquake requirements were very restrictive, and experience in California with the 1906 San Francisco earthquake as well as those at Santa Barbara (1925) and Long Beach (1933) indicated that the coefficient may have been much too high for tall framed structures (usually steel framed)."
>
> "The next era started with the 1949 edition of the Uniform Building Code, which incorporated the provisions of the then current City of Los Angeles Building Code. This used the formula for $C = 0.60/(N + 4.5)$ where N is the number of stories above the one under consideration. This coefficient was to be applied to the dead load only, neglecting all live loads. Since C for a one-story building worked out to be 13.3 percent, the net effect, considering both dead and live load requirements, was roughly comparable to the earlier 8 percent in many structures. The provisions of this formula for C, however, enabled the coefficient to be reduced for taller structures to a level that was more consistent with the experience gained on taller framed structures. The empirical formula

> was to be applicable only up to 13 stories, since Los Angeles had a 13-story height limitation. This group of building codes was the first widely used building codes that recognized the dynamic effects of an earthquake as related to the flexibility of a building, although the flexibility was crudely related only to the number of stories in a building."

> "The third era was started in 1952 because of the problem that existed in San Francisco where there was no height limit, so that the need for more rational code provisions was recognized. A committee was formed which resulted in the "dynamic approach" as exemplified by the 1956 San Francisco Building Code requirements. Subsequently, a joint North-South Committee of the Structural Engineers Association of California (SEAOC) was formed to achieve a uniform code that could be used statewide."

This resulted in the 1960 SEAOC publication, "Recommended Lateral Force Requirements," which introduced a new method of application of two variables (C and K) used in the base shear formula (V=KCW). One variable (C) directly introduced the fundamental period of the structure and defined a method of determining the period where it cannot be accurately predetermined. A second variable (K) is dependent on the type of structural system to be used. Based on research, seismic experience in the United States, and damage studies, certain types of construction are assigned bonus considerations for the seismic factors. The 1960 SEAOC recommendations appeared in the 1961 and 1964 editions of the Uniform Building Code, with little change.

> "The one change adopted by the International Conference of Building Officials, in extrapolition of the SEAOC provisions, concerned the use of zones. The SEAOC merely stated that all California should be in Zone 3 and, by inference, recognized that all areas subjected to major earthquakes of the intensity to be expected in California should be in Zone 3. Other zones were not and are not recognized by the SEAOC."

The fourth era began with the publication of the 1966 SEAOC "Recommended Lateral Force Requirements" which were subsequently incorporated into the 1967 Uniform Building Code. Some of the important revisions pertained to the following provisions:

(1) Qualifying of reinforced concrete moment-resisting

frames to the necessary ductility to permit this construction in buildings more than 160 feet in height.

(2) A distinction in defining Space Frame-Moment Resisting and Space Frame-Ductile Moment Resisting and the requirement of Ductile Moment-Resisting Space Frames in buildings over 160 feet in height.

(3) Use of K values of 0.80 or 0.67 requires a Ductile Moment-Resisting Space Frame capable of resisting 25 percent or 100 percent, respectively, of the total required lateral forces for all building heights. A Moment-Resisting Space Frame is not required to meet the special ductility requirements of a Ductile Moment-Resisting Frame if a K value of 1.00 is used in design.

The latest edition (1970) of the Uniform Building Code contains earthquake provisions based on the 1968 revision of the SEAOC "Recommended Lateral Force Requirements." After a review of the damage resulting from the earthquake in Caracus, Venezuela, in 1967, the SEAOC realized that additional consideration should be given to the ductile behavior of tied concrete columns which receive high over-turning loads but transmit little seismic shears. Accordingly, the 1968 revision provides confinement to tied column cores for their full length unless it can be demonstrated that they will not be subjected to inelastic strains during an earthquake.

## CURRENT EARTHQUAKE REGULATIONS - LOS ANGELES AREA

In the City of San Fernando, the 1967 Edition of the Uniform Building Code was in effect at the time of the February 9, 1971 earthquake. The 1970 Uniform Building Code was adopted by San Fernando on March 29, 1971.

The current City of Los Angeles Building Code became effective on May 2, 1970. The general earthquake requirements in this code are basically the same as those contained in the 1970 Uniform Building Code or the 1968 SEAOC "Recommended Lateral Force Requirements." For building permits issued after July 1, 1965, the City of Los Angeles Code requires that "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 the floor area, shall be provided with three approved recording accelerographs." These instruments are to be located in the basement, mid-portion and near the top of the building. (See chapter 3 for a discussion of the strong-motion and building-period measurements obtained in the San Fernando Earthquake.)

The current County of Los Angeles Building Code became effective July 12, 1968, and is based on the 1967 Edition of the Uniform Building Code. The previous edition of the County Code was adopted in July, 1965, and was based on the 1964 Uniform Building Code. The 1965 edition was revised in November, 1966, to include amendments based on the 1966 SEAOC recommendations. Adoption of a new code edition (1971) based on the 1970 UBC became effective November 27, 1971, by the County of Los Angeles.

The earthquake regulations for public schools in effect as of January 1, 1970, (California Administrative Code, Title 24 and 21) are based on the 1968 SEAOC recommendations. Some of the special provisions of the public school earthquake requirements are given below and outlined in Chapter 6.

Section T21-107 (d) (1).

"The value of KC as used in . . . V=KCW, shall not be less than 0.133 for all one-story buildings and not less than 0.10 for all 2- and 3-story buildings, except that the minimum KC may be 0.10 in one-story buildings with a ductile moment-resisting frame having the capacity to resist the total required lateral force.

"The coefficients, K, as listed . . . are for use under conditions of average earthquake potential hazard. These coefficients may be modified by the Office of Architecture and Construction upon the advice of seismologists, engineering geologists, or structural engineers.

"Any building or structure supported by only a single column or a single row of columns shall be designed for a minimum KC=0.30. For the building or structure with a single row of columns, this requirement shall apply only to the lateral force acting normal to the row of columns. The columns for such structures shall meet all the requirements for ductile frame design.

"Moment-resisting space frames used to resist design seismic forces in any height school building shall meet ductile frame requirements as defined. . ."

## 5.2 Case Histories of Building Damages

Case histories of buildings are discussed in this section according to building use. These are medical facilities, commercial buildings, residences, miscellaneous structures, and post offices. Schools, including the San Fernando Juvenile Hall, are covered in chapter 6. Buildings related to utilities, are covered in chapter 8.

For the most part, severely damaged buildings are discussed here; although several buildings with moderate or little damage are described. It is important to note that many buildings which were subjected to accelerations on the order of 20 per cent of gravity were not damaged.

Figure 5.2 is a general area map, designated Map 1, which indicates areas covered by detailed maps 2, 3, 4, and 5. These detailed maps are shown in figures 5.3a, 5.3b, 5.3c, and 5.3d. Damaged structures discussed below are referenced to a map and location number. Locations are numbered in general geographical order, starting from the Newhall-Saugus area in Map 2 and moving south.

There were no accelerometers in the immediate vicinity of the hardest-hit areas. There were accelerometers on the Pacoima Dam east of location 7 (Map 3), the Holiday Inn at location 25 (Map 4), the Bank of California at location 28 (Map 4), Crocker Citizens Bank at location 29 (Map 4), and the Sheraton-Universal Hotal at location 31 (Map 4). The components of ground acceleration as a fraction of gravity are shown on the location maps by a set of orthogonal arrows which indicate direction of horizontal accelerations. Magnitudes are recorded adjacent to the arrow. The value of the maximum vertical acceleration is recorded between the two orthogonal arrows. The Holiday Inn, the Crocker Citizens, and the Bank of California readings were recorded on the first floor. The Sheraton-Universal readings were recorded in the basement.

### MEDICAL FACILITIES

### Los Angeles Olive View Medical Center - Map 3, Location 4

General - The 850-bed Los Angeles Olive View Medical Center, costing approximately $23.5 million, was dedicated in November, 1970. The February 9, 1971, earthquake caused three deaths, evacuation of the structure, and a total loss of the facility. The Medical Center was constructed generally of reinforced concrete and was designed during 1965 under the 1965 Los Angeles County Building code [5.3]. Plan sheets

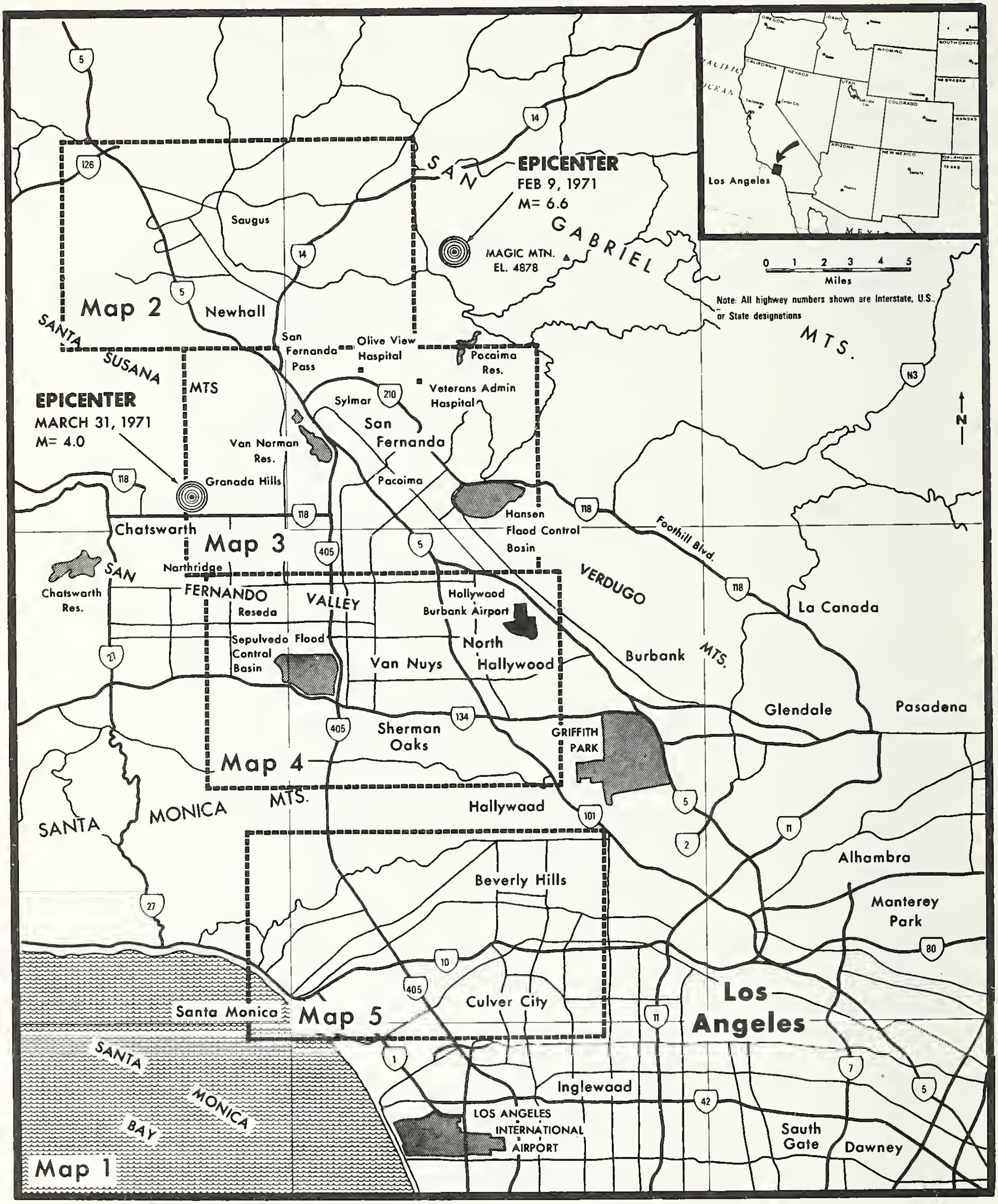



Figure 5.2 Map 1 - Los Angeles, San Fernando Valley, and Newhall.

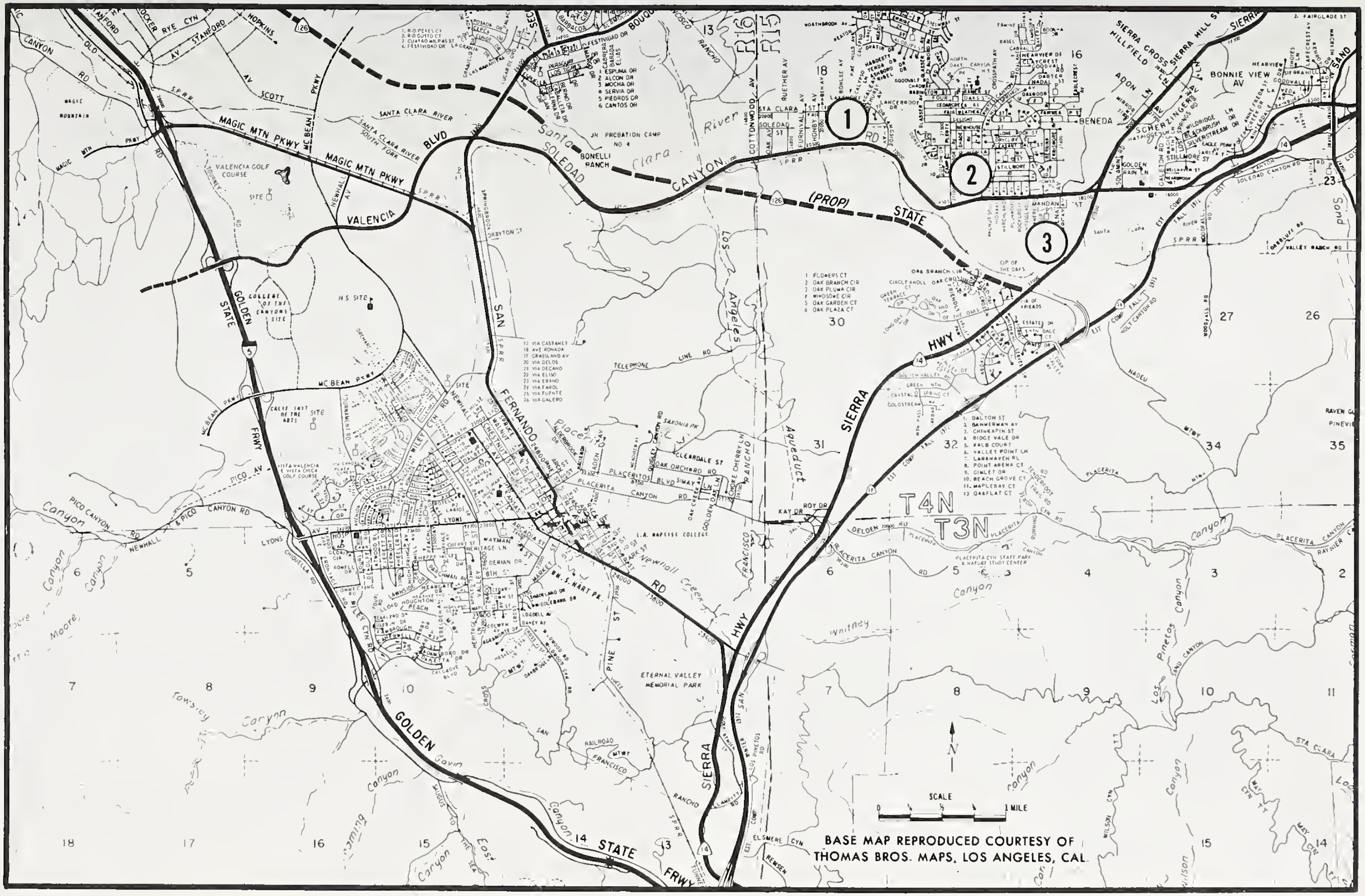



Figure 5.3a Map 2 - Newhall and Saugus, damage locations 1 through 3.

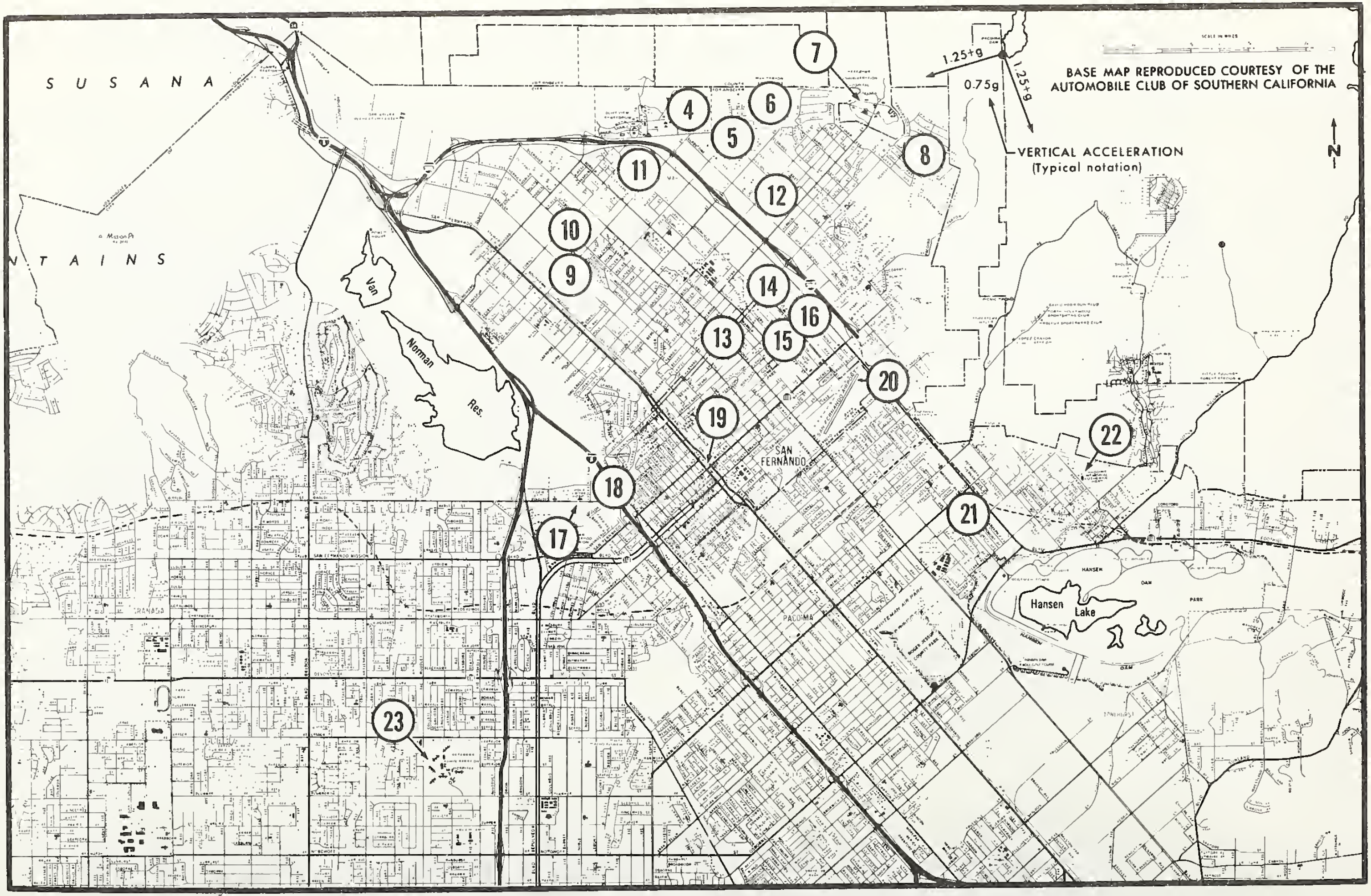



Figure 5.3b Map 3 - Upper San Fernando Valley, damage locations 4 through 23.

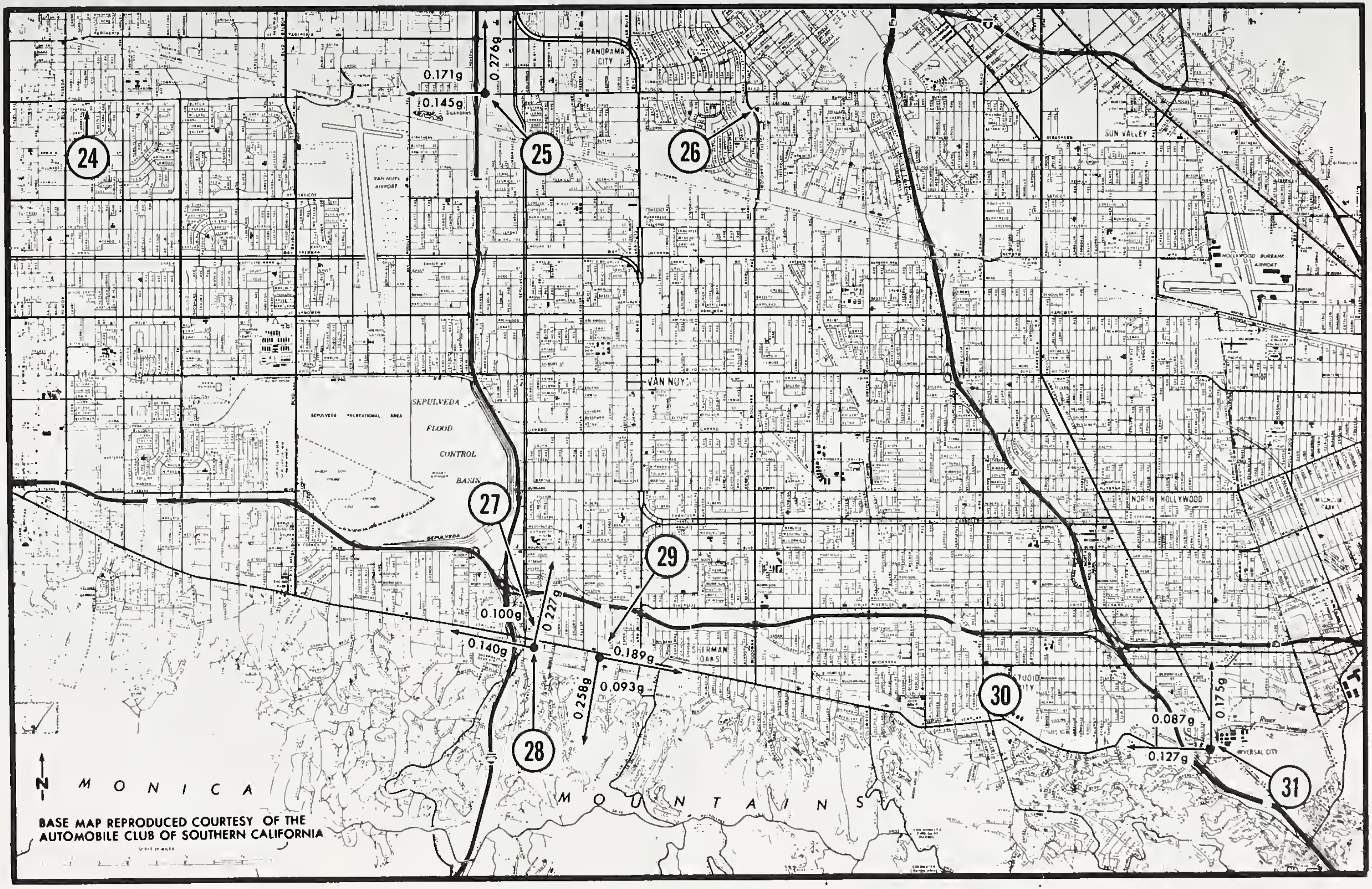



*Figure 5.3c Map 4 - Lower San Fernando Valley, damage locations 24 through 31.*

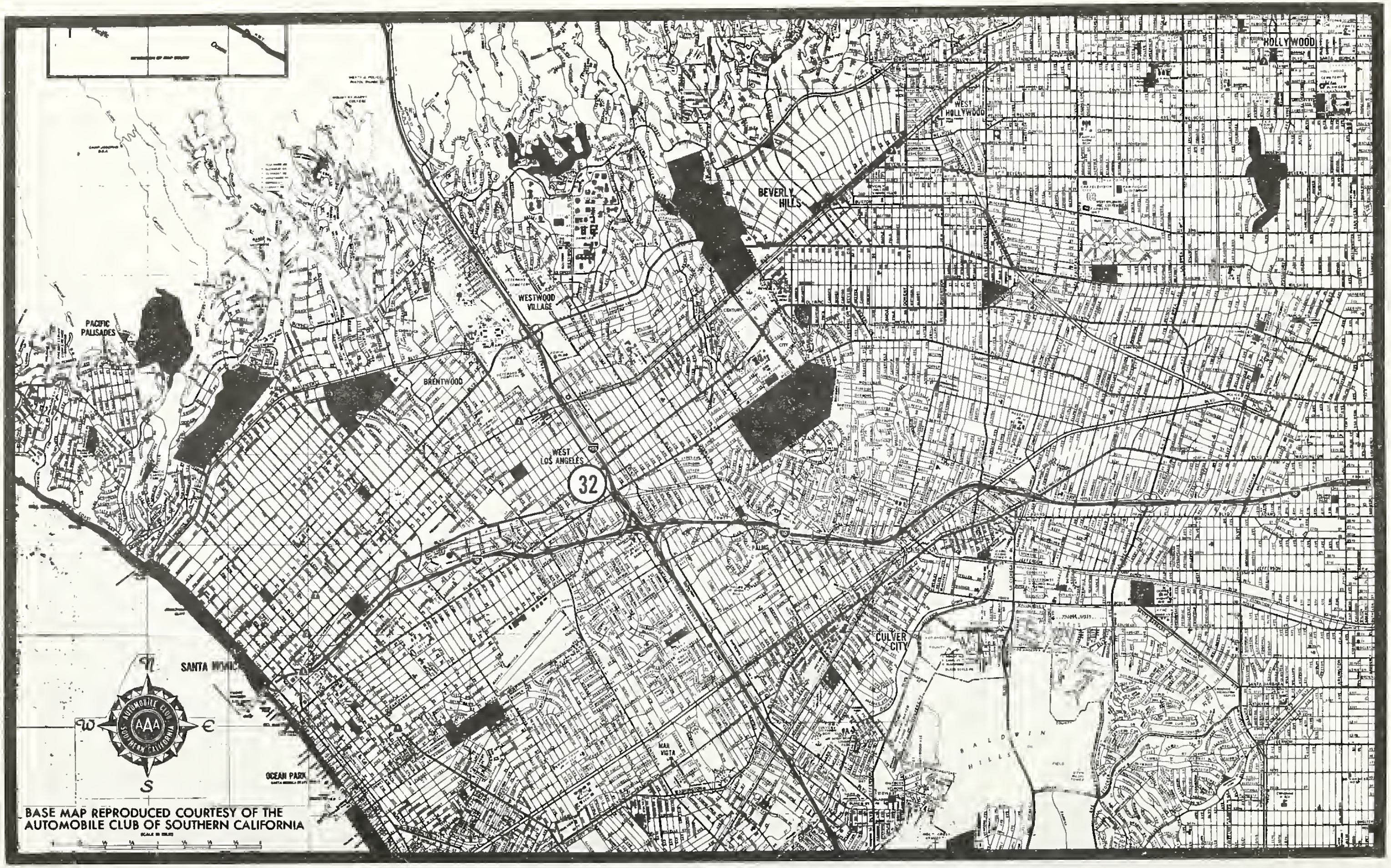



*Figure 5.3d Map 5 - West Los Angeles, damage location 32.*

are dated January 1966, well before the County Code was amended (November, 1966). Although the buildings were "modern", they were not constructed under the latest earthquake design requirements. The County Code as amended in 1966 was significantly different (with respect to column design and allowable concrete shear stress) from the County Code used in design. Specific design details are discussed later in sections describing the separate buildings in the complex. Plans and design computations for the Medical Center were provided by the Department of County Engineers, County of Los Angeles.

A site plan of the Medical Center is shown in figure 5.4. Structures identified are:

1. Main Building
2. Stair Towers
3. Assembly Building
4. Exhaust Pavilion
5. Warehouse
6. Ambulance Port
7. Psychiatric Unit
8. Canopy from Main Building to Psychiatric Unit
9. Power Plant

The main building consists of four wings-A, B, C, and D. These wings are six stories in height, including the ground story. Stair towers, separated structurally from the main building, are located at the outside ends of the four wings. These wings join to form an interior courtyard. The dotted line in figure 5.4 shows the outline of the ground story. On the north and most of the west sides, the ground story is below grade. Stair towers A, B, and D are located over the roof slab of the ground story while stair tower C is supported on footings. General damages are described below. Specific damages are discussed later in sections describing the separate buildings in the complex.

A photograph taken of the Medical Center, as viewed from the southeast, before the earthquake is shown in figure 5.5. The four wings of the main building, the psychiatric unit, the power plant, the warehouse, and the ambulance port are identified. Note that the ground story on the south and east sides is not below grade. However, the roof of the ground story on these two sides is landscaped with earth and plants. A photograph taken of the Medical Center, as viewed from the southeast (as in fig. 5.5), after the earthquake is shown in figure 5.6. All the buildings identified in figure 5.5 are also identified in figure 5.6. Three stair towers at the end of wings A, B, and D are shown toppled in figure 5.6. Although not clearly visible, the stair tower at the end of wing C is tipped to the north, but

SITE PLAN
THE LOS ANGELES OLIVE VIEW MEDICAL CENTER

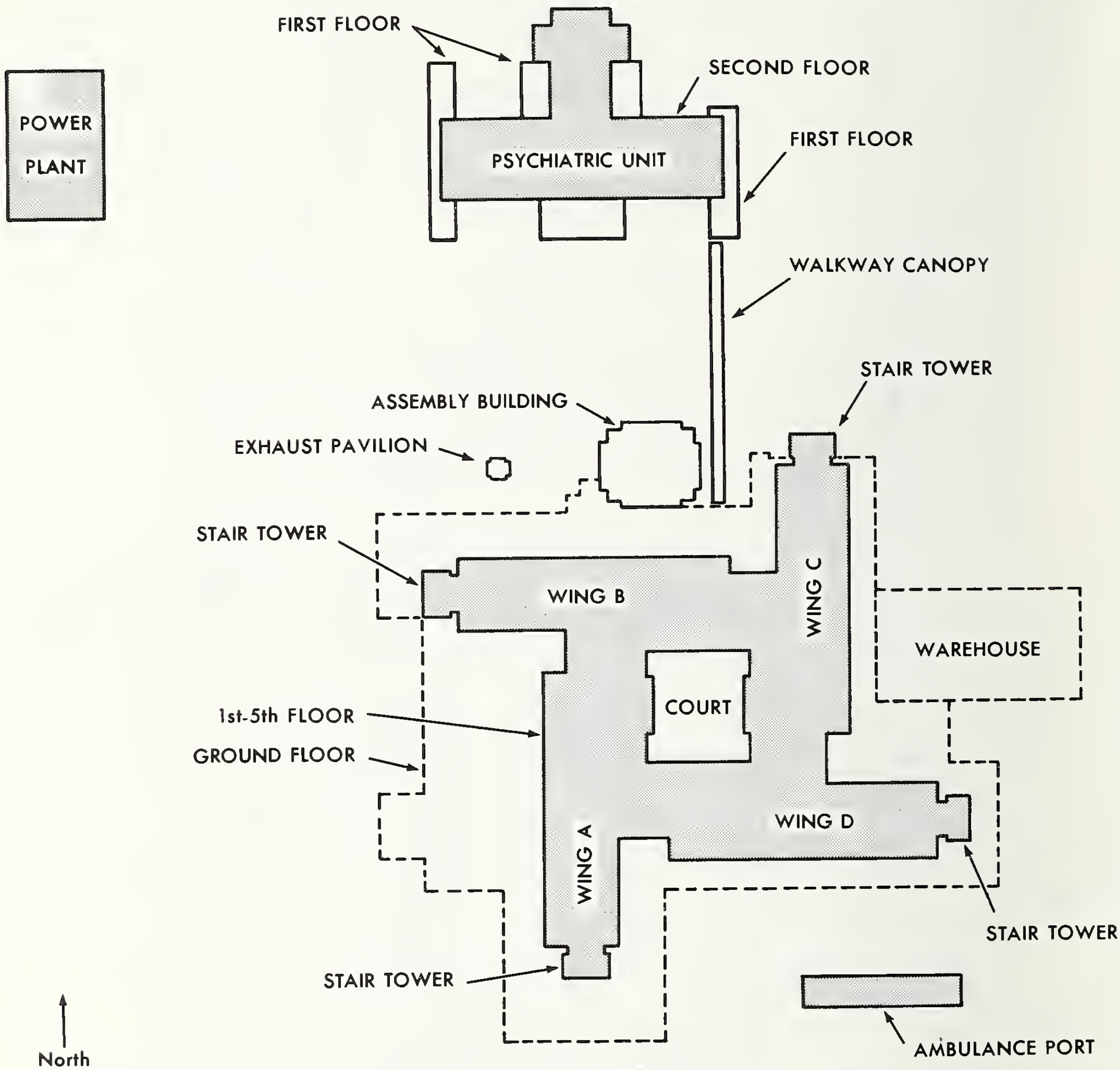



*Figure 5.4 Site plan showing major buildings of the Los Angeles County Olive View Medical Center. The main building consists of the four wings - A, B, C, and D. These wings are six stories in height including the ground floor. The dotted line surrounding the main building shows the outline of the ground floor which is below grade on the north and west sides.*

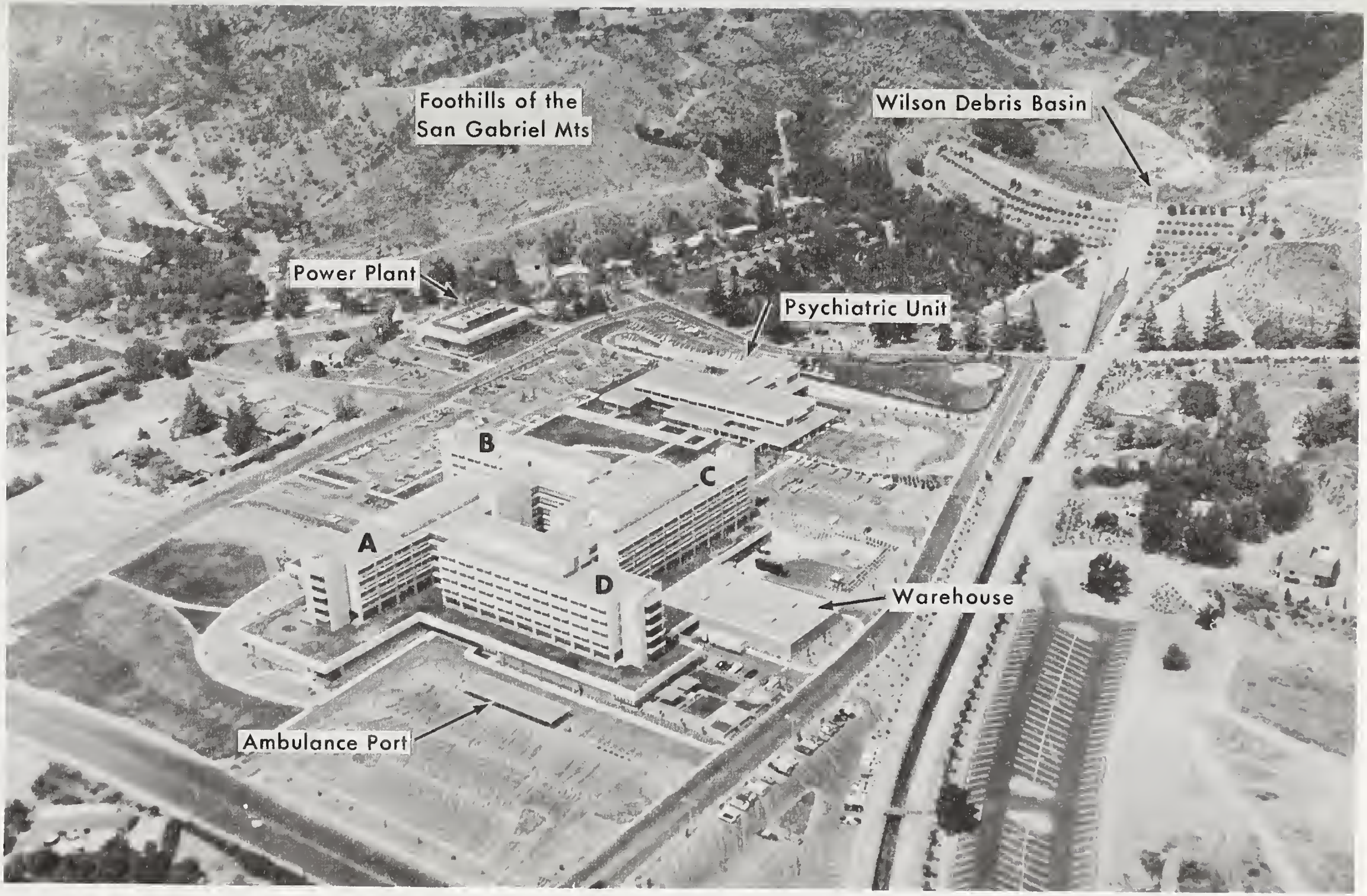



*Figure 5.5 Olive View Medical Center before the earthquake, as viewed from the southeast. Note the proximity of the hospital to the foothills of the San Gabriel Mountains (Los Angeles County Engineer Photo).*

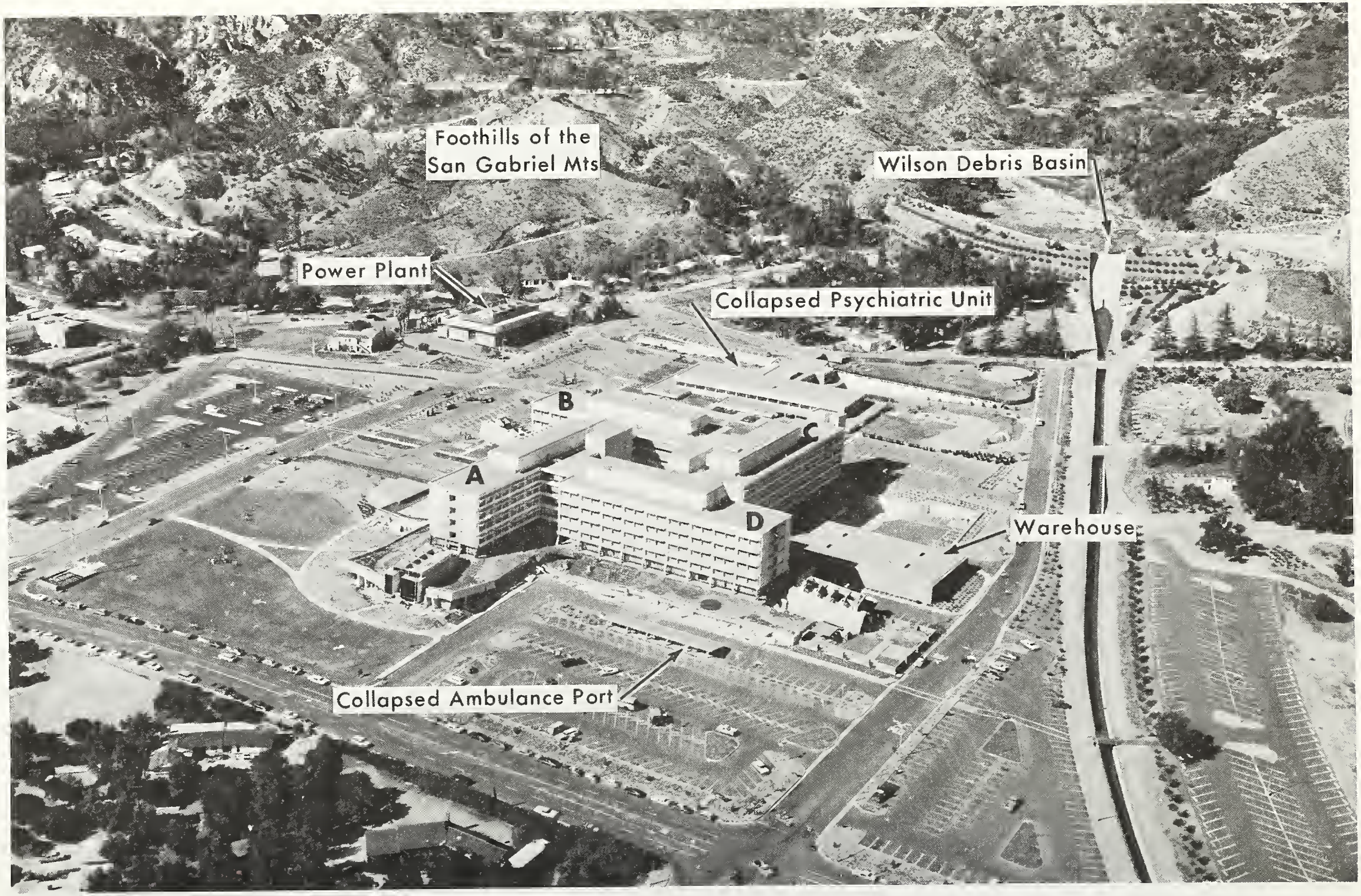



*Figure 5.6 Olive View Medical Center after the earthquake, as viewed from the southeast (Los Angeles County Engineer Photo).*

it is still standing. The three toppled towers broke through the roof of the ground story. The main building is leaning, mostly in the first story, about two feet to the north. It may also be noted on the south side of wing D, that the roof of the ground story is collapsed.

Figures 5.7 and 5.8 are photographs taken before and after the earthquake, similar to figures 5.5 and 5.6, except that the view is from the southwest. Note in figure 5.7 that the ground story is below grade on the north and most of the west sides. Close comparison of figures 5.7 and 5.8 will show that the first story of the psychiatric unit collapsed so that the building now appears to be one rather than two stories in height. Note also the collapsed ambulance port south of wing D.

A photograph of the Medical Center as viewed from the north is shown in figure 5.9. The tipped stair tower at the end of wing C is clearly visible in this photograph. The psychiatric unit, the assembly building, the exhaust pavilion, and the power plant are also shown.

The fact that only three persons lost their lives at Olive View is fortunate. Two of the three died in their iron lungs due to power failure. Only one person, a hospital employee, died as a result of structural failure. This low life loss was partly due to the early hour of the earthquake. At some later time the falling stair towers almost surely would have killed someone. Similarly, no one was occupying the first floor of the psychiatric unit when the first story totally collapsed. However, there were some forty to sixty persons sleeping in the second story who "rode" the building down without loss of life. Had the earthquake occurred a few hours later, many of these people would have been on the first floor.

All patients in the main building were brought down through the interior stairways. Elevators were inoperative due to both the power failure and the distortion of the building. The stair towers could not be used due to their collapse. A fire would have presented a serious problem due to the minimal evacuation route.

Psychiatric Unit - A photograph taken of the collapsed psychiatric unit, as viewed from the southwest, is shown in figure 5.10. As previously indicated and shown in figure 5.7, this was originally a two-story building. A plan view of the unit is shown in figure 5.11.

The psychiatric unit is discussed before the more prominent main building because it clearly establishes two very important considerations: (1) that ground accelerations

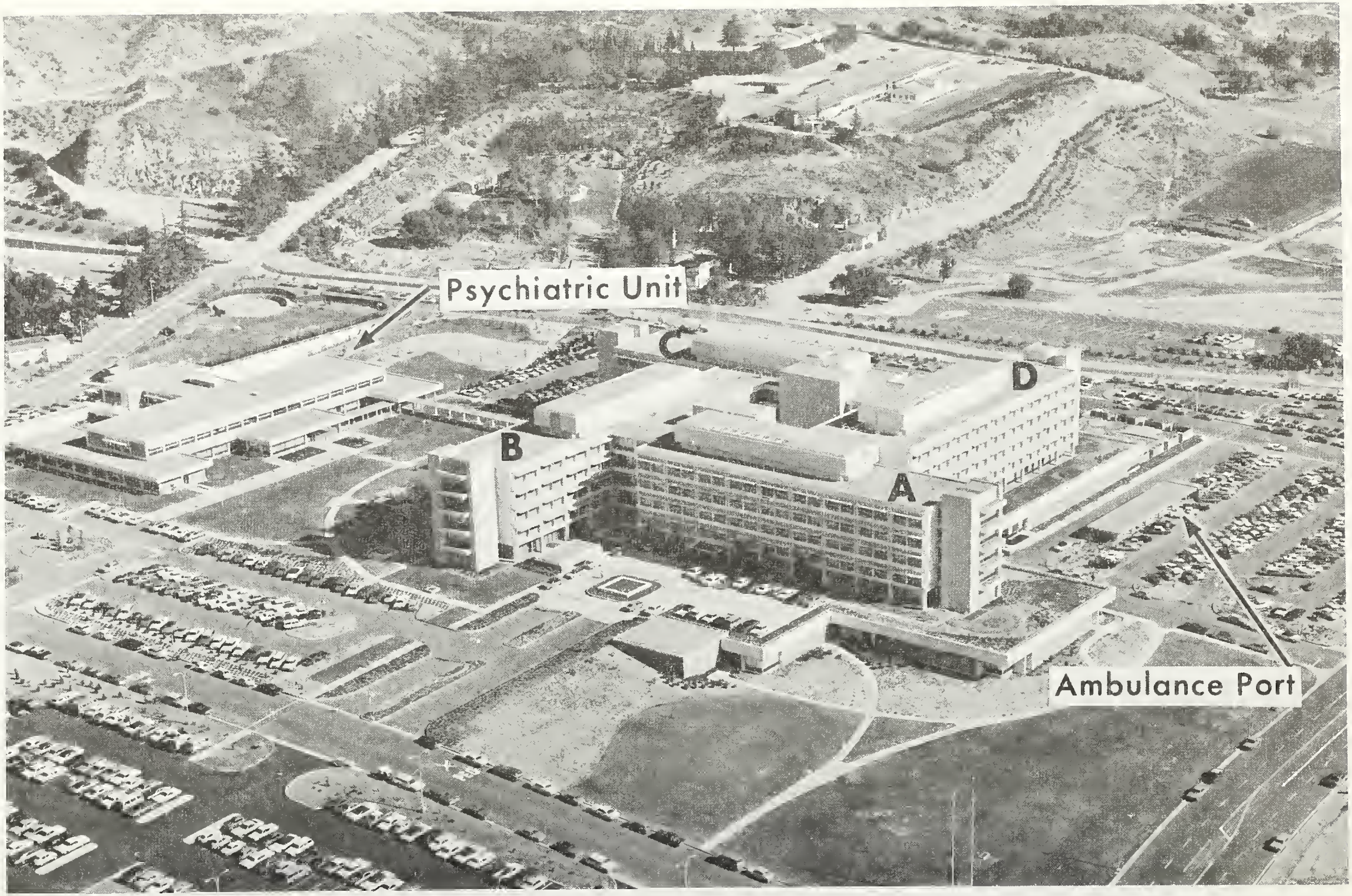



*Figure 5.7 Olive View Medical Center before the earthquake, as viewed from the southwest (Los Angeles County Engineer Photo).*


*Figure 5.8 Olive View Medical Center after the earthquake, as viewed from the southwest.*

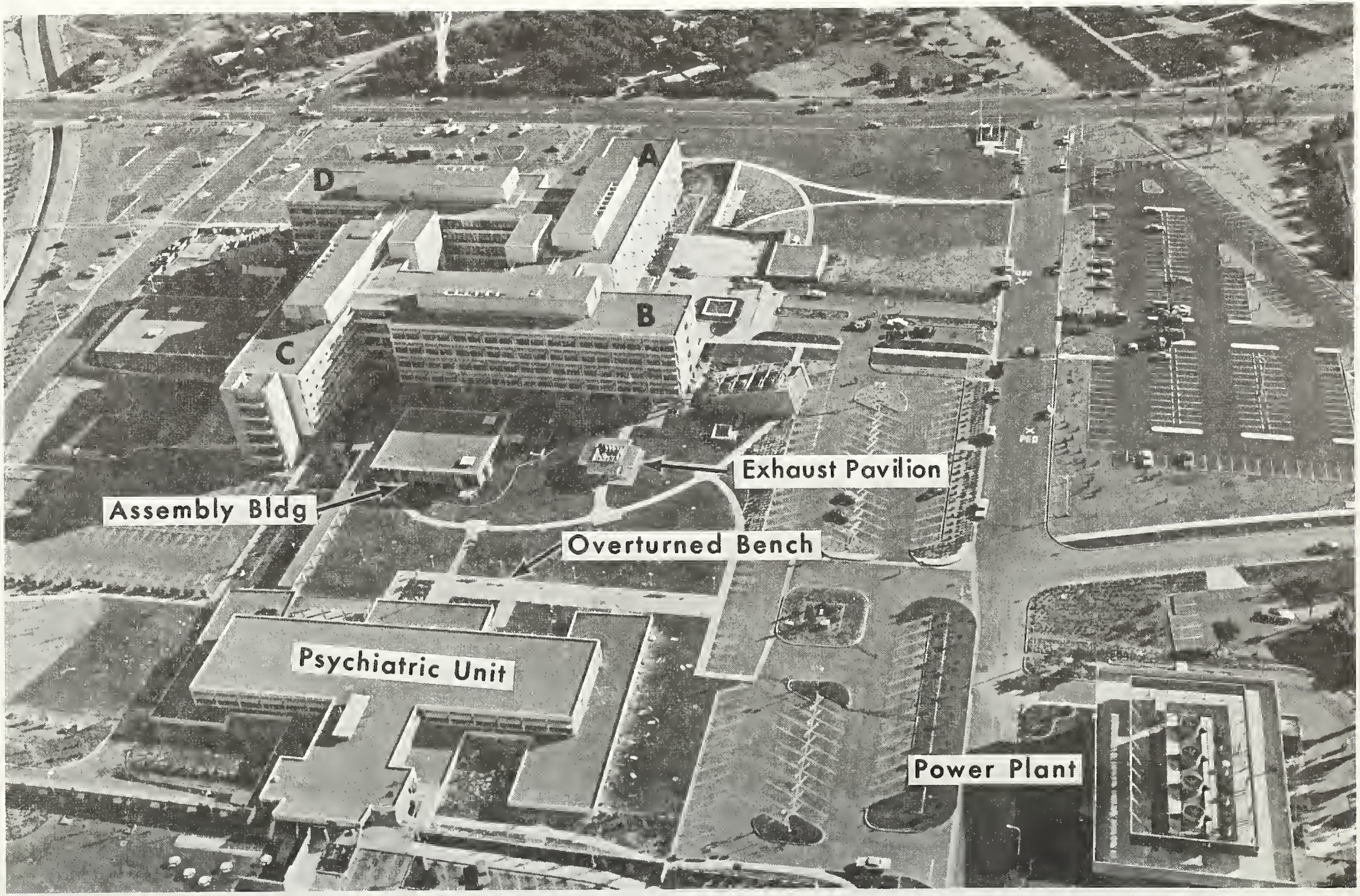



*Figure 5.9* *Olive View Medical Center as viewed from the north. The Psychiatric Unit which was originally a two-story structure collapsed and now appears to be a one-story building. (Los Angeles Times Photo)*

*Figure 5.10* *South and west elevations of the Psychiatric Unit. The ground story of this building was totally collapsed. What appears to be the ground floor was the first floor of this building.*

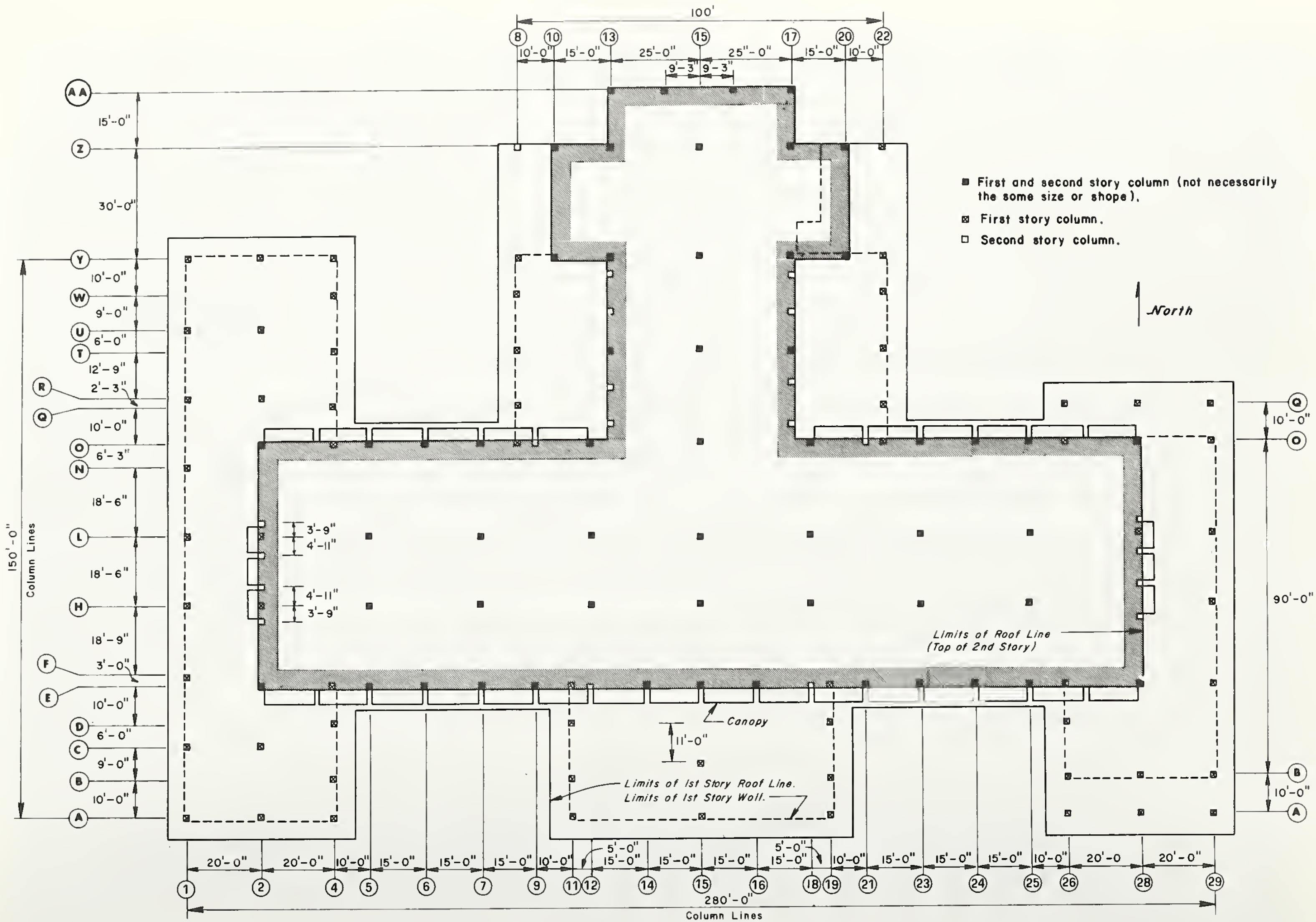



Figure 5.11 Floor plan of Psychiatric Unit.

at Olive View exceeded those anticipated by the governing building code and (2) that column design in the complex was based on then existing deficient code requirements, as compared with those in use today. Both of these findings will be discussed in more detail in following paragraphs.

The structure was a lightweight concrete building almost throughout and was designed as a moment-resisting frame under the 1960 Structural Engineers Association of Southern California recommended lateral force requirements. As a moment-resisting frame, a horizontal force factor K=0.67 was used and as a two-story building a coefficient C=0.10W was used. These values led to a seismic shear of 0.067W, where W is the dead load weight of the building.

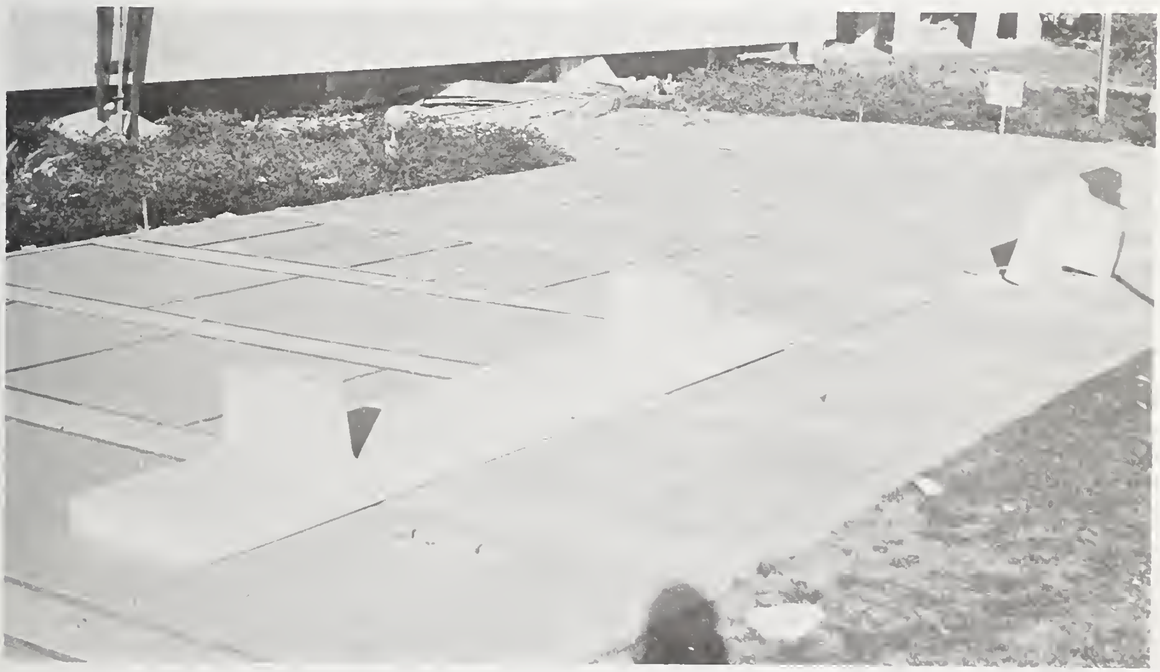
Evidence of ground motion is shown in the photograph of the west elevation of the psychiatric unit in figure 5.12. A close-up of the north entrance indicated in figure 5.12 is shown in figure 5.13. Figure 5.14 shows an overturned bench (see fig. 5.9 for location) which was bolted to the sidewalk, located south of the psychiatric unit. Calculations, which neglect the bolts, indicate that a horizontal ground acceleration of at least one-half gravity was required to overturn the bench. Similar calculations made on an overturned tombstone about three blocks south of Olive View indicated that a horizontal acceleration of one-half gravity or a horizontal acceleration of one-third gravity accompanied by a vertical acceleration of one-third gravity might have occurred in that general vicinity.

The building had an overall height of 27 ft 6 in and the upper story was 240 ft long in the east-west direction and 145 ft long in the north-south direction. Appromixate calculations of the period using the SEAOC formula would indicate values of around 0.1 sec. Under the Standard Acceleration Spectrum proposed by Biot [discussed in reference 5.4], the relative shear of a structure with a period of 0.1 sec would be 60 percent of that experienced by a structure with a period of 0.2 sec. This is the approximate critical period shown in velocity spectrum from the Pacoima Dam acceleration records. The SEAOC provisions are specifically drawn up to neglect any possible decrease in base shear for structures with periods of less than 0.2 seconds. Thus, the forces experienced by the structure before damping action would be in the range of 0.6 to 1.0 of the ground accelerations or somewhere between 0.2 and 0.5 gravity. Either of these combinations indicates that the factor of 0.067 used in design was far too low for the earthquake intensity which occurred at this location. The estimated ground accelerations are not unreasonable in view of the Pacoima Dam acceleration record.


*Figure 5.12 West elevation of the Psychiatric Unit. Damage to the surface of the parking lot, just west of this building, indicates severe ground movement.*

*Figure 5.13 Damage to the canopy at the front (north) entrance to the Psychiatric Unit.*

*Figure 5.14 This overturned concrete bench, which was bolted to the sidewalk, is located south of the Psychiatric Unit.*



The appearance of the structure after failure indicated that the columns shattered totally and pieces of lightweight concrete were hurled appreciable distances. All signs point to shear and compression failures which give virtually no ductility and so the advantages of nonlinear action and large amounts of damping in diminishing the magnitude of lateral forces are minimized.

The second floor was supported by 119 tied columns, of which 107 were 18 in by 18 in in cross section. The ties were No. 3 bars usually spaced at 18 in regardless of column size. In two instances the tie spacing exceeded the minimum column dimension. Lightweight concrete with a specified compressive strength of 3000 psi was used above the first floor. Design was based on working stress design using the general provisions of ACI 318-56 [5.5], which was the procedure in the Los Angeles County Code during the period (1965) that this particular structure was designed. This design basis provides a possible key to the collapse of the first story.

In order for the column ties to be effective as shear reinforcement they would have to be spaced at one-half the effective depth of the cross section. Since the actual ties were usually spaced at the least member thickness, they were ineffective in resisting shear. Thus all shear due to lateral load would have to be carried by the concrete. The 1956 ACI Code allowed a shear stress of 0.03 of the concrete compressive strength, or 90 psi for the concrete used (without a 33 percent overstress allowance with wind and seismic forces). The 1963 ACI Code [5.6], which was in effect in 1965 (but not yet adopted in the Los Angeles County Code) allowed a unit shear stress of 37 psi (for an unknown lightweight aggregate assuming a splitting factor $F_{sp}$=4.0). This lower stress was to correct the fact that 90 psi was known (due to several shear failures since the 1956 ACI Code) to be too high a design value for normal weight and especially some lightweight concretes. However, it should be clearly repeated that 90 psi was the legal limit in the Los Angeles County Code.

As an example of the consequences of this overly large shear stress, consider the case of column 20Z (see fig. 5.11 for location). This column had computed design shears of 17,500 lb in the north-south direction and 10,600 lb in the east-west direction for code values of lateral forces. These shears are complicated by the fact that this is a case of biaxial shear, in that very large simultaneous accelerations were noted in both the north-south and east-west directions. Hence, it is quite possible that the column was under extremely high combined shear stresses. The column is 18 in by 18 in (reinforced with 8 No. 14 bars, 2 in of clear cover). Assuming an effective depth of 15.2 in, the shear stress is 64 psi in the north-south direction and 39 psi in the east-west direction. The resultant of these two gives a shear stress of about 75 psi for the 0.067 acceleration. While this shear stress is below the 1956 ACI Code allowable value of 90 psi, it exceeds both the 1963 working-stress design value of 37 psi and the ultimate shear stress of 56 psi for an unknown lightweight concrete. When the accelerations are scaled up to what they undoubtedly were, it is no wonder that this column failed completely. In this same column the calculated axial load was very low and resulted in only slightly over 150 psi compression in the column; with the high vertical accelerations that were occurring, the compressive force probably went toward zero and would have to be disregarded.

An approximate total-story shear capacity may be obtained by multiplying the total column area (about 40,400 sq. in.) by 0.9 to obtain effective area, and then multiplying this by the 56 psi that one would obtain from the 1963 ACI Code using a splitting factor of 4.0, as specified by the Code for the case where actual values are not known. Through this procedure the total-story shear capacity would be about 2070 kips, or approximately 20 percent of the 10,327 kips dead load which the designer calculated. Assuming uniform shear in one direction and none in the other, this indicates a maximum story shear capacity of about 0.2 gravity. This analysis neglects torsional shear and biaxial shear, both of which were almost certainly present. Both of these factors would lower the maximum story shear capacity even more (note that compressive stresses were very small and probably were close to zero with the vertical accelerations which were probably present). Hence there was no reserve strength in the ground-story columns since accelerations at least as large as one-half gravity probably occurred and subsequent shear failures may have occurred before sufficient ductility was developed to relieve the acceleration forces. Thus the probable cause of failure of the structure was the inadequacy of the columns to take the seismic shears due to (1) the extremely large accelerations and (2) the inadequate shear strength of the columns.

It should be pointed out that a proper analysis of the column strength requires a knowledge of concrete column behavior under biaxial shear. There is very little research data available in this area. It should also be noted that the Los Angeles County Code was amended in 1966, one year after the plans were dated, lowering the allowable concrete shear stresses to those in the 1963 ACI Code. The maximum column tie spacing was also decreased to a maximum of one half the effective section depth for a ductile frame. Columns were also required to be designed to resist the shears accompanying full moment reversal at the column ends for moments of a magnitude equal to the column flexural capacity. All these changes would have improved the column performance although none would have changed the fact that the County Code specified horizontal forces were smaller than those which probably occurred.

Main Building - The main building was of reinforced concrete construction (with hard rock aggregate) supported on spread footings. For seismic design the main building was treated as one complete structure, consisting of wings A, B, C, and D interconnected around a center courtyard (see fig. 5.4). The ground story (which was about 16.5 ft story height) and the first story (which was about 14 ft story height) were considered as moment-resisting space frames with a capacity to resist the total required lateral force, and hence had a horizontal force factor K=0.67. The four upper stories (which were about 12.5 ft in story height) were assumed to have a dual bracing system, consisting of a moment resisting space frame with a capacity to resist not less than 25 per cent of the required seismic force and shear walls to carry the balance. Thus a horizontal force factor K=0.80 was used above the first story. Using the SEAOC procedure, the structure was designed for a lateral force of approximately 8 percent of gravity. Overturning moment was computed using a J value of unity (J has since been voted by SEAOC as always to be taken as unity). A floor plan for the main building is shown in figure 5.15. The limits of the ground story are shown dashed. Flat slab construction was generally used throughout, including the slabs under stair towers A, B, and D. Stair tower C was supported on spread footings.

The spiral columns in the ground story were supported on spread footings. Reinforced concrete flat slabs were used for the ground-story roof. The embankment on the north and west sides of the ground story was held in place by a retaining wall which was separated from the main building by a four-inch seismic joint. The seismic joint was intended to permit the building to move independently of the embankment. However, the width of the joint was inadequate as is shown in figures 5.16 and 5.17, which show compression ridges formed above the joint on the west and north sides respectively. Some damage was caused due to the building's colliding with the retaining wall.

Most columns in the first story were spirally reinforced while corner columns were tied. Shear walls from the stories above were present in the first story although they were carefully detailed to be ineffective in resisting shear by cutting them loose from the frame, since the first story was designed as a moment-resisting frame.

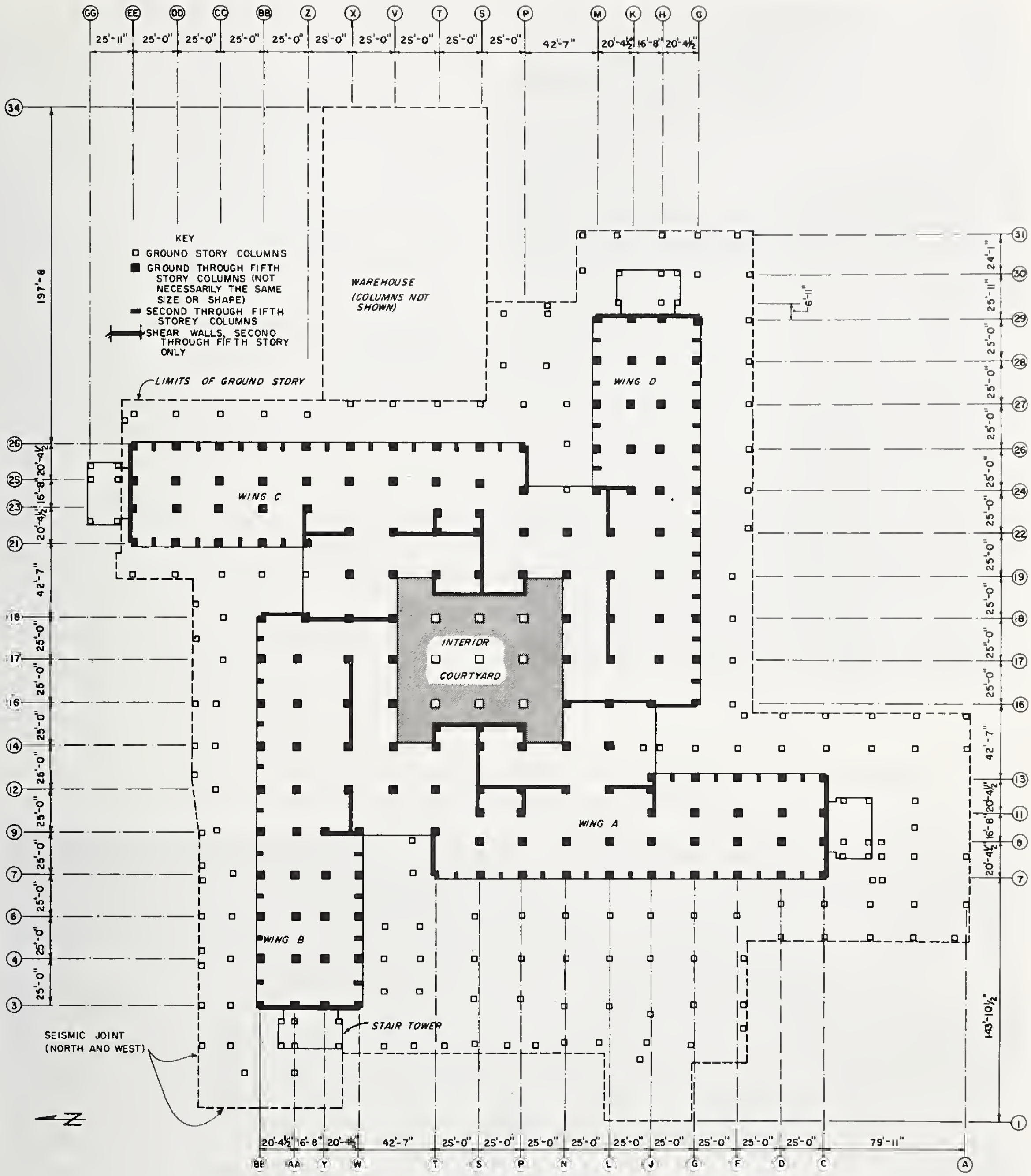



Figure 5.15 Floor plan of Main Building.

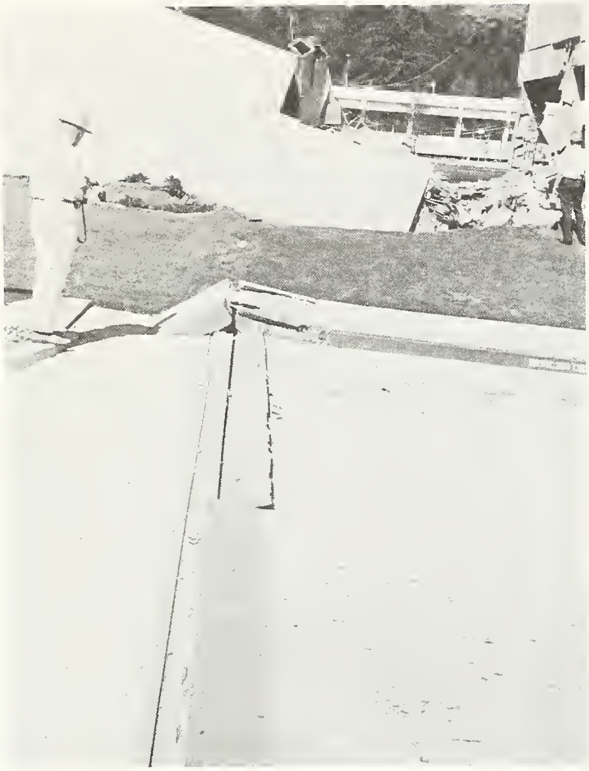
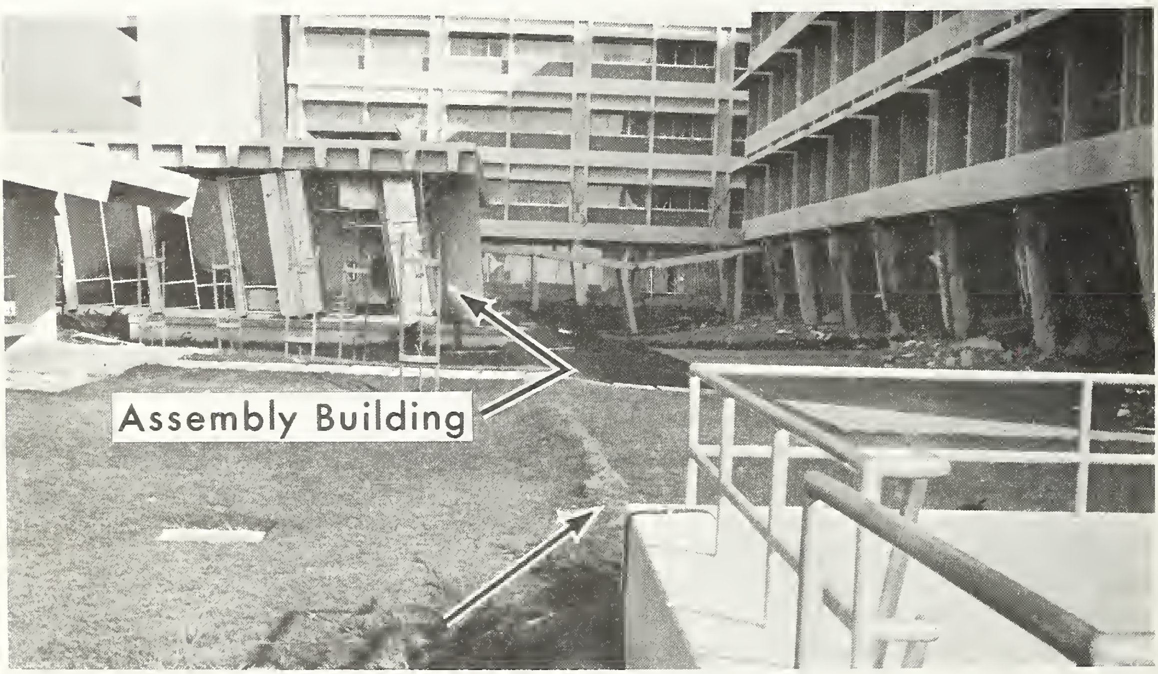


*Figure 5.16 Compression ridge on the west side of the main building. A toppled stair tower is visible in the background.*

*Figure 5.17 Compression ridge (arrow) on the north side of the main building.*



Most first-story columns were square with round spirally reinforced cores, except for the tied corner columns. These latter columns were essentially square with a corner notch out, giving the appearance of a thick L-shaped column.

The typical floor plan for the stories above the first story is similar to that of the first floor, except shear walls were present to carry lateral load. Rectangular tied columns were used above the first story, except for exterior columns which were C shaped.

The west face of Wing A is shown in figure 5.18. This view shows the large lateral displacement of the wing to the north. Most of the displacement is in the first floor. This is shown clearly in the photograph of the main entrance in figure 5.19 (location shown in fig. 5.18).

A typical first-story tied corner column is shown in figure 5.20. Note that this column was completely shattered and incapable of resisting load. Vertical bars in some similar columns actually fractured as shown in figure 5.21. On the other hand the spiral columns shown in figure 5.20, although having undergone tremendous lateral displacements, were still capable of carrying load. However, it is doubtful that they could have withstood much more strong ground motion.

The corner column in figure 5.20 had a design shear of 75 kips in the east-west direction and 108 kips for shear in the north-south direction for building code lateral force values. The L-shaped column has about 880 sq in of area and probably 800 sq in effective for shear. The tie spacing of No. 3 bars at 18 in is so large that the ties cannot count as shear reinforcement. This means the shear stresses in the north-south direction are approximately 135 psi and in the east-west direction 93 psi. These have a vector sum of approximately 160 psi. This shear stress would compare to the 1956 ACI allowable value (without a 33 percent overload allowance) of 90 psi, and 1963 ACI Code allowable value of approximately 80 psi for working stress and approximately 120 psi for the ultimate shear stress of 5000 psi concrete. This is based on the relatively low code acceleration coefficient of about 8 percent which was used in design. This indicates that this column was on the borderline of adequacy under the then existing Los Angeles County Code requirements and would be inadequate if biaxial shear effects are considered. There is no mention in the code of such biaxial shear effects. When the more restrictive 1963 ACI Code shear provisions are considered, the column would be clearly insufficient for the design lateral forces. Had the County Code been revised to include the 1963 ACI Code provision, additional lateral reinforcement would have been clearly required in this column.


*Figure 5.18* *West face of Wing A. Note the extensive damage at the first floor and the lateral displacement of the building to the north. Most glass was intact in windows above the third floor.*

*Figure 5.19* *Patient evacuation through the main entrance of Wing A. (Los Angeles Times Photo)*

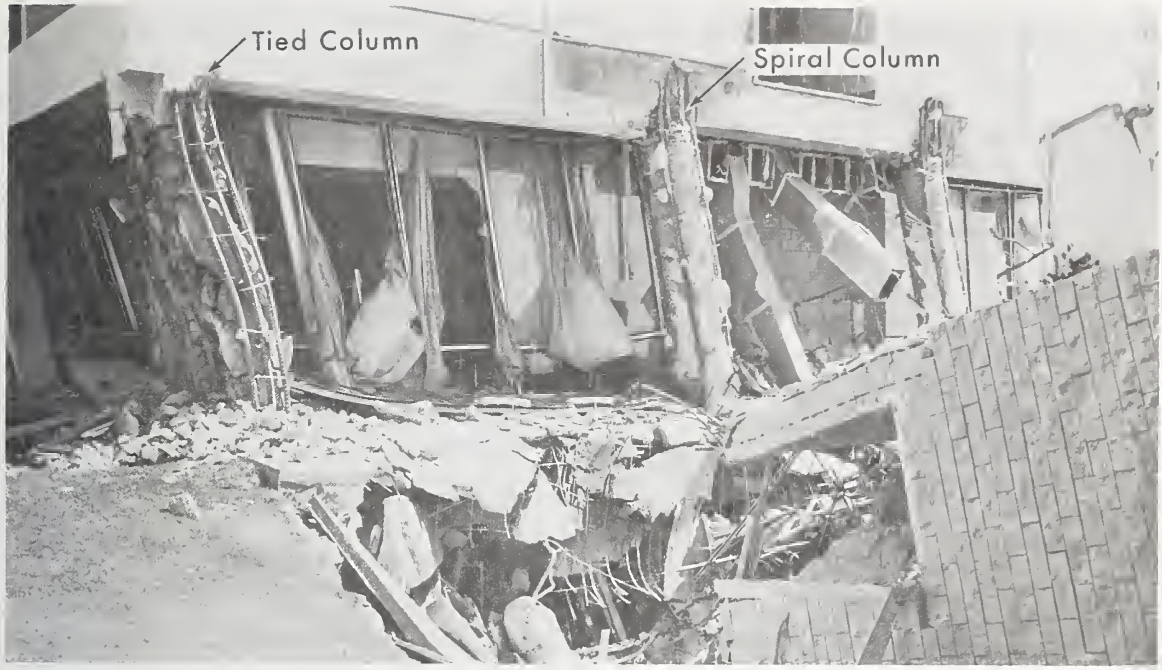
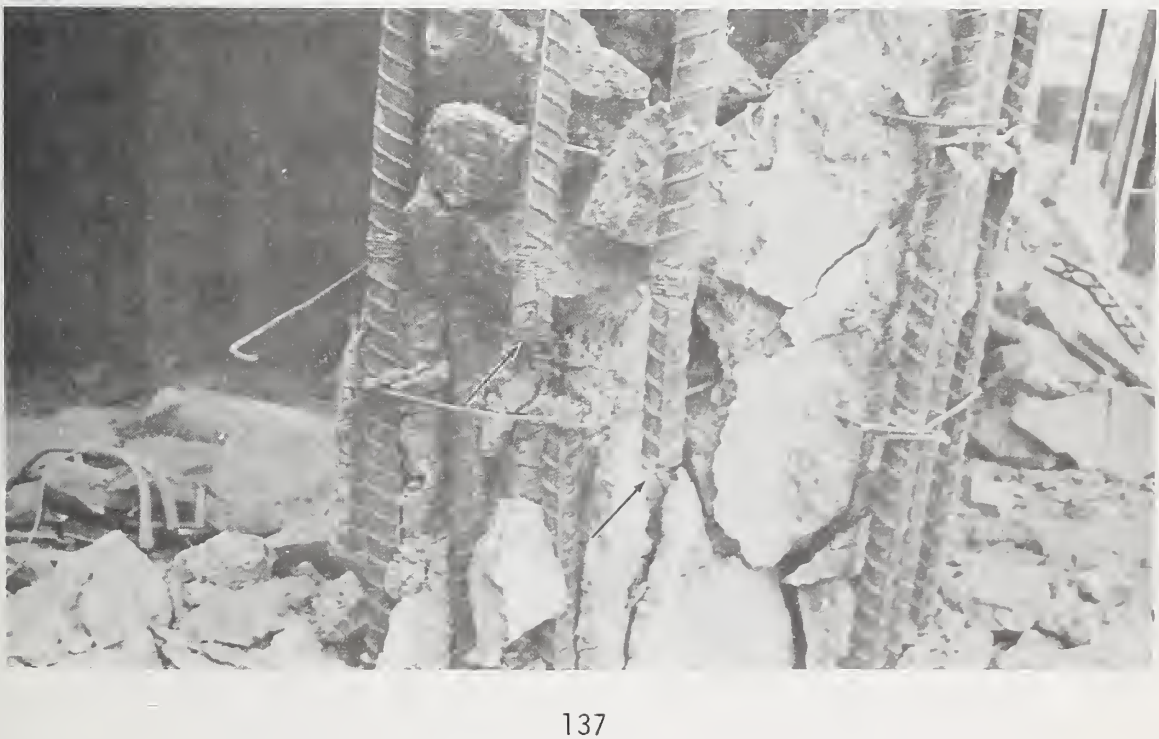



Figure 5.20 West end of Wing B. Note the contorted column and the extent of the northward drift of the building. In the right foreground is the base of the toppled stair tower. Note that this stair tower has collapsed through the roof of the ground story.

Figure 5.21 Close-up view of the first story column in the northeast corner of Wing B. Note the two fractured main reinforcing bars (arrows) and broken ties.



The same column in the basement story had design shears of 83 kips for the east-west direction and 88 kips for the north-south direction. In general the highest and most critical shears occurred in the first story rather than the ground story which had additional and larger sized columns and hence more capacity for taking shear. For instance, the particular column discussed above increased to a 32 in by 32 in spiral column in the ground story below. The reason for the transition from the spiral to a tied column was to provide the L-shape that was apparently for architectural purposes.

The east face of Wing C is shown in figure 5.22. Again the lateral displacement of the building to the north in the first floor can be seen. Note the diagonal cracking in the second-story C-shaped columns which indicates that the column was subjected to load reversal.

What appears to be an embankment in the foreground of figure 5.22 is actually a collapsed garden roof. This type of failure is shown more clearly in figure 5.23, the south face of Wing D. The garden roof which is an extension of the first floor, can be seen in its collapsed position. One hospital employee was killed at this location. Several protruding ground-story columns which originally supported the garden roof are indicated by arrows. A close-up view of one of the failed ground-story columns which originally supported the roof is shown in figure 5.24. A photograph of a ground-story column with the top completely stripped of reinforcement is shown in figure 5.25. The extensive column damage at the first floor is typical of column damage found at the first floor in all wings of the hospital.

The south row of first-story columns in Wing D is shown in figure 5.26. The first column to the left is the L-shaped corner column. The remaining columns immediately visible to the right are spiral columns with a No. 18 bar in each corner running about two-thirds of the way up the column height. These bars, which are continued from the story below, have broken out of the column corners. These bars were not confined by spirals or ties, contrary to code requirements.

The enclosed first-floor court formed by the four hospital wings, is shown in figure 5.27. The sagging of the court was caused by failure of the ground-story columns. Other interior photographs are shown in figures 5.28, 5.29 and 5.30.

Several points are illustrated by the column shown in figure 5.31. A spiral column is basically more ductile than a tied column, but in lateral motion this ductility can only be achieved if ductility exists at the column-to-beam connection. In the connection shown, two things happened-

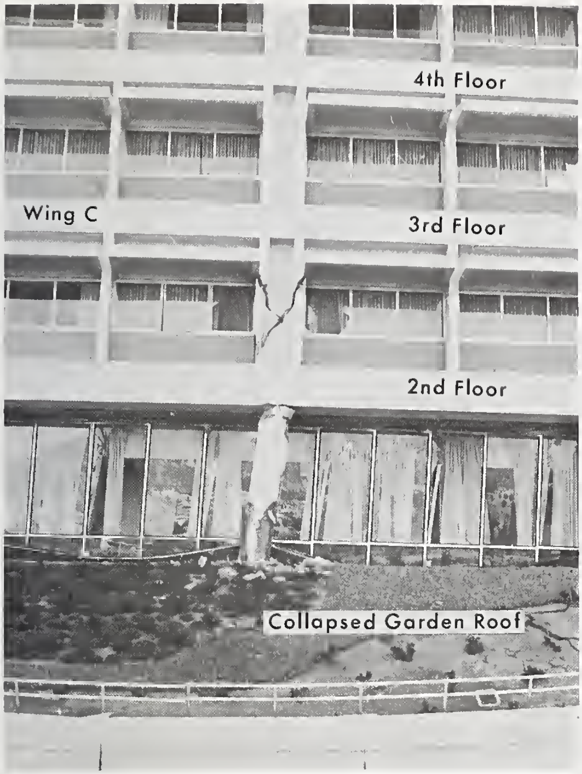



*Figure 5.22 A portion of the east face of Wing C. The collapse of the garden roof as well as the extensive damage of the first floor columns can be seen. Note the lateral displacement of the building to the north, in the first story and diagonal tension cracks in the second-story column.*

*Figure 5.23 South face of Wing D. Note that the bulk of major damage suffered by the building is limited to the ground, first, and second floors.*

*Figure 5.24 Close-up view (looking south) of one of the failed ground-story columns which originally supported the first-floor garden roof in front of Wing D. This is the same column seen in figure 5.23, the third from the right.*

*Figure 5.25 A column protruding through the garden roof at the far east end in front of Wing D. Note that in this failure the reinforcement has been completely peeled away from the upper end of the column.*

*Figure 5.26* *South row of first story columns, in Wing D; note the lack of ties on the corner bars which have broken out of the columns.*

*Figure 5.27* *Enclosed court at the first floor level. The sagging court was caused by failure of the ground-story columns.*

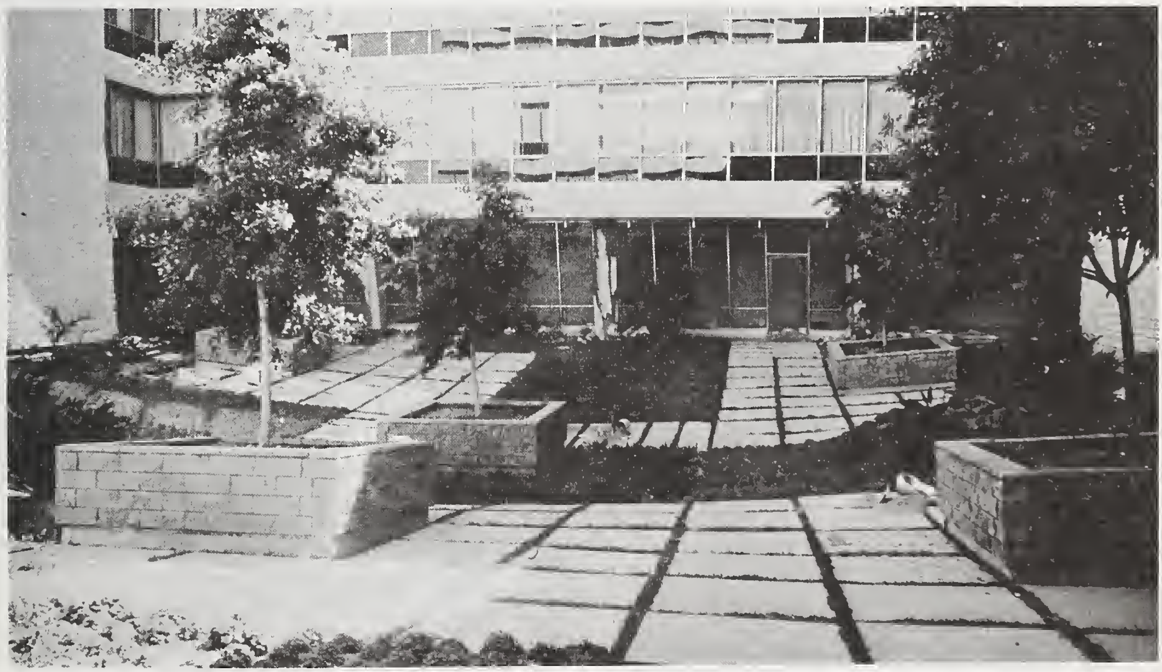


*Figure 5.28 Damaged main lobby in Wing A. Note the window mullion distortion. The planted area seen through the window is a court enclosed by the four wings.*

*Figure 5.29 Damaged elevators in the Wing A lobby.*

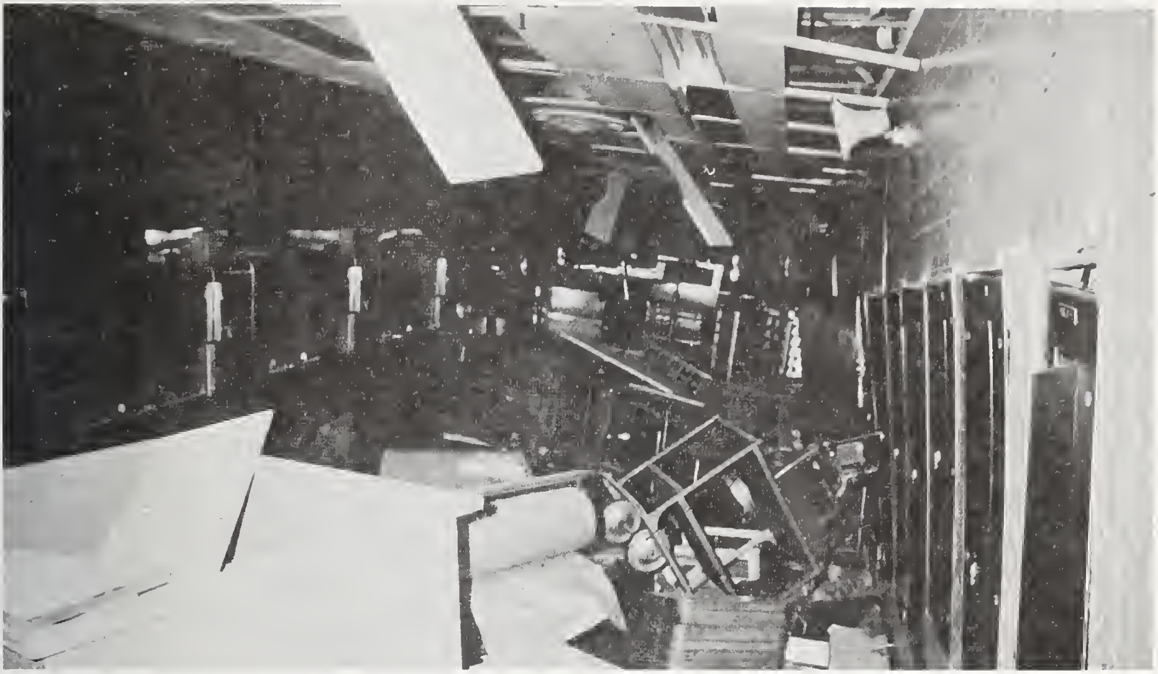


*Figure 5.30 Violent shaking of the building left the ground floor hospital kitchen in shambles.*

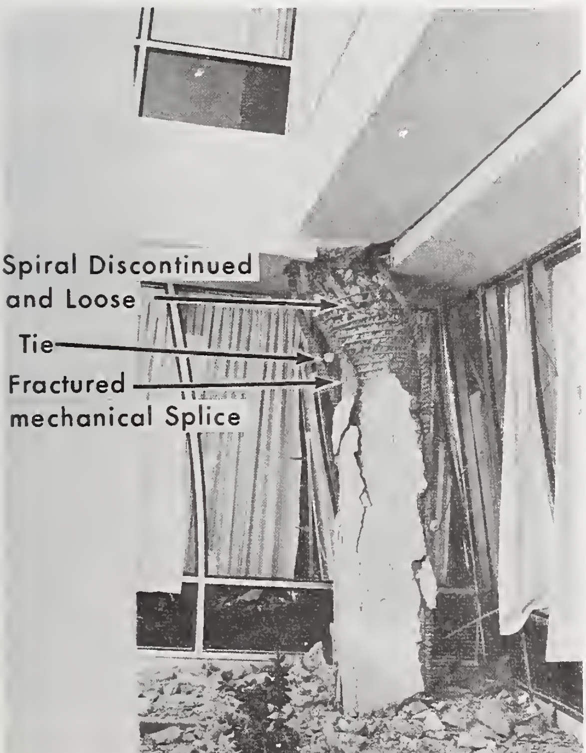



*Figure 5.31 Damaged Spiral Column.*

(1) the spiral stopped short of the connection, making it ineffective where it is needed most in lateral motion, (it should have been continued into the beam) and (2) the spiral unwound due to insufficient anchorage. Anchorage was intended to be achieved by lapping the spiral. This was not effective since the lap could not work once the covering concrete had broken away. Positive anchorage by hooks into the interior of the core is essential.

Stair Towers - A photograph of a typical stair tower before the earthquake is shown in figure 5.32. The stair towers were approximately 21 ft by 36 ft in plan with a connecting walkway into the main buildings. The towers were actually very heavy shear walls through five floors (designed with a factor K=1.0). Stair towers A, B, and D were supported on ground-story columns with loads being transferred from the tower, through the first-floor slab and into the columns. Stair tower C was supported by three columns cast with the embankment retaining wall and three footings away from the retaining wall. The towers were structurally separated from the main building by a four-inch space.

Stair towers A, B, and D overturned leaving the end of the wings exposed as shown in figure 5.33. Stair tower C, supported on footings, did not topple as is shown in figure 5.34. The overturning coefficient J was calculated as 0.557 and 0.656. Base shears were computed as 0.053 and 0.058 of the structure weight. Applying the J factor to the base shears means that the structure was designed for approximately 0.03 gravity when calculating overturning moment. All evidence indicates that accelerations at Olive View greatly exceeded this amount.

Examination of the base of stair tower B indicates bars which seemed to pull out of the first-floor slab (fig. 5.35). As previously indicated the stair tower walls were dowelled into the slab, which in turn was dowelled into supporting beams. These dowels were 36 in long (18 in into the walls and 18 in into the beams). The dowels matched the wall reinforcement which was usually No. 4 bars. This means that the overturning moment was resisted by No. 4 bars embedded 18 in. This is not adequate anchorage for such an important tensile tie under these load conditions.

Canopy from Main Building to Psychiatric Unit - The canopy connecting the main building to the psychiatric unit is shown in figure 5.36; this structure was severely damaged. The canopy roof was of lightweight concrete, supported on a pair of stone aggregate concrete columns spaced 9.5 ft across the width. This pair of columns was spaced at 15 ft along the canopy length except near the assembly building. In the latter location the spacing varied. The column

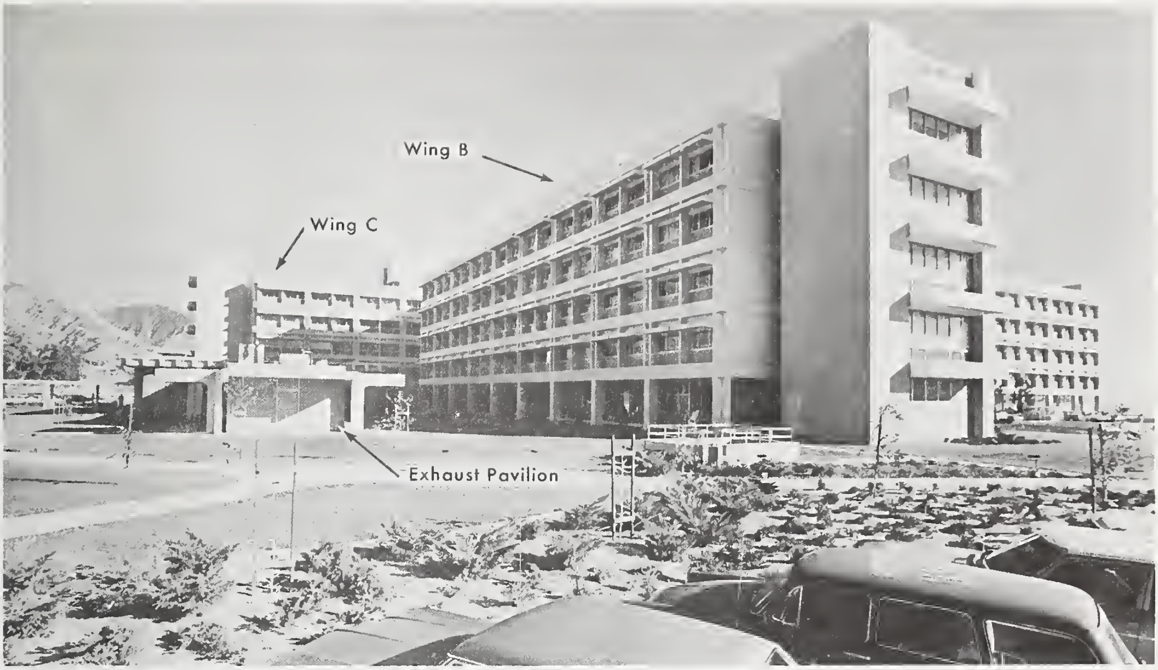
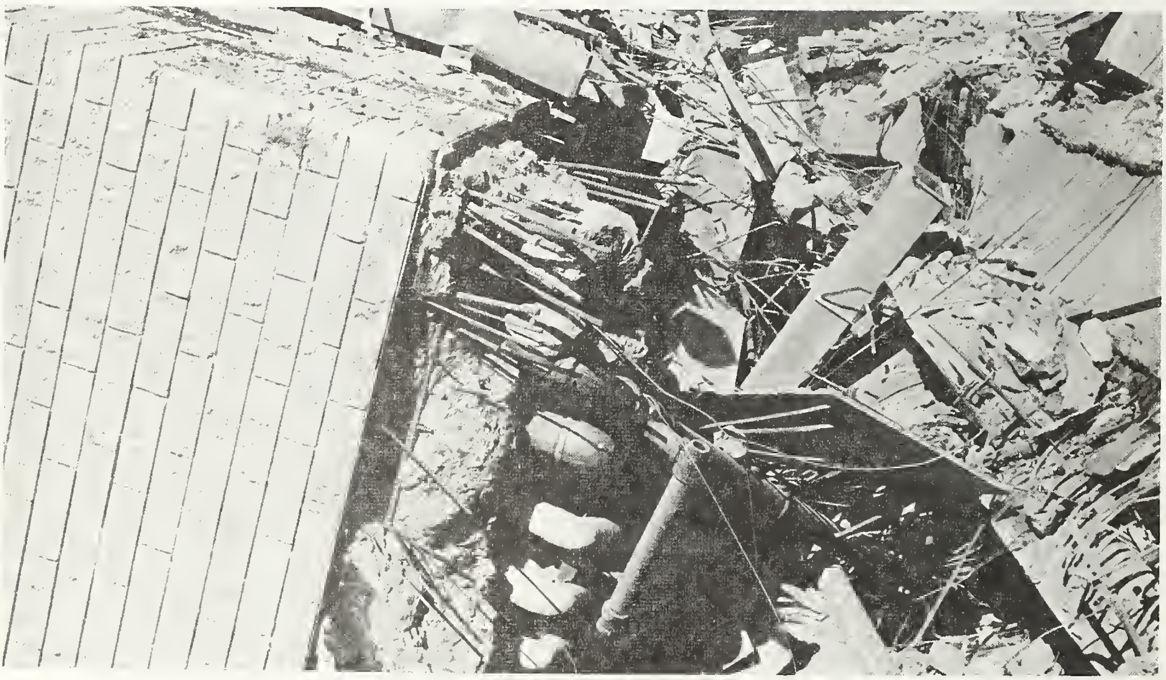



*Figure 5.32 Typical stair tower before the earthquake.*

*Figure 5.33 Wing D after stair tower overturned.*

*Figure 5.34 Tipped stair tower at the end of Wing C. This is the only stair tower that did not topple. This stair tower is supported directly on the foundation; whereas, the other three towers were supported on the ground-story columns.*

*Figure 5.35 Base of stair tower B.*



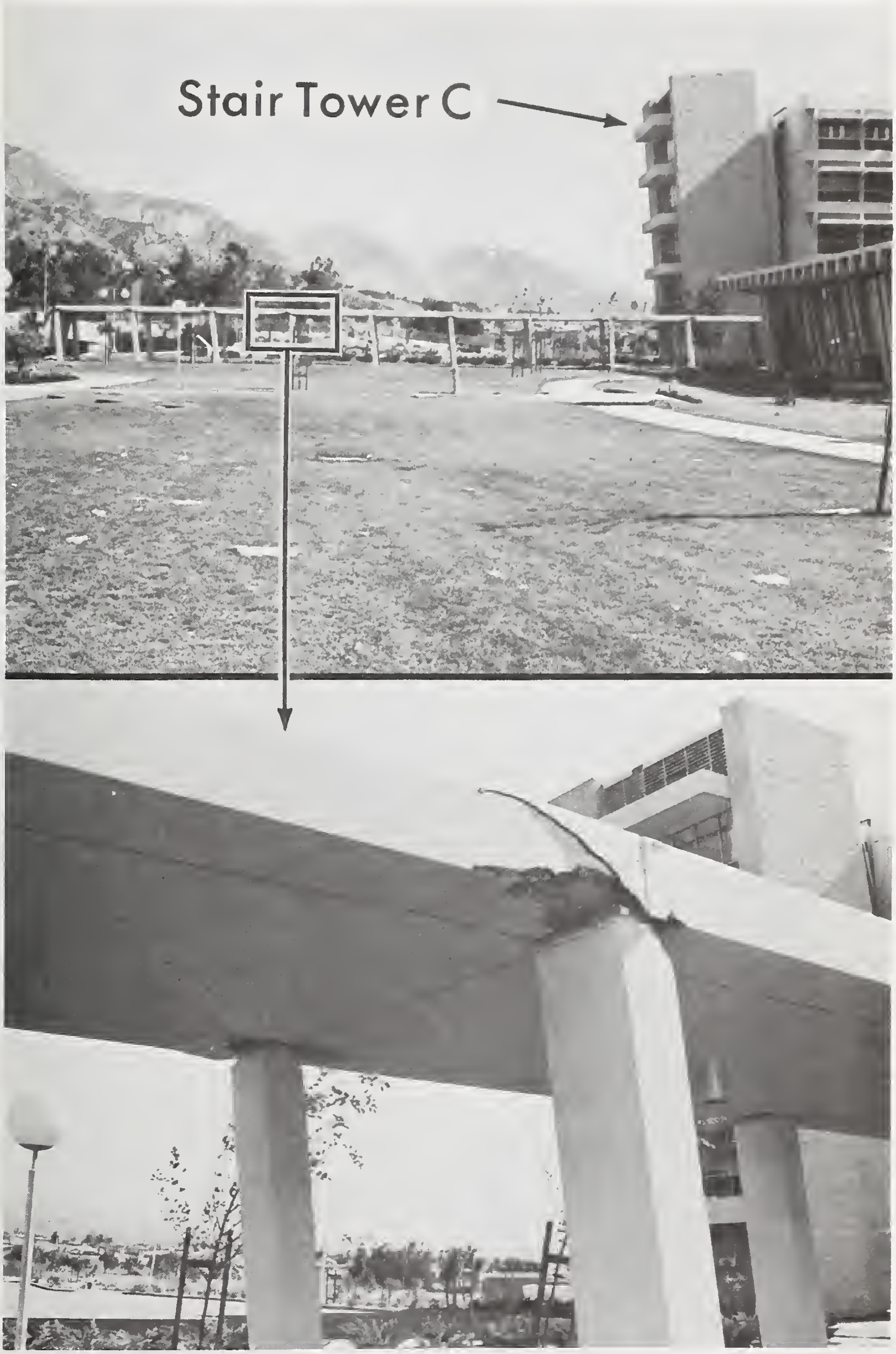



*Figure 5.36 Walkway Canopy.*

reinforcing bars extended only a few inches into the canopy slab and could not offer restraint due to bending at the top of the columns.

Assembly Building - The assembly building is shown in figures 5.9 and 5.17. As shown in the latter figure, the structure is leaning about two feet to the north.

This structure was constructed in reinforced concrete with a lightweight concrete roof and stone aggregate columns. Two of its three stories are below grade.

Exhaust Pavilion - The exhaust pavilion is shown in figures 5.9 and 5.37. This building is surrounded by an overhanging canopy (originally horizontal) which was severely damaged, indicating large vertical accelerations.

Power Plant - The power plant is a one-story rectangular building. The structure was constructed of steel frames diagonally braced at the north end with steel straps. This bracing is shown in figure 5.38. The straps on the right stretched about three feet. One of the two straps on the left fractured at a weld, as shown in the inset.

The power plant contents were severely damaged, with boilers shifting as much as four feet.

Ambulance Port - The ambulance port located south of wing D is rectangular in plan, measuring 27 ft by 117 ft. The lightweight concrete joist roof is supported on twelve reinforced concrete columns. The columns were 14 in by 18 in reinforced with No. 9 bars and ties spaced at 12 in on center.

The columns failed as shown in figure 5.39. The north columns were restrained at the base by the pavement and tried to resist most of the shear. Note the nonshattered portion of the columns remained vertical. The south columns (originally vertical) rotated about the footings.

Old Buildings - There were a number of pre-1933 buildings in the old section of the Olive View complex. Many of these buildings were either totally collapsed or severely damaged. Figure 5.40a shows a completely collapsed old brick-wall building and figure 5.40b shows a severely damaged wood-frame house.

## San Fernando V.A. Hospital - Map 3, Location 7

The San Fernando Veteran's Administration Hospital had the misfortune of being located near the earthquake epicenter.

*Figure 5.37 The damage to the overhanging canopy of the exhaust pavilion possibly indicates large vertical accelerations.*

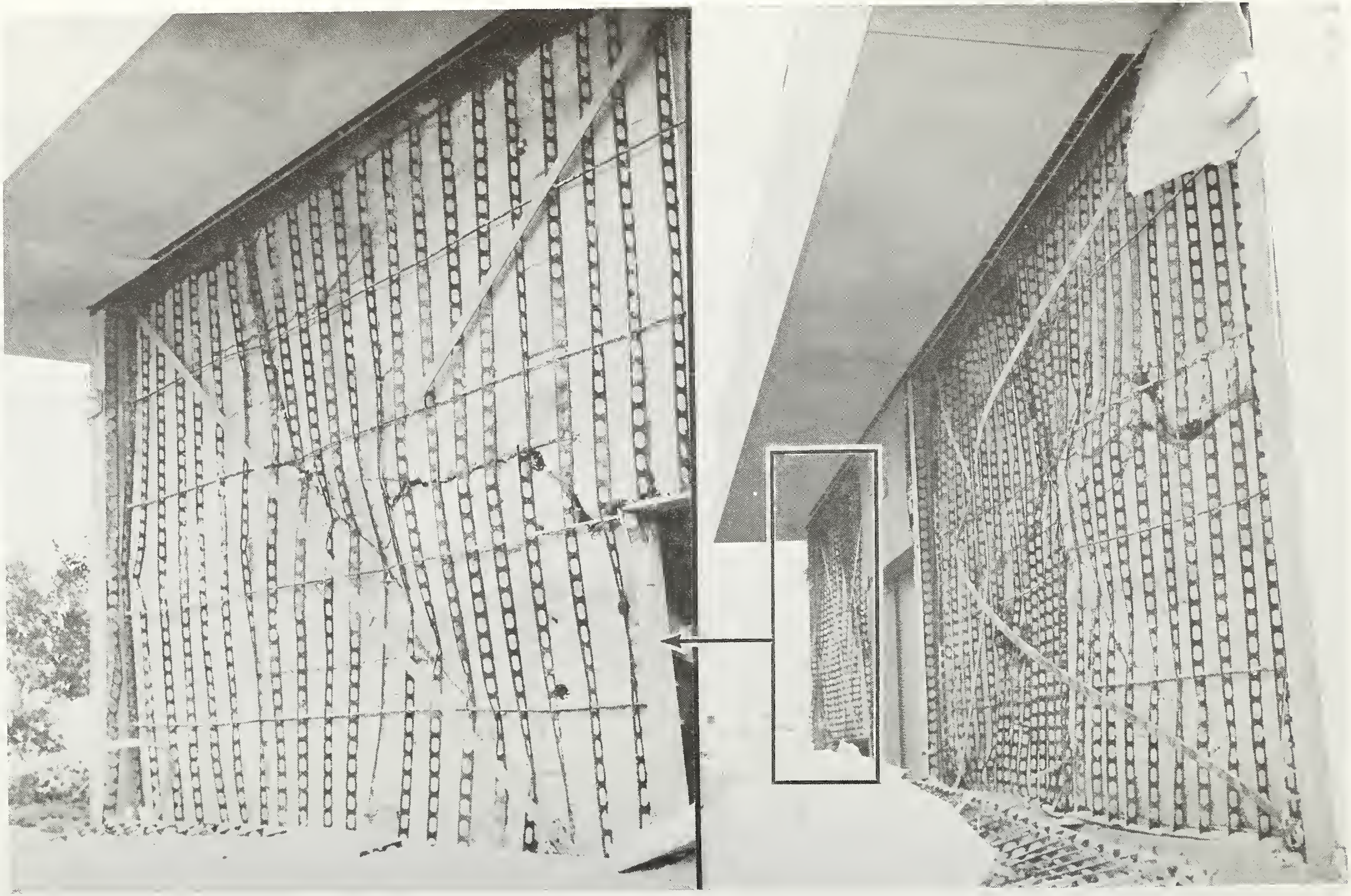


*Figure 5.38 Damaged diagonal bracing in the north wall of the power plant.*

*Figure 5.39 Damaged reinforced concrete ambulance port south of Wing D.*

*Figure 5.40a* *Totally collapsed old brick building at the old section of the Olive View Medical Center. The building was not designed to resist earthquake forces.*

*Figure 5.40b* *Severely damaged pre-1933 wood-frame structure at the old section of the Olive View Medical Center.*

The Pacoima Dam which is located about one mile east of the hospital experienced the largest ground accelerations ever recorded in an earthquake (refer to section 3.3 for its magnitudes). The hospital is located adjacent to the community of Sylmar in the northeastern section of the San Fernando Valley, approximately four miles from the center of the city of San Fernando. The hospital was built by the Veteran's Bureau, the predecessor agency of the Veteran's Administration, in 1925 and opened as a Tuberculosis Hospital on March 1, 1926. It was redesignated as a General Medical Hospital on July 1, 1963. As of February 9, 1971, the hospital consisted of 45 separate buildings and 6 support facilities which constituted a 420-bed general medical hospital and a 36-bed Nursing Home Care Unit.

Forty-six people, mostly patients, died as a result of the collapse of two major buildings shown in figure 5.41 (much of the rubble had already been cleared at the time of the photo). Efforts to obtain rescue aid were hampered by a general loss of communications, including telephone service (see chapter 8) and the HEAR System. HEAR (Hospital Emergency Assistance Radio) is a special emergency communication system among the hospitals in the Los Angeles area which can operate from emergency power supplies. Unfortunately the building which housed the power supply at the V.A. hospital collapsed, making the system useless. The room containing the transmission equipment was also inaccessible due to debris. Either of these factors alone would have prevented use of the HEAR system. It was reported that a man was sent by automobile to obtain help. Once the message was given to the police, the request for aid was confused with the collapse at the nearby Olive View Medical Center; thus, there was no response to the request. It was over an hour before the collapsed V.A. buildings were observed from a passing helicopter and assistance was sent. Communications were established soon afterwards by sending a mobile transmitting unit to the hospital.

Once rescue operations (fig. 5.42) were underway, they were handicapped by difficulty in clearing the rubble, since all material had to be moved almost by hand in the search for survivors. Rescue work continued around-the-clock until the last victim was recovered some four days after the quake. The intense efforts of rescue workers were partly rewarded when a survivor was found some 58 hours after the earthquake. The man survived by sliding under a sink. Structural members which fell across kitchen equipment apparently supported the upper floor slabs.

Table 5.1 is a schedule of buildings and ancillary structures which comprised the existing hospital complex. Also included in the table is the year in which each unit was constructed, the general type of construction, and the

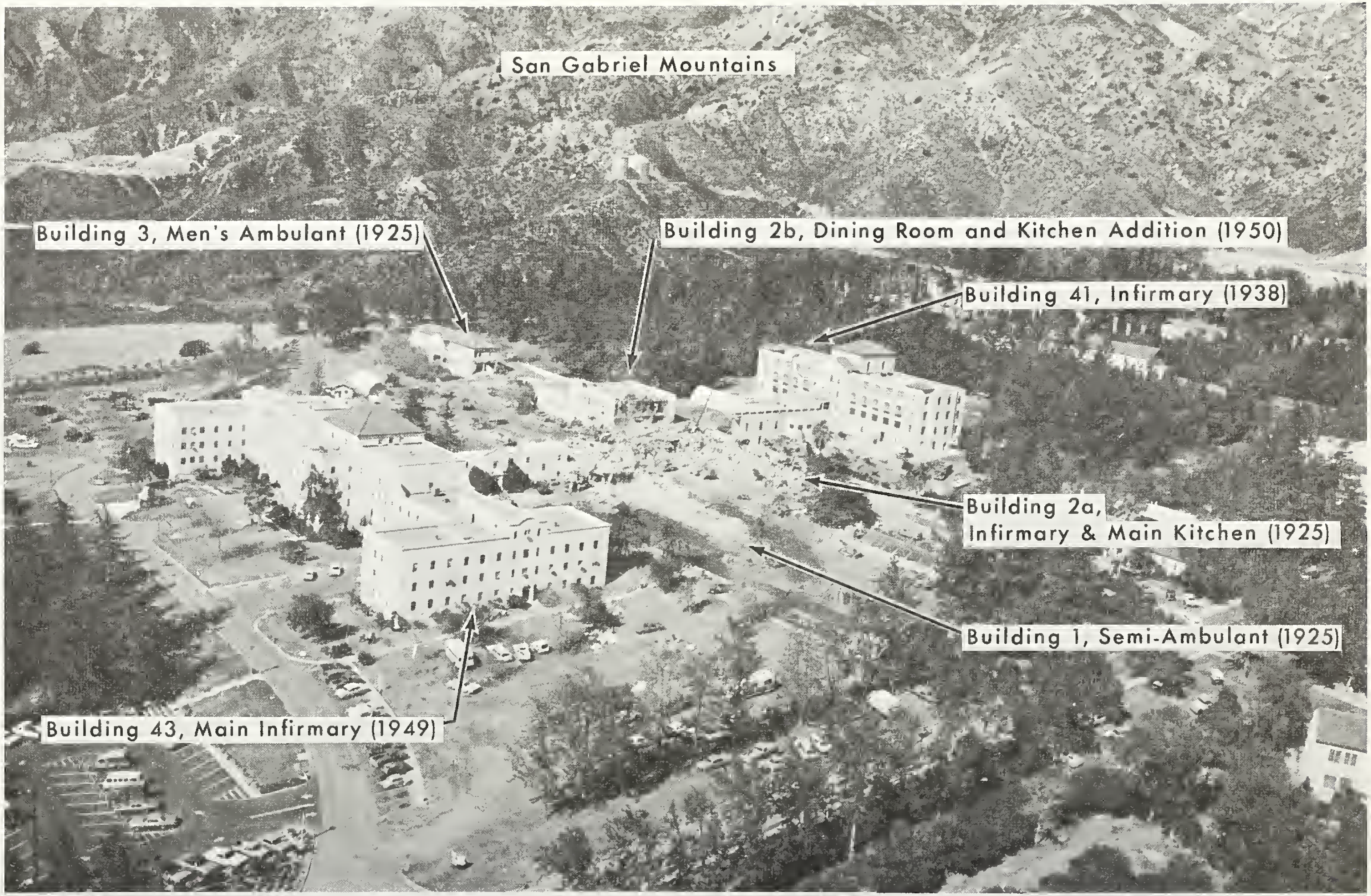



*Figure 5.41 Aerial view of the San Fernando Veterans Administration Hospital complex at Sylmar, looking northwest. Note the general setting of this hospital at the foothills of the San Gabriel mountains.*

*Figure 5.42* Rescue and clean-up operations. This collapsed building was constructed in 1925 and is a reinforced concrete frame structure infilled with unreinforced clay tile walls. (Los Angeles Times Photo).

degree of damage sustained by the building or structure during the February 9 earthquake. Figure 5.43 is a general plan of the 100-acre hospital site showing the location of the various buildings and structures. These data and the following information are based on reports prepared by the Office of Construction, Veterans Administration dated February 16, 1971, and Brandow and Johnston Associates, Structural Engineers, dated March 12 and 26, 1971.

As indicated in table 5.1, many of the original hospital buildings constructed in 1925 and 1927 had reinforced concrete structural frames with unreinforced clay tile exterior walls. Quarters buildings and other small shop buildings were constructed with wood framing. All buildings were four stories or less in height and many of them were not designed to resist seismic forces because no recognized standard for this purpose was in use at that time.

Over the years, several minor buildings were added. The first major addition to house patients was the three-story Building No. 41, constructed in 1938. The next additions to the hospital were the three-story Building No. 43 and a new boiler plant built in 1949. Since that time other buildings were erected, including the Chapel (Building No. 52) and a kitchen and dining hall addition (Building No. 2b) in 1950 and a laundry (Building No. 50) in 1951. All buildings constructed at the hospital since 1935 were designed for earthquake forces in compliance with historically applicable Los Angeles building codes.

Aerial views of the San Fernando Veterans Administration Hospital complex following the February 9 earthquake are shown in figures 5.44 and 5.45. Many buildings and their construction dates are shown in these two photos.

In general, the 26 buildings and additions built prior to 1933 sustained the greatest structural damage. Four of these buildings (No. 1 and 2a in fig. 5.44 and No. 8 and 10 in fig. 5.45) totally collapsed during the initial moments of the main shock. The floor plans showing the typical beam and column framing of Buildings 1 and 2a are shown in figures 5.46 and 5.47 respectively. These two pre-earthquake design buildings (1925) had little or no earthquake resistance and would not have been acceptable by modern standards.

Building No. 6, a two-story building approximately 50 ft by 100 ft in plan, was one of the original 1925 buildings which was severely damaged (figs. 5.48, 5.49 and 5.50). A small one-story wood frame building (Building No. T-7) constructed in 1947 adjacent to Building No. 6 suffered no apparent damage.


*Figure 5.43* *Plan of San Fernando Veterans Administration complex. Buildings or structures indicated as solid sustained relatively minor structural damage. Other outlined structures either experienced severe damage or collapsed (see table 5.1).*

*Table 5.1: Building Schedule, Veterans Administration Hospital, San Fernando, California*

| Building Number | Building Name | Year Constructed [a] | Type of Construction [b] | Degree of Damage |
|---|---|---|---|---|
| 1 | Semi-Ambulant | 1925 | I | Destroyed |
| 2a | Infirmary & Main Kitchen | 1925 | I | Destroyed |
| 2b | Dining Room & Kitchen Addn. | 1950 | II | Serviceable |
| 3 | Mens' Ambulant | 1925 | I | Severe Damage |
| 4 | Recreation | 1925 | I | Severe Damage |
| 5 | Infirmary | 1925 | I | Severe Damage |
| 6 | Infirmary | 1925 | I | Severe Damage |
| 7 | Recreation Supply & Fiscal | 1925 | I | Severe Damage |
| 8 | Garage & Firehouse | 1925 | I | Destroyed |
| 9 | Supply Warehouse | 1925 | I | Severe Damage |
| 10 | Engineering Office and Shop | 1925 | I | Destroyed |
| 11 | Nursing Home | 1925 | I | Severe Damage |
| 12 | Nurses Quarters | 1925 | I | Severe Damage |
| 13 | Quarters | 1925 | III | Severe Damage |
| 14 | Quarters | 1925 | III | Severe Damage |
| 15 | Quarters | 1925 | III | Severe Damage |
| 16 | Quarters | 1925 | III | Severe Damage |
| 17 | Quarters | 1925 | III | Severe Damage |
| 18 | Quarters | 1925 | III | Severe Damage |
| 19 | Gate house | 1925 | III | Severe Damage |
| 20 | Water Tank | 1925 | -- | Serviceable |
| 23 | Ground Maintenance | 1931 | IV | Severe Damage |
| 24 | Carpenter Shop | 1927 | IV | Severe Damage |
| 25 | Plumbing Shop | 1927 | IV | Severe Damage |
| 26 | Paint Shop | 1947 | IV | Severe Damage |
| 28 | Guest House | 1928 | III | Severe Damage |
| 29 | Paint & Oil Storage | 1930 | II | Serviceable |
| 31 | Garage | 1939 | III | Severe Damage |
| 32 | Garage | 1939 | III | Destroyed |
| 33 | Garage | 1939 | III | Destroyed |
| 34 | Green House | 1932 | V | Serviceable |
| 35 | Equipment Repair Shop | 1927 | IV | Severe Damage |
| 40 | Automatic Valve Chamber | 1925 | -- | Serviceable |
| 41 | Infirmary | 1938 | II | Serviceable |
| 42 | Pump House | 1940 | III | Severe Damage |

Table No. 5.1 (continued)

| Building Number | Building Name | Year Constructed (a) | Type of Construction (b) | Degree of Damage |
|---|---|---|---|---|
| 43 | Main Infirmary | 1949 | II | Serviceable |
| 44 | Incinerator | 1949 | II | Severe Damage |
| 45 | Boiler House | 1949 | II | Serviceable |
| 47 | Steel Water Tower | 1949 | -- | Serviceable |
| 49 | Concrete Chimney | 1949 | -- | Serviceable |
| 50 | Laundry | 1951 | II | Serviceable |
| 52 | Chapel | 1950 | II | Serviceable |
| 53 | Pump House | 1950 | II | Serviceable |
| 54 | Main Switch House | 1950 | II | Serviceable |
| 55 | Wet Salt Storage Basin | 1950 | -- | Serviceable |
| 56 | Transformer Pad | ? | -- | Serviceable |
| 57 | Transformer Vault | ? | II | Serviceable |
| 58 | Golf Club House | 1957 | IV | Serviceable |
| Temporary Buildings | | | | |
| T-7 | Patients' Recreation Room | 1947 | IV | Serviceable |
| T-8 | Canteen | 1947 | V | Serviceable |
| T-10 | Research Storage | 1947 | V | Serviceable |
| T-12 | Recreation | 1948 | V | Serviceable |
| T-13 | Animal House | 1947 | V | Serviceable |

(a) All buildings erected since 1935 were seismic resistive construction.
(b) I-Reinforced concrete frame and diaphragms with unreinforced clay tile shear walls.
II-Reinforced concrete frame, diaphragms, bearing and shear walls.
III-Wood frame and diaphragms with unreinforced clay tile bearing and shear walls.
IV-Wood frame and diaphragms with wood stud and plaster shear walls.
V-Metal frame and diaphragm and metal bracing.

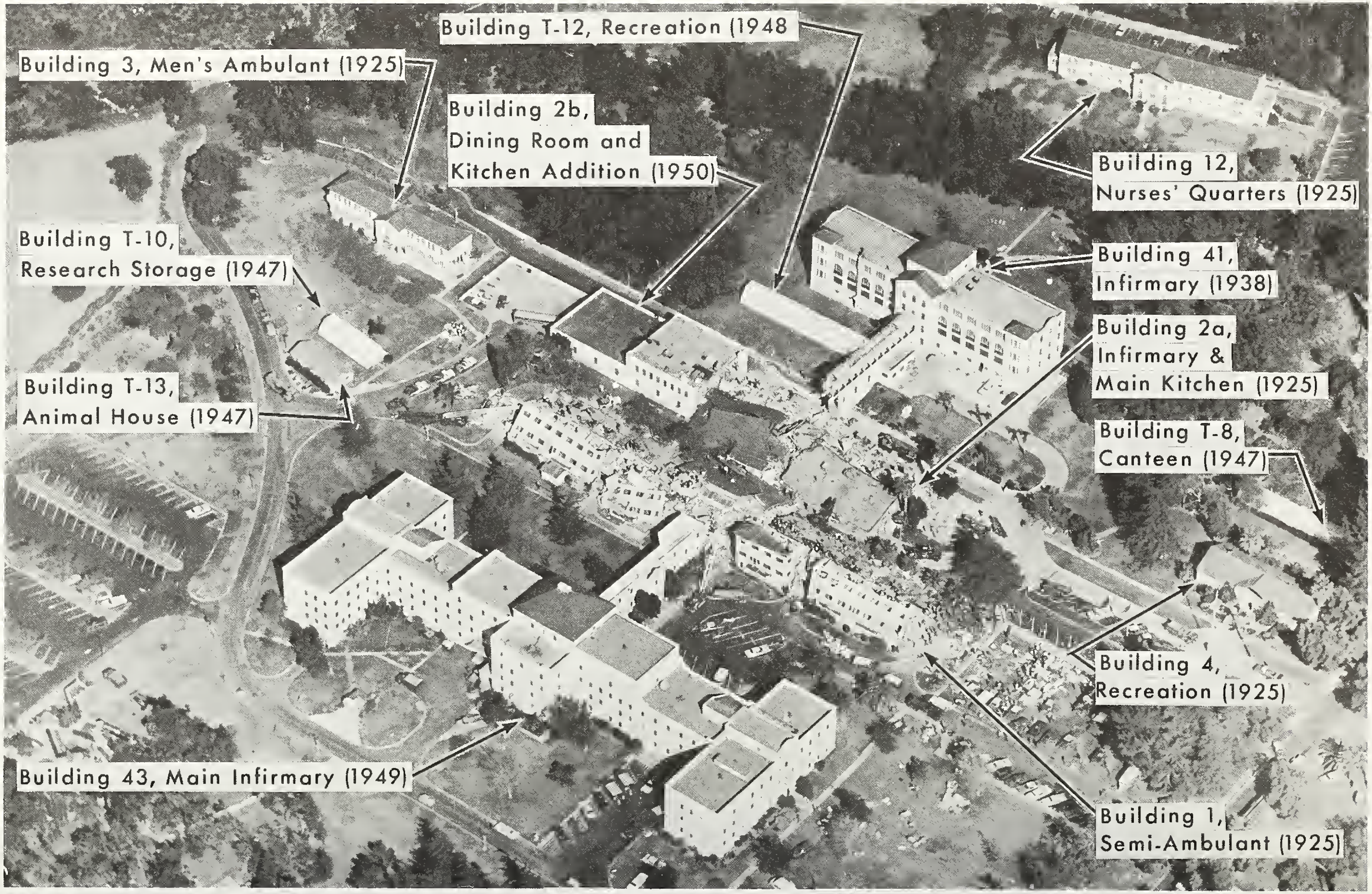



*Figure 5.44 Collapsed buildings (No. 1 and 2a) constructed in 1925 where 46 patients and hospital workers were killed. Many of the buildings, also constructed in 1925 (Buildings 3, 4, and 12), although still standing, were severely damaged. (Los Angeles Times Photo).*

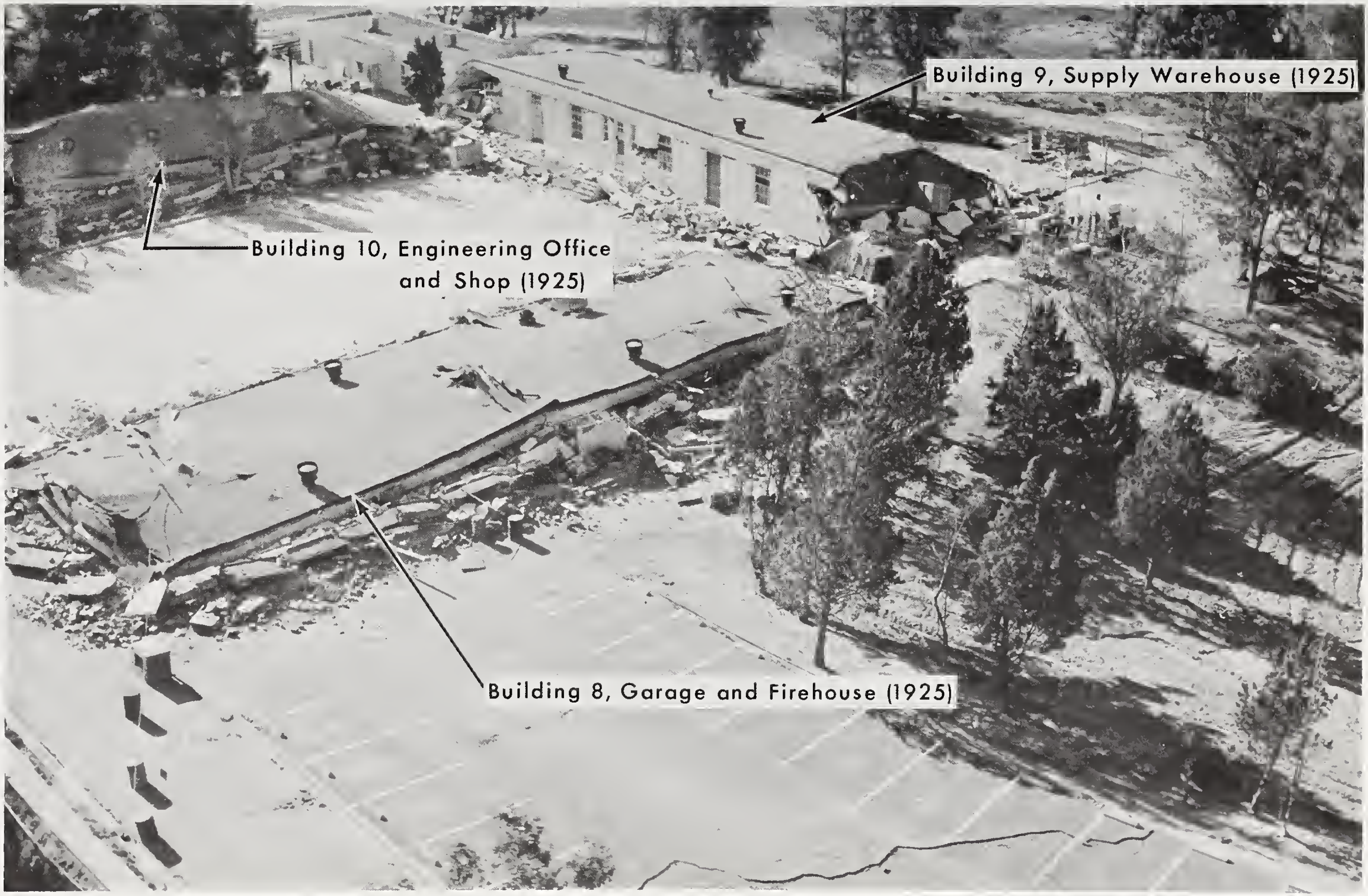
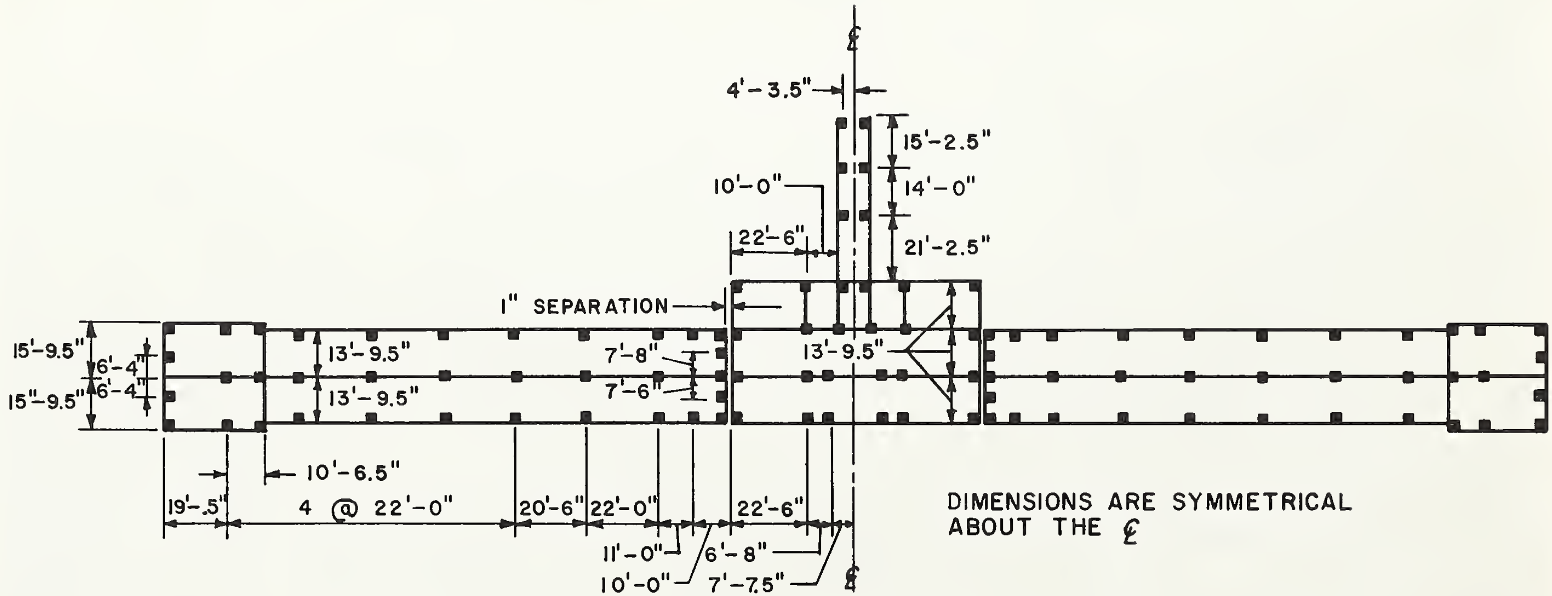



*Figure 5.45* *Collapsed one-story buildings (No. 8 and 10) located at the east end of the hospital complex. Note that two structures oriented in one direction collapsed; whereas, the building (No. 9) oriented 90 degrees to them is damaged but still standing.*




PLAN - BUILDING 1

*Figure 5.46 Plan of Building 1, Semi-Ambulant Building, a 3-story building constructed in 1925.*

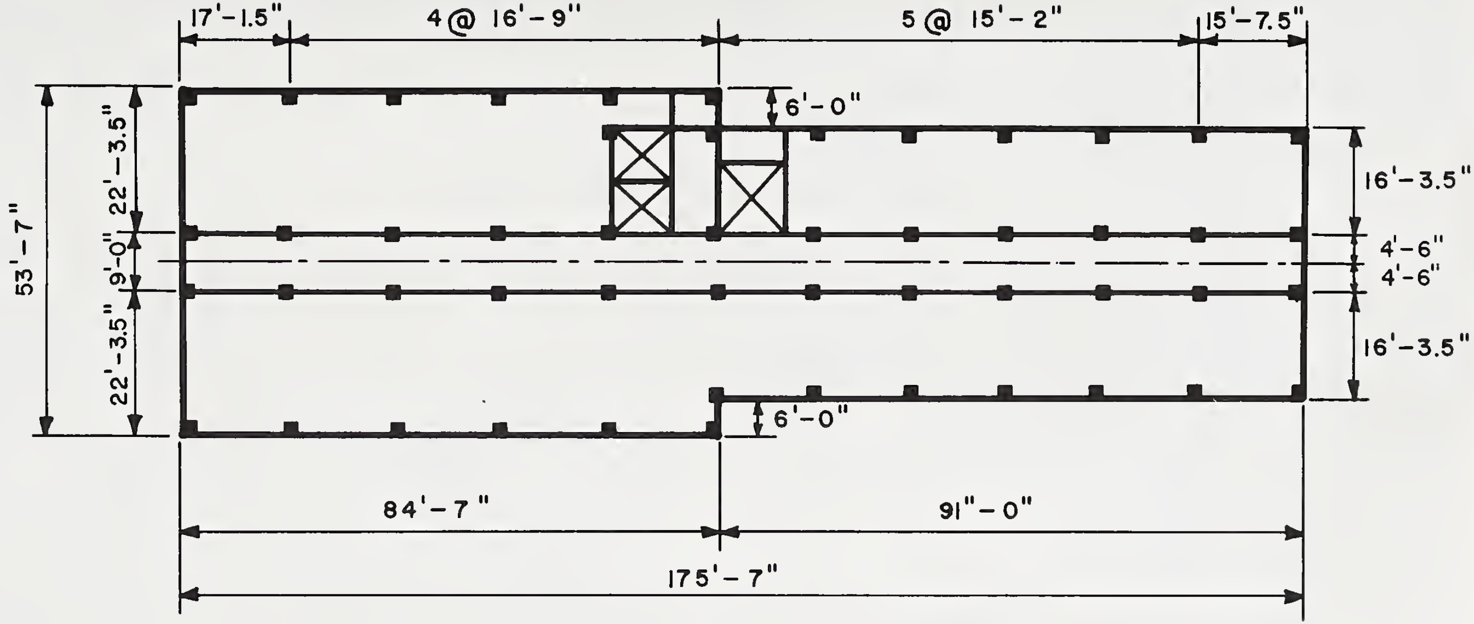



*Figure 5.47 Plan of Building 2a, Infirmary and Main Kitchen, a 3-story building constructed in 1925.*

*Figure 5.48 Building 6, located east of the main buildings. This building which was constructed in 1925 was severely damaged.*


Figure 5.49 Detailed view of one of damaged reinforced concrete columns. Exterior walls were unreinforced clay tile and stucco.

Figure 5.50 Close-up view of one of many damaged columns in Building 6. Note a lack of ties around the exposed square reinforcing bars.

Buildings constructed since 1935, including the addition to Building No. 2 (Building No. 2b, fig. 5.44), sustained relatively minor structural damage with the exception of Buildings No. 31, 32, 33 (three- and four-car garages) and Building No. 42 (Pump House). In addition, the boilers in the Boiler House were severely damaged and there was major damage to all utilities. The incinerator stack which was constructed in 1949 collapsed.

Building 41 (fig. 5.44), which was located north of collapsed Building 2a was constructed in 1938 and is a three-story (plus basement) reinforced concrete frame and bearing wall building. The structure had some diagonal hairline cracks in some of the walls but the frame was essentially intact. Figure 5.51 is a plan of Building 41 showing the general arrangement of the reinforced concrete bearing walls, shear walls, and columns. The longitudinal and exterior walls are 12-in thick and the interior transverse walls are 10-in thick. The foundation consists of interconnected spread footings.

Building 43, (fig. 5.44), which was located south of collapsed Building 1, was constructed in 1949 and is also a reinforced concrete frame and bearing wall building. It varies in height from 3 to 4 stories (plus basement) and has a spread-footing foundation. Similar to Building 41, damage consisted primarily of hairline cracks in some of the walls. The general arrangement of the structural walls and columns is shown in figure 5.52. The exterior walls are 12-in thick and transverse walls are 10-in thick in Unit A and 8-in thick in Units B and C. As indicated in figure 5.52, the walls in the end wings (Units B and C) are separated 6 inches from the adjacent parallel walls in the main center wing of the building (Unit A).

Based on preliminary seismological data reported by the California Institute of Technology, the building failures at the San Fernando V.A. Hospital were precipitated by intense areal ground motion rather than localized earth faulting or ground movements.

Buildings 2b, 41, and 43 provide excellent examples of the performance of post-earthquake design structures when they are compared with the pre-earthquake design of Buildings 1 and 2a, all of which were adjacent to each other. This series of buildings clearly demonstrates the need for determining the strength of old buildings and strengthing or demolishing them where required. The V.A. has already started such a program as a result of this earthquake.

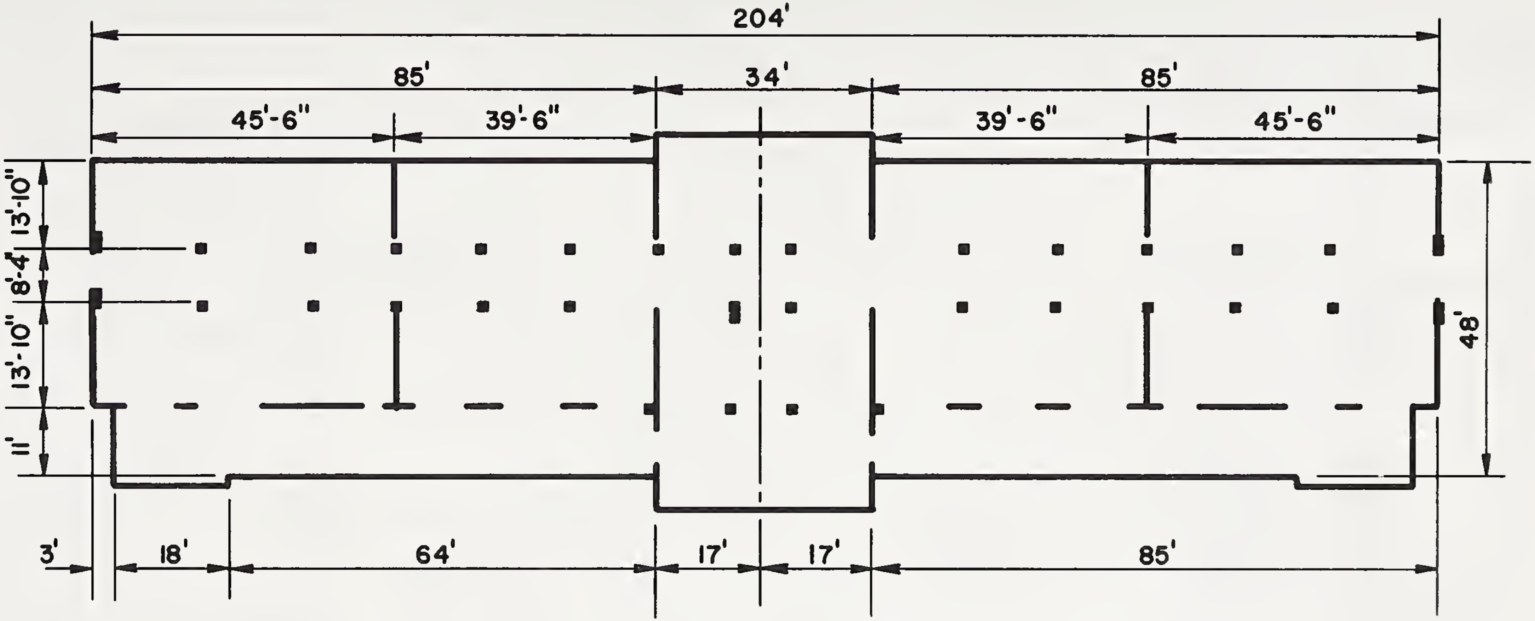



Figure 5.51 *Plan of Building 41, Infirmary Building, a three-story (plus basement) reinforced concrete frame and bearing wall building constructed in 1938.*

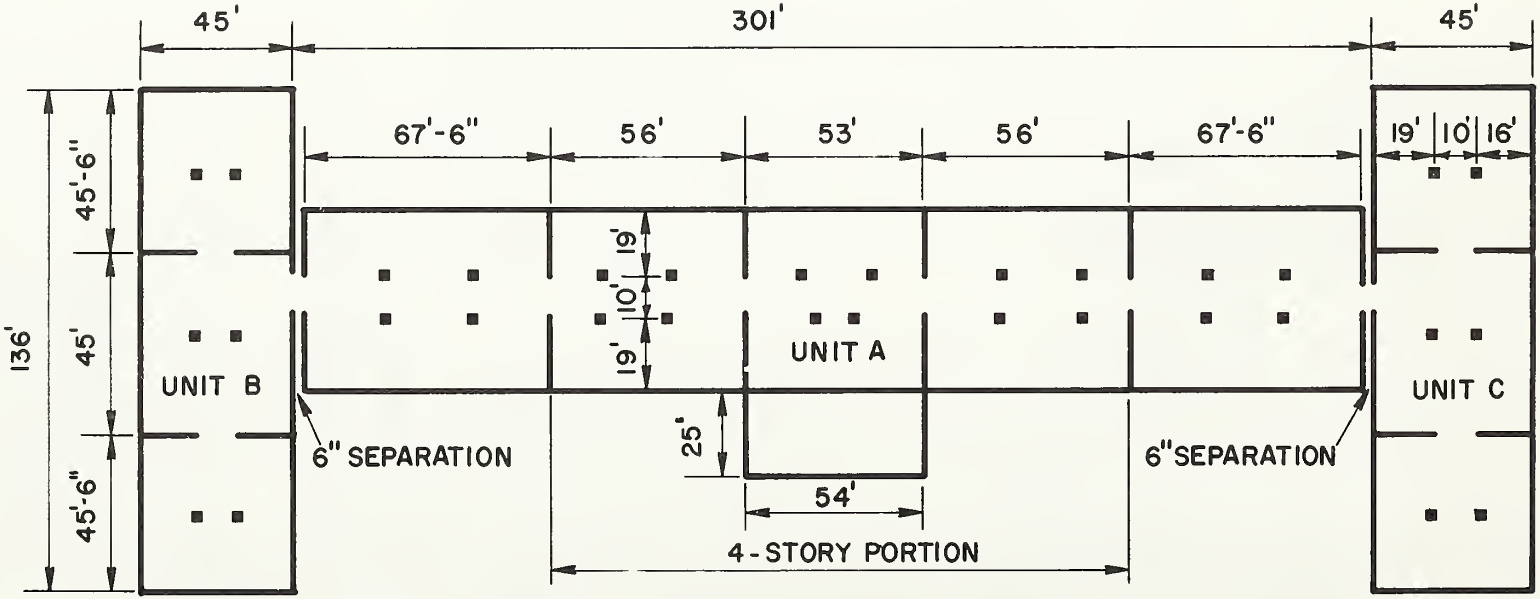



*Figure 5.52 Plan of Building 43, Main Infirmary Building, a 3 to 4-story building constructed in 1949.*

Holy Cross Hospital, Map 3, Location 17

Holy Cross Hospital which opened in June, 1961, is a seven-story building located at Rinaldi Street and Indian Hills Road. As shown in figure 5.53 the hospital complex included a boiler plant (service building) and a one-story Continuing Care Facility. Also shown in the figure is the Indian Hills Medical Center which is discussed under Commercial Buildings. A general layout of the buildings is shown in figure 5.54. Note that the long direction of the hospital is not quite in the east-west direction, although it will be considered so-oriented in the discussions which follow.

After the earthquake, patients began arriving from the nearby areas. It was only then that the extent of damage to Holy Cross was becoming apparent and evacuation of the building was found necessary. Elevators were inoperative due to counter weights leaving their tracks and tangling cables. Steel guides were also bent. About 170 patients were evacuated by means of an exterior stairway at the east end of the seven-story tower. Rescue workers entered through an interior stairwell near the west end of the tower to avoid interference with the evacuees. During the evacuation emergency power supplies and oxygen cylinders were in use. A strict no smoking or open flame policy was enforced as a safeguard against fire since there was no water available. Evacuated patients and patients received from nearby areas were treated on the lawn and in the continuing care facility.

The main building is a reinforced concrete shear wall structure with normal weight aggregate concrete columns and lightweight aggregate concrete floors. The floor framing was comprised of pan-joists and beams. The foundation was drilled cast-in-place friction piles. The building was designed in the period 1957-1958 under the Los Angeles City Building Code.

A floor plan of the building showing column and shear wall locations throughout its height is shown in figure 5.55. The building consists of a seven-story tower (a partial basement under the tower and a service tunnel running from the basement and under the north wing are not shown) with appendages of varying height located around the tower.

An aerial photo of the main building is shown in figure 5.56. The exterior columns on the north and south sides were extensively cracked, particularly in the third- and fourth-stories, as shown in figure 5.57. The concrete in the southeast fourth-story exterior column was crushed as shown in figure 5.58. Close-up views of this same column as viewed from inside are shown in figures 5.59 and 5.60 (note the splice of the column steel and the widely spaced ties).


*Figure 5.53 Aerial view of Holy Cross Hospital and Indian Hills Medical Center.*

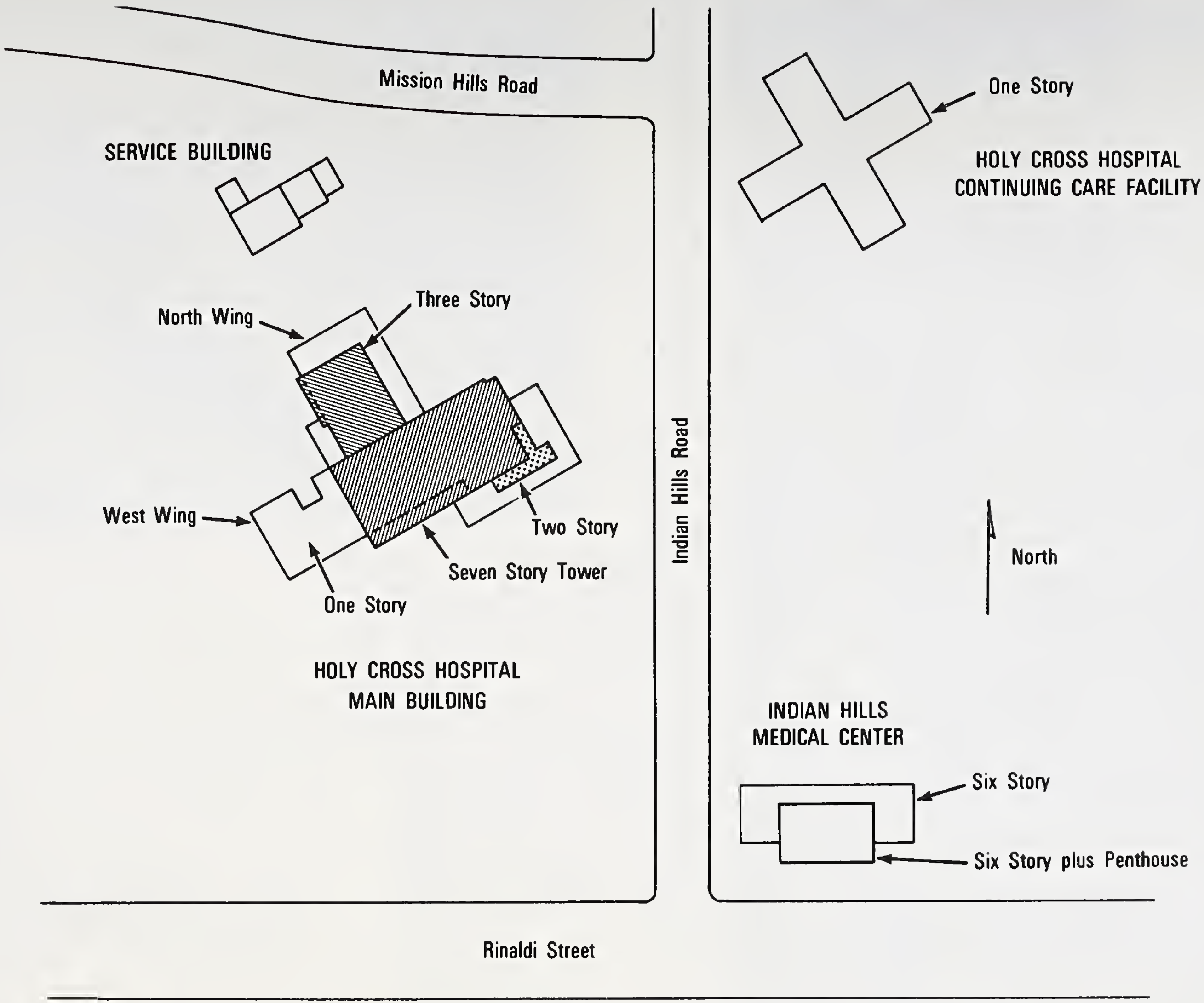



*Figure 5.54 Plan view of Holy Cross Hospital and Indian Hills Medical Center.*

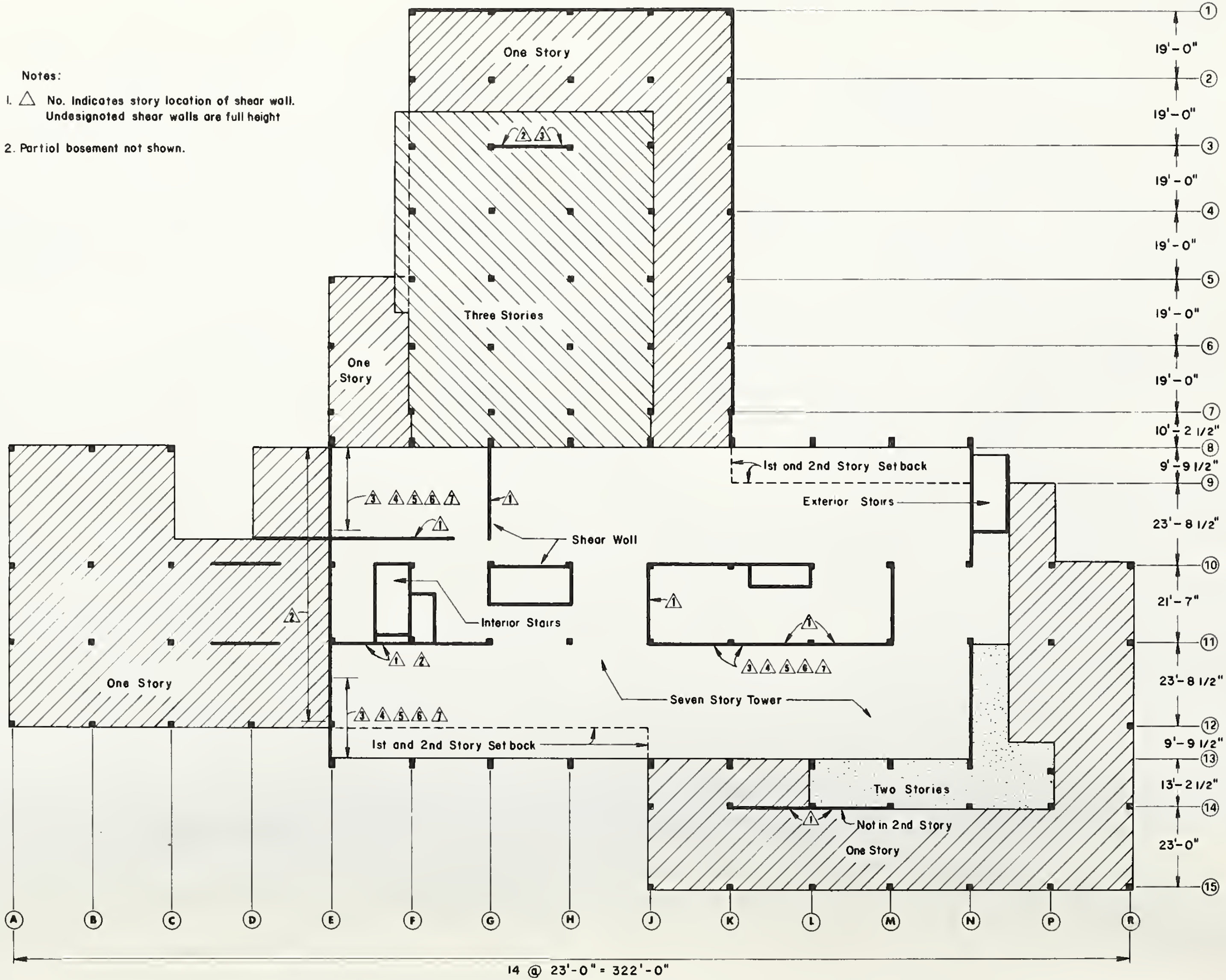



Figure 5.55 Floor plan of Holy Cross Hospital.


*Figure 5.56 Aerial view of Holy Cross Hospital. This 209-bed, reinforced concrete structure was badly damaged and has been evacuated.*

*Figure 5.57 Extensive column cracking is visible in the third and fourth stories of the south face.*

*Figure 5.58 View of the failed column at the southeast corner of the fourth story.*

*Figure 5.59 Damaged connection between the spandrel and shear wall on the southeast corner of the fourth floor.*

Further extent of damage at the fourth floor can be seen in figure 5.61. Note the massive failure at the fourth-floor construction joint.

A west elevation of the tower is shown in figure 5.62. Note the horizontal cracking at the third-floor construction joint. A close-up of this damage is shown in figure 5.63. Interior damage was extensive at the west end of the third story. Two third-story columns centered between the two end walls failed in shear. The southernmost of these columns is shown in figure 5.64. The third floor in this area was extensively damaged as shown in figure 5.65 (note the base of one column in the background and the door off to the right). A closeup of floor displacement is shown in figure 5.66. The room located through the door to the right in figure 5.65 (northwest corner) is shown in figure 5.67. Note the cracks in the shear wall, floor, and spandrel. The southwest corner room is shown in figure 5.68. Damage was similar to that in the northwest corner room.

The interior stairwell near the west end of the building, which was used by rescue workers to enter the building, is shown in figure 5.69. Note the diagonal cracking and the horizontal crack at the third-floor construction joint. Movement of the building caused concrete to spall and powder around some reinforcing bars as shown in the inset.

Limited wall cracking was found in the basement and more pronounced wall cracking was found in the first- and second-stories. Structural damage was not severe above the fourth story although some did occur as shown in the photograph of a doorway on the seventh story in figure 5.70. Shaking on the seventh floor also caused shelves to overturn, dumping their contents on the floor (figure 5.71).

The three-story north appendage, referred to as the north wing, is shown in figures 5.72 and 5.73. Note the portion of the wing supported on the second-story columns and attached to the third and fourth floors of the tower section. It was probably this configuration which caused the extensive damage to the third and fourth stories of the seven-story tower. The second-story columns shown in the east elevation were cracked as shown in figures 5.74 and 5.75. The north wing has now separated from the tower by cracking which occurred at the juncture of the wing and the tower.

Some damage, consisting mostly of movement of equipment, was found in the service building. The Continuing Care Facility shown in figure 5.76 was undamaged.

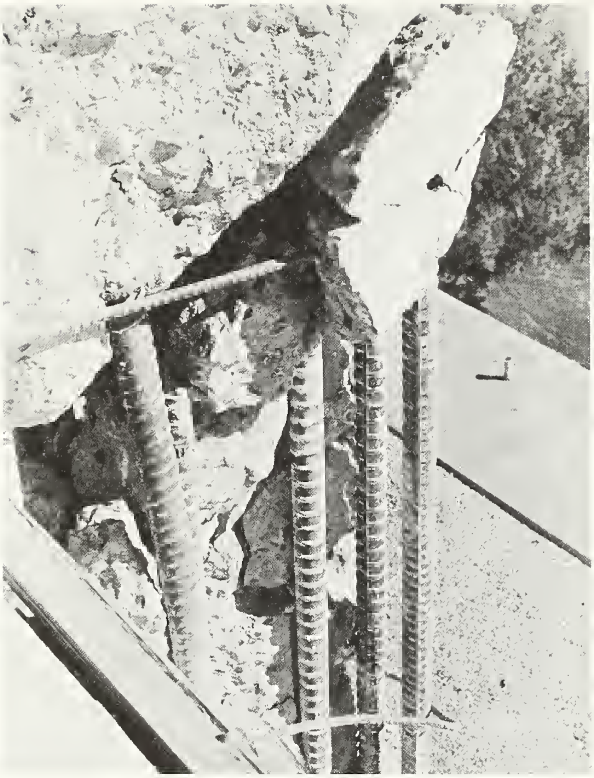


*Figure 5.60* *View of the reinforcement in the southeast corner fourth-floor column.*

*Figure 5.61* *East elevation of tower.*

*Figure 5.62 West elevation of tower.*

*Figure 5.63 Failure at third-story construction joint at the west end of the tower.*

*Figure 5.64 Reinforced concrete column at the west end of the third story.*

*Figure 5.65 Floor damage at the west end of the third floor.*

*Figure 5.66 Floor cracking at the west end of the third floor. The floor is concrete pan-joist construction with a 3-in. slab.*

*Figure 5.67* *Damaged northwest corner room on the third floor. Note shear wall, floor, and spandrel cracking.*

*Figure 5.68* *Damaged southwest corner room on the third floor. Note floor and spandrel cracking.*

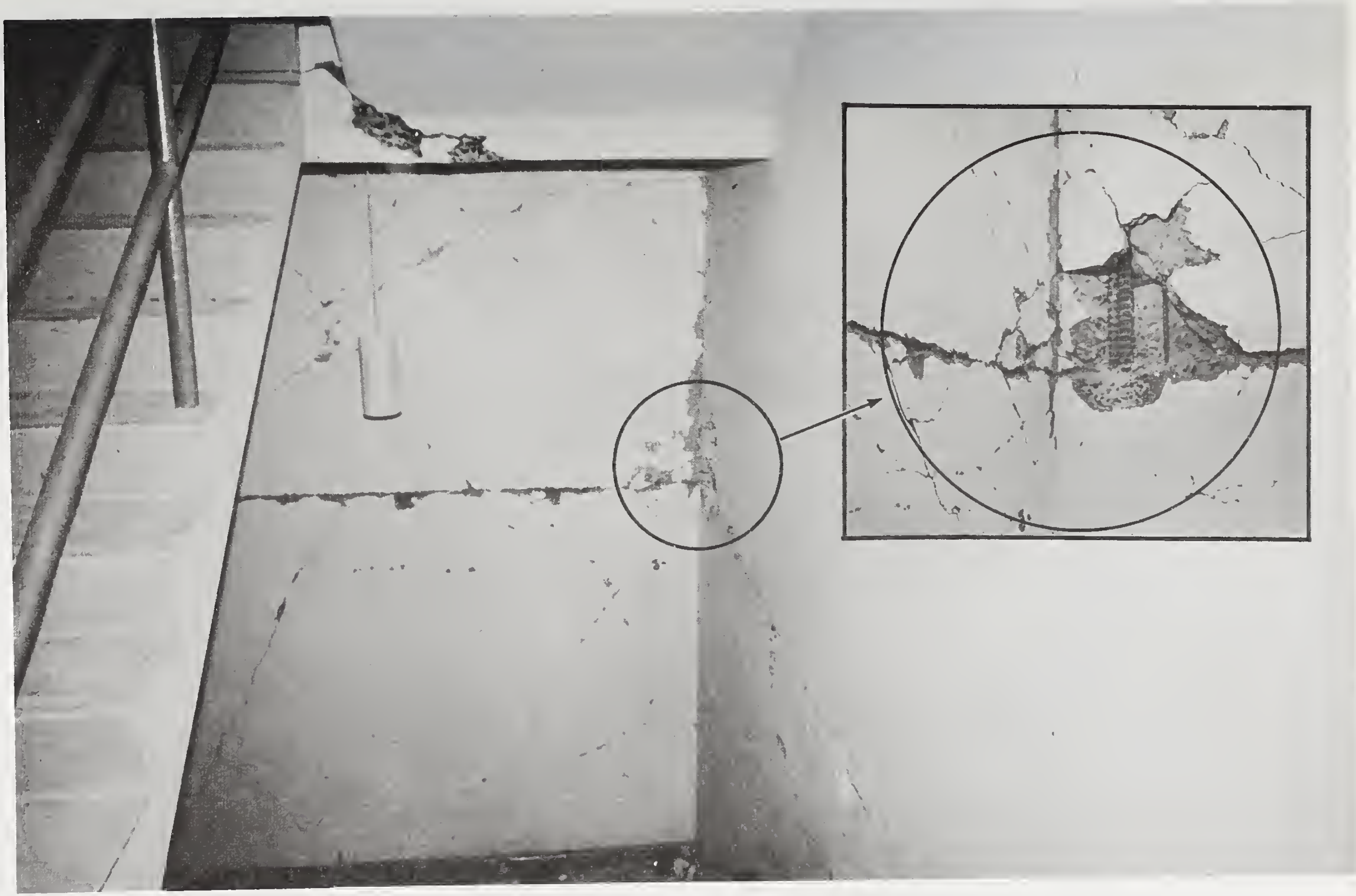


*Figure 5.69* *Diagonal and horizontal cracking in stairwell near the west end of the building. The horizontal crack is at the third floor.*

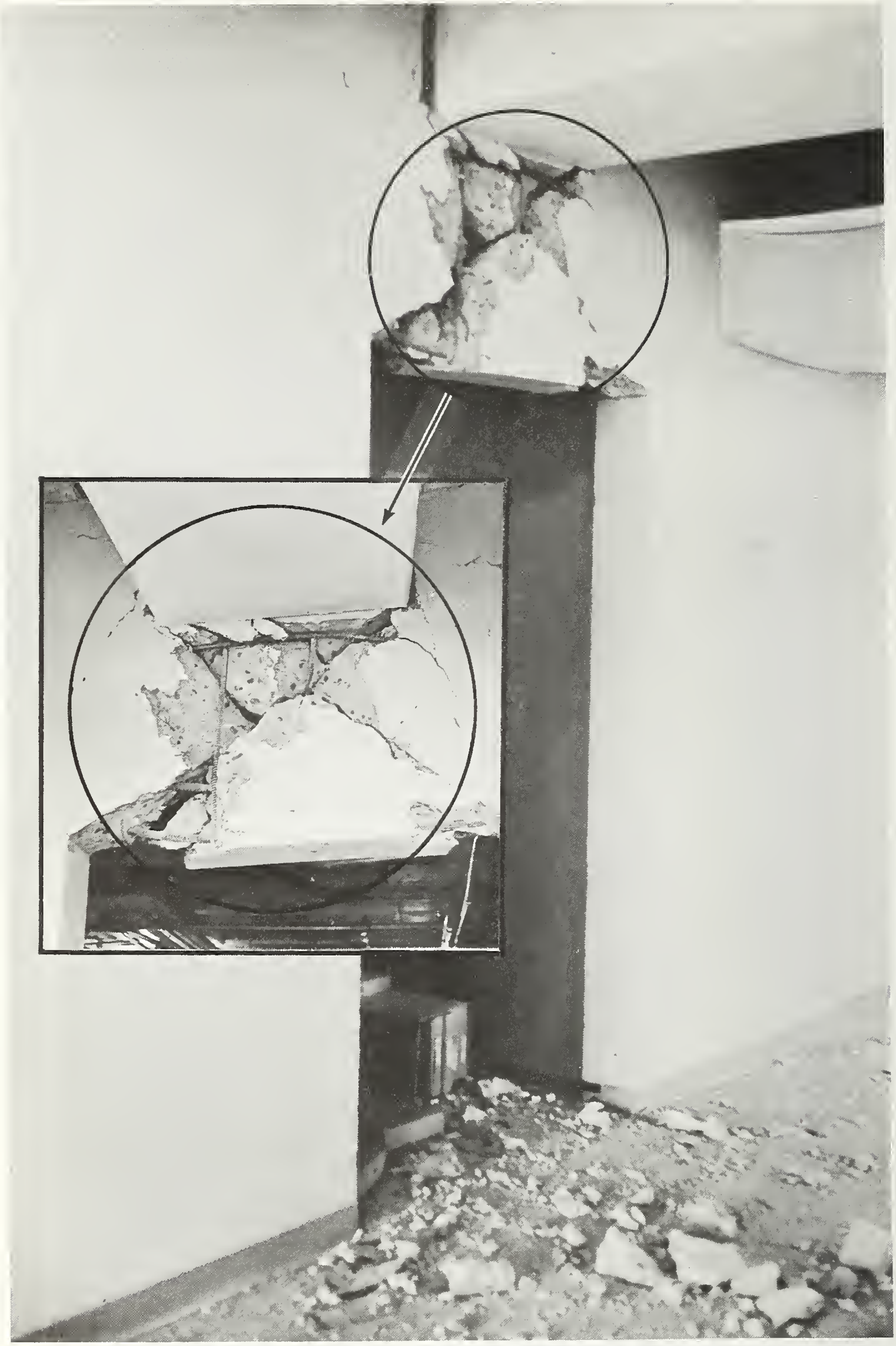


*Figure 5.70 Beam over doorway near the west end of the seventh floor. A duct passes through an opening above the beam.*

*Figure 5.71 Shaking caused shelves to overturn on the seventh floor.*

*Figure 5.72 West elevation of North Wing.*

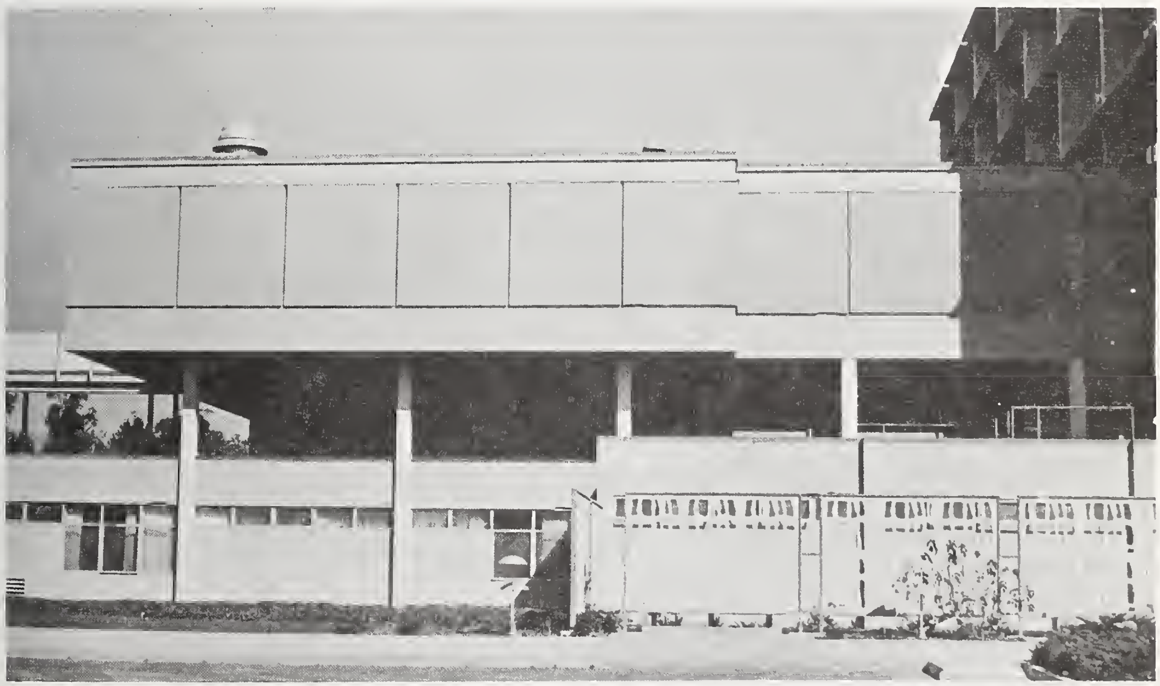


Figure 5.73 East elevation of three-story north wing. The east row of the second-story columns are shown in figure 5.74.

Figure 5.74 Torsional cracking of second-story columns in the north wing.

*Figure 5.75 Close-up view of one of the columns shown in figure 5.74*

*Figure 5.76 Holy Cross Continuing Care facility. This 50-bed single-story building was not damaged.*

## Pacoima Memorial Lutheran Hospital, Map 3, Location 22

Pacoima Memorial Lutheran Hospital is located on Eldridge Avenue. The hospital was built in 1959 and is shown in figure 5.77 along with the Golden State Mental Health Center which was under construction but almost complete. The light areas on the hills in the background are landslide areas.

All patients were evacuated from the 110-bed facility shortly after the earthquake. It was reported that the emergency facilities were initially closed but, although badly damaged, were reopened out of necessity; since this was the only hospital in this part of the San Fernando valley. The elevators were inoperative after the earthquake (one elevator was operative later). Emergency power had to be used throughout.

The north elevation of Pacoima Hospital is shown in figure 5.78. The building is a reinforced concrete shear wall structure with some reinforced masonry shear walls in the north and south sections. The floor and roof structures are reinforced concrete pan joist and beams. The foundation consists of drilled concrete piles. The structure was designed in 1958 under the Los Angeles City Building Code and was designed to resist horizontal forces of about 10 percent of gravity.

The building consists of three sections. The three story plus basement center section, the one- and two-story north section, and a one-story south section (figs. 5.77 and 5.78). A south elevation of the center section which shows the basement is shown in figure 5.79. A portion of the south section is barely visible behind the trees in the right of the figure. The reinforced brick wall behind the trees was cracked diagonally as shown in figure 5.80. This was the west wall of the cafeteria.

The east-end shear wall, a major load resisting element, was badly damaged as shown in figure 5.81. The portion of the walls outlined by a distinct rectangular crack pattern at each of three stories are referred to as "knockout panels" on the structural drawings. They were apparently provided for future expansion. An interior view of the first-story shear wall is shown in figure 5.82.

The stairwell shown in figure 5.83 was located at the east end of the center section and was damaged extensively. One of the two elevators (shown in figure 5.84) was badly distorted and inoperative. The second elevator was functioning.

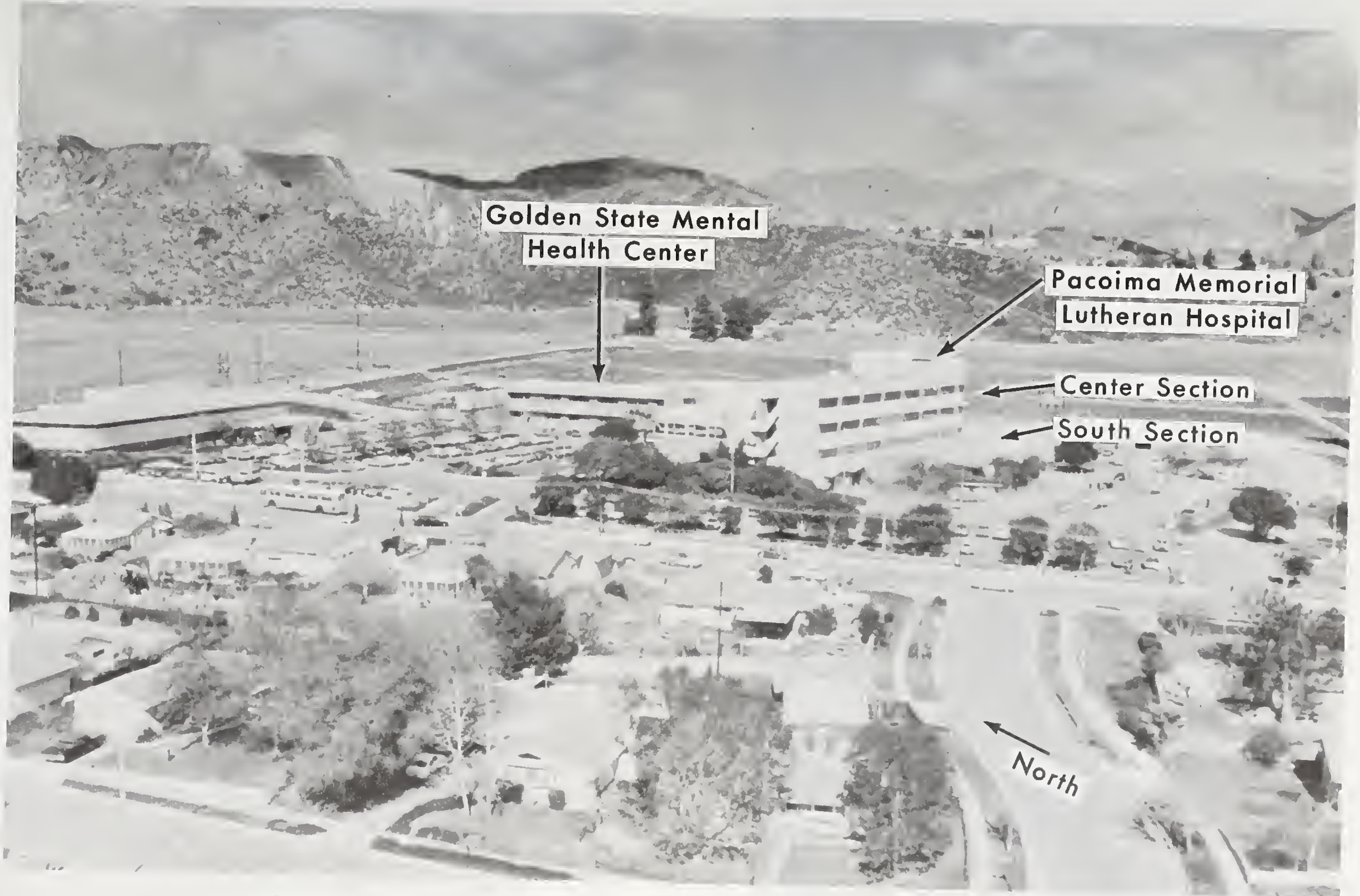



Figure 5.77 Aerial view of Pacoima Lutheran Hospital and adjoining Golden State Mental Health Center, 11600 Eldridge Avenue, Pacoima.

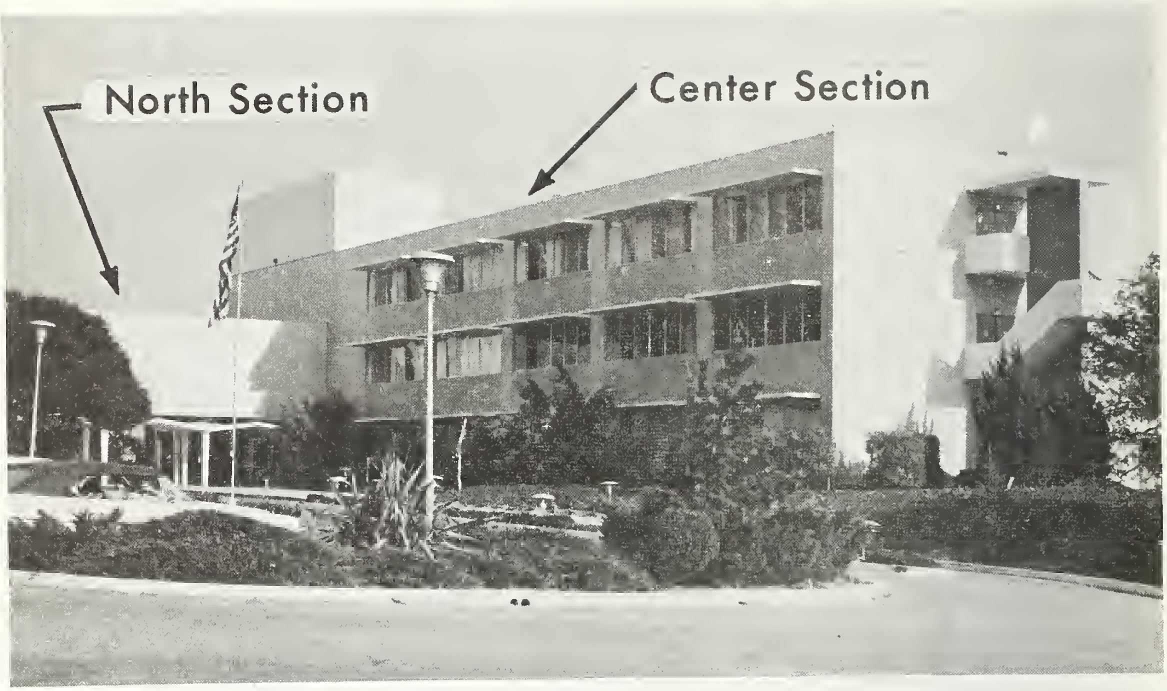



*Figure 5.78 North elevation of Pacoima Hospital.*

*Figure 5.79 South elevation of center section.*

*Figure 5.80 Reinforced brick masonry shear wall at west end of south section.*

*Figure 5.81 East-end shear wall of center section.*

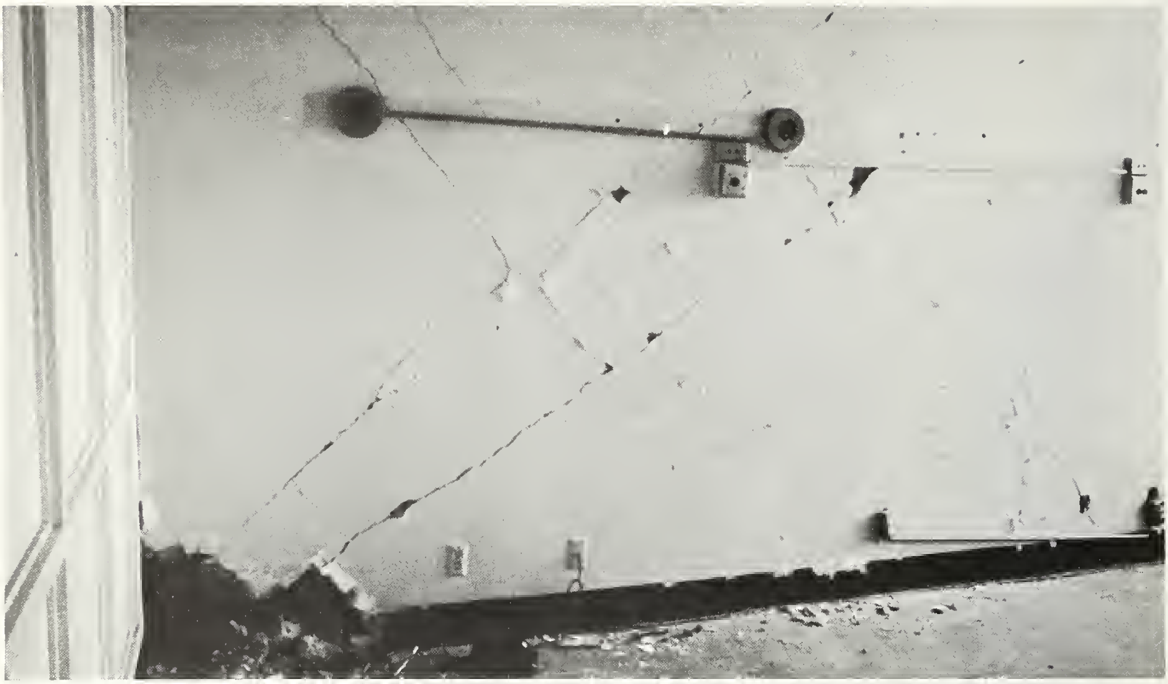


*Figure 5.82 Interior view of damaged reinforced concrete shear wall at the east end of the center section of Pacoima Hospital.*

*Figure 5.83 Damaged reinforced concrete beam and walls in the stair-well at the east end of Pacoima Hospital.*

The walkway shown in figure 5.85 connected the north section with the mental health center. Relative movement shown seemed to indicate uplift of the main building. Further relative movement is shown in figure 5.86 which shows the north side of the north section. It was not certain at the time if the displacement was due to building uplift or soil settlement. Later reports indicated that the pile caps had "jumped" off the piles.

The west elevation of the new two-story mental health center is shown in figure 5.87. Interior finishing was being rapidly completed so that this reinforced brick masonry building could be used as a general purpose facility. Damage to shear walls and other structural elements was minor as indicated by the cracking shown in figure 5.88.

Sepulveda V.A. Hospital, Map 3, Location 23

The Sepulveda Hospital (fig. 5.89), which is located about 2.5 miles south of the lower Van Norman Reservoir, was built by the Veterans Administration and opened in 1955. It is a 1000-bed campus-type complex dedicated mainly to psychiatric treatment. As of December 31, 1970, the hospital was operating at a level of 720 beds. Buildings range from one to six stories and are designed to resist earthquake forces. Patient buildings are constructed of reinforced concrete frames with reinforced masonry walls.

One of the six-story buildings experienced considerable damage to the seismic (flexible) joints between building segments. However, it was observed that these joints did minimize the damage which could have occurred. Another six-story building sustained minimal cracking in structural concrete walls and extensive plaster cracking, particularly on the fifth and sixth floors. Several buildings sustained visible cracking in reinforced concrete joist and slab floor systems. A number of elevators sustained damage to the cast iron counterweight guide shoes and rail brackets. Several other elevators sustained additional damage to hoisting machine drive motors.

Overall, there was only minimal structural damage to the Sepulveda V.A. Hospital. The operation of the hospital was not interrupted. However, extensive elevator and plaster repairs were required and a number of seismic joints required replacement.

Northridge Hospital, Map 4, Location 24

This hospital is located on Roscoe Boulevard near Reseda


*Figure 5.84 Buckled steel elevator door frame in the Pacoima Hospital.*

*Figure 5.85 Connecting enclosed walkway between Pacoima Hospital (on the left) and the Mental Health Center. The structural damage and ground condition (arrows) appeared to indicate that the hospital building had experienced several inches of uplift.*

Figure 5.86 *Relative movement between north section and ground.*

Figure 5.87 *New two-story (26-bed) Mental Health Center.*

Figure 5.88 Cracking in exterior roof beams of the Mental Health Center.

Figure 5.89 Aerial view of Sepulveda V.A. Hospital which opened in 1955.

Boulevard, some 15 miles away from the epicentral region. The epicentral distance of this hospital was about the same as that of the Holiday Inn (Orion Boulevard and Roscoe Boulevard, location 25), where ground accelerations were recorded. Thus, it is reasonable to assume that the ground accelerations at the hospital might have reached as much as 0.27g in the horizontal direction and 0.17g in the vertical direction.

The west elevation of this structure is shown in figure 5.90. It is a five-story steel frame structure with reinforced brick masonry shear walls. The exterior cladding was of brick veneer. Damage to the veneer was extensive on the east and west elevations (fig. 5.91). The shear walls in the first story were badly cracked at many places. However, it appeared that there was no damage to the steel frame as all parts of the structure were intact without a sign of significant permanent deformation. All mechanical and electrical equipment was functioning after the earthquake, including elevators.

The aftershock of March 31 (refer to section 1.1) caused additional damage to the brick veneer. Although the magnitude of this aftershock was small (4.0 on the Richter scale), because the epicenter was only three and a half miles away from this structure, the resulting damage to the exterior cladding was considerably greater than that caused by the February 9 quake. During the aftershock large pieces of the brick veneer were spalled off from the wall, exposing the reinforcement in the shear wall (fig. 5.92). The building, however, remained in full use after both the February 9 earthquake and the March 31 aftershock.

### Kaiser Foundation Hospital, Map 4, Location 26

Kaiser Foundation Hospital, 13652 Cantara Street, Panaroma City, is a 10-story reinforced lightweight concrete shear wall structure (fig. 5.93). The basement and lower 3 stories have a rectangular plan, approximately 215 ft by 125 ft (fig. 5.94). The upper 7 stories consist of two circular sections, approximately 90-ft in diameter, connected by an elevator lobby section (fig. 5.95).

There was severe cracking of the shear walls in the first, second, and third stories (fig. 5.96). The doors to the interior stairwell on the second and third floors were rendered inoperable due to the crushing of the spandrel beam in the shear wall (fig. 5.97). The fourth-floor slab, which is the transfer slab between the circular tower shear walls and the walls below, cracked and displaced vertically. There

Figure 5.90 *Northridge Hospital on Roscoe Boulevard near Reseda Boulevard.*

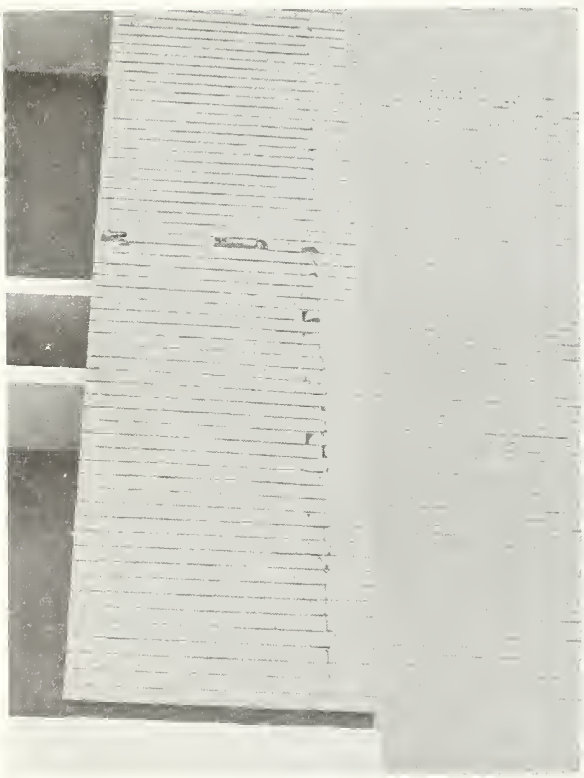


Figure 5.91 *Typical cracks in brick veneer over the west entrance.*

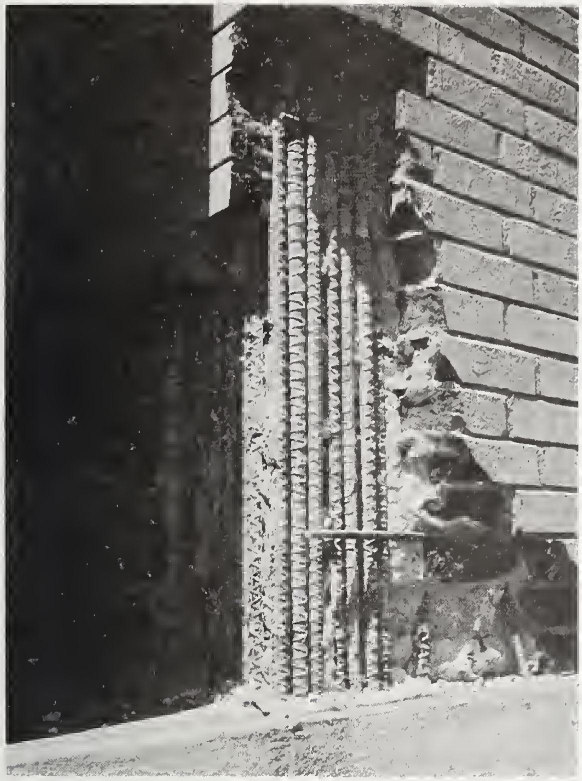


*Figure 5.92 Additional damage to brick veneer resulted from the March 31 aftershock, exposing the reinforcement in the shear wall.*

*Figure 5.93 General view of the 10-story Kaiser Foundation Hospital.*

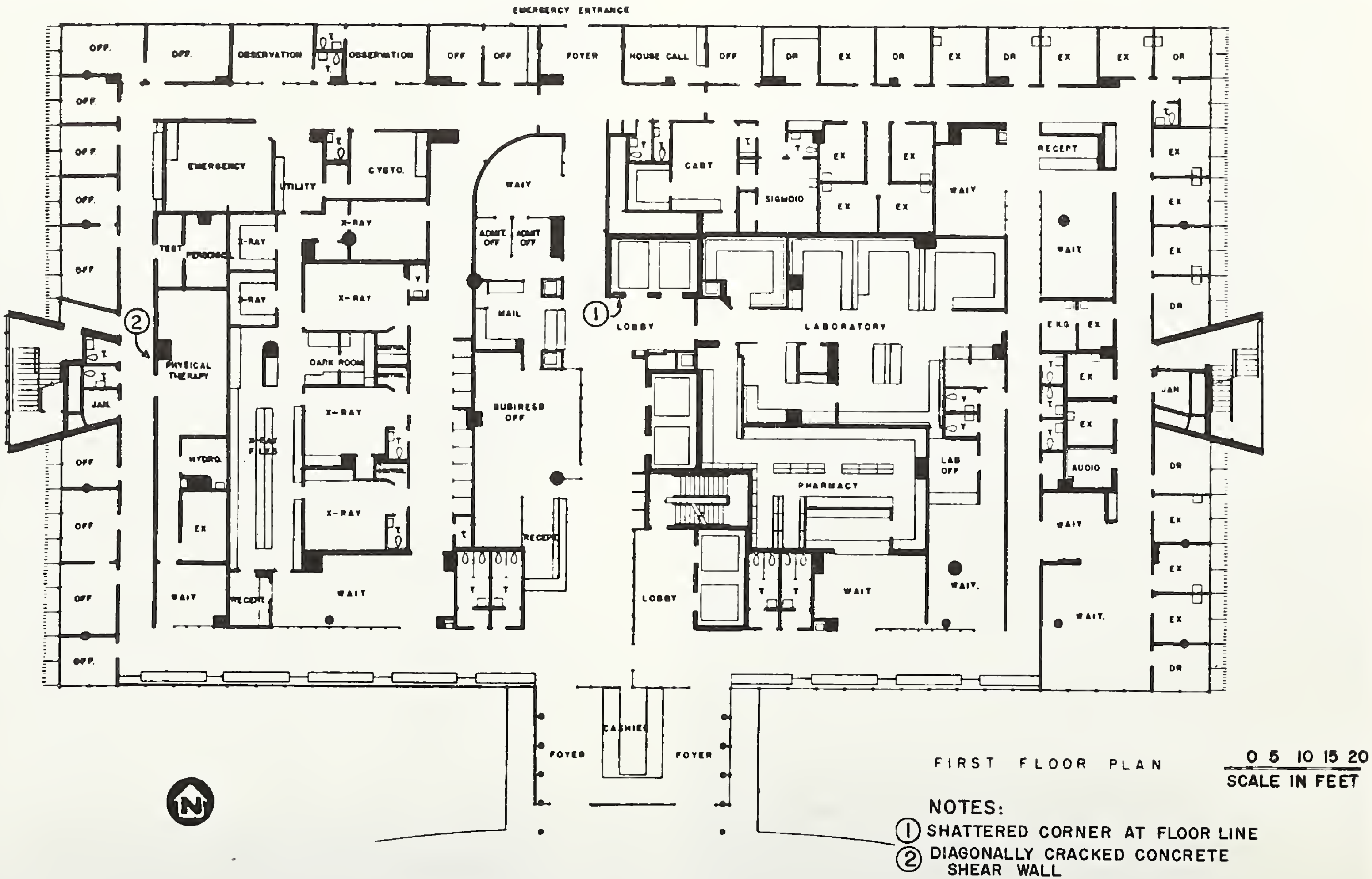
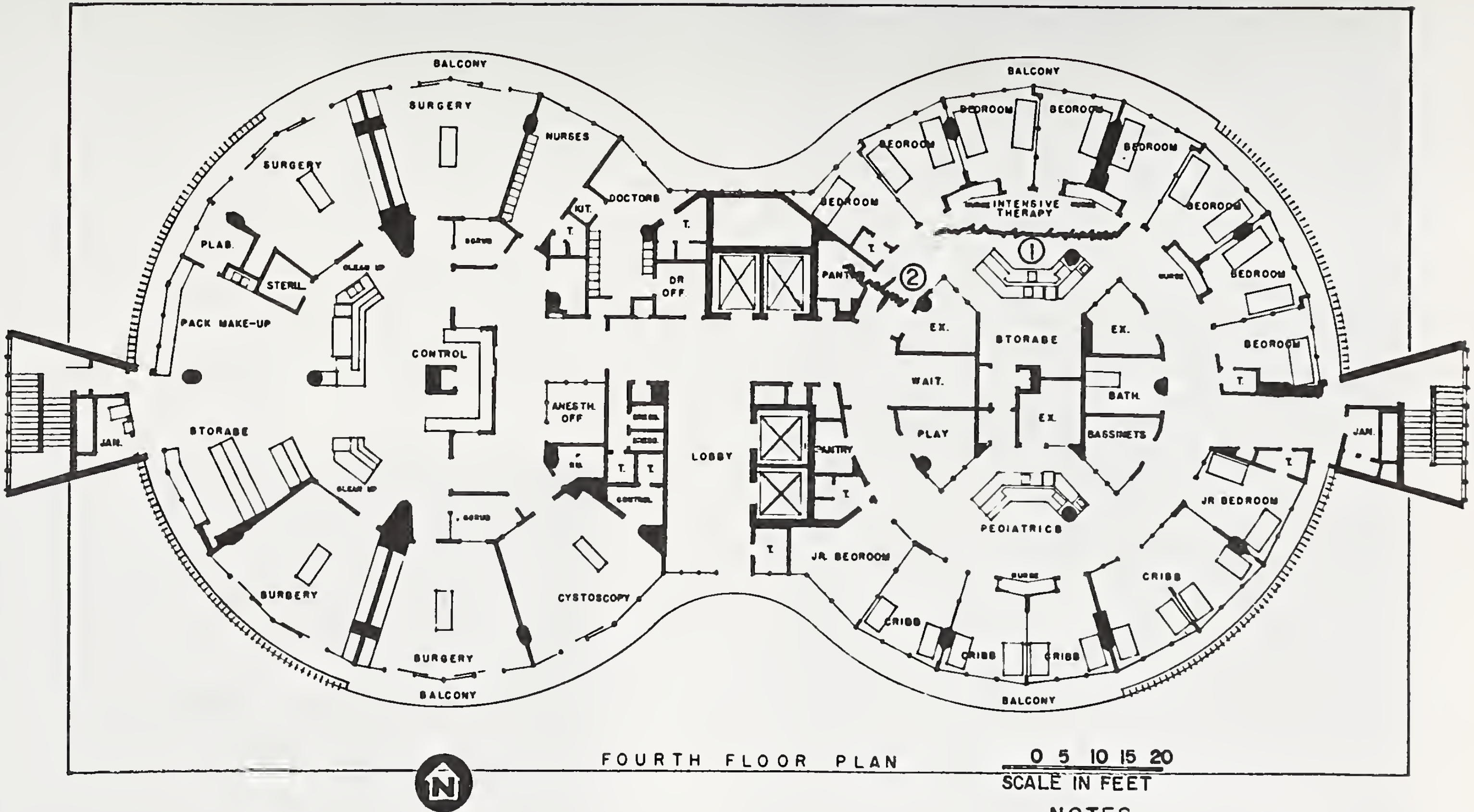



*Figure 5.94 First-floor plan, Kaiser Foundation Hospital.*




NOTES:

(1) LARGE CRACK IN SLAB (E-W)
(2) CRACK IN FLOOR WITH 1"- OF VERTICAL DISPLACEMENT

*Figure 5.95 Fourth-floor plan, Kaiser Foundation Hospital. Outline of 3rd-story roof also shown.*

*Figure 5.96 Crushed reinforced concrete elevator shaft on the first floor. Note the exposed reinforcing steel. (See fig. 5.94, location 1).*

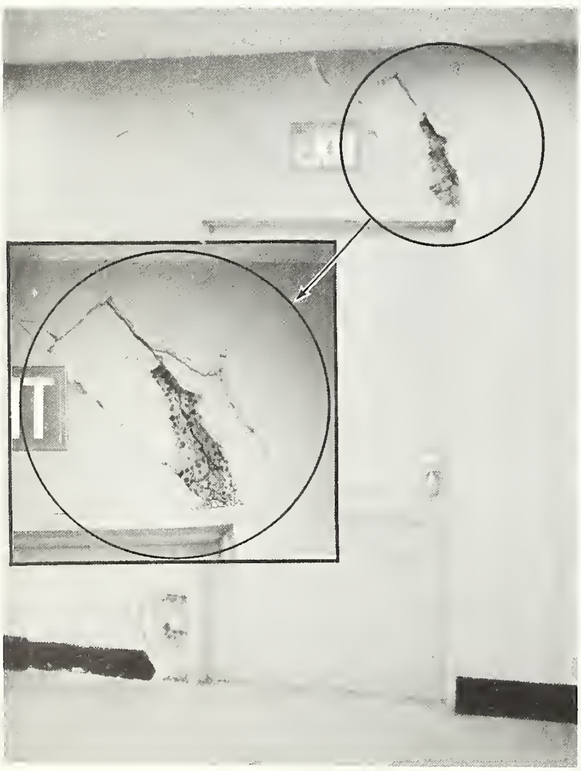


*Figure 5.97 Second-floor door to interior stairwell. This door was jammed shut.*

was no apparent sign of structural damage above the fourth floor level.

During the February 9 earthquake about 50 glass panels fell from the building. In addition many windows were blown out by wind on February 19. The chief engineer for the hospital attributed the wind damage to glass panels which were loosened in their stops during the earthquake.

All the elevators were out of service after the earthquake and one elevator was still inoperable on March 1. The emergency power system remained functional. The entire hospital remained in operation.

## COMMERCIAL BUILDINGS

### Indian Hills Medical Center, Map 3, Location 18

The Indian Hills Medical Center is a six-story reinforced concrete structure with a light steel frame penthouse. The building is located near the previously described Holy Cross Hospital and is shown in figures 5.53 and 5.54.

A view of the structure taken from Holy Cross is shown in figure 5.98. A typical floor plan is shown in figure 5.99. The first-story height is 17 ft 6 in, then 13 ft 3 in is typical for all stories above. The structure is supported by a concrete pile foundation.

Design was based on the 1966 Los Angeles City Building Code using K=1.0. The building has eight normal weight concrete shear walls with integrally cast columns in addition to a beam-and-column frame. Floor slabs are lightweight concrete.

There was horizontal cracking at the construction joint at each floor level as shown in figure 5.100. This was particularly pronounced at the second, third, and fourth floors where concrete also spalled at the corners (figs. 5.101, 5.102, 5.103, and 5.104). Note the buckled vertical reinforcement in figure 5.104. These same shear walls were also cracked diagonally for the full height of the building (fig. 5.102). There was also extensive crushing at the base of the southeast shear wall as shown in figure 5.105.

Interior damage consisted of horizontal cracking at the floor-to-wall joint of the third, fourth, and fifth floors (fig. 5.106). There were also damaged beam-column connections on these same floors (fig. 5.107). Some damage was also noted in the stairwell where the connections of metal stairs

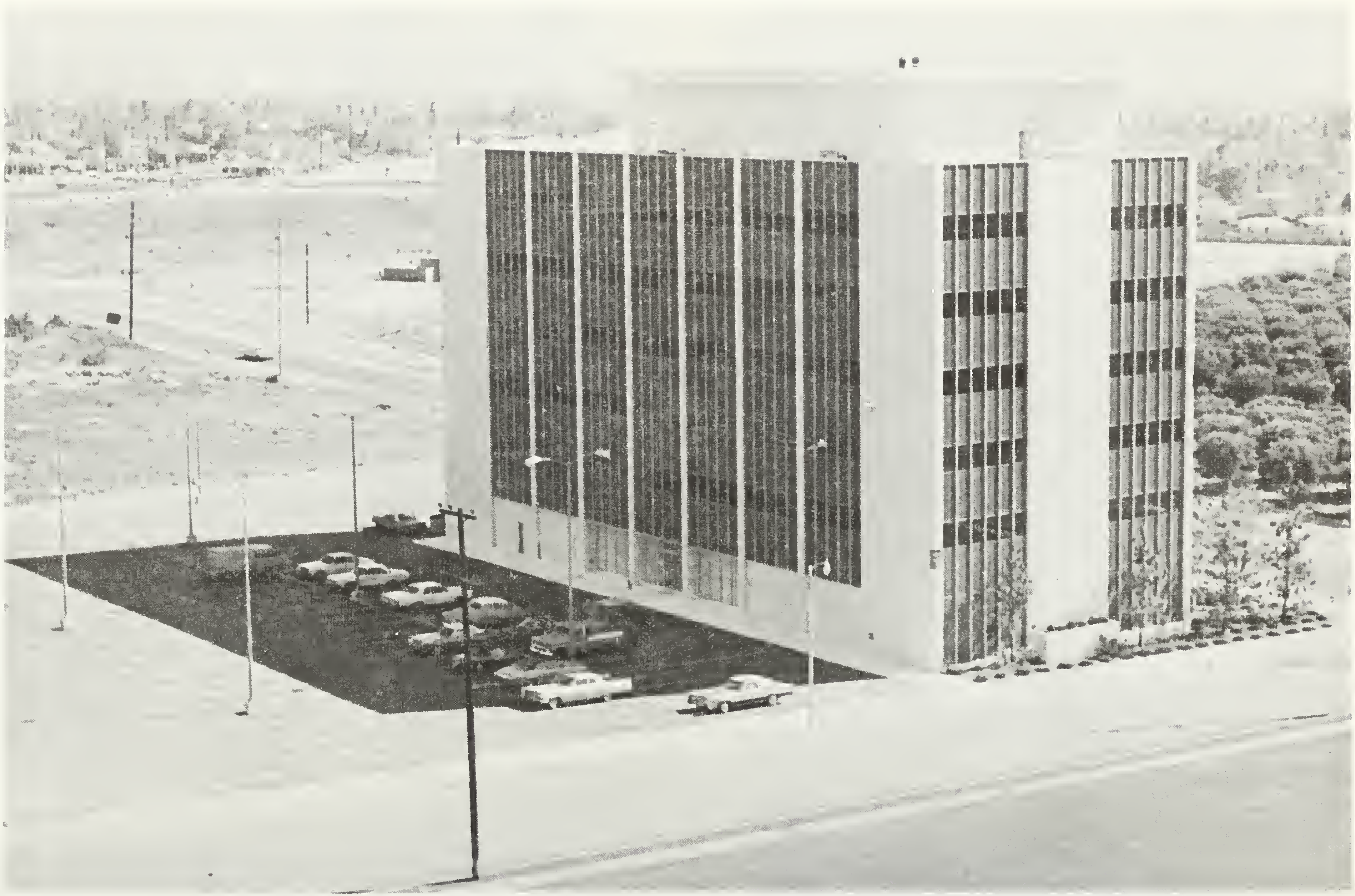


*Figure 5.98 North elevation of the Indian Hills Medical Center as viewed from Holy Crcss Hospital.*

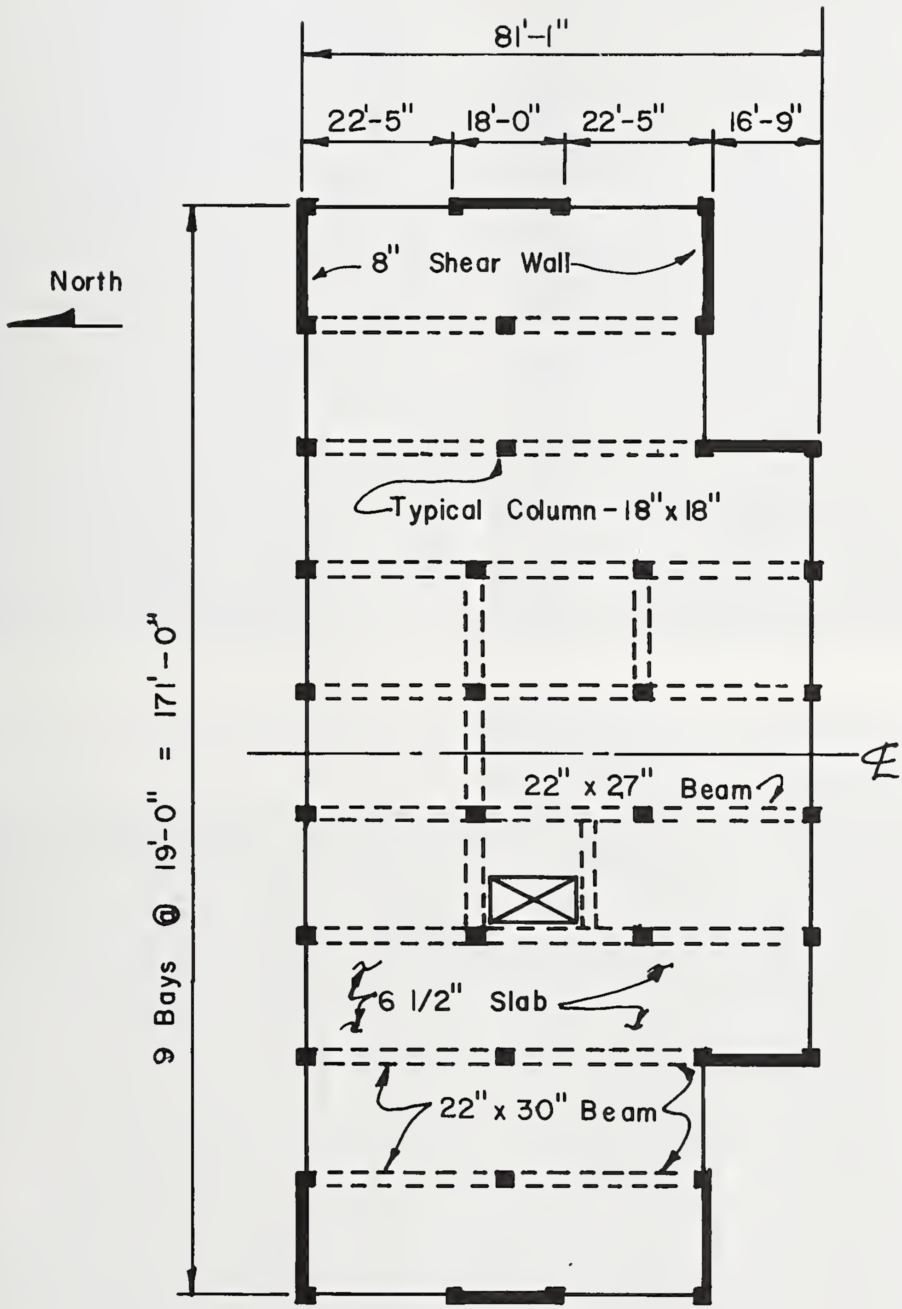



*Figure 5.99 Indian Hills Medical Center - Typical floor plan.*

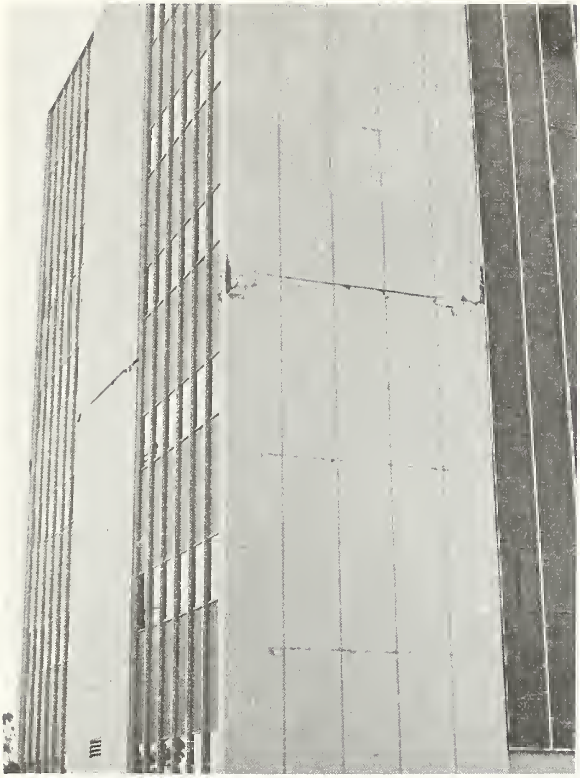
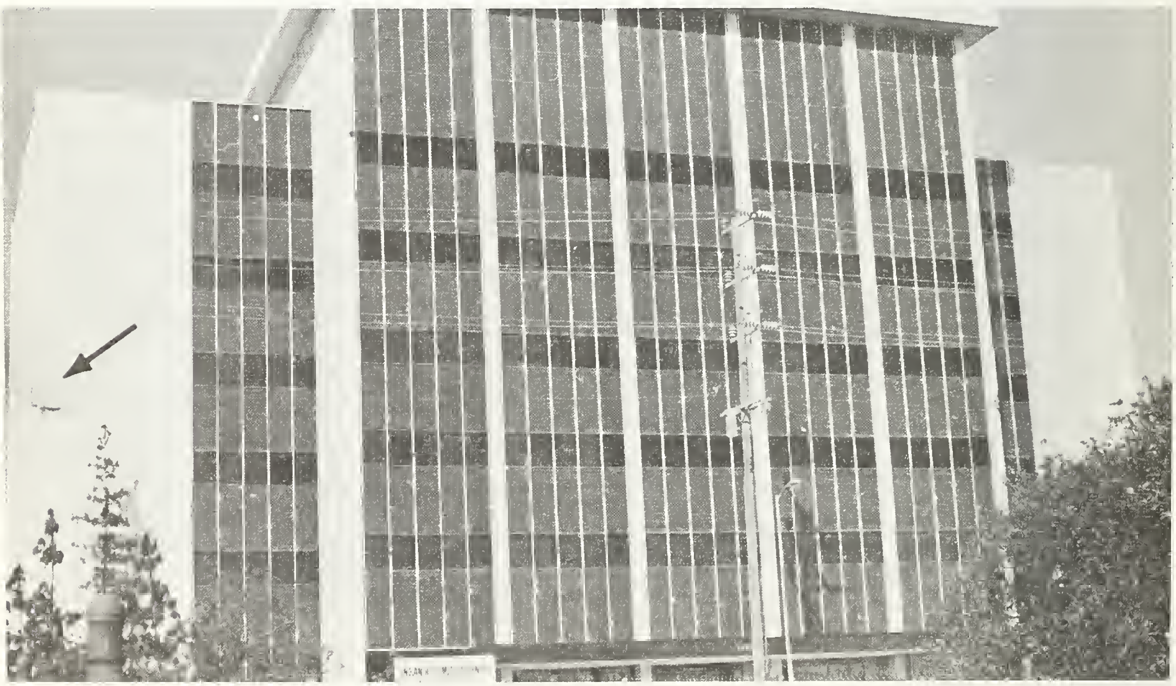


*Figure 5.100* View of the northeast corner. The north and east shear walls were cracked horizontally at the second through the sixth floors. Diagonal cracks were also present between floors.

*Figure 5.101* South elevation of the Indian Hills Medical Center. Note the horizontal crack and spalling visible on the left shear wall at the fourth-floor level.



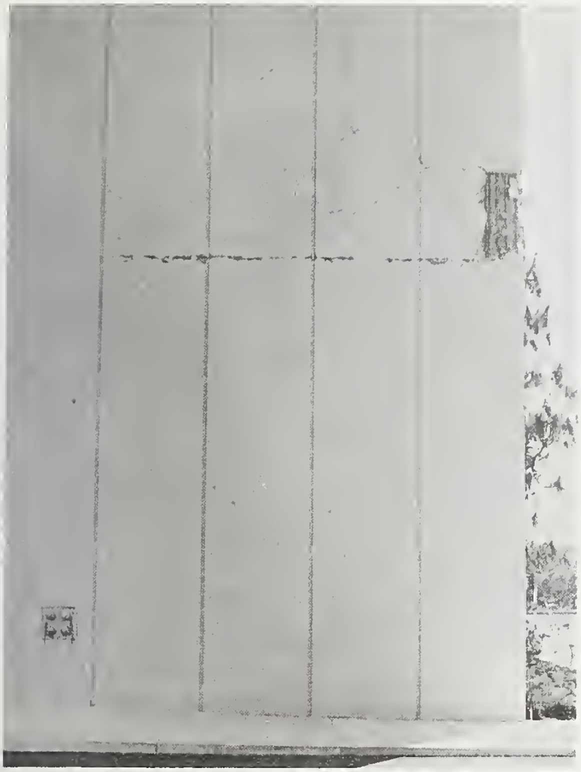


*Figure 5.102 Diagonal cracking and spalling in the northwest corner shear wall.*

*Figure 5.103 Close-up view of the spalled area of the shear wall shown in figure 5.102.*

Figure 5.104 Buckled vertical reinforcement in southeast shear wall.

Figure 5.105 Cracking at the base of the southeast corner shear wall. Note the exposed reinforcing bar.

*Figure 5.106 Interior view of horizontal cracking at the northeast corner of the fourth floor.*

*Figure 5.107 Damaged beam-column connection in the fourth story. The large piece of broken concrete is suspended by an electrical cable.*

pulled away from the concrete wall at the landings. Although usable, the stairs were "shaky." Elevators were inoperative immediately after the earthquake but were working at the time of inspection.

The structure was repaired by attaching new 8-in gunite walls to the existing shear walls and providing additional floor-slab support by means of steel angles.

Holiday Inn, Map 4, Location 25

The Holiday Inn at 8244 Orion Street, Panorama City, is a seven-story (no basement) hotel that is about four years old. The building is a reinforced concrete frame with two end shear walls (fig. 5.108).

This building had strong-motion accelerometers on the first and fourth floors and in the roof penthouse. On the first floor the maximum horizontal and vertical accelerations were 0.276g and 0.171g, respectively. The maximum horizontal and vertical accelerations recorded in the penthouse were 0.388g and 0.224g, respectively. These recordings are quite important since this was the closest instrumented building to the epicenter and the area of greatest damage.

The most severe damage was to nonstructural elements such as plaster walls and plumbing fixtures. This damage is illustrated in figure 5.109. A cracked cast-iron bathtub which has already been removed from the building is shown in figure 5.110. This type of damage was greatest on the second, third, and fourth stories. This evidence together with the recorded accelerations suggest that this building underwent severe lateral deformation during the earthquake. There were apparently some cracks in the structural frame which were repaired with epoxy.

Union Bank, Map 4, Location 27

The thirteen-story Union Bank Building is located at 15233 Ventura Boulevard across the street from the Bank of California (fig. 5.111). Accelerometers were not installed in the Union Bank; however, maximum horizontal and vertical ground accelerations of 0.227g and 0.100g were recorded at the Bank of California.

The building is constructed of reinforced concrete and was designed under the 1964 Los Angeles City Code as a moment-resisting frame (K=0.67). A typical floor plan is shown in figure 5.112. Typically the story height is 11 ft 9 in with a 23 ft 6 in first story. The total building


Figure 5.108 Elevation of Holiday Inn.

Figure 5.109 Damaged bathtub.

Figure 5.110 Cracked and chipped bathtub.


*Figure 5.111 Aerial view of the Union Bank and the Bank of California buildings located on Ventura Boulevard near Sepulveda Boulevard.*

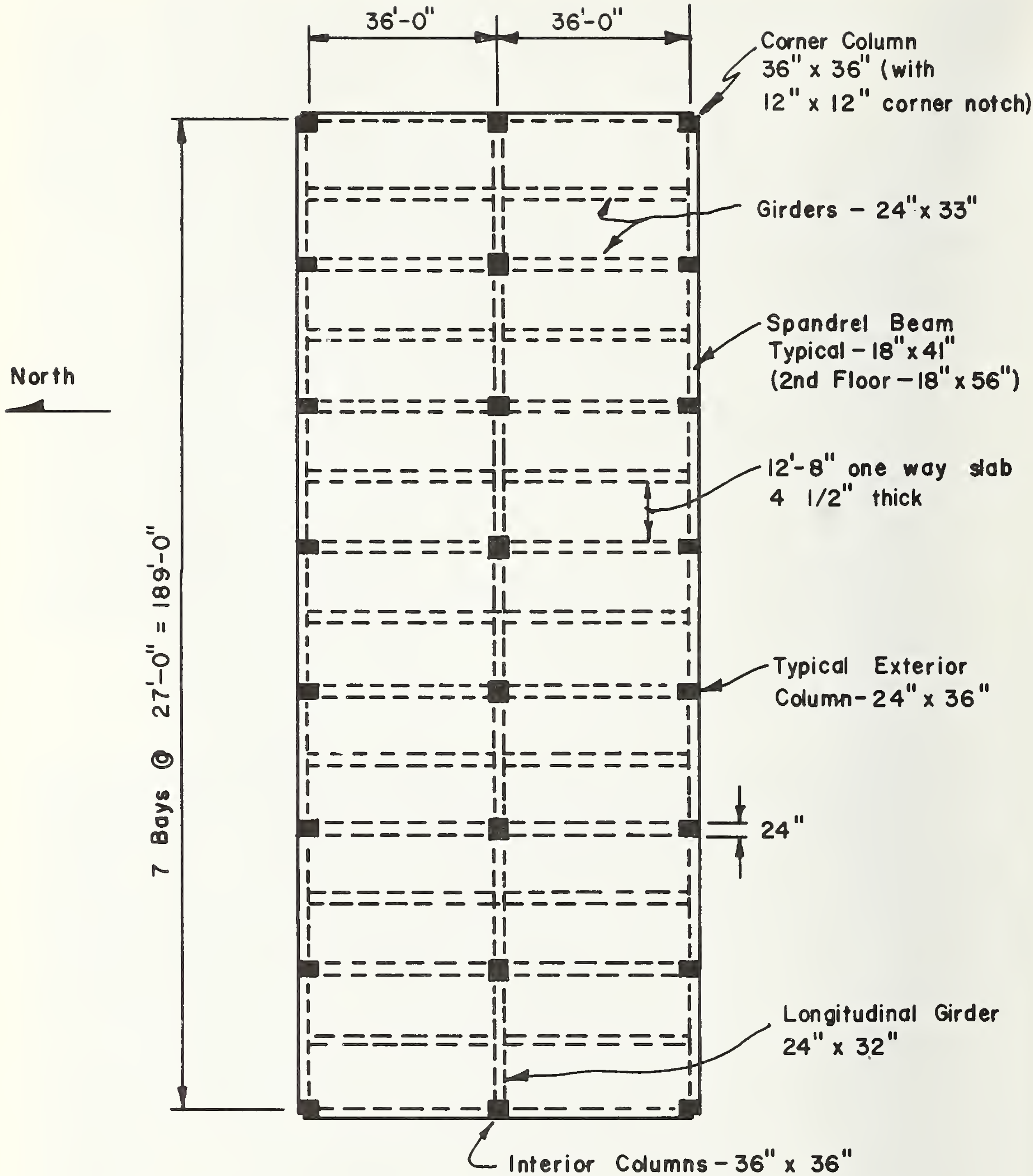



*Figure 5.112 Typical floor framing plan of the Union Bank Building.*

height is 164 ft 6 in. There is a two-level underground garage and a mechanical penthouse. The foundation is drilled cast-in-place friction piles.

An elevation of the bank building is shown in figure 5.113. Principal structural damage was in the first-story columns, primarily at the second-floor beam-column connections as shown in figure 5.114. These columns were repaired with epoxy shortly after the earthquake as shown in figure 5.115. However, the initial attempt was not successful and two additional trials were made before the repair was considered to be acceptable.

Other non-structural damage reported consisted of inoperative elevators, jammed doors on the 2nd, 3rd, 4th, and 5th floors, steel stair landings pulled away from supports, some partition damage, and cracked glass and marble veneer in the first story.

The parking garage located on the north side of the bank (see fig. 5.111) is shown in figure 5.116. The garage was constructed of precast pretensioned hollowcore planks with a cast-in-place topping supported on precast pretensioned girders. Columns are cast-in-place reinforced concrete. Most damage was limited to the columns which were cracked on the east-west faces as shown in figure 5.117. Concrete column corbels were also cracked as shown in figure 5.118.

Bank of California, Map 4, Location 28

The twelve-story Bank of California is located at 15250 Ventura Boulevard across the street from the Union Bank Building (fig. 5.111). Strong-motion accelerometers were located on the first floor, on the seventh floor, and on the roof. Maximum recorded horizontal and vertical accelerations in the basement were 0.227g and 0.100g. On the roof the maximum horizontal and vertical accelerations were 0.261g and 0.131g.

The building is constructed of reinforced concrete and was designed under the 1964 Los Angeles City Code as a shear wall building with a 25 percent frame (K=1.0). Shear walls are present only in the first two stories. The building is a frame structure for the rest of its height. An elevation is shown in figure 5.119 and a floor plan in figure 5.120. There is no basement. The building foundation is on drilled concrete piles.

There were cracks in the building frame over most of its height. These were especially pronounced in the connection of the lightweight concrete beams to the normal weight concrete


*Figure 5.113* *The south and west elevations of the thirteen-story Union Bank building. All four exterior corner columns in this reinforced concrete structure were cracked at the second floor level.*

*Figure 5.114 View of cracked southeast column.*

*Figure 5.115 Cracked beam-column connections at the first floor were repaired with epoxy.*

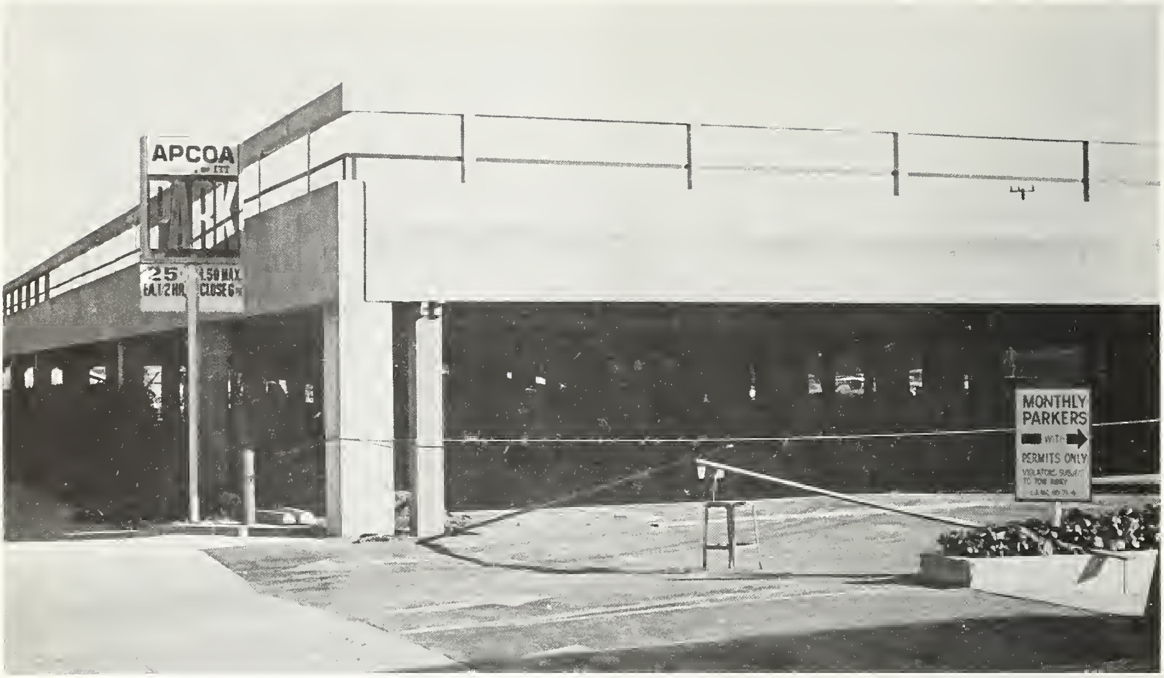
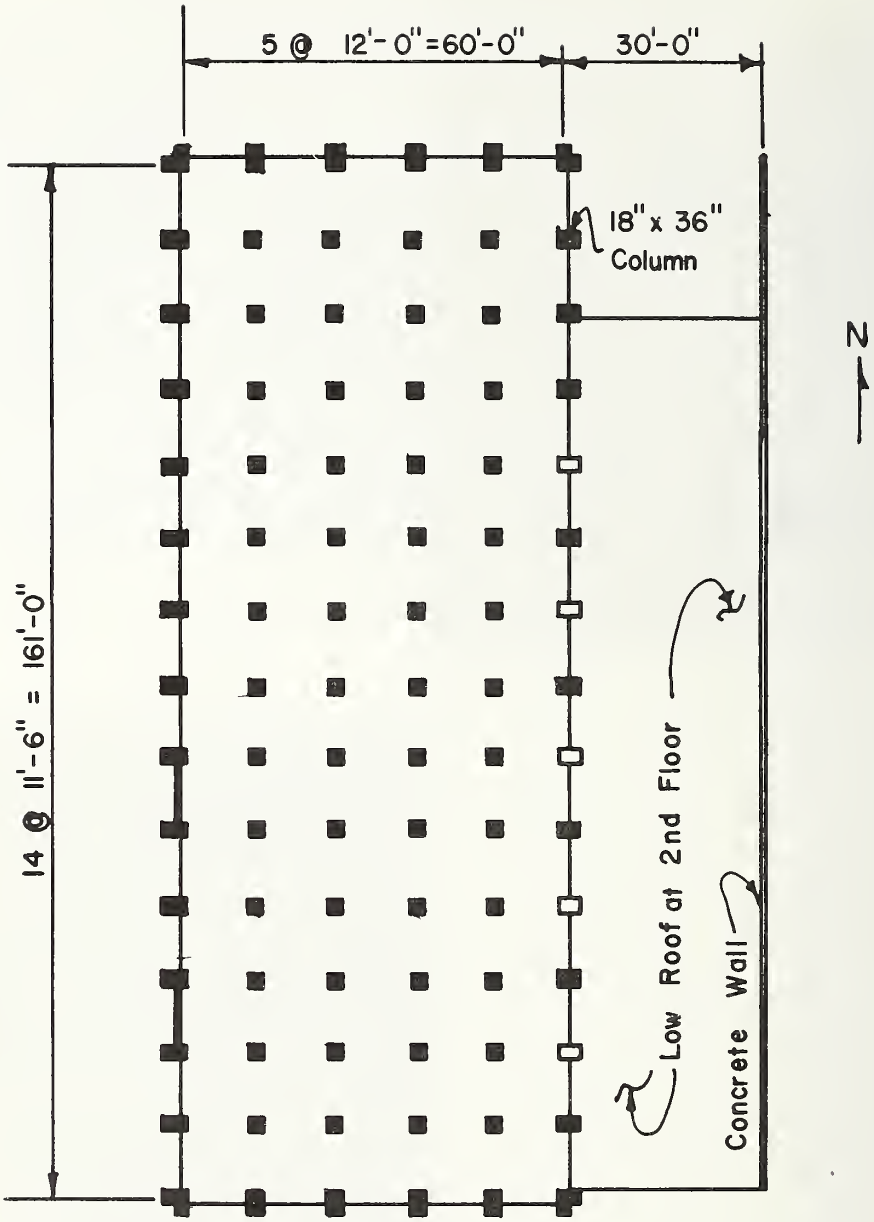



*Figure 5.116 Parking garage on the north side of the Union Bank building.*

*Figure 5.117 Typical cracked interior column. All beams with damaged columns were shored with timber posts.*

Figure 5.118 Cracking of exterior column corbel.


Figure 5.119 North and east elevations of the twelve-story Bank of California building.




Notes: 1. Columns not shaded are present only from the 3rd story up.
2. Shear walls are present only through the second story.

*Figure 5.120 Bank of California - Typical floor plan.*

columns as shown in figure 5.121. This damage was being repaired with epoxy.

Crocker Citizens Bank, Map 4, Location 29

The fourteen-story Crocker Citizens Bank shown in figure 5.122 is a reinforced concrete frame and shear wall structure located at 14724 Ventura Boulevard, very near the Union Bank and Bank of California. A typical floor plan is shown in figure 5.123. There was little or no damage reported in this building. This building had strong-motion recorders on the ground floor (no basement), sixth floor, and penthouse. First floor maximum horizontal and vertical accelerations were 0.258g and 0.093g. At the roof the maximum horizontal acceleration was 0.315g. The vertical trace was too faint to read.

Sheraton Universal Hotel, Map 4, Location 31

The 21-story Sheraton Universal Hotel, figure 5.124, is a reinforced concrete building located at 3838 Lankershim Boulevard. This 210-ft building was designed under the Los Angeles Code using the November 1966 modifications for a ductile moment-resisting frame. The floor plan of the high-rise portion is shown in figure 5.125. A cross section through the main building and a typical beam-to-column framing detail is shown in figure 5.126a and b, respectively. Apparent damage to this concrete structure was limited to a cracked mural as shown in figure 5.127.

Strong motion accelerographs in this building were installed in the basement, the 11th floor, and the 21st floor. Maximum horizontal and vertical accelerations of 0.175g and 0.087g were recorded in the basement. Both maximum accelerations were 0.226g on the 21st floor.

Grant's Department Store, Map 2, Location 2

Grant's Department Store is located on Soledad Canyon Road in Newhall, some 10 miles directly west of the epicenter. An exterior view of what appears to be an undamaged building is shown in figure 5.128. It is a single story, brick-wall structure with "panelized roofs".

The use of panelized roof systems was found to be common for commercial and industrial buildings. Roof framing usually consisted of glue-laminated wood girders spanning between interior columns and the exterior wall. Wood joists spaced at 8 ft on center span between the girders and wooden

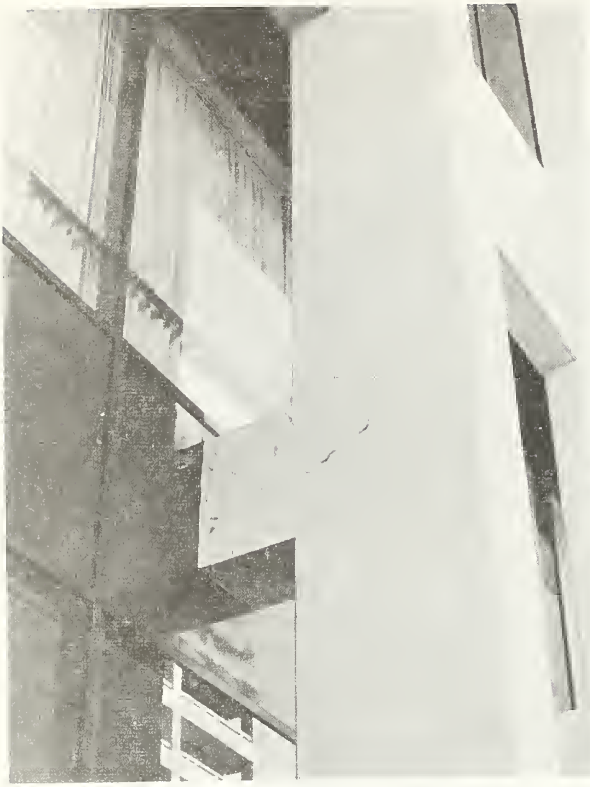


Figure 5.121 Cracking in a beam-column joint at the second floor level on the north face of the Bank of California building.

Figure 5.122 Elevation of Crocker Citizens Bank.

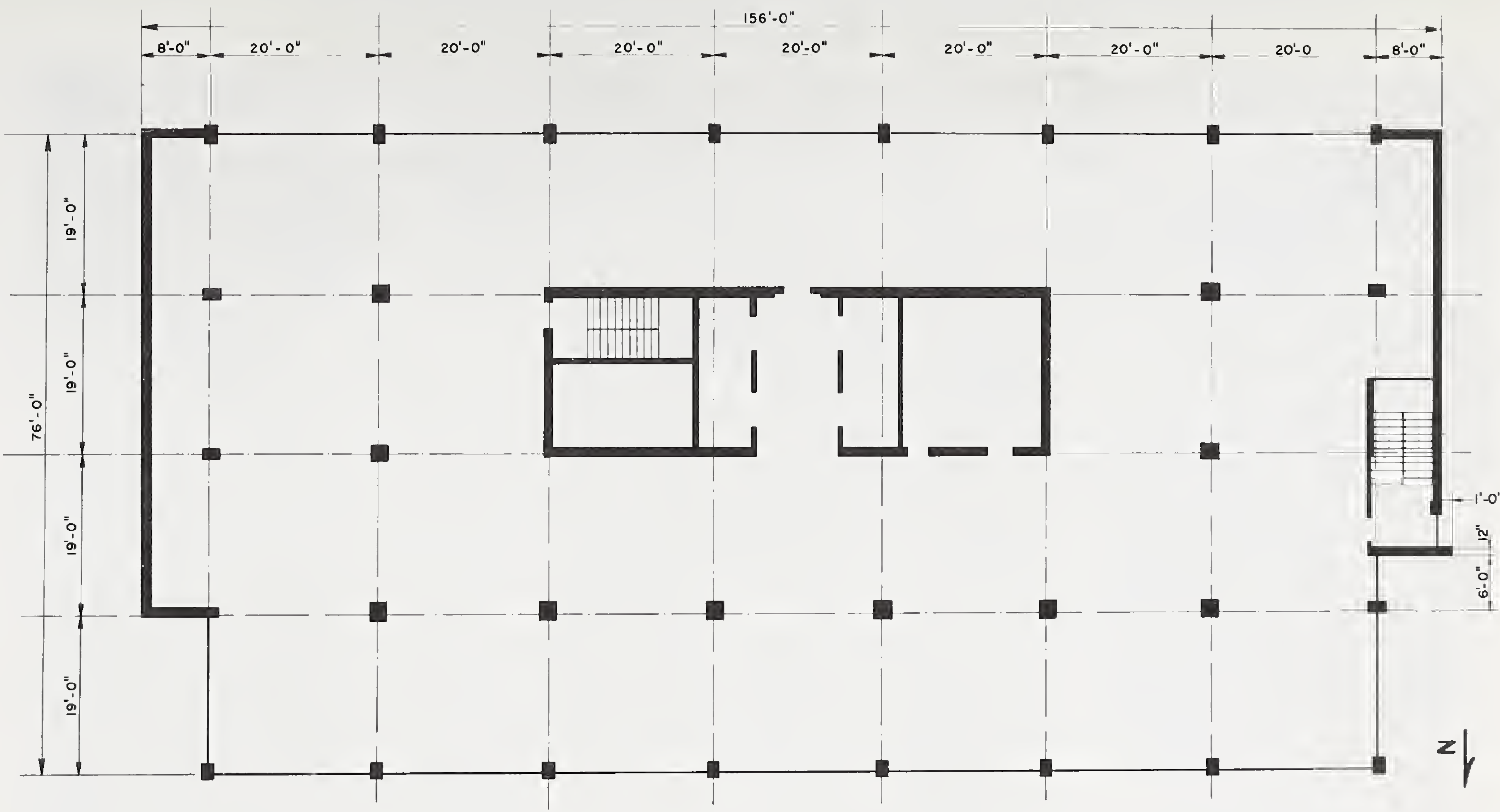



Figure 5.123 Floor plan of Crocker Citizens Bank.


*Figure 5.124 Elevation of Sheraton-Universal Hotel.*

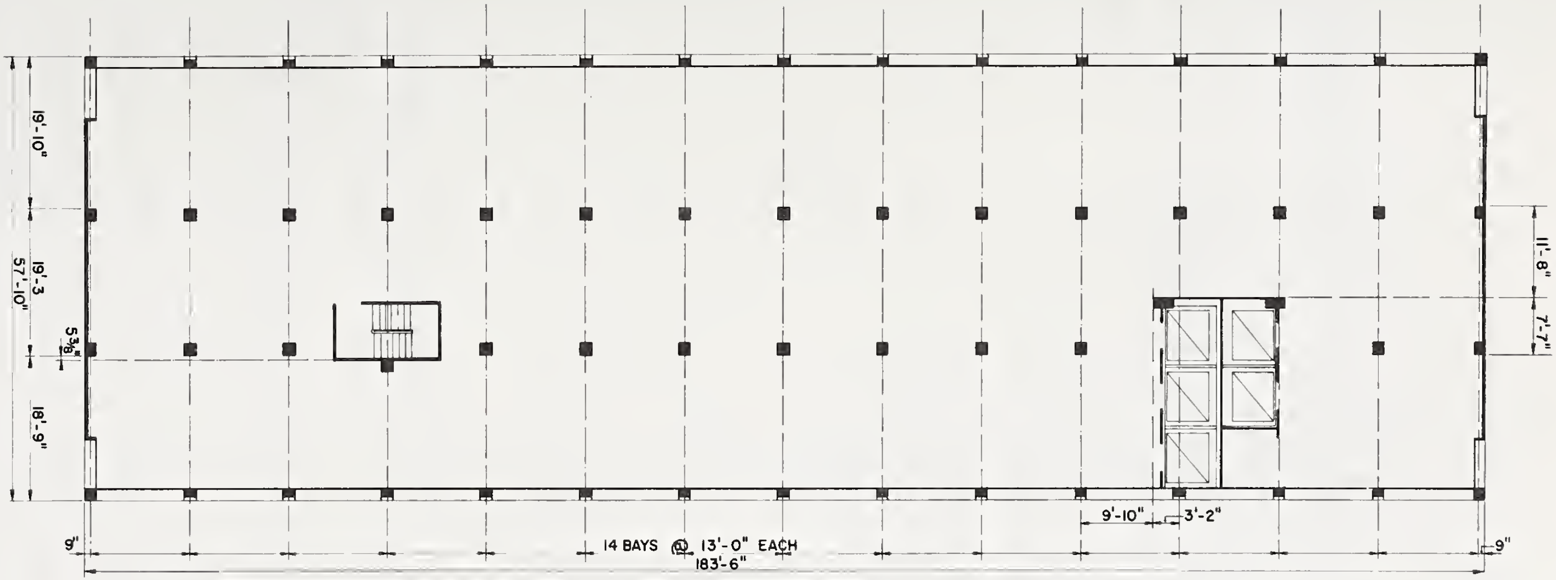



*Figure 5.125 Typical floor plan of high-rise section.*

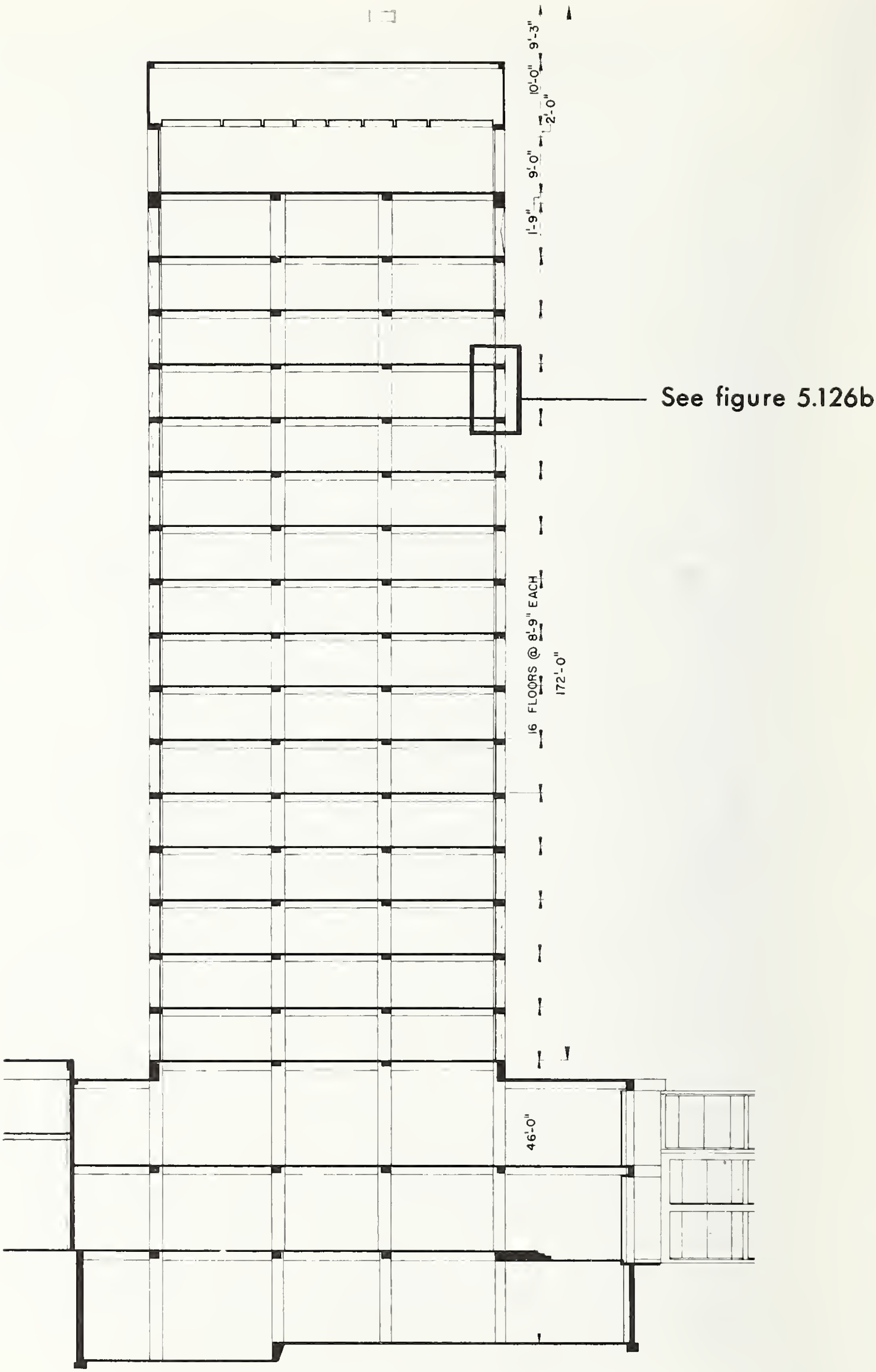



*Figure 5.126a  Typical cross section of elevation.*

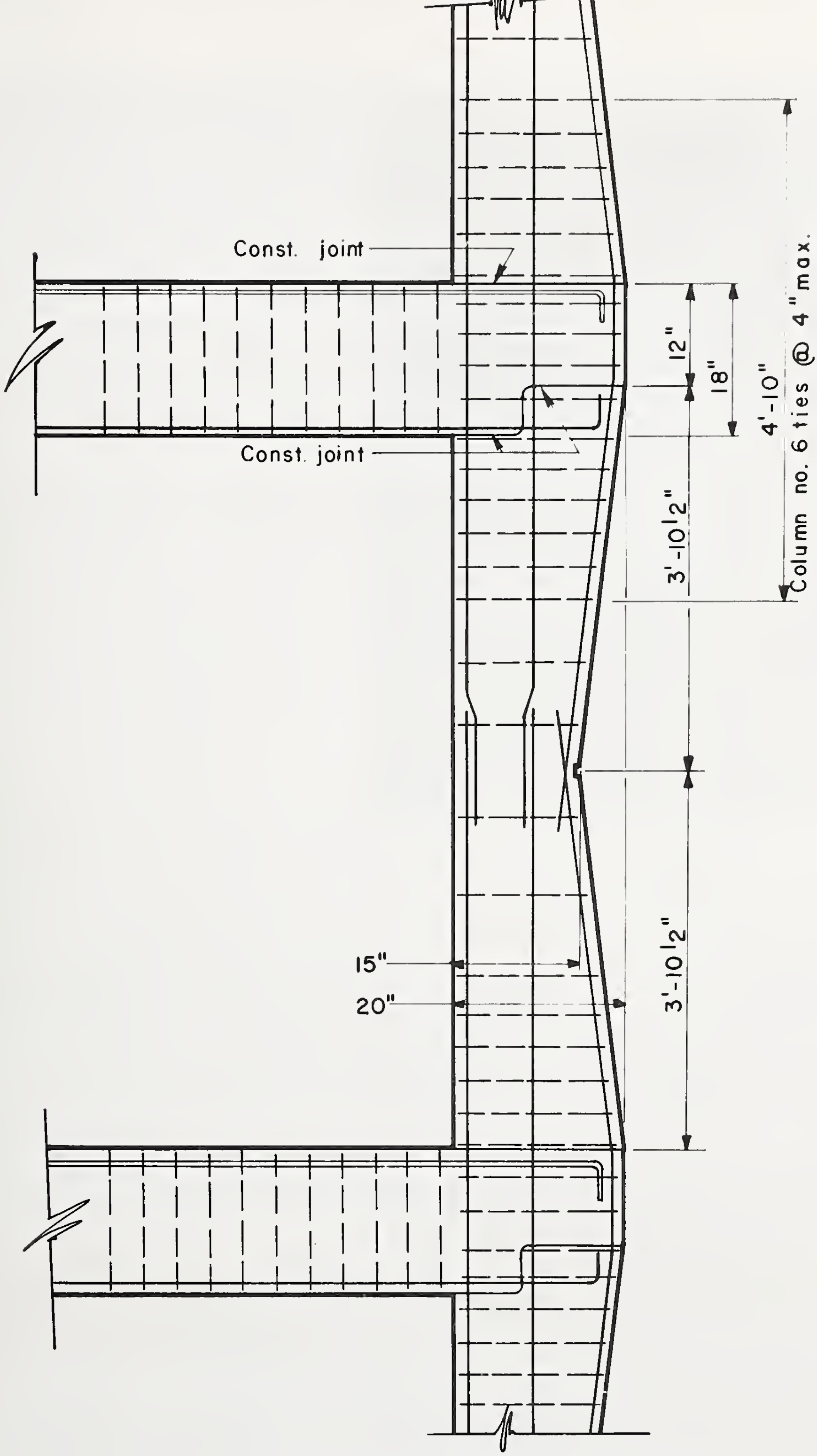



Figure 5.126b Typical framing detail.

Figure 5.127 Cracked tile mural.

Figure 5.128 Exterior view of Grants Department Store.

ledgers which are fastened to the wall by means of anchor bolts or drilled-in anchors. Usually these joists are supported on sheet metal hangers (fig. 5.129). Factory preassembled panels of 4 ft by 8 ft plywood sheet nailed to 2 in by 4 in wood members with metal hangers attached to the ends are placed between the joists. The edges of plywood are nailed to adjacent panels or to wooden ledgers attached to the exterior wall. With this framing system, the lateral support to the walls and the continuity of the roof panels is exclusively dependent upon the nailing of the plywood. There are no positive anchors at the wall to prevent it from pulling away from the roof.

When outward forces on the wall are generated by earthquakes, the tension load of the wall pulling away from the roof is resisted solely by a row of closely spaced nails pulling across the grain of the wooden ledger. Although the nailing stresses can be justified under the present building code, there is a tendency for the ledger to split as well as fail in cross bending caused by the eccentricity between the position of the anchor bolt and the top of the ledger where the plywood is attached.

Figure 5.129 shows a collapsed portion of the roof which spanned between the exterior brick wall and the interior beams. It is apparent from this photograph that the nailing through the roof panel did not have sufficient strength to accommodate lateral movement of the wall. Since the earthquake, the City of Los Angeles has proposed building code changes which would not permit the type of anchorage of masonry walls to the roof as described in this previous paragraph.

Other interior damage consisted of fallen suspended ceiling and merchandise thrown to the floor (fig. 5.130).

<u>Shopping Center at Glenoaks Boulevard and Hubbard Avenue, Map 3, Location 13</u>

A shopping center located at the corner of Glenoaks Boulevard and Hubbard Avenue, City of San Fernando, was so severely damaged that it was later razed. This center was located in the region of maximum ground movement. A post-earthquake survey of ground elevation indicated that the ground in his area was raised as much as about 5 ft (see fig. 8.23). Gas, water, and sewer lines were severely damaged in the immediate area (fig. 8.21).

Considerable ground movement was evident in the parking lot around the Boy's Market grocery store as shown in figure 5.131. The market experienced considerable damage to windows and structural elements. The construction was similar to

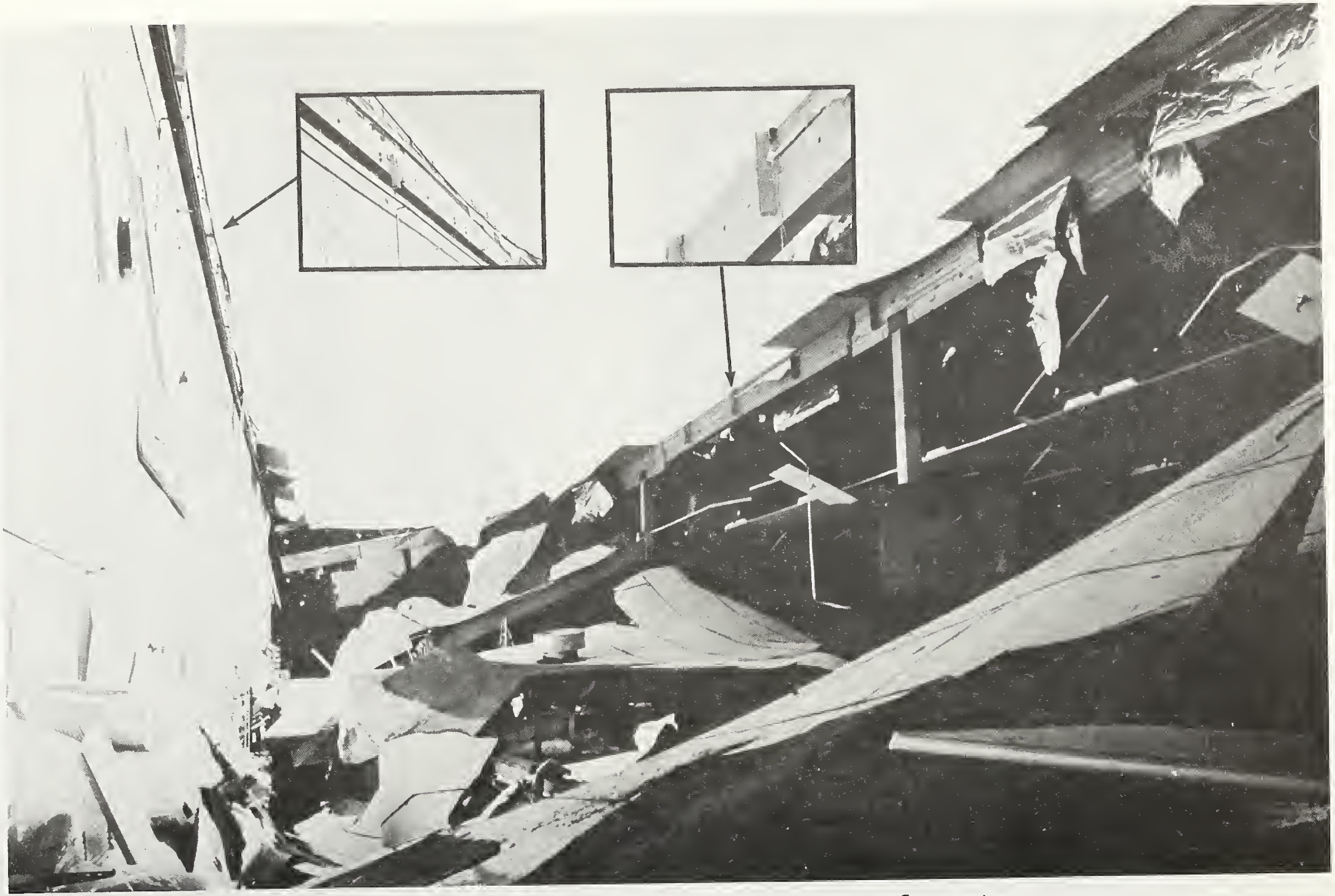


*Figure 5.129* *Collapsed roof structure at rear of Grants Department Store. Arrows indicate a wooden ledger fastened to the masonry wall (on the left) and metal beam hangers on the laminated wood girder (on the right).*

*Figure 5.130* *Interior view of Grants Department Store showing missing portions of suspended ceiling. Most of the merchandise which had been thrown to the floor had been picked up when this photograph was taken.*

*Figure 5.131* *East elevation of Boy's Market.*

Grant's Department Store (location 2) except damage was more severe. At this site the masonry wall supporting wood beams fell outward, dropping beams to the ground. The laminated wood girder which supported one end of the roof beams is visible in figure 5.132.

Another small store in the center was similarly damaged as shown in figure 5.133. Note that the reinforced brick masonry wall remained essentially intact even though it fell against an adjacent building. A view from the opposite end of the store is shown in figure 5.134. The laminated timber beam at the left spanned approximately 65 ft from a column at the front of the store to a rear masonry wall which also collapsed. Wood joists spanned from this beam to a wood ledger fastened to the wall on the right. Another view of this store is shown in figure 5.135.

The construction of the southern portion of the center shown in figure 5.136 differed from that described above. This portion had a concrete canopy supported on a light steel frame and was severely racked during the earthquake. The destructiveness of ground shaking can be envisaged by extensive cracking in the parking lot pavement. Yielding and fracture of one of the support columns is shown in figure 5.137.

San Fernando Mall, Map 3, Location 19

The San Fernando Mall, main business district in the City of San Fernando, is shown in figure 5.138. Most buildings were four stories or less in height and appeared to be built before 1930.

The hotel shown in figure 5.139 was severely damaged. The second story and roof are held up by interior partitions. One of the walls fell through the roof of the adjacent store causing severe interior damage (fig. 5.140). Fallen parapets, walls, and bricks were common as shown in figures 5.141, 5.142, and 5.143. The Kress store was severely damaged as shown in figures 5.144, 5.145, and 5.146. The diagonally cracked wall of Kress shown in figure 5.146 was common in other buildings in the mall as shown in figure 5.147.

This damage, though severe, was not unexpected considering the type of construction. A similarly constructed building, the Midnight Mission, as far away as downtown Los Angeles was damaged as shown in figure 5.148. At this latter location one man was killed when he ran outside and was struck by falling masonry. Such falling objects are common and dangerous with the pre-1933 masonry construction. It is certainly not typical of modern reinforced masonry construction,

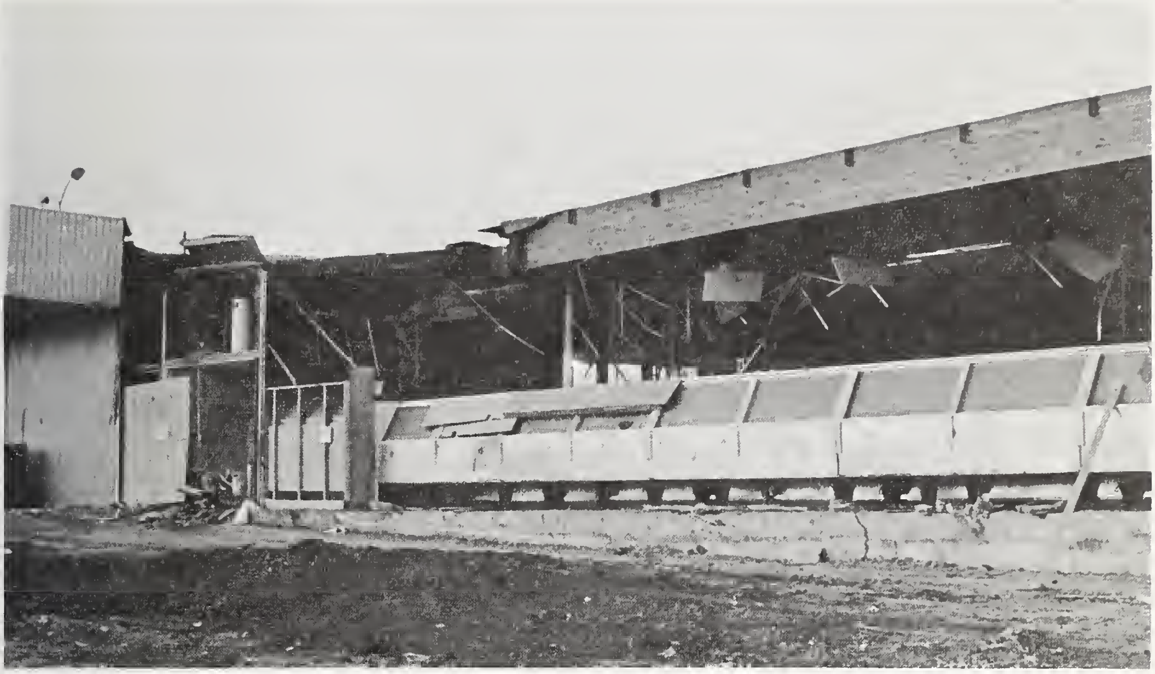


*Figure 5.132 View of the Boy's Market looking east. Most debris has been cleared.*

*Figure 5.133 Collapsed side wall of a small store in the shopping center.*

*Figure 5.134 Collapsed side wall shown in figure 5.133 as viewed from the rear of the store.*

*Figure 5.135 Another view of the store shown in figures 5.133 and 5.134.*

*Figure 5.136 Southern portion of the shopping center.*

*Figure 5.137 Yielded and fractured canopy support.*


*Figure 5.138 Main business district (San Fernando Mall) in City of San Fernando.*

*Figure 5.139 Total collapse of brick masonry wall surrounding three-story wood frame building (Mission City Hotel).*

*Figure 5.140 The wall of the building shown in figure 5.139 fell through the roof of this building.*

*Figure 5.141 Damaged parapet and wall.*


*Figure 5.142  Damage to brick masonry parapet wall.*

*Figure 5.143  Collapsed wall and parapet.*


*Figure 5.144 Front view of Kress Department Store.*

*Figure 5.145 Closeup of damaged brick wall.*


Figure 5.146 Diagonally cracked wall at Kress.

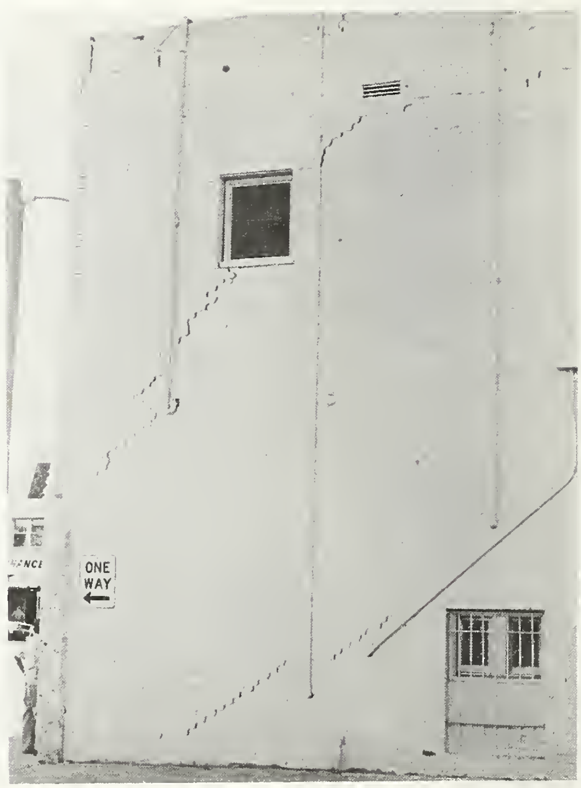



Figure 5.147 Typical diagonally cracked wall.

such as the Golden State Medical Center at the Pacoima Hospital, described previously, and the Sportsmen's Lodge Hotel, which is described later.

## Foothill Medical Center, Map 3, Location 21

The Foothill Medical Center (see fig. 5.149 and fig. 5.150) is located on Van Nuys Boulevard at Dronfield Avenue, Pacoima. This structural steel frame building (about 65 ft by 200 ft in plan) had considerable glass breakage as shown in figure 5.151. There was also some damage to the stucco covered wood-frame exterior walls as shown in figures 5.151 and 5.152.

The wallboard had been removed near the entrance to examine the lateral bracing (fig. 5.153). While the bracing showed no evidence of damage, the exposure revealed local flange buckling of the steel column as indicated by the arrow. There was considerable damage to interior partitions and stairwells (fig. 5.154).

The reinforced concrete masonry wall shown in figure 5.155 was on two sides of the center parking lot. The wall reinforcement did not always extend to the top of the wall and reinforced cells were not always grouted. Usually blocks above the reinforcing cut off were toppled (indicated by arrows). Walls were usually leaning when reinforced cells were not grouted.

## Sportsmen's Lodge Hotel, Map 4, Location 30

The nine-year old Sportsmen's Lodge Hotel shown in figure 5.156 is located on Ventura Boulevard near Coldwater Canyon Avenue. The complex consists of two buildings, one two-stories (60 ft x 170 ft) and one five-stories (68 ft x 240 ft) high. Construction is reinforced concrete masonry bearing wall and precast reinforced lightweight concrete slabs. This structure had only minor damage consisting of some cracked plaster and one broken window.

# RESIDENCES

Essentially all of the dwellings in Southern California including large two- and three-story apartment complexes are wood frame construction. Most of the single family residences are predominantly one-story structures, although some are two-story. Roofs are wood rafters covered either with plywood or one-inch thick boards. Roof coverings are varied and


*Figure 5.148* *Midnight Mission, 396 South Los Angeles Street, in downtown Los Angeles. This building, erected in 1896, provided shelter for about 200 men. One man was killed when he ran out of the front door and was struck by falling masonry. (Los Angeles Times Photo).*

*Figure 5.149* *General view of Foothill Medical Center*

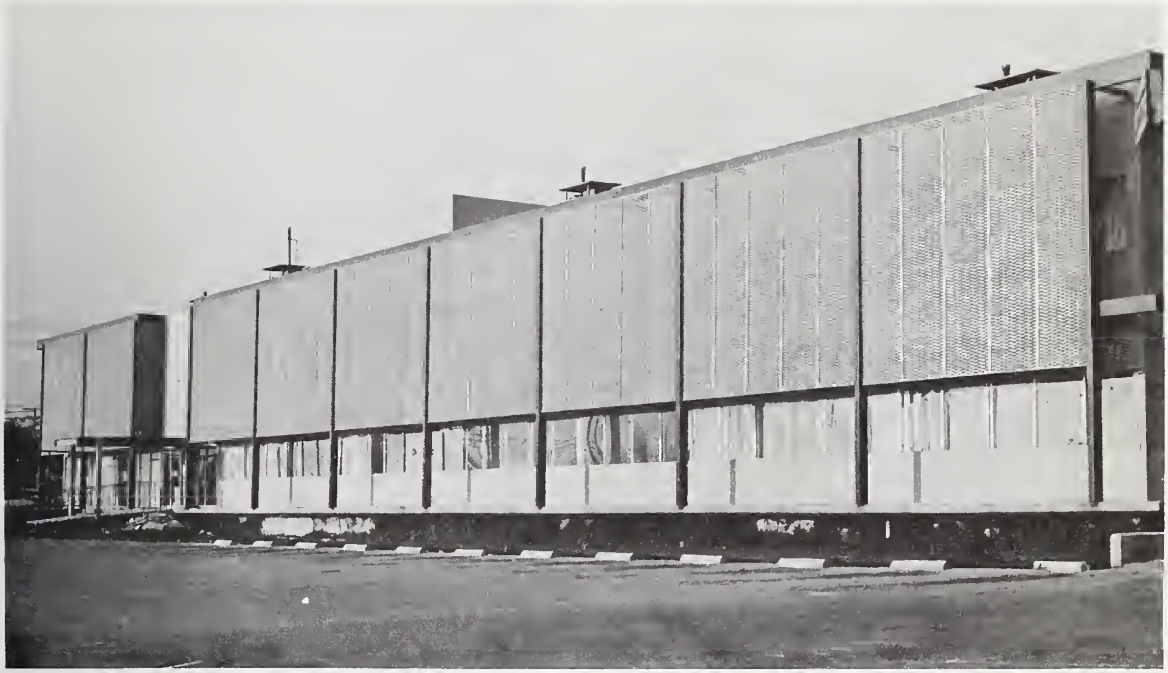


*Figure 5.150 West elevation of Foothill Medical Center, Van Nuys Boulevard at Dronfield Avenue, Pacoima.*

*Figure 5.151 East elevation showing glass breakage and cracked exterior walls.*

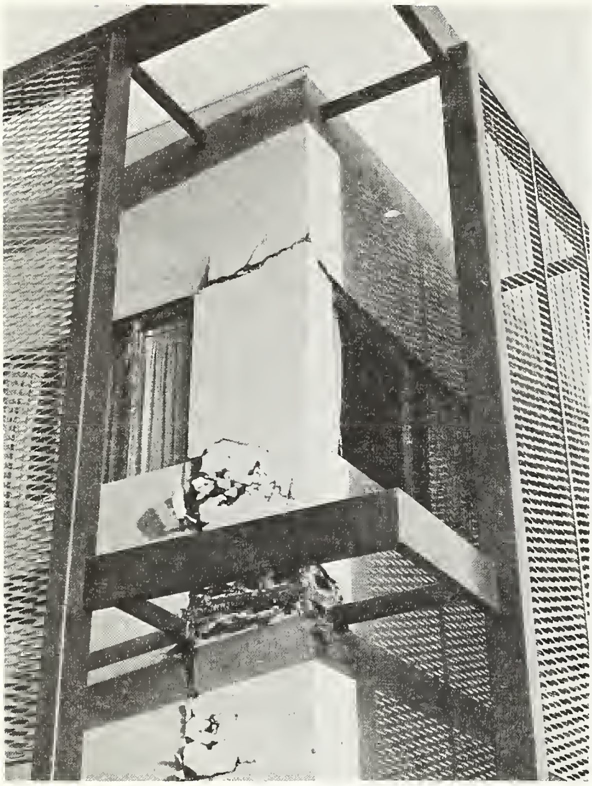


*Figure 5.152 Corner damage to the stucco covered wood-frame exterior walls.*

*Figure 5.153 Interior view of lateral bracing. The arrows locates local flange buckling at the bottom of the steel column.*

*Figure 5.154 Wallboard cracking in stairwell. The cracking pattern reflects the size of the wallboard.*

*Figure 5.155 Reinforced concrete masonry wall around the Medical Center parking lot. Arrows indicate the location of vertical reinforcement.*

include built-up asphalt impregnated asbestos material, asphalt shingles, wood shingles or shakes, and clay tile, depending on the slope of roof and cost of the structure. Walls are constructed with 2 in by 4 in wood studs spaced 16 inches apart and braced at each corner and at 25 feet intervals with a diagonal brace. The exterior of the walls are covered with cement plaster, wood siding, or plywood while the interior surfaces and ceilings are generally finished with plaster on gypsum board lath or with gypsum board "drywall". Floors are either wood joists with plywood or concrete slab on grade, and the buildings are generally supported on spread foundations.

Buildings ranging in cost from modest to very expensive employ the above construction with minor variations. Most dwellings are constructed in accordance with specification code criteria and are therefore not substantiated by engineering computations, unless there are very large rooms (over 25 feet clear span) or other unusual construction. Normally the rooms are small with enough cross partitions to "brace themselves."

Fireplaces installed in higher priced homes are usually masonry with grout and nominal reinforcement placed between the flue lining and the outer masonry shell. The chimneys are usually anchored at the roof or upper-story ceiling.

Split-level houses frequently collapsed as in the case of the house shown in figure 5.157 (map 3, location 5). This house was similar in appearance to the one shown in figure 5.158 which was not damaged. Other damaged split level houses (map 3, location 6) are shown in figures 5.159, 5.160, and 5.161. A similar house located slightly farther south (map 3, location 12) is shown in figure 5.162. In general, split-level or irregularly-shaped houses that were not adequately tied at the changes in roof or floor elevation were severely damaged either by collapse or at the junction of the wings (fig. 5.161). Collapse frequently occurred due to a wide open-space provided by the garage door in the end wall, which offered little lateral resistance (fig. 5.162).

Masonry chimneys on many residences collapsed as shown shown in figure 5.163 (map 3, location 10). Leaning chimneys were not uncommon as shown in figure 5.164 (map 3, location 11) when they were properly reinforced but inadequately tied to the house. Note the collapsed masonry fence in front of the house.

Because of their "self bracing" effect, residences usually suffered only nominal damage except at locations close to the zone of ground rupture. The houses shown in figures 5.165, 5.166, and 5.167 (map 3, location 15) are

*Figure 5.156 Aerial view of Sportsmen's Lodge Hotel.*

*Figure 5.157 Collapsed split-level residence at Fenton Avenue and Tyler Street.*

Figure 5.158 Undamaged split-level residence at Fenton Avenue and Tyler Street.

Figure 5.159 Collapsed split-level residence at Almetz Street and Leedy Avenue.

*Figure 5.160 Damaged split-level residence at Aldergrove Street and Leedy Avenue.*

*Figure 5.161 Closeup of damaged split-level residence shown in figure 5.160.*

*Figure 5.162 Split-level residence, Sylmar. Note crushed car (arrow). (Los Angeles Times Photo).*

*Figure 5.163 Damaged reinforced brick masonry chimney (arrows indicate reinforcement).*

*Figure 5.164 Leaning reinforced brick masonry chimney.*

*Figure 5.165 Wood-frame and stucco house on 8th Street, San Fernando. Horizontal cracking occurred along the top of the foundation wall. Note ground cracking in left foreground.*


*Figure 5.166 Damaged single family residence.*

*Figure 5.167 Destroyed wood-frame house.*

typical examples of the damage which occurred near severe ground rupture.

One group of partially completed residences located east of and adjacent to the San Fernando Veterans Hospital (map 3, location 8) was heavily damaged as shown in figure 5.168. The frames and roofs were essentially complete at the time of the earthquake. The walls of the houses on the left were covered with wire mesh and paper backing which was to be covered with stucco (in a few cases decorative plywood panels were also installed). This covering was essentially the only difference from the houses on the right which collapsed. It appears that a very nominal amount of material furnished considerable lateral support. (The house in the right rear background which did not collapse was apparently no different than those which did collapse. The reason it remained standing is not known.) A partially collapsed house is shown in figure 5.169.

Apartments differ little in construction from houses. The "Friday USA" Apartments (map 3, location 14) suffered severe damage as shown in figure 5.170 due to little lateral resistance provided by large open-garage areas in the first floor. The Mardette Apartments (map 3, location 16) were similarly damaged as shown in figures 5.171 and 5.172. The wide open parking areas and entranceways provide little lateral resistance as in the case of split-level houses previously discussed.

Typical damaged mobile homes are shown in figure 5.173 (map 2, location 1) and figure 5.174 (map 2, location 3). These homes are usually supported on small concrete or metal base supports. Metal curtain walls around the base were usually damaged when trailers moved sideways (there is little to resist lateral movement since these homes are usually not attached to the base supports). Frequently considerable damage was caused by the supports subsequently punching through the floor.

A major change to building code requirements relating to residential dwellings does not appear to be warranted. However, making proper use of the inherent strength of the structure and providing adequate ties at corners and chimneys would improve its behavior. Connections and details should be carefully considered by the designer and should be clearly shown on the drawings; it should not be left to the discretion of the builder.

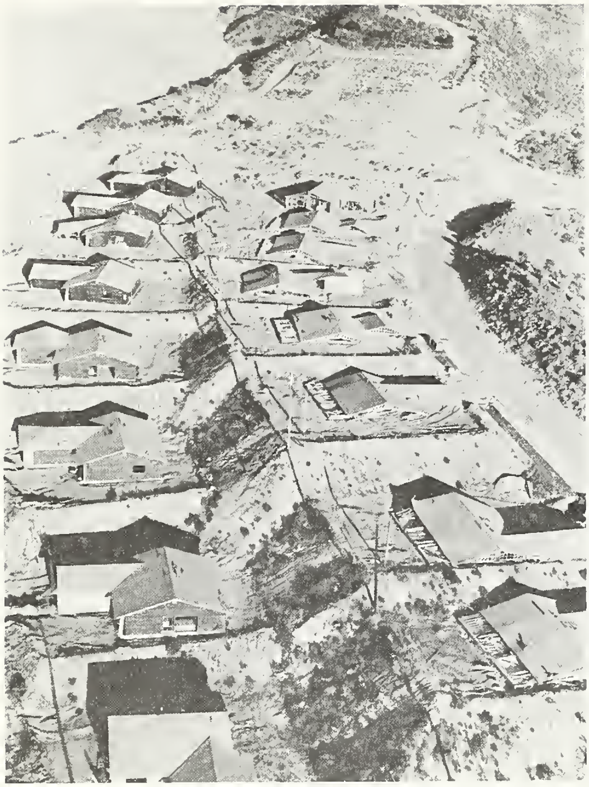


*Figure 5.168 Housing development under construction.*

*Figure 5.169 Partially collapsed houses under construction.*


*Figure 5.170 Rear view of damaged three-story "Friday USA" apartment building.*

*Figure 5.171 Rear view of damaged two-story Mardette apartment building.*


*Figure 5.172 Front view of Mardette apartment building.*

*Figure 5.173 Mobile home (Soledad Canyon Road, Newhall) which was damaged when it fell off its small concrete and metal base supports.*

## MISCELLANEOUS STRUCTURES

### Nethercutt Museum, Map 3, Location 9

The Nethercutt Museum for antique cars, which is located on Bledsoe Street near Bradley Avenue, Sylmar, is a reinforced concrete structure approximately 80 ft by 100 ft in plan and 110 ft in height (fig. 5.175). The 3-story building was designed in accordance with City of Los Angeles Building Code (K=1.33). The first story is 36 ft in height and has a partial mezzanine. The second- and third-story heights are 22 ft. The exterior concrete walls which extend up beyond the roof level are 8-in thick. Eight-inch two-way floor slabs span to 4-ft wide by 2-ft deep floor beams which are carried on columns spaced about 26-ft apart. The foundation consists of drilled concrete piles.

Damage noted on the building exterior consisted primarily of horizontal cracks and concrete spalling along the construction joints (fig. 5.176 and 5.177) and at the building corners (fig. 5.178). The penthouse free-standing cantilever concrete walls were only 12 hours old at the time of the earthquake but suffered no damage.

A severely damaged light frame store located diagonally across the corner from the museum is shown in figure 5.179.

### San Fernando Industrial Park, Map 3, Location 20

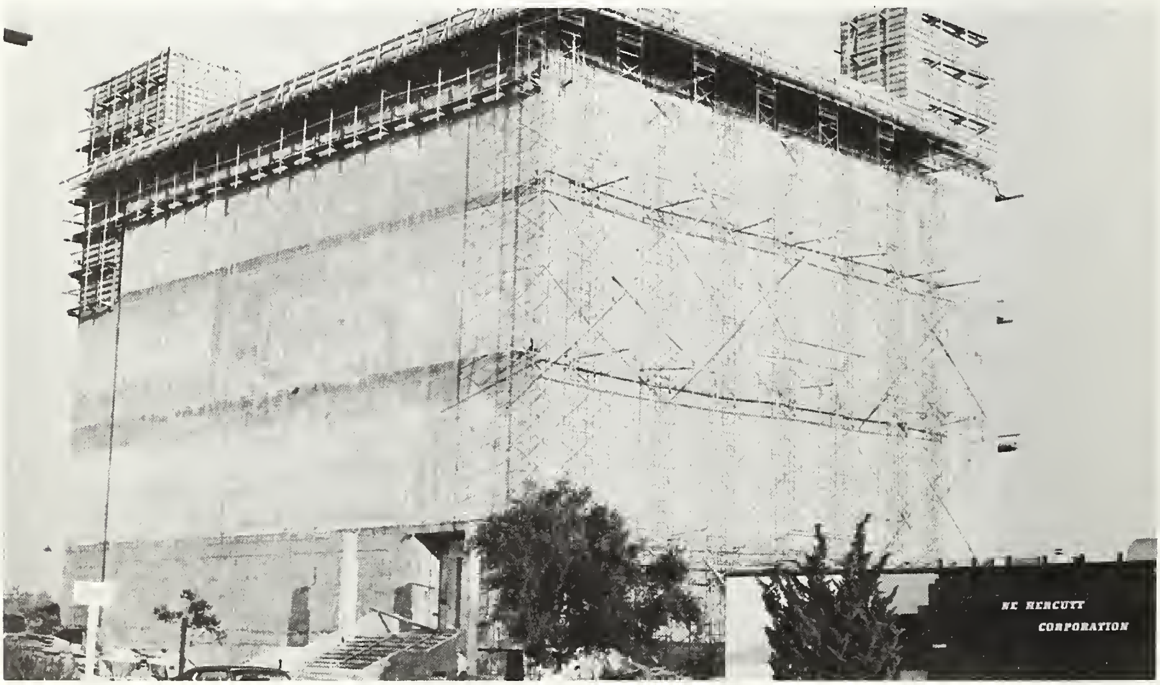
This industrial park comprised of some 30 small industrial buildings located at the northeast corner of the City of San Fernando. The complex was located in the region of strong ground motion with evidence of vertical ground displacement of as much as three feet. The primary cause of damage to many buildings, therefore, can be attributed to permanent ground displacement and faulting (see figs. 5.180 and 5.181).

The buildings are single-story structures with wood "panelized roofs". The exterior walls are either reinforced grouted masonry or tilt-up concrete panels. The floors are concrete slab-on-grade either at the street level or at truck loading level. The buildings are supported on spread footings.

The type of construction of most buildings in this industrial park is similar to that of the Grant's Department Store, described previously, thus similar types of failures occurred. Structural damage to the buildings in the industrial park consisted of portions of exterior wall pulling

*Figure 5.174 Mobile home park located along Sierra Highway 14 in Newhall. At the center of the figure are the remains of a home which was completely destroyed by fire. Note damaged metal curtain walls around the bases of the mobile homes (arrows).*

*Figure 5.175 Elevation of Nethercutt Museum.*



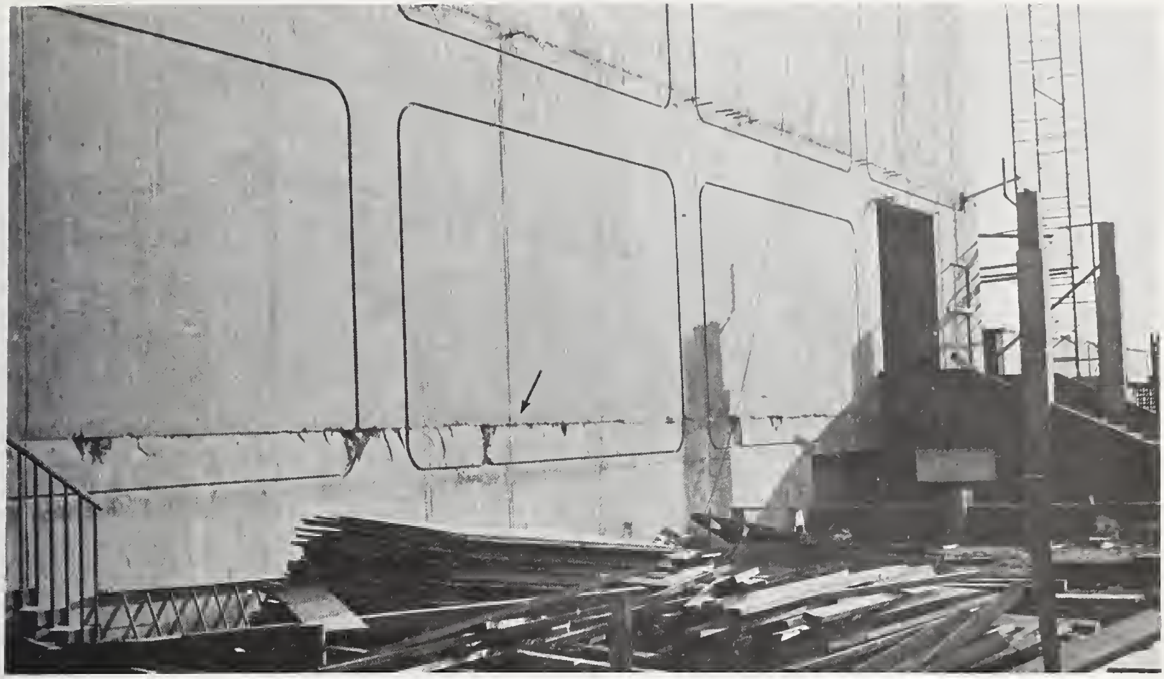


*Figure 5.176 Horizontal crack along the construction joint at the base of the structure.*

*Figure 5.177 Damage to the exterior surface of the concrete wall along the construction joint.*

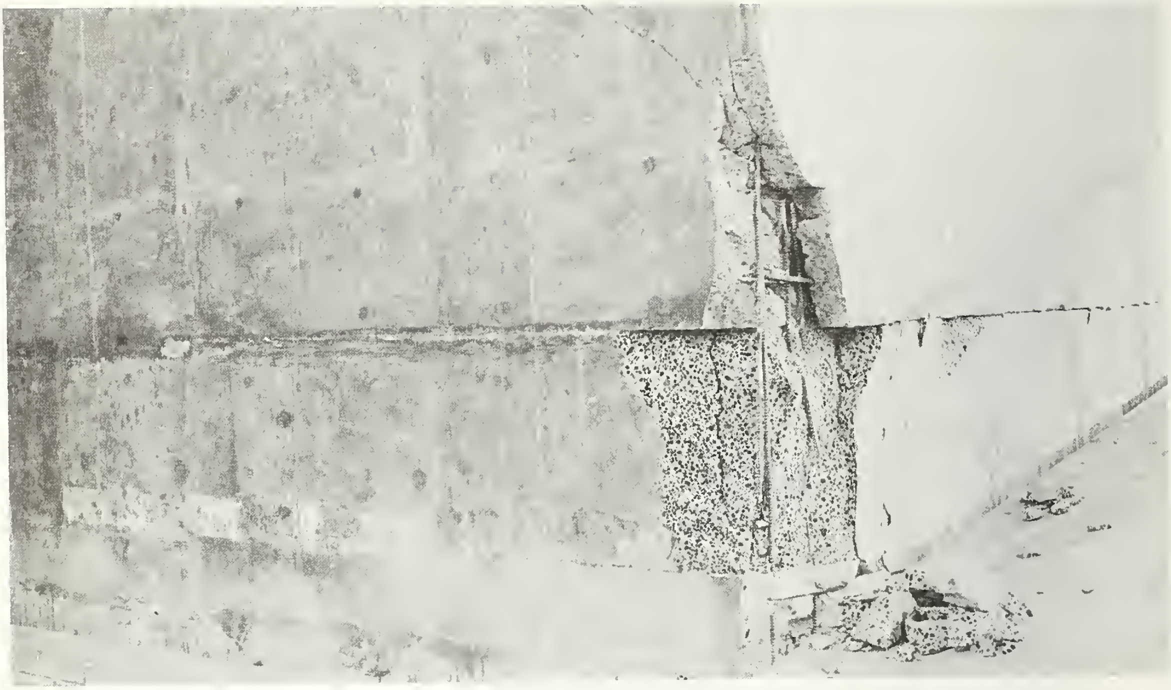


*Figure 5.178 Cracking and spalling at first floor Corner of the Nethercutt Museum.*

*Figure 5.179 This distorted light-frame store is located near the Nethercutt Museum.*


*Figure 5.180  Ground faulting caused separation of the concrete slab and steps.*

*Figure 5.181  Ground displacement caused separation of two walls.*

away from the roof and floor, thereby causing the roof to collapse. A typical failure is shown in figure 5.182.

The performance of drilled-in anchors can be inferior to cast-in anchors or to anchor bolts in resisting tensile forces since the strength is dependent on the quality of installation. Typical evidence of failure of drilled-in anchors is seen in figure 5.183. The drilled-in anchors in the wall, which were used to connect reinforcing dowels at the floor level, failed thereby contributing to the exterior wall collapse.

Because the buildings in this industrial park were located in the region of severe ground movement and faulting, the level of acceleration might have been considerably greater than the code design criteria. Consequently severe structural damage would be expected. However, it appears that the addition of positive tension anchors between the wall and framing members would significantly increase earthquake resistance and could have reduced the extent of damage.

Parking Structure in West Los Angeles, Map 5, Location 32

This five-level structure is located at 2215 Purdue Avenue in West Los Angeles, some 25 miles away from the epicentral region. Based on acceleration measurements taken at neighboring structures, it can be estimated that peak ground accelerations at this location could have been about 0.17g in the horizontal direction and 0.06g in the vertical direction.

An overall view of the structure is shown in figure 5.184. The structure is about 105 ft wide and 147 ft long. It is of steel frame construction with concrete end shear walls and diagonal bracings to resist lateral forces. The floor framing is comprised of steel joists supported on steel beams. The three-inch concrete slab was poured directly on corrugated sheet-metal decks which rest on the steel joists.

The structure was designed in accordance with the 1961 Los Angeles City Building Code using a horizontal force factor "K" of 1.33. It was designed to support a design dead load of 38 lb per sq ft and a design live load of 50 lb per sq ft. At the time of the earthquake the building was not occupied by cars.

Damage to the structure was confined to web crippling of some of main floor beams to which diagonal braces were attached. Positions of damaged beams with respect to diagonal braces and a close-up view of a damaged beam are

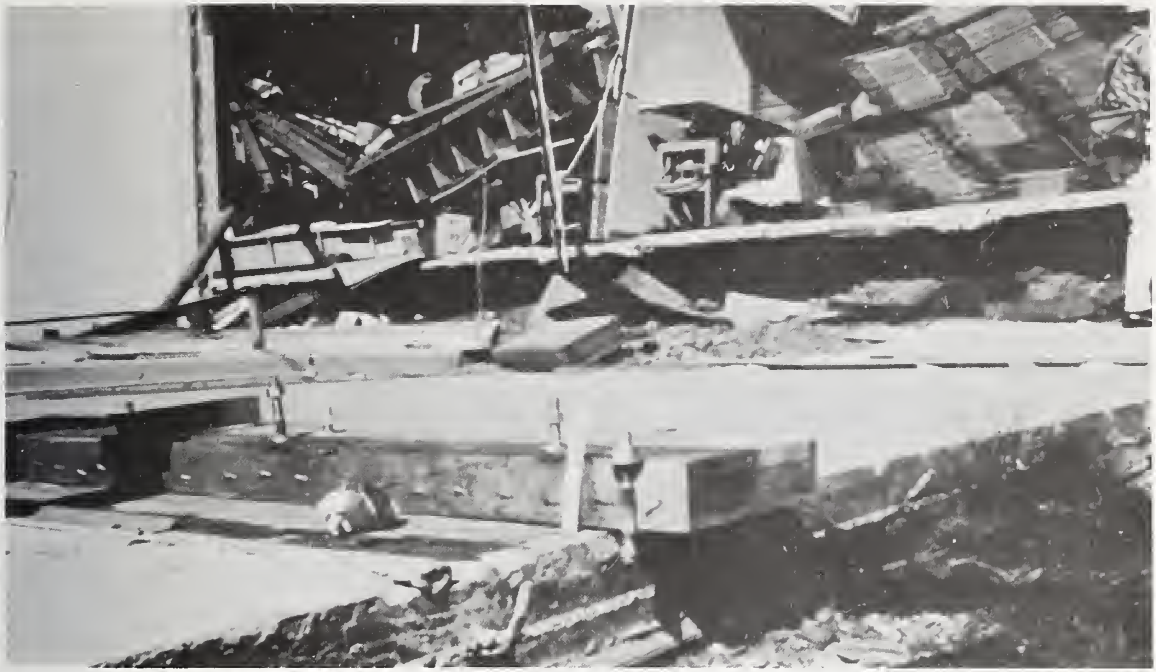


*Figure 5.182 The roof collapsed when the exterior wall pulled away.*

*Figure 5.183 Typical failure of drilled-in anchors.*

shown in figures 5.185 and 5.186. It is clear from these photographs that no vertical stiffeners were provided at the support. Figure 5.187 shows a typical repaired beam. Two vertical stiffeners were added to the beam and bottom half of the crippled web and the flange was replaced with a new section over the support. Initial installation of vertical stiffeners at a cost far less than the repair cost could have prevented the web from crippling.

POSTAL FACILITIES

There are over thirty postal facilities in the San Fernando Valley. These are generally one-story high (in at least one case leased "mobile homes") of numerous construction types. Damage was usually a function of the age of the structure more than any other single factor. The most important findings were damages to nonstructural elements. The following information was abstracted from a report to the Assistant Postmaster General, Facilities Department, U. S. Postal Service.

Suspended light fixtures were the most common source of injury, primarily by breaking loose at the fixture connections. Tiles frequently fell from acoustical tile ceilings. Battery-operated emergency lights were generally of little value, usually smashing to the floor and leaving the buildings in darkness. In cases where lights did not fall there were frequently an insufficient number of lights or beams were aimed too high to light the floors and exits. With the loss of power, exit signs and lights were useless.

Fire extinguishers were sometimes thrown from their brackets or the bracket pulled loose from its support. In some cases fire extinguishers were not in an area where emergency lighting existed.

## 5.3 References

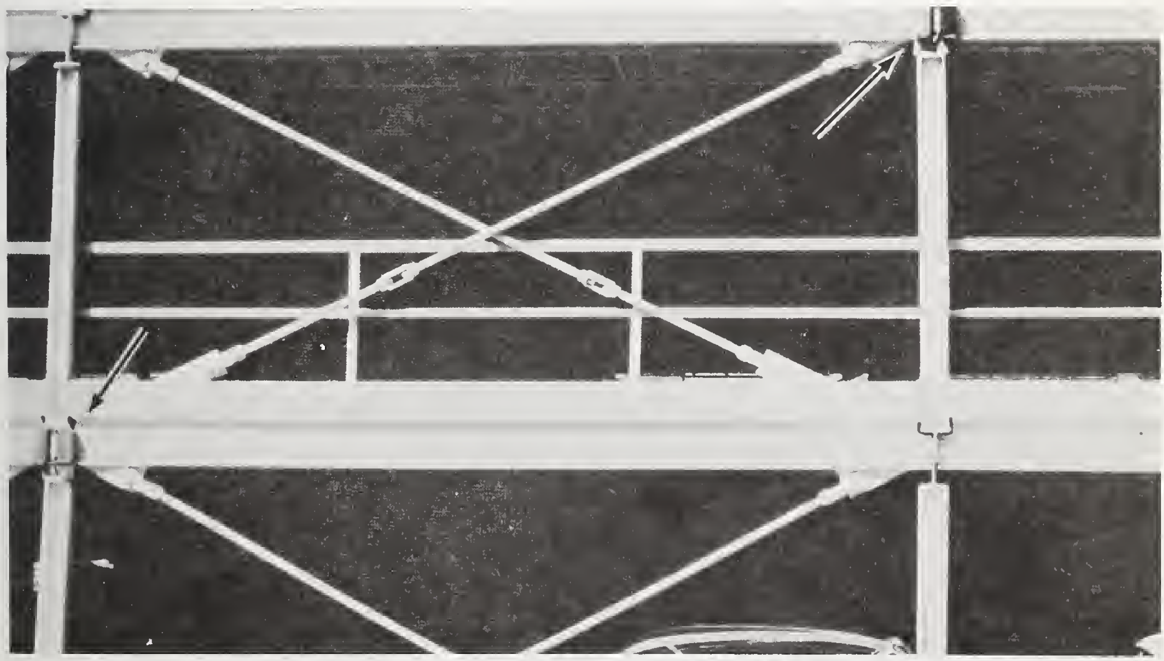

5.1 The Prince William Sound Alaska Earthquake of 1964 and Aftershocks, Volume II, Part A, Publication 10-3 (C&GS), U. S. Department of Commerce, 1967.

5.2 Structural Engineers Association of California, Recommended Lateral Force Requirements and Commentary, 1968.

5.3 Los Angeles County Building Laws, 1965 Edition, International Conference of Building Officials. Pasadena, California, 1965.

*Figure 5.184 General view of parking structure, located at 2215 Purdue Avenue in west Los Angeles.*

*Figure 5.185 Location of damaged floor beams (arrows).*



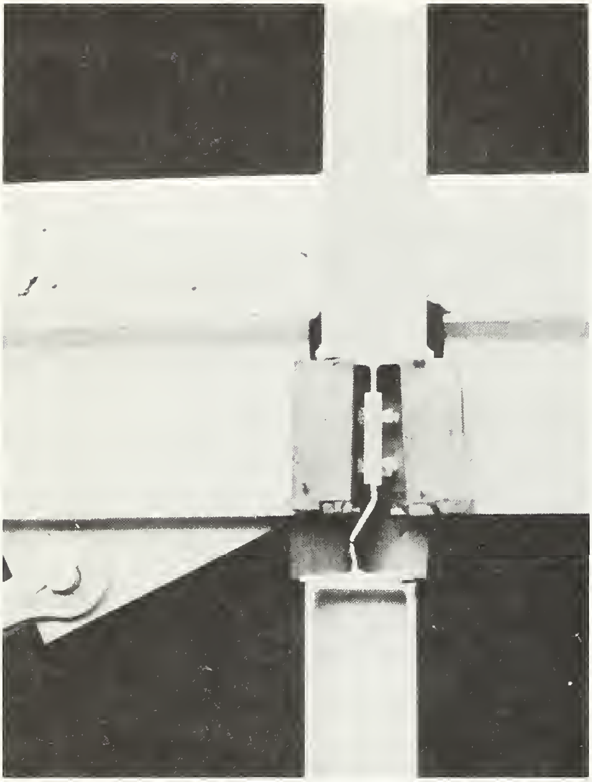
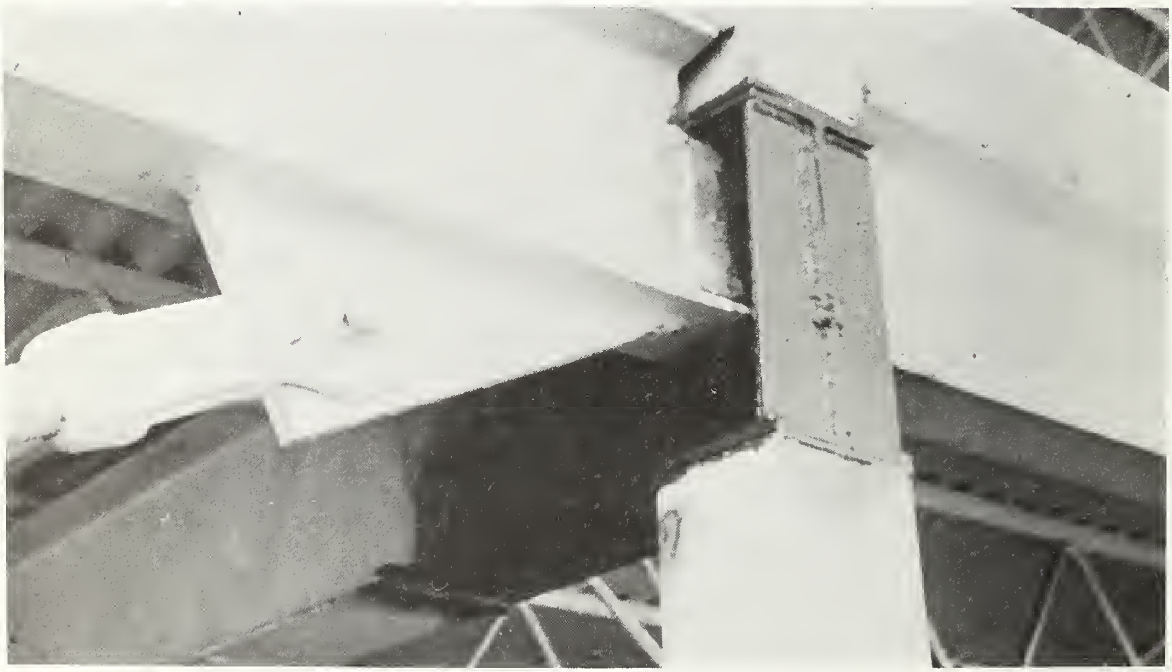


Figure 5.186 Close-up of damaged floor beam. Wood blocks were used to provide temporary support.

Figure 5.187 Repaired floor beam. The bottom half of the beam near the support was repaired with a new section and two bearing stiffeners were added.



5.4 Joint Committee on Lateral Forces, "Lateral Forces of Earthquake and Wind," ASCE Transactions, Vol. 117, 1952.

5.5 American Concrete Institute, Building Code Requirements for Reinforced Concrete (ACI 318-56), Detroit, Michigan, May, 1956.

5.6 American Concrete Institute, Building Code Requirements for Reinforced Concrete (ACI 318-63), Detroit, Michigan, June, 1963.

# CHAPTER 6 DAMAGE TO SCHOOLS

## 6.1 Introduction

The majority of the schools in Southern California are one- and two-story buildings with wood stud walls or reinforced grouted masonry walls and with wood or light steel roof construction. However, there are some buildings of every conceivable type of construction. There are also many temporary or portable classroom buildings, many on "substandard" foundations of wood sills and short cripple studs directly on the ground. At many school sites, the portable buildings moved on their support, requiring that they be reset.

There are approximately 200 public schools within the area subjected to strong ground motion (ten mile radius of San Fernando) and at least 25 on or within one mile of the zone of ground rupture. Although there was structural damage to modern school buildings (those constructed according to the requirements of the Field Act of 1933), only one such school which was within the zone of ground rupture was damaged to the extent that the school board considered abandoning it. The school did not collapse, and after the school board and their structural engineer reviewed the extent of damage and estimated cost of repairs, the engineer was commissioned to prepare plans for repairs. Many schools close to the zone of major disaster reported no damage whatsoever. In contrast, 13 older (pre-Field Act) buildings at 10 school sites in the Los Angeles Unified School District, as much as 20 miles from the central disaster area, were so seriously damaged that they were immediately vacated and scheduled for demolition.

## 6.2 Laws Governing School Construction

On the evening of March 10, 1933, the Long Beach-Compton area experienced a severe earthquake (Magnitude 6.3) which left most of the schools in shambles. Had the quake occurred during school hours, the loss of life and injury would have been devastating. The California Legislature took immediate action to avoid a future disaster and one month later, on April 10, 1933, adopted a bill authored by Assemblyman Don Field of Glendale, California to amend the Education Code.

The new law, usually referred to as the Field Act, as now amended provides the following:

1. All new buildings regardless of cost and additions

or alterations costing over $10,000 shall be constructed under the supervision of the Department of General Services.

2. The school board shall submit plans and specifications to the Department of General Services for review and obtain approval prior to awarding a contract for the work, and shall not pay for work not in accordance with the approval.

3. All plans and specifications shall be prepared by an architect or a structural engineer licensed by the state. The construction shall also be supervised by an architect or structural engineer.

4. The school board must provide a qualified full-time resident inspector who is satisfactory to the architect or structural engineer and to the Department of General Services.

5. Verified reports shall be submitted by the architect, engineer, contractor, and inspector certifying that all work complies with the approved documents.

6. The Department of General Services shall make such field inspections during the progress of the construction as in its judgement is necessary and proper to enforce the act.

7. The Department of General Services shall make rules and regulations (building code) to carry out the provisions of the act.

8. A procedure is established whereby school boards may have the Department of General Services examine old schools to determine if they are safe for occupancy.

9. Any violation of the Field Act or false statement on a verified report or affidavit is a felony.

The Department of General Services has delegated the enforcement of the Field Act to the State Architect who in turn delegated the enforcement, except for making the rules and regulations, to Chief Structural Engineer, Schoolhouse Section of the Office of Architecture and Construction. The Schoolhouse Section does, however, act in an advisory capacity to the Building Standards Commission in establishing the rules and regulations.

The code regulating the design and construction of schools is the Basic Building Regulations (California Administrative Code Title 24). For those sections of the Basic Building Regulations not yet finalized, sections of Chapter 1 of California Administrative Code Title 21 are still in use.

The requirements for schools set forth in Title 24 and Title 21 are somewhat more stringent than some of the local building codes. Some of the more restrictive requirements of Title 24 and Title 21 are:

1. Additional requirements for material testing and inspections.

2. Limitation of deflection, drift, and span-depth ratio.

3. Higher seismic forces for some buildings three stories or less in height.

4. Lower allowable masonry stresses.

5. Limitations of slope of grain for "machine stress rated" lumber.

6. Lower allowable stress for wood joists, rafters, and other members spaced two feet or less.

7. Greater minimum spacing of adjacent concrete reinforcing bars to reduce congestion.

8. Requirement that steel ties in concrete columns be spaced at half the normal required spacing (twice as many ties) within the top and bottom two feet.

9. Lower allowable capacity for shear connectors for composite (steel beam and concrete slab) construction.

10. Greater minimum thickness of light gage steel used for structural members.

However, the primary difference between school construction and other work is in the scrutiny given to details and connections, and in the added job site surveillance by the architect, engineer, inspector, and a field engineer of the Schoolhouse Section of the Office of Architecture and Construction.

Other sections of the Education Code also control the construction and occupancy of schools.

An article entitled Fitness of Buildings for Occupancy, as amended in 1968, provides for the examination of all pre-Field Act school buildings prior to January 1, 1970. Buildings found to be unsafe as defined in the Code must be reinforced or abandoned not later than June 30, 1975. In determining the safety of a building, the sole determination shall be protection of life and the prevention of personal injury at a level of safety equivalent to that established by the Field Act and the rules and regulations adopted thereunder, disregarding insofar as possible, such building damage not jeopardizing life which would be expected from one disturbance of nature of the intensity used for design purposes in said rules and regulations.

A Section relating to Investigation of Prospective School Sites, added to the statutes in 1967, requires a school district to have a proposed site investigated by competent personnel to ensure that all factors affecting public interest are included, not cost alone. "The investigation shall include such geological and engineering studies as will preclude siting of a school over or within a fault, on or below a slide area, or in any other location where the geological characteristics are such that the construction effort required to make the site safe for occupancy is economically unfeasible."

## 6.3 Case Histories of Individual School Damages

### SCHOOLS CONSTRUCTED BEFORE 1933

Los Angeles High School (Main Building) was constructed in 1917. The school is located approximately 20 miles southeast of the City of San Fernando (location 1, fig. 6.1). The building was a four-story concrete frame with unreinforced brick filler walls and had a 35-ft central tower which had a steel frame (fig. 6.2). The roof and classroom floors were wood framing and sheathing; the concrete frame was designed for gravity loads only; the exterior brick filler walls were unreinforced and laid in lime mortar; the partitions were hollow clay tile. The building was supported on spread foundations. A portion of the parapet fell through the roof of a teacher's work room and came to rest on a table (fig. 6.3). Much of the brickwork fell from the parapet directly onto the rear exit stairs and into the courtyard (fig. 6.4). There was serious cracking of the brick filler walls and the clay tile partitions. The building has been abandoned and was demolished.

Morningside School (Building #2) in the City of San Fernando was constructed in 1928. The school was located adjacent to the area of ground rupture (location 2, fig. 6.1).

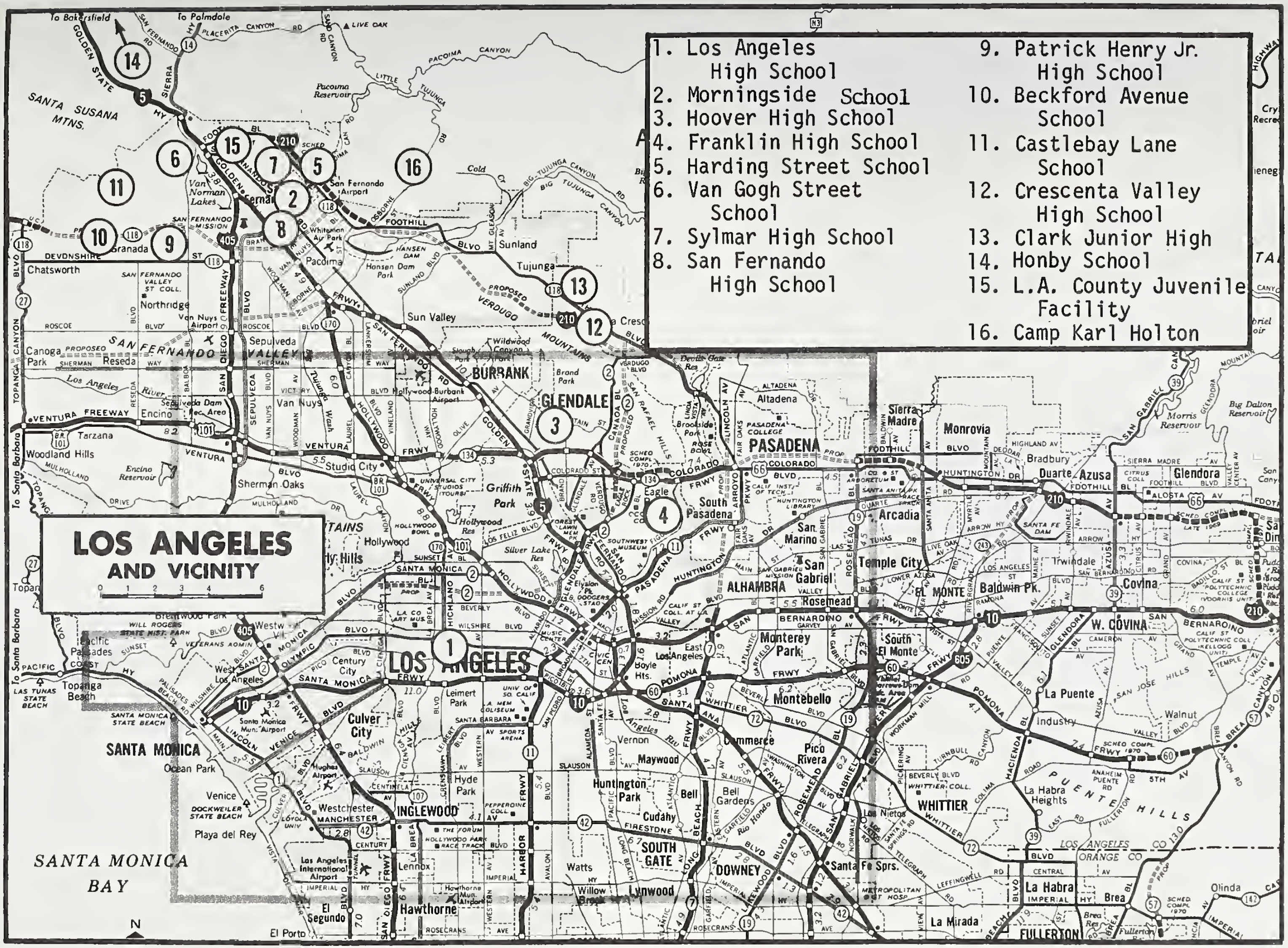



*Figure 6.1 Location of damaged schools.*

*Figure 6.2 Main building at Los Angeles High School before demolition.*

*Figure 6.3 A portion of the parapet wall fell through the roof and came to rest on a table, Los Angeles High School.*

The structure was a two-story double loaded corridor classroom building 60-ft wide and 100-ft long with unreinforced brick bearing walls laid in lime mortar. The structure had a wood framed gable roof covered with heavy slate roofing, wood framed floors in the classrooms, and concrete slabs in the corridor. The walls were unreinforced brick with a concrete bond beam at the roof. The building rested on spread footings. The structure did not collapse, but sustained serious cracking of the brick walls (fig. 6.5), brick separated at the bond beam and was displaced at the gable end (fig. 6.6). The building has been abandoned and was demolished.

## REINFORCED BUILDINGS - CONSTRUCTED PRIOR TO 1933 - REINFORCED TO RESIST EARTHQUAKE FORCES AFTER 1933

Morningside School (Building #1) in the City of San Fernando, was constructed in 1915 and is located adjacent to Building #2 previously described. The building is a two-story "T"-shaped concrete bearing wall structure (fig. 6.7). The original construction had a wood framed gabled roof with heavy slate roofing. Floor framing consisted of wood joists and sheathing. The exterior walls were concrete having an ultimate compressive strength of approximately 900 psi. The partitions were wood studs with plaster on wood strip lath. The building rests on spread foundations. In 1964, the building was reinforced by removing the heavy slate roofing, installing new plywood sheathing in the plane of the second-story ceiling, anchoring the floors and roof to the walls, installing a plywood shear-resisting element in the attic to stabilize the gabled roof framing in the longitudinal direction, and the installation of additional roof supports. New concrete shear-resisting wall panels were added since Title 21 considers concrete with a compressive strength below 1500 psi the same as unreinforced brickwork. There was nominal structural damage. The plywood sheathing on the longitudinal wall in the attic pulled loose from the studs, and some of the roof-to-wall anchors were damaged. Most of the interior plaster on wood strip lath was loosened. All the lath and plaster has been removed and replaced with drywall.

Hoover High School (Auditorium) in Glendale was constructed in 1930. The school is located approximately 14 miles southeast of the City of San Fernando (location 3, fig. 6.1). The original construction consisted of steel roof trusses and wood joists and sheathing supported on steel columns encased in exterior unreinforced brick walls laid in lime mortar. The interior was plastered directly on the brick. The floor system is wood framing and the building rests on spread foundations. In 1937, the building was reinforced by removing the wood roof joists and sheathing, the top three feet of the brick walls, and the outer face of brick on the

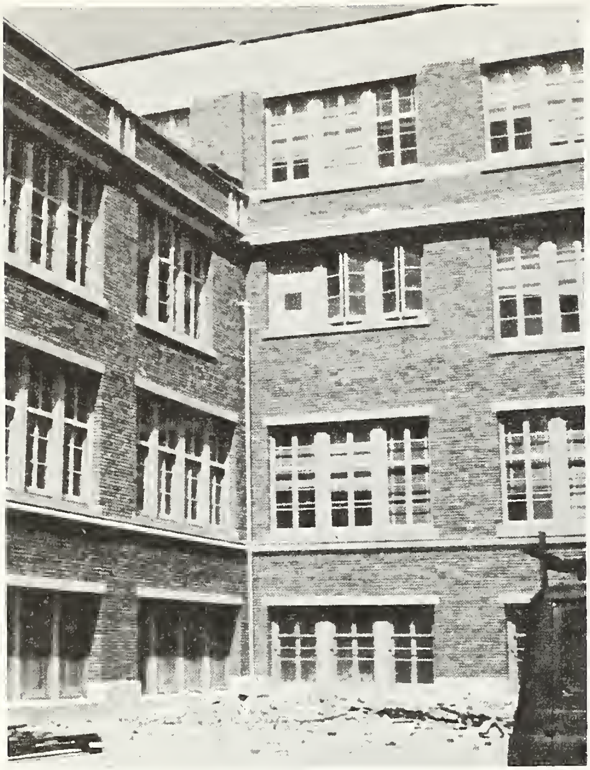


*Figure 6.4 Brickwork fell from the parapet into the courtyard, Los Angeles High School.*

*Figure 6.5 Brick walls sustained serious cracking, Morningside School Building #2.*

*Figure 6.6 Brick wall was displaced at the gable end, Morningside School Building #2.*

*Figure 6.7 Morningside School Building #1 was constructed in 1915 and strengthened to resist earthquake forces in 1964.*

walls. New steel roof joists were then installed. New reinforced concrete was cast against the remaining brick and a new concrete roof slab was cast.

There was no structural damage to this building. Some interior plaster, which was installed at the time of the original construction, flaked from the wall. Tapping the surface with a hammer indicated that the plaster is no longer bonded to the brick at many locations. Newer brick bearing wall structures at this school suffered only superficial damage, such as minor plaster cracks and displacement of structural separation joint covers. The entire school is occupied.

Franklin High School (Auditorium) in the Highland Park area of Los Angeles was constructed in 1928. The school is located approximately 19 miles southeast of the City of San Fernando (location 4, fig. 6.1). Original construction consisted of steel roof trusses with wood rafters and sheathing, wood ceiling joists, and metal lath and plaster. The walls were unreinforced brick bearing walls laid in lime mortar. The building rests on spread foundations. The building was reinforced in 1960 by removing the exterior face of brick from the wall, cutting chases for stiffening ribs, placing reinforcing steel near the wall and in the chases, and applying a blanket of gunite. The walls were tied to the roof construction and additional bracing was installed in the plane of the roof.

There was no apparent structural damage. Some of the decorative plaster on the ceiling, which was installed at the time of the original construction, has fallen. The ceiling has been replaced. Other buildings at this site sustained no apparent damage.

## SCHOOLS CONSTRUCTED AFTER 1933 IN ACCORDANCE WITH THE FIELD ACT

Harding Street School in the City of San Fernando was constructed in 1964. The school is in the area of ground rupture (location 5, fig. 6.1). The construction is one-story wood frame and stucco. The roof framing is comprised of wood joists and plywood sheathing. There are plywood shear-resisting elements, usually located in the walls. The floor is a concrete slab on grade and the building rests on spread footings.

There was minor structural damage to the walls and slab on grade. The slab cracked in two classrooms, presumably located directly above ground rupture, with a vertical displacement of approximately 3 1/2 in (fig. 6.8). The walls racked slightly causing some doors to bind in the steel frames (fig. 6.9). There was also plaster cracking. The two classrooms in which the floor was cracked were repaired. The entire school is occupied.

Van Gogh Street School, Granada Hills was constructed in 1968. The school is in an area of ground rupture directly west of the Upper Van Norman Reservoir (location 6, fig. 6.1). The construction is a single-story wood frame and stucco. The roof framing is comprised of wood joists and plywood sheathing. There are plywood shear-resisting elements, usually located in the walls. The floor is concrete slab on grade. The building rests on concrete spread footings. The lunch shelter is a wooden roof structure supported on 6-in square steel tube columns cantilevering from the foundation. Although there was no collapse, there was major structural damage which consisted of buckling of slabs on grade (fig. 6.10), fracturing of concrete foundation walls (fig. 6.11), and displacement of bearing walls in the zone of cracked foundations (fig. 6.12). The lunch shelter columns were permanently bent approximately 15 degrees at the base at the point of weakness created by openings for electrical outlet boxes (fig. 6.13). A cantilever concrete block garden wall leaned approximately ten degrees but remained standing (fig. 6.14). The underground utilities were severely damaged. The school has been repaired and is fully occupied.

Sylmar High School in Sylmar was constructed in 1961. The school is in the zone of earth rupture adjacent to the City of San Fernando (location 7, fig. 6.1). The school consisted of a complex of single-story wood frame and stucco buildings. The gymnasium is a concrete structure. The buildings have wood rafters and plywood sheathing, wood stud bearing walls with plywood shear-resisting elements. Floors are slab on grade except for the cafeteria kitchen which has a raised structural concrete slab and beam with intermediate concrete piers. The buildings rest on spread foundations. The gymnasium has a prestressed concrete folded plate roof, concrete exterior walls, concrete slab on grade and rests on drilled concrete friction piles. The covered arcades are wood roof structures supported on pipe columns cantilevering from the foundation.

Principle structural damage was caused by the differential stiffness of the buildings and the covered arcades. The arcades pulled loose both from the buildings and from each other and distorted their supporting pipe columns (fig. 6.15 and 6.16). There was a slight spalling of the gymnasium prestressed-concrete roof member where a steel cross beam was attached to support gymnastic equipment (fig. 6.17). The ground under the gymnasium subsided approximately 1 1/2 in leaving a void beneath the slab on grade adjacent to the

*Figure 6.8 Cracked slab with a vertical displacement of approximately 3 1/2 inches, Harding Street School.*

*Figure 6.9 Plaster cracks over door at Harding Street School. Racking of wall caused doors to bind in the steel frames.*

*Figure 6.10 Buckled slab on grade at Van Gogh Street School.*

*Figure 6.11 Fractured concrete foundation wall, Van Gogh Street School.*

*Figure 6.12 Displaced bearing wall, Van Gogh Street School.*

*Figure 6.13 Lunch shelter columns were permanently bent at opening for electrical outlet box, Van Gogh Street School.*

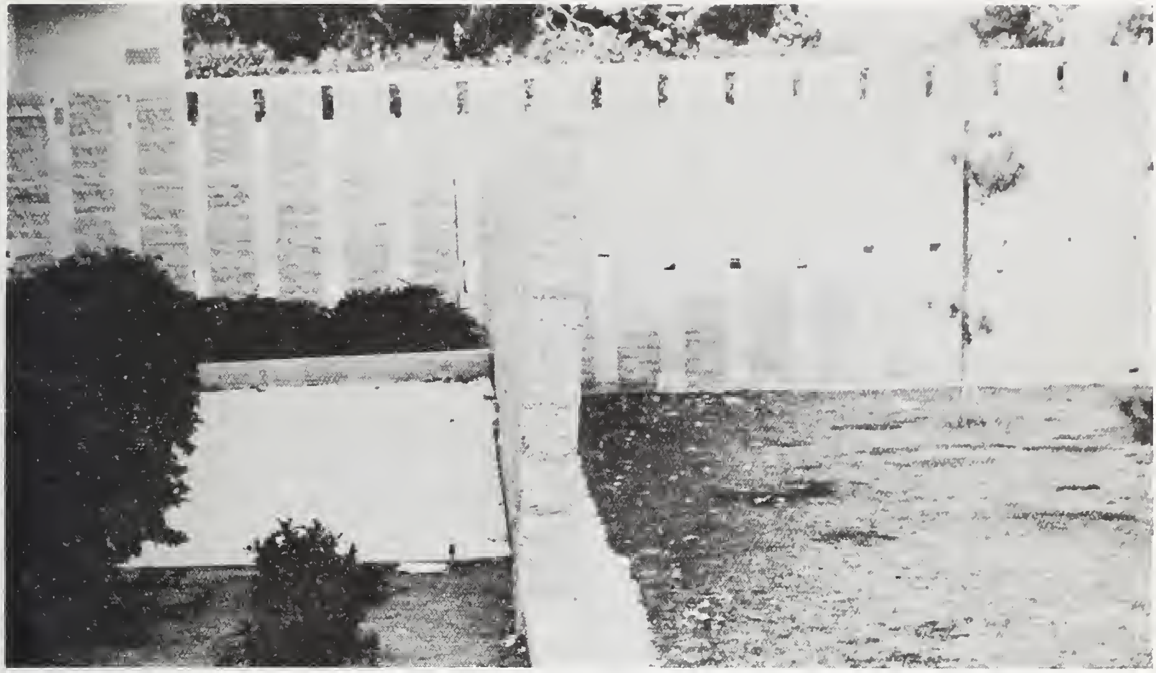
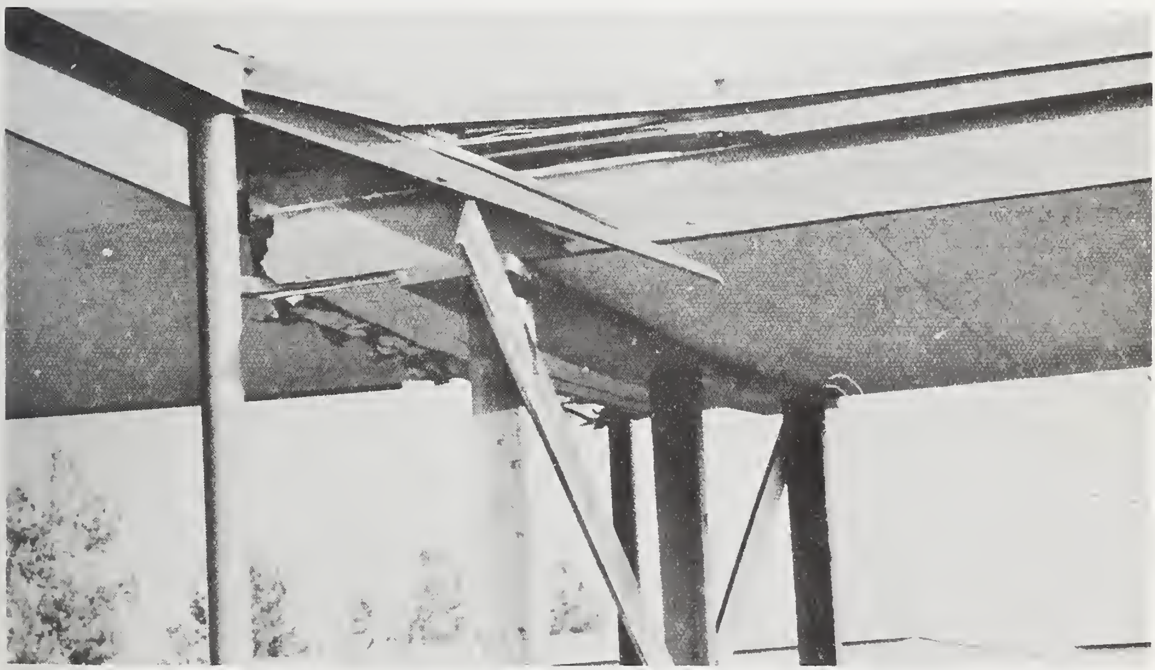


Figure 6.14 Cantilever concrete block garden wall leaned approximately ten degrees, Van Gogh Street School.

Figure 6.15 Arcades pulled loose from each other at Sylmar High School.



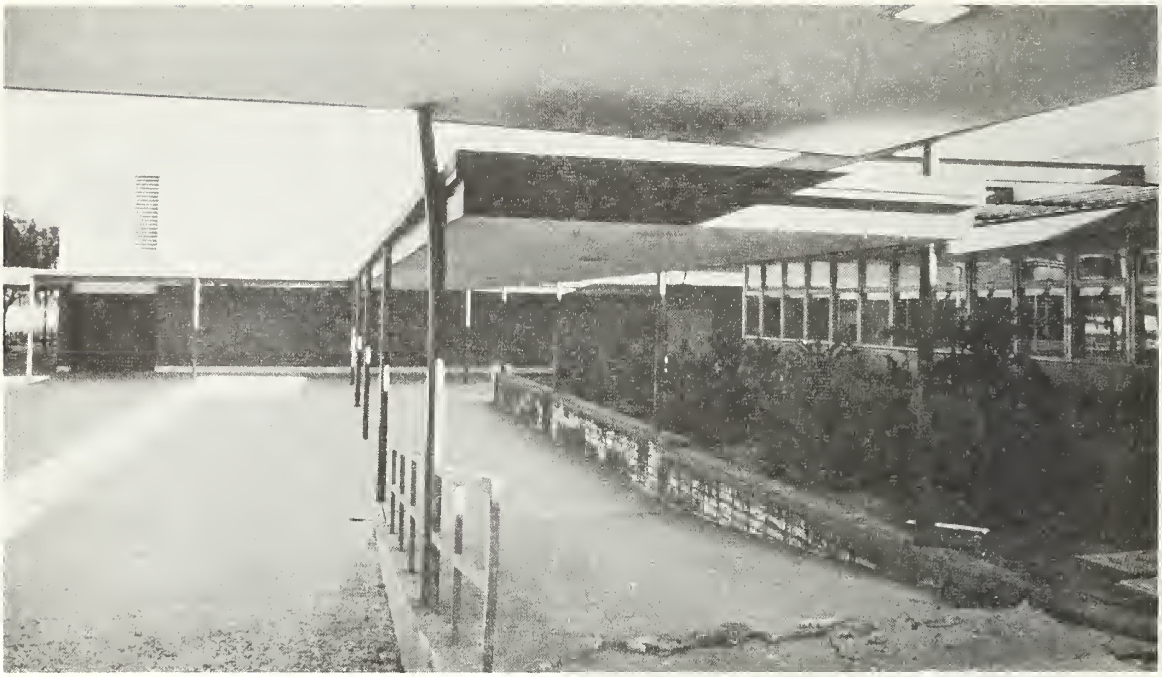
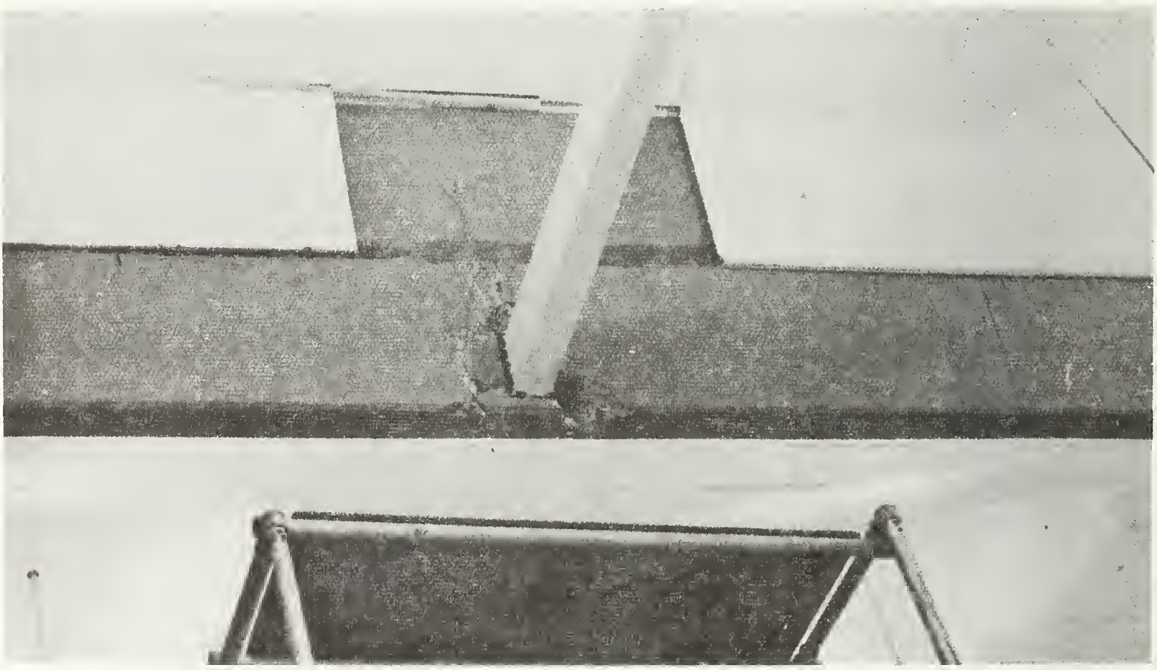


*Figure 6.16 Distorted pipe columns at arcade, Sylmar High School.*

*Figure 6.17 Spalled prestressed concrete roof member at gymnastic equipment support, Sylmar High School.*



walls. The floor slab sagged down toward the middle of the room. The gymnasium walls which rested on piles did not subside. Slabs on grade were extensively cracked and buckled (fig. 6.18 and 6.19). The interior supporting piers for the structural floor in the kitchen were severely damaged and shoring was required to provide for safe occupancy. A reinforced grouted brick transformer vault, constructed partially below grade, was extensively damaged. There was considerable damage to lighting fixtures, acoustical ceilings, and plaster. The school is presently occupied.

San Fernando High School in the City of San Fernando was constructed in several increments starting in about 1952. The school is located adjacent to the area of ground rupture (location 8, fig. 6.1). The buildings are primarily one-and two-story concrete bearing wall structures with the classroom wings having double loaded corridors. The roof has concrete beams and slabs except for the auditorium which has metal decking supported on steel trusses with a horizontal bracing system in the plane of the lower chord. The gymnasium has a steel sawtooth roof framing system with horizontal rod bracing in the plane of the lower chord. The walls are reinforced concrete and the floors are concrete beams and slabs. The buildings rest on spread footings. In addition there are two frame and stucco classroom buildings.

There was limited structural damage; in the gymnasium, the diagonal rods in the horizontal bracing system pulled out of clevises where oversized clevises were erroneously installed (fig. 6.20). In the auditorium several 5x3 angles in the horizontal bracing system buckled (fig. 6.21).

The administration building sustained water damage to the ceiling caused by a broken fire sprinkler pipe. The two-story classroom buildings suffered damage to plaster at the junction of the transverse partitions and the concrete exterior walls. Some of the cabinets along the transverse classroom partitions were displaced and racked. Several window panels in the sawtooth roof of the gymnasium fell onto the roof. The "T"-bar ceiling in the wood framed classrooms was damaged primarily at the perimeter; however, no lighting fixtures dropped. The extent of damage was minimal considering the school's close proximity to the earthquake.

Patrick Henry Junior High School in Granada Hills was constructed in 1959. The school is located approximately four miles southwest of the City of San Fernando and is approximately two miles southeast of the epicenter of the strong aftershock of March 31, 1971 (location 9, fig. 6.1). The buildings are primarily one- and two-story reinforced grouted brick and concrete bearing wall structures. The

Figure 6.18 Slabs on grade were extensively cracked and buckled, Sylmar High School.

Figure 6.19 Slabs on grade were extensively cracked and buckled, Sylmar High School.

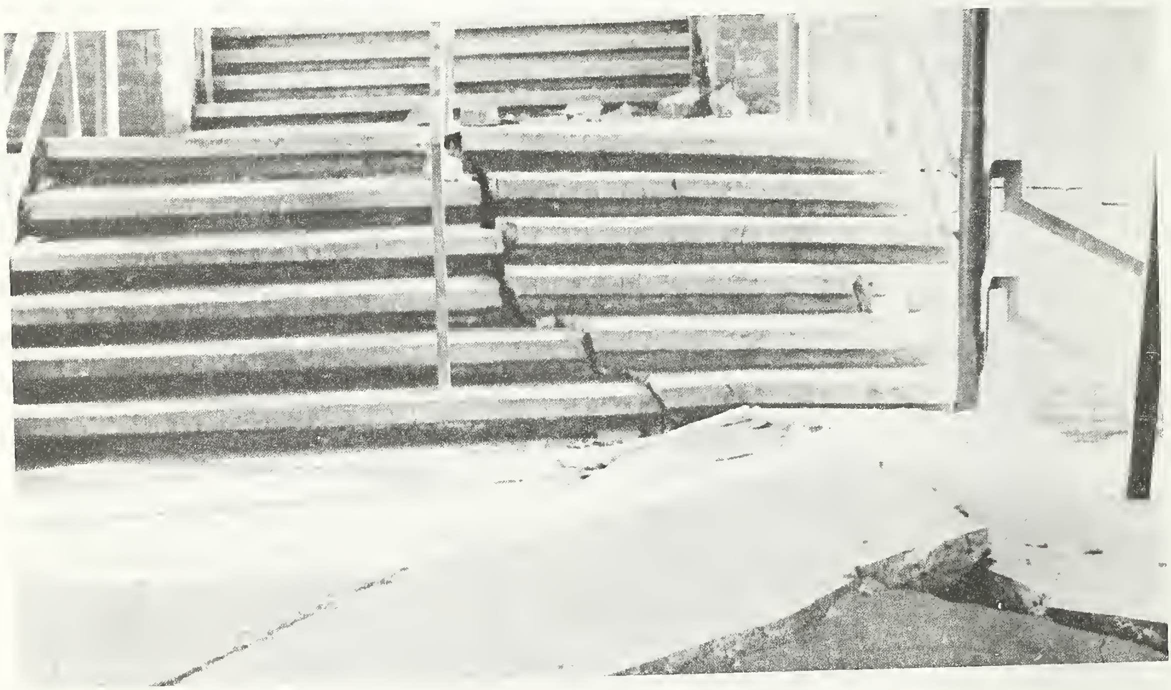


*Figure 6.20 Horizontal bracing rod pulled out of clevis, San Fernando High School.*

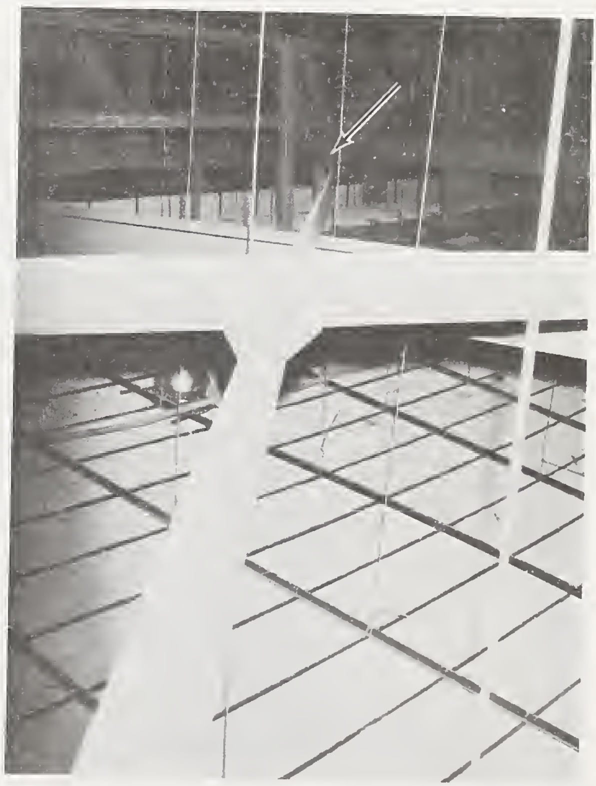


*Figure 6.21 Buckled 5 x 3 angle brace, San Fernando High School.*

classroom wings are constructed back-to-back (longitudinal wall at center of building with rooms on both sides) with an exit to an exterior walk or balcony on each side. The buildings have concrete roof and floor slabs (first floor is concrete slab on grade), and rest on spread footings. Mechanical penthouses on the roof are constructed with a braced structural steel frame and covered with corrugated cement-asbestos board.

The major structural damage consisted of fracturing of four second-story columns of a two-story covered arcade (fig. 6.22 and 6.23). The concrete roof of the arcade was shored to prevent collapse. The longitudinal concrete wall in the classroom buildings, extending the full length of the buildings sustained some hairline diagonal cracks near both ends of the building. The cracking can be attributed to shrinkage stresses that existed prior to the earthquake. A concrete curb at the corner of the mechanical penthouse fractured under the steel column at the brace connection (fig. 6.24 and 6.25). Structural damage was minimal; however, a major loss of lighting fixtures constituted a real hazard. Approximately 1500 lighting fixture stems failed at the ceiling canopy or at the fixture. All fixtures have been reinstalled. There was nominal damage to acoustical ceiling tile. Electrical conduits were damaged at structural separation joints where they were rigidly connected to both sides (fig. 6.26). The earthquake aftershock of March 31, 1971, caused little additional damage. There were a few new plaster cracks, a few damaged ceiling tiles, and some additional flaking of concrete on the previously damaged arcade columns. It is possible that the aftershock could have caused collapse of the arcade if it had not been shored. The damaged arcade has been removed. The entire school is in use.

<u>Beckford Avenue School</u> in Northridge was constructed in 1966. This school is located essentially over the epicenter of the March 31, 1971 aftershock and approximately 6 miles west of the City of San Fernando (location 10, fig. 6.1). The school is a single-story wood frame and stucco complex. The roof framing consists of wood joists and plywood sheathing. Ceilings are "T" bar with lay-in acoustical tile. Lateral bracing is provided by plywood shear-resisting elements. The floor is concrete slab on grade and the foundations are continuous footings. There are two temporary classroom buildings on wood mudsills resting on the asphalt pavement.

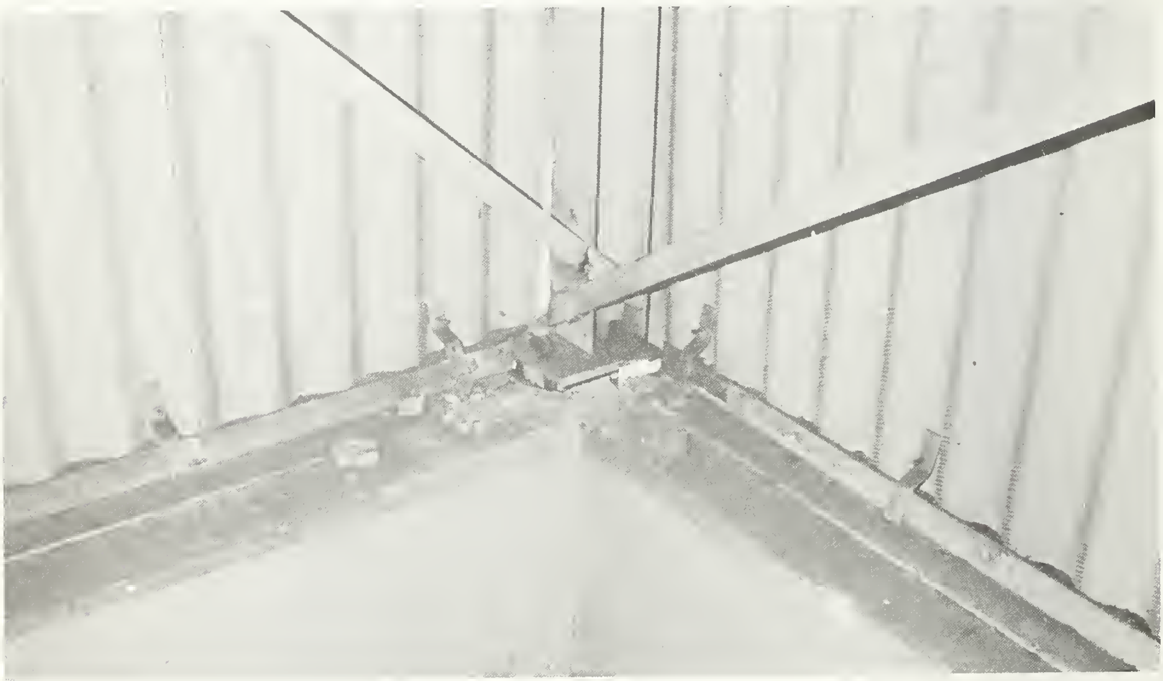
There was no damage caused by the main shock of February 9, 1971; however, the March 31, 1971, aftershock caused considerable nonstructural damage. Much of the ceiling tile and "T" bars at the perimeter of the rooms fell. Most of the diffuser lenses on the light fixtures fell. One

Figure 6.22 Fractured concrete at second-story covered arcade columns, Patrick Henry Jr. High School.

Figure 6.23 Close up of fractured concrete second-story covered arcade column, Patrick Henry Jr. High School.

*Figure 6.24* *Corner of mechanical penthouse fractured under steel column (exterior view), Patrick Henry Jr. High School.*

*Figure 6.25* *Corner of mechanical penthouse under steel column (interior view), Patrick Henry Jr. High School.*



of the portable classroom buildings moved approximately two inches. All damaged ceiling sections were removed and the school was reopened. There was extensive damage and collapse of chimneys and masonry garden walls in the residential area directly adjacent to and surrounding the school.

Castlebay Lane School in Northridge is still under construction but nearing completion. The school is located approximately two miles north of the epicenter of the March 31, 1971, aftershock and about six miles west of the City of San Fernando (location 11, fig. 6.1). The school is a two-story wood frame and stucco structure. The roof and second-floor framing consist of wood "Trus-Joists" and plywood sheathing. Lateral bracing is provided by plywood shear-resisting elements. The first floor is concrete on grade. The building rests on concrete spread footings. Exterior finish is stucco with some ceramic tile veneer. Interior walls are plastered on gypsum board lath. Classroom ceilings are acoustical plaster, corridors have "T"-bar acoustical ceilings.

There was no structural damage caused by either the earthquake of February 9, 1971, or the aftershock of March 31, 1971. However, at the time of the main shock some ceramic tile veneer fell from the wall (fig. 6.27). The interior plaster cracked at many locations, the "T"-bar ceiling was damaged, the structural separation joint covers distorted, and mechanical equipment located on the roof moved off the isolation mounts (fig. 6.28). Repairs were underway when the aftershock of March 31, 1971, occurred causing additional ceramic veneer to fall from the wall, damage to corridor "T"-bar ceilings which had already been repaired (fig. 6.29) and extensive cracking of interior plaster at the joints of the gypsum board lath (fig. 6.30).

Crescenta Valley High School (Administration Classroom Building) in La Crescenta was constructed in 1955. The school is located approximately twelve miles southeast of the City of San Fernando (location 12, fig. 6.1). The building is a three-story concrete bearing wall, double loaded corridor, classroom structure with a one-story and basement administrative office wing at the north end (fig. 6.31). The roof and upper-story floors are concrete beams and slabs, the lower floor is a concrete slab on grade. The building is supported on spread footings. There was major cracking of the piers in the north wall of the administration wing (fig. 6.32), which is a continuation of the three-story shear wall of the classroom section. Since there are only minor cracks in the three-story portion of the wall, it is difficult to attribute the pier cracking to horizontal forces; it appears that the damage was caused primarily by vertical movement of the administration wing relative to the main classroom area.

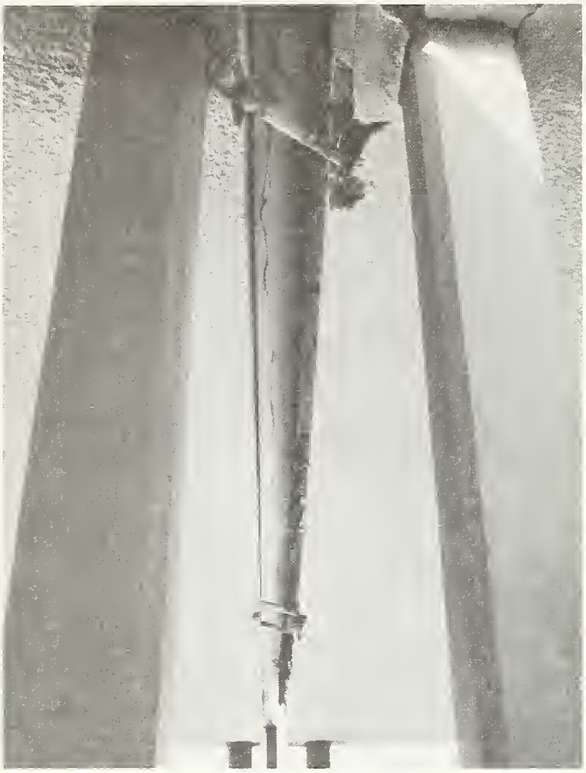
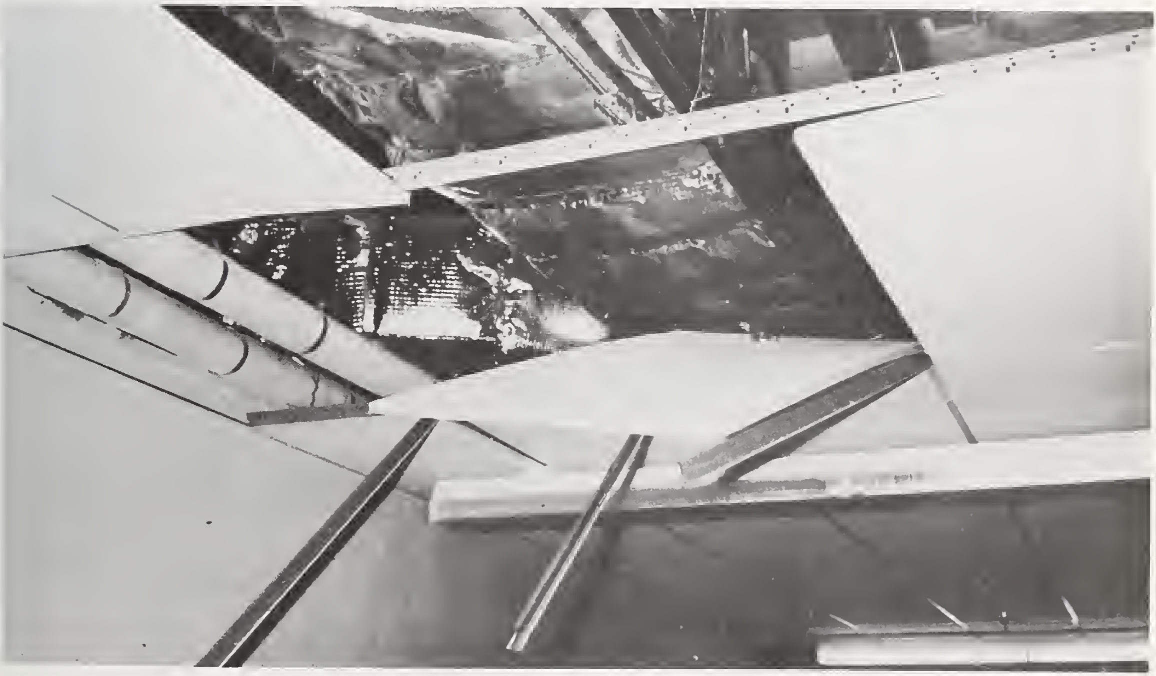


*Figure 6.26 Electrical conduits were damaged at structural separation joints, Patrick Henry Jr. High School.*

*Figure 6.27 Ceramic tile fell from wall, Castlebay Lane School.*

*Figure 6.28 Mechanical equipment moved off the isolation mounts, Castlebay Lane School.*

*Figure 6.29 "T" bar ceilings were damaged, Castlebay Lane School.*



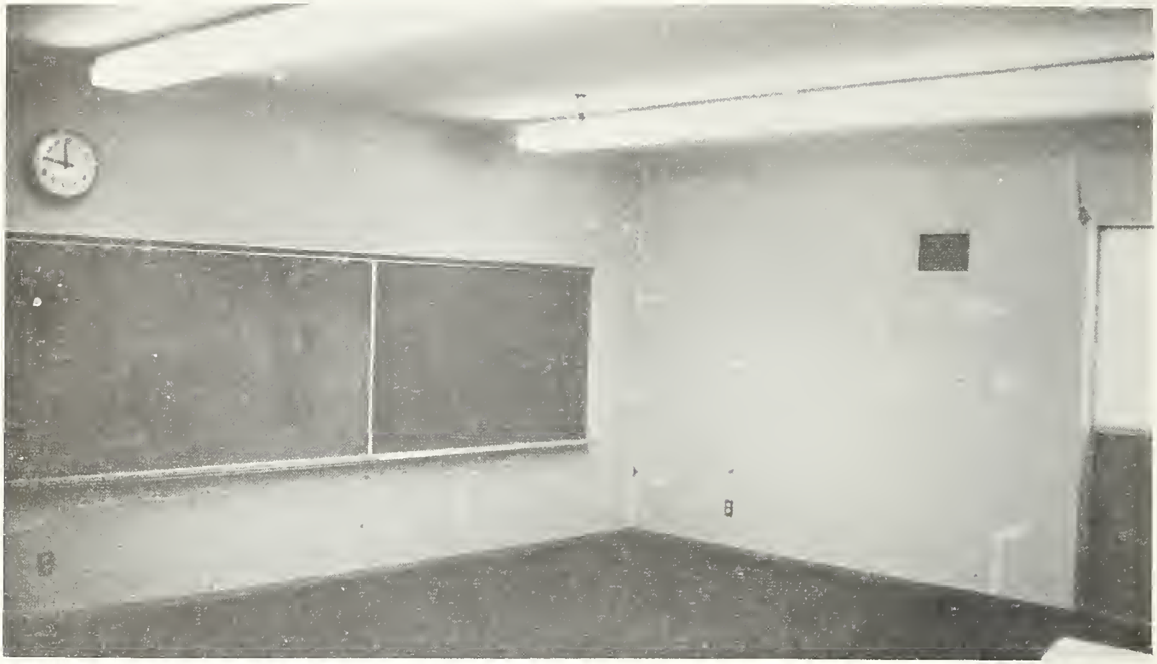
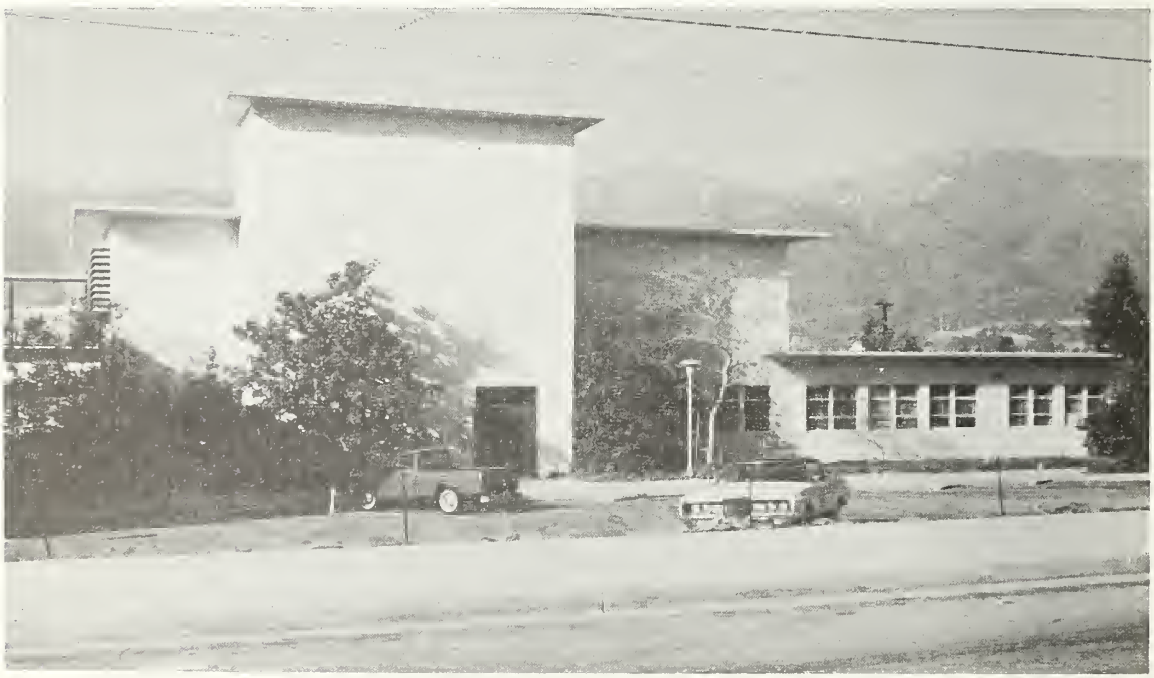


*Figure 6.30 Plaster cracks at joints of gypsum board lath, Castlebay Lane School.*

*Figure 6.31 North elevation of Crescenta Valley High School.*



There are also transverse cracks near the middle and near the south end of the classroom portion. There was considerable light fixture damage; in addition, many ventilating louvers weighing about 40 pounds pulled loose from the corridor wall above the exit doors and fell to the floor (fig. 6.33). There was only minimal damage reported on the rest of the school site. However, the glass in the enclosure surrounding the exit stairwell of the gymnasium shattered and fell into the stairwell and could have been a serious hazard. The entire school is in use.

Clark Junior High School in La Crescenta was constructed in 1961. The school is located approximately eleven miles southeast of the City of San Fernando (location 13, fig. 6.1). The school has two-story brick bearing wall, double loaded, corridor classroom buildings. The roofs are poured gypsum on light steel framing; the second floor is concrete slab and joist; and the first floor is a concrete slab on grade. The upper-story intermediate transverse shear-resisting elements are flat-steel-bar diagonal braces concealed in the transverse partitions. The end transverse shear elements in the upper story and all of the elements in the lower story are reinforced grouted brick masonry. The buildings rest on spread footings. The major structural damage was caused by excessive elongation of the steel diagonal braces (fig. 6.34). This permitted the gypsum roof diaphragm to deflect excessively and thus greatly increased the bending stresses near the middle and the shearing stresses at the ends of the roof diaphragm. Cracks were noted in the roof diaphragm. The increased shears at the ends of the roof diaphragm also caused some minor cracking of the end brick shear walls. There was extensive nonstructural damage including damage of lighting fixtures (fig. 6.35) and cracked plaster partitions due to excessive drift of the top story. The school is in use.

Honby School in Saugus was constructed in 1967. The school is located in a zone of ground rupture approximately ten miles northwest of the City of San Fernando (location 14, fig. 6.1). The school consists of a single-story group of reinforced grouted brick bearing wall, hexagonal shaped buildings. The roof construction is wood joists and plywood sheathing; the walls are reinforced grouted brick with a concrete bond beam at the top; the floor is a concrete slab on grade and the buildings rest on concrete spread foundations. There was visible cracking in the concrete bond beam directly over the cracks in the ground surface and vertically displaced slabs on grade. No other damage was reported, and the school is fully occupied.

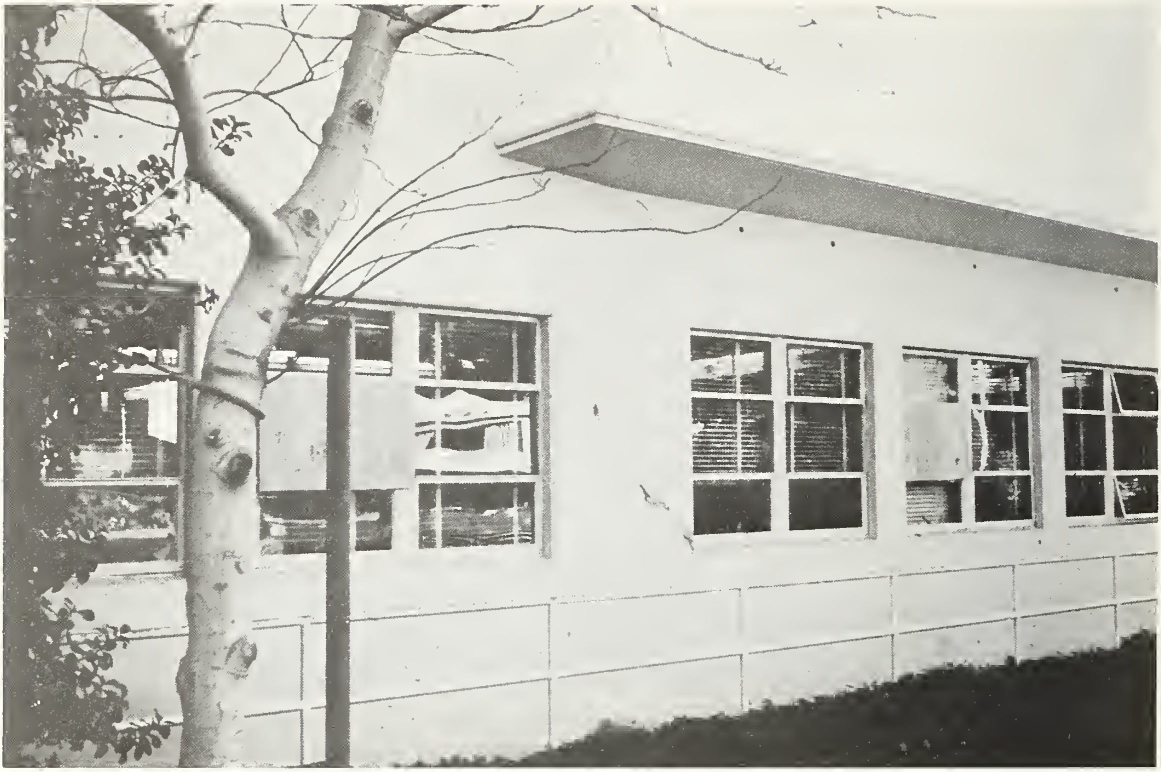


*Figure 6.32 Cracked pier in north wall of administration wing, Crescenta Valley High School.*

*Figure 6.33 Ventilating louvers pulled loose from corridor wall above classroom doors (louver placed on table for photo), Crescenta Valley High School.*

*Figure 6.34* Flat steel bar braces concealed in transverse partitions, Clark Junior High School. Damage to partition caused by buckling of brace and striking the studs.

*Figure 6.35* Damage to lighting fixtures at Clark Junior High School.

## SPECIAL INSTITUTIONS WITH SCHOOL STRUCTURES

The school portions of these special institutions were constructed in compliance with the Field Act; whereas, the remainder of the structures were constructed under the less restrictive requirements of the Los Angeles County Building Code.

Los Angeles County Juvenile Facility, in Sylmar was constructed in 1965. This institution is located in a zone of extreme ground fracture approximately three miles northwest of the City of San Fernando (location 15, fig. 6.1). The institution is primarily a group of one- and two-story concrete block bearing wall buildings, some of which are joined at split levels to accomodate a sloping site (fig. 6.36). The roof and second floor consist of concrete joists and slabs. Walls are concrete block. The first floor is a concrete slab on grade. The gymnasium has a metal deck roof supported on steel trusses resting on top of concrete walls with clerestory windows on four sides. Diagonal rods provide the lateral bracing of the steel trusses from the roof level to the top of the walls. All buildings rest on spread footings. The concrete block construction of the school buildings in this complex is different from that of other buildings. The rules and regulations for school buildings require uniform distribution of reinforcement in both horizontal and vertical directions and all block cells must be fully grouted. On this project, reinforcement was spaced at two feet on center in both directions. For other buildings the Code permits the concentration of horizontal reinforcement in the bond beam and only the reinforced cell need be grouted.

There was a collapse of a portion of the administration wing which occurred directly over the area of maximum ground fracture (fig. 6.37), and there was extensive damage without collapse at other areas of ground rupture (fig. 6.38 and 6.39). In the area adjacent to the actual ground cracking, the degree of damage was variable.

The school buildings at this institution survived the quake very well (fig. 6.40, 6.41 and 6.42). The west end wall of the boys' school was severely damaged and one window fell inwardly (fig. 6.43). Some of the perimeter ceiling in the west classroom was damaged; however, the contents of the room remained intact and it appears that the classroom would have been safe (fig. 6.44). There was some minor displacement of concrete blocks at the tops of transverse partitions (fig. 6.45). The walls at the junction of the boys' school and the gymnasium were cracked (fig. 6.46). The glass in the east and west clerestory between the trusses in the gymnasium fell to the ground and the bracing rods between the roof and the top of the walls either stretched or broke

*Figure 6.36 Some buildings were constructed with split levels to accommodate a sloping site at the Los Angeles County Juvenile Facility.*

*Figure 6.37 A portion of the Administration Wing collapsed at the Los Angeles County Juvenile Facility. Note the distortion of the ground surface.*

Figure 6.38 Ground crack adjacent to girls' living unit, Los Angeles County Juvenile Facility. Man standing in the crack is 6 ft 0 in tall.

Figure 6.39 Large displacement in floor of girls' living unit, Los Angeles County Juvenile Facility. Structure did not collapse.

*Figure 6.40 Boys' school and gymnasium, Los Angeles County Juvenile Facility.*

*Figure 6.41 West wall of girls' school, Los Angeles County Juvenile Facility. Note displacement at left (north) end of building and severe ground rupture (see figure 6.48).*

*Figure 6.42 East wall of girls' school, Los Angeles County Juvenile Facility. Note distress at right (north) end adjacent to taller (cafeteria) building and ground rupture.*

*Figure 6.43 Damage to west wall of boys' school, Los Angeles County Juvenile Facility. Window fell inward.*

*Figure 6.44 West classroom of boys' school, Los Angeles County Juvenile Facility. It appears the room would have been safe.*

*Figure 6.45 Displacement of concrete block at top of transverse partition, Los Angeles County Juvenile Facility.*

(fig. 6.47). There was damage to the portion of the girls' school adjacent to the cafeteria at a location of vertical ground fracture; however, the solidly grouted concrete block wall resisted the distortion (fig. 6.48), and prevented collapse. The floor was badly cracked in this area. There was minimal damage to the rest of the girls' school.

Major damage on other buildings consisted of shattering of walls at locations of change in roof elevations (fig. 6.49 and 6.50) and fracturing of concrete block walls constructed with "stacked bond", in which the horizontal reinforcing was concentrated only at the bond beam (fig. 6.51 and 6.52). The better behavior of the school buildings can be attributed to the more restrictive rules and regulations of the Field Act since the other factors (Architects, Engineers, Contractors and Inspectors) were the same for both types of buildings. The school buildings could be economically repaired; however, the entire facility has been vacated and may be abandoned.

Camp Karl Holton, Los Angeles County Probation Camp, located in Little Tujunga Canyon was constructed in 1959. The camp is located in an area of extreme ground fracture (fig. 6.53 and 6.54) approximately four miles northeast of the City of San Fernanco (location 16, fig. 6.1). This camp contains a group of single-story concrete block bearing wall buildings which include administration, service facilities, school, living quarters, and a recreation building. Construction consists of wood joists and roof sheathing, concrete block walls, and concrete slabs on grade. The buildings rest on spread footings. There is one wooden portable classroom building near the masonry school. A concrete block security wall surrounds the institution. Much of the perimeter security wall has fallen or is severely tilted (fig. 6.55). As previously noted, concrete block masonry for schools requires uniform distribution of horizontal and vertical reinforcement with all block cells fully grouted whereas for other structures horizontal reinforcement may be concentrated in the bond beam and only reinforced cells must be grouted.

The recreation building suffered severe structural damage. The concrete block walls which were grouted only in the reinforced cells shattered and were displaced as much as four inches laterally at the base (fig. 6.56 and 6.57). The living unit was undergoing major alterations at the time of the earthquake. Thus, the damages sustained do not necessarily reflect the condition of the building. The vehicle storage building has a broken bottom chord at one gabled wooden roof truss, and the wood sheathing appears to be supporting the roof by plate action. Approximately six inches of vertical displacement between the masonry classroom building and the portable classroom building was noted (fig. 6.58). Either

*Figure 6.46 Cracked wall at junction of boys' school and gymnasium, Los Angeles County Juvenile Facility.*

*Figure 6.47 Clerestory windows of gymnasium fell out and bracing rods stretched, Los Angeles County Juvenile Facility.*

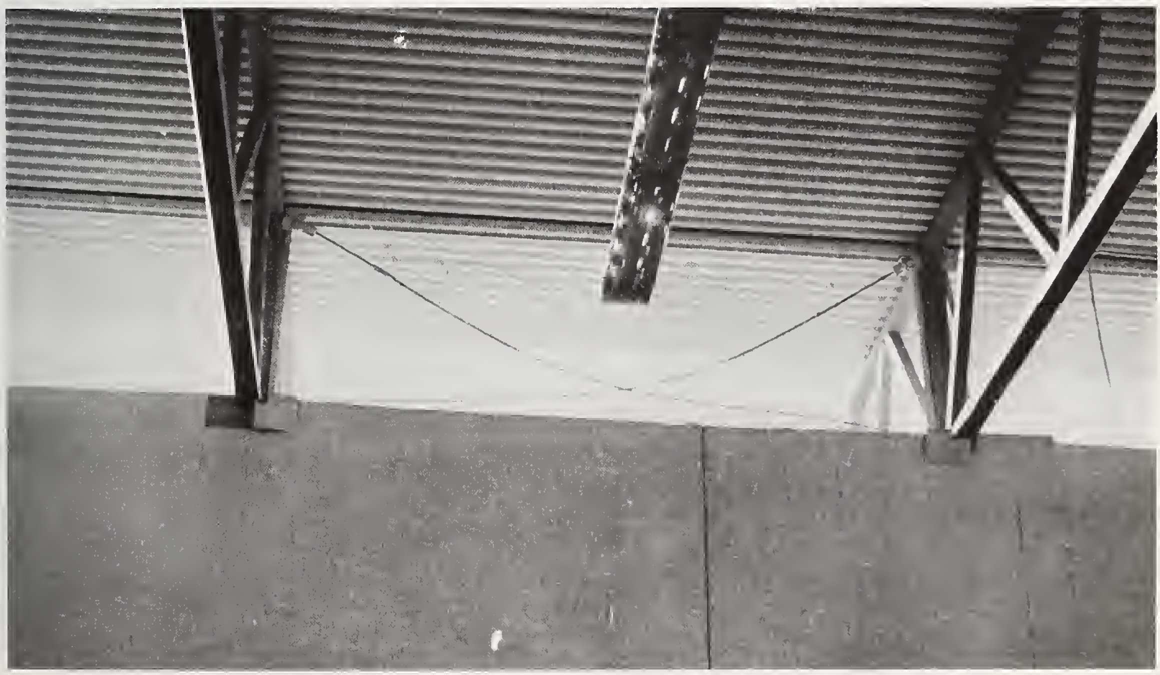


Figure 6.48 Damage to girls' school adjacent to cafeteria, Los Angeles County Juvenile Facility. The solidly grouted concrete block wall resisted the distortion.

*Figure 6.49 Damage to wall at change in roof elevation (exterior), Los Angeles County Juvenile Facility.*

*Figure 6.50 Damage to wall at change in roof elevation (interior), Los Angeles County Juvenile Facility.*

Figure 6.51 Damage to concrete block wall, Los Angeles County Juvenile Facility. Note absence of horizontal reinforcement. Only reinforced vertical cells are filled with grout.

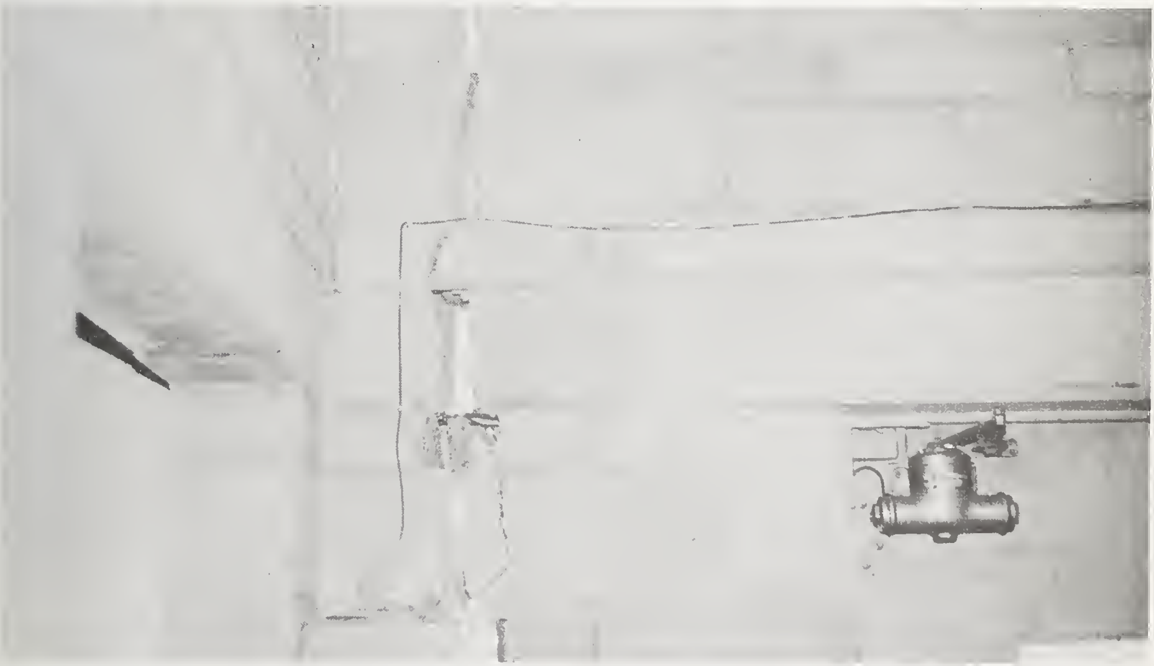

*Figure 6.52* *Damage to concrete block wall, Los Angeles County Juvenile Facility. Note absence of horizontal reinforcement. Only reinforced vertical cells are filled with grout.*

Figure 6.53 Eight-inch vertical displacement in out-door recreation area, Camp Karl Holton.

Figure 6.54 Extensive cracking of parking lot pavement, Camp Karl Holton.

*Figure 6.55 A portion of perimeter security wall has fallen; some is severely tilted, Camp Karl Holton.*

*Figure 6.56 Concrete block wall grouted only in vertically reinforced cells shattered, Camp Karl Holton.*



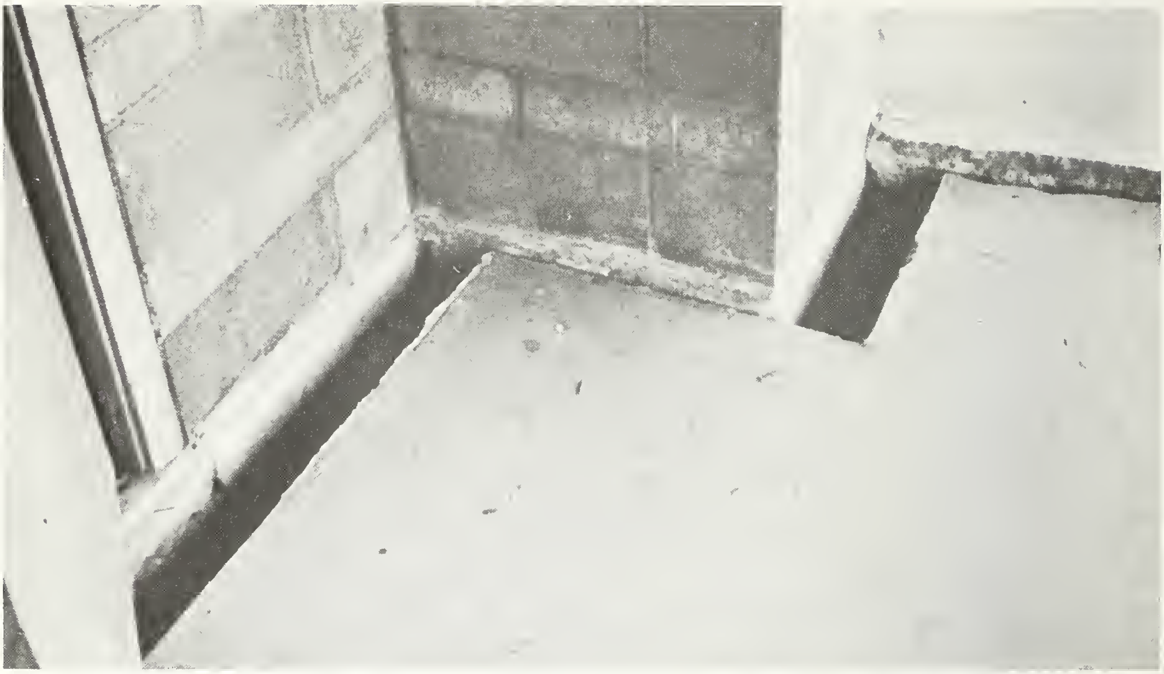


*Figure 6.57 Recreation Building at Camp Karl Holton. Four-inch displacement between slab and wall.*

*Figure 6.58 Approximately 6 inches of vertical displacement between masonry classroom building and the portable classroom building, Camp Karl Holton.*

the portable building and the ground on which it is supported dropped or the adjacent area, together with the building thereon, was raised. The building remained intact although two covered arcade pipe columns were displaced a few inches at the base where the slab on grade pulled apart (fig. 6.59). The stem mounted lighting fixtures in all buildings were severely damaged and most fell to the floor (fig. 6.60). Damage to the portable classroom was minimal. A sheet metal foundation clip was distorted and an interior partition was offset a quarter of an inch at a joint in the wallboard. The classroom building could be used in its present condition if the light fixtures were reinstalled; however, the complex has been vacated pending a study of the disposition of the camp.

## 6.4 Summary

School buildings as a group survived the earthquake exceptionally well. It is again noted that, although structural damage was experienced, none of the new buildings collapsed. It appears that no one would have been seriously injured as a result of such structural damage. The cost to repair this damage will be relatively minimal. Fortunately, there was no total collapse of older pre-Field Act buildings, but there was partial collapse and extensive damage requiring extensive repair and reinforcement or replacement prior to reoccupying these older structures. As previously noted, thirteen buildings at ten schools in the Los Angeles Unified School District were vacated immediately; all of the other old (pre-Field Act) schools within the district were reviewed by a panel of engineers. Approximately 85 buildings (approximately 900 classrooms) were found to be potentially dangerous in the event of the reoccurrence of a quake of similar magnitude closer to these schools. These schools are now vacated, and depending on cost, will either be strengthened or replaced prior to July 1, 1975, as required by Article 5 of the California Education Code. Older buildings which were strengthened in accordance with the intent of the Field Act suffered relatively minor structural damage.

Structural damage to new schools included displacement of slabs on grade and foundations at locations of ground rupture; permanent deformation of cantilever columns supporting lunch shelters or arcades; cracked arcade columns; cracked shear wall piers; stretched steel strap bracing with consequently large deflections and cracking of poured gypsum roof diaphragm. Damage to concrete and masonry consisted of minor cracking.

The present code makes no provision for forces caused by vertical acceleration of an earthquake. When this is coupled with the present code provision, permitting a one-

*Figure 6.59* *Arcade pipe columns were displaced at base where slab on grade pulled apart, Camp Karl Holton. (Damaged lighting fixtures were removed from buildings and stored on walkway.)*

*Figure 6.60* *Stem mounted lighting fixtures were severely damaged, Camp Karl Holton.*

third (1/3) increase in the working stresses when seismic loading is combined with vertical loading, the margin of safety is reduced to an unacceptable value. Further, permitting the one-third (1/3) increase of stresses in steel bracing also increases the elongation of the brace and therefore the drift of the building, thereby increasing the amount of nonstructural damage.

Comfort must not be sought in the fact that there was only minimal structural damage to the new buildings because, in some instances, these same buildings suffered considerable nonstructural damage which could have been an extreme hazard to its occupants. Many of the suspended "T"-bar acoustical ceiling systems failed, dropping acoustic tile and "T" bars, principally at perimeters of rooms. Lighting fixtures attached only to the "T" bars pulled away and in many cases dropped to the floor. Lighting fixtures with a swivel connection at the ceiling but not at the fixture fractured at the stem and dropped to the floor. Glass shattered because it was tightly set in the frames. Heavy ventilating grilles dropped from the ceiling or walls. Relative displacement between structures at separtion joints sheared electrical conduits which were rigidly connected to both sides of the joint, thereby causing a failure in the lighting system. There was plaster and ceramic tile damage primarily at junctions of rigid to flexible elements.

## 6.5 Recommendations

School structures resisted the earthquake forces very well, and are in effect a testimonial to good design and construction procedures within the present state of the art. The first objective of earthquake resistive design, "prevention of loss of life or personal injury" has been met. However, the reduction of personal hazard and damage primarily from nonstructural elements is still a major consideration. Therefore, the following changes to the rules and regulations should be considered:

1. Provide for vertical seismic forces as well as lateral seismic forces.

2. Eliminate the permitted one-third (1/3) increase to the allowable working stresses when considering seismic loading combined with vertical loading.

3. Apply an occupancy multiplier of perhaps 1.5 for schools to the seismic shear determined for ordinary structures.

4. Reinforce all concrete column-beam connections as required for a ductile frame.

5. Limit the drift of structures to be compatible with contiguous rigid elements, and make provision for differential movement between buildings or portions of buildings.

6. Provide for the design and detailing of all "nonstructural" elements that could fall; such as, ceiling construction, lighting fixtures and accessories, mechanical duct and louvers, mechanical equipment, curtain walls, veneers, glazing, etc.

## CHAPTER 7 DAMAGE TO BRIDGES AND HIGHWAYS

### 7.1 Introduction

The State of California has been widely acknowledged as a leader in the development of street, highway, and freeway nets and the northern San Fernando Valley had a wide variety of such transportation arteries. Post-quake reports to the California State Highway Commission indicated that the earthquake damaged approximately 11 miles of multilane freeways and 6 miles of conventional state highways, in addition to numerous city and county streets. Estimates of total damage to roads and highways varied from $27.5 million to $33 million, with approximately $15 million damage related to the freeway system shown in figure 7.1. Of this total, approximately $7 million represented damages to sixty bridges. Approximately 10 percent of these bridges either collapsed or were so badly damaged that they will have to be removed and replaced. The remainder varied widely in degree of damage, but have the possibility of complete restoration.

Two major interchanges in the area were under varying degrees of construction at the time of the earthquake, so that the loss represents estimated losses of forms, materials, and other construction equipment. In addition to the material loss, two men lost their lives when a freeway overpass fell on their pickup truck as they passed beneath it during the quake. Considerable disruption of traffic flow was caused by the collapses and pavement ruptures but was quickly brought under control by provision of temporary detours and alternate routing. The heavily traveled Golden State Freeway (Interstate 5) was reopened to traffic using emergency detours within five days after the earthquake.

The major damage to bridge structures was concentrated in the area shown in figure 7.1, with approximately 80 percent of the reported estimated costs of bridge repairs and replacements being for structures in this area, which undoubtedly experienced substantial accelerations and large earth motions. In addition, several of the aftershocks had epicenters which apparently bracketed the Interstate 5/210 intersection. This area is immediately adjacent to the Sylmar Converter Station and the San Fernando Juvenile Hall (both of which experienced substantial damage), and is just north of the Van Norman reservoirs.

### 7.2 Case Histories of Bridge Damages

INTERSECTION OF INTERSTATE 5 AND CALIFORNIA 14 (Palmdale Interchange--Location A on fig. 7.1)

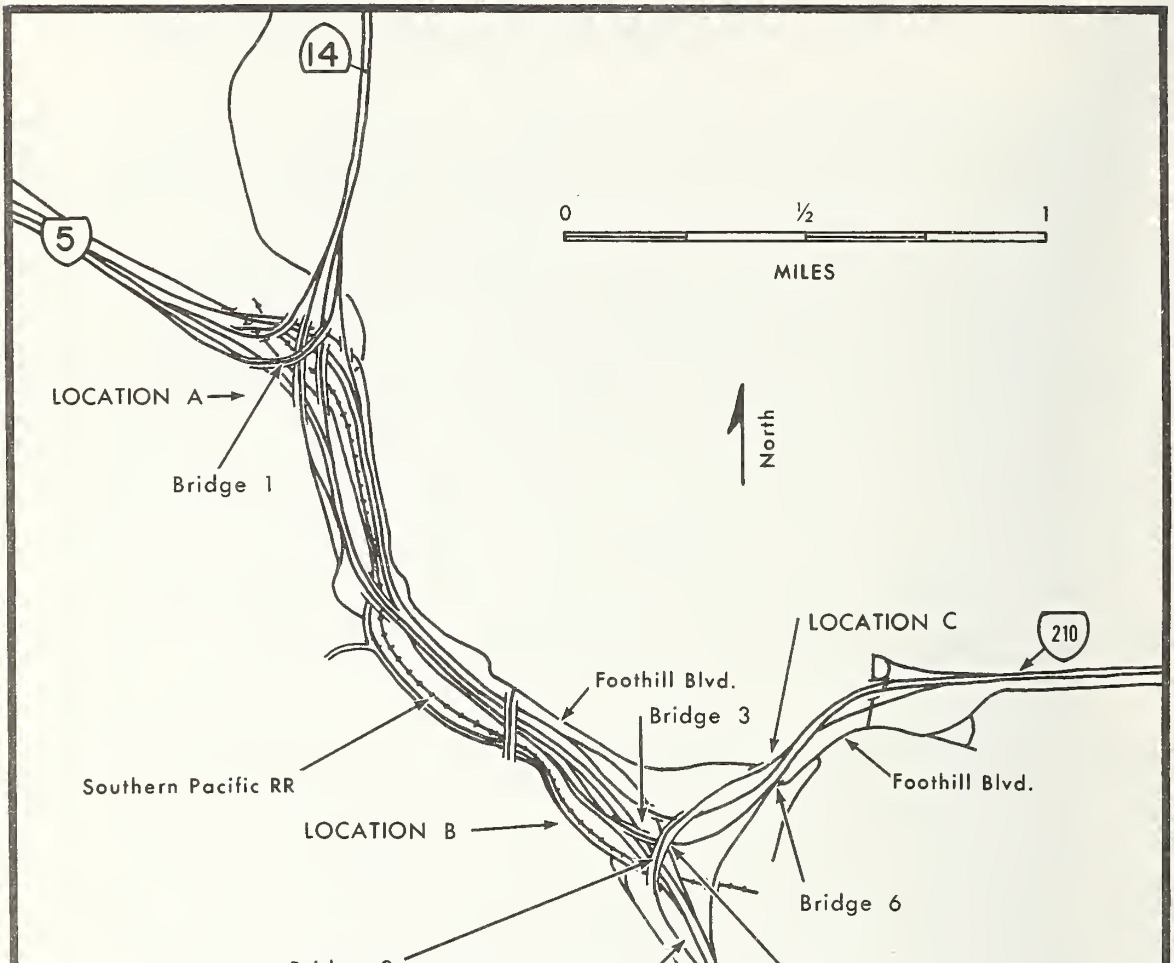



*Figure 7.1 Layout of freeway interchanges.*

This impressive interchange which linked the Golden State and the Antelope Valley freeways had been designed in the latter part of 1968, and was under active construction. The major damage at the interchange was sustained by the Southern Connector Overcrossing, which is labeled Bridge No. 1 in figure 7.2. At the time of the earthquake the superstructure had been essentially completed, as indicated by the dashed lines on figure 7.2. Two spans with the intermediate supporting column collapsed, and the debris can be seen in the bottom of the canyon. The two bridges shown with falsework in the foreground of figure 7.2 were under construction and suffered substantial damage due to shifting and settling of the falsework, which caused cracking in the girder soffits and stems. In addition, one of the piers was damaged by the collapse of the South Connector Overcrossing (Bridge No. 1), which struck it during its fall.

The South Connector Overcrossing was a nine span, continuous box girder bridge, as shown in figure 7.3. The bridge was curved in plan, and consisted essentially of two cast-in-place reinforced concrete box girder spans, a short overhang and hinge, two cast-in-place post-tensioned box girder spans, another hinge and short overhang, a cast-in-place reinforced box girder span, three cast-in-place post-tensioned box girder spans, and another cast-in-place nonprestressed concrete box girder span. The portion of the bridge which collapsed was the 354-ft cast-in-place prestressed box girder section which spanned between carefully detailed hinges which were located 14-ft inside piers 3 and 5. The section which collapsed was cast integrally with a very slender column which was founded on sandstone. The box girder section had three cells and a 34-ft overall width, as shown in figure 7.4. The girder dead load was approximately 9000 lb per ft of span. The girder was 7-ft deep with a 7-in top slab, 6-in bottom slab, and nominal 12-in webs.

The supporting pier which failed was nominally 10 ft by 6 ft in cross section, with an octagonal form as shown in figure 7.4. It was reinforced with 52 No. 18 bars, which were Grade 60 steel. Most other bars were Grade 40 steel. While all other piers were specified to be Class A concrete, this pier was specified to have a concrete compressive strength of 4000 psi. This particular pier was founded at elevation 1417, with a nominal ground elevation of 1463, and the bottom of the box girder was at approximately elevation 1604. Therefore, the column had nominal clear height of 142 ft above the ground surface, and the overall pier was approximately 187 ft in height. Since the earth would not give full fixity at ground level, the practical unsupported height of the column would be around 150 ft. This would indicate a slenderness ratio of 25 in the longitudinal

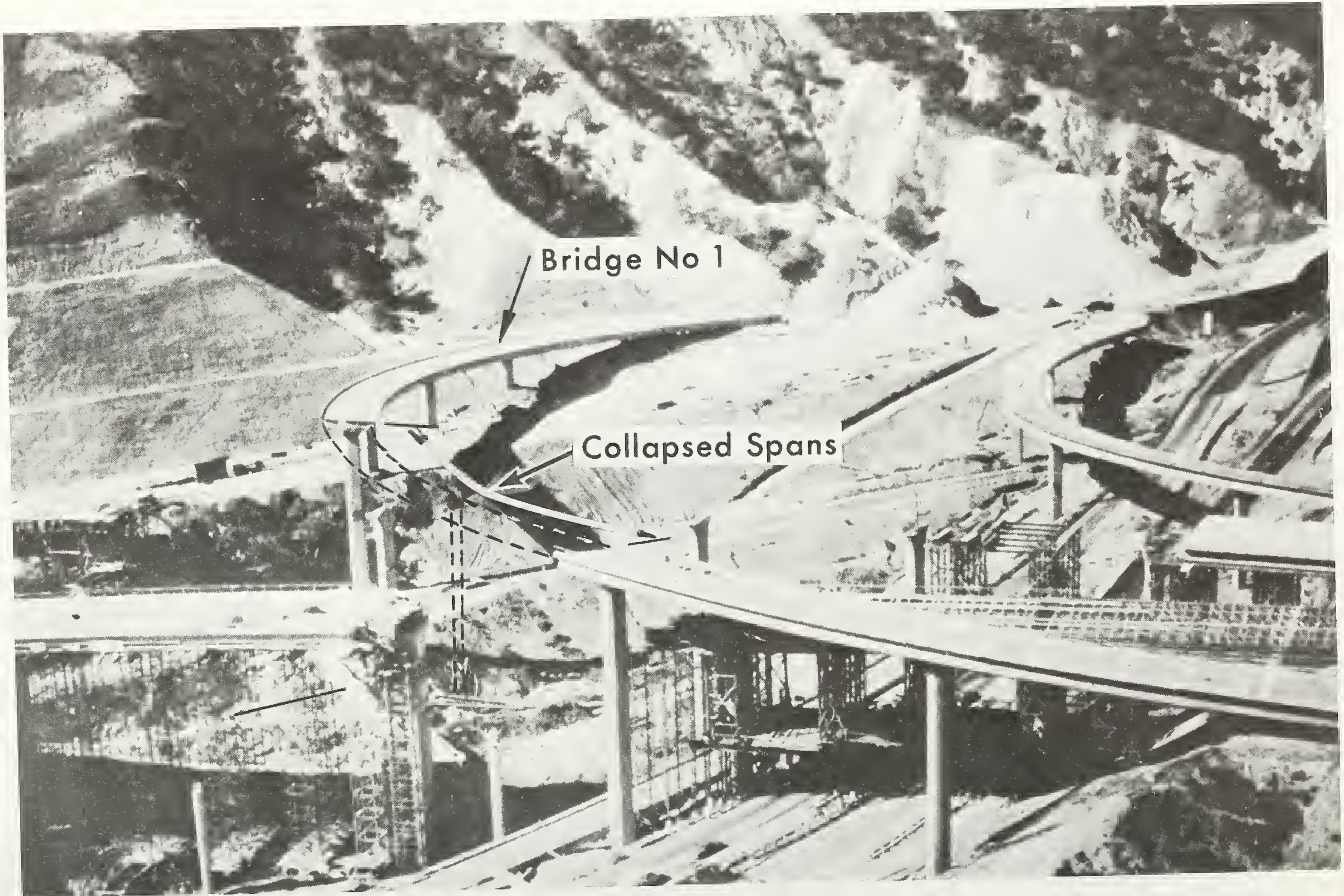



*Figure 7.2 Collapse of the South Connector Overcrossing at Palmdale interchange.*

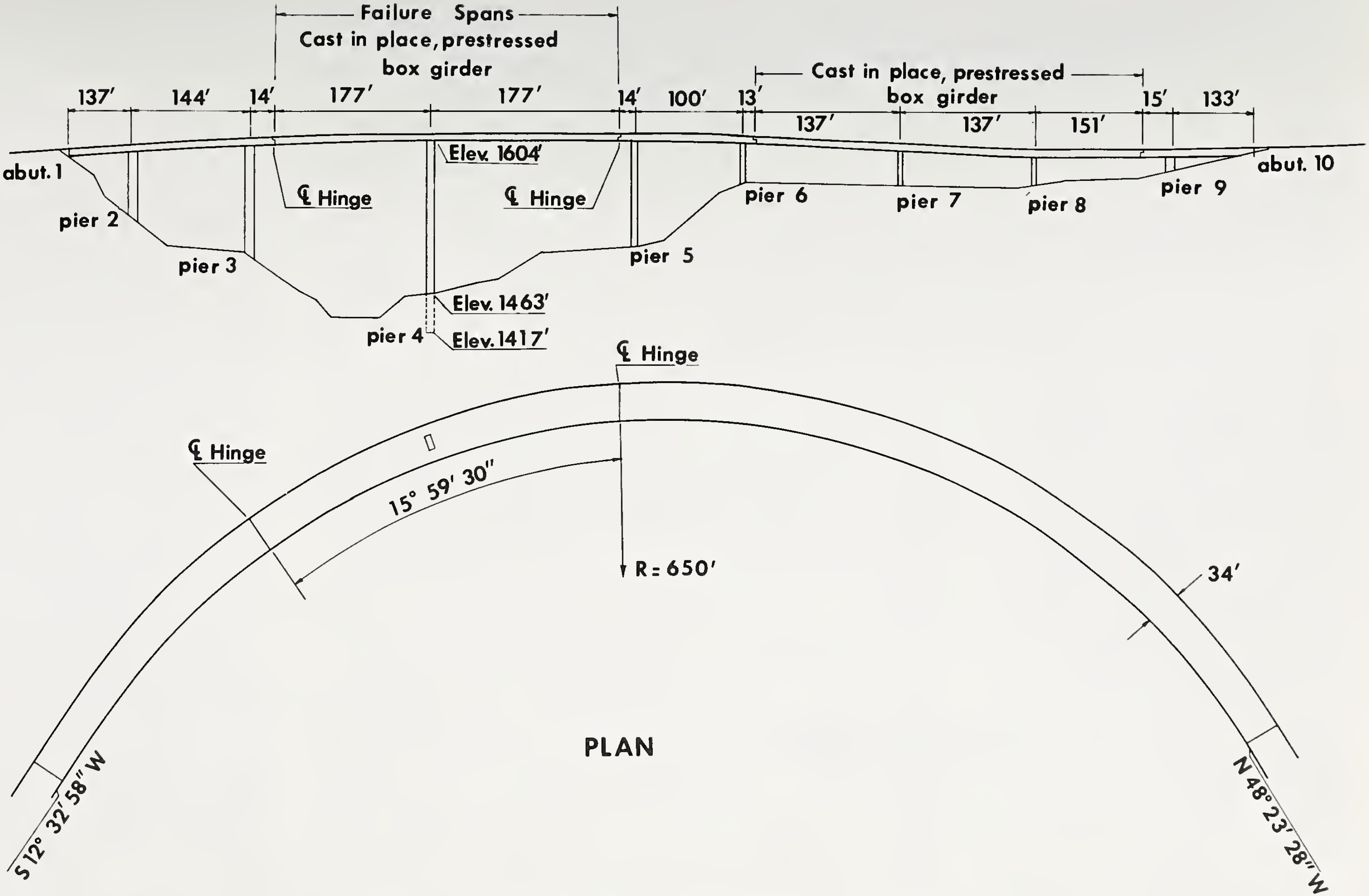



*Figure 7.3 Plan and elevation of South Connector Overcrossing.*

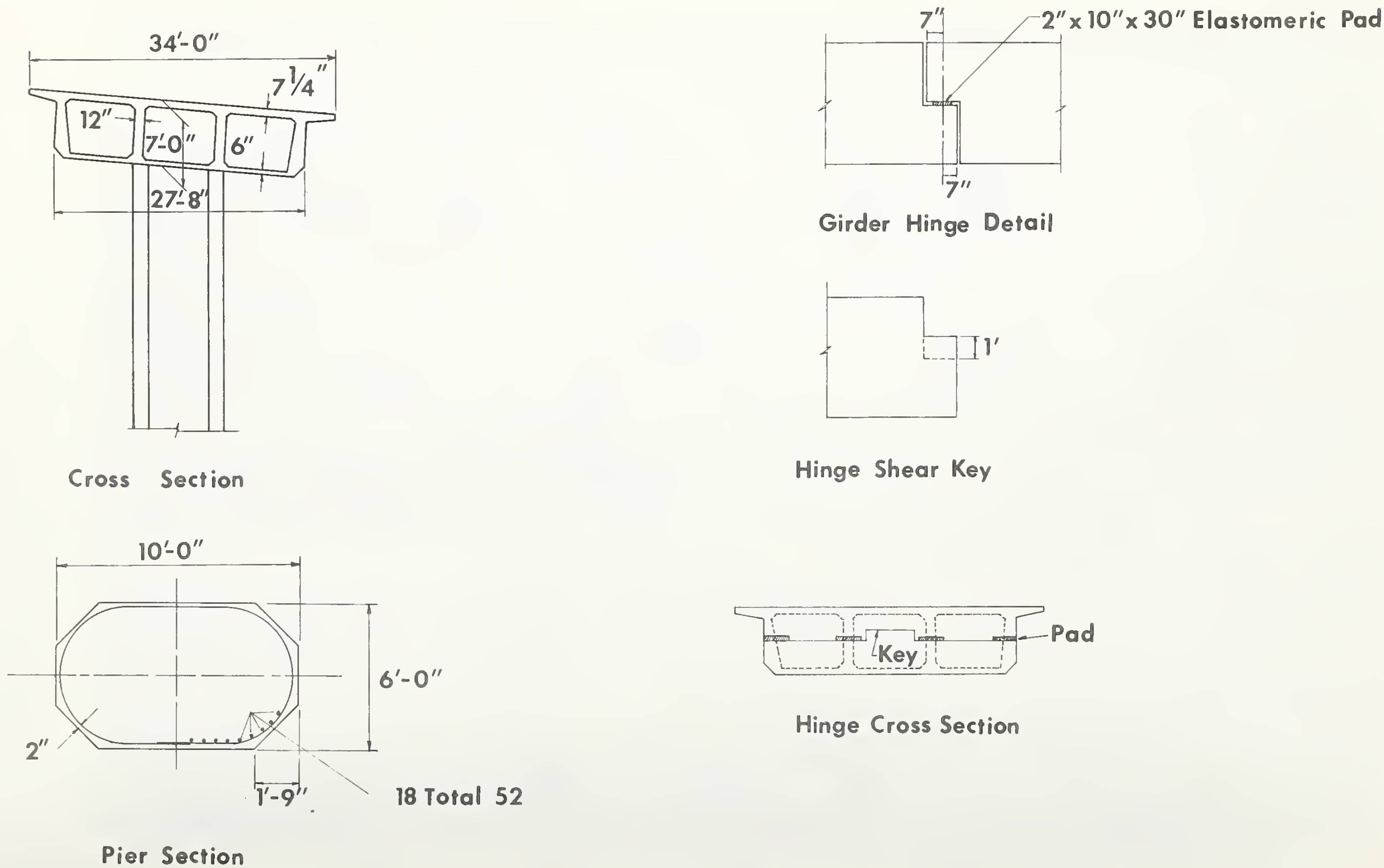



*Figure 7.4 Details of the South Connector Overcrossing.*

direction and 15 in the transverse direction. The slenderness of the column and the nature of the hinge seat can be seen in figure 7.5, which is a post-failure view of pier 5. Figure 7.6 shows the hinge cross section immediately adjacent to pier 3. Reference to the hinge detail as shown in figure 7.4 indicates that each web of the box girder rested on a 2-in thick by 10-in deep by 30-in wide elastomeric bearing pad and the center bay of the hinge seat had a 1-ft high raised shear key, which was approximately 5-ft 6-in wide. The raised cross section of this shear key is visible in figure 7.6. The only positive ties in the longitudinal direction were 1 1/2-in equalizing bolts, one of which was placed in the center of each of the three cells, parallel to the axis of the bridge.

The box girder had stiffening diaphragms at the center lines of the span and a 9-ft 6-in wide solid diaphragm above the pier, with the column reinforcement extending up into this diaphragm to within 1 ft of the top deck surface. Figure 7.7 indicates the debris after the collapse. Sections of the box girder can be seen at the upper right of the picture (resting against the pier from the other undercross bridge). The column can be seen lying across the center of the picture, and additional sections of the box girder can be seen in the lower left. There were obvious marks along the upper edge of the column standing in the upper right of the picture, which indicated that the girder had brushed against it with substantial force while dropping. Additional failure detail can be seen in figure 7.8, which shows the column top as it would be seen from underneath the structure. It can be observed that the column stub and column bars have pulled out of the cast-in-place diaphragm. This can be seen with greater clarity in figure 7.9, which shows the top of the column in the center of the picture, resting on a wrecked hoisting crane. It is extremely interesting to note that none of the heavy No. 18 cross bars, which were used as the main top reinforcement in the wide diaphragm, appear in the picture. These bars were reported to be in the rubble at the base of the column.

From the appearance of the failure debris and the remnants, as well as the reports of large ground motion in the area, it seems probable that the various sections of the bridge began to deflect with different magnitudes and frequencies and the amount of movement exceeded the seating capacity at the hinge, throwing a tremendous moment on the single slender pier. It may be significant that the section which collapsed was the only portion of the bridge which depended on a single column, as can be seen from figure 7.2. The other prestressed box girder section was supported on two columns and, hence, had some possibility of further stiffening if hinge capacities were exceeded. The raised

*Figure 7.5 Post-failure view of Pier 5 and hinge seat.*

*Figure 7.6 Post-failure view of Pier 3 and hinge seat with shear key.*

*Figure 7.7 Debris from the South Connector Overcrossing.*

*Figure 7.8 The top of Pier 5 and girder debris from the South Connector Overcrossing.*

*Figure 7.9 The top of Pier 5, which was embedded in the girder diaphragm, resting on the construction crane after collapse.*

shear key appears to be reasonably effective for preventing transverse motion at the hinges, as the hinge shear keys at both ends were visible from ground inspection, although possibly damaged. In spite of the apparent slenderness of the columns, very rough checks indicate that the moment capacity would be adequate for the specified earthquake loads used with the California Highway Department standard design procedures. However, the structural system used was vulnerable to the coupling of extremely high accelerations with large ground motions, since there is no alternate load path if the deflection capacity of the hinge is exceeded. Cursory examination of the box girder debris indicated that the girder section itself had probably performed well. In a number of cases the tendons were still intact although the girders were ruptured.

INTERSECTION OF INTERSTATE 5 AND INTERSTATE 210 (Golden State Freeway and Foothill Freeway Interchange--Location B on fig. 7.1)

This complex interchange, much of which was completed and open to traffic, suffered extensive damage and must be almost completely rebuilt. Figure 7.10 is an aerial view of the freeway interchange looking north toward San Fernando Pass and Newhall. The collapsed IS 210/5 Separation and Overhead caused major damage to the Golden State Freeway (IS 5), the Foothill Freeway (IS 210), the San Fernando Road (which paralleled the railroad), and the Southern Pacific Railroad tracks. The collapse of the Separation and Overhead resulted in the loss of the lives of two men who were passing underneath it in a pickup truck. Additional detail can be seen in figure 7.11. In addition to the collapsed Separation and Overhead structure (Bridge 2), the Northwest Connector Overcrossing (Bridge 3) was severely damaged from the impact of Bridge 2 and must be removed and replaced. The series of essentially parallel spans carrying IS 5 South and IS 5 North, and the northbound truck routes over the Southern Pacific Railroad tracks and San Fernando Road (Bridge Group 4) were also destroyed. Some spans of this bridge group were impacted by the falling IS 210/5 Separation and Overhead, while one steel girder span collapsed by dropping off its bearings. This complex represented various types of bridges, including reinforced concrete box girders, precast prestressed I-beam girders, and concrete-steel composite girder bridges. Immediately to the east of this group, Bridge 5 (as shown in fig. 7.12) was a two span cast-in-place prestressed box girder with a longest span of 122 ft. It was supported by a single column bent founded on 16-in diameter cast-in-place drilled shaft piles. The structure suffered major column damage and was removed. Because of the need to rapidly remove the destroyed and damaged bridges in order to reopen the


Figure 7.10 *Aerial view of freeway interchange at the junction of Golden State Freeway (IS 5) and Foothill Freeway (IS 210), looking north toward San Fernando Pass and Newhall.*

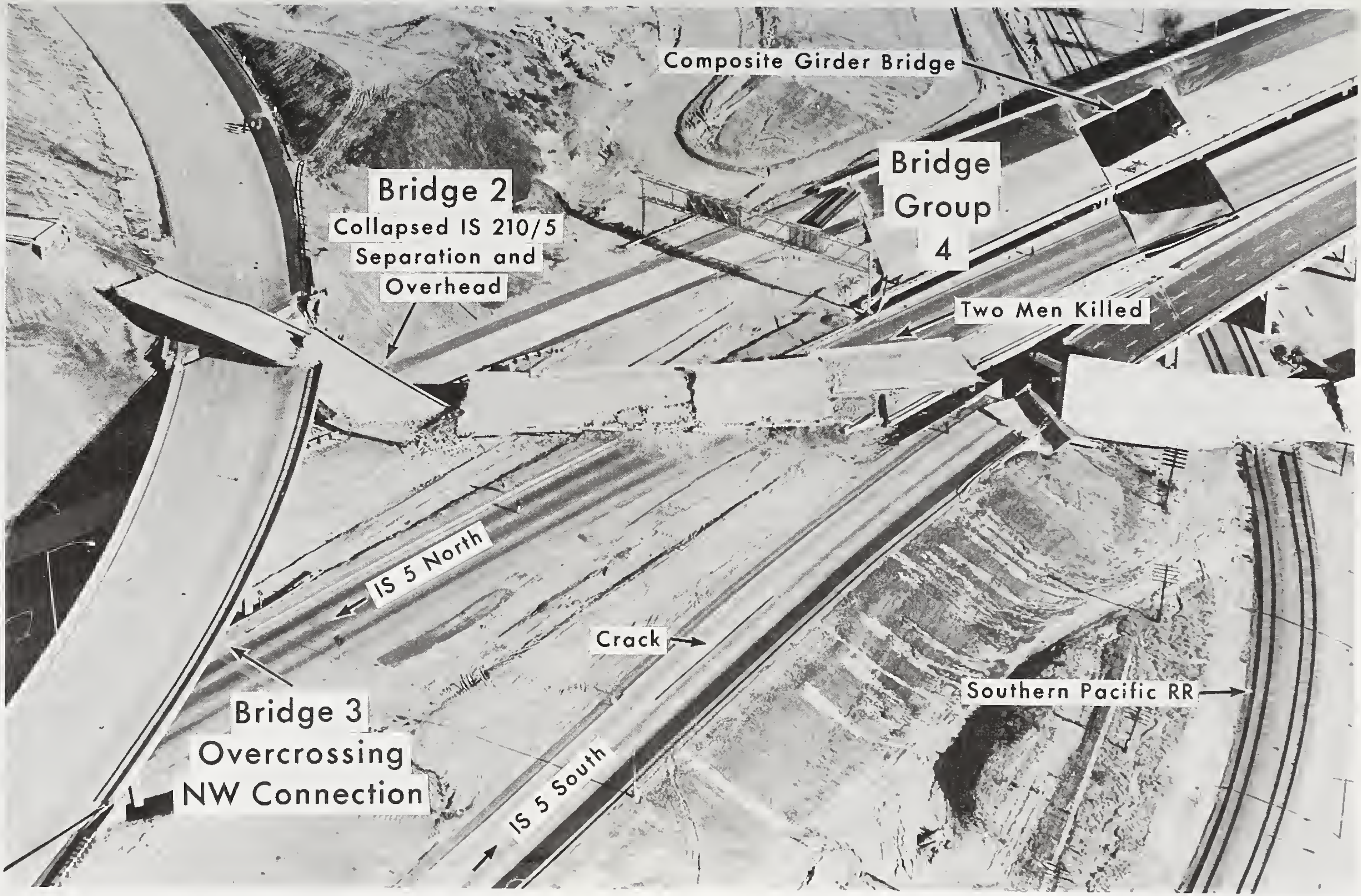



*Figure 7.11 View looking east of the collapsed Separation and Overhead (Los Angeles Times Photo).*

railroad tracks, immediate destruction of the remains of the bridges was undertaken and very little information was gathered concerning Bridge Group 4 or Bridge 5.

The epicenters of two of the moderate aftershocks were reported to have bracketed this bridge interchange and substantial evidence of ground motion and cracking was apparent. The closeness of this interchange to other severely damaged modern facilities such as the San Fernando Juvenile Hall and Olive View Hospital can be seen in figure 7.13. Because of the absence of recording accelerometers in the immediate area, the magnitude of actual accelerations is unknown. However, based on the general distribution of other accelerometer records and the extensive damage observed, it is probable that the site underwent horizontal accelerations of from one-third to one-half gravity and possible vertical accelerations of approximately one-half that amount. These would be substantially above the design values contemplated as will be seen in a later section. Very severe ground motion was noted on the Southern Pacific Railroad tracks adjacent to the San Fernando Juvenile Hall and the entire area around the Juvenile Hall was suspected to be a portion of a large slide area.

The collapsed IS 210/5 Separation and Overhead structure (Bridge 2) was a 7-span, nonprestressed, reinforced concrete box girder structure, carried on single column bents as shown in figure 7.14. This bridge was also curved in plan and had hinge details generally similar to those described previously for the bridge at the Palmdale interchange. The bridge was supported on elastomeric bearing pads on the pile-supported end abutments shown in the upper left of figure 7.15. A closeup of the abutment bearings at the north end is shown in figure 7.16. One of the elastomeric bearing pads can be seen lying next to the concrete bearing block. Another of these pads was found in a relatively undamaged state adhering to the under side of a box girder web after the girder had fallen approximately halfway down the slope. The appearance of the concrete pedestals in figure 7.16 indicates that the girders slid off the pedestals fairly rapidly.

The girder cross section is shown in figure 7.17, along with typical pier details. The three-cell girder was 6-ft 6-in deep, with a 7 1/4-in thick top slab, 6 1/4-in thick bottom slab and 8-in webs. The southernmost columns were carried on spread footings mounted on Class 2 piles, while the northern group of columns was mounted on 6-ft diameter circular concrete shafts which were cast in drilled holes. As noted in the discussions in the section on soils and foundations, the foundations for these columns were carried

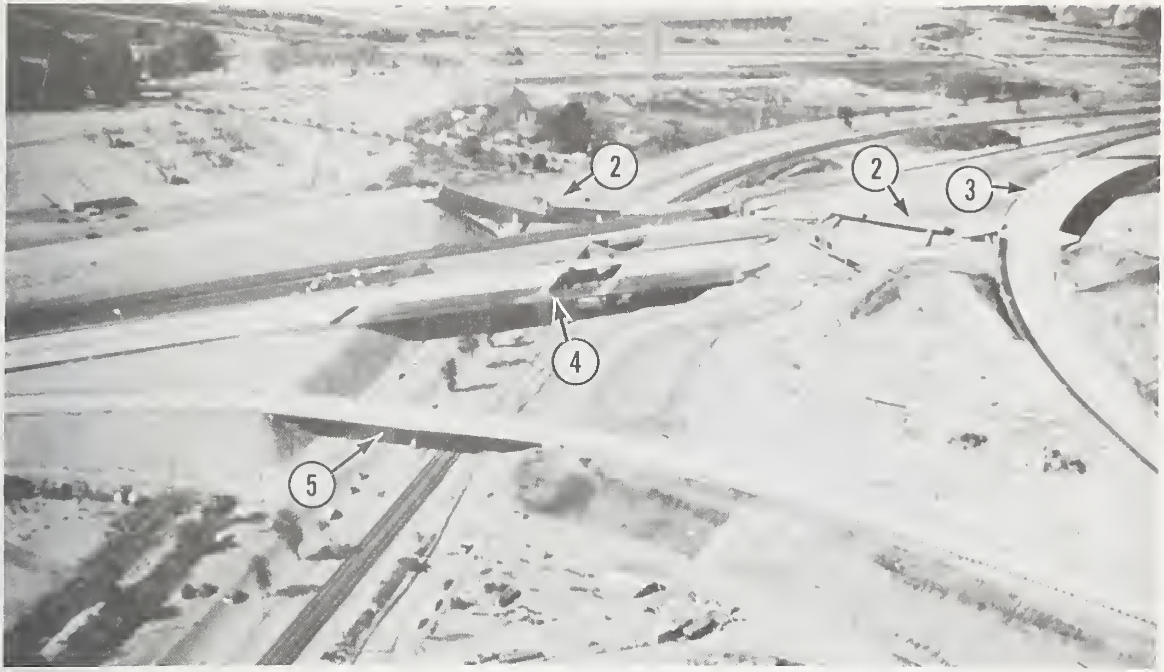
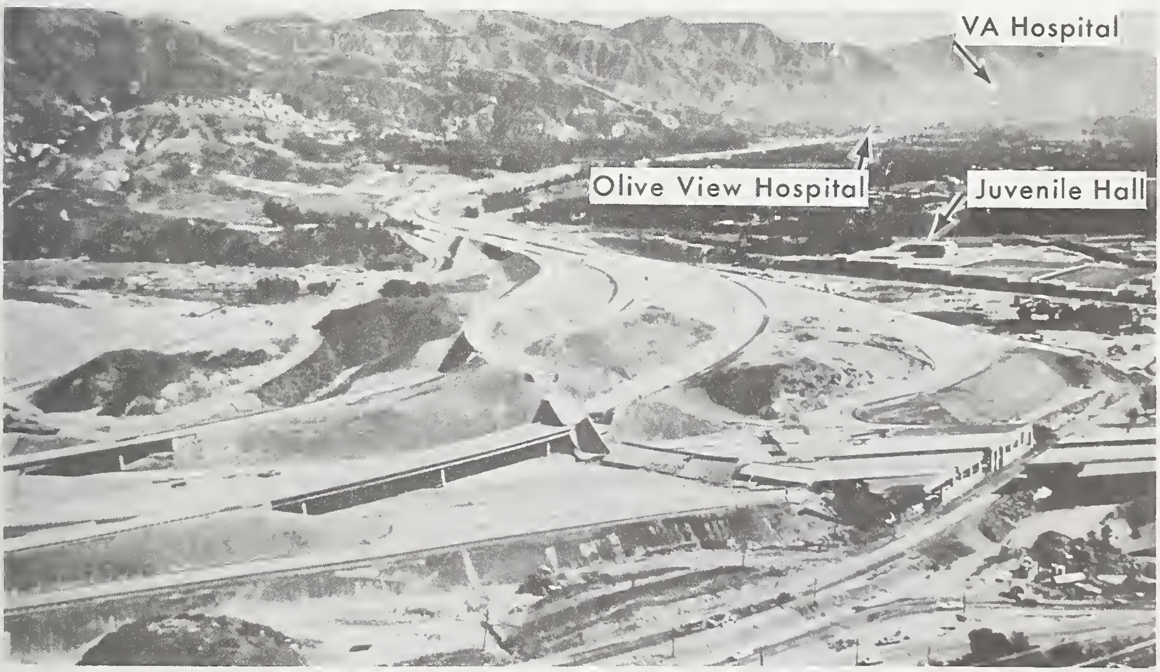



Figure 7.12 View looking northwest at the IS 5 and IS 210 interchange, showing Bridge group 4 and Bridge 5 immediately after the quake and before removal to restore railroad service.

Figure 7.13 Aerial view of IS 5 and IS 210 interchange looking east toward Olive View and Veterans Administration Hospital.



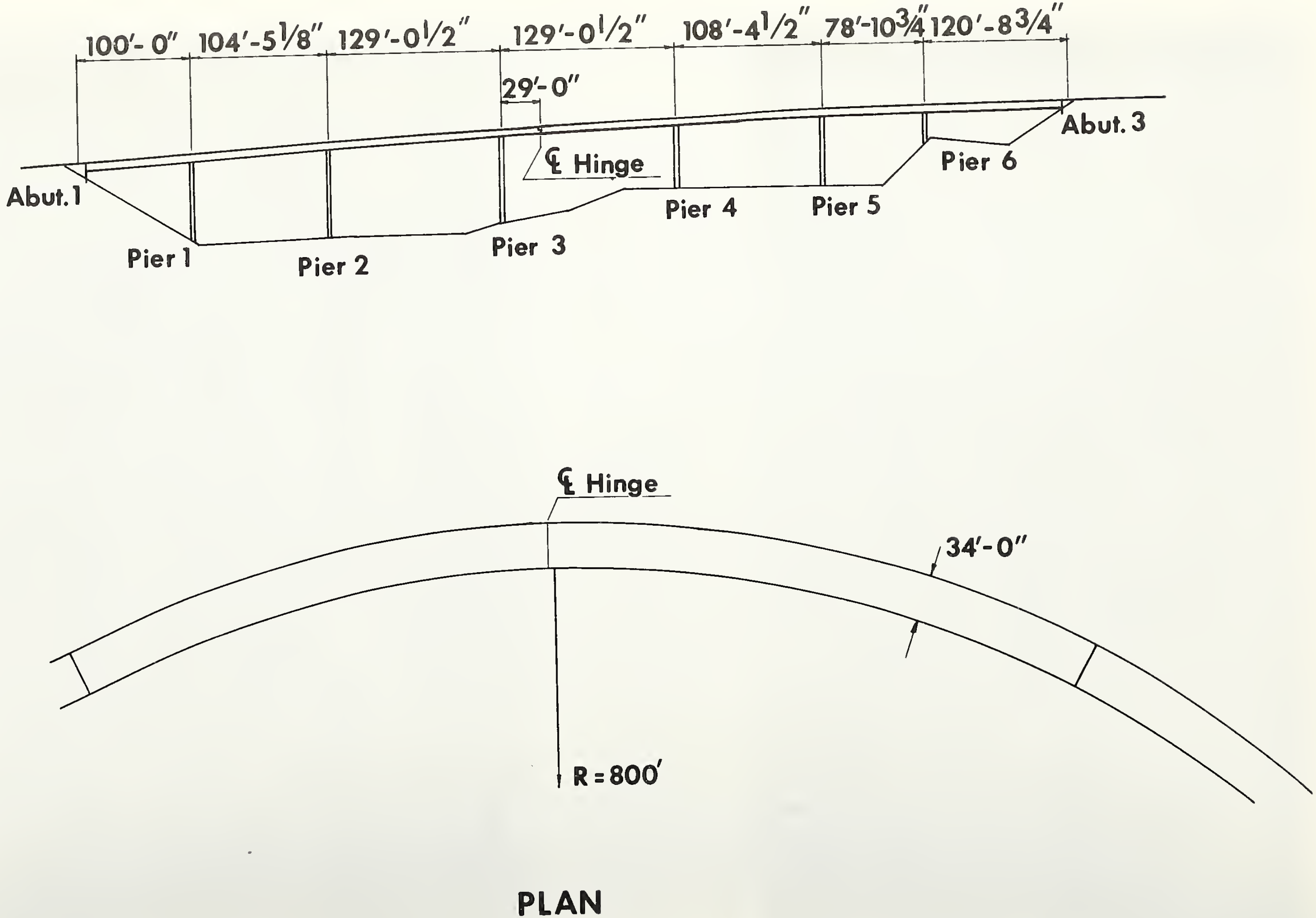



Figure 7.14 Plan and elevation of IS 210/5 Separation and Overhead structure (Bridge 2).

*Figure 7.15 East abutment of IS 210/5 Separation and Overhead structure.*

*Figure 7.16 Close up of east abutment and elastomeric bearing pad.*

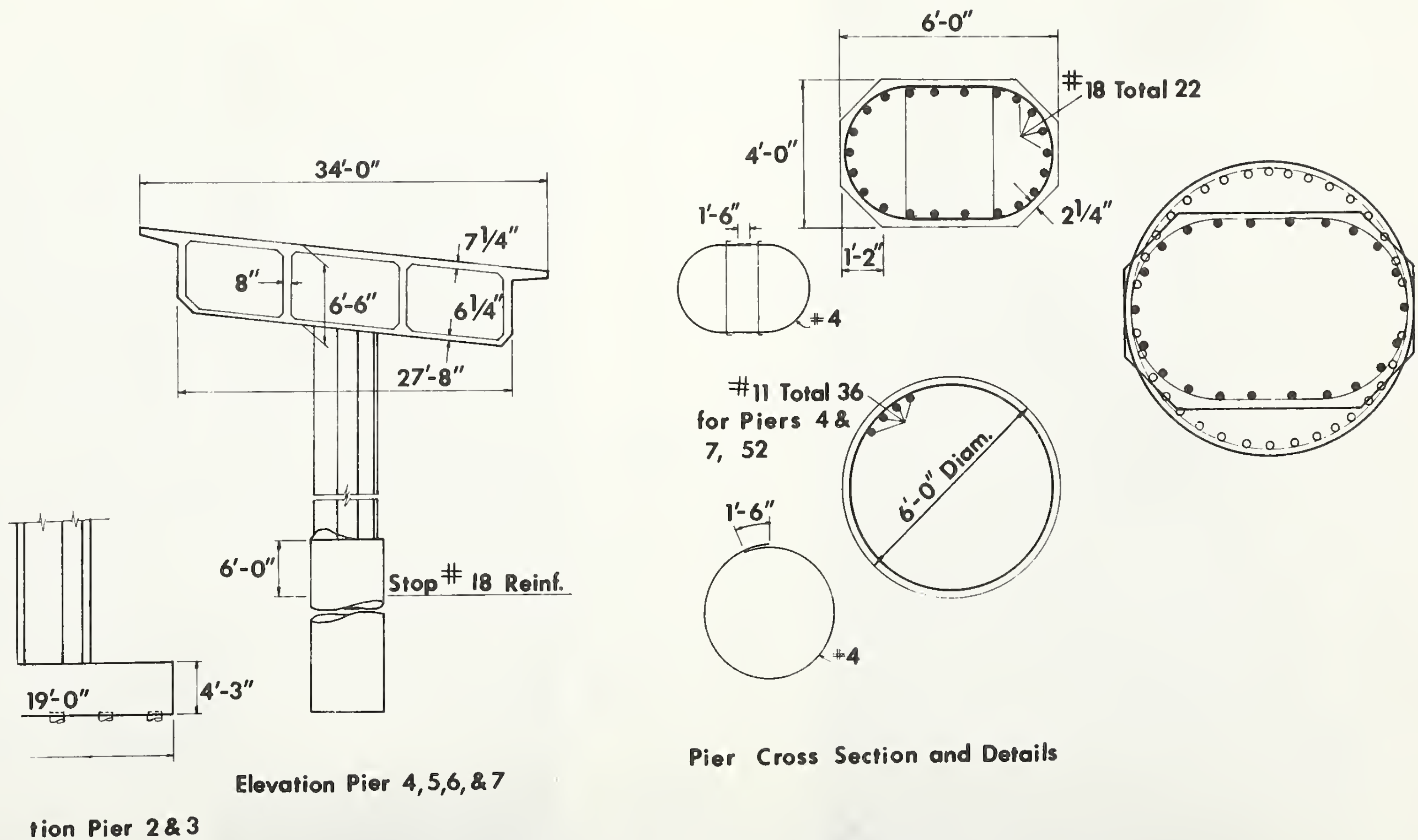



*Figure 7.17 Typical details for the IS 210/5 Separation and Overhead structure.*

down to strata of sands and gravels which showed very high penetration resistance. This differs from the Palmdale interchange which was founded on sandstone and possibly explains the more extensive damage occurring in this area. As can be seen from figure 7.17, the transition from the octagonal column to the circular cast-in-place drilled shaft is essentially a massive splice wherein all of the No. 18 bars in the column are lap-spliced to the No. 11 bars in the drilled shaft, with a nominal 6-ft lap length. This is possibly a severe interpretation of what would be considered a foundation anchorage, but the absence of reasonable percentages of transverse flexural reinforcement that would occur in a normal footing and the need for a transition zone before the earth can give full confinement to the drilled shaft allow one to visualize the action almost as that of a splice. As can be seen in figure 7.18, the columns both pulled out from the cast-in-place drilled shafts and developed massive plastic hinges at the column-girder connection. In some cases the girders appeared relatively undamaged and uncracked at the column connection (see fig. 7.19) and had shattered through at intermediate span points. In other cases, as shown in figure 7.20, the reinforcement in the lower slab of the girder had been completely stripped out as well as the column having separated from the cast-in-place drilled shaft. In this latter figure, the close spacing of the No. 18 bars can be seen as well as the almost total absence of transverse reinforcement at the splice zone. As shown in figure 7.17, the octagonal columns were nominally 4 ft by 6 ft in cross section and a typical column was reinforced with 22 No. 18 bars of Grade 40 steel. The longitudinal slenderness ratios varied from 11 to 16 and transverse slenderness ratios from 7 to 11. The tied columns had No. 4 hoops at 12-in spacing and these hoops were a single pair of U-bars which overlapped 18 in, as shown in figure 7.17. An extra pair of cross ties was also provided. At failure, some hoops separated and pulled out, as may be seen in figure 7.20. The drilled shafts were reinforced with 36 to 52 No. 11 bars, with No. 4 hoops at 12-in spacing enclosing the bars. The hoops were lapped 18-in and had no hooks. The combination of lap splices of No. 18 bars, splicing at essentially the point of maximum moment, the absence of large amounts of confining transverse reinforcement, and the splicing of all bars at one section compound the difficulty of developing this joint and should be avoided where possible in the future.

As in the case of the Palmdale interchange, it is extremely difficult to determine whether the bridge came off its abutments before the column joint failed, but from the appearance of the collapsed structure it can be concluded that the bridge was unable to withstand large ground movements and the column-to-foundation and column-to-girder connections

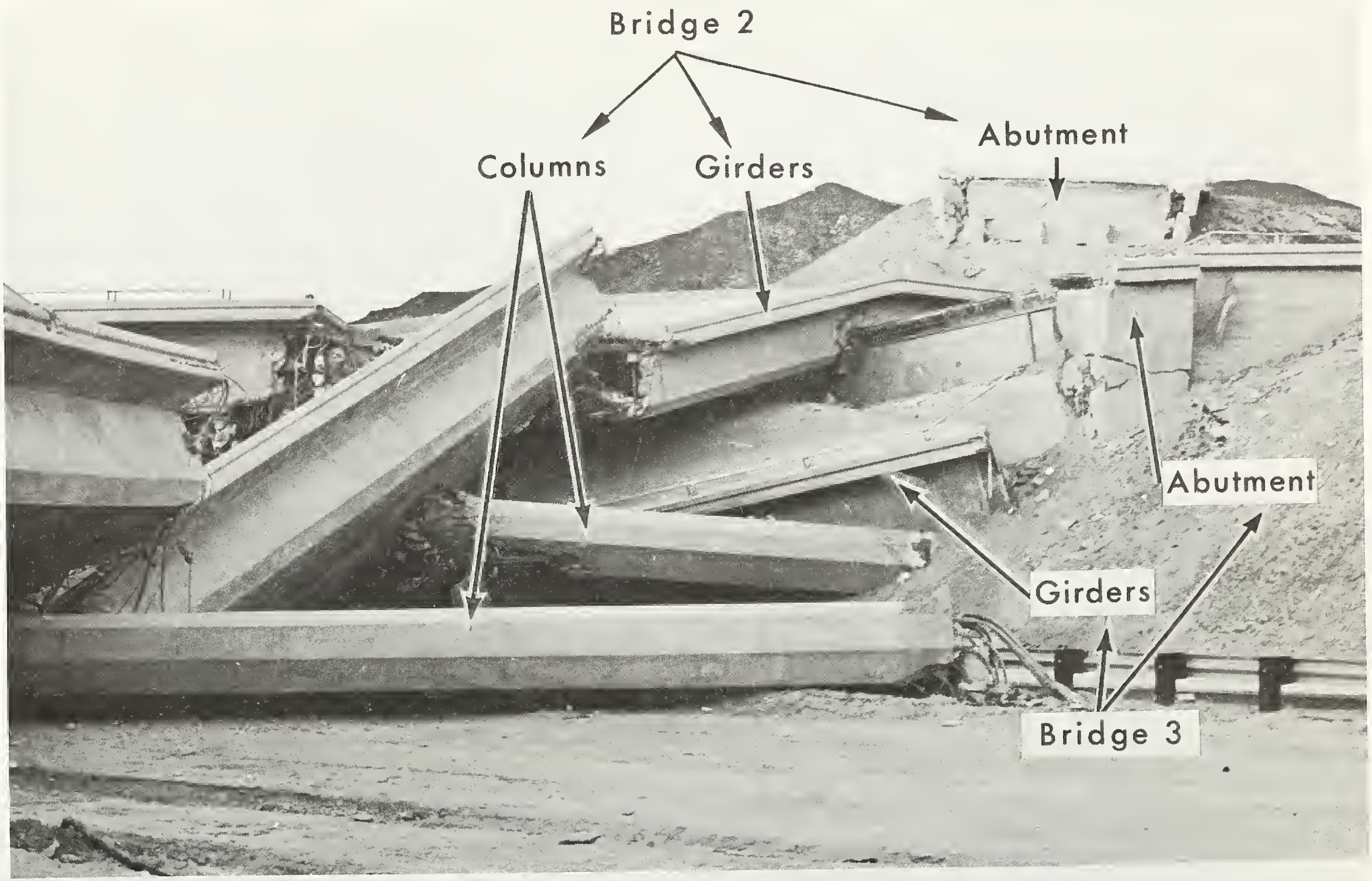



*Figure 7.18* *Collapsed girders and columns of the IS 210/5 Separation and Overhead structure (Bridge 2) on top of the collapsed Northwest Connector Overcrossing bridge (Bridge 3).*

*Figure 7.19* *Column-girder connection with major distress in the column. Note the separated column hoops.*

*Figure 7.20* *Base of the column which pulled out from the 6-ft diameter cast-in-place drilled shaft. Note the stripping of the reinforcement of the bottom of the girder in the background of the picture.*

were unable to withstand the combination of seismic and deflection produced moments. Again, with single columns at points of support, the column-to-foundation connection became critical and there was no chance for development of an alternate load path.

The Northern Connector Overcrossing (Bridge 3 in figs. 7.1 and 7.10) was a skewed, curved, 7-cell cast-in-place reinforced concrete box girder bridge. It had four spans of 70 ft, 180 ft, 162 ft, and 100 ft, and was carried on three- and four-columns bents, as shown in figure 7.21. The columns were octagonal, inscribed in a 4 ft square as shown and were reinforced with 16 to 20 No. 18 bars, which were surrounded by a No. 5 spiral at a 5-in pitch. Reinforcement was Grade 40 and column concrete was 4000 psi compressive strength. The columns rested on 6-ft diameter cast-in-place drilled shaft piles which were reinforced with 22 No. 11 bars, which extended approximately 15 ft into the pile. The columns in all bents were essentially hinged at the base where they connected to the cast-in-place pile using a 2-ft diameter by 1 5/8-in thick keyway, with only four No. 11 bars crossing it, as shown in the key detail in figure 7.21.

This structure was struck by the falling South Connector Overcrossing at its south and east end, as shown in figure 7.18. This impact added to the natural acceleration forced the structure northward, and because of the skew angle it slid in a northeast direction at the northern end, as shown in figure 7.22. The line of columns at bent 2 on the north edge collapsed with increasing distress from west to east, as shown in figure 7.23. Note that the columns are virtually uncracked near their base and over the lower two-thirds of their height. The action of the No. 5 spirals in holding the columns together is apparent and can be seen in detail in figure 7.24. In this same photo the relatively undamaged appearance of the girder at the column-girder connection can also be seen. Because of the damage to the substructure and in order to expedite the restoration of traffic, the entire structure is slated for removal and replacement.

## FOOTHILL BOULEVARD UNDERCROSSING OF INTERSTATE 210 (Location C on fig. 7.1)

The Foothill Boulevard Undercrossing was a pair of four span continuous reinforced concrete box girder skewed bridges, which carred IS 210 above Foothill Boulevard. The southernmost bridge of the pair was the more extremely damaged. A typical cross section is shown in figure 7.25, along with column and pier details. The intermediate bents consisted of octagonal columns inscribed in a 4 ft square and reinforced typically with 22 No. 14 bars or 18 No. 18 bars.

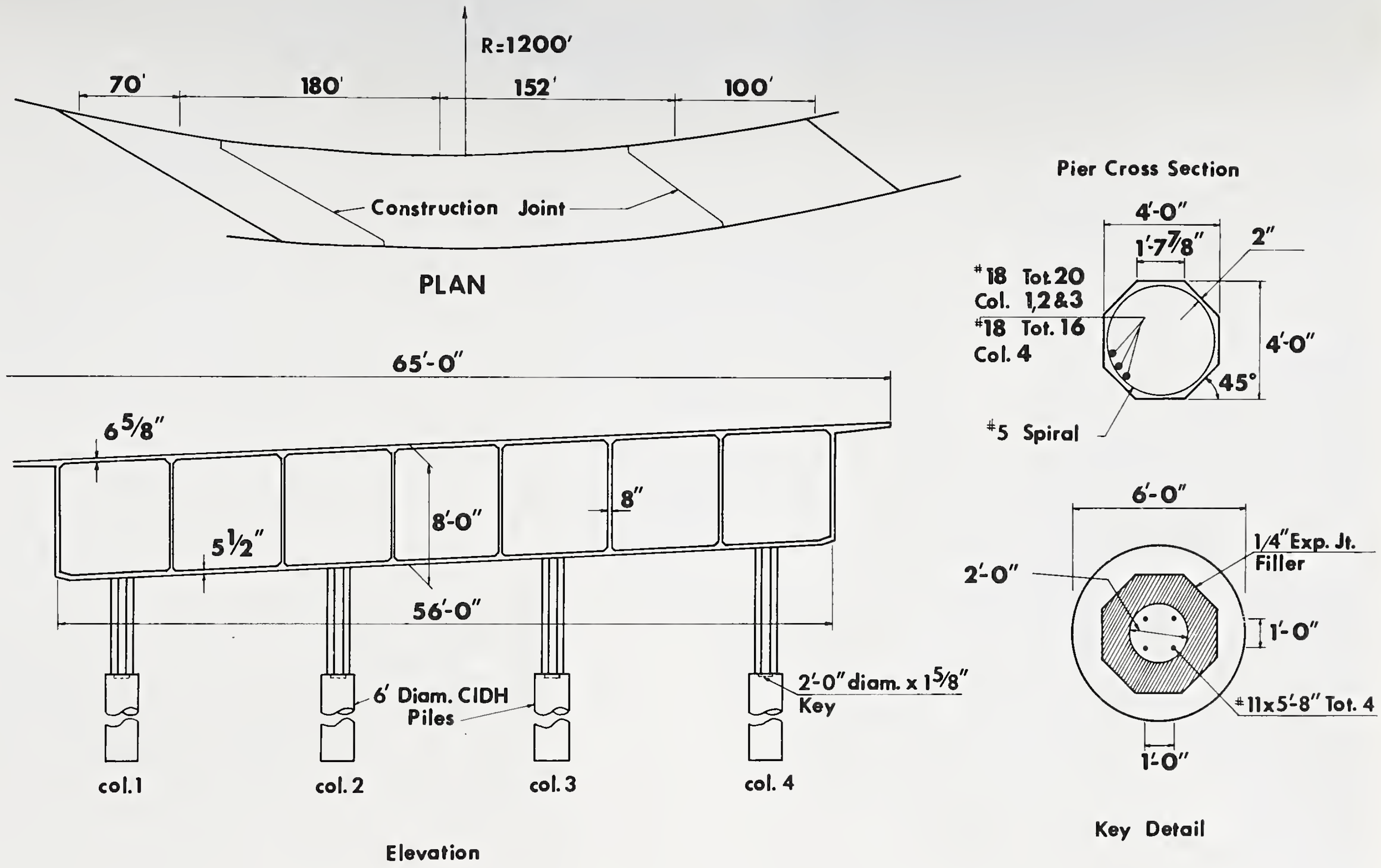



Figure 7.21 Important details of the Northwest Connector Overcrossing.

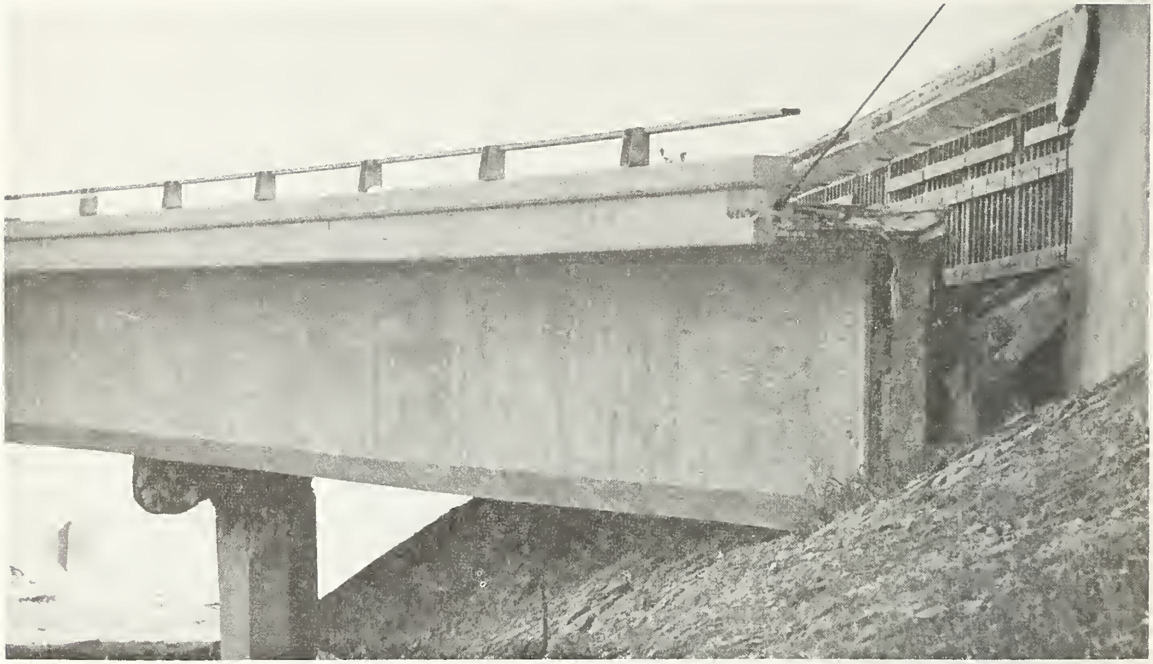


*Figure 7.22 Abutment and pier damage at the western end of the Northwest Connector Overcrossing.*

*Figure 7.23 Spiral column failures in bent 2 at the west end of the Northwest Connector Overcrossing.*

*Figure 7.24 Close up of exterior spiral column.*

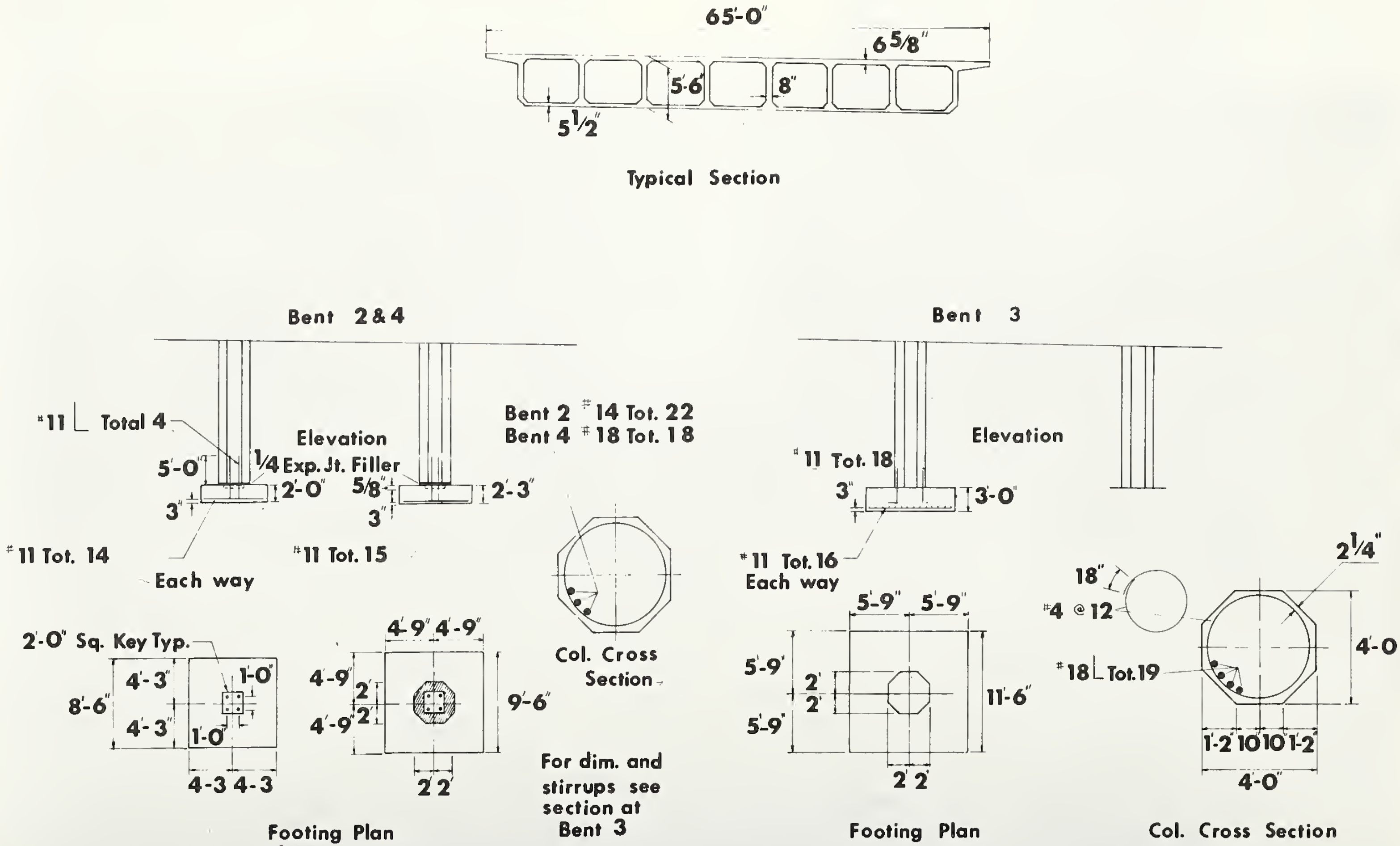



*Figure 7.25 Details of the Foothill Boulevard Undercrossing.*

The columns were provided with No. 4 hoop ties at 12-in centers and the hoops had an 18-in lap length around the circumference. There was an important difference in the types of foundations between the northern or left bridge and the more heavily damaged southern or right bridge. The less damaged northern (left) bridge was founded on spread footings which were placed on top of piles in interior bents 2 and 3 and spread footings without piles in bent 4. In contrast, the southern or right bridge was carried on spread footings on all bents with no piles being used. The right bridge was also narrower in cross section because of the ramp space provided in the left bridge. The typical base detail at the bottom of the columns in bents 2 and 4 provided for a 2-ft square by 1 5/8-in thick keyway, with 4 No. 11 bars extending from the column into the footing into the center of the key, thus providing "hinged" bases on these column lines. In contrast, the middle bent 3 had full dowel splices into the footings for all bars.

The major damages noted in this bridge was sustained by the southern or right bridge and were mostly in the columns and abutments, as can be seen in figure 7.26, which shows bent 3 of the right bridge in the foreground. The bridge had apparently twisted in a clockwise direction and it appeared that it had rotated about the third column from the right hand side of the photo, which is relatively undamaged. Severe damage was encountered in the other columns, as can be seen from the closeup in figure 7.27, which indicates complete destruction of the core concrete and buckling of the bars, as well as loss of the relatively light hoops provided. Note the hoops lying in the rubble in the foreground. The columns of bents 2 and 4 were set in a series of closure walls and movement could take place only at the top, so the damage to these columns was fairly well confined to the very top portion, as shown in figure 7.28. In spite of the massive damage to the support elements, the girders appeared to be in surprisingly sound shape, and visual examination did not indicate any appreciable evidences of concrete cracks large enough to indicate reinforcement yielding.

## MISCELLANEOUS BRIDGE DAMAGE

Throughout the remainder of the north San Fernando Valley, numerous bridges were damaged in a much less dramatic manner. A typical sight was extensive cracking in the abutments, similar to that shown in figure 7.29. In this case a major crack ran through the earth fill up to the bridge abutment, through the abutment and out the other side of the abutment. The bridge in this case was displaced

*Figure 7.26 Column failures in bent 3 of Foothill Boulevard Undercrossing.*

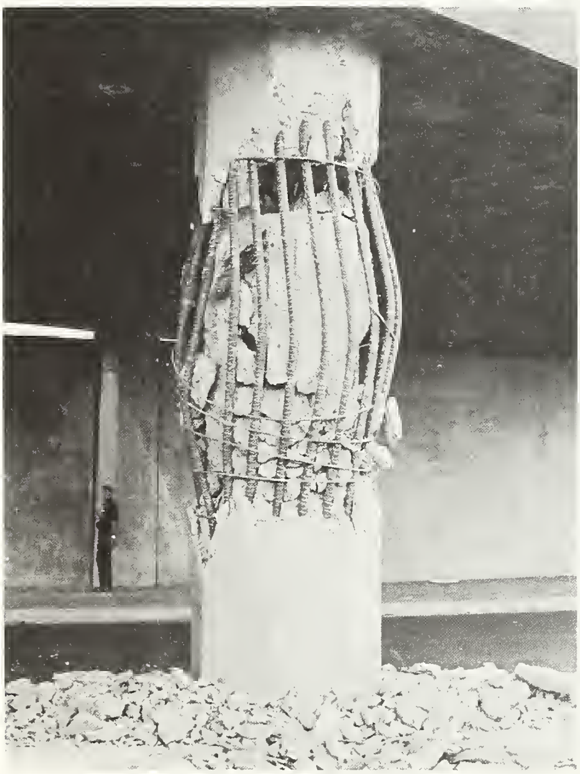


*Figure 7.27 Close-up view of failure of a tied column in bent 3. Note hoops on ground in debris.*

*Figure 7.28 Damaged columns in bent 4 with 10-in closure wall which was separated from the columns by 1/2-in filled expansion joints.*

*Figure 7.29 Typical abutment damage.*

both longitudinally and transversely over 6 in, as can be seen in figure 7.30. This particular structure was a continuous reinforced concrete box girder bridge, located between the Palmdale and Foothill interchanges.

Bridges supported on tall, slender columns and piers seemed particularly susceptible to movement and bridges located several miles north of the Palmdale interchange indicated substantial movement at hinges, as shown in figure 7.31, and extensive butting of one section against another causing spalling as shown in figure 7.32. However, these damages can be classified as minor. Many of the bridges with heavy abutments came through with relatively light damage to concrete slope paving and no visible damage to the structure, as did the single span prestressed box girder bridge shown in figure 7.33. This bridge was located on the Foothill Freeway at Glenoaks Boulevard and is only a few blocks from Olive View Hospital.

## 7.3 Damage to Roads and Streets

Throughout the entire northern San Fernando Valley ground movement caused extensive damage to roads, streets, and highways. Only a few examples of damage which occurred at the locations shown in figure 7.34 are discussed here.

BLUCHER AVENUE, Location 1, Figure 7.34

Blucher Avenue (figs. 7.35 through 7.37) was extensively damaged due to lateral and vertical movements of several feet. The extent of ground movement in the area is further illustrated by the ground cave-in shown in figure 7.38.

HUBBARD AVENUE AND GLENOAKS BOULEVARD, Location 2, Figure 7.34

Many major streets; such as Hubbard Avenue, San Fernando Road, and Glenoaks Boulevard were badly damaged and total reconstruction was reported as the only solution.

Some of the most spectacular damage occurred at the intersection of Hubbard and Glenoaks. As shown in figure 7.39, repair work was underway almost immediately. Considerable street damage was caused by gas line explosions. Further evidence of the ground movement is shown in figure 7.40.

ORANGE GROVE AVENUE, Location 3, Figure 7.34

Damage to streets, driveways, and sidewalks is illustrated is figures 7.41 and 7.42. The ground movements in this area were on the order of four feet horizontally.

*Figure 7.30* *Longitudinal and transverse movement of cast-in-place box girders with respect to abutment sections.*

*Figure 7.31* *Typical movement at a hinge joint of a slender box girder bridge.*

*Figure 7.32 Typical movement and spalling at a joint in an elevated box girder bridge, caused by battering at the joints.*

*Figure 7.33 Typical damage to wing walls and sloped paving on a single span prestressed concrete box girder bridge.*

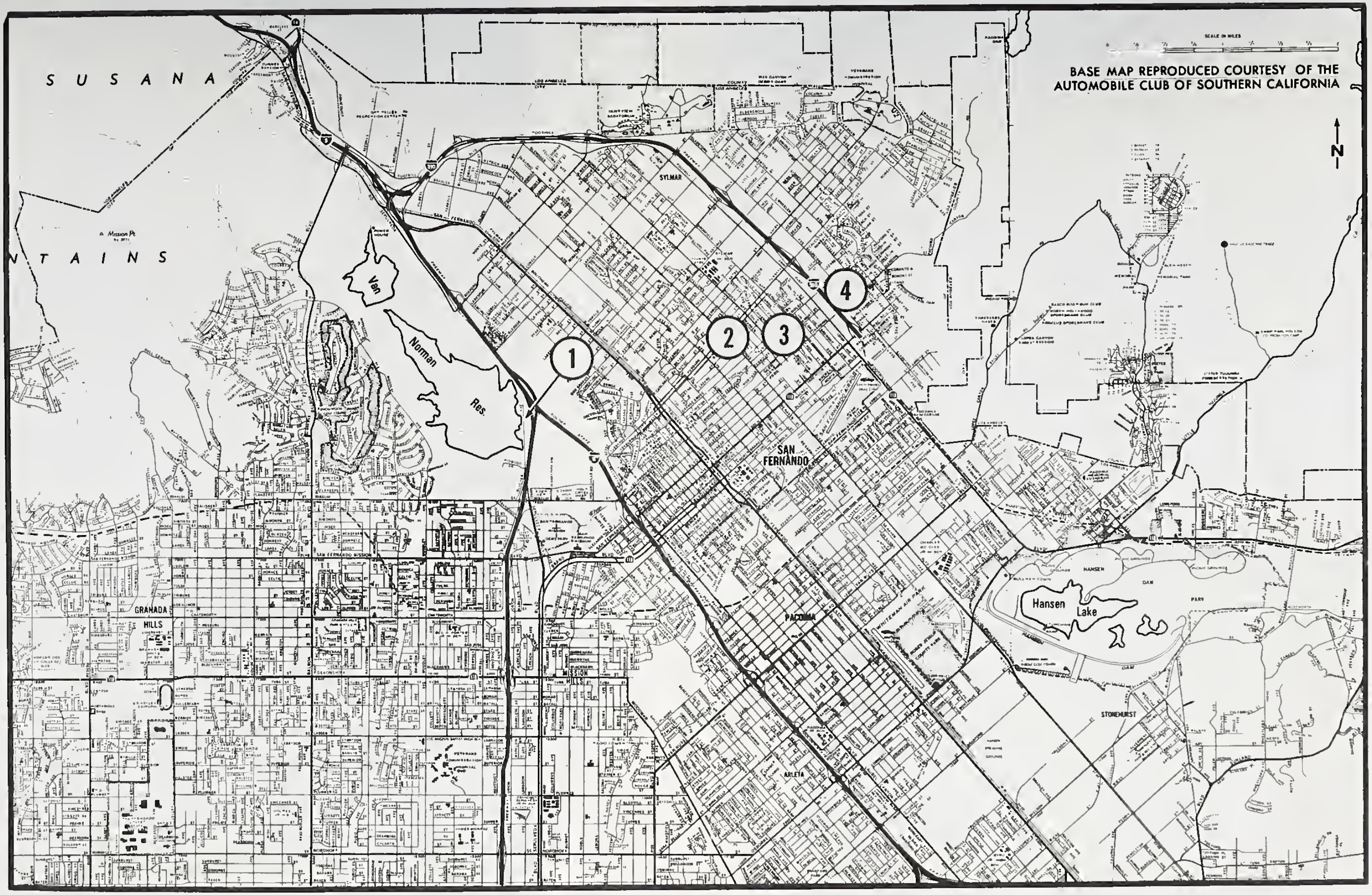


Figure 7.34 Some Locations of Damaged Roadways.

*Figure 7.35 Road cave-in and ground cracks along Blucher Avenue along the eastern edge of the lower Van Norman Reservoir.*

*Figure 7.36 Concrete road slabs were crushed by the ground motion in this location.*

*Figure 7.37* One of ground cracks along the road had a depth of at least 14 ft.

*Figure 7.38* Ground cave-in (2-ft wide) near the damaged road slabs shown in figure 7.35.

*Figure 7.39* *Broken gas, water, and sewer lines being repaired near intersection of Hubbard Avenue and Glenoaks Boulevard, San Fernando.*

*Figure 7.40* *Considerable ground movement is evident in the parking lot of the shopping center at the corner of Hubbard Avenue and Glenoaks Boulevard.*

*Figure 7.41 Sidewalk along Knox Street, near Orange Grove Avenue, San Fernando. Horizontal ground movement is clearly evident.*

*Figure 7.42 Damaged street and sidewalk along 8th Street, between Orange Grove Avenue and Fernmont Street, San Fernando.*

HARDING ELEMENTARY SCHOOL, Location 4, Figure 7.34

Street damage in Sylmar near the Harding Elementary School is shown in figure 7.43.

Both portland cement and asphaltic concrete pavements were subject to fracture in areas of appreciable ground movement. Fractures were of both the tensile and compressive type, depending on the nature and magnitude of the movement. In a number of cases rupture of the pavement was caused by breakage in the utility lines underground, or by erosion of the substrata, due to the flow of escaped liquids. Throughout the mountain area a number of roads were blocked and in some cases badly eroded by slide action. However, this type of damage has been widely seen in previous earthquakes and is certainly to be expected in view of the large displacements of soil masses reported in this particular earthquake.

## 7.4 Bridge Design Procedures

In general the major structures which suffered severe damage or destruction in the earthquake were designed under the general provisions of the American Association of State Highway Officials Standard Specifications for Highway Bridges, 9th edition, 1965, with revisions and as supplemented by the California Highway Department Bridge Planning and Design Manual. Design live loading was based on HS20-44 and Alternative. Reinforced concrete design was based on working stress theory with an allowable concrete compressive stress in flexure of 1200 psi, based on a concrete compressive strength of 3000 psi. In a few of the important columns the concrete strength was increased to 4000 psi. The modular ratio n was ordinarily taken as 10 and the allowable steel stresses as 20,000 psi, based on an intermediate grade reinforcement (Grade 40). In the No. 18 bars used in the Palmdale interchange, Grade 60 steel was used with an allowable tensile stress of 24,000 psi.

### SEISMIC DESIGN

The proper design of bridges in seismic areas is extremely complex. It is not simply a question of the adequacy of estimating the seismic force or seismic coefficient, but rather one of the entire design, detailing, and construction procedures. Any assumptions made in setting the best estimate of the seismic forces should be integrated in the design criteria and must be carried out in all subsequent details and considered for all possible failure modes. The method of analysis used, the determination of limit states, and the overall stability of the structure

*Figure 7.43  Buckled street in Sylmar near Harding Elementary School. (Los Angeles Times Photo.)*

must be considered on the same rational basis. Design considerations should not be limited to questions of strength but should also consider deformations and deflections since the inability of a support detail to either restrain deformation or provide positive support under extreme deflections can cause a precipitous failure. Details concerning the reinforcement of individual members, joints, and connections must be developed carefully to ensure that the full ductility assumed in the analysis can be developed. In special cases where very slender structures are involved or where differences can exist between centroids of lateral stiffnesses and lateral forces, comprehensive analyses should be carried out which consider the dynamic nature of the problem. In addition, on alluvial-type soils the interaction of the foundation with the soil and with the structure should be examined under both static and dynamic conditions. Where a considerable backlog of experience with a particular type of design exists in an earthquake area, the design procedures can be simplified to equivalent static load cases based on satisfactory performance of structures in previous earthquakes. However, the design of new and somewhat unconventional structures should be carefully checked both analytically and experimentally for possible weaknesses under earthquake-type loading.

In the subsequent sections information will be provided concerning the general provisions for seismic design and several different guideline specifications. Wherever any comparisons are made between specifications, it will be on the basis of earthquake forces only and using as a general criterion the working stress and yield point values of Grade 40 reinforcement, which was the type of reinforcement most widely used in the highway structures in the San Fernando Valley area.

Most specifications for earthquake forces to be applied to bridge structures use the concept of the equivalent static lateral load and are expressed on a force basis with little written emphasis on the need to check deflections and deformations, particularly as they might effect stability of the overall structure. The equivalent static load expresses the postulated earthquake induced dynamic loadings as static forces similar to wind loading, using a coefficient which relates the lateral or vertical acceleration to the acceleration of gravity. The choices of these coefficients vary widely under various specifications.

## 1965 AASHO SPECIFICATIONS

The 1965 AASHO Specifications were basically a working stress design specification in which Article 1.2.22, governing loading combinations, indicated that in loading combinations with earthquake loading (Group VII) a one-third increase in the basic unit stresses would be allowed. The lateral forces specified by Article 1.2.20 were expressed as a percentage of the dead load of the structure and the live load could be disregarded. The values of the lateral force coefficients range from 2 percent for structures on spread footings on material rated at less than 4 tons per square foot to 6 percent for structures founded on piles.

The magnitudes of these coefficients are far less than the accelerations reported in the San Fernando earthquake and thus would demand a high degree of dynamic damping and ductility for structures in severe earthquake regions. The specification makes no mention of vertical accelerations and does not include any factor which would reduce or increase the coefficients according to geographical location and probability of seismic activity.

## CALIFORNIA BRIDGE PLANNING AND DESIGN MANUAL

The California State Highway Department has implemented a somewhat more comprehensive requirement, which was adapted from the 1960 Structural Engineers Association of California "Recommended Lateral Force Requirements". This seismic force computational procedure was outlined in the March 1968 California State Highway Department Bridge Planning and Design Manual, which is referenced in the plans for a number of the damaged freeway structures. The California requirement is as follows:

2-25 Seismic Forces

All structures except underground structures and retaining walls shall be designed to resist earthquake forces (EQ) in accordance with the following equations. A nomograph is available on Page 5-65 of Vol. III of the Bridge Planning and Design Manual for solving equation (2).

(1) EQ = KCD

EQ = The force applied horizontally at the center of gravity of the structure. This force shall be distributed to supports according to their relative stiffnesses.

K = Numerical coefficient representing energy absorption of the structure:

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

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

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

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

C = Numerical coefficient representing structure stiffness

(3) $T = 0.32\sqrt{\frac{D}{P}}$ for single story structures only

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

The EQ forces calculated above shall never be less than 0.02D. Special consideration shall be given to structures founded in soft materials capable of large earthquake movements, and to large structures having massive piers.

Very approximate checks were made using these equations for a bridge similar to the tall overcrossing at the Palmdale interchange and the lateral force coefficient was found to be between 3 and 4 percent. Insufficient design detail was available for a more exact check. Again, the magnitude of these coefficients would imply extremely large ductility factors and in view of the reported accelerations during the San Fernando earthquake would appear to be inadequate. The California specification does caution the designer to give special consideration to structures on soft materials and to large structures with massive piers, but does not specify any vertical acceleration levels or vertical force coefficients.

## SURVEY OF JAPANESE DESIGN REQUIREMENTS FOR BRIDGES

The various agencies responsible for the design of railroad and highway bridges and expressways in Japan have adopted considerably more severe equivalent static load coefficients than those outlined for either AASHO or the California State Highway Department. A direct comparison is difficult because of the different values for allowable and ultimate material properties. In addition, in most of the specifications the Japanese allow a 50 percent increase in the allowable stresses for both concrete and steel reinforcement for earthquake load in contrast to those for dead and live loads. However, when both the design stresses and the overload factors are considered, the ratio of allowable earthquake design stress to yield point for the reinforcement is of approximately the same magnitude in Japanese practice and in U. S. practice. Hence, the seismic design coefficients used by the Japanese highway groups can be compared to those in use in the United States on an order of magnitude basis with reasonable dependability.

The 1964 editions of the Japanese Road Association, "Specifications for the Design of Reinforced Concrete Highway Bridges," and "Specifications for the Design of Steel Highway Bridges," specify horizontal seismic coefficients for highway bridges as a function of probability of earthquake intensity and foundation conditions as shown in table 7.1. It can be seen that in areas of high seismic probability the lateral force coefficients vary from 0.15 to 0.35, or substantially above the values in the U. S. specifications. In addition, a value of 0.10 is specified for the standard vertical seismic coefficient in all cases.

The Tokyo Expressway Public Corporation, 1967, "Aseismic Design Standard," is used for expressways in the Tokyo metropolitan area and is more closely tied to soil conditions. The design coefficients are shown in tables 7.2 and 7.3 (In these tables the value N is defined as the number of blows necessary to produce a penetration of 30 centimeters when a hammer weighing 63.5 kilograms is dropped onto a standard sampler from a height of 75 centimeters). Again it can be seen that the range of seismic coefficients is appreciably above those called for in the U. S. specifications.

In addition to the increased basic design coefficients, various Japanese specifications call for an increase in the design seismic coefficient for those parts of the structure higher than 10 to 15 meters above the ground surface. Those increases in coefficients for tall structures can actually result in total horizontal coefficients of as much as one-half gravity. Suggested distributions for the application of seismic coefficients below the ground surfaces are also made. An excellent summary of the various design provisions is contained in "Earthquake Resistant Design for Civil Engineering Structures, Earth Structures, and Foundations in Japan," compiled by the Japan Society of Civil Engineers in November 1968.

## 7.5 Summary and Recommendations

In spite of evident conscientiousness in design, detailing, construction, and maintenance, a number of important highway structures failed completely and catas-

Table 7.1 Japan Road Association, Horizontal Seismic Coefficients for Highway Bridges

| Locality \ Ground Condition | Soft | Medium | Hard |
|---|---|---|---|
| Where strong earthquakes have frequently occurred. | 0.35 to 0.30 | 0.30 to 0.20 | 0.20 to 0.15 |
| Where a strong earthquake is known to have occurred. | 0.30 to 0.20 | 0.20 to 0.15 | 0.15 to 0.10 |
| Other localities. | 0.20 | 0.15 | 0.10 |

Table 7.2 Design Seismic Coefficients for Tokyo Metropolitan Expressways

| Seismic Coeff. \ Type of Ground* | I | II | III | IV |
|---|---|---|---|---|
| Horizontal | 0.20 | 0.24 | 0.27 | 0.30 |
| Vertical | 0.10 | 0.10 | 0.10 | 0.10 |

*The numbers indicating the types of grounds correspond to the numbers given in table 7.3.

Table 7.3 Classification of Grounds

| Thickness of Alluvium and Kanto Loam Layer \ Ground Conditions of Alluvium and Kanto Loam Layer | Sand & Gravel | Sand, Clay, & Kanto Loam Layer with $N \geqq 5$. | Soft Ground | |
|---|---|---|---|---|
| | | | $2 \leqq N < 5$ | $N < 2$ |
| 0 - 3 m | I (Foundations must be constructed after completely removing this thin layers) | | | |
| 3 - 10 m | III (II)* | III (II) | IV (III) | IV (III) |
| 10 - 25 m | III (II) | IV (III) | IV (III) | IV (IV) |
| Greater than 25 m | IV (III) | IV (III) | IV (IV) | IV (IV) |

*The numbers in brackets maybe used as the type of ground if very rigid foundations such as caissons are used and the length of the foundation is less than 3 times the diameter or the shorter linear dimension of the bottom surface.

trophically in the San Fernando earthquake. Due to the fortunate combination of circumstances, in that a number of the bridges and roads had not been opened to traffic and the earthquake occurred at a time when very few vehicles were on the highways, the actual loss of life and injury to vehicular occupants was small. However, in a more extensive earthquake or in one occurring closer to a built-up region or during a peak traffic period, collapses like those which occurred in the interchanges could produce large numbers of casualties and extensive disruption of vital transportation arteries required by emergency services. In view of this potential danger, all lessons which may be learned from failures should be intensively pursued and design and construction procedures altered as necessary.

The recurring themes which seem to appear in discussions of numerous failures of bridges in earthquakes are true in the bridge failures of the San Fernando earthquake. Generally, superstructures behaved well with major damage to girders occurring upon impact when they fell to the ground. Except for such impacts on falling, the superstructures generally showed only minor damage to joints and near abutments. Throughout the area extensive damage was incurred at abutments where large foundation movements took place and where consolidation and compaction of loose strata produced large differential movements. Consequent settlement and both transverse and longitudinal movement of abutments and piers literally pulled the supports out from under girders, producing heavy demands on other columns and abutments to attempt to stabilize the structure. Lack of restraining devices at bearings, hinges, and joints permitted the girders to pull free from the supports and fall to the ground. In a number of other cases the lateral and torsional forces imposed on the supporting members in addition to the high rate of vertical accelerations caused column stresses which were particularly serious in lightly tied columns, since the poorly confined concrete then failed completely. Both girder-to-column and column-to-pile connections proved especially vulnerable because of the large size reinforcing bars and small amounts of confining reinforcement utilized. Such failures are catastrophic in single column bents since no load redistribution possibility exists and the lack of redundancy or alternate load paths means that loss of a poorly detailed section results in a total collapse.

In view of the extent of serious collapse of important highway bridges and in view of the extensive reports of recorded horizontal and vertical accelerations far beyond those envisioned in formulation of design criteria, it is recommended that an immediate review be made of the adequacy of present requirements for seismic design of highway bridges. This study should not focus completely on the lateral and vertical dynamic or equivalent static design forces, but should consider the complete seismic design problem. Assessment should be made of the adequacy of specifications for ensuring that methods of analysis are capable of correctly representing important structural variables such as appreciable curvature, skewness of span, variation in mass and resistance centroids, and effect of typical foundation conditions.

Particular attention must be paid to the overall design concept to ensure that the structure is provided with an alternate load path or with some type of aseismic connectors or restrainers, so that in the event of an earthquake of such a magnitude, if it is not practical to attempt to keep the structure from being damaged, total collapse will not ensue. In construction similar to that observed in this earthquake, increased widths could be provided on abutment pads and on hinge supports, so that increased longitudinal displacements would be possible without the girders coming completely off the supports. Alternatively, types of connections could be devised which would allow the girder to travel an amount which would be more than ample to overcome shrinkage, temperature, or other ordinary volume changes, but which would then cause additional stiffening and continuity in the system for displacements of the magnitude of those occurring with large foundation disturbances. In the case of the single column bent, where all of the moment must be taken by a cantilever-type action, extreme cases might indicate that the bent will have to be changed to a multiple column design, so that the lateral force moments can be resisted by a couple, with a substantially enlarged moment arm. In any case, it will be especially necessary for tall, slender structures to consider the possibility of deflection of alternate supports in opposite directions, thus imposing severe deformation requirements on the girder and girder seats. The entire structural system must be designed for the forces required to stabilize such a structural condition.

Revised regulations for highway bridge construction should include a factor which expresses the probability of the occurrance of an earthquake of high intensity, a factor which reflects a general soil and foundation condition, a factor which reflects the importance of the structure and the degree of danger in its collapse, and a factor which is an accurate appraisal of the amount of ductility which can be expected in the particular type of structural design and materials. It is not realistic to expect all highway related agencies in the United States to design all bridges to the same identical specifications. However, the guideline specifications should be flexible enough to clearly distinguish between regions and to encourage more accurate microzoning within a particular region. The specifications should also require material properties and connection details which are consistent with the assumed amount of ductility allowed for in dissipation of the seismic energy. Many of the details of the structures in the San Fernando region were inadequate for the development of ductility factors of the order of 4 or more. Specific recommendations to increase the ductility factor should be studied in detail. Among these are:

(1) Reassessment of lateral reinforcement requirements in columns and piers. In many cases the size, spacing, and volumetric percentage of lateral reinforcement seem to be only a token contribution when compared to the load-carrying capacity of the main longitudinal reinforcement. Minimum percentages for ties or hoops, approximately either a portion of the volumetric percentage required for spiral reinforcement, or the minimum requirements for shear reinforcement would be a step in the right direction.

(2) Maximum spacing of column ties and hoops should be limited to one-half the effective depth of the cross section, so as to ensure effectiveness as shear reinforcement.

(3) Additional column ties, hoops, or spiral wraps should be provided in the end sections of all columns and into the beam column or girder column connections where possible.

(4) If the column drilled shaft pier is to be considered as a column-to-footing connection, then additional transverse reinforcement should be placed in the top of the pier section to prevent concentration of overlapping splitting stresses from the column reinforcement. It is probably more logical to consider this connection as a type of indirect splice and to follow modified splice rules which would call for special precautions for the splicing of No. 18 bars, caution against having a splice at the point of maximum tensile stress, and call for additional lateral reinforcement in the splice region.

(5) The use of lapped splices for lateral reinforcement, such as column hoops, should be discouraged and all hoops or ties should be positively anchored by hooks or embedment into the core. Wherever close spacing of multiple, large diameter reinforcing bars, such as No. 14 or No. 18, are required in an anchorage zone (as at pier top-girder diaphragm connections) a portion of the bars should be positively anchored to the main transverse reinforcement in the anchor member.

As a result of the observations made following the earthquake, it is apparent that additional research and study is needed in the following areas of importance to bridge designers:

(1) The dynamic behavior of curved and skewed bridges. It is particularly apparent that these structures are susceptible to severe transverse and oblique movements and comprehensive design procedures are required to ensure safe design.

(2) Behavior of splices of closely spaced and large bars under dynamic loading conditions.

(3) The general field of bond and anchorage requirements for large size reinforcing bars under dynamic loading conditions, with emphasis on a study of the anchorage requirements for column bars in drilled shaft-type footings when subjected to alternating dynamic loading.

(4) Distribution of dynamic loads to cast-in-place drilled shafts in alluvium strata and the effective length of pier-column combinations with various connection details.

(5) Investigation of procedures for developing more effective hinge details to withstand seismic forces and restrain large deformations while still allowing service load level movements.

The California State Highway Department and the many local street and road departments reacted quickly and expertly in restoring the transportation net in the San Fernando Valley to a high degree of capacity within days of the earthquake. Their prompt and efficient action minimized the impact of the millions of dollars of destruction done to the highway and bridge systems, and indicated that a certain degree of damage is certainly acceptable and practical. The goal of future specifications and code revisions should be directed at preventing total collapse and to control other damage to a manageable, repairable degree, minimizing public loss within an acceptable cost framework.

# CHAPTER 8 DAMAGE TO PUBLIC SERVICES AND FLOOD CONTROL FACILITIES

## 8.1 Water Supplies

### CITY OF LOS ANGELES

The City of Los Angeles receives water from several sources: underground wells; High Sierra water through the two Owen river aqueducts; and the Colorado River through its aqueducts. As a result of the earthquake, the Los Angeles water system facilities in the north end of San Fernando Valley were severely damaged.

Immediately following the earthquake, approximately five percent of the area of the City of Los Angeles, primarily around the northern part of the San Fernando Valley, had its water service interrupted (see fig. 8.1). Water service was restored to most of this area within four to five days. However, because of extensive damage to water distribution systems, several areas did not receive water until eleven days after the quake occurred.

### DEPARTMENT OF WATER AND POWER FACILITIES (CITY OF LOS ANGELES)

Total damages to the water supply and distribution systems under the Department of Water and Power are estimated at $48 million. Half or more of this amount will result from replacement cost of the water storage and related facilities at the Upper and Lower Van Norman Reservoirs.

Major damage to the water distribution facilities was confined primarily to the Van Norman Reservoir complex and trunk lines in the Sylmar-Granada Hills area. The complex consisted of two major reservoirs formed by the Upper and Lower San Fernando Dams, a smaller bypass reservoir, a complex of bypass pipelines, penstock for power stations, bypass channels, chlorination stations, and standby pumping stations. The complex is designed so that normal water supply to the City of Los Angeles can be maintained even if one or more facilities are out of service. Other damages in the Upper San Fernando Valley included damage to Maclay Reservoir, 750 broken mains, 756 damaged service connections, 35 gate valves, 44 fire hydrants, 20 safety valves, and 2 storage tanks.

Damages to the Van Norman Reservoir complex and other distribution systems are discussed below with reference to the various items identified by numbers in figure 8.2.

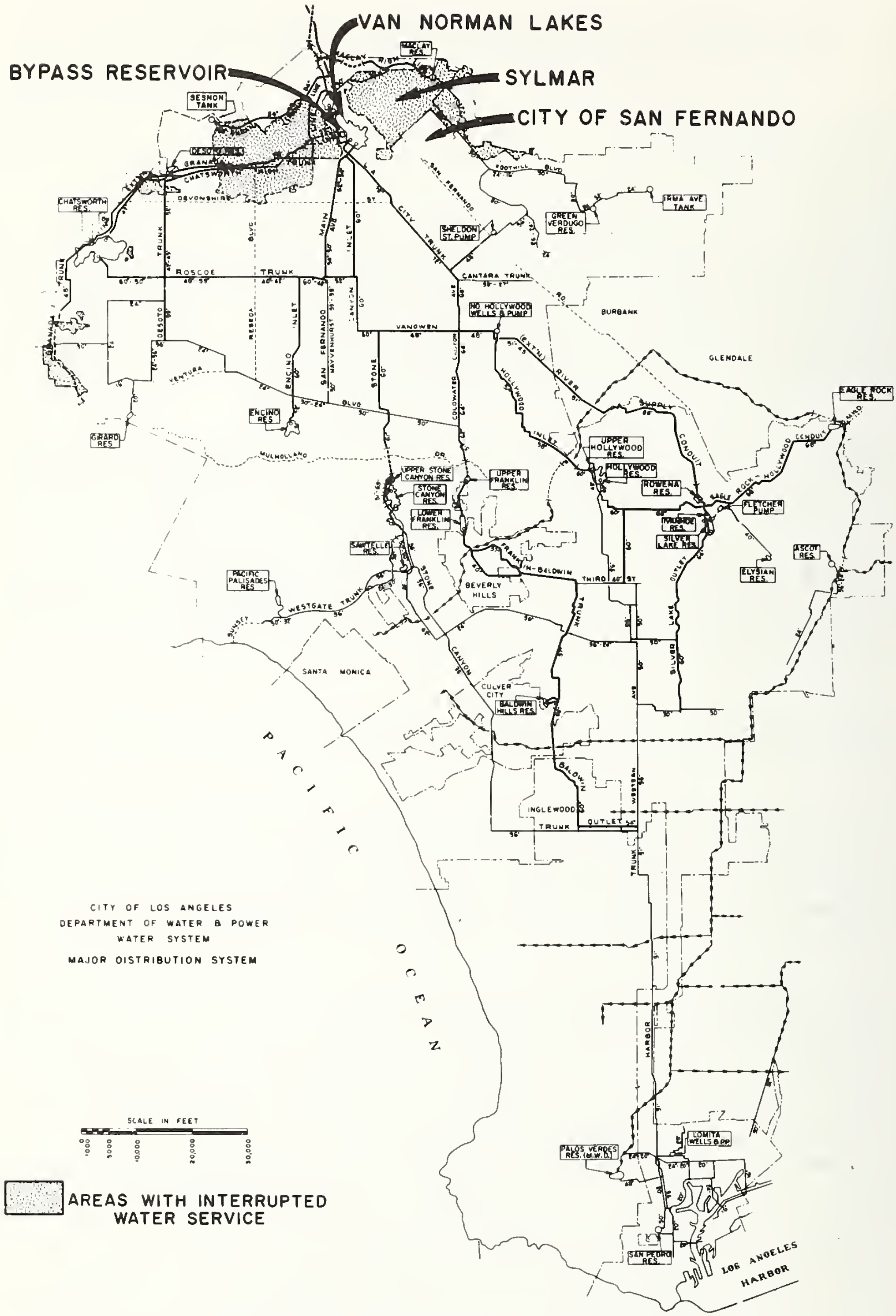



*Figure 8.1 Major water distribution system of the City of Los Angeles.*

1. Both Los Angeles Aqueducts, which deliver 80 percent of the total supply to the city, were damaged in the area immediately above the Cascades. The first aqueduct, built in 1913, delivers some 495 cu ft per second at a hydraulic gradient of 1449 feet. It was damaged at an inverted siphon transition segment (location 1) and was restored to full service condition in two days. The second aqueduct (Saugus Pipeline) delivers 250 cu ft per second at a hydraulic gradient of 1805 feet. It was moderately damaged with ruptures in the 77-inch welded steel pipes (location 2). The Los Angeles Department of Water and Power returned the line to service April 8, 1971.

2. Two major trunk lines, the 54-inch Susana and the 48-inch Granada Trunk lines, (locations 3 and 4) serving areas west of the Van Norman Reservoirs sustained numerous breaks in all-welded steel construction. The Maclay High Line (location 5), serving areas east of the reservoirs, is an old unreinforced concrete culvert. It sustained little damage.

3. The old penstock (location 6), which is made of riveted steel construction, was not damaged. However, the power house (location 7) which is located at the upstream side of the Upper Van Norman Reservoir dropped about three feet, thereby shearing off the end of the penstock. The penstock has been sealed and a bypass pipe (location 8) has been installed to carry water from the penstock into the upper reservoir.

4. The Upper Van Norman Reservoir (location 9) with a capacity of 1800 acre-feet, was damaged. The Upper San Fernando dam, which was built in 1919-1921, is about 1200 ft in length along its axis and about 60 ft high. It settled about three feet and displaced laterally about five feet at the crest. However, the dam did not collapse and has been in use at about one-third capacity. Of the two intakes, the old one (Outlet No. 1) has cracked at the base; whereas, the new one (Outlet No. 2) did not sustain damage. Some water seepage appeared at the downstream side of the upper reservoir dam (fig. 8.3) as a result of damage in the drain system.

5. The 99-inch bypass pipeline (location 10) around the Lower Van Norman Reservoir and the 240 acre-feet bypass reservoir and dam (location 11), both completed in 1970, were undamaged.

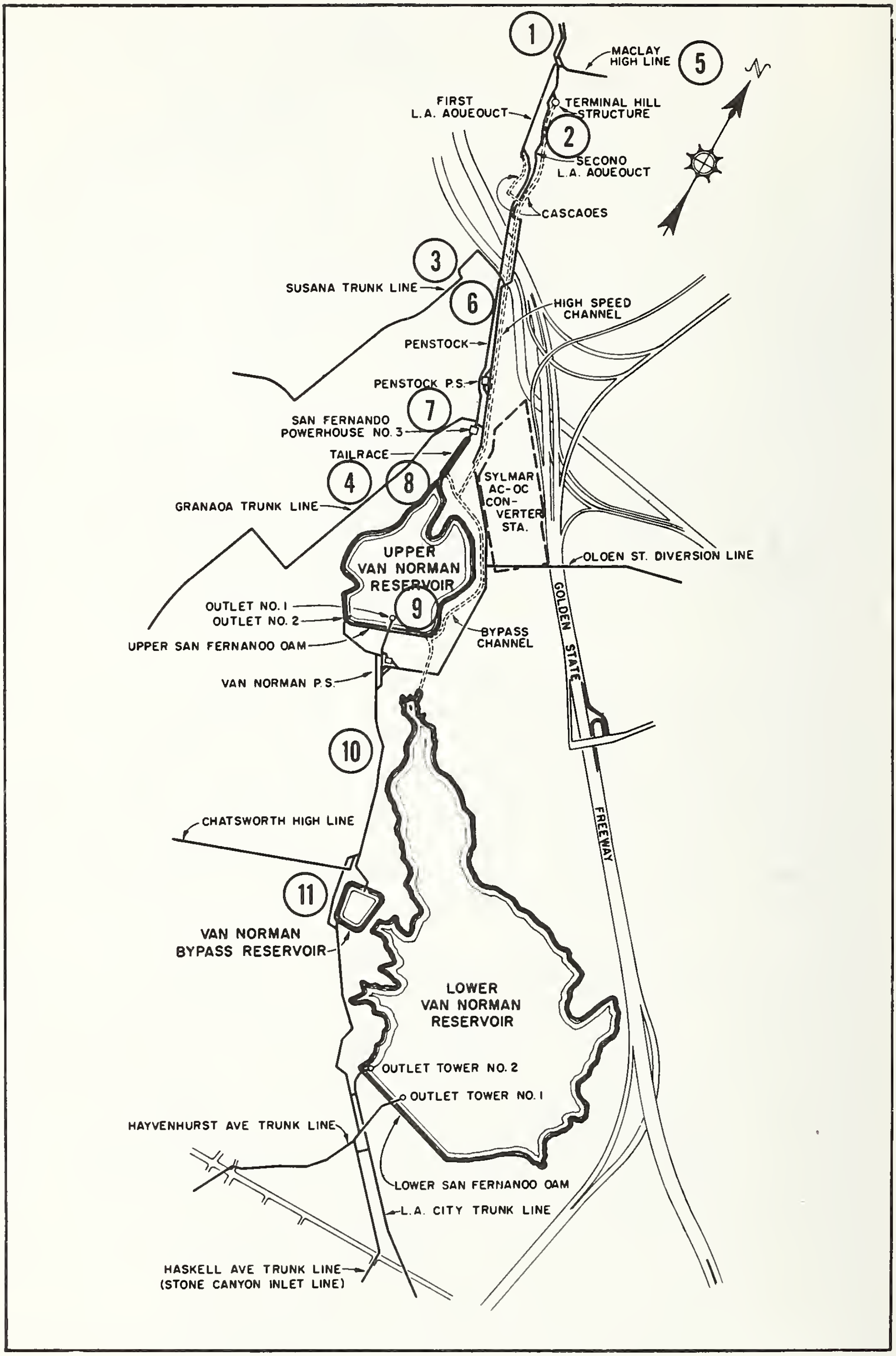



*Figure 8.2 Van Norman Reservoir Complex.*

Both the bypass line and reservoir are now being used to convey water from the Upper Van Norman Reservoir to the Chatsworth High Line and major trunk lines at the outlet to the Lower Van Norman Reservoir.

6. The Lower San Fernando Dam was severely damaged but did not collapse. It was built by the hydraulic fill method in 1913-1915 and raised and improved in 1930 and again in 1940. The reservoir can hold some 20,000 acre-feet of water at the water level of 1134 feet. At the time of the quake, the water level was 1110 feet, containing about 11,000 acre-feet. With this lower water level, although the crest of the dam collapsed during the quake (see fig. 8.4), the dam held back the water. With only a few feet of freeboard to relieve the water pressure on the damaged dam, the water was immediately drained through the collapsed intake at a rate of 800 cubic feet per second (cfs) and discharged by 11 pumps at a rate of 190 cfs. The Lower Van Norman Reservoir is now out of service and is almost completely drained.

## METROPOLITAN WATER DISTRICT FACILITIES

The Metropolitan Water District (MWD) distributes water throughout Southern California. It channels water to the Los Angeles Department of Water and Power through a 242-mile Colorado River Aqueduct. The aqueduct together with its distribution lines escaped major damage. However, MWD's Joseph Jansen Filtration Plant, a $24.8 million facility under construction (85 percent complete at the time of the earthquake) in the Sylmar area, experienced damage to both steel and concrete installations.

The intensity of ground movement at the Jensen Filtration Plant can be envisaged from the buckled 1.9-million-gallon water storage tank shown in figure 8.5. The tank, which was made of 3/8-in thick steel plate, was 30-ft high and 100-ft in diameter. This tank was located in the northwest area of the plant and was reported to be approximately one-half full at the time of the earthquake. Some of the anchor bolts, which held the tank to its foundation, were pulled out of their anchorage. The amount of pullout was, in general, greater on the north and south sides than the east and west sides. This indicates that some rocking motion of the tank occurred during the earthquake about an axis oriented in the east-west direction. The amount of uplift of the tank was greater on the south side than on the north side, as indicated by the amount of pull-out of the anchor bolts. The amount of pulled-out length was several inches on the

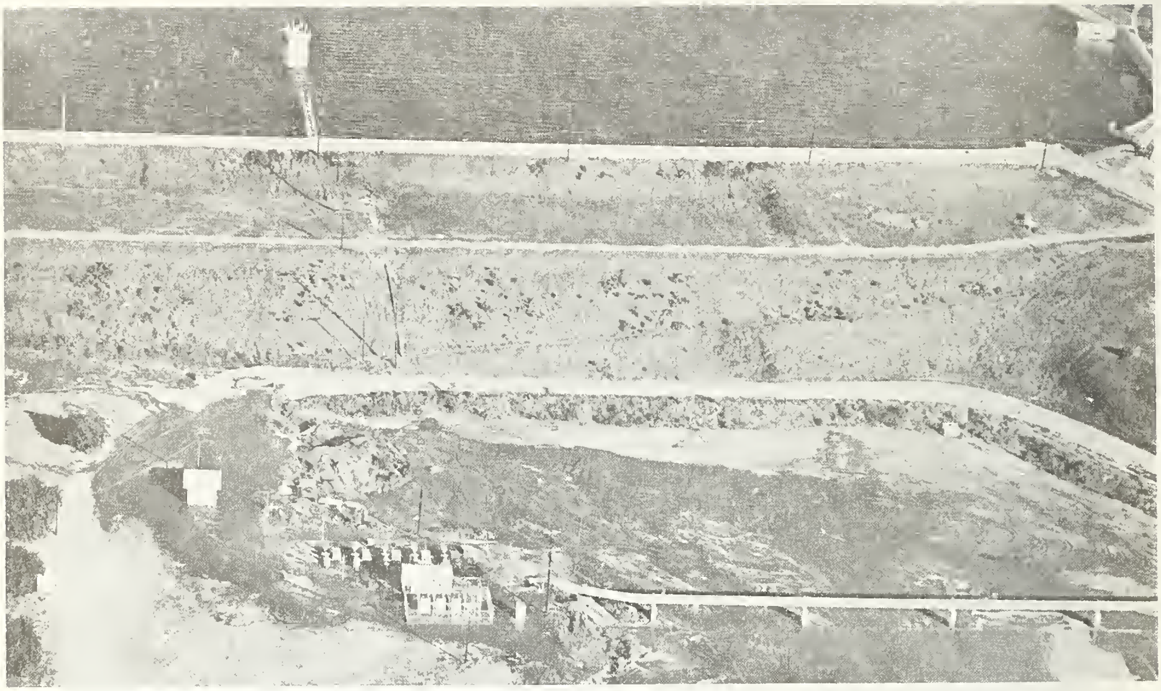
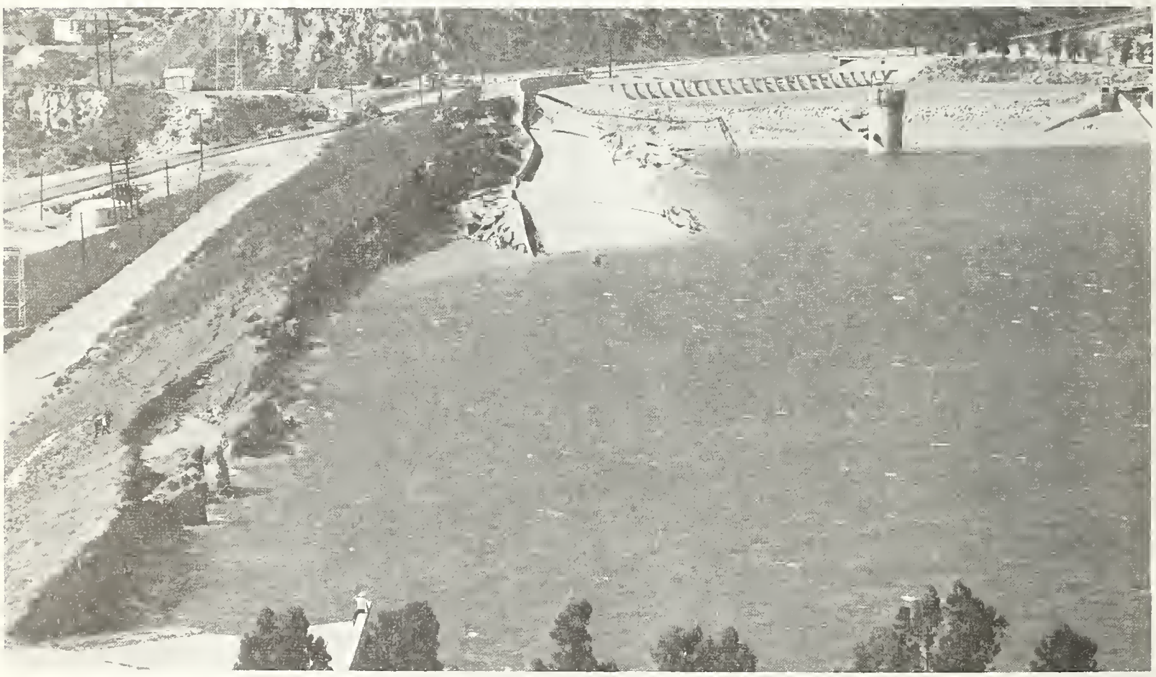


*Figure 8.3 Downstream face of the Upper San Fernando Dam. Note the seepage at the foot of the dam.*

*Figure 8.4 Collapsed crest of the Lower San Fernando Dam.*



north side and as much as 19 inches on the south side (fig. 8.6). This difference in the amount of pull-out can possibly be explained by heavy machinery which was attached to the north side of the tank and apparently restrained the uplift.

In addition to damages to the structures above the ground, the reinforced concrete wall, the 14-in thick concrete roof slab, the 16-in thick concrete floor slab, and the spiral columns of the plant's 50-million-gallon underground reservoir were severely damaged during the earthquake. It has been reported that most damages at the plant were on filled ground which was consolidated by the ground shaking.

Total damages to the MWD are estimated at about $7 million as of April 1971. This damage figure includes the damages sustained by the Jensen Filtration Plant, the Balboa inlet tunnel, the San Fernando tunnel, and other MWD's water distribution facilities in Los Angeles and Ventura counties.

### CITY OF SAN FERNANDO

Water consumed by more than 400,000 inhabitants in the City of San Fernando was supplied by seven underground wells located within and around the city. All seven wells suffered damage during the earthquake. Shortly after the earthquake, two wells were restored to service.

Within the city limits, the water supply system in the extreme north corner, bounded by Harding Avenue, Glenoaks Boulevard and Hubbard and 8th Streets, was completely destroyed. The original six-to-ten-inch diameter cast iron riveted pipes were removed and replaced.

## 8.2 Electricity

Electric power in the Los Angeles area is distributed by the Los Angeles Department of Water and Power (DWP) and the Southern California Edison Company (SCE). The DWP provides the power to customers within the Los Angeles city limits and the SCE to customers in the areas exclusive of the City of Los Angeles. Facilities of both DWP and SCE sustained damages from the earthquake.

### DEPARTMENT OF WATER AND POWER (CITY OF LOS ANGELES)

The DWP suffered an estimated $45 million damage to its power facilities with most of it occurring at four locations. These are the Sylmar Convertor Station, the Sylmar Switching

*Figure 8.5 Buckled steel (3/8 in. thick) backwash tank at Joseph Jensen Filtration Plant.*

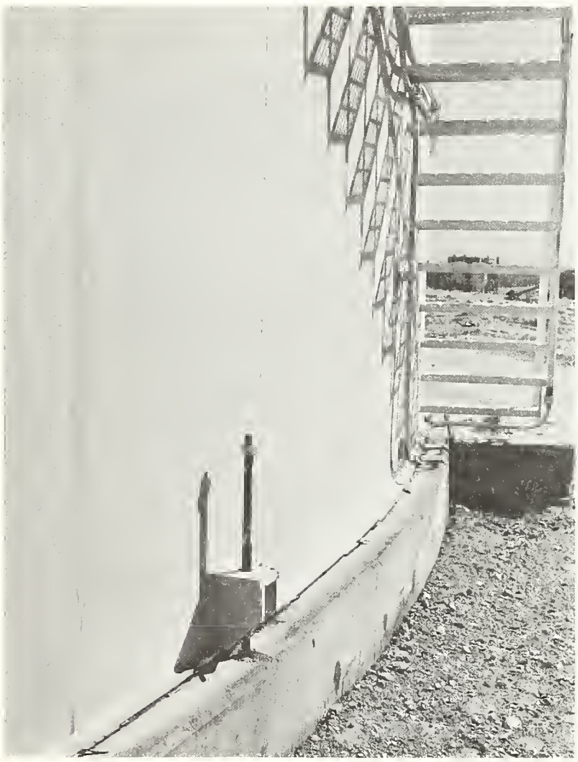


*Figure 8.6 Vertical Movement of tank pulled one-inch diameter anchor bolt out of concrete base.*

Station, the Olive Switching Station, and the San Fernando Power Plant (see fig. 8.7).

As a result of power distribution facility damages, services to 363,000 customers (representing a load of 418,000 kilowatts) was temporarily disrupted. Most service was restored to customers within a period ranging from 15 minutes to 1 1/2 hours. Most power service was restored by the evening of February 11, some 60 hours after the initial shock, with the exception of the evacuated area below the Lower Van Norman Reservoir. Power was restored to this section on February 12, when residents returned from evacuation.

## SYLMAR CONVERTOR STATION

The most serious damage to the DWP's power facilities was the loss of service of the Sylmar d.c. Convertor Station, with an estimated $30 million damage. The station is the southern terminus of the 846 mile, direct current, 800,000 - volt line bringing power from hydroelectric plants on the Columbia River (fig. 8.8). The DWP shares ownership of the station and the 581 mile southern section of the transmission line with the Southern California Edison Company and the cities of Burbank, Glendale, and Pasadena.

With the loss of the d.c. convertor station, which was dedicated in September 1970, the DWP lost 525,000 kilowatts of capacity which constitutes 12.7 percent of its total system capacity of 4,144,000 kilowatts. Anticipated peak load for August 1971 is 3,340,000 kilowatts. As of June 1971, the station was scheduled to resume partial operation in early December 1971, and should be capable of receiving full capacity of 1,440,000 kilowatts by August 1972.

An aerial view of the Sylmar Convertor Station is shown in figure 8.9. The station consists of the main steel frame structure and two adjoining radio-shielded open steel-frame structures. The three-story main structure has 6- to 8-in concrete floor slabs on steel floor beams and metal deck roof. The steel frames have vertical "X" bracings to resist lateral loads. The exposed steel frames which cover the transformer yards have horizontal and vertical "X" bracings and are covered with wire mesh.

The main building structure performed well and damage to it was minimal, although many wide cracks in the parking lot pavement indicate that the structure was subjected to a severe ground motion. Most damages in the main building occurred in the basement where service and equipment sections were joined. Steel columns of the two sections were supported on common concrete columns at this juncture. The upper portion

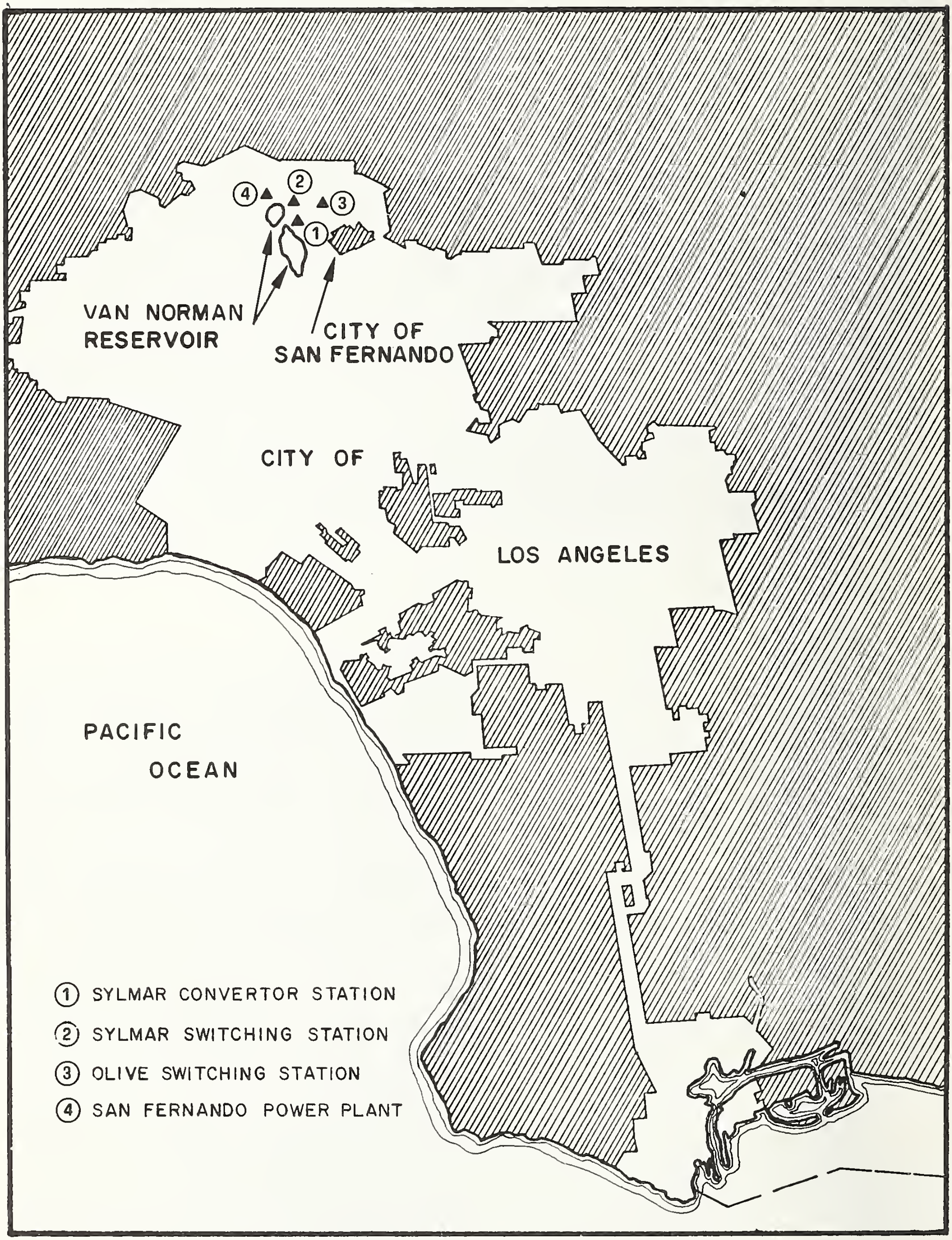



*Figure 8.7 Location of damaged major electrical facilities.*

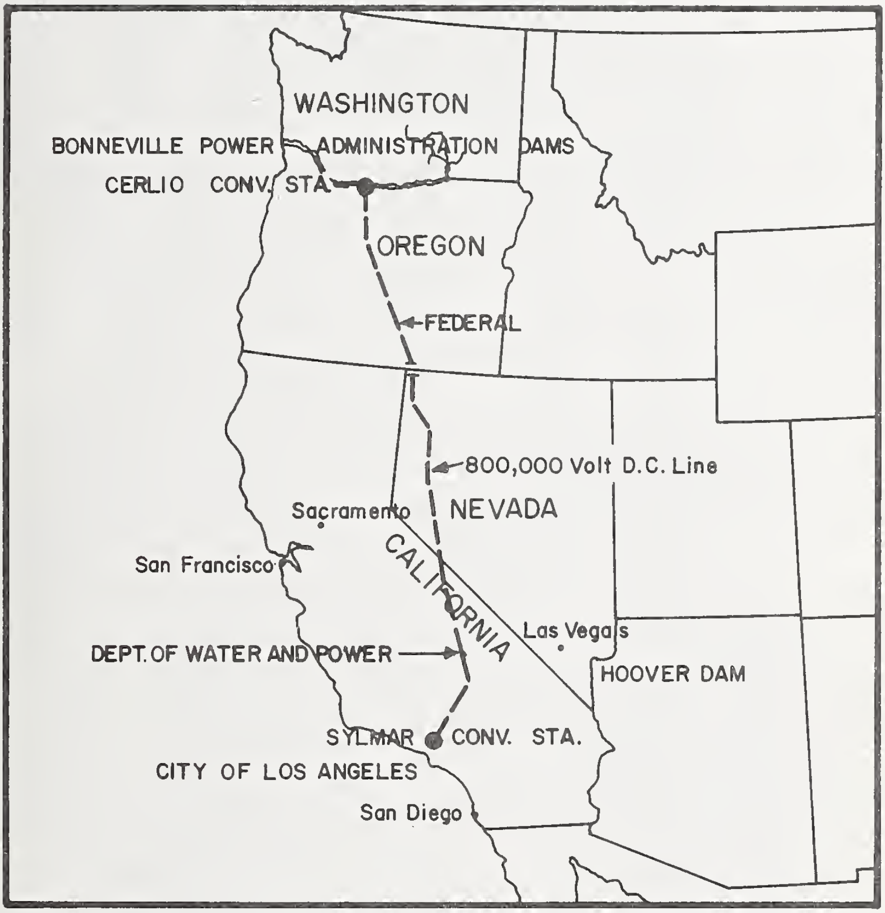



*Figure 8.8 The Pacific Intertie System.*

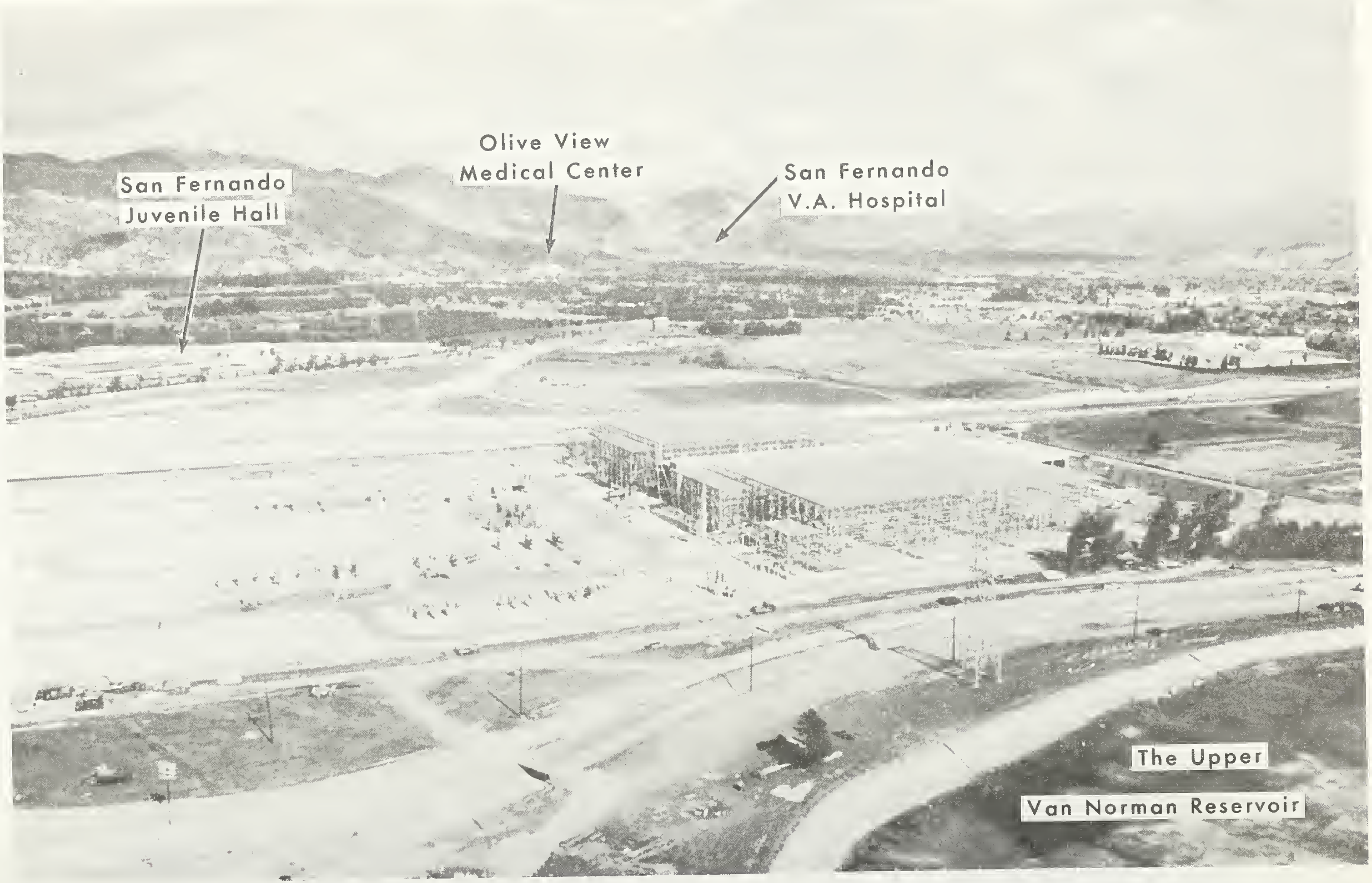



*Figure 8.9 An aerial view of the Sylmar Convertor Station.*

of these columns was damaged due to what appears to be independent movement that took place between the two sections of the building. The floor slab and the basement wall in this vicinity were also damaged. On the other hand, numerous damages were sustained by the bare steel frame structures over the transformer yards. Many braces were buckled, stretched, and broken at gusset plates (fig. 8.10 and 8.11). Some columns were bent and, in several places, cracks occurred between the column pedestal and the concrete wall which surrounded the yard (fig. 8.12).

The most severe damage, which resulted in more than $28 million loss, occurred in the valve bays and outdoor electrical equipment yards. The valve bays house 42 mercury arc convertor valves. These valves, each costing $0.5 million, are designed to change direct current to alternating current, or vice versa. The upper half of the valve is suspended by four insulated wires and the lower half is supported on four insulated supports. Both the upper and lower supports were fractured and cracked, apparently due to vertical and lateral-torsional motions of the valve during the earthquake.

Much outdoor electrical equipment was supported on insulators, (fig. 8.13). Equipment collapsed due to failure of these insulators during the earthquake as shown in figure 8.14.

Several large oscillators were toppled and dislodged due to failure of their anchorage to the foundation pad (fig. 8.15). Each oscillator was attached to four 12-in by 12-in by 3/4-in anchor plates. Each of these plates was anchored to the concrete pad with four 1/2-in studs. A close examination showed that all studs were fractured at the base metal. Some fractured studs showed only partial fusion to the base anchor plate. The base steel plates and a concrete pad to which the plates were embedded are shown in figures 8.16 and 8.17, respectively. In figure 8.17 the broken studs are indicated by arrows.

Numerous large transformers in the radio-shielded yards were moved laterally on their foundation slabs as anchor bolts were sheared off due to lateral forces. A typical example of such a failure is shown in figure 8.18.

## SYLMAR SWITCHING STATION

The Sylmar Switching station is located just north of the Sylmar Convertor Station. Damage to this station caused a disruption to the DWP interconnection to the Southern California Edison system. A temporary interconnection has been established since March 1971.

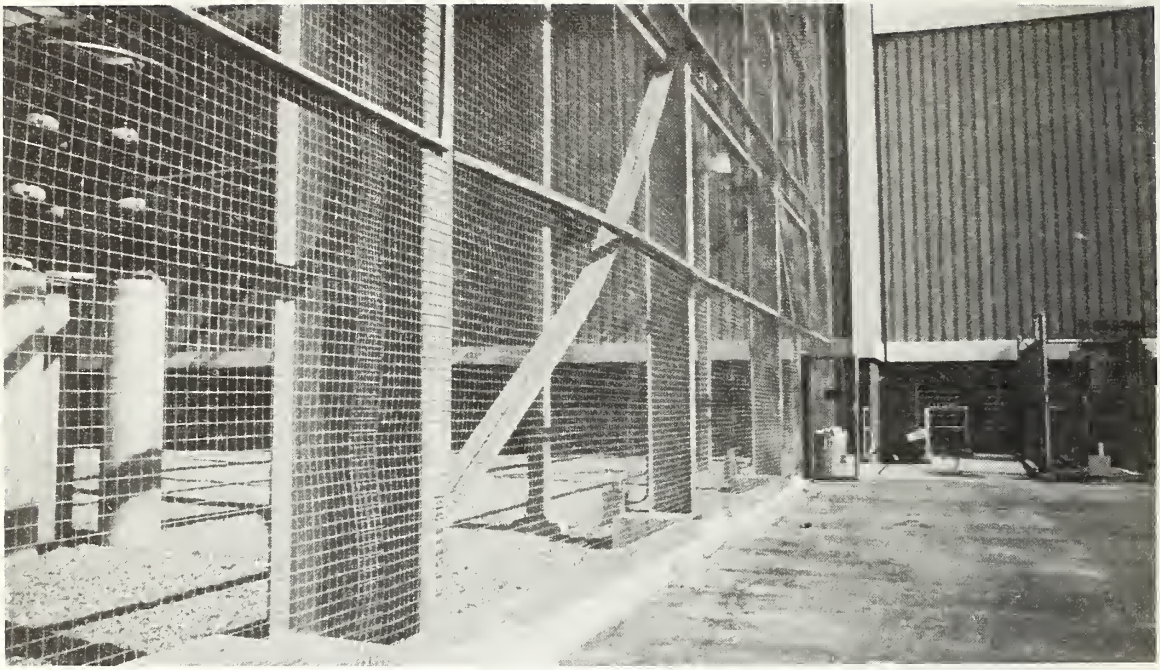


*Figure 8.10 Buckled lateral bracing.*

*Figure 8.11 Fractured gusset plate.*

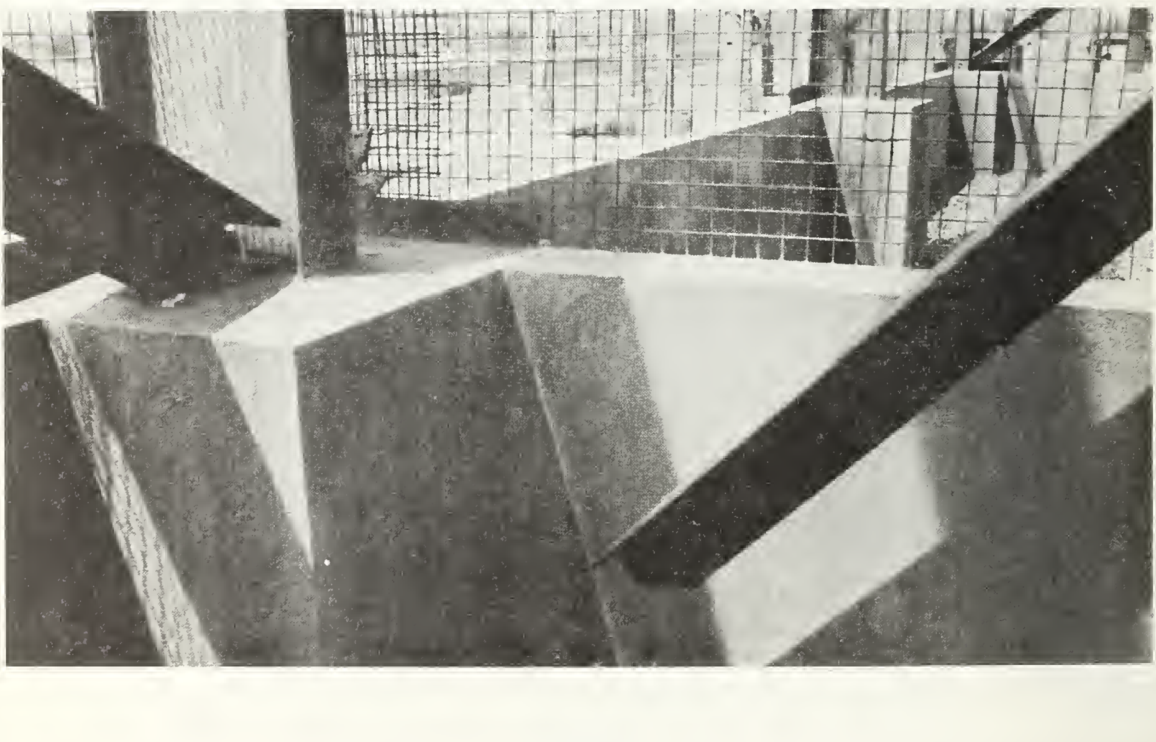


*Figure 8.12 Concrete spalling at one of the joints between a column pedestal and an enclosure wall surrounding the electrical transformer yard.*

*Figure 8.13 Outdoor electrical equipment supported on insulators.*

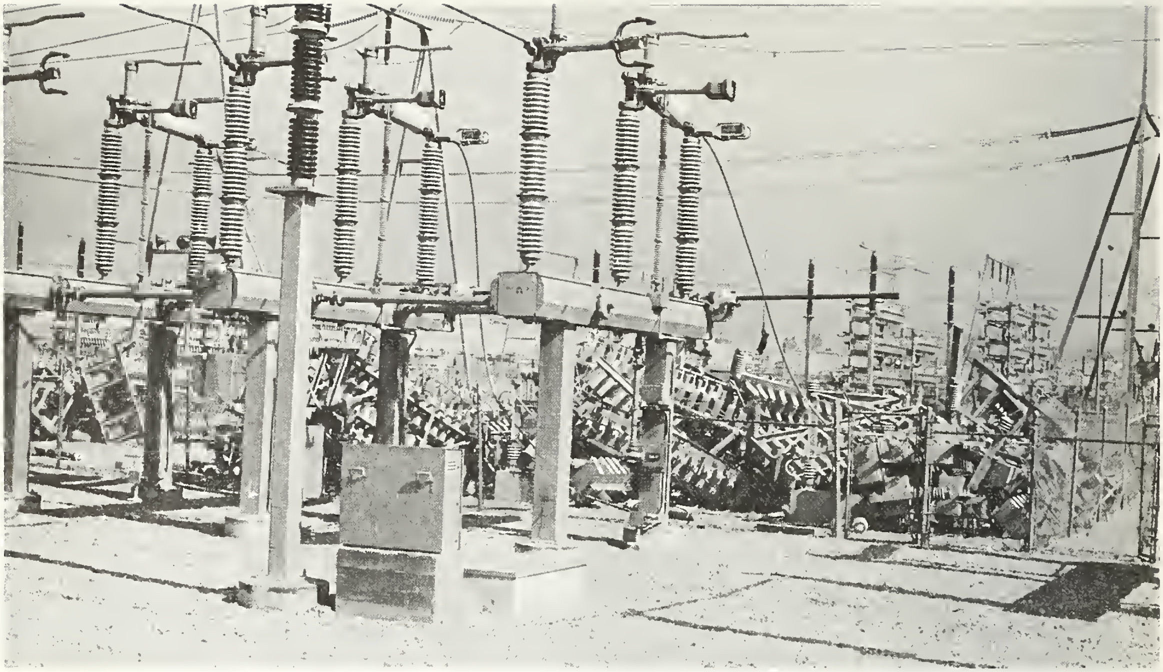


*Figure 8.14 Destroyed electrical capacitor banks (in background) at Sylmar Convertor Station.*

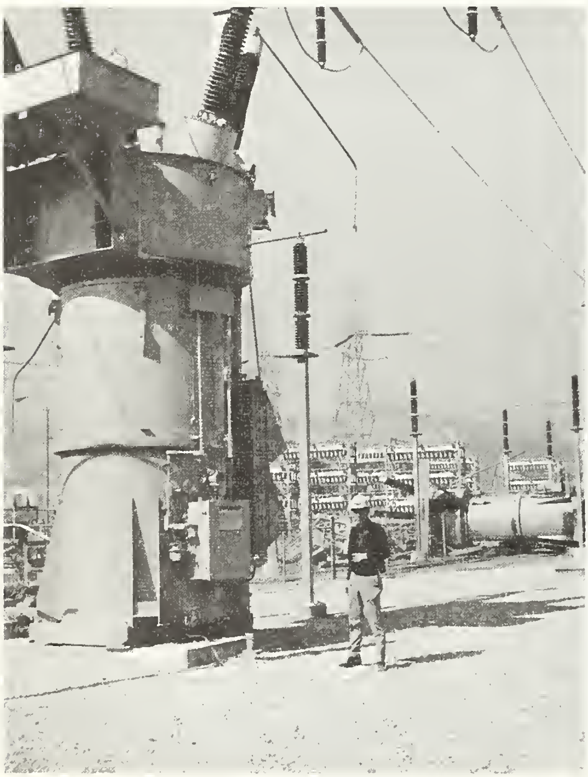


*Figure 8.15 Toppled and dislodged oscillators. (J. H. Rainer photo)*

*Figure 8.16* *View showing bottom of toppled oscillator with its steel base plates and top of the foundation slab.*

*Figure 8.17* *Concrete pad showing the broken studs embeded in it. Porosity in the weld of the upper-right corner stud can be seen in this photograph.*

Damage to equipment at this station included broken circuit breakers and transformers which fell from their supports.

## OLIVE SWITCHING STATION

Olive Switching Station is located on San Fernando Road near Olden Street. This Station, which received power from Owens Valley and the San Fernando Power Plant, sustained damage to power receiving and distribution systems. Much of the insulator-supported equipment in the yard collapsed and transformer banks rolled over.

By the night of February 9, a temporary bypass was constructed to connect the Owens Gorge Transmission line to Receiving Station J, thus making the Owens Gorge Power available to the DWP system.

## SAN FERNANDO POWER PLANT

The power plant which is located just north of the Upper Van Norman Reservoir was dropped about three feet, thereby shearing off the end of the penstock and breaking the bypass gate. This station had a generating capacity of 6000 kilowatts and is out of service indefinitely. Severe ground failure around the plant is shown in figure 8.19.

## OTHER DAMAGED FACILITIES

Two distribution stations in the Sylmar-Granada Hills area suffered damages. One station in Sylmar, located at the corner of Glenoaks Boulevard and Polk Street, had its high voltage lines and voltage regulators damaged. The other station which is located near Balboa Boulevard and Woodley Avenue received mainly structural damages to the tilt-up walls supporting steel columns. Many steel columns were bent and some ruptured in the web.

Other damages to DWP included collapsed underground vaults and many fractured ducts. In Tujunga canyon, several power towers shifted up to 20 feet due to land slides. Others had their lightning arrestors cracked.

## SOUTHERN CALIFORNIA EDISON COMPANY

SCE suffered an estimated damage of $720,000. Damage to the SCE systems were minor compared with those of DWP. Except for that portion of damage related to the Sylmar Convertor

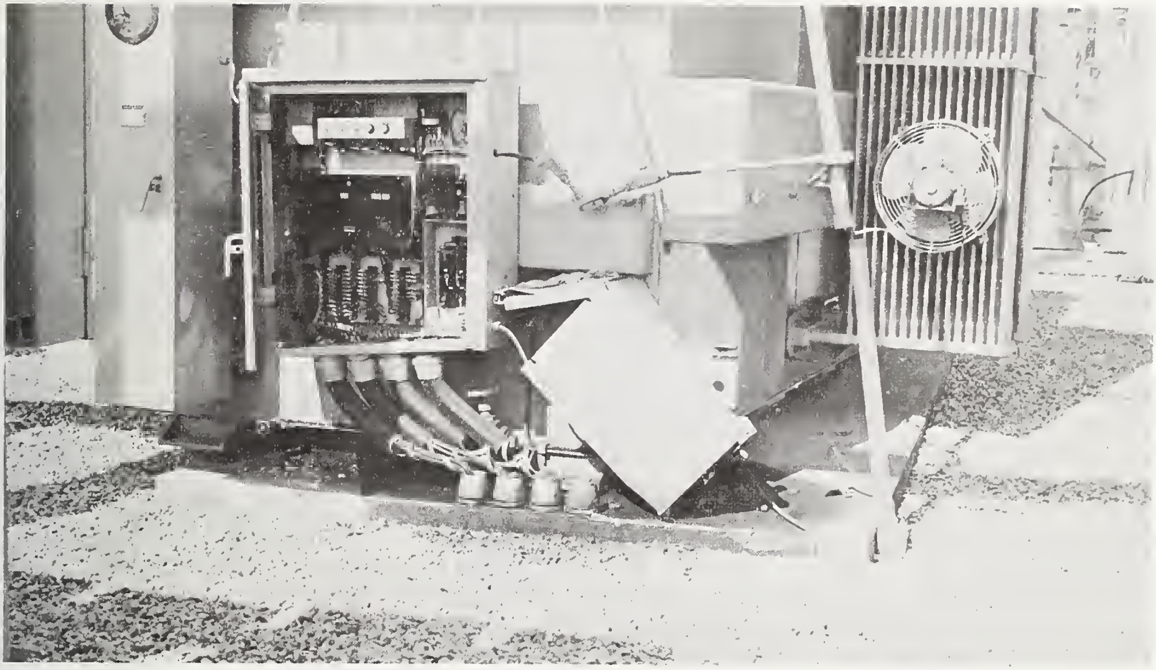
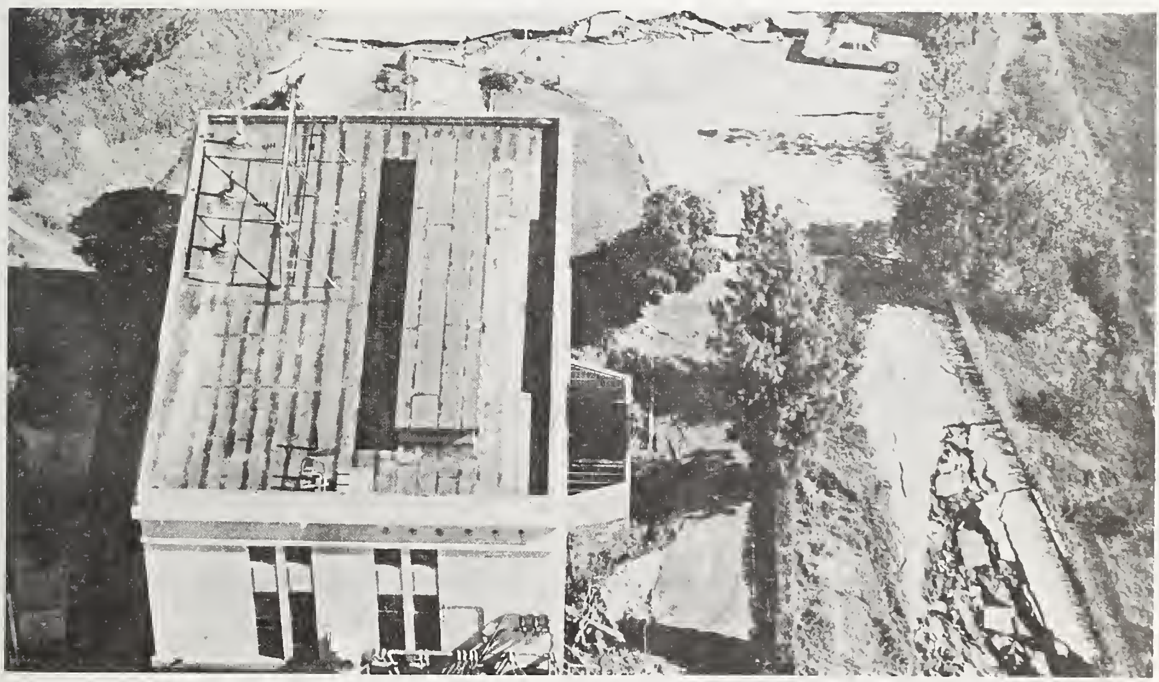


*Figure 8.18 Electrical transformer which moved laterally on its foundation slab.*

*Figure 8.19 Aerial view of the San Fernando Power Plant.*



Station, of which SCE is part owner, damages were confined to transmission lines. A total of 47 towers were found to be unstable due to loss or movement of footings. In addition, while not in imminent danger, as a consequence of deep fissures through and around the tower, a moderate rain could result in excessive erosion and land slides.

Some 330,000 customers were affected by power interruption. Approximately 180,000 customers were interrupted for less than one minute, and less than 1,000 customers had interruptions up to a maximum of 32 hours.

## 8.3 Gas Supplies

The Southern California Gas Company distributes natural gas throughout the Los Angeles area. Estimated damage to the company's facilities was about $2.0 million, mostly in distribution lines, which are typically two- to six-inch diameter steel pipes which are buried three-feet deep. The only area where the distribution piping system was seriously damaged occurred within the 11- to 12-square-mile area in the northeast section of the San Fernando Valley. The area was bounded by Golden State Freeway, Foothill Boulevard, and Pacoima Wash (fig. 8.20 - Area 1). Gas in this area was supplied by major lines at a pressure of 170 psi. Shortly after the quake the pressure was reduced to 35 psi in the distribution systems. Gas supply in the damaged area was shut off within two hours and the gas in the distribution system dissipated into the atmosphere.

The following report was given by W. C. Mosteller, Manager of Engineering of the Southern California Gas Company before the Los Angeles County Earthquake Commission on March 17, 1971.

> "In general, the distribution piping system withstood the stresses of the quake remarkably well. This is borne out by the fact that in many areas where buildings sustained appreciable damage, the system remained intact."
>
> "The major transmission lines from out-of-state, 30-in diameter and above, did not suffer any apparent damage. Service to the great majority of the company's 3.1 million customers was little affected."
>
> "The transmission system handling the deliveries of California gas from the San Joaquin Valley to the Los Angeles basin was damaged to the extent that four lines had to be shut down in

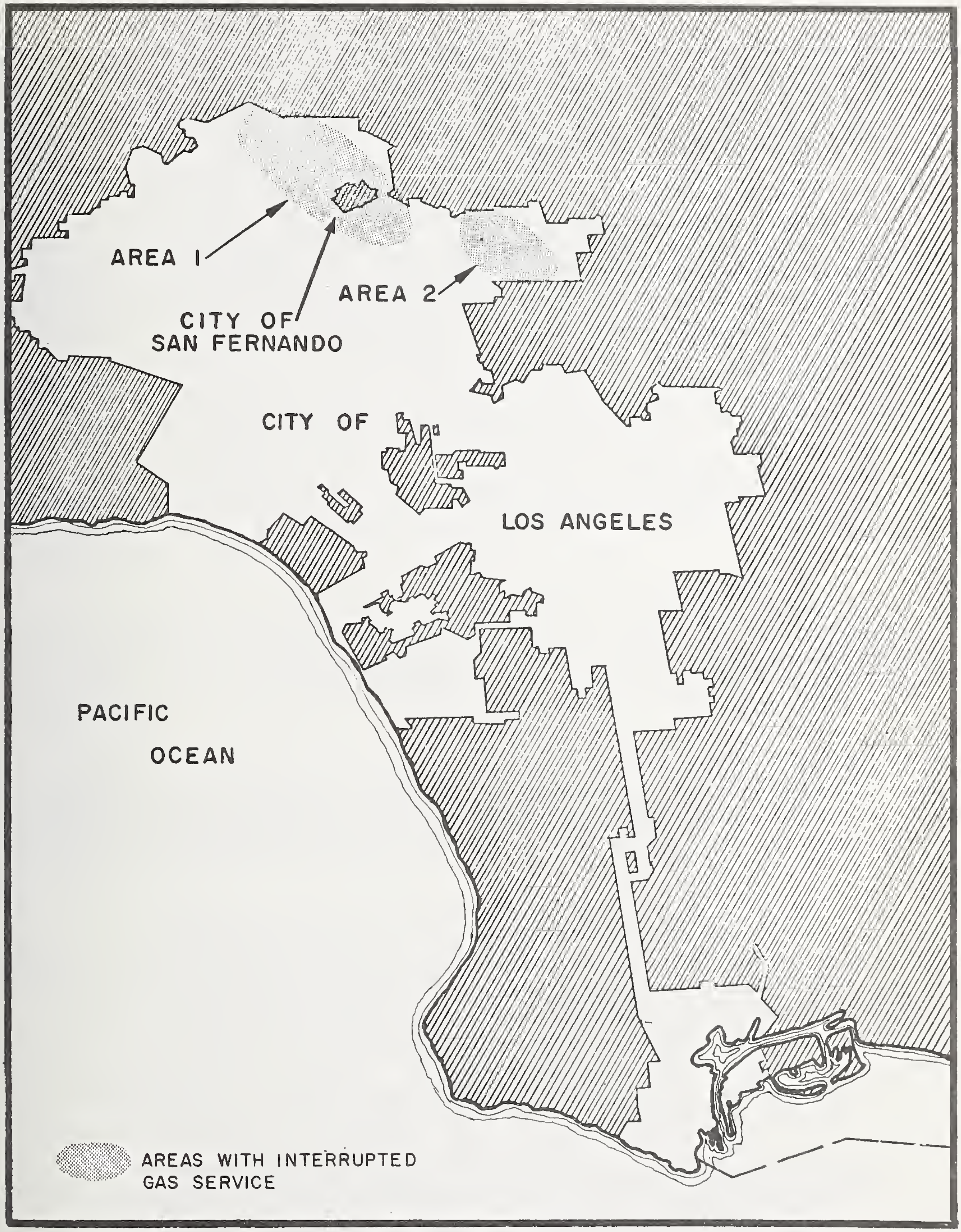



*Figure 8.20 Areas with interrupted gas service.*

the San Fernando area. Damage to the four lines, which range from 12 to 26 inches in diameter, occurred between Newhall and San Fernando and resulted in the loss of supply to the distribution system in the Sylmar-San Fernando area."

"The distribution system in this hard-hit area consists principally of a network of 2-, 3-, and 4-in welded steel mains serving approximately 17,000 customers. The violent earth movement there pulled, compressed, and twisted the piping system, resulting in broken mains, valves and service risers. Approximately 450 breaks have been discovered."

"Damage to the remainder of the Company's distribution system was confined to isolated fractures or pipe separations that did not result in serious impairment of service to customers."

"Repairs to the damaged gas mains in the San Fernando area were commenced almost immediately after the quake. Gas service to customers was restored commencing Sunday, February 14, and completed for the most part by Saturday, February 20th."

There were several gas fires within streets at broken gas lines, one at the intersection of Glenoaks Boulevard and Hubbard Street. A number of other fires damaged buildings after the quake; however, none had been identified as associated with gas.

Many broken gas lines were flooded with water from broken water mains. The greatest damage of this type occurred in the northern corner of the City of San Fernando where, as shown in figure 8.21, ruptured gas, water, and sewer lines were buried adjacent to each other. Some 10 days after the quake, a problem of a similar nature occurred in the Sunland-Tujunga area, some 6 miles east of the City of San Fernando, where earthquake-loosened soil worked its way into the broken gas lines and filters in a regulator station. This disrupted gas service to about 10,000 customers for a period of up to one day, in the area bounded by Sherman Grove Avenue, Haynes Canyon Avenue, the San Gabriel Mountains and Verdugo Mountains (fig. 8.20, Area 2).

## 8.4 Telephone Facilities

The General Telephone Company sustained severe damage to its buildings and equipment. It is estimated that the damage to the company was approximately $4.7 million. Of this figure, about $0.2 million is attributed to building damages and $4.5 million to equipment damages.

These figures suggest that the damage to building structure was miminal. Of a total 27 buildings inspected, one was vacated as unsafe, one was structurally damaged and nine suffered non-structural damages.

The equipment damage was concentrated at the Sylmar Central Office where all the switching equipment was rolled over (fig. 8.22). Loss of this central office left 9,500 Sylmar customers without telephone service.

The buildings were designed in compliance with all governing building and safety codes at their respective locations. In addition, the equipment areas were designed for 150 lbs per square foot of live load.

It appears from what happened at the Sylmar Central Office that while the buildings are, in general, adequately designed for lateral forces, the equipment supporting frames did not have adequate lateral supports to prevent them from toppling.

It was reported that, at present, there are two basic systems for equipment structural support. These systems are used in California and other states. One system which is supplied by many of the telephone equipment manufacturers consists of the following:

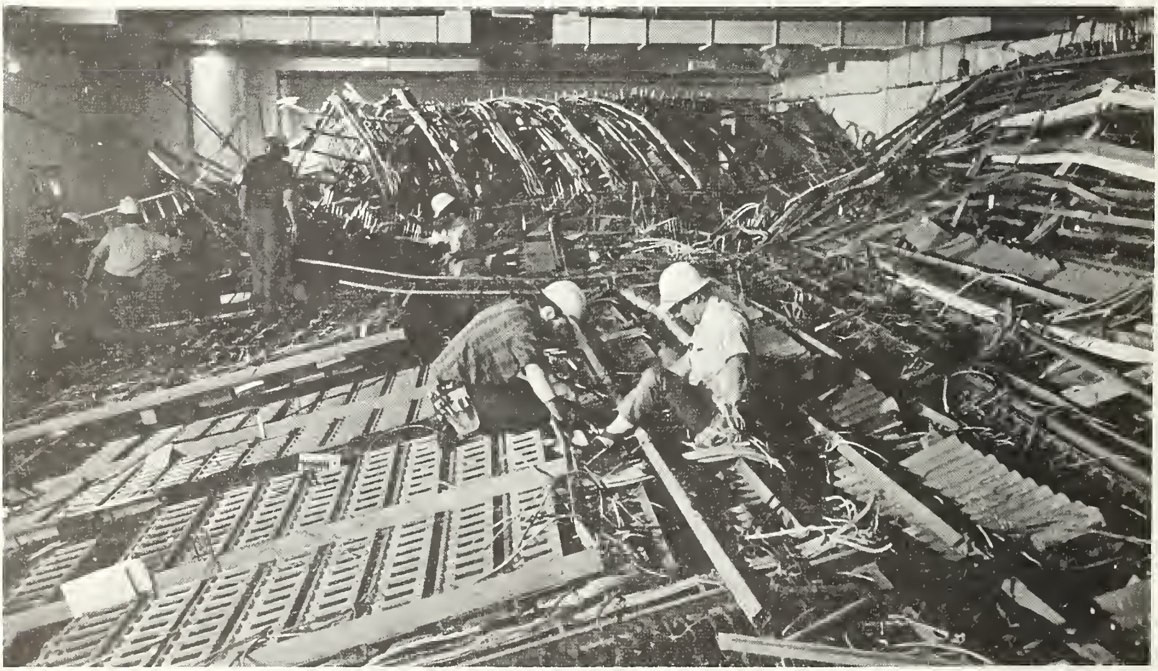
> The placing of a steel angle ledger on the perimeter walls of the equipment room. At right angles to this ledger and perpendicular to the equipment frame a steel channel is bolted to the ledger and the equipment is framed to it by means of mechanical connections.

The other system, primarily in use by the General Telephone Company, consists of the following:

> Continuous dovetail anchors are embedded in the roof structure and/or 5 lb/ft channel section is bolted via inserts to the structure with threaded rods dropped down to support cable racks and secondary steel superstructure run perpendicular to the equipment frames. As an

Figure 8.21 Close-up view of ruptured sewer, water, and gas lines.

Figure 8.22 Toppled and damaged telephone switching equipment at Sylmar Central Office of General Telephone Company.



integral part of this structure, diagonal bracing is used to provide a Warren truss. Equipment frames are erected under the above described structure and fastened by means of mechanical connections.

In both systems the equipment frames are anchored to the floor by power-driven concrete inserts and/or steel anchors, shields, etc.

The equipment frames consist of vertical angle sections supporting horizontal rows of equipment fastened to the two supporting systems described above.

The post-earthquake survey revealed that improvement in the mechanical connections that provide the lateral support to the equipment frame is required.

The mechanical connections which are presently used should be re-evaluated for strength and need to be improved to insure that the equipment supporting structure has at least equivalent structural integrity to that of the building.

## 8.5 Sewers and Storm Drains

The preliminary estimate (March 9, 1971) made by the Department of Public Works, City of Los Angeles, for the sewer and storm drain repair work required to place the damaged facilities in a satisfactory condition is as follows:

| | |
|---|---|
| Sewers | $1,000,000 |
| Storm Drains | 800,000 |
| Total | $1,800,000 |

These estimates are based upon the results of physical examinations made by the Department of Public Works up to March 9, and further study may indicate that additional construction may be needed.

The Department of Public Works of the City of Los Angeles has conducted a survey to determine the changes in elevation caused by the earthquake. Their results are presented in figure 8.23. The numbers indicate changes of evaluation, assuming that the reference point--the intersection of Van Nuys and Foothill Boulevards--has remained at the original elevation. The maximum change of 7.5 ft was measured in Lopez Canyon, with a rise of about 5 ft near the northern part of the City of San Fernando (intersection of Harding Street and Foothill Boulevard). Figure 8.23 also

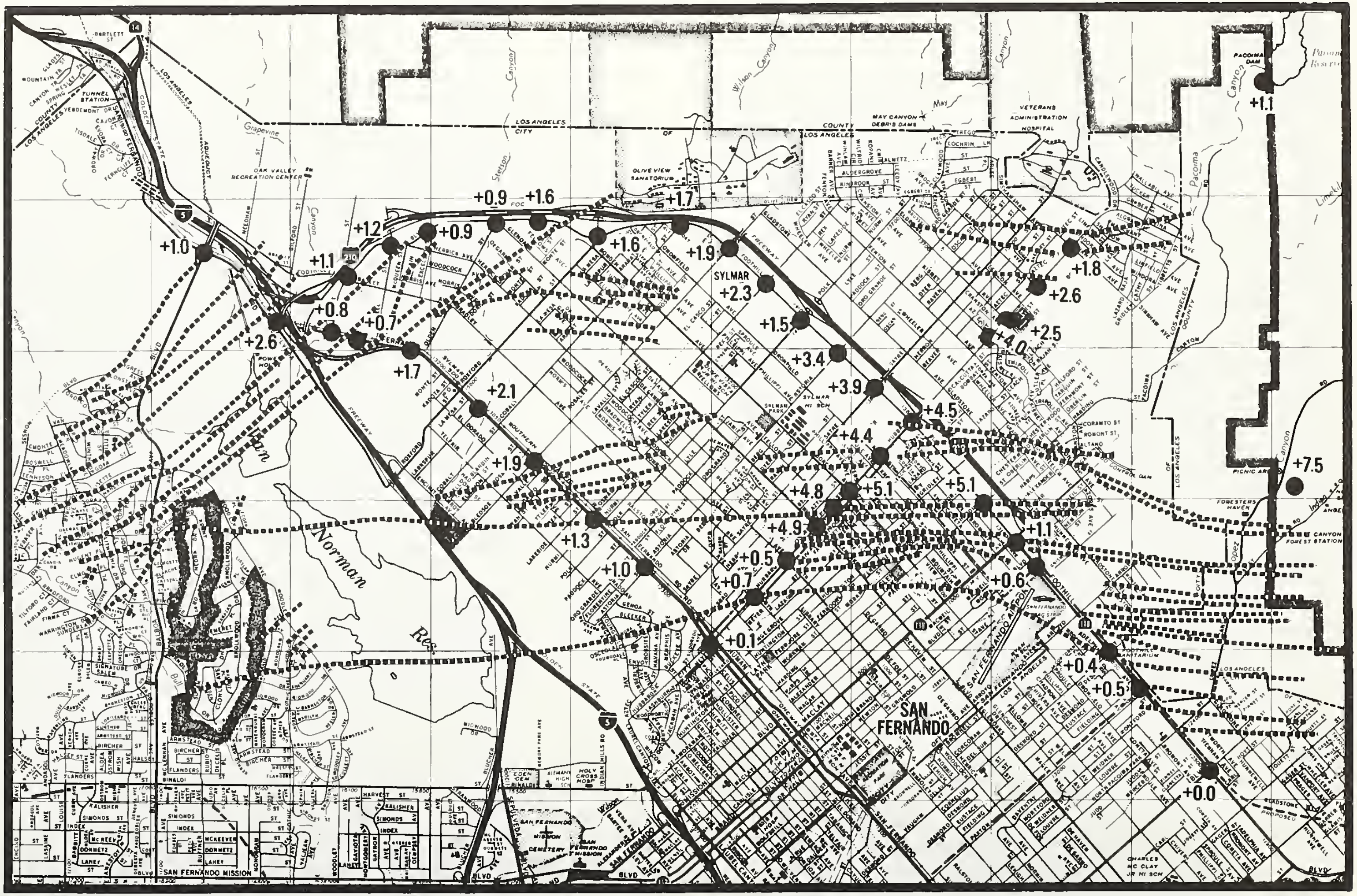



Figure 8.23 Changes in elevation (in feet) caused by earthquake. Dashed lines indicate estimated location of ground surface breaks.

shows the observed and estimated location of many surface breaks and faults in the area. Much damage to sewers and storm drains occurred in these areas which experienced considerable ground movement.

Most of the damaged sewers were 10 to 12 inches in diameter and were located 8-ft deep. Of the 120 miles of sewers in the Sylmar and San Fernando area, about 10 percent have been examined by television as of March 11. (The cost of this television work ranges from $0.80 to $1.00 per lineal foot.) Damage has been found in about eight percent of the sewers examined which lie along the northeast-southwest streets and about one to two percent along the northwest-southeast streets.

Damage has also been detected in some of the deeper sewers. For example, the 18-in diameter, 25 ft deep sewer on Telfair Avenue, east of the Lower Van Norman Reservoir, is extensively damaged.

In the City of San Fernando complete rebuilding of the sewer system will be needed in the area bounded by Hubbard Street, Glenoaks Boulevard, Harding Street, and Eighth Street (see fig. 8.23 for location).

Storm drain facilities to be repaired consist of main-line storm drains, sidewalk culverts, manholes, open channels, and reinforced concrete box structures. Despite the damage, the Department of Public Works reported that most of the drains are intact and functioning well.

## 8.6 Fire Damage and Protection

Relatively little fire damage occurred as a result of the February 9 earthquake. A single-family dwelling and several commercial buildings destroyed by fire are shown in figures 8.24 and 8.25. The number and types of incidents that occurred during the three 24-hour period following the earthquake are given in table 8.1. To serve as comparison, the 1970 daily averages are also given in the table.

The resources of the Los Angeles Fire Department (LAFD) were actively involved in the post-quake activities. With the telephone system out in many areas, and no street boxes in the San Fernando Valley, helicopter surveilance was used and radio communications became vital. The LAFD was called to help in the rescue operations at the San Fernando Veterans Administration Hospital. They also had some 17 pumps operating (total 2492 pump-hours) to supplement the water supply.

Based on preliminary reports, only five Los Angeles

*Figure 8.24 House in Knollwood area destroyed by fire.*

*Figure 8.25 Commercial buildings destroyed by fire (Chatsworth Street and Zelzah Avenue, Granda Hills).*

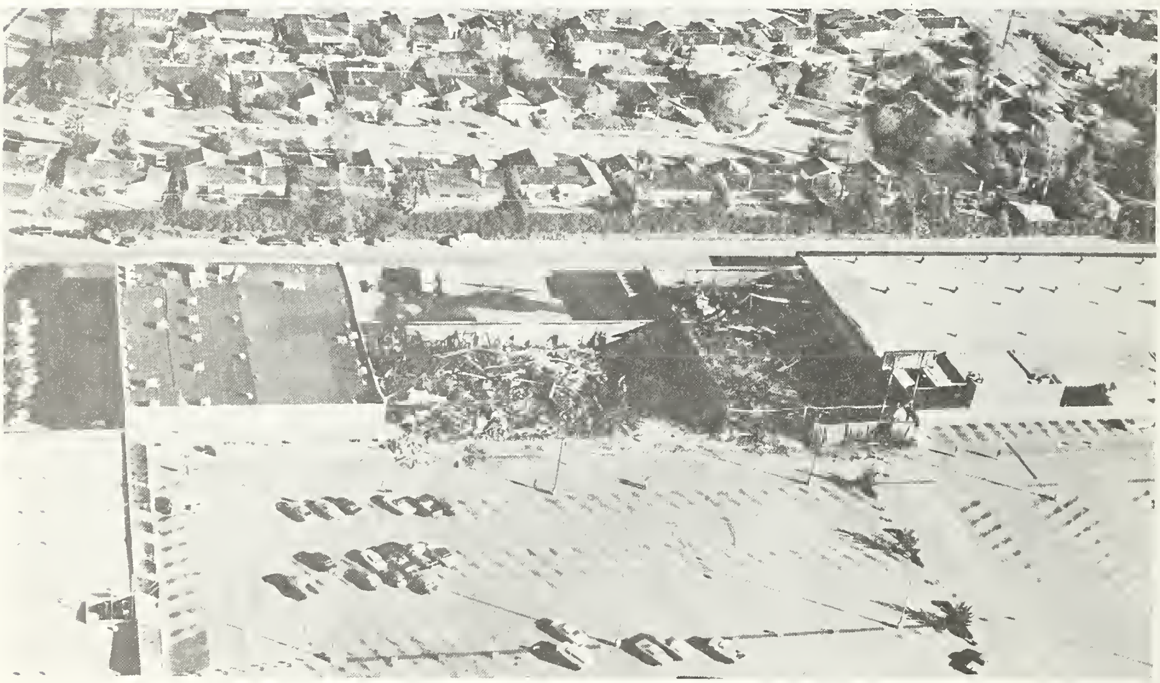


Table 8.1 Earthquake Related Incidents (Based on data compiled by Los Angeles Fire Department February 23, 1971)

| Type of Incident | Feb. 9 (0600) to Feb. 10 (0600) | Feb. 10 (0600) to Feb. 11 (0600) | Feb. 11 (0600) to Feb. 12 (0600) | Total | 1970 Daily Average |
|---|---|---|---|---|---|
| Electrical Failure | 23 | 12 | 9 | 44 | 6.5 |
| Explosion-Flammable Gas | -- | 1 | -- | 1 | 2/month |
| False Alarm--Accidental | 26 | 8 | 11 | 45 | -- |
| False Alarm--Malicious | 21 | 32 | 27 | 80 | 30 |
| Fire Not in Structure | 49 | 51 | 40 | 140 | 60 |
| Fire in Structure | 62 | 34 | 35 | 131 | 29 |
| Fire Not in Structure - Out | 8 | 14 | 16 | 38 | -- |
| Fire in Structure - Out | 9 | 5 | 9 | 23 | -- |
| Food on Stove | 8 | 7 | 3 | 18 | -- |
| Invest. Other Than Accident | 23 | 3 | 1 | 27 | 1 |
| Leak, Natural Gas | 28 | 13 | 5 | 46 | 2 |
| Leak, Refrigerant | -- | -- | 1 | 1 | -- |
| Power Lines Down | 52 | 2 | 1 | 55 | 1.5 |
| Rescue | 9 | 3 | 4 | 16 | -- |
| Smoke Scare | 21 | 12 | 10 | 43 | 8 |
| Spill, Chemical | 4 | 1 | 1 | 6 | 1/week |
| Spill, Flammable Liquid | 1 | 3 | 1 | 5 | -- |
| Washdown, Gasoline | 3 | 7 | 7 | 17 | -- |
| Water Flow | 17 | 4 | 1 | 22 | 1.5 |
| Water Surge | 1 | 1 | 1 | 3 | 3/week |

Fire Department stations (out of a total of 104 stations) suffered severe wall cracking or major structural damage.

The fire protection of many old buildings, particularly in downtown Los Angeles was affected by the collapse of the protecting material. In some of these buildings the fire protection consisted of terracotta hollow tiles covered with cement mortar and gypsum plaster.

## 8.7 Flood Control Facilities

Three separate agencies, the U. S. Army Corps of Engineers, Los Angeles County Flood Control District, and the City of Los Angeles Department of Public Works are responsible for drainage and flood control in Los Angeles. In general, the Corps of Engineers is responsible for the improvement of the larger streams which traverse the metropolitan area. Its projects are to protect against more severe storms then those that fall under the jurisdiction of the Flood Control District and the City; storms of such intensity occur on an average of about once every one hundred years. The Flood Control District's channels are built to accommodate the waters of a fifty-year storm while the Los Angeles City and the Flood Control District normally design their underground drainage facilities to provide relief from the effects of a ten-year storm [8.1].

Damage Estimates - Damage to the Los Angeles Flood Control District facilities has been estimated at $2.5 million [8.2]. The only Corps of Engineers project which suffered significant damage was the Lopez Debris Basin [8.2 and 8.3]. The cost to repair this project has not been reported at this time.

Dams - Pacoima Dam, built in 1928 as a 370 ft high concrete-arch structure, is the only major District dam in the Sylmar area (see fig. 8.26). Significant damage occurred to the gunite abutments (figs. 8.27 and 8.28), and massive rockslides occurred in the vicinity of the dam (fig. 8.29) [8.4]. Although damaged evaluation is not complete, preliminary reports indicate there was no structural damage to this dam. Water level behind the dam was low at the time of the earthquake.

Debris Basins - At the Wilson Canyon Debris Basin, damage consisted of slides on the access road, cracked and destroyed roadway, cracked upstream concrete facing, fallen right-of-way fencing, displaced headwall and cracked earth levee. At the Schoolhouse Debris Basin, damage consisted of slides on the access road, destroyed right-of-way fencing, cracked spillway and spillway walls out of plumb. At the Stetson

*Figure 8.26 Pacoima Dam, a 370-ft high concrete-arch dam, built in 1928.*

*Figure 8.27 Damage to gunite left abutment, Pacoima Dam.*

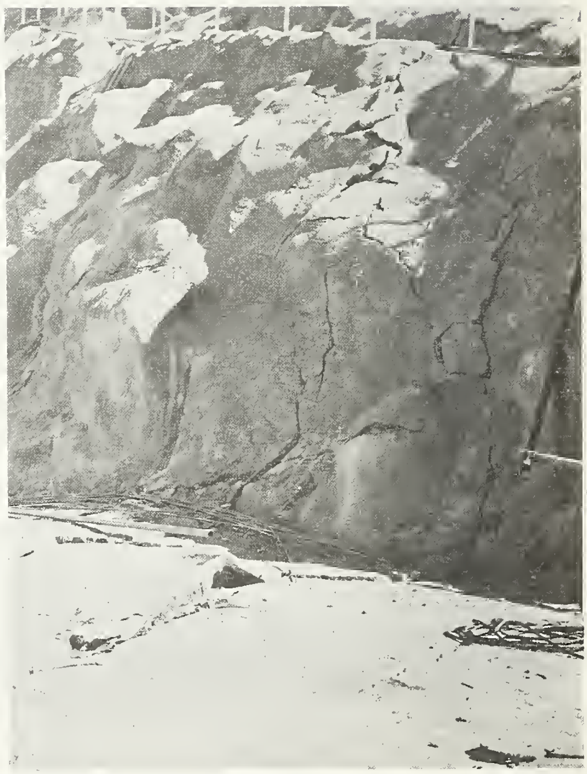


*Figure 8.28 Cracks in gunite, left abutment downstream of Pacoima Dam.*

*Figure 8.29 Slides on access road along covered walkway, Pacoima Dam.*

Debris Basin, there were cracks in the dam fill on both sides of the spillway (fig. 8.30) and in the facing slab parallel to the spillway invert (fig. 8.31). Hog Debris Basin and Sombrero Debris Basin also suffered similar damage.

Channels - The Wilson Canyon Channel has a reinforced concrete box which was badly damaged (figs. 8.32 and 8.33). Concrete walls along the Lopez Canyon Channel were also badly damaged (figs. 8.34 and 8.35). Other channels which were damaged are the Mansfield Channel and the Pacoima Wash Channel.

## 8.8 References


8.1 Drainage Plan - City of Los Angeles; Publication prepared by Department of City Planning and the Department of Public Works Engineering Bureau, 1968.

8.2 C. Martin Duke, "Damage to Water Supply Systems, The San Fernando, California Earthquake of February 9, 1971," Geological Survey Professional Paper 733, U. S. Government Printing Office, 1971.

8.3 Los Angeles District, Corps of Engineers, Post Earthquake Inspection and Evaluation Survey, February 18, 1971.

8.4 Damage Report, Earthquake of February 9, 1971, Los Angeles County Flood Control District, Operation and Maintenance Division Preliminary Report.

*Figure 8.30 Looking northwesterly toward basin showing crack in the pavement and a 2-inch vertical displacement of paved crest of dam (Stetson Debris Basin).*

*Figure 8.31 Looking easterly showing damage to facing slab; Stetson Debris Basin.*

*Figure 8.32 Left wall buckled inward at station 45+10. Wilson Canyon Channel.*

*Figure 8.33 Reinforced concrete box has completely separated (4 to 12 inches) at several locations between stations 44+00 and 49+93; Wilson Canyon Channel.*

*Figure 8.34 Lopez Canyon Channel. A 50-foot section of the wall has failed on the right bank. The 50-foot section adjacent upstream also shows severe structural damage.*

*Figure 8.35 Lopez Canyon Channel. A 2-foot displacement of the channel had caused complete failure of the walls and invert, with the upstream portion moving over the downstream portion.*

# CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations given in this chapter are based on the site investigations which were made immediately following the San Fernando earthquake and the subsequent study of a number of damaged structures as well as seismic code requirements. The recommendations, however, are not limited only to the San Fernando area but are also applicable to other earthquake-prone areas in the United States.

## 1. REVIEW OF CURRENT DESIGN PRACTICE

*An immediate review should be made of the adequacy of present design requirements for seismic design.*

Most buildings near the epicenter, which were severely damaged, such as the Olive View Medical Center, San Fernando V.A. Hospital, Pacoima Memorial Lutheran Hospital, and Holy Cross Hospital were not designed under the latest building code requirements (1968 SEAOC or 1970 UBC). Nevertheless, the performance of these structures should be reviewed in detail, with proper consideration given to the code requirements used in their design, so that these data and information can be used to assess the adequacy of present seismic design requirements for buildings.

Many bridges designed under present specifications collapsed. Thus, these design requirements are in need of review.

Numerous recordings of horizontal accelerations which greatly exceeded those envisioned in formulating the current lateral force requirements are reason to review and possibly increase seismic coefficients currently in use. It may be necessary in the future to adopt a design procedure which reflects the dynamic response characteristics of structures. However, as an intermediate action, the existing values of the coefficients used in design equations should be modified.

In view of records of vertical ground accelerations of the same order of magnitude as horizontal accelerations, the presence of vertical seismic forces can no longer be ignored.

## 2. UPDATING DESIGN REGULATIONS

*Present procedures used to update design regulations should be reviewed in order to evaluate more expeditious ways to incorporate new knowledge into design.*

Building codes are continually-changing documents. As codes are updated, design requirements may be made more liberal or more conservative. It is important that unsafe requirements be corrected as rapidly as possible.

The Olive View Medical Center provides an excellent example of the delay in revising building codes to change requirements which had been found to be unconservative. As indicated in the discussion of Olive View (sec. 5.2), a possible cause of the collapse of the Psychiatric Unit was insufficient first-story column shear strength. The concrete unit shear stresses used in design were based on the 1956 ACI Building Code. The 1956 ACI unit shear stresses were found to be unconservative. Thus, the 1963 ACI Code adopted more restrictive unit shear stresses than the 1956 Code. It was not until November 1966 that the Los Angeles County Code was updated to use the newest ACI Code unit shear stresses. The use of the more restrictive unit shear stresses would have required the use of additional lateral reinforcement in the columns, which could possibly have prevented the collapse of the Psychiatric Unit. This can not be stated conclusively, since it must be noted that ground accelerations were much larger than those anticipated in formulating code design equations.

There does not appear to be a simple solution to the problem of updating codes more rapidly. However, as pointed out above, it may be legally possible to design an unsafe structure. Thus, the problem is clearly in need of study.

## 3. CRITICAL FACILITIES

*Critical facilities should be designed so that they remain functional after a severe earthquake.*

Facilities, in this context, include hospitals, emergency services (fire, police, etc.), utilities, communications, transportation networks, schools, and high occupancy buildings. Design requirements should reflect the importance of the facility and the degree of danger involved in its failure.

Four hospitals-San Fernando V.A. Hospital, Olive View Medical Center, Holy Cross Hospital, and Pacoima Memorial Lutheran were critical facilities which were severely damaged and were unable to fulfill their functions.

Public facilities which have a high occupancy use should be considered critical due to the potential loss of a large number of lives involved in a collapse.

Water, sewerage, gas, and electricity were severely damaged in the San Fernando Valley area. Many communities were without these facilities for a considerable period of time.

The collapse of bridges at the intersection of the Golden State Freeway and Foothill Freeway blocked an extremely important potential evacuation route from the Los Angeles area. Although crews worked remarkably well in constructing detours, these routes would not have been immediately available for evacuation as well as for rescue operations had the damage been more widespread.

The damage at the Sylmar Convertor Station indicates the potential for disruption of power. The damage of the telephone switching equipment at the Sylmar Central Office disrupted communications to the extent that full service was not restored until 6 1/2 weeks after the earthquake.

Due to such disruptions of required services in a disaster it is recommended that a seismic occupancy multiplier should be applied to the lateral force design and should be a function of the hazard potential of the facility. Emergency facilities such as hospitals, police stations, fire stations, utilities, and other installations that must remain operational after a disaster should have a seismic multiplier of possibly 2 or greater. Structures such as schools, places of assembly, and other establishments subject to a high panic potential should have a multiplier of perhaps 1 1/2.

## 4. REHABILITATION AND STRUCTURES UNDER CONSTRUCTION

*Buildings in existence or under construction should conform to up-to-date design regulations.*

Evaluation of the earthquake hazard of structures built under older codes should begin immediately. This is particularly important for critical public buildings. For instance the San Fernando V.A. hospital buildings which collapsed were built well before the existence of earthquake requirements. Where required, critical public buildings should be scheduled for rehabilitation or removal. Such a program is feasible as was demonstrated by the performance of rehabilitated schools (chapter 6) which were constructed prior to 1933 but were reinforced after 1933 to resist earthquake forces.

A problem of equal importance is that of establishing a mechanism by which a structure under construction can be re-evaluated in light of important changes in design regulations. An example of this is the Olive View Medical Center. During, or shortly before, the construction of the medical center, the 1965 L.A. County Code was amended in November 1966 to include significant changes in design procedure.

## 5. CONSTRUCTION INSPECTION

> *Revised design regulations should include provisions for construction inspection of critical facilities by a qualified structural engineer.*

In general schools constructed under the Field Act performed well during the earthquake. Several schools constructed prior to the Field Act performed poorly and had to be demolished. Possibly the most significant provision of the Act is the requirement for strict construction supervision.

Although no apparent signs of faulty construction were observed in the major structures surveyed, this requirement seems to be particularly important for critical public buildings.

## 6. DEFORMATION AND DEFLECTION

> *Earthquake design should consider deformation and deflection as well as strength.*

Deformation and deflection, as well as strength, should be considered in earthquake-resistant design. This is illustrated by the horizontal and vertical movements which caused bridge girders to move off their supporting abutments and piers. Ground displacements must be studied carefully to determine appropriate magnitudes of movement which should be accounted for in design. Details must be developed to allow for such movement.

## 7. DUCTILITY

> *An accurate appraisal must be made of the amount of ductility which can be expected in a particular type of structural design and material.*

Design regulations should require material properties and connection details which are consistent with the amount of ductility allowed in dissipation of seismic energy. For example, the specific recommendations listed below to increase the ductility in reinforced concrete should be studied in detail.

(a) The tied column behavior at Olive View illustrates that the maximum spacing of column ties should be limited to one-half the effective depth of the cross section to insure effectiveness as shear reinforcement.

(b) Columns and piers should be designed to resist a shear at least equivalent to that imposed by a complete moment reversal on the column equal to the flexural capacity of the column. This is necessary to prevent a shear type failure since such a failure is not ductile.

(c) Extra column ties or spiral wraps should be provided in the end sections of all columns and into the beam-column connections.

(d) The use of lap slices for lateral reinforcement such as column ties should be discouraged. All ties should be anchored by hooks or embedment into the core in order to remain effective after covering concrete has spalled off.

## 8. MOMENT-RESISTING CONCRETE FRAMES

> *Consideration should be given to modifying the requirements for moment-resisting concrete frames to include at least some of the reinforcing requirements for*

*ductile moment-resisting frames.*

The first-story column failure of the Psychiatric Unit at Olive View illustrates the consequences of the failure of a column in shear (as indicated in chapter 5 it is felt that shear was the most likely cause of failure). Once a concrete member has cracked in shear, its shear capacity becomes essentially zero. This is in contrast to the case of a flexural type of failure where the member may continue to deform but can still carry load.

The psychiatric unit was designed as a moment-resisting frame using K=0.67. Under current practice such a frame would have to be a ductile moment-resisting frame which has special reinforcing requirements beyond those of the moment-resisting frame. For example, ties should be spaced no farther apart than one-half the least column dimension. The moment-resisting frame with a K=1.0 does not have these special requirements. For practical purposes, the Psychiatric Unit was not a ductile frame. Under current practice a K=1.0 would be used (with the same reinforcement details). However, it is entirely likely that the same type of failure would have occurred, even if the design had been based on somewhat larger lateral forces.

Thus to provide some shear resistance, after shear cracking of concrete, it is recommended that ties be spaced no greater than one-half the effective depth of the column cross section in moment-resisting frames as well as ductile moment-resisting frames.

## 9. DESIGN OF APPENDAGE CONNECTIONS

*Connections of appendages should be designed to resist the same seismic forces as the structure which supports them.*

Appendages such as light fixtures, emergency lights, suspended ceilings, and other overhead objects fell during the earthquake. The connection of such appendages to structural elements should be given engineering consideration since its failure presents a hazard.

## 10. DWELLINGS AND MOBILE HOMES

*Specific recommendations for dwellings and mobile homes are listed below.*

(a) Positive means of transferring lateral forces are required where large openings exist in walls and where changes occur in configuration. Numerous failures were observed in apartments and single family homes where large openings in walls were provided for garages or entranceways. Numerous single-family dwellings collapsed or were near collapse due to the inadequate tying together of adjacent units.

(b) Chimneys should be adequately reinforced and anchored to the main structure. Numerous failures due to lack of reinforcement and anchorage were observed.

(c) Improved methods for supporting mobile homes should be developed and implemented.

## 11. DAMS

*Seismic resistance and design of dams whose failure would present hazards to heavily populated areas should be reviewed.*

The near-failure of the Lower San Fernando Dam which is located above a densely populated residential area required the evacuation of about 79,000 people. This is sufficient cause for the review of the seismic resistance of such structures. All existing dams located close to a dense population should be examined for strength and stability due to ground faulting and acceleration.

The adequacy of present design requirements for the seismic design of dams should be reviewed. A vertical acceleration component should be considered, particularly for gravity dams.

## 12. UNDERGROUND INSTALLATIONS

*Underground installations such as water, gas, and sewer pipes should be designed to minimize service interruption.*

Gas, water, and sewer lines may fracture during severe ground movements. Seepage of sewage into the water line is a source of contamination. Seepage of water and sewage into gas lines causes extensive system damage. A possible means of minimizing damage is to install flexible joints and automatic cut-off valves. Alternate service routes are also desirable.

## 13. MACHINERY AND EQUIPMENT

*Machinery and equipment should be anchored to resist the same seismic forces as the structure which supports them.*

Heavy electrical equipment suffered damage at the Sylmar Convertor Station due to inadequate support or anchorage. Boilers at Olive View and Juvenile Hall caused damage to buildings which housed them due to their movement. Proper anchorage of such equipment to structural elements is essential. Telephone switching banks were also destroyed in Sylmar due to inadequate support.

## 14. ELEVATORS

*The design of elevator systems should be reviewed to insure their operation after a disaster.*

As indicated in this report, many elevators could not be operated after the earthquake. This was due either to power loss, bent guide rails, and/or cables tangled by loose counterweights. Thus, elevators were not available for use during or after a disaster. Furthermore it seems possible that, had the earthquake occurred during hours of heavy use, lives would have been endangered. This could have occurred due to counterweights falling through the car. Had fires occurred, the passengers of immobile elevators almost surely would have died. It should be pointed out; however, that such fires did not occur as a result of this earthquake.

## 15. GEOLOGIC HAZARDS AND FAULT ACTIVITY

*Improved methods should be developed to identify the potential activity of faults and associated geologic hazards.*

The San Fernando Earthquake, as well as the Kern County Earthquake, occurred along faults not generally recognized as active before the earthquake. These earthquakes point towards the need for developing improved methods for characterizing the potential activities of faults and applying these methods in geologic field studies of faults in seismic areas. Information gained from such studies can then be taken into account in planning for the use of land.

## 16. SEISMIC RISK ZONING

*More detailed and concise seismic risk maps should be developed.*

Present seismic risk maps, such as the ones given in the Uniform Building Code, are based on the Modified Mercalli Intensity scale. As a result only short-period (less than one second) effects are considered. Since more and more highrise structures will be built in the future, a seismic risk map incorporating long period effects will be needed.

Furthermore, the present risk map reflects potential seismic risks on a large regional basis. Micro-regional seismic risk maps are urgently needed to cover potential seismic risks representing a small region, such as the San Fernando Valley. Such a micro-seismic risk map should consider (1) distribution and frequency of earthquake, (2) local (site oriented) geology and (3) regional geologic features, such as ground faulting, etc.

## 17. EXPANSION OF SEISMOGRAPH NETWORK

*The present seismic monitoring program should be expanded and improved.*

Information provided by the strong-motion seismograph is single best source of scientific data that can be used in post earthquake studies of structural performance. Even in the Los Angeles area, with the largest number of strong-motion seismographs installed in buildings in the United States, the seismic monitoring network was not as complete as it could have been. For example, while many buildings in downtown Los Angeles had strong-motion seismographs, none of the severely damaged buildings in the upper San Fernando Valley had the seismographs. In view of this, to complete the seismic monitoring network, strong-motion seismographs should be installed in public structures in regions where large buildings are not present.

It should also be emphasized that comprehensive seismic monitoring programs are needed not only in the western states of the U. S. but in all earthquake-prone regions of the country.

In general the system of accelographs in the Los Angeles area performed well and obtained much useful data. However, there were cases where data were not obtained due to dead batteries, traces too faint to read, or lack of paper in the instrument. Adequate funding should be made available for maintenance of the system.



☆ U. S. GOVERNMENT PRINTING OFFICE : 1971 O - 451-866

FORM NBS-114A (1-71)

| U.S. DEPT. OF COMM. BIBLIOGRAPHIC DATA SHEET | 1. PUBLICATION OR REPORT NO. | 2. Gov't Accession No. | 3. Recipient's Accession No. |
|---|---|---|---|
| | NBS BSS-40 | | |

| | |
|---|---|
| 4. TITLE AND SUBTITLE<br>Engineering Aspects of the 1971 San Fernando Earthquake | 5. Publication Date<br>December 1971 |
| | 6. Performing Organization Code |
| 7. AUTHOR(S)<br>Hai Sang Lew, Edgar V. Leyendecker, and Robert D. Dikkers | 8. Performing Organization |
| 9. PERFORMING ORGANIZATION NAME AND ADDRESS<br>NATIONAL BUREAU OF STANDARDS<br>DEPARTMENT OF COMMERCE<br>WASHINGTON, D.C. 20234 | 10. Project/Task/Work Unit No.<br>4213116 |
| | 11. Contract/Grant No. |
| 12. Sponsoring Organization Name and Address<br>Office of Emergency Preparedness, Washington, D. C. 20504<br>Office of Civil Defense, Washington, D. C. 20310<br>National Bureau of Standards, Washington, D. C. 20234 | 13. Type of Report & Period Covered<br>Final |
| | 14. Sponsoring Agency Code |

15. SUPPLEMENTARY NOTES

16. ABSTRACT (A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)

Immediately following the San Fernando, California earthquake (February 9, 1971), a four-man team from the Building Research Division of the National Bureau of Standards' Institute for Applied Technology, surveyed the damage to buildings and other structures. This report is based primarily on the data gathered during the survey but includes some data provided by other agencies and individuals. Based on study of these data and observations made during the survey, recommendations are made pertaining to the improvement of building and other structural design and construction practices.

17. KEY WORDS (Alphabetical order, separated by semicolons) Bridge; building; codes; dams; earthquakes; earthquake damage; foundation geology; highways; housing; hospital; mobile home; seismic; standards; structural engineering.

| 18. AVAILABILITY STATEMENT | 19. SECURITY CLASS (THIS REPORT) | 21. NO. OF PAGES |
|---|---|---|
| [X] UNLIMITED. | UNCLASSIFIED | 419 |
| [ ] FOR OFFICIAL DISTRIBUTION. DO NOT RELEASE TO NTIS. | 20. SECURITY CLASS (THIS PAGE)<br>UNCLASSIFIED | 22. Price<br>$ 3.00 |

USCOMM-DC 66244-P71

U.S. DEPARTMENT OF COMMERCE
National Bureau of Standards
Washington, D.C. 20234

OFFICIAL BUSINESS

Penalty for Private Use, $300

POSTAGE AND FEES PAID
U.S. DEPARTMENT OF COMMERCE

U.S.MA