

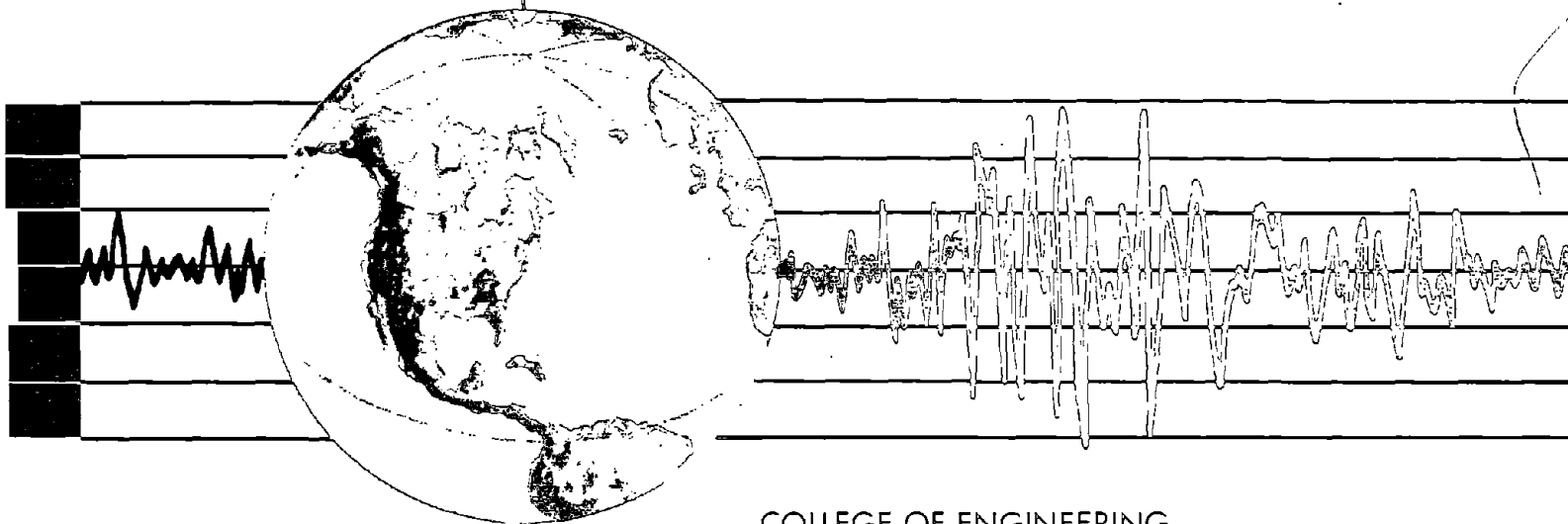
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EARTHQUAKE ENGINEERING RESEARCH CENTER

PRELIMINARY REPORT ON THE
SEISMOLOGICAL AND ENGINEERING ASPECTS
OF THE JANUARY 17, 1994
NORTHRIDGE EARTHQUAKE

Preliminary findings from field investigations by teams from the University of California at Berkeley immediately following the earthquake.



COLLEGE OF ENGINEERING

UNIVERSITY OF CALIFORNIA AT BERKELEY

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THE SEISMOLOGICAL AND ENGINEERING ASPECTS OF
THE JANUARY 17, 1994 NORTHRIDGE EARTHQUAKE

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| 5-1 to 5-2 | Reference to bent 10 should be changed to bent 2.
Reference to bent 9 should be changed to bent 3.
Reference to bent 8 should be changed to bent 4. |
| 5-8 | Fig. 5.5: Bent 2 of Interstate 5/Route 14, South Overhead |
| 5-9 | Fig. 5.6: Bent 3 of Interstate 5/Route 14, South Overhead |
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PREFACE

This report on the seismological and engineering aspects of the 17 January, 1994, Northridge earthquake was printed on 24 January, 1994, one week after the main event. Its purpose is to provide a brief overview of preliminary observations related to the earthquake. The primary audience is seismologists, engineers and related professionals in the earthquake hazard and earthquake risk mitigation field. The report is preliminary in the sense that significant data collection and analysis remain to be completed. Reports containing more complete data and analysis may be issued at a later date.

ABSTRACT

Immediately following the 17 January, 1994, Northridge earthquake, the Earthquake Engineering Research Center dispatched a reconnaissance team to the epicentral region. This report, issued one week after the earthquake, provides an overview of the seismological and engineering aspects of the earthquake and associated aftershocks.

SLIDE SET

A slide set containing approximately 100 slides obtained during the reconnaissance, including all slides and photographs in this report, is being prepared. Copies of the set are available at cost. To obtain a set, write to EERC Reports, 1301 S. 46th Street, Richmond, California 94804, e-mail to reports@eerc.berkeley.edu, call 510-231-9468, or fax 510-231-9461.



Epicenter

LOS ANGELES

PACIFIC

CHAPTER 1

INTRODUCTION

The Northridge earthquake struck the San Fernando Valley region of Southern California at 4:30 a.m. local time, Monday, 17 January 1994. The Seismographic Stations at the University of California at Berkeley assessed the main event at moment magnitude 6.7. According to current accounts, the earthquake resulted in at least 55 deaths and as many as 5000 injuries. The Red Cross estimates 25,000 dwellings are uninhabitable. Very preliminary damage estimates range from \$15-30 billion, which, if correct, would make this the most costly natural disaster in U.S. history.

Studies of aftershocks and permanent ground deformations are providing data from which will emerge a clear image of the earthquake mechanism and related geological phenomena. Early evidence suggests that the earthquake had a focal depth of about 14 km and a thrust mechanism. The epicenter is approximately 25 km southwest of the epicenter for the 1971 San Fernando earthquake. Ongoing analytical and field work will clarify details of the mechanism.

Ground motion records already have been made available from several sources. Durations of strong shaking (peak accelerations exceeding 0.05g) are about 20 seconds in many locations. Several records indicate peak vertical accelerations equal to or exceeding peak horizontal accelerations. Early and approximate analyses of the records suggest that the ground motion intensities may exceed levels commonly used in current engineering design.

Preliminary assessments of engineered structures indicate that the majority performed well during the earthquake; however, there is significant and costly damage over a wide geographic region. In most cases the damage appears to have occurred in older structures, the proportions and details of which do not satisfy current requirements for construction. In other cases, damage has occurred in more recent construction. The efficacy of seismic retrofitting and of technologies such as seismic isolation is often evident. Though a significant amount of data has been gathered, the full impact of the earthquake on structural and nonstructural systems will only be understood many months into the future.

Immediately following the earthquake, a research team comprising about 50 individuals from the Earthquake Engineering Research Center, Seismographic Stations, and Lawrence Berkeley Laboratories pooled their energies and talents to gather perishable and valuable data on the earthquake and its engineering effects. The team focused its attention on seismology, geology, geotechnical engineering, and structural engineering (transportation and building structures). The five remaining chapters in this report provide brief and preliminary summaries of our findings at the end of one week following the main shock. More detailed summaries and analyses will be made available later.

Although the Northridge earthquake and its effects have been tragic in the total loss of life, personal injury, and economic losses, we must use this time to advance our knowledge and construction practices. The acceptable performance of the majority of constructed facilities, and the comparatively small number of deaths compared with earthquakes of similar magnitude elsewhere in the world, emphasizes the overwhelming success of several earthquake risk reduction efforts at the national, state, and local levels. It is imperative that these programs continue and expand so that the tragedy of future earthquakes will be reduced.

CHAPTER 2

SEISMOLOGICAL AND GEOLOGICAL OBSERVATIONS

This is a preliminary report on the geological and seismological aspects of the January 17, 1994 Northridge earthquake, which occurred at 4:30 am (PST) under the north-western end of the San Fernando Valley, Los Angeles, CA (epicentral location: $34^{\circ}13'$ North, $118^{\circ}32'$ W, from Caltech). This report is based on main shock and aftershock data from Caltech/USGS, information from analysis of broadband and strong motion records available to UC Berkeley Seismographic Station scientists during the first 5 days following the main shock, as well as geological information obtained in aerial reconnaissance and field investigation conducted by UC Berkeley and Caltech.

Seismological observations

The results of UC Berkeley's preliminary modelling of broadband records for the main shock from the TERRAScope network and the Berkeley Digital Seismic Network (BDSN) indicate a moment magnitude of 6.7 (local magnitude 6.6 by Caltech), focal depth of ~ 14 km and a thrust mechanism, with both planes striking approximately 10° North of West and dipping approximately 45° (Fig. 2.1). This is consistent with the first motion mechanism released by Caltech several hours after the event. Preliminary results of the empirical Green's function deconvolution analysis in which the effects of source radiation pattern, regional wave propagation, local site conditions and attenuation are removed from the mainshock records, reveals a source duration of approximately 6 seconds (Fig. 2.2). There appears to be a slight directivity towards the North indicating that the event ruptured updip, towards the north, along a south dipping fault. The distribution of aftershocks, covers an area roughly 30km wide (San Fernando to Santa Suzanna) by 25 km long (North Ridge to Santa Clarita Valley) primarily North of the mainshock epicenter, with shallowing depth towards the North (Caltech solutions). The actual fault plane thus appears to be the south dipping plane.

The aftershock frequency distribution appears to be consistent with the general trends in California. Several aftershocks of magnitude larger than 5 occurred during the first 5 days after the main shock (Table 2.1). The largest one in that time period occurred at 3:33 PM PST on January 17 and has a preliminary moment magnitude of 6.0 (UC Berkeley; Harvard gives 5.9) and a similar mechanism to that of the main shock (Fig. 2.1), with a depth of ~ 8 km. Reliable moment tensor solutions for some of the largest aftershocks have been obtained at UC Berkeley using body waveform modelling and, independently, surface wave spectral domain inversion. Most indicate thrust mechanisms similar to that of the main shock, although some have slightly rotated strikes towards North of West. There are several strike-slip mechanisms in the center of the aftershock zone (Fig. 2.1).

Preliminary analysis of strong motion records from TERRAScope and BDSN stations indicate a duration of shaking of ~25-30 sec and a possibly complicated rupture with at least 2 shocks separated by several sec (Fig. 2.3). The two shocks appear to be also resolvable in the preliminary deconvolution of the broadband source time function (Fig. 2.2).

The Northridge earthquake is the latest and so far the largest, in a series of significant earthquakes that have occurred since 1987 in this part of the transverse ranges. The largest of these were the 1987 Whittier Narrows earthquake (M_l=5.9) and the 1991 Sierra Madre earthquake (M_l=5.8). Both these earthquakes occurred to the east of the Northridge epicenter (figure 2.4). All these earthquakes had similar thrust mechanisms. In contrast to the Northridge event however, they occurred on north -dipping planes, as did the San Fernando (Sylmar) earthquake of February 9, 1971 (M_l= 6.4). The San Fernando event occurred at a depth of 13 km (Heaton and Helmberger, 1979; Langston, 1978; Hanks, 1974) on a previously unmapped fault. For this earthquake, evidence of surface rupture was found in a zone directly to the East of the surface projection of the Northridge fault plane. The epicenter of the Sylmar earthquake was located about 25 km northeast of the Northridge event.

All these earthquakes are expressions of the north-south compressive deformation occurring across the Transverse Ranges of southern California. This deformation results from the convergence across the "big bend" of the San Andreas fault system between Gorman and Cajon Pass.

The thrust mechanism of this earthquake may explain the unusually strong shaking experienced in some areas.

Field observations

We used the aftershock pattern from the southern California seismic network, the mainshock focal mechanism from UC Berkeley, published geological mapping, and field reports from Caltech to plan a helicopter reconnaissance along the surface projection of the Northridge earthquake fault plane. The reconnaissance was flown on Wednesday January 19th with cooperation from the United States Coast Guard.

We observed three areas of extensional ground breakage to the south of the surface projection of the Northridge rupture plane (Fig. 2.5). We believe the primary fault plane is manifested by a broad upwarp, and the upward bending of the mountains has resulted in the opening of many extensional fractures. This is reminiscent of the rupture pattern observed in the Loma Prieta earthquake. Our observations indicate that extensional surface strain is prevalent across a large part of the Santa Susana Mountains (Fig. 2.6 to 2.9).

The pattern of faulting from all data indicates that a major south dipping fault system, possibly an eastward extension of the Oak Ridge fault, produced the Northridge earthquake.

The Interstate 5 and Route 1475 bridge failures are within the zone of extensional ground failure that we observed. It is possible that the extensional strain contributed to these bridge failures, and ground deformation is being documented in the vicinity of these bridges.

Table 2.1: Preliminary Berkeley locations and magnitudes for Northridge events with magnitudes ≥ 4.5 from 01/17 to 01/21/1994. Magnitudes are M_L from synthetic Wood-Anderson records, and M_W , from broadband moment tensor inversion. Locations are less reliable than those given by Caltech and are subject to change. Magnitude estimates are more robust.

Date	Time(UTC)	Latitude	Longitude	M_L	M_W
01/17/94	12:31	34.17	-118.57	6.8	6.7
01/17/94	13:06	34.20	-118.56	4.8	
01/17/94	13:26	34.29	-118.46	4.7	4.9
01/17/94	13:56	34.26	-118.65	4.8	
01/17/94	14:14	34.47	-118.61	4.5	
01/17/94	15:54	34.35	-118.65	5.0	
01/17/94	17:56	34.21	-118.59	4.6	4.6
01/17/94	20:46	34.25	-118.61	5.3	5.0
01/17/94	23:33	34.35	-118.79	6.2	6.0
01/18/94	00:43	34.14	-118.13	5.7	
01/18/94	04:01	34.31	-118.67	4.6	4.4
01/18/94	15:23	34.34	-118.60	5.1	
01/19/94	04:40	34.33	-118.58	4.6	4.3
01/19/94	21:09	34.55	-118.59	5.5	5.3
01/21/94	18:39	34.26	-118.51	4.7	4.6
01/21/94	18:52	34.23	-118.46	4.6	

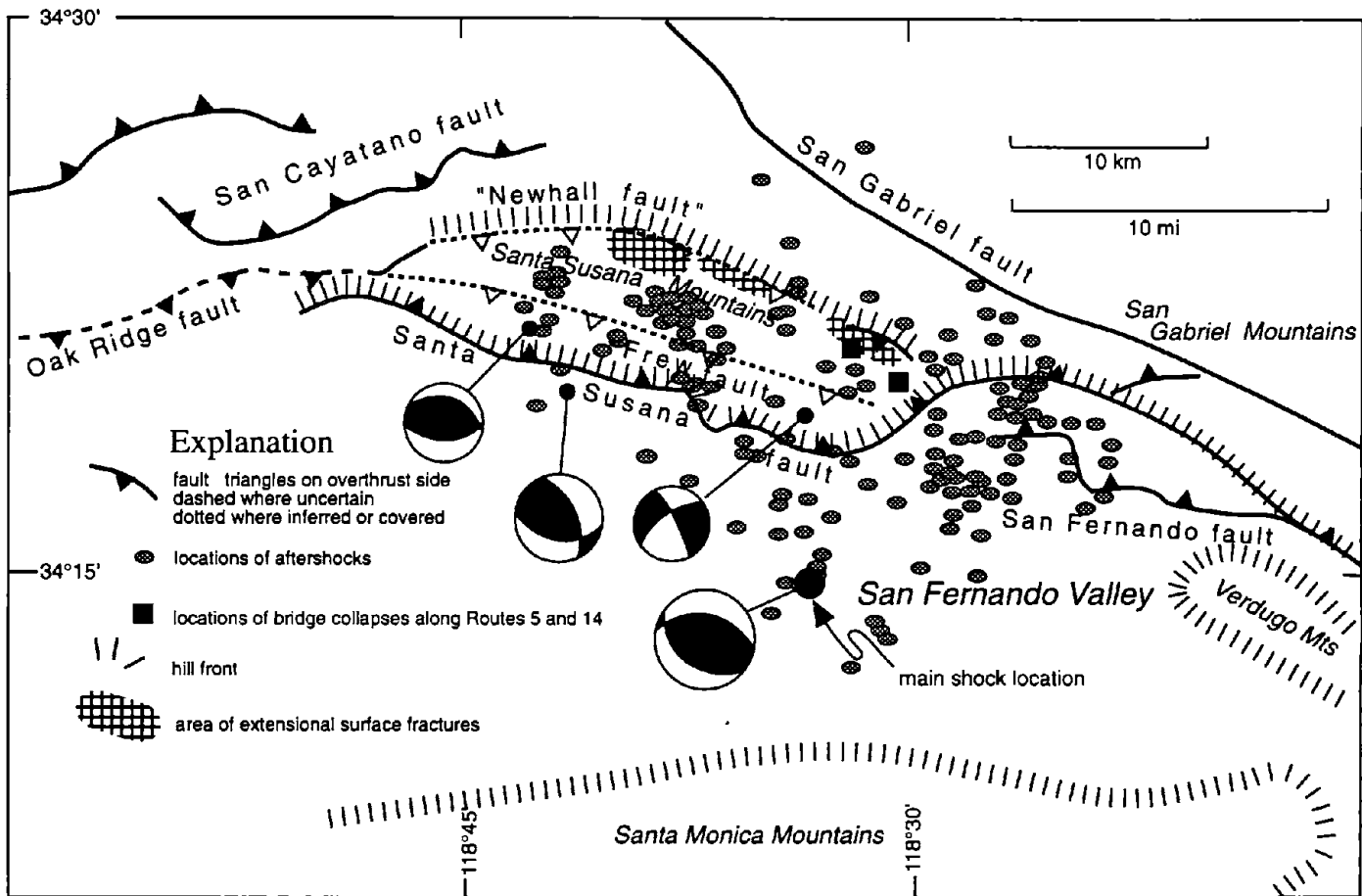


Figure 2.1: Map of key seismological features in the area of the Northridge earthquake. The focal mechanism of mainshock indicates pure thrust faulting. Aftershocks indicate that the south-dipping focal plane, is the rupture plane. This plane projects to the surface near the northern edge of the Santa Susana Mountains (where the "Newhall fault" is provisionally located). Aftershocks approximate the location of the Northridge rupture plane. Extensional surface fracturing was documented in the areas marked with hatures. Bridge failures within the zones of extension are located. Note that the 1971 San Fernando (Sylmar) earthquake ruptured the adjacent north-dipping San Fernando fault.

Source Time Functions for 9401171231 Northridge Earthquake Estimated from Empirical Green's Function Deconvolution

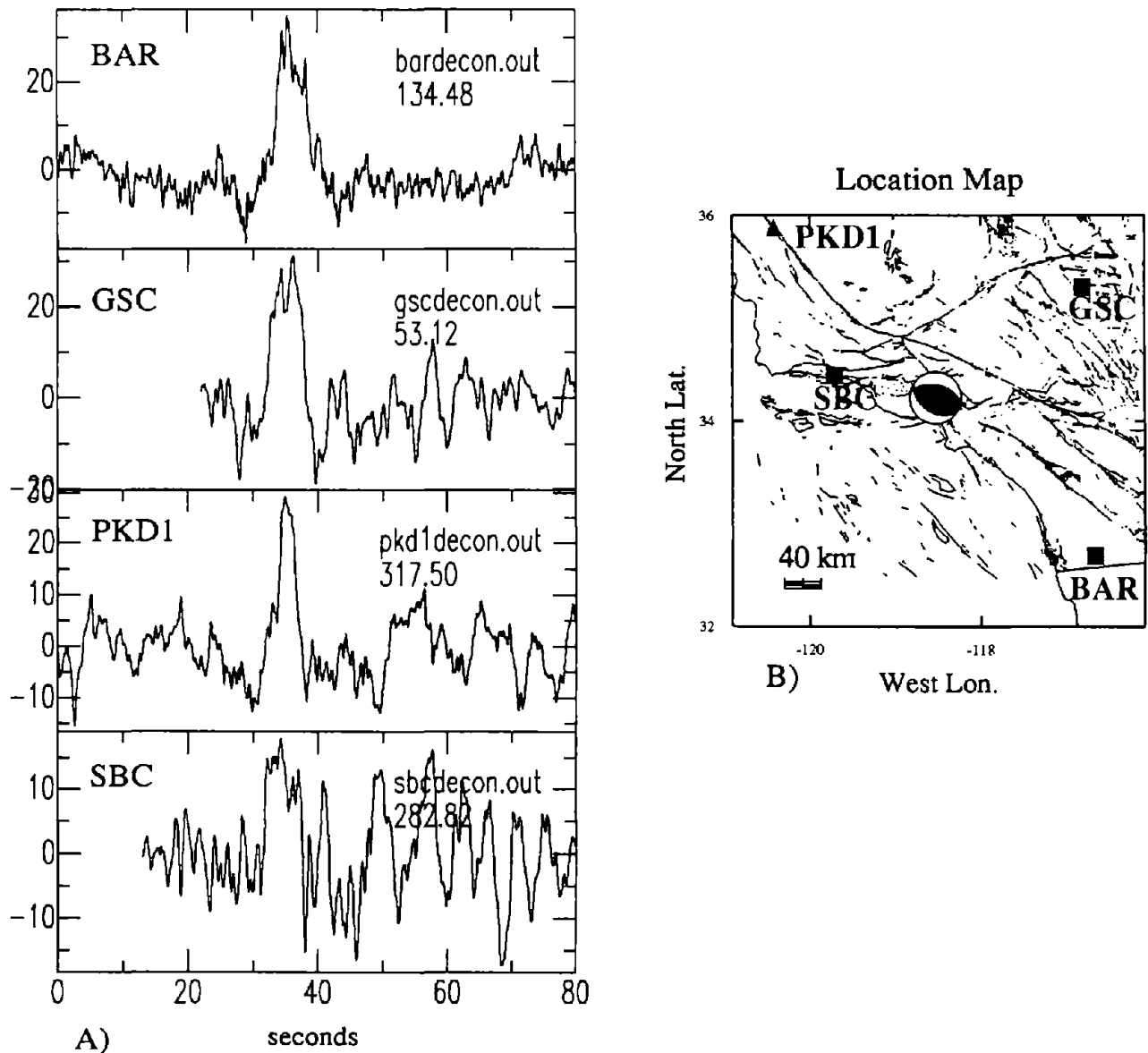


Fig 2.2. a) source time functions obtained by deconvolving the motions of a nearly co-located aftershock with a focal mechanism similar to that of the mainshock. The deconvolution was performed in the spectral domain and the empirical Green's function spectra were corrected with 1% water-level to minimize instability introduced during the deconvolution process. TERRAscope stations BAR, GSC and SBC reveal 6 second source durations. The duration at BDSN station PKD1 is shorter (4.9 s) indicating a component of northward directivity during the earthquake rupture. Assuming a circular fault, a duration of 6 seconds gives a fault radius of 8.2 km. Considering the seismic moment obtained from inversion of complete waveforms ($1.2 \cdot 10^{26}$ dyne-cm) and a rigidity of $3 \cdot 10^{11}$ dyne/cm², the average slip on the fault plane is estimated to be approximately 1.9 meters. b) shows the locations of stations used in the analysis.

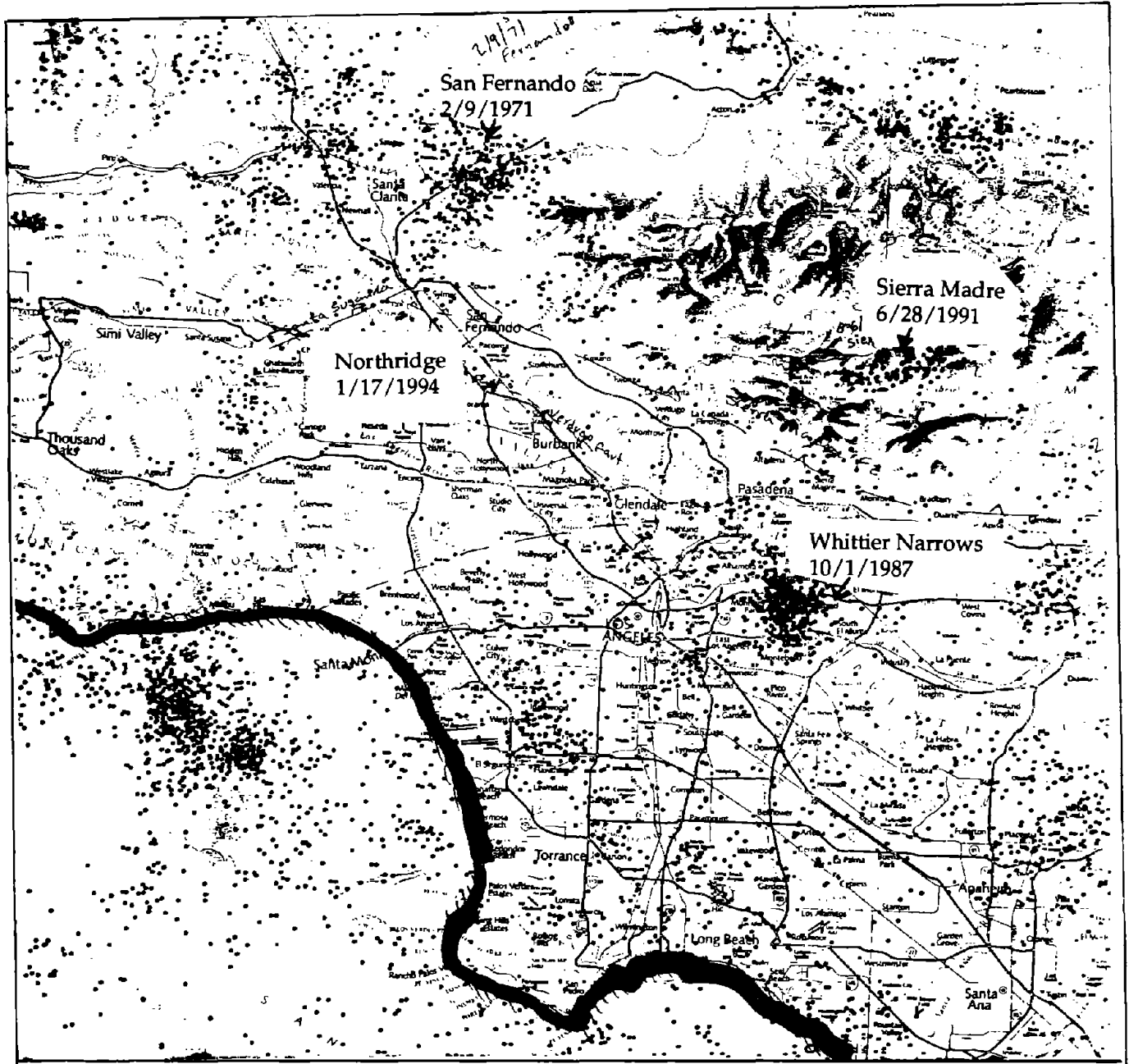


Fig. 2.3. Excerpt from National Earthquake Information Center epicenter map for the area north-west of Los Angeles, showing epicentral locations of recent large events relative to that of the Northridge earthquake. The thick line is the coast. The scale is approximately 3 cm = 10 miles.

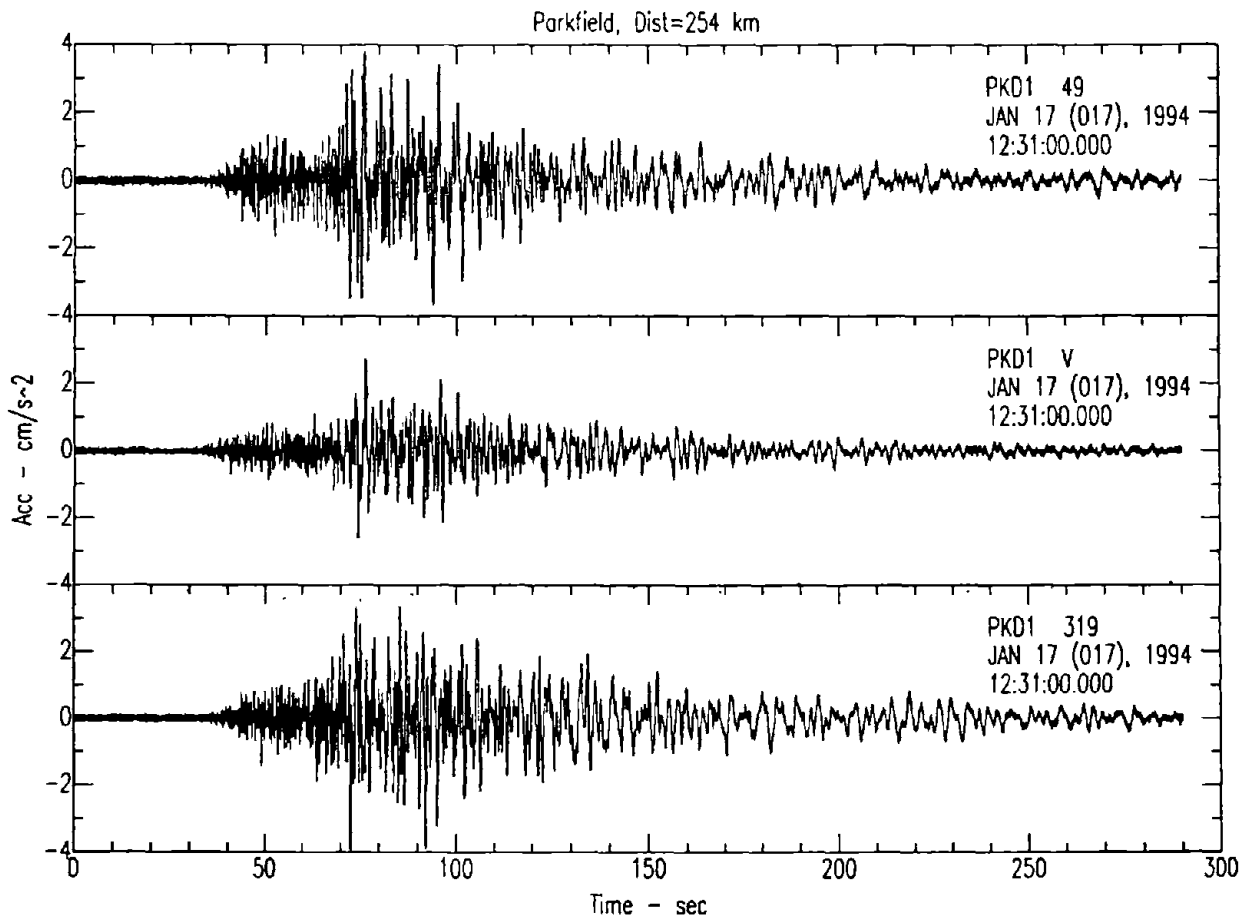


Fig. 2.4. Strong ground motion accelerograms recorded at the BDSN site at Parkfield from the January 17, 1994 Northridge earthquake (epicentral distance 254 km). Three traces are transverse, vertical and radial from top to bottom respectively. Peak ground accelerations were 3.70 cm/s^2 (transverse), 2.61 cm/s^2 (vertical) and 3.89 cm/s^2 (radial).

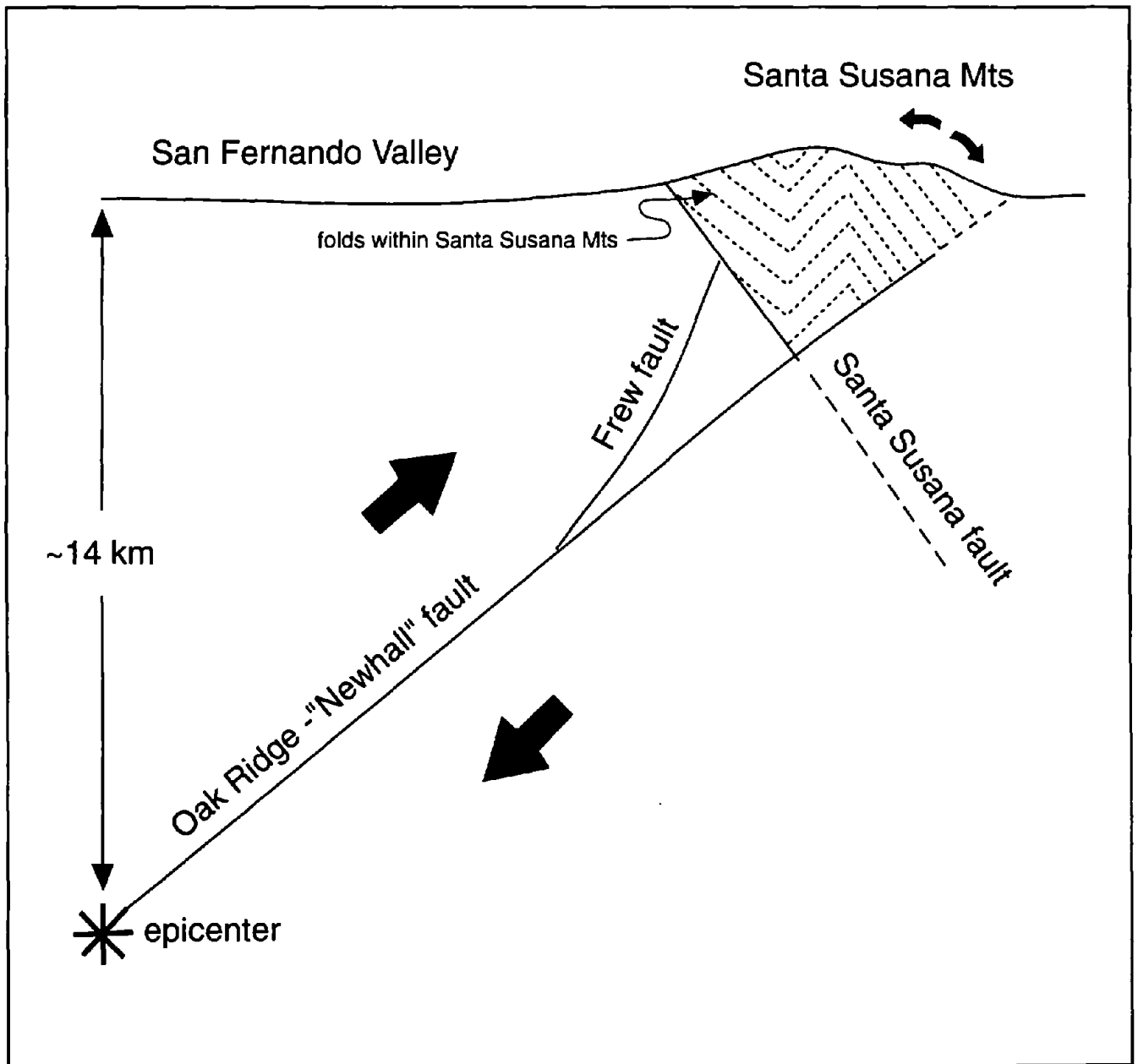


Figure 2.5: Geological interpretation of preliminary earthquake source data. A fault plane dipping about 45° to the south is inferred from the focal mechanism and aftershock data. The existence of the Oak Ridge "Newhall fault" is inferred from the seismological data and from preliminary field data. Note that extension at the surface can be produced by the transfer of slip into broad folding near the tip of a thrust.



Figure 2.6: Ground ruptures within the Newhall-Potrero Oil Field along the northern edge of the Santa Susana Mountains.



Figure 2.7: Ground ruptures within the Newhall-Potrero Oil Field along the northern edge of the Santa Susana Mountains.

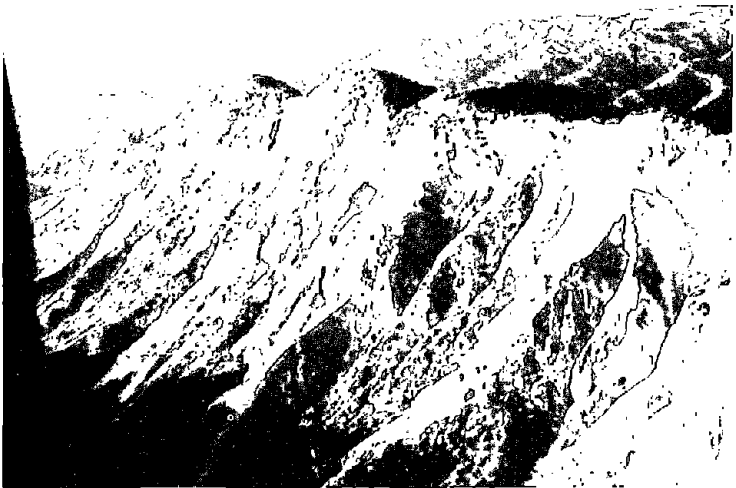


Figure 2.8: Displacements along steeply south-dipping bedding planes to the east of the Newhall-Potrero oilfield.

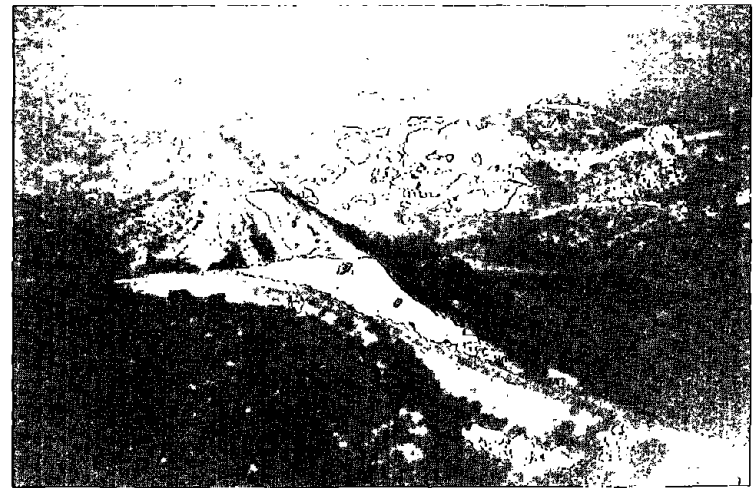


Figure 2.9: Typical rockfall, northwest of the Route 14/5 interchange, Santa Susana Mountains.

CHAPTER 3

STRONG GROUND MOTION

The Northridge Earthquake of January 17, 1994 generated a large number of strong motion recordings over a wide variety of geologic site conditions, including free-field stations on rock and soil as well as recordings of motions from instrumented structures of varying types of construction. Several agencies, such as the California Division of Mines and Geology (CDMG) Strong Motion Instrumentation Program (CSMIP), the U.S. Geological Survey (USGS), and California Institute of Technology (Caltech) each maintain relatively extensive strong-motion instrumentation networks in the affected region.

As of January 21, 1994, the only strong motion records that have been preliminarily processed and made publicly available are those from 44 instrumentation stations of the CSMIP network (1994). Refer to Table 3.1. Figure 3.1 is a map of the epicentral region showing the locations of selected CSMIP stations.

Thirty-eight out of the 44 available accelerograms from Strong Motion Instrumentation Program (CSMIP) stations were analyzed, as information regarding the site geology at six of the sites is not yet available. Table 3.2 presents a brief summary overview of the general site geology at each of the 38 stations based on data provided by CSMIP. Figure 3.2 is a plot of peak horizontal ground acceleration vs. epicentral distance, for both free-field records and records obtained at the bases of structures. These are further separated by use of different symbols for records obtained at stations sited "on soil" or "on rock". It should be noted that epicentral distance is a generally poor measure of "distance", especially in the near-field, and that closest distance to the fault rupture surface is generally to be preferred. Unfortunately, there continues to be debate regarding the precise location of the rupture surface, so epicentral distance has been used herein.

As shown in Figure 3.2, all recorded peak horizontal accelerations from the free-field rock sites plotted above the mean attenuation relationship for rock as proposed by Joyner and Boore (1988). The "closest" (based on epicentral distance) free-field instrument on rock, located at Pacoima-Kagel Canyon Fire Station #74 approximately 17 km northeast of the epicenter, recorded a peak horizontal acceleration of 0.44g. The largest free-field peak acceleration recorded on rock was 0.49g, recorded at the Los Angeles 7-story University Hospital, 36 km southeast of the epicenter.

The closest free-field instrument on soil (approximately 10m of alluvium over siltstone) is located at Tarzana-Cedar Hill Nursery, approximately 7 km south of the epicenter. Peak horizontal and vertical accelerations of 1.82g and 1.18g, respectively were recorded. It should be noted that the Tarzana station recorded much higher accelerations than stations with similar epicentral distances during the 1994 Northridge earthquake, as well

and 1991 Sierra Madre Earthquakes. However, recordings from this station during the 1992 Big Bear and 1992 Landers Earthquakes show reasonable accelerations for this site.

Peak horizontal ground accelerations from strong-motion instruments located at the bases of structures are also shown in Figure 3.2. It is likely that these values are, on the average, slightly lower than what would be recorded at a free-field site since at sites where both free-field and ground/basement floor recordings were available, the peak horizontal accelerations recorded at the structure sites were generally lower than the free-field measurements by approximately 10 to 30 percent.

Unlike the 1989 Loma Prieta Earthquake, no clear trends in amplification of ground motions at soil sites is apparent for the initial 38 stations studied in this report; however, a preliminary map of heavily damaged (unsafe) buildings prepared by the City of Los Angeles Department of Building Safety shows clusters of damage concentrated in east-west trending zones along Interstate 10 between Santa Monica and east Los Angeles, through Hollywood between Interstate 101 and Interstate 5, along Highway 134 east of Interstate 405 in Sherman Oaks, as well as in the epicentral area. Further investigation of site geology, structural basin effects, seismological and structural considerations will be required to determine how local site conditions may have contributed to these significant clusters of damage.

Figure 3.3 presents a plot of peak horizontal accelerations recorded at the 38 CSMIP stations vs. the peak vertical accelerations recorded at these stations. Although at a few stations vertical accelerations recorded were nearly equal to the recorded horizontal accelerations, and at one station the peak vertical acceleration was higher than the peak horizontal acceleration, in general, peak vertical accelerations were more typically equal to approximately two-thirds of the peak horizontal accelerations.

Table 3.3 summarizes the results of preliminary analysis of selected records obtained in and near to the epicentral region. The predominant period was read directly, due to the fact that no digitized records are available yet. The records used for this purpose are only the five nearest to the epicenter. They include both the Tarzana and the Sylmar records, which have shown unusually high values of peak acceleration. As shown in Table 3.3 the records studied show a predominant period of about .25 to .4 seconds. Also, when considering the level of acceleration above 0.05g as indicative of the duration of the stronger phase of shaking, it appears that the duration of strong shaking was on the order of 15 to 20 seconds at and near the epicentral region. These preliminary results indicate that the destructive potential of this earthquake was somewhat higher than the levels observed in the urban areas of Northern California during the Loma Prieta earthquake of 1989, as both higher near-field accelerations and a slightly longer duration of strong shaking appear to have been produced by the Northridge Earthquake fault rupture.

Finally, Figure 3.4 shows plots of the free-field acceleration time histories recorded at (a) the Tarzana-Cedar Hill Nursery (CSMIP Station #24436) and (b) the Sylmar County Hospital Parking Lot (CSMIP Station #24514). These plots are taken directly from the CSMIP preliminary reports, and are poorly reproduced. Nonetheless, they serve to illustrate the character of the motions at these sites, which are notable for their considerable duration of relatively strong shaking. The Cedar Hill Nursery site was the station that recorded the highest "level ground" accelerations released to date, and the County Hospital station is of particular interest as it is adjacent to the site of the Olive View Hospital which fared poorly in the previous (1971) San Fernando Earthquake.

Table 3.1: Data Recovered from Selected Stations of the California Strong Motion Instrumentation Program (CSMIP) for the 17 January 1994 Northridge/San Fernando Valley Earthquake

No.	Station Name	Station Coordinates		Epicentral Distance	Maximum Acceleration		
		N.Lat	W.Long		Free-field	Base	Struct.
24386	Van Nuys - 7-story Hotel	34.221	118.471	6 km	---	0.47g H 0.30g V	0.59g H
24436	Tarzana - Cedar Hill Nursery	34.160	118.534	7 km	1.82g H 1.18g V	---	---
24087	Arleta - Nordhoff Ave. Fire Station	34.236	118.439	9 km	0.35g H 0.59g V	---	---
24322	Sherman Oaks - 13-story Commercial Bldg.	34.154	118.465	10 km	---	0.46g H 0.18g V	0.90g H
24514	Sylmar - 6-story County Hospital	34.326	118.444	15 km	0.91g H 0.60g V	0.82g H 0.34g V	2.31g H
24088	Pacoima - Kagel Canyon Fire Sta. #74	34.288	118.375	17 km	0.44g H 0.19g V	---	---
24207	Pacoima Dam	34.334	118.396	18 km	---	0.54g H 0.43g V	>2.3g H >1.7g V
24464	North Hollywood - 20-story Hotel	34.138	118.359	19 km	---	0.33g H 0.15g V	0.66g H
24231	Los Angeles - 7-story University Bldg.	34.069	118.442	19 km	---	0.29g H 0.25g V	0.77g H
24389	Century City - LACC North	34.064	118.417	20 km	0.27g H 0.15g V	---	---
24643	Los Angeles - 19-story Office Bldg.	34.059	118.416	21 km	---	0.32g H 0.13g V	0.65g H
24385	Burbank - 10-story Residential Bldg.	34.187	118.311	21 km	---	0.30g H 0.13g V	0.79g H
24370	Burbank - 6-story Commercial Bldg.	34.185	118.308	22 km	---	0.35g H 0.15g V	0.49g H
24670	Los Angeles - 110/405 Interchange Bridge	34.031	118.433	23 km	---	---	1.00g H 1.83g V
24303	Los Angeles - Hollywood Storage Bldg Free Field	34.090	118.339	23 km	0.41g H 0.19g V	---	---
24236	Los Angeles - Hollywood Storage Bldg.	34.090	118.338	23 km	0.41g H 0.19g V	0.29g H 0.11g V	1.61g H
24538	Santa Monica - City Hall Grounds	34.011	118.490	24 km	0.93g H 0.25g V	---	---
24251	Wood Ranch Dam	34.240	118.820	26 km	---	---	0.39g H 0.18g V
24157	LA - Baldwin Hills	34.009	118.361	28 km	0.24g H 0.10g V	---	---
24612	Los Angeles - Pico and Sentous	34.043	118.271	31 km	0.19g H 0.07g V	---	---
24602	Los Angeles - 52-story Office Bldg.	34.051	118.259	32 km	---	0.15g H 0.11g V	0.41g H
24611	Los Angeles - Temple and Hope	34.059	118.246	32 km	0.19g H 0.10g V	---	---
24655	Los Angeles - 6-story Parking Structure	34.021	118.289	32 km	---	0.29g H 0.22g V	1.21g H 0.52g V
24629	Los Angeles - 54-story Office Bldg.	34.048	118.260	32 km	---	0.14g H 0.08g V	0.19g H
24652	Los Angeles - 6-story Office Building	34.021	118.287	32 km	---	0.24g H 0.08g V	0.59g H 0.18g V

Table 3.1: Continued

No.	Station Name	Station Coordinates		Epicentral Distance*	Maximum Acceleration		
		N.Lat	W.Long		Free-field	Base	Struct.
24569	Los Angeles - 15-story Govt. Office Bldg.	34.058	118.249	32 km	---	0.21g H 0.07g V	0.29g H
24579	Los Angeles - 9-story Office Bldg.	34.044	118.261	32 km	---	0.18g H 0.12g V	0.34g H
24283	Moorpark	34.288	118.881	33 km	0.30g H 0.15g V	---	---
14654	El Segundo - 14-story Office Building	33.920	118.390	36 km	---	0.13g H 0.04g V	0.25g H 0.17g V
24605	Los Angeles - 7-story University Hospital (Base Isolated)	34.062	118.198	36 km	0.49g H 0.12g V	0.37g H 0.09g V	0.21g H 0.13g V
24541	Pasadena - 6-story Office Building	34.146	118.147	37 km	---	0.17g H 0.09g V	0.21g H
24468	Los Angeles - 8-story CSULA Admin. Bldg.	34.067	118.168	38 km	---	0.17g H 0.06g V	0.25g H 0.17g V
24592	Los Angeles - City Terrace	34.053	118.171	39 km	0.32g H 0.13g V	---	---
24580	Los Angeles - Fire Command Control Bldg. (Base Isolated)	34.053	118.171	39 km	0.32g H 0.13g V	0.22g H 0.11g V	0.35g H 0.30g V
24401	San Marino - Southwestern Academy	34.115	118.130	39 km	0.16g H 0.09g V	---	---
14606	Whittier - 8-story Hotel	33.975	118.036	54 km	---	0.19g H 0.10g V	0.49g H
14406	Los Angeles - Vincent Thomas Bridge	33.750	118.271	58 km	---	0.25g H 0.08g V	0.65g H 0.44g V
14560	Long Beach - City Hall Grounds	33.768	118.196	59 km	0.06g H 0.03g V	---	---
14533	Long Beach - 15-story Govt. Office Bldg.	33.768	118.195	59 km	0.06g H 0.03g V	0.04g H 0.03g V	0.06g H 0.05g V
14578	Seal Beach - 8-story Office Bldg. (Base Isolated)	33.757	118.084	66 km	0.09g H 0.04g V	0.08g H 0.03g V	0.15g H 0.16g V
23622	San Bernardino - 1-story Commercial Bldg.	34.098	117.293	115 km	---	0.05g H 0.02g V	0.15g H
23631	San Bernardino - Hwy 110/215 Free Field	34.065	117.292	115 km	0.10g H 0.04g V	---	---
23631	San Bernardino - 110/215 Interchange	34.064	117.296	115 km	0.10g H 0.04g V	0.13g H 0.04g V	0.47g H 0.31g V
12636	Sage - Fire Station	33.580	116.931	165 km	0.03g H 0.02g V	---	---

Table 3.2: Site Geology at Selected CSMIP Stations

No.	Station Name	Site Geology
24386	Van Nuys - 7-story Hotel	Alluvium
24436	Tarzana - Cedar Hill Nursery	Shallow alluvium (~10m) over siltstone
24087	Arleta - Nordhoff Ave. Fire Station	Deep alluvium
24322	Sherman Oaks-13-story Commercial Bldg.	Alluvium
24514	Sylmar - 6-story County Hospital	Alluvium
24088	Pacoima - Kagel Canyon Fire Station #74	Sandstone
24207	Pacoima Dam	Metamorphic dioritic gneiss
24464	North Hollywood - 20-story Hotel	Sandstone/shale
24231	LA - 7-story University Building	Terrace deposits
24389	Century City - LACC North	Terrace deposits
24385	Burbank - 10-story Residential Bldg.	Alluvium
24370	Burbank - 6-story Commercial Bldg.	Alluvium
24303	LA - Hollywood Storage Bldg. Free Field	130m alluvium
24236	LA - Hollywood Storage Bldg.	130m alluvium
24538	Santa Monica - City Hall Grounds	Terrace deposits
24157	LA - Baldwin Hills	1m fill over shale/sandstone
24612	LA - Pico and Sentous	Alluvium
24602	LA - 52-story Office Building	7m alluvium over sedimentary rock
24611	LA - Temple and Hope	Siltstone

No.	Station Name	Site Geology
24629	LA - 54-story Office Building	Alluvium over sedimentary rock
24569	LA - 15-story Government Office Bldg.	Siltstone
24579	LA - 9-story Office Building	Alluvium
24283	Moorpark	Alluvium
24605	LA-7-story Univ. Hospital (Base Isolated)	Siltstone
24541	Pasadena - 6-story Office Building	Deep alluvial fan
24468	LA - 8-story CSULA Admin. Bldg.	Siltstone
24592	LA - City Terrace	Siltstone
24580	LA - Fire Command Control Bldg. (Base Isolated)	Siltstone
24401	San Marino - Southwestern Academy	Deep alluvial fan
14606	Whittier - 8-story Hotel	Shallow alluvium over sedimentary bedrock
14406	LA - Vincent Thomas Bridge	Alluvium
14560	Long Beach - City Hall Grounds	Terrace deposits
14533	Long Beach - 15-story Govt. Office Bldg.	Terrace deposits
14578	Seal Beach - 8-story Office Bldg (Base Isolated)	Alluvium
23622	San Bernardino - 1-story Comm. Bldg.	Deep alluvium
23631	San Bernardino - Hwy I10/215 Free Field	Alluvium
23631	San Bernardino-Hwy I10/215 Interchange	Alluvium
12636	Sage - Fire Station	Shallow alluvium over granitic bedrock

Table 3.3: Preliminary Summary of Data for 5 CSMIP Stations At and Near To the Epicentral Region

Station Name	Site Condition	Epicentral Distance (km)	Predominant Period (sec)	Duration (sec)	Characteristic Frequency (Hz)
Sylmar (E-W)	Alluvium	15	0.35	14	5.71
Arleta (E-W)	Deep Alluvium	9	0.40	16.5	4.55
Tarzana (E-W)	Alluvium (10m?) over siltstone	7	0.35	20.5	7.80
LA Storage (N-S)	Alluvium (130m?) over sandstone shale	23	0.24	15	6.40
LA Pico (N-S)	Alluvium	31	0.40	13	4.15

***Duration of accelerations greater than 0.05g.**

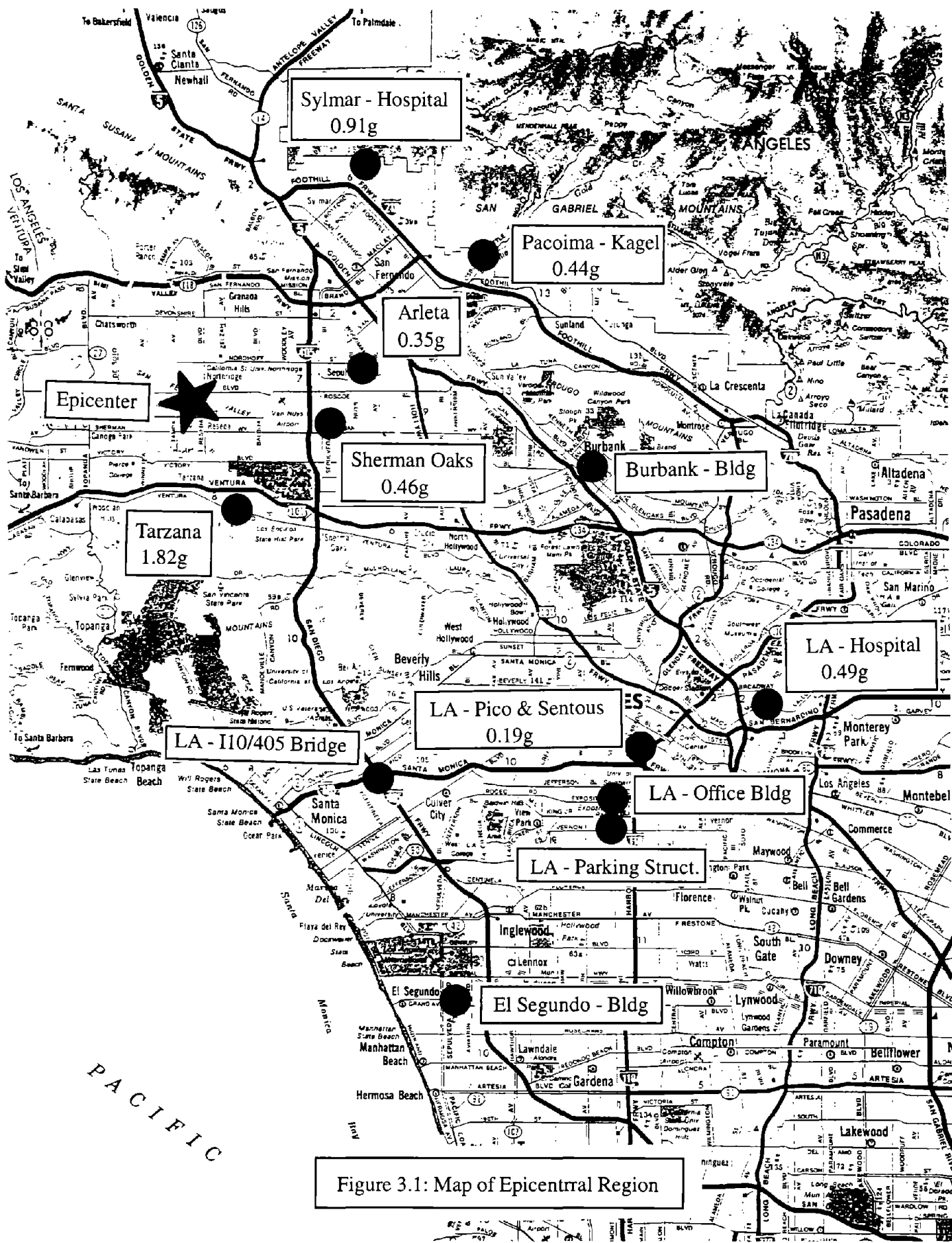


Figure 3.1: Map of Epicentral Region

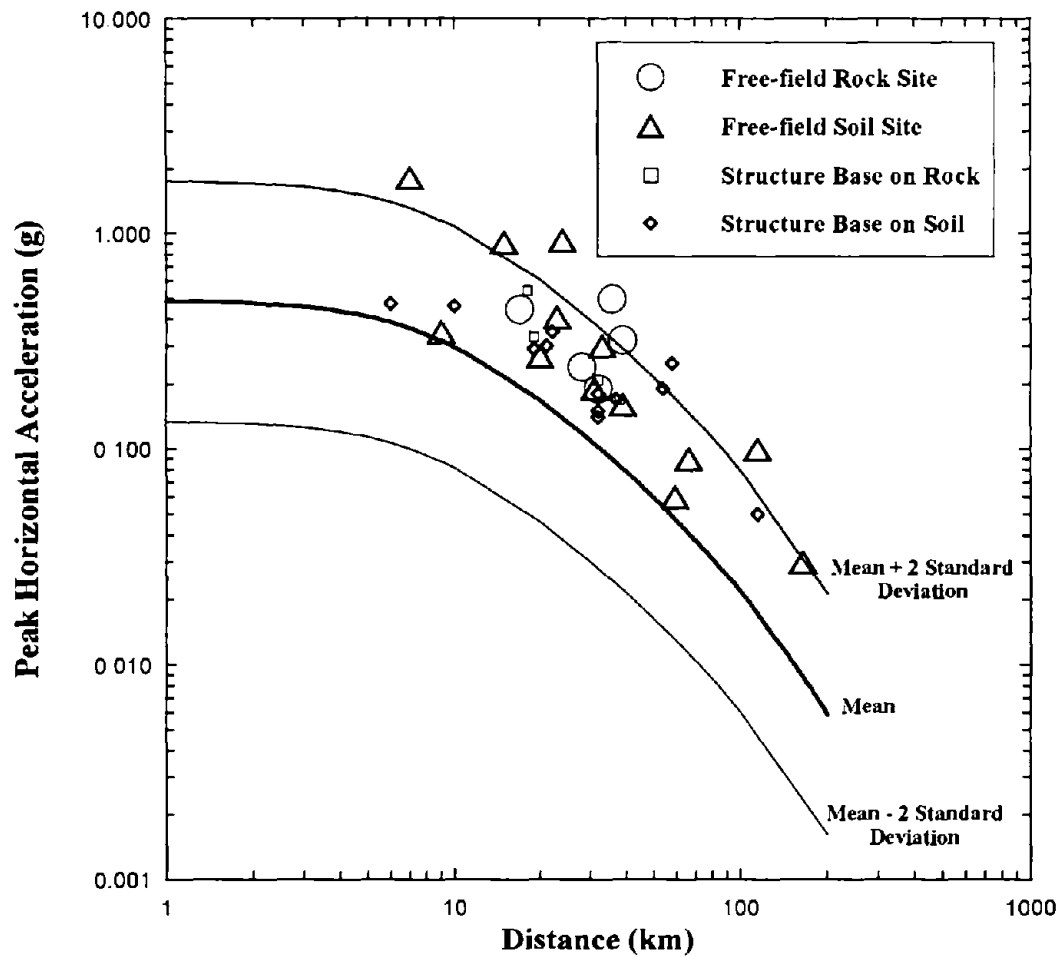


Figure 3.2: Peak Horizontal Acceleration vs. Epicentral Distance, and the Joyner and Boore (1988) Attenuation Relationship.

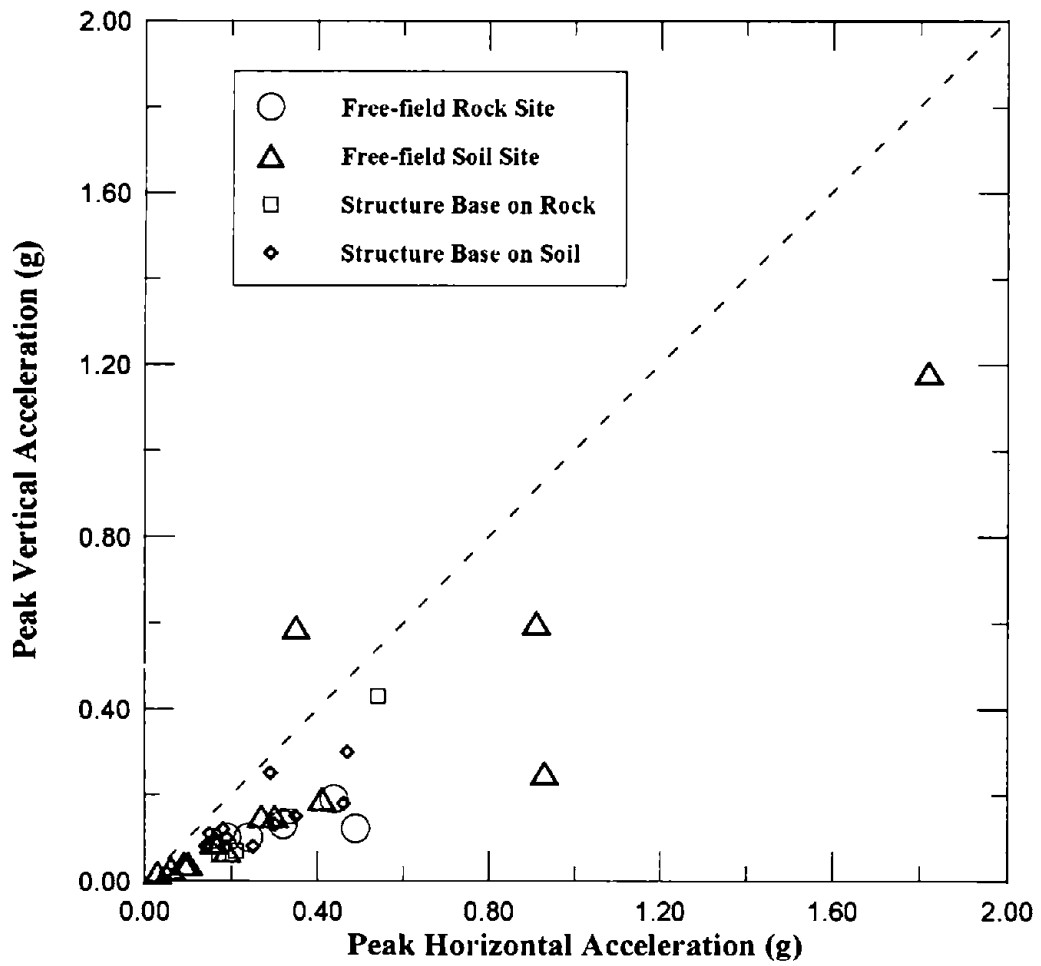
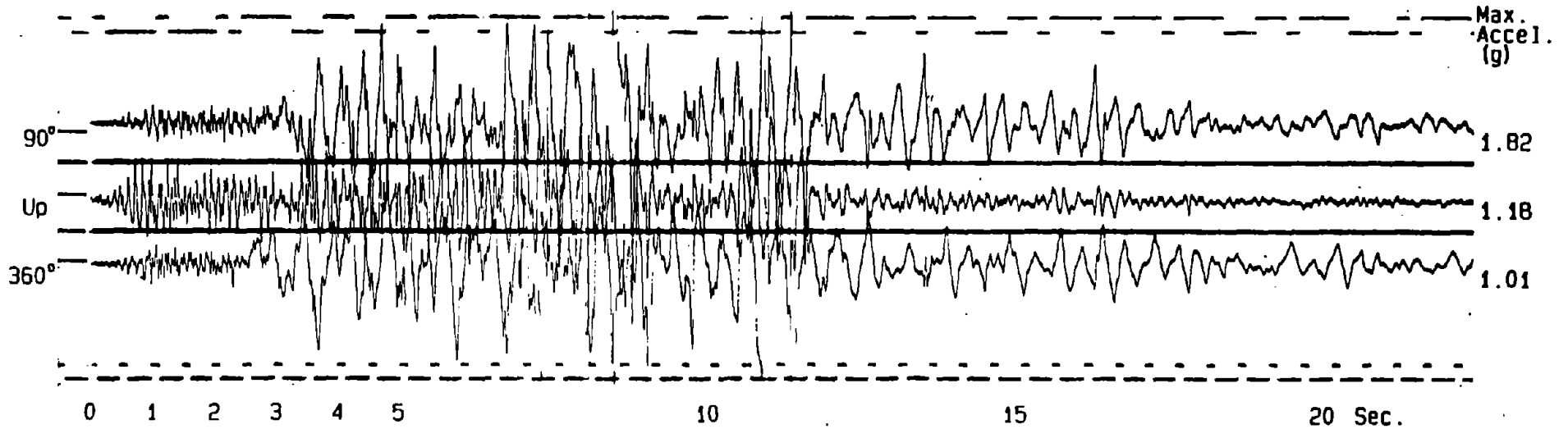


Figure 3.3: Plot of Peak Horizontal Acceleration vs. Peak Vertical Acceleration at 38 CSMIP Strong Motion Recording Stations

Tarzana - Cedar Hill Nursery
(CSMIP Station 24436)

Record 24436-S1614-94017



3-11

Sylmar - 6-story County Hospital Parking Lot
(CSMIP Station 24514)

Record 24514-S5254-94017

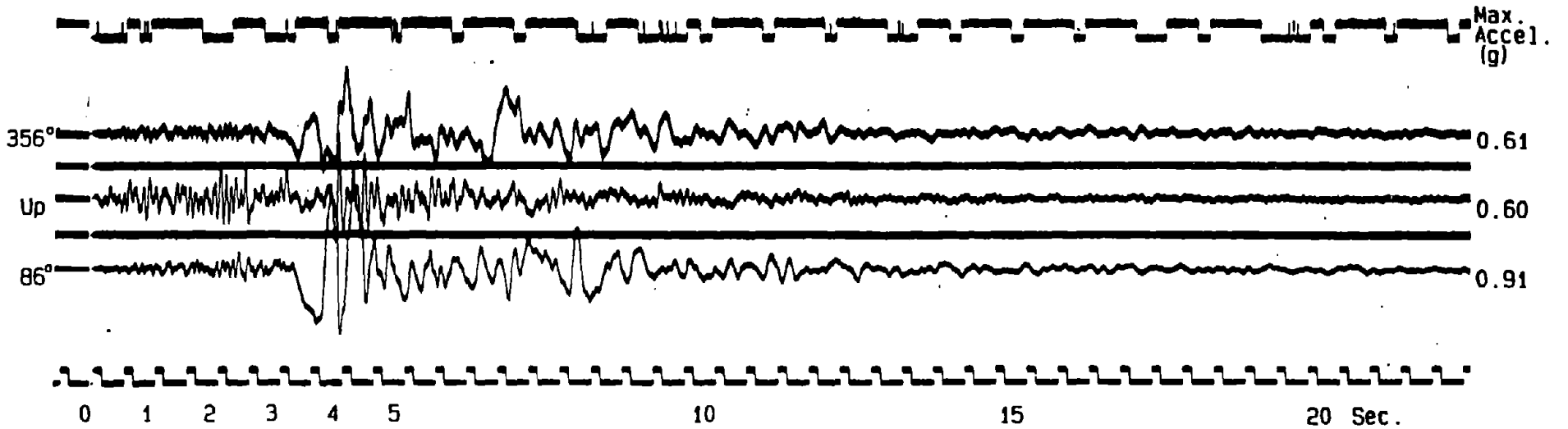


Figure 3.4: Free-Field Acceleration Time Histories Recorded at CSMIP Stations #24436 and 24514 (CSMIP, 1994)

CHAPTER 4

GEOTECHNICAL CONSIDERATIONS

The Northridge earthquake of January 17, 1994 caused extensive damage throughout the epicentral region and in several surrounding areas. Based on preliminary reconnaissance work performed immediately following the earthquake, the major geotechnical aspects of this event were found to include the following:

1. Pronounced ground movements were observed at Potrero Canyon, southwest of Magic Mountain. The observed movements mainly consisted of extensional features, but localized compressional features were also found. This ground breakage most likely resulted from areal subsidence and lateral spreading at the bedrock-alluvium contact; but the widespread surface distress in the regions overlying the northern edges of the apparent shallow southerly dipping thrust fault rupture plane may have been due in part to fracturing and distress within the folded upthrown bedrock adjacent to the primary thrust fault.
2. Local site conditions do not appear to have exerted as dominant an influence on ground shaking levels as in the recent 1989 Loma Prieta Earthquake, and the largest concentration of structural damages appears to have occurred in the heart of the epicentral region at and near Northridge. Several significant concentrations of damage occurred away from this epicentral region, however, (a) at Hollywood, north of Santa Monica Blvd. and between Highways 5 and 101, (b) at Sherman Oaks, near Highway 101 just east of Highway 405, and (c) along an arc in central Los Angeles just to the northeast of Culver City. All three of these regions of concentrated "clusters" of structural damage appear to be underlain by pronounced alluvial basins. The effects of deeper, structural basins on ground motions may also be significant, as they appeared to be in this same region in the previous 1971 San Fernando Earthquake, but there are not yet enough strong motion records available to properly investigate this.
3. Soil liquefaction and lateral spreading occurred over large areas in Northridge, near the junctures of Highways 5 and 210 and 5 and 405, and in Simi Valley. Although of relatively large overall areal extent, liquefaction and lateral spreading were of minor severity throughout most of these regions, and appeared to contribute little to the structural damages that resulted primarily from strong shaking and inertial forces in these regions. Liquefaction and lateral spreading (and compression) was evidenced mainly by curb and pavement damages, and resulted in numerous small pipe breaks in these regions. More pronounced liquefaction occurred at and near the Jensen Filtration Plant near Upper Van Norman Lake, but damage to the facility itself was relatively minor. The Juvenile Hall landslide, which occurred as a result of liquefaction near the juncture of Highways 5 and 405 during the 1971 San Fernando Earthquake, experienced minor downslope movements, offsetting curbs by approximately 3 to 6 inches or less along the approximate boundaries of the 1971 slide zone. Signs of liquefaction (minor lateral spreading, compression and/or sand boils) were also detected at various sites up to 27 miles from the epicenter, including

along the Santa Clara River between Fillmore and Highway 5, in Potrero Canyon, in the dry lakebed behind Hansen Dam, in Santa Monica, and on the coast at Marina del Rey, at Kings Harbor at Redondo Beach, and at the western end of the Port of Los Angeles.

4. Numerous landslides and rockfalls occurred near the coast at Pacific Palisades and in sparsely populated regions in the Santa Monica Mountains, the San Gabriel Mountains east of the Highway 5 and 14 interchange, and the Santa Susana Mountains. A significant coastal bluff failure occurred at Pacific Palisades, destroying several homes and closing the Pacific Coast Highway (Hwy. 1). Slope movements may also have been instrumental in disabling two aqueduct pipelines, resulting in loss of water service to the Simi Valley region. Overall, however, the widespread occurrence of slope failures, rockfalls and raveling in natural slopes and talus occurred primarily in undeveloped areas and caused little damage.
5. There are a large number of earth and rockfill dams in the strongly shaken region. A number of these experienced minor deformation and cracking, and minor slope movements occurred in several natural abutment slopes. Several embankments suffered minor damages at and near their crests, and the Pacoima Dam (a concrete dam) suffered damages very similar to those it experienced in the 1972 San Fernando Earthquake. The single "failure" of a "dam" was actually the loss of a small dike (approximately 20 feet high) retaining a minor pond at the influent basin of the DWP treatment facility near Van Norman Reservoir. There were, however, no significant occurrences of distress at major dams posing significant risk of failure, and overall performance of earth and rockfill dams appears to have been good.
6. Nine major solid waste landfills in the strongly shaken region were inspected. Several of these sustained some minor cracking within their surface cover soils, necessitating some minor re-working and re-compaction of the cover soils to reduce gas leakage (and odor). There were, however, no indications of significant distress to slopes or geosynthetic liner systems, and overall stability and performance of these fills appears to have been very good. One of the major landfills (the OII Landfill) is well-instrumented with survey monuments, inclinometers, and a pair of strong motion recording stations (on the crest and adjacent to the toe of the fill). Well-documented seismic performance data for waste landfills is currently very sparse, and the data provided by this event can be expected to be of major value to designers of waste landfills.
7. Numerous small pipe breaks occurred in areas affected by liquefaction and/or minor lateral spreading, including Northridge and the greater western San Fernando Valley area, and Simi Valley. Damage to two major aqueduct pipelines resulted in prolonged loss of water service in the Simi Valley area. A rupture in an oil pipeline resulted in significant contamination of the Santa Clara River at and west of Highway 5. Overall, performance of water systems was very good, and water service was restored to most areas by Wednesday evening.

Although minor in impact, damages related to geotechnical considerations were widespread, as shown in Figure 4.1. Due to the brief time period between the completion

of our field reconnaissance and the publication of this report, most photographs of the various features described herein were unavailable for this first report. We regret this inconvenience, but refer the reader to an upcoming report to be published through the EERC which will more thoroughly document the geotechnical aspects of this earthquake.

Ground Failure

The Northridge Earthquake caused ground failures at several locations within the San Fernando Valley and the Los Angeles basin from Highway 126 in the north to the Port of Los Angeles in the south. Although these phenomena were concentrated mainly in the epicentral region, incidents of ground failure or ground deformation did occur at distances of up to 36 miles from the epicenter.

Significant surface breakage occurred on the north flank of the Santa Susana Mountains, south of State Road 126, at Potrero Canyon. The valley, which is approximately 2.5 miles long and less than a mile wide, is covered mainly by landslide debris and alluvial material. The area was inspected and where surface breakage occurred detailed maps were developed.

Linear ground breakage features, which are roughly parallel to the east-west trend of the mountain ridges, were observed along the northern and southern margins of the valleys within Potrero Canyon. The ground fractures tend to follow the topographic contours around the base of the hills, but they do cut linearly across alluvial deposits at numerous locations. On the northern margins of the valleys, the fractures are primarily extensional, with minor right-lateral offsets. Multiple ground fractures within zones 5 to 30 feet wide accommodate as much as 2 feet of vertical movement (see Figure 4.2). The width of the fractures vary from less than ¼-inch to as much as 4 inches. Extensional features are also observed along the southern, eastern and western margins of the valleys. Minor left-lateral offsets occur along the southern margins of the valleys. Compressional features, however, are found along the southern margins of the valleys at a number of locations. These features include shallow thrusting along distinct shear surfaces that dip to the south at approximately 30 to 40 degrees with up to 6 inches of dip-slip displacement. Headscarps in the hills above these thrust features, which might have indicated that they represented toes of landslides, could not be found. Evidence of localized compression was also noted at the entrance to valleys (e.g., see Figure 4.3), but a majority of the significant ground fracturing in this area was extensional.

This site is being investigated for possible evidence of surface fault rupture, but it appears that much of these previously stated observations can be explained by earthquake shaking-induced compaction of alluvial sediments and large-scale lateral spreading. The surface area of the alluvial deposits bounded by these ground fractures, which are predominantly extensional, is over 2000 acres. The geology in this region is extremely complex and this situation is exacerbated by the fact that oil and water have been withdrawn from the area over the last hundred years. Further investigation, including trenching and surveying, is warranted at this site.

Soil Liquefaction and Lateral Spreading

Evidence of soil liquefaction including sand boils, ground settlement, and lateral spreading was found over a fairly widespread area, as shown in Figure 4.1. Damage associated with liquefaction generally included breakage of buried pipelines and pavement cracking/buckling. Based on the preliminary results of our reconnaissance, liquefaction does not appear to have directly contributed significantly to any structural failures of buildings or highway structures, and was a relatively minor factor as compared to strong shaking in terms of damage to structures in most areas. Much of the "lateral spreading" damage noted in urban areas appears likely to have resulted either from minor liquefaction at depth, with non-liquefied overlying soils largely mitigating surface distress, or from cyclic compaction of non-saturated alluvium.

The northernmost area found to show evidence of liquefaction to date was along the Santa Clara River between Highways 23 and 5. A thorough reconnaissance of this area was impossible due to an oil cleanup operation. However, sand boils were found in Potrero Canyon and surrounding areas, and adjacent to bridge piers for the crossing of Highway 23 over the Santa Clara River. An example of the Potrero Canyon sand boils, which have a significant fines content, is shown in Figure 4.4. Near a particular pier under construction at the bridge for the Highway 23 crossing over the Santa Clara River, sand boils were observed near the pier and cracks induced by lateral spreading were found approximately 15 feet away from the pier. No significant damage occurred to the bridge structure as a result of this liquefaction. Liquefaction was also observed by a local resident at a site along the Santa Clara River near Piru. Based on the reports of this individual, sand boils emerged during both the main shock and a magnitude 5.5 aftershock, but not during somewhat smaller 5.1 aftershocks. Having thus bracketed the earthquake magnitudes wherein liquefaction occurred, this site may represent an interesting case history against which to calibrate liquefaction analysis procedures.

In the 1971 San Fernando earthquake, the Lower San Fernando dam suffered extensive damage and was nearly overtopped due to sliding of the upstream shell induced by liquefaction of a portion of the shell material. Following the 1971 earthquake, the dam embankment was repaired and the crest lowered, but the repaired embankment was not intended to impound water. In the recent earthquake, this area again experienced liquefaction. Sand boils, sand fissures, and earth fissures were observed approximately 120 to 500 feet from the upstream toe of the now inactive embankment. The earth fissures were oriented parallel to the axis of the dam, were up to 8 inches wide with 8-inch vertical offsets, and were generally 120 to 250 feet from the upstream toe. Earth and sand fissures oriented perpendicular to the axis of the dam were more prevalent in the region beyond 300 feet from the upstream toe.

The Jensen Filtration Plant, adjacent to the San Fernando Dam complex, experienced damage from ground movement and lateral spreading which forced a shutdown of the facility. This facility had been heavily damaged as a result of extensive liquefaction in the 1971 San Fernando Earthquake, and the damages in the current (Northridge) event were generally similar but were significantly less severe. Damage at the site included:

- Earth fissures 200 feet long and up to 3 inches wide with a maximum of 8

inches of vertical offset in the parking lot of the main control building. This parking lot is located at the top of a 40 foot high slope, just above the DWP Treatment Facility.

- Settlement of the ground surface adjacent to the main control building with a maximum settlement of about 4 inches.
- Several pipeline breaks, including the main influent aqueduct, as well as irrigation lines and chlorination lines.
- Minor horizontal and vertical movements across construction joints in the pipeline gallery below the main control building and reportedly in the sedimentation basins. Many of these joints had also moved differentially during the 1971 San Fernando Earthquake.
- Apparent partial floating of an underground 50 million gallon finished water reservoir, with movement on the order of 2 to 4 inches.

The ground movement causing the damage noted above may have resulted in part from liquefaction of the loose alluvial soils underlying nonliquefied fills placed during construction at the treatment facility, as occurred in the 1971 San Fernando Earthquake. No sand boils or other direct evidences of liquefaction were observed on the site; however, sand boils were observed at the base of the slope west of the facility within the DWP treatment facility.

A large ground movement, encompassing a portion of the San Fernando Juvenile Hall facility, occurred as a result of soil liquefaction during the 1971 San Fernando earthquake. The slide appears to have been partially reactivated during the 1994 Northridge earthquake, as large fissures were observed in the parking lot at the southeast corner of the facility. These fissures were parallel to a large sewer line, running southwest-northeast below the parking lot, and were up to 4 inches wide with little or no vertical offset. Additional evidence of partial slide reactivation was gathered along San Fernando Road, southwest of the Juvenile Hall facility. Cracking of the pavement and cracking and buckling of curbs were observed along the east side of San Fernando Road in the vicinity of the previously mapped landslide boundary. In addition, ground cracking was observed in a DWP facility on the west side of San Fernando Road which also appeared to correspond with the previously mapped landslide boundary.

Farther east, liquefaction was observed in the dry lakebed behind Hansen Dam, a U.S. Army Corps of Engineers flood control dam that at the time of the field inspection was not impounding water. Sand boils up to 3 feet in diameter, and sand fissures up to 50 feet long and 6 inches wide, were observed upstream of the reservoir flood zone across an approximately 300 by 1000-foot area near several ponds. Sand boils from this area are shown on Figures 4.5 and 4.6. Surprisingly large flows were observed to have been exuded from several boils and fissures which resulted in localized erosion. Lateral spreading of up to 3 feet and settlements of about one foot were also noted in some areas.

The city of Granada Hills, located north of Northridge, and northern and central Northridge experienced significant ground movement as evidenced by numerous cracks in streets and broken and buckled curbs (Figures 4.7 and 4.8). Significant ground fracturing was found west of Woodley Ave., south of Midwood Dr., east of Shoshone Avenue, and north of Highway 118. Figure 4.9 is an example of a compressional feature due to lateral spreading in this general area, whereas Figure 4.10 is an example of lateral spreading clearly

related to soil liquefaction. The cracks are typically up to 2 inches wide in the asphalt concrete pavement, although in one case a 6 inch separation between a house foundation and the adjacent ground was noted. The ground cracking generally trends east to west and continues for more than one mile. This area was also the site of numerous water and gas pipe breaks. The water pipes (68 and 48 inch diameter) and the gas pipe (12 inch diameter) on Balboa Boulevard just north of Rinaldi Street, were separated by 8 to 12 inches according to the repair crew at the site. At one location nearby, a maintenance crew reported that a 1-inch diameter pipe separated six inches and was displaced six inches laterally.

Ground failure occurred on the slope at the south side of Highway 118 just east of the Balboa Boulevard overpass. Cracks up to 6 inches wide with up to 8 inches of vertical offset were observed on the south side of Highway 118 just east of the overpass. At the eastbound on-ramp to Highway 118 at Balboa, earth fissures subparallel to the highway were found near both sides of the on-ramp. Erosion from a broken water main near the overpass caused a large void to form (exposing several piers) near the south abutment.

Many large cracks in the pavement were observed along Highway 126 between Fillmore and Interstate 5. The probable cause of these cracks is settlement and lateral spreading of the fills underlying the roadway. In Fillmore, severe cracking of asphalt pavement and concrete curbs was noted at the intersection of Celis and Wolfskill. A water line also appeared to have been broken in the area.

Lateral spreading and settlement occurred at numerous locations throughout the eastern end of the Simi Valley area, causing minor slope displacements and damage to pavements and buried utility lines. Numerous pipeline breaks occurred in this area. These movements may have resulted in part from liquefaction of loose alluvial sands underlying nonliquefied surficial soils; however, no sand boils were observed and the occurrence of liquefaction cannot be confirmed. The most dramatic example of such movements occurred at Rory Lane just north of the Arroyo Simi drainage channel where approximately 8 to 12 inches of lateral and vertical offsets were observed. A large block of material appeared to have displaced southwards towards the channel.

Evidence of liquefaction in the form of sand fissures and sand boils was observed in the southern half of the northwest parking lot at the Santa Monica Municipal Pier. Extensive cracking of the 5-inch thick asphalt pavement was typically oriented subparallel to the coastline. Lateral and vertical offsets were generally about 1½ inches, although extension cracks of up to 5 inches were also recorded. No signs of liquefaction were observed below the Municipal Pier or on the beach adjacent to the parking lot.

Damage due to liquefaction was also observed in the King Harbor area of Redondo Beach. At an artificial (man-made) beach and swimming area south of Portofino Way and Harbor Drive, numerous sand boils of up to 4 feet in diameter were found. Cracks with vertical offsets of up to 2 inches were located concentrically around the swimming area.

At Marina del Rey, a large fissure possibly due to liquefaction was reported by a representative of the Department of Harbors and Beaches at an artificial beach between Palawan Way and Panay Way. Representatives of the EERC were unable to inspect the damage, as clean-up of the area occurred prior to their arrival.

North of this area, extensive damage occurred at Marina Way and Harbor Drive. Although the cause of the damage was not apparent, two sand boils were observed at the site, and the south retaining wall along Marina Way had bulged and displaced southward. Damage in the area consisted of a broken 8-inch sewer line along the center of Marina Way, and cracking and buckling of asphalt pavement along most of the length of Marina Way.

Evidence of ground movement was also observed along Nagoya Way on the western side of the Port of Los Angeles. The area of significant ground movement was located between Berths 83 and 76, and possibly extended south to Berth 74. A reinforced concrete bulkhead was located along its entire length. In the parking lots at the north end of Ports O' Call Village, the ground adjacent to the bulkhead settled as much as 1 to 1½ inches. Cracks in the asphalt pavement (between 1/16 and 3/4 inches wide) or new separations between pavement and sidewalk were observed subparallel to the bulkhead, extending through most of the length of the parking lots. Cracks in the concrete slabs-on-grade were also observed within some of the buildings in this area. Cracks subparallel to the bulkhead as wide as ½ inch were observed in the brick-covered walkways between buildings. The worst cracking was observed near Berth 77, which opened up at the top of the bulkhead, cracking the brick patio and the slabs-on-grade inside the structures. Evidence of wall movement was also observed at this location, and a gas pipe broke nearby. Adjacent to and north of Berth 83, the paved areas around the Los Angeles Maritime Museum experienced extensive cracking and settlement.

In the northwest corner of the port facility, the America President Lines container terminal experienced lateral spreading, according to engineering staff of the Port of Los Angeles. A pile-reinforced dike at this location was displaced outward into the harbor, causing up to 1 foot of settlement in the backfill. The damage to the berth was extensive enough to require repair before it could be put back into service on Friday, January 21, 1994. These repairs were largely completed before the damage could be assessed by staff of the Earthquake Engineering Research Center, so the extent of the damage is unknown.

Landslides

The Northridge Earthquake caused scattered minor rockfalls and landslides throughout Los Angeles and Ventura Counties. Major landslides occurred in the Santa Monica and San Gabriel Mountains closing roads and destroying homes, as described below. In addition, shattered ridges were observed in the Santa Susana Mountains, north of the epicentral region.

The most damaging landslides occurred in the coastal bluffs of the Pacific Palisades in Santa Monica. Here, the northbound lanes of the Pacific Coast Highway remained closed between Temescal Canyon Road and Chautauqua Boulevard for at least 4 days following the earthquake.

Four large landslides were observed in this area, along with several smaller slides. These failures occurred in Quaternary and Pleistocene age deposits of weakly cemented sand (Jennings and Strand, 1969). The slopes where the failures occurred were 120 to 200 feet

in height, and moderately steep (between 45 and 60 degrees). The failure masses appeared to be only a few yards thick, subparallel to the slope, and had widths on the order of 300 feet. The slide debris was predominantly loose sand. The most damaging of these landslides occurred just north of Chautauqua Boulevard on the Pacific Coast Highway. This slide carried a portion of a house down the slope, and on adjacent properties, shallow concrete piers and H-piles were observed to be hanging in mid-air near the crest of the slope. Three homes at the crest of the bluff were condemned. Some evidence of topographic amplification of shaking was also observed in this residential development, as the most severe damage to homes tended to be at sites on the southeast corner of the bluff near the crest.

Santa Susana Canyon Road is a two-lane roadway that parallels Highway 118 and connects the San Fernando Valley with Simi Valley. One section of the road closed due to slides is approximately 5 miles from the epicenter and is built with cut slopes into cemented sand and weak sandstone canyon walls that form slopes of 2H:1V and greater. Slope failures varied from 25 feet to more than 100 feet in height. Slope failures and landslides occurred both downslope and upslope of the road. Debris from upslope failures generally was large enough to block the near lane of the road. Blocks as large as 5 feet in diameter were noted and at least one of the slides appeared to be a failure along intersecting joint planes. Downslope failures created extensional cracks 10 to 30 feet away from the edge of the slope and parallel with the road. One larger slide caused vertical subsidence of 5 inches in the roadway and an additional 12-18 inches along the shoulder.

Several earthquake-induced landslides were observed in the Angeles National Forest of the San Gabriel Mountains. Two major slides occurred at Dillon Divide, along the Little Tujunga Road that links Highways 210 and 14. No casualties were reported, but the volume of debris and the large size of the fallen rocks kept the road closed for four days. Once reopened, this road served as a main alternative access to Santa Clarita. At least 10 other slides were observed along the Little Tujunga Road.

Also along the Little Tujunga Road, major pavement cracks were observed at Bear Canyon and Sand Canyon. These fresh cracks were up to 1 in wide, continuous, and hemispherically shaped, indicating deformation of the underlying fill. Retaining structures, consisting mainly of reinforced concrete crib walls, were inspected and no damage was observed. Several reinforced concrete crib walls, built by the Angeles National Forest as debris basins along Schoolhouse Canyon and the West Fork canyon, were reported to have suffered no damage by local authorities. Road crews also reported a major rockslide on Placerita Canyon Road.

North of Simi Valley in the Big Mountains (near the north end of Tapo Canyon Road), landsliding occurred in an embankment adjacent to a quarry debris basin which generally contains a small amount of water. The embankment is composed of clean sand with some gravel and concrete debris. Only the base of the embankment was saturated (by water from the debris basin), and it appears that liquefaction of saturated soils near the toe may have contributed to the failure. The unsaturated soils at the top of the slope fractured into discrete blocks as they slid downslope (Figure 4.12). Workers at the quarry reported that slope movements were initiated by the main shock, but that additional movements occurred

with each of the large aftershocks.

Several shallow rockslides were observed along the hills on the north side of Highway 126 between Fillmore and Interstate 5. An example of a raveling failure in this general area is presented in Figure 4.11. Closer inspection of a particular rockslide north of the city of Piru indicated the failure occurred in weathered sandstone.

Earthquake-induced landsliding was also observed in Universal City, where a 24-foot high landslide was observed on Cahuenga Blvd.

Earth Structures

A total of nine earth or rockfill embankment dams were inspected by the EERC team after the 1994 Northridge Earthquake: Castaic, Encino Lake, Hansen, Lower San Fernando, Santa Felicia, Sepulveda, Upper Van Norman, an asphalt lined storage reservoir at the DWP water treatment facility, and a small earth embankment impounding an influent basin/reservoir. The California Division of Safety of Dams (DSOD) has undertaken an extensive inspection of all major dams within the strongly shaken region.

A number of dams suffered relatively minor cracking. The Upper Van Norman Lake dam experienced minor cracking along the crest of the embankment. About twenty feet down the west side of the downstream face, three to four inches of settlement was observed around what appeared to be concrete "mini-piles", about two to three inches in diameter. The eastern portion of the downstream face experienced moderate cracking with less cracking toward the western end of the embankment.

The asphalt lining of the DWP storage reservoir cracked at several locations, with one crack extending below the water level. There were also some broken pipes at the crest of the reservoir, but it is unknown whether the earthquake caused these breaks.

The small embankment at the southern (downstream) end of the influent basin at the DWP water treatment facility was breached and washed out following the earthquake. The embankment, which was on the order of 15 feet high and impounded a small pond, was reported to have been overtopped by DWP personnel who were at the site during the earthquake. Based on the absence of water marks on either side of the breached section and no evidence of flows higher than 2 feet in the channel below the dam, it is unlikely that the embankment was significantly overtopped. It appears likely that liquefaction of the soils below the embankment and consequent ground movements caused cracking of the embankment, which then failed due to piping and erosion, releasing the small amount of water in the pond relatively slowly. Evidence of liquefaction (sand boils) was observed along the west side of the basin, and evidence of slope movement was apparent immediately adjacent to the basin. The slope movement caused buckling of the shotcrete lining of the basin in two areas. In addition, the basin is located immediately downslope from the Jensen Filtration Plant, which was the site of large scale lateral spreading.

There was no observed damage to Castaic Dam, but some fresh cracks were observed in the asphalt parking area on Lake Hughes Road (along the right abutment). A small slide,

50 to 60 feet wide, occurred in the area north of the right abutment and upstream of the dam, towards the reservoir. Many small surficial slides along the edge of the reservoir were also observed, and the lift gate mechanism of the outlet works was damaged. Although there was no observable damage to the San Felicia embankment, there was evidence of minor renewed slide activity in areas of previous instability along the approach road above the left abutment.

Several mechanically stabilized walls were inspected along U.S. Highway 101, Interstate Highway 110, and California State Highway 2. Of these structures, signs of earthquake-induced movements were only found along Highway 101 at Universal City. At this location, there are three large, approximately 40 feet tall walls, only one of which was damaged during the earthquake. The damaged crib wall is part of the onramp access to Highway 101 from Coral Drive. A crack parallel to the wall facing, approximately 120 feet long with 1 inch of vertical and horizontal displacement, was observed 6 feet behind the face of the wall. A second crack, also parallel to the facing, was located approximately 25 feet from the wall. The two additional crib walls at this location showed no signs of distress. Retaining structures were also inspected in the Angeles National Forest. These walls were primarily reinforced concrete crib walls, and although several landslides were observed in the region, no damage to the walls was observed.

Several other earth and/or rockfill dams experienced minor cracking and distress, and several suffered damages to their abutments and/or reservoir slopes. Details can best be obtained at this early juncture through DSOD. Similarly, the Pacoima Dam (a concrete structure) suffered damages very similar to those it suffered in 1971. It is interesting to note that peak accelerations of approximated 2g were recorded at both the crest and an abutment station at Pacoima Dam.

Overall, no major dams or embankments suffered significant damages posing any threat of failure, and the performance of dams was generally good.

Solid Waste Landfills

The 1994 Northridge earthquake provided important observational data on the response of landfills to strong levels of earthquake shaking. A large number of landfills in the Los Angeles area were located close to the epicenter and experienced strong levels of shaking, and nine of these were inspected after the event. Although no landfills demonstrated any signs of a major instability, several experienced minor levels of damage (cracking).

The Simi Valley and Puente Hills landfills experienced minor cracking as a result of the earthquake shaking. At the Sunshine Canyon landfill, longitudinal cracks were observed along the crest of the waste fill. Similar minor cracking occurred at several locations on the faces of slopes of the Operating Industries Inc. (OII) Landfill, mainly at or near to berm roads. The cracks, however, did not extend through the soil cover system at these landfills, and were minor in extent, being generally on the order of 1 or 2 inches or less at their widest point and showing little or no shear offset. The cracking appeared to represent simple brittle cracking of the stiffer compacted cover soil veneers overlying the more ductile waste fill, and did not represent any threat of incipient instability. At the Lopez Canyon

landfill (shown on Figure 4.13), minor cracking was observed at the interface between the waste fill and the natural canyon slopes. Preliminary studies indicate that the relatively slight damage observed at these landfills poses no significant risk and can be easily repaired.

At the Chiquito Canyon landfill, a minor amount of damage from the earthquake was reported by the owner. The landfill was accepting waste at the time of the earthquake. Longitudinal cracks were observed at the crest of the landfill along the interface between the landfill liner and the waste fill. The slopes in this area were graded at approximately 2H:1V. The cracks were several inches wide with a vertical offset of several inches causing, in one area of the landfill, a small tear in the HDPE liner. The report on this landfill is preliminary and the site survey information will have to be closely examined before any conclusions can be drawn about its performance.

In general, the performance of the major landfills, several of which appear to have been subjected to peak bedrock (input) accelerations of 0.2g to 0.5g, was very good.

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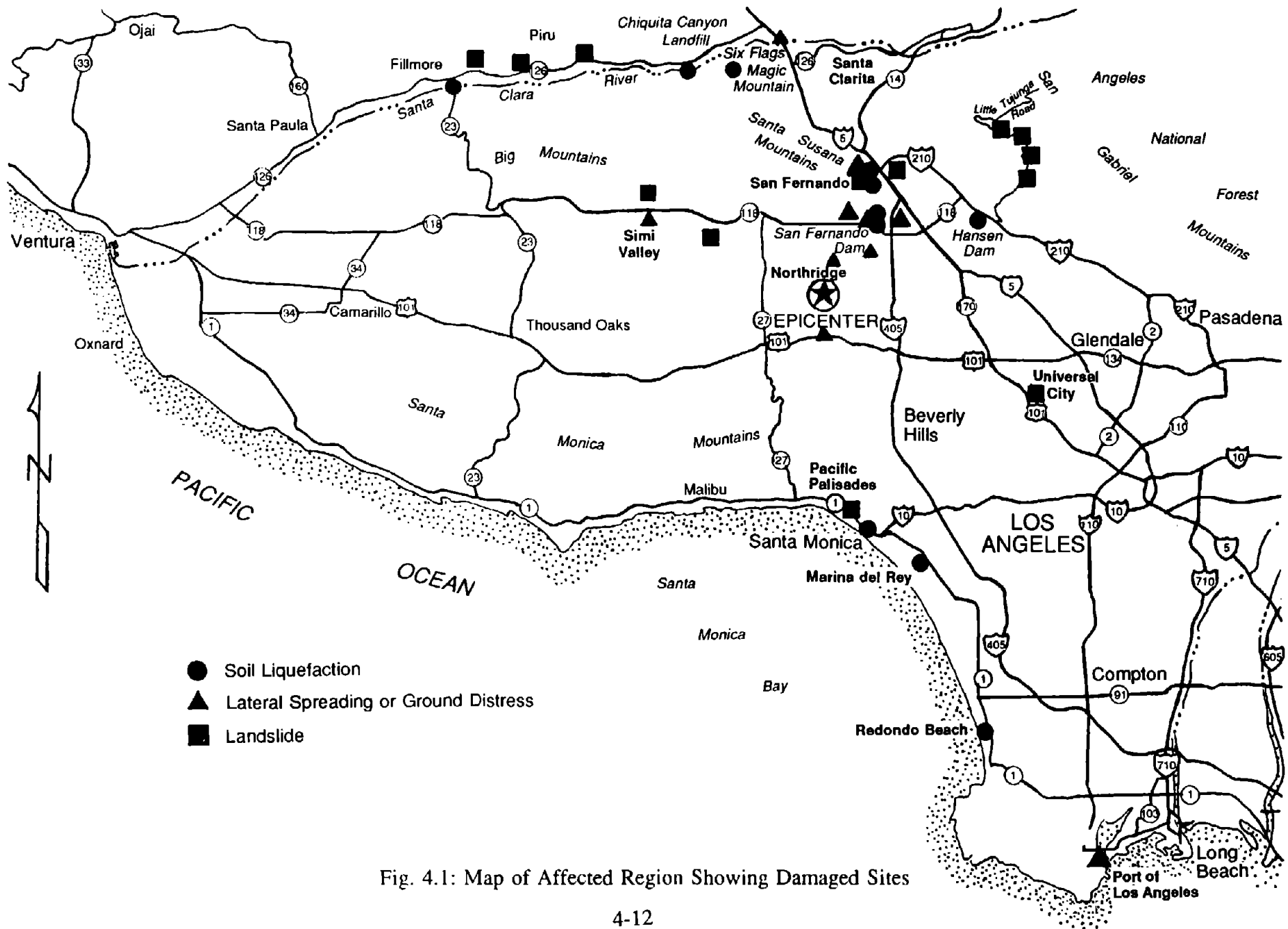


Fig. 4.1: Map of Affected Region Showing Damaged Sites



Fig. 4.2: Extension feature at soil to bedrock contact in Potrero Canyon. Note 1-foot long ruler at right side of photograph.



Fig. 4.3: Evidence of localized compression in Potrero Canyon. Note that originally straight pipe was pushed up and laterally; shortening across its length was 5 inches.



Fig. 4.4: Liquefaction of sand deposit with appreciable fines content in Potrero Canyon

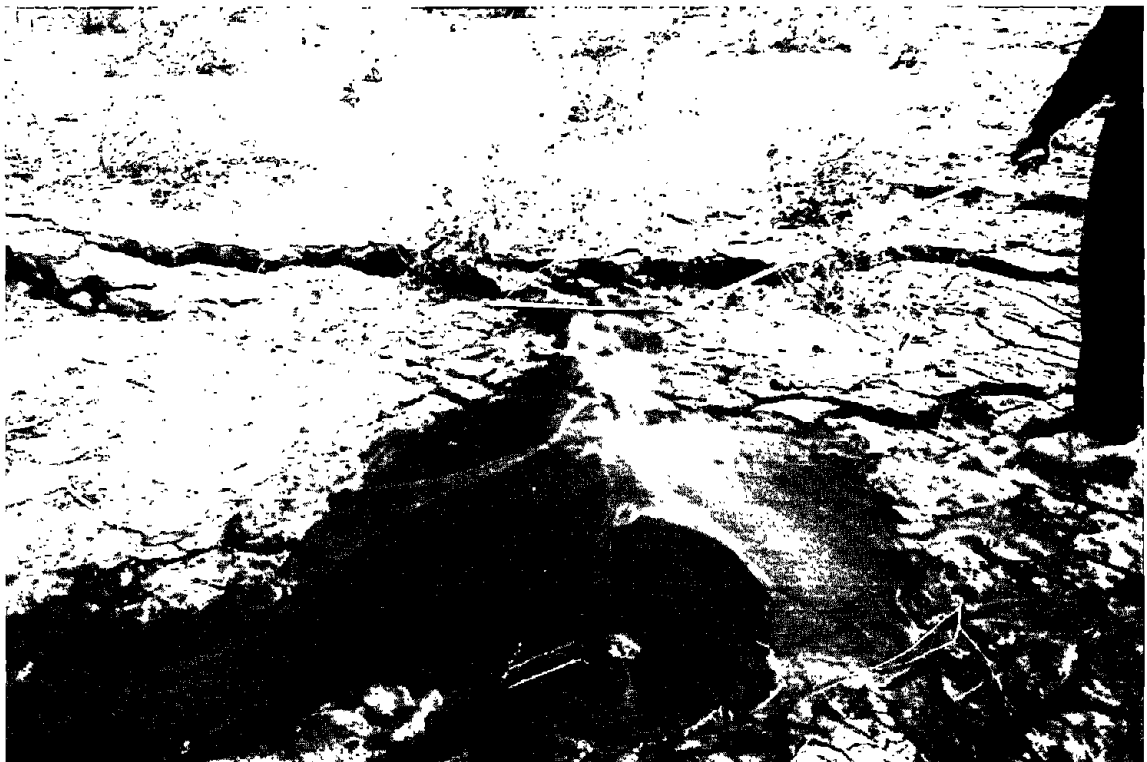


Fig. 4.5: Lateral spreading fissure and sand boil at dry lakebed behind Hansen Dam



Fig. 4.6: Roadway damaged by liquefaction and lateral spreading failure at dry lakebed behind Hansen Dam



Fig. 4.7: Damage from lateral spreading, Northridge Hospital



Fig. 4.8: Buckling of curb from lateral spreading, Northridge Hospital



Fig. 4.9: Localized compression feature and pavement damage in Northridge near Highway 118



Fig. 4.10: Evidence of liquefaction and lateral spreading, northeast Northridge



Fig. 4.11: Bluff failure and ravelling on Highway 126 near Piru

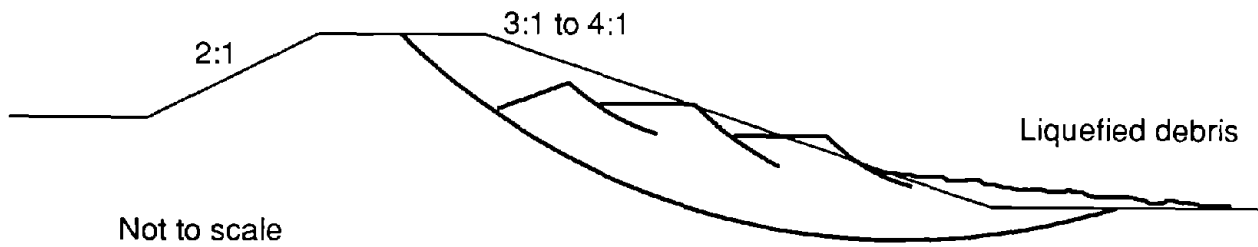


Fig. 4.12: Schematic of Slope Failure Geometry at Quarry Embankment

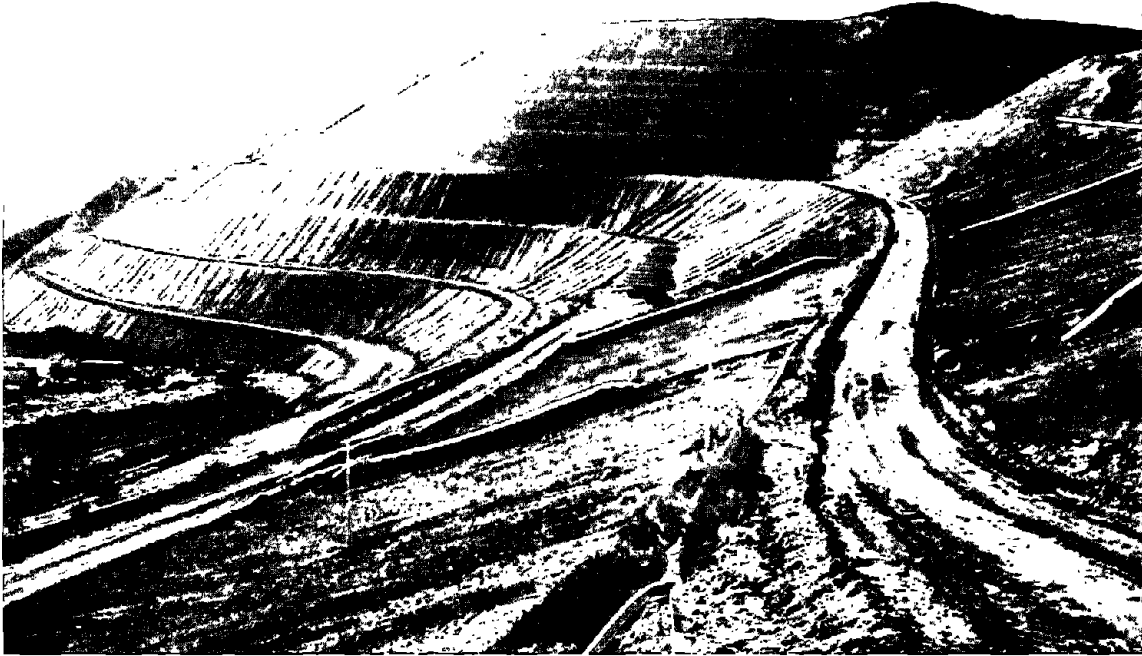


Fig. 4.13: View of Lopez Canyon solid waste landfill

CHAPTER 5

TRANSPORTATION STRUCTURES

The metropolitan Los Angeles area is highly dependent on its transportation systems. Most of the 600 mile freeway system survived the Northridge earthquake with minimal or easily repairable damage. However, the extensive damage or collapse of approximately ten freeway structures caused widespread disruption after the earthquake. The structures retrofitted by Caltrans since the 1989 Loma Prieta earthquake performed very well in most cases. Structures designed to current standards appear to have performed well, indicating that if the damaged structures had been designed to current standards many of the observed failures would not have occurred. This preliminary investigation includes reconnaissance of the freeway structures by the EERC team, review of available structural drawings, and tentative conclusions concerning the seismic performance of the structures. Unless stated otherwise, all the bridges described in this chapter are constructed of reinforced and prestressed concrete with multi-cell box girders (cast in place).

Interstate 5/Route 14 Interchange

An overall view of the Interstate 5/Route 14 interchange in Fig. 5.1 shows that the most significant damage was in the North Connector Overcrossing (west 14 to north 5) and the South Overhead (west 14 to north 5). The connectors are box girders supported by single column bents. While under construction, several structures in the interchange collapsed or were damaged in the 1971 San Fernando earthquake. The North Connector was repaired shortly after that earthquake.

North Connector (Bridge No. 53-1964F)

The eastern end frame of the North Connector collapsed, as shown in Fig. 5.2. The frame consists of a simply supported span between abutment 1 and bent 2 and continuous spans over bents 2 to 4. The simple span fell off the seat abutment, but the transverse shear keys remained intact. A shear failure in the bent 2 column appears to have initiated the collapse (Fig. 5.3). The 4 ft by 8 ft column has 42 #18 bars longitudinally and #4 ties at 12 in. with 3 #4 crossties. The column is roughly one-half as tall as the adjacent column at bent 3 and, therefore, probably attracted larger shear forces. After the shear failure of bent 2 and resulting loss of gravity load capacity, the box girder formed a hinge at bent 3 and subsequently tore out. One of the restrainer units at the hinge near bent 2 pulled out the diaphragm, and in the other unit the restrainers pulled out of the bearing plate. The next hinge, near bent 4, was barely providing support on about 2 in. of the 14 in. seat.

South Overhead (Bridge 53-1960F)

As shown in Fig. 5.4, the southern end frame of the South Overhead collapsed. The frame consists of a seat abutment and continuous spans over bents 10 and 9 with a hinge near bent 8. The shear crack in the bent 9 column (Fig. 5.6) indicates motion towards the abutment. The column at bent 10 most likely failed first because it is only about one-third the height of bent 9 and, therefore, would have attracted a larger shear force than bent 9. The loss of bent 10 could have caused the box girder to hinge at that location as evidenced by flexural cracks at the bottom of the girder (Fig. 5.5), and pulled the girder off the abutment. The increased gravity load at bent 9

then appears to have caused the cap beam failure, with the subsequent punch through of the column, and pulled the box girder off the hinge near bent 8. The splitting of the bent cap into a wedge shape along bent 9 could be due to flexural hinging of the deck as it collapsed on either side. The top reinforcement from the bent cap lay unbent across the deck near the column base.

Interstate 5/Route 118 Interchange, Southwest Connector (Bridge No. 53-2329)

Fig. 5.7 shows bent 2 in the end frame just before the connector crosses Sharp Ave. The column experienced large longitudinal forces as evidenced by the shear cracks and soil displacement on the east side of the column base. The reinforcement of the 8 ft octagonal column consists of 64 #11 longitudinal bars and #4 spirals at 3.5 in. pitch. The incipient shear failure of the column may be due to larger forces at the stiff end frame or a higher point of maximum moment in the CIDH (cast in place drilled hole) foundation than assumed for the design in 1972. The hinges, with 2 ft seat width, showed evidence of pounding, and abutment 1 was damaged.

Interstate 5/210 Interchange, Southwest Connector (Bridge No. 53-1989F)

This connector, which carries traffic from east 210 to south 5, is part of an interchange that was heavily damaged in the 1971 San Fernando earthquake. The connector has seven single column bents, seat abutments, and no intermediate hinges. In the Northridge earthquake, both abutments suffered extensive damage. At abutment 1 (northeast end) the box girder pulled out at least 3 in. and there was evidence of pounding longitudinally and transversely. Abutment 9 (southwest end) had extensive pounding damage and it spalled in a pattern consistent with twisting of the box girder. Bent 2, which is very short, had considerable spalling at the top and bottom of the column. Flexural spalling and cracking was observed at bents 3, 4, and 5. The column for bent 6 passes through an opening in the box girder of the San Fernando Undercrossing (Bridge No. 53-1730). The nominal 6 in. gap between the column and undercrossing deck pounded during the earthquake, most likely due to displacement of the deck (see below).

Interstate 5, San Fernando Road Undercrossings

Interstate 5 crosses over San Fernando Road at the interchange with Interstate 210 (Bridge No. 53-1730) and again further north near the interchange with Route 14. At both locations the undercrossing is skewed, and there is evidence of pounding and pullout at the abutments and damage to the wingwalls. The intermediate hinges also showed slight pounding damage. Several columns in the multi-column bents had minor spalling near the soffit.

Interstate 405/Interstate 10 Interchange

The connectors and overpasses at the Interstate 405/10 interchange were retrofitted with full-length steel jackets and hinge restrainers, and the foundations were strengthened. Inspection of the structure showed relative movement and pounding at most of the hinges. The connector from westbound Interstate 10 to southbound Interstate 405 experienced shear cracking of the girder seat and vertical and horizontal offset of the roadway at the hinge atop a T-shaped column, as shown in Fig. 5.8. There was no visible damage to the retrofitted northbound Interstate 405 to westbound

Interstate 10 connector. Strong motion instruments at that connector showed a peak vertical acceleration of 1.83 g as the box girder separated and pounded on the abutment.

Route 118, Bull Creek Canyon Channel Bridge (Bridge No. 53-2206)

The two bridges were designed in 1973. The westbound structure is supported by a four-column bent (bent 2) and a five-column bent (bent 3). The eastbound structure has the same geometry, but both bents have five columns. The abutments are at different skew angles. The transverse reinforcement in the columns is #5 smooth spirals at 4 in. pitch for one column diameter at the top and bottom, and 12 in. pitch in the middle. The two southernmost columns of bent 2 showed plastic hinging in and below the confined length, with fractured spirals and buckled longitudinal bars (see Fig. 5.9). The soffit adjacent to the hinged bent 2 columns were spalled with large cracks parallel to the bent. The retaining wall for the channel is located directly against the base of all the columns in bent 3. As a result of the restraint by the channel wall, all columns along bent 3 failed in shear just above the wall and the confined zone, as shown in Fig. 5.10. Both abutments appeared undamaged from below. However, the approach slab on the southeast side had been pulled south about 13 in. This may indicate that the abutment or backfill was flexible enough to allow the structure to displace transversely, increasing the forces on the columns.

Route 118, Mission-Gothic Undercrossing (Bridge No. 2205)

The Mission-Gothic Undercrossing consists of two parallel structures designed in 1973. The abutments have a 90 degree difference in skew because of the intersection of the two streets below, as shown in the schematic plan in Fig. 5.11. The westbound structure is 506 ft long with two bents, and the eastbound structure is 566 ft long with three bents. The 6 ft octagonal columns are flared at the top. During the main event and aftershocks, the eastbound structure came off the east abutment and collapsed. The westbound structure partially collapsed but remained on the abutments (Fig. 5.12). The earthquake displaced both columns in bent 3-left of the westbound structure transversely, with a plastic hinge forming below the flares (Fig. 5.13). The columns in bents 4-right and 3-right of the eastbound structure also had plastic hinges below the flares. There was additional damage to shearing deformation in the hinge. The two columns forming bent 2-left of the westbound structure were displaced in the longitudinal direction, producing hinging about the weak axis of the flare near the soffit. At the eastern abutment supporting the westbound structure, the recessed shear keys had completely spalled, while the raised shear keys were not damaged. The pattern of abutment damage and column deformation seems to indicate that the eastbound structure rotated about the western abutment. The westbound structure also rotated about the western abutment, but to a lesser degree, probably because of its shorter length.

Interstate 10, Fairfax-Washington Undercrossing (Bridge No. 53-1580)

The damaged region of this undercrossing is between the west abutment and the first hinge (Fig. 5.14). The frame is supported by a seven-column bent. Shear cracking, compressive crushing of the concrete, and symmetrical longitudinal bar buckling were evident in all the columns of the bent, as shown in Fig. 5.15. The columns at the next bent, east of the hinge, had diagonal shear cracks. As a result of the crushing of the columns, the box girder lifted off the west abutment rockers, and formed plastic hinges in the girders near lap splices of the girder

reinforcement. The adjacent Cadillac Undercrossing, which had been retrofitted with full-length steel jackets, suffered no visible damage.

Interstate 10, La Cienega–Venice Undercrossing (Bridge No. 53–1609)

This collapsed undercrossing consists of two structures, each with eight three-column bents and two end pier walls (Fig. 5.16), some of which are skewed. Hinges are located between bents 3 and 4, and bents 6 and 7; they apparently had not been retrofitted with restrainers. Shear cracking, compressive crushing of the concrete, and symmetrical longitudinal bar buckling were evident in most of the columns for the westbound structure, as shown in Figs. 5.17 and 5.18. The deck unseated at the hinge between bents 6 and 7 and apparently failed in shear at the location of the lap splices for the girder reinforcement. The eastbound structure suffered similar damage, but to a lesser extent, possibly because of restraint by a nearby ramp structure. Observation of the damaged columns revealed transverse reinforcement of #4 spliced hoops at 12 in. In some of the damaged columns, the hoops had either fractured or the lap splices had opened.

Interstate 5, Gavin Canyon Undercrossing (Bridge No. 53–1797)

Interstate 5 crosses Gavin Canyon on two skewed five-span bridges (Fig. 5.19) constructed in 1967. The central span of each structure is prestressed and cantilevered beyond the bents to provide support for the adjacent spans with an 8 in. seat at the hinge. Minor cracking at the bases of the columns supporting the center spans suggested east–west displacement or rotation in the plan of the superstructure. It appears that the displacements were sufficiently large to cause loss of bearing support at the hinge seats. The unseating apparently caused flexural failure of the box girders. Cable restrainers had been installed across the hinge seats in a 1974 retrofit.

Route 405, Jefferson Blvd. Undercrossing (Bridge No. 53–1255)

The undercrossing was designed and constructed in the early 1960's. During later construction of the Route 90 interchange, columns were built atop the outriggers supporting Interstate 405. During the earthquake, shear cracks in both directions extending into the lower column were evident on all the exterior outrigger joints of the double-deck columns (Fig. 5.20). Connectors with single column bents had been retrofit with full-length and partial steel jackets. There was no visible damage in these retrofitted connectors.

Interstate 5, Santa Clara River Bridge (Bridge No. 53–0687)

Interstate 5 crosses the Santa Clara River on two skewed seven-span bridges, each consisting of a concrete deck over six steel plate girders bearing on concrete pier walls and abutments. Pounding damage was observed between the concrete barrier rails and abutments, and at intermediate hinges. Concrete spalled about the plate girder anchorage at the top corner of the pier wall closest to the abutments (Fig. 5.21). Anchor bolts (1.5 in. diameter) attaching the eastern girders of each bridge to concrete pedestals were sheared off. Cable restrainers had broken at these locations, some by pullout of the cable from the swaged end and others by fracture of the cable (Fig. 5.22). Lock nuts on several cable restrainer connections were observed to be loose by several turns. Relative displacements of at least 3.25 in. longitudinally and 4 in. transversely were

required to open a 5 in.-diameter pipe rail at the west side of the northbound structure. The observed damage was consistent with a clockwise rotation of superstructure frames.

Route 101, Los Virgenes Overcrossing (Bridge No. 53-1442)

Los Virgenes crosses over Route 101 on a four-span bridge constructed in 1974. The bridge consists of a concrete deck over steel girders supported on three multi-column concrete bents and concrete abutments. There are hinges in the superstructure at the first and last bents and along the centerline of the deck. Pounding damage was observed at the hinges. Abutment fill settlements of approximately 4 in. were observed. Caltrans employees excavated soil around piles supporting the south abutment. Damage to a pile at the east end of the abutment was observed (Fig. 5.23); no damage was observed at the west excavation. The potential for damage to piles in other bridges with relatively minor superstructure and abutment damage may be worth further investigation.

Interstate 5/Route 126 Separation (Bridge No. 53-1626)

Route 126 west diverges from Interstate 5 on two four-span bridges constructed in 1964. Each structure is a box girder supported on three-column bents and the abutments. The columns showed evidence of rocking and each had spalled at the southeast corner near the box girder soffit. Concrete had spalled and vertical steel was exposed and deformed at the abutment connection to the box girder at the east abutments. No structural damage was present at the west abutments. Ground cracks were observed at the eastern median strip and on the eastern abutment fill slopes. These cracks increased in size to the east of the bridge, approaching 2 inches in width, 2 to 3 inches in vertical offset, and tens to hundreds of feet in length (Fig. 5.24). The abutment fill at the west approach slab of the northern roadway settled about 2 in.

Other Damage

Soundwalls constructed on concrete barriers along the northern edges of Route 101, just east of Interstate 405, were observed in various conditions. Some walls, shown in Fig. 5.25, have remained vertical, others have rotated, and another has collapsed to the south. The collapsed wall has #5 longitudinal reinforcement placed in every fourth cell (approximately 32 in. on center) of a precast masonry panel. Lap splices of approximately 28 in. in length begin at the base of the wall. Concrete had been placed in the cells containing longitudinal reinforcement with one exception. At every location the wall reinforcement had failed near the connection to the concrete barrier rail. Approximately one-half of the bars failed by fracture. Signs of necking were not observed, and the fractures occurred as far as 10 in. below the wall/barrier interface. The remaining bars had lap splice failures. The precast masonry was disrupted at the end of the wall where a lap splice failure was observed.

As of 9:51 a.m. on 19 January 1994, Caltrans reported damage to approximately thirty additional structures. Most of the reports consisted of approach slab settlement, abutment damage, bearing damage, or minor column spalling. A later report indicated damage to connectors at the Route 134/2 interchange which had been retrofitted with steel jackets. The City of Los Angeles reported damage to a pedestrian overcrossing at Wilbur Ave. in San Fernando, in addition to settlement at approach spans and shear key damage in several city bridges. Railroad companies reported inconsequential damage and nearly full operation of their bridges.

Conclusions

Although it is too soon to state definitive conclusions about the seismic performance of freeway structures in the Northridge earthquake, the types of damage do indicate important areas for further investigation and research. As has been known for some time, inadequate transverse reinforcement can lead to catastrophic shear failure of columns. This can be exacerbated by unanticipated rotational restraint at the keys into the foundation and architectural flares not spalling off as intended. Steel jacketing of columns and foundation upgrading appears to have been successful judging from the performance of retrofitted structures. In general, the structures designed using the current earthquake-resistant standards appeared to have performed well in this earthquake.

The ground motion produced by the Northridge earthquake was intense. Using the large amount of new strong motion data, further investigation will determine whether seismic zonation, site effects, and design response spectra adequately represent the ground motion in the earthquake. Evaluation of vertical ground motion may indicate that it is necessary to consider the vertical component in design. In some cases, such as the Interstate 10 and Route 118 overcrossing, local site effects may have amplified the ground motion experienced by the structures. The canyon in which the Interstate 5/Route 14 interchange is located may have experienced large spatial variations of ground motion that affected the two partially collapsed connectors.

The damage to the connectors indicated that stiff end frames may experience larger demands than predicted by the current analysis procedures. The larger forces can be caused by pounding of adjacent, more flexible frames, or sudden release of the abutment after experiencing damaging deformations. In general, the abutment damage indicates the need for improved earthquake-resistant design of abutments. Except for the Gavin Canyon Undercrossing, it does not appear that unseating of hinges precipitated other collapses. However, several cases of damage or collapse were observed in bridges with skewed hinges in the superstructure. Further investigation is needed, however, to determine the effectiveness of restrainers and seat extenders in preventing unseating failures.

Acknowledgements

The post-earthquake investigation would not have been possible without the assistance of Caltrans. Immediately after the earthquake, James E. Roberts, Interim Deputy Director, Department of Transportation, authorized access to all Caltrans facilities by the EERC team. James Gates, Division of Structures, quickly provided drawings for many of the damaged structures, and numerous Caltrans engineers provided information about the bridges.



Fig. 5.1: Aerial View of Interstate 5/Route 14 Interchange Looking South



Fig. 5.2: Overview of Interstate 5/Route 14, North Connector Failure



Fig. 5.3: Bent 2 of Interstate 5/Route 14, North Connector Failure

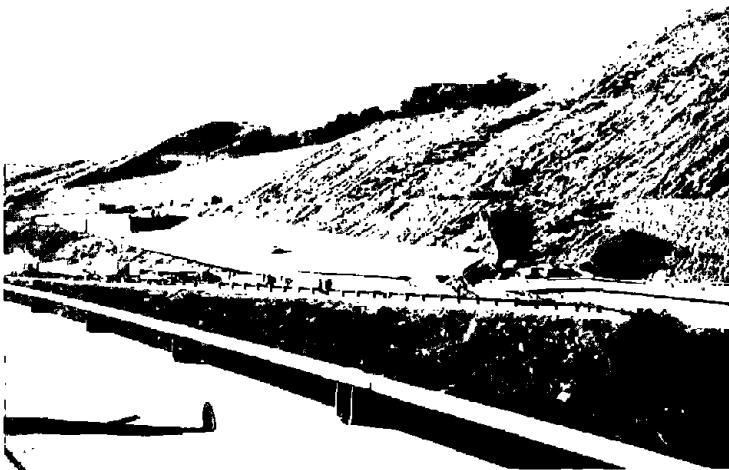


Fig. 5.4: Overview of Interstate 5/Route 14, South Overhead Failure

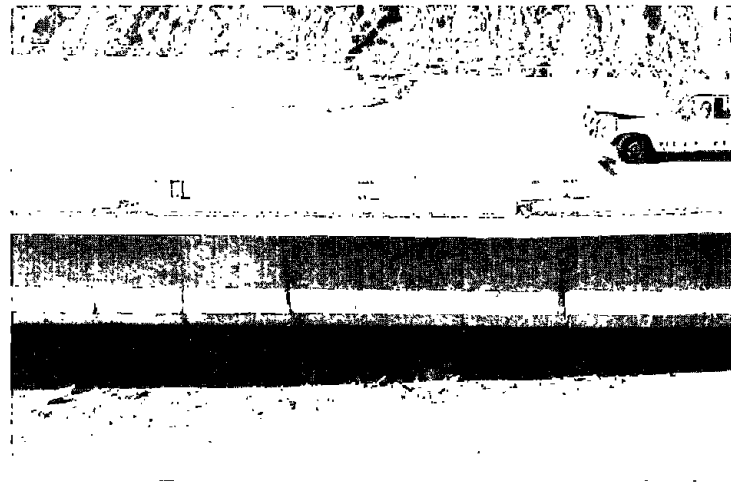


Fig. 5.5: Bent 10 of Interstate 5/Route 14, South Overhead Failure



Fig. 5.6: Bent 9 of Interstate 5/Route 14, South Overhead

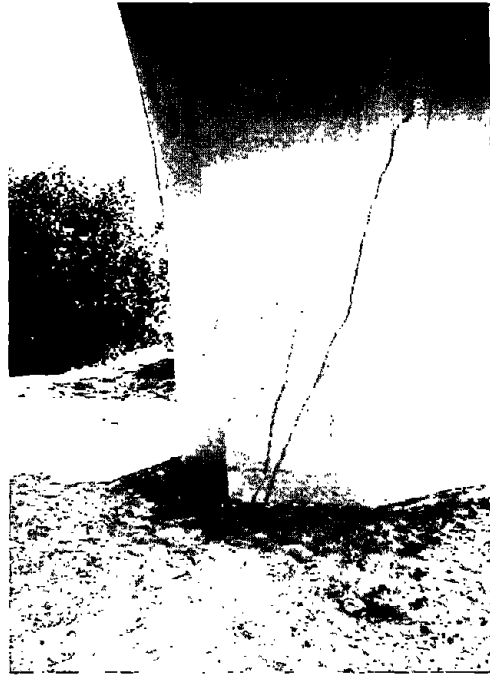


Fig. 5.7: Bent 2 of Interstate 405/Route 118 Southwest Connector



Fig. 5.8: Seat Failure at Connector from West Interstate 10 to South Interstate 405



Fig. 5.9: Column Failure at Bent 2, Bull Creek Canyon Channel Bridge

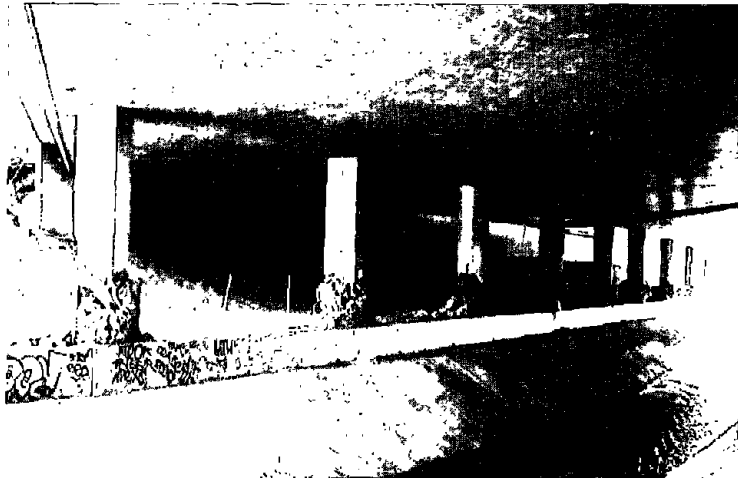


Fig. 5.10: Column Failure at Bent 3, Bull Creek Canyon Channel Bridge

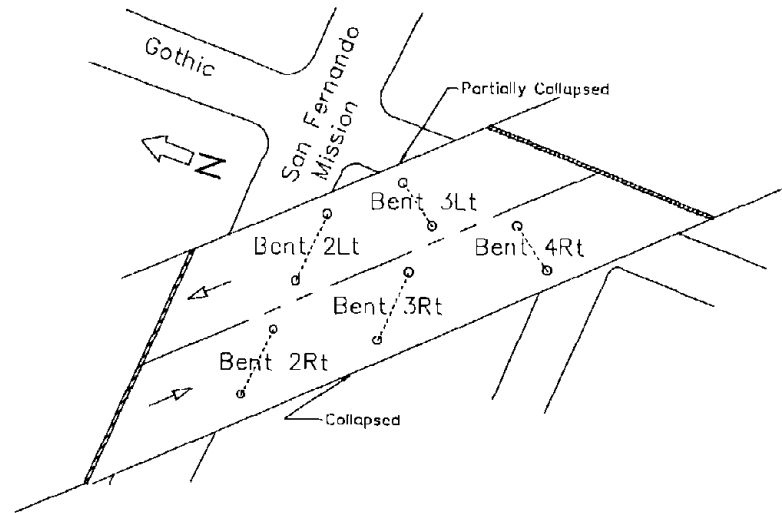


Fig. 5.11: Schematic Plan of Mission-Gothic Undercrossing Bridge

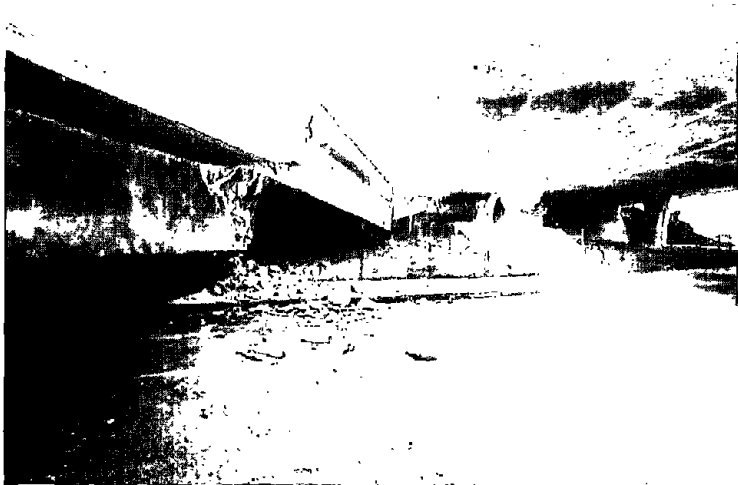


Fig. 5.12: West View of Mission-Gothic Undercrossing

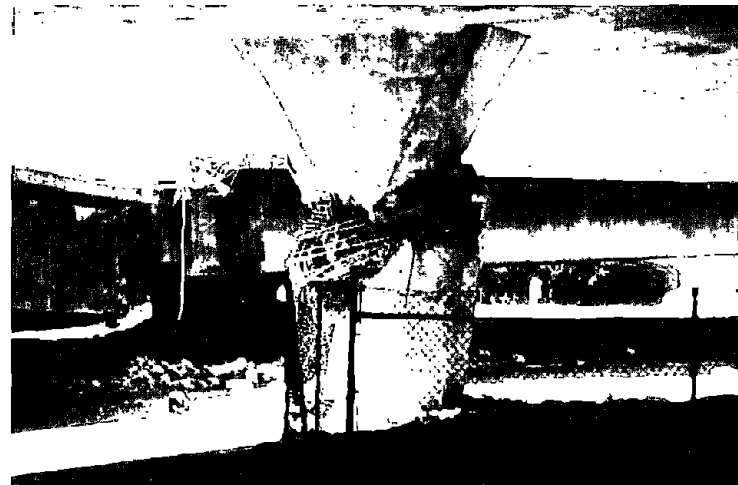


Fig. 5.13: South Column of Bent 3-Left, Mission-Gothic Undercrossing

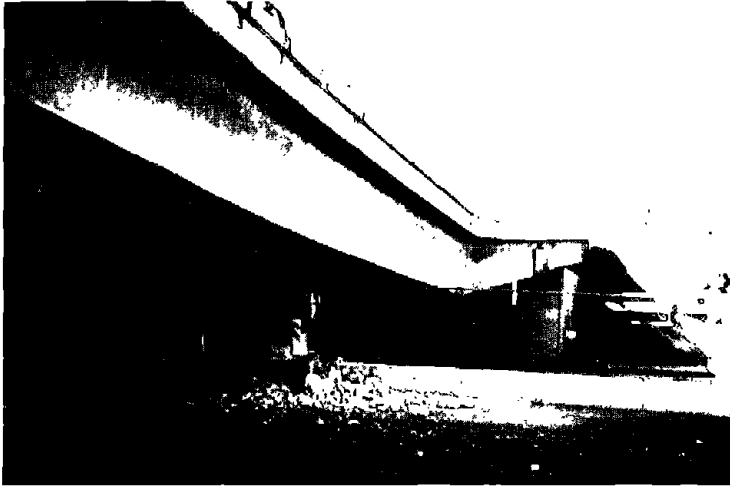


Fig. 5.14: Overview of Fairfax Undercrossing Collapse



Fig. 5.15: Column Failure at Fairfax Overcrossing

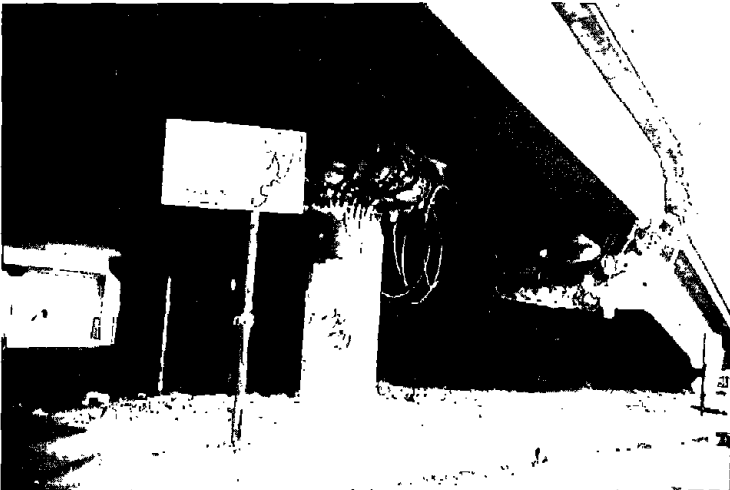


Fig. 5.16: Overview of La Cienega-Venice Undercrossing Collapse

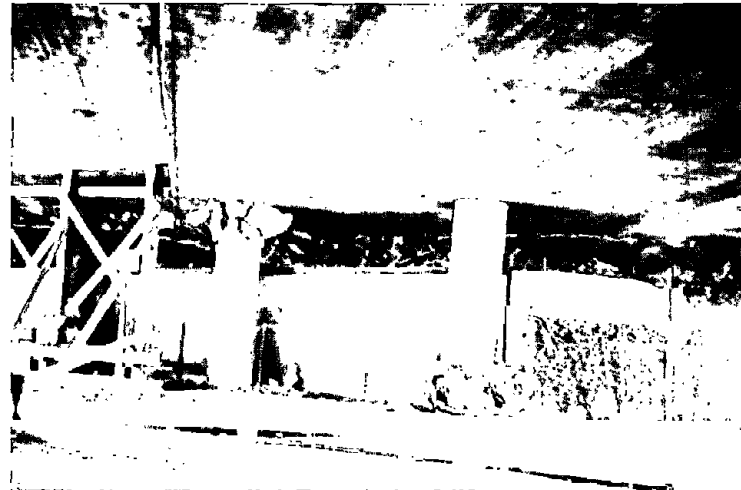


Fig. 5.17: Column Failure at Bent 7, La Cienega-Venice Undercrossing

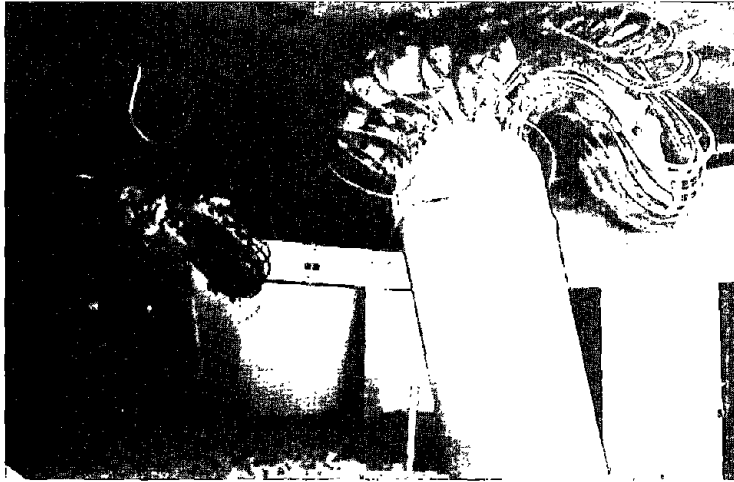


Fig. 5.18: Column Failure at Bent 6, La Cienega–Venice Undercrossing



Fig. 5.19: Collapsed Spans at Gavin Canyon Undercrossing



Fig. 5.20: Joint Failure at Jefferson Blvd. Undercrossing

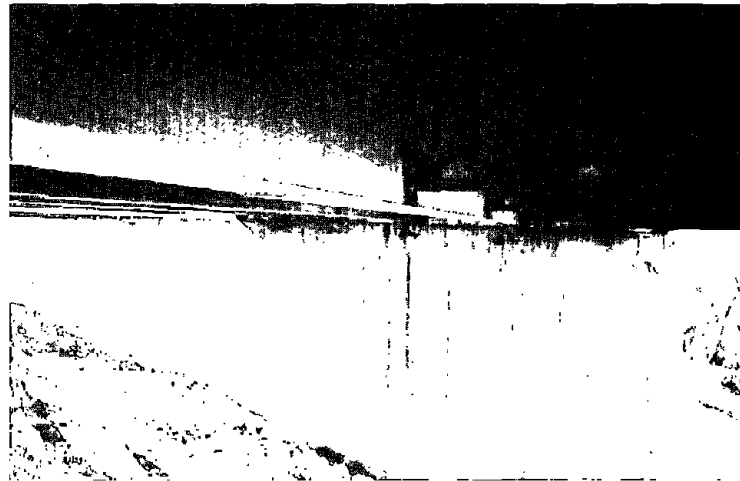


Fig. 5.21: Pier Wall at Southbound Span of Santa Clara River Bridge Looking South



Fig. 5.22: Restrainer at Santa Clara River Bridge



Fig. 5.23: Pile Damage at Los Virgenes Overcrossing



Fig. 5.24: Ground Cracks at East Approach of Interstate 5/Route 126 Separation

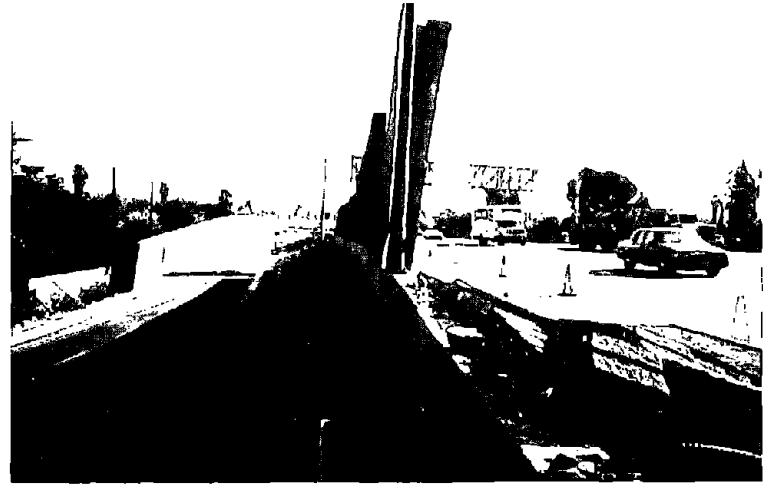


Fig. 5.25: Sound Walls at Route 101 near Interstate 405

CHAPTER 6

BUILDING STRUCTURES

General

The January 17, 1994 Northridge Earthquake affected a very large and densely populated urban and suburban area with a wide range of structural types. While the epicenter was located in the city of Northridge in the San Fernando Valley, high horizontal accelerations were recorded at sites as far as 36 km from the epicenter in downtown Los Angeles. Several free field instruments at a distance between 10 and 15 km in the north/north-east direction from the epicenter registered very high horizontal and high vertical accelerations: 0.91g horizontal acceleration at the Sylmar Country Hospital and 0.59g vertical acceleration at the Nordhoff Avenue Fire Station. The large number of buildings and structural types that were affected by the earthquake make a thorough evaluation impossible at this early stage. This preliminary report contains information on the following types of structures for which several instances of significant structural or non-structural damage, partial or total collapse were observed: reinforced concrete buildings, parking structures, hospitals, unreinforced masonry buildings, wood residential structures and base isolated structures. Parking structures had a large incidence of partial and total collapse cases among modern engineered structures and are, therefore, treated separately and at some length in this report. Hospitals are also treated separately, because of their importance and the high incidence of non-structural damage that was observed. No reports of significant damage to steel buildings were obtained at this early stage.

Reinforced Concrete Buildings

Reinforced concrete buildings suffered significant structural damage. Two buildings suffered partial collapse, while another suffered such serious structural damage that it had to be immediately demolished. The information in this report is based on structures located in Sherman Oaks (approx. 10 km from the epicenter), Culver City (approx. 25 km from the epicenter) and Santa Monica (approx. 25 km from the epicenter). All types of structural systems from ductile frames, shear walls, coupled shear walls and dual systems to non-ductile reinforced concrete frames suffered structural damage.

The observed types of failure in reinforced concrete buildings include shear cracking and compression spalling in poorly reinforced beam-column joints in the moment-resisting frame in Fig. 6.1, shear and bond-splitting failure in the column ends of the third story of the 7-story ductile moment-resisting frame in Fig. 6.2, shear failure of poorly detailed beam and column ends and beam-column joints in the five story non-ductile reinforced concrete frame in Fig. 6.3 which suffered partial collapse, short column shear from the second to the fifth floor in the six story frame in Fig. 6.4, splice failure at the corner shear wall of the dual wall-frame system in Fig. 6.5 and shear failure of the coupling beams in the 15-story coupled wall system in Fig. 6.6.

Fig. 6.1 is representative of the type of problem that was observed in several cast in place reinforced concrete parking structures which are discussed in detail in the following section.

The 7-story moment resisting frame in Fig. 6.2 is located in Van Nuys approx. 7 km east of the epicenter and represents the closest instrumented building. It was built in the mid sixties and suffered non-structural damage totaling \$400,000 during the 1971 San Fernando earthquake under a peak horizontal acceleration of 0.27g. [1]. During the Northridge earthquake it experienced a peak horizontal acceleration of 0.47g at the base and 0.59g at the roof. The strong ground motion lasted for about 15 sec and included a significant peak vertical acceleration of 0.30g which appears to precede the strong part of the horizontal ground motion by about 5 sec. The significant vertical component of the ground motion that preceded the horizontal component is evident in several records in the vicinity of the epicenter. The building suffered serious structural damage in all columns of the third floor with signs of shear-bond splitting type of failure.

Fig. 6.3 shows a detail of the non-ductile reinforced concrete frame of the 5-story Kaiser Permanente office building that suffered partial collapse. The lack of any transverse steel in the beam-column joint region appears to be an important reason of its poor behavior. Most importantly, the poor proportioning and detailing of the beams and columns of the moment resisting frame led to a soft second story mechanism.

Fig. 6.4 shows the six story moment-resisting frame of a commercial building at the corner of Olympic Blvd. and Barrington. It suffered very serious structural damage during the main shock and started showing signs of total collapse after major aftershocks that it had to be demolished. The exterior frame showed large X-shaped shear cracks in the columns of the second to the fifth story. These cracks coincided in height with the location of the window panes and suggest that the attachment of the cladding gave rise to the undesirable short column behavior.

Fig. 6.5 belongs to a six story reinforced concrete building in Santa Monica and shows the observed structural damage to the shear wall system. The lateral load resisting system of this building is a dual frame-wall system in both directions. Longitudinally the lateral load system is made up of a six bay moment resisting frame and corner shear walls. In the transverse direction a two bay frame is coupled with a shear wall that extended over a length approximately equivalent to two bays. Above each frame window a non structural concrete window shade had been placed. Heavy spandrel beams connect the columns. The majority of the damage occurred in the shear walls and frame columns. Cracks in the columns ran from the lower portion of the story windows at a 45 degree angle indicating short column behavior. At the edge of the column the crack extended up vertically. Damage to the shear walls was concentrated between the second and third stories. Crushing of the concrete at the edge of the walls was prominent, exposing terminated longitudinal reinforcing bars. A horizontal crack projected from the point of concrete crushing along part of the wall width.

Fig. 6.6 shows portion of a fifteen story apartment building in Santa Monica that was badly damaged. The longitudinal direction consisted of an eight bay non-ductile reinforced concrete frame. A coupled wall system provided lateral resistance in the transverse direction. Significant structural damage was observed in the lower half of the building in both directions.

The coupled wall system experienced significant shear damage in the coupling beams and no damage in the walls. The nonductile frame suffered damage in the columns, but not the beams.

Other interesting cases of earthquake damage include a 13 story commercial building in Sherman Oaks at a distance of 10 km from the epicenter. The building had a shear wall system in the N-S and a moment resisting frame in the E-W direction. This building experienced a peak horizontal acceleration of 0.46g at the base and 0.90g at the roof. Since the predominant horizontal acceleration along Ventura Blvd. in Sherman Oaks appeared to be in the N-S direction structural damage occurred exclusively in the shear wall system with several shear and flexural cracks extending halfway up the wall. During a strong aftershock three days after the main shock additional cracks appeared in the wall and the building had to be temporarily evacuated. This building experienced a peak horizontal acceleration of 0.26g at the base and 0.32g at the roof during the 1971 San Fernando earthquake with no reported damage [2].

Finally, the collapse of the two upper floors of the Bullocks store in the Fashion Island Mall of Northridge provided an interesting case of apparent punching shear failure of the waffle slab floor system of the roof and floor under large horizontal and vertical accelerations.

Parking Structures

Parking structures represent the category of modern engineered structures that appear to have suffered the largest incidence of partial or total collapse cases. These cases occurred, both, in the immediate vicinity of the epicenter with several spectacular failures in the Northridge Fashion Island Mall and on the campus of the California State University at Northridge (CSUN), but also at a distance from the epicenter, as the parking structures in the Glendale Civic Center (2), at the Kaiser Permanente in Culver City (2) and in Santa Monica demonstrate. Typical forms of the observed structural damage in parking structures are shown in Figs. 6.7-6.12.

Most cases of partial or complete collapse involve modern precast parking structures which either lack a lateral load resisting system in one direction or, otherwise, have a very flexible lateral load resisting system in one or both directions. Several such structures virtually “imploded” in the Northridge earthquake with Fig. 6.7 providing a spectacular example of the parking structure in the CSUN campus. Possible causes of such total collapse might be the unseating of the precast girders due to large lateral movement at the short corbel seats or the shear-compression failure of the columns. In all cases the prestressing tendons in the floor slab provided a catenary action that caused the spectacular “implosion” of part or of the entire structure. Other areas of weakness appear to be the connections of precast girders to the corbel seats at the columns. These connections commonly involve the welding of a plate at the bottom of the girder to an angle at the free corner of the corbel. Weld failures were observed in the post-earthquake survey of damage, as was the “chipping-of” of the corner of the corbel that reduced the seating area of the precast girder (Fig. 6.9). The latter cause could have precipitated the unseating of the precast girder particularly under the high vertical accelerations (e.g. 0.59g in the girder of a parking garage in downtown Los Angeles at a distance of 32 km from the epicenter).

Another area of weakness in modern precast parking structures is the flexibility of the thin cast-in-place topping slab that forms the horizontal floor and roof diaphragms. Significant compression crushing was evident in the roof diaphragm of City Hall Parking Structure, where the addition of another parking floor with insufficient lateral load resistance appears to be the cause of the partial roof collapse (Fig. 6.11). The falling debris from the supporting beam and a planter punched through two floors of the three story parking structure.

In older cast-in-place concrete parking structures a short column behavior was observed in the columns of perimeter frames due to partial masonry infills, as shown in Fig. 6.8. Shear-compression failure is evident in the interior column of the parking structure in Fig. 6.10. The large spacing of the transverse reinforcement could not prevent the bucking of longitudinal steel under the combined action of gravity with large lateral displacements.

Finally, the shear cracking in the columns of the parking structure in Fig. 6.12 can be attributed to inadequate detailing of the perimeter frame. Since shear walls were provided in both directions of the parking structure, it is not clear whether the participation of the frame in the lateral load resistance of the structure was intentional or not. Even so, the observed significant out-of-plane movement of the walls perpendicular to the direction of shear cracking offered further proof of the flexibility of lateral load systems in many parking structures.

It is apparent from the earthquake damage survey of parking structures that was completed to date, that revisions of the earthquake resistant design of this type of structure are necessary in order to account for the combination of high horizontal and vertical accelerations that were recorded in the vicinity of the epicenter.

Unreinforced Masonry (URM) Buildings

Unreinforced masonry structures have long been identified as buildings prone to severe structural and nonstructural damage in moderate and strong ground motions. Extensive damage to this type of structure was also evident in this preliminary post-earthquake survey. What is of particular interest in connection with the Northridge earthquake is, that several buildings in the area of strong shaking were retrofit in accordance with the earthquake risk mitigation program for URM buildings that was adopted by the City of Los Angeles in 1981. While it was very difficult for this reconnaissance team to identify in this short period which buildings were retrofit and which were not, some instances of damage to retrofit URM buildings were observed.

Fig. 6.13 shows a two story URM building in Culver City that was retrofit by tying the wall to the roof diaphragm with steel tendons. Since no such tie-back was provided at the floor level, the three wythe wall bowed out-of-plane and failed in flexure at the first floor level at mid-height of the wall.

In addition to the out-of-plane masonry wall behavior the other prevalent failure mode for URM buildings is the in-plane shear failure that is evident in the large X-shaped of the masonry pier in Fig. 6.14.

The combination of the in-plane and out-of-plane behavior was evident in the structures in Figs. 6.15 and 6.16. Fig. 6.15 shows the front of a four story apartment building located in Santa Monica. Brick bearing wall thickness was three wythes throughout. The roof was wood construction supported by a four foot wooden truss. The reconnaissance team could not identify the presence of a plywood diaphragm. Significant damage occurred at the top two stories of the structure. Out of plane failure of the walls was evident. In the portion of the front wall that was intact, in plane shear cracks existed at the height of the windows. Prominent shear cracking occurred in the masonry piers that were supporting the balconies. There was little evidence of damage to the side walls. Fig. 6.16 shows the typical failure of a two story nonretrofitted masonry building: significant damage of the front wall that has fallen out of plane with prominent in plane shear cracks clearly visible at the side wall.

Hospitals

Much as was the case in the 1971 San Fernando earthquake several hospitals in the area of strong shaking suffered structural and very serious non-structural damage. The Indian Hills Hospital suffered structural damage in the shear walls with concrete crushing and apparent lap splice failure at the construction joint at the fourth floor level (Fig. 6.17). The Veterans Hill Hospital in the immediate vicinity of the epicenter suffered very serious damage to contents and equipment. Water from the ruptured sprinkler system ran for several hours and flooded a good portion of the building according to preliminary reports. Fig. 6.18 shows the state of a typical ground floor office at the Veterans Hospital. Severe structural damage at the St. John's Hospital in Santa Monica caused its immediate evacuation.

Of particular interest to earthquake engineers is the fact that the Olive View Hospital which was rebuilt after its collapse following the 1971 San Fernando earthquake recorded a peak horizontal acceleration of 2.3g in the mechanical penthouse at the roof level, but did not suffer serious damage.

Residential Structures

Most residential structures in the vicinity of the epicenter were one story wood houses and two to three story wood apartment buildings. One story houses seem at this stage to have suffered little structural damage. Several chimneys broke at the roof line and fell, while quite a few buildings displaced horizontally from the foundation due to inadequate anchorage.

By contrast, two to three story apartment buildings in the vicinity of the epicenter suffered extensive structural damage and several first floor partial or total collapses were responsible for the majority of deaths in the Northridge earthquake. Most apartment buildings in the area were poorly engineered wood frame buildings covered with stucco walls only and lacked a lateral load resisting system due to the absence of plywood shear walls. Most of these buildings feature carports in the ground floor with the apartments located in the floors above. This led to a large incidence of soft first story collapse mechanisms (Figs. 6.19 and 6.20). In one instance part of the

ground floor was also filled with apartments and gave rise to the highest single incidence of deaths (Figs. 6.19 and 6.20).

Base Isolated Structures

Three seismically-isolated structures in the Los Angeles area were subjected to strong ground shaking during the Northridge earthquake. Two of these are supported on elastomeric isolators - the University of Southern California Teaching Hospital(USC) and the Los Angeles County Fire Command and Control Facility (FCCF) -while the third is supported on a helical steel spring and viscous dashpot system (GERB). Preliminary accelerograms have been released by the California Strong Motion Instrumentation Program (SMIP) from the USC and FCCF buildings. The USGS has recently recovered accelerograms from the GERB structure, but these were not available at the time of this report.

The USC hospital is an 8-story braced steel frame supported on 68 lead-rubber isolators and 81 elastomeric isolators. It is located east of downtown Los Angeles, approximately 36 km from the earthquake epicenter. The strongest motions recorded at the site were in the north-south direction. The peak free-field acceleration was 0.49 g, and the peak foundation acceleration was 0.37 g. The peak structure accelerations were 0.13 g and 0.21 g at the base and roof, respectively, implying amplification ratios of 0.32 and 0.57 relative to the input motion at the foundation level. These ratios are in the expected range for a seismically-isolated structure under this level of ground acceleration. For comparison, the amplification factors in the east-west direction were 1.0 and 1.2 at the base and roof, respectively; the peak foundation acceleration in this direction was only 0.16 g. The hospital remained completely functional during and after the earthquake, and there were no reports of damage to equipment inside the building. This event has been the most significant test to date of a full-size seismically-isolated building and provides a valuable data set for further study of this type of structures.

The FCCF is a 2-story braced steel frame supported on 32 high-damping rubber isolators and serves as the headquarters from which fire equipment is dispatched throughout Los Angeles county. It is located east of downtown, approximately 39 km from the epicenter of the Northridge earthquake. The recorded response of the FCCF in this event was unusual for a base-isolated structure because several high-frequency spikes were apparent in the east-west acceleration records. Although the peak foundation accelerations in this direction were between 0.19 and 0.22 g, the first floor accelerations were between 0.21 and 0.35 g, and the roof accelerations were between 0.24 and 0.32 g. The corresponding amplification ratios are therefore substantially greater than 1.0. In the north-south direction the building performed as expected, with amplification ratios of approximately 0.4 and 0.5 at the 1st floor and roof, respectively. The peak foundation acceleration in this direction was approximately 0.18 g. The FCCF remained fully functional during and after the earthquake. An inspection of the site after the earthquake revealed that architectural details at an east-facing tile entryway near the north wall of the building may have compromised the isolation gap in the east-west direction. The tiles are not part of the isolated portion of the building and were designed as sacrificial elements that would be dislodged by the steel grillwork that overhangs the isolation gap from the main structure. The tiles had last

been replaced after being damaged in the 1992 Landers earthquake. However, it appears that the newly installed tiles provided more lateral restraint than expected to the overhanging grill, imparting an impulsive force at the first floor of the structure as the grill pounded on the tiles. This mechanism is consistent with the observation that the high-frequency acceleration spikes are larger at the north side of the building near the entryway than at the south wall, and the fact that the spikes indicate amplified accelerations only toward the west. No high-frequency response is seen in the north-south acceleration time histories.

The last isolated structures were two identical 3-story braced steel frame residences in Santa Monica, each supported at its corners by GERB helical springs and viscous dashpots. Additional springs are distributed around the building perimeter. The site is approximately 24 km from the epicenter, and although the accelerograms recorded here have not yet been processed by USGS, SMIP records from the Santa Monica City Hall Grounds nearby show peak horizontal and vertical accelerations of 0.93 g and 0.25 g, respectively. It appears from a survey of these buildings that the isolation system is more effective vertically than horizontally since several details limit horizontal movement. For example, slight damage was observed at locations where steel girders from the isolated portion of the structure framed into a concrete footing and a masonry block wall that was attached to the ground. However, a series of square glass blocks distributed around the perimeter of the structures indicated that the building experienced a vertical displacement of 3/4" to 1".

Conclusions

These preliminary observations of the post-earthquake damage from the 1994 Northridge earthquake indicate that a wide range of structural types suffered significant structural damage. While it is premature to draw any conclusions, it is safe to say that the wealth of recorded data from the main shock and several strong aftershocks will help engineers further their understanding of the seismic performance of old and new structures. Of particular interest to the designers is the fact that high horizontal accelerations were combined with significant vertical accelerations in many areas of strong ground shaking. The relatively poor performance of several parking structures requires special attention for improving the design of new and devising effective retrofit measures for existing structures. Finally, a set of very interesting records were obtained for base-isolated structures. The records from the USC hospital are particularly encouraging in that they represent the most severe test of an isolated building structure to date. The results from the FCCF and GERB structures are also worthy of further study, and illustrate the importance of careful maintenance of the seismic gap around isolated structures.

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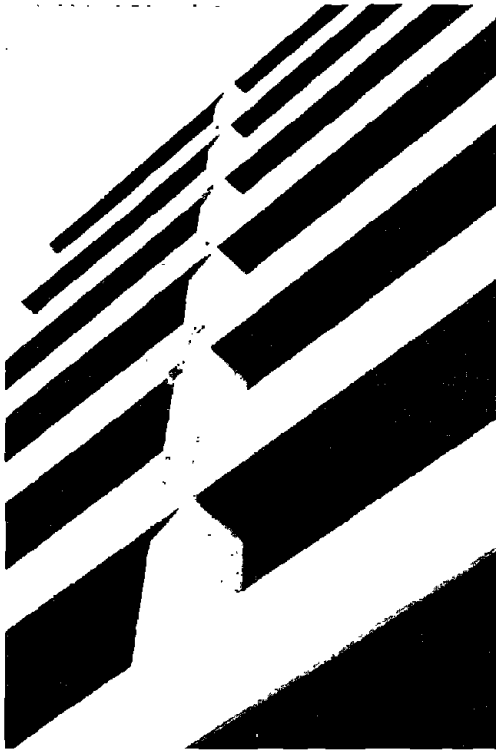


Fig. 6.1 Cast-In-Place Parking Garage With Joint Shear Failure

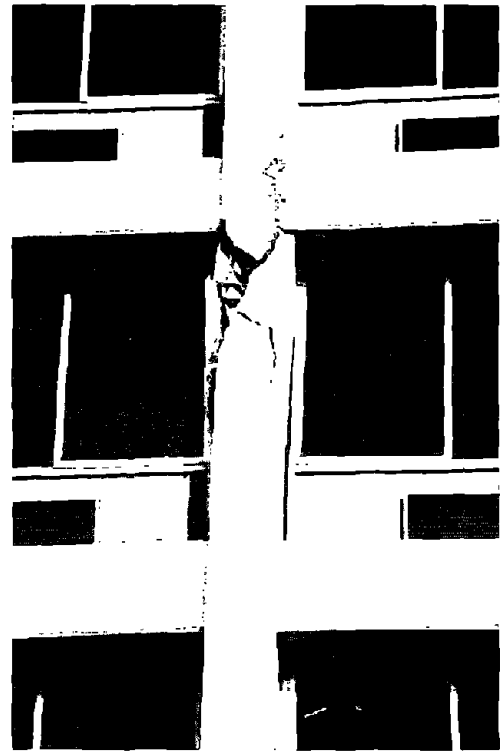


Fig. 6.2 Shear-Bond Failure in Columns of 7-Story Moment Resisting Frame



Fig. 6.3 Nonductile Reinforce Concrete (RC) Frame with Soft Story Mechanism



Fig. 6.4 Short Column Behavior in 6-story RC Moment Resisting Frame



Fig. 6.5 Damage to Shear Wall of Dual Frame-Wall System

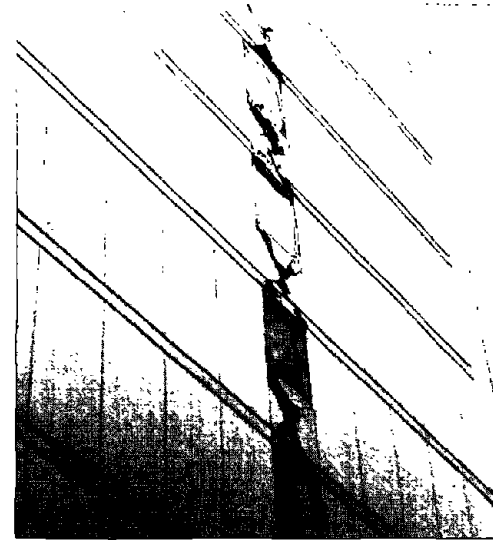


Fig. 6.6 Shear Failure of Coupling Beams in 15-story Coupled Shear Wall

6-9



Fig. 6.7 Characteristic "Implosion" of Precast Parking Structure



Fig. 6.8 Short Column Effect due to Partial Infill

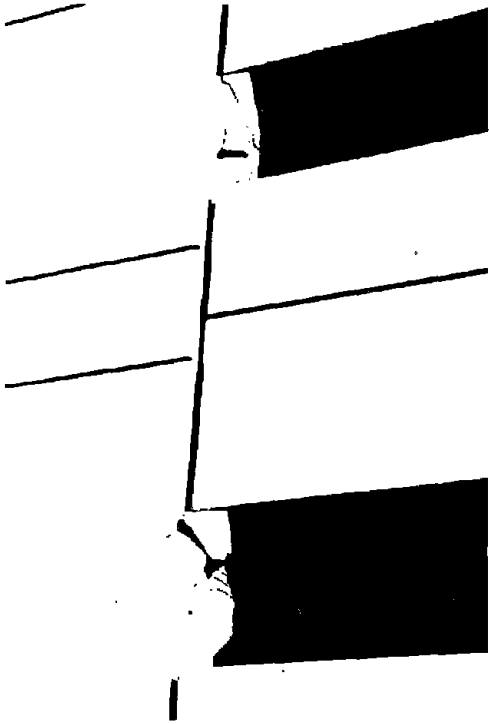


Fig. 6.9 Typical Corbel Damage in Precast Frame of Parking Garage



Fig. 6.10 Shear-Compression Failure of Cast-in-Place Interior Column



Fig. 6.11 Partial Roof Collapse with Subsequent Punching of Floors Below

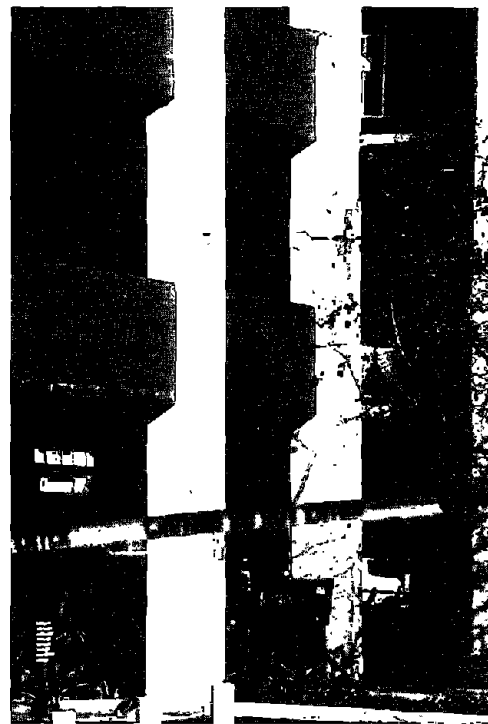


Fig. 6.12 Shear Cracking in Columns

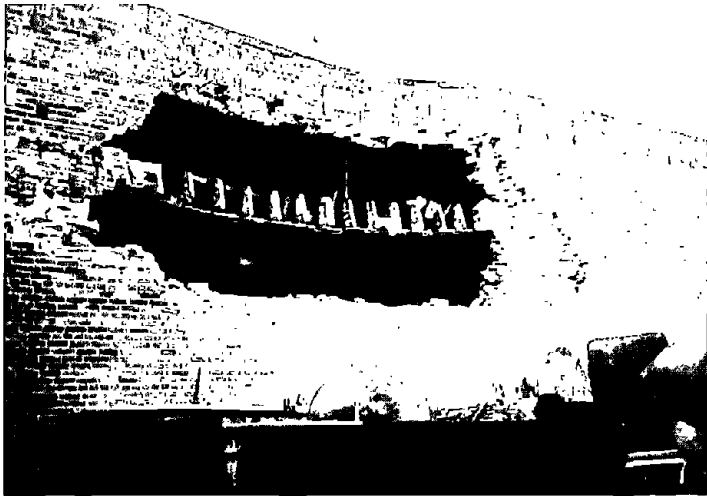


Fig. 6.13 Out of Plane Bending Failure of Retrofit URM Building



Fig. 6.14 Shear Cracking in Non-Bearing Masonry Wall

8-11



Fig. 6.15 Typical Shear and Out-of-Plane URM Failure

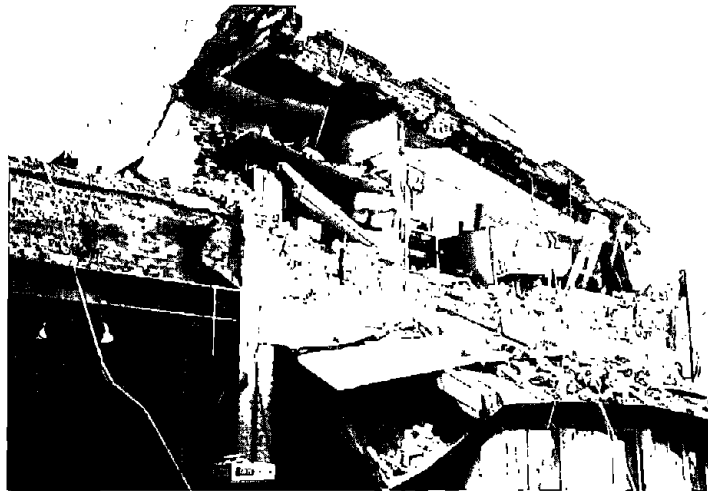


Fig. 6.16 Typical Shear and Out-of-Plane URM Failure



Fig. 6.17 Shear Wall Damage at Indian Hills Hospital



Fig. 6.18 Toppled Bookcases, Ground Floor of Veterans Hospital



Fig. 6.19 Soft Story Collapse of Wood Apartment Building



Fig. 6.20 Soft Story Collapse of Wood Apartment Building

6-12

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