THE NORTHRIDGE, CALIFORNIA EARTHQUAKE OF 17 JANUARY 1994

A FIELD REPORT BY EEFIT

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1. INTRODUCTION

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1.1 The earthquake

The Northridge earthquake occurred at 4.31am Pacific Standard Time on Monday 17 January 1994. Its magnitude was 6.8 on the Richter scale and its focus 18.5km below the Northridge area of Los Angeles, a suburb built in the San Fernando valley 30km to the north of downtown Los Angeles. The earthquake resulted in 57 deaths and over 9,000 confirmed injuries. Approximately 25,000 homes were rendered uninhabitable and the financial loss is estimated as $15-30 billion.

The earthquake was of particular interest to UK earthquake engineers for two reasons. Firstly, it was of the type and size which represents the maximum credible earthquake in the UK, and it occurred in a built-up area with construction types ranging from 1920s unreinforced masonry to state-of-the-art seismically designed structures. Secondly, the earthquake occurred within a few miles of the similarly sized San Fernando earthquake of 1971, a turning point for seismic design in the USA because it demonstrated the vulnerability of buildings, bridges, lifeline systems and dams to a moderate earthquake. As a result, major revisions were incorporated into the Uniform Building Code (UBC), introducing the concept of ductility and provisions for connection between the floor diaphragms and the resisting elements of the structure. The 1971 earthquake also led to the introduction of a major programme of seismic retrofitting in California.

1.2 The EEFIT mission

The Earthquake Engineering Field Investigation Team (EEFIT) sent a group of 14 engineers to Northridge. The team, led by Dr A. Blakeborough and Dr P.A. Merriman, comprised six UK engineers currently working in industry, four UK academics and four European academics. The EEFIT team left for Los Angeles on 27 January 1994 and spent seven days in the affected region.

The main purposes of the mission were to observe and report on damage caused by the earthquake, to develop collaboration between the team members and to build links with US earthquake engineers.

Prior to the main mission, from 19 to 22 January 1994, EEFIT member Dr M.S. Williams visited Los Angeles as part of a mission mounted by the Canadian Association of Earthquake Engineering. This enabled observations to be made of many structures that were demolished prior to the arrival of the main EEFIT mission. A briefing and list of contacts were provided to the mission leaders before their departure, as aids to effective planning.

1.3 Organisation and method of working

On arrival, the team appointed group leaders who would have responsibility for a particular section of the final report, e.g. bridges, engineered structures and dams. Members did not spend all of their time working with one group, but were encouraged to put most of their efforts into one area. The group leader was responsible for ensuring that an area was adequately covered.

After initial orientation, the team worked by dividing into sub-groups to visit specific sites of interest. Each evening, all members of the team met for a debriefing session. The group leaders gave progress reports and passed on information of use to others. The meeting then agreed the next day's activities. This proved an excellent method of organisation and is recommended for future missions of this type.

1.4 Principal features of the earthquake

This section briefly introduces the main topics of interest; these are covered in depth in the subsequent chapters.

1.4.1 Seismology (see Chapter 2)

The earthquake epicentre was in the San Fernando Valley, which lies between the Santa Monica mountains in the south and the Santa Suzanna and San Gabriel mountains to the north. The region is very seismically active, suffering ten major ($M > 7.0$) earthquakes and one great ($M > 8.0$) earthquake since 1769. The major tectonic feature is the San Andreas Fault, which runs some 60km to the east of Los Angeles. In the Los Angeles region there are numerous E-W thrust faults to the north and SE-NW strike slip faults to the south. The Northridge earthquake occurred on a previously unknown E-W reverse thrust fault under the San Fernando Valley. Peak ground accelerations greater than 1g horizontally and 0.5g vertically were measured in the epicentral region.

1.4.2 Geotechnical aspects (Chapter 3)

There were several major soil failures caused by the Northridge earthquake and the distribution of damage indicated areas of amplified ground motion caused by soil effects. In the region of severe damage, the affected areas appear as a chequered pattern on a map indicating strong amplification in some areas. Large relative ground movements occurred in the epicentral region causing fractures of gas, water, oil and sewerage pipes in a few locations. Several earthquake-induced failures emphasised the effects of soil structure interaction on structural response. Sand boils were observed at several sites.

1.4.3 Dams (Chapter 4)

Generally, the dams in the area performed well, with minor cracking occurring at a few embankment dams, but there were two cases of more notable damage. Pacoima Dam, a concrete arch structure, experienced a major crack in the thrust block to its left abutment, and at the Los Angeles Reservoir the main earth dam suffered cracking in the asphalt lining on its upstream slope. There were two instances of the localised failure of the bearings of access bridges to the intake tower; one at the Los Angeles Reservoir and one at Castaic Lake. Another less serious case of damage to a tower and bridge occurred at the Upper Hollywood reservoir.

1.4.4 Bridges (Chapter 5)

The highway system is essential to the functioning of Los Angeles. As a result of the earthquake, ten bridge structures collapsed, or were damaged badly enough to be closed, causing severe traffic congestion. The highway system is so important that reconstruction started immediately. Recently designed bridges, and those retrofitted by encasing the columns with steel jackets performed well. Damage to bridge structures can be classified by two distinct failure mechanisms. A few bridge decks simply fell off their bearings in the large motions of the earthquake. However, by far the most common mechanism was the shear failure of short columns.

1.4.5 Buildings (Chapter 6)

Damage to buildings was widespread. In the San Fernando valley alone 184 buildings were reported destroyed and 5,564 damaged. Most deaths occurred in building collapses. This number of collapses represents less than 0.1% of the building stock. While the seismic provisions clearly are having an effect in reducing loss of life and instances of damage, the economic cost remains immense.

Unreinforced masonry structures fared badly. However, retrofitting, by inserting steel bars through the floor spaces tied to steel plates on the outside walls, was effective in many instances. Many mid-rise reinforced concrete buildings of all structural types were severely damaged and a few collapsed. Most of these were older buildings and the mechanisms are nowadays recognised as typical. Of the modern structures, parking facilities, of which there are many, suffered particularly badly. The most probable reason is the poor ductility of the welded connections to the precast beams which are used extensively in this type of construction. It is expected that the failure of this modern type of construction will precipitate a thorough revision of the ductility requirements in buildings. Cracking of welded beam-column joints in steel frames was also widespread.

1.4.6 Industrial Facilities (Chapter 7)

The performance of the main structures at industrial sites was generally good. Most failures occurred at the interface between structural and mechanical components. Equipment generally proved to be robust and readily able to be reinstated.

1.4.7 Lifelines (Chapter 8)

The earthquake caused widespread loss of electrical supplies. There was a complete blackout of the Los Angeles region for a short period, and hundreds of thousands of homes were without supplies for more than 24 hours. There was also severe disruption of water and gas networks, with over 1,000 reported pipe breaks in each case, but telecommunications systems suffered comparatively few problems.

1.5 Acknowledgements

EEFIT is grateful to the following for their assistance with this mission:

- The Engineering and Physical Sciences Research Council, who provided funding to the UK academics on the EEFIT team under grant GR/J98400.

- The members of the Canadian Association for Earthquake Engineering who provided funding and technical assistance to Dr Williams.

- The staff of Caltrans (the State of California Transportation Authority), who provided access to and details of damaged bridges in the immediate aftermath of the earthquake.

- The US Earthquake Engineering Research Institute, who provided briefing and orientation for the many international teams visiting the region.

- Mr P. Lahr and Mr C. Bargus of the Los Angeles Department of Water and Power.

- Mr F. Meccia, Mr J. de Santiago, Mr J. Patrick and Mr D. Kessler of the State of California, Department of Water Resources.

- Mr R. Kroll of the Los Angeles Department of Public Works.
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- Professor F. Seible of the University of California at San Diego.
- Professor J. Hall of the California Institute of Technology.

2. SEISMOLOGY AND STRONG MOTIONS

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2.1 Introduction

On Monday 17 January 1994 at 4:31am pacific standard time, the San Fernando Valley in Southern California was shaken by an earthquake measuring 6.8 on the Richter scale. The epicentre was just southwest of Northridge (34.21°N, 118.54°W), which is approximately 30km north of Los Angeles. The focal depth of the earthquake was 18.5km, estimated by Caltech. Thousands of aftershocks have been observed following the main event, the two largest were a magnitude 5.3 event at 3:30pm on the 17 January and a magnitude 5.1 event at 4:00 on the 29 January.

2.2 Seismology and Geology

The Southern California region has experienced 10 major ($M \geq 7$) and one great earthquake ($M \geq 8$) since 1769. The largest was the magnitude 8.1 Tejon Pass event on the 9 January 1857, which caused a 350km surface rupture along the San Andreas fault (Swiss Re, 1991). The largest earthquake known to have occurred within the vicinity of Los Angeles was the magnitude 7.0 event on the 9 February 1890 as shown in Figure 2.1. Furthermore this figure shows that there have now been three earthquakes greater than magnitude 6 this century around the Los Angeles region. The 1933 Long Beach earthquake (magnitude 6.3), whose epicentre was south of Los Angeles, ranks as one of the major disasters in Southern California's short history. The majority of damage was suffered by structures which are now regarded as substandard or to structures which were located on filled or saturated ground. The 1971 San Fernando earthquake (magnitude 6.4) caused at least $500 million of damage and 65 deaths. The 1994 Northridge earthquake is the most recent of the magnitude greater than 6 events this century.

Numerous active or potentially active faults exist in Southern California. The major tectonic structure in California is the San Andreas fault, a strike-slip fault, which forms part of the tectonic boundary between the Pacific and American Plates. In the region of Los Angeles the San Andreas fault runs SE-NW, and at its closest is some 60km east of central Los Angeles. Figure 2.1 shows the approximate locations of the secondary faults. About 100km north west of Los Angeles the San Andreas fault turns a more northerly direction. This abrupt change in direction is the main reason why the faults north of Los Angeles are E-W trending thrust faults whilst those south of Los Angeles are SE-NW trending slip faults similar to the San Andreas fault. Table 2.1 gives the details of the slip rates and maximum credible earthquakes associated with the faults in the Los Angeles region after Slemmons (1979), Mark (1977) and Greensfelder (1974). The values given in this table are typical of those used by designers to calculate response spectra for design purposes.

Figure 2.2 shows a schematic outline of the geological setting of the Los Angeles region. The Northridge earthquake occurred in the San Fernando Valley, which is a deep alluvial basin that lies between the Santa Monica Mountains to the south and the Santa Susana and San Gabriel Mountains to the north. The Santa Monica and Santa Susana Mountains are formed mainly from Tertiary sedimentary and volcanic deposits whilst the San Gabriel Mountains are formed from older igneous and metamorphic formations. South of the Santa Monica Mountains is the Los

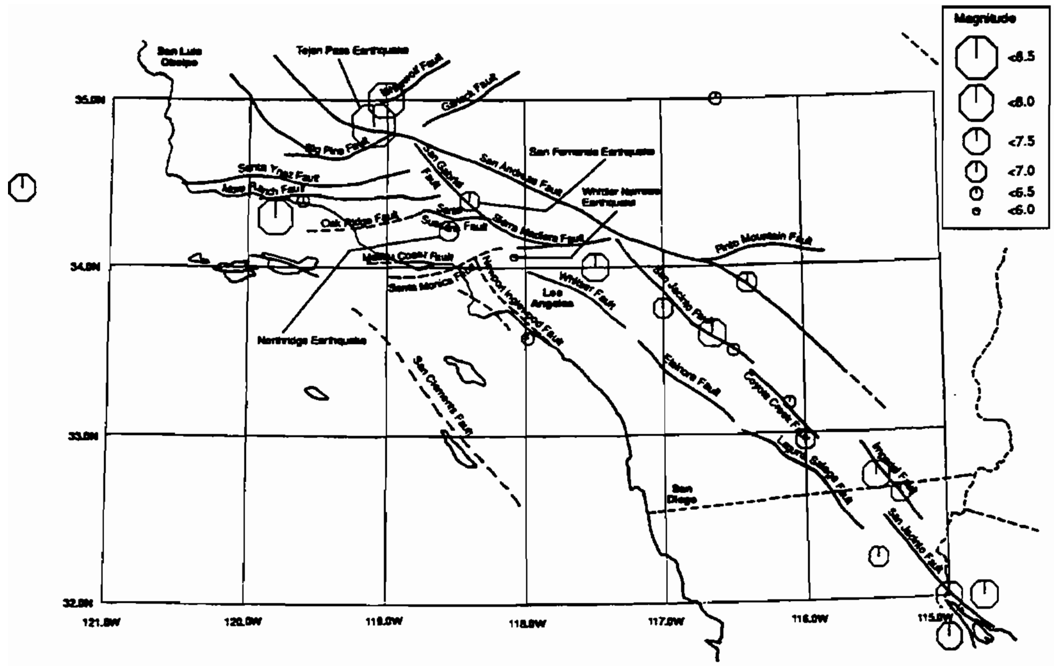

Figure 2.1 : Seismicity of Southern California (1769-1994)

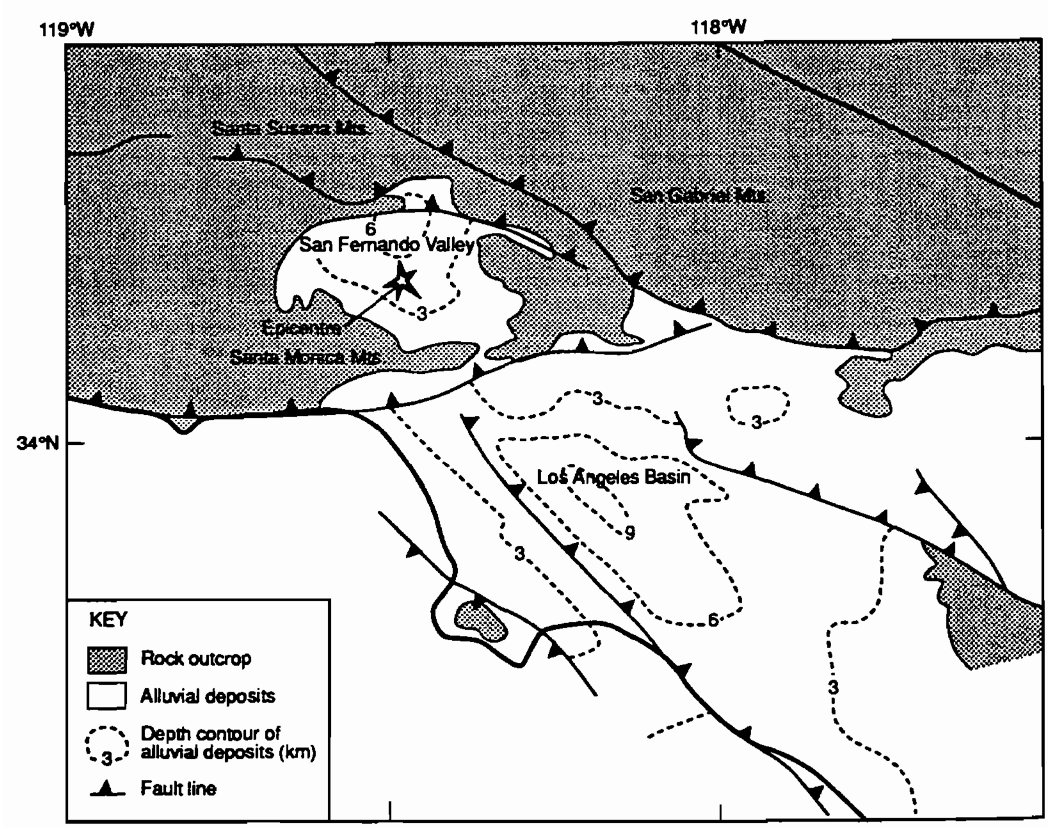

Figure 2.2 : Geological Setting of the Los Angeles Region

Angeles basin which is composed of a mixture of recent alluvial, Pleistocene marine and dune sand deposits. Furthermore this figure also shows the depth contours (in kilometres) of the alluvial deposits in the San Fernando Valley and the Los Angeles basin (after Yomogida et al, 1993). The San Fernando Valley is seen to have a nearly vertical interface about 6km deep on its northern edge whilst the Los Angeles basin has a nearly elliptical shape about 9km deep.

Table 2.1 : Faults in the Los Angeles Region

Fault	Maximum Credible Earthquake	Slip Rate (mm/year)
Cucamonga	6.5	3
Elsinore	7.5	4
Malibu Coast	7.0	0.7
Newport-Inglewood	7.0	1
Northridge Hills	6.5	-
Norwalk	6.4	-
Oak Ridge	7.5	3.5
Palos Verdes	7.0	0.3
San Andreas	8.3	25
San Fernando	6.5	1
San Gabriel	7.5	0.4
Santa Monica-Hollywood	6.8	-
Santa Susana	6.5	4
Sierra Madre	7.5	1
Whittier	7.0	4

Following the earthquake a network of over 75 portable instruments were installed to measure the aftershock pattern which is typical of significant earthquakes in California. Figure 2.3 shows the distribution of the aftershocks for the first ten days after the earthquake and a typical cross-section showing the aftershocks on a south dipping plane. This aftershock pattern has questioned the previously held theories that suggested all the faults north of Los Angeles were north dipping. A possible interpretation suggested at Caltech is that the Oak Ridge fault whose surface expression is mapped to the NW of the San Fernando Valley is the causative fault. The theory suggests that the Oak Ridge fault extends east but is overridden by the Santa Susana fault, hence it does not have a surface expression in the vicinity of the San Fernando valley. This may account for the lack of a clear surface fracture due to this earthquake. A number of similar theories have been postulated for the geological mechanism responsible for this earthquake. It is clear that the deep geological structure around Los Angeles needs further re-evaluation since a number of earthquakes have revealed new faults or caused a re-definition of the existing fault pattern.

2.3 Strong Ground Motion

The Northridge earthquake was measured by over 100 strong motion instruments which were installed by the California Division of Mines and Geology (CDMG), the United States Geological Survey (USGS) and the University of Southern California (USC). Peak horizontal ground accelerations (PHGA) in excess of 1g were measured in the epicentral region with peak vertical accelerations in excess of 0.5g. Figure 2.4 shows the PHGA contours proposed by Trifunac et al (1994) derived from the USC database, with reference to the preliminary reports of the CDMG and the USGS. Visual inspection of strong motion records indicate overall strong motion duration of between 6 and 10 seconds, with a second arrival at about 4 seconds. This corresponds with eye witness accounts which identified two distinct pulses of motion during the earthquake. The PHGA contour map corresponds well with the distribution of damage, concentrating in the San Fernando Valley, Santa Monica and Hollywood where the largest accelerations were observed. Table 2.2 shows the peak ground acceleration of the strong motion instruments which triggered within 20km of the epicentre.

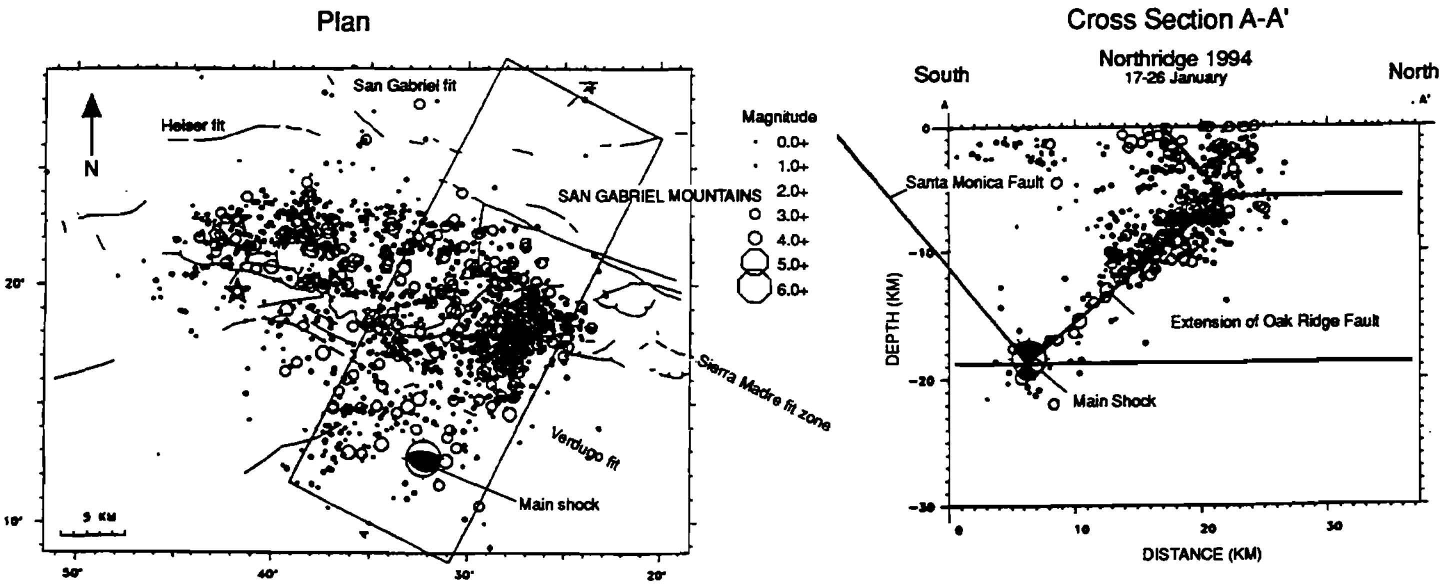

Figure 2.3 : Distribution of aftershocks (17-26 January)

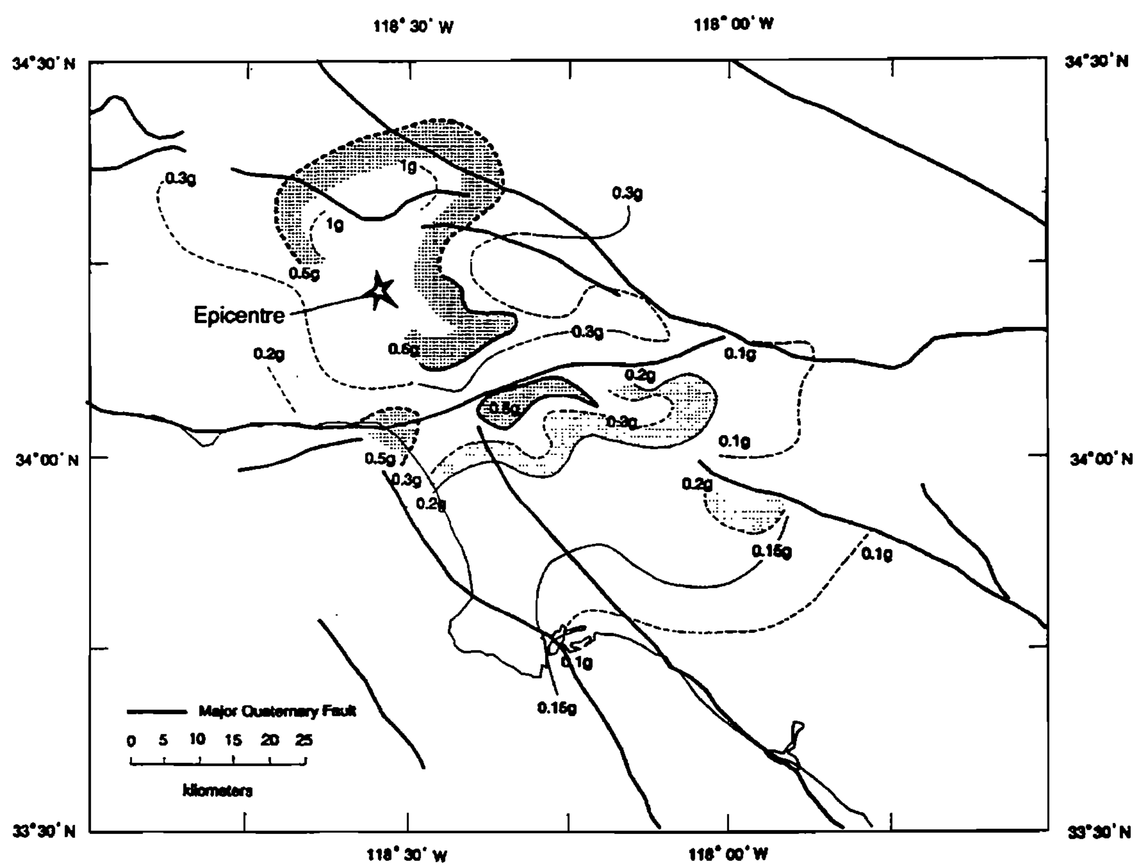

Figure 2.4 : PHGA contour map

Table 2.2 : Strong motion data for the Northridge Earthquake within 20 km of the epicentre

No	Owner	Epicentral Distance (km)	Maximum Acceleration (g)		
			Free Field	Base	Structure
1	USC	2.1	0.51H	-	-
2	USGS	5	-	-	0.48H,0.48V
3	USC	5.9	0.46H,0.63V	-	-
4	CDMG	6	-	0.47H,0.30V	0.59H
5	CDMG	7	1.82H,1.18V	-	-
6	USGS	7	0.94H,0.48V	-	-
7	USGS	8	-	-	0.76H,0.50V
8	USGS	8	-	-	0.41H,0.37V
9	USGS	8	-	-	0.61H,0.43V
10	CDMG	9	0.35H,0.59V	-	-
11	CDMG	10	-	0.46H,0.18V	0.90H
12	USC	11.5	0.51H,0.34V	-	-
13	USGS	12	-	0.62H,0.40V	-
14	USGS	12	0.98H,0.56V	-	-
15	USGS	12	-	-	0.84H,0.51V
16	USC	12.2	0.34H,0.34V	-	-
17	USC	13	0.51H,0.40V	-	-
18	USGS	14	0.43H,0.16V	-	-
19	USC	14.9	0.23H,0.17V	-	-
20	USGS	15	0.34H,0.21V	-	-
21	CDMG	15	0.91H,0.60V	0.82H,0.34V	2.31H
22	USC	15.7	0.71H,0.34V	-	-
23	USGS	16	0.34H,0.23V	-	0.76H,0.66V
24	USC	16.7	0.34H,0.23V	-	-
25	CDMG	17	0.44H,0.19V	-	-
26	CDMG	18	0.44H,0.20V	0.54H,0.43V	2.3H,1.7V
27	USGS	18	0.18H,0.14V	-	-
28	USC	18.5	0.17H,0.11V	-	-
29	CDMG	19	0.63H,0.62V	-	-
30	CDMG	19	-	0.33H,0.15V	0.66H
31	CDMG	19	-	0.29H,0.25V	0.77H
32	USGS	19	-	-	0.17H,0.24V
33	USGS	19	-	-	0.40H,0.39V
34	USGS	19	-	-	1.00H,0.51V
35	USGS	19	0.39H,0.17V	0.22H,0.09V	0.56H
36	USGS	20	-	-	0.32H,0.37V
37	USGS	20	-	-	0.32H,0.46V
38	CDMG	20	0.27H,0.15V	-	-

The largest measured PHGA values were for a site 7km south of the epicentre at Tarzana. This site showed peak accelerations of 1.82g horizontally and 1.18g vertically. However, this site is thought to be exceptional. The instrument is located on a small hill, so topographical effects may account partially for the large motions measured. However, it should be noted that the slopes leading up to the strong motion instrument are a maximum of 30° and the instrument is set well back from the main ridge line. Therefore according to the French Code the topographical effect would be minimal. The Tarzana site also produced very high accelerations during the 1987 Whittier earthquake which further suggests Tarzana is an exceptional site. Located at the site were four caravans which were erected on slender wooden frames. One caravan was dislodged during the earthquake, however it was supported only on simple wooden posts, the other three

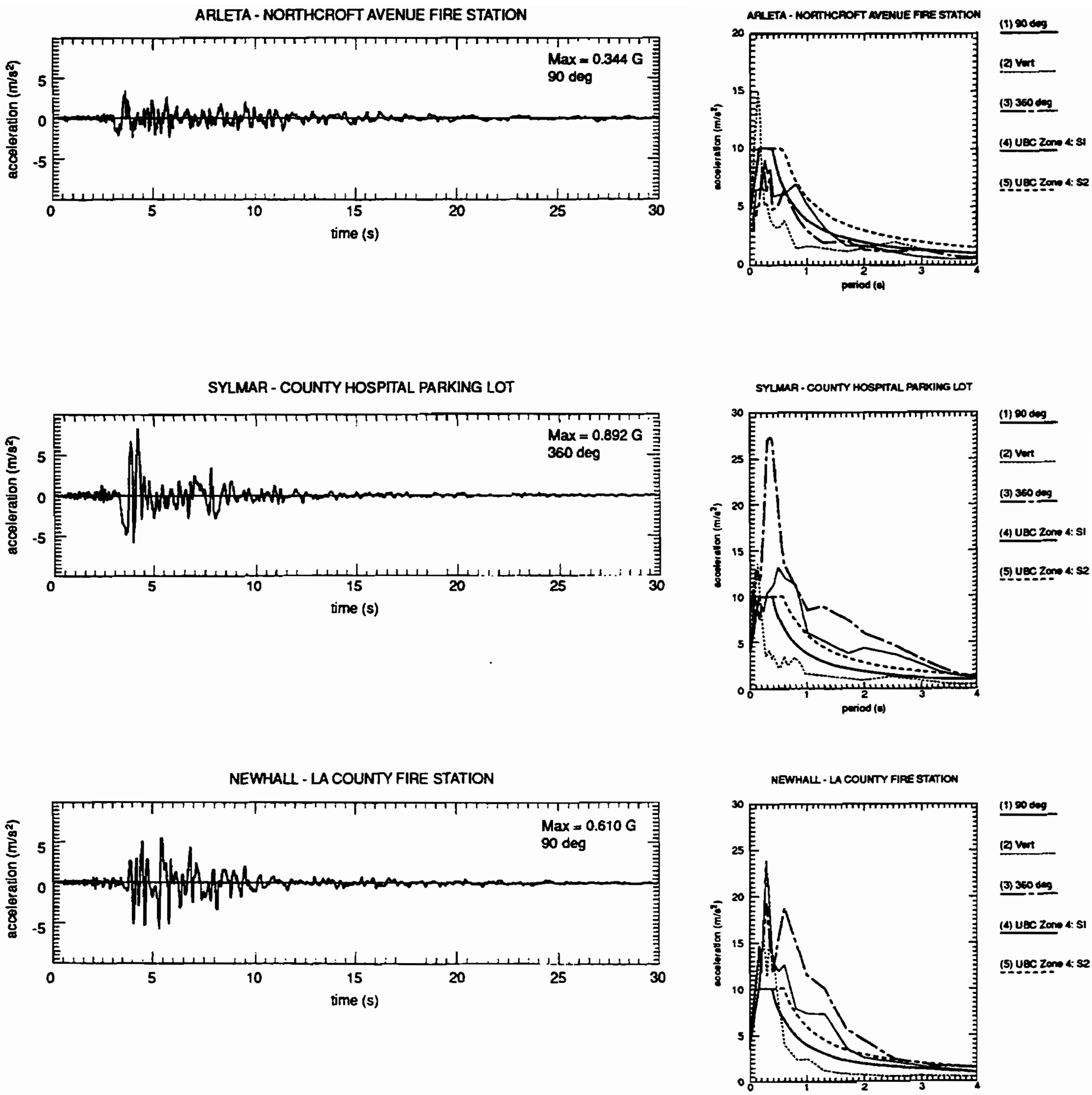

Figure 2.5 : Typical time histories and response spectra

showed no evidence of damage or significant movement which seems inconsistent with the accelerations measured. Following the earthquake, temporary instruments to measure the aftershocks were installed (by CDMG) adjacent to the main instrument location and at the bottom of the hill to attempt to understand the strong motion characteristics of this site.

Figure 2.5 shows the time histories, horizontal and vertical, for strong ground motion stations 10 (Arletta, Nordoff Ave Fire Station), 21 (Sylmar Hospital Parking Lot) and 29 (Newhall, LA County Fire Station). These are the three closest stations to the epicentre which have had their records processed by the CDMG. The figure also shows the 5% damped response spectra which correspond to the time histories. The standard Uniform Building Code (1991) spectra for zone 4 soil types S1 (rock) and S2 (deep soil site) are shown as a comparison. The locations of these three sites are also shown on Figure 2.4. The response spectra show that the natural period of the ground motions varies significantly from site to site, for both horizontal and vertical directions. It is interesting to note that for all but Arletta the response spectra exceed significantly the UBC Zone 4 spectra. However, for this case the vertical response is greater than the horizontal whilst the vertical response is less at the other two sites.

The contour map of PHGA shows that the motions generally reduce with distance from the epicentre. Figure 2.6 shows the variation of peak horizontal ground acceleration with hypocentral distance and compares this data with the peak ground acceleration attenuation law proposed by Boore and Joyner (1991) for deep soil sites in Eastern North America. (A deep soil site is defined as S2 in the Uniform Building Code). The $+1\sigma$ and -1σ attenuation laws ($\sigma = 0.76$) are also shown on the figure to help examine the scatter of data points. Though this relationship is specifically designed for Eastern North America it was developed using earthquake data from Western North America and hence appears appropriate for the deep alluvial basins around Los Angeles. The scatter of data appears to remain within the $\pm\sigma$ bounds except for a couple of sites.

2.4 Seismic Microzoning

The approximate distribution of red tagged buildings (i.e buildings that were determined to be unsafe following the earthquake) in the cites of Los Angeles and Santa Monica is shown in Figure 2.7 overlain on the geological map. It is interesting to note the clustering of damage both in the San Fernando Valley and in Los Angeles at some distance from the earthquake epicentre, such as Sherman Oaks, Hollywood, Santa Monica and Culver City.

The clustering of damage in the San Fernando Valley may be influenced by basin effects. However, other factors such as the buildings age, height and construction type may also influence the observed pattern of damage. It should be noted that the distribution of buildings (residential and office) is fairly constant across the whole valley. Furthermore the buildings are of similar construction type, residential buildings are timber framed whilst offices are mainly reinforced concrete. A detailed survey of building types would be needed to clarify these variations.

Yomogida et al (1993) have examined the basin effects in Los Angeles due to the Whittier Narrows earthquake and conclude that when the primary wave reaches the edge of the basins, surface waves with large amplitudes are created due to the impedance contrast between the sediment and the basement. This is a plausible theory for the damage in Sherman Oaks, which is close to the southern boundary of the San Fernando Valley. Furthermore Yomogida et al (1993) predicted a significant later arrival about 5 seconds after the primary S wave, especially in the south part of the Los Angeles basin. This was observed in a number of the time histories obtained for this earthquake.

In Los Angles the clustering may also be due to local soft soil conditions. For example site effects may have aided in the collapse of the I10 highway bridge over La Cienega Boulevard near Culver City. It is interesting to note that La Cienega means The Swamp in Spanish. There may have also been amplification of PHGA at Redondo Beach which suffered significant liquefaction. Despite these examples no significant soil effects were observed during this earthquake.

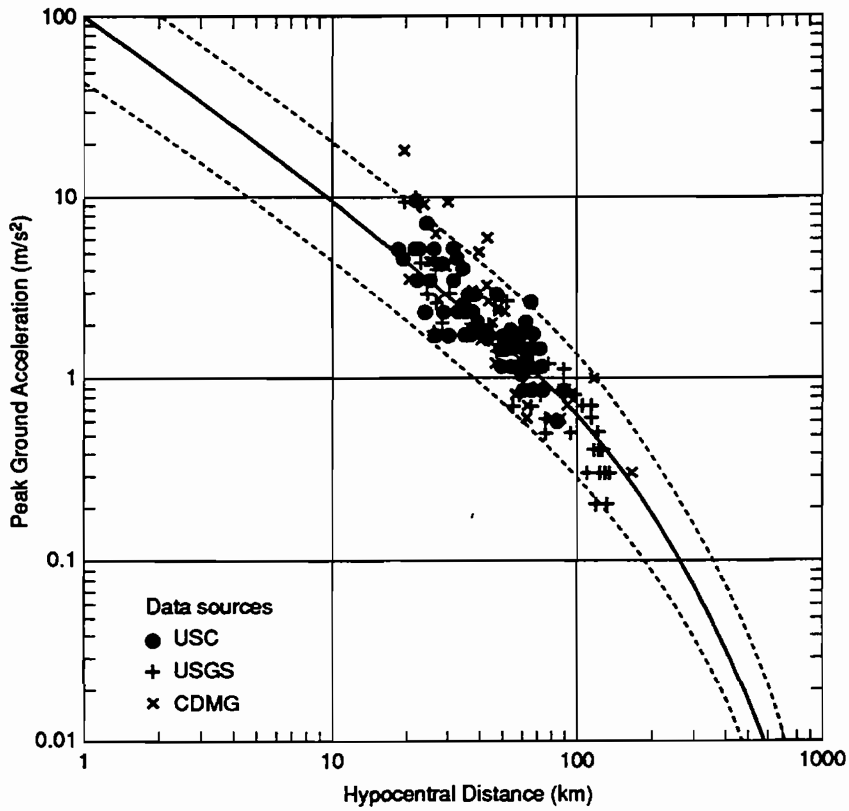

Figure 2.6 : Comparison of PHGA and Boore and Joyner (1991) attenuation relationship

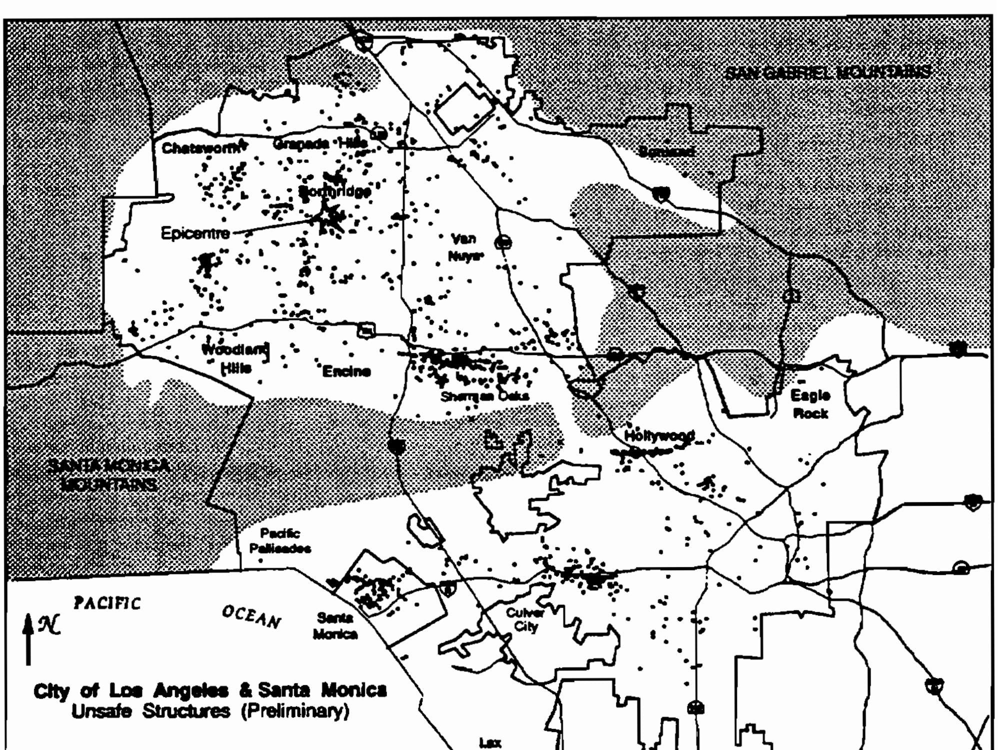

Figure 2.7 : Distribution of red-tagged buildings compared to geological map

Topographic effects were observed at Pacific Palisades where an amplification to the ground motion of about 160% occurred compared to surrounding areas. This caused the partial collapse of a 400m long by 35m high section of very steep ($\theta \geq 60°$) cliff and the destruction of a number of homes at the top of the cliff. If the requirements of the French Code are applied to this scenario a topographical coefficient (τ) of 1.4 is calculated which has to be applied a distance of 11.25m behind the ridge line, then reducing to 1.0 at 20m from the ridge line (τ is the factor by which motions are increased, compared with level ground). The measured ground motions and damage observed at Pacific Palisades suggests that such topographical effects can be significant.

2.5 Conclusions

The Northridge earthquake has re-focused the seismological, geological and engineering communities to the effects of large earthquakes in metropolitan areas such as Los Angeles. Several conclusions can be drawn from the effects of the earthquake.

(a) Ground motion exceeded UBC design motion for a 500 year return period in the San Fernando Valley and at Santa Monica - by a factor of up to 250% over an area of about 1100km^2. However, the shape of the UBC response spectrum appears to provide a reasonable bound to the recorded spectral shapes.

(b) Strong motion data from the earthquake has shown there was nothing anomalous about attenuation of ground motion or the horizontal/vertical acceleration ratio.

(c) Evidence from the earthquake suggests local amplification effects may have been significant. This will have important consequences for microzoning and will require further studies. Basin effects were probably significant, due to wave reflections in the alluvial material of the San Fernando Valley. There is evidence that topographical effects may have been important, and the approach in the French code (subsequently adopted by Eurocode 8) appears a good predictor.

(d) This earthquake has clearly shown the geological structure beneath northern Los Angeles is still not well understood, so designers must be careful in defining the level of the maximum credible earthquake. It may be prudent to assume that a magnitude 7.0 earthquake can occur anywhere in this region.

2.6 References

AFPS (1990) "Recommendations for the redaction of rules relative to the structures and buildings built in regions prone to earthquakes", French Association for Earthquake Engineering, Saint-Remy-les-Chevreuse, France.

Boore D.M. and Joyner W.B. (1991) "Estimation of ground motion at deep soil sites in Eastern North America", Bull. Seismo. Soc. of Am., Vol 81, No 6, pp 2167-2185.

CEN (1993) "Draft Eurocode 8 ENV-1998-5 : Structures in seismic regions. Part 5 Foundations, retaining structures and geotechnical aspects", European Committee for Standardization, Brussels.

Greensfelder R.W. (1974) "Maximum credible rock acceleration from earthquakes in California", CDMG Map sheet 23.

Mark R.K. (1977) "Application of linear statistical models of earthquake magnitude versus fault length in estimating maximum expectable earthquakes", Geology, Vol 5, pp464-466.

Slemmons D.B. (1979) "Evaluation of geomorphic features of active faults for engineering design and siting studies", AEG Short Course.

Trifunac M.D., Todorovska M.I. and Ivanović S.S. (1994) "A note on distribution of uncorrected peak ground accelerations during the Northridge, California, Earthquake of 17 January 1994", Soil Dynamics and Earthquake Engineering, (to be published).

Uniform Building Code (1991) "UBC, Chapter 23, Section 2312 : Earthquake regulations", International Conference of Building Officials, Whittier, California, USA.

Yomogida K. and Etgen J.T. (1993) "3-D wave propagation in the Los Angeles basin for the Whittier-Narrows earthquake", Bull. Seismo. Soc. of Am., Vol 83, No 5, pp 1325-1344.

3. GEOTECHNICAL ASPECTS

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3.1 Summary

The magnitude 6.6 earthquake which shook the city of Los Angeles in the early hours of January 17, 1994 caused extensive damage to the epicentral region in Northridge and its surrounding districts. Earthquake-induced landslides occurred at different locations, the most spectacular of which was on the east side of Pacific Coast Highway, north of Venice Beach. Large ground movements were noticed near the epicentre, particularly across Balboa Boulevard causing damage to lifeline facilities, especially the gas mains which resulted in large fires. Some liquefaction-induced failures were observed in Redondo Beach on the Pacific Ocean front and near the Los Angeles Dam site. In the vicinity of the Los Angeles Dam the effects of liquefaction were comparable to the earthquake of 1971 (Seed et al, 1975) which also measured 6.6 on the Richter scale.

Large horizontal ground accelerations of up to 1.6g were recorded at several sites, with the N-S component usually grater than the E-W one. Large vertical accelerations were also recorded in almost all of the earthquake affected districts, with peak vertical accelerations greater than 1.0g at some locations. There was, however, nothing anomalous about the horizontal/vertical acceleration ratio. (See Chapter 2 for details of the strong ground motions). The districts in which there was severe damage to structures and lifeline facilities are located in a chequered pattern around the epicentre. This suggests strong amplification of ground motion and the resulting dynamic soil-structure interaction. The disparity of ground motion amplification can be readily observed by comparing the severe damage in the Santa Monica district, near Northridge with very little damage sustained by the neighbouring Marina Del Ray district, while the Redondo Beach front at a relatively large distance from the epicentre suffered severe damage.

Several earthquake induced failures seem to emphasise the importance of dynamic soil-structure interaction in determining the behaviour of a structure subjected to earthquake loading. The dramatic failure of columns supporting a bridge on State Highway 118 suggested foundation settlement and a definite interaction of the bridge deck, column and foundation system during the earthquake. Columns supporting a pipeline on the upstream side of the Upper San Fernando Dam and uplift of pylons supporting transmission lines showed significant interaction with foundations during the earthquake.

Sand boils were observed at several locations on the downstream side of the Lower San Fernando Dam. As much as 5 cubic metres of sand flowed out from a single sand boil immediately after the earthquake at this site. Also the generation of excess pore pressures due to the incoming shear waves resulted in large sand boils in the beach sands on the Pacific Ocean front. A large sand boil was observed in the Los Angeles Port area.

3.2 Introduction

The city of Los Angeles is close to a number of fault lines including the San Andreas, San Gabriel and Santa Sussana faults. This geographic location resulted in many earthquakes in the past. In February 1971 the city experienced an earthquake of magnitude 6.6 which resulted in severe damage to the civil engineering structures especially the Lower and Upper San Fernando Dams. The Northridge earthquake of January 17, 1994 also measured 6.6 on the Richter scale with many smaller earthquakes occurring in the period between 1971 and 1994. The topography of the city, with deep alluvial soil deposits in some districts surrounded by mountain ranges like San Bernardino and San Gabriel, results in significant amplification of the stress waves arriving from a fault plane during an earthquake. This modification of the bed rock acceleration resulted in a chequered pattern of damaged districts surrounding the epicentre. The EEFIT team visited a number of specific failures that occurred as a result of the Northridge earthquake but did not attempt to carry out a rigorous survey of all such failures. A comprehensive list of all the geotechnical failures in the area was however made available to the EEFIT team by the Office of the Emergency Services (OES) in Pasadena.

Geotechnical events resulting from the present earthquake can be classified as landslides resulting from the earthquake, interaction of soil-structure systems during earthquake loading, occurrence of sand boils, liquefaction-induced failures and large ground movements. In this section the significant aspects of these failures will be presented with some examples. The damage suffered by the earth dams in the Los Angeles area will be dealt with separately in Chapter 4 of this report.

3.3 Earthquake-Induced Land Slides

Several land slides were reported in the earthquake affected districts. The most spectacular of the land slides occurred on the Pacific Coast Highway north of Venice Beach. The extent of the land slide was about 400m in width and about 45m in height running in the N-S direction. A residential building at the lip of the slide failed during this earthquake with its roof coming down the slope (Plate 3.1). The foundation piles of several buildings in this location were exposed after the slide (Plate 3.2). On the northern side of the slope a horizontal layer of wetting was observed at about the mid height of the slide (Plate 3.3). This may be due to the breakage of a water main up the slope which resulted in the wetting of the slope plane and initiated the slope failure. Another major land slide was observed along the Canyon Boulevard in the Hollywood district. Less significant land slides comprising of small boulders and foliage were observed near the Upper Hollywood Dam site (Plate 3.4).

3.4 Dynamic Soil-Structure Interaction

Some of the failures which resulted from the earthquake can be understood more completely by considering the soil-structure interaction effects. One of the examples for this is the failure of columns supporting a bridge on State Highway 118. One half of the structure collapsed completely and the columns supporting the other half of the bridge suffered severe compression/shear failure with reinforcement popping out of concrete cover. The sag in the bridge deck was more than that accounted for by the failure of columns, indicating a foundation settlement (see Chapter 5). This failure may be understood by considering the deck-column-foundation interaction under large vertical and horizontal accelerations caused by the earthquake. Some of the columns supporting the Santa Monica Freeway have shown cracks up to 200mm deep in the soil adjacent to the columns (see Fig. 3.1). The dynamic behaviour of such a column-foundation system depends strongly on the stiffness of the soil surrounding the column and the foundation compliance (Lee, 1994). Also some of the bridge abutments have suffered rotation under the dynamic earth pressure caused by the earthquake.

The elevated outlet pipeline supported on columns on the downstream side of the Upper San Fernando Dam (Plate 3.5) is another example which deconstructed the interaction effects. During the present earthquake the pipeline-column-foundation system functioned as a single unit until the columns rotated and the fixings securing the pipe to the column failed (Plate 3.6). Wide gaps opened in the soil adjacent to the base of the column.

3.5 Sand Boils

Severe sand boils were observed during the present earthquake on the upstream side of the Lower San Fernando Dam (Plate 3.7). The soil flow in the largest sand boil was measured to be 5 cubic metres in volume. The loose sandy soils in this region also liquefied in the 1971 earthquake (Seed et al, 1975). The shallow water table due to the presence of a small lake in the vicinity and predominantly sandy nature of the soil resulted in the generation of excess pore pressures during the earthquake causing these sand boils. Several sand boils were observed in the Redondo Beach area along the Pacific coast with soil flows occurring up to several hours after the earthquake. A major sand boil was observed in the Los Angeles Port area.

3.6 Liquefaction-Induced Failures

The quay wall at the King Harbour site in the Redondo Beach area suffered a severe liquefaction-induced failure. A 30m section of the south quay wall moved by about 6m into the sea forming a bow section in the plan view (see Fig.3.2). The north quay wall however suffered little damage (Plates 3.8 and 3.9). There was severe ground subsidence between the quay walls (Plate 3.10). The quay wall itself showed severe rotation with its base moving outwards, confirming the soil movement in that direction. This is further verified by the concrete anchor pillars bent in the same direction (Plate 3.11). Two structures located on the harbour suffered severe damage due to ground subsidence; one of these is shown in Plate 3.12. The Marina Del Ray district adjacent to the severely effected Santa Monica district suffered little damage while Redondo Beach, which is at a relatively large distance from the epicentre, suffered severe damage. This emphasises the site amplification characteristics in these two marina districts.

The pump house near the north dike of the Los Angeles reservoir suffered severe liquefaction failure. There was extensive cracking of the building due to ground subsidence which measured up to 350mm (Plate 3.13). A minor dam in this area failed during the earthquake. Also, the filter beds of Jensen water plant in the vicinity suffered some settlement and cracking.

3.7 Large Ground Movements

Large ground movements and extensive cracking resulted in severe damage to the pipelines. The most significant of these was the damage to the outlet pipe from the Los Angeles reservoir (Plate 3.14). The erosion resulting from the pipe breakage removed the traces of ground cracking but the cracks in the foundation of the pump house indicated the ground movement (Plate 3.15). The ground movement across Balboa Boulevard resulted in the upward buckling of the lifelines (Plate 3.16). This is further confirmed by the upward movement of up to 225mm of the pavement blocks (Plate 3.17).

3.8 Conclusions

The Northridge earthquake of 17 January 1994 caused extensive damage to the Freeways and supporting structures. The geotechnical failures included massive landslides, sand boils and liquefaction-induced failures. There are major lessons to be learnt in understanding the amplification of ground motion in soft soils even at relatively large distances from the epicentre. The proper understanding of soil-structure interaction during earthquakes may provide a key while studying some of the failures. These issues may be critical in the seismic risk assessment of structures during an earthquake of significantly larger intensity, which is a distinct possibility in Los Angeles.

3.9 References

R Lee (1994) Personal communication during ASTM Conference on Dynamic Geotechnical Testing, Calcrans, Sacramento.

H B Seed, K H Lee and I M Idriss (1975) "The slides in the San Fernando dams during the earthquake of February 9, 1971", ASCE Jnl of Geotechnical Engineering Div, GT7, pp 651-688.

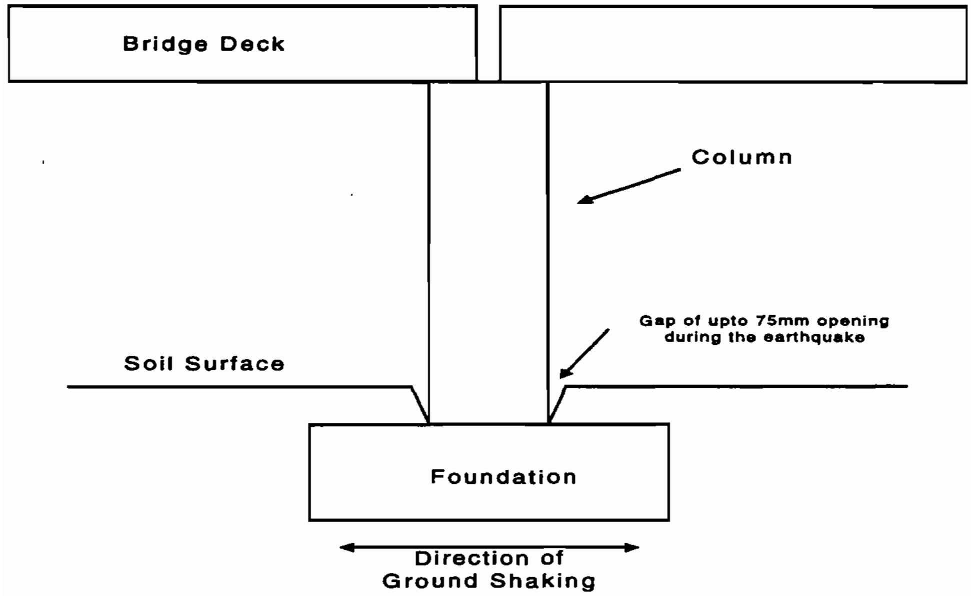

Figure 3.1: Schematic Diagram of Gaps Opening at the Base of the Column

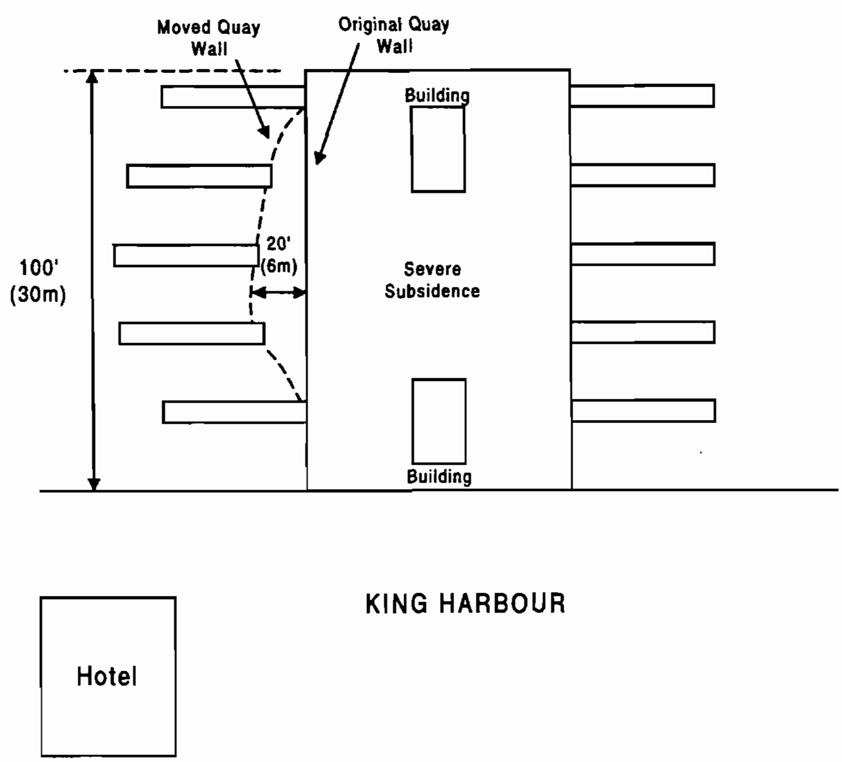

Figure 3.2: Sketch Plan of King Harbour

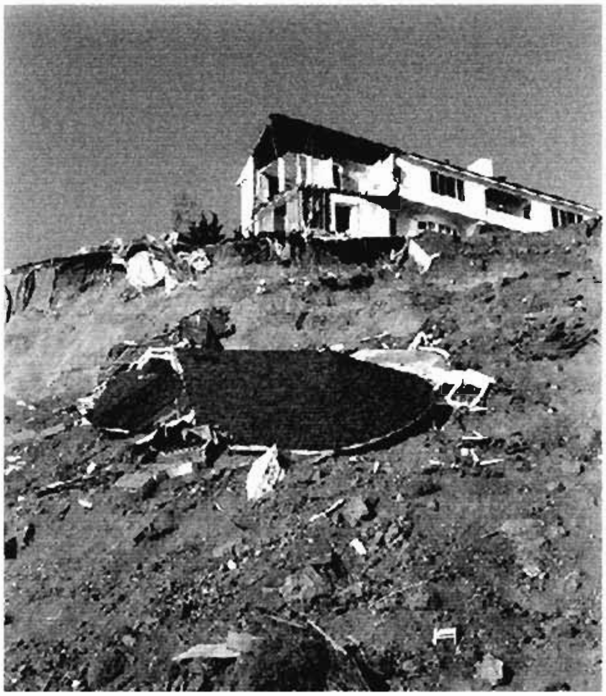

Plate 3.1: Land slide on Pacific front; plate shows the dislodged roof of the house after the earthquake

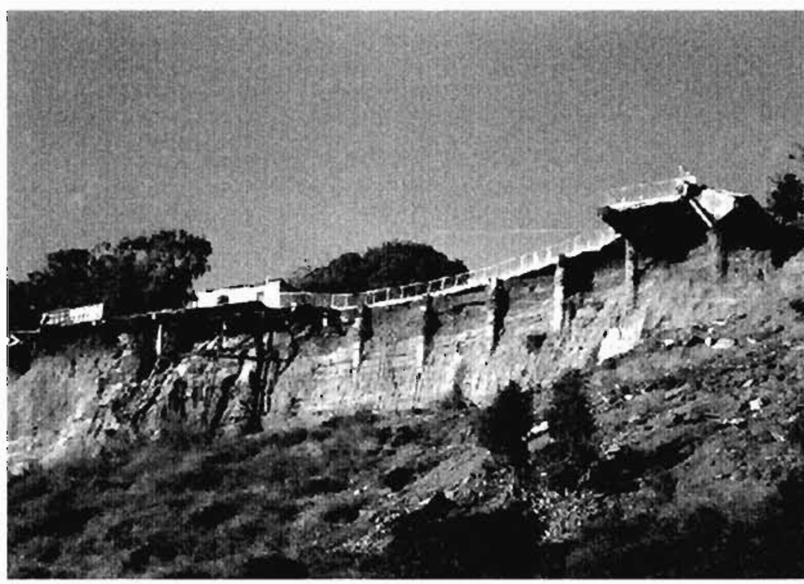

Plate 3.2: Exposed pile foundations of structures following the land slide

Plate 3.3: Northern end of land slide on Pacific front; plate shows wetting of soil, possibly due to a broken water main

Plate 3.4: Land slide near the Hollywood Dam; the extent of the land slide was camouflaged by the foliage at the site

Plate 3.5: Pipeline on downstream side of Upper San Fernando Dam; the ground motion at this site was along the pipeline

Plate 3.6: Sheared connection between the pipe and column; plate also shows leaning of the column and the gap between the column and foundation soil

Plate 3.7: Large sand boil in the Los Angeles Dam complex; plate shows the extent of the sand boil

Plate 3.8: Quay wall at the King Harbour before the earthquake

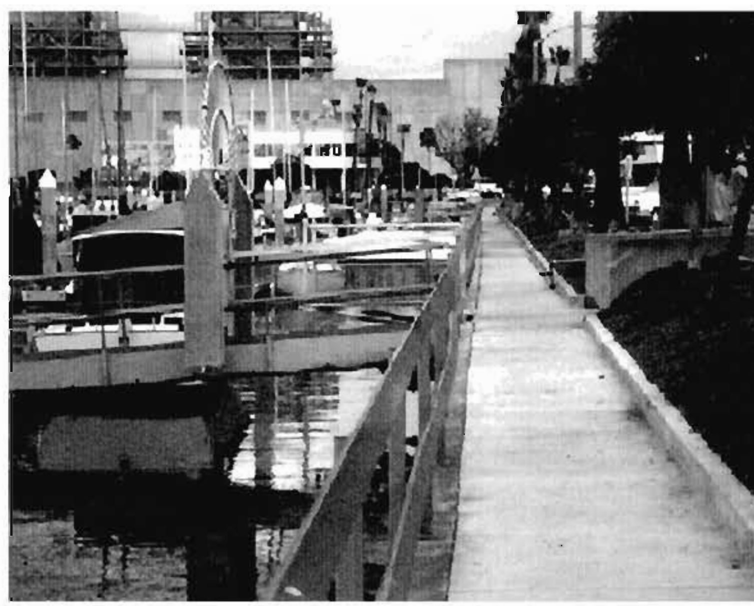

Plate 3.9: Quay wall after the earthquake; a 30m section has moved 6m into the sea

Plate 3.10: Subsidence between the quay walls; plate shows original level flush with the manhole cover and the subsidised ground level

Plate 3.11: Bent concrete anchor pillars indicate direction of soild flow following liquefaction

Plate 3.12: Structural damage due to subsidence following liquefaction; plate shows left hand side of building higher than right

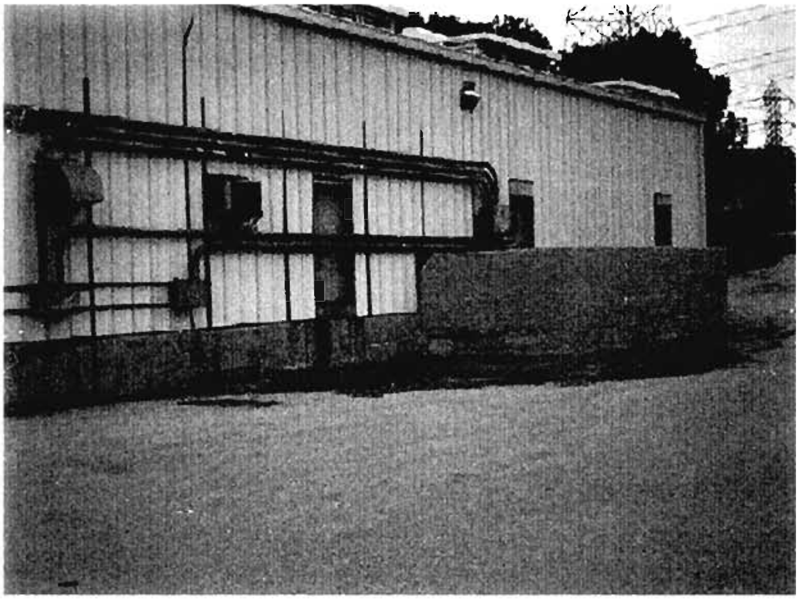

Plate 3.13: Subsidence following liquefaction adjacent to a pump house in the Los Angeles Dam complex; a minor dam near this site failed following the earthquake

Plate 3.14: Breakage of main outlet pipe of Los Angeles Dam; plate shows recent repairs to the pipe following the earthquake

Plate 3.15: Ground movement causing failure of outlet pipe is confirmed by failure of foundation block of nearby pump house

Plate 3.16: Upward bending of lifelines on Balboa Boulevard; burst gas mains caused extensive fires here

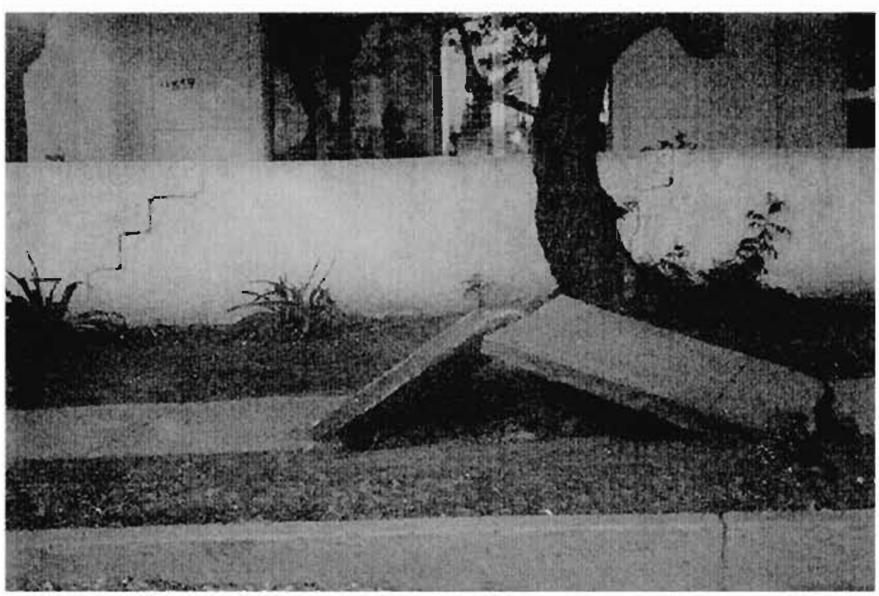

Plate 3.17: Dislodged pavement blocks due to ground movement

4. PERFORMANCE OF DAMS AND APPURTENANT WORKS

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4.1 Introduction

The aim of this section of the EEFIT mission was to study the seismic performance of the dams in the Los Angeles vicinity, their appurtenant structures, pipe lines and electrical and mechanical plant. A study of a detailed map of the Los Angeles area and the ICOLD World Register of Dams (ICOLD, 1979) reveals the sites of numerous dams; at least 14 within a 32km radius of the Northridge epicentre.

Generally, the dams performed well with minor cracking occurring at a few embankment dams and two cases of more notable damage. Pacoima Dam, a concrete arch structure, experienced a major crack in the thrust block to its left abutment. At the Los Angeles Reservoir, the main earth dam, suffered cracking in the asphalt lining on its upstream slope. Of lesser importance, but still of interest, is the damage to the Lower and Upper San Fernando embankment dams which were taken out of service after failing during the 1971 San Fernando earthquake (Seed et al, 1975).

There were two instances of the localised failure of the bearings to intake tower bridges; one at the Los Angeles Reservoir and one at Castaic Lake. These cases indicate that the movement of the towers may have caused the failure in the bridges. Another less serious case of damage to a tower and bridge occurred at the Upper Hollywood reservoir. It is believed that major pipe work and electrical and mechanical plant generally performed well with the exception of a burst in the main outlet pipe from the Los Angeles Reservoir.

All of these incidents of damage will be reported in the following sections in order of the distance of their sites from the epicentre of the Northridge earthquake.

4.2 Los Angeles Reservoir

The Los Angeles Reservoir lies about 5km north-east of the Northridge epicentre. Its construction was completed in 1977 and includes two earth dams; the main dam, 39m high and 1040m long and the smaller North Dike. The upstream slopes of both dams are protected by asphalt lining. A reinforced concrete intake tower serves as the outlet works to the main 3m diameter outlet pipe. Access to the tower is provided by an 80m long bridge consisting of three simply supported spans constructed from steel plate girders and a steel deck. The bridge is supported on a concrete abutment at the crest of the dam, on two concrete piers and on corbels at the tower. The tower and bridge are shown in Plate 4.1.

As a result of the earthquake, both the main dam and the North Dike developed cracks, the access bridge to the tower was damaged and the main outlet pipe burst.

4.2.1 Damage to the Los Angeles Dam

A series of diagonal cracks (Plate 4.2) developed in the asphalt lining on the upstream slope along the full length of the dam. They extended from crest level to below the reservoir water level. The width of these cracks was very fine at the crest and generally increased going down the slope, reaching 25mm in places. A number of other cracks in the asphalt lining appeared emanating from the vertices of the octagonal section of the bridge pier embedded in the dam. In plan, the axis of the dam has a straight central portion and curves at both ends and a very fine transverse crack in the asphalt lining occurred at each end of the straight section of the dam. Small scale localised slipping was observed in a few areas on the upstream face.

Seepage through the main dam was reported to have increased to twice its normal rate immediately after the earthquake and subsequently to be reducing.

4.2.2 Damage to the North Dike

Along the crest of the North Dike, an open trench of about 1.2m depth and 0.8m width had been excavated for the installation of pipe work. One end of this trench had been back-filled with mass concrete, a material considerably stiffer than the natural fill material of the dike. The earthquake caused a crack in the dam initiating from the exposed face of the back fill and running across the crest and the downstream slope. In another location, a transverse crack occurred which could be seen on both faces of the dike and also running around the inside face of the trench.

4.2.3 Damage to the intake tower bridge

The central span of the bridge was dislodged sideways by about 50mm at one support and 0.4m at the other (Plate 4.3). The greater displacement, at the tower end of the span, resulted from the steel girders coming out from their lateral restraints (Plate 4.4). These restraints appear to be designed to hold the girders down on their bearings and to restrain longitudinal movement by frictional resistance only. The freed end of the central span had collided with the span adjacent to the tower resulting in the crushing and buckling of the steel plates of their decks and the bending of end stiffeners on girders. The damage to the stiffeners was caused by the bridge decks cutting into them once the central span had been offset laterally.

The tower and the adjacent bridge span collided also causing crushing and buckling of the bridge deck. There was evidence of the closing of the 125mm gap which was measured between the face of the tower and the stiffening plates on the girders. The stiffening plates are covered with a neoprene pad of about 25mm thickness and these had made a black impression on the tower which corresponded to a pattern of greyish-white marks, from the concrete on to the neoprene.

The nature of the damage to the bridge indicates that the tower moved excessively, hitting the adjacent bridge span and forcing it to slide on its bearings so that it shunted and damaged the central span. Alternatively, it can be hypothesised that the inertia forces on the bridge spans alone caused them to move excessively. However, had this been the case then evidence of similar degrees of movement for each of the three spans would have been expected and the span supported at the dam showed signs of very slight movement only. Of course, the differing supports would influence the movement of the spans but without a numerical analysis, a conclusion concerning the damage mechanism cannot be justified.

4.2.4 Main outlet pipe

During the earthquake, the main 3m diameter outlet pipe from the reservoir burst at a point along its buried length (Plate 4.5) flooding the Rinaldi Receiving Station (electrical power) which lies downstream of the damaged section of pipe.

4.2.5 Significance of damage at Los Angeles Reservoir

Although the damage to the access bridge was relatively minor, its nature is significant when considering the potential effects of the strongest earthquake expected to occur at this site. In the event of the maximum credible earthquake, it is conceivable that the central span of the bridge could fall from its supports, restricting access to the intake tower. In addition, the damage to the dam could be more extensive, possibly endangering its safety and posing an imminent threat of a further disaster. If the valve to the main outlet pipe was closed, it would have to be opened from the intake tower to enable the rapid draw down of the reservoir to reduce the load on the dam. The efficient evacuation of the reservoir water would be delayed until access to the tower had been achieved. The difficulty of this task would depend on the extent of the damage to the tower itself.

An alternative scenario is where the dam is still safe, the access bridge has collapsed and the main outlet pipe has burst and is flooding property downstream. Delayed access to the tower would result in more severe flooding causing further damage to property.

Finally, the impact of the collapsed bridge span with the dam could initiate a failure in the upstream slope.

The damage at the Los Angeles reservoir highlights the importance of the continued functioning of the intake tower and its access bridge in the overall safety of the dam. Also, it indicates that the relative movement between the tower and bridge was greater than expected and that there may have been influential factors contributing to their response which are not understood fully and therefore could not be modelled accurately in the original numerical analysis. Such factors would include dynamic soil-structure interaction and fluid-structure interaction which are fields of interest continuing to attract considerable research since this dam was constructed.

4.3 Upper and Lower San Fernando Dams

The Upper and Lower San Fernando Dams are earth dams constructed in the earlier half of this century by the hydraulic fill method. They were taken out of service as a result of the damage they suffered during the 1971 San Fernando earthquake. In 1971, the 43m high, Lower San Fernando Dam experienced a significant slide in its upstream face threatening a major disaster and there was a notable slide in the downstream slope of the 25m high Upper San Fernando Dam. Since then, their storage facility has been replaced by the Los Angeles Reservoir constructed at the same site between the two abandoned reservoirs. The Lower San Fernando Dam has been repaired and with the reservoir basin empty, functions for flood control should a failure of the Los Angeles Dam occur. Details of these dams, their failures and repairs can be found in Seed et al (1975).

4.3.1 Damage to the Lower San Fernando Dam

The Northridge earthquake caused numerous longitudinal cracks to develop along the crest of the Lower San Fernando Dam (Plate 4.6) indicating that lateral spreading of the dam may have occurred. Cracks as wide as 50mm and as deep as 1.5m have been measured. Upstream of the dam, lines of sand boils were found in the dry reservoir basin. In 1971, evidence of sand boiling was found at the same locations once the reservoir had been drained.

4.3.2 Damage to the Upper San Fernando Dam

The Northridge earthquake caused further damage to the Upper San Fernando Dam with severe cracks occurring on both abutments and in the concrete spillway structure. Longitudinal cracks on the crest appeared at locations almost identical to those which occurred in 1971 and there were indications that the crest had settled further by about 75mm.

4.4 Pacoima Dam

Pacoima Dam, a 113m high concrete arch structure, lies approximately 18km north-east of the Northridge epicentre. Peak ground accelerations of 0.44g horizontally and 0.2g vertically were recorded at the site (CSMIP, 1994) and considerable damage occurred as a result of the shaking.

The dam with a crest length of 180m is located in an extremely narrow, steep-walled canyon. The predominant rock type at the site is gneissic quartz diorite, a crystalline, granite type rock. A combined pattern of joints and shears is prevalent dividing the rock into angular blocks. The rock is geologically young and hence, prone to rock falls so the abutments to the dam are covered with a gunite layer to protect the utility from falling rock debris. A low gravity thrust block was provided at the left abutment of the arch because of unfavourable localised foundation conditions.

4.4.1 Previous damage in 1971 San Fernando Earthquake

The damage to Pacoima Dam caused by the 1971 San Fernando earthquake and the subsequent repairs have been reported in detail by Swanson and Sharma (1979) and by Sharma and Sasaki (1985). The principal damage was in the region of the left abutment where movement of large blocks of the rock mass occurred. A crack developed in the left thrust block and the vertical contraction joint between the dam and the thrust block opened. The crack and open joint were repaired by sealing and grouting, and the left abutment rock mass supporting the thrust block was stabilised using post-tensioned rock anchors.

4.4.2 Damage from 1994 Northridge Earthquake

During the Northridge earthquake, peak accelerations at the crest of the dam of 2.3g horizontally and 1.5g vertically were recorded (CSMIP, 1994) although higher vertical accelerations were measured on the dam adjacent to its left abutment. The shaking resulted in damage similar to that experienced in 1971.

A new crack developed in the left abutment thrust block close to the repaired crack and the vertical contraction joint between the dam and the left abutment opened by about 50mm. The thrust block was noted to have moved downstream by about 12mm. There was extensive cracking and sliding of the gunite on both abutments which damaged and restricted some of the access routes to the crest of the dam. Considerable rock falls occurred in the canyon damaging the braced spillway chute and blocking it with rock debris.

Fortunately, at the time of the earthquake, the reservoir was below the level of the cracks, otherwise the additional hydrostatic and hydrodynamic loads on the dam may have exacerbated the damage and caused some flooding downstream.

4.5 Castaic Lake

4.5.1 General description of dam, intake tower and access bridge

Castaic Lake, situated about 40km north of the Northridge epicentre experienced peak ground accelerations of 0.52g horizontally and 0.22g vertically. The only reported damage at this site was to the intake tower bridge and to a crane mounted on the top of the tower. A 130m high zoned-earth embankment dam, constructed in the 1970s, impounds the reservoir and is founded on a bedrock composed of silty shale interbedded with sandstone. The outlet works comprise of a low level intake and a multiple level 65m high, reinforced concrete intake tower of approximately 12m diameter. The tower connects to the outlet tunnel via a 43m high vertical reinforced concrete shaft. Access to the tower is provided by a 150m long bridge made up from four simply supported spans of welded-plate girders acting compositely with a lightweight concrete deck (Plate 4.7). The bridge is supported on the intake tower, on reinforced concrete piers, and on a reinforced concrete abutment. The bridge superstructure was designed to be highly articulated and includes features detailed to accommodate extreme earthquake movements. One such detail is the roller bearing at the end of each span which allows for longitudinal movement (Plate 4.8).

4.5.2 Damage to intake tower bridge

Two of these bearings failed during the earthquake. Longitudinal movement of the second bridge span from the tower was so extreme that at the tower end, the girders rolled off of the roller bearings and came to rest on another part of the support (Plate 4.9).

The bridge span adjacent to the tower experienced notable longitudinal displacements and this was evident at the tower support where steelwork in contact with the roller bearings had been abraded as the rollers had moved to and fro. At this joint, two steel cover-plates cantilevering from the tower buckled when they collided with the bridge cutting into a reinforced concrete upstand. These plates were not structural and restricted the relative movement between the bridge and tower allowed for in the structural detailing of this joint. The impact between these plates and the adjacent span may have caused the shunting of the next span along pushing it off of its roller bearings.

The mechanism which caused the damage to this bridge appears to be similar to that which occurred at the Los Angeles Reservoir. The tower and its adjacent bridge span collided and the next span failed at its bearings nearest the tower. At the Los Angeles reservoir, there was evidence of the collision between the two bridge spans. At this site there was not, but the detailing is such that a longitudinal force could be transferred across the joint. The hypothesis that the extreme movement of the tower is responsible for the damage is supported by the strong motion records from Castaic which reveal that at bridge deck level, the tower moved a maximum of 0.16m horizontally towards the bridge (Figures 4.1 and 4.2).

4.5.3 Response of the tower

The accelerations at the top floor level of the tower were recorded in three orthogonal directions, vertically and horizontally in directions parallel and transverse to the longitudinal axis of the dam. The peak vertical acceleration was 0.17g and the peak horizontal accelerations were 0.78g transverse to the dam axis and 0.64g parallel to the dam axis. The equipment housed in the top of the tower appeared to be unaffected by these accelerations although, externally, a crane mounted on top of the tower had been displaced from its rails.

Frequency analysis of the accelerographs reveals that the natural frequencies of the tower in its lowest cantilever modes along orthogonal principal axes were 1.20Hz and 1.31Hz. These figures compare well with the frequency of 1.28Hz calculated from the original modal analysis of the tower reported by the Department of Water Resources (1974).

4.5.4 Significance of the damage

Similar to the bridge at Los Angeles Reservoir, the damage at this site was relatively minor although again, its significance should not be under estimated. Castaic Dam is situated 5km from the San Gabriel Fault and 20km from the San Andreas Fault so its appurtenant structures would have been designed to resist much stronger ground shaking. In the event of a stronger earthquake, the damage to the bridge could be more serious, possibly leading to a collapse. The consequences of the loss of access to an intake tower have been discussed previously (Section 4.2.5).

4.6 Upper Hollywood Reservoir

A further example of damage to a tower and access bridge occurred at the Upper Hollywood Reservoir. Again the apparent damage was minor although its probable cause is of interest.

The reservoir is impounded by a 25m high earth dam constructed during the 1930s. A reinforced concrete tower functions as the outlet structure and access is provided by a 35m long, continuous, five span bridge (Plate 4.10). The bridge and its balustrades are constructed in reinforced concrete.

In the natural embankment just above the bridge abutment, a land slide occurred and part of the slope appears to have moved in the direction of the longitudinal axis of the bridge. The nature of the damage on the bridge and tower indicates that this slip may have pushed the bridge into the tower. Concrete had spalled from the face of the tower from bridge deck level up to the top balustrade where the tower and bridge had impacted (Plate 4.11). In addition, the tower appeared to be leaning slightly away from the bridge although this has not been confirmed by a survey.

4.7 Conclusions

There were two notable cases of damage to dams but they were not serious enough to pose the threat of a further disaster. Considering their abundance, the dams in the Los Angeles vicinity can be considered to have performed very well during this moderate intensity earthquake.

The damage to the bridges at Castaic Lake and Los Angeles Reservoir appears to have been caused by excessive motion of their intake towers. As these structures were designed seismically, and for a stronger earthquake, there is an indication that factors influencing the response of intake towers are not understood sufficiently to be modelled accurately in numerical response analyses. This is an area requiring further research and development.

The damaged bridge at Castaic Lake demonstrates that where structural joints are designed for relative movement, non-structural elements require similar attention as poor detailing may impair the performance of the joint.

4.8 References

CSMIP (1994) "Quick report on CSMIP Strong-motion data from the Northridge/San Fernando Valley earthquake of January 17, 1994", Reports OSMS 94-04 and OSMS 94-05, California Strong Motion Instrumentation Program, Sate of California, Division of Mines and Geology.

Department of Water Resources (1974) "California State Water Project", Bulletin No. 200, Vols I-VI, State of California, Department of Water Resources.

ICOLD (1979) "World register of dams", International Commission on Large Dams.

Seed H.B., Lee K.L., Idriss, I.M. and Makolisi F.I (1975) "The slides in the San Fernando Dams during the earthquake of February 9, 1971", J. Geotech. Eng. Divn. ASCE, Vol 101, No. GT7, 651-688.

Sharma R.P. and Sasaki B.T. (1985) "Rehabilitation of earthquake shaken Pacoima arch dam", Transactions, 15th ICOLD Congress, Lausanne.

Swanson A.A. and Sharma, R.P. (1979) "Effects of the 1971 San Fernando earthquake on Pacoima Dam", Transactions, 13th ICOLD Congress, New Delhi.

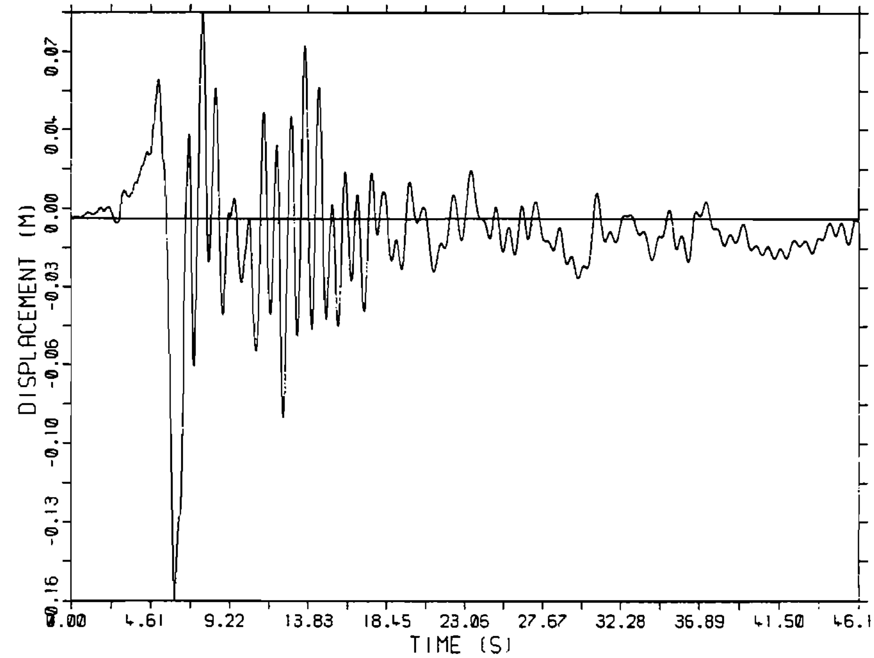


Figure 4.1: Castaic Lake - Longitudinal displacement of intake tower at bridge deck level

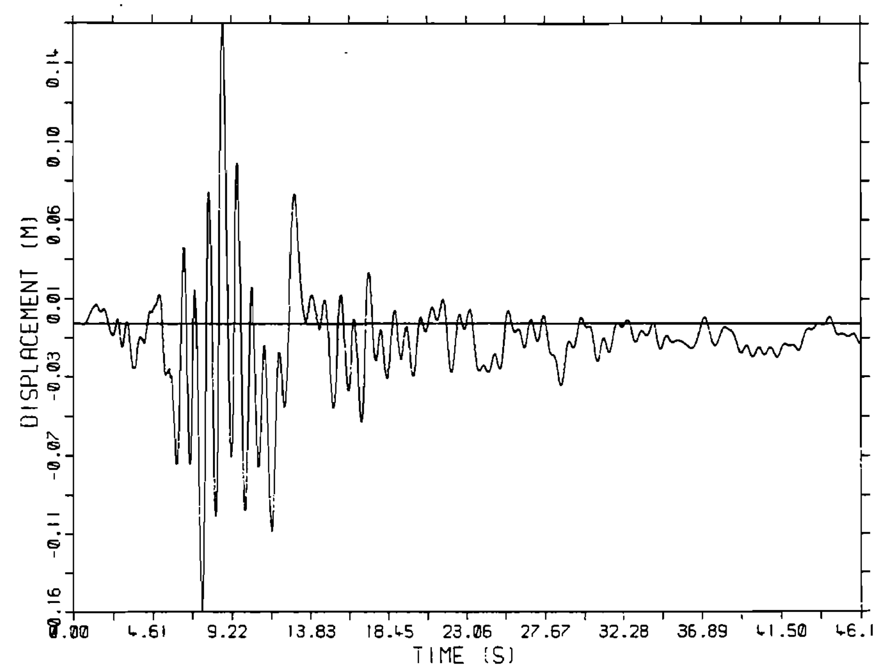


Figure 4.2: Castaic Lake - Transverse displacement of intake tower at bridge deck level

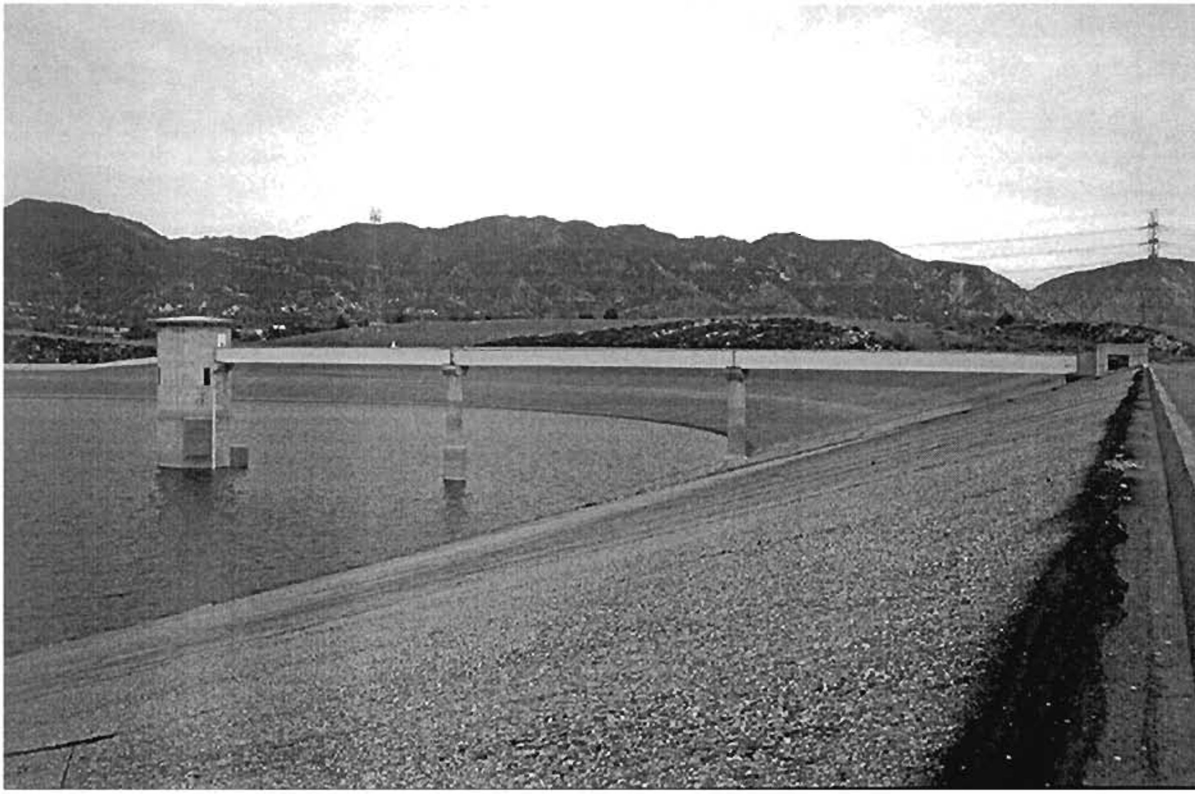


Plate 4.1: Los Angeles Reservoir - Dam, Intake Tower and Access Bridge

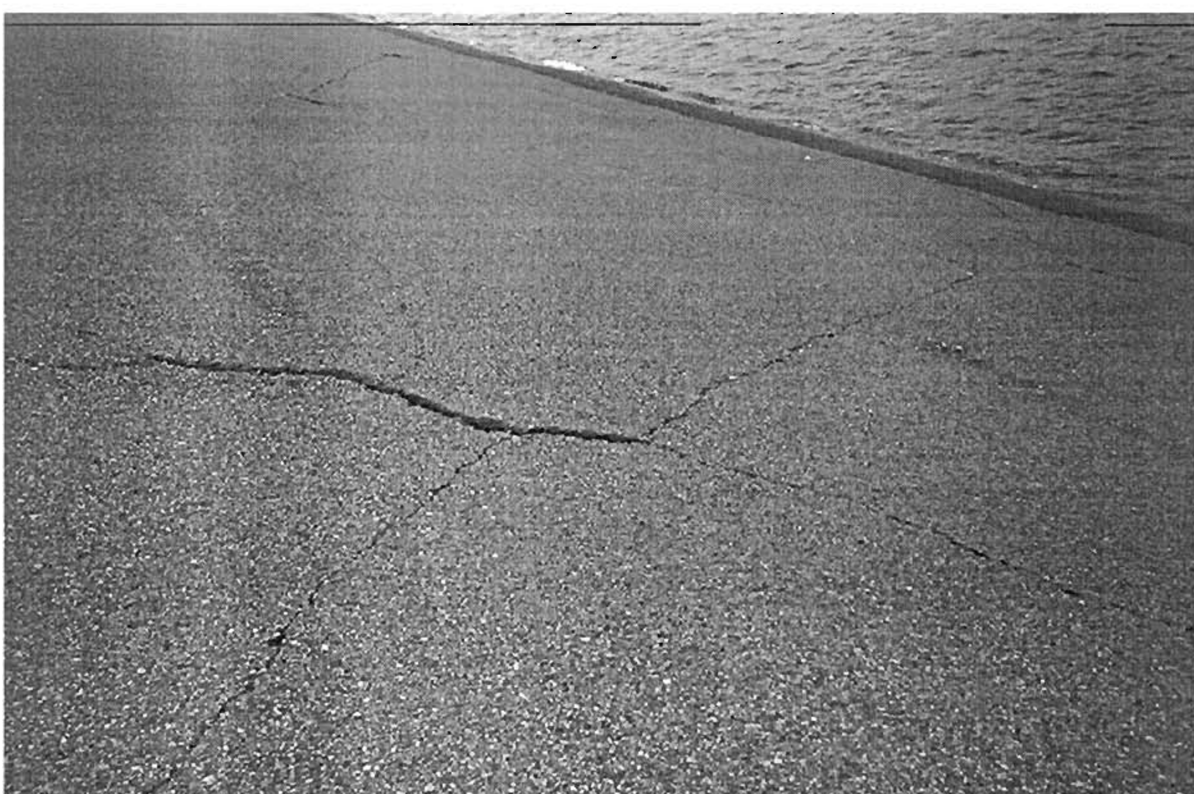
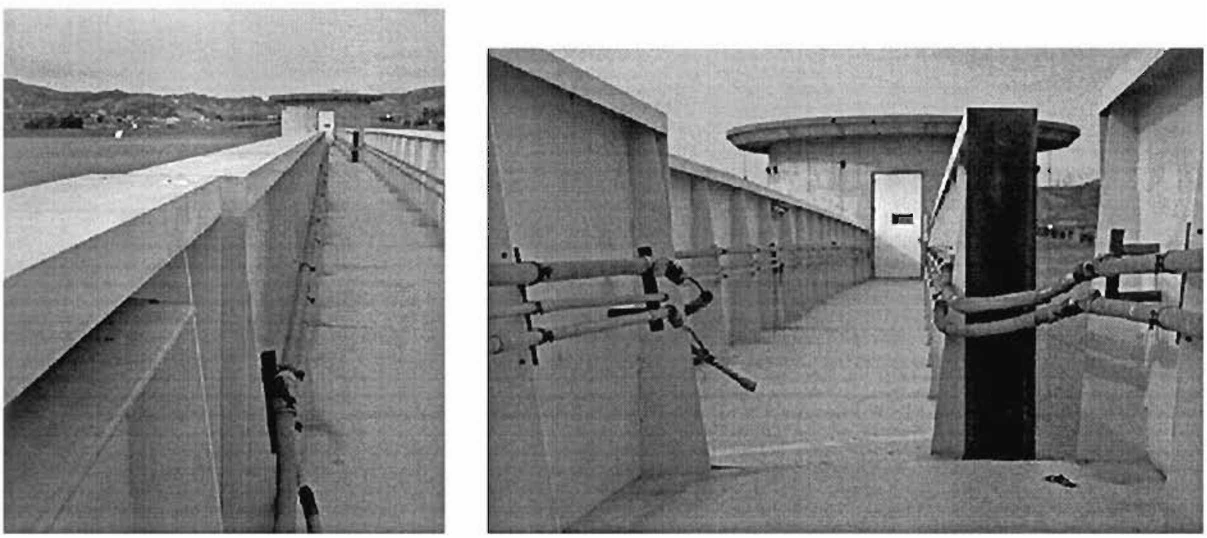


Plate 4.2: Los Angeles Dam - Diagonal Cracks in Asphalt Lining on Upstream Face



(a) (b)

Plate 4.3: Los Angeles Reservoir - View along bridge showing laterally offset central span

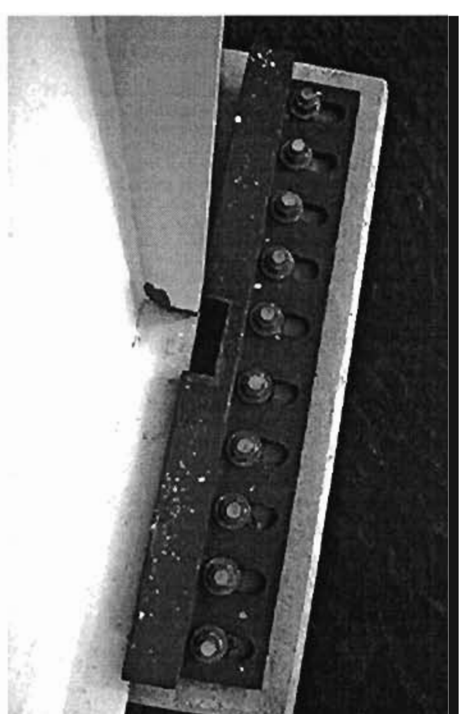


Plate 4.4: Los Angeles Reservoir - Detail of failed restraint at intake tower bridge support

Plate 4.5: Los Angeles Reservoir
Repaired outlet pipe

Plate 4.6: Lower San Fernando Dam
Longitudinal crack in crest

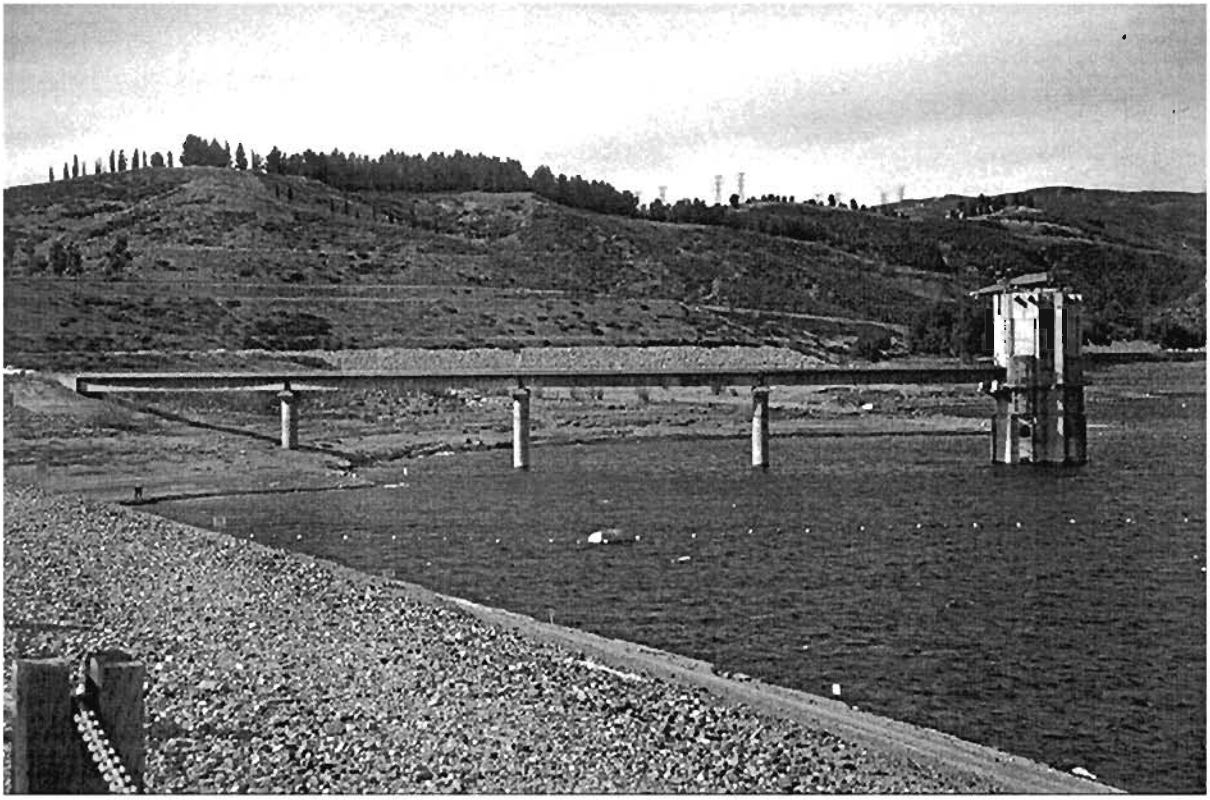


Plate 4.7: Castaic Lake - Dam, Intake Tower and Bridge

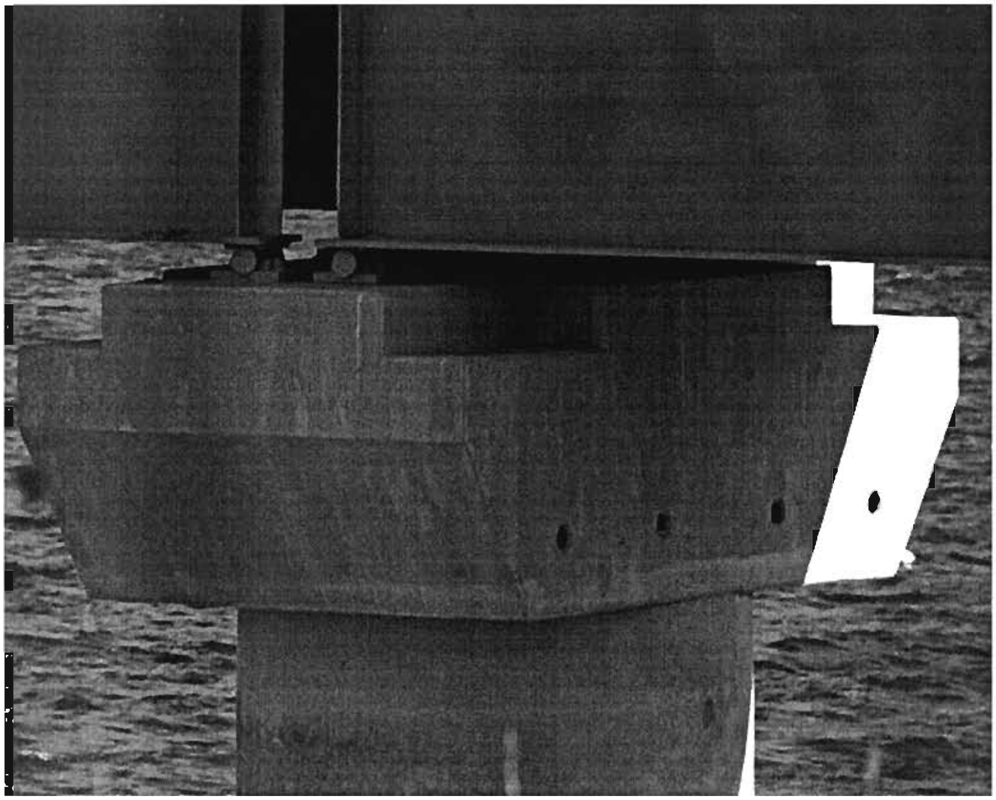


Plate 4.8: Castaic Lake - Typical detail of roller bearings to intake tower bridge

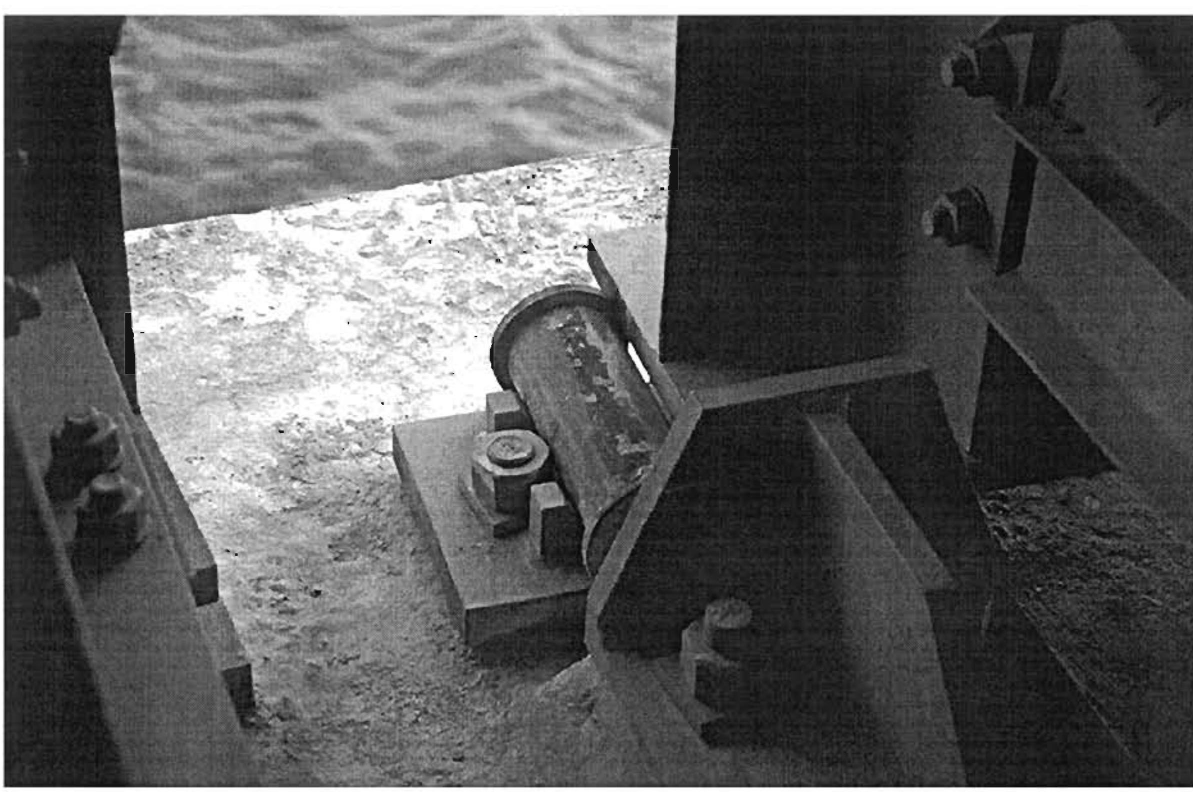


Plate 4.9: Castaic Lake - Failed roller bearing

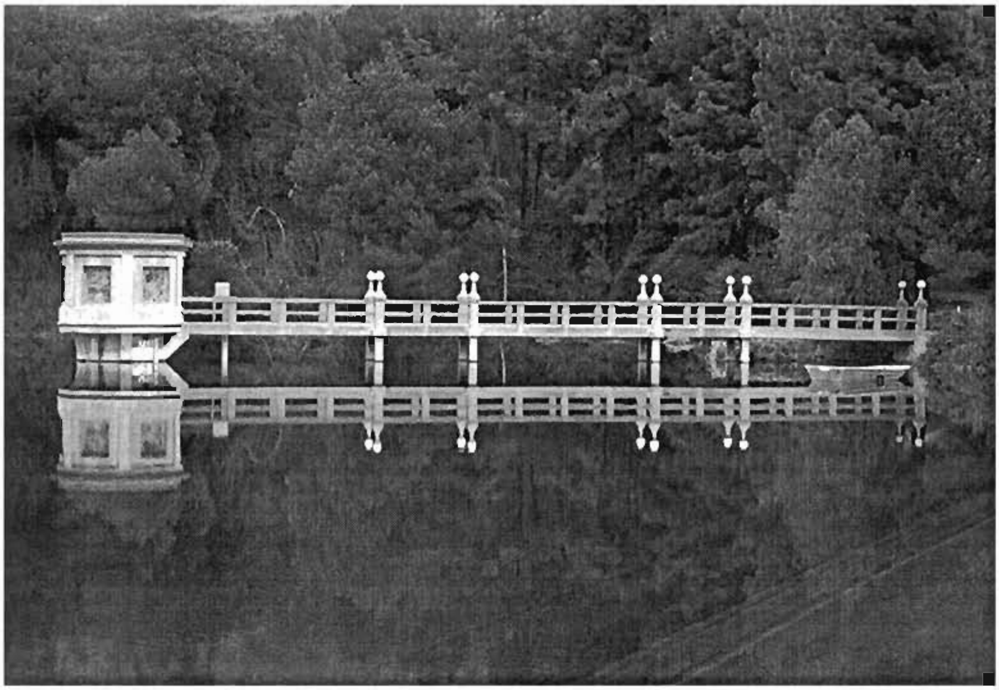


Plate 4.10: Upper Hollywood Reservoir - Outlet tower and access bridge

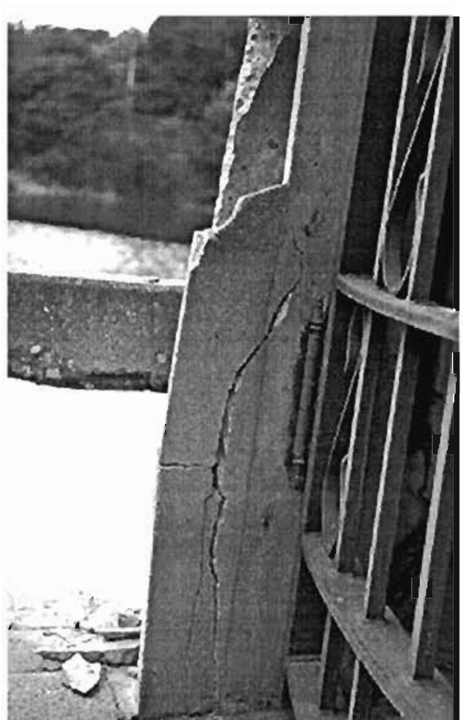


Plate 4.11: Upper Hollywood Reservoir - Damage to face of tower

5. BRIDGES

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5.1 Introduction

The image of the January 17 Northridge Earthquake portrayed by the British media was of widespread devastation. Coverage of bridges was no exception - all national newspapers had pictures of bridges that had collapsed, there was talk of tailbacks on the freeways and ironic features on Angelinos having discovered the train. What a contrast then to drive from Los Angeles International Airport to Hollywood during a weekday rush hour just ten days after the event - one damaged overbridge, heavily shored but in use, traffic heavy, but moving freely and no real sign of anything out of the ordinary.

Contrary to early reports, most of the highway infrastructure survived intact or requiring only little repair. Of the several thousand bridges in the affected region, only nine collapsed or were damaged beyond repair. Damage tended to be in pockets widely spaced about the city (Figure 5.1) although there was a heavy concentration of damage along Highway 118 which was close to the epicentre. The lessons that need to be learnt from this event must therefore centre on why certain bridges collapsed whilst others did not.

The aim of this chapter is to present the findings of the bridge investigations made during the EEFIT visit, and to draw conclusion relevant to these lessons. After a brief outline of local bridge engineering practice, the chapter will consider those structures that collapsed or were damaged in the earthquake. It shall then review the retrofitting techniques employed by CALTRANS and discuss their performance during the earthquake. Finally it will look at the reaction of the authorities to the event, considering their efforts to minimise disruption caused, their assessment and repair of damage, and their rebuilding of collapsed or condemned structures.

5.2 Background

The Los Angeles region possesses several thousand freeway bridges, nearly all of the same construction type. They consist of in situ concrete box girder superstructures with relatively long spans and often with a curved or skew alignment. Seat widths at expansion joints are normally around 150mm in the older bridges, but much larger values have been used since the mid-1970s. Bridges with narrow seats have mostly been fitted with cable restrainers to prevent excessive movements at the joints during earthquakes. Substructures usually consist of single or multiple column bents on pad or pile footings.

Previous earthquakes in California have caused considerable damage to bridge structures. Most notable among these was the 1971 San Fernando earthquake, with an epicentre very close to Northridge. This event exposed several design and construction deficiencies, including low design lateral force levels, inadequate confining reinforcement, lack of longitudinal restraint across expansion joints and insufficient seat widths at joints and supports. This led to a number of changes in seismic design provisions, and to the introduction of a major bridge retrofitting programme, of which the first phase was the provision of restrainers and seat extenders ar expansion joints. More recently, and particularly since the 1989 Loma Prieta earthquake, the pace of retrofitting has increased, with greater attention paid to the strengthening of columns, usually by the provision of grouted steel jackets.

5.3 Case histories

5.3.1 *Interstate 5/State Highway 14 interchange*

The interchange between Interstate 5 (I5) and State Highway 14 (H14) is situated in the hills to the North of the San Fernando valley approximately 15km from the epicentre. The layout of the junction is shown in Figure 5.2 and is complicated by the road network being partly duplicated to allow separate routes for trucks and cars, and by the old roads that were replaced by the freeways. The interchange was designed and built in the late 1960's and early 1970's. Although the interchange was badly damaged during the 1971 San Fernando earthquake whilst still under construction, the pre 1971 details were retained to avoid cost penalties. However, some retrofitting was carried out and the hinges were fitted with cable restrainers during the final stages of construction (CALTRANS, 1994). Further retrofits had been designed and were due to be implemented later this year (Civil Engineering, 1994).

The Northridge earthquake is thought to have resulted in peak horizontal accelerations at this site of between 0.7 and 1.0g (Yashinsky, 1994), and caused major collapses in the connectors from westbound Highway 14 to southbound I5 (W14-S5), and from westbound Highway 14 to northbound I-5 (W14-N5). Both connectors had similar structural forms, c.2m deep precast and reinforced concrete box girders supported on single flared columns of varying heights. Spans between columns varied from 40 to 60m, with expansion joints, or hinges, every two or three spans.

Connector W14-S5 failed at its southern end where the columns were shortest (Figure 5.3). The prestressed section forming the southernmost three spans collapsed. Bent 2, closest to the abutment, and at 7m the shortest column, was completely destroyed. Bent 3 remained standing with the deck falling down either side of it, the diaphragm remaining intact (Plate 5.1).

Calculations (Priestly et al, 1994) suggest that brittle shear failure of bent 2 initiated the collapse. This in turn would have led to bending failure of the deck at bent 3, with the resulting large displacements causing restrainer failure at the hinge (Plate 5.2) and loss of seating at the hinge and abutment.

Collapse of connector W14-N5 followed a similar sequence although the locations of hinges were different (Figure 5.4). Again, the shortest column (bent 2, 5.2m) adjacent to the abutment failed, most likely in shear. This led to failure of the deck in bending over bent 3 (Plate 5.3), and also loss of seating for the end span, which in this case was a short drop-in span between the hinge and the abutment.

Although only two lengths of bridge collapsed there was substantial damage to connectors that remained standing. Both abutments of the connector between southbound I5 and eastbound Highway 14 (S5-E14) were damaged, especially at the Highway 14 end where movement of the abutment wall was evident together with loss of concrete cover and buckling of reinforcement (Plate 5.4). There was also damage to the hinges on this connector; bearing pads had been completely destroyed, leaving the structure with significant horizontal and vertical displacements at the hinges, and there were signs of pounding between adjacent sections of the bridge. It is worth commenting here on the performance of the restrainers installed as part of the retrofitting process. As none of these sections came off the seats at the hinges it can be concluded that the cable restrainers fulfilled their primary purpose of holding the bridge together. However, the restrainer details allowed sufficient movement to destroy the bearings and for significant pounding damage to occur. In the event one seat was almost destroyed by pounding.

The behaviour of the hinges at this interchange (damage to or destruction of bearing pads and pounding between adjacent sections) was observed at several other sites. However, it is now the policy of CALTRANS to build these bridges as continuous structures, or at least with fewer hinges; typically bridges that had 90 to 120m between hinges now have over 300m between hinges. Such a bridge was observed further down the I5 and this showed substantial damage at both abutments, including destruction of the neoprene bearing pads and also plastic deformation in the columns. It should be noted here that the hinges can be repaired and some were being repaired during the visit to the I5/Highway 14 interchange.

5.3.2 Overpass of San Fernando Road at Interstate 5

This bridge, constructed in 1974, crosses the I5 just south of its intersection with the I210, and only a few kilometres away from some of the most severe bridge damage caused by the earthquake. There was evidence of permanent ground movement in this area; expansion joints at some abutments showed permanent openings of up to 50mm, and open cracks in roadway pavements were of the order of 10 to 20mm. The bridge consists of eight spans supported on single tall columns, and has a substantial curvature. It is the upper of two bridges at this location, one of its supporting columns passing up through an opening in the deck of the I5 from another road below. It is constructed without joints along its length, longitudinal movements being accommodated only at the abutments.

The bridge appears to have performed very well in the earthquake, despite undergoing substantial displacements. A section of detached handrail at the west end of the bridge suggested that the deck had moved at least 200mm relative to the abutment during the earthquake, and this had caused some minor spalling of the abutment. Some pounding damage occurred at the location where a column passed through the deck of the I5 suggesting that the clearance of approximately 100mm around the column was insufficient to cope with the level of seismic displacements sustained by the two structures.

5.3.3 Gavin Canyon Bridge

This bridge, constructed in 1955, carried the Golden State Freeway (I5) over a minor road 5km north of the I5/Highway 14 interchange. The ground motions in this area are thought to be of the order of 0.6g (Yashinsky, 1994). The northbound and southbound lanes were carried by separate, parallel structures. The box girder decks were placed at a very large skew of 66°, and were supported on flared twin-column bents. The structures had a total of five spans. The central section of each bridge consisted of a single-span deck with short cantilever overhangs at each end. End segments were two span decks built into the abutment at one end and supported on the central section cantilever at the other, and continuous over intermediate columns. The two central bents had columns approximately 20m in height while the columns adjacent to the abutments were of the order of 10m (Figure 5.5).

Each joint had a seat width of just 200mm, and had been retrofitted in 1974 with five sets of restrainers, each set consisting of one galvanised rod, probably 38mm in diameter, and a 7-strand prestressing tendon passing though cored holes in the end diaphragms. The cable restrainers were fitted as part of the CALTRANS seismic retrofit programme in the aftermath of the 1971 San Fernando earthquake. CALTRANS has since abandoned this restrainer design, preferring to use smaller cables which impose less severe loads on the diaphragms (Yashinsky, 1994). All the retainers were oriented in the span direction, thus affording little resistance to movements normal to the joint.

Both of the twin bridges suffered loss of span failures at the expansion joints, leaving the central span intact (Plate 5.5). The large skew and the fact the other deck segments were each supported on only one bent almost certainly gave rise to very large torsional movements during the earthquake, causing relative motion between the deck segments at the skew joints, and hence loss of seating. The longitudinal restrainers would have provided little resistance to this motion, and appeared to have failed by punching through the end diaphragms as the outer deck sections fell; intact restrainers with bearing plates still in place could be seen hanging from the undamaged central section of the deck. This is thought to be the only bridge in which loss of seating was the primary cause of failure.

5.3.4 State Highway 118

State Highway 118 (H118) was the closest freeway to the epicentre and as a consequence suffered very heavy damage during the earthquake. Damage was evident along the whole length studied, from the I210 interchange to the Balboa Boulevard overbridge. Four of the structures visited will be considered here noting differences and similarities in failure modes. Although the direction of the freeway changes along its length the two carriageways will be referred to as eastbound (on the south side) and westbound (on the north side) for simplicity.

(a) Interchange between H118 and I405

Although none of the structures collapsed at this interchange there was significant damage which had to be repaired before the roads could be reopened. Firstly, there was damage to hinges on flyovers similar to that at the I5/Highway 14 interchange, though there was less evidence of pounding. Secondly, fill material had settled behind several bridge abutments causing damage to approach slabs which dropped below the level of the bridge deck. This was being repaired by mud jacking - pumping cement grout under the approach slabs to lift them back up to the level of the bridge deck.

Thirdly, the column closest to the west abutment of the ramp connecting I405 north with H118 west had been damaged. there were diagonal shear cracks running down two faces and considerable spalling of concrete cover (Plate 5.6). The direction of the cracking indicated flexural shear type behaviour. The reinforcement cage for this column was intact and the spalled areas were to be repaired with sprayed concrete. Several factors are worth noting about this column. Firstly, there is a cutting for a local road between it and the adjacent abutment, the column itself being built on top of the embankment. Secondly, the column is flared at the top, although the cracking occurred at right angles to the flare indicating that shaking was predominantly along the bridge. This should be compared with the behaviour of the columns at Mission and Gothic (see below) where the orientation of the flares was a key factor determining the behaviour of the columns. Finally, comparing with the I5/H14 interchange described above, it is again the shortest column, adjacent to the abutments, that has suffered most damage.

(b) Bull Creek Canyon Channel

Highway 118 is carried over Bull Creek by two adjacent structures, one for each carriageway (Figure 5.6). The crossing was designed and built between 1969 and 1976. Both abutments are skewed, but at different angles, making the westbound bridge shorter than the eastbound. The close proximity of a junction means that both bridges are wider at the east abutment than at the west. The bridges are supported on two column bents. The columns in the eastern bent, which has an extra column because of the tapered deck, are built into the wall of the channel. The prestressed concrete bridge deck is a multicell box and is integral with the abutment walls.

During the earthquake, the two bridges separated, the eastbound carriageway moving southwards. All nine columns in the eastern bent failed just above the channel wall (Plate 5.7). As a result, the deck developed a large sag, with some evidence of cracking in the parapet wall, and the eastern abutment wall tilted inwards, rotating about its base. Only two of the columns failed in the western bent, those at the southern end. In contrast to the eastern bent these failed at the top where there was extensive cracking of the soffit.

Several comments can be made from these observations. Firstly, building the eastern columns into the channel walls had the effect of shortening them, making them more likely to fail in shear. This was exacerbated by the reinforcement detail; additional reinforcement was specified in the regions just below the soffit and just above the footings where maximum combinations of shear force and bending movement were expected. However, from the drawings it does not appear that additional reinforcement was included above the interface with the wall. Secondly, the layout of the bridge would have influenced its dynamic response. The skew and eccentricity of the centres of mass and resistance would have promoted a torsional response about the vertical axis. Moreover, the skewed and tapered abutments would only allow movement towards the south. Finally, as there was no tie between the foundations along the bridge, any differential movement between them had to be absorbed by flexing of the deck or columns. This led to failure being concentrated in the weakest link, that is the columns.

(c) Mission and Gothic

As at Bull Creek, H118 was carried over the intersection of San Fernando Mission Boulevard and Gothic Avenue by two adjacent structures, one for each carriageway. Again both the abutments were skewed (Figure 5.7) but in this case the taper was symmetrical and almost exactly 90 degrees to align with the road grid below. There was consequently a much greater difference in length between the structures carrying the eastbound and westbound carriageways. The bridges were prestressed concrete, multicell box girders designed and built between 1973 and 1976. They were continuous over interior twin column bents with movement taken up at the abutments. The westbound bridge was supported on two interior bents and the longer eastbound structure on three. The orientation of the bents varied depending on location, the eastern bents in each case aligned more closely with the eastern abutment. The columns were octagonal in cross-section, the width between opposite faces being 1.8m, and the heights varying between 6.5 and 7m. Heavy reinforcement was provided, with forty five 35mm diameter longitudinal bars and 16mm spirals spaced at 89 mm centres. The columns had one-way flares from around their mid-heights, the flares oriented along the line of the column bent; it is thought that the flares were not intended to affect the strength or mode of failure of the columns. The columns were effectively pinned at their feet.

The earthquake caused collapse of the eastbound and severe damage to the westbound bridge. In the westbound structure, the columns in bent 3L failed by formation of flexural hinges at the bottom of the flared section. The large deformations caused fracture of the spiral reinforcement and consequent buckling of the main steel (Plate 5.8). Failure of these columns caused a large sag to develop in the deck. However, the amount of sag appeared to exceed the shortening of the columns due to hinge formation. This could indicate an associated failure of the foundations, though the footing detail suggests that rotation of the column base was more likely. The columns in bent 2L were also damaged, but they behaved as intended with ductile flexural hinging leading to spalling in the flared region (Plate 5.9). These observations suggest that the flare altered the column failure mode for loading in one direction, but not the other. For loading along the line of the flare, the flare may have resulted in a substantial increase in flexural strength at the column top, causing hinging to occur lower down. For loading normal to the flare, the hinge occurred at the top. The failures of the various columns were therefore governed by the orientation relative to the direction of strongest ground motion.

As with the Bull Canyon bridge the mode of failure seems to have been influenced by the deck layout. The heavily tapered plan led to an eccentricity of the centres of mass and stiffness, and the shape of the abutments would only allow the deck to move in a southerly direction. This ties in with the noted failure patterns in the columns of the westbound bridge, where those in bent 3L, with flares aligned approximately north/south failed totally whereas those in bent 2L remained standing. It also helps explain why the eastbound structure suffered greater damage. Again, one column bent (4R) was oriented differently to the rest, and so suffered much greater damage. However, due to the bridge layout, there were both greater forces acting on this structure and less resistance to it moving. Consequently, the shear key at the eastern abutment failed and the structure came off its seating. The remaining columns were damaged and the bridge was left inclined at an angle supported only by the western abutment and the remains of column bents 2R and 3R.

(d) Balboa Boulevard Overbridge

This four-lane bridge was designed in 1973 and built in 1976. It consists of two spans of 34m and 45m, supported on abutments and continuous over a central twin-column bent. There are high retaining walls at the north abutment. The octagonal columns are 1.5m in diameter and approximately 15m tall, with one-way flares at their tops. Transverse reinforcement consists of 16mm spirals at 75mm centres. The area surrounding this bridge was badly damaged, with evidence of large soil movements. A few hundred metres to the north of this bridge a large steel water main and a gas main had fractured.

Aside from a small amount of spalling at the tops of the columns the bridge did not suffer any noticeable structural damage. At the south abutment a water main had burst undermining the abutment and approach paving, exposing the cast in place piles. This had not caused any permanent structural damage. At the north abutment Styrofoam packing between the deck and side wall had pulled out by 130 to 150mm, suggesting a movement which would be compatible with the spalling at the tops of the tall columns. This would indicate that displacements of the columns were large enough for the columns to reach their ultimate movement capacity, but because of their slenderness they did not fail in shear, and so remained serviceable. There was also some movement of the ground on the north side of the freeway, with obvious settlement behind the retaining walls and on the slope below the abutment wall.

5.3.5 Interstate 10

The I10 Santa Monica Freeway is reputedly the busiest road in the USA, with a traffic flow of 300,000 vehicles per day. The La Cienega/Venice overpass was designed in 1962, constructed in 1964 and was scheduled for retrofit in 1994/95 (Civil Engineering, 1994). The eight-lane bridge consisted of nine spans of approximately 30m each, with expansion joints in the third and sixth spans (Figure 5.8). Seat widths at the expansion joints were 150mm; the joints had been retrofitted with 32mm high strength bar restrainers in 1978 (CALTRANS, 1994).

The 1.9m deep multi-cell box girder deck was supported on multi-column bents having circular columns 1.2m in diameter, with heights varying in the range 4.5 to 6m. Orientation of the bents varied gradually across the structure to accommodate the skew of 41° at the east abutment. Column longitudinal reinforcement levels varied substantially, but the transverse reinforcement always consisted of 12.7 mm hoops at 300mm centres. In addition to the main bridge structure, the complex included two elevated access ramps on the south side, one having a very high curvature.

The bridge was located in an area thought to contain soft soil deposits (La Cienega is Spanish for The Swamp), which may have caused amplification of the ground motions - there was significant damage to local buildings. This possibility was supported by inspection of a structurally similar bridge a few hundred metres away, which appeared to have suffered no damage.

The earthquake caused failure of nearly all columns in bents 3, 4, 5 and 6 and loss of seating of the westbound section at the movement joint adjacent to bent 6. As a result, the deck dropped approximately 1.5m, and ended up resting on a number of masonry lock-up garages situated below it (Plate 5.10). The structurally separate access ramps also suffered damage, including damage to supporting columns. The column failures appeared to be either brittle shear failures, or more ductile flexure-shear failures. The location of the failure varied form column to column, with damage at top, bottom and mid-height observed. The importance of column height was further emphasised by observations of three columns located very close to one another (Plate 5.11). The short column had suffered severe shear cracking, the intermediate length column showed a shear failure but the damage was less, while the longest column showed only spalling at the top.

Some simple order-of-magnitude calculations were performed for the columns of this bridge (Anderson et al, 1994). The horizontal displacement at the top of a column corresponding to formation of plastic hinges at each end was found to be of the order of 25mm, the exact value depending on the longitudinal steel ratio. The shear corresponding to the formation of plastic hinges at each end was of the same order of magnitude as the shear capacity, assuming the full concrete shear strength. Any rotational ductility demand during the earthquake would tend to reduce the shear strength; using a reduced value for the concrete shear strength, the column capacity was found to be well below the shear demand.

The calculations therefore show good agreement with the observations of damage, with some columns suffering brittle shear failures and others undergoing shear failure at low ductility. Given the very low displacements required to cause column failure, it is likely that this was the primary cause of collapse, with the observed joint failure caused by the resulting large deck movements. With the exception of this one joint failure, the restrainers in this bridge worked well and kept the superstructure together.

Severe damage was also sustained by the I10 underbridge crossing Fairfax and Washington Streets, about 2km east of the La Cienega/Venice structure. This structure was demolished extremely quickly and was therefore not inspected by EEFIT. However, reports suggest that its age, form of construction and mode of failure were all similar to those of the La Cienega/Venice bridge.

5.3.6 Composite bridges over Interstate 5

The overwhelming majority of bridges in the Los Angeles area are concrete structures, predominantly prestressed box or multicell box girders. All of the major damage occurred in bridges of this type. The small number of other types of bridge generally performed well, although some damage was sustained. Two composite bridges were visited - crossing the I5 close to Santa Clara to the north of the region most badly affected by the earthquake. Both bridges had moved during the earthquake and were displaced significantly from their correct alignment. However, even though they had moved such a long way, both bridges were still in use, in contrast to the concrete bridges described above.

The Peco Lyons Bridge has a single span which is curved in plan. The bridge has been widened since it was first built, with an extra carriageway being added. The original bridge was supported on rollers whereas the new section was supported on rubber pads. During the earthquake the bridge moved considerably on its bearings resulting in damage to holding down bolts for the new section and to the rollers themselves for the older section. The steel girders were connected by diagonal bracing above the abutments and this failed in a regular pattern. On the southern side of each bay the gusset plate had buckled, indicating failure in compression (Plate 5.12), whereas on the northern side the bolts had been torn out of the gusset (Plate 5.13) indicating failure in tension. This pattern was repeated across every bay.

The second bridge was a smaller single carriageway structure situated a few kilometres north of Peco Lyons. There were two spans continuous over a central reinforced concrete column. As with the Peco Lyons bridge this structure was a long way out of position (Plate 5.14), at one end by as much as 200mm. Both ends had displaced in the same direction indicating that the central column was bending rather than twisting. As with the older section of Peco Lyons, this bridge was supported on roller bearings - these had moved sideways causing considerably damage. However, the full displacement of the structure arose from movements of the abutments as well as the bearings.

5.3.7 Steel bridge over Santa Clara River

One steel bridge was inspected; this carried an old highway over the Santa Clara river. This five span truss bridge had been closed to traffic after the earthquake. Each span was simply supported, with roller bearings at one support and rockers at the other. As with the composite bridges this structure showed signs of considerable movement, both longitudinally and transversely where it was some 200mm out of alignment. However the most serious damage was failure of the K bracing connecting the trusses above the piers. At each support the lower chord and one of the diagonals had buckled, a type of failure which is typical for K braced structures.

5.4 Retrofitting

One of the more important aspects of this earthquake from an engineering point of view was the opportunity it gave to assess the performance of retrofitted bridges. Several types of retrofitting have been applied and these shall be considered in turn.

5.4.1 Retrofitting of hinges

The retrofitting of hinges was introduced following the large number of loss-of-seating collapses observed in the 1971 San Fernando earthquake. Three types of retrofitting have been applied to the hinges at the various sections of the highway network:

(a) Hinge extenders were added to some of the older hinges where the seat width was considered too narrow. These usually took the form of steel tubes passing through the half joint allowing the joint to separate completely without loss of bearing.

(b) Horizontal cable restrainers have been installed across most hinges to stop the two halves of the joint moving apart. Groups of cables were passed through the hinge and tied back to the diaphragm above the nearest pier. This procedure was very successful as only one collapse can be directly attributable to hinge separation. In all but the very earliest retrofits (such as the Gavin Canyon Bridge, see section 5.3.3) the restrainers were designed so that the cables would break before damage to the structure occurred. In most cases this was achieved, for example the cables broke in several of the hinges at the I5/H14 interchange. There were a few isolated cases where the desired failure mode was not achieved, one being the I10/I405 interchange, where the diaphragm failed before the cables and the bridge will need to be rebuilt.

(c) Vertical cable restrainers have been installed more recently to prevent vertical motion across the hinge. Again these were reported to have performed well in the earthquake.

5.4.2 Retrofitting of columns

Reinforced concrete columns have been the subject of a concerted retrofitting campaign in recent years; over one hundred bridges in the Los Angeles Basin now have column and/or footing retrofits. The case histories presented above, in which column failure was by far the most common cause of collapse, clearly demonstrate the need for more such retrofits. The collapse of part of the I10 at La Cienega Boulevard was especially pointed as the columns here were due to be retrofitted within months.

The prime weakness in older bridge columns is the inadequate provision of shear and confining reinforcement; retrofits have therefore been designed to increase the shear resistance and flexural ductility of the columns. Several methods of achieving this have been used, the method chosen depending on the type of column, its location and the as built reinforcing details. It is important to note that not all columns on a bridge would be chosen for retrofitting, usually only the shorter columns would be treated, in keeping with discussions above, and those that formed a single column bent. Of the methods employed the most common is jacketting of the columns in a steel shell, usually circular or elliptical in cross section. Variations on this include casing the whole column, the top and bottom thirds or just the top third. Usually grout is introduced between the shell and the column, but in some cases a slip surface is used to avoid increasing the flexural stiffness. An alternative method is to build walls between two columns in a pair so that they are braced together.

Jacketted columns inspected by the team appeared to have suffered no damage in the earthquake. However, one concern about this type of retrofit is that damage to the column within the jacket would not be externally visible. CALTRANS therefore undertook a further survey, in which jackets were removed from several columns in areas where ground accelerations were thought to have been high. In all cases, the interior concrete was undamaged.

5.4.3 Retrofitting of composite bridges

The behaviour of the two composite bridges visited showed that there was considerable movement at the bearings, especially those with rollers. On one bridge, Peco Lyons, attempts had been made to limit this movement by adding steel plates which fixed the lower flange of the steel girders to the abutment wall. Unfortunately all of these retrofit details failed during the earthquake (Plate 5.15), the connection to the concrete wall being unable to resist the lateral shear.

5.5 Post earthquake actions

As well as considering the performance of individual bridges during the earthquake it is also important to look at the response to the event of local authorities and engineers. CALTRANS, who are responsible for the freeway bridges in the area, reacted very impressively to the disaster. Within a few hours of the earthquake CALTRANS engineers were on site assessing the safety of the bridges. This was an enormous task as there were thousands of bridges to inspect and many had to be reinspected following aftershocks. Within twenty four hours additional teams of engineers had been flown in from the headquarters of CALTRANS in Sacramento.

In contrast to the aftermath of the Loma Prieta earthquake, CALTRANS moved very quickly to clear collapsed or severely damaged structures. Three days after the earthquake the I5/Highway 14 interchange and the I10 collapse at Fairfax and Washington had been cleared completely, and demolition was underway at all of the other major bridge collapses. Within weeks the design team, based in Sacramento, had produced designs for replacement structures. All this activity illustrates the high level of disaster planning that CALTRANS had put in. A few specific aspects of the post earthquake response are considered in more detail below.

5.5.1 *Rerouting of traffic*

With several major highways closed, rerouting of traffic represented a major problem. Several solutions were found depending on the location of the closure and the type of traffic using it. For the I5, damaged at Gavin Canyon and at the I5/H14 interchange, temporary relief roads were installed following the lines of the old highway. For H118 the westbound carriageway was propped and used to carry traffic whilst the eastbound was demolished and rebuilt. Traffic could then be swapped onto the new carriageway and the other side rebuilt. On the I10, a one way system was installed in the blocks surrounding the damaged structures so that traffic could be fed down from the freeway and through side streets with as little congestion as possible. Of course this led to considerable local congestion and tailbacks. Nevertheless, traffic congestion over the affected region in the days immediately following the earthquake was not severe, perhaps partly because many people followed the authorities' advice and stayed at home.

5.5.2 *Rapid reconstruction of bridges*

Reconstruction of most of the damaged bridges in the region proceeded extremely quickly, providing a major boost to the city's shattered self-confidence. The most celebrated example was the I10 Santa Monica Freeway, which reopened to traffic on 12 April 1994, just 84 days after the earthquake. Contractors worked around the clock to ensure early completion, spurred on by a bonus of $200,000 for every day cut from the original deadline of 24 June 1994. Considerable time savings were made by using deck formwork capable of carrying traffic loads.

Another example of very rapid reconstruction was the bridge carrying the I5 over Gavin Canyon, a few miles north of the interchange with H14. Within ten days the old bridge had been completely demolished, the old road through the valley floor temporarily widened, the new bridge designed and the contract for its construction let. This contract was to run for 130 days and included bonuses and penalties of $135,000 a day for early or late completion as appropriate. The importance of the I5 as the major north-south link in California explains the great effort to replace the structure quickly, and the large penalties and bonuses on offer.

5.5.3 *Changes to design practice*

While the earthquake caused damage which was mostly predictable, it has nevertheless resulted in some changes to CALTRANS bridge designs. New bridges are designed with an increased emphasis on simplicity and regularity, particularly with regard to column lengths and stiffnesses, and efforts are made to avoid skew spans wherever possible. Expansion joints are used as infrequently as possible, typically at 300m intervals. Where joints are used, support is provided by pairs of columns positioned on either side of the joint, so that even if there is complete loss of seating there will be no collapse.

5.6 Conclusions

1. Bridges generally performed well in the Northridge earthquake, with only a very small proportion of structures collapsing or experiencing severe damage. Nearly all of the damage occurred in older bridges which had not been designed to modern seismic standards and had not been upgraded in the most recent retrofit programme.

2. While it is often difficult to be certain of the exact sequence of events which led to collapse, it is clear that most of the bridge damage was caused by the failure of short, stocky columns. Occasionally these were non-ductile flexural failures due to inadequate confining reinforcement. In most cases, however, the columns failed in shear, either before reaching their flexural yield capacity, or at very low rotational ductilities. In some cases the lack of shear capacity was exacerbated by the failure to take account of features such as column flares or restraints from adjoining walls, which tended to increase the applied shear forces.

3. With very few exceptions, hinge restrainer retrofits succeeded in preventing loss of seating at expansion joints. Where large joint movements did occur, they were in bridges with very large skews and/or very early retrofit designs. However, widespread minor, mostly repairable damage was noted due to relative movements at deck joints and abutments.

4. Columns retrofitted with steel jackets performed extremely well, with no reported instances of damage.

5. The principal changes in Californian design practice since the earthquake have been greater efforts to avoid variations in column length over a structure, the avoidance of skew spans wherever possible, and improvements in support details around expansion joints.

5.7 References

D L Anderson, M S Williams and D W Kennedy (1994) "Performance of bridges in the Northridge earthquake", in Preliminary Report on the Northridge, California Earthquake of January 17, 1994, Canadian Association for Earthquake Engineering, Vancouver, 110-128.

CALTRANS (1994) "The Northridge earthquake", Post Earthquake Investigation Report, CALTRANS, Sacramento Ca, 146pp.

Civil Engineering (1994) "Northridge earthquake", ASCE Civil Engineering Magazine, March, 40-47.

M J N Priestley, F Seible and C M Uang (1994) "The Northridge earthquake of January 17, 1994: damage analysis of selected freeway bridges", University of California, San Diego, Report No. SSRP-94/06, 260pp.

M Yashinsky (1994) "The performance of seismic retrofits in the Northridge earthquake", CALTRANS Office of Earthquake Engineering, Sacramento, 23pp.

Figure 5.1: Locations of major highway bridge collapses

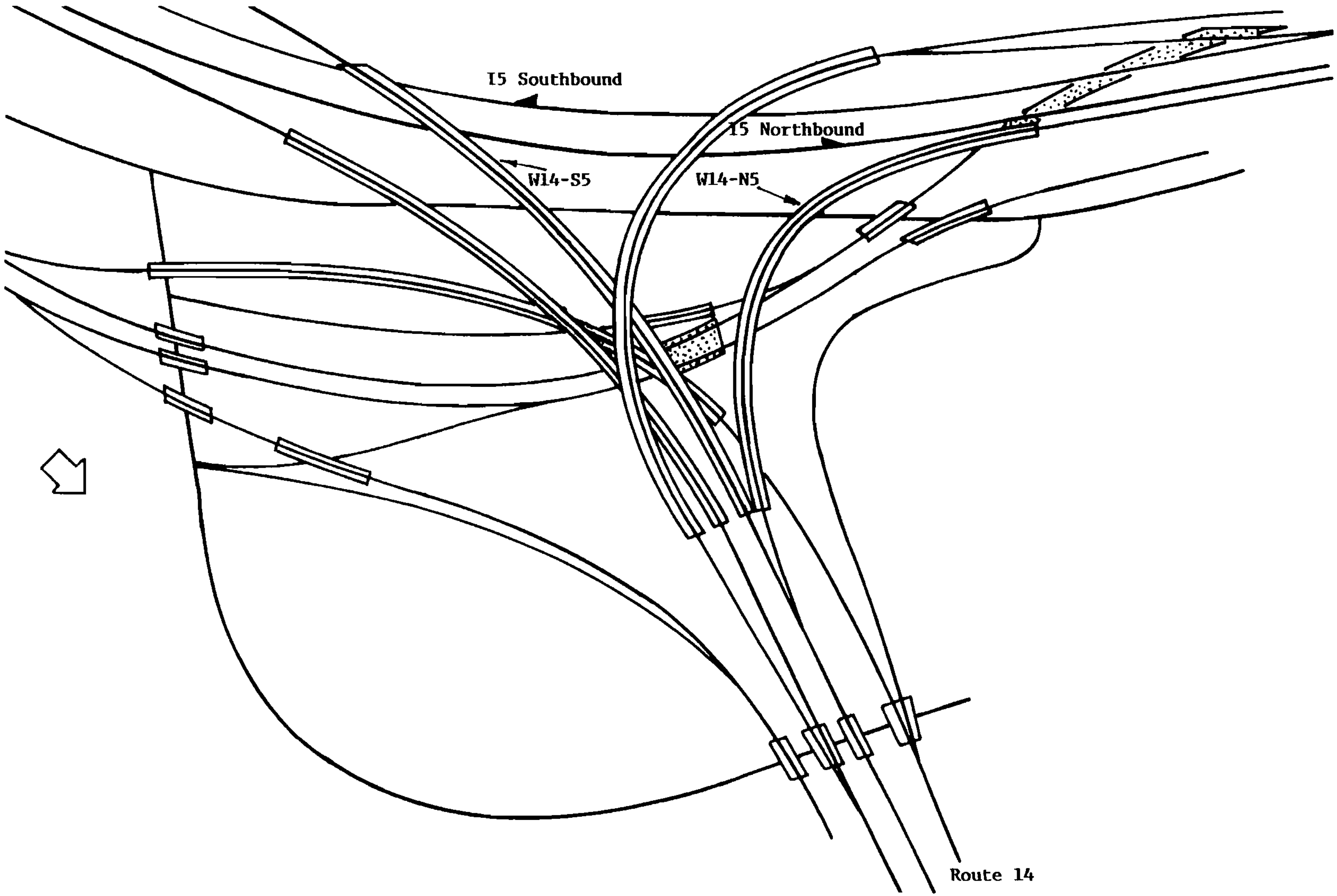


Figure 5.2: Plan of Interstate 5/Highway 14 interchange

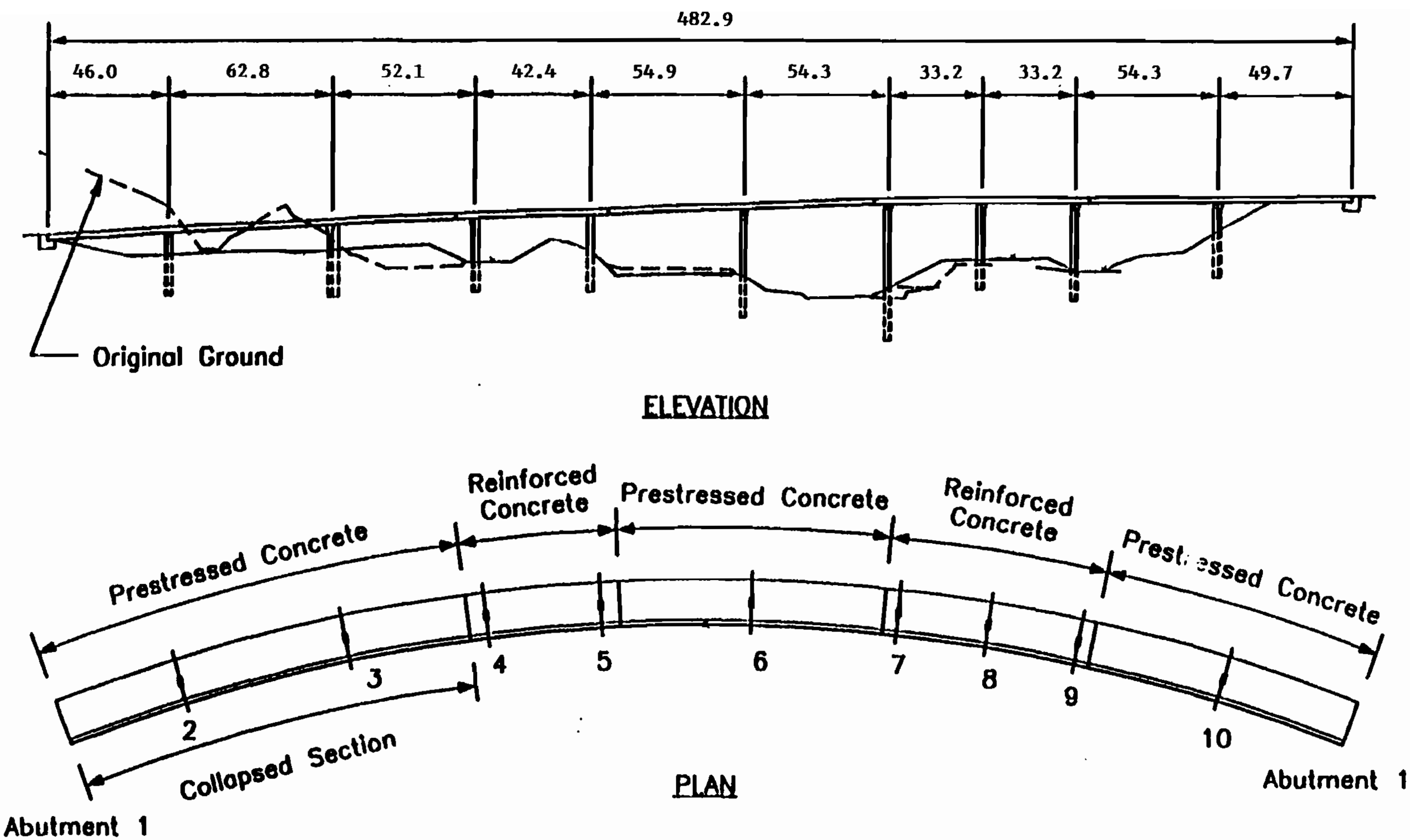


Figure 5.3: Plan and elevation of the W14-S5 connector at the I5/H14 interchange

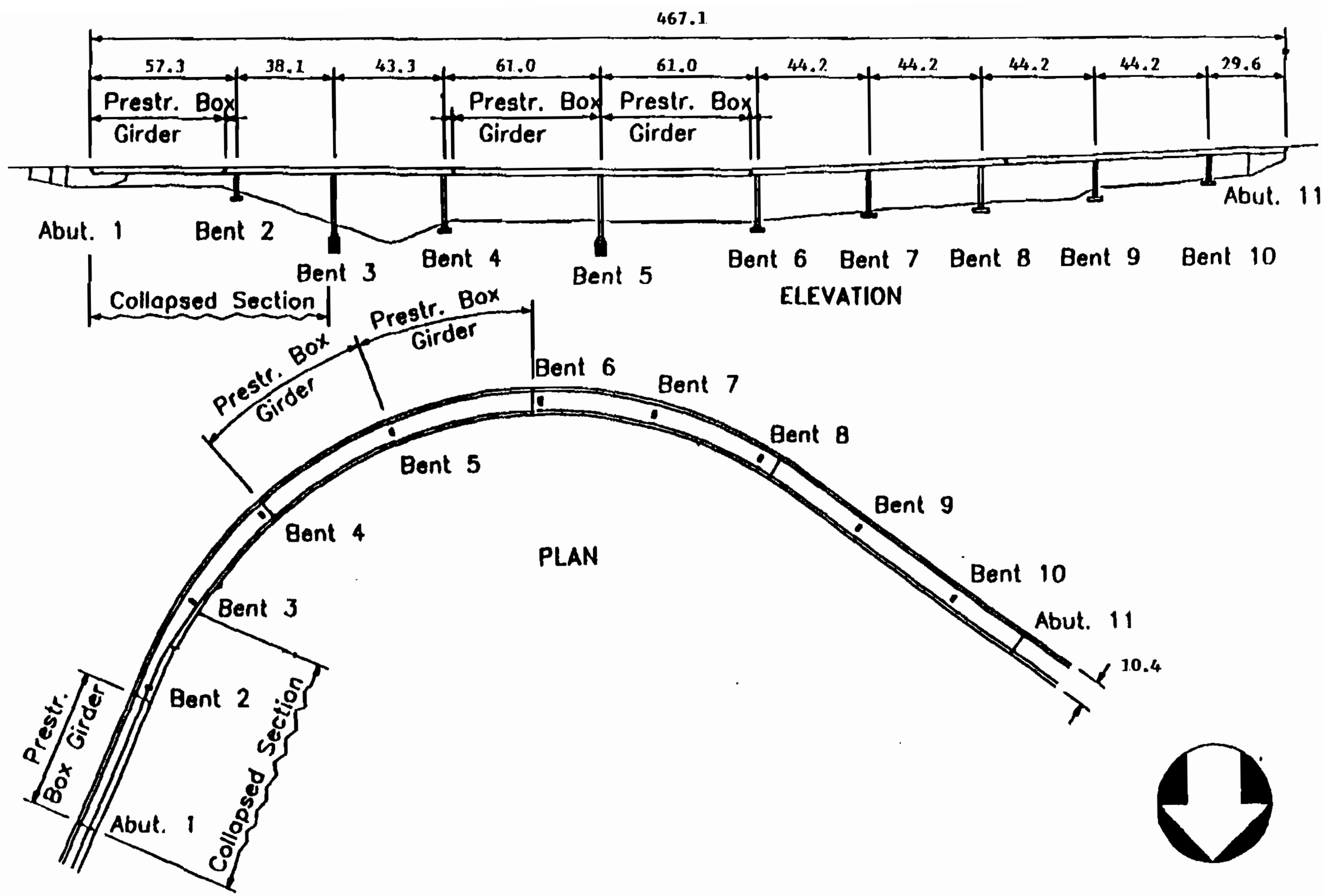
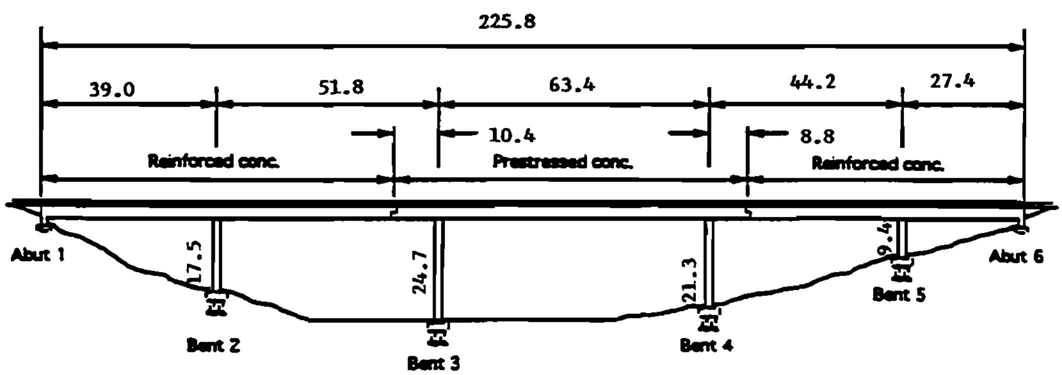
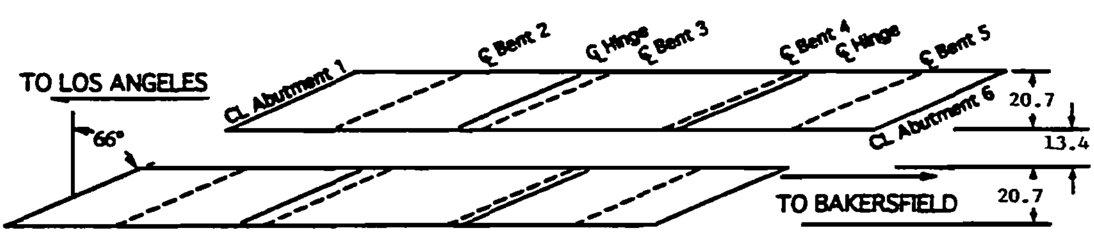


Figure 5.4: Plan and elevation of the W14-N5 connector at the I5/H14 interchange



ELEVATION IN THE DIRECTION OF SKEW



PLAN

Figure 5.5: Plan and elevation of Gavin Canyon Bridge on Interstate 5

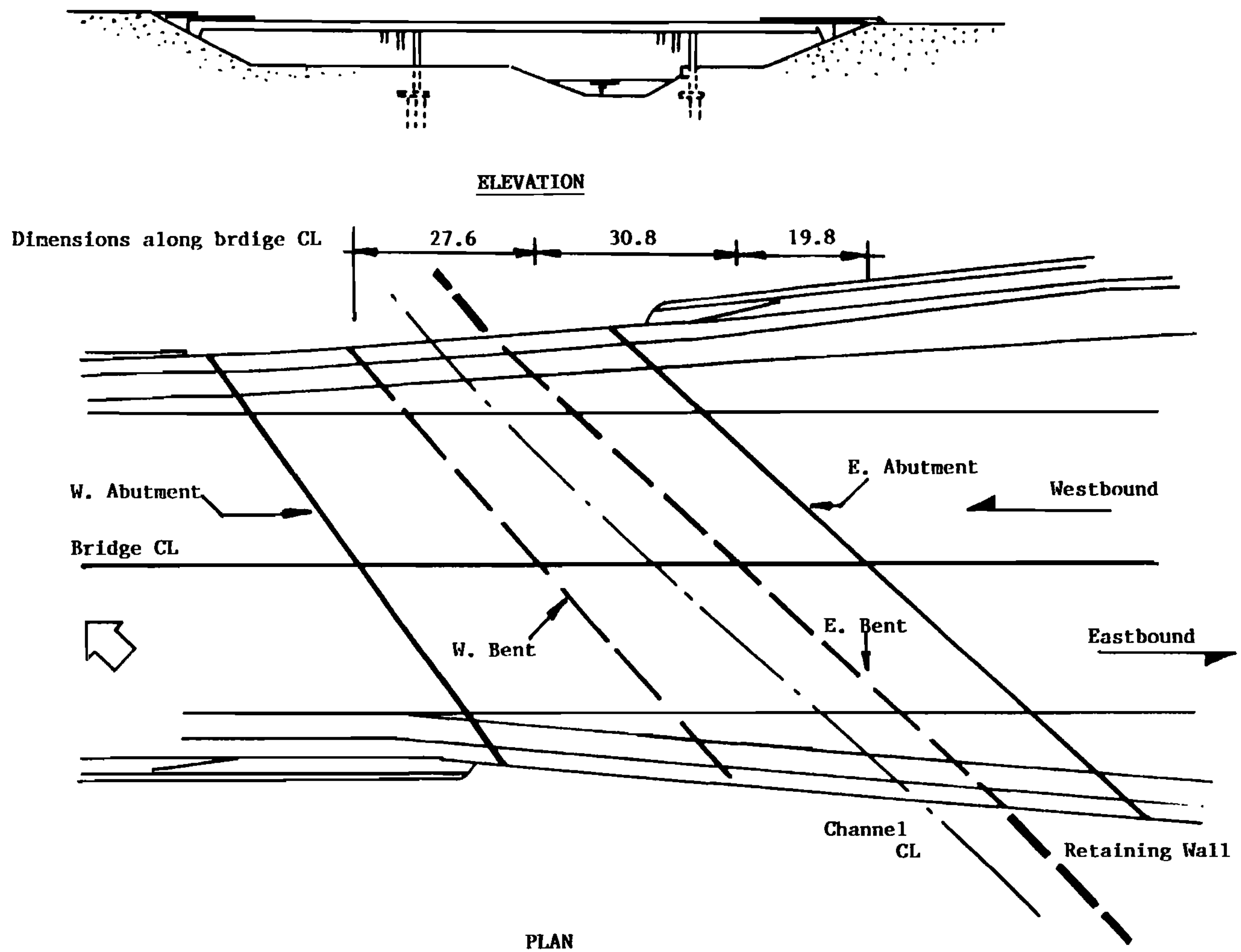


Figure 5.6: Plan and elevation of Bull Creek Canyon Channel Bridge on Highway 118

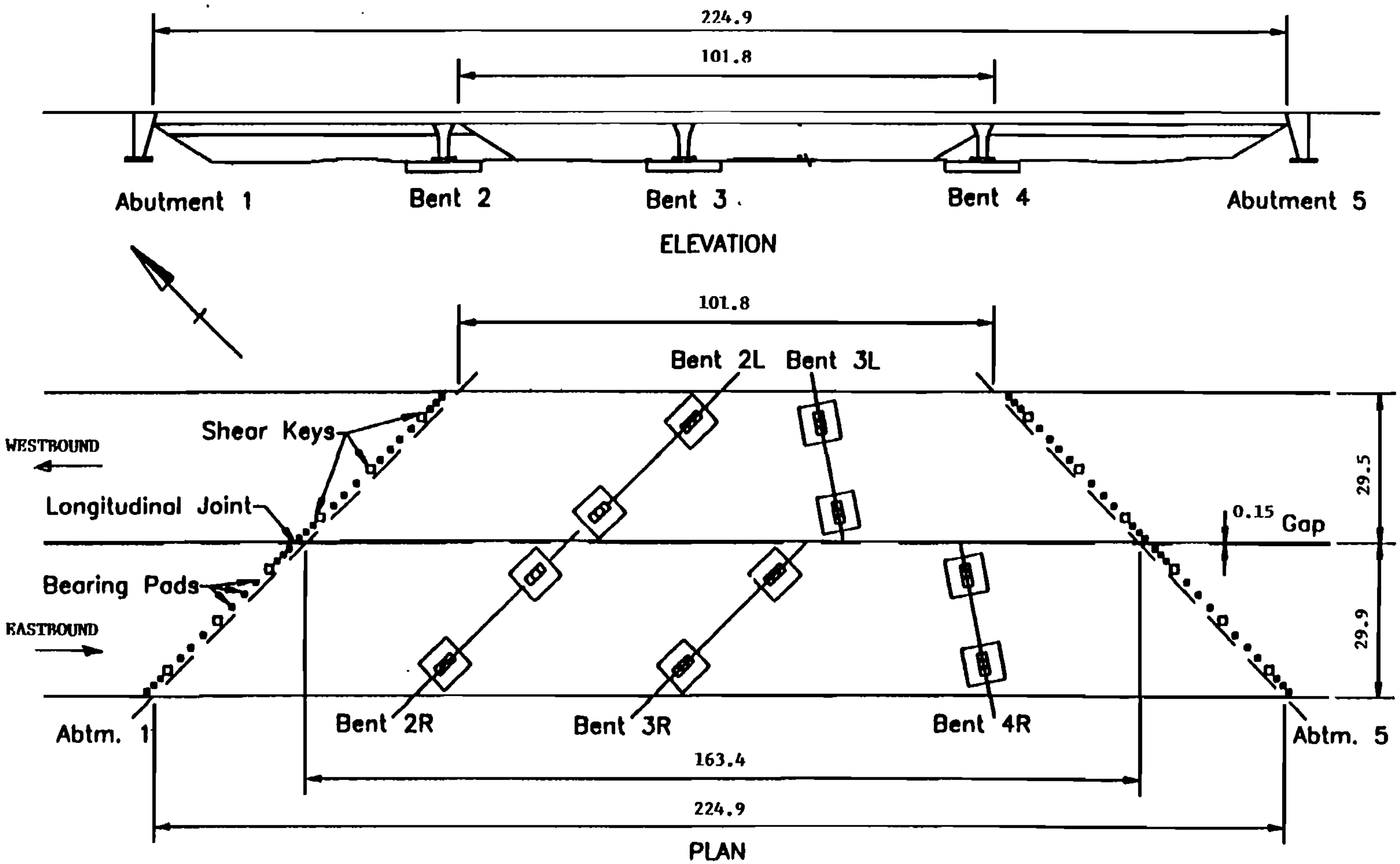


Figure 5.7: Plan and elevation of the Highway 118 undercrossing at Mission and Gothic

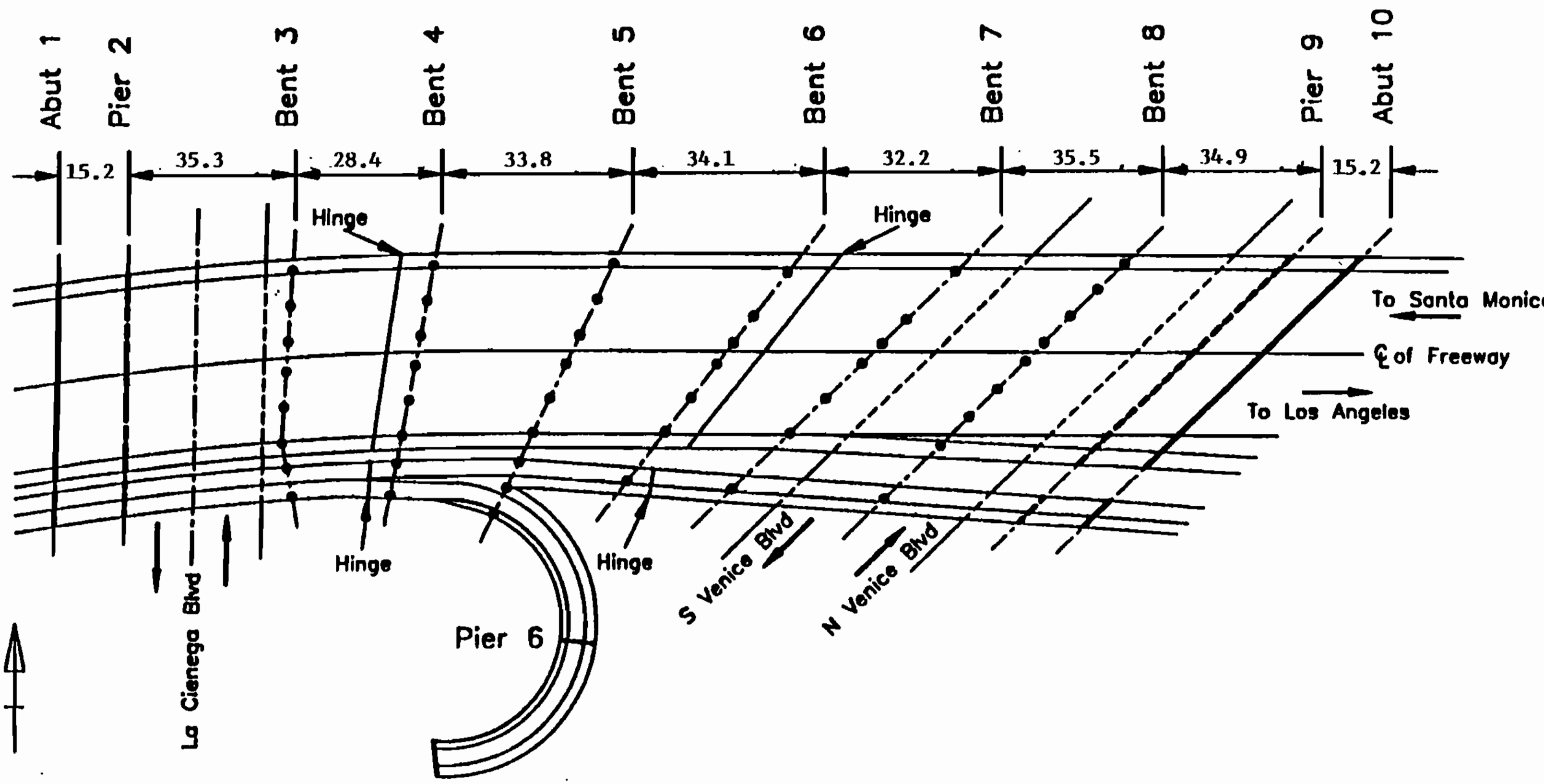


Figure 5.8: Plan of the Santa Monica Freeway (Incrstate 10) undercrossing at La Cienega and Venice

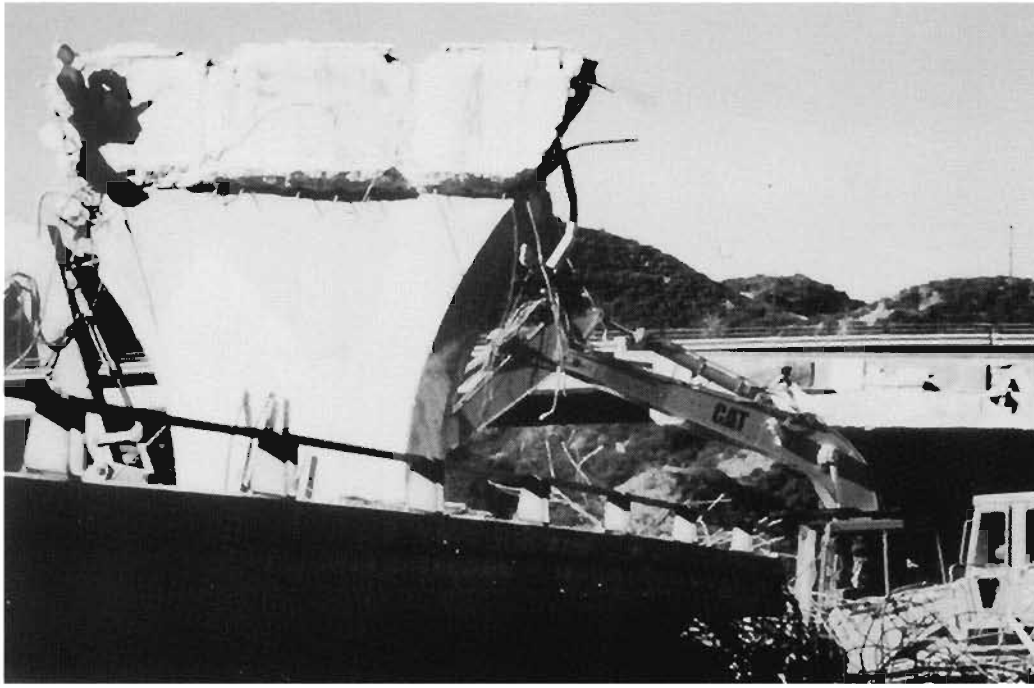


Plate 5.1: W14-S5 connector - collapse of deck at Bent 3

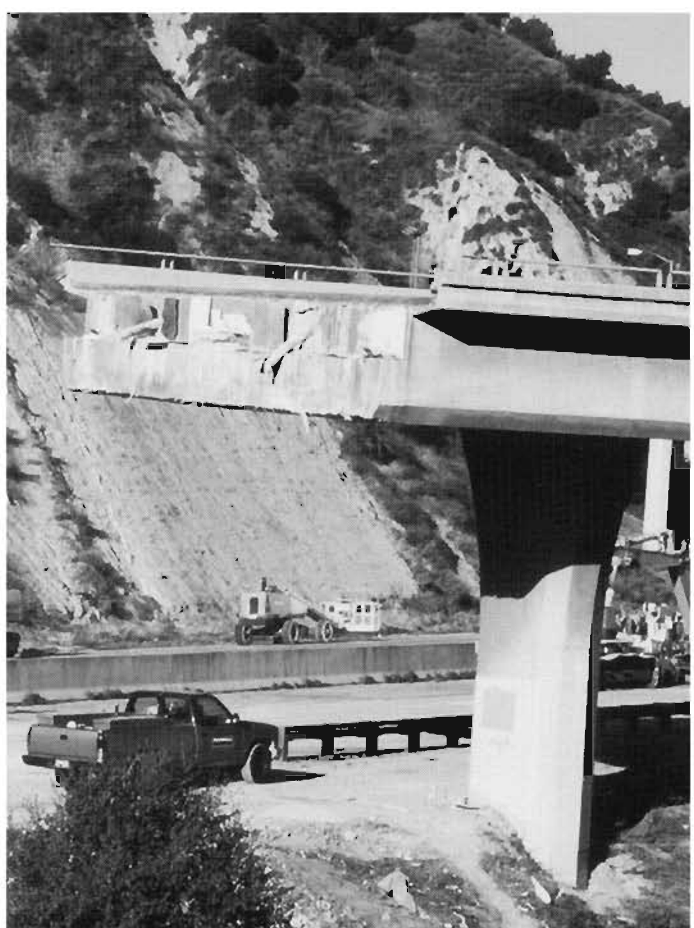


Plate 5.2: W14-S5 connector - view of intact deck section at Bent 4
after unseating of adjacent section

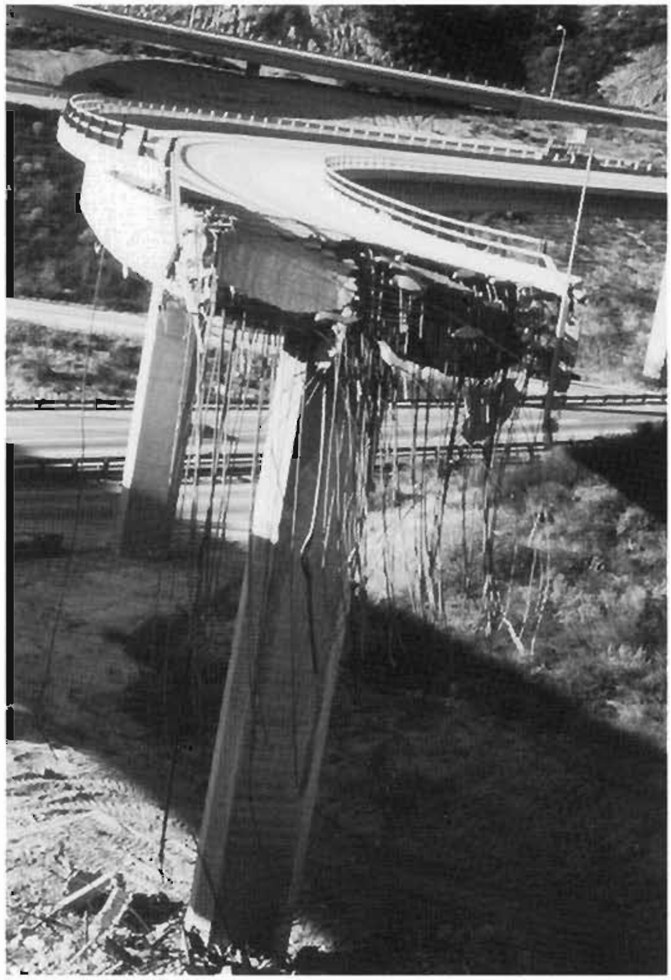


Plate 5.3: W14-N5 connector - bending failure of deck at Bent 3
(initiated by column shear failure elsewhere)

Plate 5.4: S5-E14 connector - damage to abutment wall

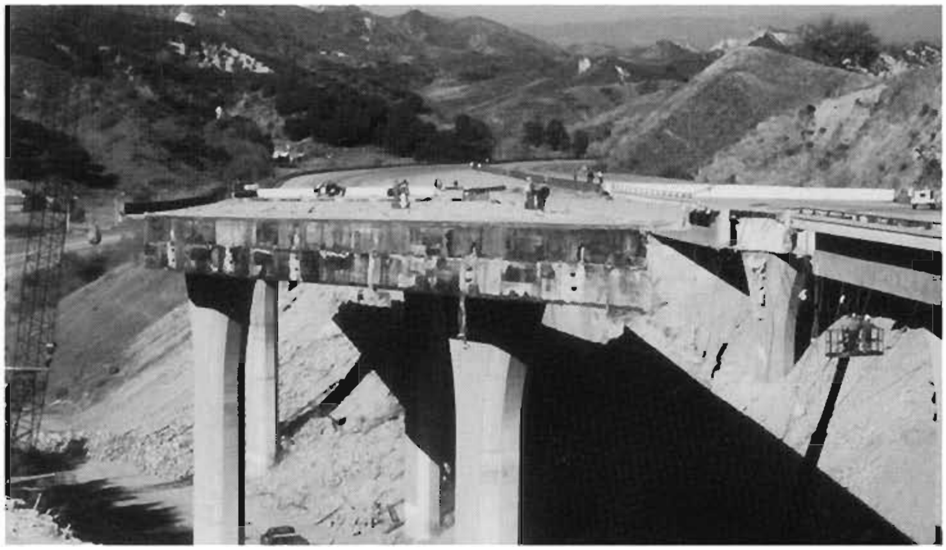


Plate 5.5: Gavin Canyon Bridge - view of intact central span after unseating of side spans

Plate 5.6: I405/H118 interchange - shear damage at base of column

Plate 5.7: Column failures at Bull Creek Canyon bridge

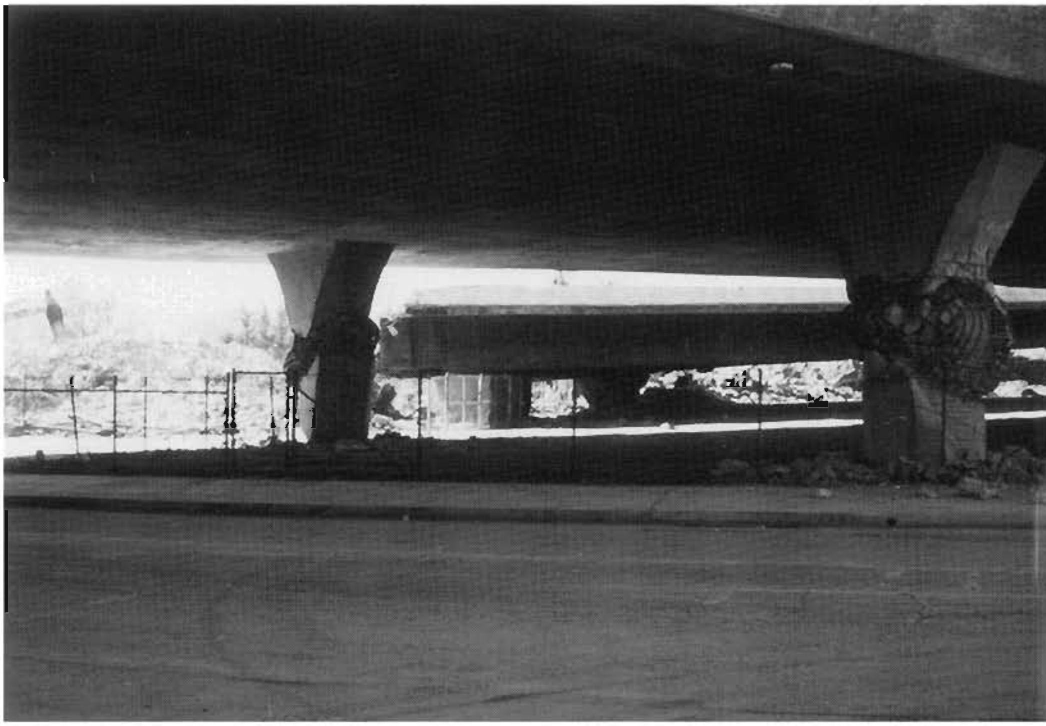


Plate 5.8: H118 at Mission and Gothic - column failures in Bent 3L

Plate 5.9: H118 at Mission and Gothic - column failures in Bent 2L

Plate 5.10: Santa Monica Freeway at La Cienega and Venice

Plate 5.11: Santa Monica Freeway at La Cienega and Venice - influence of column height

Plate 5.12: Peco Lyons Bridge - buckling of gusset plate

Plate 5.13: Peco Lyons Bridge - tension failure of cross-bracing

Plate 5.14: Lateral movement of composite bridge over Interstate 5

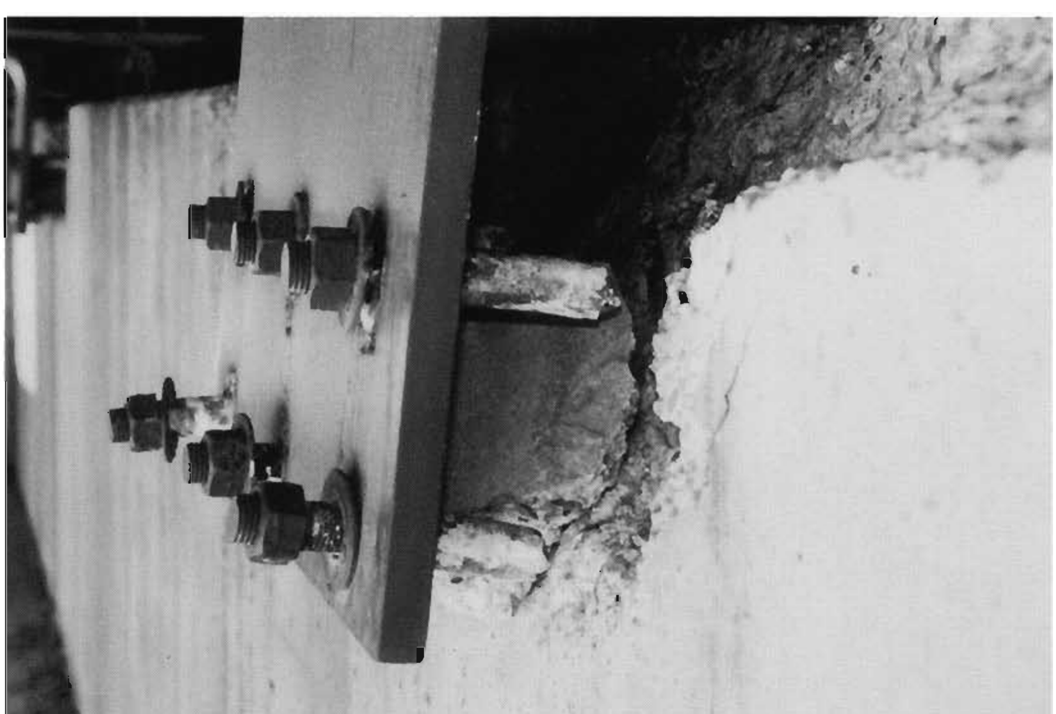


Plate 5.15: Peco Lyons Bridge - failure of retrofitted restrainer plates at abutment

6. BUILDINGS

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6.1 Introduction

6.1.1 Acknowledgements

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- William C Taylor, Taylor & Associates, Architects
- Terry Dawson, California State University, Northridge

The chapter comprises the following sub-sections:

6.1.2 General

The Northridge earthquake struck a densely populated area of Los Angeles subjecting a number of structures to ground accelerations of about 0.4g - the design value in the Uniform Building Code (UBC, 1994) for the most active part of California. The damage to the buildings mainly occurred in the northern part of Los Angeles to the north of the Hollywood Hills, but with significant damage on the coastal strip to the southwest, in particular Santa Monica, and Filmore on the high ground to the north west (see Chapter 2). The most extensive damage to structures was in the region of Canoga Park, Northridge, San Fernando and Sherman Oaks, all located in the San Fernando Valley. The California Department of Housing and Community Development reported a week after the event that there were 184 destroyed units and 5564 damaged ones.

The buildings in the area most affected include large commercial properties and large apartment blocks lining the main thoroughfares, and detached wooden houses elsewhere. There were significant administration properties affected as well as a number of hospitals. Experience from this earthquake is particularly valuable because large numbers of newer buildings were subject to ground accelerations within ±50% of the design values whilst many older buildings were sometimes subject to accelerations up to about three times their capacity assessed by more recent codes. The damage to structures in the Loma Prieta Earthquake in Northern California of 1989 (EEFIT, 1993) only provided an indication of the type of damage to be expected in the event of such an earthquake.

Damage to buildings in the area subject to the high ground motions was generally less than to bridges, the notable exception being multi-storey car parks which were often of precast concrete construction. Therefore, these merit special attention. Damage to steel structures, which was not known at the time of the visit, is summarised in 6.8 together with some comments on the seismic suitability of some US details.

The performance of structures in Los Angeles is particularly interesting for the following reasons:

1. The development of the San Fernando Valley greatly increased after 1960 and therefore most of the buildings are of recent construction.

2. The region had been subjected to a major earthquake in 1971. This had led to an extensive review of the Uniform Building Code, the main changes being the requirements to ensure ductility at plastic hinges (confinement in reinforced concrete columns, bi-diagonal bars in coupled beams and short columns) and provisions for the tying of the vertical load/lateral load resisting elements into the diaphragms at floor and foundation level.

3. The Building Regulations are more vigorously applied than in other parts of California, the threat of the "Big One" being a real concern.

4. There has been a trend to base isolate important structures in the last few years in the area. Five base isolated structures experienced large shaking, four of them being instrumented.

5. It is to be expected that this earthquake will produce an enormous quantity of valuable data, which will enable the earthquake design rules to be significantly improved. In the 1970s it appears there was a local regulation in Los Angeles that all buildings over 8 storeys should be instrumented, and whilst this rule has subsequently been relaxed due to difficulties in maintaining the equipment there is a large quantity of information available on the accelerations in these and in lower buildings. Many buildings were instrumented under the Californian Strong Motion Instrumentation Program (CSMIP) by the California Division of Mines and Geology (CDMG) to measure ground motion and structural response.

6. It is understood that FEMA, the federal emergency agency responsible for administering disaster relief, requires that photographs or videos be made of the damage before it will consider compensation. If this information is combined it is inevitable that statistics will be prepared highlighting details, whether irregularity in plan just induces local damage or affects buildings more generally and whether it affects the overall amount of energy absorbed by structures. These issues are all relevant to the rules in the recently drafted Eurocode 8.

7. There is presently a dearth of reliable information on the likely damage to buildings properly designed to resist earthquakes when subject to ground accelerations equal to or greater than the design accelerations. Present best estimates of the damage to be expected in buildings designed for horizontal accelerations between 0.1g and 0.4g using the UBC/SEAOC codes (which in some respects are optimistic)when subjected to ground accelerations of 0.4g and 0.7g are summarised in Table 6.1, together with compatible data in regards to damage indices for intensities of MM VIII and MM IX.

Throughout this report the floor numbering follows the English system in which the ground floor is at street level (the first floor in the US) and the first floor is the first floor of superstructure (the second in the US).

6.2 Base isolated structures

6.2.1 Introduction

Over the last few years there has been a gradual increase in the use of base isolation as a method of reducing the seismic loading on structures. Five seismically base isolated structures have recently been built in the Los Angeles area:

- the University of Southern California (USC) Teaching Hospital (1989);
- the County Fire Department's Fire Command and Control Facility (1989);
- the County Emergency Operations Centre (1992);
- two residential houses.

A sixth structure, the Rockwell International Information System Centre Building in Seal Beach (1989), has been retrofitted using base isolation.

Four of the above mentioned structures were subjected to strong ground shaking during the earthquake. The other felt only low levels of vibration without, therefore, any significant possibility of evaluating its behaviour. The same is true of the Foothill Communities Law and Justice Centre (1986) in nearby San Bernardino County.

Table 6.2 summarises the main features of the above mentioned buildings.

6.2.2 LA County Fire Department's Fire Command and Control Facility (FCCF)

This structure (Plate 6.1 and Figure 6.1), is located about 39 km from the epicentre. It is a two storey steel braced frame rectangular shaped building about 57 m long and 26 m wide. The building is supported by 32 wide flange steel columns which are in turn supported by high-damping natural rubber isolators, 8 m apart in the transverse direction and either 8 or 8.5 m apart in the longitudinal direction (Plate 6.2). The first and second floors are concrete slabs supported on metal decking. The decking is puddle welded to the steel floor system to provide rigid diaphragm action. The roof consists of metal decking which in turn supports an insulating concrete slab. The roof deck was treated in design as a flexible diaphragm (Backmann et al, 1990). The design displacement on the bearings is 240mm.

The fundamental periods of the isolated structure are about 2.2s in the horizontal directions, 0.1s vertically with a torsion mode at 1.8s (Backmann et al, 1990).

The building is extensively instrumented to measure ground motion and structural response (CSMIP Station No 24580). Figures 6.1 and 6.2 show the sensor locations and the recorded responses due to the earthquake. In the N-S direction (positions 5, 8 and 13) the recorded accelerations are low and no amplification phenomena were detected. High peak accelerations are present in the east-west direction. In particular spikes can be detected at position 11 located near the seismic joint at the north east building entrance. The other measurement positions in the east-west direction show a similar phenomenon. These joints were repaired after the 1992 Landers earthquake with a very strong bedding grout and mesh reinforcement. This repair resulted in no 'rattlespace' which in turn caused significant pounding accelerations in the E-W direction as the joint separated and came together. Figure 6.3 shows details of these joints.

No significant torsional movement was detected. Some amplification can be detected in this direction but it is consistent with the design hypothesis.

The peak displacement during the earthquake was about 30 mm. As with the USC hospital (see 6.2.3), the isolation system was stiffer than expected for the larger design earthquake. According to the designer (see Table 6.2) some lights swayed and hit the ceiling and some ceiling panels loosened. No other damage of any kind was reported or observed and the building continued to function during and after the earthquake.

6.2.3 USC Teaching Hospital

This building (Plate 6.3) is located east of Downtown LA about 36km from the epicentre . It was opened in May 1991 and is the first seismically isolated hospital in the world. It is an eight storey, 275 bed teaching hospital, providing 32,000m² of floor space. The structure is a steel braced frame with an asymmetric floor plan. The seismic isolation system is a combination of 68 lead-rubber bearings at the exterior braced frame columns and 81 elastomeric bearings at the interior vertical load bearing columns (see Plate 6.4 and Figure 6.4 for bearing locations and characteristics). The fundamental period of the isolated structure is about 2 s, compared with a non-isolated period of about 0.8 s.

The building is instrumented and Figures 6.5 and 6.6 show the recorded response of the hospital during the earthquake (CSMIP Station 24605). The strongest motions were in the N-S direction. Figure 6.7 illustrates the peak values of acceleration recorded with reference to N-S, E-W and UP directions. The input motion was filtered in the horizontal direction, whilst some torsional motion, with centre of rotation near to the north wall, can be detected. The vertical motion was transmitted without amplification. The peak displacement during the earthquake was about 40 mm, compared with the 260 mm design displacement. Due to the elastic non-linear behaviour of the isolation system, the structure was stiffer than would be expected for the design event. This means that during a stronger earthquake, the filtering effect will be more significant. No residual displacement was detected after the earthquake. The hospital remained in service during and after the earthquake without any damage. There were no reports of damage to equipment inside the building.

6.2.4 Rockwell International Information Systems Centre

Rockwell International Information Systems Centre is in Seal Beach (Plates 6.5 and 6.6), about 66km from the epicentre. It is an 8-storey, nonductile, concrete frame building retrofitted in 1990 with base isolation. It is supported on a combination of 78 lead-rubber and natural rubber isolators installed at mid-column height at the lower floor.

The building is instrumented by CDMG to measure ground motion and structural response. Figures 6.8 and 6.9 show the sensor locations and the available records. It has to be noted that the free field peak accelerations were very low: 0.09g and 0.06g in the horizontal directions and 0.04g vertically. It is therefore unlikely that the bearings were active during the event. The building was not accessible for security reasons.

6.2.5 County Emergency Operation Centre (EOC)

The new County EOC (Plate 6.7) will provide the environment and technology to enable the County to coordinate and manage operations in response to a disaster. Because of its critical importance it is being designed to function throughout the course of a disaster, including an 8.3 earthquake.

The EOC is located about 39km from the epicentre approximately 1km from the Fire Command Centre. The building, still under construction at the time of the event, is rectangular in plan (54m long and 31m wide) with an area of about 3,600m². The building is supported on 28 high-damping rubber bearings (Plate 6.8), whose design displacement is about 400mm, corresponding to a shear strain of 200%. The structure is a steel frame construction consisting of wide flange steel beams and columns with steel tube bracing in both directions. The building is seismically separated from the surrounding concrete retaining walls by a continuous separation joint which is 500 mm wide. Neoprene bumpers are rigidly attached to the retaining walls with a clear distance of 400 mm from the edge of the floor. The bumpers, which are designed to serve as an ultimate restraint, will prevent the building from moving beyond the design displacement in case of a failure or reduction in stiffness of the bearings (Cho and Retanal, 1993).

A monitoring system is foreseen, but during the earthquake it was not mounted and consequently no records are available. There were no reports of any damage to the structure or the internal plant which was being installed.

6.2.6 Two residential buildings with helical springs and viscodampers

These two identical houses, located in Santa Monica about 24km from the epicentre, comprise 3-storey steel braced frames. Plate 6.9 shows the houses. The total weight of each house is about 70 Tonnes. The houses are supported, at each single column, by two or four helical springs and, at the corners, by elements with springs and viscodampers (Plate 6.10). The design fundamental periods are 0.45s vertically and approximately 0.7s for rocking in both horizontal directions. The non-isolated fundamental period is predicted at about 0.13s. The viscodampers provide 25%-30% of critical damping. For the design earthquake the displacements are about 30mm horizontally and 20mm vertically (Hueffmann).

A monitoring system is installed in one of the houses, but the records were not available. CSMIP records from the Santa Monica City Hall grounds nearby show peak horizontal and vertical accelerations of 0.93g and 0.25g respectively. Some square glass blocks of different height were distributed around the perimeter of the structures, and indicated that the buildings experienced vertical displacements, including the rocking effects, of about 25mm. No structural damage was observed although some furniture inside overturned during the earthquake (EERC, 1994).

6.2.7 Foothill Communities Law and Justice Centre, San Bernardino

The Foothill Communities Law and Justice Centre (Plate 6.11), located in Rancho Cuamonga at about 100km from the epicentre, is a 4-storey building supported by 98 high-damping rubber bearings which are interposed between the basement of the structure and the foundation (Papageorgiu and Lin, 1989). The superstructure consists of a steel space-frame stiffened in various bays by braced frames. The maximum displacement at the corner bearings was estimated to be 380 mm. The building is instrumented by CSMIP. Given the epicentre distance, the amount of shaking at the site is likely to have been small.

6.2.8 Conclusions

The Northridge event was the first strong motion event in which base-isolated structures have been significantly tested. The behaviour of the buildings demonstrates the following features.

- Records from the USC hospital go some way to demonstrating the ability of base isolation techniques to successfully mitigate the effects of seismic events both from the point of view of safety of people and integrity of contents. The benefit due to non-linear material characteristics is likely to increase as the magnitude of the event increases.

- Problems occurred at the Fire Command Centre which outlined the importance seismic joints have for these kind of structures; the same of course applies for the importance of seismic gaps and their proper detailing.

- Besides the absence of structural damage, the advantage of base isolation is the ability of the structure to remain in service and to preserve internal systems. This is of great importance for strategic and high risk structures and also when sophisticated and costly equipment is present.

- Base isolation can be applied as an alternative technique for retrofitting purposes, in particular to structures where the safeguard of internals is important.

6.3 Parking structures

6.3.1 Introduction

Reinforced concrete parking structures fared far worse than any other structural group. These were characterised by:

- extensive use of precast concrete, with the beams prestressed but not the columns, with the precast units often connected by welding together steel plates welded to the reinforcement
- thin decks, sometimes with precast concrete soffits and very thin toppings
- deep parapet beams, sometimes notched at the columns or precast/prestressed slabs sometimes punctuated by windows
- lateral resistance achieved by a variety of means, including specially designed bents and shear walls, sometimes formed by infilled frames
- lack of other architectural cladding.

6.3.2 Zelzah Avenue Car Park at California State University, Northridge

The enormous 3-storey car park built in 1991 was the only building on the campus at CSU Northridge to collapse. It comprised (Plate 6.12) an external frame of precast full-height column units with half-span outriggers on either side, with minimal continuity between adjacent units which it was understood was designed as a ductile moment resistant frame, stabilising the whole structure.

Internally there were precast, pretensioned beams spanning in the north-south direction supported on corbels at the columns (Plate 6.13). It has been suggested there were cast-in fixings in the precast units but whether this applied throughout was unknown. Internally continuity along the beams was provided by top bars passing through holes in the internal columns. Between these beams spanned a cast-in place post-tensioned slab (there were no internal beams in the east-west direction), with the anchorages visible on the exterior faces of the perimeter beams (Plate 6.14). Otherwise continuity between the perimeter frame and the internal structure was reliant upon small diameter bars projecting from the beam which were lapped into the slab reinforcement (Plates 6.13 and 6.14).

Collapse appears to have been initiated by shear failure of the internal columns (Plate 6.16), attributed by the designers to fixing of these pin-end designed columns by the contractors to facilitate erection. As the internal columns failed the floors fell, pulling in the exterior frame and demonstrating the appreciable out-of-plane ductility of the ductile moment resistant frame. In-plane however there was no indication of any inelastic behaviour, attributable no doubt to the low midspan shear resistance between the precast units (Plates 6.18 and 6.19) and the high stiffness of the deep edge beams, which had a span-to-depth ratio of about 3.5 (Plate 6.18). However the notches in the top face of the beams abutting the columns may have been sufficient to avoid the X-cracking noted in the columns in 6.3.3. The ties holding the precast concrete perimeter turrets broke as the beams collapsed and they fell outwards (Plate 6.15).

With only two modestly reinforced moment resistant frames in each direction (Plate 6.17), and with twenty three lines of pin-ended columns between in one direction and seventeen in the other, it is unlikely there could have been sufficient lateral resistance to survive an earthquake of even moderate size. It is suspected that this poor structural configuration is in part attributable to the exceedingly over-optimistic R_w values in the UBC (12 for a ductile moment resistant frame) and in part to the absence in the UBC for a requirement for redundancy (the UBC requires only continuity in relation to horizontal force). The design would not have been practical using the more conservative approach in other codes. The out-of-plane deformation of the columns indicates how effectively distributed inelasticity can occur in long members with shallow moment gradients (Plates 6.18 to 6.22).

6.3.3 Northridge Fashion Centre Car Parks

In the Northridge shopping mall were two two-storey garages, with precast concrete beams spanning between precast concrete columns in one direction and with precast double-Ts spanning the other way forming the deck (Holmes and Somers, 1996). The lateral loading was carried by a single short shear wall on each face and as in the Zelzah Avenue car park the columns were designed only for gravity loading, so despite being constructed about 1988 they lacked confining reinforcement. Failure occurred in the form of collapse of the upper storey columns and disintegration of the diaphragms. The shear walls were undamaged.

6.3.4 Denny's, Van Nuys

This is the car park of American Pacific State Bank. It is a predominantly nine storey structure, situated in Sherman Oaks at the south east corner of the intersection of Ventura and Sepulveda (Plate 6.23). The structure comprised four symmetrically placed shear walls with cast-in-place columns and post-tensioned beams. The deep parapets cast monolithic with the front of the columns either crushed at the interface with the columns or caused heavy spalling of the columns, sometimes with the X-formation common in strong beam/weak column structures (Plates 6.24 and 6.25). Fortunately the spalling rarely extended beyond the plane of the parapet. Remedial action involved inserting a slit in the parapet at the face of the column (Plate 6.26).

The damage extended the full height of the building (except at roof level where the construction may have been different) at both ends of the bays either side of the shear wall. This could have been caused either by rotation of the base of the shear wall, or by relative vertical movements of the shear wall base and column bases (Figure 6.10). Cracking at the base of the partial roof storey suggested horizontal movements.

An extreme form of vertical irregularity is the beam support to the columns above the entrance (Plate 6.27). The incipient shear cracking (Plate 6.28) in the supported column is unlikely to have been caused by vertical movement alone, and is likely to have been at least partly due to the horizontal loading and the exceptionally stiff beam and the supporting columns, which were probably tied at foundation level. The shear cracking in the stiff supporting column however is in the direction to be expected from the vertical loading and the protection provided by the close proximity to the shear wall would suggest this cracking was not primarily caused by horizontal loading.

It is considered that vertical and horizontal accelerations both contributed to the damage noted, but much of the damage would have been avoided if the shear walls had more extensive foundations.

The end stair well (Plate 6.29) separated from the main structure, presumably because it was inadequately tied.

6.3.5 Car Park of the Transworld Bank, Sherman Oaks

This is a single storey two bay car park (Plates 6.30 to 6.35) designed in the 1960s (at the same time as the Bank - see 6.6.5) with pretensioned beams and a deck of precast hollow planks with in-situ topping. Horizontal cracks had previously occurred over the development length at the ends of the beams and to compensate for the lack of shear resistance extra columns had been constructed on the north and south sides of the original columns (Plates 6.33 to 6.35), which served as retrofitting in protecting the interior of the building against shaking in the predominant north/south direction. The external columns on the south facade had suffered extensive shear cracking due to movement in the east/west direction attributable to the inadequate shear reinforcement (Plate 6.31) and the ramp collapsed (Plate 6.32).

6.3.6 Car Park at the Radisson Centre, Sherman Oaks

The structure was badly damaged on account of it being constructed monolithically with the main part of the building (Plate 6.36). The connection of the beams at different levels on the two sides of the column led to the failure of the beam-column joints (Plate 6.37).

6.3.7 Car Park at the Kaiser Permanente, West Los Angeles

The structure is located in west Los Angeles, along La Cienega Boulevard at the intersection with Interstate 10, about 25km from the epicentre. At the closest recording station (CSMIP No 24157, Baldwin Hills, Los Angeles) peak values of 0.24g horizontal and 0.10g vertical acceleration were measured. It is a five-storey precast structure, which suffered shear cracking of the exterior columns (Plate 6.38) and large horizontal movements.

6.3.8 Car Park at the Saint John's Hospital, Santa Monica

The structure suffered large cracks in shear wall panels at the connections between the walls (Plate 6.39) and at the corner of large wall openings (Plate 6.40). In another car park Holmes and Somers (1996) record a sliding shear failure at a construction joint in an in-situ RC wall.

6.3.9 Cigna Garage, Granada Hills

The structure (Plates 6.41 and 6.42) is located in Granada Hills, on Balboa Boulevard at the intersection with Kingsbury Street, about 5.5km from the epicentre. At the closest recording station (No 24386, Holiday Inn Hotel in Van Nuys) peak values of 0.47g horizontal and 0.30g vertical acceleration were measured. It is a three-storey precast concrete structure. The building suffered a large amount of damage, springing apart at the corners (Plate 6.43) accompanied with tearing at the top of the shear walls (Plate 6.44). Corbel damage was extensive and clearly visible externally (Plates 6.45 and 6.46), with spalling of the concrete outside the reinforcement which was most severe at the lower levels (Plate 6.46). However another corbel of fabricated steel plate infilled with concrete (Plate 6.47), which was near the top, appeared to be close to collapse. Such instances of damage concentrated towards the top of structures are interesting as here, despite the high accelerations, inertial forces are low. They are usually the result of:

- failure or absence of connections at intermediate levels, or
- failure of an internal element, or
- frame/shear wall interaction.

6.3.10 Car Park of Armed Forces Recruiting Centre, Santa Monica

This is a stepped predominantly 3-storey car park detached from the main building (See 6.6.7 and Plate 6.48). The lateral loads had been mainly resisted by infilled frames in both directions causing some cracking between the frame and the infill and crushing the corners of some of the infills (Plate 6.49), both indicating the occurrence of relative movement between the frame and the unusually long infills. There were one-way shear cracks at the base of some isolated columns on the south east face (Plate 6.50) and transverse cracking was noted at cold joints in a column (Plate 6.51). The damage to the structure however was minor and typical of that in old structures with infilled frames.

6.3.11 Multi storey steel car parks

Steel car parks fared much better than those of precast concrete construction, but in Los Angeles they were few in number. Plate 6.52 shows a new mechanical car park at CSU Northridge, which was undamaged.

6.3.12 Conclusions: parking structures

The high rate of failure of car parking structures would appear to be principally associated with defects relating to the features associated with precast concrete construction, namely:

- precast elements were inadequately connected

- connections between the precast elements tended to be non-ductile; in monolithic construction the intersections are where most of the energy dissipation occurs

- the failure of the welded or other connections between precast units caused large relative displacements between the beams and columns and resulted in the beams falling off the corbels

- corbel failure in the region of the epicentre was probably associated with the high vertical acceleration

The performance of some of the car parks raises serious doubt under what circumstances it is permissible to design some vertical elements solely to resist gravity loading on the presumption that lateral loading will be resisted elsewhere in the structure. Precast concrete multi-storey car parks designed on this basis proved particularly vulnerable to catastrophic collapse. Collapse of the columns intended to carry only gravity loading has been noted not only in structures with highly ductile "SMRFs" (special moment resistant frames) but also in structures with rigid shear walls. The failure of these columns is attributable to their:

- forming part of the lateral load resisting system;
- suffering distortions they could not sustain;
- being subject to exceptional vertical acceleration;
- separating from the floors due to inadequate connection between the floors and columns.

Where this form of construction is employed consideration should be given to provision of many more moment resisting elements distributed throughout the structure.

There are many more vulnerable multi-storey car parks in Los Angeles. Plate 6.53 shows a vulnerable car park in Southern Los Angeles.

The design of multi-storey car parks is being reviewed by the authors of the UBC. One factor likely to be considered is that the allowance for imposed loads for car parks hitherto has been appreciably less than for residential, commercial and industrial premises.

6.4 Medical/mental health centres and hospitals

6.4.1 Introduction

Most Los Angeles hospitals remained open at near normal operations, six Disaster Medical Assistance Teams were assisting and nine Mobile Health Clinics were operational. Several health-care buildings suffered structural damage, had major non-structural damage or damage to contents and technical equipment. According to the California Office of Emergency Services, two Los Angeles County hospitals were closed and two had limited capabilities. Three county-operated and three non-county-operated health centres were closed. Many hospitals had water floods after the earthquake or had been trucking in water. This shortfall in health-care facilities was mitigated by mobile health units.

There is a concentration of hospitals in the coastal district of Santa Monica, three of which were badly damaged. At the closest recording station, (CSMIP No 24538, Santa Monica City Hall Grounds) peak accelerations measured were 0.93g horizontally and 0.26g vertically.

Before 1971 hospitals had been under the City control, but following the serious damage to hospitals after the San Fernando earthquake and the consequences of that damage control passed to the State. More stringent rules were adopted for hospitals than ordinary buildings and the horizontal force they were to withstand was increased by 50%, a requirement incorporated in the subsequent version of the UBC in 1973. In 1983 an act required more bracing and more pipe supports.

There has been no legislation requiring old hospitals to be retrofitted, like that for schools discussed in 6.5.1, but proposals for such retrofitting are under consideration and it is notable that all the serious damage occurred in hospitals constructed before 1971-1973.

The response of the University of Southern California Teaching Hospital (USC) is considered in 6.2.2. This is an eight-storey base isolated structure, with a recorded peak free-field acceleration of 0.49g (the most severe test for an isolated building structure). It was operating fully both during and after the seismic event and this is very encouraging for the future of this innovative technique.

6.4.2 St John's Hospital, Santa Monica

St John's is located at 20-23rd Street, between Arizona and Santa Monica, about 23km from the epicentre. This large hospital comprised several buildings, most of which suffered damage. The main building, the north wing, an eight-storey externally perforated 10" thick shear wall structure was constructed in the late 1940s and designed for a horizontal force corresponding to an acceleration of 0.08g, about half the net force now used for ordinary buildings in Los Angeles. The rear wall of this building (Plate 6.54) had a regular pattern of openings over most of its height, but significantly more openings at first floor which was also a short storey on account of its window heights being less than those in the rest of the building and particularly in the ground floor storey. Severe X-cracking occurred in the wall/columns all along this storey and less severely in a few of the coupling beams (Plate 6.55). There was no damage in the other (short) direction. The small amount of transverse reinforcement in the walls is likely to have contributed to their lack of ductility. For a given area of windows, tall narrow windows would appear to be preferable structurally. In-plane flexural failure had occurred at the outer edge of the walls.

Despite the severity of the damage this building was not evacuated until 20 January, three days after the earthquake. The damage is so severe that full repair and retrofitting were thought to be uneconomic. If so the course of action first considered (due to the shortage of hospital accommodation in Los Angeles) was to repair it sufficiently to resist gravity loading and demolish it in three to five years, the time to build its replacement.

Another building, a masonry infilled RC frame structure, was severely damaged with large concentrated diagonal cracks in the infills and hinges had formed in the columns (Plate 6.56). It is noted that diagonal cracks in infills are sometimes considered to be an ultimate limit state condition, unlike sliding cracks on bedding planes.

6.4.3 Kaiser Permanente Health Institution, Granada Hills

The building is located on Balboa Boulevard at the intersection with San Jose Street, about 5.5km from the epicentre. At the closest recording station (CSMIP No 24386 the Holiday Inn Hotel in Van Nuys) peak values of 0.47g of horizontal acceleration and 0.30g of vertical acceleration were measured. It is a five storey non-ductile moment-resisting frame structure. Partial collapse occurred in the form of pancaking of a weak-storey at the second storey, attributable to the formation of a weak-column-strong-beam mechanism and lack of confining reinforcement at the joint (Plate 6.57).

6.4.4 The Kaiser Panorama

The Kaiser Panorama building in Panorama City on Roscoe Boulevard at the intersection with Sepulveda, is a reinforced concrete shear wall structure constructed in 1965 and damaged in the 1971 earthquake, after which it was repaired with epoxy resin injection. Substantial gunite walls were then constructed alongside the existing shear wall and attached to it, after grit blasting the surface, with shear keys in the form of 16 mm bolts at 600 mm pitch. Guniting of the densely reinforced wall was difficult and it is understood in Los Angeles shotcrete would nowadays be used instead. It survived the Northridge Earthquake undamaged.

6.4.5 Barrington Medical Centre, Santa Monica

This exclusive private clinic, located at the intersection of Barrington and Olympic was constructed in 1968 (Plate 6.58). It is a six-storey reinforced concrete building, L-shaped in plan, with horizontal load resistance provided by irregularly placed shear walls. Deep spandrel beams created short columns at all levels except the ground floor. It is likely that the irregular planform caused substantial torsional movements of the building during the earthquake, resulting in severe X-cracking of the short columns, which had non-ductile reinforcement details (Plate 6.59). The shear walls, which were undamaged, did not protect these columns. The building was evacuated immediately after the earthquake when it was badly damaged. It seemed to be in danger of imminent collapse on sustaining further damage during an after shock and was demolished with its contents intact on 22 January 1994.

6.4.6 Santa Monica Hospital, Santa Monica

This hospital is located between 15th-17th Street, and between Arizona and Wiltshire, about 23km from the epicentre. It comprised two large buildings. The older building, which had been evacuated after the earthquake, was a regular, eight-storey reinforced concrete perforated shear wall structure (Plate 6.60), and like St John's was constructed in the 1940s and was designed for the same force coefficient. Along the two shorter sides of the building the shear walls were perforated by single vertical line of narrow windows, such that the perforations occupied no more than 40% of the height. At the north of the building diagonal cracking occurred in each of the coupling beams (Plate 6.61), whilst at the south the damage was concentrated at the lower corner of one of the walls (Plate 6.62), but with minor damage in the coupling beams. There was inadequate transverse reinforcement in both walls and coupling beams. The longer faces of the building were undamaged (right hand side of plate 6.63). Repair was considered to pose no particular problems.

The second building, which appeared very new (less than five years old), had been constructed in a similar style to the first, but was only four storeys high. This building had suffered no structural damage, though there had been quite widespread spalling of non-structural finishes, repair of which was completed by 20 January.

6.4.7 Indian Hills Medical Centre, Indian Hills

This private clinic is in Mission Hills, along Rinaldi Street at the intersection with Indian Hills Road, about 10km from the epicentre. At the closest recording station (CSMIP No 24514, Sylmar County Hospital) peak values of 0.91g horizontal and 0.60g vertical acceleration were measured. The building is a regular six-storey frame structure (Plates 6.64 to 6.65), with five bays along the facades and two groups of three shear walls with side-columns forming two C-shaped cores at the ends. The main lateral load resisting elements were shear walls with columns either end forming boundary elements, which were similar to the load bearing columns.

Damage was mainly concentrated at the shear walls and columns of the fourth level (Plates 6.66 to 6.68), but there was some damage also lower down (Plate 6.69). It is considered that the damage in Plate 6.62 at least is due to insufficient splice lengths, rather than the more usual lack of confinement, responsible for comparable damage in 6.6.7. Elsewhere diagonal cracking and spalling around main reinforcement is reported. Some crushing took place at the ground level in the side-columns of the shear walls. The building had to be evacuated.

Construction had not been quite completed at the time of the 1971 earthquake, when in suffered major damage to the shear walls. Repair included extensive crack injection and new shotcrete shear walls on existing shear walls and welding of some splices.

6.4.8 Holy Cross Hospital

In this hospital at 15031 Rinaldi Street in Mission Hills there was no structural damage, but mechanical damage was reported in the penthouse where bolts had sheared. In a non-medical building with a steel moment resisting frame cracks were reported in the columns of welded beam connections.

6.4.9 Northridge Hospital Medical Centre, Northridge

Northridge Hospital at 18300 Roscoe Boulevard experienced minor damage to the older part on spread footings which settled by 150mm. There was no settlement of the newer part on piles constructed in the late 1980s. In the older part there was some cracking in a shear wall and pipes had broken.

6.4.10 Veteran Hill Hospital

This hospital in the immediate vicinity of the epicentre suffered serious damage to the contents and equipment and was completely flooded.

6.4.11 Olive View Medical Centre

(a) Introduction

The Olive View Medical Centre in Sylmar (Plate 6.70), to the north of San Fernando township, is the county hospital and a modern 400 bed teaching hospital. It is of historic importance in earthquake engineering as it occupies the site of the hospital, designed to the 1964 building code, which was so badly damaged in the San Fernando earthquake of 1971 (Jennings, 1971) that new rules for the earthquake resistant design of hospital were introduced. During the earthquake of 1994 the structure of the main building was undamaged, despite being subject to accelerations of 0.82g at the base and 2.31g at roof level and a vertical acceleration of 0.34g (Figure 6.11). Ancillary structures in the hospital grounds however were damaged in the earthquake of 1994. It is noted that the base accelerations were less than the free field accelerations with the free field vertical acceleration being 0.60g, 76% greater than the base acceleration.

(b) Effect of the earthquake on the services provided

The normal operation of the hospital was disrupted by the high accelerations; the oxygen tank and cooling towers came off their foundations and patients records were scattered around the floor (Plate 6.71). There was severe flooding on the 6th floor and below, after which the water supply failed (Plate 6.72). Staff who mostly lived in Santa Clarita, north of the collapsed I5/H14 intersection, were unable to get to work: of the administrative staff only the Assistant Administrator, who lived in Pasadena, arrived on time and he found it impossible to run the hospital single handed. In particular there were inadequate staff to attend to the psychiatric patients on the top floor who had been much disturbed by the earthquake.

The emergency diesel generator, which had been tested weekly, started but cut out under load and could not be restarted. The emergency gas turbine also started, but could not be run due to insufficient water to cool the bearings.

The telephone system in the hospital was not damaged, but outgoing calls beyond the area covered by the local area code could not be made due to the unexpected blanket restrictions imposed by the telephone company on all users, which caused surprise and disruption to the emergency services.

It was possible for the hospital to offer only limited services under these conditions, so the emergency and ancillary services were temporarily suspended and the hospital was effectively closed, except that first aid continued to be available.

Water trucks were brought in, which enabled the gas turbine to be started. There was an ample supply of bottled oxygen and the hospital reopened by 11am, having been largely out of action for 5 hours. The normal power supply was not available for 8 days.

The hospital's earthquake drill and contingency procedures did not take account of the low staff levels available and the loss of use of the phones and high level discussion between the hospital, staff officials and the telephone company to avoid repetition of their action were already under way.

(c) The structure

The structure of the main building (Troy, 1987) comprised a 19.5 m high four storey steel tower of cruciform shape, of overall size 91 by 91 m placed symmetrically on a two storey RC structure of 91 m by 137 m. It was designed for an earthquake of magnitude 8.5. In the 1970s when it was constructed it stood unclad for some years and was not finally opened until 1987 and as it was irregular (both in plan and elevation) it was analysed using modal analysis ('dynamic analysis' in US terminology). An additional check was provided using time history analysis, which presumably included the accelerograms for the San Fernando earthquake, which were then available.

The lateral forces on the tower were resisted by stiffened steel walls of 16 mm and 19 mm plate (Plate 6.73), which had been designed disregarding tension field action (Troy, 1987). These also carried a portion of the vertical loading. The lateral forces on the structure below were carried by reinforced concrete shear walls, also designed elastically.

The high accelerations may be attributed to the lack of inelasticity in the structure due to conservatism in the design. Ferritto (1984), in his paper advocating the merits of elastic design, comments on the high accelerations in structures so designed and the care needed in the design and anchorage of the ceilings, the mechanical and electrical equipment and in particular the lights. With adequate measures in these respects the Olive View Medical Centre may, in the event of the much feared Magnitude 8 earthquake, even outperform the base isolated hospitals, which performed so well in the present earthquake.

(d) Ancillary structures

The ancillary structures contemporary with the earlier hospital suffered damage. In the tilt-up warehouse of 1961 (Plate 6.74), which was portalised in one direction (Plate 6.75) the panels separated at some joints (Plate 6.76) and a wall panel bowed out in one place (Plate 6.76) and the roof was damaged when a plywood panel was broken (Plate 6.75). However the warehouse was in use when inspected, suggesting that, unlike similar structures of its vintage which had performed poorly elsewhere, the concrete wall panels were well tied in at roof level.

Other structures which were even earlier were also damaged (Plate 6.77). No buildings collapsed as in 1971 (Jennings, 1971).

6.5 Schools and universities

The Los Angeles Unified School District is responsible for 640 elementary and secondary schools and a further 160 continuing and adult education facilities and day-care centres. All of these were closed for a full week after the earthquake. 97 schools, all in the San Fernando Valley west of the San Diego Freeway, were damaged severely enough to remain closed beyond the first week, affecting 100,000 pupils. A further 200 suffered light damage, but were nevertheless expected to reopen on 25 January. It is estimated only 1,500 classrooms out of a total of 30,000 were usable and the displaced pupils were accommodated in other schools.

School district officials stated that no school site was sufficiently badly damaged to remain permanently closed. Staff at all schools were required to report for work on Monday 24 January in order to prepare earthquake-related lessons and to assist with light clean-up duties before pupils returned on Tuesday, 25 January.

6.5.1 Schools

In the nineteenth century schools had been predominantly wood framed but these suffered severe fires during earthquakes and in the early years of the 20th century schools were constructed of masonry. These suffered badly in the Long Beach earthquake of 1933 and a law was passed that year requiring all such buildings to be retrofitted (see 6.10.4).

In Los Angeles schools tend to be in older buildings. In the Northridge earthquake schools, including the 1933 retrofitted buildings, performed relatively well.

A problem arose in that 127 schools were surveyed using the ATC 20 rapid evaluation procedures (ATC, 1988), but the critical evaluation was too conservative and only 21 were pronounced safe (green tagged). Of the remainder:

- 24 were rated dangerous (red tagged), though few were in fact considered likely to collapse in the event of a strong aftershock. The damage largely comprised fallen pieces of masonry (but not complete panels), spalling, buckled bracing, diagonal cracking in shear walls and spalling at beam/column joints. Included in this category were instances when ground cracks ran through buildings. Serious damage was confined to pre 1971-73 buildings.

- 82 had limited entry rating (yellow tagged), though this mostly related to cases where structural damage was considered possible or where there were non-structural hazards.

There was however significant damage to covered walkways and 'lunch shelters' and other light canopy structures which proved far more sway susceptible than other structures. They suffered loosened connections, minor (permanent) racking, pounding and unseating of the occasional beam.

Inadequately braced pre-1974 buildings suffered racking of the crawlspace, mostly due to rotting and the associated loosening of nails, but none collapsed.

The most hazardous non-structural features were pendant mounted lights predating the code safety wire requirements, but the most usual non-structural damage was the collapse of unanchored and unbracketed bookshelves, filing cabinets and the like, as the majority of schools had disregarded suggestions for anchoring and bracing them. Falling plaster is a hazard in the larger rooms, particularly assembly halls. The risk was from the older wood lath plaster on walls and ceilings, and not the more modern wire mesh reinforced plasters.

6.5.2 California State University (CSU), Northridge

(a) Introduction

This is a compact university campus of 30,000 students, situated at an estimated distance of 3km from the epicentre, and is considered in some detail as it contains the greatest concentration of engineered buildings in the epicentral region. Figure 6.12 is a part-plan indicating the structures discussed. The multi-storey car park which collapsed is considered in 6.3.2. The shake there was in three distinct phases, first it was North-South, then vertical, then North-South again. The nearest recording station was the Holiday Inn (See 6.6.2).

The EEFIT members present were taken on a tour of the campus by car and allowed to take photographs, but were not admitted to the interiors of the damaged structures. The internal damage however was described, and in particular the amount of the damage caused by the aftershocks. The soil is firm sandy loam.

The most notable feature of the earthquake damage in the vicinity of Northridge was the amount of damage to which vertical excitation may have contributed, which would be consistent with its epicentral location. Damage to horizontal cantilevers is the clearest sign of high vertical acceleration, as damage to vertical load bearing elements is affected by complex stress states and by overall rocking of the building. Most simple structures without wide balconies performed moderately well, despite the high horizontal accelerations.

(b) The Administration Building

The Administration Building (Plate 6.78) was superficially damaged. Some window panes had been broken on the south facade and the stairwell tower separated from the offices and was considered to be on the point of collapse (Plate 6.79). In the single storey attached annex transformers "shifted around and tried to leave the room".

(c) The Oviatt Library

This library (Plate 6.80), was built in two phases. The central part, built in 1971, is an RC shear wall structure and there are more modern extensions either end with steel columns. Around the perimeter is a large but light canopy supported by precast columns cantilevered from the ground, with a nominal connection at the top to the main roof provided by the canopy. The main damage was to the steel columns in the extensions (as described in Section 6.8) and significant damage to the cantilevers on three sides (Plates 6.80 to 6.83). There was little structural damage internally, though there was severe ceiling damage in the top storey and minor ceiling damage in the lower storeys and there was minor cracking in the shear walls.. The shelving was undamaged, though most of the books were thrown onto the floor. The damage had significantly increased due to aftershocks between 20th January and 3rd February.

(d) South Library

The old four storey South library was deemed to be undamaged following an inspection after the earthquake, though there was a large amount of minor cracking and broken glass. However an aftershock, which scared workmen on the roof, caused appreciable cracking internally and after a subsequent inspection the structure was found to be in a dangerous condition. Sliding shear cracks were noted between precast panels of a shear wall on the east side at four levels, and these were not confined to the lower part of the building (Plate 6.85). It was noted however that precast shear walls were being used in the construction of a new six storey block nearby.

(e) Other buildings

The physical education block, with one large gym in the lowest storey and two smaller gyms in the second "performed well". It was described as "ductile", though from its facade it appeared to be largely unperforated masonry at the lower level with light metal cladding above (Plate 6.86).

In the engineering building there was only minor damage, but there was so much asbestos scattered around that no one was allowed inside (Plate 6.87).

Bridges between two parts of the science block fell (Plate 6.88) and a replacement bridge was ready for erection (Plate 6.89).

In the chemistry laboratory in the second storey of the Science 2 building there was a fire which was difficult to control due to the fumes from various chemicals (Plate 6.90).

In the two-storey bookstore a precast concrete shear wall collapsed adjacent to a large irregular cut-out of flimsy construction (Plate 6.91).

The speech-drama/music centre is a massive masonry clad building without windows, divided in two by the entrance lobby (Plate 6.92). Despite its size and lack of movement joints it was not damaged by the earthquake.

The speech drama centre (near part of Plate 6.92) appeared undamaged on the outside on 3rd February, but by then internal damage had occurred during an aftershock.

There was significant damage in the new low and irregular student health centre, though there was no damage externally (Plate 6.93).

An eight-storey residential block on the campus had been abandoned prior to the earthquake because of code deficiencies. The building consisted of a frame at ground floor level, with upper storeys comprising walls made up of two skins of masonry with reinforced concrete in between (Plate 6.94). Shear cracking was observed in both concrete and brickwork at first floor level. Separation of about 75mm occurred at expansion joints between the main structure and the end stair wells. The outer masonry had fallen away in places due to the lack of wall ties (just visible behind left hand lamp post).

A number of other buildings on the campus appeared to be recently constructed, stiff, regular reinforced masonry structures. None of these showed any sign of damage, eg the Faculty Office (Plate 6.95) opposite the damaged bookstore. The performance of the reinforced masonry is considered further in 6.10.5. A flimsy open sided steel framed shed sheltering agricultural equipment and a well constructed tilt-up building were also undamaged. So too was a new steel multi-storey car park with automatic parking (though whether the parking equipment was affected is unknown). In the modular single storey electrical station there was no damage. The observatory near the student health centre, built by the University, was undamaged (Plate 6.93).

Holmes and Somers (1996) refer to damage to beam and truss connections over some of the larger lecture theatres and damage due to inadequate anchorage (mainly to RC elements) at the supports, a foretaste of the damage to large space frame roofs in Kobe (EEFIT, 1997).

In two to four storey apartment buildings surrounding the university, balconies and walkways between blocks were severely damaged and many had fallen. One off-campus apartment building occupied by students had completely collapsed, and this was the only building in which there was sufficient loss of life to justify reconsideration of the suitability of the structural form for use in earthquake areas. It was suspected that besides the balconies, which themselves impose dynamic forces on the load bearing part of the structure, there were insufficient ties between the structural elements.

The repair costs to the university buildings per unit floor area, based on information given by Holmes and Somers (1996), have been summarised according to the structural type and date of construction in Table 6.3. Compared to the unit costs for the predominant form of 1960s RC shear wall construction, those for later RC buildings were not noticeably less and those for the later braced and sway steel framed structures were significantly higher. The unit costs were noticeably lower for the smaller buildings.

6.6 Taller buildings (seven storeys or more)

6.6.1 Introduction

Whilst the tall buildings of downtown Los Angeles were not much affected and the damage to the steel buildings was unknown at the time of the visit significant damage had been suffered by a number of buildings in the 7-16 storey height range.

6.6.2 Holiday Inn Express, Van Nuys

(a) Introduction

The seven storey Holiday Inn Express (Plates 6.96 and 6.97) at the intersection of Roscoe Boulevard and Orion Avenue was the closest instrumented building to the epicentre, at a distance of about 6km. At this location the shake experienced was firstly in the east-west direction, then vertically and then east-west again. Whilst the two phases in the east-west motion can be identified from the accelerograms (Figure 6.13) it is not immediately apparent why the vertical component predominated between. The absence of the perceived north-south shake must be attributed to the greater human sensitivity to moderately large amplitude slow swaying motion than to smaller amplitude shorter period movements of the structure in the stiffer direction.

The building is of broadly similar appearance to the Holiday Inn in Santa Monica, and is probably of a standard design except that it had recently been upgraded to 'Express' status. This entailed complete refurbishment of the ground floor to accommodate a room suitable to offering complementary breakfasts and the addition of a low tower by the entrance, but without modification to the main structure. There was no restaurant. The hotel was built in 1967 and opened in 1968. The performance of this building might have implications on the adequacy of buildings of similar vintage throughout California.

(b) The structure

The building is piled, with tie beams between the pile caps in both directions. The soil is predominantly fine and the water table very low (Blume et al). The main building measured 160ft (48.8m) east-west and 62ft (18.9m) north-south, with a one bay square full height extension over the entrance and a one bay wide single storey extension along the south face. Its main characteristics are summarised in Table 6.4. There were shear walls formed by infilled frames either end (on the east and west faces). There were spandrel beams on the two facades which with the perimeter columns could be regarded simplistically as forming the main load resisting system in that direction, although the shear wall at the west end returned two bays along the south face and returned over the top by a further bay (Plate 6.97). On the north face four bays were infilled at ground floor, so the lowest storey was well restrained all around. There are nominal 25mm gaps against the columns and 12.5mm gaps beneath the beams, but as they are described as "movement joints" they may be absent in the transverse frames. A further complication was a five bay long spine down the centre of the roof at the west end.

Internally the floor was of flat slab construction 203 to 254mm thick, though there were beams in the end bays and at roof level. The end bays were about twice as stiff as the internal bays.

The columns measured 14" by 20" (356mm by 508mm) with the lesser dimension in the plane of the facade. Where the columns were damaged (discussed later) it could be seen that there were six one inch (25mm) deformed bars longitudinally, three in each of the longer faces, with ¼" (6.3mm) ties of mild steel at 12" (305mm) centres. This reinforcement is no more than might be expected in a structure designed only to resist wind, so it is possible that - if earthquake resistant detailing was used anywhere - it was used only in the levels below where the damage occurred.

(c) Damage in earlier earthquakes

According to the then requirements of the Los Angeles City Building Code the building had been instrumented and it was in fact the closest instrumented building to the estimated epicentre of the San Fernando earthquake of 1971. The acceleration at the base was 0.25g and that at the top reached 0.40g (see Table 6.5). It is reported that there was extensive damage to the plaster and plumbing fixtures in the second, third and fourth storeys. The spandrel beam at first floor level on the north face at the east end had split vertically against the column (Department of Commerce, 1973), a result of overall rocking and in-plane stiffening of the north side by the infills on that face at ground floor and the deeper first floor spandrel beams. Also on the north side unreinforced "architectural" fins on one of the columns had crushed. On the south side some of the columns had fine horizontal cracks through the construction joints at the underside of the spandrel beams.

It is recorded (Department of Commerce, 1973) that the structural frame "received some cracks which were repaired with epoxy mortar". However the location and depth of most of the cracks was not recorded in the literature studied. The fact that epoxy mortar was used instead of a less viscous repair material suggests that the cracks were wide, but they may have been superficial. Such a repair material would have been prone to spalling in subsequent earthquakes due to the greater Young's modulus.

This building was recorded as one of the success stories of the 1971 earthquake and it was concluded "the building response was elastic or very nearly so as far as the structural frame is concerned" and "emphasises the fact that buildings designed according to the code possess, in general, a level of elastic resistance that substantially exceeds that indicated by the lateral loads specified by the code. Furthermore, in the case of the Holiday Inn, the inelastic ductility of the structure provides a margin of safety against motions even greater that it experienced in the San Fernando earthquake" (Department of Commerce, 1973).

In the Whittier earthquake of 1982 there had been some cracking in the stairwells and damage to the tiling in the bathrooms.

(d) Damage to the structure

The maximum longitudinal ground acceleration was 0.47g (see Table 6.3), almost 90% greater than that experienced in the San Fernando earthquake, but that at the top was only 48% greater, reflecting the softening effect from the damage lower down.

Damage judged sufficient to cause collapse during a severe aftershock was confined to X-cracking of the fourth storey columns on the south elevation over the height of the windows (Plates 6.99, 6.100, 6.103 and 6.104) where there was heavy spalling of the concrete cover on the side faces (it extended up the columns further on the east side than the west) and the distortions had been so large that there was significant permanent set across the cracks, such that light shone through the columns (Plate 6.104). One of the links had split and there may have been others. Where the link had split the column bars bowed outwards spalling the concrete cover over a considerable distance, a consequence of the large cover to core width ratio (common in columns of small cross section), and possibly associated with high vertical forces to which the stiff roof structure may have contributed.

There was extensive fine cracking on the joints, beams and columns up to 6th floor level on the south side (Fig 8 of Lynn et al, 1996) which was not observed during the EEFIT visit and may have been caused by a subsequent aftershock.

Elsewhere damage appeared to be confined to vertical cracks running up the most westerly column on the north side over the full height of the third storey, with vertical cracks through the spandrel beams above and below (Plate 6.98), comparable to the damage at the east end of this facade in (c) above, though without assistance from the infills. Also on this facade there were signs of light cracking in the panel zones (Plate 6.98), which had no counterpart on the south side. There was some damage (which may not have been structural) to the new tower at the west end. There were horizontal sliding cracks between the panels of the shear wall at the east end and vertical cracking against the columns on the south face (Plate 6.106). Internally there were cracks in a column in the second storey at the west end and relatively minor cracking in the shear wall at the east end (Plate 6.107).

(e) Non-structural damage

There was little damage to the finishes on the ground floor storey, due no doubt to the rigid annex at this level. In the second storey the door frames distorted so much that 5 guests on the north side were trapped in their rooms, however the lack of damage to the plaster and door frames suggests the cracking of the panels over its height was unlikely to have exceeded about 15mm. At the 4th floor the plasterboard wall linings to the corridor were cracked on the north side and had spalled where they had rubbed against the floor slab (Plate 6.109). Also on this floor the bathroom tiling, the most vulnerable feature of the finishes to distortion, was badly cracked and was also damaged in the second and third storey, and to a lesser extent elsewhere.

There was however virtually no damage on the top storey which is where the accelerations were the greatest, but where the storey sway was probably the least, suggesting that distortion and not acceleration determines the amount of damage (distortion is known to be the factor determining damage to non-structural partitions). Confirmation that the east-west accelerations were greatest at this level is provided by the fact that all the televisions and the television tables, all against cross-walls, had fallen over (Plate 6.108). The fact that this occurred on both sides of the building and at no other levels confirm the general distribution of inertial forces with height assumed in structural analysis.

The most successful features were the windows which held without loss of glass, despite severe distortion of the surrounding frames and the permanent set of the structure in the fourth floor (Plate 6.104). They may even have provided some lateral restraint by holding the concrete cover in place on the sides of the most heavily damaged columns.

After the earthquake the lifts were all out of action, but by far the most serious consequence was the loss from them of hydraulic fluid, which poured down the facade and the stair well making descent from the upper floors quite treacherous. When visited the hazard had been reduced but not eliminated by sinking carpeting into the oil (Plate 6.110). The stairwells were otherwise usable, though there was appreciable damage to the flights of the precast concrete stairs at their landing connections (Plate 6.105).

From the more human perspective the maintenance engineer and his son had been asleep in adjoining rooms on the second floor at the time of the earthquake. His son, despite being nearly thrown out of bed, did not awake till awoken by his father shouting "Jason wake up" from the adjoining room. On the more eerie side someone keeps calling from one of the rooms, yet all rooms have been checked and there are no cars left in the car park.

(f) The swimming pool

The large open air swimming pool in the east of the hotel appeared to be undamaged, but was reported to be leaking.

(g) Analysis of the damage

Whilst the long front (north) facade contains a tall first storey, the columns are larger than those above, and as three of the bays are infilled by masonry (Plate 6.96) and despite the gaps around the infills (see (b) above) it is unlikely that they produced a weak storey effect, though they undoubtedly constituted a minor vertical irregularity. Over most of the length of the other long face (the south side) was a single storey service annex without windows (Plate 6.97). It is inferred that the torsion resulting from the restraint to movement in this storey and the flexibility on the opposite face was carried by the shear walls on the short ends of the building, with the first storey columns on the other side carrying little force. Such restraint is considered to have contributed to the damage to the building considered in 6.6.8.

With such a system severe damage might be expected in the second storey columns above the annex. The fact that damage occurred in the fourth storey may in part be due to changes in stiffnesses due to damage and the repairs carried out after the earlier earthquakes; this work was most extensive in the second and third storey. The damage was consistent with the repairs being most extensive towards the south side of the structure.

The splitting of the links described in (d) could have been an indication of exceptional vertical forces from the vertical excitation and rocking, though rocking may have been less in this building than in others due to the light partitions. Shear lag in the spandrel beams would have mitigated substantially the rocking effect from the end walls; shear lag would have been less in the ground floor columns on the north face and the effects of rocking were therefore more pronounced.

An interesting feature of the design is the lack of significant variation in the vertical column reinforcement between the first floor and the roof. This is a characteristic of some of the older RC buildings in Kobe (EEFIT, 1997), which exhibited similar incipient hinge formations, particularly in regards to the lack of damage in the corner columns.

(h) Remedial action or demolition

The building had been "red tagged" immediately after the earthquake and the decision as to whether it was repairable was so marginal that the fate of the building was in the balance for some time. It is likely that client confidence was a significant consideration. As the region where the column reinforcement cages was damaged was so localised the option existed of retrofitting the structure when the aftershocks ceased. Taking into account the extent of the cracking, the inadequate transverse reinforcement and the smallness of the core it would have been exceedingly difficult to reinstate the structure to even its original strength. The building was eventually declared unsafe and scheduled for demolition. However, possibly due to its historical importance, the decision was subsequently taken to repair it. Temporary inverted-V bracing was installed in three of the bays up to the damaged storey and the cracks, including the fine cracks at higher levels, were resin grouted. This measure is beneficial in old structures since they are likely to deteriorate rapidly on cracking if the reinforcement is subject to tensile yielding (EEFIT, 1997).

6.6.3 AT & T Building, Sherman Oaks

The building is on Ventura Boulevard at Kester Avenue, about 10.5km from the epicentre. The closest recording station was the nearby Transworld Bank (see 6.6.5). It is an eight-storey building, which suffered a surprising amount of cracking considering the ample provision of shear walls (Plate 6.111). Suggestions have been made that upper floors could have been added at a second stage.

6.6.4 First Interstate Bank Building, Sherman Oaks

The building is located on Ventura Boulevard at the intersection with Cedros Avenue, about 11km from the epicentre. The closest recording station was the Transworld Bank (see 6.6.5). It is a 12-storey dual structure, with a moment resisting frame in the east-west direction and shear walls along the east and west faces. The walls had flexural cracks which extended up to the fifth level (Plate 6.112). The extensive spalling of the tension side at the fifth level may be due to insufficient splice lengths. The fact that the damage took place in the shear walls only is in agreement with the recorded predominant north/south ground motion in this zone.

6.6.5 Transworld Bank, Sherman Oaks

This building is situated on the north east corner of Ventura and Sepulveda intersection at about 9km from the epicentre. The building (Plate 6.113) was designed in 1964 in accordance with the Los Angeles City Code. The plan dimensions are 58.8m east-west by 22.9m north-south; each of the 14 storeys has a height of 3.6m, with the exception of the ground floor, which is 7.2m. Allowing for the effect of the balcony beams, which were monolithic with the columns, the unsupported length in the ground floor storey is about 2.5 times that at higher levels, which constitutes a significant soft storey, and exceptionally soft for such a tall building and compared to others in the vicinity. Lateral forces are resisted by reinforced concrete moment-resisting frames, except at basement levels, where shear walls are provided in both north-south and east-west directions.

During the 1971 San Fernando earthquake, the building suffered damage to the four corner columns in the vicinity of the deep first floor beams. These were repaired, and the joint regions strengthened by the addition of post-tensioned tendons. Minor cracks in one of the basement shear walls were left unrepaired. Damage due to the Northridge earthquake was very similar to that which occurred in 1971, with cracking in the columns around the joints with the first floor beams (Plate 6.114). These cracks were repaired by epoxy grouting three days after the earthquake and there were no visible signs of any damage when visited subsequently on 3 February.

The building was instrumented (CSMIP No 24322 - Figure 6.14). Peak accelerations recorded at basement level were 0.46g north-south, 0.24g east-west and 0.18g vertically. Both the magnitudes and frequency contents of the ground level and basement records were significantly higher than those of the upper storeys, suggesting that the building response was dominated by soft storey behaviour of the high ground floor. In particular, the peak horizontal acceleration of 0.9g at ground floor level is reduced to 0.56g at the first floor.

6.6.6 Radisson Centre Hotel, Sherman Oaks

The building is located on Ventura Boulevard at Orion Avenue, about 10km from the epicentre. The closest recording station was the Transworld Bank (see 6.6.5). It is a 13-storey shear wall structure, having blocks of different heights (Plate 6.115). The main building is, in fact, connected on the front to a three-storey block, and on the back to a parking structure. It appears that the lower blocks constrained the main part of the structure, or even pounded against it with concentrated damage adjacent to the roof of the lower structure and at the level above (Plates 6.116 and 6.117, see also 6.3.6). There was sliding shear damage to the cladding panels below the region of the impact (Plate 6.116). In the lower block there was diagonal cracking in the panel below the region of impact. This block with a vehicle access below was open at ground floor level, where several of the columns had suffered shear failure and a soft-storey mechanism was developing, with the cladding panels above protecting the upper storeys (Plate 6.116). The building remained in use for several days after the earthquake, but was red-tagged on 20th January 1994.

6.6.7 Armed Forces Recruiting Centre, Santa Monica

The building is located at the Santa Monica Boulevard crossing 20th Street, about 23km from the epicentre. At the closest recording station (CSMIP No. 24538, Santa Monica City Hall Grounds) peak values of 0.93g horizontal and 0.25g vertical acceleration were measured. It is a six storey dual frame/shear wall structure with the shear walls, which had openings, located at one end (Plate 6.118). These shear walls had flexural/shear cracks (Plate 6.119), with flexural/compression failure at the tips (Plates 6.120 and 6.121). The frame-shear wall connection failed at the first storey.

In the frame at ground floor level on the north side (facing the street) a column had split down the centre, from top to bottom (Plate 6.122), damage which could only be attributable to axial compression. The adjacent column however was cracked transversely top and bottom (Plate 6.123) due to direct tension suggesting that there was a local couple between the two columns. A couple could be induced here by the upstand beam along the street, which terminated at the column cracked in direct tension. High vertical acceleration may have contributed to the damage.

6.6.8 Champagne Towers, Santa Monica

The building (Plates 6.124, 6.129 and 6.130) is located in Santa Monica, at the intersection of Ocean Avenue and Wiltshire Boulevard, about 24km from the epicentre. At the closest recording station (CSMIP No 24538, Santa Monica City Hall Grounds) peak values of 0.93g horizontal and 0.25g vertical acceleration were measured. It is the central of three high rise buildings on the front at Santa Monica (Plate 6.124). Being the oldest of the three, it was designed to the UBC of 1962, whereas the others were designed to post 1971 editions. Being built by the same developer they may be assumed to represent roughly the same standard of construction to the various editions of the UBC. Plates 6.125 to 6.128 show the adjacent Sumitomo Bank, which suffered only minor damage.

Champagne Towers is a 16 storey building. Horizontal loads are resisted transversely by coupled shear walls and longitudinally by non-ductile sway frames, though the columns are no larger and no more densely reinforced than might be expected of columns carrying only gravity loading.

Over the lower part of the building the columns have severe X-cracking on the north-east face (Plate 6.129, 6.131 to 6.133), as did the coupling beams on the south-east shear wall up to the 12th floor (Plate 6.134). The damage could have occurred due to the stiff lower storey (as in the Holiday Inn) on the north-east face (not visible in Plate 6.129 due to a low building in front), in which case it would have propagated from the second storey upwards. A contributory cause of the damage is the stiff parapet beam on the damaged face cast monolithically with the columns (Plate 6.129), in contrast with the flexible rail on the undamaged west face (Plate 6.130). This alone may have been sufficient to cause the cracking on the facade, but it could not have produced different forces in the connecting beams in the shorter faces either end. With all the structure on one facade stiffer than on the facade opposite it could have induced a torsional response. However, since the internal layout is unknown, torsional behaviour could have been caused by an eccentric core.

Temporary strengthening had been provided by plates and external ties around the fractured columns and supplementary steel columns outside the parapets.

It is noteworthy that the damage is most severe in the central part of the facade in which there is an external corridor, and where the columns are unrestrained over part of their height. Columns with cladding between are far less damaged. It is likely that the external parapet stiffened the external columns and attracted load from the internal columns. Less damage would be expected internally.

6.6.9 Buildings with cracked beams

Of the tall office buildings in downtown Santa Monica, up to just over 20 storeys a number were slightly damaged lower down, as in an 11-storey office building and a 13-storey apartment building, both of which had predominantly horizontal cracking in and around window panels in the lower storeys, probably of no structural significance (see Plates 6.135 and 6.136).

The most heavily damaged was a similar building with X-cracking in the beams (Plate 6.137), which was the only framed building observed by the team with cracking in the beams and not columns. There was nevertheless a possibility of cracking behind the precast concrete cladding units. Comparable damage with minor structural significance is discussed in 6.12. The damage to these buildings is representative of the damage most commonly experienced by most medium rise buildings in the epicentral region.

6.7 Lower engineered buildings (up to six storeys)

6.7.1 Introduction

This section covers the lower engineered buildings, excluding the semi-engineered tilt-ups considered in 6.9, the semi-engineered wood framed apartment buildings considered in 6.10.3 and the masonry buildings considered in 6.5.1 and 6.10.4.

According to a local structural engineer, William C Taylor, most of the three to six storey buildings in California built before 1933 were (like the schools discussed in 6.5.1) of unreinforced brick bearing wall construction. From 1933 to 1973 most of the buildings in this size range were of reinforced concrete shear wall or reinforced concrete non-ductile moment resistant frames, though a number were of steel. The reinforced concrete structures had been perceived to perform less well than the steel structures in the San Fernando Earthquake and after 1973 most buildings were of ductile steel construction (which includes braced frames and moment resistant frames), with a minority in more ductile reinforced concrete construction (which includes shear walls).

6.7.2 San Fernando County Building, San Fernando

This building is at the intersection of McNeil and Brand Streets in San Fernando. It was red-tagged at the time of the inspection and was by far the most important municipal structure badly damaged in the earthquake, and the only one complying with current standards, being constructed in the early 1980s. It is a four storey concrete shear wall structure, with liberal dispersion of 300 mm thick shear walls (Plate 6.138), which had cracked right through in a number of places. The main hazard was posed by the extensive shear failure in a number of unusually short gravity load bearing columns, without adequate confining reinforcement, trimming clerestory height windows between the shear walls (Plates 6.139 and 6.140). It is notable that, even with such an abundance of stocky shear walls, short columns are inadmissible in areas of high seismic activity. The damage was probably due to in-plane rocking of the shear walls. The structure provides a warning against the philosophy of employing an earthquake resistant structure to provide lateral support to structures not so designed, a type of construction permitted in the UBC under the description of "bearing wall system". The performance of this building could have serious implications for the retrofitting of existing buildings.

6.7.3 Triad Properties Apartment, Van Nuys

Most of the ground floor of this two bay, three storey apartment building on Sherman Way, near the intersection with Sepulveda was occupied by a car park. The floors throughout were of flat slab construction and punching shear failure occurred in the first floor slab around the internal columns, with the sides of the building collapsing inwards (Plates 6.141 to 6.143).

6.8 Steel structures

The 50 storey skyscrapers of downtown Los Angeles, well to the south of the epicentre, which are mostly of steel, were barely affected the earthquake with peak ground accelerations typically in the range of 0.15g to 0.20g. There were no reports of damage to steel framed structures when the team visited, but later inspections revealed that there has been cracking in the beam to column joint welds in low and medium rise structures, though none in the skyscrapers.

The design concept of fully restrained moment connections is often used and the California Seismic Safety Committee and a steel industry task committee was established to review the current design codes. The major damage was on the six storey Getty Centre and the four storey Oviatt Library where the cracks extend from the column flanges into the web (Figure 6.15). In the extensively damaged Santa Clarita City Hall and the four storey US Bovax headquarters cracking was confined to the welds. To date structural steel experts have tentatively ruled out poor welding techniques on site and low quality steel.

Most of the damaged steel buildings appear to be medium rise buildings of 1970s construction in the Santa Clarita area, about 19km to the north of the epicentre (Bertero et al, 1994). The closest recording station is Los Angeles County Fire Station in Newhall (CSMIP No 24279), with peak ground accelerations of 0.63g horizontally and 0.62g vertically. The details used in steel moment resistant frames in the United States incorporate some features which facilitate erection, for example bolted connections (Figure 6.16) but despite the general use of full penetration butt welds in the flanges they are less ductile than the details for which the ductilities in Eurocode 8 have been derived, Figure 6.17 (Scott Wilson Kirkpatrick, 1990). A problem arises, in that in the UBC (as supplemented by AISC practice) the overstrength factor used in the design of the connections is unity for butt welded connections, but it may be reduced to the strength of the intersection zone if lower. Eurocode 8 adopts the same overstrength factor, but does not take into account the intersection zone.

The modest ductility of the type of connection shown in Figure 6.15 has been recognised for some time; Tsai and Popov (1988) published rotation capacities of a range of variations on the detail in Figure 6.15, only a few of which would nowadays be considered satisfactory. A solution to the pattern has been suggested in the form of a reduced flange width to reduce the moment capacity of the member. This unfortunately has been patented, which will restrict its use and possibly its development.

At the time of the visit the only steel structure known to have been damaged by the earthquake was a large scoreboard attached to the roof of the Anaheim stadium, which partially collapsed. Damage at the stadium however was extensive. In contrast to the poor performance of concrete parking structures (see 6.3.12), those of steel performed well (Bertero et al, 1994).

6.9 Tilt-up buildings

Tilt-up buildings have substantial precast concrete wall panels. In the Los Angeles area, where there are 200 or more, they are normally one storey or shed type structures up to 300 m long by 60 m wide. Recently, however, multi-storey forms have been introduced, which are viewed with concern by those responsible for public safety. Internal partitions, if any, are flimsy.

Until 1976 the connection at the top comprised a ledger bolted to the wall panel and nailed into the plywood roof. In a third of the pre-1976 tilt-ups in the area most affected by the earthquake the nails pulled out; in a few the roof diaphragms failed in shear. More recent buildings of this type, by law must have substantial ties at the top and these buildings performed well in the earthquake. When failure occurred it was:

(i) at the corners where the maximum force occurs and there was inadequate connection between adjacent precast units;

(ii) the cracking of the precast units when they were used as shear walls in the external walls;

(iii)inadequate connections at the top to the roof diaphragm.

Since collapse outwards of the wall panels is invariably fatal for passers by, a new law was passed on 4 February 1994 requiring that all the older tilt-up buildings should be retrofitted.

In a modern tilt up near the Northridge Fashion Centre a rear wall collapsed after bolts pulled through a wooden beam. However the deformation of the alluvium may have contributed to this failure (tilt-ups are very rigid structures).

In a large two-storey tilt-up with semi-continuous fenestration irregularly dispersed around the perimeter, the shear over the height of the windows was carried in isolated RC panels. The side on which the first floor slab had been replaced by a light service floor fell (Plate 6.144), revealing a lack of horizontal ties. One of the corners opened (Plate 6.145) and some of the shear panels suffered shear cracking so severe that the concrete had fallen leaving only the reinforcement exposed (Plates 6.146 to 6.148).

6.10 Domestic housing

6.10.1 Introduction

With the exception of some older masonry houses and buildings with ground floor soft storeys, domestic housing performed well although the contents were severely damaged. Most of the housing stock is of light flexible construction.

6.10.2 Timber houses

In the San Fernando Valley, single storey timber framed houses (Plates 6.149 to 6.154), more often than not with timber cladding, are the most common buildings. The major development occurred after the 1960s, but the oldest (which were the more prone to damage due to their condition and construction) are up to 40 years older. Therefore, there are fewer houses without braced crawl spaces (the under floor area), which constituted a soft storey, than in the Loma Prieta earthquake. So collapse of the crawl space was rare in Los Angeles, though cases were noted (Plate 6.153). Overall, chimneys performed better in the

Northridge than in the Loma Prieta earthquake (EEFIT, 1993), which is attributable to the higher proportion of flexible chimneys in Southern California. Roof damage however was observed around undamaged chimneys (Plate 6.149). It is understood there were appreciable numbers of failures of internal brick chimneys and there were also many separated external brick chimneys, some of which collapsed, as in the Loma Prieta earthquake. However collapsed and severely racked porches were by far the most common form of damage (Plates 6.151 and 6.152).

6.10.3 Lower apartment buildings

The apartment blocks, frequented by students and by the recently arrived Hispanic community, are typically wood framed buildings of two or three stories without insulation and with weather protection provided by stucco. In the earthquake this was often damaged (Plate 6.155) and sometimes detached over the height of the crawlspace, and just above it (Plates 6.156 and 6.157). The thin timber backing to the stucco, common in new and old houses alike in North California, is apparently used for its insulation properties and is therefore considered to be unnecessary in the warmer climate of Southern California. Most of the buildings in the Northridge area had the construction form shown in Figure 6.18 and Plate 6.157. Insulation, when used, took the form of soft packing, providing no additional support (Figure 6.18 and Plate 6.157). However, this is an older form of construction, and unacceptable without bracing under current regulations. In the San Fernando Valley cracking of the stucco was commonplace and occasionally it had entirely collapsed. Most of the newer buildings (and some older ones) have plywood sheets or thin timbers nailed onto the timber frame (Figure 6.19 and Plate 6.156) in order to provide greater horizontal stiffness and strength.

Partitions between apartments must be designed for both sound insulation and fire resistance. For the latter reason, masonry partitions are often used. The main alternative is to provide two structurally separate timber and plasterboard walls, separated by a lead sheet (Figure 6.20).

The form of domestic housing which suffered the greatest damage was apartment blocks with soft ground stories. Whilst collapse of these buildings was rare, most had experienced significant movement and were perceived as unsafe by the former occupants, who comprised most of those temporarily housed in the tent cities administered by FEMA.

A number of smaller apartments along Dickens Street (running east-west), one block south of Ventura Boulevard, were badly damaged. The buildings along this street were mostly two to four storey timber framed houses/apartment buildings with concrete basement walls. About a quarter had open parking areas at the bottom of the structure (Plate 6.159). Many had suffered stucco cracking, most often in the form of horizontal cracks at the interface between the timber structure and the concrete basement walls. Diagonal cracking from window corners was also common. One had suffered some damage in a soft storey area at the front (Plate 6.159), but much more severe damage at the rear, where a three storey section of the structure had broken away from the remainder, and dropped by a storey, with collapse of the part between (left hand building in Plate 6.158 and the building in the rear in 6.160).

A close inspection was made of the damage in a three storey condominium (Plate 6.161) in which substantial timber posts were part of the main vertical load bearing system. The structure was weakened by car ports (Plate 6.162). The damage was concentrated in a band close to the ground (Plates 6.162 to 6.165) with damage somewhat higher up only in a wall in the car port (Plate 6.166). The building is of principal interest because of its close similarity to the Northridge Meadows Apartments, the collapse of which caused the most concentrated fatalities in this earthquake. An interesting feature was the vertical splitting of the timber of the post in the car port, which carried the upper floors. The metal in the bolted connection was undamaged (Plate 6.165).

6.10.4 Brick buildings

There are few buildings with unreinforced masonry walls in the San Fernando Valley, a fact attributable to its recent development (see 6.1.2). There were some in Sylmar to the east of the epicentral region, which had been badly damaged in the 1971 earthquake, and a few in San Fernando, but these had mostly been replaced by more modern structures. Most of those affected by the Northridge earthquake were in Santa Monica (which though remote from the epicentre was affected by strong shaking horizontally), in the vicinity of the collapsed I10 bridges (also remote from the epicentre - see Chapter 5) and along a five mile stretch of the Hollywood Boulevard.

In the few masonry buildings, mostly of older construction and some with shop fronts, the rafters, trusses and floor beams generally span from side to side, so the back and front, being unloaded vertically are more vulnerable than the sides and transverse walls. For this reason the fronts of an occasional house and shop collapsed (Plates 6.167 and 6.168). Often however the roof construction would appear from the number of side walls fallen away below an undamaged roof (Plate 6.169) to span the other way. Even so parapets are the feature of masonry buildings most vulnerable to collapse (as discussed in 6.11.2.).

The brick walls in Los Angeles are generally of two leaves without ties or cross-bonded bricks but an old wall was found in Santa Monica which had one cross-bonded course every seven courses. These were effective in holding the wall together, but in one cross-bonded course some bricks failed and the outer leaf was lost down to the cross-bonded course below, (Plate 6.170 and 6.171). Lower down the mortar had cracked and the pointing spalled away (Plate 6.172).

6.10.5 Reinforced masonry

Some of the columns in the facades of old masonry shops when heavily damaged were found to be an early form of reinforced masonry. These were possibly as old as 1920 or were constructed soon after the Long Beach Earthquake of 1933 (also see 6.5.1) (Plate 6.173). In these the masonry columns had cores comprising substantial steel posts of diameter about 75 mm. These were unbonded to the masonry and impacting of the posts upon the masonry is undoubtedly the cause of the pronounced longitudinal splitting of such columns (Plate 6.174). Indeed this form of damage may be considered a good indication of the presence of these posts, except when the cracking is confined to the column feet, when it is likely to be caused by the indirect tensile stress. Where unprotected (in Santa Monica) the iron posts were corroded to a depth of 3 mm (Plate 6.175), but they were also found (in San Fernando) protected by a tar-based fabric which was undamaged in the earthquake (Plates 6.176 to 6.178).

A large modern brick masonry building with very long clerestory type windows (Plate 6.179) reinforced in every bed course suffered in-plane cracking at the corner of a window, followed by local out-of-plane failure of the zone affected (Plate 6.180). As this was along the Hollywood Boulevard the ground acceleration is likely to have been in excess of 0.30g and the movement of the ground at the base of a tree due to root tensions (somewhat unusual for this earthquake) in the street suggests it may have been higher (Plate 6.181).

The highest building of reinforced masonry noted was the eight storey residential block at CSU Northridge discussed in 6.5.2. Severe disruption occurred to the external masonry shear walls comprising two leaves (called 'wythes') of brickwork with a single grid of reinforcement embedded in a thin mortar layer between the bricks. The construction is clearly visible in Plate 6.182, and, apart from the use of larger horizontal bars at greater spacing, is the same as the construction visible in Plate 6.180. Both buildings performed less well than the ten storey Dream Inn Hotel in Santa Cruz in the Loma Prieta earthquake (EEFIT, 1993), which like the CSU Northridge building was of 1960s construction.

Some modern buildings (mostly shops and small factories where there is no surface coating), suffered extensive cracks with the outer brick breaking away from the reinforced core. An example of this was seen on the five storey Berkeley Convalescent Hospital (Plate 6.184), where 0.93g horizontal acceleration had been measured at the closest recording station. There was damage around much of the perimeter in the first storey (Plate 6.185). The short walls between the windows at the first storey suffered serious X-cracking, with buckling of the unrestrained vertical reinforcing bars (Plates 6.186 and 6.187). Similarly the bars in the vertical cores (RC posts integral with the masonry) at the end of the walls buckled outwards (Plate 6.188). In both situations the damage was attributable to the lack of horizontal ties restraining the vertical reinforcement. In this regard the construction did not comply with the requirements for reinforced masonry in the UBC.

6.11 Retrofitted buildings

6.11.1 Engineered buildings

Attempts were made to locate older reinforced concrete and steel buildings which had been retrofitted on being found wanting after an assessment using ATC 14 (ATC, 1987). Whilst it is suspected that many structures were effectively retrofitted in repairing the damage after the 1971 earthquake the only instance encountered is that reported in 6.6.3. Two reinforced concrete buildings described as retrofitted in conversation (the buildings in 6.3.4 and 6.6.2) had not in fact been deliberately seismically retrofitted (though one of them could be said to have been effectively partially retrofitted). On the last day of the visit one seismically retrofitted reinforced concrete commercial structure was identified.

Isolated cases of retrofitted industrial structures which performed well are reported in the literature. These include an old reinforced concrete structure (of undefined construction) which was strengthened by adding shear walls, precast concrete shear wall buildings (presumably including tilt-ups) and a building with a very congested interior which was retrofitted with an external braced frame (EQE, 1994).

It is understood the reason why seismic retrofitting in the non-industrial areas is so rare is other legislation which obliges owners when upgrading buildings also to:

- upgrade the sprinkler system,
- replace asbestos, and
- comply with the Americans with Disabilities Act,

but there is no requirement to retrofit them, unless they are tilt-ups (see 6.9).

6.11.2 Retrofitted masonry buildings

The programme for retrofitting unreinforced masonry buildings was well advanced by the time of the 1971 earthquake and 80% are now completed with half of the remainder being classed as dispensable. Generally, the retrofit takes the form of steel rods inserted through the walls anchored at the exterior by steel plates and internally by timber blocks at floor beams. Raking rods anchoring the parapets are tied to the roof diaphragms. From a survey of a number of these buildings made by the team the system was found to be quite effective as many of the two and three storey buildings suffered no damage. Several retrofitted masonry buildings are shown in Plates 6.189 to 6.207, with damage levels ranging from very light cracking and delamination to localised collapse.

One failure of a two storey building occurred on Washington Boulevard, approximately 25 km from the epicentre where peak values of 0.24g horizontal and 0.10g vertical acceleration were measured at the closest station Baldwin Hills. The building had been strengthened only at roof level (Plates 6.189 and 6.191); therefore the outer walls suffered out-of-plane failure as they bowed away from the unconnected first floor diaphragm (Plate 6.190).

Cases were noted where the anchor plates at roof level in wide buildings appeared to have pulled through the wall, contributing to the damage (Plates 6.193 and 6.195). The most common feature was the failure away from the anchors, either in areas without anchors or where they were insufficiently close to hold the outer wall to the floor diaphragm. When flats had been used instead of anchor plates on a few buildings, they had performed well (Plates 6.198 and 6.207). Some of the larger and discretely retrofitted buildings in Santa Monica which performed well are show in Plates 6.208 and 6.211.

A particularly interesting case reported in the Daily News (30 January 1994) concerned a three storey masonry apartment in Hollywood with car ports on the ground floor, which had been retrofitted by steel frames around the car ports, as had been noted on buildings in San Francisco (EEFIT, 1993). The retrofitting is considered to have prevented collapse, though there was damage.

6.11.3 Retrofitted RC buildings

Very few older RC buildings had been retrofitted in the area affected by the earthquake. Of the five identified by local engineers (Holmes and Somers, 1996), three were 7 storey halls of residence at CSU Northridge. These were of 1960s construction and the columns were deficient in both confining and shear reinforcement. They also suffered from discontinuities in the shear walls in the lowest storey and from strong beam/weak column characteristics. The buildings had been retrofitted by jacketing the frames (excluding the beams at higher levels), eliminating the discontinuity in the shear walls or strengthening the columns beneath them.

In Santa Clarita a 2-storey post-tensioned flat slab building, had been retrofitted by adding upstand beams, whilst preserving the weak beam/strong column characteristics of the original construction. This was undamaged despite the severe shaking.

An old three storey RC building with shear walls providing the lateral stability, with a weak storey at ground level, had been retrofitted with two shotcrete walls. The retrofit walls sheared right through, severing the adjoining columns. They were considered to have carried the entire lateral force as the shear walls had slid on the construction joint at the floor level, partly no doubt on account of insufficient composite action with the new walls at the top.

An 8-storey industrial shear wall building in Van Nuys had been retrofitted by thickening the existing shear walls and adding new ones. The walls suffered minor cracking around openings, some extending cracks caused by the San Fernando earthquake.

The repair and strengthening to the Indian Hills Medical Centre described in 6.4.7 after the San Fernando earthquake had clearly strengthened the structure, since it suffered significantly less damage in the Northridge earthquake.

Bullocks Store in the Northridge Fashion Centre is the only retrofitted RC building which failed. It is a 3-storey structure with floors of coffered flat slab construction. Punching failure occurred on the column perimeters revealing a lack of slab reinforcement through the columns. The first and second floors collapsed over a large part of the building, leaving many of the columns unsupported laterally over three storeys. Shotcrete shear walls had been added and these performed well in resisting the lateral loads, despite a probable lack of mechanical connection to the frames. The collapse is further evidence of the vulnerability of elements designed only for gravity loads, which during earthquakes may experience frame moments far in excess of any used in the initial design, in addition to the possible effects of vertical acceleration.

6.12 Irregular buildings

Irregular buildings are of particular interest as a new perspective has been added to the observation that excessive stiffness on one side of a building can cause damage on the opposite face (EEFIT, 1993). It now appears from studies of two buildings that if the excessive stiffness on the one side is discontinuous with height the structure immediately above the level at which it terminates is more vulnerable to damage than the structure on the remote face. These two buildings had a common feature which may be necessary for this to occur. This is that the opposite faces of the building resist loads in opposite directions, and therefore carry most of the torsion unless there is severe discontinuity of the floor diaphragms (which is a consideration which did not apply to either building).

In the Sumitomo Bank in Santa Monica (the more northerly of the buildings next to Champagne Towers, see 6.6.8) in Plates 6.125 and 6.126, damage occurred in a particularly irregular first storey, where there were massive columns on each corner and where one side was infilled. It is probable, however, that the damage was confined to a part of the structure which was neither part of the intended vertical nor horizontal load resisting systems (Plates 6.127 and 6.128).

Cladding in its various forms suffered particularly at discontinuities, as in a masonry clad office building with continuous fenestration generally, except at an unperforated service core one end. The masonry had a vertical crack between the core and the offices which was emphasised visually by the mismatch of the colour of the bricks used in the repair (Plates 6.212 and 6.213).

Samples of the more irregular recent structures are shown in Plates 6.214 to 6.219, which illustrate the extent to which regularity rules can be infringed, provided the irregularity is adequately taken into account in the design.

Two masonry clad buildings with abnormal vertical discontinuities are shown in Plates 6.214 and 6.215. The lower was undamaged and the taller suffered minor cracking through the column cladding at the springing points of the arch.

The only damage found in an exceedingly irregular shopping complex on the coast in Santa Monica was a diagonal crack in the first floor tiling at a corner column; evidence of diaphragm action from the discontinuities in the vertical load resisting structure (Plates 6.216 and 6.217).

The two exceptionally irregular buildings shown in Plates 6.218 and 6.219 were undamaged, remarkable because even earthquake resistant design assumes some damage.

Some low rise buildings (timber framed, reinforced concrete and masonry) were highly irregular in ways other than the soft ground storey defect responsible for so much damage in the Loma Prieta earthquake. Considering the generally greater severity of the ground shaking in this earthquake, damage to such structures was less common than might have been expected (cases are considered in 6.10.3). The penalty for extreme irregularity in plan was minor damage. For example, in a church in San Fernando (Plates 6.220 and 6.221) there was cracking in the stucco at the change in horizontal and vertical geometry above the porch. In one of the wings of an attractive U-shaped masonry building in Santa Monica (Plate 6.225) there was local loss of stucco over the height of the crawl space

Vertical irregularity of low rise buildings with stiff external walls and irregular penetrations led to extensive cracking. In a very long low building with stiff unperforated cores each end, rotation of the core foundations probably contributed to diagonal cracks from the corners of windows and vertical cracking of spandrel beams over the entire length of the building (Plate 6.222). In a shorter building with an unperforated wall one end, damage was less severe and more evenly distributed, with possible wall rotation occurring from a compression failure in the wall (Plate 6.223). In a comparable building with a car port at one end, damage was confined to cracking at the external wall above the entrance (Plate 6.224). The features responsible for such damage are generally clear and they may sometimes be justified by such overriding architectural considerations.

Even a house perched precariously on the ridge of an escarpment in a similar location to that causing the collapse of houses in the Loma Prieta earthquake was undamaged (Plate 6.226), presumably on account of it being founded on rock. This was close to the landslips of the Palisades.

6.13 Infilled frames

There are fewer masonry infilled frames in Los Angeles than in San Francisco and local bylaws require that the infills are tied into the frames. Nevertheless there are more infilled frames than is at first apparent as they are disguised by wall finishes, which are surprisingly ductile (see 6.6.2). There are no records of either in-plane or out-of-plane failure of infilled frames at Northridge, indicating the efficiency of the ties.

A number of instances of panel cracking was observed; representative situations are discussed in 6.3.10 and 6.4.2. Damage to part of a frame was also noted, also in 6.4.2.

A special study was made of older infilled RC frame structures (Holmes and Somers, 1996) which showed that the most damaged were concrete frame buildings with clay-tile infills. In one the spandrel beams had cracked. In another, one clay tile infill panel had collapsed. Instances were noted of diagonal cracking in terra-cotta facings, cracking and some falling of poorly bonded plaster (poor bonding was most common with clay tiles). However in no case was the damage sufficient to reduce the lateral load resistance of the building significantly, even in the building with the fallen panel. It was concluded that of the older concrete framed buildings, those with infills tended to be safer than those with none. The infills tended to prevent the large deformations which cause so much damage in poorly confined RC elements.

6.14 Cladding, fixtures, fittings and contents

The stucco on the timber framed apartment buildings is discussed in 6.10.3 and the flexible wall finish believed to be common on reinforced concrete buildings is discussed in 6.13.

A number of the more prestigious modern commercial properties were clad in masonry. They tended to be somewhat irregular in the ground floor storey with large entrance lobbies and large masonry-clad columns supporting decorated masonry arches of relatively long span. These were generally well built and none of those observed had suffered extensive damage. Nevertheless in a high proportion there was significant cracking and loss of masonry at the tops of the columns and large panes of glass were often broken.

Cracking in cladding and stucco alike was often most evident when repaired and in the case of brick masonry there was seldom any attempt made to disguise the cracking.

Damage to unrestrained items is considered in 6.5.1, 6.6.2, and 6.10.1 and to fixtures in 6.4.10, 6.4.11, 6.5.2 and 6.6.2.

Of particular interest is how and where in buildings in seismically active regions it is best to store sensitive equipment and fragile objects likely to be damaged in earthquakes. Failing base isolation there would appear to be considerable advantage in housing them in a protected area at the base of the building as demonstrated in Table 6.6. It is seen in the table that at the base accelerations were on average 75% of the free field horizontal accelerations and 79% of the free field vertical accelerations, a feature noted for the smaller number of appropriately instrumented buildings in the Loma Prieta earthquake. Strangely there is no pronounced increase in the effect for the taller and more heavily laden structures (Figure 6.21) and it may be that low aspect ratio is the predominant effect. There is however a pronounced variation with epicentral distance (Figure 6.22), with the effect on the horizontal accelerations reducing with the distance and the vertical accelerations increasing. These comments take no account of the ground conditions, true mass per unit area or the rigidity, all likely to be significant parameters influencing this effect.

In the Getty Museum the approach taken was to base isolate the individual exhibits; one however toppled (BBC broadcast).

6.15 Conclusions of engineering significance

1. The use of isolation techniques to reduce accelerations by "detuning" was successful, all the controls and equipment remaining intact so that the vital facilities were operational after the event. One case study where there was restricted movement of the bearings highlighted the importance of a maintenance programme to ensure the physical gap at all times. A larger earthquake with this construction error may have severely damaged the building above.

2. The practice in design and construction of car parks should change due to:

   (i) The inadequate connection details between precast elements and the in-situ concrete.

   (ii) The higher dynamic effects than conventional buildings, where cladding etc acts as a natural dampener.

3. The practice of employing ductile moment resistant frames only on the perimeters of buildings should be discontinued as it leads to a lack of continuity between beams and columns, and provides a load resisting system with very limited redundancy.

4. The relatively poor performance of hospitals may be attributed to either the torsional effects due to irregular building shapes or the poor detailing, a major fault being the inadequacy of diagonal bars in the coupled beams of shear walls. As health care facilities are of importance during the post earthquake emergency phase, their design approach is likely to be reviewed. The possible solution of providing an elastic design as at the Olive View Medical Centre highlights the greater accelerations occurring which requires the services to be more heavily secured to ensure operations after the event. A half way solution may be to permit limited ductility to occur, which will not affect operational performance.

5. In zones which may be close to epicentres of future earthquakes, vertical effects will be important. Therefore cantilevers should be eliminated by bracing and suspended walkways tied down.

6. Frame-shear wall interaction may lead to unforeseen behaviour, for example base rotation of the walls. The interface structure should be carefully detailed (eg Denny's car park and the Holiday Inn).

7. The practice of combining within the same structure vertical elements, some of which are capable and some incapable of resisting lateral loading, is unsafe.

8. When detailing:

    (i) The concrete plastic length must not be under estimated.

    (ii) Structural elements must be designed to ensure that the shear strength is greater than the flexural strength.

    (iii) Stiff non-structural elements may change the structural response of a building and reduce ductility. Such elements should be isolated from the main structure.

    (iv) Floors must have sufficient in-plane strength to transfer horizontal inertial loads to the vertical elements, to which they must be adequately anchored.

    (v) Precast concrete shear walls must be adequately tied into the main structure.

9. In regards to masonry buildings:

    (i) Unreinforced masonry buildings tend to shake apart during strong earthquakes, with separation of the elements far more common than in-plane shear failure.

    (ii) Simple retrofitting techniques can greatly improve the performance, although it is doubtful if in most cases the performance can be made equal to that of reinforced concrete or steel structures.

    (iii) Reinforced cores in masonry buildings require throughout their lengths both links and ties into adjacent panels.

10. Buildings having blocks of different height and different ductility may perform badly (eg Radisson Centre) and the blocks should be separated.

11. Repair of the lower part of a building, if it appreciably alters the distribution of stiffness in plan or in elevation may alter the distribution of the horizontal forces higher up. It is probable that asymmetrical retrofitting of damaged parts could attract force to adjacent unretrofitted parts with serious consequences.

12. The fact that buildings are undamaged externally does not necessarily indicate the internal structure is sound. In fact cases were found in which the interiors of such buildings were so badly damaged that demolition was being considered by the buildings owners.

13. The structural codes used for the designs of buildings in California assume a peak horizontal ground acceleration of 0.4g (UBC, 1994), which represents the effect of a large earthquake at some distance from highly developed areas. It is implicit in this assumption that closer to the epicentre the degree of damage may be high, but as the extent of the area so affected is relatively small and on statistical grounds has been considered to be relatively sparsely populated, the total cost of the damage spread over the whole of the state on an annual basis is appreciably less than designing all buildings for higher ground motions. However California is now so intensively developed that it can no longer be assumed that the epicentre will be remote from highly developed areas, so the typical design situation is now better regarded as an earthquake of 6.5 to 7.2 in the immediate vicinity. In the recent past such a high proportion of these earthquakes have been in highly developed areas that there is a significant probability that a very large earthquake, which may be expected once or at most twice a century, will also occur in such areas. Consequently the economic justification for the present rules, together with appreciable loss of life to be expected in such an earthquake, may justify more stringent earthquake resistant measures and higher design accelerations, lower allowances for inelasticity and greater attention to regularity considerations.

14. Non-structurally it would appear:

    (i) Emergency drills by individual organisations do not adequately anticipate exceptional measures taken by others or the disruption to the highway network.

    (ii) Windows generally performed well in Los Angeles, without the widespread breakage of glass common in many earthquakes.

    (iii) Hydraulic fluid from lifts is treacherous and measures are required to contain it should it leak.

    (iv) Fixtures such as tiling may be more affected by sway displacement than by high acceleration.

    (v) Protected areas in the basement may be the best place to store sensitive and fragile equipment.

6.16 References

Applied Technology Council (ATC) (1987) "ATC 14: Evaluating the Seismic Resistance of Existing Buildings", Applied Technology Council for National Science Foundation, Redwood City, Ca.

E Backmann, M J Gomez and K C Change (1990) "Verification analysis of the base isolated Los Angeles County Fire Command and Control Facility", Proc. 4th US National Conference on Earthquake Eng., Palm Springs, Ca, Vol. 3.

V V Bertero, J C Anderson and H Krawinkler (1994) "Performance of steel building structures during the Northridge Earthquake", Report UBC/EERC - No. 9.

J A Blume and Associates "Holiday Inn: 8244 Orion Avenue, Van Nuys", Paper 29, Proc. of Conf. on the San Fernando Earthquake of 1971.

D M Cho and E Retamal (1993) "The Los Angeles County Emergency Operations Centre on high damping rubber bearings to withstand an earthquake bigger than the 'Big One'", Applied Technology Council Symposium, San Francisco, Ca.

Department of Commerce (1973) "San Fernando, California, Earthquake of February 9, 1971", US Department of Commerce: National Oceanic and Atmospheric Administration/ Environmental Research Laboratories, Washington, DC.

EEFIT (1993) "The Loma Prieta, USA, Earthquake of 17 October 1989", EEFIT Field Report, Earthquake Engineering Field Investigation Team, London.

EEFIT (1997) "The Kobe Earthquake of 17 January 1995", EEFIT Field Report, Earthquake Engineering Field Investigation Team, London.

EERC (1994) "Preliminary report on the seismological and engineering aspects of the 17 January 1994 Northridge earthquake", EERC Report UCB/EERC-94/01.

EQE (1994) "The January 17, 1994, Northridge, California Earthquake", Summary Report, EQE International.

FEMA (1988) "Rapid visual screening of buildings for potential seismic hazards: a handbook", Federal Emergency Management Agency, Earthquake Hazard Reduction Series 41. (To be revised to ATC-20)

J M Ferritto (1984) "Economics of seismic design for new buildings", J. Struct. Eng., ASCE, Vol 110, No 12, 2925-2938.

W T Holmes and P Somers (eds) (1996) "Northridge Earthquake of January 17th 1994: Reconnaissance Report - Volume 2". Earthquake Spectra, Earthquake Engineering Research Institute, Publication 95-03/2.

G K Hueffmann "Base isolation of two residential buildings in Los Angeles with helical springs and viscodampers", GERB Vibration Control Systems Inc, Westmond, IL.

P C Jennings (1971) "Engineering Features of the San Fernando Earthquake, Feb 9, 1971" Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, Ca.

C A Lynn, J P Moehle and A S Whittaker (1996) "Evaluation of existing seven-storey reinforced concrete building", 7th US-Japan Workshop on the Improvement of Structural Design and Construction Practices.

A Papageorgiu and Lin Bing-Chang (1989) "Study of the earthquake response of the home-isolated Law and Justice Centre in Rancho Cucamonga", Earthquake Eng. and Struct. Dyn., Vol 18, 1189-1200.

Scott Wilson Kirkpatrick (1990) "Inelasticity in Eurocode 8", Preliminary Studies on the Evaluation of Eurocode 8 for the Building Research Establishment, Scott Wilson Kirkpatrick.

R G Troy (1987) "Steel plate shear walls", Proc. Int. Conf. on the Design and Construction of Non-Conventional Structures, held at Institution of Civil Engineers, London, published by Civil-Comp Press, Edinburgh.

K-C Tsai and E P Popov (1988) "Steel beam-column joints in seismic moment resisting frames", Report UCB/EERC-88/19.

UBC (1994) "The Uniform Building Code", International Conference of Building Officials, Whittier, Ca.

Design acceleration	0.1g	0.2g	0.3g	0.4g
Ground acceleration	Probability of collapse			
0.4g	100%	45%	7%	2%
0.7g	100%	100%	65%	25-30%
Intensity	Damage Indices*			
MM VIII**	54%	18%	10%	5%
MM IX	100%	100%	47%	22%

* an index of 0 indicates no damage and 100% indicates structural collapse
** less severe than a ground acceleration of 0.4g

Table 6.1: Damage in Relation to Ground Acceleration

Structure	Owner	Engineers	Isolator	No of Isolators	Date
USC University Teaching Hospital	USC & National Medical Enterprises	KPFF	LRB	149	1989
Fire Command and Control Facility	Los Angeles County	Fluor Daniel	HDR	32	1989
County Emergency Operations Centre	Los Angeles County	DMJM & BTA	HDR	28	1992
Residential houses	Dr Ming-Li Lowe	Dr Ming-Li Lowe	GERB Resistant base	14	1991
Rockwell Building 80	Rockwell International	Englekirk & Hart	LRB	78	1990
Foothill Communities Law & Justice Centre	County of San Bernardino	Taylor & Gaines Reid & Tarics	HDR	98	1989

Table 6.2: Seismically Isolated Buildings in Los Angeles Area

Structural Type	Date (Number)	Floor Area (m^2)	Repair Cost ($\$/m^2$)
RC shear wall structures	1960-65 (9) 1973 (1) 1991 (1)	134,000 17,900 7,040	0.57 0.86 0.95
RC SMRF (car park)	1991 (1)	62.490	0.44
RC tilt-ups	1960-65 (1)	5,140	0.05
Load bearing masonry & braced steel frames (for lateral load)	1976 (1) 1984 (1)	2,540 2,590	0.22 0.03
Braced steel frames	1960 (1) 1991 (1)	590 10,290	0.02 0.75
Welded steel MR frames	1989-94 (3)	24,250	0.88
RC & reinforced brick	1959 (1)	3,250	0.10
RC & masonry shear wall	1967 (1)	13,630	2.6
Reinforced blockwork	1982 (1)	2,790	0.02
Wood frame & plywood shear walls	1985-91	72,000	0.24
Wood frame	1981	8,130	0.03

Table 6.3: Repair Costs at CSU Northridge

Floor or Storey	1st	2nd to 6th	7th
Dimensions:			
Vertically	4.1 m	2.65 m	2.64 m
Longitudinally	9 bays @ 5.7 m	as at 1st	as at 1st
Transversely	2 bays @ 6.1 m & 1 @ 6.35 m		
Columns:			
Internal	508 x 508 mm	457 x 711 mm	457 x 457 mm
External	356 x 508 mm	356 x 508 mm	356 x 508 mm
Beams:			
Longitudinal	762 x 406 mm	572 x 406 mm	559 x 406 mm
Transverse	762 x 356 mm	572 x 356 mm	559 x 356 mm
Slab:			
Thickness	254 mm	216 mm	203 mm
Reinforcement (assessed)			
Columns:			
Longitudinal	4 No 29 mm bars	4 No 22 mm bars	4 No 22 mm bars
Transverse	10 mm links @ 76 mm c/c for 380 mm, then 305 mm c/c, then 457 mm c/c		
Beams:			
Longitudinal	2 No 32 mm bars top & 2 No 19 mm bars bottom		
Transverse	10 mm links @ 127 mm c/c, then @ 152 mm c/c, then @ 254 mm c/c		

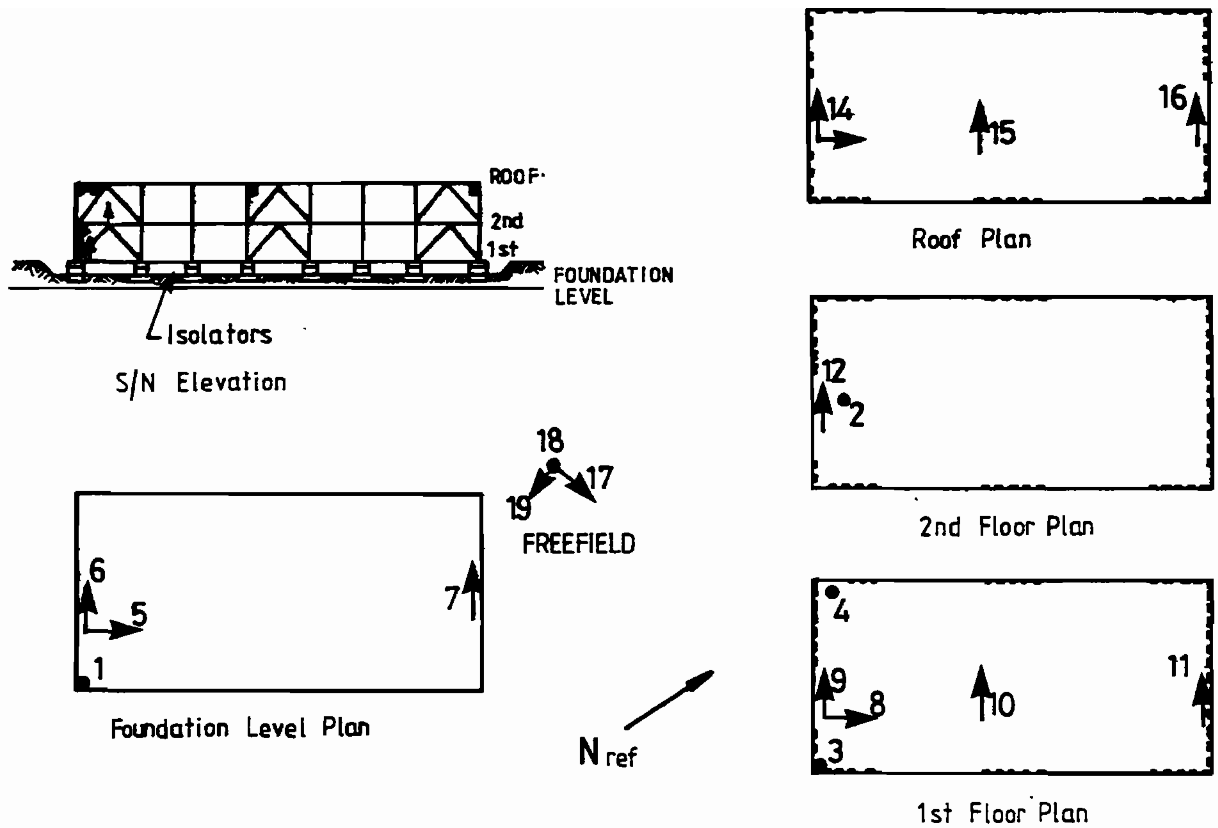
Table 6.4: Main Characteristics of the Holiday Inn, Van Nuys

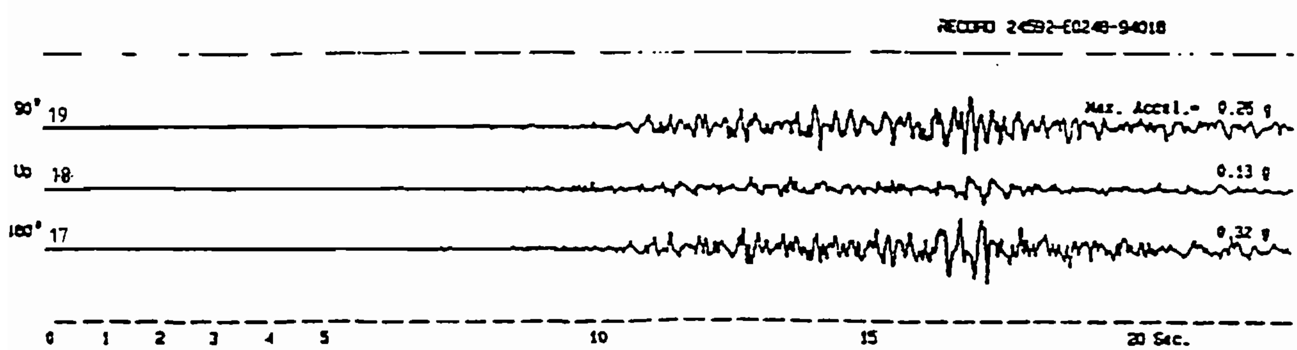
	1971 Earthquake			1994 Earthquake		
Direction	L	T	V	L	T	V
Top	0.04g	0.34g	0.26g	0.59g	0.59g	-
Mid height	0.21g	0.25g	0.24g	0.38g	0.43g	-
Ground floor	0.25g	0.15g	0.19g	0.47g	0.41g	0.30g

Table 6.5: Peak Accelerations in the Holiday Inn

Structure	Storeys	Location CSMIP Station No/ Epicentral distance	Free Field (a)	Base (b)	Structure (c)	(a)/(b)
Buildings						
Government office buildings	15	Long Beach No 14533 @ 59km	0.06gH 0.03gV	0.04gH 0.03gV	0.06gH 0.05gV	0.67 1.00
Base isolated office building	8	Seal Beach No 14578 @ 66km	0.09gH 0.04gV	0.08gH 0.03gV	0.15gH 0.16gV	0.89 0.75
Base isolated hospital	7	East Los Angeles No 24605 @ 36km	0.49gH 0.12gV	0.37gH 0.09gV	0.21gH 0.13gV	0.76 0.75
County hospital	6	Sylmar No 24514 @ 15km	0.91gH 0.60gV	0.82gH 0.34gV	2.31gH	0.90 0.57
Base isolated law and justice centre	4	Rancho Cucamonga No 23497 @ 90km	0.08gH 0.03gV	0.05gH 0.03gV	0.10gH 0.03gV	0.63 1.00
Base isolated fire command control building	2	Los Angeles No 24580 @ 39km	0.32gH 0.13gV	0.22gH 0.11gV	0.35gH 0.30gV	0.69 0.85
Garage building	1?	Hollywood No 24236 @ 23km	0.41gH 0.19gV	0.29gH 0.11gV	1.61gH	0.71 0.58
Average		Horizontally Vertically				0.75 0.79
Bridges						
I10/F210 Intersection	-	San Bernardino No 23631 @ 115km	0.10gH 0.04gV	0.13gH 0.04gV	0.47gH 0.31gV	1.30 1.00

Table 6.6: Relationship Between Base and Free Field Accelerations

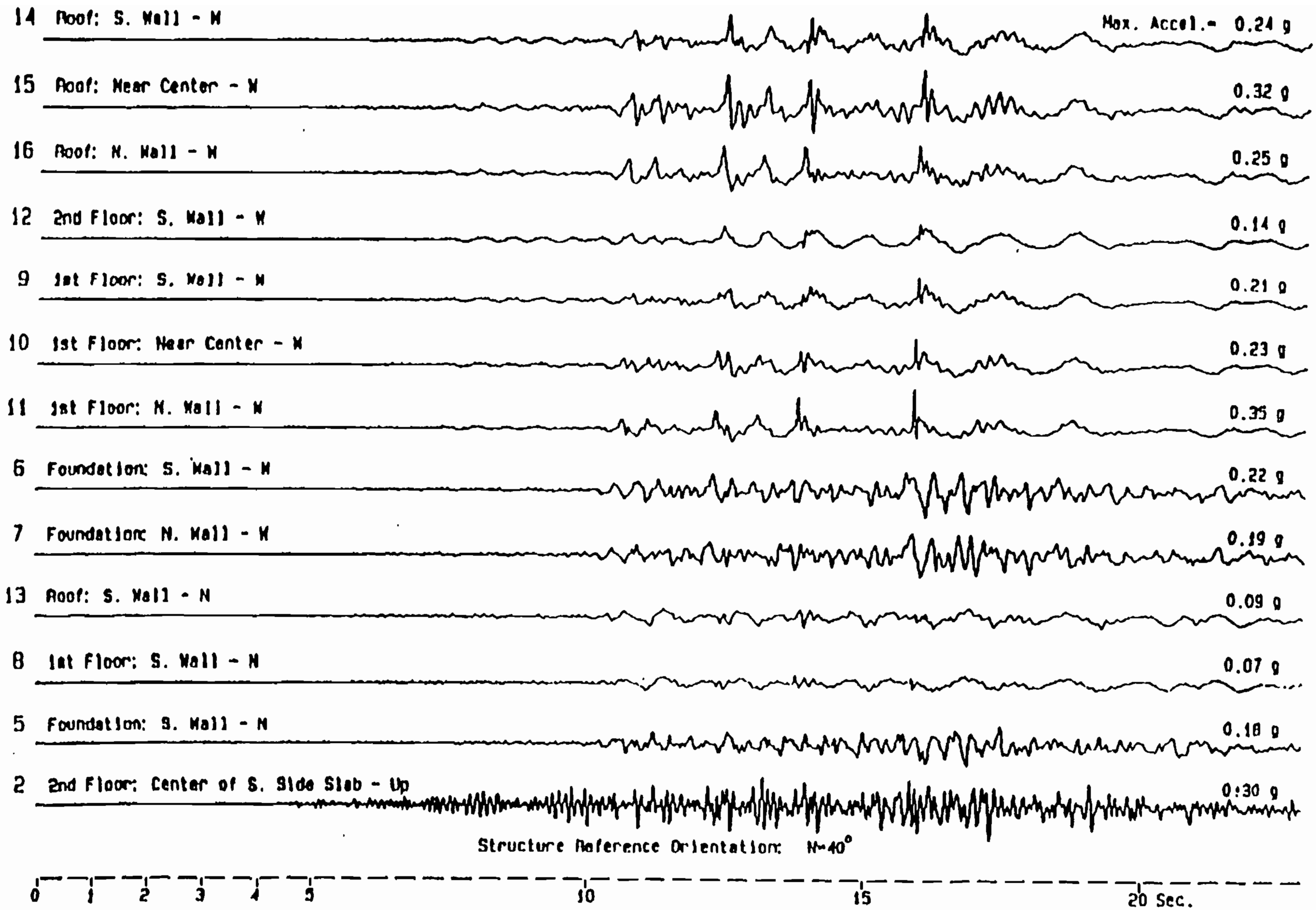




Los Angeles - City Terrace &Base Isolated Building

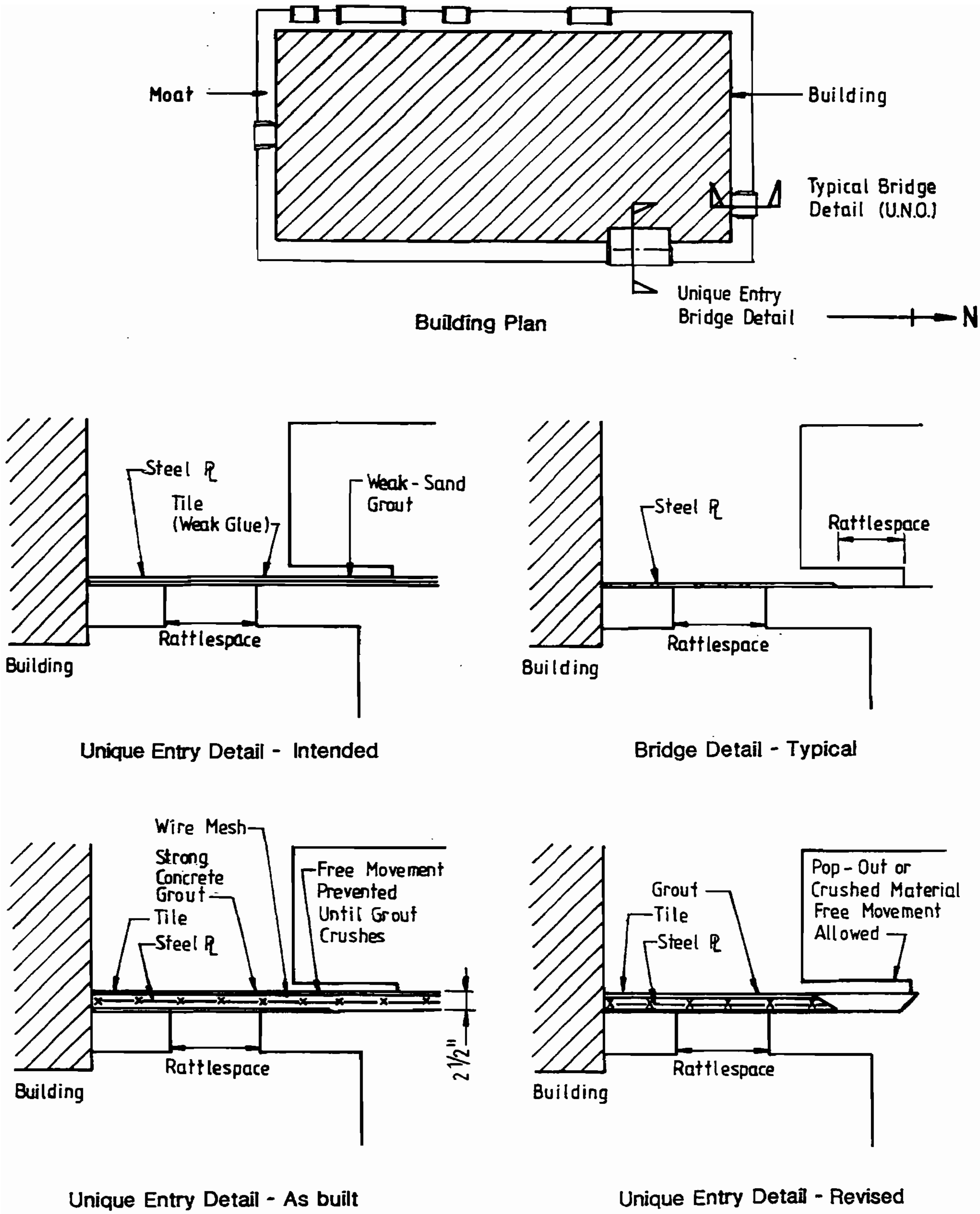
Los Angeles 2 Storey Fire Command Control Building
CSMIP Station 24580

Figure 6.1



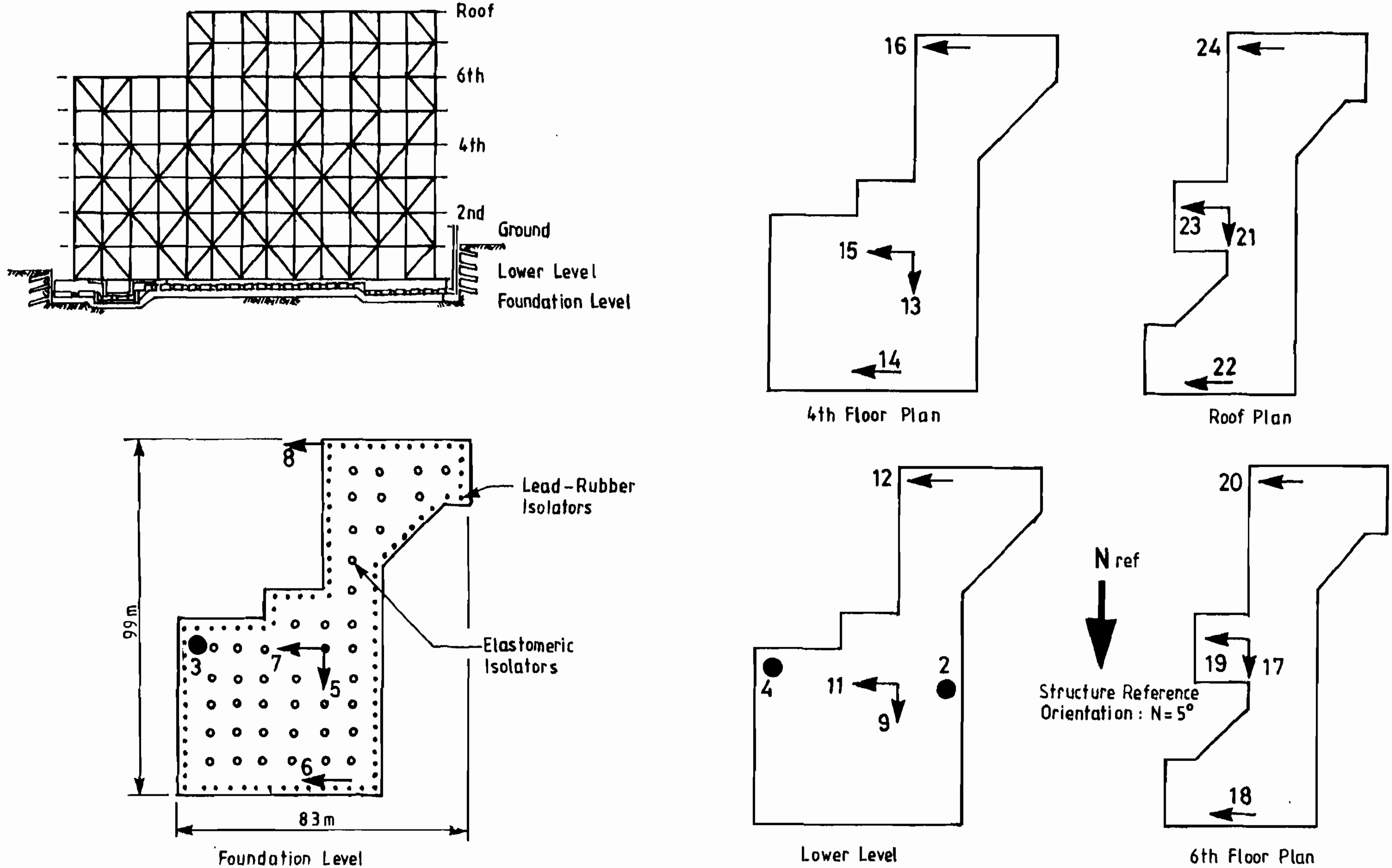
Los Angeles 2 Storey Fire Command Control Building
CSMIP Station 24580

Figure 6.2



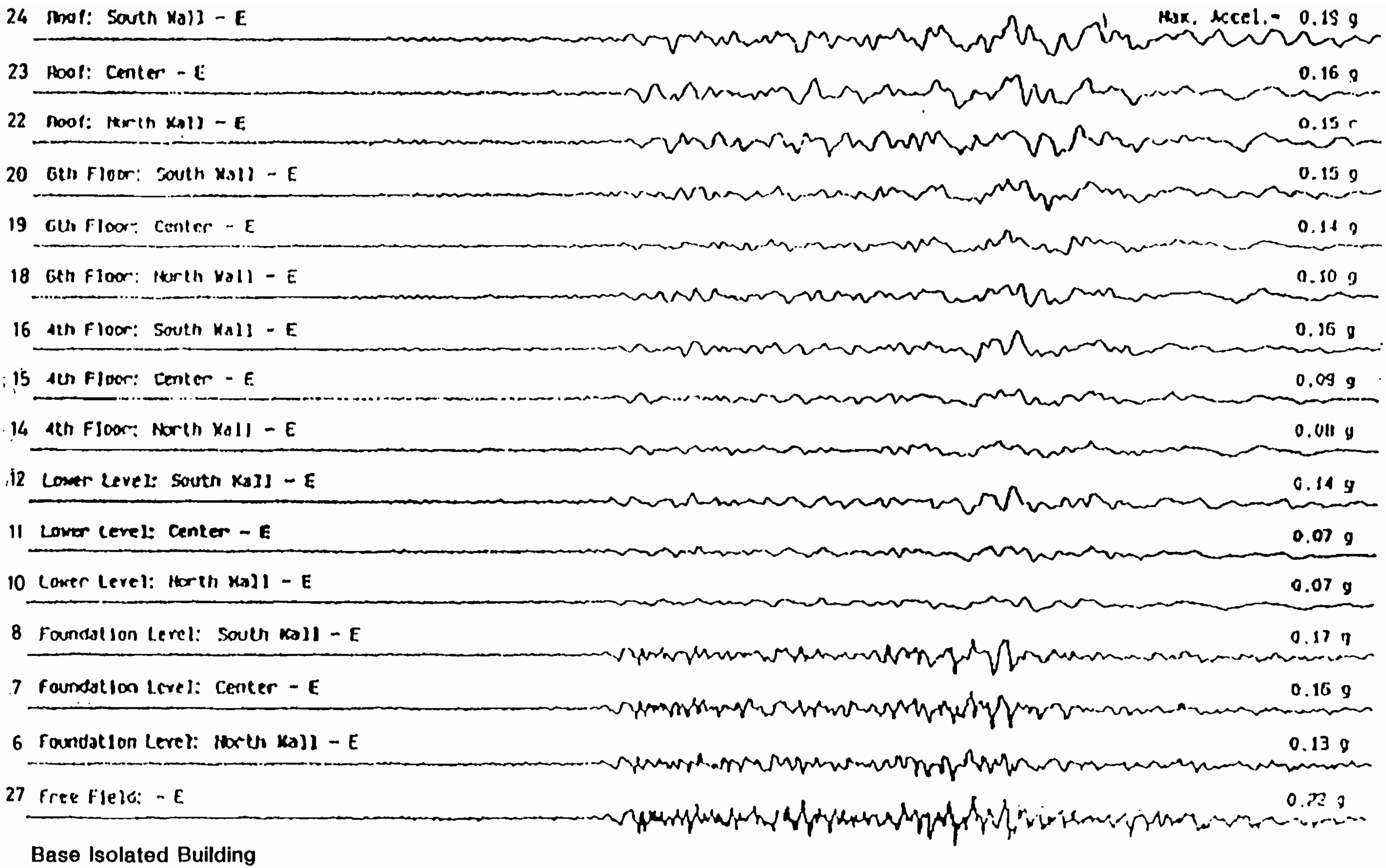
Los Angeles Fire Command & Control Facility (911)
Tile Entry Detail

Figure 6.3



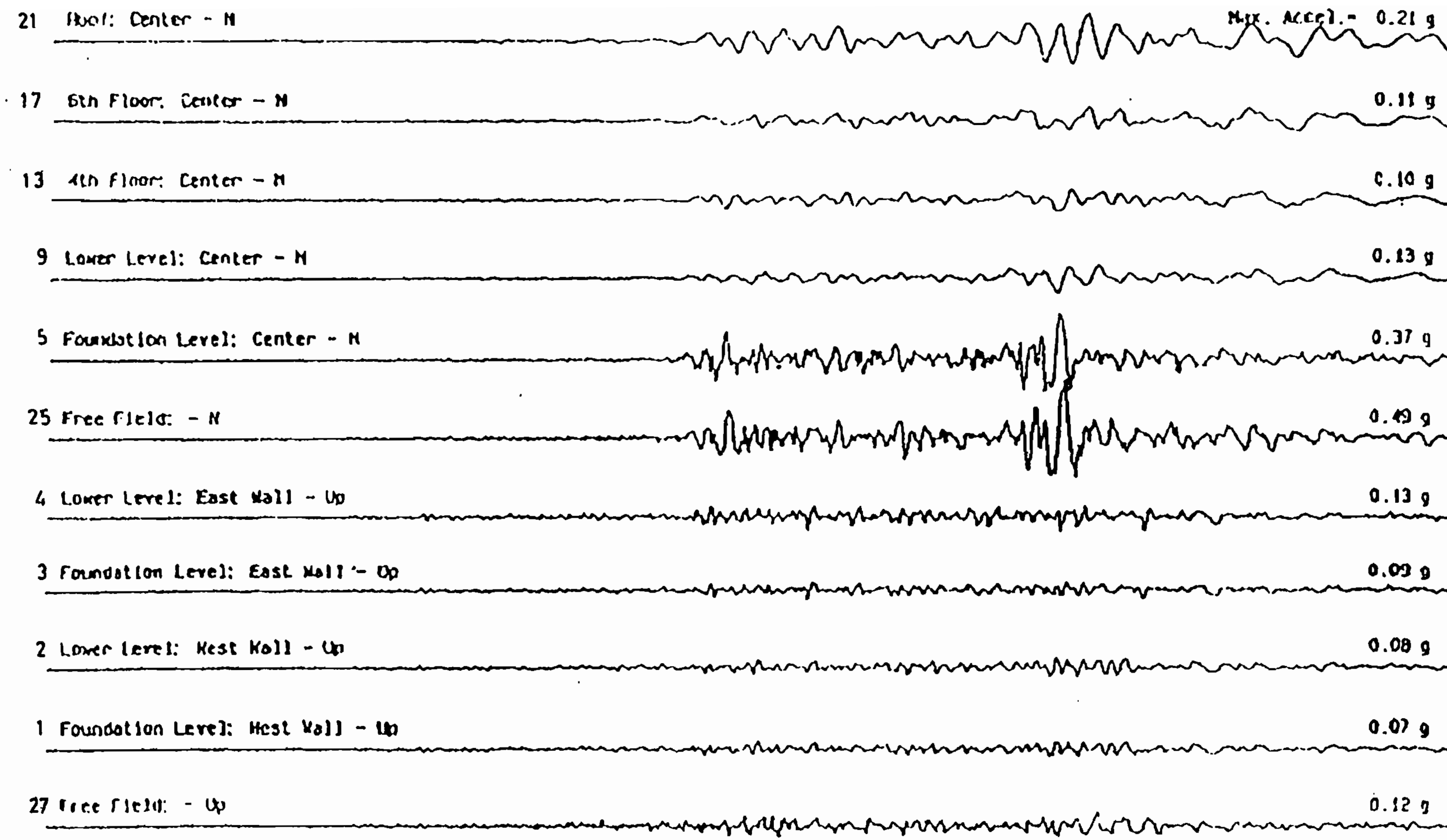
Los Angeles 7 Storey University Hospital
CSMIP Station No. 24605 Sensor Locations

Figure 6.4



Los Angeles 7 Storey University Hospital
CSMIP Station No. 24605

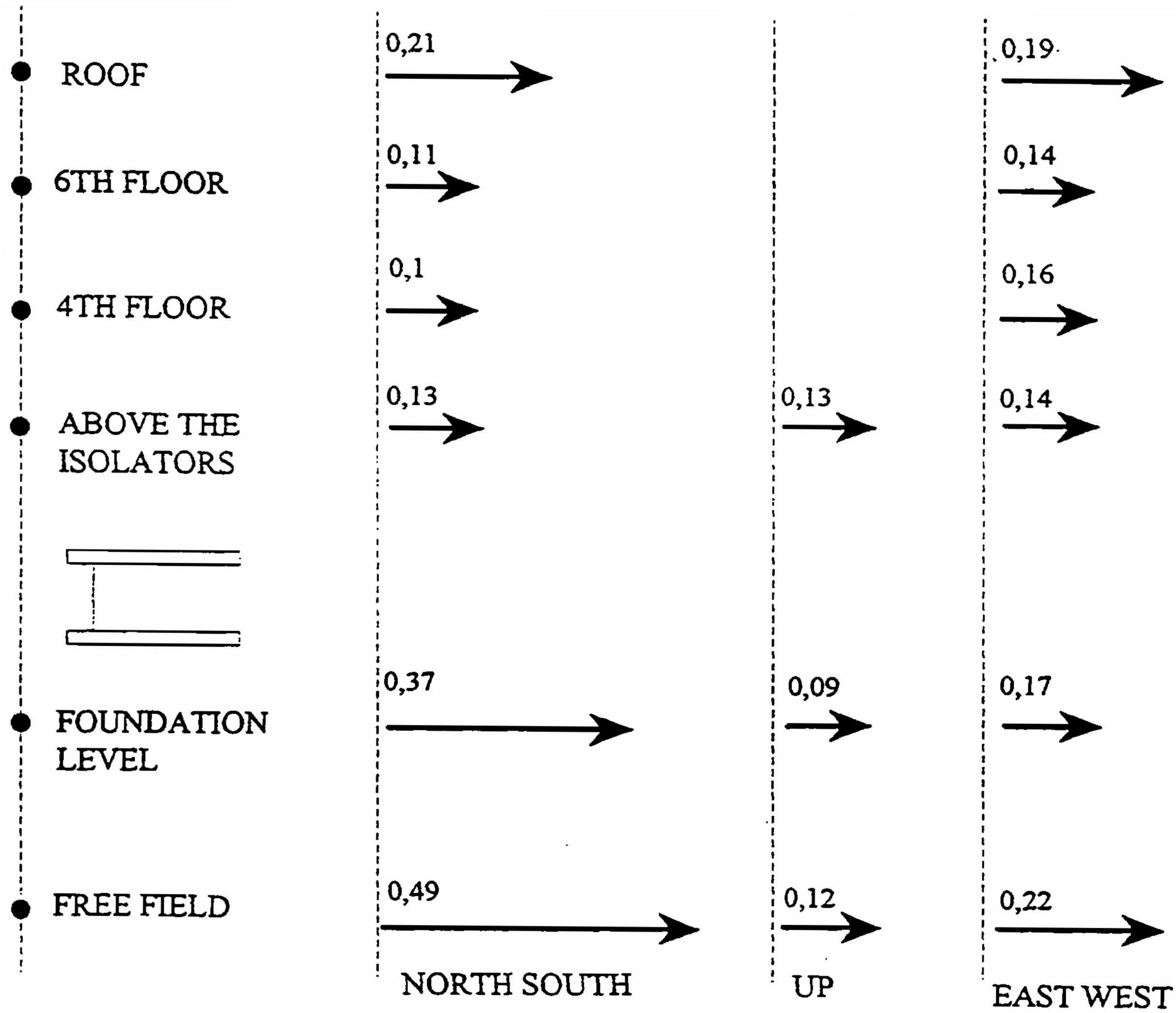
Figure 6.5



Base Isolated Building

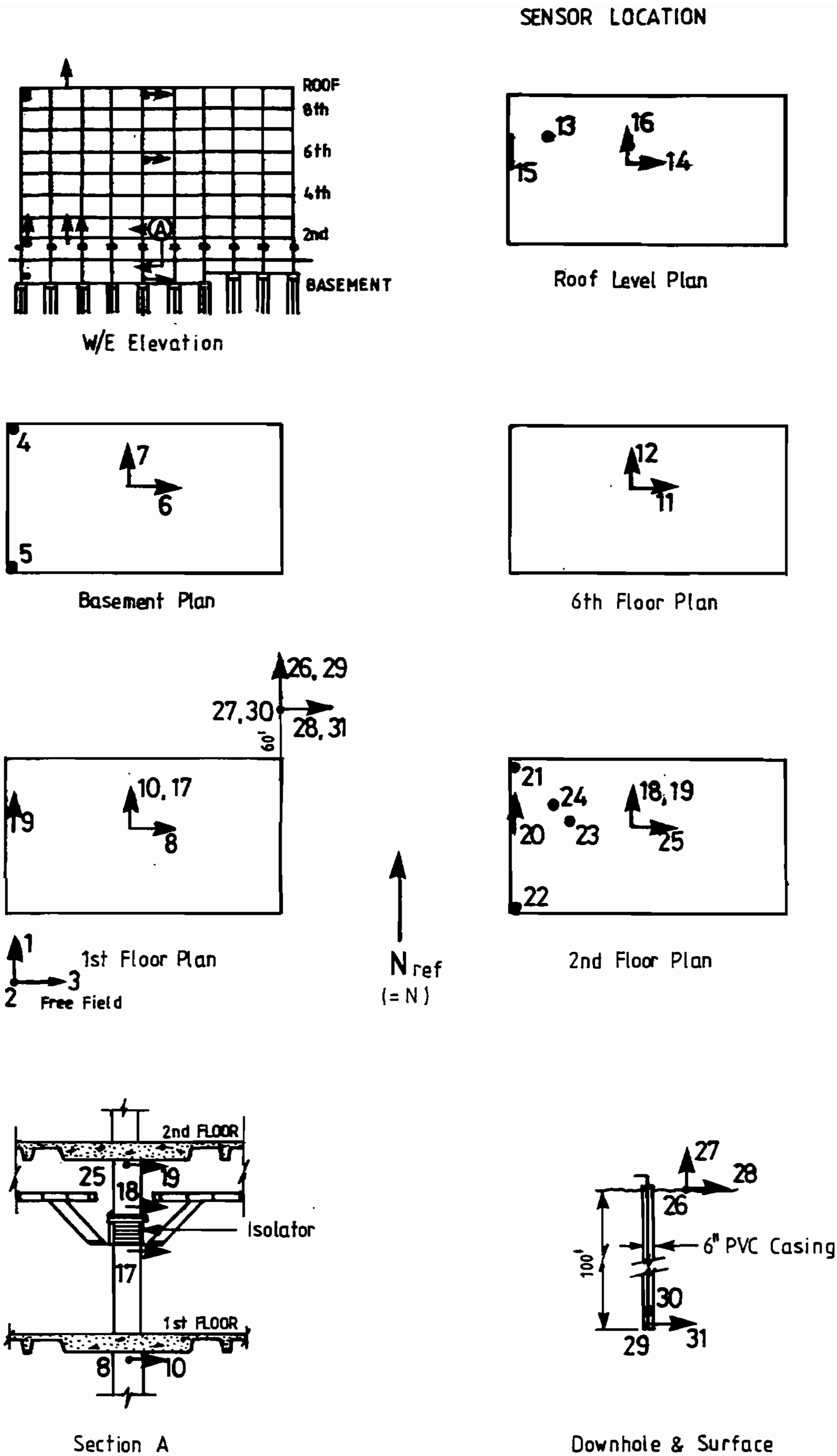
Los Angeles 7 Storey University Hospital
CSMIP Station 24605

Figure 6.6



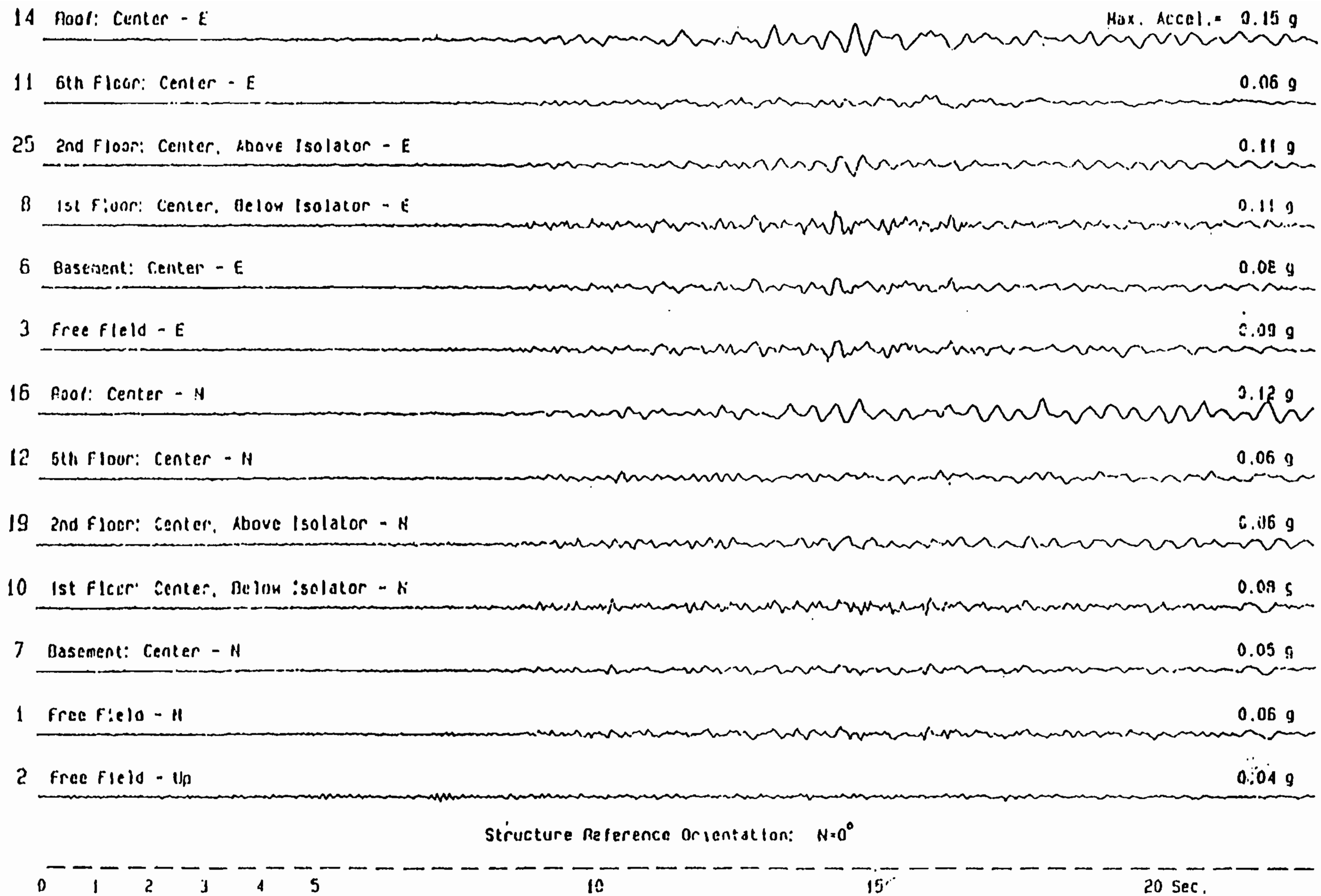
USC Hospital : Peak Values of Acceleration (g)

Figure 6.7



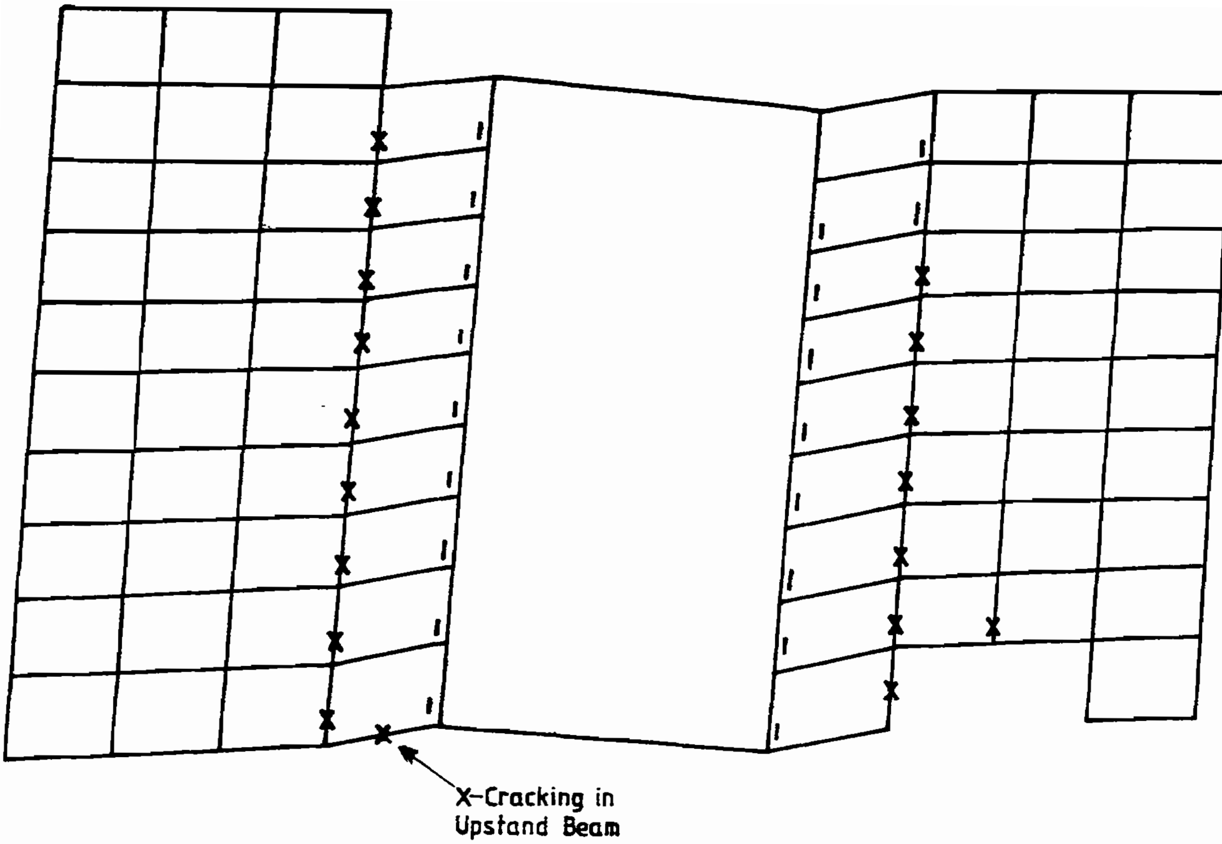
Seal Beach-8 Storey Office Building
CSMIP Station No.14578

Figure 6.8

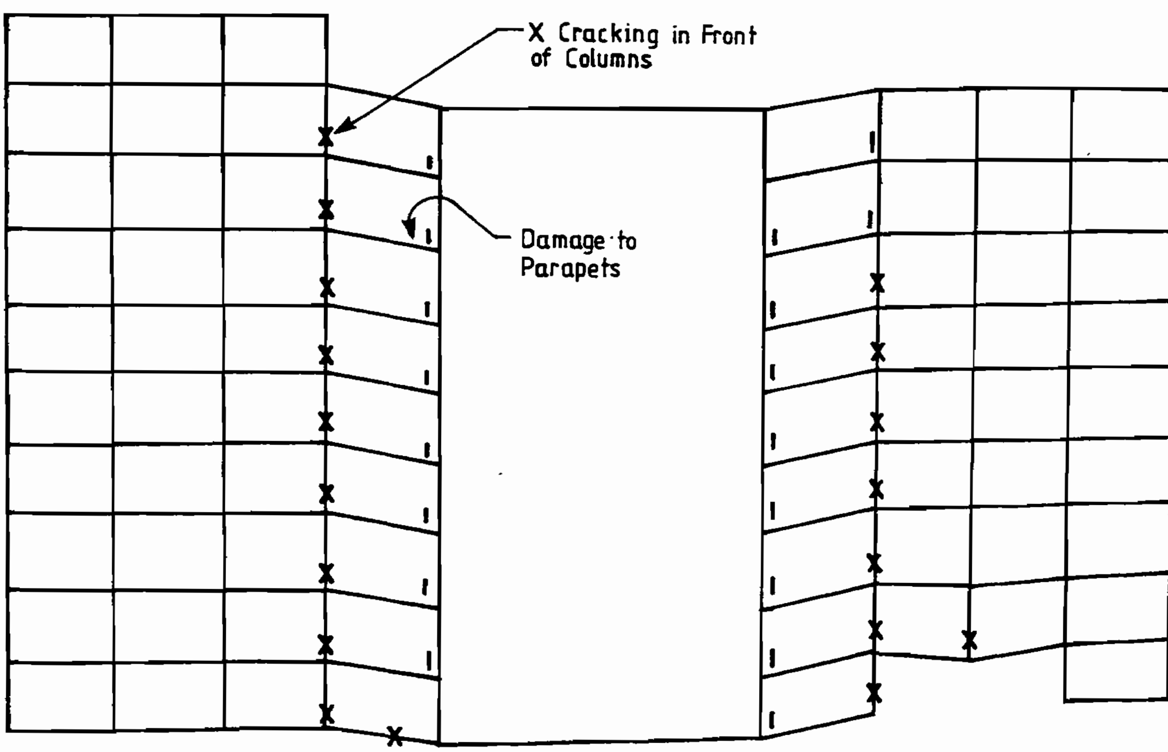


Seal Beach - 8 Storey Office Building
CSMIP Station 17578

Figure 6.9



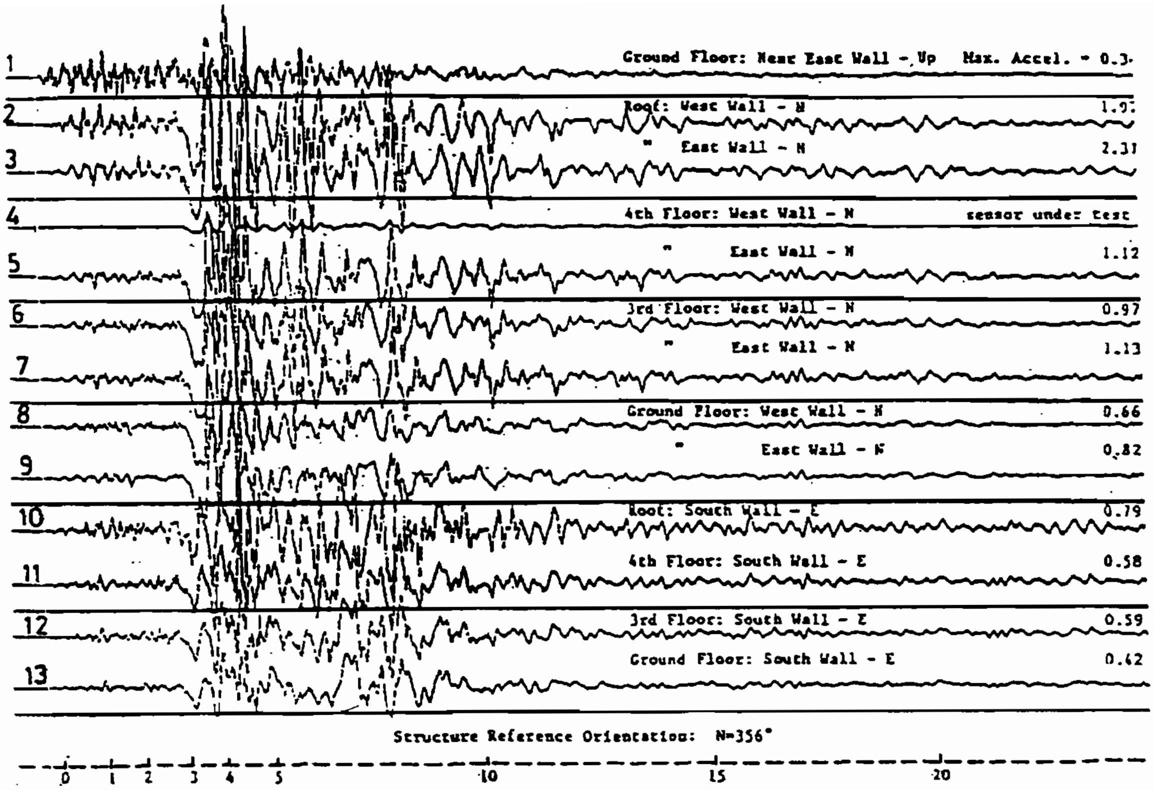
(a) Shear Wall Rotation



(b) Shear Wall Settlement

Sketch Showing Foundation Rotation in Denny's Car Park

Figure 6.10



Sylmar 6 Storey County Hospital
CSMIP Station No. 24514

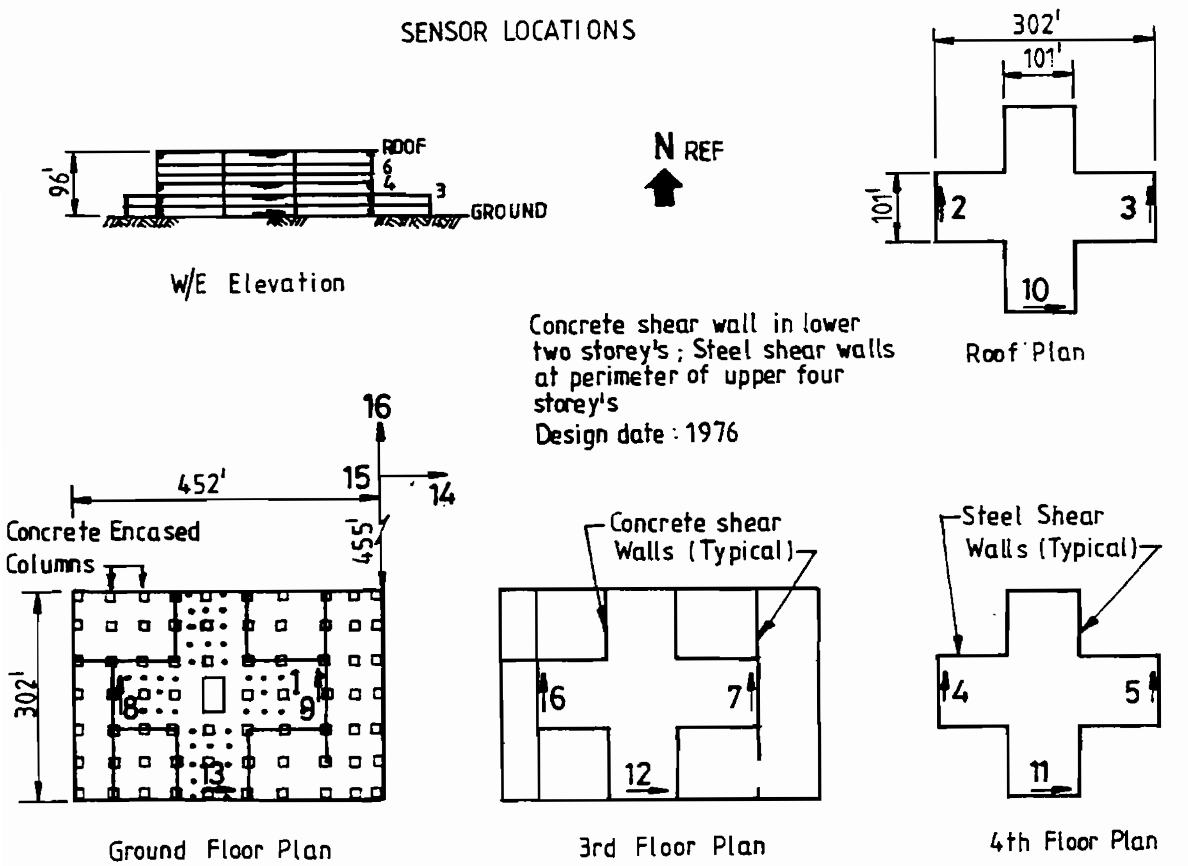


Figure 6.11

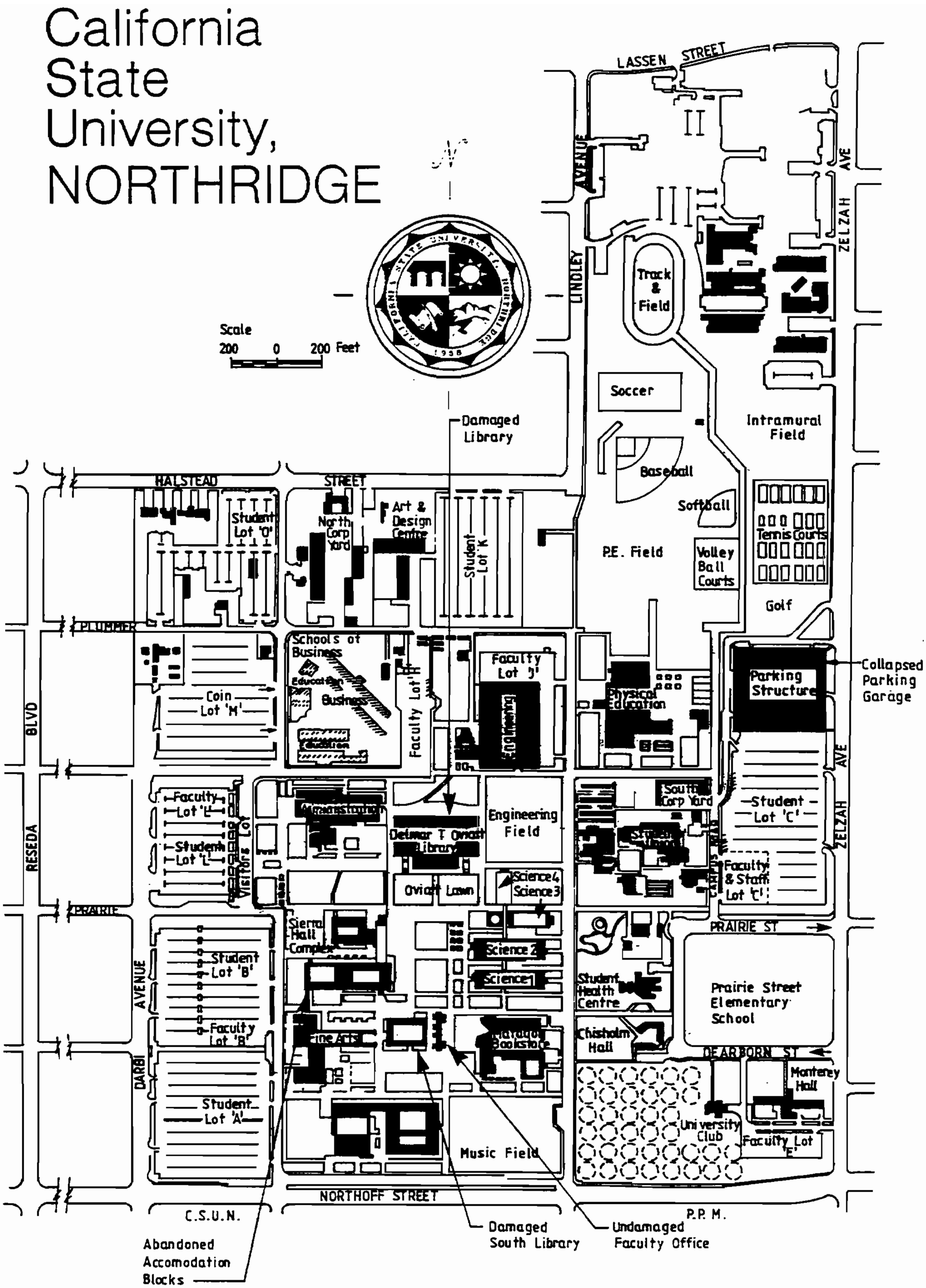

California State University, NORTHRIDGE
Scale
200 0 200 Feet
LASSEN STREET
LINDLEY
AVENUE
ZELZAH AVE
Track & Field
Soccer
Intramural Field
Baseball
Softball
P.E. Field
Volley Ball Courts
Tennis Courts
Golf
Damaged Library
HALSTEAD STREET
Student Lot 'O'
North Corp Yard
Art & Design Centre
Student Lot K
PLUMMER
Schools of Business
Education
Business
Faculty Lot 'J'
Faculty Lot
Physical Education
Parking Structure
Collapsed Parking Garage
Coin Lot 'M'
RESEDA BLVD
Faculty Lot 'L'
Student Lot 'L'
Visitors Lot
Administration
Delmar T. Oviatt Library
Engineering Field
South Corp Yard
Student Lot 'C'
Faculty & Staff Lot 'C'
Oviatt Lawn
Science 4
Science 3
PRAIRIE
PRAIRIE ST
Sierra Hall Complex
Science 2
Science 1
Student Lot 'B'
Faculty Lot 'B'
Student Health Centre
Prairie Street Elementary School
DARBI AVENUE
Fine Arts
Bookstore
Chisholm Hall
DEARBORN ST
Monterey Hall
Student Lot 'A'
Music Field
University Club
Faculty Lot 'E'
NORTHOFF STREET
C.S.U.N.
P.P. M.
Abandoned Accomodation Blocks
Damaged South Library
Undamaged Faculty Office

Figure 6.12

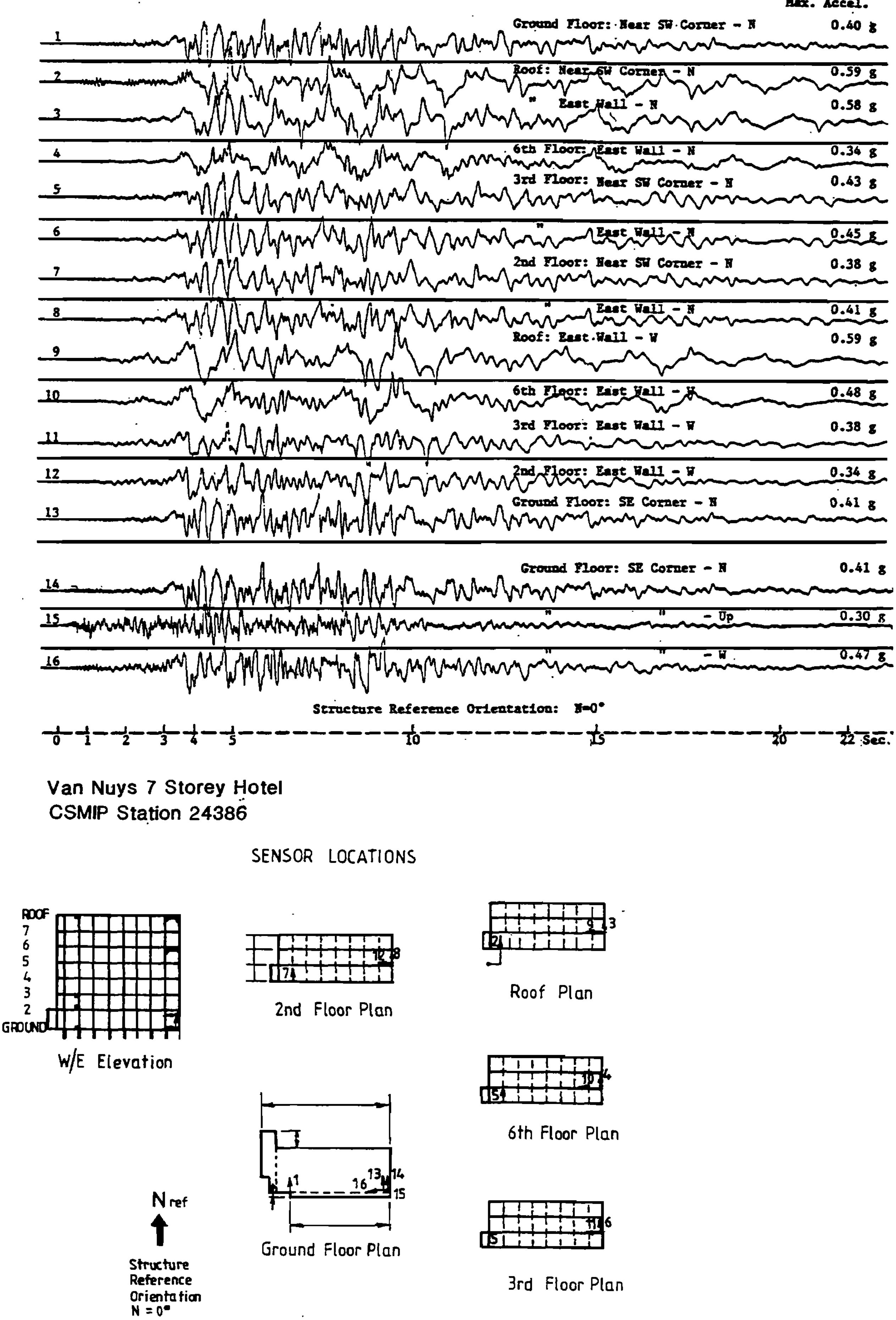
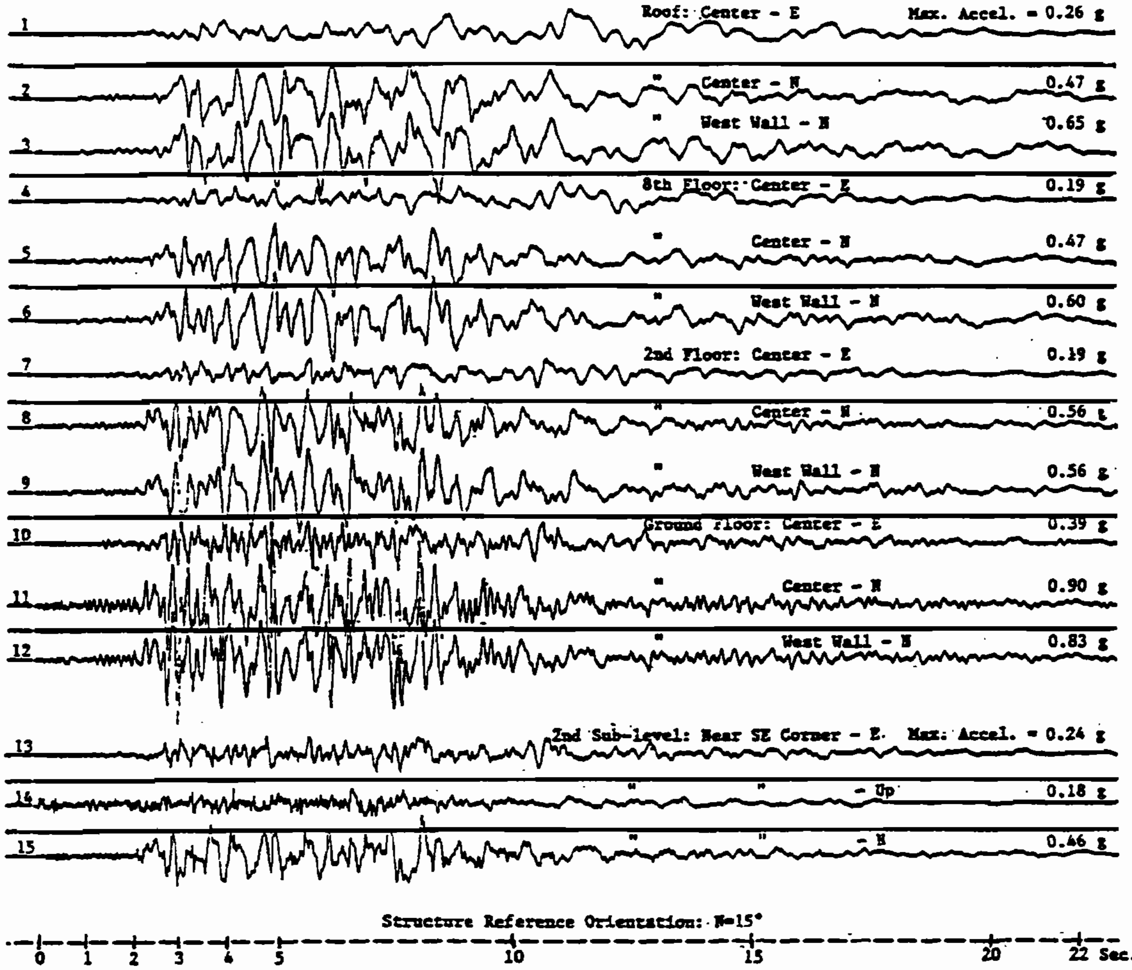


Figure 3 - Sensor Locations and Corresponding Strong Motion Records Obtained at the Holiday Inn in Van Nuys During the 1994 Northridge Earthquake

Figure 6.13



Sherman Oaks 13 Storey Commercial Building
CSMIP Station 24322

SENSOR LOCATIONS

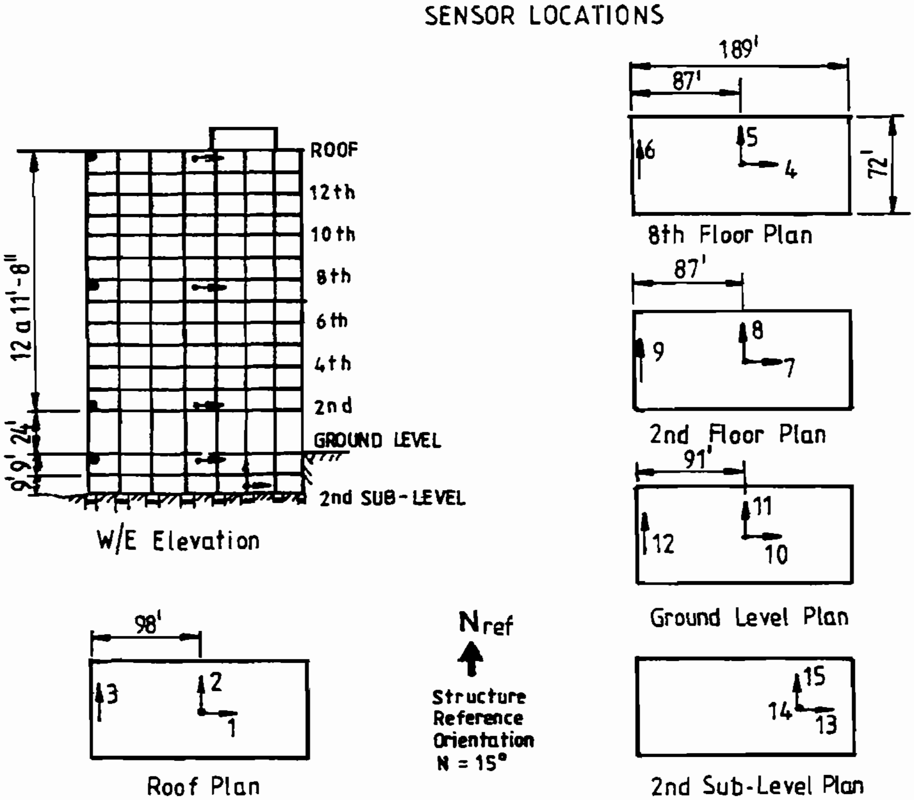
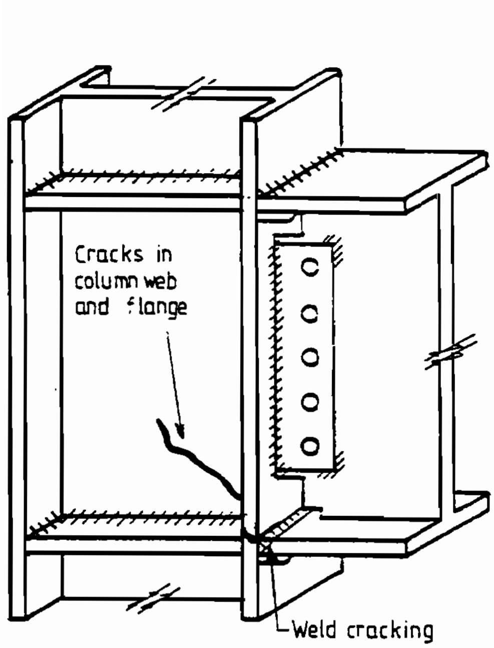
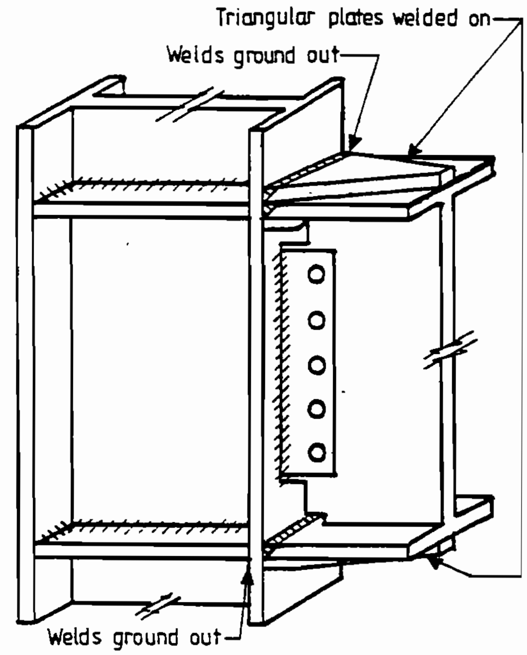


Figure 2 - Sensor Locations and Corresponding Strong Motion Records Obtained at the 13 Storey Commercial Building in Sherman Oaks

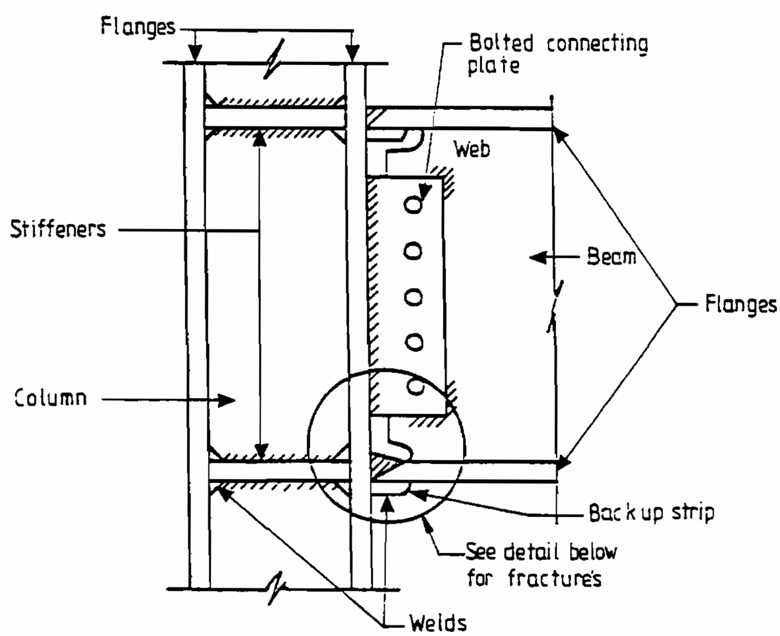
Figure 6.14



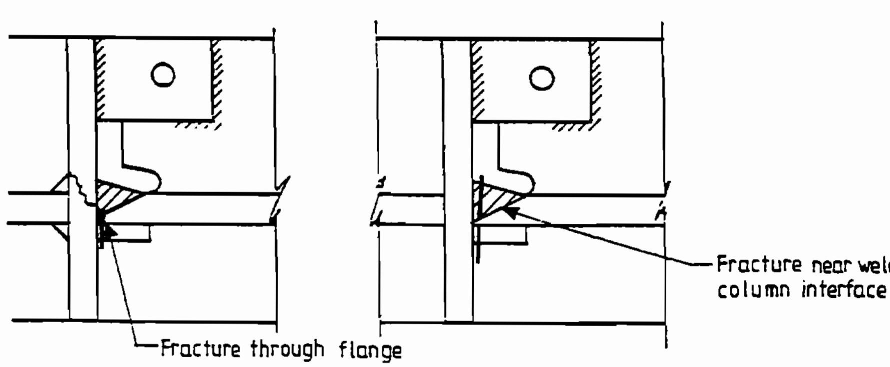
Crack in Stanchion Web



Proposed Alteration to Detail



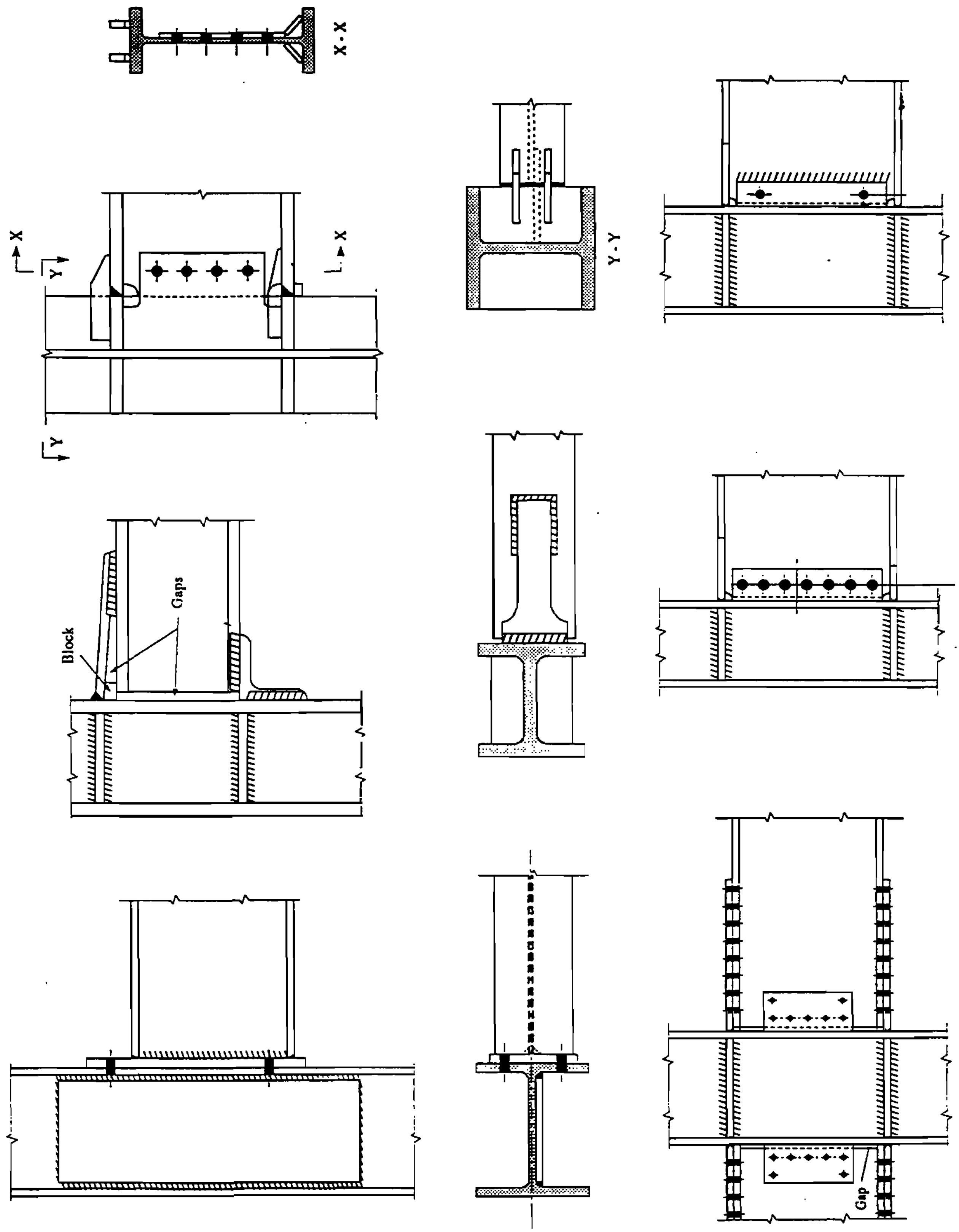
Most Typical Damage



Damage at Lower Flange Connections

Damage at Column Beam Connections in Steel Moment-Resisting Frames

Figure 6.15



US Details for Moment Resisting Frames

Figure 6.16

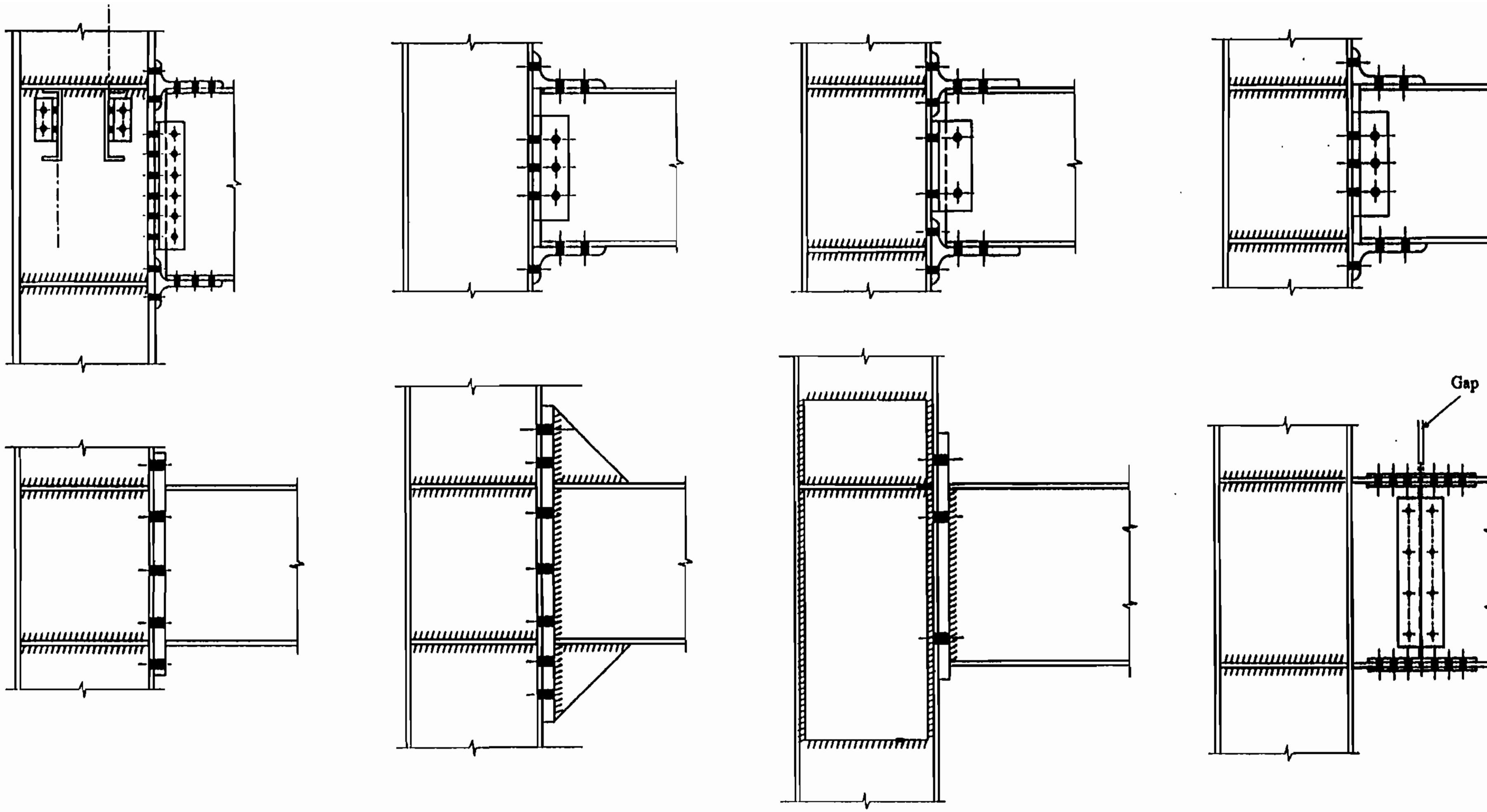


Figure 6.17

Column Beam - Connections in Moment -Resisting Frames - European Details

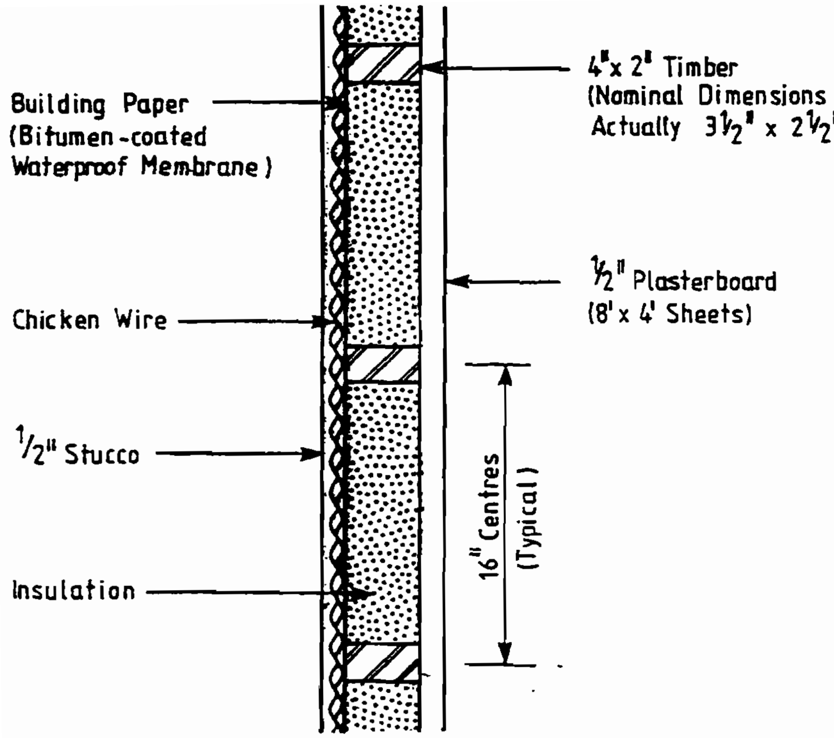


Figure 6.18

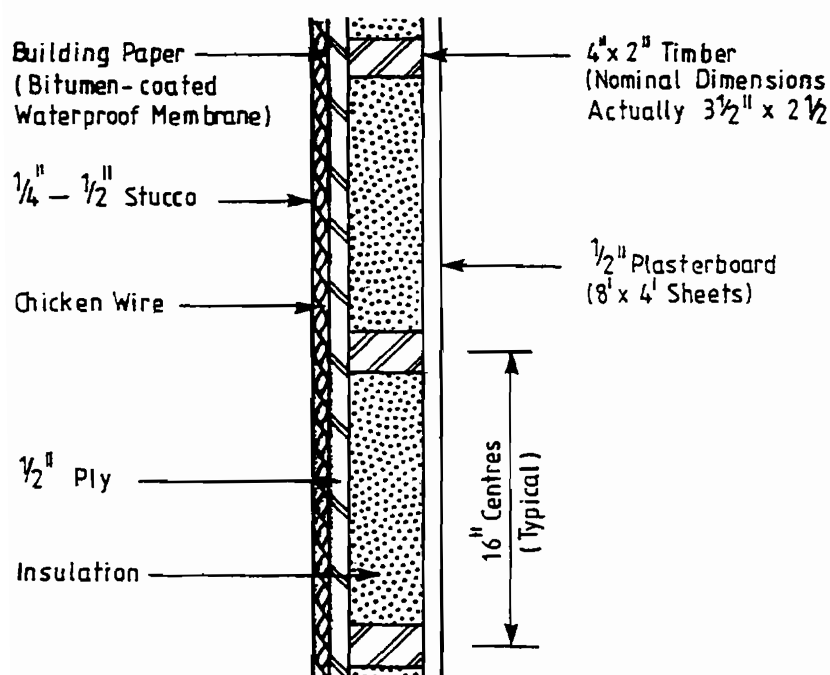


Figure 6.19

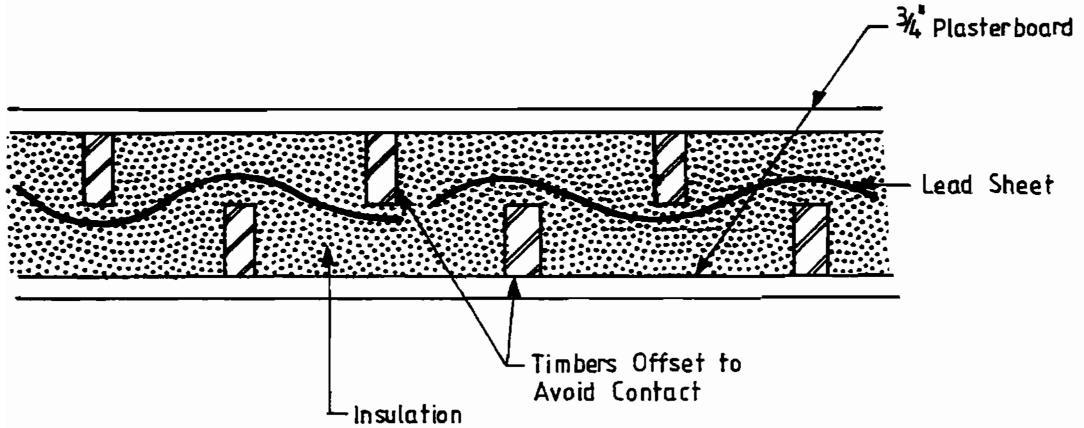
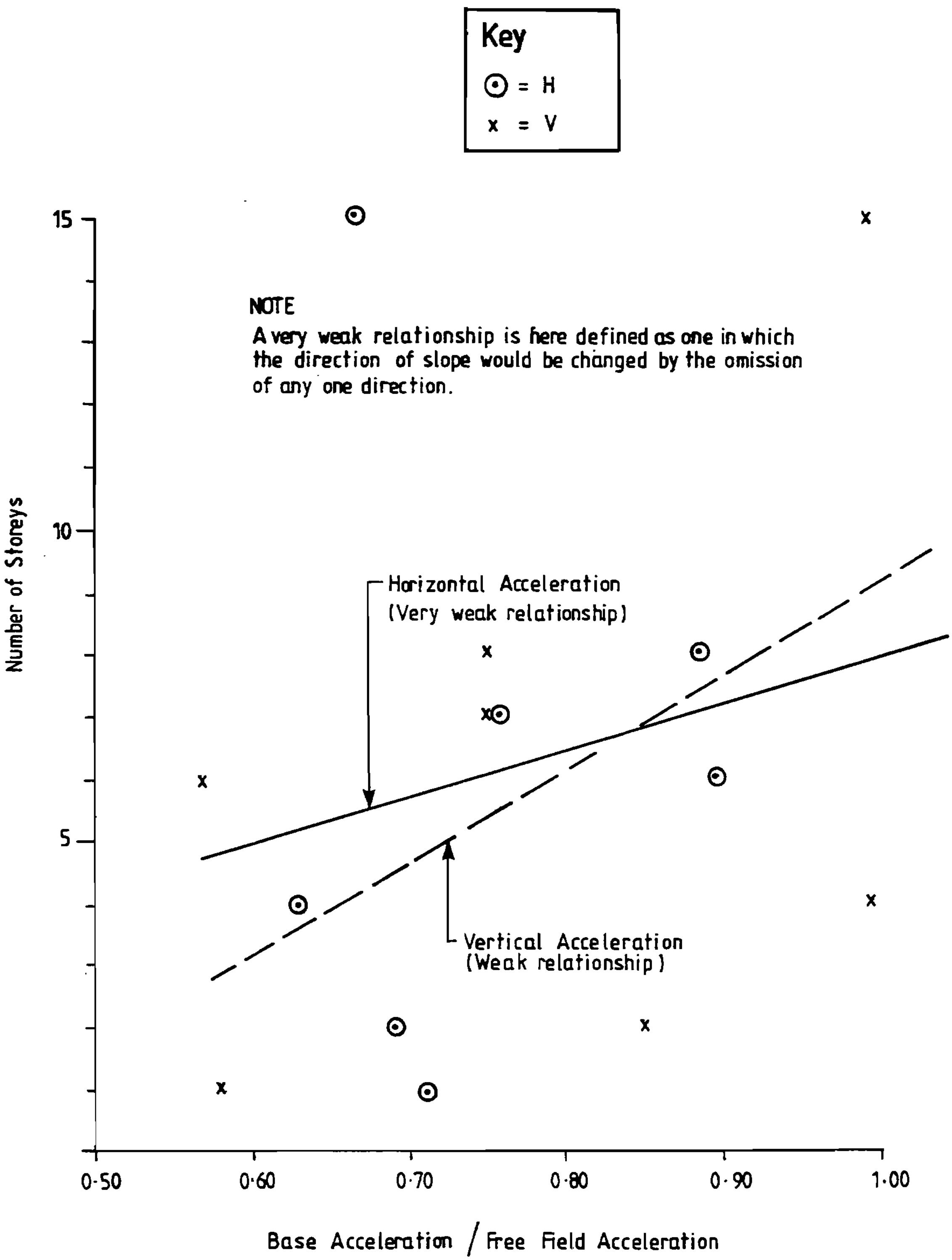
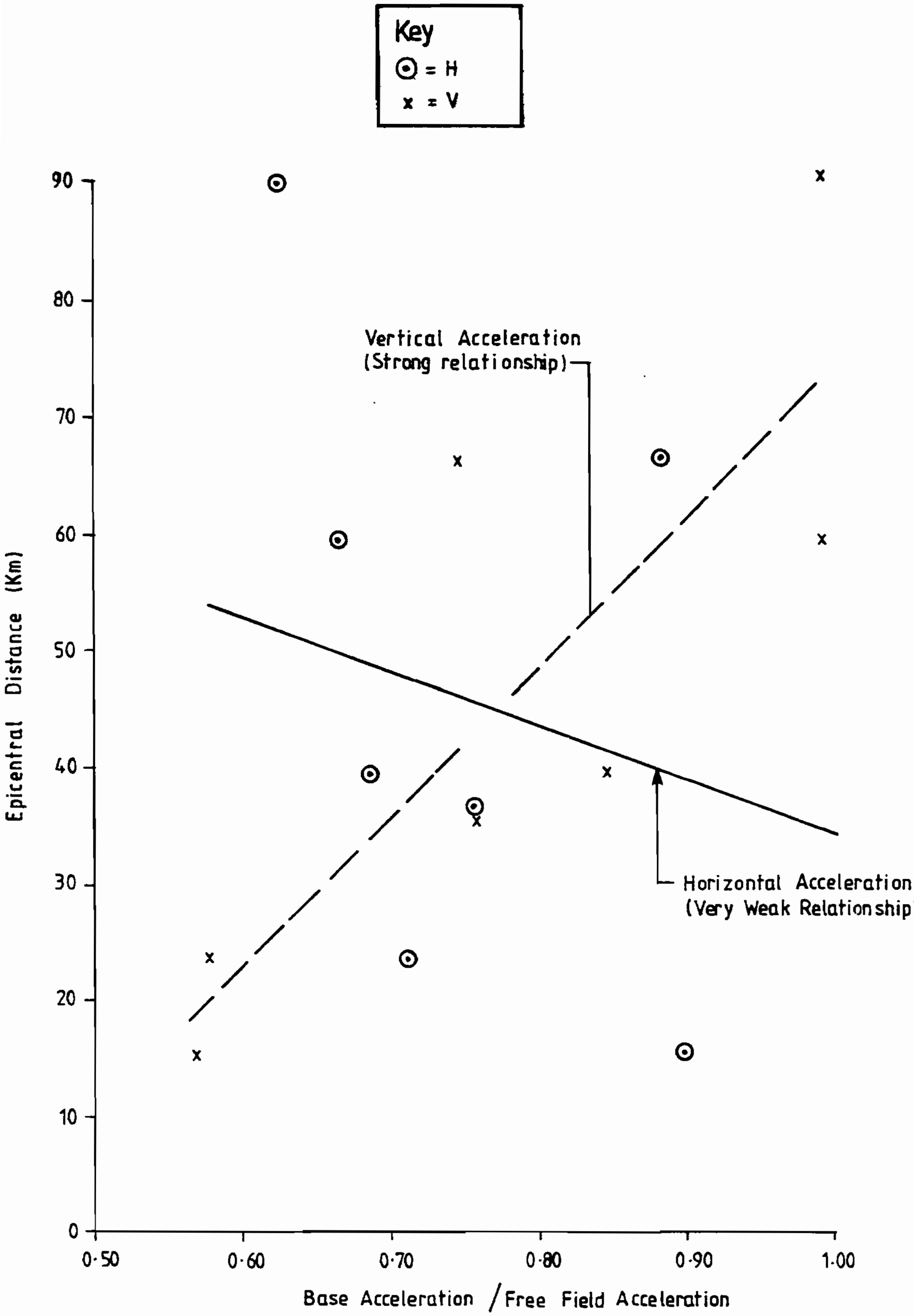

3/4" Plasterboard
Lead Sheet
Timbers Offset to
Avoid Contact
Insulation

Figure 6.20



Base to Free Field Acceleration Ratio Versus Number of Storeys

Figure 6.21



Base to Free Field Acceleration Ratio Versus Epicentral Distance

Figure 6.22

Plate 6.1: Los Angeles Fire Command and Control Facility

Plate 6.3: USC University Teaching Hospital

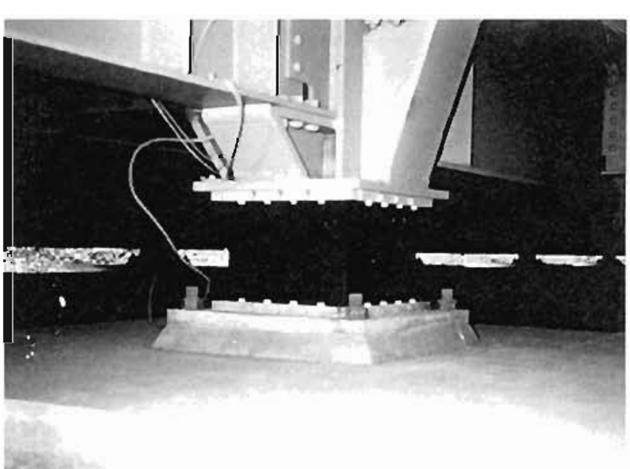


Plate 6.2: LA Fire Command : Natural rubber bearing isolators

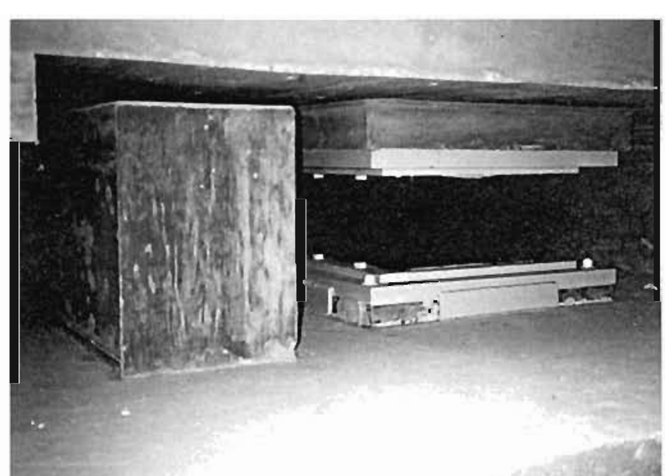


Plate 6.4: USCUTH : Bearing Location and measuring position

Plate 6.5: Rockwell International Information Systems Centre

Plate 6.6: Extent of scaffold provides evidence of damage sustained

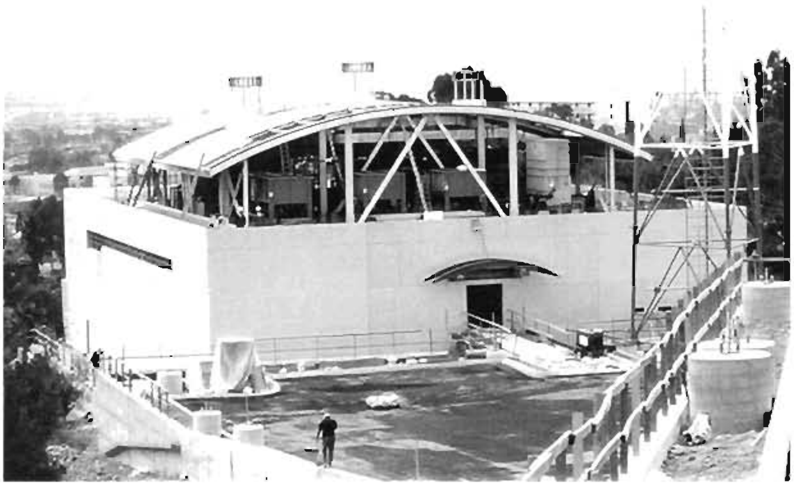


Plate 6.7: County Emergency Operation Centre

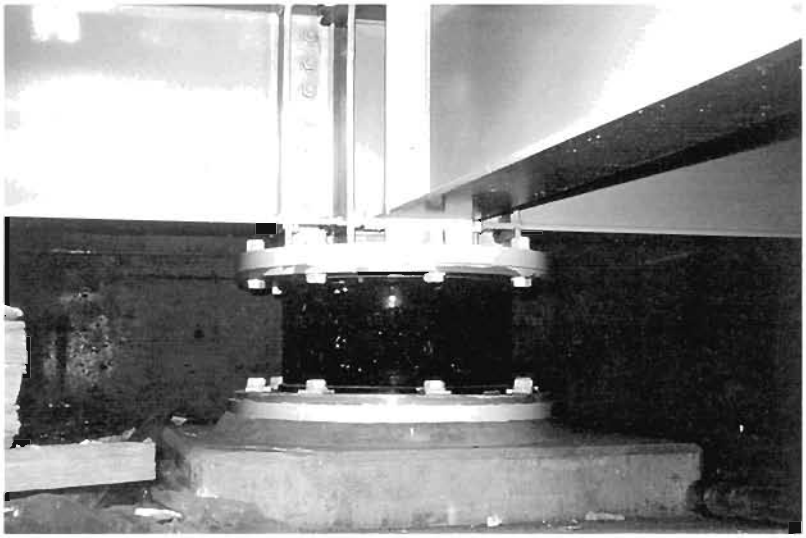


Plate 6.8: High damping rubber bearing

Plate 6.9: Two identical base isolated houses with braced steel frames

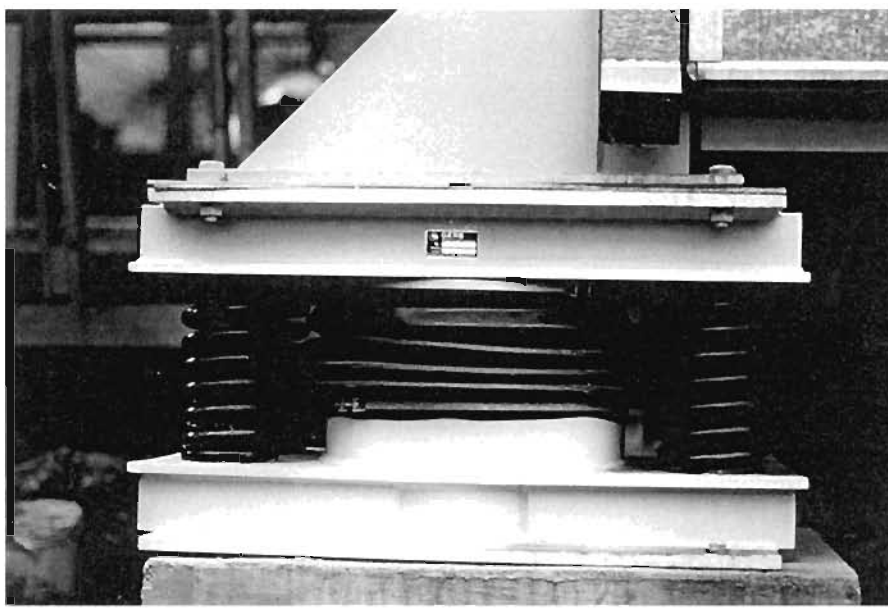


Plate 6.10: Helical springs and viscodampers

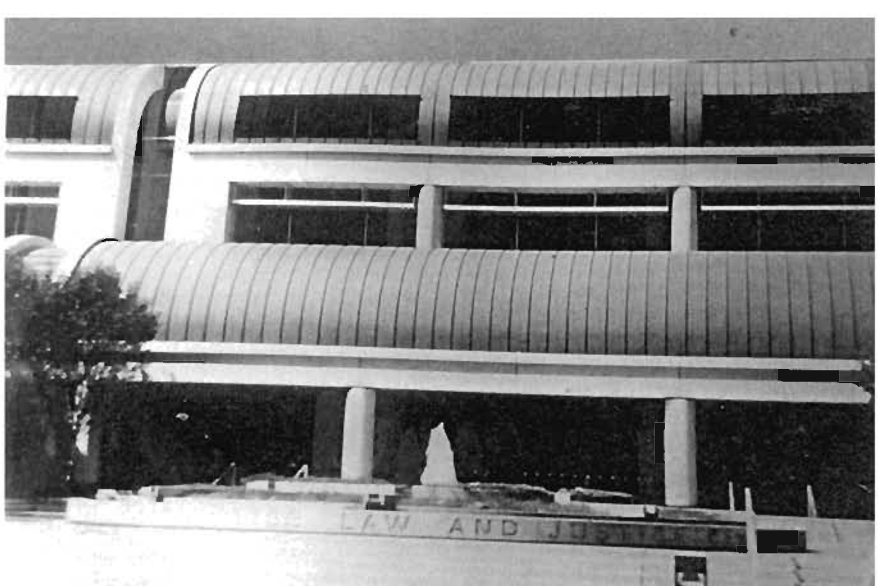


Plate 6.11: Foothill Communities Law and Justice Centre

Plate 6.12: CSU Northridge : Zelzah Avenue Car Park

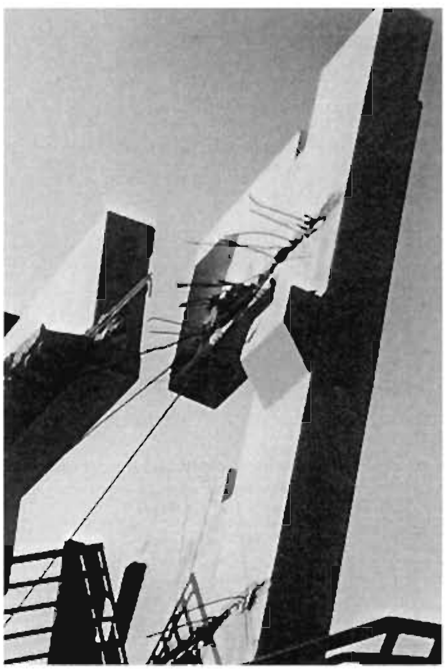


Plate 6.13: Corbels, tendons and ties

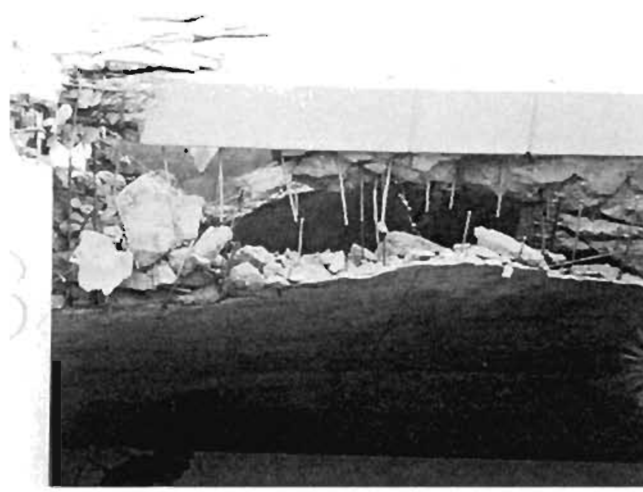


Plate 6.14: Underside of edge beam with tendons

Plate 6.15: Spiral staircase

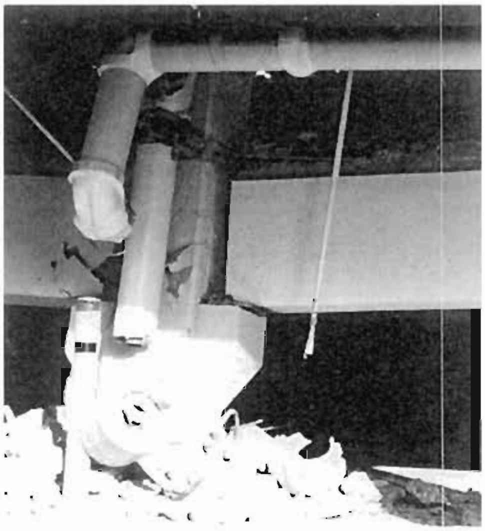


Plate 6.16: Fallen failed internal column

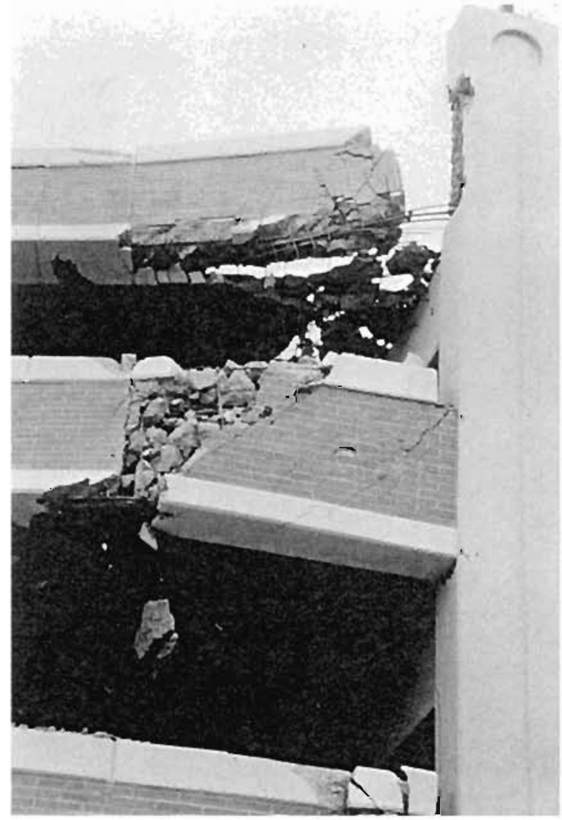


Plate 6.17: In-plane flexural failure of perimeter frame

Plates 6.18 / 19: Out-of-plane separation between precast column units, showing minimal longitudinal continuity, suggesting low beam shear resistance

Plate 6.18:

Plate 6.19

Plate 6.20: Distributed out-of-plane inelasticity in columns

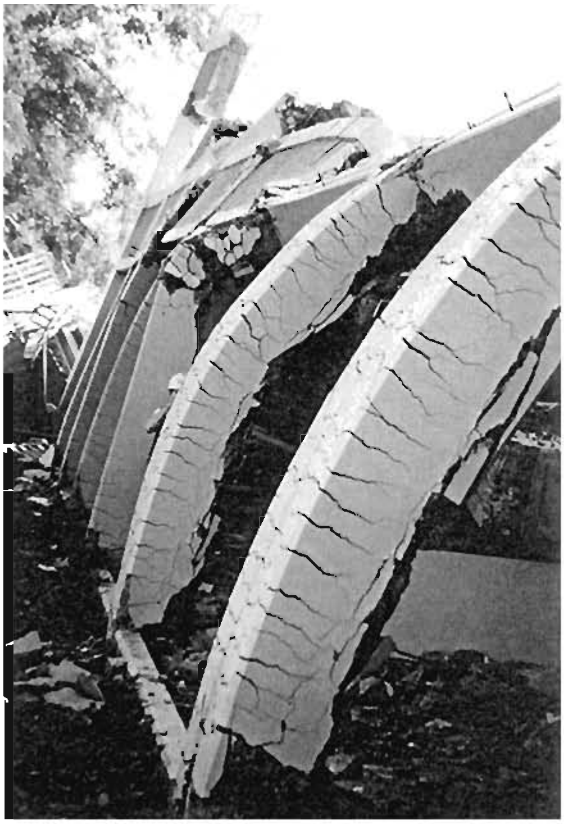


Plate 6.21: Closely spaced flexural cracks almost reaching compression face

Plate 6.22: Spalling outside the well confined column cage

Plate 6.23: Denny's Car Park

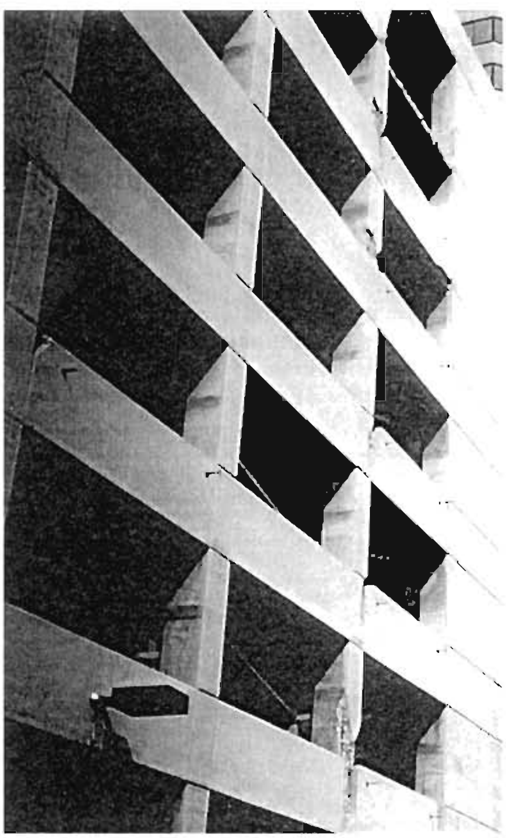


Plate 6.24: Spalling of column cover in plane of parapet

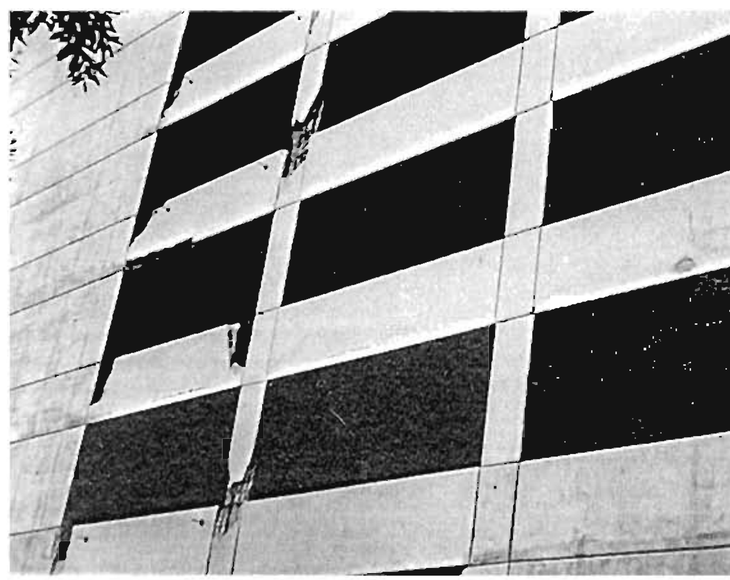


Plate 6.25: Crushing of parapet in bay next to shear wall

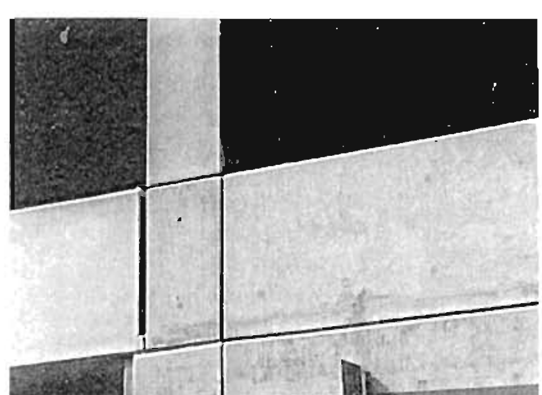


Plate 6.26: Remedial slit in parapet

Plate 6.27: Extreme vertical irregularity at entrance

Plate 6.28: Shear cracking of column

Plate 6.29: Separated stair well

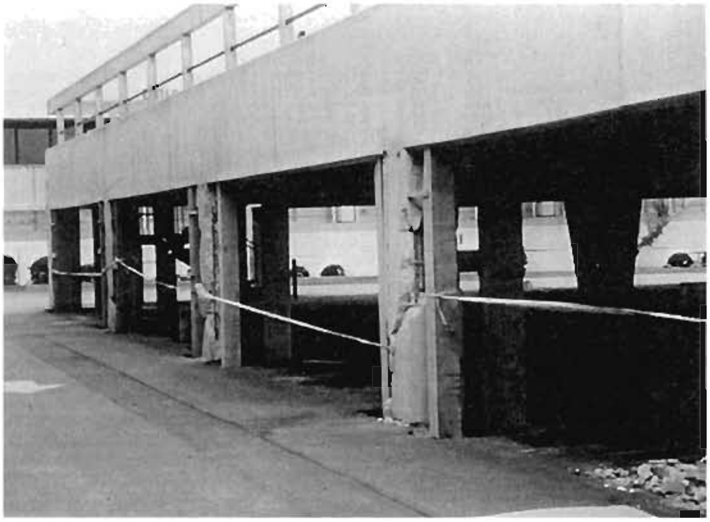


Plate 6.30: Car park of Transworld Bank

Plate 6.31: Shear cracking in columns on south side

Plate 6.33: Internal columns sandwiched between replacements

Plate 6.32: Collapsed car park ramp

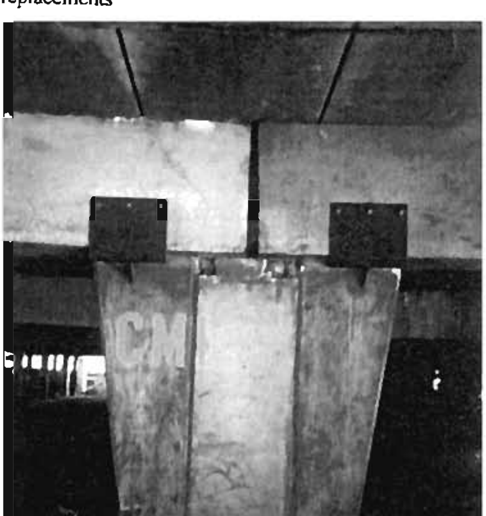


Plate 6.34: Seating of pre-stressed beams

Plate 6.35: Old cracking at end of beams

Plate 6.37: Shear cracks in columns due to change in floor level

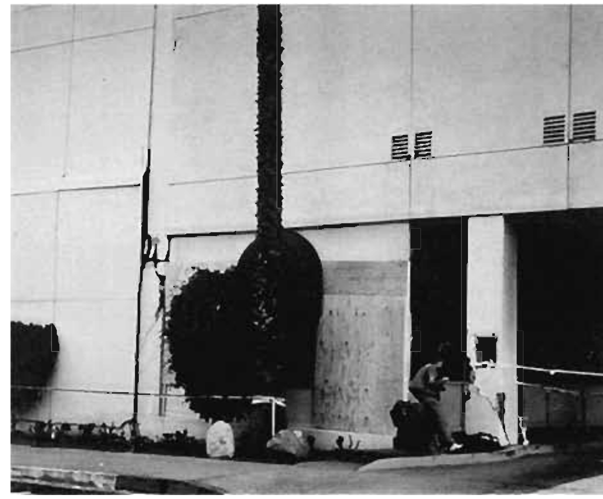


Plate 6.36: Radisson Centre : shear cracks in columns due to buffeting

Plate 6.39: St. John's Hospital car park : cracked concrete at corners of opening in walls.

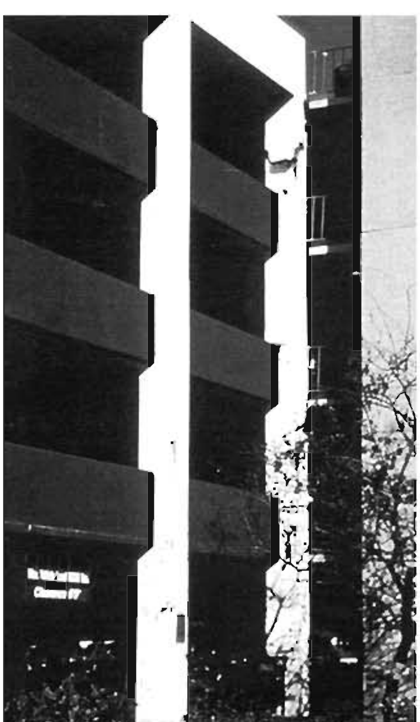


Plate 6.38: Damaged columns at the Kaiser Permanent car park

Plate 6.40: X cracking in shear wall panels at St John's Hospital car park

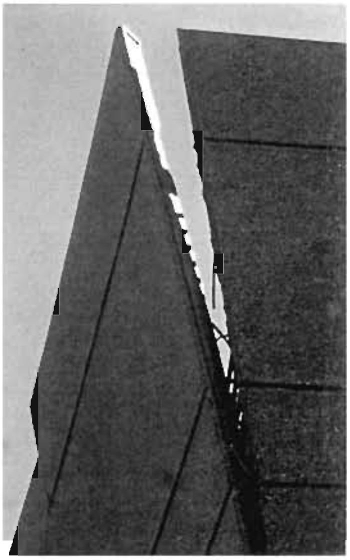


Plate 6.43: Sprung corner at Cigna Garage

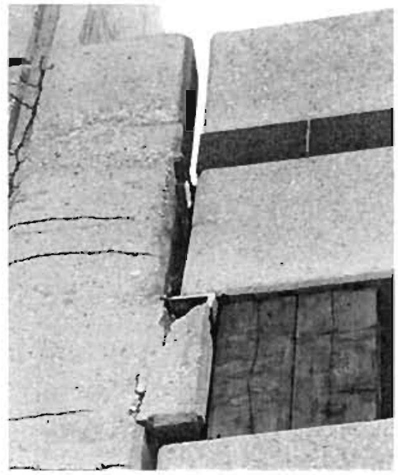


Plate 6.47: Damage to fabricated steelwork at high-level corbel

Plate 6.44: Shear wall torn at high level

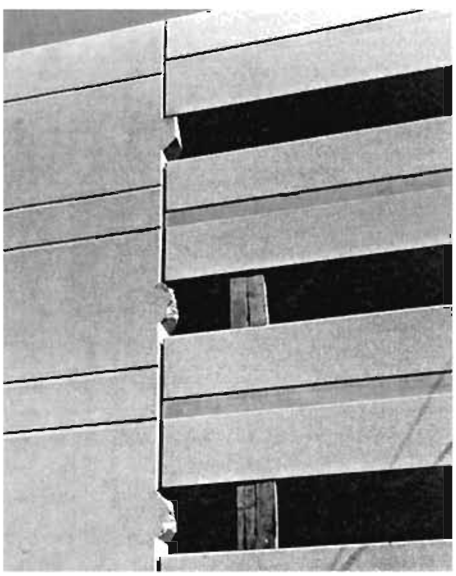


Plate 6.46: Elevation showing variation of corbel damage with height

Plate 6.41: Undamaged curved wall of Cigna Garage

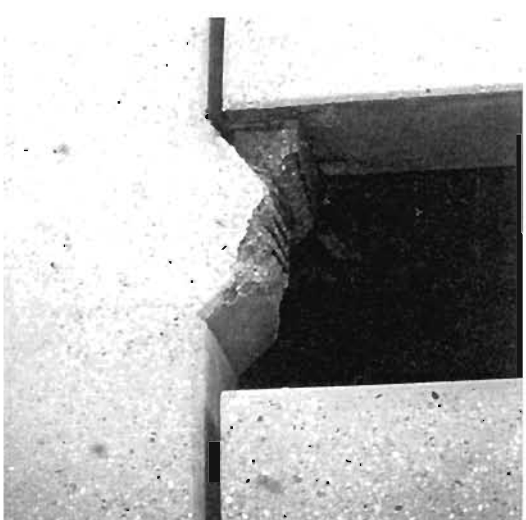


Plate 6.45: Spalling of cover concrete on underside of corbel

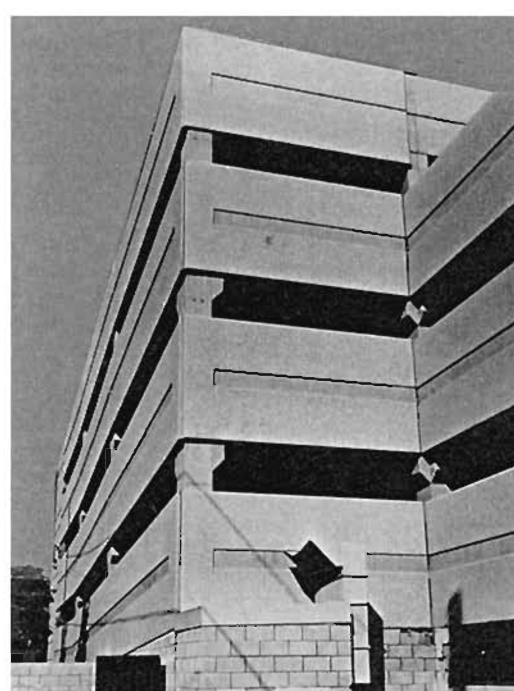


Plate 6.42: Undamaged corner without corbel construction

Plate 6.48: Armed Forces Recruiting Centre car park

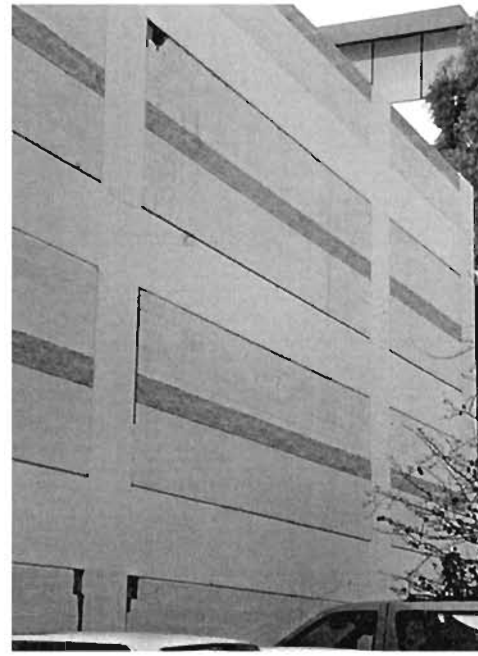


Plate 6.49: Crushed corners of infills in infilled frame

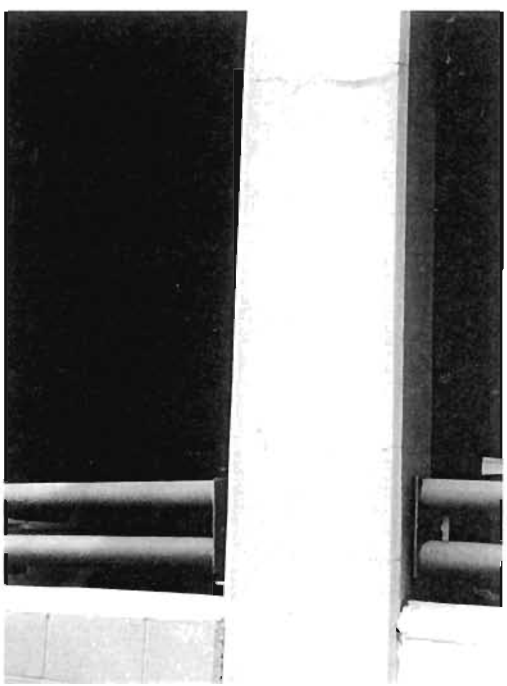


Plate 6.51: Flexural cracks in isolated column.

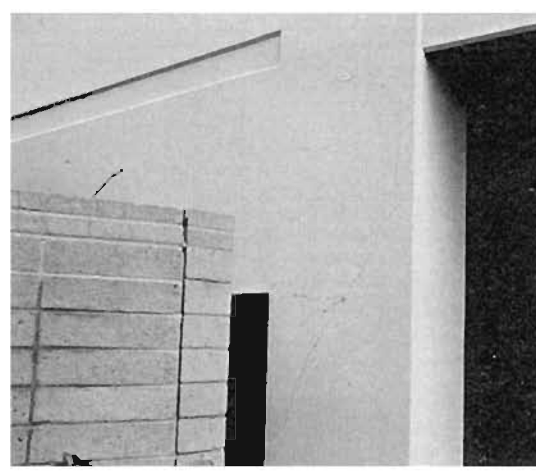


Plate 6.50: Shear crack in column at entrance

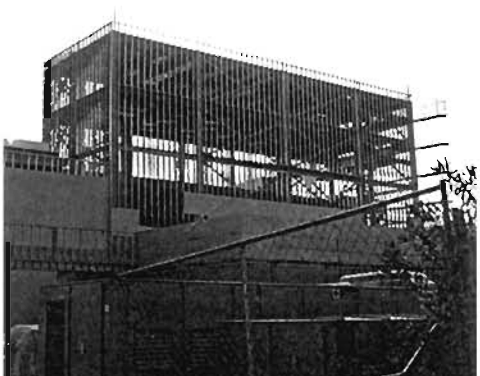


Plate 6.52: Steel multi-storey mechanical car park

Plate 6.53: Vulnerable multi-storey car park in South LA

Plate 6.54: St John's Hospital, Santa Monica

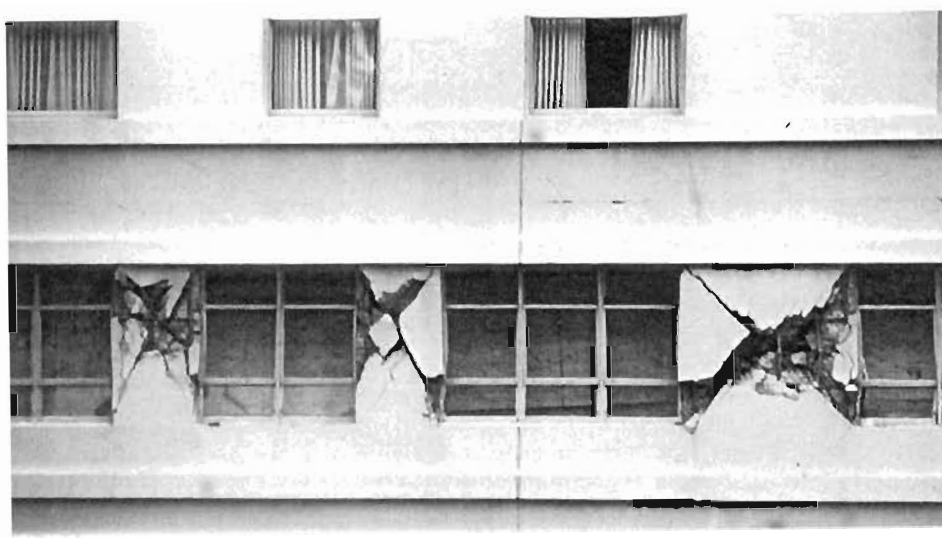


Plate 6.55: X-cracking in perforated shear wall

Plate 6.56: Infilled frame with infill and column cracked

Plate 6.57: Kaiser Permanent Health Institution

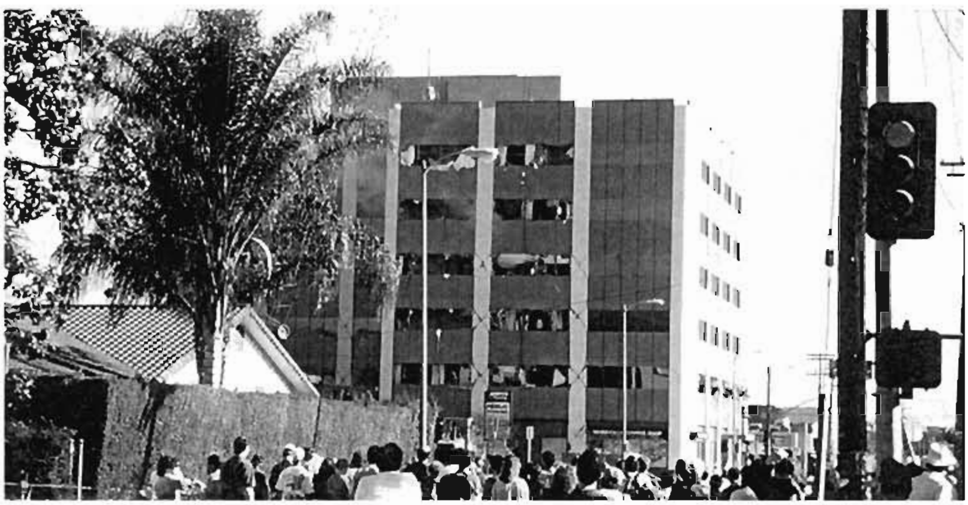


Plate 6.58: Barrington Medical Centre, Santa Monica

Plate 6.59: X-cracking in columns

Plate 6.60: Santa Monica Hospital

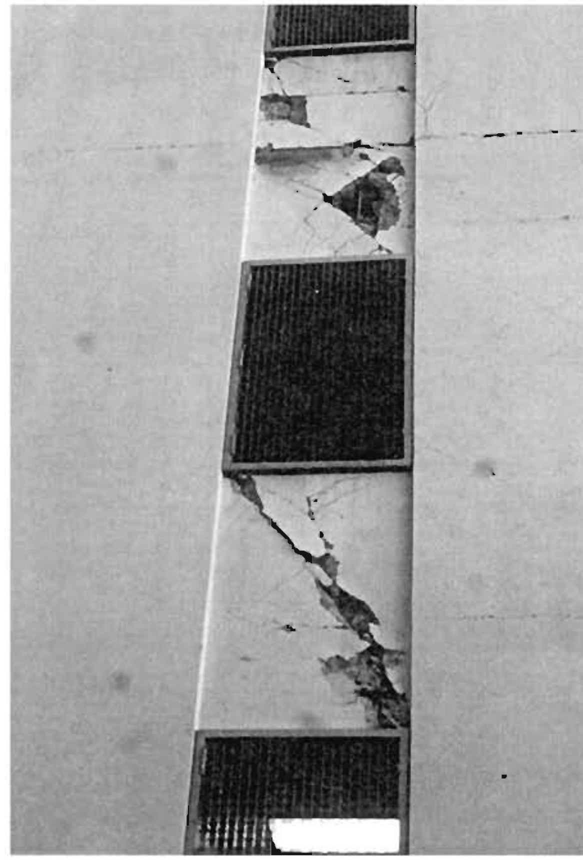


Plate 6.61: Diagonal cracking in connecting beam


Plate 6.62: In-plane flexural crushing and surface spalling of side wall

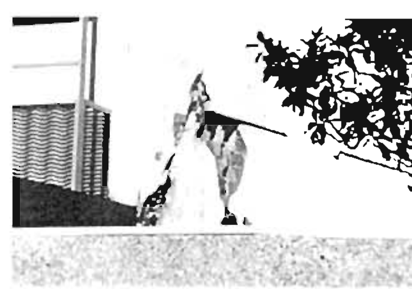


Plate 6.63: Close-up of crushing and spalling

Plate 6.64: Indian Hills Medical Centre

Plate 6.65: Rear view showing spalling at end walls

Plate 6.69: Side wall damage at front on first floor

Plate 6.66: Side wall damage at fourth floor

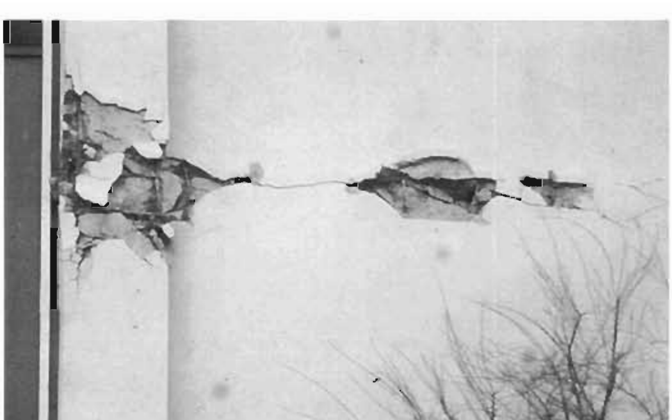


Plate 6.67: Close-up of damage in Plate 6.66

Plate 6.68: Side wall damage on opposite face

Plate 6.70: Olive View Medical Centre

Plate 6.71: Dissaray of shelved items

Plate 6.72: Flooded 6th floor corridor

Plate 6.73: The steel plate walls during construction

Plate 6.74: Old tilt-up in hospital grounds

Plate 6.75: Interior and damage to plywood roof

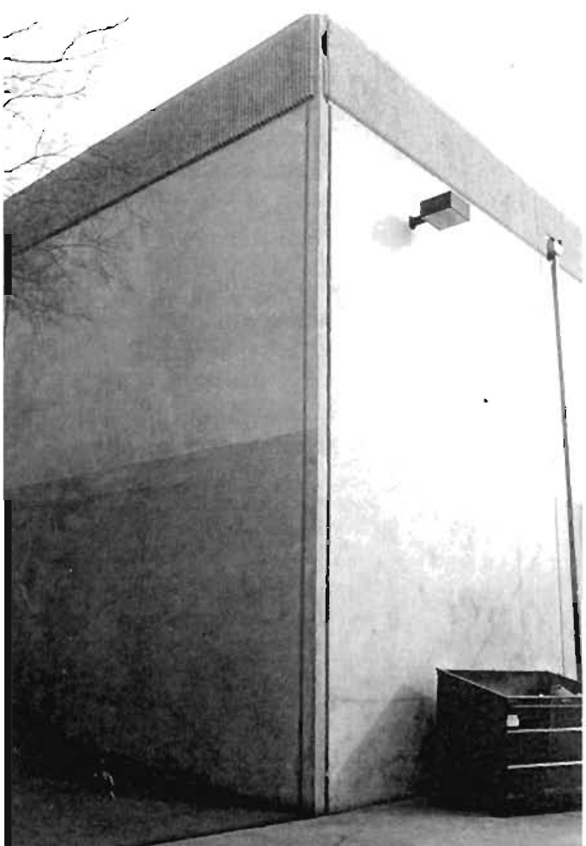


Plate 6.76: Panel separation and corner damage

Plate 6.77: Damage to lightly clad older building

Plate 6.78: CSU Northridge : Administration block

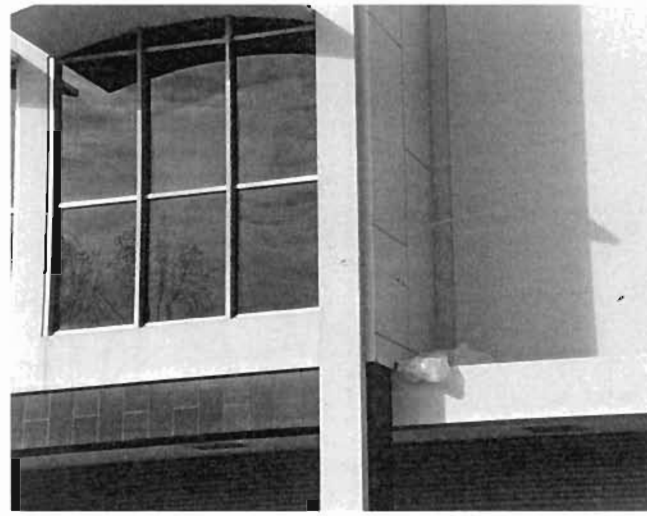


Plate 6.79: Administration block : separated stair well

Plate 6.80: Front facade of Oviatt Library

Plate 6.81: Front facade : fallen canopy

Plate 6.82: After removal of debris

Plate 6.83: Fallen canopy in re entrant corner at rear


Plate 6.84: Fine Arts block


Plate 6.85: Old South Library with damaged shear walls

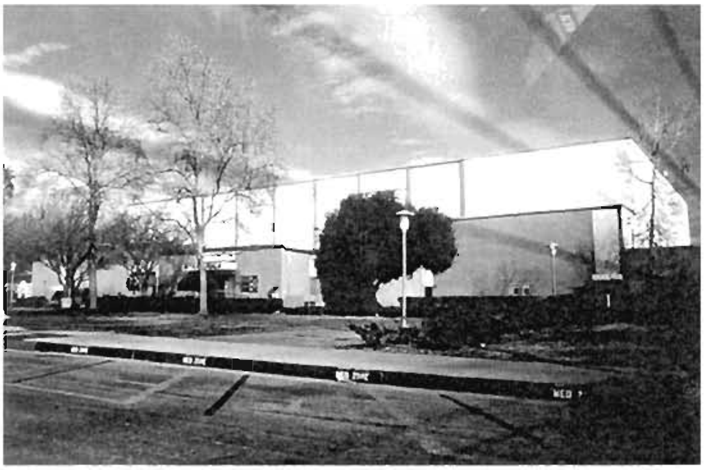


Plate 6.86: Physical Education Centre


Plate 6.87: Engineering block


Plate 6.88: Science block showing bridge supports

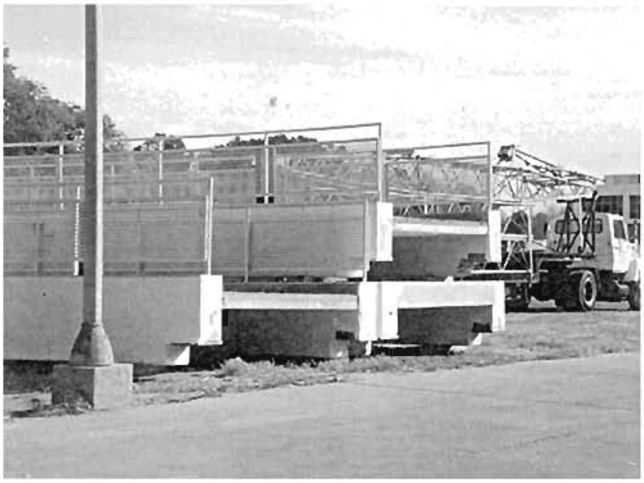


Plate 6.89: Replacement bridges

Plate 6.90: Fire damaged chemistry block

Plate 6.91: Steel framed book store with fallen tilt-up panels

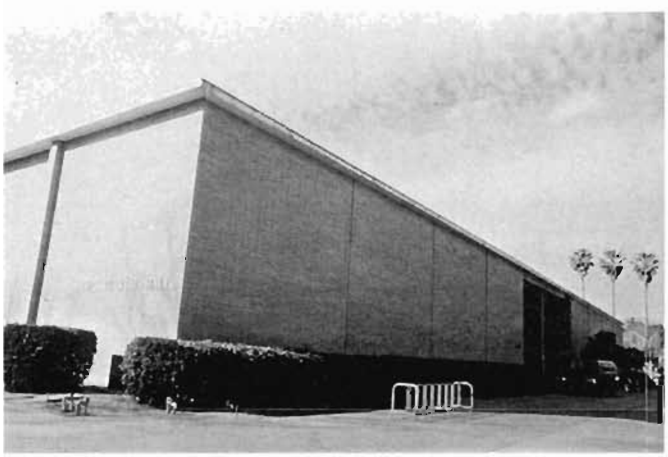


Plate 6.92: Speech drama/music building

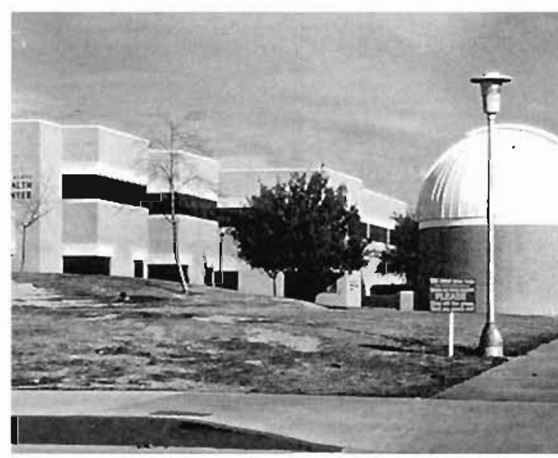


Plate 6.93: Health Centre and Observatory

Plate 6.94: Abandoned accommodation block

Plate 6.95: Undamaged faculty office

Plate 6.96: Holiday Inn, Van Nuys : North side

Plate 6.97: South side (rear)

Plate 6.98: Close-up of cracking damage at second floor on North side

Plate 6.99: Damaged fourth floor columns, South side

Plate 6.100: Close-up of X-cracking in columns

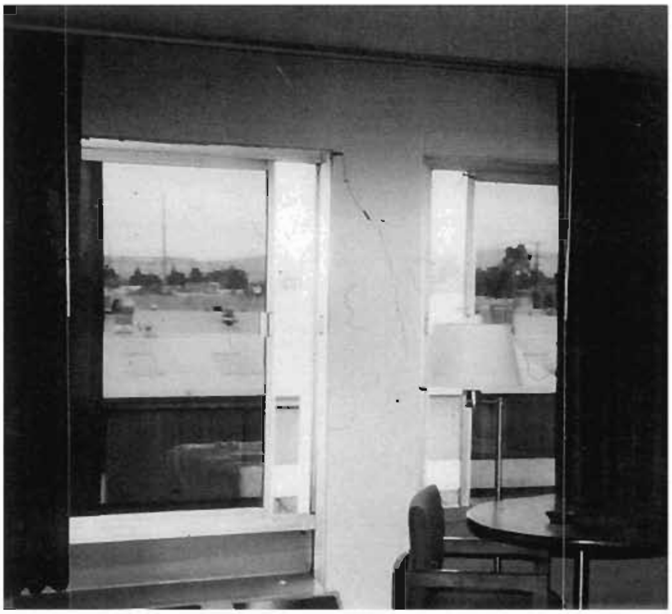


Plate 6.101: Column cracking at first floor on South side.
(only visible internally)

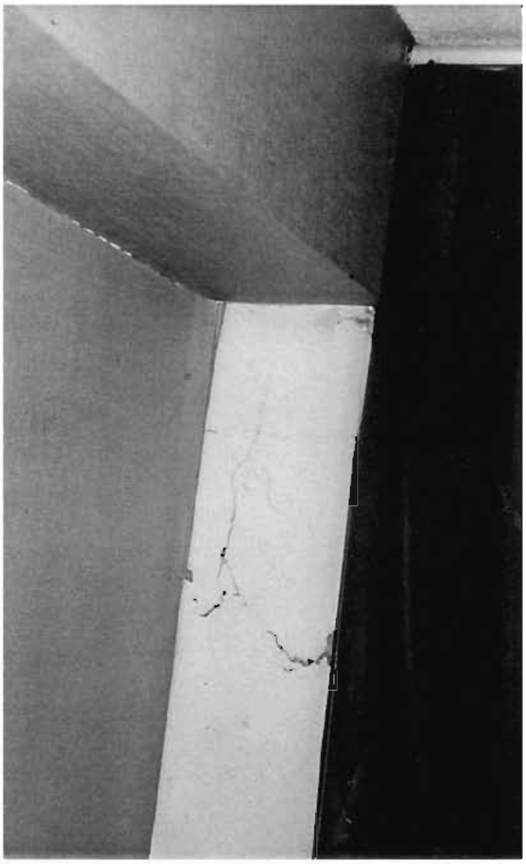


Plate 6.102: As plate 6.95 but by end wall

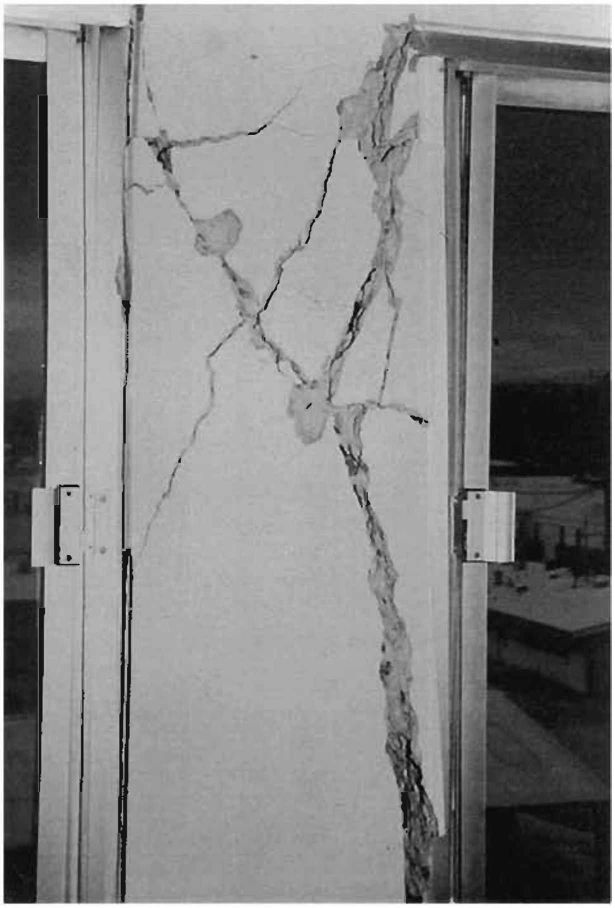


Plate 6.103: Internal view of X-cracking on 4th storey column

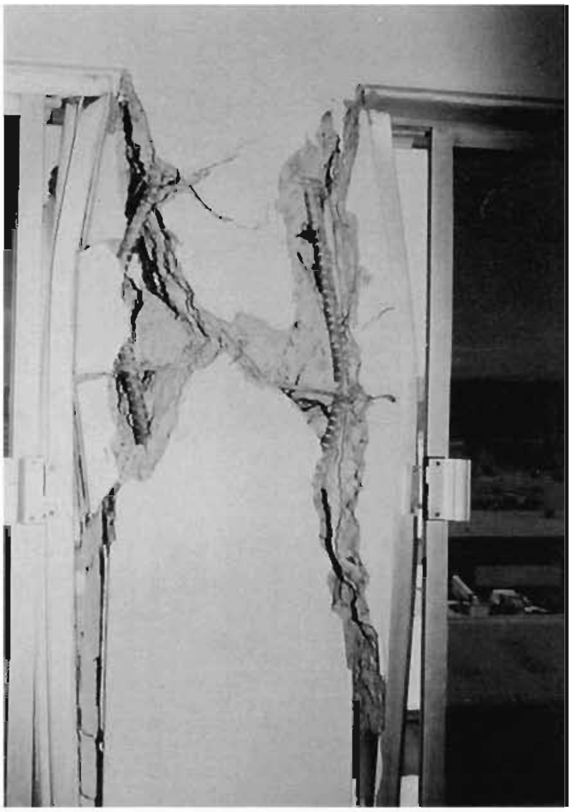


Plate 6.104: Internal view of X-cracking where tie failed

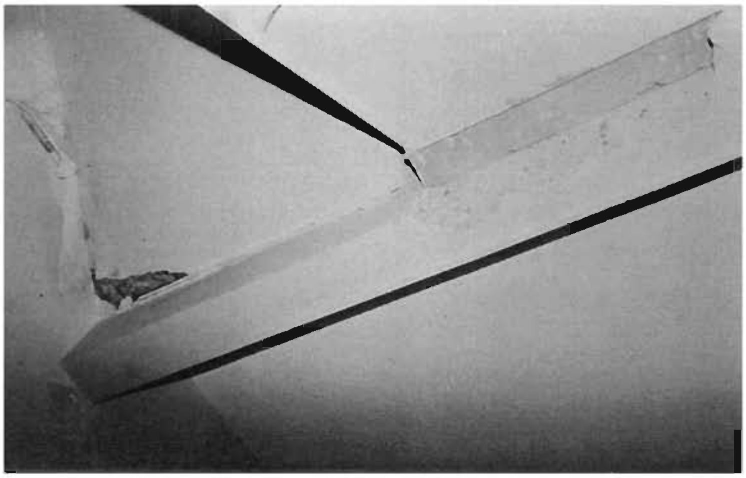


Plate 6.105: Damage to precast flights at landings

Plate 6.107: Minor cracking to internal columns at West end

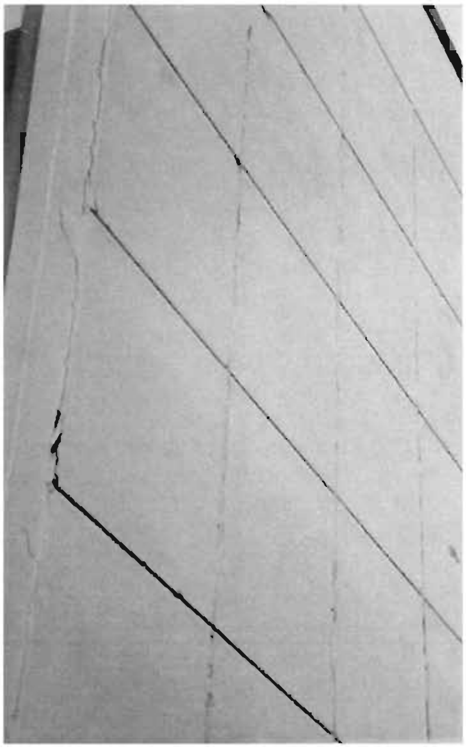


Plate 6.106. Vertical and horizontal cracks around panels on East face

Plate 6.108: Overturned furniture on top floor

Plate 6.109: Minor damage to wall linings at 4th floor

Plate 6.110: Carpetting and hydraulic fluid in stairwells by lift

Plate 6.111: Extensive wall cracking in AT&T building

Plate 6.112: Flexural wall cracks in First Interstate Bank building

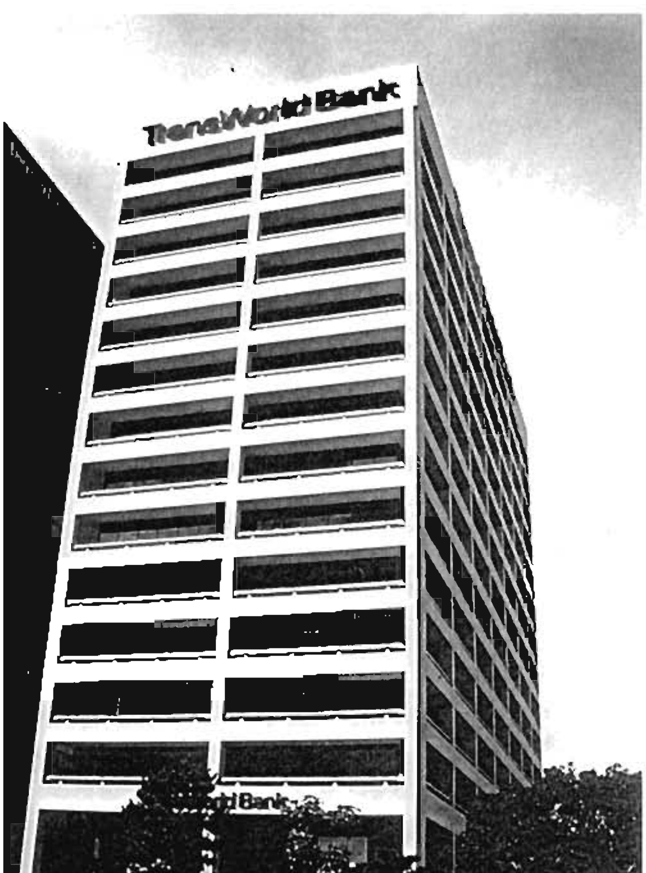


Plate 6.113: Transworld Bank

Plate 6.114: Repairing the cracks

Plate 6.115: Radisson Centre Hotel

Plate 6.116: Cracked shear wall above low level roof

Plate 6.117: Buckled shear wall reinforcement above low level roof

Plate 6.118: Armed Forces Recruiting Centre, Santa Monica

Plate 6.119: Wall cracks in coupled shear walls (NW corner)

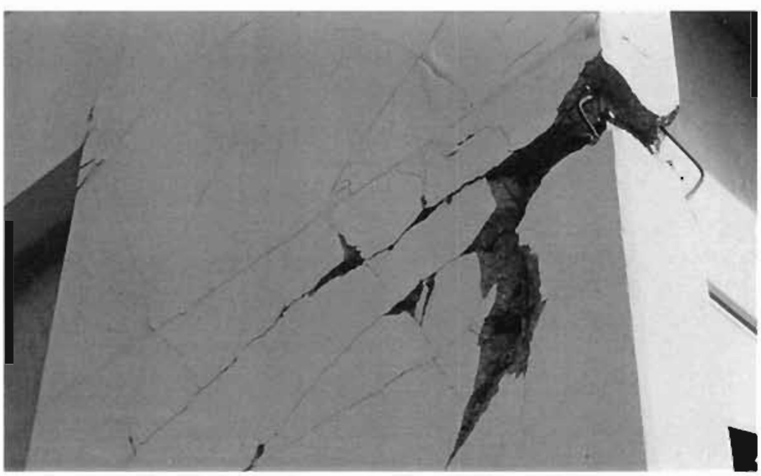


Plate 6.120: Flexural compression failure at first floor level

Plate 6.121: Comparable damage in SW corner

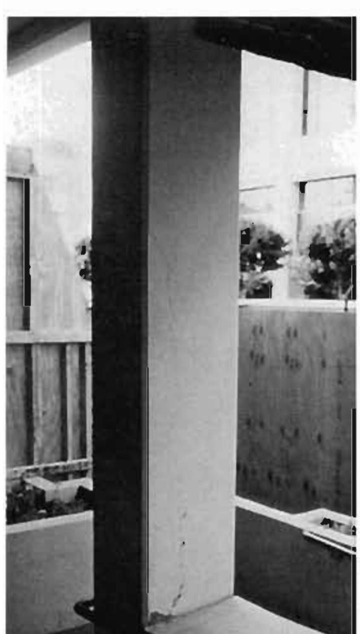


Plate 6.123: Transverse crack in column

Plate 6.122: Vertical splitting of column

Plate 6.124:
Building designed to pre-1971 edition of UBC between two buildings designed to post -1971 editions

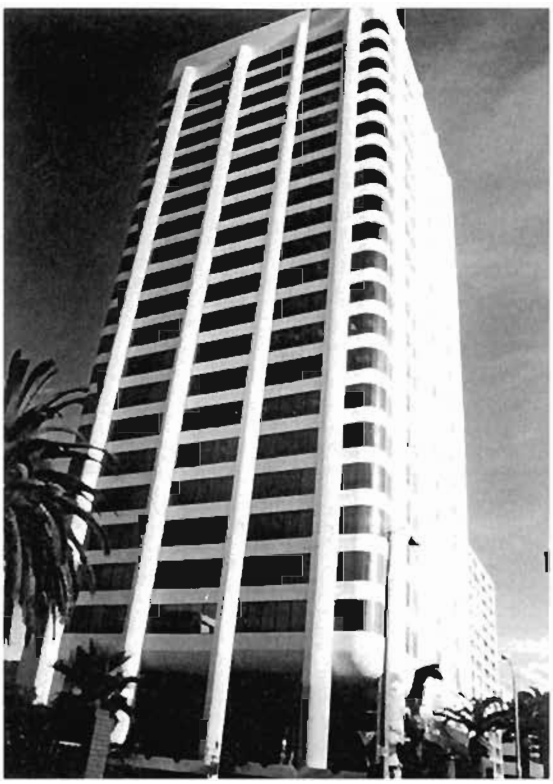


Plate 6.125: Sumitomo Bank. North face

Plate 6.126: East face

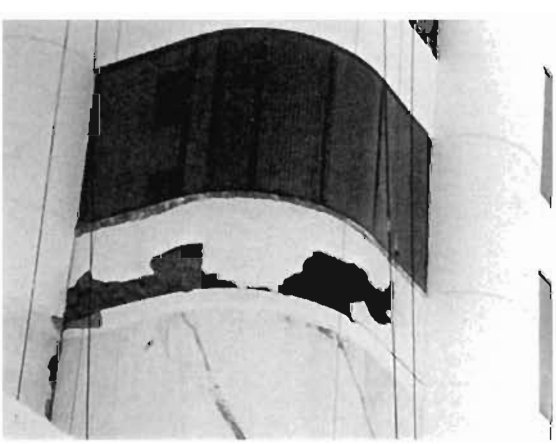


Plate 6.127: Damage at SE corner

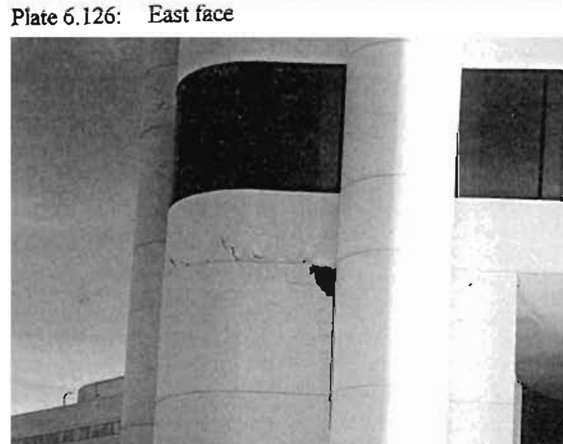


Plate 6.128: Damage at NE corner

Plate 6.129: East face of Champagne Towers, Santa Monica

Plate 6.130: West face

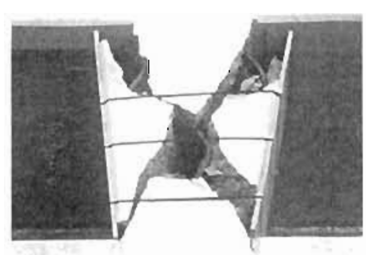


Plate 6.132: Temporary ties and plates

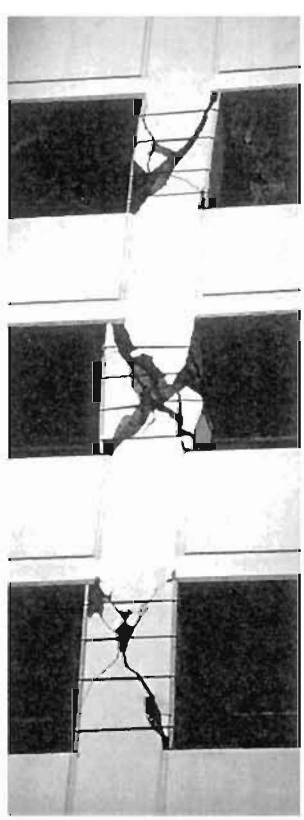


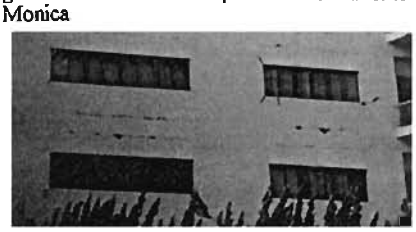
Plate 6.131: X-cracking in columns on East face

Plate 6.134: X-cracking in coupling beams on South face

Plate 6.133: Temporary support

Plate 6.135: Horizontal cracking around window panels in first floor beam in office building,

Plate 6.136: Damage to first floor window panels in apartments in Santa Monica



Close-up of damage in plate 6.136

Plate 6.137: X-cracking in beams of 12 storey building in Santa Monica

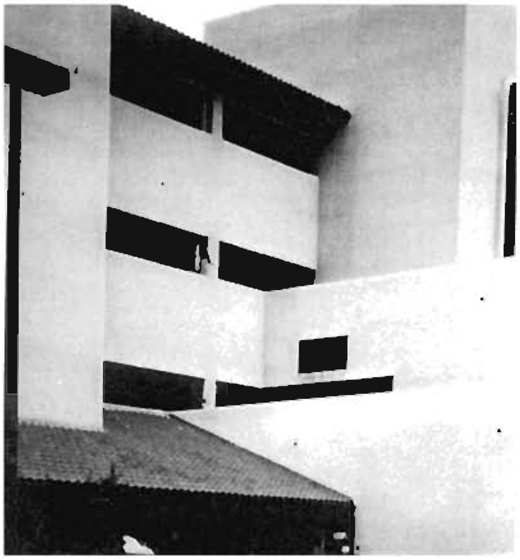
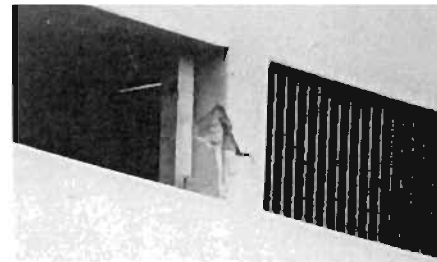
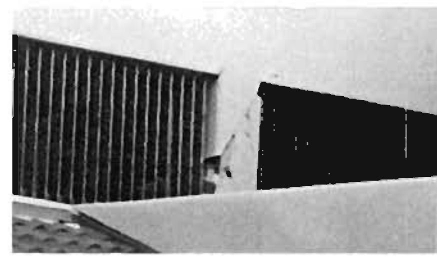
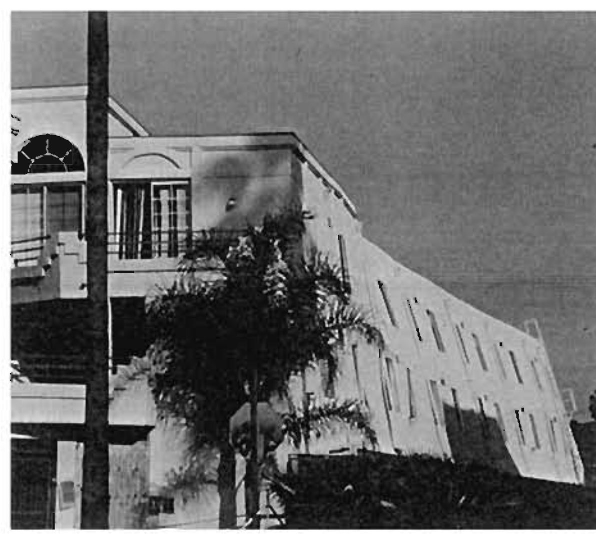


Plate 6.138: San Fernando County Building





Plates 6.139/140: X-cracking in stub columns



Plates 6.141/142: Triad Apartments: Inwards list of East and West walls after internal collapse

Plate 6.143: Close-up of East wall

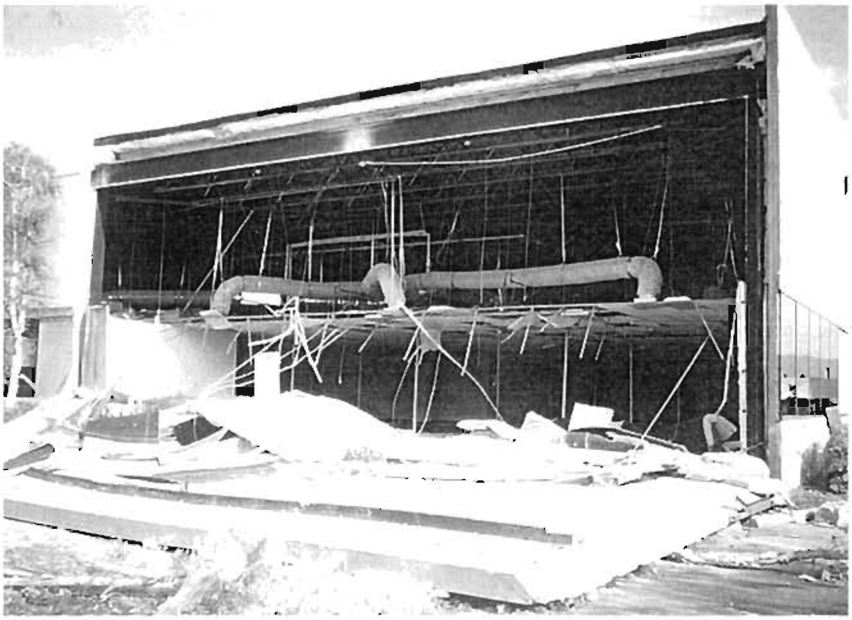
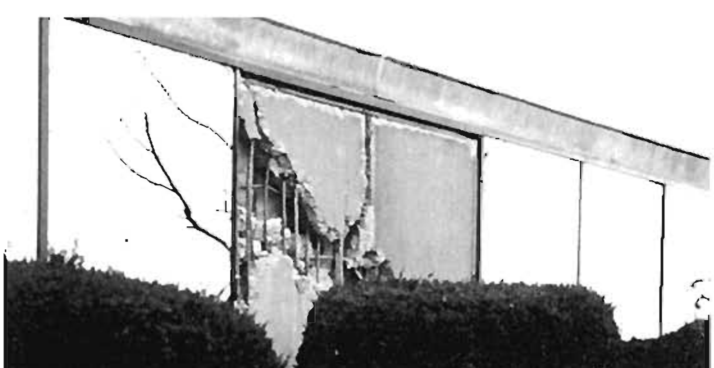


Plate 6.144: Collapsed wall of 2-storey tilt-up



Plates 6.146/148: Failed shear panels over height of ground floor fenestration

Plate 6.149: Collapsed chimneys

Plate 6.150: Damaged roof

Plate 6.151: Collapsing porch

Plate 6.152: Racked porch columns

Plate 6.153: Collapsed crawl space

Plate 6.154: Damaged crawl space

Plate 6.155: Stucco damage at crawl space in apartments

Plate 6.156: Stucco fallen from plywood sheet

Plate 6.157: Stucco fallen from insulated timber frame apartment

Plate 6.158: Collapsed part and part with collapsed soft storey

Plate 6.159: Ground floor weak storey accommodating car ports

Plate 6.160: Streetwise movement of building in plate 6.158

Plate 6.161. 3-storey condominium

Plate 6.163: Low level horizontal crack despite absence of crawl

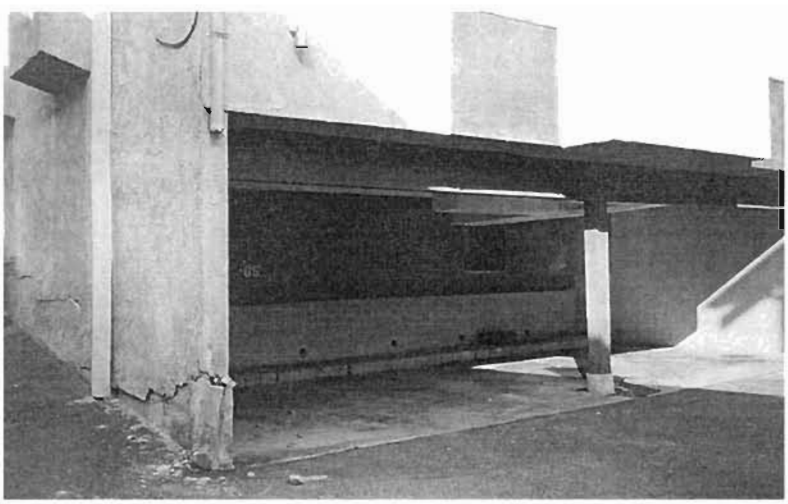


Plate 6.162: Damaged wall of car port

Plate 6.164: Close-up of timber poist

Plate 6.165: Damage exposing wall interior

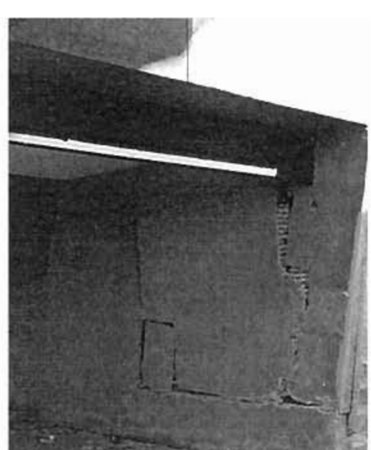


Plate 6.166· Damaged wall other end car port

Plate 6.167: Collapsed house front

Plate 6.169: Top storey side wall collapse

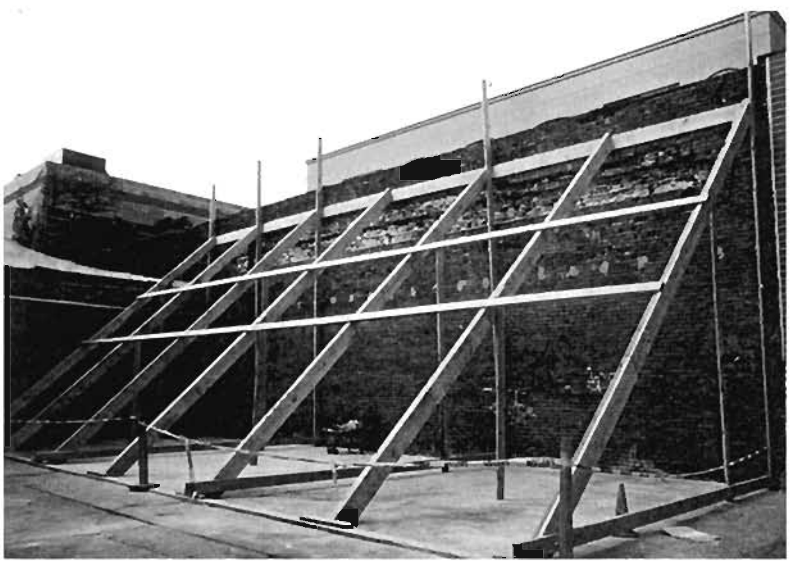


Plate 6.170: Brick wall in Santa Monica

Plate 6.168: Collapsed shop front

Plate 6.171: Delaminated outer skin

Plate 6.172: Cracking lower down

Plate 6.173: Old shops with reinforced masonry columns with embedded steel posts in Santa Monica

Plate 6.174: Disintegrating columns with steel posts

Plate 6.175: Corroded steel post

Plate 6.176: Similar construction in San Fernando

Plate 6.177: The pier-mounted tube

Plate 6.178: Corrosion protection

Plate 6 179· Reinforced masonry building

Plate 6.181: Tilted tree at the Hamlet

Plate 6.180: The damage

Plate 6 183. Fallen masonry at end of wall

Plate 6.182:
Low level disintegration of reinforced masonry

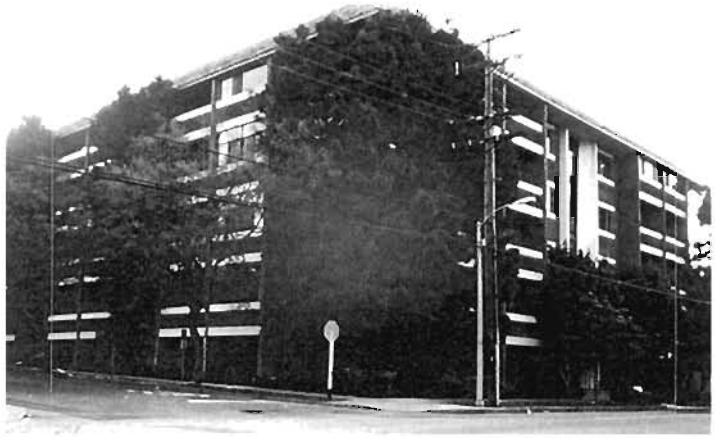


Plate 6.184: Berkeley Convalescent Hospital

Plate 6.185: Low level disintegration of reinforced masonry

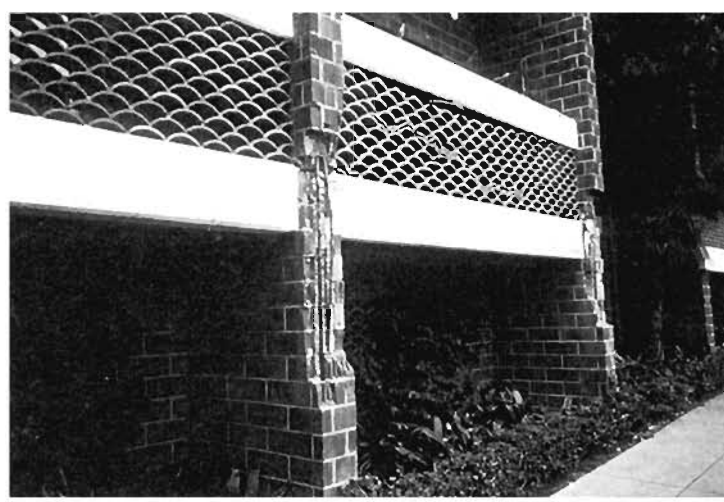


Plate 6.188: Comparable buckling at wall ends

Plates 6.186/187: Buckling of unrestrained vertical bars between windows

Plate 6.189: Local collapse in building retrofitted only at roof

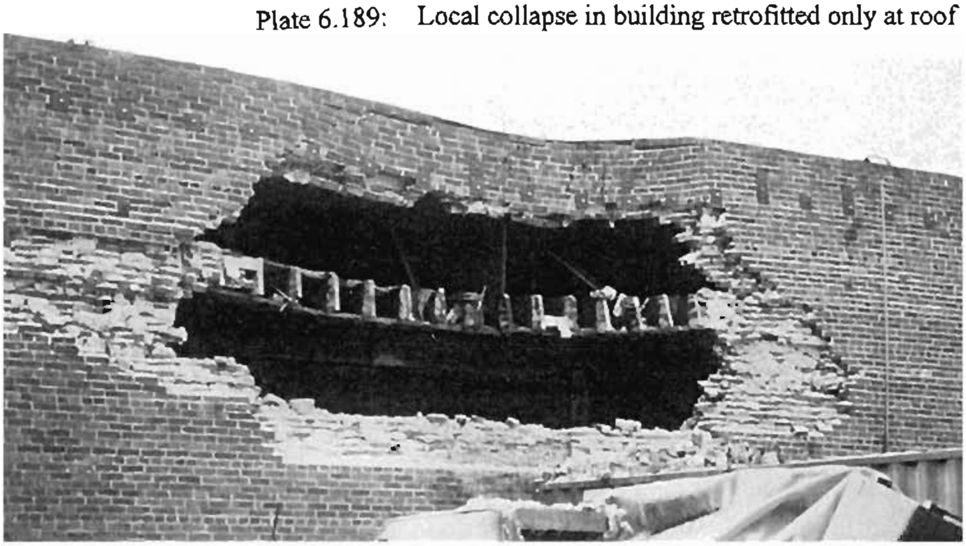


Plate 6.190: Wall collapse at unretrofitted intermediate floor

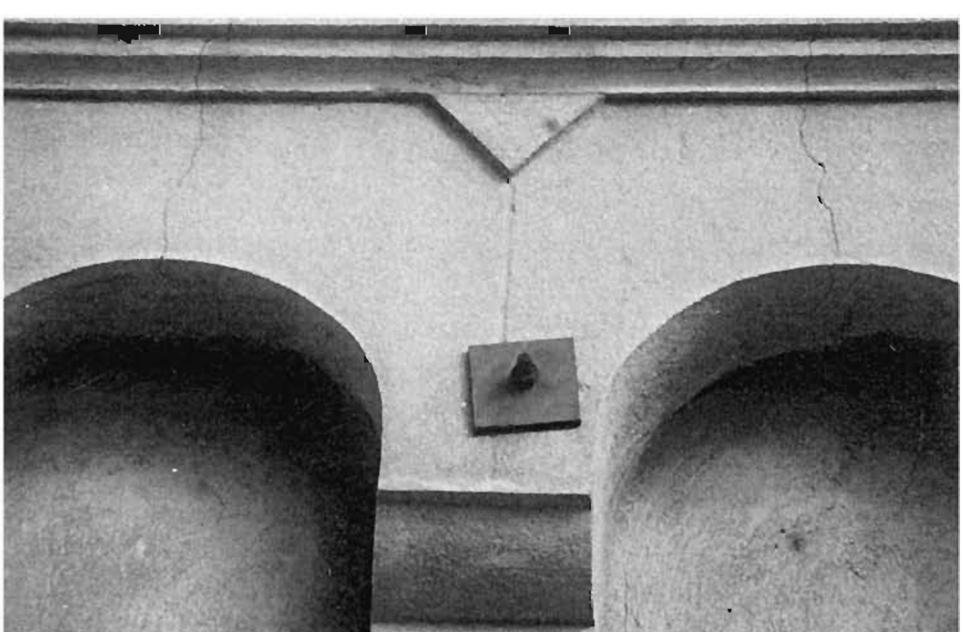


Plate 6.191: Anchor plate of tie

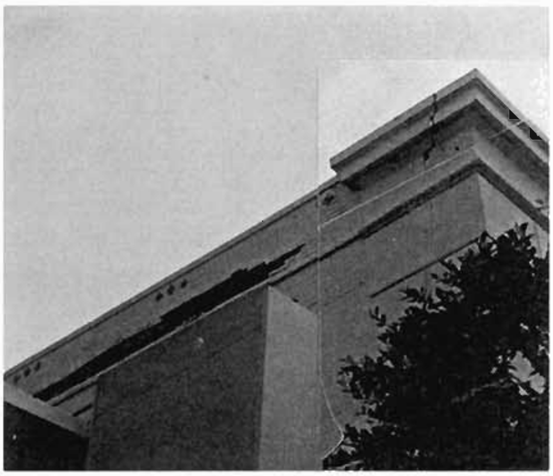


Plate 6.192: Parapet cracking due to tie flexibility

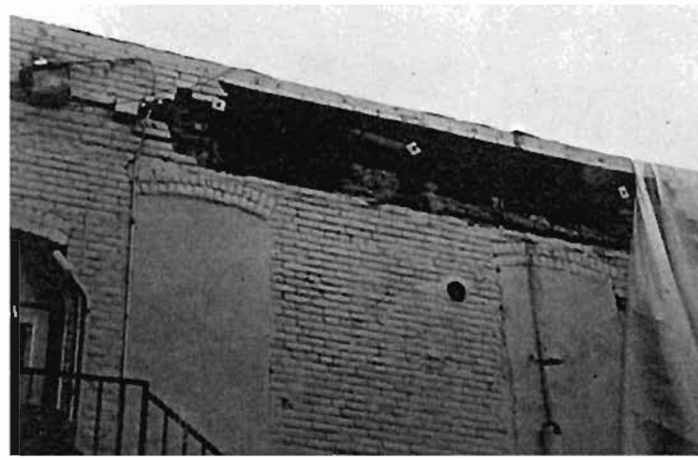


Plate 6.193: Fallen masonry in vicinity of wall ties

Plate 6.194: Fallen parapet with stays but no roof ties

Plate 6.195: Failure of masonry in vicinity of wall ties

Plate 6.196: Partial collapse of retrofitted parapet

Plate 6.197: Parapet/upper wall failure at rounded corner

Plate 6.198: The first brick building in Santa Monica

Plate 6.199: In-plane cracking of masonry by door

Plate 6.200: Commemorative plaque

Plate 6.201: The oldest substantial building in Santa Monica (1926)

Plate 6.202: Retrofitting and separation at corner

Plate 6.205: Retrofitted masonry on timber substrate

Plate 6.203: Loss of outer skin below the ties in building retrofitted only at roof level

Plate 6.206: 9" wall retrofitted with ties

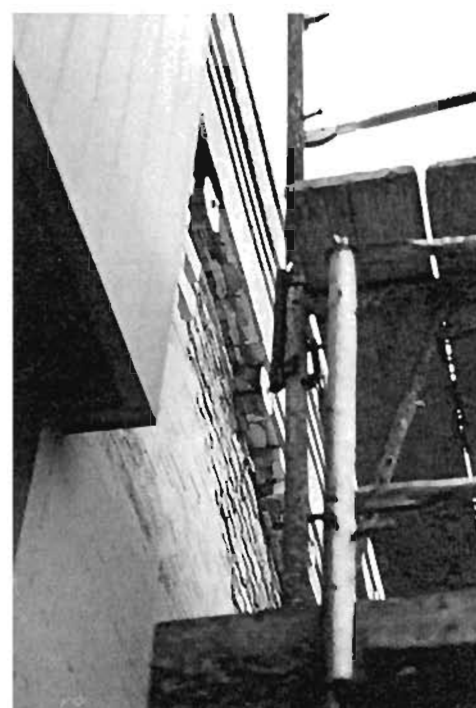


Plate 6.204: Plane of delamination and ties limiting extent of failure

Plate 6.207: 9" wall retrofitted with flats above windows

Plate 6 208· A discretely retrofitted masonry clad building with shop fronts

Plate 6 209· A discretely retrofitted masonry clad building

Plate 6.210: Close-ups of retrofitting

Plate 6 211: Medium rise building with retrofitted cladding

Plate 6.212· A masonry clad framed building with continuous fenestration except one end

Plate 6.213: Structural damage at end of fenestration

Plate 6.216. Highly irregular shopping area. Santa Monica

Plate 6 217: Diagonal crack in floor

Plate 6 219: Undamaged building irregular in both plan and elevation

Plate 6.215: Undamaged masonry clad building with vertical irregularity

Plate 6.214: Undamaged building with extreme vertical irregularity

Plate 6.218. Undamaged building with extreme vertical irregularity

Plate 6.220: Church of light construction in San Fernando

Plate 6.221: Cracking in irregularity above porch

Plate 6.222: Long low apartments with rigid walls bolt end in Santa Monica

Plate 6.223: Damage in end shear wall in apartments in Santa Monica

Plate 6.224: Crack above car park entrance at end of building

Plate 6.225: Stucco damage in old decorative U-shaped building in Santa Monica

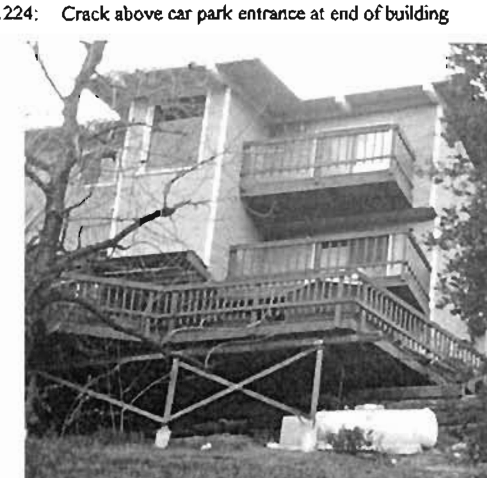


Plate 6.226: Undamaged building on crest of hill

7. INDUSTRIAL FACILITIES

I R Morris
British Nuclear Fuels plc

P A Merriman
British Nuclear Fuels plc

7.1 Introduction

The San Fernando Valley area of the Los Angeles conurbation, which was subjected to the strongest ground shaking in the Northridge earthquake, is largely residential with some commercial developments and educational establishments. Industrial facilities in the Valley are typically housed in low-rise (one or two storey) buildings and mostly serve the aerospace industry. Notable amongst the companies trading in the affected area are the Lockheed Aircraft Company around Glendale/Burbank airport in the east of the valley (16km from Northridge) and the Rockwell International Corporation Rocketdyne Division at Canoga Park towards the west end of the valley (7km from Northridge). Heavy industries are largely absent from the most affected areas although there is a large Anheuser-Busch brewery at North Hills (Northridge 6km) and aggregate handling plants at Sun Valley (Northridge 13km). In the north eastern corner of the San Fernando Valley (Northridge 11km) there is a gas fired power station and the Sylmar Converter Station (internationally famous for its contribution to the electrical supply industry and seismic databases). On the principal sites visited, the following subsections will primarily consider the chemical, electrical and mechanical handling aspects of industrial facilities.

7.2 Aggregate plants

Two aggregate handling facilities were visited at Sun Valley 13km to the East of Northridge (plants adjacent to each other on Sheldon St. by its junction with San Fernando Rd.). The smaller site was operated by Industrial Asphalt and had typical aggregate handling, screening and tar coating plant. The larger site operated by CalMat (California Materials) had aggregate crushing, screening and handling plant including that necessary to supply a large stockyard. At the time of the visit (a Sunday morning) no work was being carried out the at the Industrial Asphalt plant but the CalMat plant was busy supplying aggregates to sites where earthquake damage was being repaired. It was reported to us by site staff that both the Industrial Asphalt and CalMat facilities were unaffected by the earthquake.

The main structures on both sites are steel framed with light weather cladding in some cases. Many of the structures were framework towers and single span truss bridges to support conveyor systems (Plate 7.1). The towers for the CalMat stockyard area are tall but of average slenderness and not noticeably different in construction from UK practice.

Some damage to the stanchion supports to conveyors was observed but this was consistent with them being struck by road vehicles ie. heavy dump trucks, rather than earthquake damage. The damage, although very noticeable consisting of a partial buckle of the whole column section, did not seem to impair the function of the stanchion sufficiently to warrant action. Another light steel framed support structure with rod and turnbuckle type X-bracing had one broken tie (Plate 7.2). Whether this damage was a result of the earthquake or just ordinary wear and tear could not be ascertained. Again this damage did not impair the function of the supported plant.

Apart from the conveyor systems the Industrial Asphalt plant had a variety of screening and loading plant and a rotary kiln for coating aggregate with tar. Support systems for these were varied, with some being single tubular stanchions well anchored to individual concrete plinths. At the CalMat facility there were substantial vibrations on the crushing plant during normal operation. This suggests that there would be reserves of strength to resist externally imposed vibration. Hence it is hardly surprising that for the Northridge earthquake which occurred at night when the plant was not operational, the heavily constructed plant should be unaffected.

7.3 Brewery - North Hills

Anheuser-Busch, well known for the brewing of Budweiser beer, operate a large and modern brewery producing 12 million barrels of beer per year at 15800 Roscoe Boulevard, North Hills, 6km to the east of Northridge.

A tour of the principal parts of the plant was made accompanied by the plant manager Gary Lee. His first priority on reaching the site in the early hours of 17th January was to account for the safety of all of his employees. Mr.Lee also discussed with us the overall seismic performance of the plant which was operating at the time of the earthquake since the large brewery is essentially a continuous operation plant albeit with some aspects of the brewing process being batch operated to a 28 day cycle.

The principal types of plant at the brewery are process vessels, pipework, refrigeration and heating plant, water treatment and carbon dioxide handling equipment, product canning, warehousing and dispatch. Overlaid on these are the attendant instrumentation and control equipment and power supplies.

Overall the brewery sustained very light damage. All the buildings remained standing but with some loss of cladding to the largest steel framed building and bulging of the walls of tilt-up concrete structures (which was typical of this building type). In the warehousing area some stacks of cans were reported as having fallen over but had not caused significant failure of the warehouse building through impact loading. Piping suffered some leakage but no tanks were reported as having lost their contents. Glass lined tanks were present on the site (to hold cleaning acids for example) and at least one tank lining failed. The air systems which were of metal construction had only low damage. Electrical power was lost from the power utility. Power was restored using standby generators some of which were obtained from as far away as San Francisco. These were retained for some days until the mains power was considered reliable. Other damage reported was a pasteuriser moving off its pedestal and some conveyor falls.

Immediately following the earthquake, the main technical concern was for the security of the refrigeration system which uses ammonia. This system was fitted with automatic seismic trips set to 1g acceleration to isolate the system and ammonia in bulk. The trips worked successfully. Liquid levels in tanks were remotely monitored and some sensors gave spurious indications. Return to operation of the plant required the various systems to be functional but also required the water supplies to be of good quality for which they were subject to additional monitoring.

In visiting the site a variety of measures to protect against earthquakes could be seen in the construction of the plant. Only the more notable are considered here. Vertical cylindrical tanks supported by legs had diagonal bracing between the legs (Plate 7.3). Where unbraced legs were used for tank support the legs were of larger cross-section than for the unbraced types and the were attached to the vessel shell rather than the vessel head. (Plate 7.4). Horizontal cylindrical tanks were mounted on saddles with rings around the vessel (Plate 7.5). Within the main brewery building some large horizontal cylindrical stock tanks (about 3m diameter and 12m long) were also in ring type saddles but with additional diagonal bracing to strengthen the saddles in the out-of-plane (vessel axial) direction. Despite these tanks being large and heavy they were mounted one upon another on common supports with the design being adequate to cope with the large loads that would have been developed in the supporting system. Heavy steelwork connections similar to those in the UK nuclear industry were observed.

Overall the brewery, despite being very large (on a site a half by a quarter of a mile) with large, tall and heavy plant, and being close to the earthquake epicentre suffered only light damage. Importantly for the brewery none of the stock in production, amounting to 28 days of product, was lost nor was the quality of the product affected.

7.4 Aircraft component manufacturing

Although there are many aerospace facilities in the San Fernando Valley which would have been of interest to visit there was only time available to visit the larger sites closest to the earthquake epicentre. Two sites where aircraft components are manufactured/tested were visited. These were at Canoga Park 7km west of Northridge and at Van Nuys airport only 3km to the east of Northridge. The performance of the plant at each site is described.

7.4.1 Van Nuys

Kaiser Marquardt operate a plant on the west side of Van Nuys airport in support of the aerospace industry. A wide range of equipment and process services are present and date from the original construction in the 1930s through to the present. Due to the diversity of the equipment only brief notes on seismic performance with appropriate illustrations are given in Table 7-1.

All the older buildings performed satisfactorily with only one bracing member being observed to be bowed out of position but still intact. The single storey brick workshops dating from the 1940s, which had wooden truss roofs with some steel tension elements, performed well. One wooden truss joint had suffered damage but this could readily be removed and replaced. One of the larger, two storey modern buildings adjacent to the Kaiser Marquardt facility which had a steel frame, suffered extensive lateral movement causing windows to break and suspended ceilings to fall. The major risk to life may have been in this administration area where collapse of office equipment, eg. filing systems, computer equipment, in a highly occupied area would have caused injuries if the earthquake had occurred in working hours. The administration area was attached to a tilt up factory unit which had suffered damage requiring temporary propping of the walls to enable repairs to be carried out.

7.4.2 Canoga Park

Rockwell International Rocketdyne Division at Canoga Park, have a large manufacturing facility. Unfortunately due to insufficient time it was not possible to arrange security clearance to tour the works. However Rus Nester, a seismic engineer for Rockwell, kindly discussed at length the seismic performance of the plant and equipment at the site. The following notes are based on this discussion.

Rockwell have instituted a programme of improving earthquake safety since the 1987 Whittier Narrows earthquake. This programme has concentrated on life safety issues. An example of this is the anchoring and provision of snubbers to three large low/medium pressure boilers. Two years ago the boilers were not anchored nor provided with flexible connection lines. In general the majority of the newer replacements and additions to equipment at the factory performed well in the earthquake although some styles did not.

Table 7-2 summarises the performance of the various equipment types to be found at the Rockwell Canoga Park plant.

Equipment type	Seismic performance
Tanks	Three cylindrical horizontal tanks of 30,000 gallon capacity and mounted on concrete saddles were present. One of the tanks (Plate 7.6) rotated in its saddles (as evidenced by the lettering no longer being horizontal). This together with any overall rocking of the tank and saddle assembly strained the pipework inter-connections to be seen at the top of the tanks causing them to leak. Fortunately no fire broke out. However the tanks had to be emptied into road tankers as a matter of urgency before repairs could be effected. This was done successfully although it was difficult to provide sufficient tankers at short notice. Fire at this site was a particular hazard since in addition to propane, bulk supplies of hydrogen and hydrazine are present. Despite the cause of failure of the pipe inter-connections being well understood by the plant operators, the repair was of identical pattern to that which failed in the earthquake due to the urgency of reinstating pressure containment. Anchorages on skirt and flat bottomed tanks had been strained by the earthquake. Four identical and adjacent skirt supported 26m high compressed air receivers had strained anchors (Plate 7.7). These were strained by greater amounts towards one end of the row of vessels. The holding down bolts had been vigorously retightened before the authors' visit (breaking the wrench in the process). The air receivers were of all welded construction with the support skirt attached to the vessel shell rather than the vessel head. An older all-bolted flat bottomed tank had strained its anchorages and bulged a little but did not appear to have leaked (Plate 7.8). A vertical tank mounted on brackets and within a framework but unrestrained to the ground had moved both in translation and rotation (Plate 7.9).
Machine tools	Machine tools (milling, planing, etc. and more specialist equipment eg. an electron beam welder) which were all unanchored had slid across the floor. Movements of 0.6m were measured. Connection lines were disrupted by this movement but the machines were generally not otherwise damaged. It is fortunate that no personnel were at work at the time of the earthquake as injuries would certainly have resulted from the different machines' movement taking up all space been the machines. It is notable that the final positions of different equipment indicated overall movements be in different directions with some equipment also rotating in plan.
Miscellaneous	The spurious operation of the sprinkler system filled the sump in which a vapour degreaser machine was located. The machine was not anchored and was floating within the sump at the time of the authors' visit, restrained from turning turtle by an access platform. To protect plant from a hydrogen explosion should part of the hydrogen system fail, a 2.4m high blast wall made of blockwork had been erected around a pipework and valve assembly. Although the hydrogen system and the associated fire-fighting system was intact the blast wall had been damaged by the earthquake due to inadequate continuity of the wall at a corner (Plate 7.10). An overhead travelling crane was present in one building and appeared intact. In the machine shops old fluorescent lighting units which were much heavier than current designs had fallen. Difficulty would have been experienced in evacuation of the machine shops without further injuries because of failure of the lighting, movement of the machine tools and debris on the floor. The lighting had still had not been restored at the time of the authors' visit although some natural lighting was available.

Table 7-1 - Seismic performance of plant and equipment at Kaiser Marquardt, Van Nuys.

Equipment type	Seismic performance
Machine tools	There are many machine tools in the factory. Some are of the largest sizes and most modern types eg. 5 axis tape driven numerically controlled machines. The manufacturers' installation specifications require that the machines are not anchored to the floor. All the machines slid causing breakages to coolant and oil lines. Movements ranged from 50 to 300mm. Replacement of the machines and repairs were effected within a week, after making safe overhead hazards.
Tanks	Various tanks both open (non-pressurised) and closed (pressurised) are present, designed to withstand 0.5g acceleration. Some tanks are quite large eg. polypropylene dip tanks to 100 tons capacity. Fibreglass and polypropylene tanks for deionized water were subject to failures. One tank rocked/tipped over showing a brittle failure. The sudden release of liquid demolished an adjacent wall and damaged other equipment. Another identical tank ruptured more slowly and a third destroyed an adjacent door when it moved and broke into three pieces. Open rectangular tanks anchored to the floor are used for plating and pickling components. Up to a third of the contents of the tanks was lost through sloshing.
Fire fighting tanks	Rockwell fire fighting staff extinguished the only fire within half an hour. The fire fighting water tanks had seismically protected pipelines from the tanks. The tanks themselves moved up/down by 300mm and by 150mm horizontally.
Pipelines	At the time of the authors' visit the plant operators were studying which pipelines failed and why. Gas lines were fitted with commercially available shut off valves from various manufacturers. These all worked. It was noted that there is a statutory requirement for these to operate at 0.1g and in around 0.4 seconds.
Cranes	EOT (Electric overhead travelling) cranes are used in the factory. Crane hangers and lateral bracing failures occurred. These are failures of the crane supporting structures rather than the cranes themselves. Mr.Nester expressed concern that the codes of practice for crane design and for the design of the supporting building structures do not form an adequate basis for the seismic design of EOT cranes and buildings. (Codes give little guidance in general eg. UBC only requires a 0.2g lateral load from the crane to be considered in the building design.)
Air handling equipment	Older equipment from the original plant construction in the 1950s and early 1960s is present. The principal failures were dislodging of anti-vibration (AV) mounts. Air handlers at the factory number in the hundreds. Motor sizes are to 20hp (15kW). The smaller units seemed to fare better than the large ones.
Lighting equipment	Long rows of fluorescent light units were formerly used to illuminate work areas. These had been abandoned in place when more effective metal halide units were installed. The large vertical accelerations in the earthquake snapped the clips which retained the fluorescent units on their supports producing widescale falls of the units. Portable lighting units are used which when they fell over led to a secondary hazard (in addition to the first hazard of the falling over of the units); the wiring connections failed leading to loose exposed wires which may have been electrically live. Battery powered emergency exit lights ran for about 6 hours before the batteries drained.
Miscellaneous	A water flow test structure had an anchorage failure. This was through deterioration of the anchor cross-section to as little as 5% of the original section. This was not apparent on inspection from above.

Table 7-2 - Seismic performance of plant and equipment at Rockwell International, Rocketdyne Division, Canoga Park.

7.5 Conclusions

At all the industrial sites visited there was good seismic performance of the main structural resisting systems under acceleration levels in excess of those recommended by the Uniform Building Code. Most of the damage occurred at the interface between the structural and mechanical components - anchorage failure being a common feature. In other cases the omission of anchorage support or the failure to accommodate relative movements of the equipment lead to damage. The failures represented only a small percentage of the total equipment. Overall the majority of industrial equipment showed itself to be robust and readily able to be reinstated for further use. Therefore design effort should be concentrated on identifying equipment that may cause a personnel safety risk or significant commercial risk and in ensuring that good seismic practice is carried out.

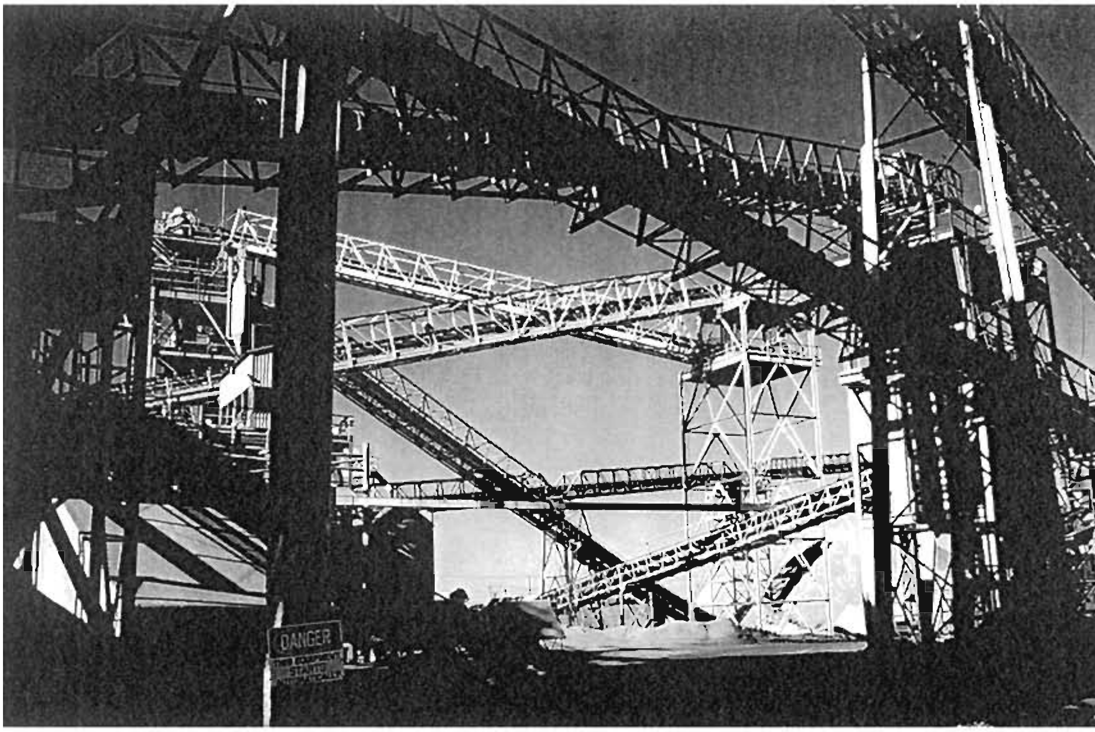


Plate 7-1 : Typical average slenderness structures at aggregate handling facility, Sun Valley

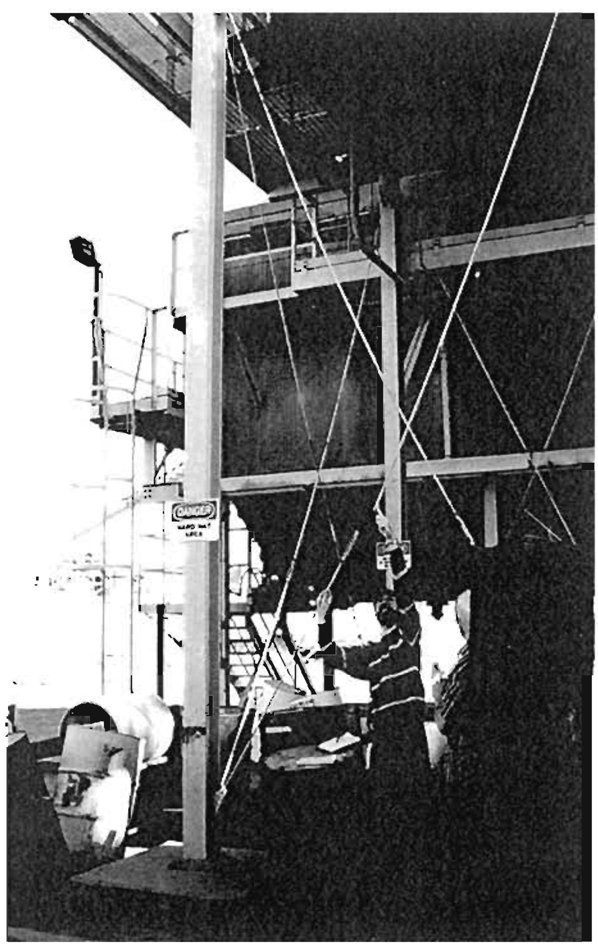


Plate 7-2 : Broken tie on light steel framed structure at aggregate handling facility, Sun Valley

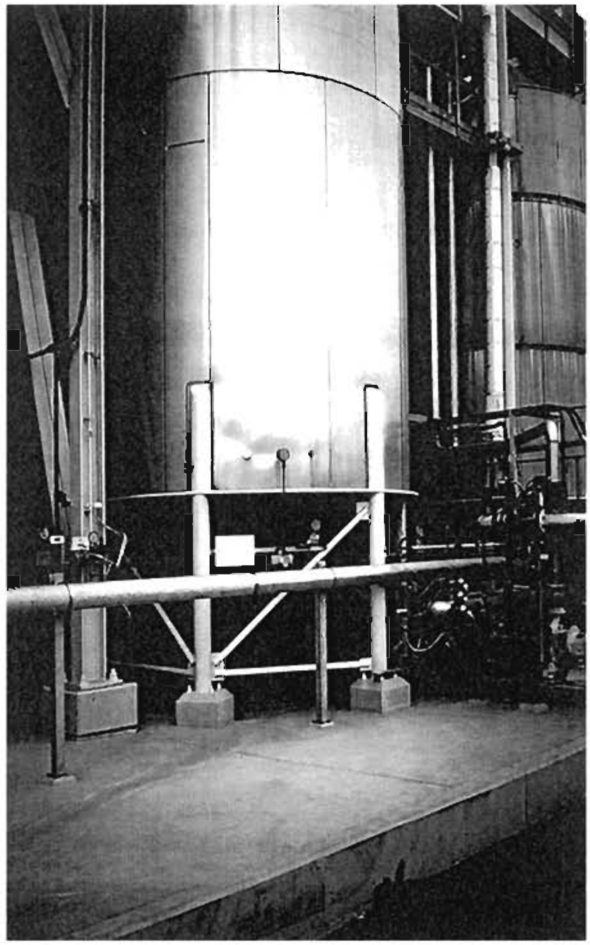


Plate 7-3 : Slender braced legs to vertical cylindrical tank, North Hills

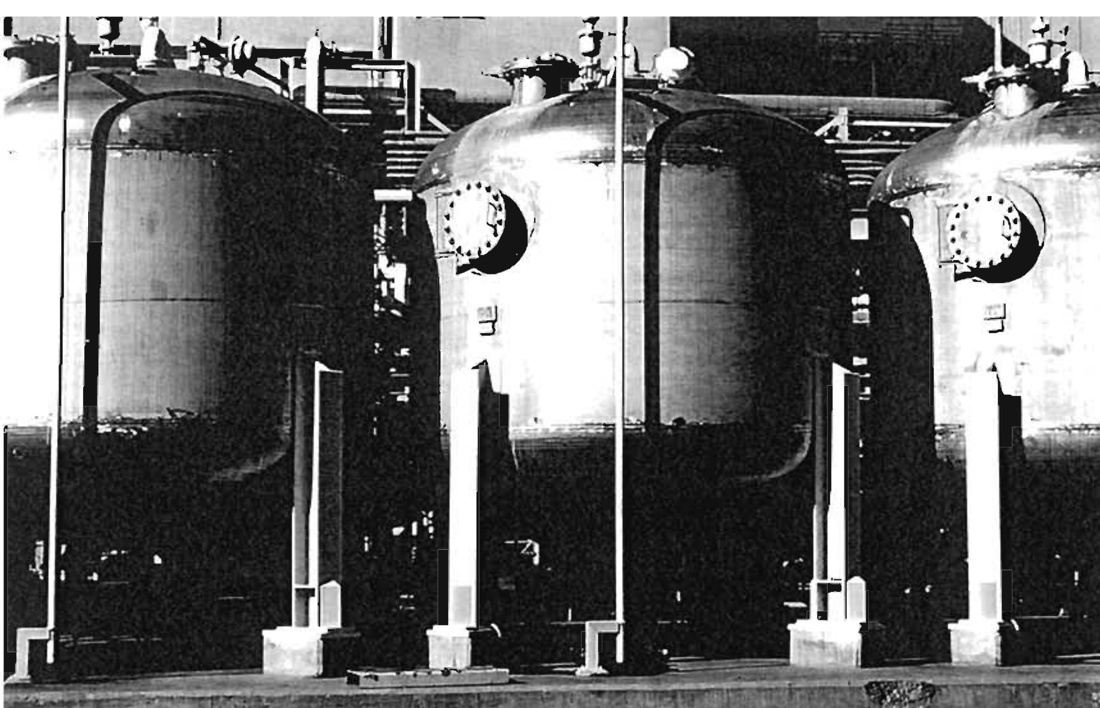


Plate 7-4 : Unbraced legs to vertical cylindrical tanks, North Hills

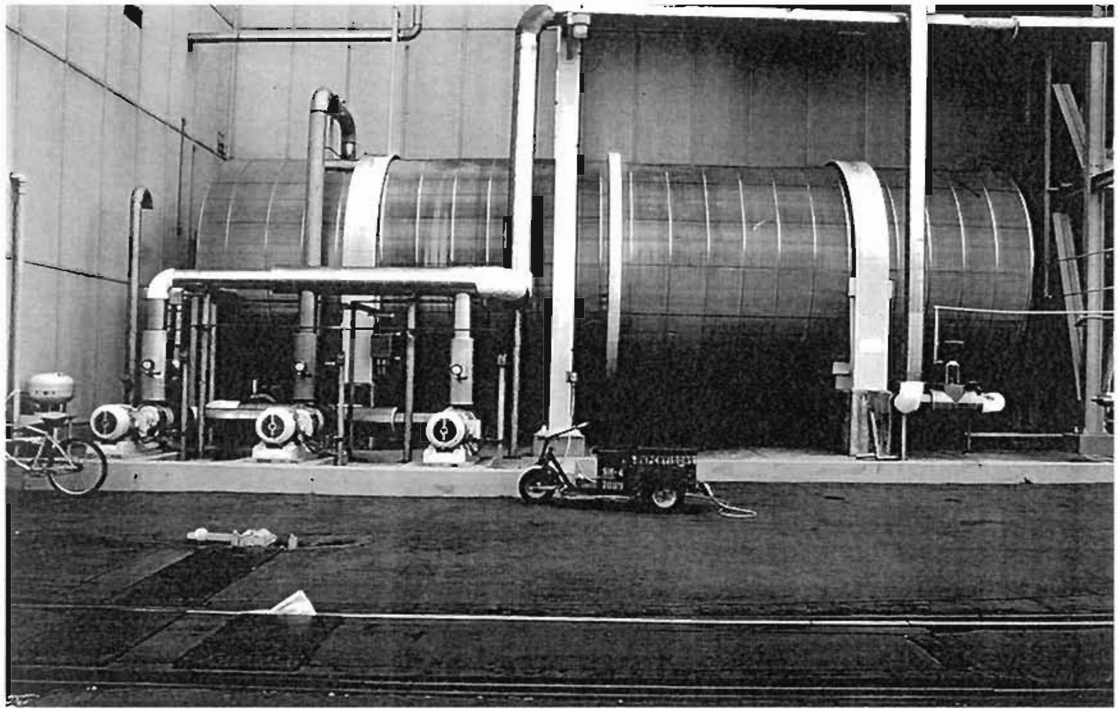


Plate 7-5 : Horizontal cylindrical vessel with ring type saddles, North Hills

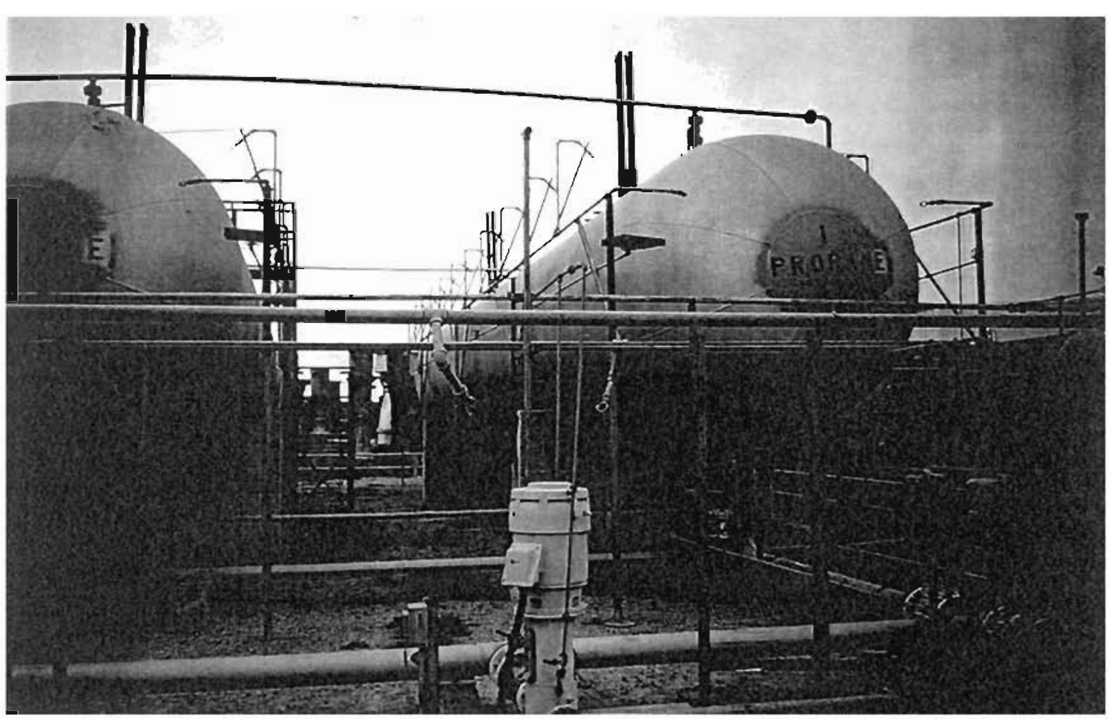


Plate 7-6 : Rotated horizontal cylindrical tank supported on concrete saddles, Van Nuys

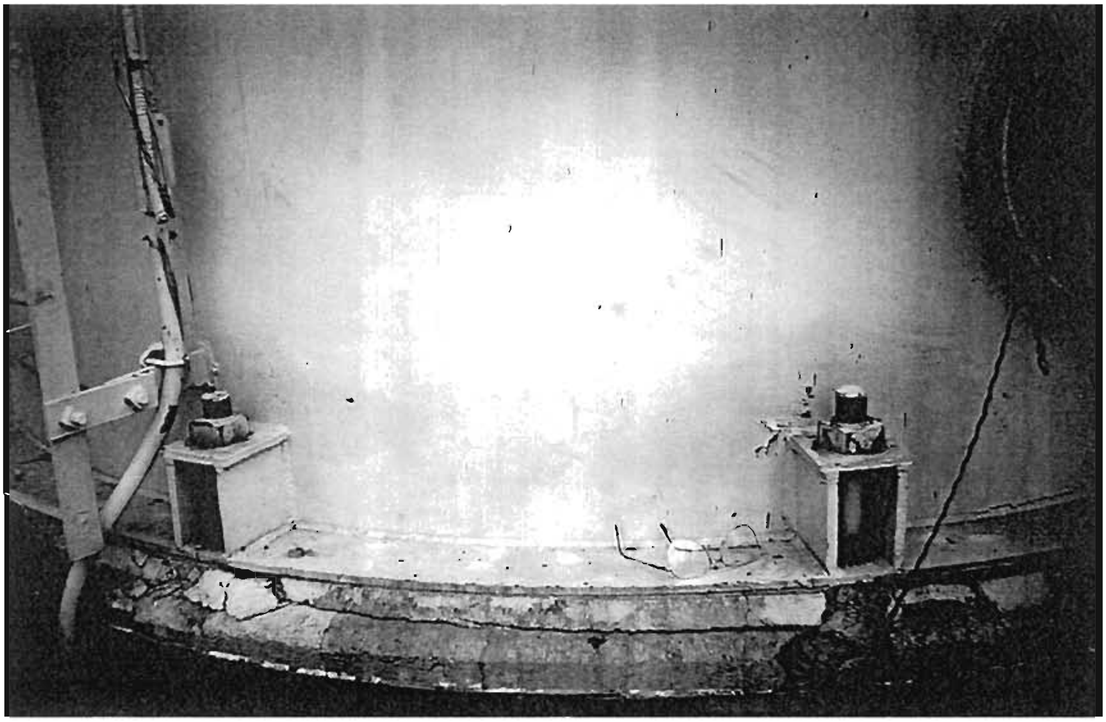


Plate 7-7 : Strained anchors (after re-tightening) on skirt support of tall air receiver, Van Nuys

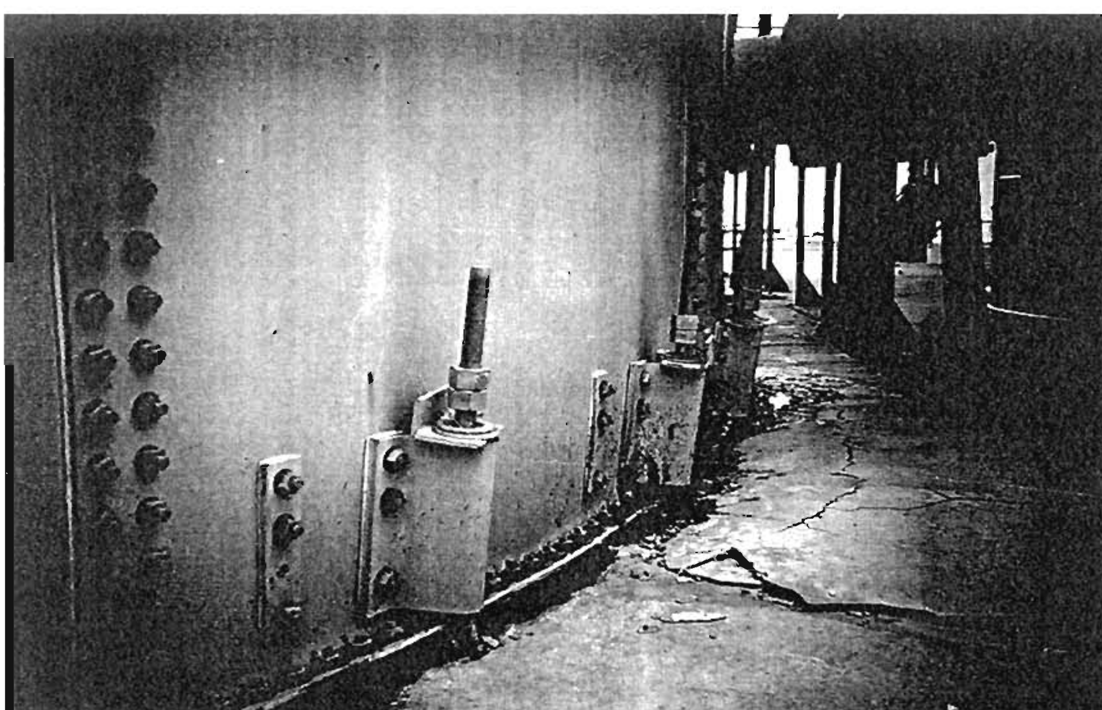
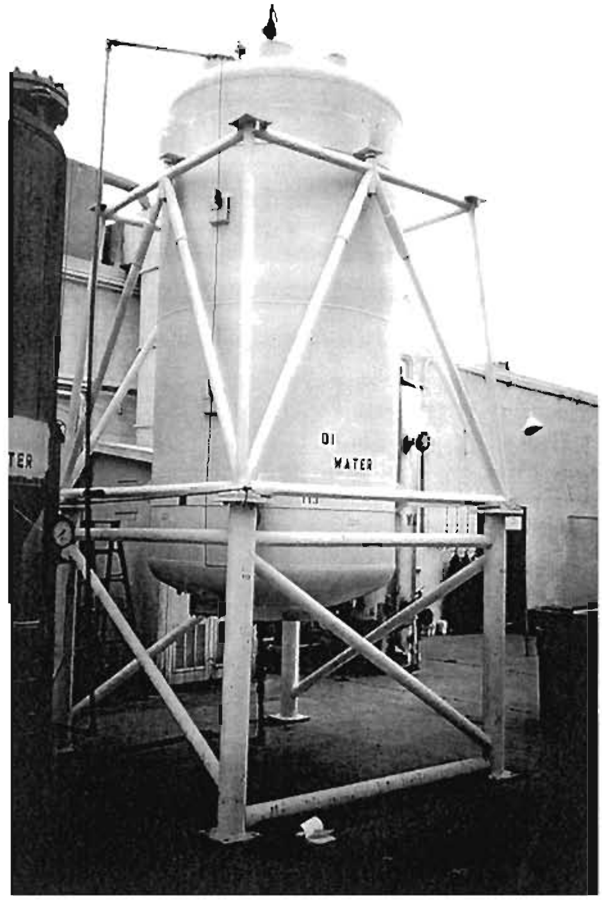


Plate 7-8 : Strained anchors and bulged tank wall to older flat-bottomed tank, Van Nuys

Plate 7-9 : Vertical tank on unanchored support frame moved in translation and rotation, Van Nuys



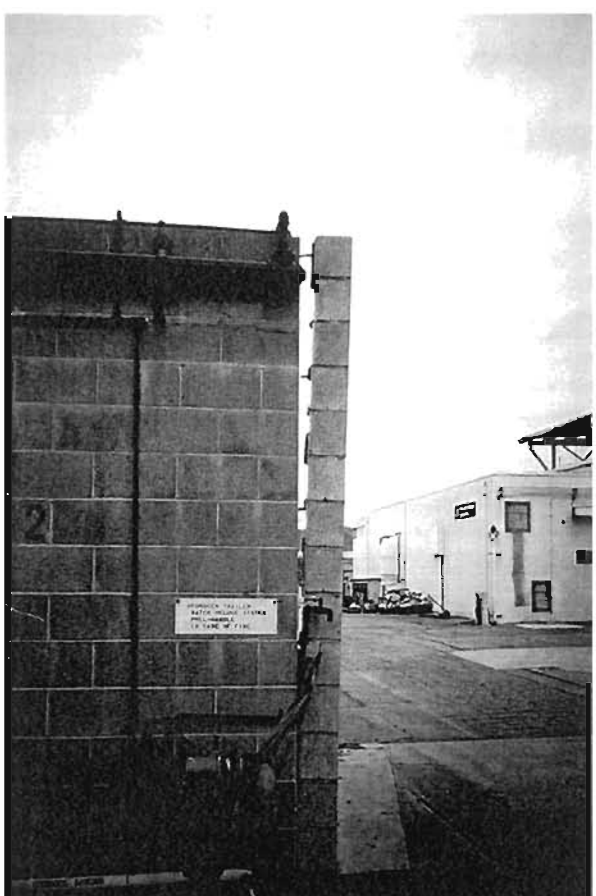


Plate 7-10 : Damage to blast wall due to inadequate continuity, Van Nuys

8. LIFELINES

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8.1 Introduction

The Northridge earthquake is considered to be the most costly natural disaster in the history of the USA. A significant proportion of the losses incurred were due to the direct damage caused to the lifeline networks and the consequential losses from this damage. The performance of a selection of the lifeline networks (electricity, water, gas, telephone etc.) is described in the following sub-sections.

8.2 Electric power

Electrical power in the San Fernando Valley and the major part of the earthquake affected area is primarily supplied by the Los Angeles Department of Water and Power (LADWP). Some counties further from the earthquake epicentre are the service area of Southern California Edison (SCE). The authors are most grateful to Ron Tognazzini and Pat Dang of LADWP and Jeff May and Torrey Yee of SCE who very kindly arranged and accompanied the authors on site visits and furnished commentaries on the performance of their companies' facilities.

The Northridge earthquake caused the first complete blackout in the history of the Los Angeles area. Supplies to two million LADWP customers were interrupted. Within two hours power was restored to approximately one million customers and a further 500,000 by nightfall. By the following day all but 72,500 LADWP customers were able to receive supplies. Total restoration took more than one week. 1.1 million of SCE's 4.2 million customers suffered a break in electricity supplies, mainly in the Thousand Oaks (27km), Ventura (66km), and Santa Barbara (109km) area (distances from Northridge; all lie to the west of Northridge). A quarter of SCE affected customers were re-energised within one minute through the operation of automatic equipment. All but 150,000 SCE customers were still disconnected in the afternoon of the earthquake and all but 800,000 at the end of the next 24 hours.

Many substations suffered significant damage and several transmission towers were damaged, with some tower collapses. There was little damage to generating stations. Although the earthquake affected many of the same facilities as the 1971 San Fernando earthquake, damage was considerably lighter despite the higher acceleration levels. This was mainly due to many improvements made in the intervening years to both seismic qualification requirements and installation practices.

8.2.1 Generation

LADWP operates a number of generation stations in the Los Angeles area, but only the 40 year old Valley generating station in Sun Valley 12km to the east of Northridge, is in the San Fernando Valley. No problems were reported with this steam plant although it went off-line due to relay chatter and then blew steam due to the lack of load to supply. There was no loss of oil pressure, which needs to be maintained due to the very long run down times (up to 1 day) of large steam turbines. Similarly, no significant damage was noted with other LADWP generation stations which are variously steam, gas and combined cycle plants. However one power station had no black start capability so needed external power supplies to be restored to it before it could again generate electricity itself.

Only two of the SCE steam plants operating at the time of the earthquake became unable to deliver electricity to the network. One was again operational within 24 hours whilst minor repairs at the other delayed operation for four days. Gas turbine peak load units were used in substitution of the non-operational steam units.

8.2.2 Distribution

As is usual in major earthquakes the electrical distribution system suffered much disruption. The principal cause was damage in the switchyards of substations in which ceramic insulators and bus-bar connections failures featured predominantly. Perhaps unusually transmission tower failures occurred, which also caused interruptions to electrical supplies.

Rinaldi substation and Sylmar Converter Station were both visited. The Rinaldi substation has an extensive switchyard and small control building. It is situated on gently sloping land immediately downstream of the Van Norman Lakes flood control dam. In the narrow plot between the substation and the dam are large above ground LADWP water supply pipelines. A variety of failures were observed again typically involving ceramic insulators. Insufficient capacity to cater for relative movement caused failures in a variety of bus-bar systems. These varied from relative movements of sections of very long bus-bars supported by chain, to cable suspended radio frequency isolators relatively rigidly connected to switchgear (Plate 8.1, showing reinstated condition using spares) and to too rigid connections between transformers and bus-bars (Plate 8.2). The rocking movement of this transformer and its plinth relative to the ground (Plate 8.3) was no doubt a contributory factor in the latter failure. It was noted that failures of the equipment and the ground motion recordings made implied ground movements rather larger than hitherto predicted for large earthquakes. The presence of large displacements or 'fling' would have implications on the design of equipment seismic isolation systems.

The performance of the support frames (Plates 8.4, 5) to two identical sets of switchgear were pointed out by Mr.Tognazzini. The light framework had been designed and analysed using response spectrum methods whilst the heavier framework had been designed to pass a sine sweep shake table test. Both frames survived intact but in neither case did the electrical equipment that they support survive, somewhat negating the effort employed in ensuring the support systems were resistant to a large earthquake.

A curious circumstance of seismic interaction between different plant occurred at the Rinaldi substation location. The large water supply pipes adjacent to the substation fractured in the earthquake, releasing large quantities of water into the substation site. The water became impounded by the solid perimeter wall and the large solid access gate that protects the site from the adjacent roadway. The access gate, being such a heavy item, is power operated. The loss of electrical supplies due to the direct action of the earthquake and the partial submergence of the drive in water rendered the gate moveable only by manual effort. Fortunately the operators were able to wind the gate open and release the water before it reached the wiring and relays in the switch cabinets in the control building although the building floor and lowest parts of the cabinets had become inundated before this could be achieved. Flooding of the switch cabinets would have delayed the restoration of electrical feeds through the substation.

Sylmar Converter Station in the north eastern corner of the San Fernando Valley (Northridge 11km) was notable in that it featured in the 1971 San Fernando earthquake and is present in equipment seismic experience databases. The Converter Station's function is to convert a two line 500,000 Volt DC connection between out of California State power stations to the LADWP high voltage AC distribution network. The primary items of equipment are the converters, which are each very large pieces of equipment approximately 20m high, and 5m by 3m in plan. These are suspended in enclosed halls from frames at roof level. The frames are on sprung supports with passive lateral restraints. The building in which the converters are housed is of steel construction and includes a control room, offices and building services. Other electrical equipment, mainly housed outside, includes AC/DC filter banks and the more usual switchgear, transformers and bus-bar connections.

Damage at Sylmar Converter Station was mainly to the outside electrical plant. Failure of ceramic insulators was the principal cause of the failures. Failure of the insulators and subsequent toppling of the equipment was observed (Plate 8.6) as well as failure of individual isolators on what appeared to be otherwise intact equipment. In the switchyard it was noted that there seemed to be a greater number of failures amongst ceramics on single concrete support plinths than amongst ceramics that shared plinths with each other.

Of the transformers present one suffered leakage of the cooling oil pipe connections between the cooling fins and main body of the transformer. These fins were braced to each other by flat bars which buckled and were evidently insufficient to carry the loads imposed on them (Plate 8.7). A pair of flat-bottomed tanks were also present with above ground bellows connections to buried pipelines providing protection from relative movement between the tank and ground (Plate 8.8).

Inside the converter building the fire fighting sprinklers were triggered by the earthquake and suspended ceiling tiles were dislodged. This resulted in damage to office equipment and furnishings from the water and the ceiling tile and water mix turning to a porridge-like material.

Overall damage at Sylmar was stated to have been about one third of that in 1971 despite there being 60% more equipment now present at the site.

8.2.3 Control centres

The LADWP control centre for the electricity distribution network experienced 0.25g maximum horizontal ground acceleration with 0.2g vertical acceleration. The centre controls the network for 1200km^2 and the 3.5 million population resident therein. The peak load is 5500MW. Since 1981 the network has been controlled by computer systems. It was said that this has lead to a 'de-skilling' of the operators who now lack the desire to hand control the network if they cannot meter what is happening remotely. Internal LADWP telephones were operating after the earthquake but some of the FM radio links were lost, including the important link between Los Angeles and the San Fernando Valley. The loss of this radio link was due to equipment at a repeater station falling from a table. Control of the network was thus seen as an arduous task for the operators.

The control centre equipment did not all perform as required. The back up generator diesel did not automatically start. Once started it would not pick up load successfully. This was despite weekly 1 hour tests of the equipment. Chiller equipment became faulty and the Halon fire protection system (usual for computer areas) dumped in error.

A feature which worked well at the control centre, and which appeared unusual in construction to the EEFIT members who observed it, was the raised floor system which has strong vertical posts to carry vertical loads and apparently vertical turnbuckles pinned at top and bottom which carry the horizontal loads when the floor moves laterally on the top of the posts. The design load for this system was 1g.

The stand-by lead acid batteries were commented upon by LADWP seismic engineering staff. The batteries were originally floor mounted to provide seismic ruggedness. However on inspection by seismic engineering staff some while prior to the earthquake they were found to be placed on an ordinary table (no leg bracing) for the convenience of staff who occasionally had to top up the battery acid. The batteries were returned to floor level and survived the earthquake without incident.

8.3 San Onofre - Nuclear Power Station (PWR)

A visit was made to the San Onofre Nuclear Power Station, located on the coast approximately 100km south east of Los Angeles in San Diego County. The station comprises 3 P.W.R. units. Unit 1, now shut down was started in 1964 and came on-line four years later. Its rated capacity was 456 MW. Units 2 and 3 are identical with a rated capacity of 1,127MW each. Construction began on these units in 1974 but was not completed until a decade later. The primary containment on units 2 and 3 comprises a 6mm liner plate and a 1m thick reinforced post-tensioned spherical concrete dome. The post-tensioning cables are in both the vertical and horizontal directions, the latter arranged around the periphery and prescribing 270°. The units are structurally isolated but share the same auxiliary building.

The location of the station was carefully chosen in respect of its geological and seismological characteristics. The plant is situated 59, 72 and 96 km from the nearest points on the active Whittier-Elisnore, San Jacinto and San Andreas faults respectively. No evidence of seismic activity over the last 120,000 years was revealed by the geological studies around the site. The design, was based on the possibility of a magnitude 7.0 event occurring 8km offshore. This corresponds to a DBE of 0.67g horizontally and 0.45g vertically. The majority of the plant is Seismic Category 1 which requires elastic performance during the event, the exception being the turbine house, which can deform plastically but must not collapse. Other extreme hazards are the 7.5m high tsunami, winds in excess of 45m/s and rain of 180mm per hour.

There is an extensive monitoring network with a series of accelerometers situated at various locations both in the free field and within the plant. These are automatically monitored and triggered but at various levels typically 0.19g. The safe shutdown has been set at 0.05g but the major earthquakes in the area, Whittier Narrows (1987) and Northridge have not produced accelerations of this magnitude at the site. Therefore, it is not surprising that there was no plant or structural damage.

Since being commissioned, there have been reviews of the safety procedure for PWRs by the National Regulatory Commission in the light of the Three Mile Island accident. These have made it economically effective for the station to provide its own qualifications table for light mechanical and control services equipment. The shake table is a 3-axis ANCO-R7 specification with a maximum load of 0.7 tonnes capable of being accelerated to 3g. The maximum velocity is 1.14m/s with a displacement limit of 100mm.

On large cylindrical tanks, additional external restraints had been provided at the four edges. This ensured that horizontal loads from the tanks would be transferred to the surrounding concrete shear walls, Plate 8.9. Inside the reactors service room columns had been stiffened with additional plates. Reserve energy supplies were held by a large tanker braced to the ground in a designated area (Plate 8.10).

Many aspects of good seismic design were seen; the large heavy restraints to prevent uplift to the crane, Plate 8.11, flexible pipe supports to cater for differential displacement, Plate 8.12, and the extensive use of unistruts or tubes to support cable trays (rods not being used in a nuclear safety environment).

8.4 Buried pipelines

To date seismic damage to buried pipelines has been considered to be of minor importance compared to the damage caused to buildings and other above earth structures. This has been due to three main reasons :

1. The first impressions gained of a severe earthquake are from the structures (above ground) that can be examined instantly. For pipelines, excavations that cost time and money are normally needed.

2. Even if excavations are performed, it is difficult to estimate if a rupture is due to the earthquake or to the earthquake just making a pre-existing malfunction appear.

3. Seismic design codes and regulations do not provide any detailed guidance for buried pipelines as they are mainly focused on the effects of inertia forces and no inertia forces are considered to act on buried structures.

In the following subsections data concerning the performance of the pipeline networks in the Northridge earthquake are reported. Since the majority of the pipelines are embedded such a report cannot be provided unless based on information already gathered by authorised organisations and individuals. In this case Le Val Lund (responsible for the Chapter on Lifelines Performance of the EERI Report on the Northridge earthquake) provided much information and Woody Savage provided details of the performance of the natural gas network.

8.4.1 Water supply network

Four main water supply pipelines were disrupted by the earthquake. These pipelines serve the Santa Clarita and San Fernando Valleys. The pipelines were all of steel and of large diameters (3.0m, 2.15m, 1.95m and 1.4m). Though treatment plants received some damage they kept providing water. The primary cause of interruption of supply to customers was the damage to the distribution system. Preliminary reports indicate over 1,100 leaks in the San Fernando Valley and approximately 300 in the Santa Clarita Valley.

The main cause of damage was the axial forces induced in the pipes due to the permanent relative displacements along the pipeline length. Vibration and pre-existing corrosion also seem to have played a part in the number of section breaks. The repair procedure consists of draining the pipeline, excavation prior to repair (where the lines are below ground level), repair, filling the pipe for testing and chlorination. This procedure has sometimes to be repeated until final restoration of supplies. It is a very time consuming process for large diameter pipes where more time is required for draining and refilling. Until the final restoration of the network, water was provided by contractors familiar with water utility work and the water agencies themselves.

8.4.2 Natural gas network

About 1,000 breaks and leaks in the gas network were reported. Approximately 95% of the leaks occurred in the distribution lines and the balance of 5% occurred in the transmission lines. The distribution system consists of steel and plastic pipes with pressures limited to 4 atmospheres. There was no damage reported on to the plastic pipelines.

People are very sensitive to gas leaks because they cause fires. About 130,000 customers asked for rather unnecessary interruptions of their supplies whilst necessary interruptions amounted to about 20,000 in number. Restoration of supply involves checking of all the gas appliances on a branch of the network before turning the service on. This is very time consuming.

The earthquake caused large ground cracks, landslides and similar phenomena. Even though pipelines may have not been damaged action was needed since damage may occur later. In such cases careful inspection of the whole network has to be performed.

Demolition of damaged bridges required special protection of the adjacent gas pipelines from impact and penetration of debris dislodged or blasted from the bridge structures.

In Balboa Boulevard, after excavation had been performed, two breaks of a 560mm line became visible. One break was due to tensile failure whereas the other was a compressive failure. This is in accordance with recent theoretical approaches yo the vulnerability of buried pipelines subjected to earthquakes. These particular breaks caused a fire in the locality which gutted four homes. At the time of the fire the fire brigade could not use water from the network whilst the adjacent road was flooded (like a river according to eyewitness reports) due to leakage from a mains water pipe.

8.4.3 *Liquid fuel*

One of the three major pipelines which transport crude oil to refineries located in the Los Angeles basin leaked near the Santa Clarita River in the Santa Clarita Valley. The leak polluted the river shore for 19km. The leakage occurred at a cracked weld. Six more welds were also said to have cracked in the pipeline.

8.4.4 *Waste water network*

Sewer lines are normally of the unpressurised gravity flow type. Damage to sewers does not make itself known instantly. It takes time (weeks or even months) before leakage will be spotted. The important parameter is, in this case, the non-stop function of the two existing sewage treatment plants. Actually they lost power for about eight hours but they did not lose their biological system.

8.4.5 *An interesting visit*

An interesting collocation of nine lifelines occurred on Balboa Boulevard in Granada Hills near the junction with Rinaldi Street (about 5km from Northridge, near to a collapse section of the I-118 highway). Evidence of large permanent ground deformations exists in the whole nearby area (sliding of bridges on their supports, cracks on the surface of the road decks, inclination of electricity network columns, etc.). Nine buried pipelines are located there (three gas, three water, two sewerage and one oil pipeline). The deformations of the ground caused leakage to some of the pipelines with extensive flooding while there was no pressure in the water supply network and water could not be used to confront several fires that occurred to homes, due to the gas leakage. Plate 8.13 shows the affected area, the collocation of pipelines and a quick pipe repair to a pipeline.

8.5 Telecommunications

The main switching/transmission equipment performed extremely well. The main cause of line disruption was circuit boards being unseated from their connectors or malfunction of the cards. There were failures of telephone exchange ventilation and air conditioning units caused by power failure and equipment failure. Fortunately the earthquake did not occur in the summer when this kind of failure could have been disastrous.

Overall the telephone network performed very well in the Northridge earthquake. Relatively few problems were directly caused by the ground shaking. However many telephone handsets were dislodged by the shaking which at the telephone exchange gives an indication as though a call is being attempted. This can lead to blocking of outgoing calls as claimed by one hospital administration.

The loss of external electricity supplies tested the telephone network standby power supplies. Typically these are batteries capable of supplying the network for at least 4 hours. In addition, as soon as they can be started standby diesel or gas turbine generators are used to replace the battery supply. Some of the standby generators were able to start but would not pick up the electrical load successfully. The Pacific Bell exchanges now tend to use standby diesels in preference to their gas turbines which are leased to the AT&T network. The higher maintenance costs associated with 1 hour/month running to validate standby usage and fuel costs were reasons for this change, although gas turbines are still well liked for continuous operation.

Telephone exchange buildings survived the earthquake. Some cracks in walls and spalling of plaster occurred but not so much as to render inoperable any building or the equipment within it. Primary exchange equipment for telephonic communication operated well although secondary equipment that services the primary equipment fared less well. The cooling of exchange equipment using chilled HVAC systems was interrupted for some days. At least one instance of pipe rupture was noted as a cause of this. As a short term measure doors were opened to allow outside air to circulate, aided by fans, to provide some cooling and try and keep temperatures below the 50°C short term design limit for the equipment. It was noted that equipment reliability may be reduced as a result.

To route calls, the network is able to use diverse routes both above and below ground level, which allows for some cable/communication link failures. Collapse of a freeway caused isolation of part of the network in the Santa Clarita Valley which had no duplication of cabling or alternative route into the area. In another instance the collapse of a freeway bridge had taken up all the slack on the telephone cables but did not interrupt the use of the cable. One fibre optic cable was severed by a gas main fire.

8.6 Rail

A 64 wagon train happened to be travelling through Northridge at the time the earthquake occurred. 35,000 litres of sulphuric acid that had been carried by 16 tank wagons were spilled and also 8,000 litres of diesel fuel from the locomotive was spilled. The train service was restored 48 hours after the event.

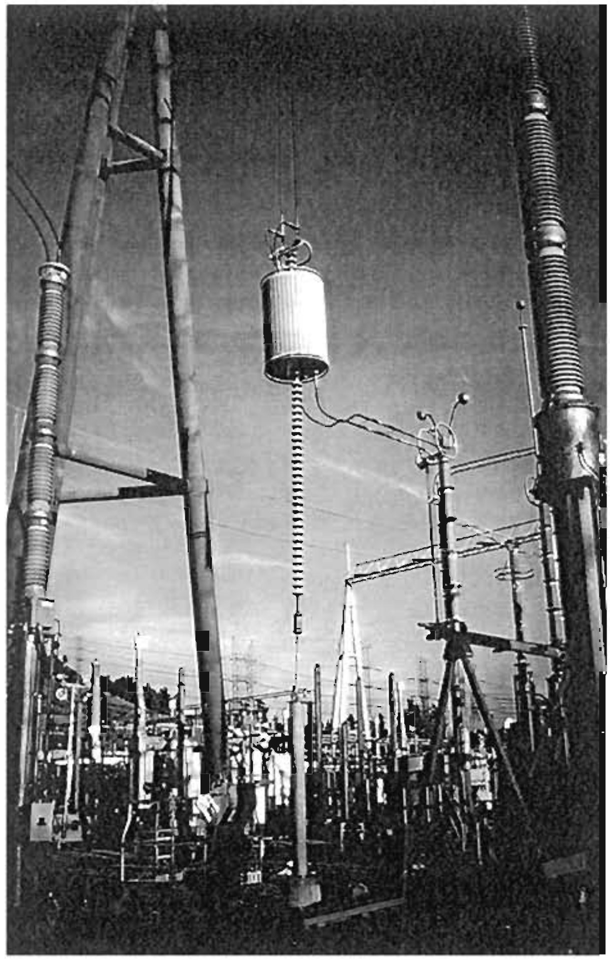

Plate 8-1 : Cable suspended radio frequency isolators with relatively rigid bar connection to switchgear, Rinaldi substation

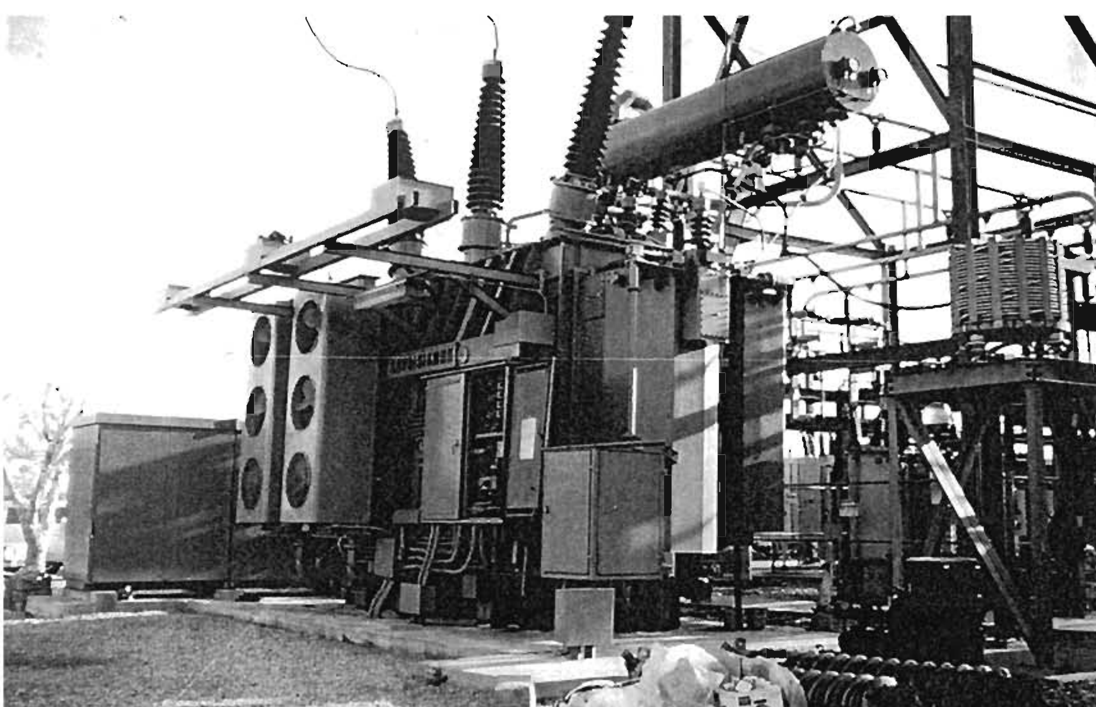

Plate 8-2 : Transformer with rigid bus-bar connections, Rinaldi substation

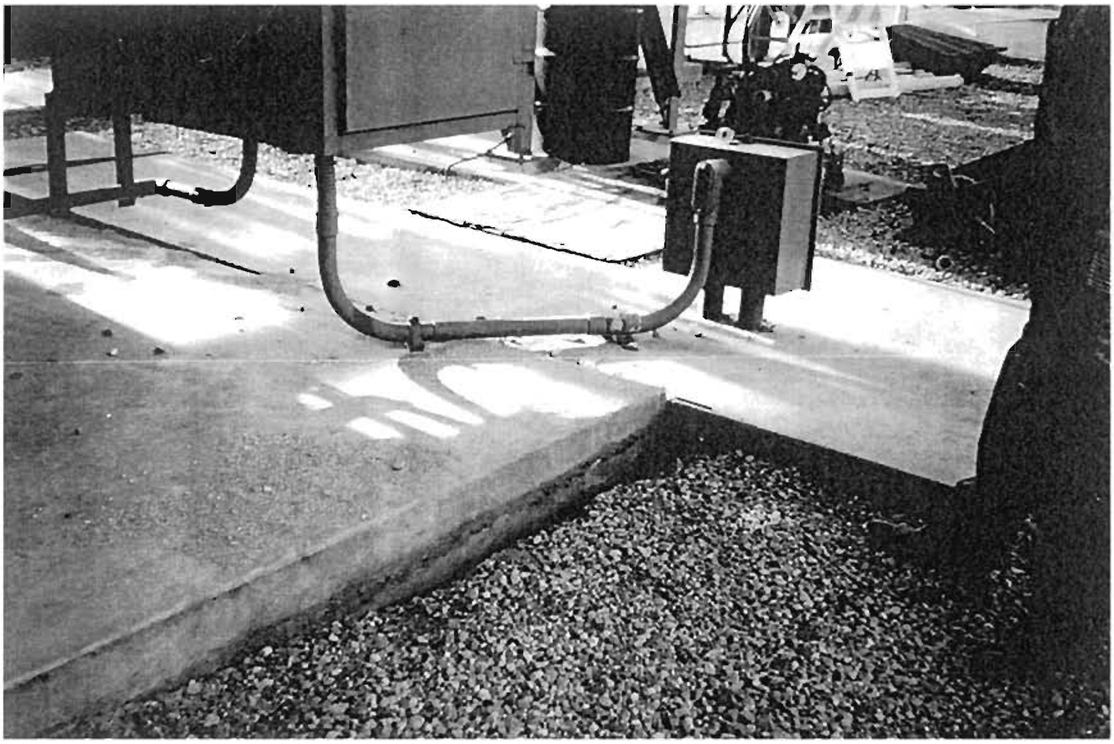

Plate 8-3 : Movement of transformer plinth of Plate 8-2 relative to ground, Rinaldi substation

Plate 8-4 : Switchgear support frame analysed by response spectrum method (*cf* Plate 8-5), Rinaldi substation

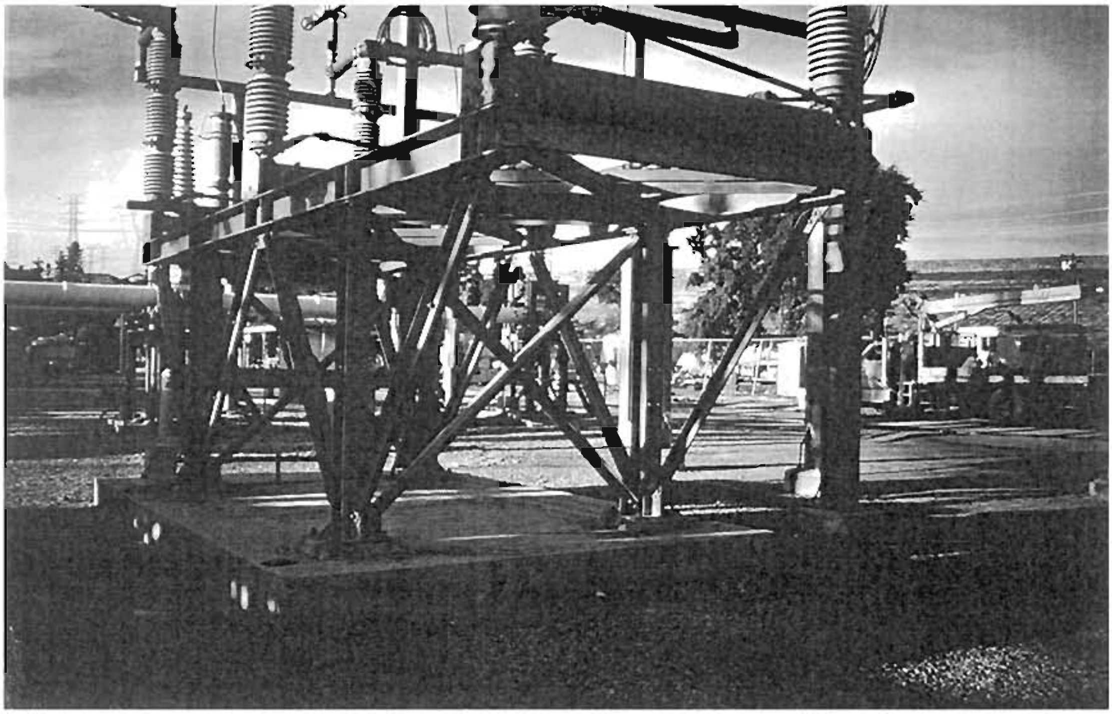

Plate 8-5 : Switchgear support frame design to pass sine sweep test (*cf* Plate 8-4), Rinaldi substation

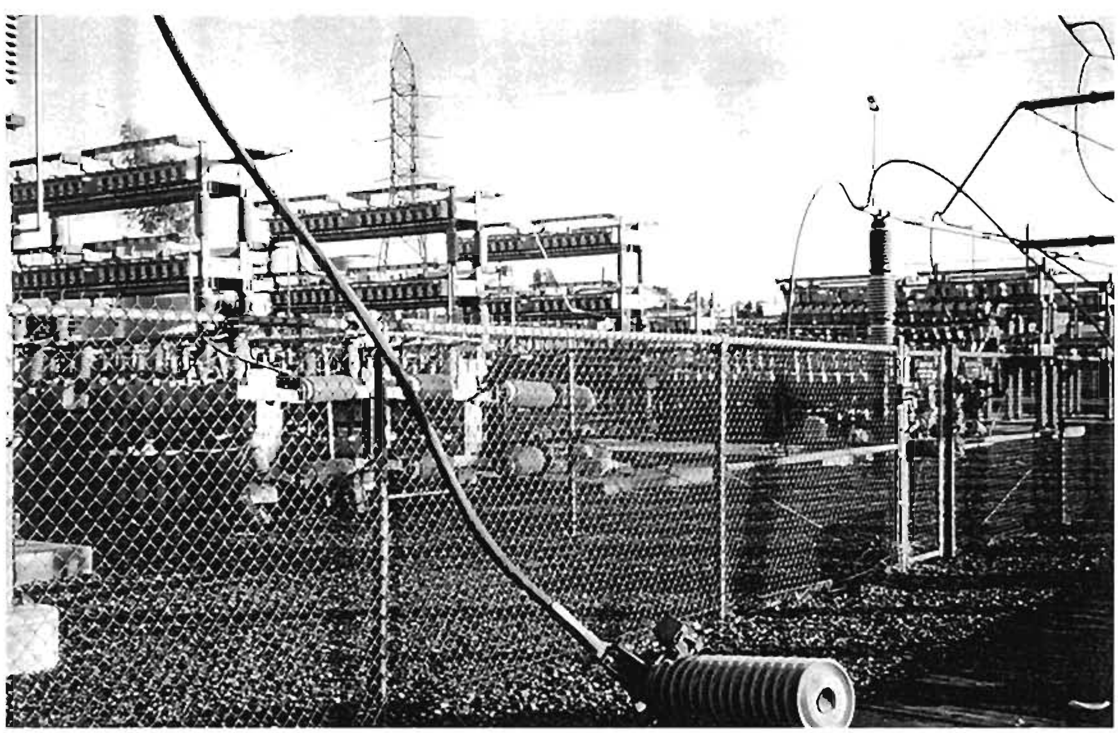

Plate 8-6 : Toppled capacitor filter banks (behind fence), Sylmar converter station

Plate 8-7 : Buckled brace and oil leakage at cooling fins of transformer, Sylmar converter station

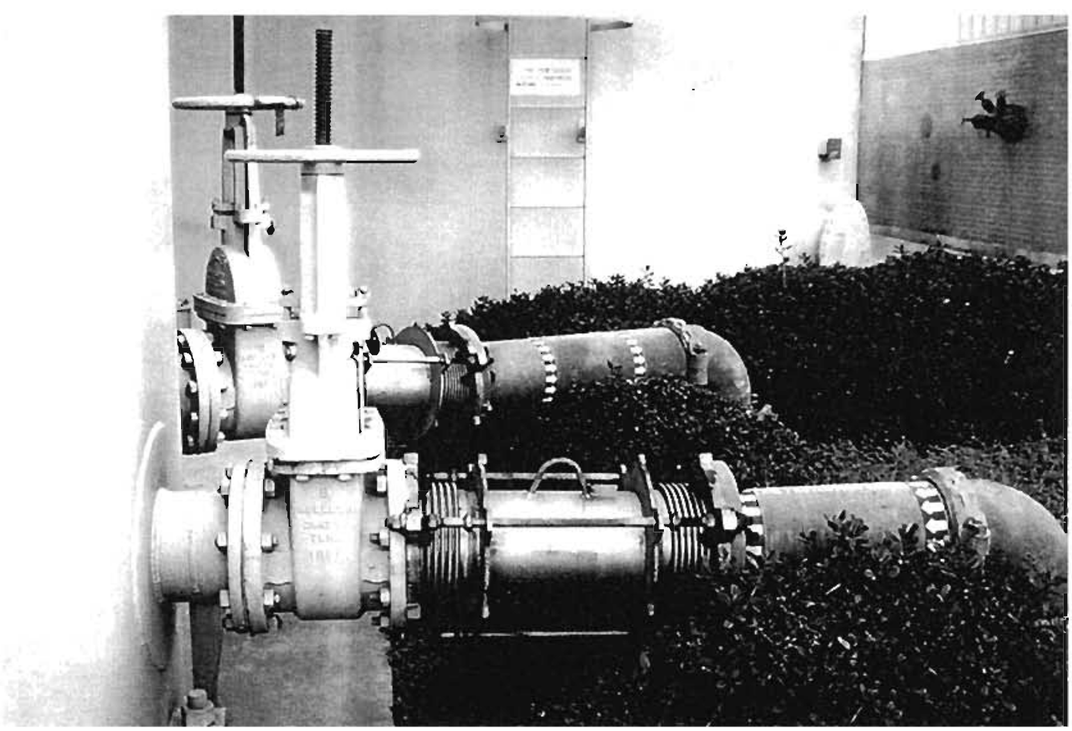

Plate 8-8 : Above ground bellows connections between buried pipes and storage tank, Sylmar converter station

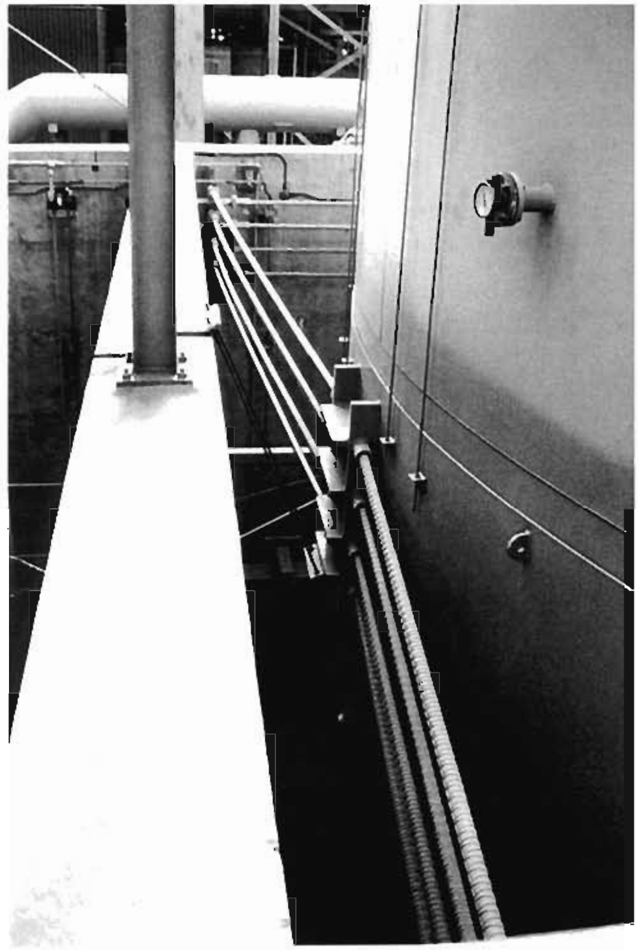

Plate 8-9 : Retrofitted additional external restraints to storage tank, San Onofre NPP

Plate 8-10 : Additional water supply designated for seismic emergency, San Onofre NPP

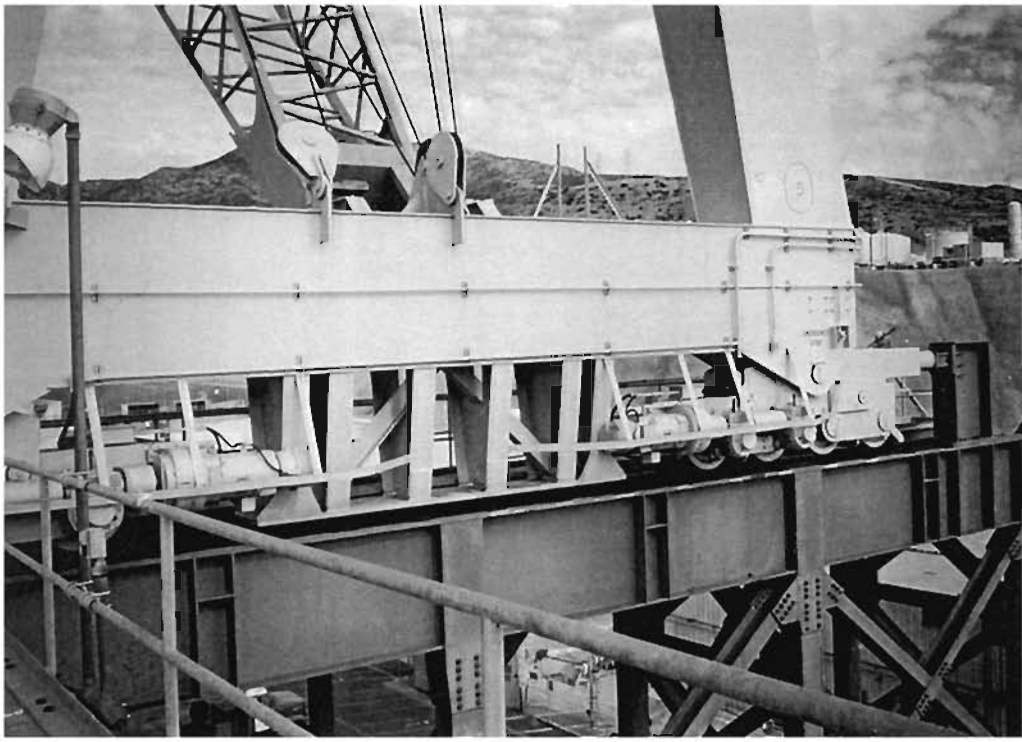

Plate 8-11 : Uplift restraints to large gantry crane, San Onofre NPP

Plate 8-12 : Conduit support, San Onofre NPP

Plate 8-13 : Typical location of gas and water pipe breakage, Granada Hills

9. CONCLUSIONS

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9.1 General

This chapter presents the main findings and conclusions of the report. More detailed conclusions can be found at the end of the relevant chapter.

The earthquake occurred at 4.31 a.m. Pacific Standard Time. Its epicentre was just south-west of Northridge in the San Fernando Valley, approximately 30 km north of central Los Angeles, and the focal depth was approximately 18.5 km. The main shock measured 6.8 on the Richter scale.

The earthquake caused 57 fatalities, around 9,000 confirmed injuries and damage whose total cost is estimated as US$15-30 billion (excluding indirect costs due to loss of function). Given the proximity of the epicentre to densely populated areas and the magnitudes of accelerations recorded, the overall performance of structures and civil engineering works in the affected region was reasonable.

9.2 Seismology and strong motions (Chapter 2)

The earthquake has shown that the geological structure beneath northern Los Angeles is less well understood than had previously been thought. Some doubt persists over the exact geological mechanism which caused the earthquake.

Recorded ground accelerations over a wide area were considerably greater than current UBC design levels. The UBC values are based on the assumption of an earthquake occurring at some distance from a densely populated area; this may be overly optimistic for a city of the size and location of Los Angeles. It may therefore be necessary to revise upwards the code ground accelerations for such areas.

Contrary to some early reports, attenuation effects and vertical/horizontal acceleration ratios were normal. There were some instances of local amplification due to soft soils and topographic effects, and some evidence of a basin effect.

9.3 Geotechnical aspects (Chapter 3)

Geotechnical failures observed at Northridge included massive landslides, sand boils and liquefaction-induced failures. Several structural failures appeared to be due to dynamic soil-structure interaction effects; such effects may prove critical in an earthquake of significantly larger intensity, which is a distinct possibility in Los Angeles.

9.4 Dams (Chapter 4)

Dams in the Los Angeles area performed well in this moderate intensity earthquake, with only two instances of significant cracking. In neither case was the damage sufficient to pose a serious threat of collapse. There were also some cases of damage to intake tower bridges, thought to be due to excessive motion of the towers themselves. The dynamic response of these towers appears to be insufficiently understood, and is an area requiring further research.

9.5 Bridges (Chapter 5)

Bridges generally performed well, with only a small number of older bridges experiencing severe damage or collapse. Nearly all instances of major damage were initiated by non-ductile failure of short, stocky columns, which attracted large loads due to irregularities in the structural layout. In some cases problems were exacerbated by the failure to take adequate account of features such as column flares or restraints from adjoining walls.

Retrofits to bridges also performed well. With very few exceptions, restrainers at expansion joints prevented loss of seating,and no damage occurred in columns retrofitted by jacketting.

9.6 Buildings (Chapter 6)

Base isolation of buildings proved extremely successful, allowing several vital facilities to remain fully operational during and immediately after the earthquake.

Precast concrete car parks performed particularly poorly, due to inadequate connection details between the precast elements and the lack of damping, much of which is provided by cladding in other building types.

The practice of employing ductile moment resistant frames only on the perimeters of buildings proved unsafe, and should be discontinued. Similarly, the use within the same structure of vertical elements, some of which are capable and some incapable of resisting lateral loading, is unsafe.

The earthquake revealed lack of ductility in the behaviour of welded steel beam-column joints in low and medium rise structures. This has prompted a major review of design procedures.

The earthquake again highlighted the damaging effects of irregularity in buildings. Examples included:

- irregularities in plan layout causing significant torsional response (e.g. many hospitals in the affected region);
- frame-shear wall buildings, where the interface between the two requires very careful detailing;
- buildings having blocks of differing heights, where separation should be provided to prevent damage at the interfaces;
- buildings where repairs have caused irregularities in stiffness (e.g. the Holiday Inn Express in Van Nuys).

It may be appropriate to reduce allowable ductilities in the design of critical structures such as hospitals, as it is important that these should be operational immediately after the earthquake.

Damage to masonry buildings mostly took the form of shaking apart or separation of elements, rather than in-plane shear failure. Simple retrofits proved effective in improving the performance of masonry buildings.

9.7 Industrial facilities (Chapter 7)

At the industrial sites visited, there was good seismic performance of the main structures, and only a small amount of equipment damage. Most failures occurred at the interface between structural and mechanical elements, with anchorage failures being particularly common. Equipment generally proved to be robust and readily able to be reinstated for further use.

9.8 Lifelines (Chapter 8)

The earthquake caused severe disruption to the Los Angeles electrical supply system, due mainly to damage to distribution facilities. A particular problem was the failure of brittle elements such as ceramic insulators, whose connections details were frequently too rigid. Unusually, there were also some failures of transmission towers. Generation stations and control centres suffered only minor disruption.

There was widespread damage to buried pipelines, often caused by axial forces set up by permanent ground deformations over the pipeline length. These resulted in considerable disruption to water and gas supplies, though there were few fires.

Telecommunications systems suffered few problems, with most failures related to loss of power supply rather than to damage of the telecommunications facilities themselves.