THE LOMA PRIETA, CALIFORNIA, EARTHQUAKE OF OCTOBER 17, 1989

Report of the NZNSEE Reconnaissance Team

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SUMMARY

This report on the Loma Prieta earthquake of October 1989 describes the earthquake itself, its effects on the surrounding region, and San Francisco in particular, and reports on the resulting damage along with the effect on local communities.

TABLE OF CONTENTS

1. INTRODUCTION	7.2 Modern High-rise Buildings
2. GEOLOGICAL - SEISMOTECTONIC SETTING	(a) Overview
3. THE LOMA PRIETA EARTHQUAKE SEQUENCE	(b) 101 California
4. GROUND MOTION RECORDS	(c) Marriott Hotel
4.1 Introduction	(d) Kaiser Plaza Building
4.2 Attenuation	(e) AT & T Exchange, Oakland
4.3 Correlation with Geology	(f) Albany Hill Condominium Block
4.4 Near-Field Records	7.3 Older Commercial Buildings
4.5 Felt Intensity	(a) Overview
5. GROUND FAILURE	(b) Southern Pacific Company
5.1 Ground types	(c) Emporium Capwell, Oakland
5.2 Cracking	(d) Granklin/21st Streets
5.3 Landslides	(e) I Magnin & Co
5.4 Liquefaction	(f) Apartment Block in Oakland
5.5 The Marina District	(g) Impact Damage
6. SOIL STRUCTURES	7.4 Modern Low-rise Commercial Buildings
6.1 Artificial Fill	(a) Overview
6.2 Earth Dams	(b) AT & T Offices
7. BUILDINGS	(c) Pacific Coast Savings & Loan
7.0 Building Inspections	(d) 150 Fourth Street
7.1 Damage Patterns	(e) San Francisco Airport
(a) Soil Type	(f) Office Buildings, San Mateo
(b) Structural Types	(g) Graduate Business School, Stanford University
(c) Building Configuration	(h) Oakland Coliseum Sports Centre
(d) Summary	7.5 Historic Buildings
	(a) Overview
	(b) City Hall, San Francisco
	(c) City Library
	(d) Churches

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- 7.6 Unreinforced Masonry Buildings
 - (a) Overview
 - (b) Epicentral Area
 - (c) Stanford University
 - (d) South of Market and Oakland
- 7.7 Strengthened Buildings
- 7.8 Timber Structures
 - (a) Overview
 - (b) Houses, Epicentral Area
 - (c) Residential Buildings, Marina
 - (d) Madison Park Apartments, Oakland
- 7.9 Schools
- 7.10 Non-Structural Elements
 - (a) General
 - (b) Windows
 - (c) Claddings
 - (d) Partitions
 - (e) Ceilings
 - (f) Contents and Equipment
 - (g) Building Services
- 8. BRIDGES
 - 8.1 Introduction
 - 8.2 General Performance
 - 8.3 I-280 Extension
 - 8.4 I-280 Army Street
 - 8.5 I-880 Cypress Viaduct
 - 8.6 I-480 Embarcadero Viaduct
 - 8.7 Mococo Ramp
 - 8.8 Struve Slough Bridges
 - 8.9 Madrone Drive Overhead
 - 8.10 Grand Avenue On Ramps
 - 8.11 San Francisco-Oakland Bay Bridge
 - 8.12 San Mateo-Heyward Bridge
 - 8.13 Carquinez Bridge
 - 8.14 Conclusions
- 9. LIFELINE SYSTEMS
 - 9.1 General
 - 9.2 Rooding
 - 9.3 Rail
 - 9.4 Electricity
 - 9.5 Gas
 - 9.6 Water
 - 9.7 Drainage
 - 9.8 Airports
 - 9.9 Sea
 - 9.10 Communications
- 10. THE RESPONSE
 - 10.1 Introduction
 - 10.2 People's Immediate Response
 - 10.3 Organisational Response
 - (a) Rescue
 - (b) Fire Suppression
 - (c) Law and Order
 - (d) Re-establishing Lifelines
 - (e) Building Inspections/Classifications
 - 10.4 Lessons-Recommendations
- 11. CONCLUSIONS/RECOMMENDATIONS
 - 11.1 Points Made
 - 11.2 The N.Z.N.S.E.E. Response
 - 11.3 Conclusion
- 12. ACKNOWLEDGEMENTS
- 13. REFERENCES

1. INTRODUCTION

In late September 1989, a New Zealand National Society for Earthquake Engineering five-member reconnaissance team travelled to the Union of Soviet Socialist Republics to visit the site of the devastating December 7 1988 earthquake at Spitak, Armenia. On the return journey, the team stopped in London to have discussions on shared findings in Armenia and future cooperative earthquake reconnaissance with members of the European Earthquake Field Investigation Team organisation, and again in San Francisco for similar discussions with the Earthquake Engineering Research Institute (EERI). This meeting in San Francisco took place on Monday 16 October. The following day team members followed their own interests, some visiting the University of California, Berkeley and the Earthquake Engineering Research Centre at Richmond, some making contact with the United States Geological Survey, and shopping.

Thus when the Loma Prieta earthquake struck at 5.04 pm on Tuesday 17 October, the team were scattered around various parts of San Francisco City.

Each team member had a different experience. One in a basement bookshop felt a small earthquake, noticed some books fall, went on to complete the purchase but couldn't do so because the electricity power supply stopped. Nearby, another team member was in the ground floor of a parking building, felt dizzy, realised it was an earthquake movement, panicked, and then sought a safe place of refuge and observed reaction. These two members coincidentally met shortly afterward and together walked through the city observing damage and volunteered and assisted San Francisco City officials with initial building inspections late into the night. Another member was walking up Powell Street (Cable-car fame), felt jolting, noticed a reaction among people in the street, observed puffs of dust from between pounding buildings, heard glass breaking, and found himself in the centre of the street. The team "accountant" was on his knees counting money on his hotel bed when he heard the sounds, felt the motion of the building and sought refuge in the doorway. The seismologist member of the team, shopping on the ground floor of a six-storey building was walking about, felt the P wave motion and then the S waves, carried out an instant assessment of the earthquake and observed the deformations in the building, finally egressing into the street clouded with dust from the pounding buildings along Market Street.

The earthquake motion experienced was very distinct but did not cause difficulty in standing. It seemed to be of about 15 seconds in duration. The reaction of people ranged from concern, suppressed excitement, some loudly expressed emotion, apparent distress, but not panic. Many seemed to congregate in open spaces and stand around. The lack of electricity in the central city had a most pronounced effect on activity, particularly on transport and later in simply leaving the

city in darkness.

There was a complete, prepared, reconnaissance team right on the spot, safe and unharmed. Most of the city could function in some restricted form; in some areas there was incredible disaster and devastation, and there was sufficient effect to be of significant technical interest.

The team comprised Bruce Shephard, building structural engineer, Peter North, civil and bridge engineer, Neil Gillon, lifelines engineer, John Berrill, geotechnical engineer and engineering seismologist, and Peter Wood, geophysicist. The team was joined by Alan Perry, architect, and kept close contact with a New Zealand Civil Defence team (Doug Bent, Brian Toomey and Margaret Laird) and DSIR physicist, Bill Stephenson with interest in soft ground earthquake amplification effects, who all travelled to San Francisco shortly following the earthquake.

In the days following the earthquake, team members individually and collectively followed lines of reconnaissance interest, joining American counterparts in inspections and evaluations of bridges, building, lifelines and ground damage as part of groups and with counterpart contacts established with EERI, the United States Geological Survey and several local authorities.

Being pre-conditioned for earthquake reconnaissance and having no local business commitments was of great benefit in undertaking wide ranging inspections and obtaining good overview appreciation.

The following reports the observations and data obtained by the team from their reconnaissance in the week following the earthquake and includes background from various sources. It was not possible for the team to stay longer and undertake more detailed investigations having already been away from New Zealand some 4 weeks. It is realised that some content of this report may be superseded as more detailed technical evaluations are published.

2. GEOLOGICAL - SEISMOTECTONIC SETTING

The Loma Prieta earthquake takes its name from the highest elevation (1157m) in the Santa Cruz Mountains, which lies within the epicentral region. The Santa Cruz Mountains form part of California's Coastal Range province, to the south of the base of the San Francisco peninsula, and north-east of Monterey Bay. Loma Prieta is approximately 100km south-east of San Francisco City¹ (Fig. 1).

The San Andreas Fault Zone runs northwest - southeast through the Santa Cruz Mountains. The fault zone is part of

the diffuse transform fault system that separates the Pacific tectonic plate from the North American plate. Relative movement between the two plates, estimated presently to be 56mm/yr, has juxtaposed differing basement rock types on either side of the San Andreas fault. To the east the exposed metasedimentary rocks of the Franciscan Complex are probably underlain at a depth of about 5km by associated mafic volcanics. To the west, granitic rocks are exposed in the mountains north of Santa Cruz, but the intervening folded Tertiary strata may be underlain at a depth of 5 - 10km by mafic crystalline rock (Bailey, 1966; Staff USGS, in press).

The basement rocks of the region are overlain (according to Lajoie and Helley, 1975) by late Pleistocene alluvium (10,000 - 70,000 years old), Holocene alluvium (0 - 5,000 years old), and Holocene estuarine deposits (0 - 9,000 years old). The late Pleistocene alluvium is in the main slightly consolidated and indurated alluvial fan deposits consisting primarily of gravel and sand with some silt; maximum thickness is at least 45 m. The Holocene deposits are all poorly consolidated. They have been differentiated as:-

Bay Mud - water saturated estuarine mud, predominately clay and silty clay underlying marshlands and tidal mudflats of San Francisco Bay, occasional lenses of well sorted fine sand and silt, occasional shelly and peaty layers, 0 - 40m thick.

Fine grained alluvium - plastic, poorly sorted carbonaceous clay and silty clay, interfingers with Bay Mud and coarse grained alluvium, generally less than 5m thick;

Medium grained alluvium - loose, moderately drained, moderately sorted sands generally less than 5m thick;

Coarse grained alluvium - loose, well drained, moderately sorted, permeable sand and gravel, up to 15m thick.

The distribution of the poorly consolidated deposits has a significant relationship to the distribution of damage due to the Loma Prieta earthquake. Such a correlation had already been noted for damage from the 1906 San Francisco earthquake.

California is characterised by high seismicity, on a world wide scale. This is because of the location of the State on the boundary between the Pacific and North American plates. The relative movement of the two plates causes accumulation of strain in the crustal rocks. Strain release is in the main by earthquake and associated fault slip. A close correlation exists in California between shallow historic seismicity and mapped surface

¹ Loma Prieta has been associated since 1884 with geodetic measurements of crustal deformation, including: the measurement of slip in the 1906 San Francisco earthquake; the development of the elastic rebound theory by H.F. Reid (1908); and currently, monthly geodetic measurements, which, although not showing a significant precursory signal for the Loma Prieta earthquake, enabled modelling of the coseismic dislocation.

FIGURE 1 LOCATION OF THE LOMA PRIETA EARTHQUAKE, CALIFORNIA.

LOMA PRIETA MAINSHOCK and AFTERSHOCKS: Oct 18-30 1989

FIGURE 2 DISTRIBUTION OF AFTERSHOCKS OF THE LOMA PRIETA EARTHQUAKE (FROM PLAFKER AND GALLOWAY, 1989).

faulting, in contrast to New Zealand which has a similar, but not identical, tectonic setting (Scholz 1977, Yeats and Berryman, 1987).

3. THE LOMA PRIETA EARTHQUAKE SEQUENCE

The Loma Prieta earthquake sequence commenced at 17hrs 04min 15 secs, 17 October 1989, Pacific Daylight time, with an epicentre location of 37°02' N, 121°53' W. The crustal rocks ruptured from a depth of about 18km. During the next 7 to 10 seconds the rupture propagated upwards to within 4km of the ground surface and to the northwest and to the southeast for some 20km, ultimately involving slip on some 300 square kilometres of buried fault surface area. The magnitude of the earthquake has been calculated to be 7.1, from surface wave observations from around the world (Plafker and Galloway, 1989).

The main and aftershock locations define the rupture surface (Fig. 2) which dips some 70° to the SW. The right lateral reverse oblique slip caused geodetically detectable horizontal and vertical displacement of the ground surface (Fig 3). The rupture surface is believed to coincide with the San Andreas fault, rerupturing the southernmost segment of the fault break of the San Francisco earthquake of 1906 (Plafker and Galloway, 1989).

Aftershocks included two events near M5 within 48 hours, all others were of smaller magnitude. The number of aftershocks greater than M3 decreased with time, closely following a pattern typical for a Californian sequence (Plafker and Galloway, 1989; Fig 4).

The San Andreas fault has a creeping segment, between San Juan Bautista and Parkfield, that produces numerous small earthquakes. Immediately north of this segment the fault has been relatively

FIGURE 3 SCHEMATIC DIAGRAM SHOWING INFERRED MOTION ON THE SAN ANDREAS FAULT DURING THE LOMA PRIETA EARTHQUAKE (AFTER PLAFKER AND GALLOWAY, 1989).

FIGURE 4 TIME DISTRIBUTION (LOGARITHMIC) OF AFTERSHOCKS (DOTS) OF MAGNITUDE (M) 3 OR GREATER OF THE LOMA PRIETA EARTHQUAKE. CURVE IS OF EXPECTED CALIFORNIA AFTERSHOCK TIME BEHAVIOUR, AND SHOWS CLOSE AGREEMENT (FROM PLAFKER AND GALLOWAY, 1989).

aseismic since 1906. What little seismicity that has occurred for the last 20 years outlined an aseismic gap between San Juan Bautista and Portola Valley that was filled by the Loma Prieta earthquake sequence (Plafker and Galloway, 1989).

4. GROUND MOTION RECORDS

4.1 Introduction

California is well covered by strong-motion instruments; the principal accelerograph networks are maintained by the US Geological Survey (USGS) and by the State of California Strong Motion Instrumentation Program (CSMIP). In the Loma Prieta earthquake, 38 USGS stations and over 90 CSMIP instruments were triggered. Figure 5 shows the instrument sites and ground-level peak acceleration values that were available at the end of November, 1989, taken from the reports of Maley et al. (1989) and Shakal et al. (1989). At the time of writing, no spectra were available.

The peak acceleration data exhibit considerable scatter, and show stronger motions to the northwest than in other directions at similar distances. This may be due to focusing of energy to the northwest, as well as to amplification by the soft sediments along the Bay margin. The greater concentration of accelerographs in that direction also increases the chance of recording extreme values.

The wide range of site conditions and the number of records obtained close to the source are particularly valuable features of this set of accelerograms. Apart from the accelerogram from the $M = 7.3$ Tabas, Iran, earthquake of 1978 and the 1976 Gazli, USSR record, they are the only near-field recordings from a non-subduction earthquake with magnitude greater than 7.

4.2 Attenuation

Figure 6 shows horizontal peak acceleration values plotted against distance. The bulk of the data lie within 30 to 100 km of the epicentre, and over this band the pattern of scatter is fairly uniform, indicating that in a statistical sense the data set is reasonably complete. At any fixed distance within this range we notice that the values are scattered over about one order of magnitude. Some of the scatter may be related to site soil conditions, but directivity and transmission path effects must also be important.

The attenuation expression of Joyner and Boore (1981) is plotted in Figure 6. We note that it predicts the near-field peaks very well, but that it strongly underestimates the distant values.

4.3 Correlation with Geology

The influence of site geology on the strength of shaking was pronounced in the Loma Prieta earthquake, with sites on fill and on Bay Mud (an unconsolidated clayey silt, described in section 2) often recording two to three times the peak accelerations of rock sites at similar

distances. This is well-illustrated in Figure 7 by the cluster of data with epicentral distances of 90 to 100 km. All but 5 of the 34 stations in this range are in the San Francisco city and Oakland areas, and thus fairly close together. Simple descriptions of site soil conditions are given by Shakal et al. (1989) for 20 of the 34 sites; peak accelerations from the 20 sites are shown in Figure 8, as a function of soil condition. The average horizontal peak acceleration from rock sites is 0.08 g, while the three sites with fill over Bay Mud averaged about 0.17 g and the two Bay Mud sites recorded 0.26 and 0.29 g. Clearly, the softer the site, the stronger the peak acceleration. It will be interesting to compare spectra from these stations to see how the site amplification varies with frequency.

Another interesting set of data for which detailed site data is available was recorded at 7 sites in the APEEL array, whose layout and site geology is shown in Figure 9 and Table 1. The sites lie within 44 to 54 km from the rupture, and their peak accelerations, plotted in Figure 10, are well correlated with the softness of site materials. Because of the large size of San Francisco Bay, the Bay Muds are in the main softer than one would normally find in an estuary; hence their capacity to amplify seismic waves is also greater than that of the coarser and less-gently deposited sediments which one would expect to find deposited in a smaller body of water.

4.4 Near-Field Records

Twenty five strong-motion stations recorded accelerograms within 40 km, about one source dimension, of the epicentre. Six instruments recorded peak accelerations of greater than 0.5g. Five of the six were within 29 km of the epicentre and within 15 km of the rupture surface inferred from the aftershock zone. The strongest horizontal peak acceleration of 0.64g came from the Corralitos station, 1 km from the fault and close to the epicentre. A selection of the strongest near-field records is presented in Figure 11 and Table 2.

The larger horizontal peak acceleration values for sites out to 30 km from the rupture surface are plotted in Figure 12, together with the attenuation model of Joyner and Boore (1981). The Joyner and Boore model predicts the close-in peaks well, but it underestimates the data at distances greater than 10 to 15 km. For these records, an explanation cannot be found in exceptionally soft site conditions. For example, the two high peaks at 15 km and 18 km are from stations 1 and 3 of the USGS Gilroy array. While station 3 is founded on Holocene interfluvial basin deposits, station 1 is on sandstone with a shear wave velocity of nearly 800 m/s (Fumal et al., 1987).

Also noteworthy are the strong vertical peak values of 0.66g and 0.60g recorded at Capitola and Watsonville respectively, both to the west of the fault on the upthrow side.

FIGURE 5 LARGER HORIZONTAL PEAK ACCELERATIONS RECORDED BY USGS AND CSMIP NETWORKS.

FIGURE 6 PEAK ACCELERATION PLOTTED AGAINST DISTANCE. THE LARGER HORIZONTAL COMPONENT FROM EACH RECORD IS SHOWN. THE SOLID LINE SHOWS JOYNER AND BOORE'S ATTENUATION EXPRESSION.

FIGURE 7 PEAK ACCELERATIONS RECORDED IN SAN FRANCISCO AND OAKLAND, TOGETHER WITH SUPERFICIAL GEOLOGY.

FIGURE 8 PEAK ACCELERATIONS AT DISTANCES OF 90 TO 100 KM SHOWING RELATIONSHIP WITH SITE GEOLOGY.

FIGURE 9 LOCATION AND SITE GEOLOGY AT APEEL ARRAY. (FROM HANSEN ET AL., 1973).

FIGURE 10 PEAK ACCELERATIONS RECORDED AT THE APEEL ARRAY SITES. NOTE THE CORRELATION OF SHAKING WITH SITE GEOLOGY.

TABLE 1 - APEEL ARRAY ACCELERATION AND SITE GEOLOGICAL DATA

APEEL Site No	Name	No.	Distance to Epicentre (km)	Rupture (km)	Acceleration Direction (degrees)	Peak (g)	Site Geology (Hansen et al. 1973)
1	Foster City Redwood Shores	375	63	44	90 up 360	0.29 0.11 0.26	20m Bay Mud over 220m Quaternary alluvium over sandstone
2	Redwood City	002	63		133 up 43	0.23 0.08 0.28	8m Bay Mud over 100m of Quaternary alluvium
7	Upper Crystal Springs Reservoir Pulgas Water Temple	378	63	44	90 up 360	0.09 0.06 0.16	Tertiary Sandstone
9	Crystal Springs Reservoir	161	62		227 up 137	0.11 0.06 0.12	Sandstone clasts
10	Upper Crystal Springs Reservoir Skyline Blvd		63	44	90 up 360	0.09 0.04 0.10	Sandstone
2E	Hayward, Muir School	121	71	53	54 up 324	0.13 0.06 0.16	Quaternary stream alluvium and slope wash
		393			90 up 360	0.14 0.10 0.18	
3E	Hayward CSUH Stadium	219	71	53	90 up 360	0.08 0.05 0.08	Quaternary rhyolite

Corralitos - Eureka Canyon Rd.
(CSMIP Station 57007)

Record 57007-S4809-89292.01

Capitola - Fire Station
(CSMIP Station 47125)

Record 47125-S1679-89291.04

Santa Cruz - UCSC/Lick Lab.
(CSMIP Station 58135)

Record 58135-S1682-89292.04

Saratoga - Aloha Ave.
(CSMIP Station 58065)

FIGURE 11 NEAR FIELD ACCELEROGRAMS (FROM SHAKEL ET AL., 1989).

FIGURE 12 PEAK ACCELERATIONS RECORDED WITHIN 30 KM OF THE RUPTURE SURFACE. LARGER HORIZONTAL COMPONENT SHOWN, TOGETHER WITH JOYNER & BOORE'S ATTENUATION EXPRESSION.

TABLE 2 - NEAR-FIELD ACCELEROGRAMS

No.	Station Name	Distance to		Max Acceleration	
		Epiceentre km	Rupture km	Horizontal g	Vertical g
57007	Corralitos Eureka Canyon Rd	7	1	0.64	0.47
57180	Lexington Dam	19	5	0.45	0.15
47459	Watsonville	18	7	0.39	0.66
58235	Saratoga 1-storey Gymnasium	27	9	0.33	0.27
58065	Saratoga Aloha Ave	27	9	0.53	0.41
47125	Capitola Fire Station	9	9	0.54	0.6
57562	San Jose 3-storey office building	21	10	0.2	0.14
57563	San Jose Santa Teresa Hills	21	10	0.28	0.22
47379	Gilroy #1 Gavilan College	29	15	0.5	0.22
47006	Gilroy Gavilan College	29	15	0.37	0.2
57476	Gilroy 2-storey Historic Comm.	28	16	0.28	0.15
47380	Gilroy #2 Hwy 101/Bolsa Road	30	16	0.37	0.31
58135	Santa Cruz UCSC/Lick Obs	16	16	0.47	0.4
47381	Gilroy #3 Gilroy Sewage Plant	31	18	0.55	0.38
47315	San Juan Bautista Hwy 101/156	33	19	0.15	0.1
1652	Anderson Dam	27	20	0.08	0.05
57355	San Jose 10-storey com. bldg.	33	20	0.11	0.09
57356	San Jose 10-storey res. bldg.	33	20	0.13	0.09
57382	Gilroy #4 San Ysidro School	32	20	0.42	0.17
57357	San Jose 13-storey Govt. Office	35	21	0.11	0.1
57217	Coyote Lake Dam Abutment	31	21	0.49	0.08
57504	Coyote Lake Dam Downstream	31	21	0.19	0.1
1571	San Jose Interchange	34	23	0.18	0.08
57383	Gilroy #6 San Ysidro	35	24	0.17	0.1
57066	Agnew, Agnews State Hospital	40	25	0.17	0.1
57425	Gilroy #7 Mantelli Ranch	40	28	0.33	0.12
57191	Halls Valley Grant Park	37	31	0.13	0.06

FIGURE 13 PRELIMINARY MAP SHOWING DISTRIBUTION OF MODIFIED MERCALLI INTENSITY FOR THE LOMA PRIETA EARTHQUAKE. ROMAN NUMERALS REPRESENT INTENSITY LEVEL BETWEEN ISOSEISMAL LINES WHILE NUMERALS INDICATE OBSERVED INTENSITY VALUES.

4.5 Felt Intensity

Preliminary assessment of the Modified Mercalli intensities by Plafker and Galloway (1989) are shown in Fig. 13. The isoseismal contours clearly show the focussing of energy to the north-west and south-east, in the directions of rupture propagation. Another striking feature is the assessment of MMIX at Oakland and the Marina District, about 100 km from the epicentre while the highest value assigned to the epicentral area is MMVIII. However, Plafker and Galloway acknowledge shortcomings of the MM scale and emphasise the difficulty in making these assessments which are based on the failure of buildings in the Marina area and of the Nimitz Freeway structure in Oakland. Since the Marina failures occurred in structures with low shearing resistance in the bottom floor caused by the presence of garage doors and the Nimitz Freeway failure had its origin in inadequate joint shear capacity, it is doubtful that these structures meet the criteria for the assignment of a MMIX intensity.

5. GROUND FAILURE

5.1 Ground types

Failed ground occurred in exposures of deeply weathered or fractured basement rock, exposed in steep natural and artificial cuts; the late Pleistocene alluvium; Holocene alluvium; Holocene estuarine deposits - Bay Mud; and some, generally older, artificial fills.

5.2 Ground Cracking

Ground cracking was associated with almost all failed ground. Extensional, compressional and shear cracking occurred, all with accompanying deformation.

Extensive mapping of ground cracking was undertaken in the epicentral region on difficult mountainous and vegetated topography over the San Andreas, Sargent, and Zyante faults. The more distant Haywards and Calaveras faults were also checked but showed only rare occurrences of minor shear cracks, (notably at Anderson Lake and East of Gilroy). No fault rupture of the ground surface was recognised.

The most extensive area of cracking occurred within the epicentral region and within the previously mapped zone of the San Andreas Fault, (Sarna-Wojcicki et al. 1975) about Summit Road, southeast of Highway 17, and Skyland Ridge, south of the old San Jose Road. The cracks show dominantly extension and left lateral shear, with a trend oblique to that of the San Andreas fault (Figs 14-17).

An explanation of these cracks is that they originated in response to the extension and shear deformation occurring above the buried fault rupture (see Fig. 3), combined with lateral spreading of the ridge tops during strong ground motions, and with some structural control from the bedding of underlying tertiary sedimentary rocks (Plafker and Galloway 1989).

Many of these cracks appeared to have been reactivated, rather than created, by the Loma Prieta earthquake.

Compressional cracking, with associated damage to structures, occurred in discontinuous zones along the northeastern margin of the Santa Cruz mountains, Los Gatos to Palo Alto. It has been suggested that these compressional zones may be related to small secondary movement on one or more of the southwest dipping thrust faults, such as the Berrocal Fault that parallel the Santa Cruz Mountain front (Plafker and Galloway 1989).

Ubiquitous cracking occurred in areas of ground failure, associated with mass movement, and liquefaction.

5.3 Landslides

The strong ground motions of the Loma Prieta earthquake triggered thousands of landslides at distances up to 130 km from the epicentre. They were most numerous in the epicentral region in the steep, rugged and heavily vegetated Santa Cruz mountains (Fig. 14) where they have occurred historically due to earthquakes and heavy rain. The triggering of landslides by seismic shaking is very dependent on ground water conditions. The Loma Prieta earthquake occurred after two dry years and after only about 50 mm of rain had fallen in the preceding five months.

Rock falls, and shallow rock and soil slides were the most common form of landsliding. Reported volumes were mostly less than 100 cubic metres, several with volumes in the range of 100-10,000 cubic metres and at least one with a volume of more than 40,000 cubic metres. Such landslides are reported to have killed two people (Plafker and Galloway, 1989).

Deeper seated rotational slumps and translational block slides detached in hundreds of places following the earthquake, but typically moved less than a metre. They buckled the ground and left scarps and fissures; damaging roads, utilities, houses, and other structures. The largest slide known of, between the community of Laurel and Summit Road, south of Highway 17 in the Santa Cruz mountains, is reported to cover 75 Hectares and to contain several dozen houses, many of which sustained substantial damage (Figs 18-20). The ground cracking associated with such slides could in most cases be separated from that discussed earlier in section 5.2. Another form of slope failure - lateral spreading - was predominantly associated with liquefaction and is discussed below in section 5.4.

5.4 Liquefaction

A notable feature of the Loma Prieta earthquake was widespread occurrence of liquefaction, and the damage that it caused; not only in the spectacular failures such as those at the Marina District of San Francisco, but also in many less-dramatic but nevertheless costly instances. Damage caused by liquefaction included lateral spreading of river banks

EXPLANATION

	Fault rupture of Loma Prieta earthquake—Arrows show relative horizontal movement; small arrow and number show direction and amount of dip; U, upthrown side; D, downthrown side		Area of ground cracks possibly related to faulting
	Fault—Dashed where approximately located		Approximate limit of structural damage
	Main shock epicenter		Approximate limit of landslides
	Area of aftershocks and abundant landslides		Prominent area of liquefaction
			Damaged bridge
			Damaged dam
			Damaged two-story viaduct

FIGURE 14 MAINSHOCK EPICENTRE OF THE LOMA PRIETA EARTHQUAKE AND INFERRED FAULT RUPTURE RELATIVE TO AREAS OF LARGER AFTERSHOCKS, ABUNDANT GROUND CRACKS, AND LANDSLIDES AND TO LIMITS OF STRUCTURAL DAMAGE. ALSO SHOWN ARE LOCATIONS OF MAJOR DAMAGED STRUCTURES AND PRINCIPAL AREAS OF GROUND CRACKS AND LIQUEFACTION (FROM PLAFKER AND GALLOWAY, 1989)

FIGURE 15 THE MOST SEVERE GROUND CRACKING, IN THE EPICENTRAL REGION SUMMIT ROAD, SOUTH OF HIGHWAY 17.

FIGURE 16 SUMMIT ROAD, SOUTH OF HIGHWAY 17, LOOKING NORTH. CENTRE LINE SHOWS LATERAL OFFSETS; ROAD GRADE WAS CONSTANT PRIOR TO LOMA PRIETA EARTHQUAKE.

FIGURE 17 TYPICAL GROUND CRACK, SUMMIT ROAD.

FIGURE 18 HEADWALL SCARP AND FISSURE OF
LARGE TRANSLATIONAL BLOCK
LANDSLIDE; REACTIVATED BY LOMA
PRIETA EARTHQUAKE. BLOCK HAS
SEPARATED FROM HEAD WALL BY ABOUT
1 M.

FIGURE 19 CRACKING AND DEFORMATION OF
GROUND WITHIN TRANSLATIONAL BLOCK
LANDSLIDE.

FIGURE 20 VIEW OF SUMMIT ROAD - LAUREL
STREAM AREA. LARGE TRANSLATIONAL
BLOCK LANDSLIDE IS OBSCURED BY
REDWOOD FOREST.

FIGURE 21 PRINCIPAL SITES OF LIQUEFACTION

and levees, most notably on the Salinas, Pajaro and San Lorenzo Rivers, floatation of buried tanks, settlement and differential settlement of buildings, and failure of pavement and buried services due to block-motion of the ground surface.

Damage to levees posed an urgent problem with the winter rains coming. At Santa Cruz, for example, less than a week after the earthquake some members of the team met an engineer from the US Army Corps of Engineers who was assessing damage to stopbanks on the San Lorenzo River, with the intention of rebuilding about 5 to 10 km of bank on that river alone before the winter.

The map in Figure 21 shows liquefaction sites either visited personally by team members, or noted from first-hand descriptions from other engineers or scientists. It is likely that there were other occurrences besides these, that were not obvious while the team was in California. It is significant that many of the sites shown in Figure 21 had liquefied in 1906 and in other earthquakes (Youd and Hoose, 1978), adding further to the evidence of the repeatability of liquefaction.

As in previous earthquakes in California and elsewhere, bridges suffered from lateral spreading of the river banks. The Southern Pacific Railroad bridge over the

Pajaro River at Watsonville is shown in Figure 22. Here the banks of the river had converged by about one metre, closing the expansion joints of the bridge, failing the south abutment shown in Figure 23, and deforming the southern, timber approach spans and piles. The southern rail embankment had settled by around 500 mm and the adjacent flood-control levees had cracked due to lateral spreading. Sand boils in the river bed confirmed the role of liquefaction.

Five hundred metres upstream, to the east, the Main St bridge in Watsonville suffered in a similar fashion. But the effect on this much stiffer five-lane concrete structure was less severe. Nevertheless, expansion joints had closed, and a 30 mm crack had opened near the base of at least one of the concrete piers of the bridge, consistent with convergence of the banks. The bridge had been closed to traffic and excavation of test pits to examine piles commenced 2 days after the earthquake.

Another area seriously damaged by liquefaction was the causeway forming the eastern approach to the San Francisco Bay Bridge. Here, the approach roads and pavement in the toll plaza area suffered severe cracking due to settlement and lateral spreading and there were several sand boils of 3 to 4 m in diameter in the unpaved median to the west of the toll gates. By the following Saturday, the larger cracks in the traffic lanes had been patched with asphalt and with lean concrete. See Figures 24 to 27.

Other notable liquefaction sites included Moss Landing, south of Santa Cruz, and the Foot of Market area near the Embarcadero Freeway, the South of Market area, and the Mission area, all in San Francisco itself. All three San Francisco sites have sand fill over Bay Mud and all three liquefied in 1906, in a much more severe fashion (Youd and Hoose, 1978) than in 1989. At Moss Landing, a marine laboratory was severely damaged, jetty piles displaced (Fig. 28), and several other buildings affected by lateral spreading. The oil tanks shown in Figure 29 tilted and settled due to foundation failure in liquefied sand.

5.5 The Marina District

The Marina District of San Francisco provides an example of the interplay of site effects, amplification and liquefaction, with a localized area of fairly intense damage to buildings, pavements and buried services. The Marina is a much sought-after residential area of 3 to 4 storey timber-framed houses and apartment buildings, fronting onto San Francisco Bay near the Golden Gate bridge. It is constructed on land reclaimed by hydraulic fill, for the 1915 Panama Pacific Exhibition. Ironically, the exhibition was undertaken as a confidence boosting exercise after the great 1906 earthquake and fire. The bay that was filled can be seen in the 1857 view of San Francisco, shown in Figure 30. Typical soil conditions are shown in Figure 31.

FIGURE 22 SOUTHERN PACIFIC RAILROAD BRIDGE
OVER THE PAJARO RIVER OF
WATSONVILLE.

FIGURE 23 SOUTH ABUTMENT OF THE SP RAILROAD
BRIDGE. LATERAL CONVERGENCE OF
RIVER BANKS CLOSED EXPANSION
JOINTS AND FAILED ABUTMENT.

FIGURE 24 TYPICAL CRACKS DUE TO LATERAL
SPREADING AND SETTLEMENT OF EAST
APPROACH TO BAY BRIDGE.

FIGURE 25 CRACKS IN PAVEMENT OF BAY BRIDGE
APPROACH BEING FILLED IN, 4 DAYS
FOLLOWING THE EARTHQUAKE.

FIGURE 26 TYPICAL SAND BOIL, IN MEDIAN AREA
NEAR BAY BRIDGE TOLL PLAZA.

FIGURE 27 SETTLEMENT OF FILL AROUND
ADMINISTRATION BUILDING AT BAY
BRIDGE TOLL PLAZA.

FIGURE 28 TILTED POLE IN LIQUEFIED SAND AT MOSS LANDING.

FIGURE 29 OIL TANKS AT MOSS LANDING. THE TANKS HAD SETTLED AS WELL AS TILTED. EJECTED SAND AROUND FOUNDATIONS CONFIRMS PRESENCE OF LIQUEFACTION.

FIGURE 30 1873 VIEW OF SAN FRANCISCO, LOOKING SOUTHWEST. THE MARINA BAY IS SEEN ON THE LEFT OF THE PICTURE. MISSION BAY, AT CENTRAL RIGHT OF THE VIEW IS ANOTHER MAJOR AREA OF FILL THAT LIQUEFIED IN 1989, AS WELL AS IN 1906 (FROM THE CALIFORNIA WATER ATLAS).

FIGURE 31 GENERALISED SOIL PROFILE AT THE MARINA DISTRICT.

Damage in The Marina was confined to an area about 6 blocks square, corresponding to the area of filling. Settlement of buildings, by up to 200 mm, compressional failure of pavements, and sand boils were found throughout the 6-block square, as was cracking of stucco and plaster. Figures 32 to 35 show typical examples. A number of buildings were severely racked in the main shock, and some collapsed, partly due to their bottom storeys having been weakened by insertion of garage doors (see Section 7.8(c)).

It appears that liquefaction, aided by amplification of shaking by the Bay Mud, was the main cause of the ground damage, and that weak bottom storeys played a major part in severe building damage.

There were no strong-motion accelerographs at the Marina. The nearest record was obtained at the Presidio site, with horizontal peaks of 0.21 g and 0.10 g and a vertical peak of 0.06g (Shakel et al., 1989). However, this site was on rock. It is more likely that peak accelerations here were similar to the values of 0.26 g and 0.29 g obtained at similar distances at the Emeryville and Oakland Wharf sites on Bay Mud. The importance of amplification by the Bay Mud is emphasized by noting that the epicentral distance of the site is 98 km, which is well beyond the normal limit of 74 km for liquefaction in an M=7.1 earthquake, according to the criterion of Tatsuoka and Kuribayashi (1976).

The almost total absence of tensile cracking and the apparently random direction of the pervasive, compressional ground deformations was a curious feature of the pavement damage in the Marina area (M.J. Bennett, pers. comm.). There was no evidence of systematic movement down the slight slope, of about 1:200, towards the Bay. But what was evident, was compression away from buildings. Compressive features such as ridges in asphalt were frequently seen both in front of and behind the same house. Most intersections showed movement of all four corners towards the centre of the intersection. In addition, many buildings had settled, especially in the north west of the affected area, by up to 200 mm. A general mechanism similar to the classical foundation failure slip field is proposed to explain both the settlement and the compressive deformations at ground level. This is illustrated schematically in Figure 36.

6. SOIL STRUCTURES

6.1 Artificial Fill

A large variation in the performance of artificial fill occurred. The Marina District being an example of poor performance, attributable to the level of engineering of the hydraulic fill placed on unconsolidated Bay Muds as discussed in section 5.4.

Well engineered fills performed well, good examples being at Foster City and Redwood City, both on the south west margin of San

Francisco Bay and at distances of 50 km and 46 km respectively from the fault plane of the mainshock.

Accelerograms from Redwood City show a peak horizontal ground acceleration of 0.28g (Maley et al. 1989). A value similar to that estimated for the Marina District in section 5.5. The team heard no reports of significant ground failure at Redwood City, in contrast to what was observed in the Marina District.

6.2 Earth Dams

No catastrophic failures of earth dams were reported from this earthquake, and the team received no reports of damage to dams while they were in California. However, according to the "New Civil Engineer" of November 2, 1989, at least three dams in the epicentral region suffered severe cracking. These are the 56 m high Austrian dam 5 km from the epicentre and impounding Lake Elsman, the Newell Dam, about 20 km west of the epicentre, and the Lexington dam, 15 km northwest of the epicentre. All showed cracking and settlements of the order of around 100 mm.

7. BUILDINGS

7.0 Building Inspections

The local authority, e.g. the City and County of San Francisco, undertook the task of inspecting each building in the areas of reported damage. Most of these inspections included both the exterior and interior of the buildings.

Enormous resources were required to undertake this task. A week after the earthquake some 700 building inspectors had assembled in San Francisco from all states.

The inspection procedures were based on those presented in the Applied Technology Council's document ATC20 "Procedures for Postearthquake Safety Evaluation of Buildings". Timing of the 1989 publication and workshop presentation on September 21 in San Francisco was most fortunate.

The procedures are based on a three stage evaluation process:

Rapid Evaluation, undertaken by qualified building inspectors, civil/structural engineers or architects, which is a rapid assessment of safety. Used to quickly post unsafe buildings and identify those requiring further evaluation.

Detailed Evaluation, undertaken by a structural engineer, being a considered visual assessment of building damage to ascertain safety or need for more investigation.

Engineering Evaluation, carried out by the owners engineering consultant, as a very detailed evaluation of thorough inspection, review of drawings, and analysis by calculation.

FIGURE 32 (a) HOUSE IN CENTRE HAS SETTLED AND TILTED TO RIGHT, MARINA BLVD., MARINA DISTRICT.

FIGURE 33 COMPRESSIONAL DEFORMATION OF PAVEMENT; AXIS OF COMPRESSION WAS GENERALLY PERPENDICULAR TO ADJACENT BUILDING, MARINA DISTRICT.

FIGURE 32 (b) CLOSE-UP OF TILTING BUILDING, MARINA DISTRICT.

FIGURE 34 PAVEMENT DEFORMED BY INWARD, COMPRESSIONAL, DISPLACEMENT OF ALL FOUR CORNERS OF INTERSECTION OF BEACH AND ? , MARINA DISTRICT.

FIGURE 35 TYPICAL SAND BOIL, HERE IN BACKYARD OF HOUSE ON RICO BLVD., MARINA DISTRICT.

POSSIBLE MECHANISM

FIGURE 36 SUGGESTED MECHANISM FOR PERSUASIVE SETTLEMENT AND COMPRESSION OF GROUND.

The ranking or posting of inspected buildings is in three categories:

Inspected, where the building is apparently safe, and can continue to be used even though it may require to be repaired.

Unsafe, and should not be entered by anyone. (In practice this absolute prohibition of entry is not practical, but entry should be limited to inspection and repair or demolition personnel under controlled conditions and fully understanding of the situation and risk).

Limited Entry, which restricts access to those necessary for further evaluation or repair.

California operates an Office of Emergency Services "State Safety Assessment Plan for Volunteer Engineers", and it is known that many engineers worked under this plan on the inspection and posting of buildings.

Members of the NZSEE Society's reconnaissance team made their services available to San Francisco City authorities on the evening following the earthquake by presenting themselves at the City Command Centre. They were utilised in responding to reports of damaged buildings inspecting and reporting with recommended action. For example, later in the night of the earthquake a visual survey was made of the main streets of Chinatown where the predominantly unreinforced brick buildings showed an unexpected low level of damage.

The following day services were again volunteered, the ATC20 procedures were issued as standard forms, and the Geary Blvd and western parts of the Marina District were inspected, reported and posted. The immediate post-earthquake experience was enhanced by the need to make decisions on building safety in an environment of potential damaging aftershocks. The procedures used are adaptable to the New Zealand situation.

7.1 Damage Patterns

(a) Soil Type

Distinct geographic patterns were evident in the distribution of earthquake damage to buildings. The strongest factor in this distribution, particularly away from the epicentral region, was the type of natural foundation, the worst areas of damage all having poorly consolidated soils. These included the Marina and South of Market districts of San Francisco, downtown Oakland and the Pacific Garden Mall area of Santa Cruz. Buildings of similar types on firm ground, showed markedly less damage.

The nature of the ground was also significant on steeper land near the epicentre, where numbers of houses were affected by large-scale landslides.

(b) Structural Types

Damage distribution also reflected different structural types. Performance of

structures within each category provided few surprises and is described in detail in the following sections, which also includes a summary of the performance of non-structural elements.

One of the most significant structural failures occurred not in a building but in the Cypress elevated freeway, discussed in Section 8.5. This was a timely reminder of the vulnerability of non-ductile reinforced concrete.

(c) Building Configuration

A third major element in damage patterns was that of building form and configuration. Most damaged buildings appeared to demonstrate some vulnerability of configuration, though confirmation of causes of damage in some cases requires more detailed investigation. The factor seemed particularly prevalent in newer buildings which suffered damage, emphasising the importance of an understanding by architects of its significance.

The most common example, demonstrated both in the Marina and among older commercial buildings, was that of soft storeys, mainly at ground floor level. Other weaknesses of building form noted included structural eccentricity, discontinuous structural elements, concentration of flexing by stiff spandrels, inadequate building separation, and irregular plan forms. Examples of failures apparently resulting from each of these weaknesses are provided in the following sections.

(d) Summary

Overall, the damage observed to building structures was predictable, generally in terms of the three factors of soil type, structural type and building configuration. Damage to secondary elements and building contents emphasised the equally important requirements of provision for relative movement and adequate securing.

It should be emphasised that the total cost of damage to buildings, while far from insignificant, represented a small proportion of the total value of building development in the affected area. Encouragement can be taken from the generally good performance, particularly of more recent structures. At the same time, this must be seen in the context of the generally moderate level of shaking in the most densely developed areas affected.

7.2 Modern High-rise Buildings

(a) Overview

Almost without exception modern high-rise buildings, concentrated in the city centres of San Francisco and Oakland, showed no externally obvious ill-effects from the earthquake. The buildings in this category are generally of twenty or more storeys, typically with steel frames and thin, relatively light claddings (see Fig. 37). The detailing of brittle claddings provides for a degree of flexing of the structure, and we heard almost no reports of glass

FIGURE 37 HIGH-RISE BUILDINGS, SAN FRANCISCO. THESE SPECTACULAR MODERN BUILDINGS PERFORMED WELL.

FIGURE 38 101 CALIFORNIA, SAN FRANCISCO.

breakages among them. It is not known of course, whether the provisions for relative movement would have been adequate for the stronger ground movements which a similar sized earthquake centred nearby would cause.

Our observations of buildings of this type were principally of the exteriors from street level, but sometimes from within or from adjacent buildings, and we spoke to occupants of several. We were told of instances of minor damage, principally to furniture and contents, but had no reports of significant damage to the building structure or fabric, other than the AT and T exchange building in Oakland. Observations of large numbers of these buildings on the Monday five days after the earthquake showed them to be in use, and apparently performing as usual.

(b) 101 California (Figure 38)

The visual inspections included 101 California Street, San Francisco. The spectacular form of this building at ground level, with slender perimeter columns unsupported for the equivalent of six storeys, and a dramatic diagonal glazed roof over the foyer, provoked some speculation before the earthquake in relation to its likely seismic performance.

Some displacement was apparent to stone facings over the seismic movement joint between the tower and the lower podium structure, but no other ill effects were observed.

(c) Marriott Hotel (Figure 39)

The Marriott Hotel, at Fourth and Howard Streets, is a new forty-storey building in the South-of-Market district of San Francisco, where many older buildings were damaged. Its stepped glazed facades culminate in spectacular reflective glass fans at roof level. Opened for the first time on the day of the earthquake, it appeared when visited the following week to be undamaged and in normal operation.

This impression was substantially supported by a later detailed report from the Hotel's Project Administrator, Don Chandler. According to this, a single glass pane broke, this being in one of the decorative fans at roof level, and one fibre-reinforced concrete panel was cracked by impact from an adjacent building. Internally, a high barrel-vaulted ceiling to a top-level lounge suffered breakages, including two sprinkler heads which caused water damage in that area. Some plasterboard in stairwells had cracked but was readily repaired; and ceilings, other

FIGURE 39 MARRIOTT HOTEL, SAN FRANCISCO

FIGURE 40 HIGH-RISE BUILDINGS, CENTRAL OAKLAND. TO THE RIGHT IS THE KAIZER PLAZA BUILDING (see section 7.2(b)) THE LOWER BRICK BUILDING SUFFERED CONSIDERABLE DAMAGE (see section 7.3(d)).

FIGURE 41 UNDAMAGED 6 M HIGH FINNED GLAZING TO KAIZER PLAZA BUILDING.

than the barrel vault, were unaffected. There was fairly widespread damage to ornaments and furniture, some of which had not been completely installed. Two elevators still under construction suffered damage to cables and rails because of incompletely secured weights. Those elevators which had been commissioned, and other building services, suffered no damage, and the three emergency generators operated as intended. The hotel was reported to be back in full operation the second day after the earthquake.

(d) Kaiser Plaza Building (Figures 40, 41)

This relatively new metal-and-glass-clad building of about thirty storeys, at 21st Street in Kaiser Plaza, Oakland, has a ground floor enclosed by high sealant-jointed glass panels, and it is linked by a bridge to an adjacent parking building. No damage was seen to the building exterior, including the ground floor glazing and bridge junctions. An occupant reported that the earthquake as experienced on the sixth floor was undramatic and that she had suffered no damage. Nearby older buildings showed considerable damage.

(e) A T & T Exchange, Oakland (Figures 42, 43)

An exception to the rule of little damage to modern high-rise, this was particularly interesting in showing obvious damage to main structural elements. This is apparently a composite structure of moment-resisting steel frame and concrete shear walls, the external walls being of poured concrete, faced on the two adjacent street sides with precast permanent formwork. These two walls, penetrated by windows at the upper levels, are omitted over the ground storey and substituted at this level by walls set back several metres from the building face. The edge of the first floor slab provides a transfer diaphragm between these walls.

The recessed shear walls, evidently constructed of lightweight concrete, showed severe damage and encasement to the ground floor columns was extensively spalled. The appearance of the building suggests that eccentricity arising from the uneven stiffness of the external walls on the upper floors has been greatly increased at ground floor level. It seems that the off-setting of the shear walls on the two street frontages, and their construction of lightweight concrete, has created a soft ground storey on these sides. It will be interesting to learn whether detailed investigation supports this initial assessment.

(f) Albany Hill Condominium Block

This structure is a 15-storey reinforced masonry residential block that was under construction when a NZPCA study tour visited San Francisco in 1977.

The current code does not permit reinforced masonry up to that height and the team made enquiries about its performance from the designers Shapiro Okino Horn and Associates. They were pleased to be able to

report that it performed well and suffered no damage.

7.3 Older Commercial Buildings

(a) Overview

This relatively diverse category, ranges in height from three to around fifteen storeys, and covers a wide span of time extending into the 1960's. Unreinforced masonry buildings are excluded, and covered in Section 7.6. A high proportion of the buildings in this category are steel framed, with heavy brick facings. Also included are some commercial buildings of moderate height constructed to more recent standards and materials, such as that at the corner of Franklin and 21st Streets in Oakland.

While few buildings in this category appeared to suffer significant structural damage, there were many examples of damage to glass, brick and other brittle claddings, and in some cases this was severe. The effects tended to be particularly intense at ground floor level, as a result of greater structural flexibility, but there were also many examples of damage at higher levels. Numerous instances were observed of damage to claddings caused by impact between buildings, but these were generally less severe than might have been expected in view of the very small clearances. We heard reports of damage to interior fabric and building services, but had limited opportunity to verify them.

The recurrent theme in the damage in this category was that of insufficient allowance in the heavy brittle claddings for movement of the structural frames under seismic loadings.

(b) Southern Pacific Company (Figure 44)

The Southern Pacific Company building at the east end of Market Street, San Francisco, suffered damage to ground floor windows and facings to columns, indicating soft-storey movement at this level. On the upper levels, ends of the terracotta sills were extensively broken, dropping debris into the street. When inspected six days after the earthquake, the building was partially in use, but many of the elevators remained out of commission.

(c) Emporium Capwell, Oakland (Figures 45, 46)

This older five-storey building in Central Oakland, with steel frame clad in masonry, reportedly dates from 1928. Virtually all ground floor windows were broken, and masonry facings cracked and spalled, particularly at the level of the window head. At higher levels there was also breaking of glass, and extensive shear cracking of the brickwork and terracotta or stonework, particularly above the apparently stiffer mezzanine level.

The building was completely closed when visited six days after the earthquake, and we were unable to inspect the interior. A later inspection showed that a mezzanine

FIGURE 42 AT & T EXCHANGE, OAKLAND.

FIGURE 43 AT & T EXCHANGE, DETAIL. DAMAGE TO SHEAR WALLS AND SPALLING OF PERIMETER COLUMNS AT GROUND FLOOR.

FIGURE 44 SOUTHERN PACIFIC CO. SAN FRANCISCO. SOFT STOREY DAMAGE AT GROUND FLOOR AND SPALLING OF TERRA-COTTA SILLS AT UPPER LEVELS.

FIGURE 45 EMPORIUM CAPWELL OAKLAND. CONCENTRATED LINES OF DAMAGE ARE VISIBLE TO FACINGS ABOVE AND BELOW THE SOLID WALL AT MEZZANINE LEVEL.

FIGURE 46 EMPORIUM CAPWELL CORNER DETAIL. SOFT-STOREY DAMAGE TO COLUMN FACINGS AND WINDOWS.

floor had partially collapsed, and that the building would be out of use for several months.

(d) Franklin/21st Streets (Figures 47-49)

Extensive window damage was also evident to the more recent building at the corner of 21st and Franklin Streets in Oakland. This ten-storey apparently steel-framed building with brick-faced spandrels lost many windows at most levels on both street frontages, and some brick facings at the junctions between spandrels and the narrow shear walls on the 21st Street face. A tall strip of brick-facing at the junction with an adjoining building collapsed, smashing the canopy below. From the little that could be seen of the face opposite Franklin Street, windows on that face were less affected.

The stiff spandrels clearly concentrated inter-storey deflection in the window strips, which showed no provision for seismic movement. This seems to have been accentuated by eccentricity, the Franklin Street face having no vertical bracing elements, in apparent contrast with the other sides, particularly the two away from the frontages.

Furniture and equipment was being evacuated when the building was visited five days after the earthquake, and it was still empty and unrepaired five weeks later.

(e) I Magnin & Co (Figure 50)

The ten-storey building of I Magnin & Co on San Francisco's Union Square suffered breakages to the majority of glass panes on both street frontages, affecting all levels. The facades have stone facings and

very large glass panes, but despite its modern appearance the building is reported to date from 1946. Remnants of broken panes were observed being rapidly removed in the darkness of the power cut during the night following the earthquake. They were replaced with plywood panels.

(f) Apartment Block in Oakland (Figures 51, 52)

This twelve-storey reinforced concrete building is L-shaped in plan with lift and stair shafts at the junction of the two wings. As can be seen in the figures, shear walls at the end of one wing were extensively cracked, most obviously in spandrels between windows, in the short beams between the two shear wall planes, and where the surface had spalled on reinforcing lap lines. This appears to be a classic example of damage arising from the configuration of the building.

(g) Impact Damage (Figure 53)

Clearances between buildings in this category were often very small, frequently being no more than a few centimetres. As a result, impact damage was frequent. The figure shows an example from the South of Market area, with particularly severe damage to column claddings and windows at roof level of the adjacent building. The entire facade was wrapped in wire netting to reduce danger from falling masonry.

FIGURE 47 FRANKLIN/21ST STREETS, OAKLAND.
INTERSTOREY DEFLECTION
CONCENTRATED IN WINDOW BANDS BY
STIFF SPANDRELS.

FIGURE 48 FRANKLIN/21ST STREETS. IMPACT
WITH THE ADJACENT BUILDING HAS
CAUSED THE COLLAPSE OF A VERTICAL
STRIP OF BRICKWORK SMASHING THE
CANOPY BELOW.

FIGURE 49 FRANKLIN/21ST STREETS, REAR
CORNER. TORSIONAL MOVEMENT
RESULTING FROM ECCENTRIC LOCATION
OF SHEAR WALLS HAS APPARENTLY
MAGNIFIED DEFLECTIONS ON FRONT
WALL.

FIGURE 50 I. MAGNIN & CO. GLASS LOSS IN EARLY HERMETICALLY SEALED BUILDING.

FIGURE 51 APARTMENT BLOCK, OAKLAND. L-SHAPED PLAN, SHOWING DAMAGE AT JUNCTIONS AND END OF WINGS. THE DAMAGE APPEARS TO RELATE TO THE BUILDING CONFIGURATION.

FIGURE 52 APARTMENT BLOCK, OAKLAND. DETAIL OF DAMAGE TO SHEAR WALL AND CONNECTING BEAM. EXTENSIVE HAIR CRACKING IS VISIBLE.

FIGURE 53 IMPACT DAMAGE. SEVERE DAMAGE TO WINDOWS AND COLUMN FACINGS AT THE LEVEL OF THE ADJACENT ROOF.

7.4 Modern Low-rise Commercial Buildings

(a) Overview

In contrast with recent tall buildings, a number of examples were observed of modern low-rise commercial buildings which showed significant damage. As far as was immediately apparent, most effects were limited to secondary elements, such as ceilings, partitions, windows, and building services, but the damage was sometimes severely disruptive to the usage of the buildings.

The reasons for the greater damage in this category were not clear. Doubtless the shorter period of the structures will have been a factor. In several cases, provision for movement in secondary elements was clearly not adequate, and this appears sometimes to have been exacerbated by weaknesses in structural configuration. Securing of ceilings and building services was evidently inadequate in some instances, either in design or construction.

(b) A T & T Offices (Figures 54 - 58)

The office occupied by A T & T in Central Oakland, is an attractive modern six-storey building clad in fibre-reinforced concrete and glass. Broad in proportion to height, it has a full-height internal atrium, and cut-away corner entrance, so that some columns are free-standing for three and five floors. It lost many external windows, and areas of ceiling tiles, and suffered flooding from broken sprinkler pipes. The ceiling damage was apparently largely the result of flooding from the sprinkler system, but there were areas at ceiling perimeters where tiles appeared to have fallen as a result of movement. Concentrated areas of damage occurred at seismic joints, where there was impact to building services and architectural elements including sprinkler pipe breakages, and a zone of crushed concrete to the floor slabs.

FIGURE 54 AT & T OFFICES OAKLAND. MANY WINDOWS WERE LOST, PARTICULARLY AROUND THE DRAMATIC CUT-AWAY ENTRANCE.

FIGURE 55 AT & T OFFICES. DETAIL OF CEILINGS DAMAGED BY BURST SPRINKLER PIPEWORK.

FIGURE 56 AT & T OFFICES. SPLENDID INTERNAL ATRIUM. THE BUILDING WAS EVACUATED FOR AT LEAST 2 MONTHS.

FIGURE 57 AT & T OFFICES. IMPACT DAMAGE AT SEISMIC MOVEMENT JOINT CAUSED SEVERE DAMAGE TO SLAB, SERVICES, AND CEILINGS (GIBSON).

FIGURE 58 AT & T OFFICES. IMPACT DAMAGE AT SEISMIC MOVEMENT JOINT CAUSED SEVERE DAMAGE TO SLAB, SERVICES, AND CEILINGS (GIBSON).

Several panes of reflective glass in a fully glazed external wall had broken, principally where they were penetrated by window cleaning guide fixtures.

The building was still unoccupied in early December. Movement in the structure clearly exceeded the capacity of the architectural and services detailing. It will be interesting to learn the reasons for this, and whether irregularity of the structure at the entrance was a contributing factor.

(c) Pacific Coast Savings & Loan

While adjacent taller buildings showed no obvious damage, this small two-storey modern-looking structure in San Francisco's Market street had many broken glass panes, and dislocated facings to an exterior column on one side. It was glazed full height around much of the perimeter, and had lift shaft and other solid elements to one corner. Though there were signs of ground settlement nearby, the pattern of damage suggested torsional movement about the eccentric stiff elements.

(d) 150 Fourth Street (Figure 59)

The six-storey building at 150 Fourth Street in San Francisco suffered damage which also appeared to relate to its structural configuration. It is in two sections, linked by a glazed atrium entrance. The northern section has apparently stiff walls at both ends, but at the south end of the southern section these are replaced by exposed steel bracing frames, which were damaged by the earthquake. The building frame is generally regular, but the perimeter beam is omitted at first floor level around part of the northern section, and three sides of the southern.

According to a report posted on the building, it suffered moderate repairable structural damage and non-structural

damage. In addition to the steel bracing frames at the south end, this included damage at the elevator cores and north-end stairwell, "slab punching shear failure" and hair-line cracking in shear walls. Surface cracking was visible in the first floor beams on the south and east sides.

(e) San Francisco Airport (Figure 60)

Damage occurred to new buildings at San Francisco's International Airport. The concourse spaces in the main International Terminal building suffered loss of ceiling tiles and some rail sections, particularly at edges, as well as some water damage from broken pipework to sprinklers or plant watering systems.

Airport officials reported that the new United Terminal suffered nearly complete collapse of the ceilings, and extensive water damage from broken sprinklers. Ceilings in the Control tower also collapsed together with lights, smashing glass tops of map-tables and causing some injuries. The building lost two exterior windows, destroying vital sound insulation.

Two hotels slightly south of the airport were visibly damaged. The EERI Preliminary Reconnaissance report recorded that the Amfac hotel had a roof-level water tank and penthouse fall to the ground, damaging the elevator shaft, while the Hyatt Regency suffered shear cracks in the basement walls and "severe vertical spalling of concrete floor slabs ... adjacent to one of the elevator shafts".

(f) Office Buildings, San Mateo (Figures 61, 62)

Two examples were seen of the performance of architectural and services elements in low-rise modern office buildings in the San Mateo area. The Metro Centre in Foster City is an attractive new four-storey office building in an irregular stepped

FIGURE 59 150 FOURTH STREET, SAN FRANCISCO. MODERATE STRUCTURAL AND NON-STRUCTURAL DAMAGE APPEARED TO RELATE TO STRUCTURAL IRREGULARITIES (GIBSON).

FIGURE 60 SAN FRANCISCO AIRPORT INTERNATIONAL TERMINAL. PARTIAL COLLAPSE OF SUSPENDED CEILINGS (JAMES).

FIGURE 61 METRO CENTRE FOSTER CITY. DAMAGE TO WINDOWS, CLADDING AND CEILINGS RELATED TO THE BUILDINGS IRREGULAR FORM.

FIGURE 62 METRO CENTRE, FOSTER CITY. 2-WAY CEILING GRID WHICH HAD LOST TILES THOUGH RAILS ARE BRACED AND LARGELY UNDAMAGED. OTHER AREAS WERE BROKEN BY FALLING SERVICES.

FIGURE 63 GRADUATE BUSINESS SCHOOL, STANFORD. A 1965 REINFORCED CONCRETE STRUCTURE. DAMAGE IS THOUGHT TO HAVE RESULTED FROM DISCONTINUITY AND ASYMMETRY OF SHEAR WALLS (GIBSON).

configuration, steel-framed and clad in fibre-reinforced concrete and glass. Steel bracing is visible in the plane of some exterior walls. The building suffered broken windows, at least one crack in the FRC panels and extensive damage to ceilings. We were told by an occupant that some of the ceiling damage was caused by the breaking free of air-conditioning equipment housed beneath the roof, and that water damage resulted from broken lines to cooling towers. An examination of an area where ceiling tiles had fallen showed that the ceiling rails were braced with wire ties. There appeared to be a concentration of glass panel and ceiling damage in the corners of the building.

A three-storey office building at 3130 La Selva, San Mateo, suffered extensive internal cosmetic damage. Roof-mounted air-conditioning equipment became dislodged and had to be removed. As the external windows did not open, this left the internal atmosphere unsatisfactory. Stairwell linings were damaged to the extent that much of the material was replaced. The exterior windows did not break, but some ground floor panels moved in relation to their frames to the extent that gaps of up to 15mm remained between glass-edge and frame.

(g) Graduate Business School, Stanford University (Figure 63)

The four-storey main building of the Business School, constructed in 1965 of reinforced concrete, was severely damaged resulting in its top three floors being closed to use for an extended period. A letter from the Dean circulated to alumni reported that the upper floors suffered some structural damage, and damage to ceilings, windows and lights. Asbestos insulation in the roof space was exposed by breakages in the ceiling below. The J Hugh Jackson Business Library, occupying the north end on the top three floors, suffered spectacular collapses of bookshelves, exacerbated by a broken water pipe.

The building has a column slab gravity frame with shear walls providing lateral stability. Many of these walls were not continuous between adjacent floors and their distribution was irregular. The discontinuity was exacerbated by irregular openings and by end regions often occurring between columns. The plan form of the building was also irregular with marked changes in diaphragm stiffness.

Shear walls at all levels showed significant cracking, since repaired by epoxy injection. In addition, the configurational weaknesses are to be reduced by the construction of new shear walls and the strengthening of the old.

(h) Oakland Coliseum Sports Centre

This is another structure that was being built when the NZPCA team visited in 1977. It suffered no structural damage. Although it lost a small amount of glass, it was available for use very quickly after the earthquake.

7.5 Historic Buildings

(a) Overview

Some older buildings inspected were of particular civic or historic significance, and distinctive architectural style. Buildings of this type, which were at least moderately damaged by the earthquake, occurred in a wide area, including Santa Cruz, Watsonville, San Francisco and Oakland. Some are described under other categories, for example Cooper House in Santa Cruz and Stanford's old buildings under "Unreinforced Masonry", and Oakland's Emporium Capwell under "Older Commercial". The buildings included Oakland's seventy-five year old Beaux Arts City Hall, a steel-framed masonry-clad building which was reported in the press to have suffered serious damage. Strong public debate was stirred about the merit and practicability of restoring this and several of other buildings. At least one, the hundred-year old Cooper House, has since been demolished.

FIGURE 64 CITY HALL, SAN FRANCISCO. STIFF PLASTER AND STONE LININGS SUFFERED EXTENSIVE DAMAGE, PARTICULARLY IN THE AREA SURROUNDING THE DOME.

FIGURE 65 SAN FRANCISCO LIBRARY. VERTICAL CRACK FORMED BETWEEN CONCENTRATED STACK AREA AND REMAINING STRUCTURE.

FIGURE 66 SAN FRANCISCO LIBRARY. LAP FORMED BETWEEN STACK MEZZANINE AND SUPPORTING WALL.

(b) City Hall, San Francisco (Figure 64)

San Francisco's City Hall is a framed building of four main storeys, essentially symmetrical, and broad in relation to height. Its form is monumental, with neoclassical facades, and a high central dome over a full height internal space. Claddings and linings are heavy and stiff, with large areas of marble and stone facing to the main internal spaces, and plaster on metal lathe to the ceilings of the dome and surrounding vaults.

Interior spaces suffered extensive superficial damage to the plaster and stone facings, clearly the consequence of movement in the more flexible structural frame. The damage included shedding of large marble panels, with the potential to cause serious injury particularly in stairwells. It was greatest in the region surrounding the central dome.

While much of the damage was subtle, clearly its repair will be difficult and expensive.

(c) City Library (Figures 65, 66)

San Francisco's City Library is a four-storey steel-framed building, with brick walls and partitions, built around 1917. The essentially-symmetrical plan is a hollow rectangle with central spine dividing the middle into two lightwells.

Damage to the building fabric was concentrated in the middle and east of the north wing. This corner contains the book stacks, housed on a seven-storey light steel mezzanine construction within the shell of the four-storey wing. The internal mezzanine structure is fixed to and relies on the main building for lateral support. These mezzanine floors have pulled away from the building walls by 100 mm (Fig. 66). The high loading of books in this area has created an eccentric load within the otherwise symmetrical building. A prominent vertical crack has resulted at a junction of this wing with the rest of the building extending from roof to near ground. The south face of the wing also developed a horizontal crack at the top floor level.

Other internal damage included extensive book spillage, some toppling and collapse of shelves and cabinets, and dislocation of stone facings.

(d) Churches (Figure 67)

Churches reported as having been significantly damaged included San Francisco's Orthodox Cathedral, Oakland's Roman Catholic Cathedral, Stanford University Chapel and historic Churches in Watsonville and Hollister.

The St Francis de Sales Cathedral in Oakland showed potentially serious cracking between a gable wall and adjacent side walls, and to the tower. Watsonville's St Patrick's Church suffered similar but more extensive damage, including the loss of tops of gable walls. The spire was later removed, and the future of the church was

in doubt.

7.6 Unreinforced Masonry Buildings

(a) Overview

Predictably, widespread damage was suffered by unreinforced masonry buildings, extending from the areas around the epicentre to parts of Oakland and San Francisco. The nature of the damage ranged from toppling of parapets and sections of walls to the effective destruction of a relatively few buildings. It was most evident in areas of soft soils such as San Francisco's south of the Market area, and in the vicinity of the epicentre.

If this damage held any surprise, it was that so many buildings in the category survived with little obvious effect. An inspection by members of the reconnaissance team of sections of San Francisco's Chinatown during the night following the earthquake revealed little obvious damage, and normal activity returned to this area very quickly. Closer inspections will be needed to identify whether the buildings have been weakened by subtle damage effects.

(b) Epicentral Region (Figures 68 - 71)

At the other extreme, in the Pacific Garden Mall area of Santa Cruz many buildings were severely damaged, resulting in deaths and injuries, and causing severe dislocation of business activity. Many of these buildings, such as the historic Cooper House, had been attractively refurbished, to cater to the city's important tourist trade. However, the refurbishment was generally superficial, with little attention to structural upgrading.

Other towns near the epicentre including Hollister and Watsonville also suffered severe damage to buildings in this category, which may have lasting economic and social effects.

(c) Stanford University (Figures 72, 73)

Estimates given at a briefing at Stanford's Engineering Department on 26 October put the value of damage to the University's structures at US\$160 million. Much of the damage was reported to be to unreinforced masonry buildings, including the unstrengthened Language and Geology corners and the magnificent Chapel in the Main Quadrangle. Unreinforced masonry buildings which had been strengthened since the 1950's were reported to be in good condition after the earthquake.

(d) South of Market and Oakland (Figure 74)

Examples of seriously affected unreinforced masonry buildings were observed in San Francisco's South of Market area and in Central Oakland, both areas of soft soils. The worst of these was a large four-storey brick building at Sixth and Bluxome Streets in the South of Market area which lost the top storey of an end wall, causing several deaths in cars below. A wall of similar orientation beside an internal recess in the building suffered similar damage. The

FIGURE 67 ST PATRICK'S CHURCH, WATSONVILLE. EXTENSIVE SHEAR FRACTURING OF MASONRY TOWER AND GABLE WALLS.

FIGURE 68 PACIFIC GARDEN MALL, SANTA CRUZ. CHARMINGLY REFURBISHED BUILDINGS BUT WITH LITTLE ATTENTION TO STRENGTHENING (LAIRD).

FIGURE 69 PACIFIC GARDEN MALL, SANTA CRUZ.

FIGURE 70 PACIFIC GARDEN MALL, SANTA CRUZ. THE DESTRUCTION OF THIS AREA CAUSED GREAT SOCIAL, ECONOMIC AND AESTHETIC LOSS.

FIGURE 72 STANFORD UNIVERSITY. REPAIR OF DAMAGED ARCHWAY. TOTAL DAMAGE, MAINLY TO UNREINFORCED MASONRY, WAS ESTIMATED AT US\$160 MILLION (GIBSON).

FIGURE 71 COOPER HOUSE, SANTA CRUZ. THIS HISTORIC GEM HAS SINCE BEEN DEMOLISHED.

FIGURE 73 STANFORD UNIVERSITY. THE STRENGTHENED HISTORY CORNER PICTURED WAS UNDAMAGED.

FIGURE 74 BLUXOME AND 6TH STREETS, SAN FRANCISCO. THE COLLAPSE OF THIS WALL KILLED 5 PEOPLE. ADDED TIES CAN BE SEEN RUNNING PARALLEL TO THE COLLAPSED SECTION BUT THE WALL WAS POORLY TIED AGAINST FACE LOADS.

FIGURE 76 DETAIL OF STRENGTHENED BUILDING. THIS MASONRY BUILDING WAS STRENGTHENED BY REBUILDING THE FACADE TO INCORPORATE AN ELEGANTLY DETAILED STEEL FRAME.

FIGURE 75 DAMAGED AND STRENGTHENED BUILDINGS, SAN FRANCISCO. THE CORNER BUILDING WAS BADLY DAMAGED AND IS BEING DEMOLISHED. THE STRENGTHENED BUILDING BEYOND IS INTACT.

FIGURE 77 HOUSE, HOLISTER. COLLAPSING OF PILES OR JACKSTUDS WAS A COMMON MODE OF FAILURE IN THE REGION OF THE EPICENTRE.

FIGURE 78 HOUSE, LOMA PRIETA. THE POORLY-BRACED GROUND FLOOR OF THIS HOUSE HAS COLLAPSED.

FIGURE 79 HOUSE ON LANDSLIDE, LOMA PRIETA. MOVEMENT OF THE LANDSLIDE HAS CAUSED THE COLLAPSE OF THE FRONT SECTION OF THIS HOUSE.

FIGURE 80 SOFT-STOREY DAMAGE, MARINA DISTRICT. SOME OF THESE BUILDINGS WERE LATER JACKED UPRIGHT AND BRACED.

FIGURE 81 (a) MARINA. THIS CORNER APARTMENT BLOCK LATER COLLAPSED FULLY, AS SHOWN TO THE LEFT OF FIGURE 82.

FIGURE 81 (b) GROUND FLOOR DETAIL.

FIGURE 82 DOUBLE COLLAPSE, MARINA. TWO ADJACENT BLOCKS WHOSE GROUND FLOORS COLLAPSED COMPLETELY SEVERAL DAYS AFTER THE QUAKE. THEY WERE LATER DEMOLISHED.

FIGURE 83 COLLAPSE, MARINA. THIS BUILDING LOST TWO STOREYS AT THE CORNER AND WAS QUICKLY DEMOLISHED.

FIGURE 84 VENEER, MARINA. NO TIES COULD BE SEEN BETWEEN THE VENEER AND HORIZONTAL BOARDING BEHIND.

collapses revealed long ties which had apparently been added to the structure, running parallel to the collapsed sections. The unsafe state of the buildings was reported in the press to have been advised to the owners prior to the earthquake, and litigation was underway.

7.7 Strengthened Buildings

No general review of the performance of strengthened buildings, or of the variety of standards which had been applied, was possible in the short period of the reconnaissance. However, there were a number of indications of the success of strengthening procedures in at least some instances. For example, of 56 schools strengthened under an upgrading programme during the 1970's, none were significantly damaged.

Figures 74 and 75 show an example of a commercial building in central San Francisco, strengthened by the addition of steel framework.

7.8 Timber Structures

(a) Overview

As expected, a high proportion of timber-framed buildings showed little ill-effect from the earthquake even when subjected to severe ground shaking. However, in the epicentral region a proportion of timber houses suffered dislocation from foundations, and others were affected by being situated on landslides. The other area of extensive damage to timber structures was the Marina district of San Francisco, where the building configurations gave rise to distinctive modes of failure.

(b) Houses, Epicentral Area (Figures 77 - 79)

Failures of houses in the areas close to the epicentre followed patterns familiar from earlier earthquakes such as Edgumbe and Coalinga. The most common severe failure was the collapse of sub-floor framing, in the form of piles with or without jackstuds above. Examples of this were seen over a wide area, including the vicinities of Hollister, Watsonville and Loma Prieta. Other predictable failures included loss of chimneys and brick veneers.

In the Santa Cruz mountains, houses were affected by being located on large landslides. Many were in thickly-wooded areas and while the press reported that hundreds of houses were affected, the scale of the problem, and the effects on the individual houses, were not easy to ascertain.

(c) Residential Buildings, Marina (Figures 80 - 86)

Particularly spectacular failures of timber-framed structures occurred in the exclusive Marina District of San Francisco. The buildings are characteristically two to four-storeys high in row form, independently structured but with

negligible separation, and clad in stucco or brick veneer over horizontal or diagonal timber boarding.

A number of factors combined to cause the failures. Ground motions were strongly amplified by the underlying deposits of Bay mud, overlain with poorly consolidated fill. There were widespread signs of soil liquefaction and settlement. A high proportion of the buildings had weakly-braced ground storeys, having been built or later modified to accommodate motor cars on their ground floors.

The worst-affected buildings were those on corners, where they were frequently four-storeys high and subdivided into apartments. In many cases the ground floors of those consisted of rows of garages on two frontages, with little lateral stiffness on either face. Buildings in such positions were subject to battering from houses on two adjacent sides with no compensating pressure from the street sides, and collapse when it occurred was invariably towards the street intersection.

Structural failures consisted generally of horizontal offsetting of the ground floors, and cracking of ground floor walls and column joints, leading sometimes to the total loss of one and occasionally two storeys. Other buildings showed settlement with some instances of tilting. Damage to stucco finishes and brick veneers was widespread, as to interior linings and chattels. Several of the buildings were demolished during the week following the earthquake. Others have since been jacked back to vertical.

(d) Madison Park Apartments, Oakland

This five-storey timber building, included on the National Register of Historic Places, had clearly survived the 1906 earthquake. Damage externally visible consisted of the loss of the ground floor brick veneer, which showed no ties to the diagonal boarding behind, and the consequential loss of ground floor windows. The upper storeys, including their aluminium-framed windows within wooden subframes, appeared unscathed.

7.9 Schools

Of the 145 schools in San Francisco City and County, all but four were reported to be back in use after being checked, by the Monday following the earthquake [personal communication from C. Arnold]. Of these four, one in Marina was in use as an emergency shelter and another was endangered by an adjacent structure. Outside of San Francisco and County, in the epicentral region at Summit Road at least, schools were damaged by ground failure.

7.10 Non-Structural Elements

(a) General

Outside the two categories of unreinforced masonry and timber structures, the overwhelming proportion of damage observed

FIGURE 85 STUCCO DAMAGE, MARINA. IN THIS CASE THE FRACTURING APPEARED TO RESULT FROM BUILDING SETTLEMENT. SIGNS OF LIQUEFACTION AND SETTLEMENT WERE FREQUENT.

FIGURE 86 FIRE, MARINA DISTRICT. THE FIRE FED ON FRACTURED GAS LINES AND WAS BROUGHT UNDER CONTROL WITH EXTREME DIFFICULTY, USING SEA WATER PUMPED TO THE SITE BY A FIRE BOAT.

related to architectural elements, building services, and contents. Two main causes were evident: inadequate provision for movement within and between secondary elements in relation to the flexibility of the structure, and inadequate securing and support. The importance of these aspects of New Zealand codes was therefore reinforced. In a relatively small number of cases observed, the damage was the direct result of structural failure, though there were many examples which suggested inadequate communication between structural engineers and architects or services engineers, and some indicating poor construction.

(b) Windows

One of the reassuring aspects of the earthquake was the good performance of window and cladding systems to modern high-

rise buildings. This indicated that public fears of streets being showered with glass from such buildings are not justified at least in the moderate level of shaking experienced. However, windows in older and lower buildings, particularly in the form of horizontal bands between stiff spandrels, sometimes caused serious problems. This appears to justify the New Zealand practice of providing seismic subframes in such situations, as opposed to Californian methods which rely largely on movement between glass and frames.

No instances were found of multi-storey frameless glazing with silicone joints. Six metre high frameless toughened glass panels, sealant jointed and with fin stiffeners, were observed to have performed successfully in at least one modern high-rise building.

FIGURE 87 COLLAPSED LIBRARY SHELVES,
STANFORD.

FIGURE 88 SPILLED BOOKS, SAN FRANCISCO
LIBRARY.

FIGURE 89 TOPPLED MICROFILM CABINETS, SAN
FRANCISCO LIBRARY.

(c) Claddings

The need to provide for movement around brittle claddings was emphasised by the performance of older medium-rise buildings, many of which shed fragments of brick, terracotta and glass into the streets. Frequent impact damage between buildings, sometimes serious, was a reminder of the importance of providing adequate separation between structures.

In domestic scale building, many examples were seen of damage to brick veneers and stucco claddings, as noted under Section 7.8. The use of these materials is much more common than in New Zealand, presumably as a response to the danger of fire, which appears to have a higher profile in the Californian public consciousness.

(d) Partitions

While opportunities to inspect building interiors were limited, a number of instances were observed of damage to stiff partitions by building movement. Separation of partitions from the structure and from one another was not noted in any buildings, and appears to be uncommon. The resultant damage, while doubtless expensive cumulatively, did not appear to cause serious disruption, and may have been less costly than detailing for separation. It is not known whether any instances occurred of unseparated partitions having a seriously detrimental effect on structural performance. The loss of stone facings to unseparated partitions in San Francisco's City Hall presented a real danger to building users.

(e) Ceilings

Several examples of the collapse of suspended ceilings have been given in earlier sections. Some apparently resulted from inadequate bracing of the ceilings themselves, and consequent damage to the ceiling rails. Elsewhere ceiling panels had fallen while the rail systems appeared largely undamaged. In other cases damage was reported from building services mounted in the ceiling space.

While the ceilings themselves no doubt created a hazard and nuisance, they were frequently quickly cleaned up. More lasting disruption sometimes resulted from the effects of ceiling mounted services, particularly the breakage of sprinkler heads and consequent flooding.

(f) Contents and Equipment

The most dramatic examples noted of damage to the contents of buildings were within libraries (Figures 87, 88). In Stanford's Graduate Business School, row after row of bookshelves toppled, spilling books, which were also damaged by a burst water-pipe. It was extremely lucky that no serious injuries resulted to library users or staff. San Francisco's main library also spilled large numbers of books, but a relatively small proportion of shelves toppled or collapsed. As noted earlier, the weight of shelving stacks concentrated on seven levels in one corner of the

building appeared to be principal cause of serious damage to the structure. In the same building large cabinets of microfilms toppled, posing serious threats to staff (Fig. 89).

The Marriott Hotel's Project Administrator reported extensive damage to furniture, decorative sculptures and glass-topped tables. As a result, he was taking a personal interest in the selection and fixing of such furniture. Efforts to dry out carpets in the United Terminal at San Francisco Airport, drenched by broken sprinklers, were unsuccessful, and they had later to be replaced.

Overall, damage to building contents clearly represented a significant loss both in direct financial terms and disruption to functional activities.

(g) Building Services

A principal factor in the delays in restoring electricity supplies to some areas, by up to several days, was the danger of igniting gas from leaks in supply pipework. This is relevant to those parts of New Zealand with reticulated gas supplies. The resulting disruption to human activity was significant, affecting lighting, ventilation and elevator services within buildings, as well as security of buildings and contents. Those buildings with good emergency power supplies, such as San Francisco's Hilton Hotel, were at an advantage, and well placed to provide assistance to emergency efforts.

Several other examples of effects to and by building services have been provided in the preceding text. Of particular significance were fire-sprinkler systems. While in some areas fire-fighting water supplies were lost by mains breakages, in others considerable damage was done by water from broken sprinkler systems. Examples quoted earlier include heads broken by falling ceiling elements, and pipe breakages at seismic movement joints. Other reports we heard, but were unable to verify, were of sprinkler damage to a computer building and to a hospital, the resulting damage costing millions of dollars.

Examples have also been identified earlier of air conditioning equipment falling from ceiling spaces and being dislodged from a roof, as well as the falling of a large roof-level tank from the Amfac Hotel. However, the failures appeared to be in a small minority of installations. Elevators appeared generally to perform well. While some examples were observed of delays in recommissioning, in most cases services were promptly restored once power was available.

8. BRIDGES

8.1 Introduction

Following the earthquake the Applied Technology Council (ATC) and EERI set up a combined reconnaissance group to survey the damage. This group was given permission by CalTrans to visit the sites of all bridge

failures with a view to identifying the causes and reporting on the damage. They formed three field parties to inspect the bridges so that rapid coverage could be provided.

Two members of the NZNSEE reconnaissance team were invited to join the inspection parties and in that way the New Zealand team was given an excellent opportunity to get efficient coverage and a good account of the bridge damage in California.

8.2 General Performance

There are many miles of bridges on the Californian State Highway system varying from small clover leaf ramps, and long concrete and steel viaducts up to the major crossings of the Bay.

Although most of them performed well, they should have, because the earthquake was not large and was centred some 100 km from San Francisco. However, compared to the buildings in San Francisco, the bridges in that region did not fare very well. Not only did some of the older (1950) bridges collapse or sustain severe damage but at least one of the modern reinforced concrete viaducts was badly damaged.

Furthermore, there were some serious problems with the connection of superstructures to substructures for some of the major steel bridge crossings of the Bay.

The estimated cost of restoring the bridges in California up to the pre-earthquake standard is 1.8 to 2.0 billion US dollars.

8.3 I-280 Extension

The I-280 Extension is a new structure on Interstate 280 south of the Bay Bridge in San Francisco.

At one point, where an off ramp crosses over the main viaduct with a continuous box girder, the ramp is supported on a very high bent. At the same point the route crosses China Basin and there is filled ground over a considerable depth of bay mud (possibly up to 25m). Some liquefaction was noted a short distance from the structure but not at the structure itself. However, there is evidence of quite large deflections of the piers relative to the ground (Figs. 90, 91).

At the top of the bent both knee joints have suffered diagonal cracking and spalling. The bent did not collapse but the damage is serious and the main viaduct underneath is closed. The main viaduct is also a continuous structure at this point but with stepped joints at about quarter points in various places. Some of these joints have come to rest further apart than is normal and some of the linkages have been worked very hard. There is occasional evidence of spalling caused by movement of joints.

With the exception of the failed knee joints the damage to this rather recent structure could be considered to be acceptable because it is repairable and

would not necessarily cause closure of the complete freeway.

8.4 I-280 Army Street

Near Army Street the I-280 is carried on an older structure with a section consisting of a double deck reinforced concrete box girder bridge.

At the point where the upper deck joins the lower deck the stiffness of the supports increases abruptly. The first stiff supports attracted an increased share of the lateral loads and have failed.

Also some of the first independent tall supports were close to the lower structure. The two structures swayed out of phase at this point and the lower structure impacted on the columns of the upper structure in some places causing serious shear failures in the columns (Figs. 92, 94).

8.5 I-880 Cypress Viaduct

The Cypress viaduct is a two-level reinforced concrete structure on the Interstate 880 expressway (Nimitz Highway) in Oakland. The bridge was built some 30 years ago, presumably to the standards of the day. It was included in the CalTrans superstructure retrofit programme (Phase I) which consisted in the main of adding cable ties across expansion joints and linking simply supported spans to pier caps. Cypress was known to be deficient in hoop steel (ties) in the columns by today's standards and it was scheduled for substructure retrofitting in the 1990-1992 programme. Whether such a programme would have made any difference would depend on the nature of the retrofit details.

During the earthquake one and a quarter kilometres of the upper deck collapsed onto the lower deck in a slow progression from some point (not necessarily at one end). People had time to stop their cars and get out. Some had parked under bents, believing that they were the safest place. Some of those vehicles were crushed to about 50mm thick (Fig. 95).

There has been speculation that the superstructure retrofit programme caused the failure to be more extensive than it would otherwise have been because it tied the superstructure together across joints, thus ensuring that a collapse of one span would affect the adjacent spans, creating a "domino" effect. However, that theory is somewhat countered by the fact that one span in the middle of the collapsed portion did not fail and the box section had to shear to allow it to stay up.

The bent configurations varied along the structure but the "standard" bent had two legs in both levels and the upper level had three hinges (Fig. 96). Normally the only column fixed end in the top storey was at the bottom on one side or the other. No doubt the lack of redundancy in the top storey did not help but it was not the initial cause of the failure.

The bottom storey columns were always fully continuous with the lower storey beam at

FIGURE 90 I 280 : EXTENSION - CHINA BASIN KING STREET OFF-RAMP PASSES OVER THE I 280 ON A HIGH BENT.

FIGURE 91 I 280 : FAILURE OF THE KNEE JOINT ON THE HIGH BENT SUPPORTING THE KING STREET OFF-RAMP.

FIGURE 92 I 280 : NEAR ARMY STREET WHERE THE UPPER DECK STARTS TO ALIGN WITH THE LOWER DECK. RIGHT HAND COLUMNS ARE SUPPORTED ON LOWER DECK, LEFT HAND COLUMNS ARE SUPPORTED ON GROUND LEVEL.

FIGURE 93 I 280 : NEAR ARMY STREET. TYPICAL SHORT COLUMN SHEAR FAILURE.

FIGURE 94 I 280 : NEAR ARMY STREET. TALL COLUMN POUNDED ON LOWER DECK.

FIGURE 95 CYPRESS VIADUCT.

FIGURE 96 CYPRESS VIADUCT - TYPICAL TWO LEG BENT.

FIGURE 97 CYPRESS VIADUCT - BEAM COLUMN JOINT.

the top and the pile cap at the bottom.

The failure seems to have started as a deterioration of the bottom joint with debonding of the positive and negative moment steel in the joint zone.

Bents swayed laterally creating alternate positive and negative moments at the joint between the bottom beam and the column. Bents near to the collapsed portion display some distress in this region with the bottom steel debonding first followed by the top steel which turns down into the column with a large radius bend finishing up about 300mm inside the outside face of the column (Fig. 97).

It is speculated that a diagonal shear crack then developed on the plane of the debonded negative steel. The concrete column steel had very light ties, including those in the hinge zone. Because of the hinge, the column load from the top level is applied as a point load, and unfortunately, on the wrong side of the shear plane created by the debonding negative moment reinforcing (Fig. 98).

The joint failed in shear and the column kicked out. Once one side had failed the other side was also likely to fail and in most cases it did. The top deck fell on the bottom deck with very little lateral sway or displacement (Fig. 99).

FIGURE 98 CYPRESS VIADUCT - FIRST STOREY JOINT STARTING TO FAIL NEAR THE PLANE OF THE BEAM STEEL TURNING DOWN INTO THE COLUMN.

At the section in the middle of the collapse, where one span stayed up, the pier bents are skewed and therefore longer. The designer introduced a third column in the bottom storey at this point and that reduced the moment demand at the outside beam/column joint (Fig. 100). Presumably this was sufficient to protect the outside joints from debonding of the moment steel in the joint and the onset of joint failure (Fig. 101).

8.6 I-480 Embarcadero Viaduct

The Embarcadero viaduct near the Ferry Building in downtown San Francisco is one of the approaches to the Bay Bridge. It has a lot in common with the Cypress viaduct but there are some differences. The top columns are not tapered for example (Fig. 102).

It has the same hinge configuration on the upper level and similar but not necessarily identical reinforcing details in the lower level beam/column joint.

Examination of that structure reveals that some bents exhibit a crack pattern in that joint which indicates the onset of a similar failure mechanism to that of Cypress (Fig. 103).

FIGURE 99 CYPRESS VIADUCT - FIRST STOREY JOINT EXPOSED SHOWING BEAM STEEL TURNING DOWN INTO COLUMN, PINNED JOINT DETAIL AT BASE OF TOP STOREY COLUMN AND DELAMINATED COLUMN STEEL WITH LIGHT HORIZONTAL TIES.

FIGURE 100 CYPRESS VIADUCT - SKEW SPAN WHICH DID NOT FALL.

FIGURE 101 CYPRESS VIADUCT - UNBONDED PRESTRESS IN THE TOP PIER CAP WAS RELEASED IN THE FAILURE.

FIGURE 102 EMBARCADERO - TYPICAL TWO LEG BENT.

FIGURE 103 EMBARCADERO - CRACKING AT A FIRST STOREY JOINT.

The damage seems to be greatest in the central straight section of the viaduct. This could be for a variety of reasons such as :-

The ground is soft along the water front and includes some reclamation. In the damaged zone there is evidence of considerable ground movement and some liquefaction.

The bridge is curved at each end and the curvature and ramps could be providing additional stability.

It is interesting to speculate on whether the structure is salvageable and if it is, whether it will be. Although it is a vital link, there has been an environmental lobby for its removal because it does not grace the waterfront of downtown San Francisco. It may be that if it could be removed with Federal aid on account of the earthquake, the opportunity might be taken.

8.7 Mococo Ramp

The Mococo Ramp is an off ramp with a tight spiral. It is a flat slab bridge supported by rows of "pile extensions" (Fig. 104).

Some of the shorter pile extensions show signs of yielding in bending at the underside of the slab.

It is possible that some of them have also yielded below ground level and that should be checked when they are repaired.

FIGURE 104 MOCOCO RAMP - SOME SHORT STIFF PILES AT THE OTHER END HAVE STARTED TO YIELD.

Some of the pile extensions at the high level appear to be bent and out of line, however, that is due to construction tolerance. The pattern is too random for it to have been caused by the earthquake.

8.8 Struve Slough Bridges

These are a pair of low level crossings of the slough near Watsonville within 10 miles of the epicentre.

They are 244 m long, in 22 spans of skewed continuous ribbed slabs with expansion joints in about every sixth span. Thus the superstructures were not an effective horizontal beam between abutments.

The substructures were rows of small 400mm diameter pile extensions up to the underside of the slab at each pier. There were pile caps between the beams at the piers. The bridges appear to have moved sideways a considerable amount during the earthquake. Both bridges collapsed, one completely to the ground for half of its length (Fig. 105).

Some piles sheared off at the underside of the pile cap and in places the movement has been sufficient for them to miss the pile cap and punch up through the deck (Fig. 106).

The collapse of the bridge was progressive and a motorist on it at the time felt it settling but was able to drive off.

The poor performance of the bridges may be due to micro-zone effects, the skew, and poor detailing of the pile to pile cap zone.

FIGURE 105 STRUVE SLOUGH - GENERAL VIEW. PILES HAVE SHEARED AT THE UNDERSIDE OF THE PILE CAP/DECK.

FIGURE 106 STRUVE SLOUGH - PILES HAVE PUNCHED THROUGH THE DECK.

8.9 Madrone Drive Overhead

This is a small (3 span) reinforced concrete T-beam bridge which carries the Santa Cruz to Los Gatos State Highway over a local street.

It was recently retrofitted to upgrade its seismic safety by the addition of lead/rubber base isolation bearings.

When such systems are fitted to new bridges large clearances between the superstructure and the abutment backwalls are provided so as to allow the superstructure to "float" on the base isolators.

In this case, because it was an existing structure, a decision was taken to not demolish the existing back walls and replace them with greater clearance until

after they had been damaged in an earthquake.

In this earthquake the back walls were hammered and damaged and presumably the opportunity will be taken to rebuild it with the required clearances.

Although the damaged backwall appears at first sight to amount to a failure it is damage that was predicted and is acceptable performance under the circumstances.

8.10 Grand Avenue On Ramps

These ramps connect the I-80 to the Oakland docklands area and are on another soft ground site. The ramps are typical steel beam bridges on reinforced concrete hammer head piers.

FIGURE 107 GRAND AVENUE RAMP - TYPICAL HAMMER HEAD PIER. NOTE REFLECTION OF PILE CAP AT GROUND LEVEL.

FIGURE 108 GRAND AVENUE RAMP - COLUMN BASE FLEXURAL YIELD JUST STARTING AND COVER CONCRETE SPALLING.

The pile caps are near the ground surface and some show signs of movement relative to the ground (Fig. 107) where there are also signs of liquefaction.

At a location 100 m further east on firmer ground the piers have started to yield at their base and have spalled cover concrete at the corners (Fig. 108). There is no indication that the horizontal ties have been inadequate in this case.

8.11 San Francisco-Oakland Bay Bridge

The Bay Bridge was completed some 50 years ago and it is an impressive series of structures even by today's standards.

It was designed to the best standards of the day and it is known that some account

was taken of seismic forces in the design. However, at that time seismic design was simplistic and based on the application of static horizontal forces to simulate earthquake loads.

We have not seen the design calculations and are not aware of the designers philosophy for distributing horizontal loads from the superstructure to the foundations.

We note that Pier E9 is a trestle type pier whereas the other piers to each side of it for some distance are slender in the longitudinal direction. Thus we assume that Pier 9 is an anchor pier for longitudinal horizontal loads for a considerable portion of the east structure (Fig. 109).

Link spans in the upper and lower deck systems over that pier were provided with sliding seats at the southern end. The provision for movement was small but sufficient to accommodate the end rotations of the truss spans landing on each side of the trestle pier. Those trusses (both sides) were fixed to the pier which is consistent with it being an anchor pier.

The earthquake was centred almost due south of the bridge and the bridge alignment is roughly east-west. Thus it is likely that the predominant response was longitudinal and the damage was consistent with that.

During the earthquake the bridge would have been quite lively. At the anchor pier (Pier E9), the holding down bolts to the pedestals between the eastern truss and the top of the trestle were sheared, one each side. The pedestals slid on the top plate of the trestle, probably oscillating back and forth, and to a lesser extent, side to side. They came to rest 130 mm from their original position in the direction of the road centre line and towards Oakland.

The travel was more than the 15 m deck span sliding supports could cope with and they fell off at the San Francisco end (Fig. 110).

There was some speculation that the top deck span fell first and knocked down the bottom deck span. However the movement was sufficient to drop both spans at the same time and the condition of the support brackets at the sliding end suggest that is what happened.

The bottom span fell onto a transformer housing in the top of Pier E9 and the top deck fell onto the bottom deck. If the transformer housing had not been there the two decks may have fallen right through and done much more damage to the pier.

The fault was not that the deck beam seats were too small but rather that the holding down bolts in the main truss pedestals failed. It is interesting to speculate what would have happened if they had not failed. By failing they changed the response of the structure by providing a simple form of base isolation. This may have protected the pier from greater forces than it could cope with (Fig. 111).

It is also of note that the plan area of the base plate under the pedestal was sufficient to allow considerable movement. Perhaps the holding down bolts were sized to fail at a predetermined load and act as a seismic fuse. That is considered to be unlikely because if it were the case a larger seat for the deck link expansion joints would have been desirable.

Restoration poses some problems. The 130 mm of movement has gone somewhere and it is likely to have been taken up in compression of thermal expansion joints elsewhere in the bridge. That 130 mm should be recovered.

If larger holding down bolts are provided the pier will be asked to carry much larger forces in the next earthquake of equal or

greater intensity.

Since this earthquake was a small one at this site the whole question of the seismic safety of the structure must be re-thought.

It is possible that some form of base isolation between the superstructure and the top of the substructure is indicated. Although the problems of installing such elements are major they are small compared to the consequences of substructure failure in an earthquake. This failure may have been a cheap warning.

8.12 San Mateo - Hayward Bridge

The San Mateo-Hayward Bridge is another major crossing of the Bay and the main spans consist of twin continuous orthotropic steel box girders. It was built in the early 60's.

The superstructure is supported on steel box piers with pinned joints between. During the earthquake the keepers on all of the pins on Pier 21 failed and the pins started to walk out (Fig. 112, 113).

The bridge finished up about 50 mm out of line laterally at that pier and with the pins about 100 mm out of position.

The pins were able to be pushed back into position without having to stop the traffic which was fortunate in view of the closure of the Bay Bridge further north.

The repair work includes a much more robust keeper system.

8.13 Carquinez Bridge

This is a pair of high steel cantilever truss bridge (Fig. 114). It is reported that some (approach) spans fell off their bearings but by the time the team wanted to inspect it they had been jacked up and restored.

8.14 Conclusion

It is tempting to generalise about the bearing failures on three of the large steel bridges which are important links in the Bay region. However, there is not necessarily a connection. The San Mateo bridge is the most recent and seismic design was surely a major consideration. The failure was of a small detail unrelated to the seismic design philosophy and it is not possible to generalise from it.

The failure of the holding down bolts on the Bay Bridge suggest that the forces experienced were greater than the designer anticipated and therefore a complete review of the seismic resistance of this structure is warranted.

Without seeing the details of the bearing failure of the Carquinez Bridge it is not possible to link that failure to the overall seismic design philosophy. However, that bridge also dates from a time when seismic design was not very sophisticated and a complete review may be required.

The concrete bridges in the CalTrans system have been subject to a review of seismic

FIGURE 109 BAY BRIDGE - WEST BAY CROSSING - ELEVATION ON PIER E9.

FIGURE 110 BAY BRIDGE - TOP DECK BEAMS HAVE PULLED OFF THE ANGLE CLEATS ON THE FAR SIDE TRANSOM.

FIGURE 111 BAY BRIDGE - PINNED BEARING TO EASTERN TRUSS LANDING ON PIER E9. FAILED PAINT INDICATES THAT HOLDING DOWN BOLTS HAVE SHEARED AND THE COMPLETE ASSEMBLY HAS SLID ON THE PIER HEAD.

FIGURE 112 SAN MATEO BRIDGE - PINNED BEARING BETWEEN ONE SIDE OF THE BOX GIRDER AND THE TOP OF THE PIER.

FIGURE 113 SAN MATEO BRIDGE - SPANNER ON THE KEEPER NUT ON ONE END OF A BEARING PIN.

FIGURE 114 CARQUINEZ BRIDGE.

safety since the 1971 San Fernando Earthquake. As a result of that review a two-phase programme of retrofitting was undertaken and Phase I was completed in 1988.

Phase I was designed to improve the linkage of spans through construction joints and to substructures. There is much evidence that this work was effective in the Loma Prieta earthquake.

Phase II is intended to tackle the much more difficult problem of deficient and brittle substructures. Pre 1971 columns were known to be deficient in lateral ties and the failure of some bridge columns in the Whittier earthquake of 1987 confirmed this.

The team saw no evidence that the Phase II programme had been started. There is a press clipping dated August 9, 1989 announcing that the Cypress viaduct substructure retrofit would start in 1983 at a cost of US\$7,700,000.

The retrofit was to have included wrapping the columns in steel plate in order to compensate for the deficient ties. The newspaper article makes no mention of any measures that would have solved the deficiency in the joint detail.

The failure of a beam column joint on the recently constructed 280 Extension (China Basin) adds emphasis to the need for attention to this aspect of design.

The collapse of the Cypress viaduct was a tragedy and the near collapse of Embarcadero and I-280 present additional major problems for the highway system. The intensity of the Loma Prieta earthquake was not high in San Francisco. Had it been of greater duration and/or magnitude or the epicentre closer to San Francisco, the damage would have been much greater.

9. LIFELINE SYSTEMS

9.1 General

Performance of the lifeline systems in the San Francisco Bay area during the Loma Prieta earthquake of 17 October 1989 varied greatly, depending on location relative to the epicentre and ground conditions.

Liquefaction and the resulting ground deformation had the most serious effect on lifeline systems, resulting in failure of water, gas, sewer, stormwater, telephone and electricity services, and disruption to roading.

Systems located on or in sands or silty soils performed worse.

Wave action caused by ground movement resulted in some severe damage to water treatment facilities in areas close to the epicentre.

The performance of individual lifeline systems is discussed below.

9.2 Roading

Ground deformation damaged many roads. The effects ranged from minor cracking through to major fissures such as those in Summit Road, south of Highway 17 between San Jose and Santa Cruz (Fig. 16).

In many areas where ground was subject to liquefaction, cracking in roads was noted with a differential movement of up to 50 mm. This was observed in the Santa Cruz/Watsonville area, the Marina District of San Francisco, and Oakland.

Footpaths and kerbing were also severely damaged in the abovenamed locations. Concrete kerbing was broken and displaced. Concrete footpaths were uplifted and cracked and it was observed in the Marina area that at many road intersections concrete paving slabs were uplifted due to ground contraction (Fig. 115 - 118).

Road services were severely disrupted where buildings collapsed over roadways, where emergency services prevented access and at collapsed and failed freeway structures and bridges.

Major roadways were closed due to damage or landslides. In addition to Interstates I80, I280, I480, and I880 in the San Francisco, Oakland area (see Section 8), were Highway 17 between San Jose and Santa Cruz, the Southwest Expressway (a major South Bay artery) and parts of Interstate 280 in San Jose.

9.3 Rail

In the San Francisco/Oakland area minor disruption to rail traffic occurred. The Bay Area Rapid Transit (BART) subway system was shut down immediately after the earthquake. By the morning of 18 October 1989 BART trains were running again.

To ease vehicle traffic on the undamaged bay bridges the BART service was increased on the transbay route to operate 24 hourly. By 24 October 1989 the number of passengers had increased by 50 percent. Trains running on a 24 hour basis began to experience some problems with maintenance of the units.

9.4 Electricity

In many areas electricity supply was cut to consumers shortly after the earthquake hit. Electricity supply was restored to some consumers in areas not badly affected within minutes, but to many, power was not restored for some days. One reason for the delay in restoration of power was the problem of gas leaks. The Pacific Gas and Electric Company (PG & E), suppliers of electricity and gas in the Bay Area, would not turn power back on until gas leaks were repaired and fire risk eliminated.

PG & E reported that the majority of power was restored in four to eight hours of the main earthquake. In Santa Cruz County 67,000 of the 96,000 residents had power restored by late morning of 19 October 1989.

FIGURE 115 DEFORMATION TO FOOTPATH PAVING.
MARINA AREA, SAN FRANCISCO.

FIGURE 116 FAILED FOOTPATH DUE TO GROUND
CONTRACTION. MARINA AREA, SAN FRANCISCO.

FIGURE 117 ROAD PAVEMENT DEFORMATION.
MARINA AREA, SAN FRANCISCO.

FIGURE 118 KERB DAMAGE AND ROAD SETTLEMENT. BETWEEN SANTA CRUZ AND WATSONVILLE.

Three substations were damaged. The Moss Landing Power Plant failed due to liquefaction which resulted in equipment damage. The switching yard and substation were heavily damaged. Buildings at this plant remained structurally sound.

Metcalf Substation east of San Jose was also severely damaged but it was recognised by PG & E that damage would occur in a large earthquake. In preparation for such an event spare parts were stored nearby.

PG & E reported that lesser substations elsewhere were damaged but not to the extent of the two detailed above.

One such substation was the Dolphin Substation located adjacent to the San Lorenzo River in Santa Cruz. Liquefaction in this area caused the three transformers, founded on a concrete pad, to rock. Arcing occurred between adjacent transformers and bushings were lost (Fig. 119). Earthquake restraints at the base of one transformer failed (Fig. 120).

Overall the majority of damage was at transmission facilities and comparatively lesser damage was caused to distribution facilities (Fig. 121).

A major nuclear power plant at Diablo Canyon, about 190 km from the epicentre, suffered no damage. PG & E advised that this plant had been designed for a ground acceleration of 1.2 g.

Eighty-five percent of power was restored to Santa Cruz residents by 10pm on 19 October 1989. A PG & E representative reported that 85-90 percent of power had been restored to Watsonville residents by the same date.

9.5 Gas

Gas supply was a major casualty of the Loma Prieta earthquake. Supplies particularly disrupted were those in areas affected by

liquefaction and other areas where ground deformation was significant.

In the Marina area of San Francisco disruption to supply was mainly caused by the failure of cast iron gas mains.

The Pacific Gas and Electric Company advised that some parts of the gas system are fitted with automatic shut-off valves which respond to a drop in pressure but the majority of the system is not so protected. As a result of this, gas leakage was a considerable problem. Gas leakage heightened the risk of fires in the affected areas and reportedly fuelled the fires in the Marina. Thirty blocks of the Marina area were evacuated on the night of 18 October 1989 by order of the Mayor of San Francisco because of the risk of explosion due to gas leaks.

9.6 Water

As for gas, water distribution was also severely disrupted by the earthquake, particularly in areas suffering large ground deformation.

The San Francisco Water Department (SFWD) reported that all elements of the water supply system (treatment, delivery and distribution) continued operation. The five service reservoirs in the Bay Area remained undamaged. Two of these are situated on the east side of the Bay and three are located on the peninsular. The two filtration plants, one located each side of San Francisco Bay, were also undamaged.

The main supply source for the City and County of San Francisco is the Hetch Hetchy system located approximately 250 km from San Francisco near Yosemite National Park. This is a gravity supply which was uninterrupted by the 17 October earthquake.

The SFWD supplies water from this source on a wholesale basis to other communities

FIGURE 119 DAMAGED BUSHINGS ON THREE TRANSFORMERS OF DOLPHIN SUBSTATION, SANTA CRUZ.

FIGURE 120 FAILED TRANSFORMER RESTRAINT DOLPHIN SUBSTATION, SANTA CRUZ.

FIGURE 121 TRANSFORMERS OFF THEIR TOWER MOUNTING. SUMMIT ROAD LOMA PRIETA.

FIGURE 122 LEAKING SERVICE CONNECTION TO CAST IRON WATERMAIN. NEW TAPPING ADJACENT, MARINA AREA, SAN FRANCISCO.

FIGURE 123 NEW TAPPINGS FOR POLYETHYLENE SERVICE CONNECTIONS ON CAST IRON WATERMAIN. CONNECTION AT LEFT FAILED AT SCREW FITTING. MARINA AREA, SAN FRANCISCO.

FIGURE 124 FIRE HOSES FROM FIRE BOAT PUMPING SEA WATER FROM BAY. MARINA AREA, SAN FRANCISCO.

outside the City and County of San Francisco through 30 suburban wholesale agencies. The distribution systems of these communities suffered some main breaks and power failures but service was essentially maintained.

In San Francisco City only isolated and localised distribution problems occurred. No multiblock areas were seriously affected, except for the Marina area. Within the Marina area between 65 and 70 main breaks and service leaks occurred and over 50 main breaks occurred in the balance of the City. The majority of breaks within the Marina area were on cast iron pipe laid up to the 1930s. Ductile iron pipe was used for water mains from the 1930s. The total length of distribution mains within the City is approximately 1,500 linear kilometres.

Water was restored to the mass care centre in the Marina area by the night of Wednesday, 18 October 1989. By 2pm on Saturday, 21 October 1989, pressure was restored to all but two blocks of the Marina area.

SFWD also reported that no problems were encountered where pipelines connect to structures. Automatic shut-off valves are not incorporated in the City's water supply system.

No problems occurred in relation to water storage in San Francisco. The City has a total of 1,500 Ml of distribution storage and a normal consumption of about 380 Ml/d.

Most materials for repairs were in stock for the SFWD. A special shipment of bolted couplings were required and some spare valves and pipe specials were brought in from Alabama. No shortages occurred which prevented repairs being made.

In the Marina area it was observed that many service connections to water mains had failed due to shearing and bending at the connection. No specific precautions had been taken to prevent this type of failure. The service connections were made at the side of the mains and therefore this failure type is predictable (Fig. 122, 123).

The initial estimated total cost of damage to SFWD facilities was between \$0.5 and \$1.0 million.

It is worthy of note that the San Francisco Fire Department operates a water supply system for firefighting which is separate from the potable water supply system. The fire system consists of two regulating reservoirs pump fed with sea water from the Bay. A fire main broke in Market Street during the earthquake. Because this main was situated in sand the leak was not detected and the two reservoirs emptied. Hydrants on the low pressure potable water system were used to augment the firefighting supply. This separate fire system was installed after the 1906 earthquake for use in the event of a major earthquake or other significant emergency. A fire boat was also used to pump sea water from the Bay as there was insufficient

capacity in the undamaged reticulated pipework (Fig. 124).

In San Jose a number of steel water mains were reported as failing during the earthquake. Very little cast iron or ductile iron mains exist in this area. A representative of the San Jose Water Company advised that the steel pipe was mainly failing at the "bell" (socket) of spigot and socket joints. The pipelines had rubber O-ring joints and it was reported that the bell was cracking due to compression from the spigot end. These were being repaired by cutting out the joint and replacing it with pipe and bolted couplings. Sixty-seven pipe repairs were completed within the first 48 hours of the earthquake restoring water to all distribution tanks and reservoirs. By the morning of Friday, 20 October, 155 leaks had been detected and logged for repair.

Two tanks supplied by the San Jose Water Company were damaged and subsequently taken off line. One of these was a 380 m3 capacity bolted steel tank which buckled at its base. A second steel reservoir developed a leak on the steel inlet/outlet pipe beneath the tank floor. The capacity of this second reservoir was 2.6 Ml.

A coagulation tank at a San Jose Water Company Direct Filtration Plant was made inoperative when its redwood timber baffles were reduced to kindling, reportedly as a result of wave action.

The Rinconada Water Treatment Plant operated by the Santa Clara Valley Water District, San Jose, also suffered damage. The plant is situated approximately 25 km from the earthquake's epicentre and has a capacity of 300 Ml/d with four 75 Ml/d capacity clarifiers.

Three clarifiers were in service when the earthquake hit and all were damaged. The clarifiers were reinforced concrete with radial steel launders. Many of these launders collapsed due to failure of the steel bolted connection to the concrete perimeter walls. The steel central core of each clarifier moved relative to the remainder of the structure and buckling of steel members resulted (Fig. 125 - 128).

The out-of-service clarifier and one other clarifier which suffered lesser damage were able to continue supply at 120 Ml/d.

No other damage was reported at this water treatment plant.

A bulk raw water main which supplies the Rinconada Water Treatment Plant crosses the Calaveras Fault. This main bifurcates from 2.4 m diameter steel pipe to two 1.68 diameter steel pipes where it crosses the fault. The pipe is modified prestressed steel, spigot and socket, rubber ring jointed, concrete lined and coated. A section of this pipeline at the fault line was reported to have failed but details were not available.

In Santa Cruz breaks mainly occurred on cast iron and asbestos cement mains (Fig. 129). Water supply to residents in Redwood

FIGURE 125 DAMAGED CLARIFIER IN SERVICE. RINCONADA WATER TREATMENT PLANT, SAN JOSE.

FIGURE 126 COLLAPSED LAUNDERS IN ONE OF TWO BADLY DAMAGED CLARIFIERS. RINCONADA WATER TREATMENT PLANT, SAN JOSE.

FIGURE 127 FAILED STEEL CLARIFIER LAUNDER SUPPORT. RINCONADA WATER TREATMENT PLANT, SAN JOSE.

FIGURE 128 CLARIFIER INLET PIPES.
RINCONADA WATER TREATMENT PLANT,
SAN JOSE.

FIGURE 129 BROKEN 100 MM CAST IRON
WATERMAIN SANTA CRUZ.

FIGURE 130 EMERGENCY WATER SUPPLY CENTRAL
SCHOOL, LOMA PRIETA.

Estates in the Santa Cruz Mountains area was severely damaged. Two of five reservoirs supplying the area cracked and drained. Water was being supplied to residents either canned by a brewery, or by Tanker (Fig. 130).

In Watsonville the damage was similar to Santa Cruz. Water was being supplied to residents without water from a tanker.

9.7 Drainage

Generally sewer and stormwater systems performed well. It is expected, however, that some pipework failed in those areas which also had gas and water main failures.

In Santa Cruz three main sewers cracked. One was near Schwan Lake and two were near the San Lorenzo River. Sewage from these mains ran into the river which flows through Santa Cruz and into Monterey Bay. The river was temporarily closed for bathing as a result. Repairs were estimated to have been completed by 22 October 1989.

In the Marina area surface damage was observed to cast iron sumps. These suffered badly due to the large amount of ground deformation (Fig. 131, 132).

At the time of preparation of this report the full extent of damage was unavailable.

9.8 Airports

The Oakland International Airport was back in operation by Wednesday, 18 October. Approximately 900 m of one runway was damaged through liquefaction and settling during the earthquake but flights were not affected on the days following the earthquake.

San Francisco International Airport closed for 13 hours immediately after the earthquake due to control tower damage. This damage included loss of some glass from the windows and ceiling tiles which had fallen. The airport was operating at 75 percent terminal capacity by midday of 18 October 1989. Some minor damage occurred to one of the runways.

FIGURE 131 UNDAMAGED CAST IRON SUMP. MARINA AREA, SAN FRANCISCO.

FIGURE 132 CAST IRON SUMP FAILED DUE TO GROUND DISPLACEMENT. MARINA AREA, SAN FRANCISCO.

9.9 Sea

Following the earthquake a passenger ferry service was commenced between the East Bay area and San Francisco. This was required to ease bridge traffic congestion caused as a result of the collapsed Cypress Structure of the Nimitz Freeway and the failed Bay Bridge.

Negotiations were held with ferry operators from both the North Bay and the East Bay to provide reduced fares on ferries running at half hourly intervals and increased to 20 minute intervals during peak commuter periods. Initial plans were made to bring in ferries from West Coast areas to boost the ferry fleet to 45 craft. These craft would transport 60,000 passengers per day. The Navy was to contribute 20 boats, including two amphibious landing craft.

To accommodate vehicle parking on the East Bay for ferry passengers the Navy bulldozed a car park at Alameda.

During the first few days after the earthquake ridership was low on the ferries but by the end of October the number of passengers carried had increased from 2,000 to 7,000 per day.

9.10 Communications

Except for the areas close to the epicentre and the Marina district of San Francisco, the telephone system was generally undamaged. Severe overloading of the system did occur, however, immediately following the earthquake causing disruption to this communication service. To meet the demand on this service Pacific Bell provided cellular telephones to the emergency services of Police, Fire and Ambulance, and to service authorities free of charge. Temporary telephones were provided in areas where domestic services were either damaged or unavailable due to building access restrictions, no charges were made for calls (Fig. 133).

In San Francisco the computer system for communications for the Police and Fire Departments failed due to power cuts and therefore a manual dispatch system had to be used until power was restored.

Television, radio and newspaper services continued to operate in all areas, providing coverage of the aftermath of the earthquake.

10. THE RESPONSE

10.1 Introduction

Some 48 hours after the event a small team comprising Mr Doug Bent (Civil Defence Commissioner, Wellington), Ms Margaret Laird (Wellington Regional Council Civil Defence Officer), and Mr Brian Toomey (Civil Defence Controller Lower Hutt City Council) arrived in San Francisco. The purpose of the visit was to examine the response measures taken from a civil defence viewpoint, in order that appropriate lessons may be obtained for application in New Zealand.

This was the first time that such a team made an overseas visit of this nature, so soon after an earthquake event, and to an area with a societal structure not dissimilar to New Zealand. The team knew of the NZ National Society of Earthquake Engineers representatives already being in the area, and early contact was made, and kept throughout the visit. However, they deliberately worked separately from them, on the basis that they were looking for different information and therefore had to speak to different people in various organisations. (They did think it prudent, though, to elect to stay in the same hotel which they surmised must have been safe enough).

Visits to specific scenes of damage by the C.D. team were almost co-incidental, their objective being to see the management

FIGURE 133 TEMPORARY TELEPHONE FACILITY - FREE SERVICE. MARINA AREA, SAN FRANCISCO.

systems in place rather than examine damage per se. The scenes visited and the associated topics of interest were:-

- The Marina area in San Francisco City, and the welfare aspects of displaced persons and other residents.
- The Pacific Way shopping mall of Santa Cruz City, and that jurisdiction's response headquarters.
- The Nimitz Freeway collapse, and the site control headquarters.
- San Mateo, Watsonville, Boulder Creek, and Santa Cruz, with the various American Red Cross Society facilities at each.
- Blue Ridge Drive, not far from the epicentre, where some discussions took place with affected local residents.
- Region IX of the Federal Emergency Management Agency in San Francisco.
- The Bay Area Regional Earthquake Preparedness Project (BAREPP) office in Oakland City, and the Region II headquarters in Pleasant Hill, both of the California Office of Emergency Services.
- The Office of Emergency Services, the Fire Department, and the Police Department of the City and County of San Francisco.

10.2 People's Immediate Response

Many people who were affected to varying degrees by the earthquake were spoken to. Those driving vehicles typically thought they had a puncture, until they noticed structures moving, whereupon most stopped driving. With some exceptions which one would expect, those in buildings sought shelter as best possible, and those outside tried to avoid falling objects. From questioning it has been concluded that Californians were very aware of the earthquake threat even before 17 Oct, and that generally they know what to do during the shaking. The same cannot be said for New Zealanders in general, or Wellingtonians in particular. It is a Civil Defence intention to more firmly establish this through surveys to be conducted next year.

Most individuals spoke freely of fear but none, even after prompting, described people panicking. This is in keeping with other recent earthquakes around the world. It is thus time to put aside the notion that panic is rampant in an earthquake.

After the initial shaking had stopped, and people had collected their thoughts, a community spirit and desire to help others very quickly developed. Rescue operations by people in the street, and efforts to save threatened property soon got under way, albeit with a little prompting by police etc in some instances. This spontaneous "buddy rescue" accounts for a great majority of successful rescues after an earthquake, and Loma Prieta was no

exception. Fortunately, casualties were light as a percentage of the total population (Some 62 deaths, and about 3400 reported injuries of varying severity in a population of about 6 million).

10.3 Organisational Response

On-duty emergency services crews responded without delay, and within two or three hours off-duty personnel either reported to their stations or were deployed directly to scenes from their homes. Contingency plans for this type of activity were well established, and contributed much to the success of the response efforts by the emergency services. Comments through the media from civic leaders, officials, and members of the public on the way the emergency services put their all into the response were overwhelmingly supportive.

In the United States, each of the four levels of government have their own autonomous emergency services, eg Federal, State, County, and municipalities. On the surface, this would appear to offer the opportunity for confusion and possible conflict of interests. In fact this does not happen because all emergency services departments work very closely day by day and have thus established good working relationships personally, informally, and also through various co-operative mutual support arrangements.

As in New Zealand, the prime responsibility for the emergency, once declared, rests with local government. The elected head of the organisation usually has overall control of the operation, and it is managed in much the same way as a civil defence emergency in New Zealand. If resources to hand at local level are not enough to meet the need, requests for assistance are made to the next higher level of government. These higher level support agencies fulfil a resource management co-ordination function while detailed control remains at local level.

One major difference between the systems in New Zealand and the United States is in the welfare and care of disaster victims. In New Zealand the local authority civil defence organisation manages the welfare function with support from government departments (such as Social Welfare and Housing Corporation) and from voluntary organisations such as NZ Red Cross and Salvation Army. In the United States, this function is largely picked up by volunteer organisations, principally the American Red Cross, with financial assistance given where necessary from Federal sources, largely through the Federal Emergency Management Agency (FEMA). Local jurisdictions have a lesser role in this function.

As a result, the transition from the emergency phase into the recovery and reconstruction phase is somewhat sooner than in New Zealand, although the formal state of emergency remains extant for a similar period.

Local jurisdictions, supported where necessary by other government

organisations, concentrate their efforts on the following functions: rescue, fire suppression, law and order, re-establishing lifelines, and inspection and classification of structures.

(a) Rescue

Most successful rescues were done on the "buddy rescue" basis, by people in the vicinity using whatever materials were immediately at hand. Local fire departments have trained rescue teams, equipped to deal with rescues which require expertise in locating and extricating trapped or stranded victims. On-duty teams deployed quickly, and within two or three hours off-duty personnel had reported and were set to work. The teams were supplemented with specialist expertise such as engineering advice, heat seeking devices, search dog teams, medical support, and heavy machinery for cutting and lifting large obstructions. These specialist or heavy rescue operations are difficult and sometimes dangerous, and the need for engineering expertise for advice and assistance is very real.

(b) Fire Suppression

The weather conditions on the evening of the earthquake were almost miraculously favourable. It was warm so there were very few home heating fires burning, and it was also calm so the danger of fire spreading was somewhat reduced. There were several dozen calls to attend fires in San Francisco, but most were quickly dealt with.

One building fire in particular, in the Marina District, very nearly got out of control. It was gas fed; although gas mains had been shut down it was impossible to shut off the gas to that particular building, thus the residual gas in the mains kept flowing into the blaze. The problem was exacerbated by both the normal water reticulation system in the area, and the one specially installed for fire fighting (after the 1906 event), failing (Section 9.6). The fire was contained, however, by using San Francisco's last remaining (and about to be decommissioned) fire boat, pumping from the marina itself, beyond its rated capacity, through long hoses laid mostly by volunteers at the scene. It was a magnificent example of co-operation by strangers faced with a common danger, working selflessly to overcome it.

(c) Law and Order

Despite initial fears, and some "media hype", incidents of looting were rare and did not pose any particularly serious problem to the police forces. It was no doubt helped significantly by the early setting up of cordons around vulnerable areas.

There were generally two cordons set up. The outer cordon was designed to prevent the general public from entering the area, and was one or two blocks away from the damaged area. Inside this cordon, but outside the inner cordon, authorised media and support agencies were allowed to

establish their facilities. For example, in the Marina District was the Marina Middle School which was used by the American Red Cross as a disaster victim support and accommodation centre, and nearby were the Salvation Army, SPCA, and AAA. About 100m away in the same street were some four transportable satellite TV facilities. Entry into the inner cordon had to be by special pass issued by the law enforcement agencies, restricted to residents, building owners, and authorised workers. We were able to get into these areas using letters of introduction we had with us, and by explaining clearly our reasons for being there.

The cordons were staffed by police, supplemented by National Guardsmen. It was a staff intensive task, requiring considerable organisational effort, and what seemed to be miles of yellow plastic tape and portable barricades.

(d) Re-establishing Lifelines

Descriptions of the damage to lifelines are given in Section 9, but it was clear that maximum effort was quickly put into getting services re-established as soon as possible. Much of the disruption to normal life can be attributed to loss of services and communications links, and many buildings were closed down for this reason rather than structural damage.

The cost of damage to lifelines is one matter, but possibly of greater consequence is the ongoing cost to the community in terms of lost business (several large shopping areas, hotels, and convention centres were closed, and some dozens of manufacturers suffered interruption to production). The loss of the use of major road routes, such as the Bay Bridge, the Cypress structure, and the Embarcadero added to commuting time and costs, and to transportation costs for goods and services. Everything now costs more.

(e) Building Inspections/Classifications

This is a huge task, quite beyond the capacity of the normal engineering and inspection staffs of any local council. Extra staff (hundreds in the case of San Francisco City) need to be quickly gathered together and brought in. The inspection (Section 7.0) determines, firstly the safety of a structure, and secondly its serviceability. One inspection is frequently not enough, especially if further movement is evident or even suspected. The process is thus ongoing and can last for weeks.

In addition to the local authority's requirement for these inspections, there is a demand from building owners and insurance companies for inspections to determine losses and repair or reconstruction strategies. The message is clear - the engineering community at large can expect to be placed under considerable strain for services following even a moderate earthquake event in a metropolitan area.

The question of classification of buildings needs attention. Three broad categories of

classification were applied: Unsafe (no entry), Restricted access (this tends to vary), and Safe to Use. Buildings need to be re-classified as time goes on, and the meanings of the classifications need to be clear, and perhaps modified as the risk of aftershocks diminishes. There are many implications, for engineers, for civil defence, and for both groups collectively.

10.4 Lessons and Recommendations for Civil Defence

From our observations of the response to the Loma Prieta earthquake, we can conclude that although the response structures and methodologies used in California and in New Zealand are different, the end results are the same. We can, however, follow some examples of the Californian approach to better prepare ourselves, and to improve our response to a similar event to Loma Prieta.

Our efforts in improving public awareness in general, and public education in particular need attention. Specific areas include:

- At home:
 - Mitigation measures, structural and non structural
 - Household plan for re-uniting members, and evacuation "getaway kits"
 - Emergency water and food etc
- At work:
 - Mitigation measures
 - Building emergency plans
 - Getting the business back into business
- Anywhere:
 - How to "ride" an earthquake
 - What to do when the shaking stops.

The role of the engineering community in reacting to a damaging earthquake needs examining. Inspections for research purposes are important, and so too are inspections needed for declaring structures safe for use (or otherwise), and to determine what work is needed to render damaged structures useable again. In addition, there will be a need for engineering advice and assistance during difficult rescue operations. The problems of how the required number of engineers should be gathered, deployed, and tasked into an earthquake scene should be addressed now, with appropriate contingency plans drawn up.

11. CONCLUSIONS/RECOMMENDATIONS

11.1 Points made (and remade) by the Loma Prieta earthquake are:

1. The earthquake was of generally moderate intensity in the San Francisco and Oakland area (MMVI-VII), yet localised intensities were up to two steps higher at particularly soft sites. This is attributable to ground conditions, noted prior to the earthquake. Microzoning is therefore important, but zoning itself does not reduce the hazard and appropriate

mitigation steps must also be undertaken.

2. Local variations in earthquake intensity were similar to those observed during the San Francisco earthquake of 1906. The response of structures to the Loma Prieta earthquake is an indicator of potential vulnerability to a larger (expected) earthquake.
3. Significant damage, including collapse, resulted from buildings pounding their neighbours. Building separations were frequently insufficient.
4. Any weakness in building design, construction, and maintenance, including that of secondary elements, increases seismic vulnerability.
5. Unreinforced masonry performed even better than expected, though at least one partial failure caused loss of life.
6. Inadequate seismic resistance of foundations, principally those of domestic buildings, caused the failure of otherwise sound structures. These were not ground failures (except where liquefaction occurred), but rather the failure of the unbraced jack studs between the foundation and the floor.
7. Inappropriate alterations can weaken structural integrity and cause the failure of the building, as did the insertion of garage doors without compensation for loss of bracing in many buildings in the Marina District.
8. Anchoring of items, power transformers through to filing cabinets, if inadequate or non-existent, causes needless loss of function.
9. Piped reticulation e.g. of water or gas, is vulnerable if differential movement is not allowed for and shut off valves not adequately provided.
10. The supply of electric power was shut down in many areas where reticulated gas was feared to have leaked.
11. The severity of the disaster was reduced by contingency planning; including the provision of spare materials, emergency plans and practising of emergency procedures.
12. All lifeline systems need redundancies that have differing exposure to perceived hazards. Note in particular that the two separate reticulated water systems in the Marina District failed and that firefighting there was dependant on a third, portable, system and voluntary labour.

13. Hazards can be identified successfully (e.g. microzoning), but mitigation requires the additional steps of planning and modification of existing and proposed developments.

14. Significant economic impact appears to have resulted from the Loma Prieta earthquake, not only in losses directly due to damage, but also from secondary effects such as the restriction of access to buildings. Operational businesses were forced to close if neighbouring properties were judged a hazard (e.g. Pacific Mall, Santa Cruz).

11.2 The N.Z.N.S.E.E. Response

Fortuitously, the original five members of the reconnaissance team were in place before the Loma Prieta earthquake struck. They have gained rare first hand experience of a strong earthquake in conditions of particular relevance to NZ social and physical conditions.

The augmented team and the NZ Civil Defence team observed the immediate recovery phase, and were actively involved in investigation and assessment of the earthquakes effects. Resources available were of a scale not readily expected in New Zealand, enabling, among other things, access to a wealth of rapidly provided technical information.

On returning to New Zealand the team recommended follow up visits, particularly in connection with lifelines (electrical and pipelines) and building services.

New Zealanders have gained much first hand experience of a strong earthquake from these activities. Dissemination of the knowledge gained began immediately. In particular, with the further assistance of the reconnaissance sponsors and that of the Building Research Association of New Zealand (BRANZ) and the Cement and Concrete Assn of NZ, a series of public presentations on the Loma Prieta earthquake were made by the reconnaissance team in Dunedin, Christchurch, Wellington, Hamilton and Auckland. Technical talks have also been given at the request of interested groups. Technical reports, sourced from California, on various aspects of the earthquake have been collected and distributed on request. This report outlines aspects of the reconnaissance teams observations; further technical reports are expected.

The team's experience of the Loma Prieta earthquake suggests that the NZ National Society for Earthquake Engineering should possibly consider the promotion of earthquake preparedness in addition to its promotion of seismic research and the mitigation of earthquake hazard.

11.3 Conclusion

The Loma Prieta earthquake has shown that current directions in seismic research, engineering, and hazard mitigation are generally correct. It has highlighted the

continuing problem of what to do about existing buildings, bridges, and lifelines that have elements with known poor performance, e.g. hazardous foundation conditions, unreinforced masonry, nonductile concrete, brittle pipe.

The Loma Prieta earthquake was, at its worst, of moderate intensity (MM IX). California and New Zealand have in the past experienced earthquakes with greater intensities, and can expect more. Greater effort is needed at all levels to prepare our societies for the next seismic attack.

12. ACKNOWLEDGMENTS

The team's initial reconnaissance to the USSR was taken on the initiative of the New Zealand National Society for Earthquake Engineering, with the support of the USSR Embassy in Wellington NZ, and the sponsorship of the USSR Academy of Science. Financial support was given by the Society, and particularly by the New Zealand Earthquake and War Damage Commission who sponsored the international travel costs. The Academy of Science met all costs within the USSR.

The employers of all reconnaissance team members supported them with special leave to undertake the project. Thus the DSIR Geological Survey, KRTA Ltd., Wellington Regional Council, Murray-North Ltd., Works Development and Services Corporation Ltd., and the University of Canterbury are thanked for their support.

The reconnaissance of the Loma Prieta Earthquake was additionally supported by the Earthquake Engineering Research Institute, Applied Technology Council and United States Geological Survey who are thanked for assimilating New Zealand members into their response teams and facilitating its activities.

There are very many individuals with whom members of the team spoke and obtained information. These are too numerous to list by name, but their contribution to the experience and knowledge base is acknowledged.

Particular thanks for assistance are due to the following:

K Gibson, H. James and M Laird, Wellington Regional Council for the use of photos;

Staff of the U.S. Geological Survey for their very considerable help; and

the State of California, Office of Planning and Research for permission to reproduce the 1873 view of the San Francisco waterfront from their publication, the "California Water Atlas".

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