

THE LOMA PRIETA EARTHQUAKE OF 17 OCTOBER 1989

A FIELD REPORT BY EEFIT

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CONTENTS

	Page
Acknowledgements	i
Contents	ii
Introduction	iii
S R Ledbetter and R S Steedman	
1.0 Tectonic Setting	1-1
G Woo and R Muir Wood	
2.0 Strong Motion Records	2-1
A M Chandler	
3.0 Performance of Non-Engineered Buildings	3-1
A M Chandler and D G E Smith	
4.0 Performance of Engineered Buildings	4-1
D G E Smith, A M Chandler and A Blakeborough	
5.0 Performance of Bridge Structures	5-1
J M Barr and S C Birkbeck	
6.0 Performance of Industrial Facilities	6-1
P Ford, J Donald, I Morris and P Merriman	
7.0 Geotechnical Aspects	7-1
R S Steedman	
8.0 Geotechnical Phenomena in the Epicentral Area	8-1
A Coatsworth	

INTRODUCTION

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BACKGROUND TO THE EEFIT FIELD INVESTIGATION

EEFIT is a group of engineers, architects and scientists with an interest in earthquakes. It was founded in 1982 with the aim of reporting to the UK and international engineering communities the lessons to be learnt from damaging earthquakes. EEFIT has organised a series of field investigations, each of which has resulted in the publication of a report to disseminate its findings further. The Loma Prieta Investigation involved the largest field team to date and provided a wealth of experience for the engineers who made up the team. This report is a distillation of the team's findings.

The Ms 7.1 Loma Prieta earthquake of 17 October 1989 was the largest earthquake on the San Andreas fault since 1906 and the largest in California since 1952 with its epicentre to the south of San Francisco in the Santa Cruz mountains. Although there was considerable damage over a wide area, ranging from San Francisco and Oakland to the North to Monterey Bay in the South the earthquake duration was only around half the duration that would normally be associated with an event of this magnitude. Ground motions in the Bay Area on alluvial sites were considerably greater than had been expected for such an event and caused considerable damage to engineered structures including large bridges and high rise buildings. There was severe damage to lifeline systems caused by ground failure and a major conflagration in the Marina district of San Francisco following the earthquake was only narrowly averted. News reports from the area quickly confirmed that the disaster would have major engineering significance and the decision to assemble a British team was then automatic. It was decided that an EEFIT study would be the most appropriate means of reporting the disaster to the engineering community in the UK.

THE FIELD INVESTIGATION TEAM

The EEFIT Team consisted of:

Stephen Ledbetter, University of Bath (Team Leader);

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EEFIT members Nigel Hinings (Allott & Lomax) and Robert Muir Wood (then of Principia Mechanica) travelled separately to California.

The EEFIT team was chosen to include bridge engineers, structural engineers, electrical and mechanical engineers, geotechnical engineers and building engineers. The 13 man team comprised five academics and eight engineers from consultancy or industry. The team spent seven days in California beginning on Sunday 29 October (12 days after the earthquake). The team was based in the South of San Francisco and was able to visit sites throughout the damaged area. They reformed daily into groups of 2, 3 or 4 in order to visit the maximum number of sites whilst maintaining an optimum combination of skills and expertise.

The team were greatly assisted by EQE Inc of San Francisco and the Earthquake Engineering Research Institute who advised on damage sites that were worthy of study.

The observations, findings and conclusions of the team are presented in this report as a series of papers on particular topics. The named authors are the principal authors but they have received information, help and guidance from other team members. Other experts such as Gordon Woo (then of Principia Mechanica) generously contributed to the report even though they did not participate directly in the field team. EEFIT are grateful for the assistance given by Jack Pappin (Ove Arup & Partners) and Robin Spence (Cambridge University), who reviewed the report prior to publication.

TECTONIC SETTING

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1.1 BACKGROUND

The 7.1 Ms (6.9 Mw) October 17th 1989 Loma Prieta earthquake was the largest earthquake on the San Andreas Fault since 1906, and the largest in California since 1952. Occurring only 50 to 100 kms from the merged cities and suburbs that fringe the San Francisco Bay, it has provided the most important test of recent building design and modern earthquake engineering. By their infrequency, large earthquakes in highly populated technologically advanced regions have an influence on earthquake engineering far outweighing their size. The Loma Prieta earthquake will provide the foundation for earthquake resistant design for the 1990's, as much for what survived as for what did not. Elevated to such standing, it is vital to know in what seismological ways the event was or was not typical.

Named in 1895 after the linear southeasterly valley, down which the San Andreas creek flowed, it was not until the morning of April 18th 1906 that the San Andreas fault made itself notorious by destroying San Francisco. Since that day, studies of the San Andreas fault have provided much of the basis for the scientific understanding of earthquakes. In 1910, America's foremost geological physicist, Harry Fielding Reid, recognised that the regional crustal movements identified in repeated geodetic surveys reflected continual distortion prior to the sudden elastic release of strain along the fault. His elastic rebound model for earthquake generation remains the starting point for the understanding of the earthquake source, and hence the investigation of earthquake ground motion.

In the 1960's, the significance of the San Andreas Fault was comprehended: a clear (1100 km long) example of a transform fault plate boundary, connecting the young spreading ridge that zigzags down the Gulf of California, to a complex plate triple junction location off Cape Mendocino, northern California. For much of its length, the San Andreas Fault follows an arc of a circle whose centre is the pole of relative motion between the American and Pacific plates. Generally the movement along this plate boundary is horizontal. The sense of the movement is dextral: the Pacific plate moves to the northwest relative to America. The San Andreas Fault has assembled itself along the 'lowest energy' orientation, but there are bends in the line of the fault, formed through interaction with neighbouring faults or because the fault has doglegged between pre-existing lines of weakness. At such bends, the plate boundary invariably becomes partly converted to vertical displacement. Intensive geological investigations of long-term slip rates have shown that the San Andreas Fault does not channel the whole relative plate motion of 56 mm per year, which becomes distributed over other faults to the east. Along its central section, it carries the majority of this motion (37 mm/year), whereas through the San Francisco Peninsula and into the Santa Cruz Mountains, around 15 to 25% of the plate boundary (8 - 14 mm/year) follows this route, Reference 1.

The Fort Tejon earthquake of 1857 is one of the great San Andreas Fault events to have been historically documented. Kerry Sieh (Reference 2) investigated the associated surface rupture, and found that it continued for 40 km through the mountains to the northwest of Los Angeles. Trenching the fault at the site of a drained marsh, Sieh revealed a series of past fault movements, each of which could be dated. With the brevity of California's colonial history, this work provides the best picture (1500 years) of the pattern of earthquake recurrence along the fault. Sieh also investigated the offsets (of streams, landslide scars etc.) associated with the 1857 earthquake, and found that at any one location, the displacement seemed to have been repeated in earlier fault movements. From this observation emerged the idea of the characteristic earthquake: a typical event on a particular fault section, giving rise to a specific regular displacement.

In reviewing information of fault displacements caused by the 1906 earthquake, Thatcher was the first to recognise the significance of the diminution of the displacement from 4-6m to the north of San Francisco, to 3-4m in the north San Francisco peninsula, Reference 3. Southeast of Palo Alto through to the southern end of the 1906 fault rupture, close to the Mission of San Juan Bautista (see Fig 1.1), the displacement again decreased to no more than 1.5m. As improved estimates of the long-term slip-rate became available, it was simple to estimate the date of the next major earthquake on this end of the 1906 fault rupture: as early as 1990, probably by the end of this century. But in 1987, Thatcher and Lisowski reanalysed the local geodetic observations collected after the 1906 earthquake, and claimed that the measured surface fault breaks in this region significantly underestimated (by up to 100%) the actual displacement on the buried fault, and hence that the recurrence interval could be twice as long, Reference 1. The next major earthquake along this section therefore appeared (in 1987) to be still some 60 to 100 years away.

1.2 THE LOMA PRIETA EARTHQUAKE

With hindsight, there was much information already available from which aspects of the 1989 Loma Prieta earthquake could have been predicted. Through the Santa Cruz Mountains, the San Andreas Fault swings about ten degrees anticlockwise out of alignment with the ideal transform fault orientation, and in consequence the fault is placed in compression, the rocks through which it passes converting some of the horizontal movement into uplift. The Santa Cruz Mountains comprise a series of tight, fault-bounded folds, aligned almost parallel with the San Andreas Fault. The highest point, Mt. Loma Prieta, is almost 1200m in elevation and lies close to the centre of the range.

The curious nature of the deformation in the Santa Cruz Mountains was reported in the comprehensive studies of the 1906 earthquake, Reference 4. Among more than 100 published photographs of dextral surface displacement, the two pictures from the mountains alone show sinistral displacement. The only place that the San Andreas Fault was seen to have moved in the mountains was in a railway tunnel, in which the fault-plane dipped at 70 degrees to the southwest. There was also evidence for regional compression, both from within and from outside the tunnel; a section of track even had to be cut from the railway line in the course of repairs. However at either end of the chain of the Santa Cruz Mountains, where the fault swung back to its normal orientation, the topography declined in elevation, and the simple 1906 surface fault trace returned.

Hence the 70 degree dip of the fault beneath the Santa Cruz Mountains had already been established, as had the absence of a surface outcrop of the fault, and the possibility that the Santa Cruz Mountains were a fault segment in their own right. The existence of the mountain range itself implied a vertical component to the faulting. Mountains grow downwards into the lower crust and mantle, four or five times as fast as they rise. Hence young mountains have suppressed crustal geothermal gradients, and consequently brittle faulting can extend to greater depths than is typical of neighbouring regions. In

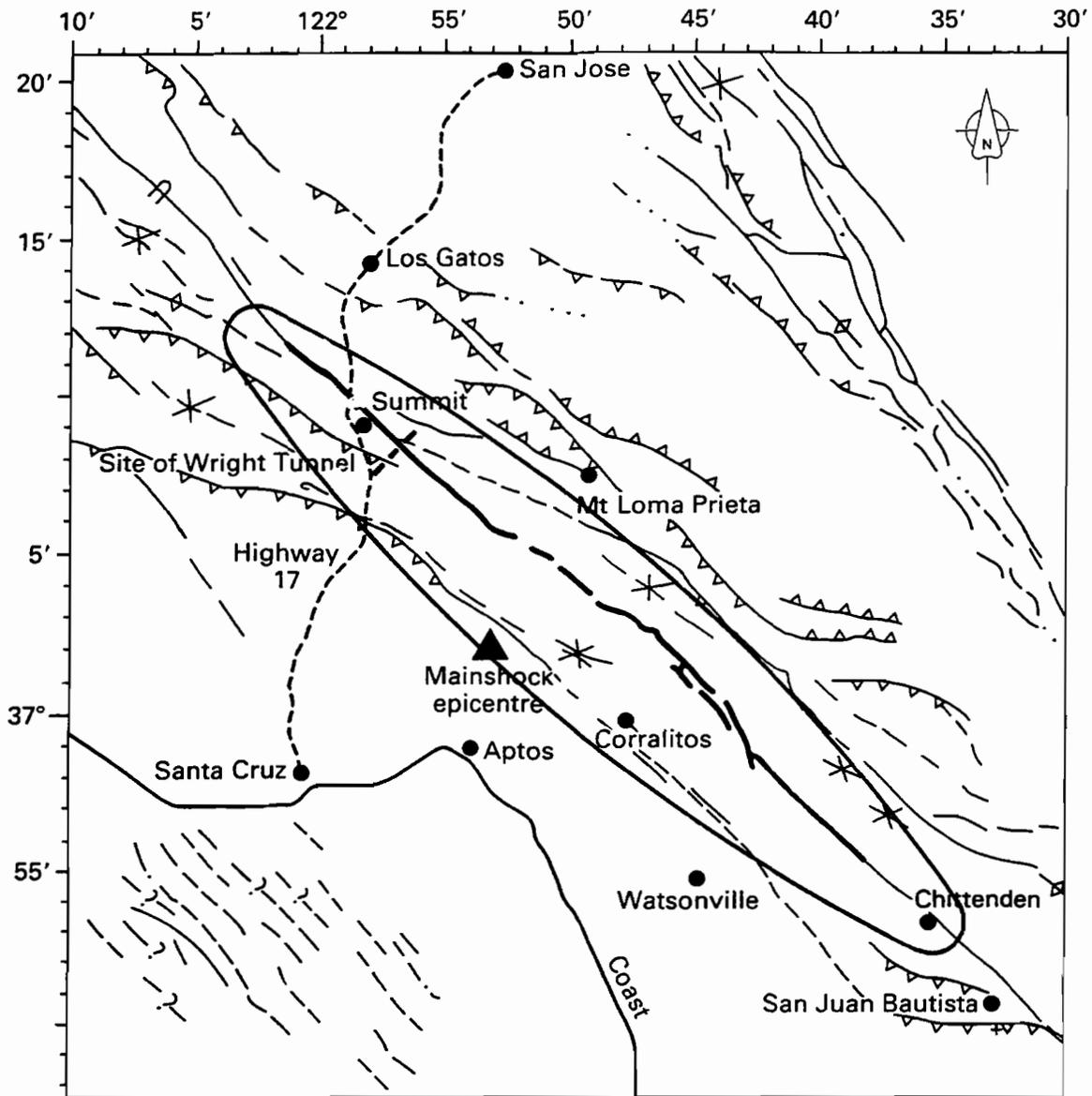
combination with the dip of the fault, this suppressed geotherm has meant that the area of the fault that ruptured, for a given length, is almost 50% greater than a typical section of the San Andreas Fault. It is for this reason that the size of the earthquake anticipated from a breakage along the Santa Cruz Mountains segment, was generally underestimated.

Until ductile processes become dominant, rocks in the crust increase in strength with depth, and so it was unsurprising that the 'preparation zone' of the October 17th 1989 earthquake should be located (at 18 km) beneath the highest point of the Santa Cruz Mountains: Loma Prieta. This in turn is almost dead centre along the line of the range: a fact that significantly influenced seismic ground motion. The fault rupture expanded bilaterally upwards and sideways, until in about six seconds the entire 40 km Santa Cruz Mountains fault segment had broken. Many fault ruptures initiate at the end of a fault segment, and consequently have twice the duration for the corresponding magnitude.

With knowledge of some of the basic seismic source parameters, theoretical seismic wave modelling techniques allow aspects of the pattern of ground motion to be discerned. Close to the rupture, severe ground shaking would be expected asymmetrically on the down-dip side of the fault, from seismic waves radiated along and upwards from the rupture, and the vertical component of motion would be expected to be substantially higher than is usual in Californian earthquakes. These expectations are borne out by the strong motion instrumental recordings in the coastal area covering Watsonville, Santa Cruz and Corralitos (see Fig 1.1). The high vertical accelerations, in some instances well exceeding the peak horizontal accelerations, are particularly noteworthy. Furthermore, in the epicentral region of the Santa Cruz Mountains, topographically amplified accelerations as high as 1g were reported (see Section 2).

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= Main area of aftershocks

Figure 1.1 : Map of epicentral region

STRONG MOTION RECORDS

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2.1 INTRODUCTION

The Ms 7.1 northern California earthquake, which occurred at 17:04 Pacific Daylight Time on 17 October 1989 was centred approximately 15 km north-east of Santa Cruz, California, and 96 km south-southeast of San Francisco (Figure 2.1) in the San Andreas Fault Zone. It was felt as far away as Reno and Las Vegas, Nevada and Los Angeles, California, and was the largest magnitude earthquake centred in northern California since 1906. It also has been reported (References 1,2) to have led to 67 deaths, and more than 2400 injuries. Damage was caused to 18,300 homes and 2600 businesses, and cost approximately \$5.6 billion.

Significant collections of strong motion data from this earthquake have been obtained from the California Division of Mines and Geology (CDMG), Office of Strong Motion Studies (Reference 3) and from the US Geological Survey (USGS) (Reference 4). Strong motion accelerographs at 38 USGS stations located at epicentral distances (measured approximately from the centre of the after shock zone) in the range 27 to 115 km were triggered by the main shock. These stations consist of 21 ground stations, 13 large buildings including 5 hospitals, 2 dams and 2 bridge abutments. Data was also recovered from 73 stations of the California Strong-Motion Instrumentation Program (CSMIP) operated by the CDMG.

The peak horizontal ground acceleration of 0.65g was recorded by CSMIP Stn. 57007 at Corralitos, 5 km from the epicentre, where the peak vertical acceleration reached 0.47g (Table 2.1). A vertical acceleration of 0.66g was recorded at the Watsonville Telephone Building (CSMIP Stn. 47459) at an epicentral distance of 11 km. Several other stations within 35 km of the epicentre recorded peak horizontal and vertical ground accelerations greater than 0.4g (Table 2.1). The Strong-Motion duration varied between 10 and 15 seconds, as shown in the selected accelerograms from the San Francisco Bay Area, plotted in Figure 2.3.

2.2 ANALYSIS OF PEAK GROUND MOTIONS AT SELECTED STATIONS

Table 2.1 lists 22 Strong-Motion recordings from the CSMIP stations, with their associated epicentral distances and peak horizontal and vertical ground accelerations. The orthogonal horizontal components have been designated 00° and 90°, measured in relation to a reference North direction which varied for each recording station according to the orientation of the accelerograph instruments. Hence the data describes the peak orthogonal horizontal ground motions without reference to the true compass directions.

Peak horizontal ground accelerations from 20 selected USGS recording stations are listed in Table 2.2. The data has been plotted on Figure 2.1, where the arrow lengths indicate the magnitude of the peak horizontal accelerations, from two orthogonal components at each recording station. The plotted data indicates that generally the N-S ground motions exceeded these in the E-W direction in the epicentral region (Stns. 1-9 in Table 2.2), whereas the E-W motion was dominant in the northern San Francisco bay area (Stns. 10-20 in Table 2.2). For example, at Stanford University (Stn. 4 in Table 2.2 and Figure 2.1), the peak N-S motion of 0.29g was more than 50% greater than the peak E-W motion (0.19g), whereas at San Francisco Golden Gate Bridge (Stn. 18) the E-W motion of 0.24g was twice that in the N-S direction (0.12g).

The ground motion records at 5 locations in the San Francisco Bay area have been studied in greater detail. The locations of the recording stations are shown in Figure 2.2, which also indicates the simplified geological features of the surface deposits in the Bay area. Three sites have been considered in the city of San Francisco: Presidio (Table 2.1, U), Rincon Hill (Q) and Pacific Heights (T). The Presidio is about 1.5 miles southwest of the Marina district, and 1 mile north of Richmond; in both these areas there was extensive damage to 4 storey wooden framed dwellings as reported in Section 3.2 and 3.3 of this report. The peak horizontal acceleration at the Presidio station was 0.21g, in approximately the E-W direction (260° component). The relatively high value compared with other nearby stations is probably attributable to the presence of relatively soft alluvial marine terrace depositions in this area (Figure 2.2), giving peak ground motions several times larger than at Pacific Heights, where only 5-6%g was recorded. This latter site, together with Rincon Hill (8-9%g) are located on hard rock formations with shallow surface deposits of stiff sand and hence exhibit relatively low amplitude, high frequency motions, as shown in Figure 2.3(a). The Rincon Hill records show a Strong-Motion duration of about 10 sec, whereas at Presidio (Figure 2.3(b)) the duration was about 14 sec, with the motion showing several cycles of low frequency motion.

Soft soil amplification of ground motions is a feature which was observed in several locations in the Bay area, and was considered to be one of the major causes of structural damage to buildings and bridge structures, as discussed in Sections 4 and 5 of this report.

Such amplification was also noted as an important feature of the Mexican earthquake of 1985 (Reference 5, 6), particularly in the Lake Zone of Mexico City where soft soil deposits induced exceptionally high ground motions compared with the nearby bedrock motions which had attenuated to a harmless amplitude of about 4%g. This ground motion amplification was caused largely by one-dimensional resonance effects in the 30-50m superficial layer of soft clay (References 5, 7). These deposits, though unusual are not unique and hence it can be expected that other earthquake-prone sites with similar soil conditions will demonstrate similar ground motion characteristics. The latest (1988) version of the SEAOC (Structural Engineers' Association of California) seismic design code (Reference 8) takes account of this feature by including a site factor $S_0 = 2.0$ (see Section 2.4) for soil profiles containing more than 12m of soft clay. Nevertheless ground motion amplifications of much greater than 2 have been observed both in Mexico city (where the amplification factor compared with bed rock motions was about 5) and also from the evidence gathered to date from the Loma Prieta earthquake of 17 October 1989. This has led to recent concern (Reference 9) that the design forces recommended in SEAOC 1988 are inadequate for structures built on soft sites such as the deep bayshore muds in San Francisco, and that consequently such structures would be under-designed to resist magnitude 8+ earthquakes (which have a return period of about 100 years in the northern California region).

The effect is apparent in the recordings from Oakland, at the outer Harbor Wharf terminal area which is a facility built on reclaimed land overlying Bay Mud (Table 2.1, P and

Figure 2.2) and at a 2-storey downtown office building in the Oakland district which is built on soft alluvium and Bay Mud deposits (Table 2.1, O). The time-histories of the horizontal ground accelerations recorded at these two sites are shown in Figure 2.3(c) and 2.3(d), respectively. Peak accelerations were in the range 0.2-0.3g, representing amplification factors of 3-5 times the base rock accelerations from the sites in San Francisco mentioned above. Strong-Motion durations of about 17 seconds were recorded, with evident low frequency components within 1.5 - 2 miles of the collapsed section of the Interstate I-880 Nimitz freeway (Figure 2.2). The ground motions recorded on alluvium/bay mud at Emeryville, 1 mile north of the collapsed freeway are shown in Figure 2.3(e), with peak horizontal accelerations of 0.22-0.26g (Table 2.2, No. 13). The records again show strong evidence of soft soil, low frequency motions.

Figure 2.4 shows the results of a time-history analysis of the response of a single mass structure with 2% damping, accounting for the orthogonal effects of the two horizontal ground acceleration records from the Emeryville station (Figure 2.3(e)). The displacement response has been plotted with reference to true North (note that the ground motions were recorded in reference directions N10°W and S80°W, Table 2.2). Figure 2.4 shows the responses of structures with natural periods T of 0.6, 0.8 and 1.0 sec, over the time range 0-20 sec (refer to Figure 2.3(e)), where it is noted that the structure is in free-vibration damped motion after T = 15 sec). The peak E-W and N-S structural responses obtained from Figure 2.4 have been summarised in Table 2.3, where the ratio of peak responses in the orthogonal directions is 1.65 for T = 0.8 sec and 1.80 for T = 1.0 sec. These figures compare with a ratio of approximately $0.26/0.22 = 1.18$ between the peak E-W and N-S ground accelerations (Table 2.2, No. 13), and confirm the earlier observation that the structural response is much stronger in general for the E-W direction in the San Francisco/Oakland area. This feature could have been influential in producing strong transverse response of the I-880 freeway deck in the collapsed section in Oakland, which is oriented approximately N-S (Figure 2.2).

2.3 RESPONSE SPECTRA AND PEAK SPECTRAL AMPLIFICATIONS OF RECORDED GROUND MOTIONS

The acceleration response spectra of selected components of the ground motions illustrated in Figure 2.3 have been plotted in Figures 2.5 to 2.8 and Figures 2.10 and 2.11 over the period range 0-3.0 sec. Figure 2.5 shows the spectra for 2% damping generated from the Oakland outer Harbor Wharf (terminal area) records, Figure 2.3(c). The peak spectral amplification factor (that is, the ratio of peak structural acceleration response to the peak ground acceleration) is exceptionally high, being 4.76 for the 90° record (maximum ground acceleration MGA = 0.27g) and 3.47 for the 00° record (MGA = 0.29g), both occurring at a period T = 0.65 sec (see Table 2.4). Spectral amplification of greater than 2.5 occurs in the 90° record over the period range 0.5 to 0.9 sec, and in the 00° record over the range 0.6-1.0 sec. The 90° record also shows a peak at T = 1.5 sec (spectral amplification factor 2.43). The distinctive peak in the range 0.5-1.0 sec in both records is indicative of fundamental mode resonance in the surficial alluvial deposits in the Harbor Wharf area.

Long period spectral amplification is particularly evident in the spectra generated from the Emeryville records, shown in Figure 2.6. The designations E-W and N-S are used to indicate the S80°W and N10°W records, respectively (Table 2.2 and Figure 2.3(e)). The E-W record with MGA = 0.26g has a peak 2% damped spectral amplification of 3.81 at T = 1.5 sec, whilst the N-S record (MGA = 0.22g) has a corresponding peak at T = 1.4 sec, with spectral amplification of 3.12. Both records have distinctive peaks at T = 0.65 sec, with spectral amplifications of 3.48 in the E-W record, and 3.67 in the N-S record. The E-W record in particular shows exceptionally high amplification over the wide period range 0.6-1.6 sec, the response averaging 0.85g (amplification 3.3) in this range. The

amplification for period $T = 1.6$ sec in the E-W record is approximately twice that for the N-S record.

Comparing the periods at which peak response amplification is observed for the Oakland outer Harbor Wharf and Emeryville records, there are clearly two significant ranges in which ground motion energy is concentrated, namely 0.6-0.7 sec and 1.3-1.6 sec. For the Oakland outer Harbor Wharf record the spectral amplification is greatest in the former range, with secondary peaks at the longer periods, whilst the reverse is observed for the Emeryville records. There are two possible explanations for this two-peak phenomenon. Common to both arguments is the fact that periods around 1.5 sec correspond to the fundamental resonance of the soft Bay Mud/fill surface deposits underlain by the stiffer alluvial deposits which are present, for example, as surficial deposits at Presidio, San Francisco (Figure 2.2). The total depth of these deposits and the superimposed Bay Mud/fill layer which exists at the Oakland outer Harbor Wharf and Emeryville sites is about 25-35m, for which the resonant period of about 1.5 sec is appropriate, following Reference 7 and based on typical values of shear wave velocity and bulk density for such materials. The period range 0.6-0.7 sec, at which both records demonstrate another significant spectral peak (Figures 2.5, 2.6), probably corresponds to the resonant period of the surficial Bay Mud/fill layer, which has an average depth of about 6 to 7 metres in these areas (Reference 9). The resonance of this layer is then particularly evident in the Oakland outer Harbor Wharf record (Figure 2.5). The alternative explanation for the (second) resonant peak around $T = 0.65$ sec is that this represents a sub-harmonic of the fundamental period, $T = 1.5$ sec, with a ratio in this case of 2.3 between the two periods. If the natural modes of the soil deposits are assumed to be in the form of standing waves set up in the softer surface layer(s) (Reference 7), then a ratio closer to 3 would be expected between the first and second resonant periods. On this basis the former explanation of the second resonant period seems more plausible in this instance.

The two dominant peaks in the response spectral curves for the various soft soil sites in the Oakland Emeryville area are emphasised in Figure 2.7 which compares the 2% damped response spectra for the Oakland harbor (90°), Emeryville (E-W) and Oakland office (90°) records (the latter is shown in Figure 2.3(d)). All three records have similar durations and peak accelerations (15-17 sec and 0.25-0.27g, respectively). The peak amplification for the Oakland office (90°) record is 3.33 at $T = 0.33$ sec, but also has secondary peaks at $T = 0.65$ sec (amplification 2.11) and $T = 1.4$ sec (2.39). The shape of the Oakland office (90°) spectrum is similar to that of the Emeryville (E-W) record, but with significantly smaller amplification ratios in the critical range $0.6 < T < 1.6$ sec. This is probably attributable to the fact that the record was obtained from the ground floor of the 2-storey masonry/steel office building, and hence unlike the other considered records does not represent a free-field ground motion. The motion has therefore been modified by the effects of structure/soil interaction, with the peak spectral amplification at $T = 0.325$ sec probably occurring at the fundamental natural period of the building, and spectral amplification factors being suppressed at longer periods due to the inertia of the structural foundation.

Figure 2.8 compares the 2% damped response spectra obtained from the Oakland Harbor (90°), Presidio (90°) and Rincon Hill (90°) records. The former has been discussed above, and the soft soil amplification in the period 0.5-1.0 sec is particularly evident when comparing with spectra from stiffer and hard soil sites, such as Presidio (MGA = 0.21g, see Figure 2.3(b)) and Rincon Hill (MGA = 0.09g, Figure 2.3(a)), respectively. The peak spectral amplifications for the Presidio record occur in the range $0.25 < T < 0.65$ sec (averaging 2.5), whilst the Rincon Hill spectrum displays 3 distinct peaks at $T = 0.25$ sec (spectral amplification 2.83), $T = 0.55$ sec (3.02) and $T = 0.8$ sec (2.68). The spectral accelerations for the Rincon Hill and Presidio records peak at 0.27g and 0.67g, respectively; these compare with a peak of 1.29g for the Oakland Harbor record.

The site amplification effect as discussed earlier with reference to the Mexican earthquake (References 5-7) may be defined as the increase in the calculated structural response to a soft soil ground acceleration record, compared with the response to the corresponding bedrock motion. By expressing the structural response in the form of response spectra as in Figures 2.5-2.8, the site amplification can be deduced (for an appropriate level of structural damping, say 2% of critical) as in Figure 2.9 by dividing the response spectra for the Emeryville, Oakland Harbor and Presidio records (Figures 2.7, 2.8) with that obtained from the rock outcrop site at Rincon Hill (Figure 2.8). The site amplification ratio plotted in Figure 2.9 is a measure of the increase of ground motion amplitudes on soft sites, as distinguished from the spectral amplification factors referred to above which are a measure of the structural response in comparison with peak ground (surface) acceleration. Hence on soft sites the two effects are combined when considering the relationship between peak structural response and the bedrock motion of the ground.

The site amplification ratio for the Oakland Harbor 90° record (Figure 2.9) shows three distinct peaks at periods of 0.65, 1.0 and 1.5 seconds. It is clear from Figure 2.7 that the intermediate period, 1.0 seconds, does not correspond to a peak in the Oakland Harbor 90° record and hence the explanation for the apparently high amplification (6.4) at this period is the exceptionally low value of the Rincon Hill 90° spectrum (Figure 2.8). The peaks of the site amplification graphs occurring at $T = 1.0$ seconds, which appear in all three records shown in Figure 2.9, are therefore considered to be spurious and do not correspond to resonance of the softer, surface materials as discussed earlier. Resonance effects do however lead to the peaks in the site amplification curve for the Oakland Harbor 90° record at $T = 0.65$ and 1.5 seconds, with values of 10.9 and 5.2 respectively. The same periods produce peaks in the Emeryville (E-W) site amplification spectrum, with ratios of 7.6 and 7.9 respectively. The results for the stiffer site at Presidio (90° record) also show a peak at a period of 0.7 seconds (amplification 3.2), but as expected the primary peak is at the lower period of 0.38 seconds, where the site amplification ratio is 5.7 (Figure 2.9). Hence there is a consistent relationship amongst these three records which reinforces the argument that site effects were of great significance in the ground motion records obtained from the San Francisco Bay regions, with a high proportion of damage occurring in those areas where the effect was most noticeable (see Figure 2.2, together with damage surveys given elsewhere in this Report, as well as in References 1 and 2).

Even at short periods (< 0.5 seconds) and at long periods (> 2.0 seconds) the site amplification averages about 2-3 (Figure 2.9), and hence exceeds the maximum factor of 2.0 provided in the SEAOC (Structural Engineers' Association of California) design recommendations, Reference 8, as discussed in Section 2.2. In the intermediate period range (0.5-2.0 sec), in which the majority of medium-rise buildings have their fundamental natural periods, the amplification for the softer sites at Oakland Harbor and Emeryville averages about 5.0 and 5.9, respectively. These results apply to structural response spectra computed for 2% damping. Corresponding results for 5% damping indicate that the site amplification is not very sensitive to the structural damping employed in the calculation, as expected. In the period range 0.5-2.0 seconds, the average site amplification ratios for the Oakland Harbor and Emeryville records are 4.7 and 5.8, respectively. These values are only 6% and 2% lower than those quoted above for more lightly damped structures.

The effect of higher damping (5%) on the spectra for the Oakland Harbor (90°) and Emeryville (E-W) records is shown in Figures 2.10 and 2.11, respectively. In the former case the peak spectral acceleration at $T = 0.65$ sec still exceeds 1.0g (amplification 3.81), whilst in the latter case the peak occurs at $T = 1.2$ sec, with spectral amplification of 2.98. The Emeryville record displays average spectral amplification of 2.7 in the critical period range 0.6-1.6 sec.

Table 2.4 summarises the most significant spectral data from Figures 2.5-2.10, together with other records from Figure 2.3 whose spectra have not been illustrated. In each case the data is presented for both 2% and 5% damping. The mean peak spectral amplification factor across all 10 component records is 3.66 for 2% damping and 2.72 for 5% damping, with corresponding median values of 4.34 and 2.66 (Table 2.5). These values can be compared with the recommendations of Newmark and Hall (Reference 10), who carried out a detailed statistical study of a large number of strong-motion earthquake records from the western US. The resulting design spectral curves indicate that the peak amplification for damping of 2% and 5% should be taken as 2.73 and 2.12, respectively, for the median response (i.e. 50% probability of exceedance). The corresponding values from Newmark and Hall's study for median plus one standard deviation response (16% probability of exceedance) are 3.66 and 2.71 for 2% and 5% damping, respectively (Table 2.5). Eurocode 8 (Reference 11) recommends that peak amplification values of 3.95 (2% damping) and 2.50 (5% damping) should be used, with a stated probability of exceedance of 20-30%.

Comparing the values quoted in Table 2.5, it is clear that both the median and median plus one standard deviation amplification factors from the 10 records considered from the Loma Prieta earthquake (Figure 2.3) exceed significantly the codified recommendations, with the exception of the Eurocode 8 value for 2% damping which gives a reasonable estimate, taking the stated probability of exceedance of 20-30% (i.e. just less than the median plus one standard deviation response). It is interesting to note that the mean spectral amplification obtained for the Loma Prieta earthquake records for both 2% and 5% damping is in exact agreement with the corresponding Newmark and Hall recommendations for 16% probability of exceedance. It must be remembered, however, that in the former case only 10 records have been considered whereas Newmark and Hall used over 90 western US records in their study. Despite this cautionary note, the Loma Prieta earthquake records have demonstrated that in certain cases very large spectral amplification factors (approaching 5 for 2% damping and 4 for 5% damping, Table 2.4) are observed, particularly for records from medium-stiff or soft soil sites. These amplification factors lead to very large spectral accelerations: in several cases values around 1.0g have been noted, and in the case of the Oakland Harbor (90°) record the exceptionally high value of 1.3g was recorded. This has some serious implications for the required level of design forces for buildings in the northern California region, which are discussed in relation to existing codified provisions in Section 2.4 below.

2.4 COMPARISON OF RESPONSE SPECTRA WITH CALIFORNIA CODE DESIGN RECOMMENDATIONS

The SEAOC - 1988 seismic code (Reference 8) recommends minimum design lateral forces in terms of the base shear coefficient V/W , where V is the total lateral force or shear at the base of the structure and W is the total seismic dead load. The ratio V/W is given by

$$\frac{V}{W} = \frac{ZIC}{R} \quad (2.1a)$$

$$\text{where } C = \frac{1.25 S_0}{T^{2/3}} \quad (2.1b)$$

The lateral force coefficient C is a function of the fundamental period of vibration, T (in seconds) of the structure in the direction under consideration, as in Eqn. (2.1b). Limiting values on C are 2.75 (upper limit, without regard to soil type or structural

period) and $0.075R$ (lower limit), where R is a coefficient accounting for the type of structural system resisting the lateral forces. The zone factor Z (Eqn. 2.1(a)) has 4 levels, depending on the seismicity of the region. The level of highest seismicity (Zone 4) is used for California, where $Z = 0.40$. The importance of a building in terms of post-earthquake recovery is accounted for in SEAOC - 1988 by an importance factor I (Eqn. (2.1a)), which increases the level of base shear applied to essential and hazardous facilities by 25%. For special and standard occupancy structures (Reference 8), I is taken to be 1.0.

The soil factor $S_0 = 1.0, 1.2, 1.5$ or 2.0 , where the factor 1.0 applies to rock or stiff soil conditions, and for stable deposits of sands, gravels, or stiff clays of a depth less than 60m. The factor $S_0 = 1.2$ applies to deep cohesionless or stiff clay soil conditions exceeding 60m in depth and where the soil types overlying rock are stable deposits of sands, gravels, or stiff clays. The factor $S_0 = 1.5$ applies to soft to medium stiff clays and sands. As mentioned in Section 2.1, following the 1985 Mexico city earthquake (References 5-7) a soil factor of $S_0 = 2.0$ was added for soil profiles containing more than 12m of soft clays.

This mirrored the changes to the Mexican earthquake design code introduced as Emergency Regulations immediately after the 1985 earthquake (see Reference 5), which increased the lateral forces for mid-to high rise structures with high overall ductility by factor of up to 2.5. SEAOC 1988 (Reference 8) also proposes that further research on the topic of ground motion amplification for soft sites be carried out, leading to the eventual replacement of the soil factor S in the lateral force requirements by site-specific design spectra. Although there has been a great deal of study and research on the topic of regional ground motions, modified to take account of the geologic and stratigraphic conditions pertaining to the site, it must be considered a somewhat controversial matter. Nevertheless, it is clear from observations that the type of faulting, the regional geology, the local site conditions and the nature of the structure all have a major influence on the motions that are experienced by the structure. Studies of the nature of the motions on sites displaying significant longer period components are summarised in Reference 12, in terms of the response spectra associated with the measured records at various sites. As Nuttli (Reference 13) has indicated, long period motions are clearly of major significance in some sites in the Western United States for large earthquakes, and this is confirmed by observations earlier in this report from the recent Loma Prieta earthquake in northern California.

The base shear coefficient V/W (from Eqn. (2.1a)) has been plotted in Figure 2.12, taking $Z = 0.4$ (zone of highest seismicity) and $I = 1.0$ (non-essential or hazardous facility). In Figure 2.12(a), the structural system factor $R = 8$, which is applicable to building frames with concrete shear walls, or concentric braced frames of steel or concrete (Reference 8). For ductile special moment-resisting space frames (SMRSF) in steel or concrete, or dual systems consisting of concrete shear walls with a SMRSF, the factor $R = 12$ as in Figure 2.12(b). In both cases, the comparison is made for the three soil types using $S_0 = 1.0, 1.5$ and 2.0 , as defined above.

The maximum seismic base shear coefficient V/W is specified by SEAOC to be 0.138 for $R = 8$, and 0.092 for $R = 12$. In both cases the minimum seismic coefficient is taken to be 0.03. The maximum design forces are applicable to period T , where $T = 0.87$ sec for the softest soil type, $S_0 = 2.0$. For $S_0 = 1.0$ and 1.5 , the corresponding periods are $T = 0.31$ sec and $T = 0.56$ sec, respectively (Figure 2.12).

SEAOC also gives guidance as to the design response spectra. Figures 2.7 and 2.8 show the resulting code spectra plotted with the observed spectra. As can be seen the soft soil sites (Figure 2.7) experienced levels of ground motion similar to the code design levels. Figure 2.8 compares the response spectra from a very stiff site (Rincon Hill), a medium

stiff site (Presidio) and a soft soil (Oakland Harbor) site with the code spectra. As can be seen the very stiff and medium stiff sites experienced ground motions considerably smaller than those required by the SEAOC code. This is not surprising for an earthquake of magnitude 7 at an epicentral distance of 100 km. The design spectra correspond to a larger and closer earthquake than that experienced in the San Francisco, Oakland area during the Loma Prieta earthquake. What was surprising, however, was that the ground motions experienced on soft soil sites were so high. This implies that if this area experiences an earthquake leading to the design level ground motions on very stiff and medium stiff sites then the soft soil sites are likely to experience ground motions well in excess of the code requirements.

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Table 2.1: Data Recovered From Selected CSMIP Stations [3]

Station Name	CSMIP Stn No	Epicentral Distance (km)	Peak Ground Acceleration (g)		
			H		V
			00°	90°	
A Corralitos	57007	5	0.64	0.50	0.47
B Watsonville: Tel.Bldg	47459	11	0.28	0.39	0.66
C Capitola	47125	14	0.54	0.47	0.60
D Gilroy #1	47379	21	0.43	0.50	0.22
E Santa Cruz	58135	23	0.47	0.44	0.40
F Gilroy #3	47381	24	0.55	0.37	0.38
G Saratoga	58065	35	0.53	0.34	0.41
H Hollister	47524	40	0.38	0.18	0.20
I San Jose: SC County Bldg	57357	40	0.11	0.10	0.10
J Foster City	58375	70	0.26	0.29	0.11
K Hayward: Muir School	58393	77	0.18	0.14	0.10
L San Francisco (SF): Airport	58223	87	0.24	0.33	0.05
M South SF: Sierra Pt Over	58536	91	0.09	0.05	0.06
N South SF: 4-storey hospital	58261	93	0.14	0.15	0.08
O Oakland: 2-storey office	58224	99	0.21	0.25	0.16
P Oakland: Outer Harbor Wharf	58472	101	0.29	0.27	0.07
Q SF: Rincon Hill	58151	102	0.08	0.09	0.03
R Yerba Buena Island	58163	102	0.03	0.06	0.03
S SF: Telegraph Hill	58133	104	0.06	0.08	0.03
T SF: Pacific Heights	58131	104	0.06	0.05	0.03
U SF: Presidio	58222	105	0.10	0.21	0.06
V Treasure Island	58117	105	0.11	0.16	0.02

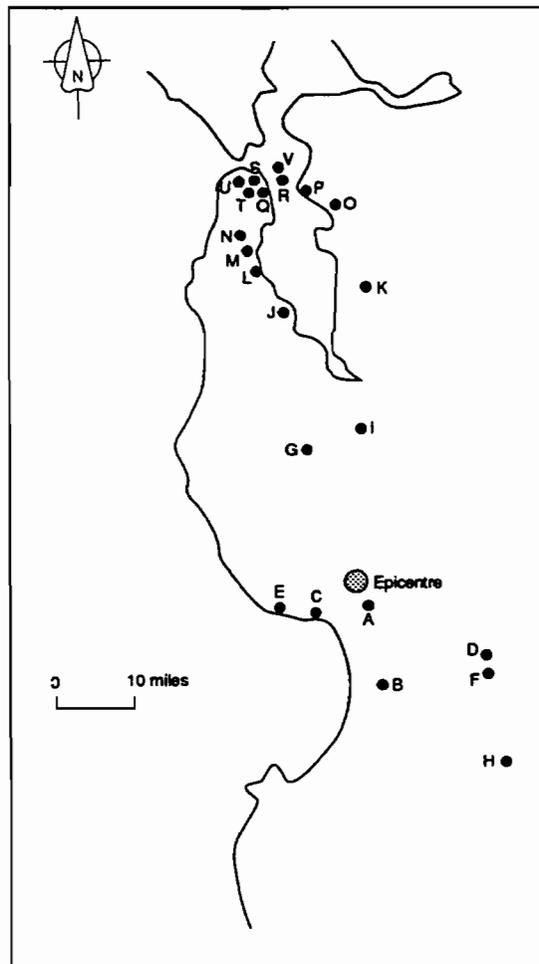


Table 2.2: Peak Horizontal Ground Accelerations, USGS Stations [4]

Station Name	CSMIP Stn No	Epicentral Distance (km)	Peak Horizontal Ground Motions			
			(g)	dir°	(g)	dir°
1 San Jose Interchange: 101/280/680 Abut	1571	34	0.18	322	0.13	232
2 Calaveras Array: Cherry Flat Res	1696	42	0.09	360	0.07	270
3 Sunnyvale: Colton Ave	1695	43	0.22	360	0.19	270
4 Stanford University	1601	51	0.29	360	0.19	270
5 Menlo Park	1230	54	0.12	110	0.27	020
6 Fremont	1686	56	0.15	180	0.20	090
7 Apeel Array: Stn 9	1161	62	0.11	227	0.12	137
8 Foster City	1515	66	0.12	360	0.11	270
9 Hayward: Muir School	1121	72	0.13	054	0.16	324
10 San Francisco (SF) Shafter St	1675	89	0.11	360	0.07	270
11 SF: State University	1116	93	0.14	270	0.11	180
12 SF: Market Street	1446	96	0.08	135	0.11	045
13 Emeryville	1662	97	0.22	350	0.26	260
14 SF: Montgomery St	1239	97	0.12	261	0.11	171
15 Berkeley: UC	1005	98	0.04	135	0.08	045
16 Berkeley: Shattuck Av	1103	99	0.09	261	0.11	171
17 SF: VA Hospital	1225	100	0.08	185	0.16	095
18 SF: Golden Gate Br	1678	100	0.12	360	0.24	270
19 Richmond: Bulk Mail	1439	101	0.08	057	0.11	327
20 Larkspur: Ferry Term	1590	105	0.10	360	0.14	270

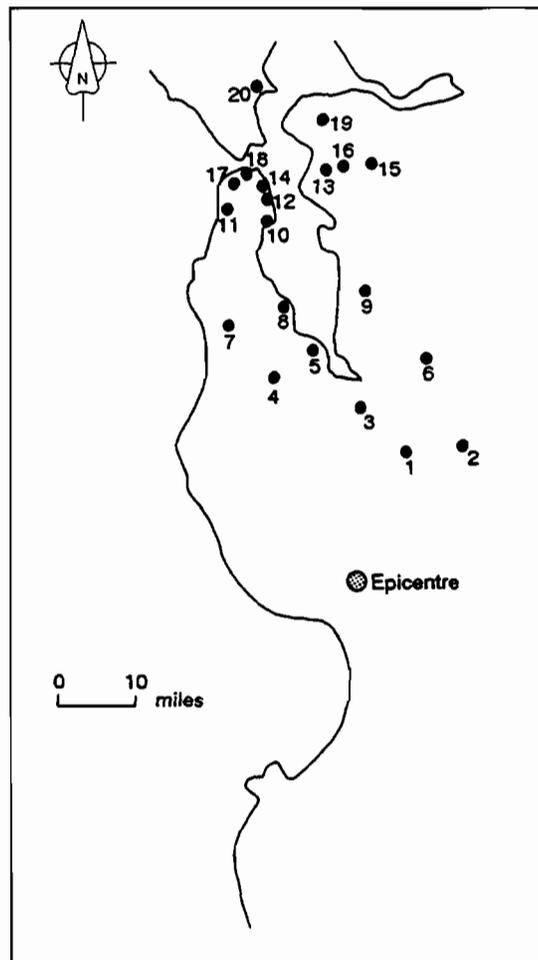


Table 2.3: Peak Displacement Responses to Emeryville Ground Motion Record

Natural Period (s)	Peak Displacement Responses (cm)		
	E-W	N-S	Ratio
0.6	5.4	4.8	1.13
0.8	12.4	7.5	1.65
1.0	21.2	11.8	1.80

Table 2.4: Peak Ground Motions and Spectral Accelerations

RECORD	Comp	MGA (g)	2% damping		5% damping		Period (s)*	
			S _a (g)	Peak SAF	S _a (g)	Peak SAF	2%	5%
San Francisco	90°	0.09	0.27	3.02	0.19	2.16	0.55	0.55
Rincon Hill	00°	0.08	0.25	3.13	0.17	2.11	0.95	0.60
San Francisco	90°	0.21	0.67	3.19	0.57	2.72	0.48	0.48
Presidio	00°	0.10	0.49	4.90	0.34	3.40	0.75	0.70
Oakland Outer Harbor	90°	0.27	1.29	4.76	1.03	3.81	0.65	0.65
Wharf	00°	0.29	1.01	3.47	0.78	2.68	0.65	0.65
Oakland 2-Storey	90°	0.25	0.83	3.33	0.52	2.07	0.33	0.48
Office Building	00°	0.21	0.69	3.27	0.55	2.64	0.30	0.30
Emeryville	(260°) E-W	0.26	0.99	3.81	0.77	2.98	1.50	1.20
	(350°) N-S	0.22	0.81	3.67	0.58	2.62	0.65	0.65

MGA = Maximum Ground Acceleration
 SAF = Spectral Amplification Factor
 S_a = Spectral Acceleration

* Peak SAF

Table 2.5 Peak Spectral Amplification Factors from Loma Prieta Earthquake, 10/17/89, and Comparison with Newmark and Hall [10] and Eurocode 8 [11] Recommendations

	2% damping		5% damping	
	50%	16%	50%	16%*
Loma Prieta 10/17/89	3.40 (mean 3.66)	4.06	2.66 (mean 2.72)	3.22
Newmark and Hall	2.73	3.66	2.12	2.71
Eurocode No 8	3.95 [†]		2.50 [†]	

* probabilities of exceedance: 50% = median response
 16% = median plus one standard deviation response

† probability of exceedance = 20-30% for Eurocode 8

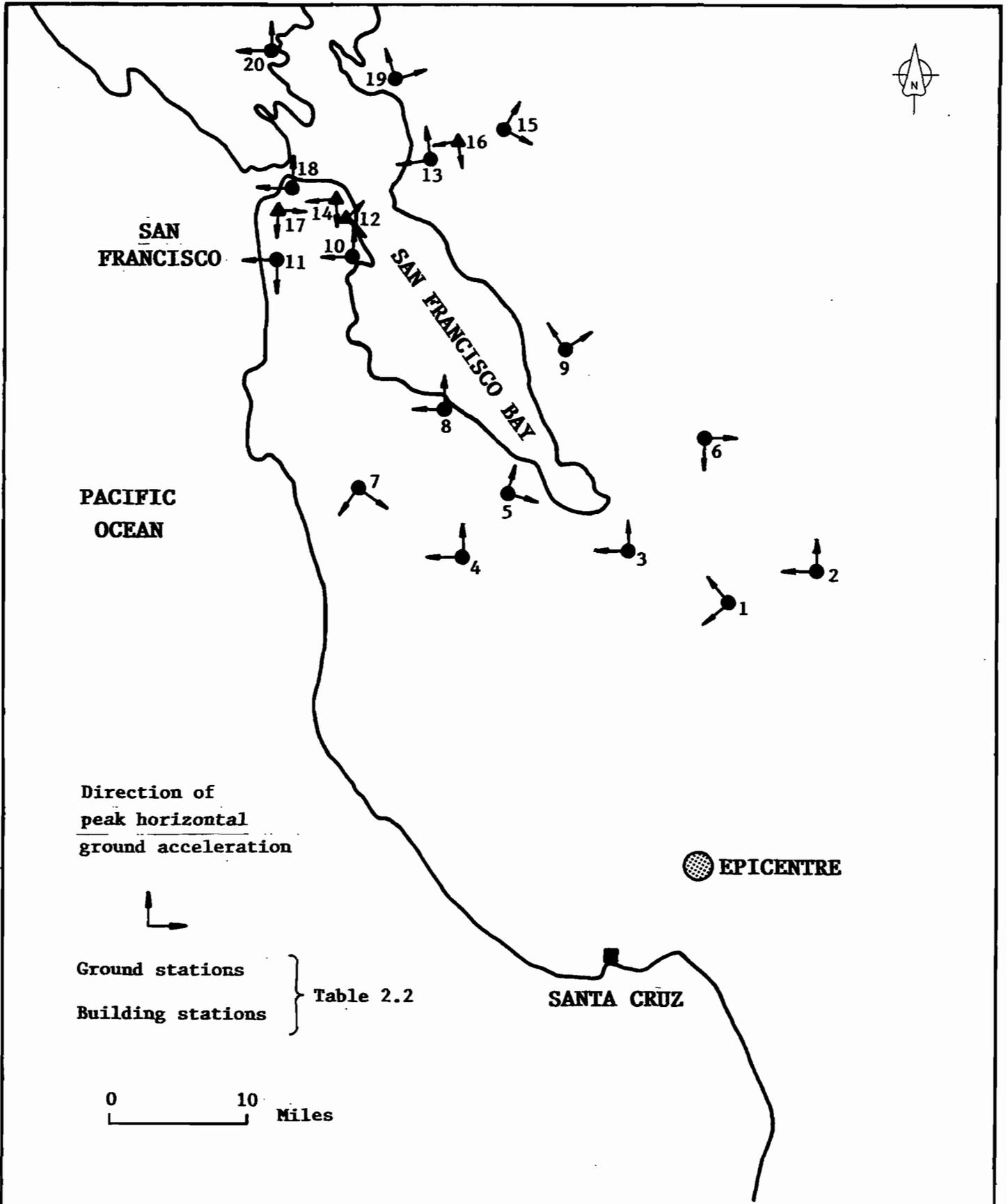


Figure 2.1 : Location map showing 20 USGS recording stations [4], Table 2.2 together with peak horizontal ground accelerations

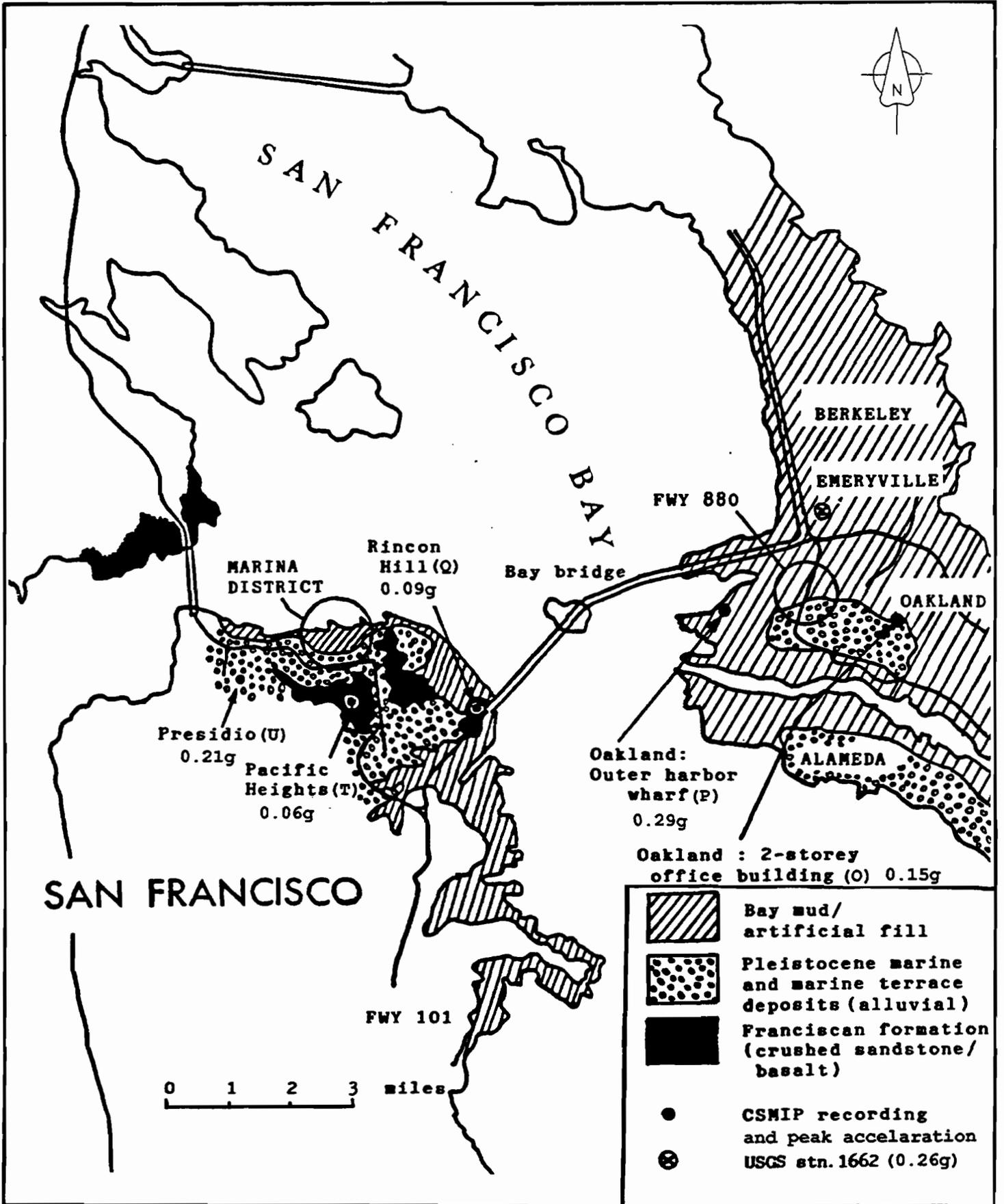


Figure 2.2 : Map of San Francisco Bay area showing surface geology and soil conditions, together with the locations of key earthquake recording stations

Loma Prieta Earthquake

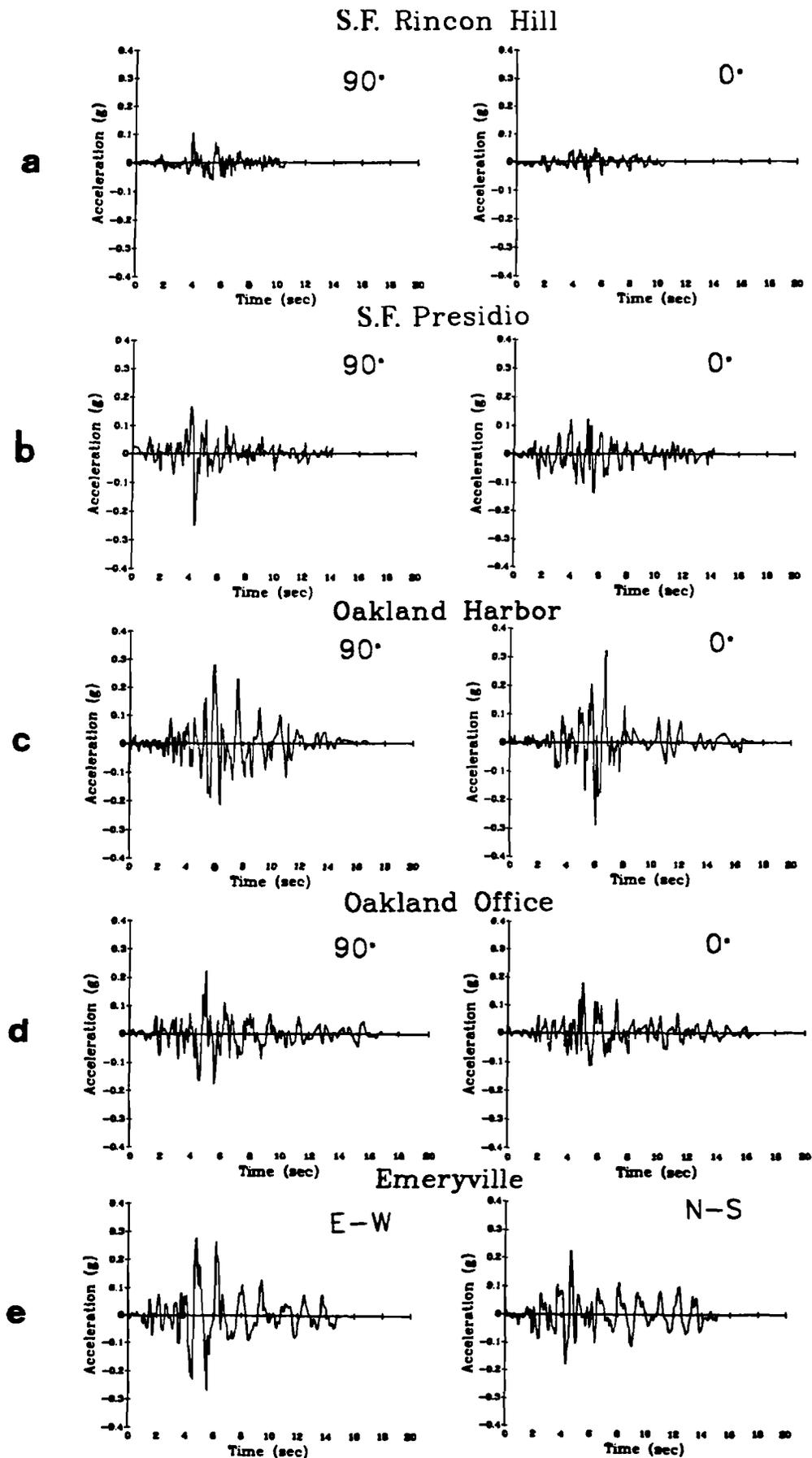


Figure 2.3 : Earthquake time-histories recorded at key sites in San Francisco (S.F.), Oakland and Emeryville

Emeryville Displacement Traces

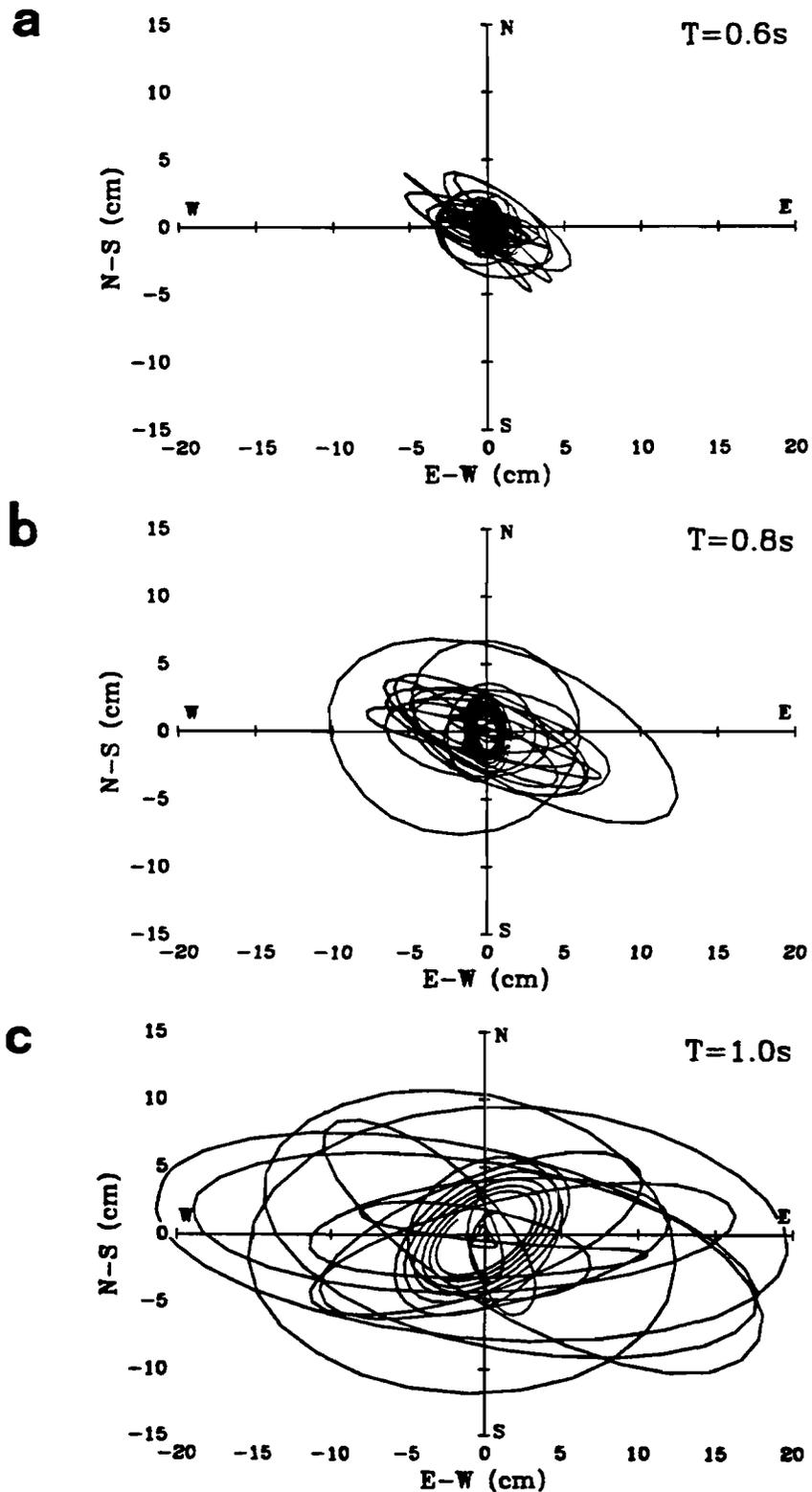


Figure 2.4 : Displacement response time-histories for single mass systems with natural periods T of 0.6 sec (a), 0.8 sec (b) and 1.0 sec (c); 2% damping

Loma Prieta, CA Earthquake 17OCT89

Acceleration Response Spectra
: 2% damping

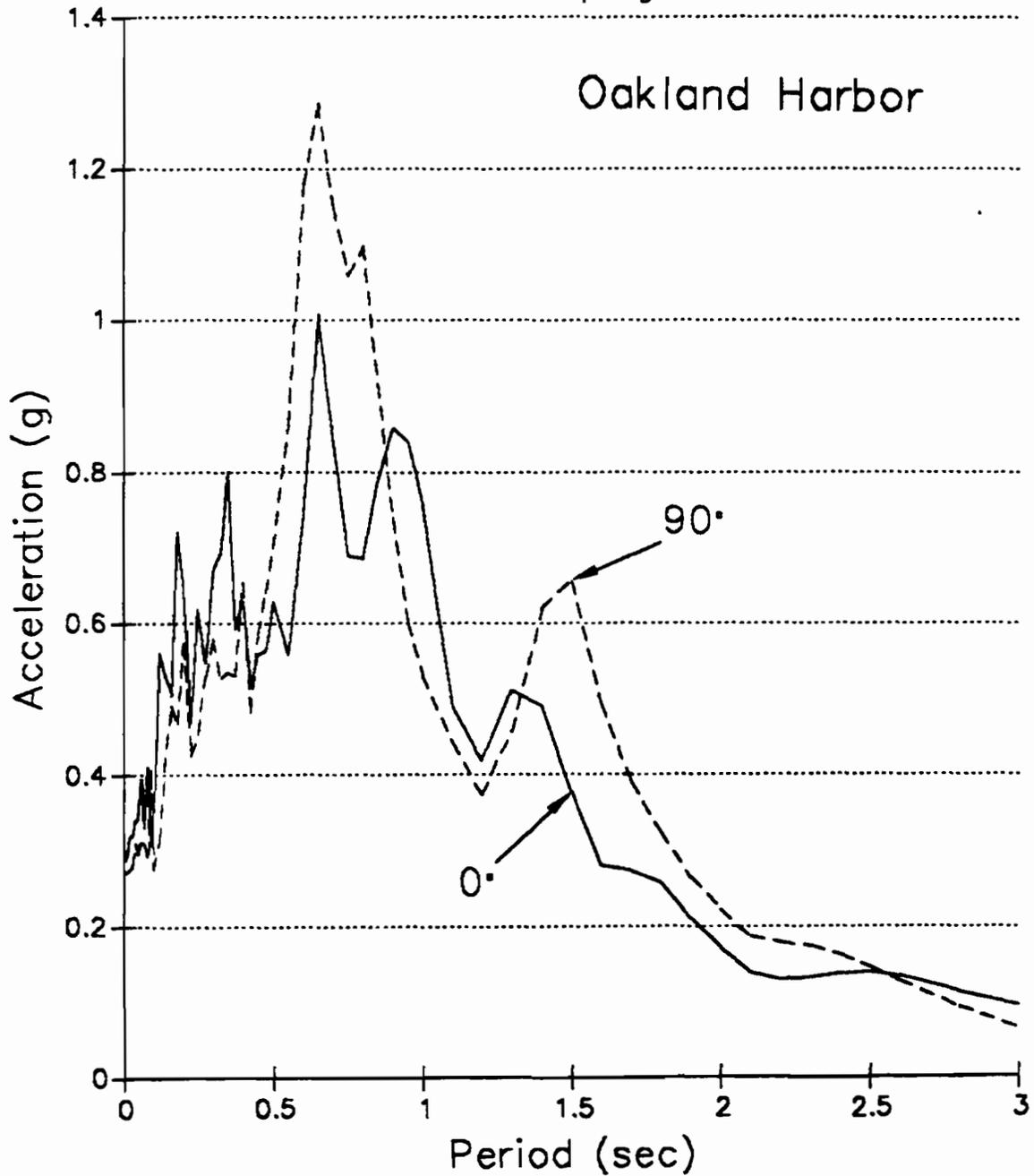


Figure 2.5 : Acceleration response spectra for the horizontal components of the Oakland Harbor Wharf record

Loma Prieta, CA Earthquake 17OCT89
Acceleration Response Spectra
: 2% damping

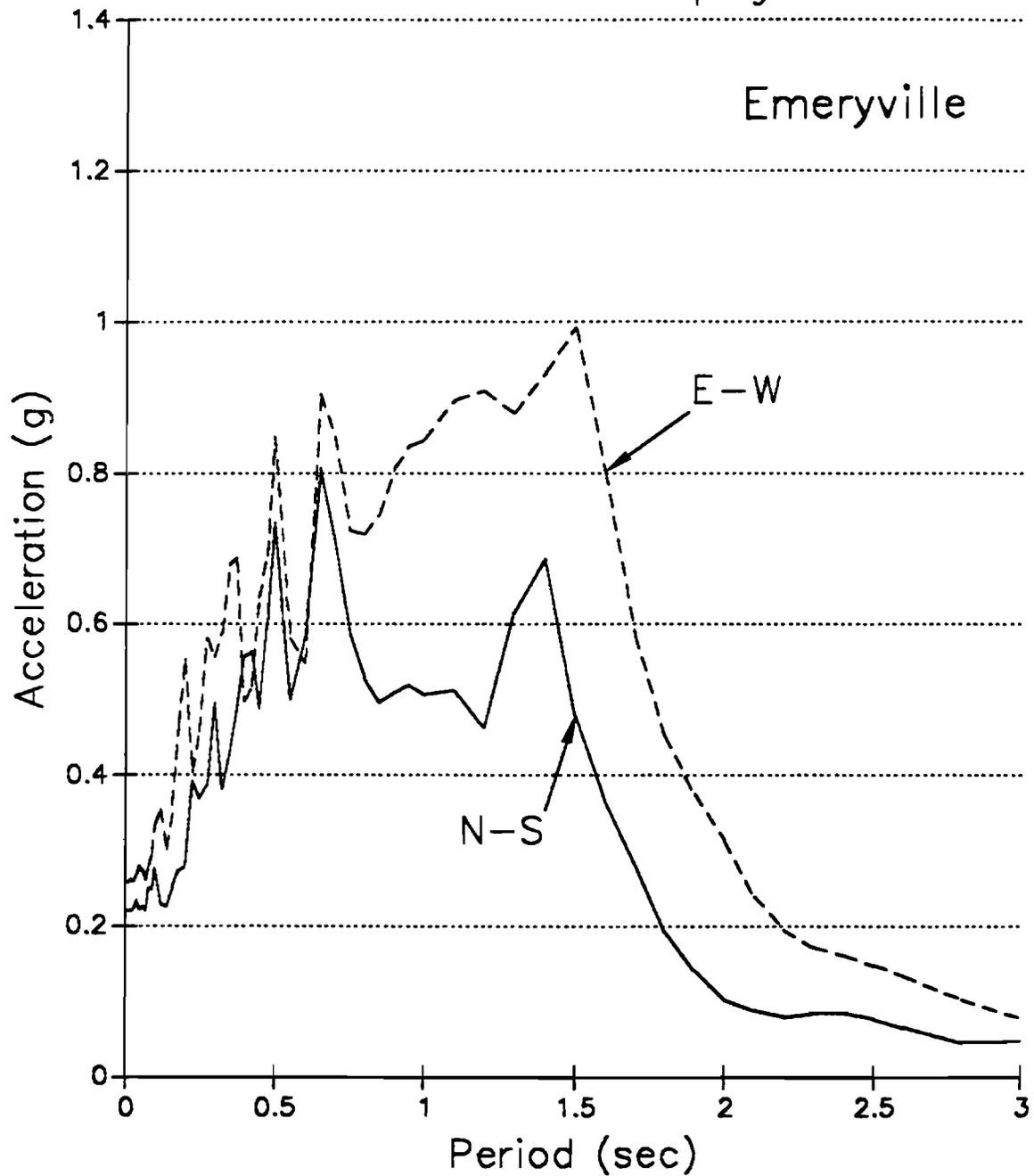


Figure 2.6 : Acceleration response spectra for the horizontal components of the Emeryville record

Loma Prieta, CA Earthquake 17OCT89
 Acceleration Response Spectra
 : 2% damping

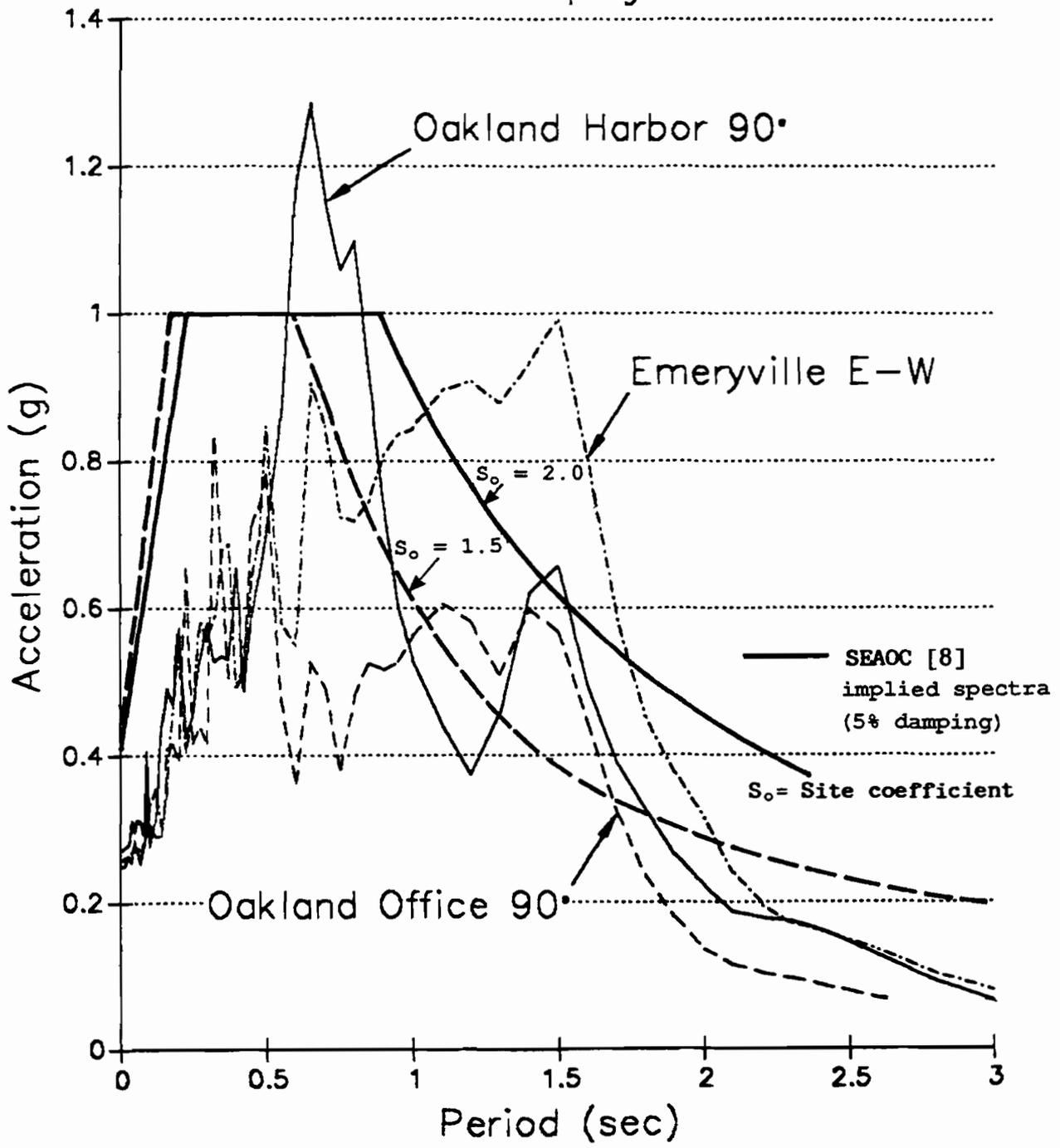


Figure 2.7 : Acceleration response spectra for soft sites in Oakland and Emeryville, and comparison with SEAOC [8] implied elastic design spectra for $Z=0.4$, $I=1.0$

Loma Prieta, CA Earthquake 17OCT89
 Acceleration Response Spectra
 : 2% damping

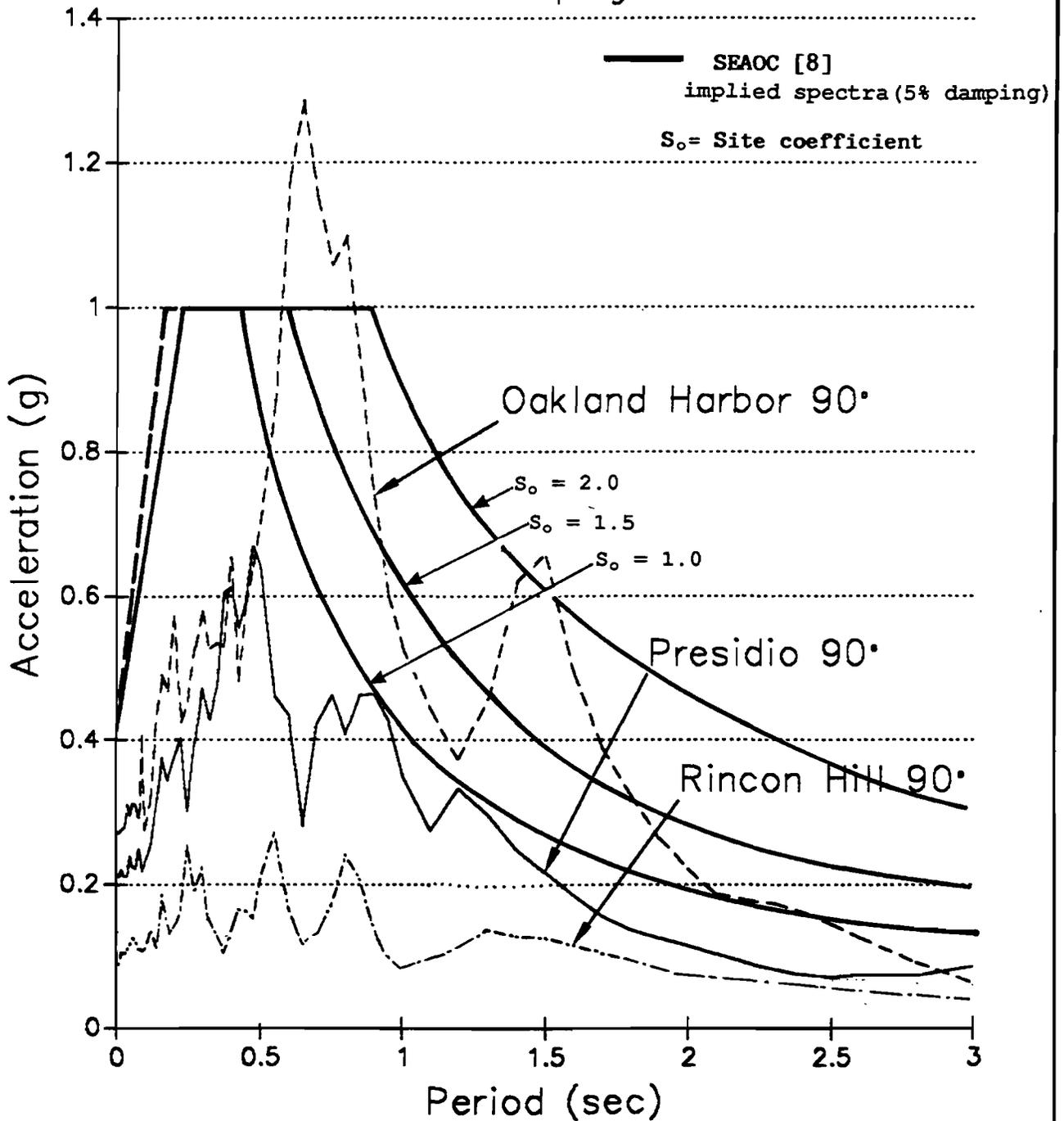


Figure 2.8 : Acceleration response spectra for soft, intermediate and bedrock sites in Oakland and San Francisco, and comparison with SEAOC [8] implied elastic design spectra; $Z=0.4$, $I=1.0$

Loma Prieta, CA Earthquake 17OCT89

Acceleration Response Spectra
Normalized to Rincon Hill 90° : 2% damping

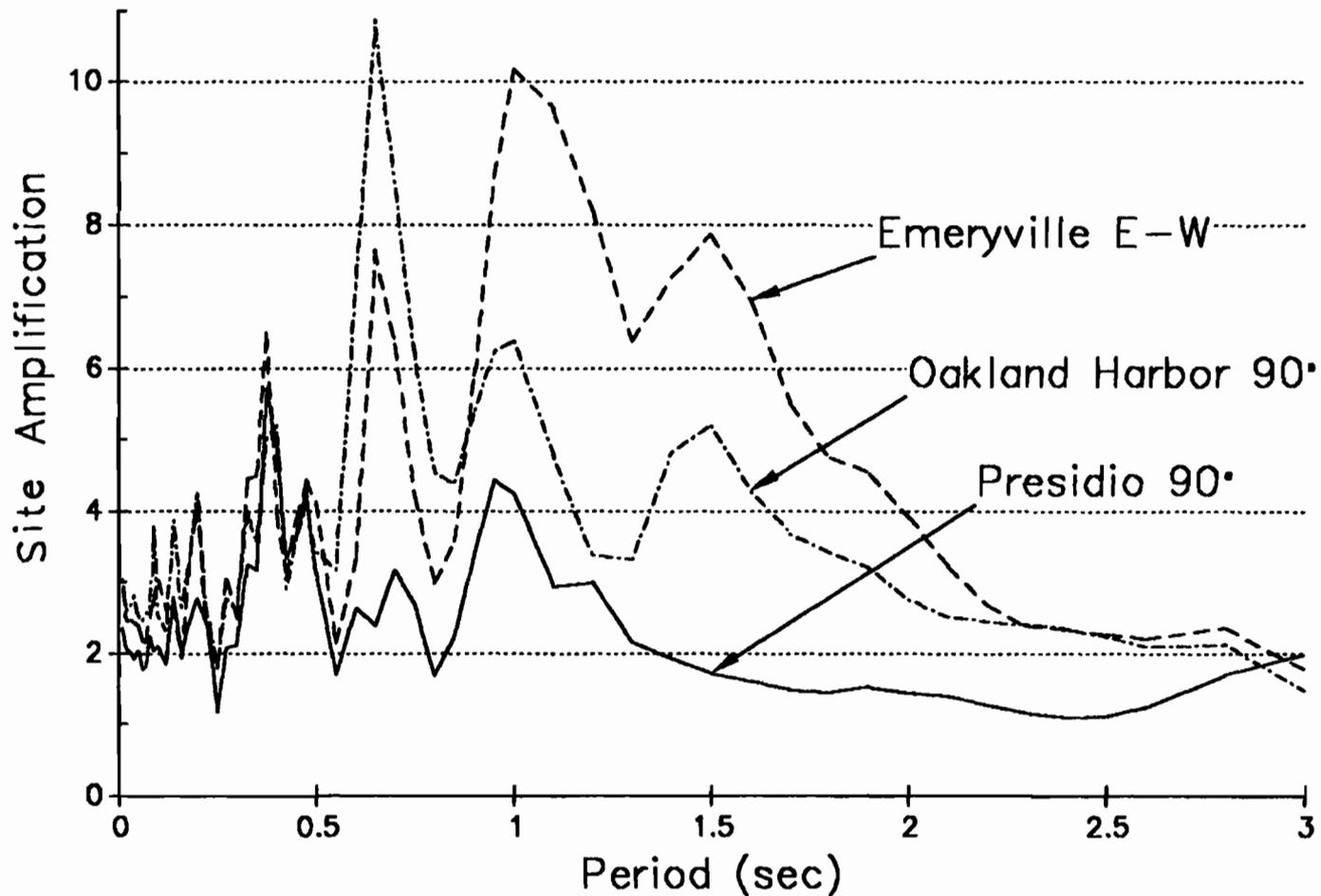


Figure 2.9 : Site amplification factor for 2% acceleration response spectra normalized to Rincon Hill (90°) bedrock record

Loma Prieta, CA Earthquake 17OCT89
Acceleration Response Spectra

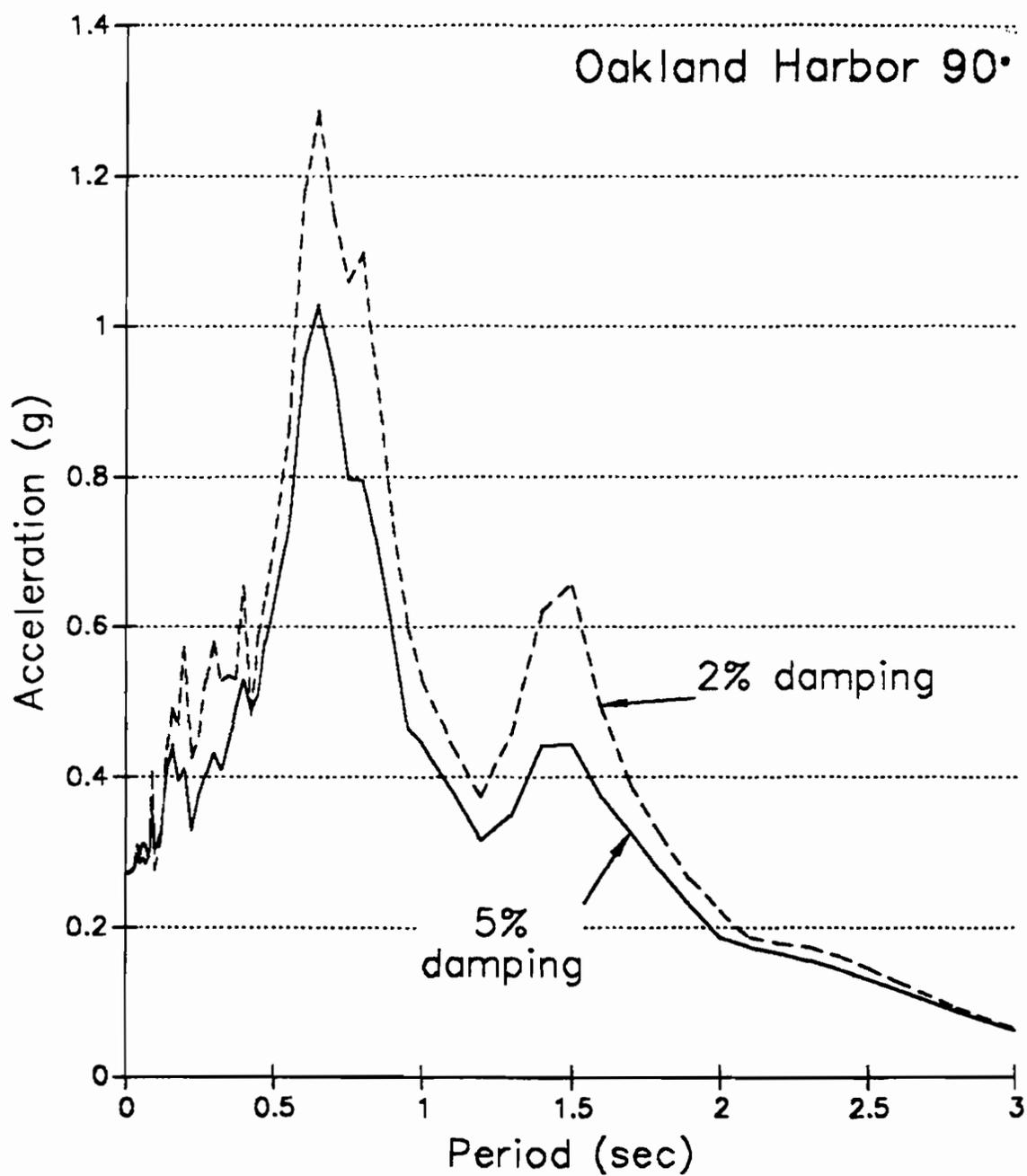


Figure 2.10 : Acceleration response spectra for Oakland Harbor Wharf 90° record

Loma Prieta, CA Earthquake 17OCT89
Acceleration Response Spectra

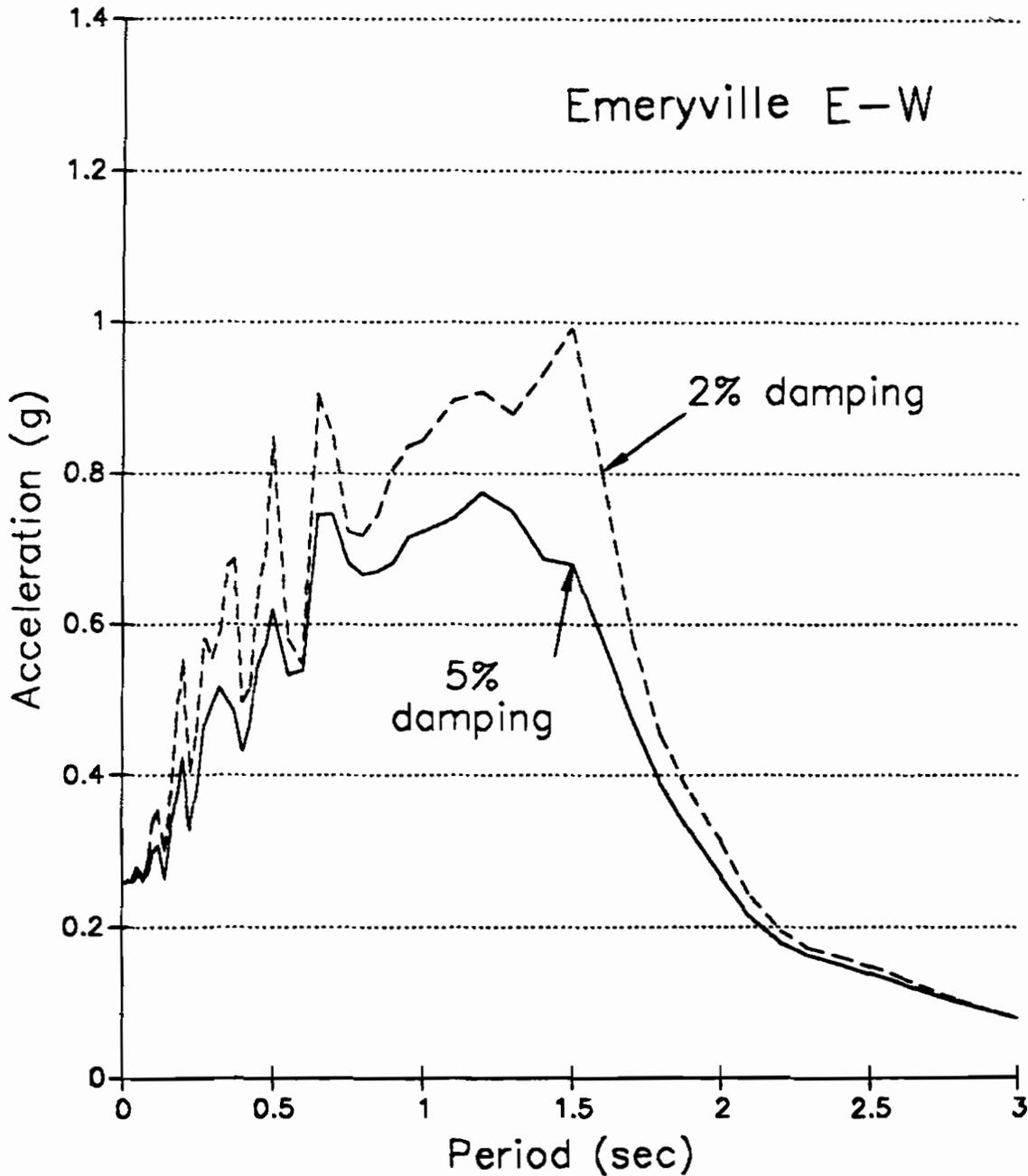


Figure 2.11 : Acceleration response spectra for Emeryville E-W record

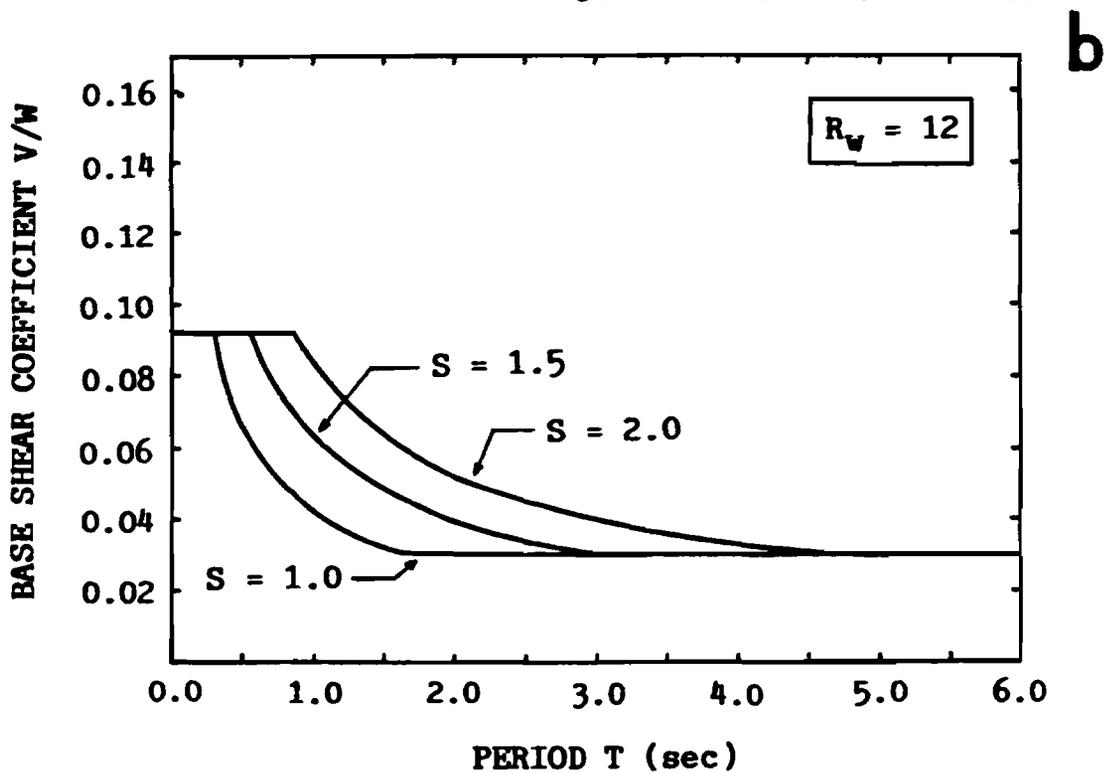
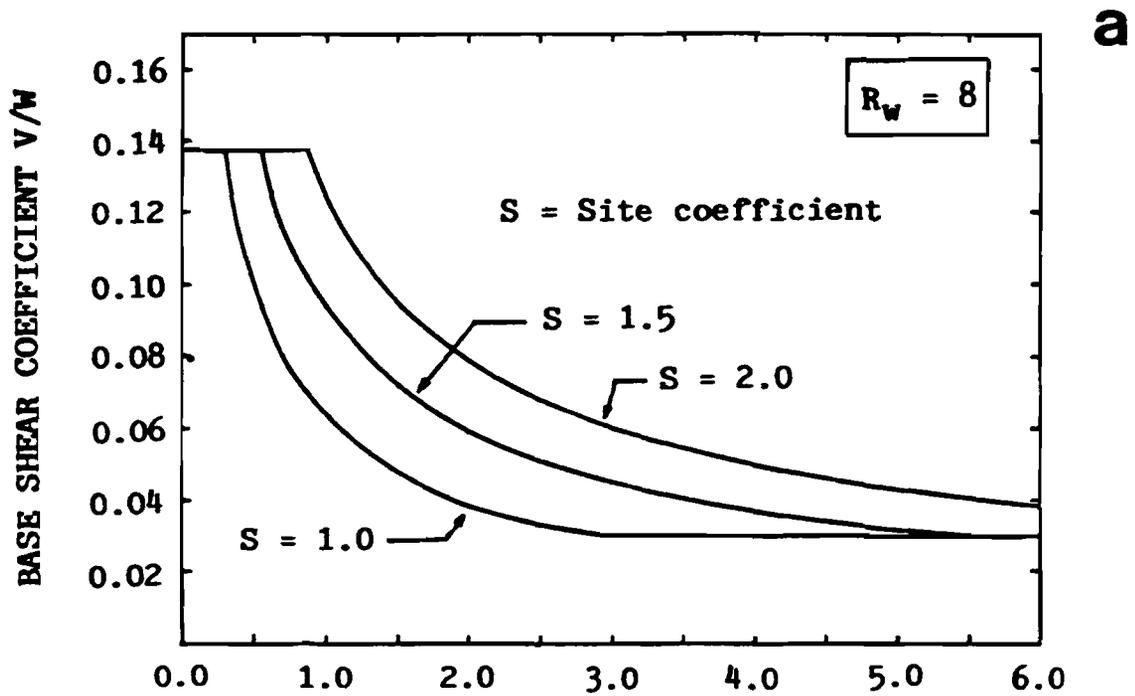


Figure 2.12 : SEAOC [8] base shear coefficients for lateral load-resisting structural systems with $R_w=8$ (a) and $R_w=12$ (b); $Z=0.4$, $I=1.0$

PERFORMANCE OF NON-ENGINEERED BUILDINGS

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P Boyce, M Frazer and S Hagblade, engineers of Santa Cruz, California.

3.1 INTRODUCTION

Most of the structural damage to buildings from the Loma Prieta earthquake was to non-engineered buildings, and structural collapse was confined to this type of structure. Masonry infills and cladding in engineered and in older semi-engineered buildings also suffered damage (see Section 4 in this report covering Engineered Buildings). However most of the damaged non-engineered buildings were on the poorer soils, for example in San Francisco the deep bay muds and hydraulic fill in the Marina District (Figure 3.1) and soil of only slightly better quality in parts of the South of Market area. There were usually no engineered buildings nearby for good comparisons to be drawn, but two cases are cited in which the engineered buildings near badly damaged masonry buildings had suffered either less damage or none whatever. There was no damage to non-engineered buildings reported from the parts of San Francisco on rock in the vicinity of the more heavily engineered buildings of the town. However in the Richmond area (Figure 3.1) there was damage to non-engineered buildings some way from the Bay and Ocean shores, where better ground conditions were recorded. Due to the stucco facades, buildings of very different age might be given a similar appearance and completely disguise the type of construction. Only where damage was appreciable was the structural form revealed. The Lettunich Building in Watsonville for example had a relatively modern structure despite its appearance (See Section 4.3.4).

The severe damage to the downtown area of Santa Cruz is largely attributable to the foundations. This area had always been subject to flooding and had formerly been a marsh, but about forty years ago the river was diverted into a deep culvert (See Section 4.6.5) after which the ground water level in the downtown area probably fell. Flooding before this diversion is reported to have caused appreciable settlement damage in at least one of the buildings. Buildings with damaged foundations are known to be particularly prone to damage during earthquakes.

In San Francisco and Oakland, Sandborne maps exist for all buildings and on these are recorded the structural details and records of earthquake damage. In the area of Santa Cruz no such records are kept but a "Damage Survey Report" (DSR) is produced for each structure. Throughout the whole affected area buildings were assessed for their engineering integrity, and "tagged" depending on the condition. Red-tagged buildings were condemned for demolition. Green-tagged buildings were those requiring no repair and orange-tagged buildings were those requiring repair and further inspection, and liable

to be red-tagged if they suffered further damage from an aftershock.

A total of 17 deaths are reported which were related to the failure of masonry and other non-engineered buildings. This is about 25% of the final death toll of 67 people (58% of which was due to the collapse of the Cypress Viaduct). These were:

- 6 in Santa Cruz,
- 4 in the Marina District of San Francisco (at 2 Cervantes Boulevard and 3701 Divisadero),
- 5 in Bluxome Street, San Francisco,
- 2 in Watsonville.

Most if not all of the structures responsible for these deaths are described below.

This earthquake triggered more than 150 strong motion observation instruments, in the near and far field. From these 90 were at ground level, under small shelters or in buildings with less than 4 storeys. In the near field (fault distance <30km) 15 instruments located on alluvial soil were triggered giving peak accelerations in the range of 0.14 to 0.64g. Another 3 instruments were located on intermediate to hard soil and registered 0.33 to 0.47g (fault distances 15-28km). It is almost certain that buildings located within the 30km radius circle from the centre of the inferred fault and on hard soil were subjected to higher peak accelerations than those in soft soil. Nevertheless from a quick examination of buildings located on harder soils as opposed to those located on softer soil (in Santa Cruz and Watsonville) it was noticed that most of the damage was in the latter. This partly justifies the lack of correlation between peak horizontal accelerations and damage to buildings situated in the near field. Not even the earthquake magnitude, which is a measure of the energy released, is a reliable indicator of the extent of damaging potential. For its magnitude the Loma Prieta earthquake could not be considered a very damaging earthquake from the point of view of buildings on good foundations. This may be related to the fact that the duration at source was only 6 seconds, which is the shortest duration likely for an earthquake of magnitude 7.1, Reference 1, and attributable to its two-way propagation of the shock from its point of initiation, rather than the more common unidirectional propagation.

For the far field (fault distance >60km) there were 42 instruments triggered by the earthquake. The horizontal peak accelerations range between 0.02 and 0.16g. At these distances the peak accelerations of soft soil are amplified by a factor of 2-4 in comparison with those in bedrock. It is expected though that the peak velocities and displacements are amplified by a larger factor (possibly 3-6 times for velocities and 4-8 times for displacements).

Although peak horizontal acceleration is certainly not the best parameter to explain damage to buildings, it must be noted that the degree of damage of vulnerable load-bearing masonry buildings was surprisingly low. Broken glass was a rare sight in Gilroy, which apparently experienced strong motion with peaks in the range of 0.17 to 0.55g. In previous earthquakes severe damage has occurred at sites that experience accelerations larger than 0.20g. As an example in the districts of Kentro and Nesaki in the city of Kalamata (Greece 1986), accelerations recorded at 0.20g and 0.30g, and half the masonry buildings were damaged beyond repair, as opposed to 1.1% of reinforced concrete framed buildings, Reference 2. Detailed damage surveys when published will shed considerable light on some of these striking differences. Nevertheless the quality of masonry buildings in California is definitely higher than in Greece, Italy, Turkey and Mexico and most poorer countries which suffer damaging earthquakes. This fact proves that well designed and carefully constructed masonry buildings can on good foundations survive moderate earthquake motions (this concerns particularly countries in the Developing World).

3.2 THE MARINA DISTRICT, SAN FRANCISCO

3.2.1 Location and Ground Conditions

San Francisco's Marina District is an area of approximately 0.6 square miles, situated on the Bay shore in the northern part of the peninsular (Figure 3.1). It is developed on hydraulic fill underlain by deep Bay Mud, the fill being placed originally for the Panama-Pacific International Exposition of 1915. Soft soil amplification recorded at similar sites in the Bay area, considered in Section 2 of this Report, is likely to have produced peak horizontal ground accelerations of 0.2 - 0.3g in the Marina District. At Oakland Harbour Wharf, for example, which is an area of reclaimed land with similar surface materials to the Marina area, the peak acceleration recorded was 0.29g. This compares with only 0.05-0.09g at stiff soil and rock sites in the San Francisco area (see Tables 2.1, 2.2 and Figures 2.1 - 2.3). That severe ground shaking and settlements occurred in the Marina District is evidenced by the distortion of roads and sidewalks in the area (Plate 3.1, see also Section 7 of this Report). Dynamic settlement induced by the earthquake was increased by the fact that the hydraulic fill had never before been consolidated by a large-magnitude earthquake. Liquefaction and sand boils were also induced by shaking of saturated sediments overlain by fill on reclaimed land throughout the Bay area.

3.2.2 Type of Construction

The majority of the buildings in the Marina District consist of 2-4 storey abutting timber-framed dwellings (Plates 3.2, 3.3). The typical street consisted of houses of similar height, usually abutting, but with the taller and generally larger buildings on corner sites. These date from the period 1915-1925, and were not designed specifically to resist lateral, and in particular earthquake forces, since they pre-date the Uniform Building Code which was introduced first in 1927. In most cases the buildings had not been strengthened against lateral loading, as evidenced by the lack of diagonal cross-bracing in the majority of the timber frames.

Figure 3.2 shows diagrammatically the elevation of a typical timber-framed 4-storey dwelling in the Marina District. Most such structures are strengthened by internal partition walls and are boarded with timber dating from the original construction. At a later date they have been modified by:

- (a) removing the first-storey external walls to provide car parking spaces beneath the living accommodation. The frontage to the street consists then of boarded timber columns supporting the first floor beams, the columns being clad with brickwork (Figure 3.2 and Plate 3.4). At the rear of the buildings the timber frame has minimal or no bracing and is faced with timber boarding, with the columns throughout being clad with galvanised steel sheeting to satisfy fire regulations (Plate 3.5). The beam and column members were simply nailed together to form the skeletal framework.
- (b) Superficial stucco panels or brickwork cladding had been added for decorative purposes to most buildings (Plates 3.6 - 3.9). This cladding was attached directly onto the timber boarding, and at the rear of buildings, in particular, provided some lateral resistance in the form of weak unreinforced shear walls. Stucco cladding was more common than brick cladding. In these buildings the brick cladding comprised a single leaf, compared to the double leaf cladding normal on the facades of the larger engineered buildings. The bricks however were the same small solid bricks used on the larger buildings and there was rarely any evidence of ties. However 1 1/2" (38mm) ties at 40" (1.02m) centres each way were noted

behind one section of fallen masonry, the collapse of which was related to the deterioration of the timber lining. The quality of the mortar used for the brick cladding was poor in many of the older buildings, but the poorest quality of all was in some of the newer buildings.

In contrast the stucco, a sand cement render, typically 10mm thick, was generally of good quality and was commonly tied to the timber lining by small protruding nails. Nails 3/4" (19mm) long and at 9" (229mm) pitch each way were found behind one fallen unreinforced stucco panel (Plate 3.7). Sometimes the stucco contained a light galvanised chicken wire. There were relatively few out-of-plane failures of the stucco, and those noted were confined to cases without mesh. Diagonal or vertical cracks were more common.

3.2.3 Damage and Collapse Due to Effect of Soft First Storey

The weakening effect of the removal of the walls at first storey level led to many dwellings in the Marina District being severely damaged (Plate 3.10), and in some cases caused total collapse (Plates 3.13 and 3.16) or partial collapse (Plates 3.14, 3.15).

The most severe damage occurred in the 4-storey buildings, particularly those isolated from the supporting effect of adjacent structures on one side in one or both elevations. The sidesway mechanism (Plates 3.4, 3.10) was initiated due to the high lateral forces at the weak first storey level (Figure 3.3), with the upper three storeys behaving as a stiffened box structure subjected to approximately uniform horizontal acceleration. The horizontal forces were amplified by:

- (a) the effect of the soft soil deposits which induced peak ground accelerations of 3-4 times the bedrock motion, which in San Francisco was recorded at 0.05 to 0.07 g, and
- (b) the dynamic structural response behaviour illustrated in Figure 3.3, where the mass of storeys 2-4 is subjected to accelerations multiplied from the base (ground) motion by factors up to 4, depending on the natural period and damping of the building. This is discussed further in Section 2 of this Report which analyses the earthquake ground motions and response spectra.

Hence combining the effects (a) and (b) above, the spectral acceleration generated in the building assuming its behaviour to be that of a simplified single degree of freedom system, would in some cases have been as large as 1.0-1.3g (see Figures 2.7, 2.8.), assuming elastic behaviour. An inelastic analysis has also been carried out (See Fig 3.4), and is discussed below.

The sidesway mechanism resulted from widespread joint failure and rotation at the first floor column/beam connections (Plate 3.10), with lateral deformation of up to 300mm (12 in) in some cases (Plate 3.17). In a few cases the first storey collapsed completely. Two structures on the point of collapse during an early visit a week after the earthquake (Plates 3.11, 3.12) had collapsed or had been demolished when visited a week later (Plate 3.13), and the corner site was soon cleared (Plate 3.14). Of the two buildings in Plate 3.11 the lower three stories of the corner building had collapsed by the time of the second visit and the top storey had moved over by two storey heights (Plate 3.13), whilst the lowest storey of the adjacent building collapsed and the remainder moved over by one storey height (3m or 10ft). This building may have contributed to the collapse of its neighbour on its left (Plate 3.11). The outline of an adjacent once lower building, graphically illustrating the drop, is shown in Fig. 3.14.

A simplified analysis of inelastic single-degree-of-freedom response has been

carried out, in order to simulate the behaviour of buildings in the Marina District to the type of motion recorded at a site with similar ground properties, namely the Emeryville 260° (approximately E-W) record at USGS station 1662, shown in Figure 3.4(a). The peak horizontal ground acceleration at this site, located about 1 mile north of the collapsed Cypress Viaduct in Oakland (Figure 2.2) was 0.26g, and the duration of strong shaking (RMS acceleration >0.05g) was about 12 seconds (Fig 2.3e). The long-period motions present in the ground acceleration trace are indicative of the soft soil conditions, and result in a response spectrum with high average spectral accelerations in the mid-period range of 0.6 to 1.6 seconds, namely 0.83g for structures with 2% damping, representing an average spectral amplification ratio of 3.2 (Fig 2.6). This parameter gives a strong indication of the extent of damage to be expected in non-engineered buildings on such soft sites. The peak spectral velocity calculated using the Emeryville East-West record is also exceptionally high, being 2.27m/sec at a period of 1.5 seconds, with an average value of 1.24m/sec over the period range 0.6 to 1.6 seconds.

In carrying out the inelastic time history analysis using the Emeryville E-W record, the controlling structural parameters were taken as the initial elastic period T (seconds), the equivalent viscous damping for the elastic system, here taken as 5% for the timber-framed dwellings, the yield displacement and the post-yield force-displacement behaviour. Analysis has been carried out assuming a yield displacement in accordance with the maximum allowable storey drift in the 1988 SEAOC (Structural Engineers' Association of California) seismic regulations, Reference 3, namely 0.005 times the storey height. In this case, assuming a storey height of 3.0m, the maximum allowable drift is 15mm for elastic analysis. A set of 3 analyses have been carried out for structures with initial elastic periods (T) of 0.6, 0.8 and 1.0 seconds. The assumptions made were a yield displacement of 15 mm and a bi-linear force displacement relationship with post-yield stiffness of 0.5% of the elastic stiffness, which implies that the structural connections generate a near-constant resisting moment once initial 'yield' or failure of the nailed joints has occurred. The time-history responses of these buildings to the Emeryville (E-W) earthquake record have been plotted in Figures 3.4(b) - (d) respectively.

Of interest in Figure 3.4 is the final and permanent displacement or deformation of the structure, which for an elastic period of 0.6 sec is -102mm which is 6.8 times the yield displacement, Figure 3.4(b). The corresponding values for periods of 0.8 and 1.0 seconds (Figures 3.4(c), (d)) are -220mm and +142mm, respectively, or 14.7 and 9.5 times the yield displacement. Although the sign associated with these values is of little significance, it is interesting to note that the structure with a period of 1.0 seconds yields in the opposite direction to the other two cases considered. The first yield occurs in all three cases just before the time (t) reaches 5 seconds (compare the response traces with the time history of ground motion in Figure 3.4(a)), but the reversal of ground acceleration in the time interval between 5.6 and 6.0 seconds causes the structures with periods of 0.6 and 0.8 seconds to yield significantly in the reverse (negative) direction, following which the displacement remains negative with slowly increasing permanent deformation. The load reversal between 5.6-6.0 seconds causes some negative yielding for structures with a period of 1.0 seconds, but this is followed by successive yield excursions in the positive displacement direction as shown in Figure 3.4 (d).

The initial elastic period of the 4-storey timber framed buildings in the Marina District is estimated to be 0.6-0.8 seconds, on the basis of:

- (a) the low overall stiffness of the first storey due to the open frontage and the lack of bracing to the timber frame at the rear of the dwellings, and

- (b) the added mass of the external masonry cladding.

The results from Figure 3.4(c), in particular, show that in this range, the permanent deformation induced in the structure could be over 200mm, which is in accordance with observations from damaged buildings in the area (Plates 3.4, 3.17).

In San Francisco as a whole, 12050 persons were displaced from their dwellings. The Marina district alone accounted for over 20% of this number, Reference 4.

There was no loss of life in either of the two buildings in the Marina District which suffered the complete destruction of the upper floors ("pancake collapse"). This contrasts with the situation when engineered reinforced concrete framed buildings with soft ground storeys collapsed, as in Mexico City and elsewhere.

3.2.4 Other Structural Damage Characteristics

A number of other common structural damage features were observed from the buildings in the Marina District, namely:

- (a) The effect of the weak first storey was accentuated in many cases by the asymmetry of the stiffness distribution, with consequent initial failure of the front columns leading to a shift in the centre of stiffness further towards the rear wall of the building. This then increased torsional movements of the structure, with large deformations at the street frontage and ultimately failure of the rear wall and frame connections, which must withstand nearly all the lateral loading once failure has occurred in the front columns.

The effects of horizontal irregularity on structural response have been discussed in relation to engineered buildings in Section 4 of this report. For buildings of 2 or 3 storeys, the damage caused by such asymmetry appears to have been limited by the stiffening effects of internal cross-walls. For taller buildings, and particularly those situated on corner sites, damage was more severe, with the weaker street frontages suffering greater damage than the stiffer, off-street, sides of such buildings (Plate 3.21). The directionality of the earthquake meant that the East-West frontage of this building in the Marina District suffered appreciably greater damage than the North-South frontage (see (e) below).

- (b) Some horizontal cracking occurred at foundation level (Plate 3.9), and other buildings slid sideways on their foundations by up to 60mm (Plate 3.18).
- (c) Disruption of gas and electricity services was widespread, and in a few cases fires had started, causing a number of buildings to be destroyed. This effect was localised, being limited to only two sites in the Marina District (Plates 3.19, 3.20).

The structural damage listed above and in Section 3.2.3 was particularly severe in the 4-storey buildings, especially the detached buildings on corner sites as mentioned above (for example Plate 3.21), and those at the end of a terrace (Plate 3.10). The 2 and 3 storey terraced buildings survived the earthquake with only minor non-structural damage, due to the lateral and torsional restraint provided by the adjacent structures which limited the forces on the front columns, and the lower period, probably less than 0.4 seconds, which led to much smaller lateral forces (see the response spectra plotted in Figures 2.7 and 2.8). It is surmised that the generally greater damage to the buildings on corner sites was due to:

- (a) their greater height and therefore greater mass, increasing the lateral force in the lower storey
- (b) the somewhat more pronounced lack of stiffness in the lower storey
- (c) the somewhat longer period of vibration due to (a) and (b)
- (d) the buffeting from the houses on one side only which would have increased the lateral displacement whilst at the same time providing a measure of support to the terrace houses, with the out-of-phase buffeting on the two sides preventing the build-up of cumulative displacements.
- (e) their greater torsional irregularity, as many had garages on two adjacent sides, though the earthquake was highly directional in the area, and most if not all of the damage could be attributed to E-W movement. Even in the vulnerable corner properties with soft stories in two directions, there was often barely enough movement to jam the garage doors in the N-S direction.

3.2.5 Non-Structural Damage

Widespread non-structural damage was observed, falling into two main categories:

- (a) There was considerable evidence of loose and broken cladding, particularly around doorways (Plate 3.17) and windows (Plate 3.6). Large panels of cladding had fallen from buildings with damage being most noticeable at the first storey level (Plate 3.7).
- (b) Disruption to services due to soil liquefaction and differential settlement in the soft fill material (see Section 7 of this Report).

3.3 OTHER DISTRICTS OF SAN FRANCISCO

3.3.1 Richmond District

Damage to 3 and 4 storey timber-framed buildings was also observed in the Richmond District of San Francisco, situated about 1.5 - 2.5 miles south-west of the Marina District (Figure 3.1). The structural damage was again due mainly to irregular stiffness distributions, particularly weakening at the first storey level as a result of alterations to incorporate car parking spaces beneath the building (Plates 3.22 to 3.25). Some of the areas affected had earlier been the site of ponds. In a new timber house under construction in the Richmond area (Plate 3.25) the lower storey comprised a portal of welded universal sections, which demonstrates that designers of smaller buildings appreciated the vulnerability of the type of construction in the Marina District. This building had suffered no damage, yet a conventional soft-storeyed building opposite had suffered badly with collapse of brick cladding and substantial damage to the ground floor timber columns.

3.3.2 South of Market Area

The South of Market Area suffers from generally poorer ground conditions than in the area to the North West of Market Street and parts are on reclaimed land. The building stock is quite old comprising mainly semi-engineered and masonry buildings, mostly prewar, and industrial properties housed commonly in steel sheds. A few masonry structures in the area and one to the North of Market Street were badly damaged. These structures were mainly condemned and demolition was in some cases well advanced.

3.4 DETACHED DWELLINGS AND MASONRY BUILDINGS IN THE SAN FRANCISCO AND EPICENTRAL REGIONS

3.4.1 Major Types of Damage

The area near the earthquake epicentre, situated about 100km SE of San Francisco (see Figure 1.1), suffered peak horizontal accelerations of 0.45 -0.65g. This area includes Los Gatos to the North, Santa Cruz in the South West and Watsonville in the South. Towns to the East of the Epicentre, in particular Hollister, Gilroy and Morgan Hill, suffered significantly less damage than other towns close to the epicentre. For example in Gilroy which is a very small town but one well instrumented, only one masonry building was damaged (Plate 3.54). The reason may be that these towns have suffered small earthquakes in the recent past, so the most vulnerable structures may already have been demolished. If so then the engineering assessments must have been excellent as residual unrepaired damage from the earlier earthquakes would have left these towns particularly vulnerable to later events. Whilst the structures in San Francisco had suffered most damage in the E-W and NW-SE direction, the structures to the South and West of the epicentre suffered more damage in the North-South direction. In particular this was noted in Boulder Creek, Santa Cruz and Watsonville. In Los Gatos just to the North West of the epicentre the evidence was conflicting, with occupants of houses being thrown in a North-South direction but a soft storeyed structure near a cutting, discussed later, had swayed in the East-West direction. The damage to non-engineered buildings in the worst affected areas, as well as in San Francisco outside the Marina and Richmond districts, as described in Sections 3.2 and 3.3 above, has been classified into the following four categories:

3.4.1.1 Construction of the Thicker Masonry Walls.

The masonry walls were often two bricks thick apparently with little or no cross-bonding. There was often a vertical layer of mortar between the walls. One instance was found in the Marina District of San Francisco with a vertical mortar layer and with cross bonding in which failure of one skin had occurred, severing the cross-bonded bricks (Plate 3.27).

3.4.1.2 Parapet Wall Failures

The most common form of masonry failure was the collapse of parapet walls. A number of the taller parapets collapsed in San Francisco, but in the downtown areas of Santa Cruz, Watsonville and Los Gatos many of the lower parapets collapsed (Plates 3.26, 3.28). Some too collapsed in Hollister. Frequently collapse was confined to the outer layer of masonry. It is noted that parapets on masonry buildings are not permitted in the Chinese seismic code.

3.4.1.3 Side and Front Wall Failures

Out-of-plane side wall failures were the next most common form of collapsed masonry, and usually only the top part of walls just below the roof was affected (Plates 3.29 and 3.30). Triangular failures with their greatest horizontal dimension at roof level were considered attributable to insufficient tying of the walls into the roof, which was sometimes aggravated by inadequate corner ties (Plate 3.34). Why failure less frequently occurred on the street frontage may be attributed to the fact the roof and floor structures generally spanned between front and back walls, tending both to restrain and compress them. In many cases parapet collapse tore away the top part of the wall (Plates 3.29, 3.33) and when this happened the front elevation too was sometimes affected and part of the roof

might collapse (Plate 3.33). Gable ends of pitched roofs are particularly vulnerable (Plate 3.30); so too are walls alongside staircases in which the flights are not tied to the walls (Plate 3.34).

3.4.1.4 Soft Storeys and Other Damage Features in Downtown Shops

In both Santa Cruz and Watsonville a high proportion of the shops had large windows, sometimes only interrupted by columns adjacent to neighbouring properties and by a central doorway (Plate 3.36). Irrespective of the material of construction, this presents a flexible ground floor structure and the damage noted on many such columns (Plates 3.40, 3.41) was due to impact between the buildings, described in California as "pounding". Such damage might be confined to the outer layer of cladding. In some 3-storey smaller shops in Santa Cruz, probably pre-war (Plate 3.39) the small columns either side of the windows were clearly of masonry. On some the stucco had spalled and the interior had suffered. Inside one such shop a pier in the party wall had split vertically suggesting a compression failure (Plate 3.37). Vertical splitting in external columns was normally confined to the lower few courses, suggesting a flexural compression failure (Plate 3.37). No evidence could be found to suggest that soft storeys lessened damage to the building above, which seemed to be the situation in the Marina District of San Francisco (Section 3.2.3).

Where there was no soft storey there tended to be some damage higher up, usually in the form of cracks to the corners of windows (Plate 3.46). Where there were no windows, as in the Del Mar Theatre in Santa Cruz, construction with rigid masonry envelopes might escape undamaged (Plate 3.44). Plate 3.43 shows a similar structure in Watsonville.

In Los Gatos the soft ground floor storey of a two storey masonry shop (Plate 3.42) had swayed to the degree noted for the soft storeyed buildings of the Marina District of San Francisco, which exceeded that in Santa Cruz and Watsonville. The shop was close to the edge of a deep road cutting estimated to be 15m deep accommodating Highway 17. The large cracks in the road suggest the cutting had amplified the ground movement. This was the only instance noted of a soft storeyed property founded in good material suffering damage (outcrops in the cutting suggested it was weathered rock), and the only instance noted of a soft storeyed property in which the upper floors had collapsed but the lowest storey had survived. This was clearly attributable to impact damage. The fact that the building swayed in the East-West direction, whereas movement generally in the area was in the North-South direction, cannot be attributed solely to the fact the building was soft only in the East-West direction. Evidence that the accelerations may have been greatest in that direction locally is provided by the fact that in the same row of commercial properties the end wall of the face immediately above the cutting fell out (Plate 3.47). The local accelerations here were clearly influenced by the North-South orientation of the cutting for Highway 17.

3.4.1.5 Shop Foundations

In Watsonville some of the smaller shops had brick foundations about 3 ft (1m) deep. One such building had a damaged facade revealing a stucco faced timber superstructure. The extent of this form of construction in downtown Santa Cruz is unknown and all the structures inspected, except Ford's Department Store, were of masonry construction. The superstructure of one building in Santa Cruz had been demolished revealing massive rubble wall foundations (Plate 3.45).

No liquefaction has been noted in the downtown areas of either Santa Cruz or Watsonville. In downtown Santa Cruz the soil is very variable but typical of the

worst ground condition is the following bore hole information.

Depth below street level	Soil
0-1 ft	Grey sand
2-3 ft	Clayey sand
3-6 ft	Mottled grey silty clay with high organic content
6-13ft	Brown silty sand, damp and loose
13-18ft	Loose to medium dense sand
17-18ft	Gravel

The water table was at 9ft below street level.

3.4.2 Damage to Notable Buildings

3.4.2.1 Fords Department Store, Watsonville

One large structure of uncertain date of construction was Ford's department store Watsonville (Plate 3.48), now demolished. This had a soft ground floor storey, but was continuously clad over the remaining stories. The structure was severed by a large vertical central crack (Plate 3.49) and there was much lesser facade damage, mostly confined to the cladding. Some of the internal structure is reported to have collapsed. A corner column (Plate 3.50) had spalled revealing mixed construction including a layer of bricks applied to a mortar layer, probably on a masonry interior.

3.4.2.2 Ford's Department Store, Santa Cruz

This was a low rise structure, possibly pre-war, part with a mezzanine floor. It had a lightweight facade and a blockwork party wall. The top of the sidewall of the taller masonry building next door had collapsed (Plate 3.51), causing the collapse of the single storey part of the store (Plate 3.52). The part with the mezzanine floor still stood. Further down the street on the same side a similar situation had occurred, with the collapse of the roof of a small shop (Plate 3.53). Most of the buildings in the vicinity and on this side of the street were demolished soon afterwards.

3.4.2.3 Compass and Rose

This was an old brick built bar (Plate 3.56) with exposed brickwork in which the masonry had a distinctly weathered appearance, suggesting both the brick and the mortar was of particularly poor quality. A close inspection inside revealed that the mortar had disintegrated over quite extensive areas (Plate 3.57) and in one place the masonry had flaked (Plate 3.55). This structure was damaged beyond repair.

3.4.2.4 Metropole Hotel

The Metropole hotel in Santa Cruz is an old hotel of load bearing masonry in the downtown area. The end wall fell out (Plate 3.54).

3.4.2.5 Churches

Churches do not feature on American maps and due to the height of buildings they may be hard to find. A brick church in Watsonville with a tall spire, reputed to be only eight years old, had suffered damage to the masonry in the form of loss of the top part of the masonry walls closing the transepts and the aisle, the loss of parapets and cracking at the corners. The spire had been prefabricated and

had cast iron spirelets, all of which were undamaged (Plate 3.58). The spire was removed shortly after the photograph was taken and laid carefully on the ground awaiting the repair of the masonry.

In Santa Cruz the Church of the Holy Cross had a cracked spire, which was subsequently rebuilt. There were sand boils in the road nearby. A replica of the original Mission building in Santa Cruz was undamaged.

3.4.2.6 Timber framed buildings

Timber framed buildings without ground floor soft stories and in good condition generally performed well, if bolted down. Large undamaged timber framed buildings noted were a hotel behind Kane Hall in Watsonville (Plate 3.61) and a timber framed Palladian style church of the appearance of an isolated two storey block in Los Gatos (Plate 3.62). On Road 152 between Watsonville and Gilroy a timber framed school was cracked on all sides just above foundation level (Plate 3.60). The damage however was attributable to rotten wood and the fact the frame was not bolted to the foundations. In downtown Watsonville a number of the smaller commercial buildings were timber framed and some of these had soft ground floor stories and had suffered loss of masonry cladding and impact damage. These were founded on alluvium. In Los Gatos similar two storey timber framed houses on two sides of the main square had suffered no significant damage, and all were green tagged. The good performance can be attributed to the facts that:

- (a) the ground floor stories were generally less soft than in many shops elsewhere,
- (b) the buildings were of similar height and structural form and,
- (c) they were probably founded on weathered rock.

3.4.3 Detached Family Houses

3.4.3.1 Type of Construction

Most isolated detached domestic houses in the epicentral and Bay areas are quite different from the houses in the Marina District of San Francisco, but they are similar in that they are predominantly of timber construction. The detached houses however are usually of painted woodwork, but some of the older more expensive houses have patterned stucco finishes, though very few have the masonry cladding common in the Marina District (Plates 3.6, 3.7).

The detached houses, whether of one or two storeys, have a common feature, namely the "cripple" or "pony" wall around an enclosed, but usually accessible, underfloor area, known as the "crawl space". As in Britain underfloor ventilation protects wooden floors from deterioration, but the greater height of the American construction enables the termite and rot damaged support to be replaced without major disruption. It is also convenient for service installation and replacement, housing meters for the services. In low lying ground it also provides some protection from flooding.

A feature of most of the houses was that the foundations were constructed on the ground surface, with minimal excavation, so many were founded on topsoil. There was sometimes a concrete ground slab, probably two to three inches thick. At the base there was a very shallow concrete strip supporting the building frame (Plate 3.66), but around the perimeter was sometimes a low masonry wall of stone (Plate 3.71) or brick (Plate 3.72), which belong respectively to the houses shown in Plates 3.69 and 3.70. This would normally be disguised behind the

cladding. It is understood that houses predating 1950 generally had masonry walls and were not bolted down. In many of the more recent houses there was no masonry, but instead a concrete slab to which the frame was bolted, with bolts typically at four feet centres.

3.4.3.2 Observed Damage

The damage suffered by the detached family houses in Santa Cruz and surrounding communities was quite different from that suffered by the predominantly terraced houses in the Marina District of San Francisco, a fact in part attributable to the greater ground acceleration nearer the epicentre. It is questionable whether these detached houses would have suffered major damage had they been situated in the Marina District.

The most common form of damage to detached houses was the collapse of chimneys on roofs, and it is estimated by loan specialists that 60% of chimneys collapsed above roof level in the towns close to and to the south of the epicentre, including Santa Cruz and Watsonville. The next most common source of damage, though far less frequent, was the separation and subsequent collapse of external chimney stacks (Plate 3.63). Along one road in Los Gatos a third of houses with this form of construction had lost their chimneys.

A very severe form of damage occurred to porches which were proud of the main building in plan. Porches recessed within the plan area suffered less damage. This is a form of soft storey damage and is attributable to:

- (a) The shear from the porch roof causing separation between the porch and the house when the porch collapsed, or
- (b) Severe damage to the columns of the porch roof just above balcony level when there was an obvious separation from the house.

The reason for (b) was unclear, but might be the horizontal flexibility of the structure of the porch roof.

A mode of failure, described colloquially as "houses sliding off their foundation", was very often just as it is described (Plate 3.66 which is a detail of the house in Plate 3.65), though the description was also applied to the folding of an inadequately braced crawlspace (Plate 3.64). When total collapse of this storey had occurred it was very often impossible to determine which mechanism had induced the collapse (Plate 3.67).

The level of the under-floor masonry varied in some cases around the perimeter and internally, and the steps provided shear keys. Clearly the internal chimney stacks had carried the brunt of the horizontal force in many houses, thereby preventing or reducing sliding, but the damage caused to the chimney stacks was more difficult to assess. The owner of one such house examined had noted diagonal cracking in the underfloor part of the chimney stack, with the danger of total collapse during an aftershock (Plate 3.73).

In the most serious cases of sliding the cripple wall collapsed and the house fell by 2 or 3 ft (0.6 - 0.9 m). The impact often caused damage elsewhere in the house and the collapse of a porch was not infrequent (Plates 3.76 and 3.77 illustrate this for the utilitarian and luxury ends of the housing stock). In some houses, common in Watsonville, the steps to the front door were of stone and the houses sometimes separated horizontally as the cripple wall collapsed (Plate 3.68).

The most heavily damaged buildings observed in the town areas were in Myrtle Street, Santa Cruz, and possibly the most severe case of damage to a two storey house is that shown in Plate 3.78. A notable feature in Myrtle Street and elsewhere was that a part or side of a street would be affected, possibly indicating a local foundation problem. A street of single storey houses so affected was Jefferson Street, Watsonville. In Watsonville most of the damaged timber framed houses were in one sector to the North of the main downtown area. In other parts of the town they were mostly undamaged.

Plate 3.79 shows a house suffering from porch and external chimney collapse.

In Myrtle Street there had been a fire, which could not be extinguished due to the fracture of a water main on West Cliff.

Buildings were sometimes stabilised against aftershocks with plywood sheets nailed across windows.

3.4.4 Hill Top Communities

West Cliff itself is a rock outcrop and examination of houses along the crest (Plate 3.75) revealed a vertical crack in one house, with two-way movements of about 5mm (1/4"), but no serious damage. At the foot of the outcrop, but still on rock, was a small paint spray shop where the paint cans had stayed on the shelf, yet the road on softer soil in front had a longitudinal crack and a wall opposite had collapsed. On West Cliff one house was found which had moved two inches away from the soil, but the significant damage to this house was attributable to rotten timber. Elsewhere on the ridge damage was minimal. There was no indication here of the possible ground motion amplification factor of two to three suggested by theory for steep cliffs.

As already noted one soft-storeyed masonry building close to the top of a deep cutting in Los Gatos had suffered severe damage. There were reports of significant damage to houses in other hill top locations such as the "Summit" area. A few modern houses on rock suffered badly, but these were of poor construction. A hill top community above Boulder Creek, where there were reported to be a number of particularly well built modern houses suffering major damage, was chosen for examination (Plates 3.80 to 3.87). These houses reflected the present thinking of using 1/2" (12mm) thick plywood walls to resist the seismic shock, and this they did effectively. A common feature was that although built on a sloping site, soil seemed to have been placed to provide a ledge to facilitate construction. The movement of this soil, observable from deep cracks in the soil and the collapse of a cliff top fence, was a major source of damage.

One building still standing but badly shaken, with a collapsed chimney and jammed garage doors, had a wide balcony cantilevered out from the top of the hillside (Plate 3.82). This building was well founded and the snapping of bracing testified to the magnitude of the horizontal force. Buildings on the other side of this road, on the crest of the hill, suffered no significant damage.

A common problem on other hillside communities, generally when the foundations were stepped, is the collapse of the unusually high cripple wall on the front face, which was attributed to rocking of the building. The solution suggested was seismic ties at the back of the building.

The measures for the strengthening of timber framed buildings to resist future earthquakes were the tying down of the buildings on their foundations with bolts (Figure 3.5) and of bracing to strengthen the external parts of chimneys.

However there was frequently insufficient weight in the foundations for this measure to be effective.

A domestic house in the Summit area near Aptos, in the fault zone, is reported to have failed with explosive violence. The roof lifted off, all the walls were heavily distorted and steel bolts had sheared. The recommended strengthening measures would not have reduced damage on this house. Further evidence that vertical accelerations had exceeded 1.0g was provided by a parked vehicle which is reported to have jumped over a fence.

3.4.5 Restoration of Damaged Buildings

The problems facing the engineers responsible for condemning and restoring structures were as follows:

- (i) In masonry buildings suffering severe discrete cracks there was doubt as to the damage to the masonry in areas still apparently solid. The capacity to resist vertical loading was unquestionable, but the problem here was in assessing the strength reduction in resisting lateral loads.
- (ii) In downtown Santa Cruz the cracks in some masonry structures were being monitored and some were opening during aftershocks, none of which were very significant. The continued movement was attributed to settlement of the soil structure, allegedly damaged previously by flooding. Most of the buildings were of insufficient importance to justify the expense of underpinning.
- (iii) There was a general reluctance to conduct repair work when there was a possibility of significant aftershocks. However the characteristics of aftershock records in California is such that the risk would seem to become acceptable, providing normal precautions are taken, only a few weeks after the earthquake.

It is likely that the expertise in restoring buildings in the United States is less than in Great Britain, as until recently there has been a tendency to demolish old buildings and construct taller ones rather than refurbish existing buildings.

3.5 CONCLUSIONS

- (i) Masonry are high risk structures, particularly when on soil subject to liquefaction and flooding.
- (ii) Structures already suffering settlement damage are particularly prone to earthquake damage.
- (iii) Masonry comprising two vertical leaves require cross-bonded bricks more frequently.
- (iv) Parapets are particularly prone to damage and require counterforts or other strengthening.
- (v) Masonry structures with soft stories are particularly prone to damage and the only unreinforced masonry structures surviving a severe earthquake intact were cubic structures with few windows.
- (vi) The tops of walls, where they carried little direct load, need to be well tied into the horizontal structure, and walls should be well tied into stairs.
- (vii) Light timber houses need to be well tied to their foundations.

- (viii) The mass of house foundations is sometimes insufficient.
- (ix) The amount of bracing over the height of the cripple wall in many houses should be increased.
- (x) Internal chimney stacks attract load but can be strengthened to carry horizontal forces.
- (xi) External chimney stacks and all chimneys should be well tied into the main structure of the house.
- (xii) The construction of houses should be avoided in those areas suffering the worst damage unless constructional defects can be identified or ground works can be justified.
- (xiii) Timber houses with 1/2" plywood panels are an excellent form of construction in seismic areas, providing the foundations are sound.
- (xiv) Houses should not be founded on topsoil.

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2. A S Elnashai and K Pilakautas, "The Kalamata (Greece) Earthquake of 13th September 1986", Engineering Seismology and Earthquake Engineering Research Reports, Imperial College, London.
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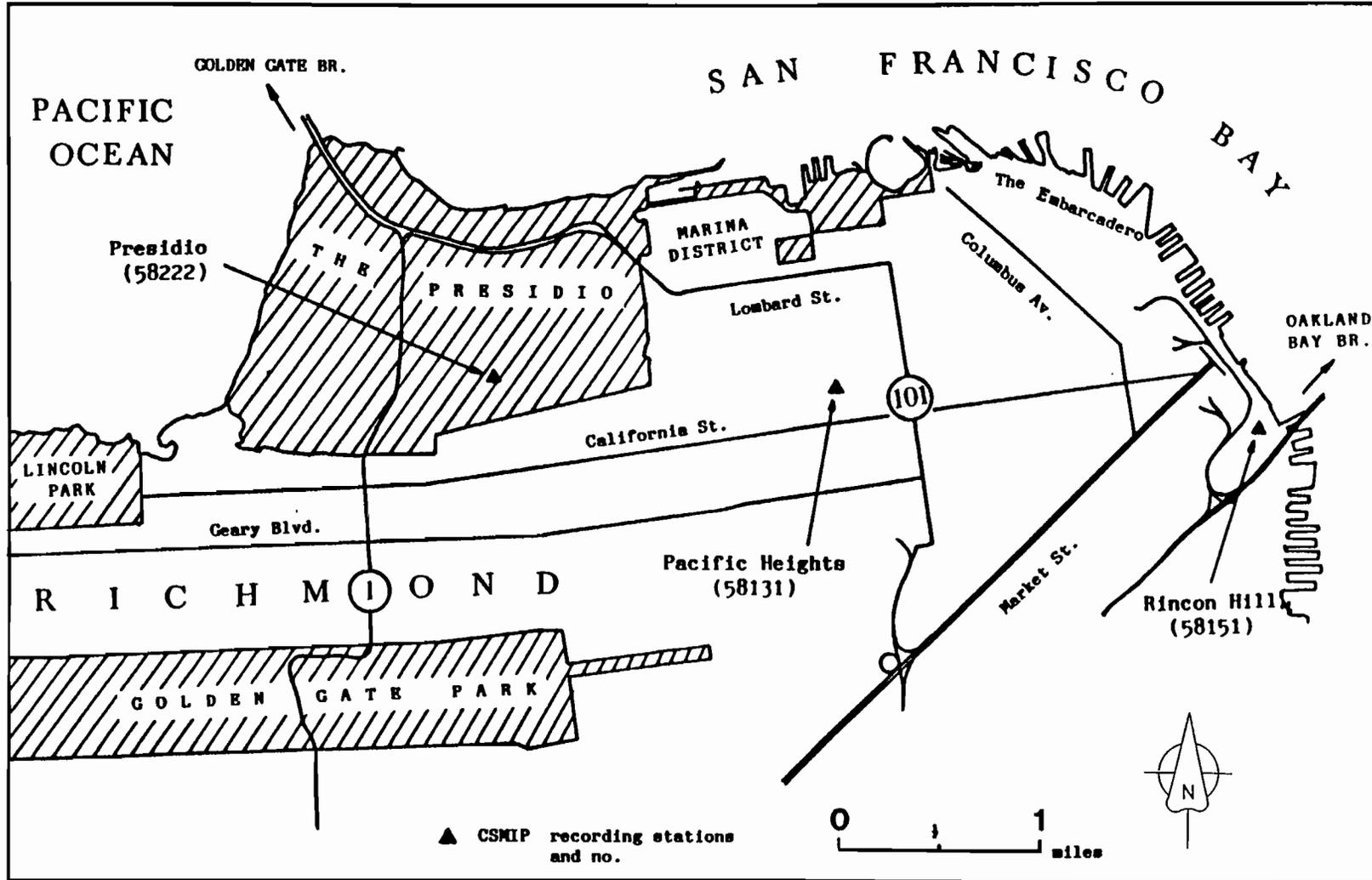


Figure 3-1 : Map of San Francisco City peninsular showing Marina and Richmond Districts.

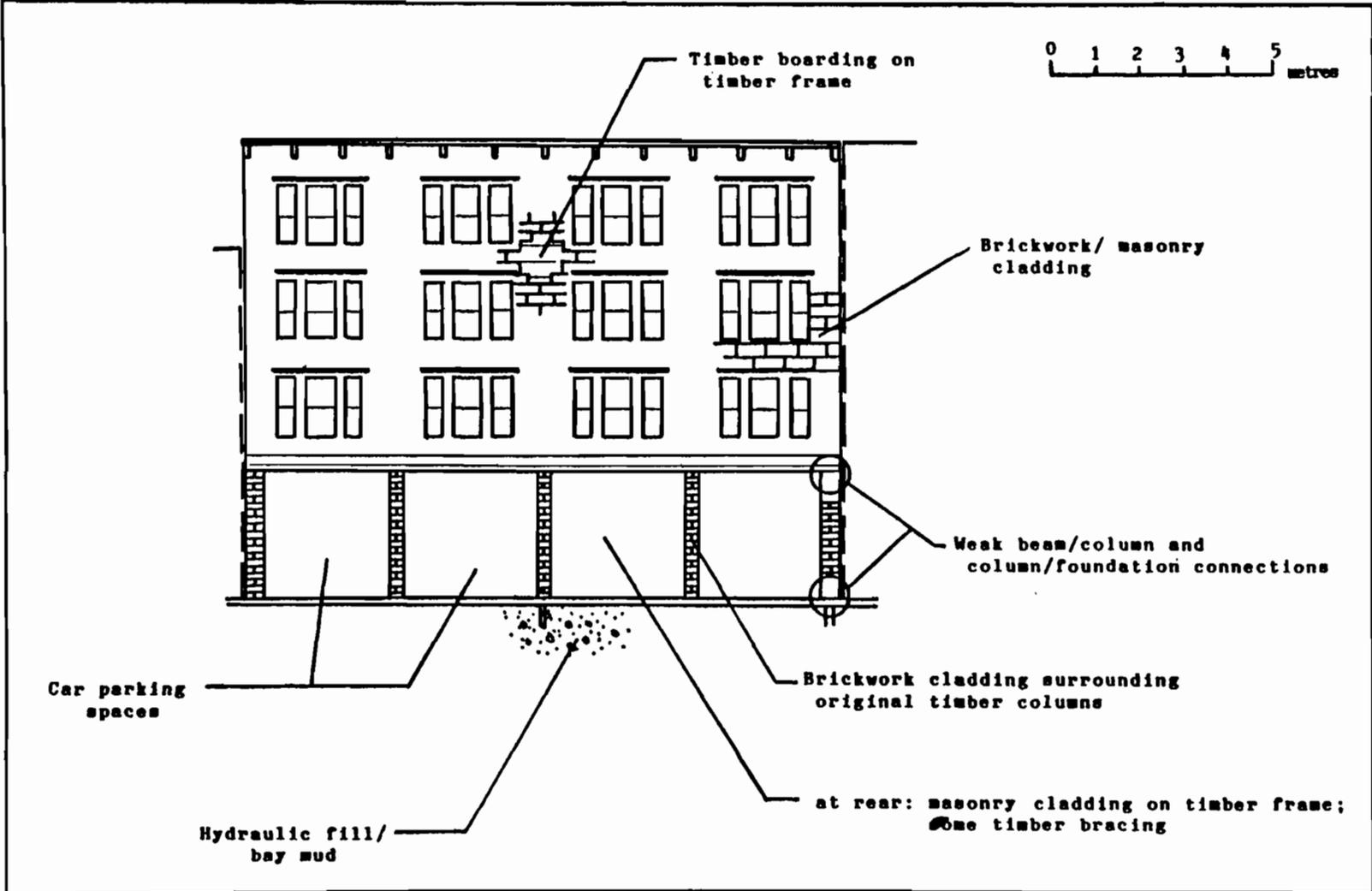
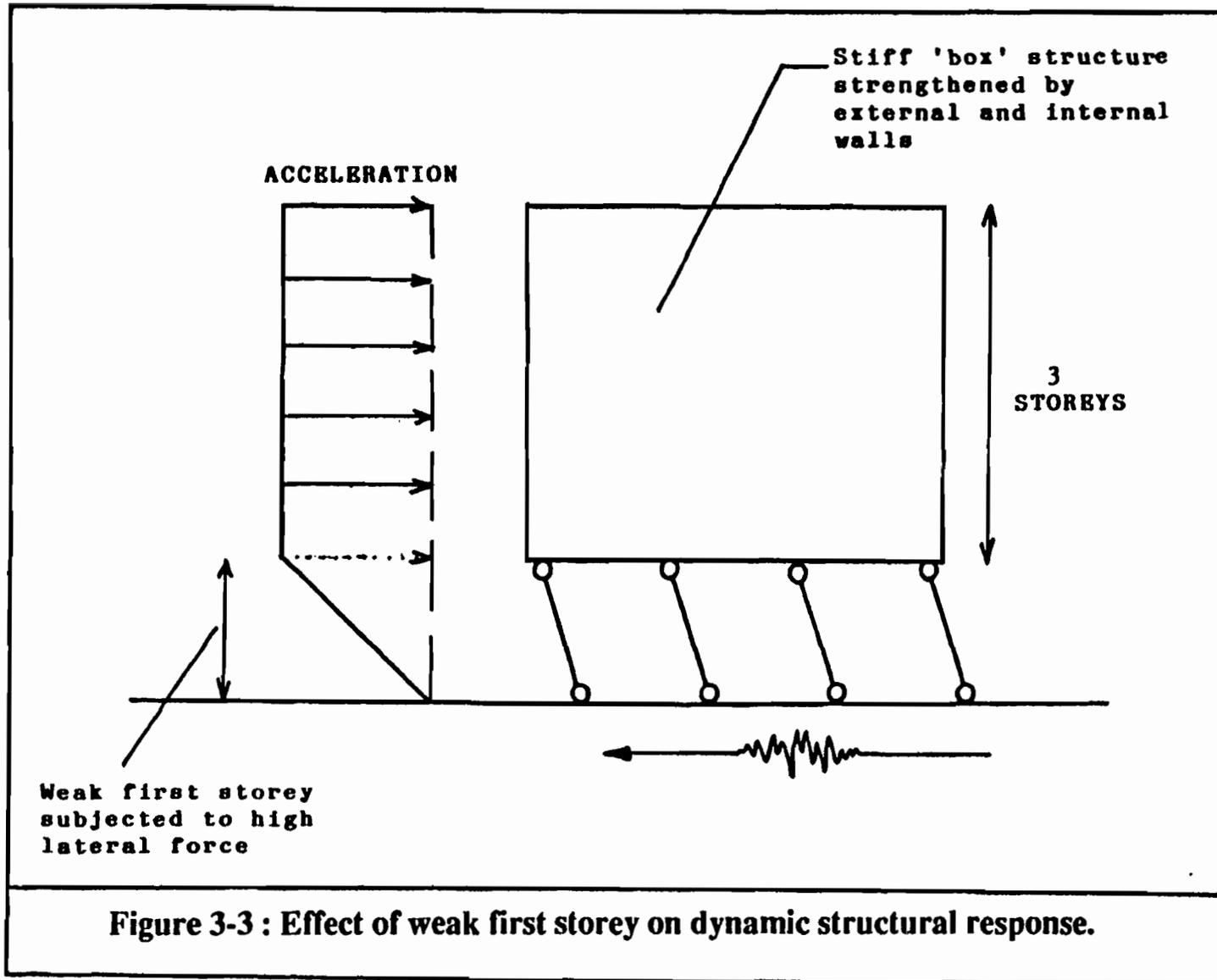
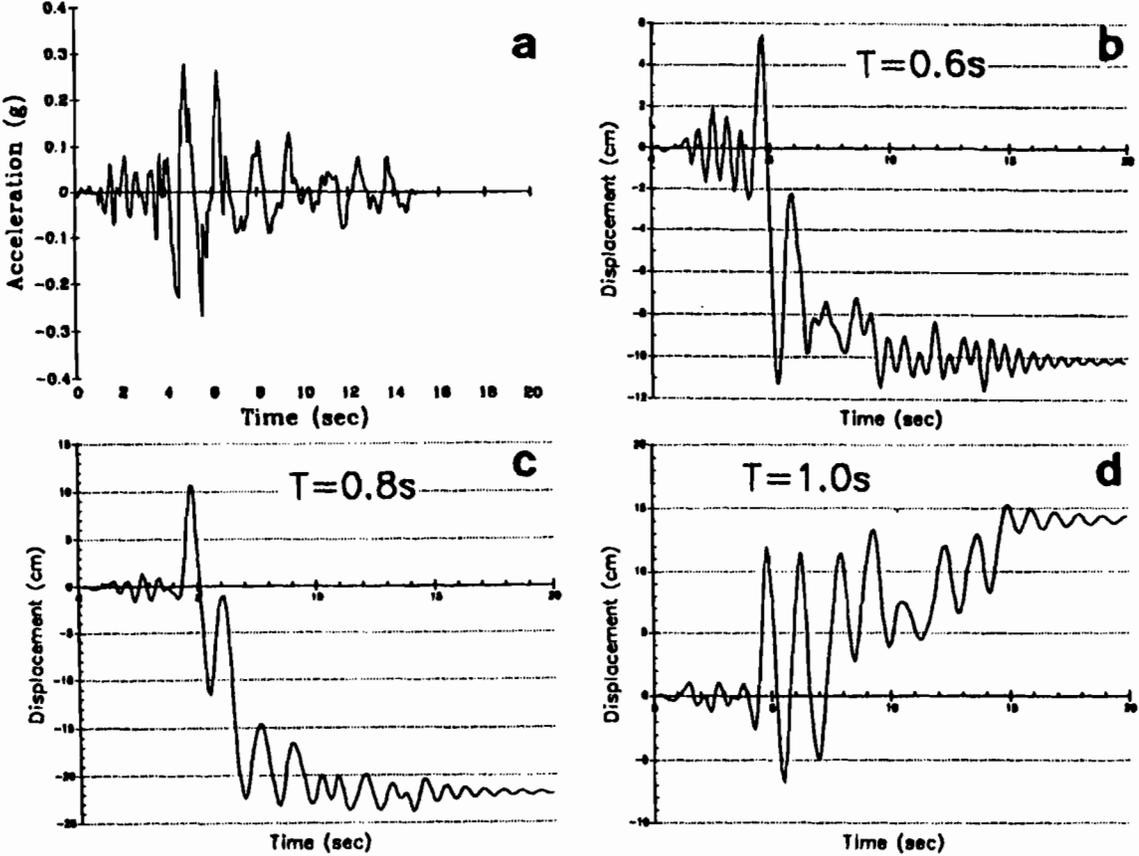


Figure 3-2 : Schematic elevation of typical timber framed 4-storey dwelling in the Marina District, San Francisco.



Loma Prieta Earthquake
Emeryville E-W

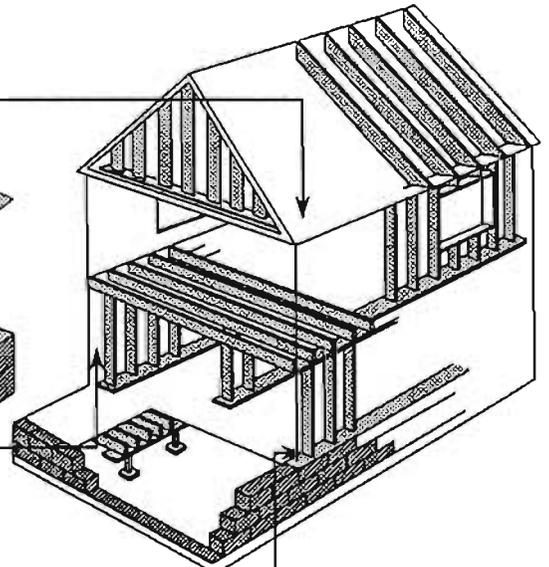
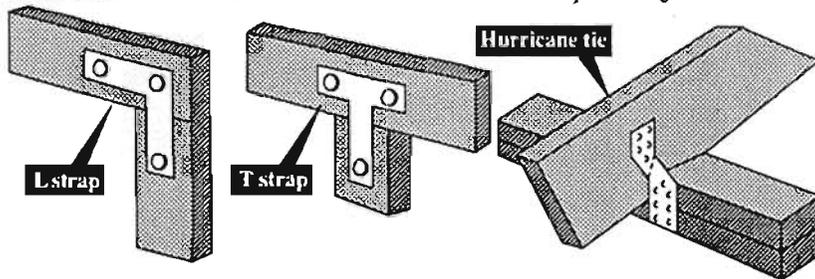


Elastic damping = 5% Yield displacement = 1.5 cm

Figure 3-4 : Inelastic dynamic response of structures with varying initial periods to the Emeryville E-W earthquake record.

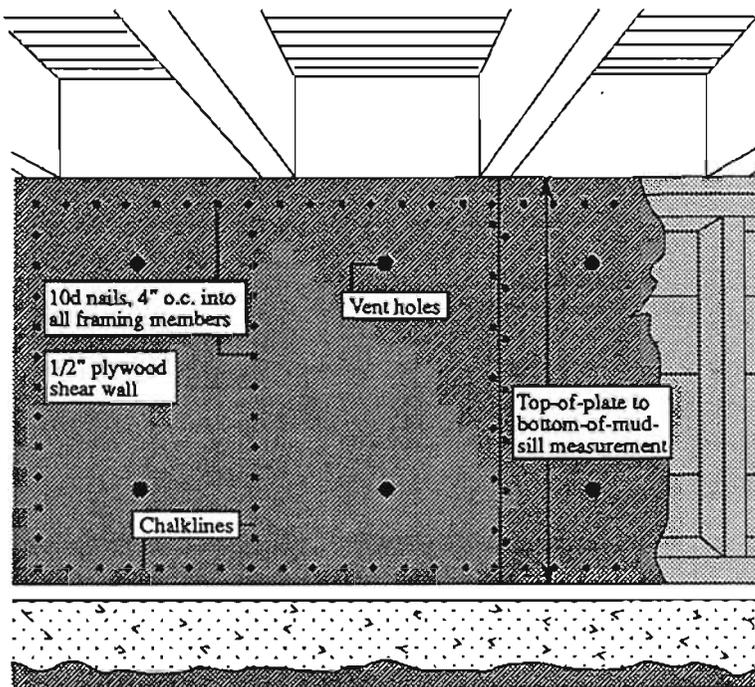
Metal connectors

Metal connectors can reinforce connections between rafters and plate and girders.



Shear - wall Installation

Plywood shear panels provide resistance and counteract lateral forces



Drop - in anchor bolts

Homes can be secured to foundations with drop - in anchor bolts. Inspect foundation first.

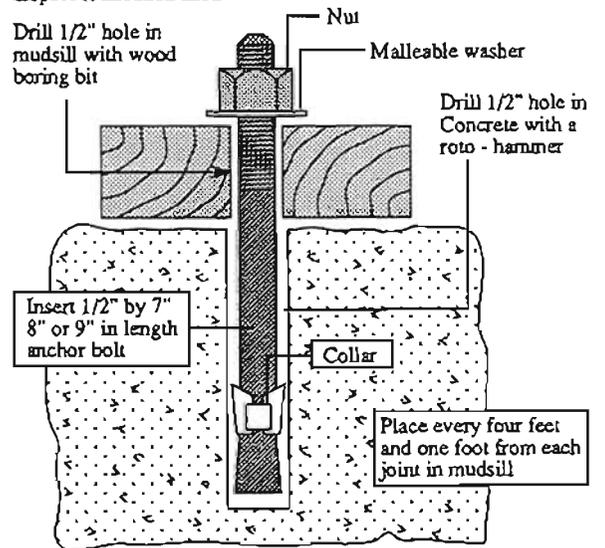


Figure 3.5 : Recommended retrofitting details for timber framed houses.

PLATE 3-1



PLATE 3-2



PLATE 3-3



PLATE 3-4



PLATE 3-5

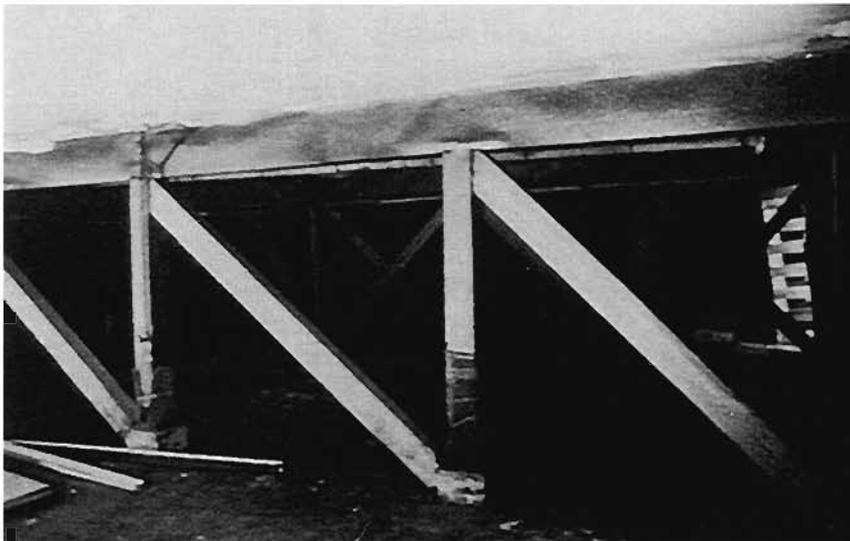


PLATE 3-6

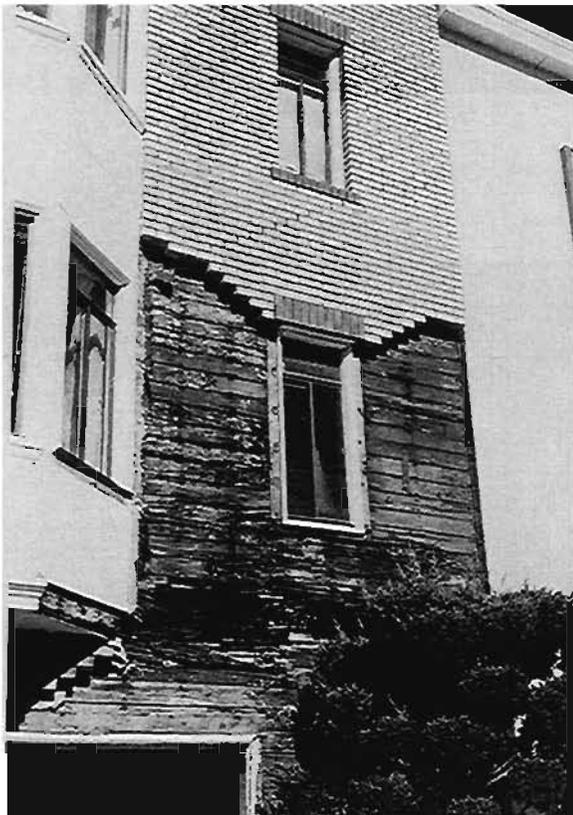


PLATE 3-7



PLATE 3-8



PLATE 3-9



PLATE 3-10



PLATE 3-11



PLATE 3-12

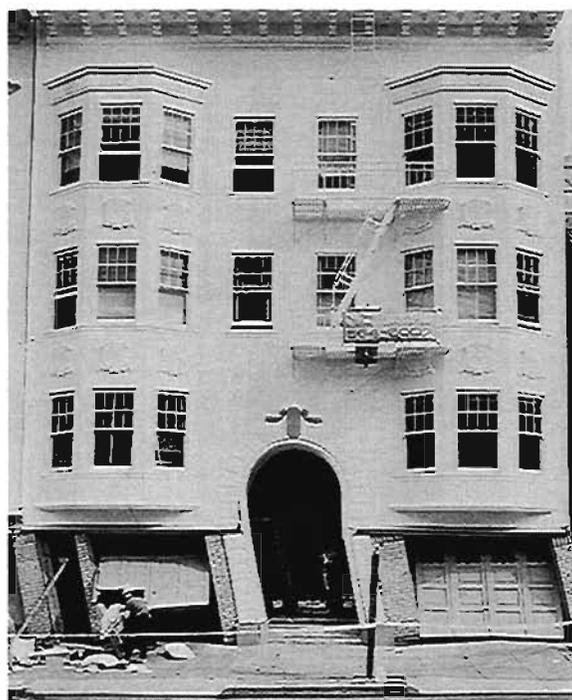


PLATE 3-13



PLATE 3-14



PLATE 3-15



PLATE 3-16



PLATE 3-17



PLATE 3-18



PLATE 3-19

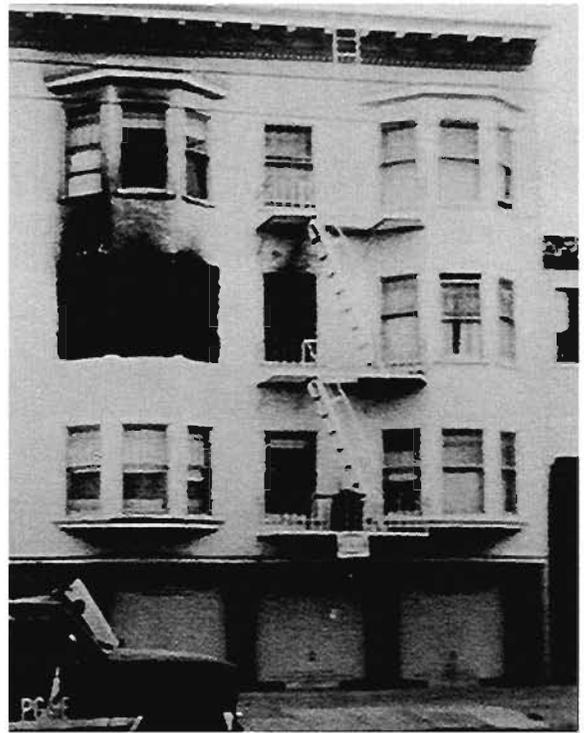


PLATE 3-20



PLATE 3-21



PLATE 3-22

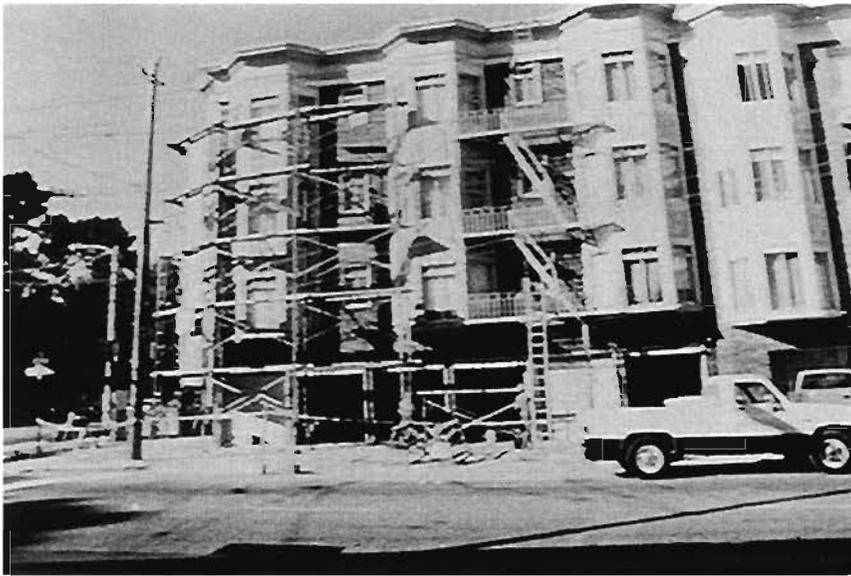


PLATE 3-23

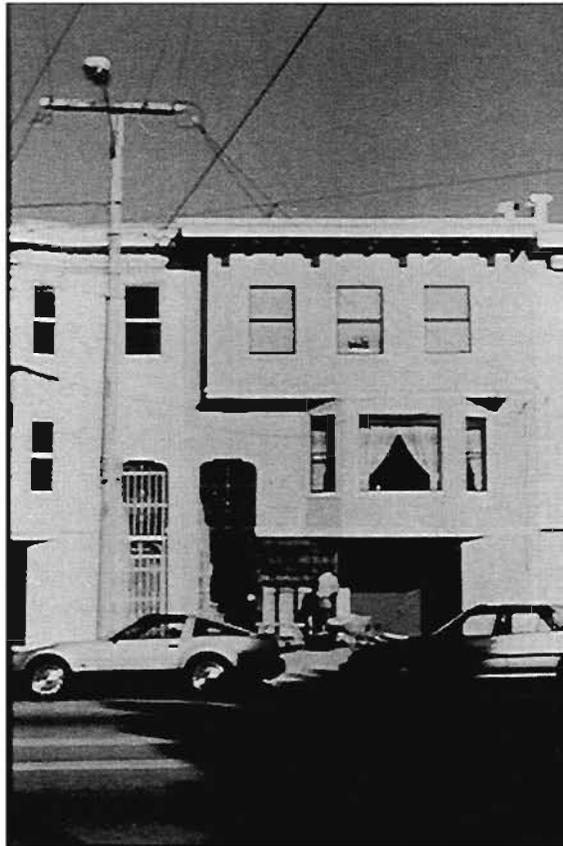


PLATE 3-24



PLATE 3-25



PLATE 3-26



PLATE 3-27



PLATE 3-28

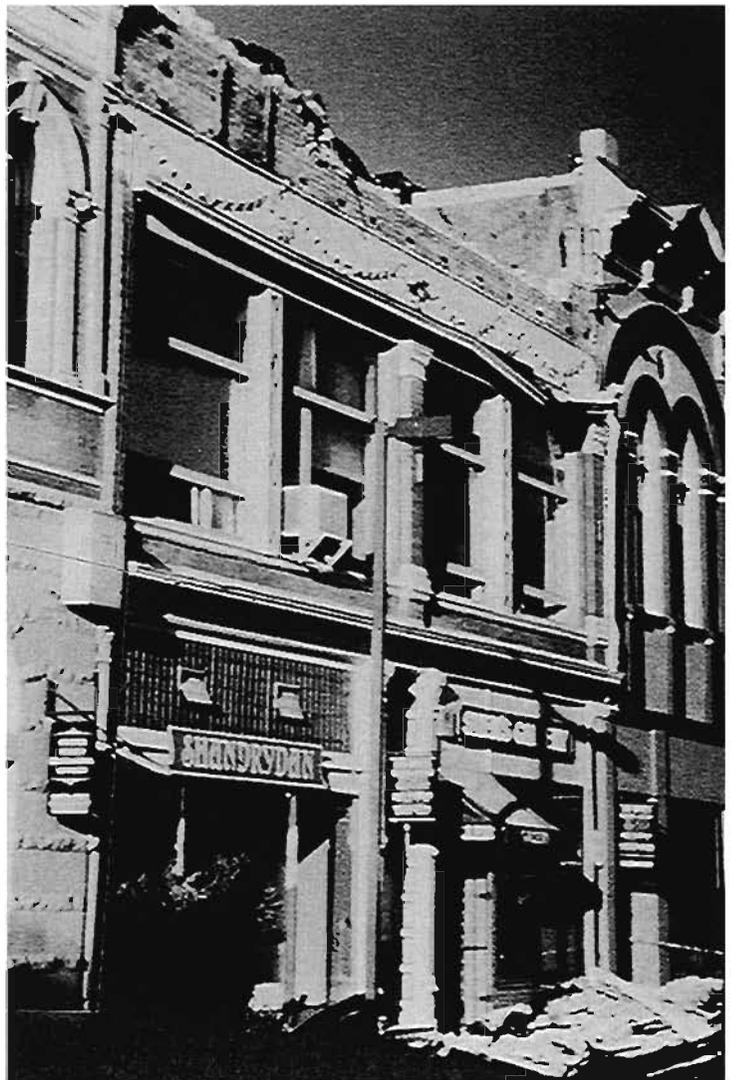


PLATE 3-29

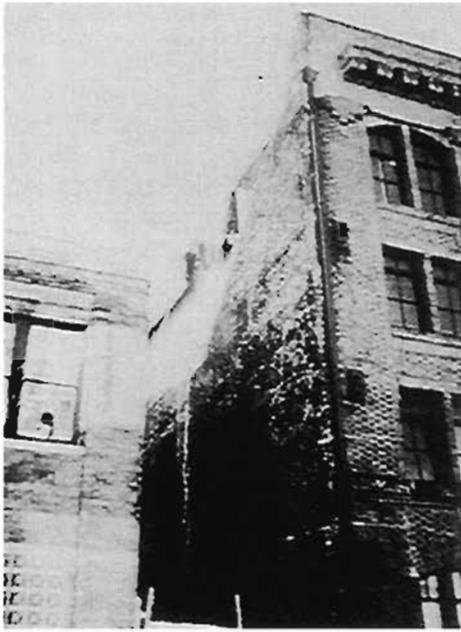


PLATE 3-30



PLATE 3-31



PLATE 3-32



PLATE 3-33



PLATE 3-34



PLATE 3-35



PLATE 3-36

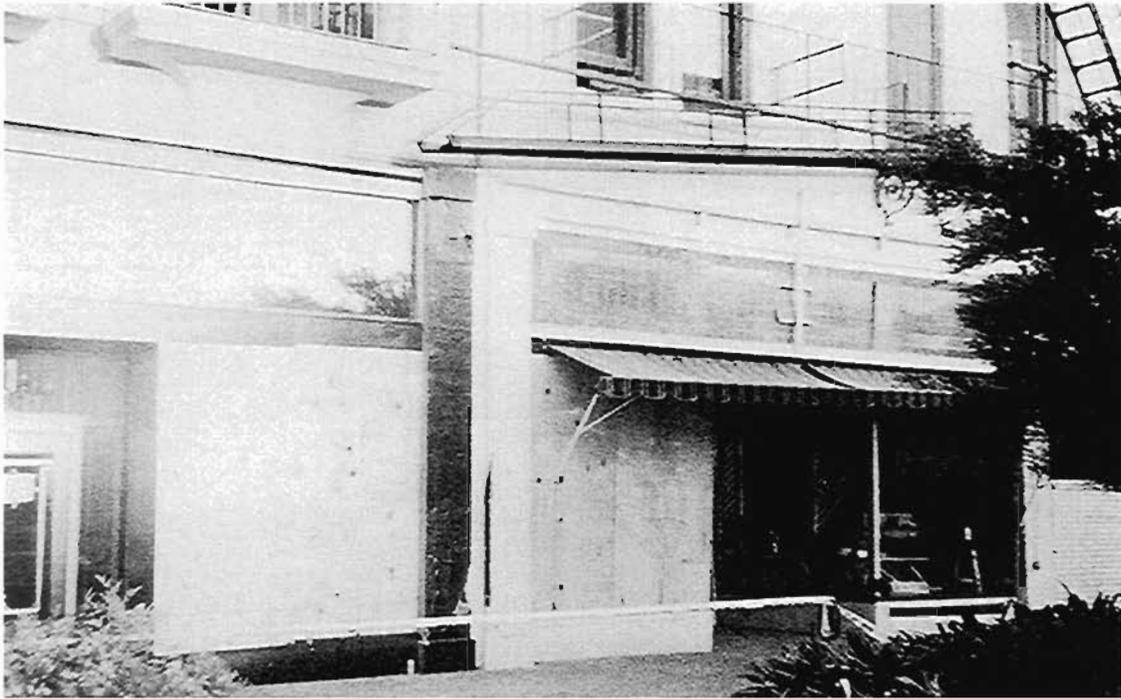


PLATE 3-37

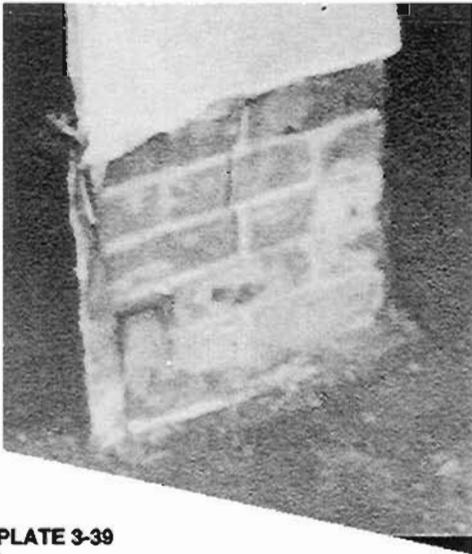


PLATE 3-38

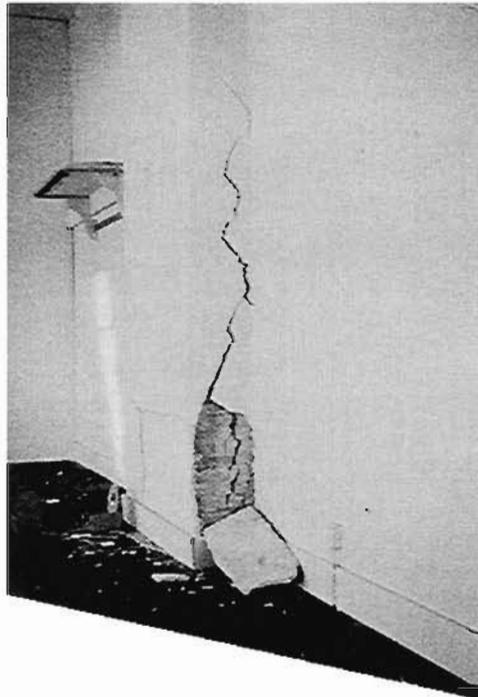


PLATE 3-39



PLATE 3-40



PLATE 3-41



PLATE 3-42



PLATE 3-43



PLATE 3-44



PLATE 3-45



PLATE 3-47

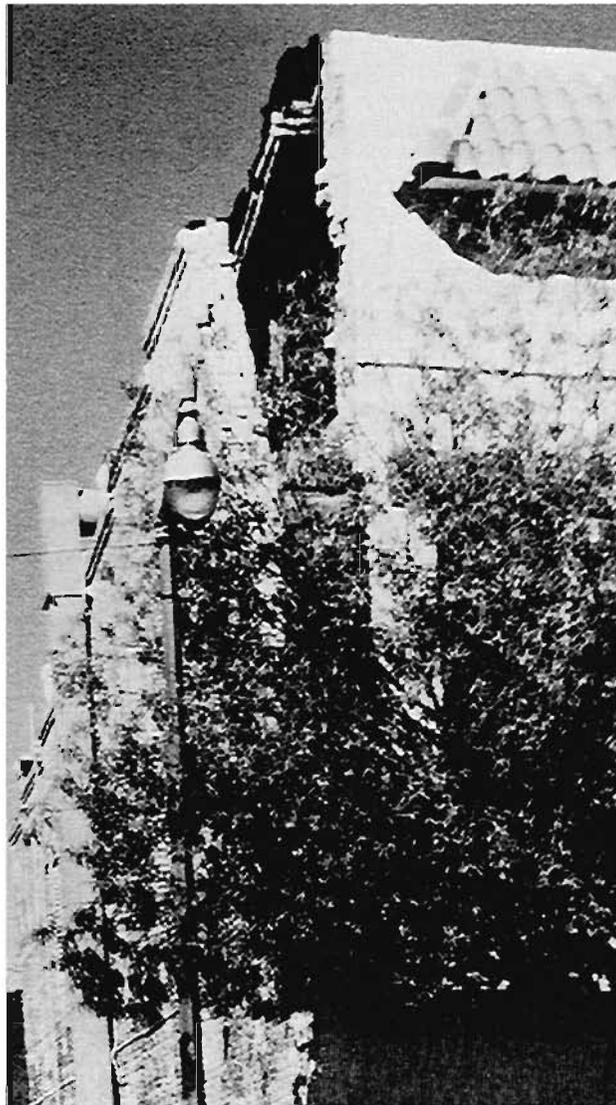


PLATE 3-46

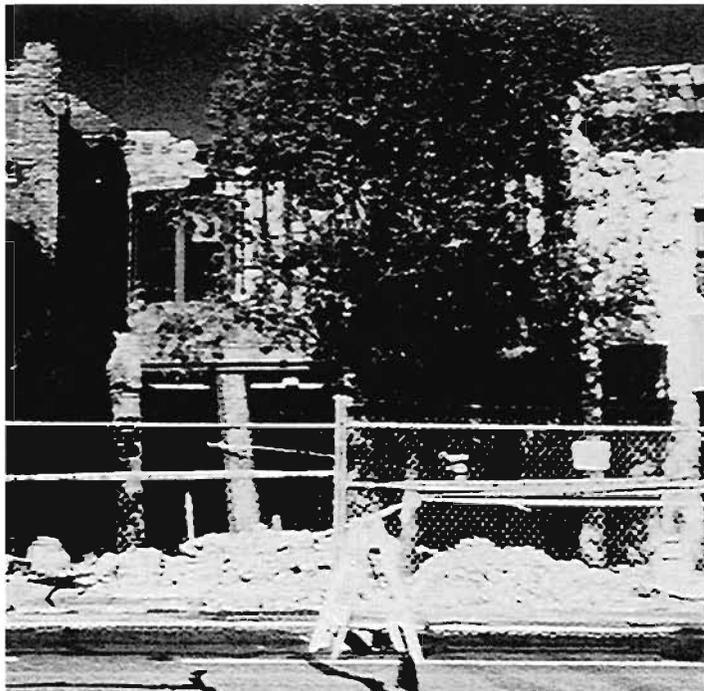


PLATE 3-48

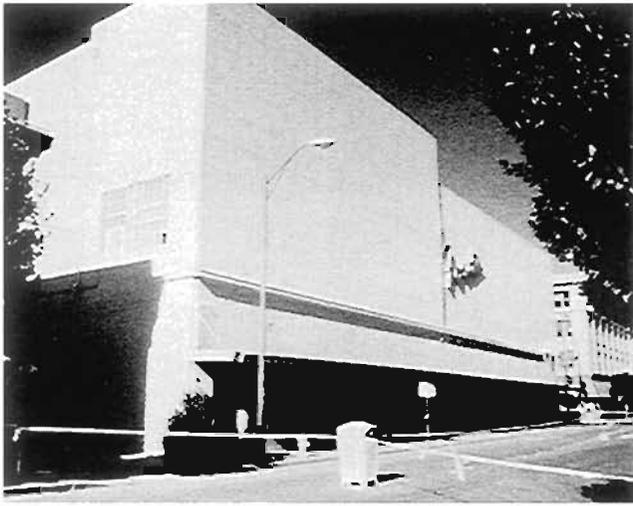


PLATE 3-49



PLATE 3-51



PLATE 3-50



PLATE 3-52



PLATE 3-54



PLATE 3-53



PLATE 3-55

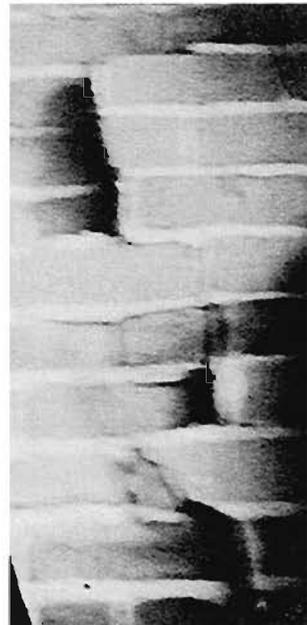


PLATE 3-56



PLATE 3-57

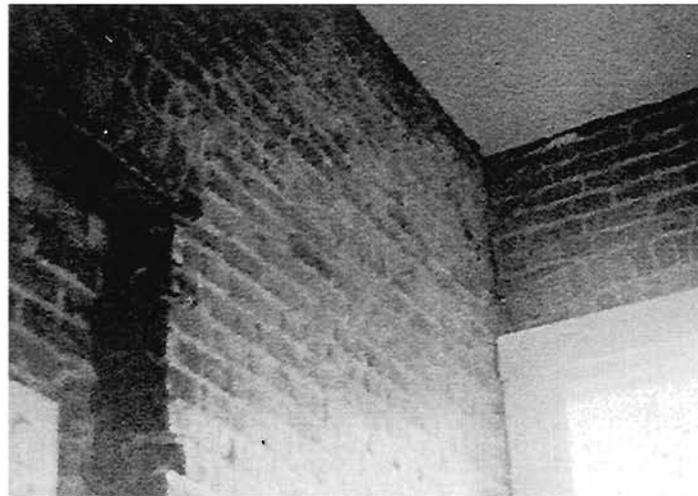


PLATE 3-58



PLATE 3-60

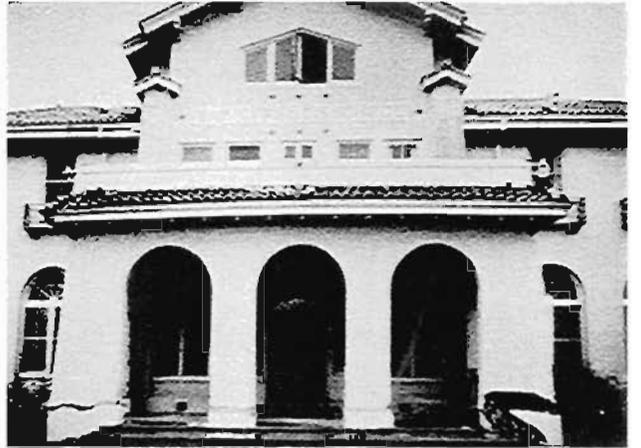


PLATE 3-61



PLATE 3-59



PLATE 3-62



PLATE 3-63



PLATE 3-64



PLATE 3-66



PLATE 3-65



PLATE 3-68

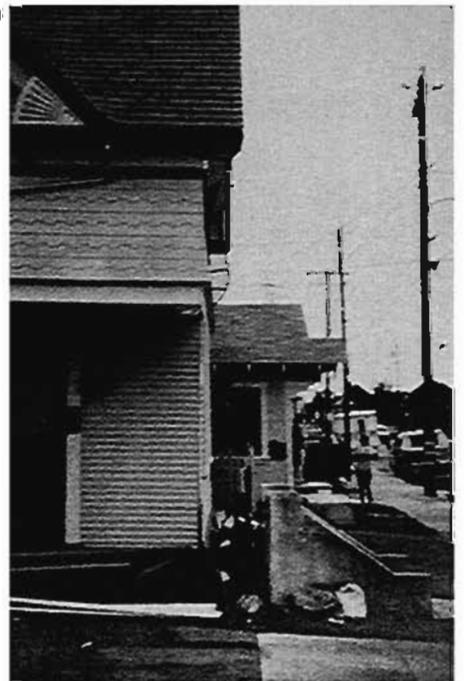


PLATE 3-67



PLATE 3-69



PLATE 3-70



PLATE 3-71

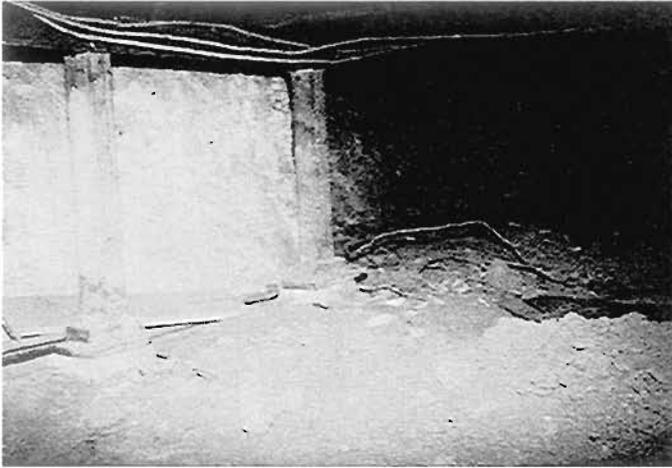


PLATE 3-72

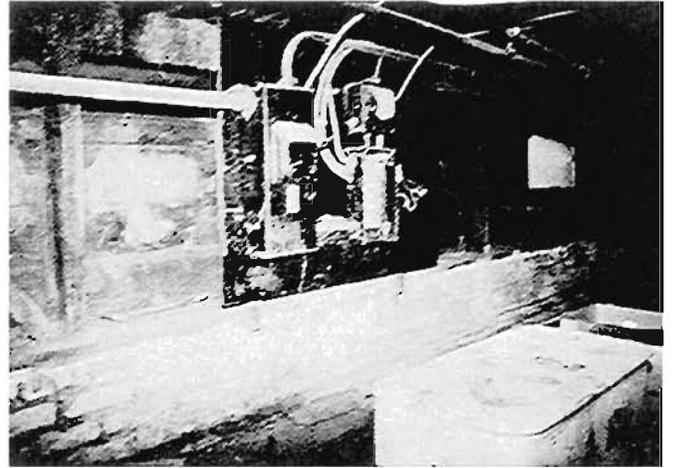


PLATE 3-73



PLATE 3-74

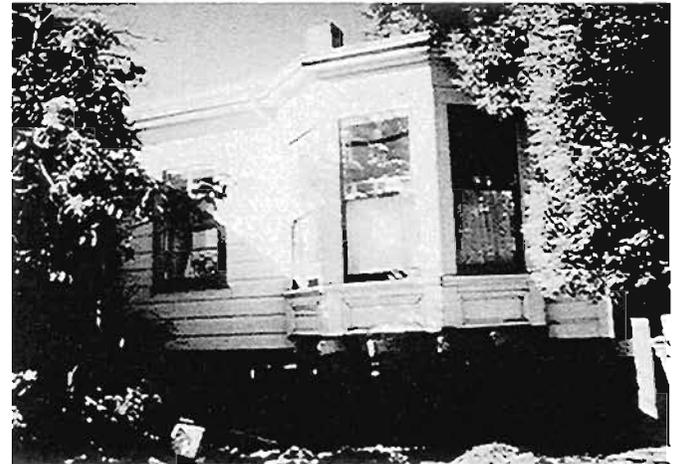


PLATE 3-75



PLATE 3-76



PLATE 3-77



PLATE 3-78



PLATE 3-79

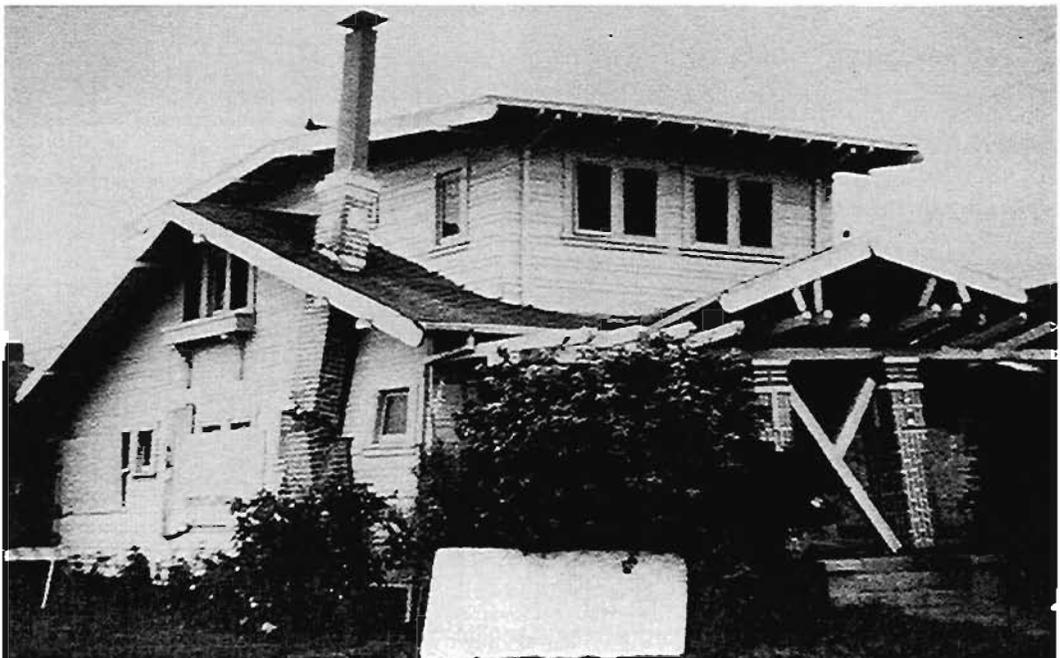


PLATE 3-80

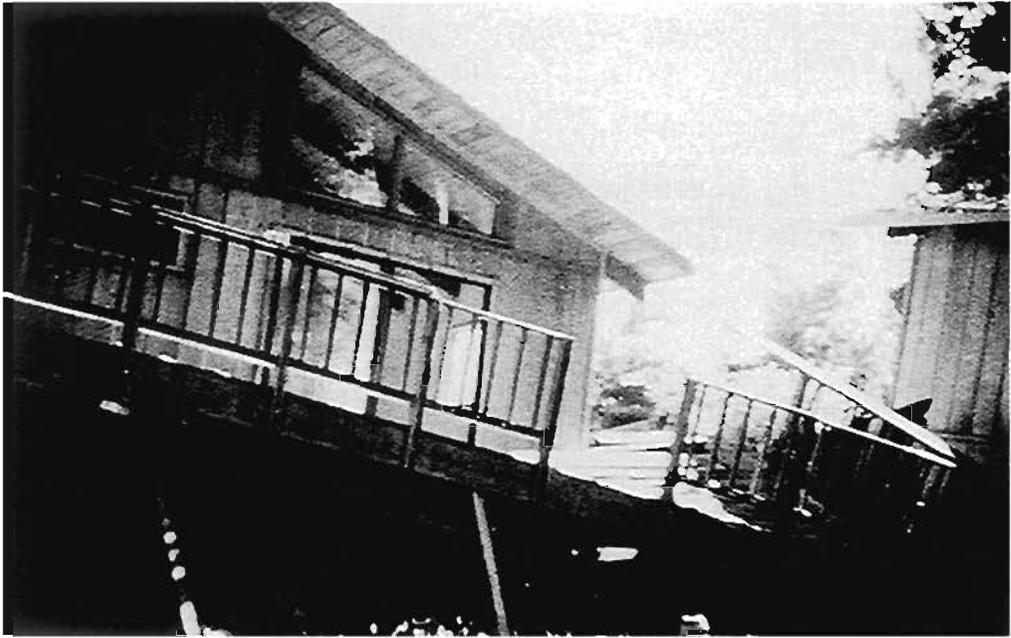


PLATE 3-81



PLATE 3-82



PLATE 3-84



PLATE 3-83

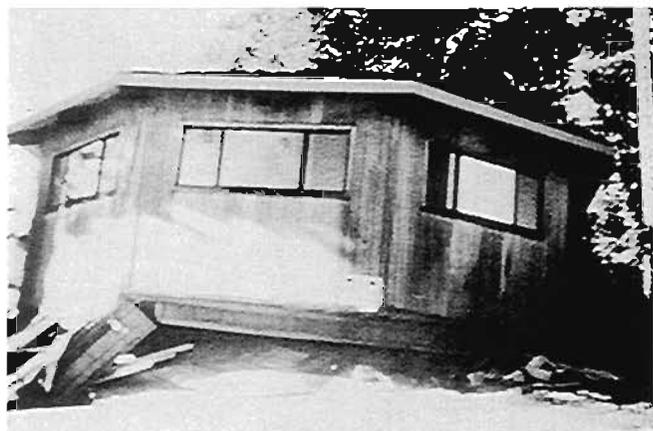


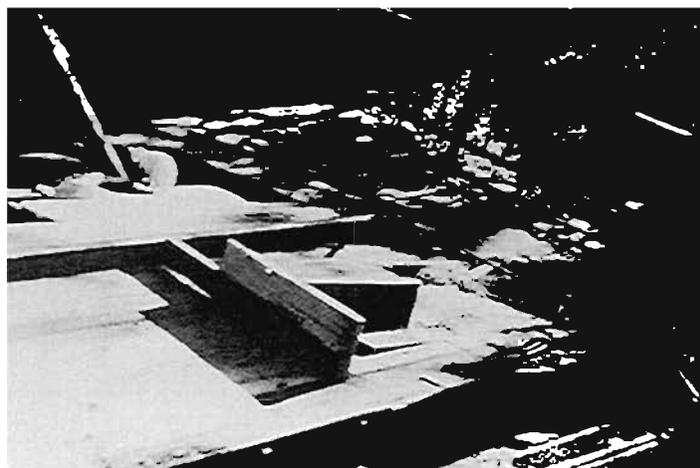
PLATE 3-85



PLATE 3-86



PLATE 3-87



PERFORMANCE OF ENGINEERED BUILDINGS

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4.1 INTRODUCTION

4.1.1 Regional Classification of Structures

In San Francisco and Oakland, where the peak ground acceleration on soft soils was typically 0.25 to 0.33g, the main interest is in ascertaining what damage has occurred, because any design features to which the damage is attributable need to be avoided in future. However in the area closer to the epicentre, where horizontal accelerations were 0.45 to 0.65g, and including Los Gatos in the North, Santa Cruz in the South West and Watsonville in the South, a reasonable level of damage might be expected. Here the interest shifts to structures which have performed well because the structural features they contain may be considered successful.

4.1.2 Foundation Conditions of Buildings Examined

It was generally possible to establish neither the exact ground conditions nor details of the foundations of the buildings considered. However, general conditions in the epicentral region can be inferred from some typical drawings and the comments of local inhabitants. In Watsonville, outline details of the telephone exchange (Fig 4.1) indicated it was constructed on pad footings at different levels, interconnected by the beams. The foundations were backfilled with sand to ground floor level. The erratic foundation level suggested the level of sound strata was 2m to 5m down. This depth would only have been reached in the older buildings with basements. Most apparently had none. As the telephone exchange was the most significant of the more recent buildings in the area it is likely that no buildings were piled. This conclusion can be tentatively extended to the whole region of the epicentre including Santa Cruz and Watsonville.

No incidences of damage to engineered buildings were found in Los Gatos, where the buildings were generally on better foundation material, typically weathered rock.

Sufficient comments were received both in Watsonville and Santa Cruz to suggest

that the old downtown areas, dating from just before the turn of the century, were probably built on the poorest ground (with the selection of sites being mainly dependent on the existing roads) and that the more modern structures were on somewhat firmer ground. The old downtown area of Santa Cruz had been subject to flooding in recent years and some structures had suffered cracking even before the earthquake.

In the San Francisco Bay area ground conditions vary widely. These are areas of beach deposited sand and of alluvial clays. There are outcrops of the underlying sandstone and shale, and on the present shoreline large areas of artificially made land of hydraulic fill. It was noticeable that damage to buildings was greater on poorer ground, the worst being on the artificial fill.

4.1.3 Facades

In San Francisco the main damage was to the facades of buildings and was in four forms.

- (i) By far the most common form of damage was local spalling of the cladding between abutting buildings, attributable to impact pounding when the buildings moved out of phase (Plates 4.1 and 4.2). The damage was most pronounced when the buildings were likely to have significantly different periods of vibration (Plates 4.3 and 4.4).
- (ii) A type of damage well recorded in the literature is the "X-cracking" between windows, both of masonry clad and load-bearing masonry buildings (Plate 4.5). This occurred in a few buildings of both types. One reason for the lower incidence of this type of cracking than might have been expected is the relatively small window area in many of the older buildings.
- (iii) In some buildings small pieces of mosaic had fallen away and monumental masonry cladding was cracked. Cases were noted of quite extensive damage to artificial masonry cladding (Plates 4.6 to 4.9). Both are attributable to the rigidity of the facade causing it to attract load (Plates 4.6 and 4.7).
- (iv) A particular type of facade damage in some isolated buildings was local spalling of masonry at corners, in buildings in which the masonry appeared to be continuous. This was attributable to longitudinal shear, flexural out-of-plane movement and the possible weakening due to local weathering at the exposed corners (Plate 4.10). Very often the brickwork was two bricks thick and only the outer layer was affected (Plate 4.11).

This type of damage is best avoided by severing the continuity of masonry in each storey; confining it to bands. This is illustrated in Plate 4.16, where the damage occurred mainly due to movement in the lowest storey.

- (v) Cracking (Plates 4.13 and 4.14) had occurred around masonry panels due to differential movement of panels and the framework illustrated in Figure 4.2. An interesting vertical crack beneath a substantial metal chimney was noted in one building (Plate 4.12), probably initiated by rocking of the chimney following pullout of the holding down bolts.
- (vi) The last form of damage, mercifully uncommon on this occasion, was the breakage of glazing. Nevertheless a few buildings lost most of their glass (Plate 4.15). The structure of these buildings did not differ appreciably from contemporary structures in which the glazing was undamaged. There is no evidence in the building illustrated of extensive damage elsewhere on the facade.

4.1.4 Torsional Response of Asymmetric Buildings

Torsional structural response, arising from pronounced asymmetry in the plan layout of lateral load-resisting members, damaged, or was evident in the recorded response of, a number of buildings in Oakland. Two buildings in particular have been selected for detailed discussion, namely the 15-storey Pacific Bell Telephone building on the corner of 17th and Franklin Streets in Oakland and a 2-storey masonry and steel office building in the Lake Merritt district. In the latter case, recordings of ground motions and structural response accelerations were made by the California Division of Mines and Geology (CSMIP Station 58224, see Figure 2.2 and Table 2.1).

4.1.5 Response of Irregular Buildings

A number of buildings examined had forms of irregularity other than those which can be represented in analysis by design torques. These include vertical irregularity in the form of soft weak stories, set-backs (the most pronounced being a multi-storey penthouse) and sloping sites. There were several forms of structural discontinuity, mass irregularity and one building with an exceptionally large atrium. The buildings examined in this section generally lacked irregularities as severe as those in the non-engineered buildings discussed in Chapter 3, and only when there were other defects was damage appreciable.

4.2 HEAVILY OR MODERATELY DAMAGED BUILDINGS

4.2.1 General

Recorded and observed instances of structural damage to engineered buildings were rare in San Francisco and Oakland; however evidence from the buildings not founded on soft ground suggested the damage there was more extensive than appreciated in the immediate aftermath of the earthquake.

- (i) In a randomly selected modern car park structure in downtown San Francisco, in which no damage had been reported (possibly located on rock), evidence was found of substantial cracking.
- (ii) The fact that in a multi-storey building there was evidence of plasticity in the strong motion records for the superstructure, though no damage in this structure is reported.

The buildings discussed in this section include two in which the damage is partly or entirely due to vertical irregularity, one to torsional irregularity, and one to the use of an inappropriate structural form for an area of high seismicity.

4.2.2 Amfac Hotel, San Francisco

The Amfac Hotel is located to the South of the San Francisco International Airport, on a strip of made ground. An unusually high three-storey penthouse at the hub of this three-winged hotel accommodated a service room. In a tank room above (Plate 4.17) a 2000 gallon water tank which had not been properly anchored collapsed into the elevator engine room below (Plate 4.18). The hotel had to be closed as a result. Fortunately no-one was in the elevators despite a convention in the hotel at the time. In terms of building codes the tank on the roof constituted a mass irregularity, and the pent house was tall enough to constitute a vertical irregularity. It is noted only single-storey pent houses are exempted from the UBC/SEAOC regularity rules.

4.2.3 Building on Bluxome Street, San Francisco

In Bluxome Street, in the South of Market area, a reinforced concrete structure, comprising circular columns with flared column heads and flat slab floor construction, is reported to have suffered structural damage. This was in the form of bad cracking through two of the column heads. In both cases the damage was confined to one side of the column and the slab was not considered to be in danger of collapse. The building was still in use when inspected. Due to the difficulty of providing ties to adequately confine the concrete in this type of construction it is not normally used in zones of high seismicity. It is not included either in Eurocode 8 or in the earthquake resistant chapter of AIC 318 (1989).

4.2.4 Pacific Bell Telephone Building, Oakland

The Pacific Bell Building is a 15-storey moment-resisting steel framed building with composite columns. It has structural reinforced concrete shear walls at the rear and on an adjacent side, and is connected by an elevated walkway at 7th Floor level to an adjacent building, visible in the overall view of the rear at the crossroads of 19th Street and Franklin Street in Plate 4.19. Plate 4.20 shows the Franklin Street facade at the junction with 17th Street. (17th street runs from left to right). The facades facing 17th Street and Franklin Street have composite steel and concrete columns at approximately 6m intervals (Plates 4.20 and 4.21). Set back at a distance of 3.5m from this line of columns on both facades are a series of narrow shear walls, approximately 4m wide without openings, which are constructed throughout the full height of the building. Each shear wall is separated by 1.5 to 2m, to allow ground level access to the building, visible in the background of Plate 4.22. The first storey has a height of about 5m, which exceeds the 3.5m height of the upper stories (Plate 4.20).

The continuous shear walls on two sides of the building give very large additional lateral stiffness to the structure, whereas the non-continuous shear walls on the street facades provide relatively little additional stiffness, especially as they were set-back from the row of external columns. Hence the overall plan centre of stiffness is eccentrically located towards the rear and off-street side of the building, whereas the mass centre is located approximately symmetrically assuming a uniform distribution of loading. The non-continuous shear walls were incorporated in the building design primarily in order to reduce the stiffness asymmetry, but on the evidence of the severe damage caused to the outer row of columns on the Franklin street facade of the building (Plates 4.21 and 4.22) torsional response behaviour contributed significantly to the building's dynamic earthquake response. These columns showed evidence of structural damage due to yielding at the beam/column connections at first floor level (Plate 4.22), as well as loss of concrete cover on at least two sides of the column. The non-continuous shear walls were also damaged (Plate 4.23), with spalling of surface concrete exposing the reinforcing bars.

Further evidence that this building experienced large torsional responses during the earthquake is given by Plate 4.24, which shows the corner column/first floor beam connection at Franklin Street and 17th Street. Inelastic deformation has caused permanent movements of the column above the floor/beam connections, with evidence of rotational as well as lateral displacements of the floors above the level shown in the plate.

At the time of inspection this building was closed for repairs. Damage was confined to the open facades in the first storey. There was no external evidence of damage at higher levels, or to the shear wall at the rear of the building (Plate 4.19).

4.2.5 Palomar Hotel, Santa Cruz

The most notable older building still standing is the Palomar Hotel (Plate 4.25), occupied before the earthquake largely by older residents. This is a massive structure of seven storeys and a basement. It has a reinforced concrete frame cast monolithic with insitu upstand walls on the facade and was constructed about 1928. It had a continuous strip foundation around the outside and pad footings under the internal columns. Although not reinforced to today's standards the reinforcement is considered substantial compared to contemporary structures. It lacked a ground floor soft storey, but pronounced diagonal cracking was noted in the appreciable sections between the relatively small windows at first floor level which accounted for no more than 50% of the plan area on the street facade, though compared to the 30% in the storeys above this clearly constituted a local weakening (Plate 4.26).

A member of EEFIT who had visited Santa Cruz before the arrival of the main team, witnessed the movement of this building during an aftershock and stated that the sway was appreciable. Inter-storey drift was detectable, demonstrating that despite its apparent solidity the building responded as a frame. Initially demolition was considered because the cracks, which were being monitored, opened progressively during the aftershocks. In February 1990 it was decided that the building would be saved and its structural integrity reinstated by epoxy grouting.

4.2.6 Watsonville Hospital

This hospital, on poor ground, is reported to have suffered significant structural damage.

4.3 LIGHTLY DAMAGED BUILDINGS

4.3.1 Introduction

Damage to engineered buildings was mostly light and the following are a representative selection from the San Francisco and the epicentral regions.

4.3.2 Shell Building - Corner of Bush Street and Battery Street, San Francisco

The Shell Building (Plate 4.27) is a tall office block in downtown San Francisco. The exterior facade of hollow porcelain bricks was cracked up each of the front corners of the building. Damage to the bricks had also occurred in the corners of the window openings, on the exterior corner at ground level where the brickwork had broken and fallen off, and from pounding by the next building on Bush Street at the 3rd and 8th floor level, but most damage was at the 8th floor.

Inside damage was localised to cracking of the partition walls and to the stairwell, which ran down beside the elevator shafts. Being the junction between the main structure and the stiff elevator shafts this is where damage would be expected. Some of the internal marble cladding had also come down. There was a great deal of work going on in the building, but a large part of this was associated with a planned renovation which was in progress when the earthquake struck.

Occupants of the building during the earthquake reported the motion was mainly parallel to Battery Street and had caused filing cabinets to be displaced.

4.3.3 Hearst Parking Center, 45 3rd Street, San Francisco

This reinforced concrete structure offered an unencumbered view of damage to a concrete frame. The Center has eight floors of parking above ground floor, partly occupied by a snack bar. Cars make their way up and down a double helix ramp enclosed in the rear half of the building. The other half is built around a well above the snack bar. The elevator shafts are between the well and the ramp.

Damage to the frame was not great and was confined to cracking, except where a slither of concrete had spalled off one of the beams near the spiral ramp, but no steel had been uncovered. The cracking occurred in the columns round the well, towards the front of the building. Plate 4.29 is a photograph of one of the top storey columns; X-cracking can be clearly seen at the head of the column. No cracking was observed at the base of the columns or in any of the exterior columns. The degree of cracking increased from the 3rd floor, where only 4 of the 12 internal columns were affected, to the 8th, where the cracking extended to all the columns round the well and the wall of the elevator shaft. The cracking was consistent with the principal motion being parallel to 3rd Street, which runs NW-SE.

4.3.4 Lettunich Building, Watsonville

This is a four storey reinforced concrete framed building of uncertain date of construction. The facade was of chicken wire reinforced stucco, made to imitate Portland stone, applied to a timber substrate. In places this system had completely failed and within the massive sham masonry piers reinforced concrete columns of small cross section could be seen (Plates 4.8, 4.9). There was no apparent damage to the structure. Although of heavy appearance this type of cladding is exceedingly light and ideal for a seismic zone.

4.3.5 Municipal and County Building, Santa Cruz

This five storey structure (Plate 4.30) with a basement is the most heavily engineered and the largest building in Santa Cruz. It has a ductile moment resisting frame of reinforced concrete infilled with precast concrete panels, lightly bolted to the frames (Plate 4.29) and with a mastic seal internally. The frame was designed to carry the lateral seismic loading. During the earthquake severe noise is reported to have occurred, and was attributed to:

- (a) the grinding of the infills within their frame due to the relative movements shown in Figure 4.2; and
- (b) to the shattering of the plate glass light diffusers.

The damage to the structure was minor, being :

- (i) cracking of the staircase across the landing between flights, where it is of no structural significance and where there is normally only light reinforcement (Plate 4.32).
- (ii) an uplift cone around a bolt at a half joint, attributable to the combined effect of reversal of shear and the high component of the uplift force (Plate 4.33).
- (iii) crushing of some calcium silicate bricks around the stair towards the top of the building. These bricks are known to be brittle and for that reason their use is disallowed in Los Angeles (Plate 4.31).

- (iv) light cracking at the bottom of columns at the pile caps and loss of cover to the tie beams between the pile caps (Plate 4.34).

4.3.6 Large Three storey Building at Intersection of Soquel and Ocean Streets, Santa Cruz

This is an irregular structure situated at the intersection of obliquely intersecting roads. It contravenes the spirit or the letter of many of the guidelines in seismic codes, with a facade stepped in plan (Plate 4.35). Besides being of flat slab construction with internal drops, it has cantilevered corners, a stepped foundation (Plate 4.35), a swimming pool at first floor level supported by deep downstand beams (Plate 4.37) and columns (Plate 4.36). These are irregularly infilled with masonry to form shear walls. The columns in the facade were similarly infilled over most of the perimeter (Plate 4.35, 4.36). They successfully carried the lateral load from the entire building. No damage could be found in the facade or in the car park below, except for two small vertical cracks in the masonry. The situation, at the top of a slight hill, suggested the ground was firm.

4.3.7 Hobees Restaurant Complex, Santa Cruz

This large three storey irregular reinforced concrete structure (Plate 4.38) was a multi-level shopping precinct. There were external balconies supported by circular columns. Besides minor cracking in one masonry infill the main problem was the extrusion of the elastomeric material around the windows. The windows however were uncracked.

4.3.8 The Travelodge, Ocean Street, Santa Cruz

The Travelodge was a two storey reinforced blockwork building, in plan measuring about 70m by 15m (Plate 4.39). The owner described the foundation as a slab. The only damage was diagonal cracking in the masonry between the windows half-way along the front face (Plate 4.40), near the stairs. This was attributed to the absence locally of the first floor slab.

4.3.9 The Islander Motel, Ocean Street, Santa Cruz

The Islander Motel was a two storey, L-shaped building (Plate 4.41), with reinforced concrete transverse walls and with blockwork walls longitudinally. In front was a long canopy supported externally by two columns. The proprietress reported that the roof above the bathroom on the road elevation had lifted off during the earthquake and fallen back into its original position (Plate 4.42).

4.3.10 Motel on the Slope to the South of West Cliff, Santa Cruz

This two storey motel (Plate 4.43) was a long narrow building which stepped up the hillside. There was minor cracking in the walls, probably due to the discontinuity at the steps in the roof slab.

4.3.11 Dominican Hospital, Santa Cruz

A four storey moment resistant framed structure at the intersection of Soquel Avenue and Highway 1, owned by the Dominican Hospital, was the only engineered structure suffering significant damage. It was designed in the 1960's for seismic zone 3. Some columns are cracked. The hospital is under the control of the State Inspectorate for Hospitals which has very stringent design requirements for earthquakes. They however did not apply to this ancillary structure. After the earthquake the damaged building was posted as "limited entry only."

4.4 UNDAMAGED BUILDINGS

4.4.1 General

One of the most pertinent lessons from the earthquake was the good performances of the engineered buildings in the vicinity of the epicentre. It is suspected that few of the downtown buildings in these areas were designed for lateral loading from earthquakes. However most would have been designed, but they are classified as semi-engineered for the present purposes and considered in the section on non-engineered buildings.

The general lack of evidence of relative deformation in the structure echoes the experience with the Coalinga Earthquake, and is likely to be associated with the short duration of the earthquake at source.

4.4.2 Two Storey Office Building : Lake Merritt District

This building is located in the downtown district of Oakland, and is constructed with masonry shear walls and a rigid steel frame. Figure 4.3 shows that the reinforced concrete block shear walls are built on two sides of the structure only, and hence the structural response behaviour would have been similar to that of the Pacific Bell building described in Section 4.2.4 above. The building is instrumented by the California Division of Mines and Geology, and the location of the 10 sensors is shown on Figure 4.3. The time-histories of the acceleration records from these 10 locations are shown in Figure 4.4, with records 6 and 10 being the ground motions at the NE corner in the reference E-W (90°) and N-S (00°) directions, with peak accelerations of 0.25 and 0.21g respectively. Note that the reference North direction is orientated at 20° to true North. Further data including response spectra of these ground motions is given in Tables 2.1 and 2.4, together with Figure 2.7.

The 6 sensors which recorded the horizontal structural responses at 2nd Floor (No's 4, 5, 9) and Roof (No's 2, 3, 8) levels give clear evidence of the anticipated large torsional motions.

Comparing the peak E-W accelerations on the stiffer side of the structure (Sensors 4 and 2 having peak responses of 0.34g and 0.37g respectively) with those on the opposite site (sensors 5 and 3 having peak accelerations of 0.55g and 0.66g) shows that torsion increased the response of the more flexible southern side of the building by about 60-80% compared with the northern side, which is stiffened by a reinforced concrete shear wall (Figure 4.3).

The peak E-W roof response of 0.66g is 2.6 times the corresponding peak ground acceleration, whereas the motions in the N-S direction taken from sensors 8-10 show that the eastern shear wall behaved nearly as a rigid system, with the peak acceleration at roof level (0.26g) being only 24% more than at ground level.

This building was designed for earthquake loading according to the UBC lateral force provisions existing at the time of construction (1964), which placed no restriction on the size of torsional asymmetry in the structure. Given the magnitude of the recent earthquake, the fact that this building suffered only superficial non-structural damage indicates that the code torsional provisions (which locate the design lateral force at the centre of mass of the building, namely the geometric centre assuming a uniform distribution of floor loading) are adequate for low-rise structures, even under relatively severe earthquake events. Under present UBC design provision, however, a building with such a pronounced asymmetry would have to be designed to more stringent conditions

based on a full three dimensional dynamic analysis of lateral and torsional response characteristics using suitable strong ground motions. It is likely that the good performance of the Oakland office building is due partly to the stiffening effects of internal cross-walls (not shown in Figure 4.3), which provide redundancy to the structure. Their effect in taller buildings may be less significant and hence torsional asymmetry should be catered for in this case by properly implemented dynamic analysis, as described above.

4.4.3 575 Market Street, San Francisco

In 575 Market Street, San Francisco there was evidence of plasticity in the strong motion records for the superstructure. No information is available to establish whether this is due to the behaviour of the cladding, minor hysteretic effects due to cracking or true plasticity. If due to cracking the extent would have been substantial. However, no damage has been reported and the circumstantial evidence of there being some may be considered insufficient to justify the expense of an investigation.

4.4.4 Telephone Exchange, Watsonville

This structure, which comprised composite columns, some reinforced concrete shear walls and reinforced concrete beams, suffered no apparent damage and the opinion is there was none, despite the foundation pads being at different levels (Figure 4.1). The tying of the foundation pads suggested the structure had been designed for earthquake loading. The maximum responses of 13 accelerograms indicated accelerations of 0.26 to 0.81g in the N-S direction, 0.39 to 1.24g in the E-W direction and 0.52 to 0.66g vertically. The good performance here of the composite columns contrasts with that of those in the Pacific Bell Building in Oakland, confirming the implications in Section 4.2.4 that the latter had been designed inadequately for the effects of lateral loading.

4.4.5 The Dream Inn Hotel, Santa Cruz

This reinforced masonry building (Plate 4.44), built into a sloping site, with ten stories facing the sea and six the other way, is reported to have suffered no damage. It is on the coast and is reported to be founded in sandstone. It is therefore on rather firmer ground than many sites in Santa Cruz, a benefit which clearly outweighed the adverse effects of the steeply sloping site. The excellent performance of this building during the earthquake has attracted the attention of American engineers. It was designed in the early 1960s and seismic forces were taken into account.

4.4.6 The Santa Cruz Sentinel Building

This is a large two storey square building, appearing from the outside to have full height imperforated infilled frames on two adjacent faces, and of similar construction but with a continuous clerestory window on the other faces (Plate 4.45). There was no evidence of lateral restraint to the panels. There was an atrium at first floor level which was far larger than that permitted in many design codes (Plate 4.46). The structure was designed as a moment resisting frame and constructed in the late 1960's. No damage was reported, despite a pronounced crack in the ground in the passage down the side of the building (Plate 4.47).

4.4.7 Five Storey Condominium, Los Gatos

The building was of cross wall construction, but was irregular in plan with an external lift/stair well. It had suffered no apparent damage despite being sited at the top of an underpass (Plate 4.48), estimated to be 15m deep.

4.4.8 Six Storey Condominium, Santa Cruz

This structure was of precast concrete panel construction with the end walls extending over the full width of the building (Plate 4.49). It had external shafts, but did not rely upon these for the lateral resistance. It was green-tagged (Plate 4.50), indicating there was no damage requiring repair.

4.4.9 Monterey Savings Building, Santa Cruz

One masonry building in Santa Cruz stood out as being very modern and of solid masonry construction (Plate 4.51). Being so new it is assumed it had been designed to carry seismic loads and it had suffered no obvious damage. It nevertheless could be classed with the solid masonry structures without openings, which performed best amongst the masonry structures (See Section 3 of this Report covering damage to on Non-engineered Buildings).

4.4.10 Car Parks in Santa Cruz

Two multi-storey car parks in Santa Cruz were examined and no cracks were found in either. One was an extensive two level car park with external shear walls all round. The other was a three storey car park with the lateral loads carried by very substantial insitu concrete raking members, which had successfully carried the lateral forces (Plates 4.52 and 4.53).

4.5 DAMAGE TO RETROFITTED BUILDINGS

4.5.1 Introduction

The University of Berkeley were studying four retrofitted buildings in San Francisco, in all of which the retrofitting had failed. Two were of masonry and two of reinforced concrete. Attempts were being made to determine the proportion of retrofitted buildings in which the retrofitting had failed, relative to comparable structures which had not been retrofitted.

4.5.2 The Cooper House, Santa Cruz

The other notable older building in downtown Santa Cruz was the Cooper House, which had been constructed at the turn of the century. It was an attractive masonry building, built in the red brickwork favoured in the churches. It has formally been the old court building and had survived the 1906 San Francisco earthquake, when it had been relatively new, but could have suffered some damage. It suffered more damage from a smaller local earthquake during the mid 1920's. Consideration had been given to demolition then, but instead the damaged areas of masonry were cut out and reinstated. Not long before the 1989 earthquake it had been extensively retrofitted by constructing circumferential ring beams and floor to wall connections in accordance with the Los Angeles Bylaws. Although this work was not quite complete the building had been completely refurbished. The exterior had suffered a significant amount of damage, particularly around the arches. Inside the damage was more severe and the building was demolished within three weeks of the earthquake with the agreement of the Historical Association of California.

4.5.3 Former Municipal Offices and Mortuary, Santa Cruz

Nearby the Cooper House site were three masonry structures used until the earthquake as offices by lawyers and other professionals. As least one had a basement (Plate 4.54). These had been retrofitted by installing rectangular posts

against long walls considered vulnerable to out-of-plane failure. The posts were attached to the brick walls by expanding anchors. Impact damage between the buildings had been eliminated by installing posts either side of the abutting walls and tying them together with through bolts.

During the earthquake the buildings suffered appreciable cracking and many of the retrofitted anchors pulled out of the walls (Plate 4.55). The retrofitting was considered to have been successful to the extent of preventing collapse while the occupants escaped. None had been allowed back by the time of the earthquake.

4.5.4 The Casa Del Rey Retirement Home, Santa Cruz

The Casa Del Rey Retirement Home was a very large structure to the South of West Cliff (Plates 4.56 and 4.57). The exterior facade had recently been reconstructed. Much of the interior is reported to have collapsed, yet the only damage visible on the exterior was permanent set in the high part of the parapet over the main entrance. Poor foundation conditions and impact between the old and new structure are possible causes of the failure.

4.6 MISCELLANEOUS STRUCTURES

4.6.1 Towers

Due to the hilly ground and plenty of reservoirs there were very few water towers in the vicinity of the epicentre. One was identified to the West of Highway 101 which appeared sound but it was not inspected.

A well-braced fire brigade training tower in Watsonville was undamaged.

4.6.2 Greenhouses, North of Watsonville

Greenhouses, common in the area of Watsonville (known as the "Strawberry Capital of the World" but also a major producer of tomatoes) sometimes suffered badly. The largest individual claim for damage outside the downtown areas was to Japanese greenhouses near Corralitos where damage in the order of \$800,000 was reported.

4.6.3 Roller Coaster, Santa Cruz

The roller coaster in Santa Cruz, which is a wooden structure with many legs founded on small concrete pads on sand at ground level, had suffered no noticeable structural distress, though it was out of action. The floor slab in an underground service area had suffered major settlement around internal columns. The beach sand appeared too coarse to have been prone to liquefaction and it is considered the settlement was due to liquefaction of finer material at a lower level. The owners considered neither structure to be safe.

4.6.4 Piers, Santa Cruz

The old pier near the Dream Inn Hotel in Santa Cruz, which had been well maintained, is reported to have suffered no significant damage. A smaller pier in the harbour, which had not been maintained, is reported to have suffered appreciable damage. The earthquake was reported to have caused an appreciable sea wave, though it was not considered high enough to have caused the damage.

4.6.5 Culvert in Santa Cruz

A deep reinforced concrete channel (See Section 3), empty at the time of the

earthquake, and a culvert under Ocean Street, Santa Cruz, had suffered no obvious damage.

4.6.6 Mobile Homes

Mobile homes, effectively demobilised caravans, were reported to have been shaken off their foundations in Watsonville.

4.6.7 Petrol Station, Ocean Street, Santa Cruz

The masonry cladding around the four RHS tubes supporting a car pump canopy had collapsed, damaging cars. The rest of the canopy was undamaged but the exposed columns were corroded at the bottom. The masonry was not to be replaced.

4.7 CONCLUSIONS

- (i) The earthquake of 17th October 1989 provided the first significant test for most of the engineered building stock of the San Francisco Bay and Santa Cruz areas, and was a retest only for the small number in the vicinity of Morgan Hill and Gilroy (see Section 3.4.1).

The short duration at source and the small number of peak accelerations close to the maximum would tend to suggest the Loma Prieta earthquake was not a very damaging earthquake for its magnitude. Of the stock of medium and high rise buildings in San Francisco and Oakland, only some on the softer soils experienced peak ground accelerations approaching two-thirds of the design ground acceleration for the area of 0.4g. The earthquake cannot therefore be regarded as providing justification for the design methods for engineered buildings so far from the epicentre. It might however have provided some comfort in the response of retrofitted buildings, which are designed to a lower standard, and to the older semi-engineered building stock.

The less severe earthquakes (or less severe fires in fire engineering) however often provide better understanding of building response than severe earthquakes, and valuable information has been provided by the performance of buildings in the San Francisco/Oakland area.

The performance of structures nearer the epicentre has a more direct bearing on design methods as the design ground acceleration was exceeded, but even this is tempered by consideration of the short duration. The buildings in this area are predominantly low rise structures and for engineered buildings in this class the earthquake provides justification of the design methods in regards to moderate earthquakes. Studies for the verifications of Eurocode 8, which has been calibrated against the UBC/SEAOC codes, Reference 1, suggest that the US codes are potentially unsafe for low period structures. On the other hand some of the design penalties placed on irregularity in the Eurocode would appear heavily over-conservative.

- (ii) The engineered buildings on good ground generally suffered least; damage was limited to the external cracking of the facade and spalling at the corners. These were isolated cases of spalling of the cover in reinforced concrete construction.
- (iii) On poor ground engineered buildings fared less well, but damage was concentrated at structural discontinuities or severe asymmetries. Some buildings, however, seem to have experienced no detectable damage despite obvious weaknesses in design.

- (iv) Severely asymmetrical buildings were at risk of having been shaken significantly in torsion and suffering extensive damage.
- (v) The peak spectral amplification of 2.6 noted in a torsionally irregular building suggests that the value of 2.0 in EC8 for regular structures on soft soil sites is acceptably conservative.
- (vi) Damage observed at the car park in San Francisco (where cracking at the heads of columns had gone undetected), and inferred in 575 Market, suggests that less well detailed structures might have suffered a degree of damage rendering them less able to resist future earthquakes.
- (vii) The behaviour of the infill framed structures that were observed indicated that they performed well.
- (viii) The piling of structures does not necessarily avoid the erratic high amplifications resulting from bad ground conditions.
- (ix) Generally semi-engineered framed buildings which were not retrofitted performed distinctly better than non-engineered load bearing masonry buildings, even those which had been retrofitted. Good performance may be expected where:
 - (a) The stiffness and strength of vertical elements are well distributed, or, in cases where the stiffness and strengths are less well distributed, the structure is stiffened by thick masonry walls abutting the structure, particularly in the lowest storey or around the perimeter.
 - (b) The structures are not waisted or have weak internal cross-sections, resulting from the omission of part of a floor (at an atrium or stairwell), unless the void is stiffened by a horizontal diaphragm, as in the Santa Cruz Sentinel Building. With this qualification low buildings might even perform well with plan irregularities appreciably exceeding those recommended in seismic codes, due to a comfortable reserve in lateral strength.
 - (c) The gaps between buildings are greater than 25mm, but there were no situations among those examined indicating what separation might be acceptable. Some gap is considered necessary to provide a fire break, which prevents buildings being tied together.
 - (d) Masonry cladding is confined to horizontal bands, and for two-leaved cladding the proportion of cross-bonded bricks should be increased.

REFERENCE

1. "Calibration/Design Studies for the National Evaluation of Eurocode 8 : Vol. 1: Design Studies" Scott Wilson Kirkpatrick for the Building Research Establishment/Department of the Environment, Feb 1992.

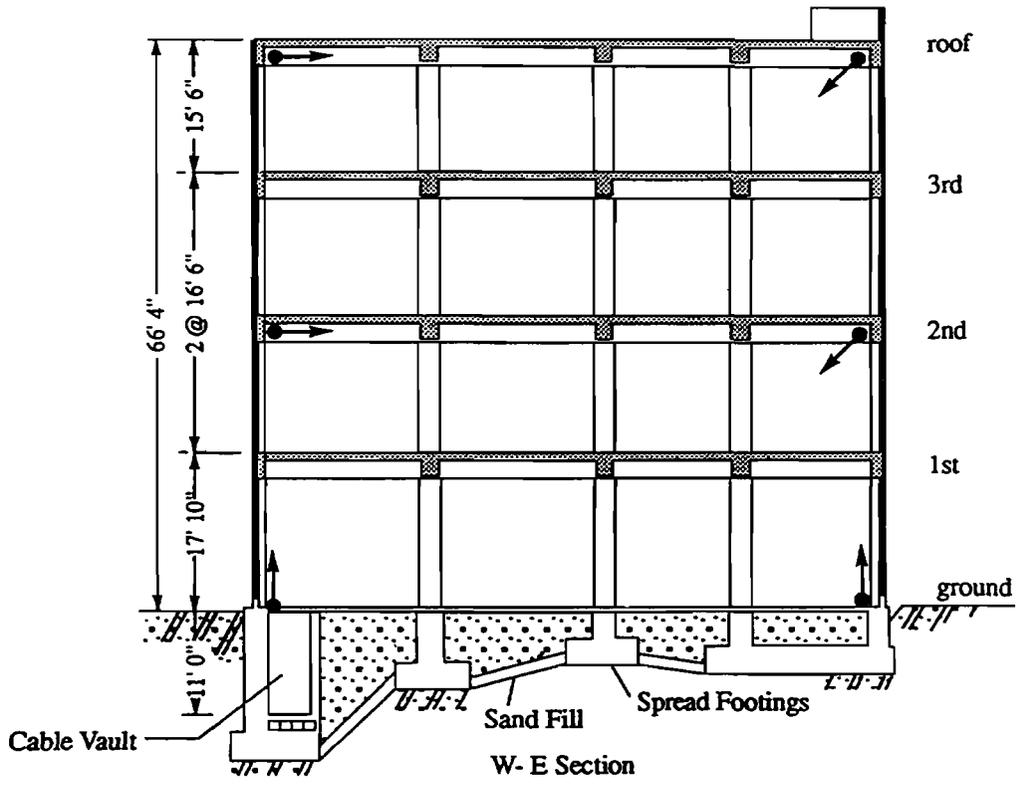


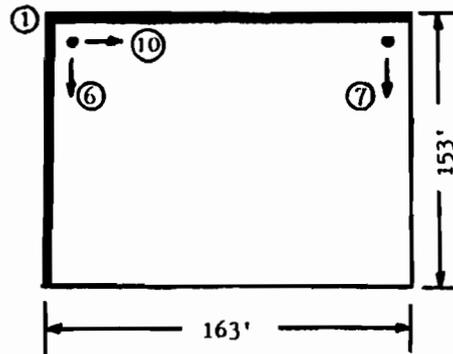
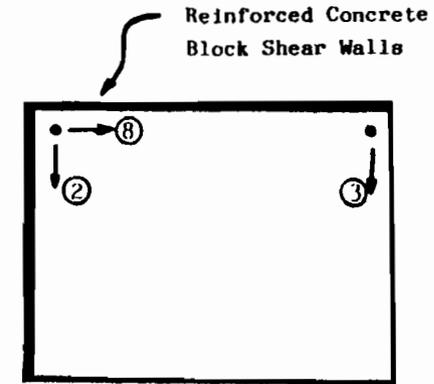
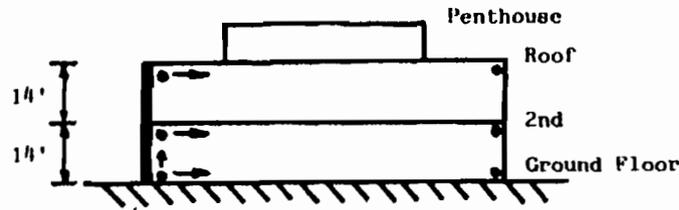
Figure 4.1 : Watsonville - Telephone building

Oakland - 2-storey Masonry/Steel Office Building (CSMIP Station No. 58224)

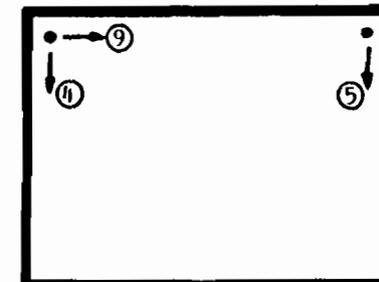
Reinforced masonry shear walls
and rigid steel frame

Design Date : 1964
Construction Date : 1966

● Sensor locations
and directions



← N_{ref}



Structure N_{ref} is 20° from True N.

Figure 4.3 : Diagrams of 2-storey office building in Oakland showing locations of sensors.

**Oakland - 2-storey Masonry/Steel Office Building
(CSMIP Station No. 58224)**

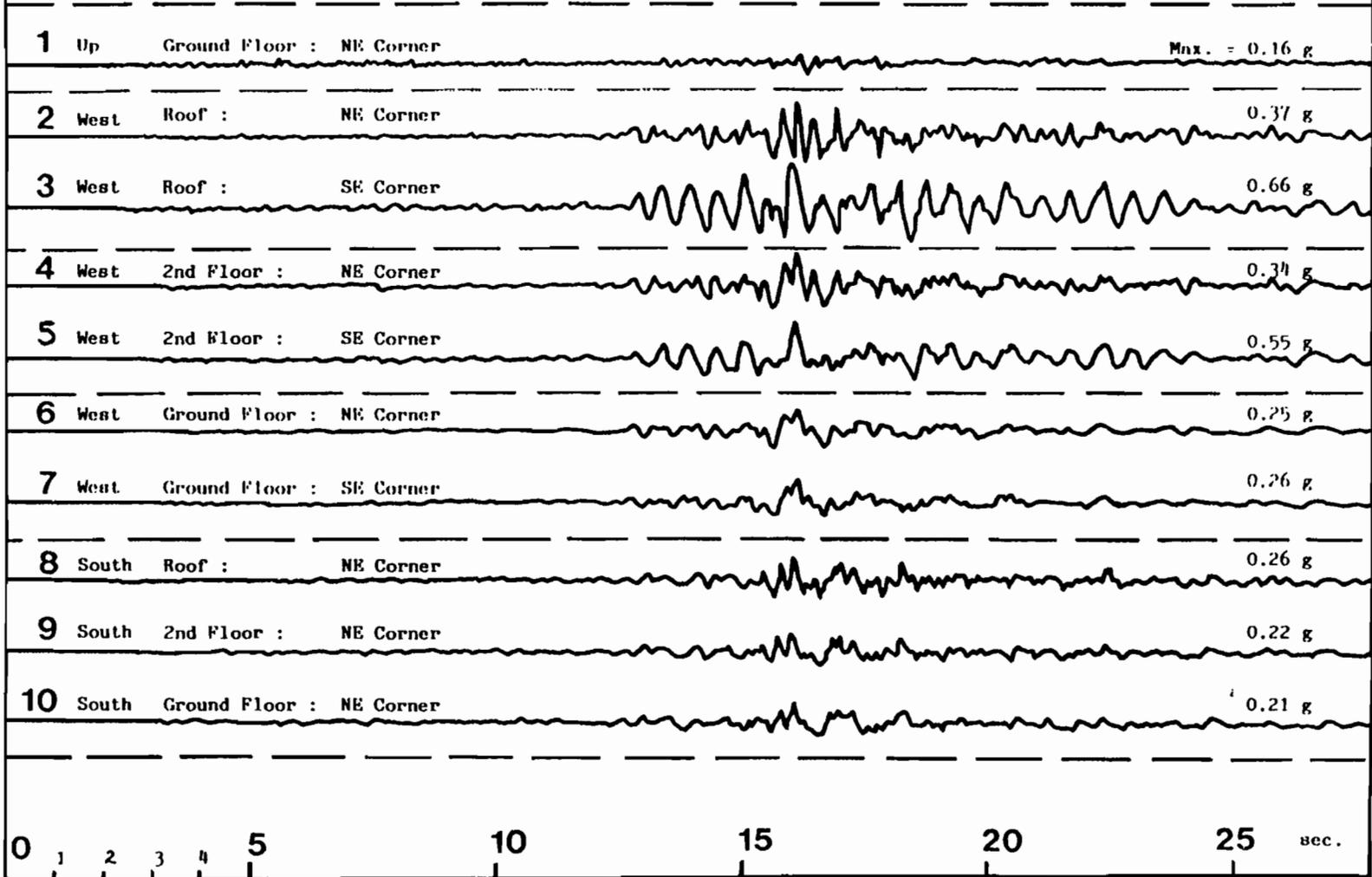


Figure 4.4 : Accelerometer recordings from 2-storey office building in Oakland.

PLATE 4-1

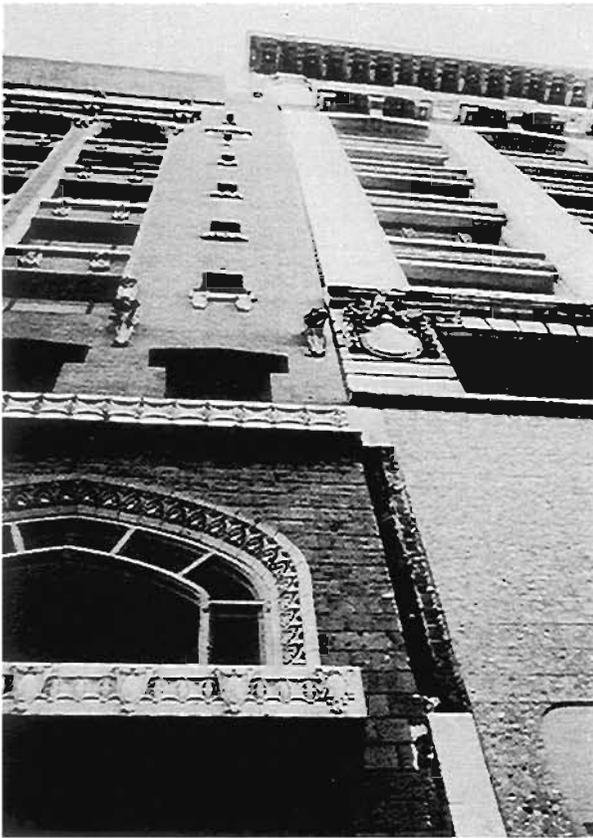


PLATE 4-2

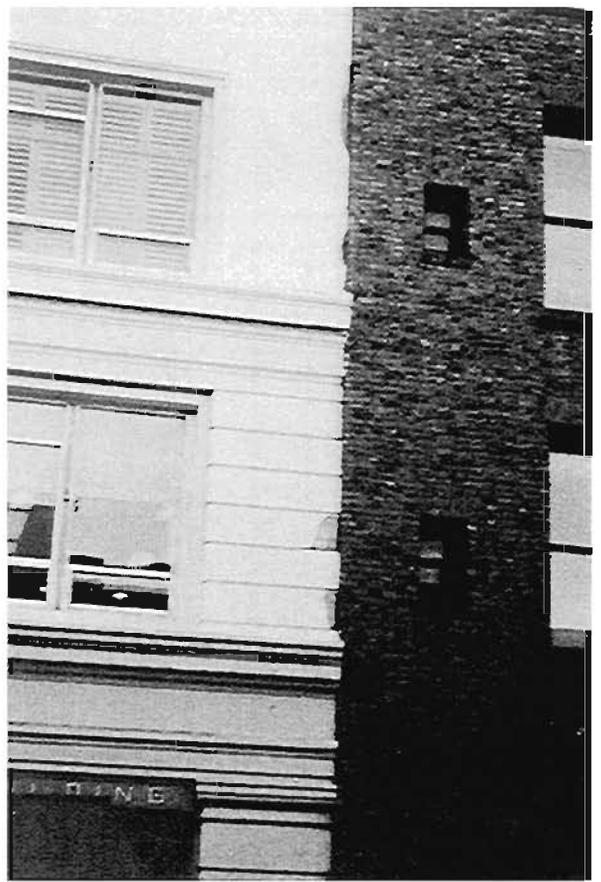


PLATE 4-3

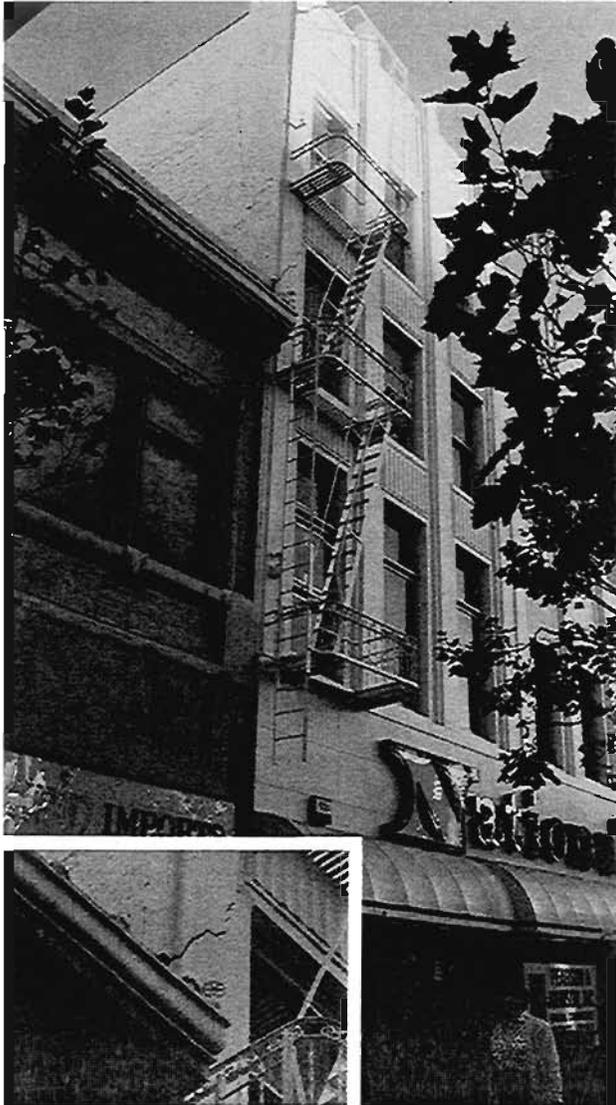


PLATE 4-4

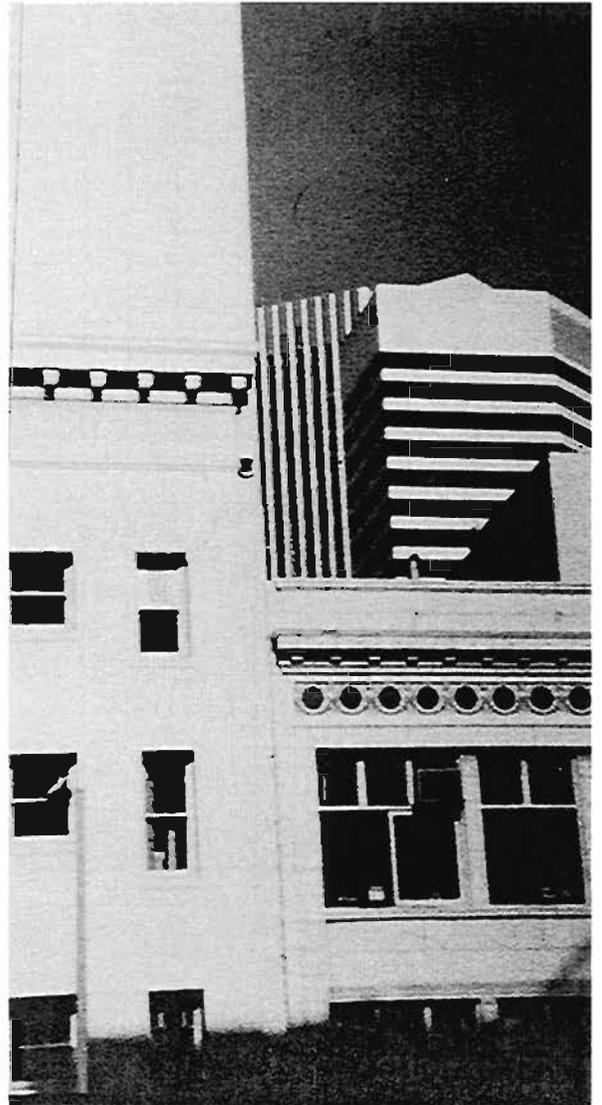


PLATE 4-5



PLATE 4-6



PLATE 4-8



PLATE 4-7

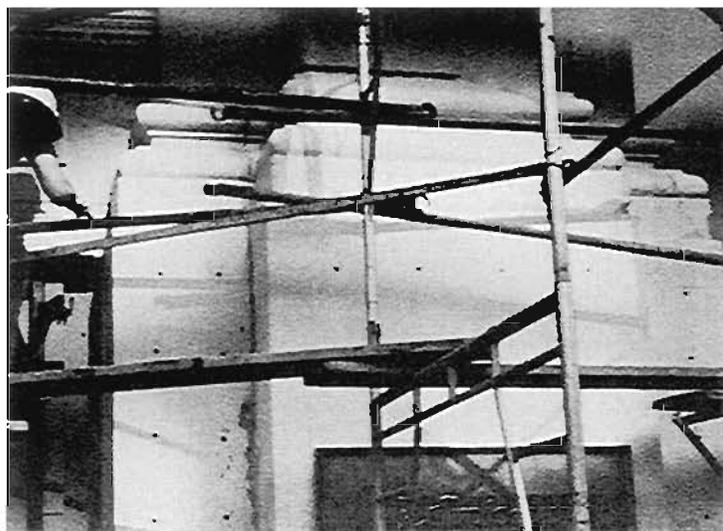


PLATE 4-9

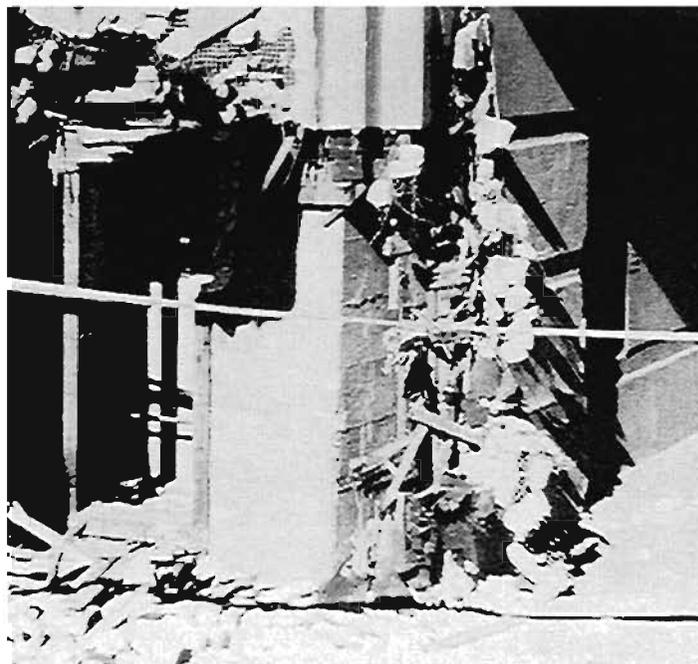


PLATE 4-10

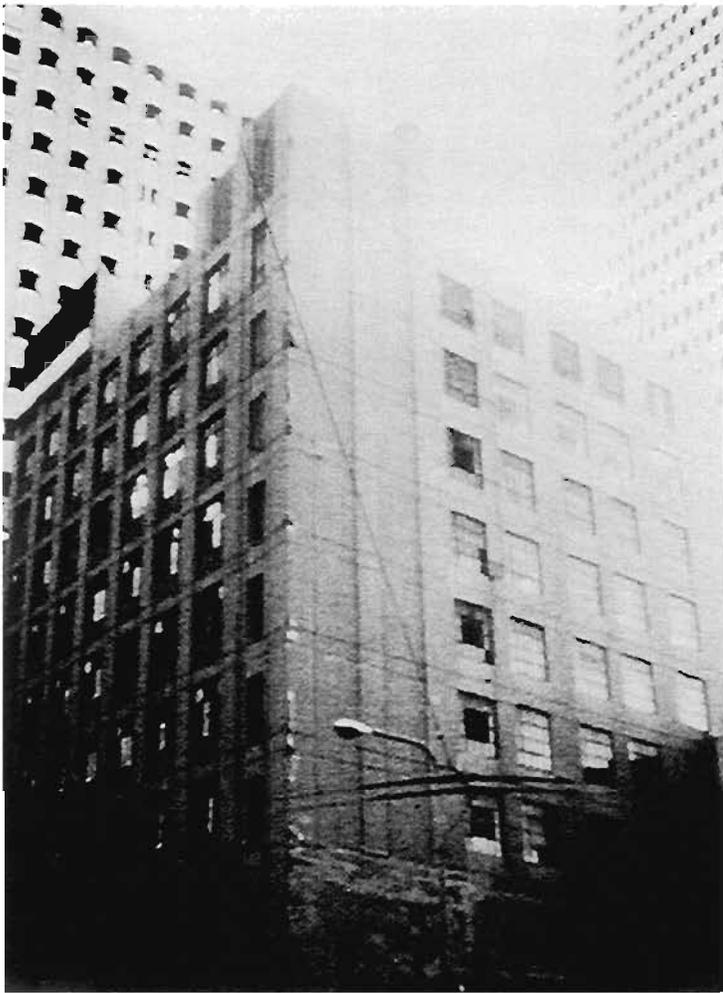


PLATE 4-11

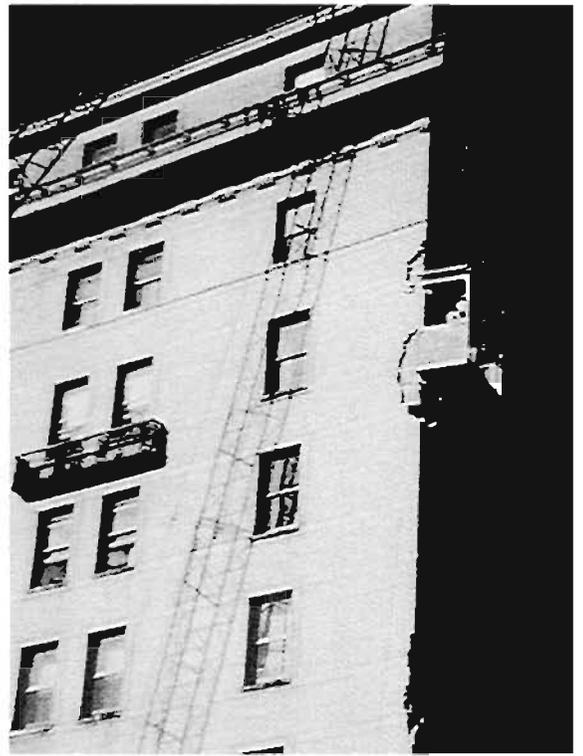


PLATE 4-12

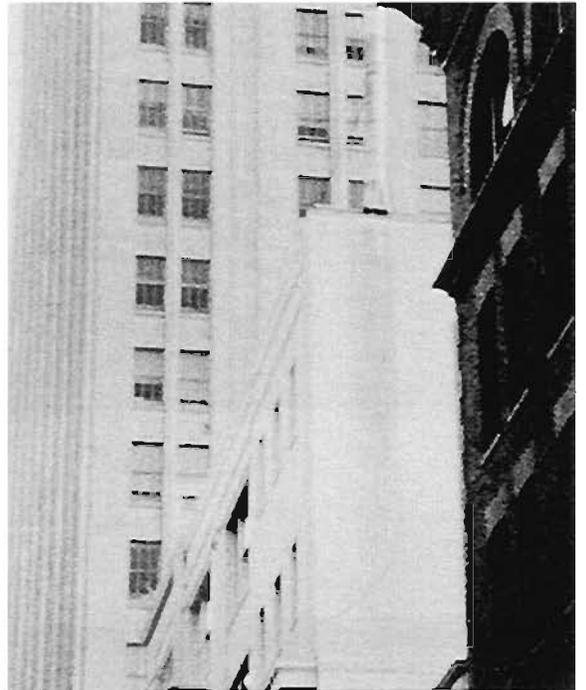


PLATE 4-13

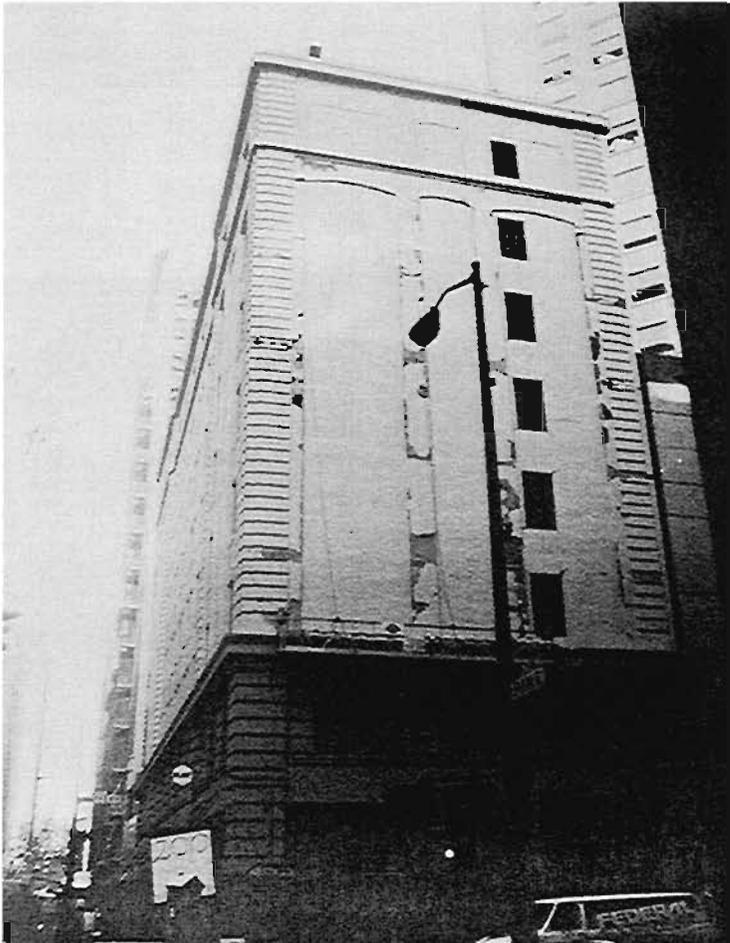


PLATE 4-14

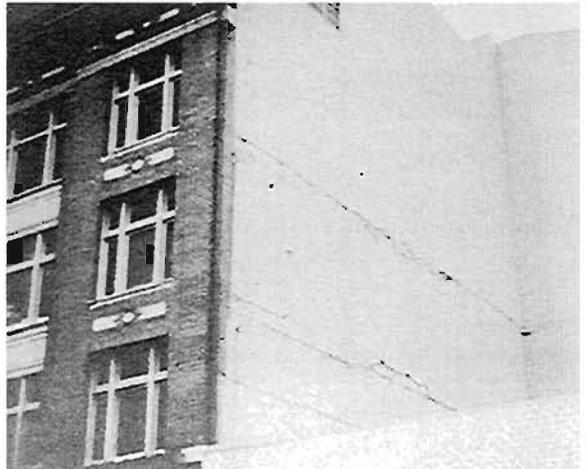


PLATE 4-15

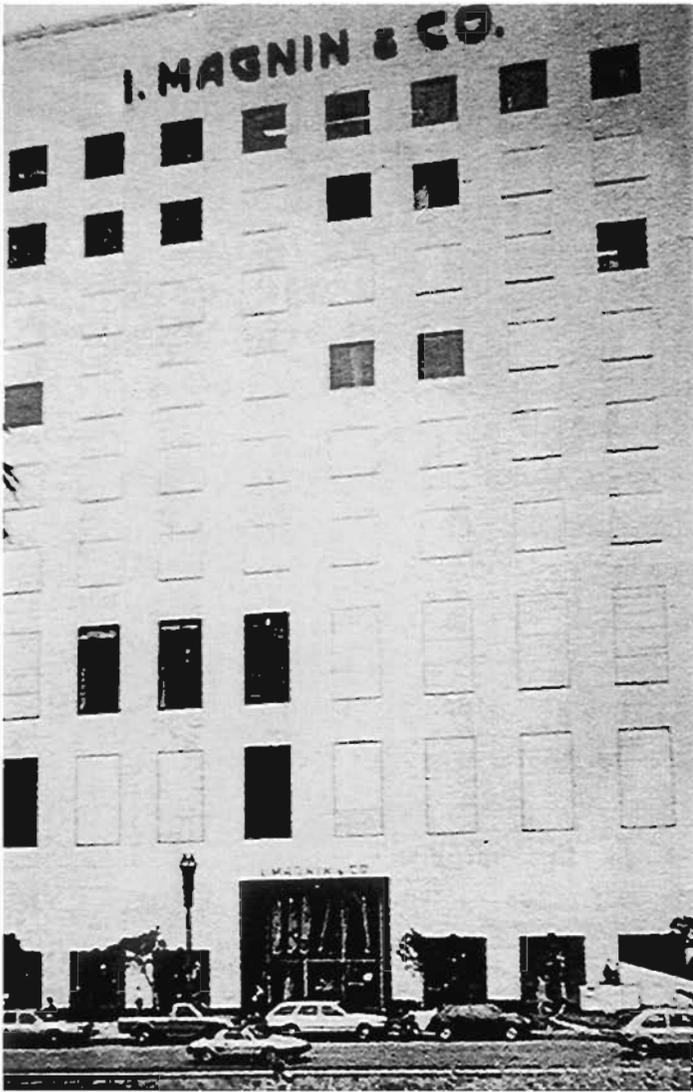


PLATE 4-16

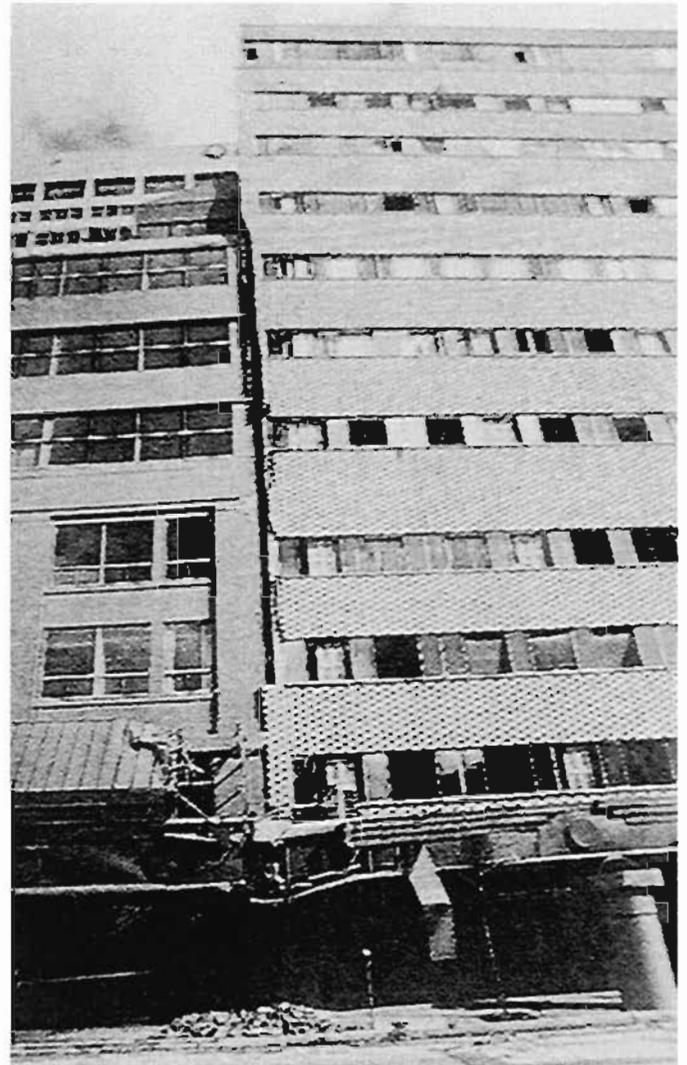


PLATE 4-17

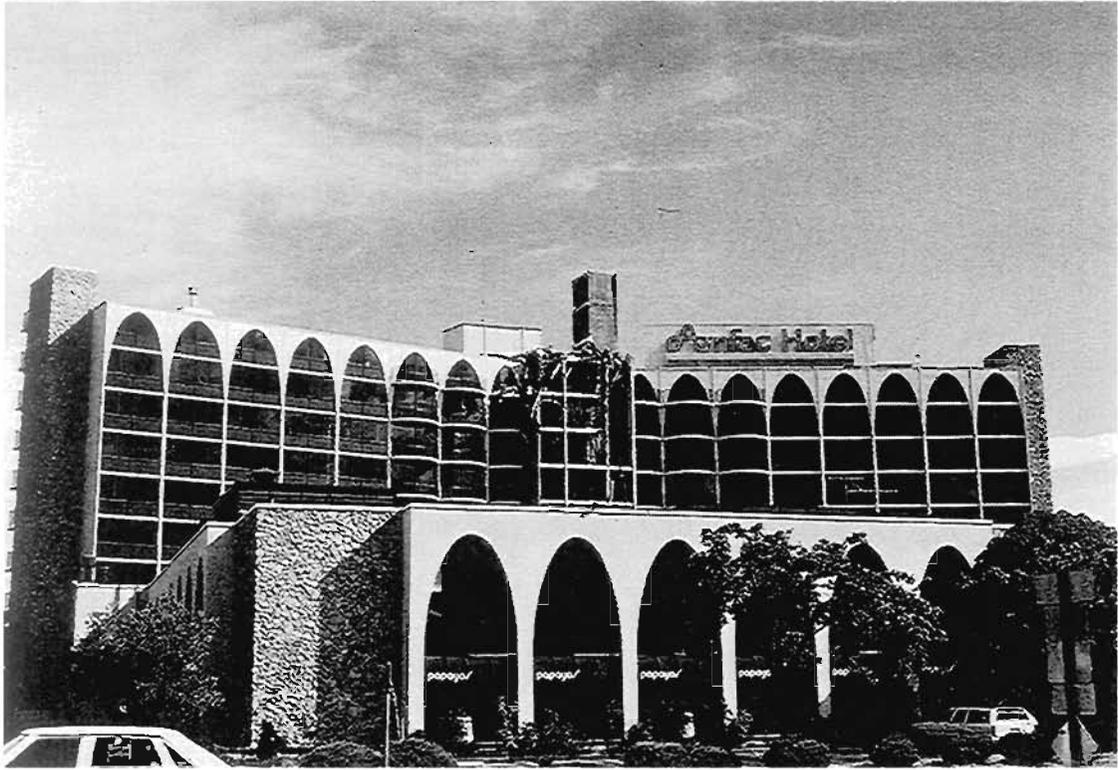


PLATE 4-18



PLATE 4-19

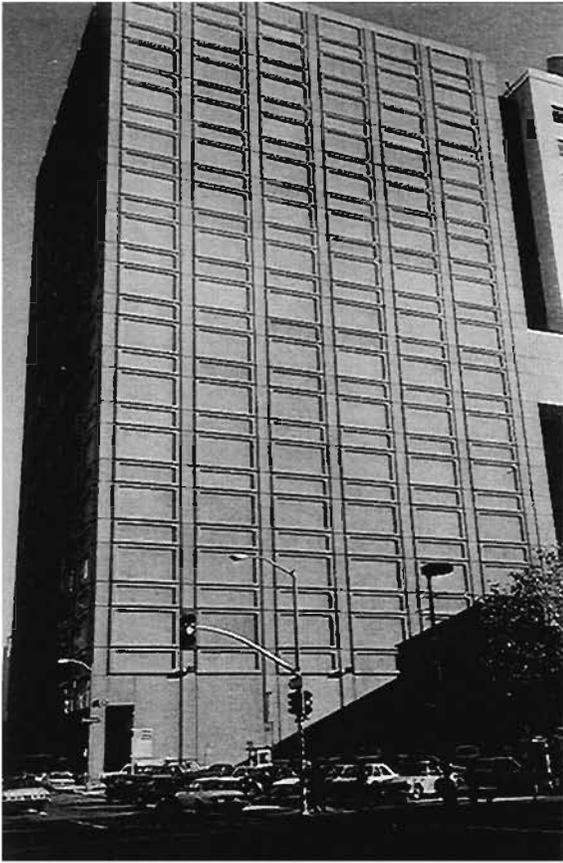


PLATE 4-20



PLATE 4-21



PLATE 4-22

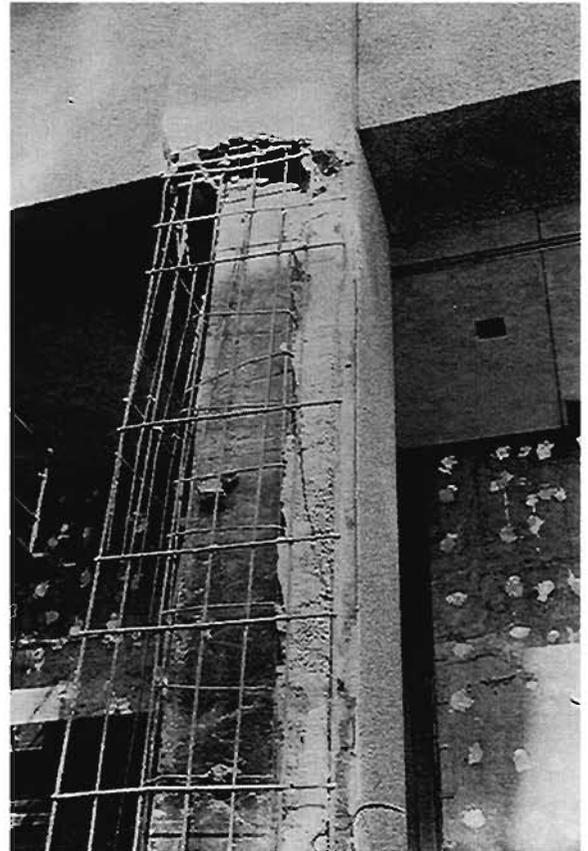


PLATE 4-23



PLATE 4-24

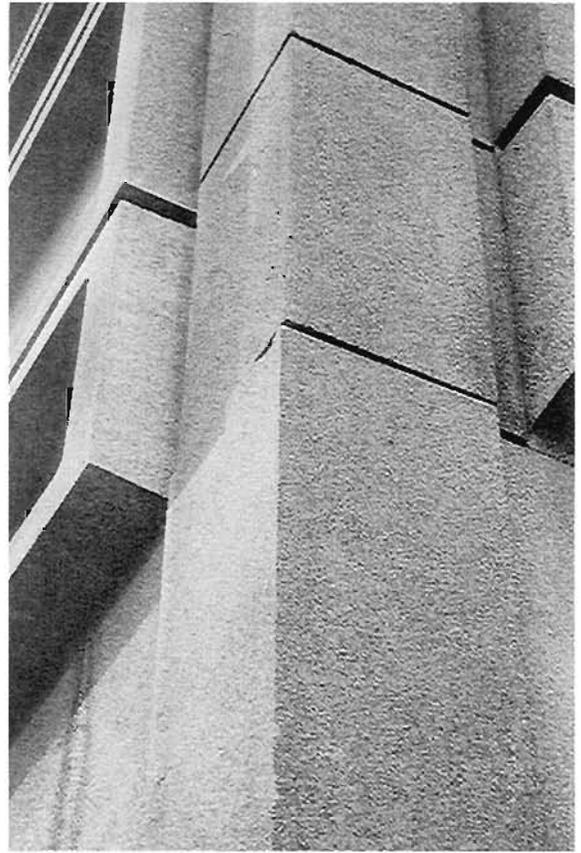


PLATE 4-25



PLATE 4-26



PLATE 4-27



PLATE 4-28

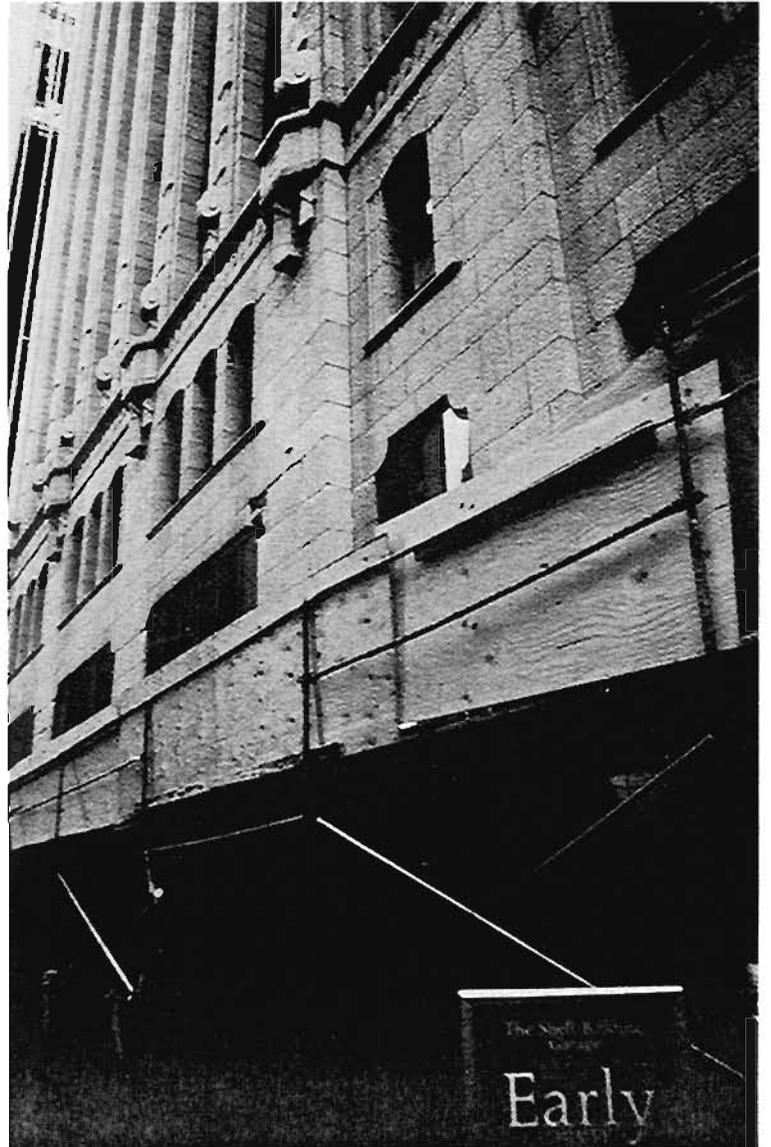


PLATE 4-29



PLATE 4-32

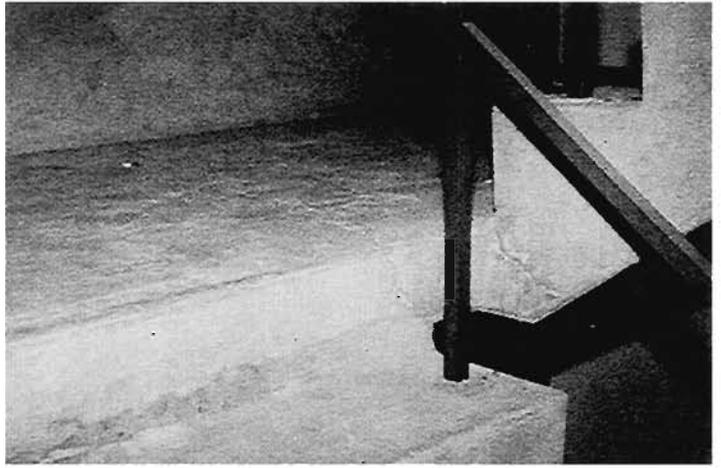


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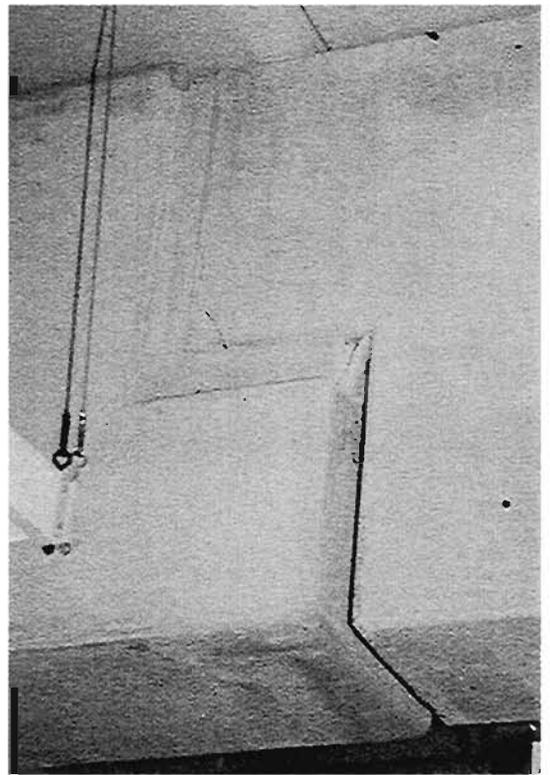


PLATE 4-30



PLATE 4-31

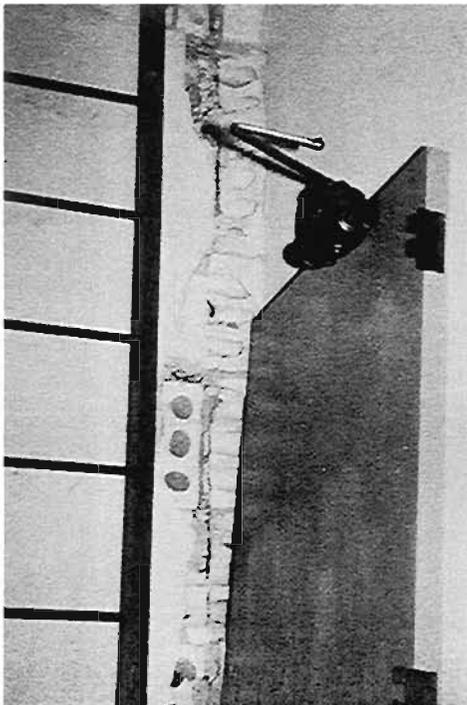


PLATE 4-34



PLATE 4-35

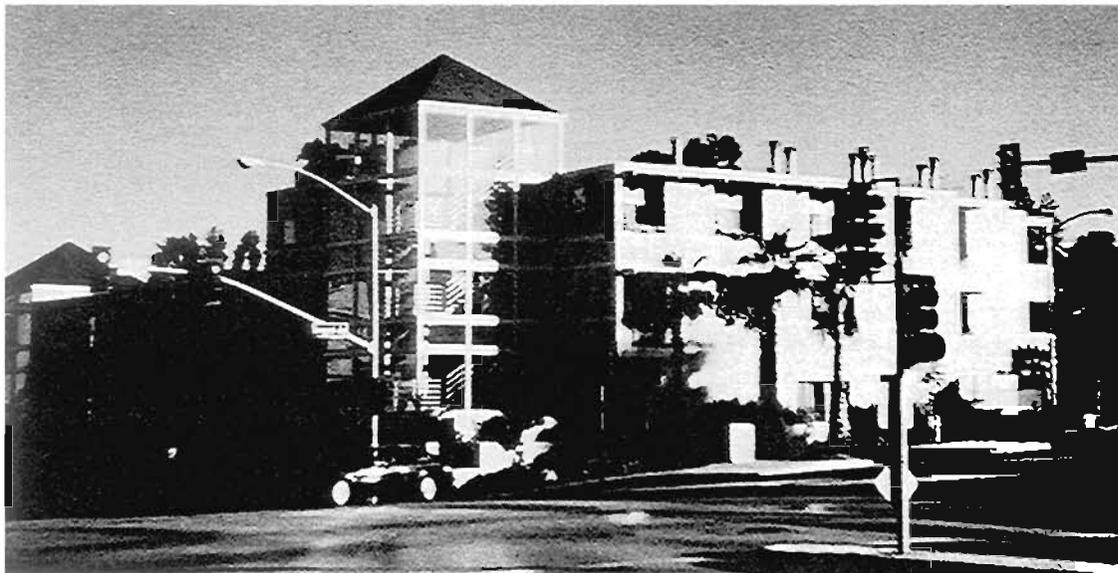


PLATE 4-36

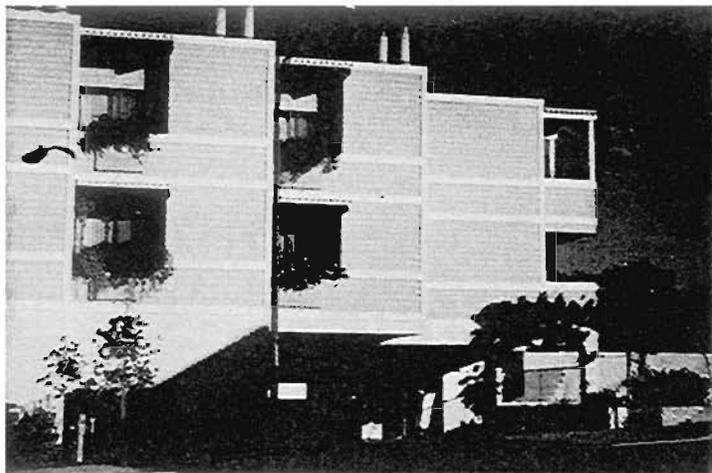


PLATE 4-37

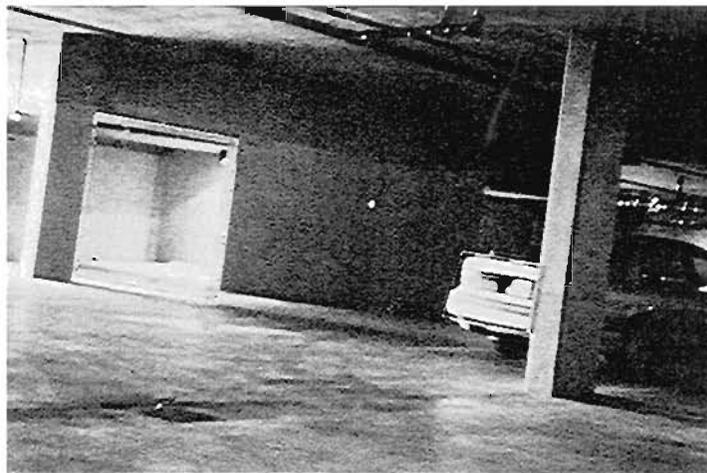
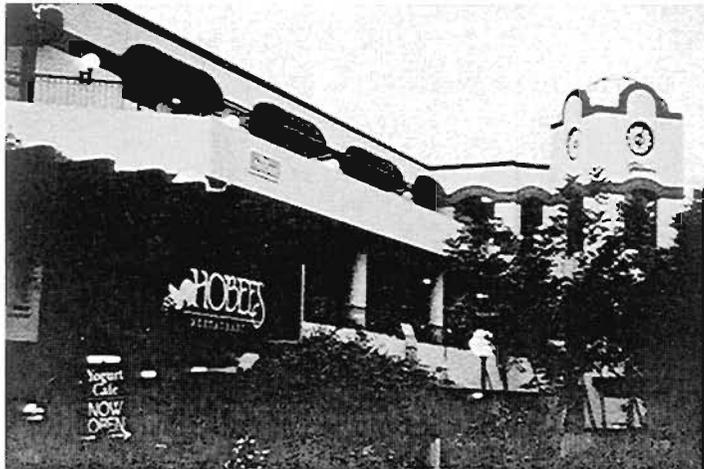


PLATE 4-38



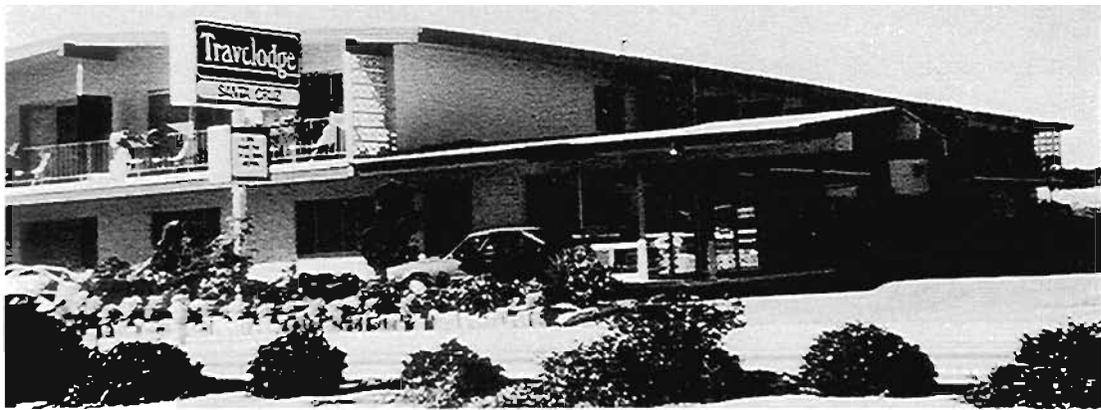


PLATE 4-39

PLATE 4-41

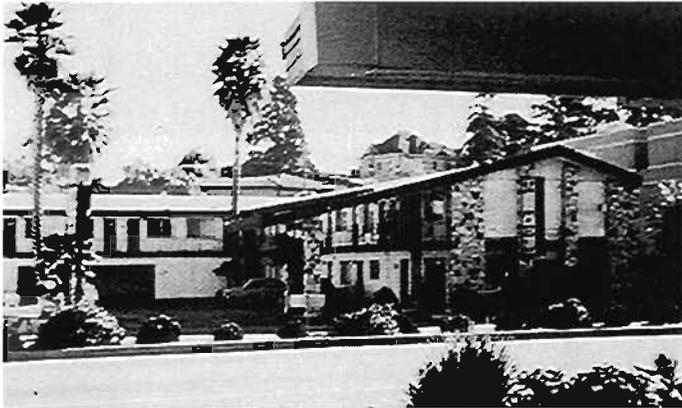


PLATE 4-40



PLATE 4-42



PLATE 4-43

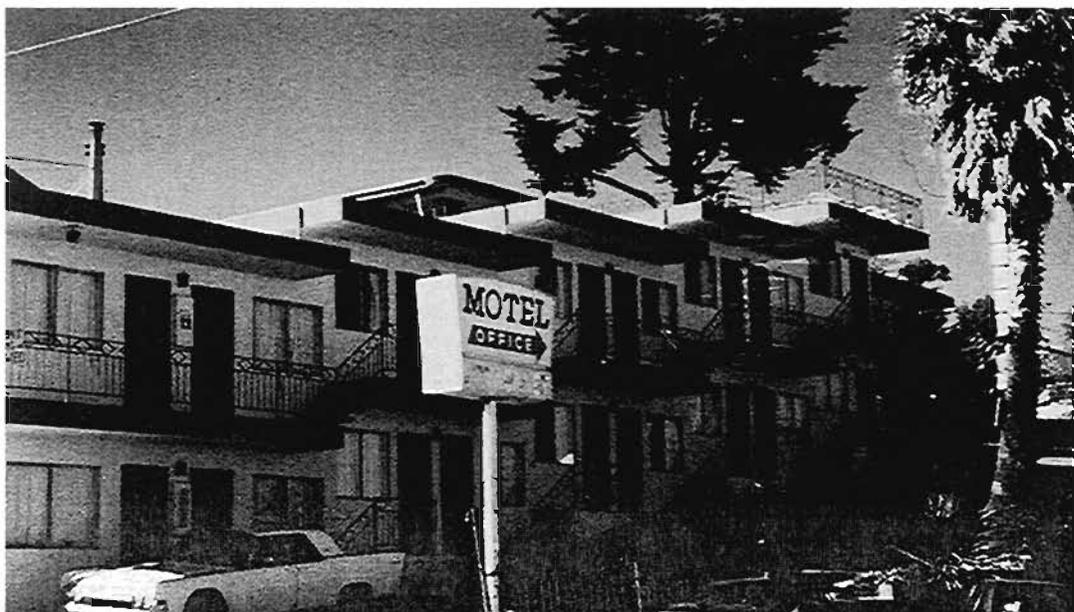


PLATE 4-44

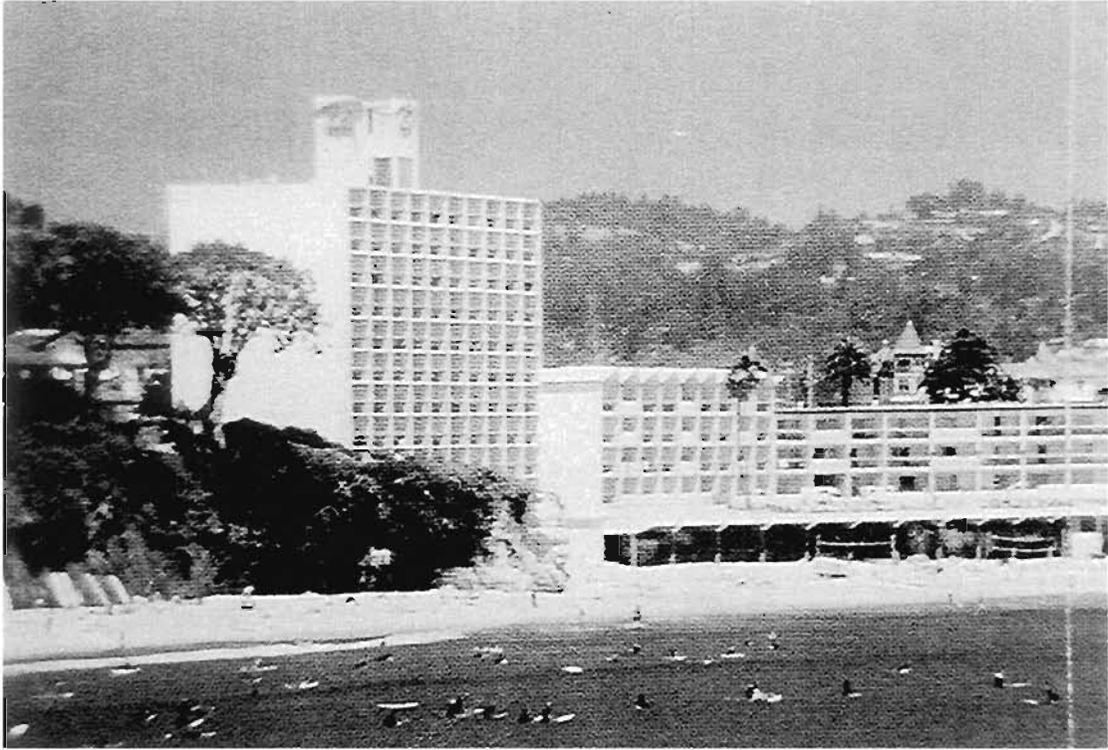


PLATE 4-45



PLATE 4-47

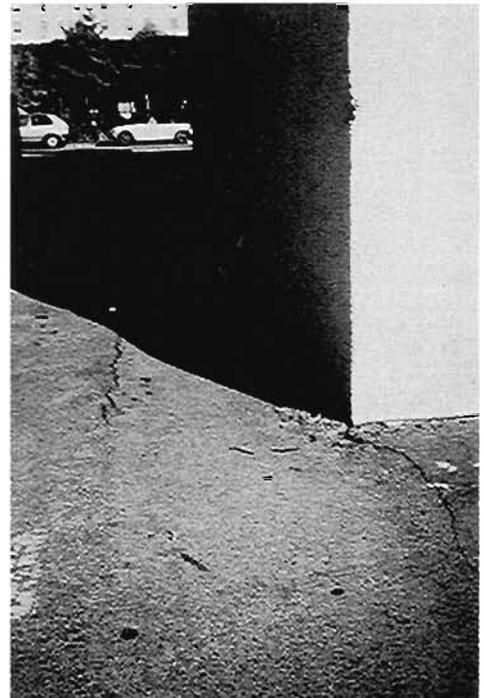


PLATE 4-46



PLATE 4-48



PLATE 4-49



PLATE 4-50

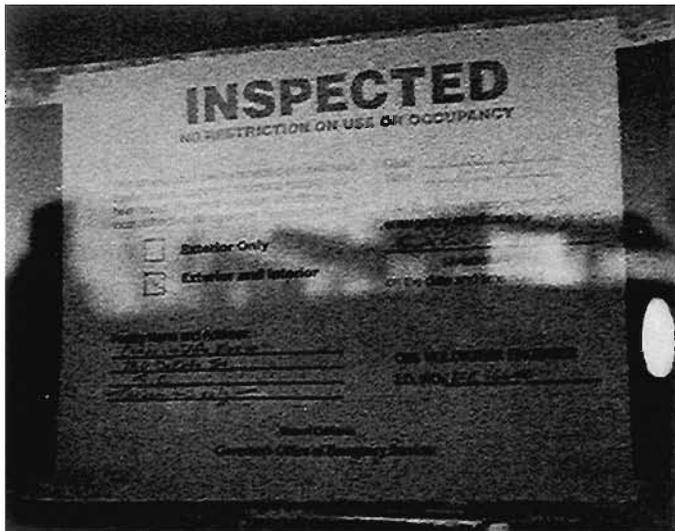


PLATE 4-51



PLATE 4-52



PLATE 4-53



PLATE 4-54

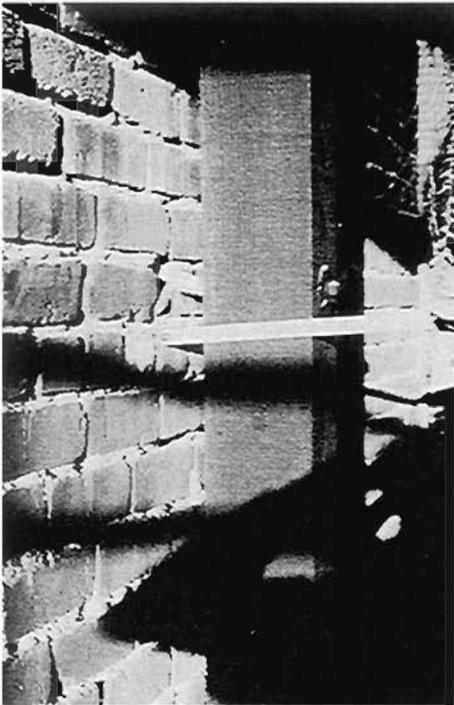


PLATE 4-55



PLATE 4-56



PLATE 4-57

PERFORMANCE OF BRIDGE STRUCTURES

J M Barr,
Rendel Palmer and Tritton

S C Birkbeck,
Ove Arup and Partners

5.1 INTRODUCTION

The bridges which have received most publicity are those in San Francisco and Oakland, where failure of the Bay Bridge and the double-decked Cypress Viaduct took the greatest toll in terms of both loss of life and economic loss. On the San Francisco side of the Bay another double-decked freeway structure, the Embarcadero Viaduct, was quite badly damaged but survived. This paper reports the observations of the two EEFIT bridge engineers on the following structures:

- Cypress Street Viaduct, Oakland
- Embarcadero Viaduct, San Francisco
- Oakland Bay Bridge, San Francisco

- Santa Cruz Bridges:
 - Laurel Street Bridge
 - Riverside Avenue Bridge
 - Cut Bias Bridge
 - Murray Street Bridge

- Struve Slough Bridge, Watsonville
- Golden Gate Bridge, San Francisco

The patient and friendly assistance lent to the EEFIT team by Dave Paulson, Lisa Murphy, and other staff of CalTrans (California Department of Transportation); by Brian Evers of Santa Cruz City Hall, and by Daniel E Mohn, District Engineer for the Golden Gate Bridge is gratefully acknowledged.

5.2 CYPRESS STREET VIADUCT, OAKLAND, SAN FRANCISCO

5.2.1 Background

This double-decker highway structure, completed in 1957, was the first structure of its type in the state of California. It carried Interstate Highway I-880 through the city of Oakland on a north-south axis, feeding through traffic to and from the Oakland Bay Bridge (see Figure 5.1). By carrying two elevated carriageways the viaduct allowed local traffic freedom of movement from downtown Oakland to its port, military and industrial areas to the west.

During the Loma Prieta earthquake more than 1300m of the upper deck collapsed

(see Plates 5.1 and 5.2), killing 35 people.

5.2.2 Structure Description

Inherent in the geometry of this type of highway structure are complexities and variations where it crosses skewed roads at ground level, where access ramps join and where the double-decker configuration changes to side-by-side carriageways. The following brief description relates primarily to the unhindered double-deck structure which could be described as typical, although seismic performance can be disproportionately affected by the complex variations.

The structure generally comprised twin-level reinforced concrete multi-celled box girder decks supported by two-storey reinforced concrete portal frames (see Figure 5.2). Where the width of the frame was larger than usual, or where it was intended to widen the portal at a later date to add another access road, the top crossbeam was prestressed. Viaduct spans ranged generally from 68ft to 90ft (20.7m to 27.4m), but were most commonly around 80ft (24.4m).

Pseudo pin-joint connections were incorporated at the base of all lower storey columns, and two or three pin joints were also built into the upper storey columns. A brief analysis of the incidence of the three different articulation arrangements can be found in Figure 5.3. These pin joints were made by creating a simple concrete hinge with just 4No. 1 3/8in (35mm diameter) bars on a 6in (150mm) square grid passing through a reduced column section. A 6in (150mm) diameter thin gauge steel drain pipe ran through the centre of the hinge (see Figure 5.4).

Not only did the articulation vary, but the level of the lower pins in the upper columns and the width of the columns changed from bent to bent.

Lower storey columns were of constant rectangular section, while upper columns tapered on their inner face from a maximum at the upper deck crossbeam to a minimum typically at the top of the lower deck parapet. Most commonly the upper storey had two pins, located at the top of the lower deck parapet. In this general case the upper columns tapered from 4ft (1.22m) at the top to 3ft (0.91m) at the level of the pin joints. Stub columns 2ft 3in (0.69m) high extended up from the top of the crossbeam to the pin joint. Crossbeams were 8ft (2.44m) deep, and lower columns were 6ft (1.82m) wide. Normal to the plane of the portal bents all columns and beams were a constant 4ft (1.22m). Transverse reinforcement in the columns and stub columns comprised only 0.5in (13mm) diameter links at 12in (305mm) centres around longitudinal bars which varied from 1.25in (32mm) to 2.25in (57mm) diameter.

Typically decks were continuous over three spans, with expansion hinge joints at the ends of each three span module located one fifth of the span from the nearest support. These half-joints had a width of interaction of just 4.75in (120mm). As part of the retrofit programme following the 1971 San Fernando earthquake, cable restrainers were installed across these narrow expansion hinge joints.

Founding soils along the length of the structure are a mixture of sands, silts and clay, with some lenses of organic material and gravels. Foundations consisting of groups of vertical piles transferred structure loads down through loose surface deposits to stiffer sands and clays at a depth understood to be generally no greater than 10m. Typically these foundations were of 20No. piles supporting a 12x15ft (3.65x4.57m) cap under each column. No tie beams linked the two caps of each portal bent.

5.2.3 Damage

Failure of the upper storey columns brought down more than 1300m of the top roadway deck. Except at joints between collapsed sections and structure still standing, the retrofitted restrainers generally kept joints closed.

There were two exceptions to this picture of consistent failure of the upper deck only. At a skewed crossing of a rail track and local road, a single span of upper deck supported by Bents 96 and 97 remained standing. Besides their skew, these portal bents and the two neighbouring bents are notable for having no pins at the base of the upper columns and for having a central column to the lower storey. The other exception was close to the north end of the failed viaduct, where both decks collapsed over the two spans between Bents 104 and 106.

The minimal provision of shear and confinement reinforcement within the columns undoubtedly played a major part in consequential column damage. However, it appeared that most failures were triggered at the column/beam connection, either within the stub column and the beam or within the depth of the beam for those cases where the pin was at a lower level. The typical failure mode is illustrated in Figure 5.5.

In virtually every case of complete failure, whether the column was pinned or continuous at its base, there was evidence of brittle shear failure due to lack of confinement at the column connection with the transverse deck beam (see Plate 5.3). The foot of the column had kicked out sideways and the upper corner of the crossbeam had been broken off and subsequently abraded by the inner column face during collapse. Outer face reinforcement of the lower column had been peeled out, delaminating cover concrete (see Plate 5.4).

The degree of damage to the column itself was controlled by its end fixity. In some cases where the column had been pinned top and bottom, following failure of its opposite number which had no pin at the bottom, lateral displacements combined with loss of seating allowed the column section to fall free almost undamaged. Along one section the columns with pins top and bottom remained in position while the other side collapsed.

In many cases during collapse the bending capacity of the upper deck prestressed crossbeams was exceeded and the prestressing tendons had ruptured, shooting out from their anchorages by up to approximately one metre.

Horizontal gaps were found in some places between pilecaps and surrounding ground showing there had been differential movement and probably some softening of the soil during the shaking.

5.2.4 Interpretation and Conclusions

Failure was essentially due to brittle shear failure in the upper deck columns and their connections with the lower deck. Design codes in the early 1950's took little cognisance of possible earthquake effects. The codes and elastic methods in use at that time did not lead designers towards an awareness that design loading might be exceeded, or to an understanding of the importance of ductility to structural behaviour beyond elastic limits. If design had been to modern standards of loading and detailing in force in California, collapse would not have occurred.

The small longitudinal and transverse displacements between the collapsed upper deck and the lower deck suggested that sequential collapse had not occurred.

At first sight it seemed remarkable that the impacting deck did not induce collapse

of the lower storey also. However, because the retrofitted restrainers had generally avoided separation of the narrow half-joints the impact was mostly transferred through the upper deck crossbeam directly into the relatively robust lower storey columns.

The large number of pin-joints built into the structure affected dynamic response by reducing potential redundancy, energy dissipation and damping, and by increasing displacements. Besides reducing thermal and differential settlement stresses, the introduction of so many pin-joints facilitated analysis. At the time when this structure was being analysed no electronic calculators or computers were available to designers. The tapered upper columns focussed high shear stresses at their base. The column lateral reinforcement was constant and minimal, and column longitudinal reinforcement was in the form of widely spaced non-confined large bars. In the case of columns without pin-jointed bases, the column/beam connection was weak with no positive provision to confine and link column reinforcement through the joint with the beam reinforcement. All column bars were lapped at the same level, just 1ft (305mm) below the top of the lower deck crossbeam, i.e. within a zone which was highly stressed in shear and bending. Lap lengths were only twenty bar diameters. In the case of columns with pin-joints, the lack of lateral confinement in the stub column and crossbeam concrete directly below the joint allowed the column base to break free and induce collapse.

Once the upper corner of the stub column and crossbeam supporting the columns had broken away, characteristically at around 45°, the columns collapsed under cyclic loading with their inner faces grinding against the remaining crossbeam concrete and the outer bend of the top reinforcement in the beam. The horizontal load induced between the inner column face and the crossbeam, which increased rapidly as the upper deck fell, would have induced a large opening moment at the top of the portal leading to failure of that joint. Where the upper joint was pinned, failure would have been rapid.

Soil conditions undoubtedly played a part in the excitation applied to the structure. Although there were no signs of liquefaction or excessive settlements, softening due to increased pore water pressure in the saturated fill would have filtered out higher frequency waves and increased displacements.

5.3 EMBARCADERO VIADUCT, DOWNTOWN SAN FRANCISCO

5.3.1 Background

The Embarcadero Viaduct extends for about 1200m along the Embarcadero quayside in San Francisco, past the site of the World Trade Center, feeding Freeway 480 traffic between ramps at Broadway, Clay Street and Washington Street and Freeway 80, which crosses the Bay Bridge to Oakland or links with Freeway 101 south towards San Jose (see Figure 5.6).

This double-decked viaduct was completed in 1963, some six years after the Cypress Street Viaduct on the other side of the Bay.

5.3.2 Structure Description

In many respects this structure is similar to Cypress Viaduct, but with some variations, including upper storey columns of constant rectangular section rather than tapered (see Plate 5.5). These 4ftx4.5ft (1.22x1.37m) upper columns were typically reinforced with 0.9% longitudinal steel and 0.5in (13mm) diameter links at 12in (305mm) centres.

Mid-depth hinge joints were provided at every third span. The width of structural interaction at these joints was increased from 4.75in (120mm) used on the Cypress Viaduct to 6in (150mm). Cable restrainers had been installed across the joints as part of the retrofit programme following the San Fernando earthquake (see Plate 5.6).

Again simple concrete hinge joints were provided at the connection between lower storey columns and pilecaps. As at the Cypress Viaduct the location of hinges in the upper storey varied depending on the structure geometry, incorporating bifurcating on/off ramps on its west side. In the general non-widened case, however, whereas Cypress had two hinges at the base of upper storey columns, Embarcadero had two hinges at the top of the columns.

Where portals had greater spans to accommodate the extra roadway for on/off ramps, the upper crossbeams were prestressed and a three-pin articulation was incorporated in the upper storey. On one side the column was pinned top and bottom, and on the other side the column was pinned at the top.

In 1985 cable restrainers were fitted across the top hinges wherever this three-pin arrangement had been used. Also horizontal restrainers were fitted between original structure and subsequently constructed ramps to avoid pounding due to non-synchronous displacements of adjacent structure.

Foundation soils in this area comprise uncompacted fill materials placed to reclaim ground for the 1915 World Trade Fair close by. Steel H-piles transfer loads to stiffer deposits at depth. No link beam was provided between individual pilecaps of each portal bent, but like Cypress Street the lower storey frame seemed stiff enough to restrain any non-synchronous base excitation effects.

5.3.3 Damage

The most severe structural damage observed was diagonal shear cracking below the base of the east side upper column at Bent No. 78, within the depth of the lower deck crossbeam. At this location spalling had occurred and some reinforcing bars were visible (see Plate 5.7). Diagonal cracking at the column/lower crossbeam junction was also observed at Bents 79, 90-93 (east side) and at Bents 76-81 (west side).

Cracking was not limited to the bents with three pins in the upper storey, but was observed at various bents with top pin joints only.

There was evidence of settlement of ground around pilecaps, particularly at Bents 73-77 and 86-87. These settlements and accompanying horizontal strains had caused cracking of asphaltic surfacing under the viaduct (see Plate 5.8).

5.3.4 Interpretation

The damage to the structure fits well with the mode of response postulated for the Cypress Street Viaduct. The greatest damage at Bents 76-78 corresponds to a section of structure with three pins in the upper storey, leaving just one monolithic joint to carry the horizontal inertial shear load. That complete collapse did not occur is likely to have been due to the lower intensity of ground shaking at this site and to some small improvements in detailing.

The pin joint restrainers on the three pin bents showed that CalTrans were aware of the potential weakness of the multiple-pin articulation, but strengthening work had not been implemented on the two-pin bents at Embarcadero nor on Cypress Viaduct. The particular form of restrainer, although simple to fit, would have a

limited range of effectiveness.

5.4 OAKLAND BAY BRIDGE, SAN FRANCISCO

5.4.1 Background

The Bay Bridge is in fact two separate bridges connecting San Francisco and Oakland. The crossing links the two cities by way of Yerba Buena Island within the Bay. The total length of the route is 8.4 miles (13.4km), and it carries the two five-lane carriageways of Interstate 80 on twin levels.

The bridges were first opened in 1936 and now carry daily an average of 260,000 vehicles. Caltrans has staff assigned on a full-time basis to inspect and maintain the structures.

5.4.2 Structure Description

The bridge connecting San Francisco and Yerba Buena Island consists of twin suspension bridges with a shared central anchorage. Its length is 9260ft (2822m) with main spans of 2310ft (704m). The towers are 526ft (160m) high and a vertical navigation clearance of 220ft (67m) is provided. The twin decks are carried at the upper and lower levels of the deck stiffening truss.

The bridge connecting Yerba Buena Island and Oakland consists of mixed steel trusses totalling 10176ft (3102m). Cantilever truss bridges with a maximum span of 1400ft (427m) and vertical clearance of 191ft (58m) supports the twin decks over the navigation channel. Mid-way between the island and Oakland, at pier E9, the structure changes to shorter approach spans and lighter shallower trusses. Again the structure is double-decked to carry twin level carriageways (see Figure 5.7).

It is presumed that the bridge piers are founded deep below the Bay mud on caissons. In 1974 hinge restrainers were installed at most expansion joints to limit large seismic relative displacements between sections, and thereby reduce the risk of spans being dislodged from their bearing shelves.

5.4.3 Damage

Major structural damage during the Loma Prieta earthquake was limited to a single section of the Yerba Buena to Oakland truss crossing. There was no reported damage to the suspension bridges.

Damage occurred at the 60m high trestle pier E9 (see Figure 5.8 and Plate 5.9) where the two types of truss bridge meet: the deep girder of the cantilever truss and the shallower trusses of the approach spans. The pier has four braced vertical legs and is rigidly connected to both truss structures. Pier E9 is understood to have provided anchorage against longitudinal forces on the deck arising between expansion joints at piers E4 and E11, a distance of some 3176ft (968m).

The twin concrete-decked spans above the pier are supported on longitudinal stringers of approximately 50ft (15m) span. The stringers are each seated on brackets bolted to the webs of transverse girders at the ends of each truss. It is understood that no restrainers had been fitted across the joints here.

Failure occurred under reported differential longitudinal movements of over 6in (150mm) at the pier. A Caltrans engineer quoted a peak differential movement of 11in (275mm) indicated by scratches on girder paintwork at the first joint to the west from pier E9. Following the earthquake the approach spans at pier E9 had

displaced relative to the pier 5in (125mm) longitudinally, and 1.5in (38mm) transversely. The seating of the stringers was not sufficient to accommodate the longitudinal movements. The upper stringers came off their seatings on the west (cantilever truss) side and rotated about their supports on the east side. Impact from the upper deck on the lower deck caused a similar failure of the lower deck. The two decks came to rest supported on steelwork of the tower beneath (see Figure 5.8). It was reported that bolts of 1in (25mm) diameter locating the Oakland span had failed. Caltrans quoted the failure load of those bolts together as being 2 million pounds (900tonnes).

During repair the approach spans were jacked back to their original position, and the contractor stated that a force of 170t was required from each of the two jacks used (see Plate 5.10).

5.4.4 Interpretation

On a long structure it is to be expected that differential substructure movements will occur due to wavelength effects and soil/structure response. It is clear that the longitudinal motion of the supporting substructure in this case was greater than could be accommodated by the stringer supports. The width of bearing for the stringers should certainly have been greater than the 5in (125mm) which was originally provided. The current AASHTO Guide Specifications for Seismic Design of Highway Bridges give minimum support widths, but these were published some 45 years after the bridge was opened and after many advances in seismic design.

The replacement stringers are to be supported on neoprene pads seated on 8in (200mm) wide brackets, Figure 5.9. This width of support is less than would be required under the guide specifications, but it is understood that some form of retrofitting is to be applied to safeguard the span under the next major earthquake. The integrity of this span over pier E9 is, of course, linked to the safety of the adjacent pier to pier spans, and the width of seating of all spans of the Bay Bridge will need to be reviewed carefully. The simply supported spans are more vulnerable to being dislodged than the cable and cantilever constructed spans.

5.5 LAUREL STREET BRIDGE, SANTA CRUZ

5.5.1 Background

This bridge carries a single carriageway road, Laurel Street, over the San Lorenzo River in Santa Cruz (see Figure 5.10 and Plate 5.11).

It was built in 1968 and carries town traffic from west to east over the river. At this point the river is tidal, and large variations in level occur.

The bridge suffered minor damage in the earthquake, and following emergency repairs was again carrying traffic at the time of the EEFIT visit.

5.5.2 Structure Description

The bridge has three spans and is constructed in reinforced/prestressed concrete. The deck comprises a multi-cellular variable depth girder with flush soffit, and a suspended span between half-joints completing the main central span.

Heavily flared reinforced concrete piers give the architectural impression of being made up from a series of rectangular slabs of increasing dimensions laid one on top of another. The two piers are monolithic with the deck.

The bankseat abutments have in-line wing walls. From their relative settlement during the earthquake it was clear that the bankseat was piled and the wing walls were on spread foundations.

5.5.3 Damage and Interpretation

At the south-west end of the bridge there had been settlement of fill behind the abutment leading to 3in (75mm) settlement of the wing walls relative to the piled abutment, and corresponding rotation of the run-on slab. In front of the abutment there had been slumping of the embankment as a result of a reduction in shear strength due to increase in pore water pressure during the shaking (see Plate 5.12). Minor impact damage had also occurred.

At the north-east abutment there had been some minor settlement. Some spalling at the junction of the end of the bridge and the wingwall copings showed there had been impact during the shaking. The gap between the two was closed.

5.6 RIVERSIDE AVENUE BRIDGE, SANTA CRUZ

5.6.1 Background and Description

This bridge carries Riverside Avenue on a north-south axis over the San Lorenzo River, downstream from the Laurel Street Bridge.

It was built in 1939. Previous scour damage had undermined one of the piers allowing it to settle and rotate. This differential settlement of the supports to the continuous superstructure had led to a tension crack above the pier. As a result of this earlier scour damage and a need to widen the structure to cope with increased traffic, the bridge was already on City Hall's replacement programme. Earthquake damage had caused its closure and will speed its replacement.

5.6.2 Structure Description

The bridge has three spans and is constructed in reinforced concrete. It has a variable depth superstructure supported on two piers in the river (see Plate 5.13). Piled cantilever wall abutments are flanked by angled walls to guide flow through the end spans, and by in-line wing walls carrying a continuation of the deck parapet. There are movement joints at each abutment.

5.6.3 Damage and Interpretation

Extensive settlement of the approaches on both north and south sides was observed. During the earthquake increased pore pressure had softened the saturated ground, and this partial liquefaction was reported to have left water "boiling" out of the embankment following the earthquake.

At the north abutment slumping and settlement of fill (see Plate 5.14) had led to settlement of the wing walls relative to the abutment which had ruptured the reinforced concrete parapets. Measurement across the parapet at the north end showed a differential settlement of up to 8in (200mm) (see Plate 5.15). The expansion joint was closed and the run-on slab had translated and rotated about its hinge at the back of the abutment wall, leaving a gap of about 4in (100mm) in the roadway surface at the north end.

The flow guide walls rotated during the earthquake leaving the top as much as 3in (75mm) forward of its original position. At the south abutment the damage was of the same pattern, but the magnitude of the settlement was less.

5.7 CUT BIAS BRIDGE, SANTA CRUZ

5.7.1 Background and Description

This reinforced concrete bridge (see Plate 5.16) was reported as having been built in 1939, although it looked somewhat younger. It comprises 6 spans of approximately 8m of in-situ beam and slab construction, with a joint over the central pier. Supports comprise piled bankseat abutments and portal piers. A water main is supported from the inner face of the north columns of the portal piers.

The arm of the river which it crosses has been extensively culverted and filled upstream, and present day flows are small.

Three of the portal bents showed signs of previous distress, in the form of cracks and rusting reinforcement at the top of north side columns (see Plate 5.17). At first this was attributed to differential settlement of the south side columns, i.e. the columns adjacent to open water. On reflection, however, it was clear that the deformation in the plane of the bent which had caused the earlier cracking could only derive from movement of the north columns themselves, independent of the crossheads and other columns. If differential settlement had occurred there would, almost certainly, have had to be a corresponding crack near the top of the south columns, and there was no cracking at this location.

The explanation lay in the upstream filling which came very close to the bridge, terminating with a steep slope adjacent to the columns (see Plate 5.17). The fill surcharge must have caused lateral spreading of underlying soft alluvial soil, either through normal long-term movements or triggered by previous tremors.

5.7.2 Damage and Interpretation

Liquefaction and slumping of fill had taken place behind the abutments, and the differential settlement between the approach road and the abutment had led to disruption of paving (see Plate 5.16). Cracks had also appeared in the embankment/levee adjacent to the abutment.

New diagonal cracks in the crossbeam of the first portal bent from the west end were probably the result of flow of liquefied soil from under the abutment during the shaking. This would have moved the toe of the south pile/column away from the abutment. As already discussed, the north column is embedded in and restrained longitudinally by the toe of the upstream fill, and so torsion would have been induced in the crosshead.

5.8 MURRAY STREET BRIDGE, SANTA CRUZ

5.8.1 Background and Description

Otherwise known as the Glen E Coolidge Memorial Bridge, this 9-span structure was completed in 1963, and carries Murray Street over the yachting marina in Santa Cruz (see Plate 5.18). A water main is also carried suspended from hangers under the southern deck edge cantilever. The superstructure comprises simply-supported prestressed beams with an in-situ deck slab. The beams are supported on rubber or neoprene pad bearings on twin column portal piers and bankseat abutments. Both the abutments and piers are piled.

The deck joint above the first pier from the western end was jammed closed at the northern edge of the deck and 2in (50mm) open at the southern edge. It was clear

from old asphaltic filler in the joint that most of this deformation predated the earthquake.

At the east abutment and the first pier from the east end scour had exposed the pilecap and tops of the 15in (380mm) diameter piles. The piles had been cast in-situ within corrugated steel tubes, which in this tidal zone had completely corroded.

5.8.2 Damage and Interpretation

The first pier from the west end was out of plumb with an inclination of approximately 1 in 50 (see Plate 5.19). The west abutment had moved some 1.5in (40mm) back from the fill in front of it (see Plate 5.20), and at deck level a crack had opened in the surfacing between the backwall and the fill. These permanent displacements were probably caused by a combination of inertia loads from the deck and some softening of the soil due to a temporary rise in pore water pressure during the earthquake.

At the second pier from the west end some spalling of column concrete had exposed reinforcement at the level where the two columns were infilled with a wall near ground level, and there was some minor cracking near the top of the north column.

The kink in the deck at the first pier from the west end which resulted in the uneven joint gap had been made slightly worse by the earthquake.

The corroded casing to the exposed piles under the first pier from the east end had been loosened by the earthquake and could be readily broken away by hand. Also the thin coating of hardened cement slurry which had been spread over the scoured surface of the bed around this pier, presumably to retard further scour, was freshly cracked (see Plate 5.21). However, there was no obvious sign of fracture of the piles.

5.9 STRUVE SLOUGH BRIDGE, WATSONVILLE

5.9.1 Background and Description

These are two reinforced concrete bridges, each carrying one carriageway of Highway 1 over the marshy area known as Struve Slough, just outside Watsonville. Each deck is of in-situ reinforced concrete beam and slab construction supported at close centres (approximately 5m) by bents of four vertical piles built in to a crossbeam linking the five beam ribs. These bents were at skews up to 25°, and comprised steel cased in-situ reinforced concrete piles below ground level, extended upwards as circular columns to meet the crosshead.

The deck was generally continuous, but at those locations where halving expansion joints were provided, cable restrainers had been retrofitted.

5.9.2 Damage and Interpretation

About half of the westernmost structure collapsed as a result of failure of the pile/deck connection, and in doing so four of the fractured piles punched through the deck slab (see Plates 5.22 and 5.23).

There had clearly been large horizontal displacements of the deck relative to the supporting trestle piles. While shaking proceeded displacements would have increased as the ground softened due to increased pore water pressure and as the pile/deck connections progressively deteriorated. At the northern abutment a 4in

(100mm) gap had opened up between the abutment and the fill behind.

At the base of each pile under the non-collapsed deck there was a circle of crushed concrete which had dropped as the pile/deck connection had undergone large cyclic rotations (see Plate 5.24). Pile binding reinforcement consisted of just 3/16in (4mm) wire wrapped at approximately 150mm centres which was totally inadequate to confine the 6No 0.75in (20mm) diameter longitudinal reinforcement and the concrete (see Plate 5.25).

The cable restrainers had served to keep a continuous roadway during collapse, which could have meant the difference between life and death to drivers. However, the protruding piles would have presented a formidable hazard to any unsuspecting driver who had the misfortune to try to cross the western bridge in the outside lane just after the earthquake. It was reported that a police patrol car was a write-off after trying to cross the bridge at speed. Because it was on the inside lane the occupants escaped unhurt.

5.10 GOLDEN GATE BRIDGE, SAN FRANCISCO

5.10.1 Background and Description

At the time of its completion in 1937 the Golden Gate Bridge (see Plate 5.26), with a main span of 4200ft (1280m), was the longest suspension bridge in the world. It links San Francisco with Marin County to the north, and its six traffic lanes carry an average of 111,000 vehicles per day.

Overall it is 6450ft (1966m) long and its towers are 227m high. At midspan a clearance of 220ft (67m) above high tide is provided, which allows clear passage for all shipping.

The bridge has only been closed three times, each time due to hazard to motorists from high winds.

Previous analysis had indicated that the main bridge could sustain a magnitude 8 earthquake. Following the 1971 San Fernando earthquake, analysis of the arch and girder approach spans suggested that they could fail in a magnitude 4-5 event. Hence between 1981 and 1982 additional holding-down bolts were incorporated together with span-to-span restrainers, and the bearing shelf at the Marin end was widened to 2ft (610mm).

Between 1982 and 1987 the 7in (178mm) thick concrete deck was replaced by an orthotropic steel deck with a 2in (50mm) epoxy asphalt wearing course.

5.10.2 Damage and Interpretation

Very little damage was sustained during the Loma Prieta earthquake. At two expansion joints on the main bridge the new concrete footway had suffered minor impact damage (see Plate 5.27), exposing the epoxy coated reinforcement. The strengthening of the approach structures seemed to have performed well.

Mr Dan Mohn, the District Engineer responsible for the bridge was in his car close to mid-span when the shaking occurred. To keep control of his car he had to slow from around 45mph to 25mph. He observed the random swinging of the hangers and estimated that at mid-span the differential horizontal movement between the deck and the cables was "*probably more than a foot*". There had been reports of damage to the coating of the short mid-span hangers but this was not confirmed.

POSTSCRIPT

The above report was written immediately following the EEFIT visit to California in 1989. In the period prior to publication various other reports have appeared, the most comprehensive and authoritative to which the authors have had access was that prepared by the Board of Inquiry to the Governor (Reference 1). An important and humbling statement made by Housner in that report was:

"The Board cautions that there is still lack of experience for bridge behaviour during the very strong and long duration shaking that would result from a major earthquake. The long-term process of understanding the impacts of earthquake ground motions has just begun. Research and experience have much yet to teach on how to design and construct new bridges and upgrade existing ones."

In the light of the information contained in that report and others, and from correspondence from Brian Evers of Santa Cruz, the following comments, quotes, additions and amendments are made. They are far from comprehensive:

Cypress Viaduct

1. The final death toll from the collapse of the Cypress Viaduct was forty-one and not thirty-five.
2. Preliminary design for the Cypress Viaduct commenced in 1949, construction was carried out between 1954 and 1957, the seismic design requirement between 1949 and 1954 was only 0.06g and this had been set in 1943. This compared with the Uniform Building Code requirement for Oakland and San Francisco of 0.16g.
3. Shortly after the earthquake contracts were let to a value of US\$3.5 million to demolish and remove the structure in its entirety.
4. Reference 1 states that *".... there is no evidence of failure of the foundation system or that the foundation contributed to the failure of the bents."*
5. Elastic response analyses estimated top storey shear to be some three times available capacity as calculated by the ACI code (average value from Reference 1). The EEFIT author had calculated a capacity based on BS5400 which gives a ratio of approximately 2.5. However, in loading tests on the southern portion of the Cypress Viaduct which remained standing the ratio between estimated top storey shear and effective failure was of the order of 1.7, showing both the ACI and BS5400 code predictions to be highly conservative in practice in this case.
6. Reference 1 states that *"Following the San Fernando earthquake of 1971 a decision was made to first utilize the limited funds available for retrofitting to install only longitudinal restrainers at the transverse expansion joints in bridge decks. This was done for the Cypress Viaduct in 1977, but unfortunately no detailed comprehensive analyses of the entire structure system were made to determine if other weaknesses existed. Such analyses, with methods available in 1977, would have predicted the failure of the Cypress Viaduct under a ground motion equivalent to that experienced in the Loma Prieta earthquake of October 17, 1989 or greater."*

Embarcadero Viaduct

After extensive studies into retrofit alternatives, the decision was taken to demolish this structure completely. Work to take down the structure was tendered at US\$3.25 million, began in March 1991 and was programmed to last 80 days

(References 2 and 3). The basis for the decision to demolish was the cost of US\$69.5 million for retrofitting and community opposition to a structure which not only had been shown to be seismically vulnerable but which also occupied a prime waterfront site in front of the historic Ferry Building. According to Reference 2 the structure is to be replaced by a £60 million underground link.

Oakland Bay Bridge

1. East of Yerba Buena Island the rock stratum slopes down very sharply. Consequently concrete caisson piers E3 to E5 extend to between -54m and -72m, passing through strata of bay mud and sandy silty clay with varying stiffness and amounts of sand, founding in sand and gravel layers. The remaining piers of the East Bay Crossing are supported on timber piles with toe levels of between -35m and -38m.
2. Retrofit carried out in 1976 comprised:
 - rods and tie-downs were installed near the east ends of the concrete stringer spans at lower deck level of bents E23 to E27, and at the upper deck ramp level of bents E34 to E38;
 - rods were installed near the east ends of steel stringer spans at upper deck level at bents E25,E27,E29 and E31;
 - steel restrainers with elastomeric pads were installed at expansion shoe locations at bents E17 to E22.

Riverside Avenue Bridge, Santa Cruz

This bridge was judged to be irreparable and was demolished (Reference 4).

Murray Street Bridge, Santa Cruz

1. At bent 6 (from the western end) the two southernmost raking piles were found to have sheared through at approximately 1.2m below pilecap level (Reference 4).
2. The columns of portal bents 2,3 and 4 were strengthened by casting a reinforced concrete casing connected into the concrete of the existing structure by hooked 0.5in diameter dowels epoxy grouted into drilled holes in the columns, and by 1.0in diameter bars epoxy grouted into holes in the pilecaps (Reference 4).

Golden Gate Bridge

According to Reference 5 it is planned to retrofit the bridge to resist an earthquake of magnitude 8.3 on the Richter Scale. Preliminary studies have been carried out by T Y Liu International of San Francisco and they concluded that under this intensity:

- the approach viaducts might collapse;
- the main cables could slide off the towers;
- the stiffening trusses might slam into the lower legs;
- the towers might rock, with uplift as much as one foot.

Final design work will be split into three contracts: the north and south approach structures and the main bridge, and will cost US\$9 million. The cost of the works is estimated to be in the region of US\$130 million.

REFERENCES

1. Housner G W, "Competing Against Time", Report to Governor George Deukmejian from the The Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake, published by the State of California, Office of Planning and Reasearch, May 1990.
2. New Civil Engineer, 11 October 1990, page 15, "California Gets Serious".
3. Engineering News Record, March 11 1991, page 9, "Adios Embarcadero".
4. Letter from Brian Evers of Santa Cruz Public Works Department dated 5 April 1990.
5. Engineering News Record, January 6 1992, page 12, "Extensive seismic retrofit planned for Golden Gate".

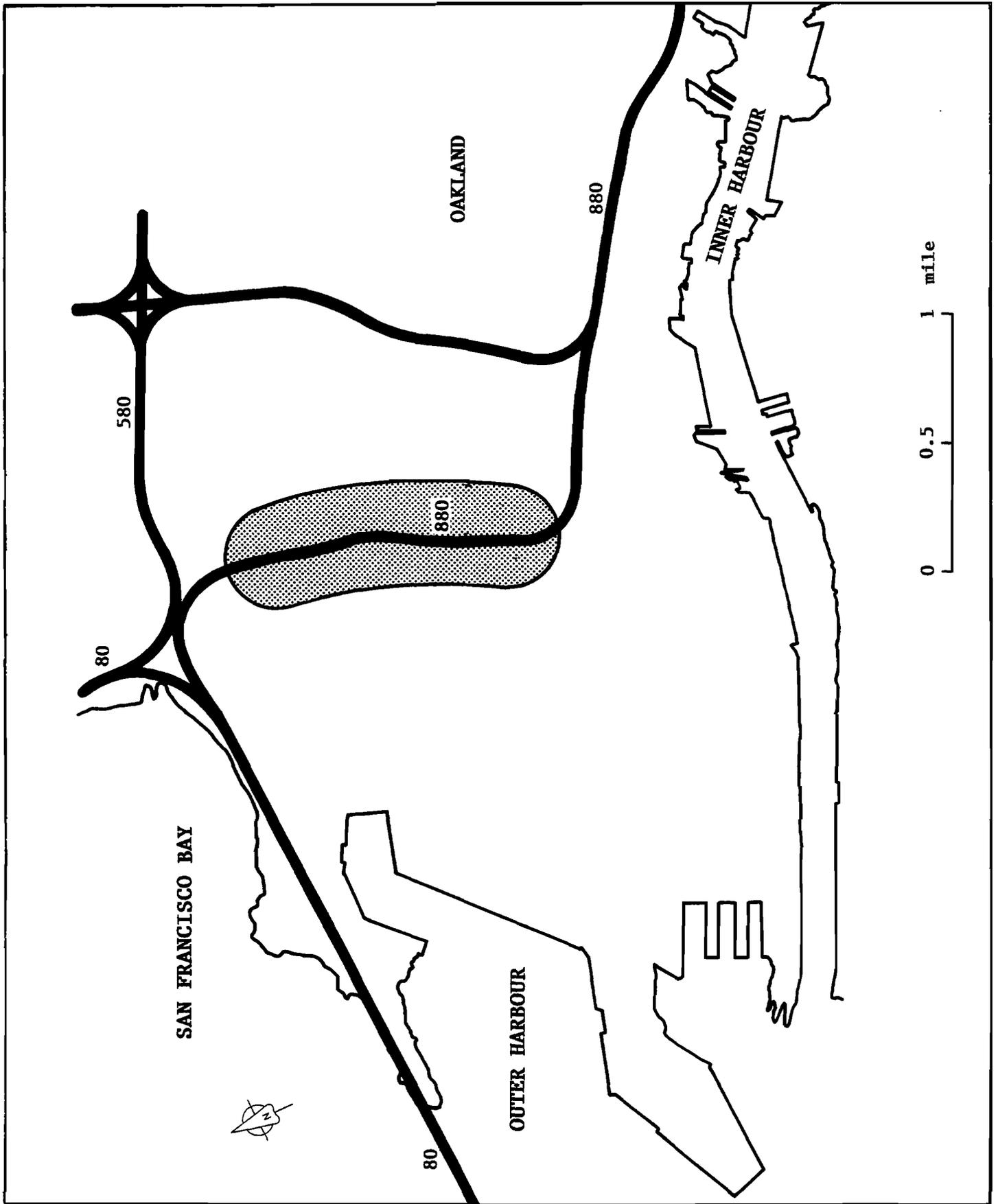


Figure 5.1 : Map showing location of Cypress Street Viaduct

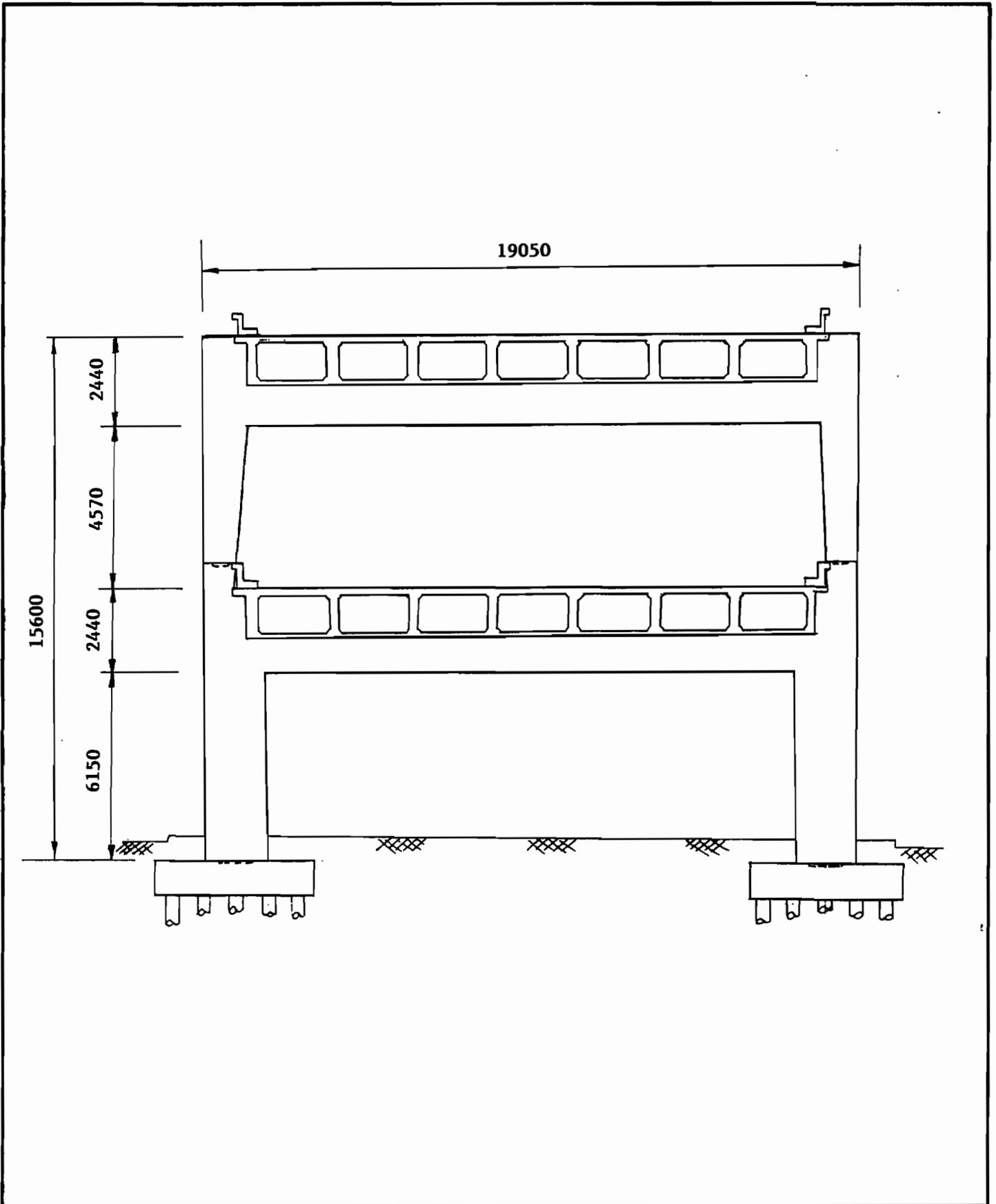


Figure 5.2 : Typical cross section of Cypress Street Viaduct

ARTICULATION AND INCIDENCE OF EACH (BENTS 62 TO 113)

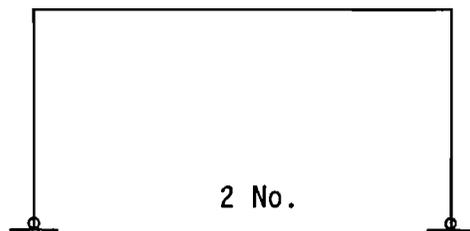
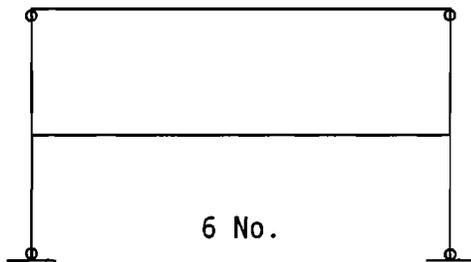
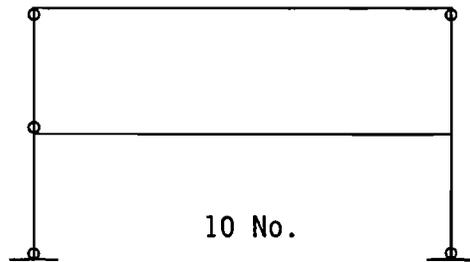
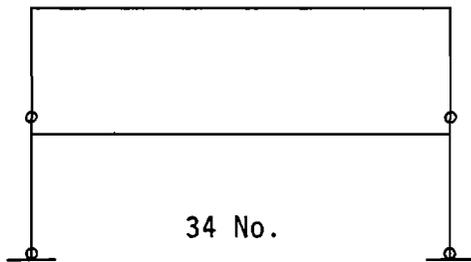


Figure 5.3 : Articulation of Portal Bents along length of Collapse

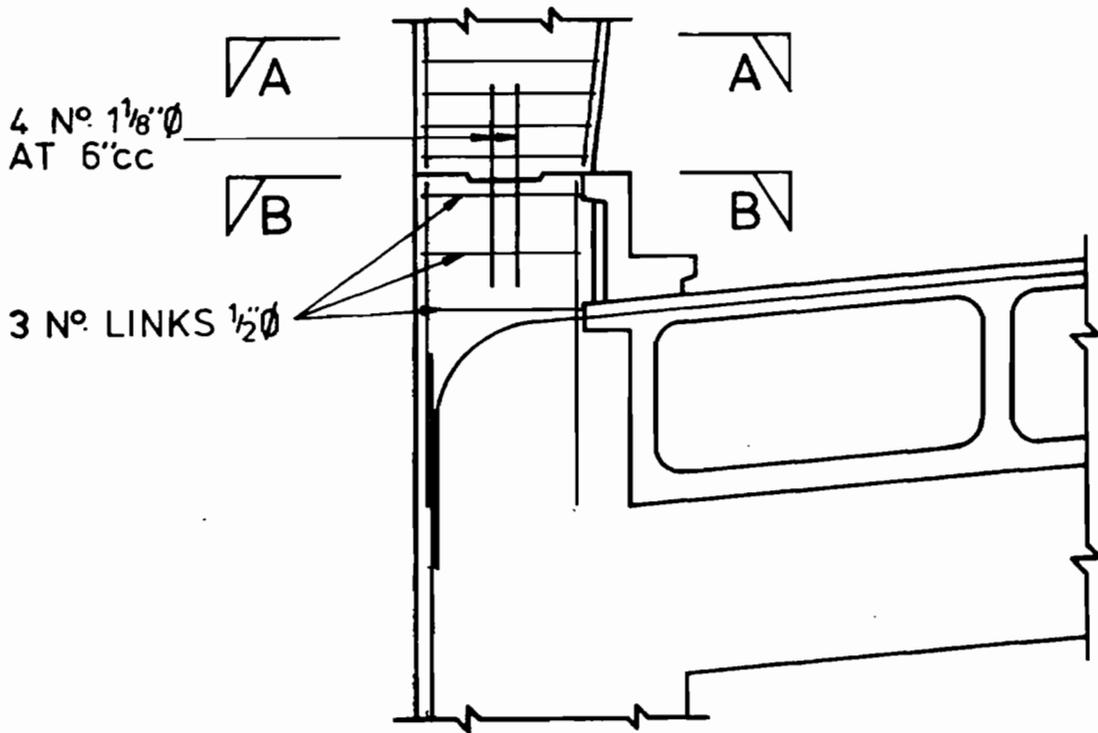
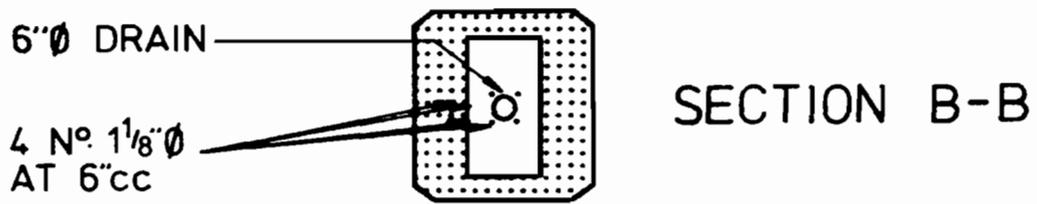
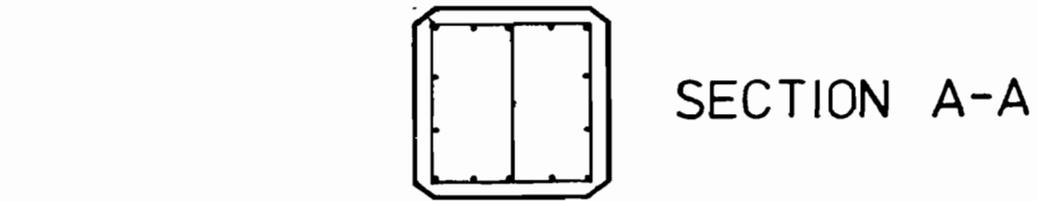


Figure 5.4 : Detail of pier joint between upper column and lower deck

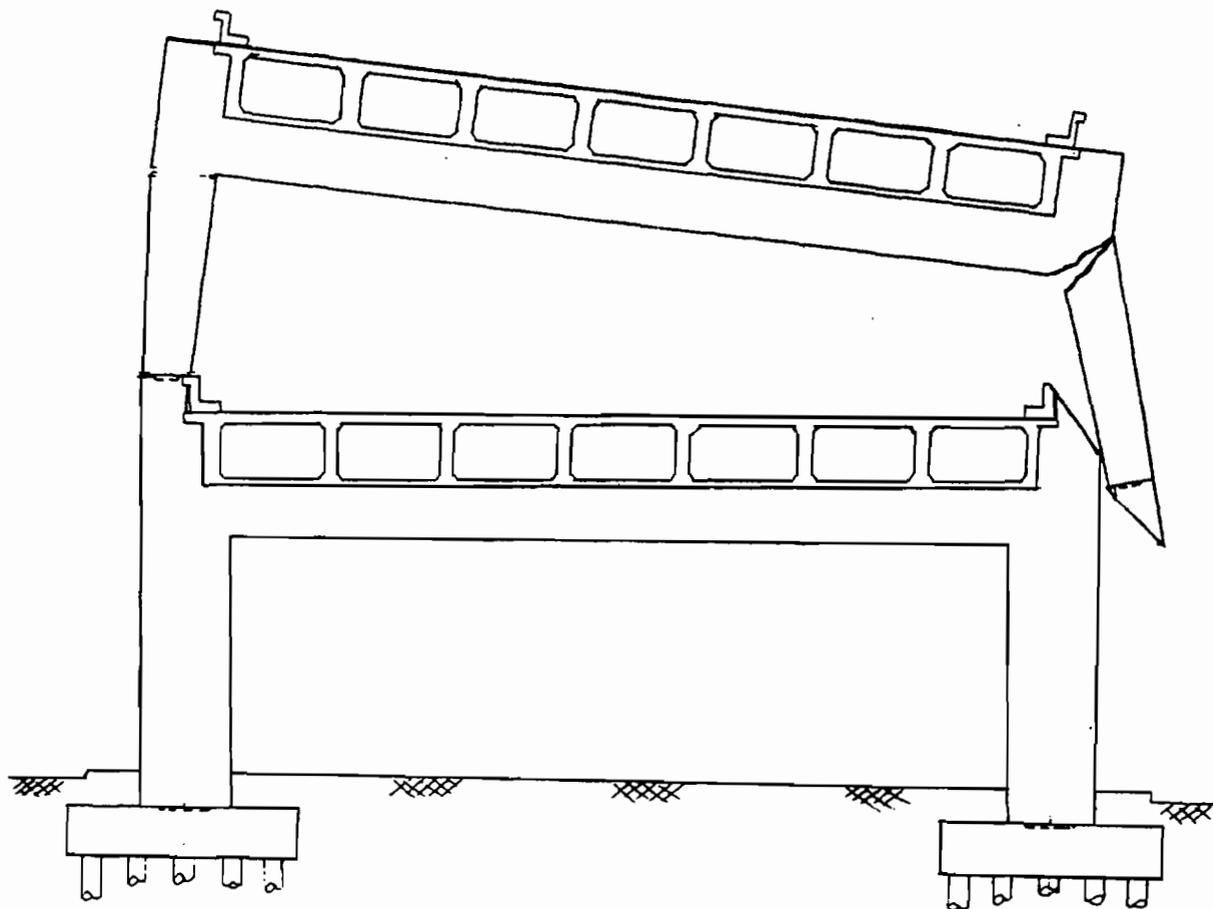


Figure 5.5 : Typical failure mode initiated by brittle shear at base of upper column

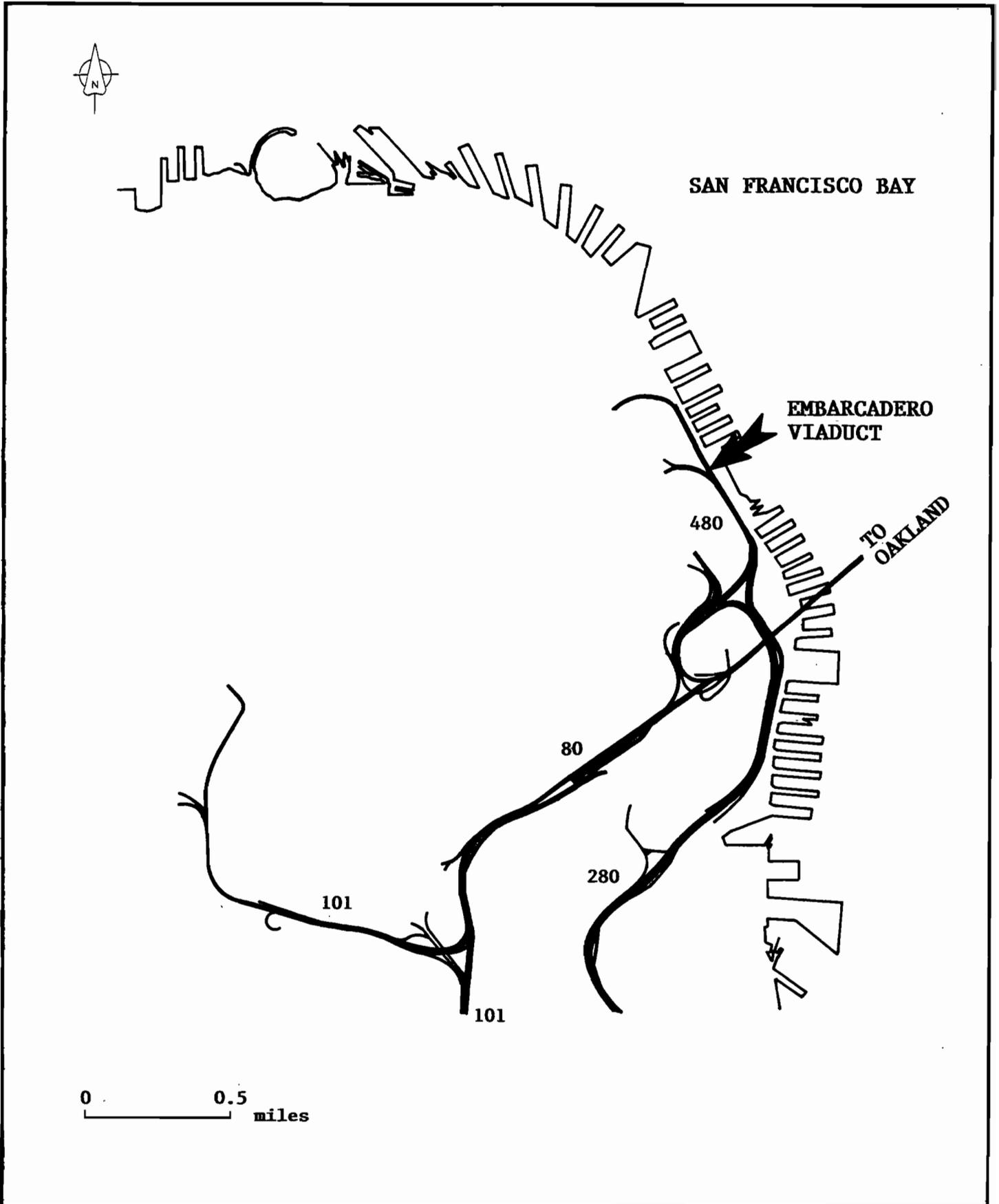


Figure 5.6 : Location of Embarcadero Viaduct, San Francisco

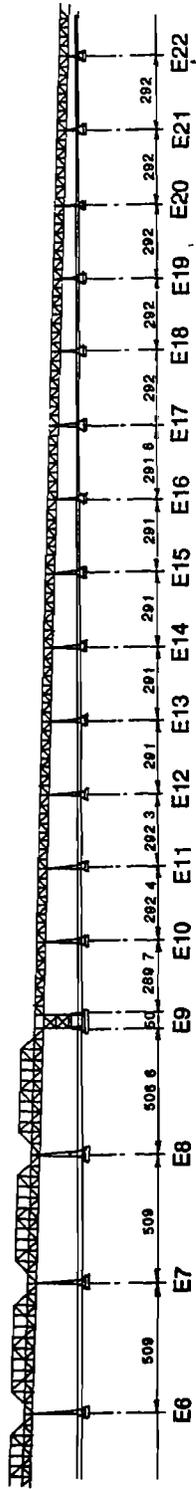


Figure 5.7 : San Francisco - Oakland Bay Bridge
Oakland Approach Spans

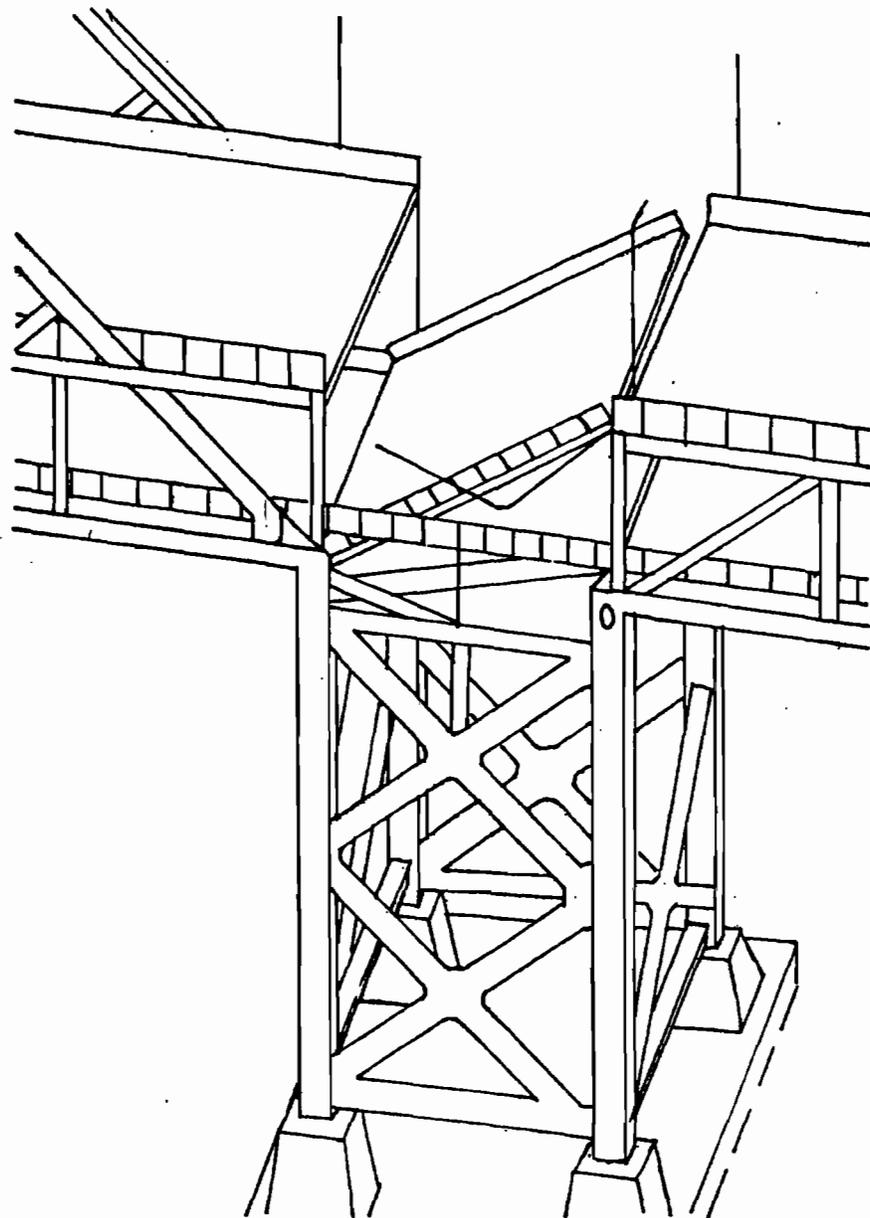
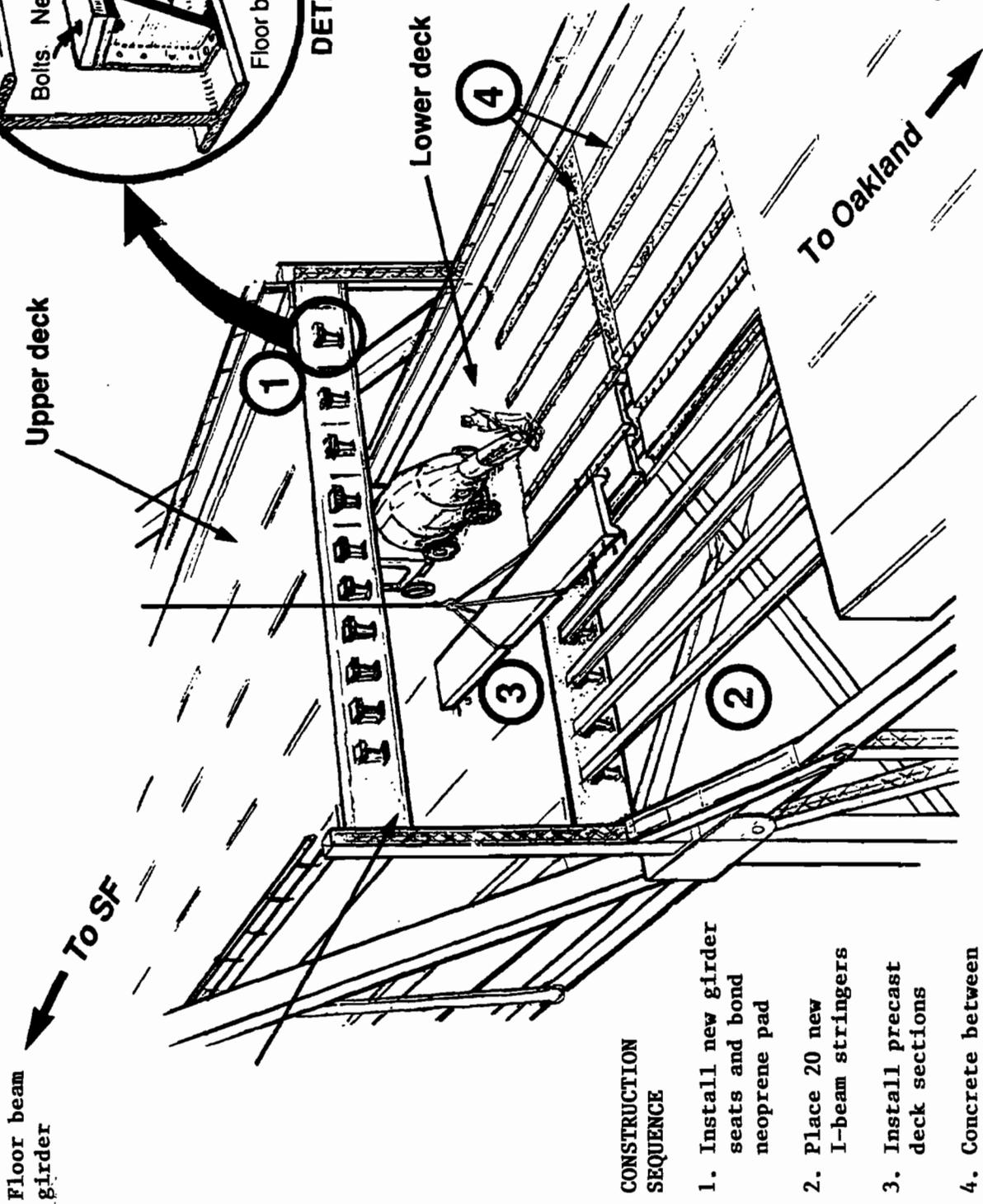
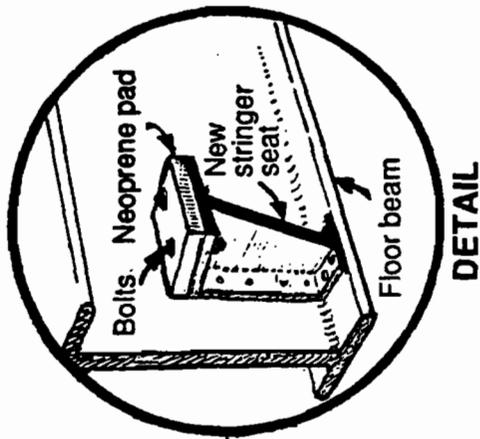


Figure 5.8 : Collapse of Bridge Decks at Pier E9



CONSTRUCTION SEQUENCE

1. Install new girder seats and bond neoprene pad
2. Place 20 new I-beam stringers
3. Install precast deck sections
4. Concrete between deck sections

Figure 5.9 : San Francisco - Oakland Bay Bridge Repair

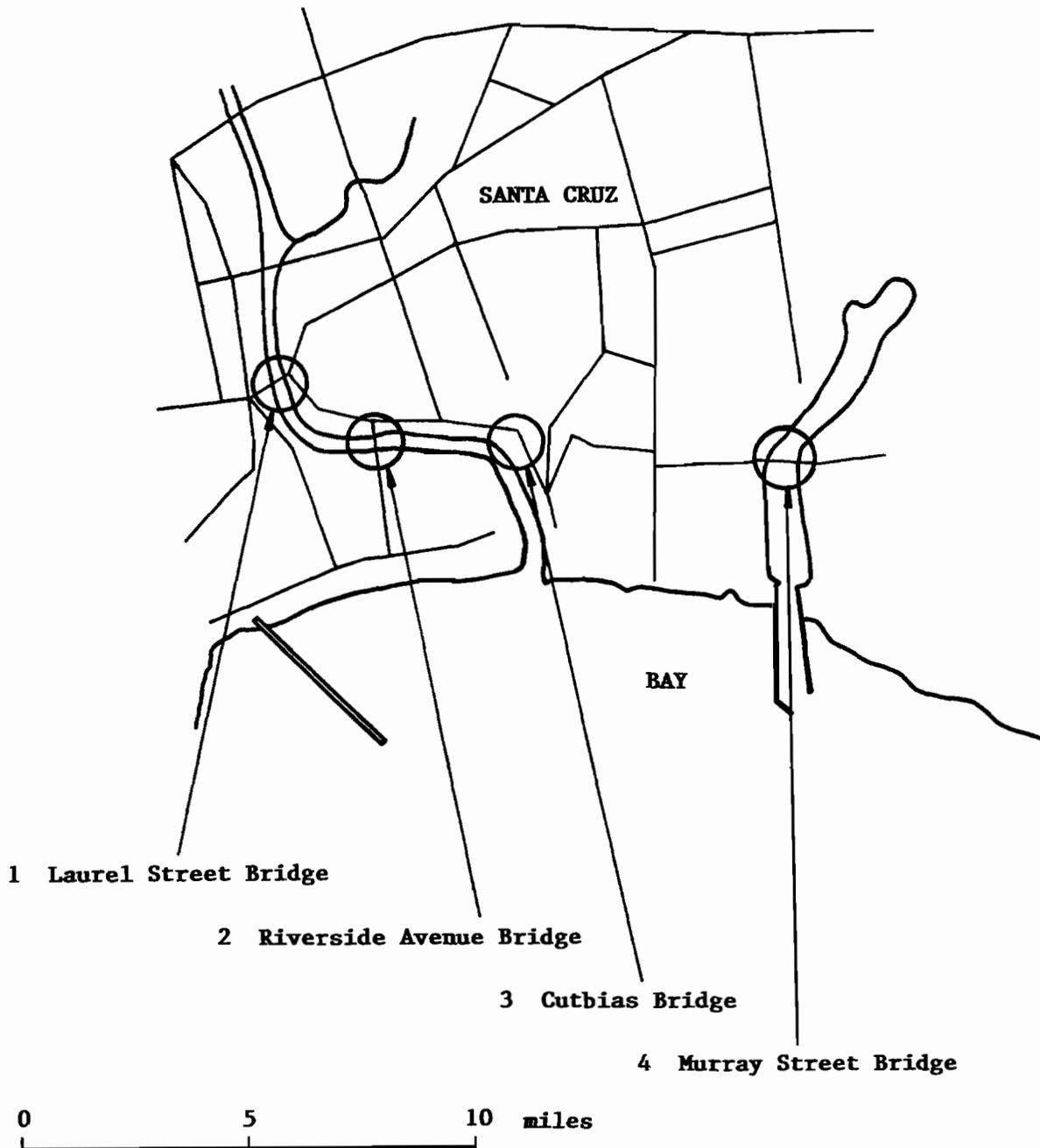
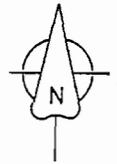


Figure 5.10 : Map of Santa Cruz



Plate 5.1 : Cypress Street Viaduct - Looking South along collapsed deck



Plate 5.2 : Cypress Street Viaduct - General view of collapsed viaduct



Plate 5.3 : Cypress Street Viaduct - Typical brittle failure of upper column



Plate 5.4 : Cypress Street Viaduct - Outer layer of lower column reinforcement and cover pushed off



Plate 5.5 : Embarcadero Viaduct - General view



Plate 5.6 : Embarcadero Viaduct - Cable restrainers (under metal covers at top left hand corner)



Plate 5.7 : Embarcadero Viaduct - Concrete spalling at beam/column connection

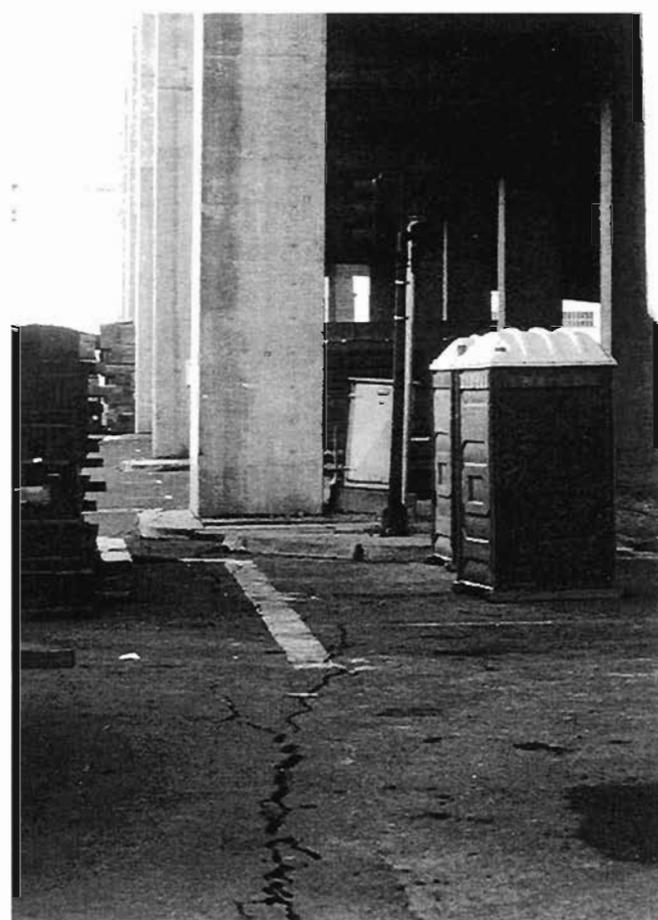


Plate 5.8 : Ground settlement and cracking

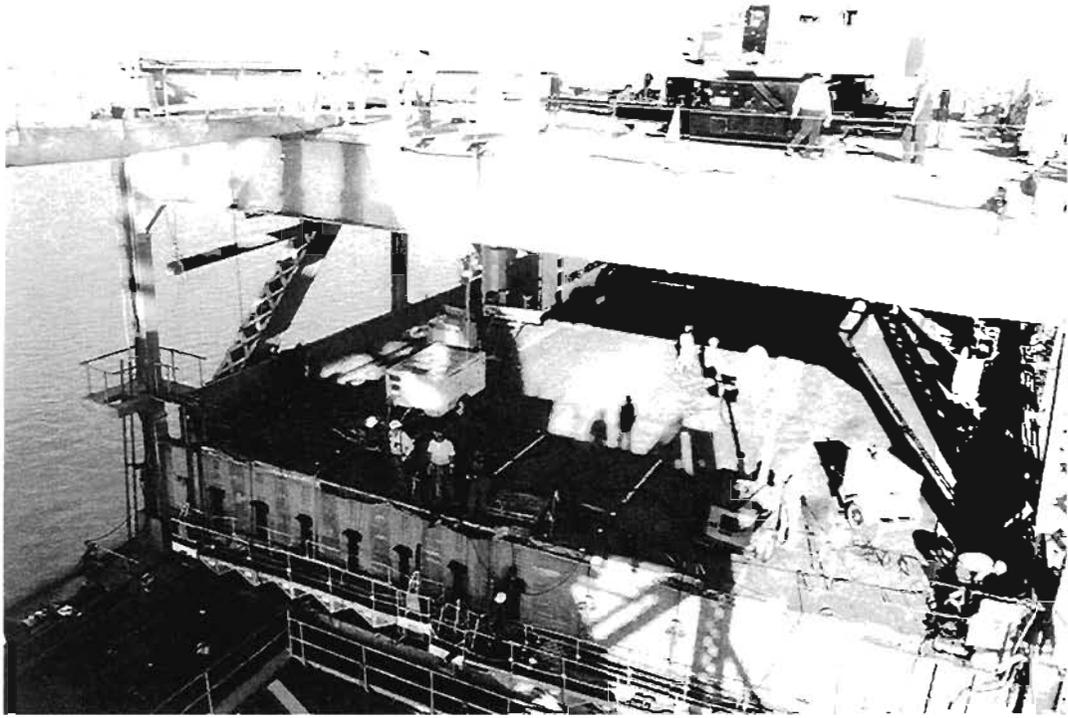


Plate 5.9 : Oakland Bay Bridge - View across collapsed deck

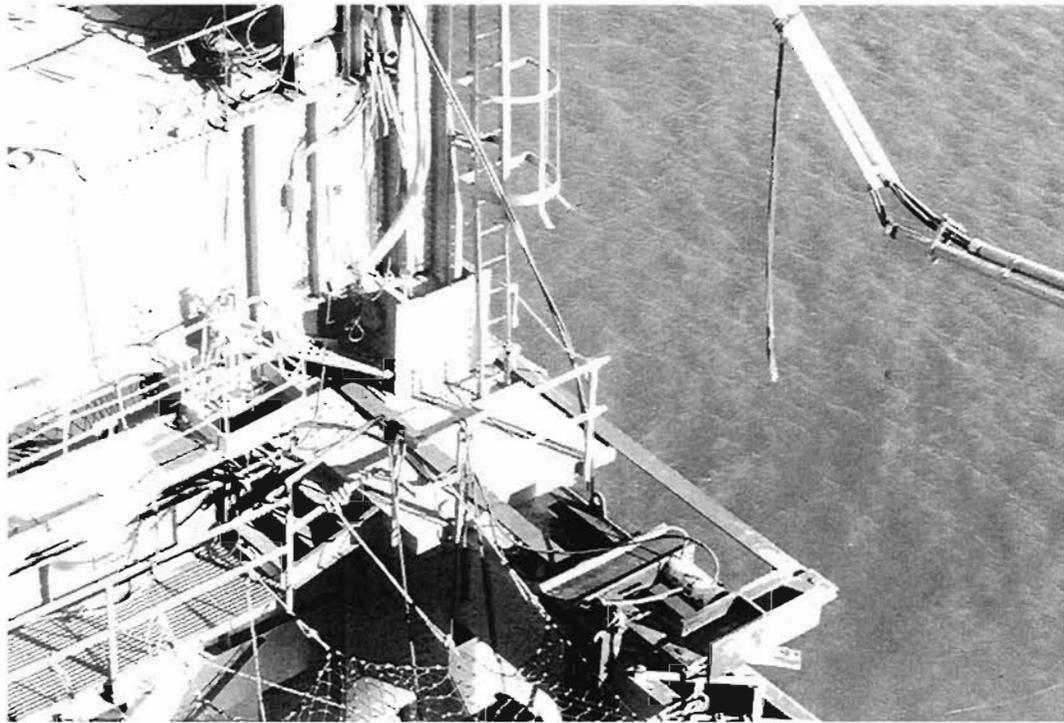


Plate 5.10 : Oakland Bay Bridge - Jacking approach spans back to original position



Plate 5.11 : Laurel Street Bridge - General view



Plate 5.12 : Laurel Street Bridge - Slumping of embankment

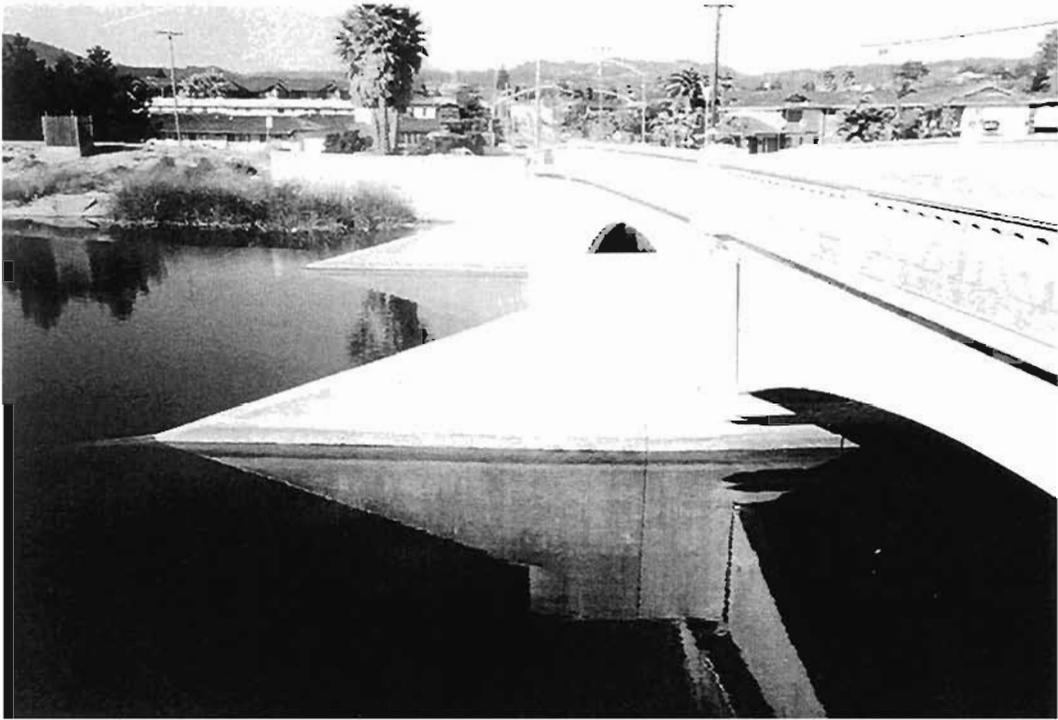


Plate 5.13 : Riverside Avenue Bridge - General view



Plate 5.14 : Riverside Avenue Bridge - Settlement of approach embankment

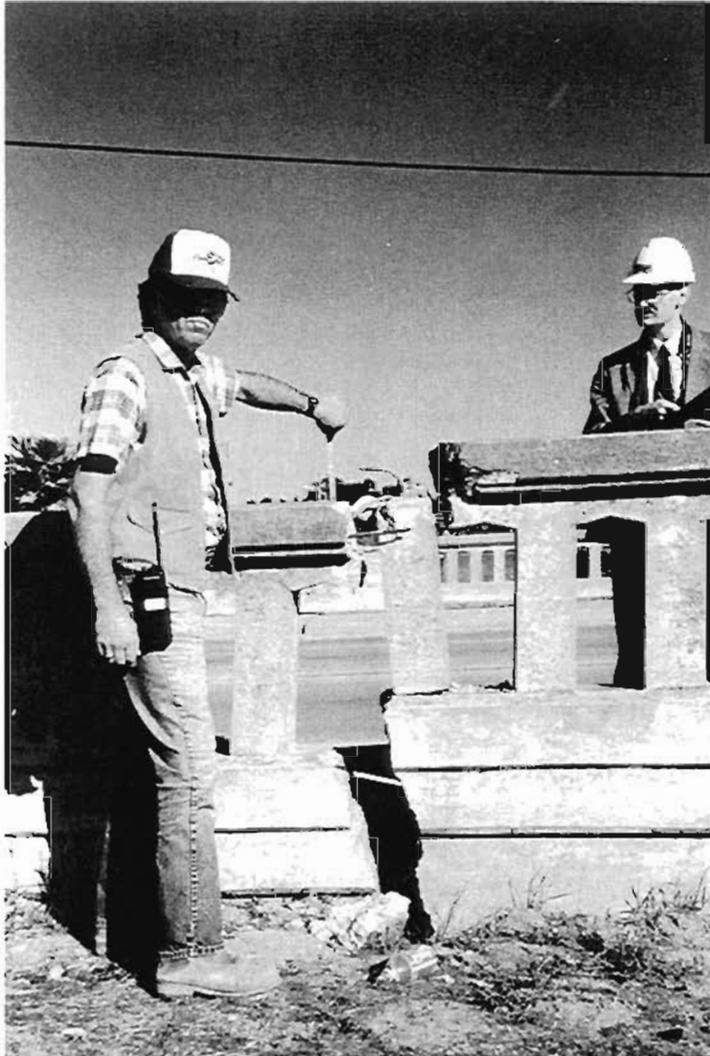


Plate 5.15 : Riverside Avenue Bridge - Damage to parapet



Plate 5.16 : Cut Bias Bridge - General view



Plate 5.17 : Cut Bias Bridge - North side column showing crushing and toe of adjacent fill



Plate 5.18 : Murray Street Bridge - General view



Plate 5.19 : Murray Street Bridge - West pier showing Brian Evans holding plumb line

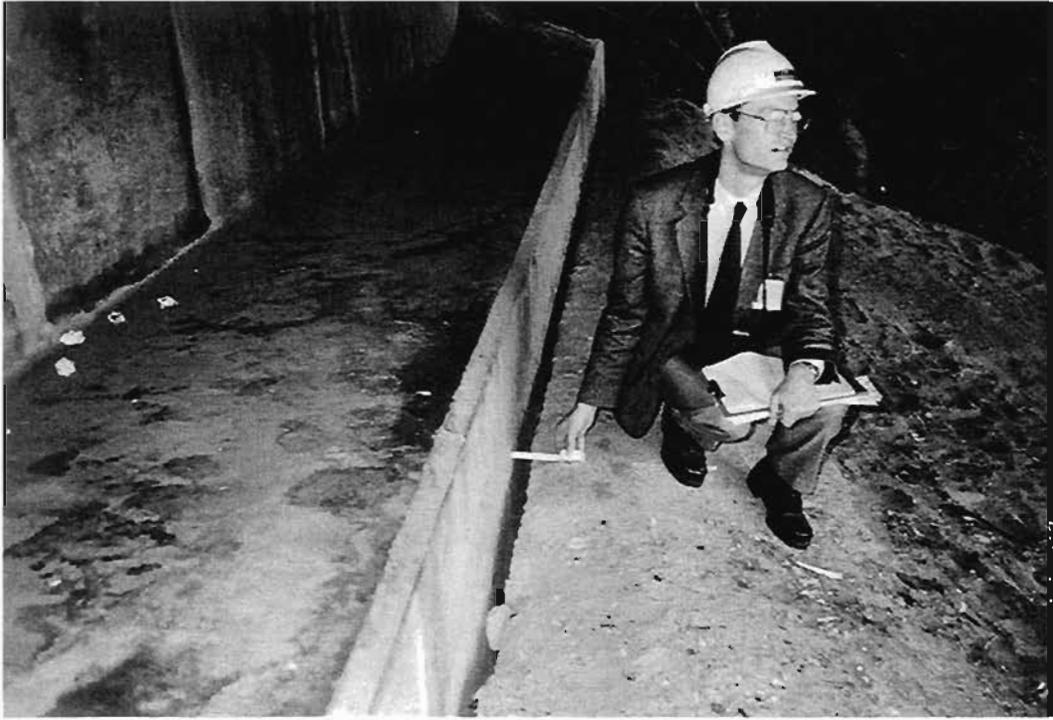


Plate 5.20 : Murray Street Bridge - Relative movement between abutment and pile



Plate 5.21 : Murray Street Bridge - Cracking of surface cement slurry

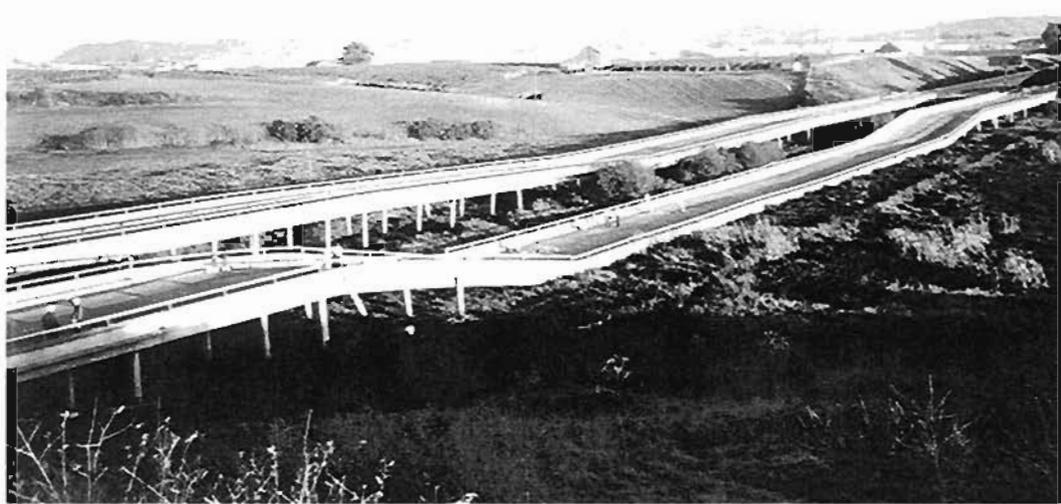


Plate 5.22 : Struve Slough Bridge - General view of collapsed section



Plate 5.23 : Struve Slough Bridge - Piles punched through deck slab



Plate 5.24 : Struve Slough Bridge - Crushed concrete from pile/deck joint above postholing around pile



Plate 5.25 : Struve Slough Bridge - Failed pile/deck joint, note inadequate spiral binding reinforcement



Plate 5.26 : Golden Gate Bridge - General View from South

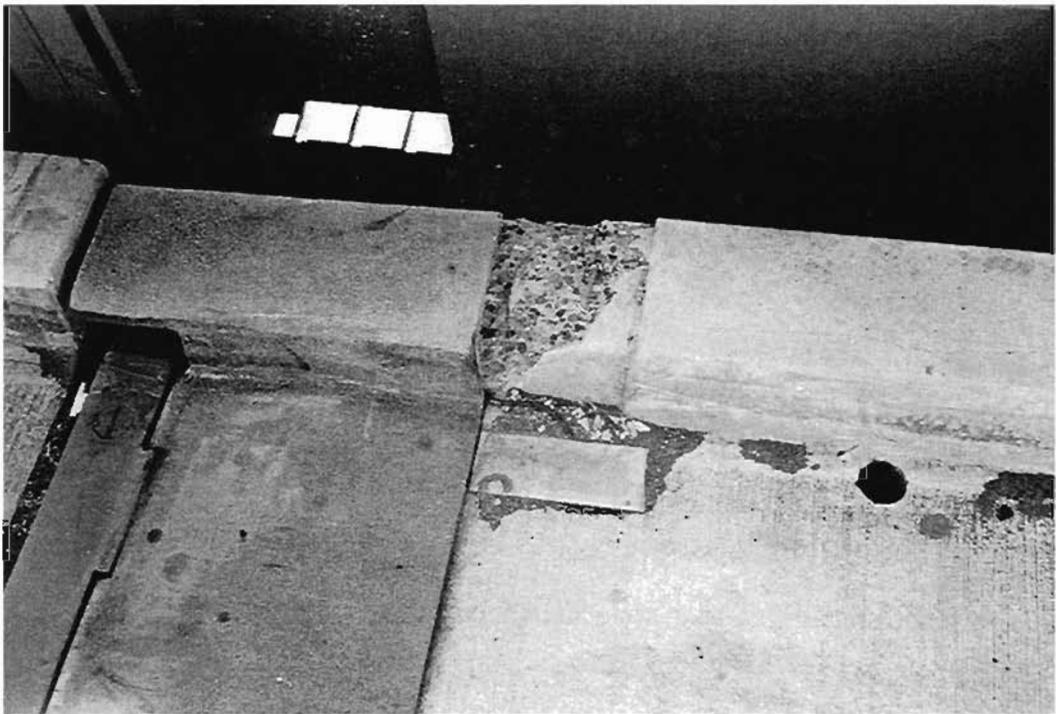


Plate 5.27 : Golden Gate Bridge - Minor impact damage adjacent to joint

PERFORMANCE OF INDUSTRIAL FACILITIES

P Ford,
AEA Technology

J Donald,
BEQE Ltd

I Morris and P Merriman,
BNF PLC

6.1 INTRODUCTION

Northern California contains many different kinds of industrial facilities, although the epicentral region around Loma Prieta, near Santa Cruz, is not itself heavily industrialised. Therefore it is not surprising in a State that has carried out extensive design of such facilities, that plant and structural failures were sporadic and generally of a minor nature; although the consequences of such failures sometimes resulted in significant plant down times or loss of product.

Four engineers from the EEFIT team surveyed several industrial sites where damage was known to have occurred, from San Francisco to the north, as far south as Gonzales, 50 miles south of the epicentre. The following industrial facilities were visited.

Antioch	Du Pont Chemical Plant
San Jose	State University, 100MW Cogen plant
Moss Landing	PGE Electricity generation site, Brickworks and Magnesia Plant
Watsonville	Soft drinks factory
Hollister	Light Industrial facilities
Gonzales	Vintners International Co.Inc., Winery
Oakland	Telephone exchange, Harbour, Nabisco Factory

The location of these sites is indicated on the map in Figure 6.1. Best estimate peak ground accelerations are also indicated and are taken from information given in Section 2.

6.2 STRUCTURAL ASPECTS

Important structural features were identified at three of the sites visited:

Du Pont	(Antioch)
PG&E	(Moss Landing)
Light Industrial Buildings	(Hollister)

The failure or damage to major structural components was noted for all sites. In terms of seismic effects and consequences, the three sites listed showed particularly interesting examples of both structures designed against earthquakes and non-seismically designed structures in the aftermath of an earthquake.

6.2.1 Du Pont – Antioch

At this chemical works, there was a wide range of buildings constructed from the early 60's to the present date. It was observed that the older structures had light bracing members very similar to that used for wind bracing on conventional buildings in the United Kingdom. In the progression to more recently constructed modern structures, there was a marked tendency to use heavy universal columns to meet current design standards (Plate 6.1). In addition, at foundation level bolts were extended to provide ductile connections. There was no reported structural damage at this site but sloshing of liquor in tanks activated alarms monitoring the liquor depth.

6.2.2 PG&E – Moss Landing

This was a large (2GW) gas/oil fired electricity generating site situated about 20 miles from the epicentre. There were several engineered structures on site (Plate 6.2). Structural damage was only reported for the large, open, steel framed structure housing the two 750 MW generation plants. This structure is approximately 120 m by 90 m by 45 m high and is constructed from very large section steel columns and beams, with large strength of member bolted connections at all major intersections. The structure contained some concrete flooring, but this was not continuous at any height, and whilst a basic structure of 7 storeys was observed, there were many discontinuities in the steelwork. From observation of the internal distribution of plant, it was clear that the loading at each level varied greatly.

Several instances of structural damage were reported, but they were mostly of a minor nature and none prevented the continued operation of the plant. There appears to be no damage to the major structural steelwork, although one element of minor bracing steelwork between the two large chimneys was observed to have buckled (Plate 6.3). A welded shear block restraining one of the large heat exchangers had failed (Plate 6.4) and some failure of heat exchanger tubes was reported. Seismic restraints to the heat exchangers and main stream lines had yielded in some cases, indicating that significant movement had occurred during the earthquake. At a point in one of the main steam lines lateral movement of the pipe by approximately half a diameter in either direction had occurred, damaging the side restraint steelwork and the thermal insulation (replacement steelwork is shown on Plate 6.5). The pipework remained undamaged.

The two large chimneys 155 m high and 20 m diameter at their base were constructed in reinforced concrete with an internal steel liner (Plate 6.6). No structural failures or concrete cracks were reported for these, even though eye witnesses reported large fundamental mode cantilever swaying of the structures.

Three connected steel chimney stacks belonging to the five 100 MW generator sets showed no sign of damage at the site (Plate 6.7). These stacks are secured to their concrete bases by long bolts and are braced to each other at 3/4 height and separately back to the main structure at 2/3 height.

6.2.3 Light Industrial Buildings – Hollister

Hollister is about 25 miles from the epicentre and was therefore subject to quite severe shaking – the peak free field ground acceleration was measured at 0.38g horizontally and 0.20g vertically. At its northern edge are sited several single storey warehousing units of various designs. Three examples that were found to have extensive structural damage were investigated. There were many other examples that appeared undamaged. In all cases where significant damage

occurred, the failure mode had been the same, that is unstable inventory falling against both structural members and cladding, resulting in partial or total collapse of that section of the building.

The first example is of a reinforced precast concrete tilt-up structure with Glulam roof (Plate 6.8). When the inventory fell against the side of the building, the structure may well have survived the partial collapse that subsequently occurred if the Glulam roof had been securely tied to the precast tilt-up beams panels. In the event, when this connection failed the affected precast panels were supported only along their base edge and along their vertical edges to adjacent panels. Out of plane forces produced by the weight of falling inventory induced bending moments along each of the three supported edges. Being relatively weak against such loads, several edge connections between adjacent panels were severed and then each of the affected panels rotated about its base edge, collapsing completely. The end panel shown clearly in Plate 6.8 has been well supported against out of plane forces by the perpendicular side wall of the building. In this case the panel has hinged about one vertical edge by about 60°, and illustrating the robustness of concrete structures.

The remaining two structures are both constructed from steel portal frames, enclosed in light sheet metal cladding. Two forms of failure were noted, the first being shown clearly in Plate 6.9. Here, inventory has fallen against the cladding and destroyed it, but the portal frames themselves are still stable. In Plate 6.10 an alternative type of failure is seen, in which the weight of inventory has caused a portal leg to twist and buckle. Lateral displacement near the base of the leg has badly twisted the roof beam to which it is attached but the connection has remained intact.

The non structural items that caused the damage were mainly tins of tomatoes stacked on wooden pallets. The pallets were stacked at most four high. The principal failure mode is depicted in Plate 6.11 which shows the group of tins in the bottom pallet of a partly toppled stack. Crushing of the bottom row of tins on one side of the stack as well as shearing between layers of tins led to significant stack drifts. Similar stacks located right up to the building walls, moving in unison, provided sufficient load on external walls to cause their failure. Stacks of 80 gallon drums of tomato products all with four drums to a pallet with pallets stacked four high were also present. Buckling of the bottom drums in a stack led to toppling of the three upper layers of pallets in some stacks. Breakage of the wooden pallets also caused toppling of the stack. In general few of the stacks of drums collapsed and those that did fall were at the free edges of a stacked area. Most remained in position undamaged. An example of a stack of drums which nearly fell, together with the typical undamaged stack is shown in Plate 6.12.

It is concluded that the warehouse building design took no account of the falling inventory. The inventory was stacked in free standing columns which for the smaller tins were insufficient to resist the large horizontal shaking that occurred. For the large drums it is possible that a stack of items neither fixed down nor rigidly fixed together or laterally restrained may withstand an earthquake without any damage.

6.3 TANKS AND VESSELS

Tanks and vessels were observed at a number of sites. Failures were observed at Moss Landing and Gonzales of cylindrical flat bottomed tanks whilst no failures of other configurations of vessels were seen.

6.3.1 Flat Bottomed Tanks

The gas/oil fired power station at Moss Landing has a variety of large diameter flat bottomed tanks for oil and water storage. The majority of the tanks are for oil storage and are typically about 50 m in diameter and 12 m high. None of these tanks suffered failures but a smaller water storage tank about 15 m diameter and 10 m high ruptured, rapidly releasing its contents (Plate 6.13). This tank was poorly founded on crushed rock and had no engineered holding down detail. Rupture occurred at the junction of the bottom plate to the tank shell and at the tank roof to shell connection. Also visible was a buckle in the tank shell at high level, approximately diametrically opposite the roof/shell connection failure.

It is postulated that the failure was initiated by uplift of the tank at the bottom rupture location possibly combined with settlement of the aggregate on the opposite side from the rupture. The tank base then receiving insufficient support directly from the foundation, overloaded the shell to base plate connection. The roof/shell connection failure and buckling of the shell may have been caused by liquid sloshing effects or may be the result of the rapid emptying of the tank creating a partial vacuum within the ullage space. The observed evidence is consistent with either possible cause.

At Gonzales ten of approximately one hundred similar flat bottomed tanks failed in a mode generic to those often observed. The tanks are constructed of stainless steel and are approximately 6 m in diameter and 10 m high. At the time of the earthquake each tank is believed to have held 75 m³ of liquor (equivalent to 100,000 bottles of wine) but it is not known whether all this was lost. The tanks stand on a concrete plinth about 0.8 m high and are fixed to it with small straps welded to the shell and embedded in the concrete. The tanks are also fixed to the base through a bottom exit pipe nozzle which is connected to a trough embedded in the concrete (Figure 6.2). The leakage failure occurred at the weld of the trough section to the tank shell wall. The connection, on rocking movement of the tank, and the uplift of the tank wall, was unable to deform sufficiently to accommodate the relative movement between tank wall and the fixed trough.

Large concrete flat bottomed tanks are used at a magnesia plant near Moss Landing; no structural damage was observed to the tanks. Wooden raceways at the perimeter of the liquor free surface in three 43 m diameter tanks with about 6 m depth of liquor were damaged by sloshing of the tank contents. One of the tanks also suffered jamming of a rotating paddle (Plate 6.14).

6.3.2 Vessels

None of the sites visited reported any failures of conventional vessels. A wide variety of sizes of both vertical and horizontal vessels supported most often by concrete saddles, steel saddles, braced and unbraced legs, skirts and brackets attached to the vessel shell were observed. The vessels were located at grade or within structures up to 75 m high. The structures were most often steel braced frames solely supporting process plant. Buildings of other construction types, such as moment frames or shear walls, were also used to house plant. Steel braced frames were typically used to support vessels and piping assemblies within these buildings. The function of the vessels observed was as diverse as their geometry. The largest were bulk powder hoppers of 1000 te capacity and the smallest process breakpots and air receivers for fractional horsepower pumps.

The absence of observed failures may in part be due to the specific sample of vessels seen. In particular the use of seismically poor unbraced legs was often compensated by good engineering attention to the necessary size of leg cross section and attachment to the vessel. Also proper care had been taken in

fabrication to prevent failure for the vessels as seen. In one case a hold-down detail was observed which performed acceptably but was a detail which is believed to have failed elsewhere when it was poorly constructed.

6.4 MECHANICAL EQUIPMENT

Heavy industrial mechanical equipment was seen at the power station, magnesia plant and brickworks at Moss Landing, a bottling plant at Watsonville and in Oakland near the Interstate 880 section which collapsed at Oakland Harbour. In general the equipment was seismically robust but a few items were seen that required repair.

The most impressive successes were the 50 m long 5 m diameter inclined rotary kilns at Moss Landing. The kilns are fired at 1815°C and must be continuously rotated to avoid failure. Diesel driven standby electrical power supplies were quickly brought into operation to preserve the kilns.

One of a pair of 2.5 te capacity dewatering batch presses (Plate 6.15) located about 25 m above grade in a braced frame building at Moss Landing became inoperable during the earthquake when the rails supporting moveable carriages deformed sufficiently to allow some carriages to fall. The carriages became wedged between the rails but were readily recovered and put back in service. It is notable that no hydraulic oil leakage occurred from the large high pressure operating cylinder nor malfunction of the complex mechanical linkages or control systems for this machine.

At Watsonville a bottling plant experienced 75 mm of lateral movement of an unrestrained bottling machine which stood at ground level. The machine was about 10 m long, 4.5 m wide and 3 m high. The mechanical systems in the machine and in adjacent bottle handling conveyors were put straight back into use when power was restored by the grid.

Overhead travelling cranes were seen in Oakland and Moss Landing. Apart from one crane rail remote from a crane when the earthquake occurred, no failures of cranes were reported. The large gantry cranes at Oakland harbour on the quays affected by liquefaction failures were not able to operate due to loss of power supplies and possible lack of support to the rails founded on fill.

Fan assisted cooling towers of wooden construction were seen at Moss Landing and Oakland city centre. One of these on top of a 76 m high building collapsed in part (Plate 6.16) whilst a similar item oriented at right angles to the collapsed tower on the roof of the adjacent building and at the same level only experienced minor damage to a few asbestos cement panels.

6.5 STANDBY AND CO-GENERATION (COGEN) GAS TURBINE POWER SUPPLY UNITS

Special attention was paid to obtaining information on the seismic performance of standby and Cogen electrical power supplies which had a gas turbine in the 2.5 MW to 4 MW range as prime mover. Gas turbines of this size are used in the UK to provide guaranteed standby electrical power at industrial and commercial facilities. The observation of the performance of the unit in the Loma Prieta earthquake gives useful additional data in demonstrating that such standby power would be available after a large earthquake in the UK. In all, information was supplied by the manufacturer for 16 units in the Bay Area of which 7 were visited. No unit was put permanently out of action either of those on cold standby or those continuously operating on load at Cogen units. The machines were located at ground level through to the top of 76 m high tower block buildings.

One Cogen unit was tripped out during the event due to sloshing of water in an exhaust

gas waste-heat recovery plant steam drum. Another was tripped out 4 minutes after the end of the earthquake shaking due to excessive load on the grid. (A turbine of 4 MW maximum rating is unable to stay on frequency and voltage when main supplies to the grid such as Moss Landing power station disconnect in an uncontrolled way). These units were supplying power again within 3½ hours of the main event occurring.

Four units at the Bank of America data processing centre in San Francisco were run continuously on load for three days to guarantee no interruption of supply which would have occurred if the external power utility grid had been relied upon. Two units used by Pacific Bell in central Oakland were employed to ensure uninterrupted supply to the telephone exchange building (see Section 4.2.4) despite damage occurring to the building structure itself.

6.6 ELECTRICITY GENERATION AND DISTRIBUTION

6.6.1 Status of Distribution network

The electricity supplies in the Bay Area were widely disrupted. This included the financial district of San Francisco and most of the large industrial facilities in this area. The primary cause was the loss of supplies through the main substation due to damage to ceramic insulators. The smaller power generation plants such as the Co-generation plant at San Jose State University suffered no widespread damage. Because of the smaller output from these plants (of the order of 4.0 MW) the supply voltages are lower and hence ceramic damage did not affect distribution systems. The Pacific Gas and Electric (PG&E, the main electricity supply utility in the area) engineers managed to restore power to most areas within 24 hours by either routing supplies around damaged substations or by fly-rigging the damaged switchyards.

6.6.2 Ceramic Damage

The 500 kV and the 220 kV switchyards at the Moss Landing PG&E site were inspected. The 500 kV yard showed damage to tall bus-bars supported on ceramics, Plate 6.17. There was also significant damage to the air circuit breakers which control throughput of all three phases of the supply.

A total of five air circuit breakers were in use at the site, four were a Westinghouse design and one was an Hitachi unit. The Westinghouse units all failed due to their location on a steel frame which was friction-clipped to the concrete pad foundation. These friction clips, which are considered a poor detail in earthquake regions, slipped during the event permitting the frame to move until it overturned when it was clear of all restraints. The Hitachi unit was positively bolted to its pad foundation and suffered no damage.

There was also evidence of ceramic damage in the 200 kV switchyard at Moss Landing and the engineers at this site also informed the group of similar ceramic damage at the Metcalf 500 kV switchyard.

Experience-based data suggests that damage to ceramics can be anticipated at pga's of 0.10g with total damage at approximately 0.30g. The damage at Moss Landing supported this data and it would be anticipated that for higher ground accelerations the Hitachi units which survived this event would also fail, even with their more substantial anchorage.

6.6.3 Transformers

The large external high voltage transformers at Moss Landing were reported to

have performed satisfactorily. This was surprising as their large mass, small footprint and the lack of positive restraint due to their mounting on rails, made them vulnerable to damage (Plate 6.18).

Smaller voltage transformers were observed inside several buildings and no damage was noted by inspection or by reports from the owners.

6.6.4 Switchboards

Switchboards and panels are typically used for control and instrumentation systems. This form of equipment was inspected at several sites and noted to be functional after the event. There was evidence of large displacement (of the order of 150 mm) on some unrestrained units but they were reported to be functional nevertheless. There were no reports of spurious trips or loss of instrumentation due to relay chatter at any of the sites investigated or of the sites contacted by telephone.

6.6.5 Motors

There were no reports of failure of electrical motors due to the seismic event. This form of equipment is considered to be rugged with most problems occurring due to control or instrumentation faults. Motors surveyed included very small air conditioning units through to very large 100 Hp units used to drive the rotating kilns at the Moss Landing brickworks site.

6.6.6 Batteries and Backup Power Supplies

There were no reports of battery backup power supplies failing due to battery damage. Indeed an instrumented telephone building in Watsonville had batteries on its top floor which experienced 1.24 g horizontally. These batteries acted to power up a backup diesel generator system on the ground floor (0.39g horizontal, 0.66 g vertical) successfully. The only significant problem occurred at Moss Landing where the batteries had insufficient capacity to maintain the oil pumps which fed the low pressure turbine bearings. Hence, after approximately four hours of hot shutdown, the oil pressure reduced to the point where the turbine bearings overheated and were damaged. A number of possible causes for this failure may be surmised. The battery storage capacity may have degraded with age, the batteries may have been insufficiently charged or had not been specified correctly during the design. Which of these possibilities applies could not be confirmed.

6.6.7 Cable Trays

Cable trays were inspected at Moss Landing, Antioch and San Jose. The cable trays at Moss Landing were significantly more substantial than the others with short spans and large support steelwork. The system at San Jose was very light and used friction clip anchors on the bottom flange of the steelwork floor above. This form of support detail is not recommended in seismic design but suffered no damage during this event. It was noted however that the conduits on the trays at San Jose were thin walled box structures and appeared to contribute most of the system stiffness and hence probably acted to prevent the large sway displacements which would have failed the friction clip hanger anchors.

6.7 PIPING SYSTEMS

6.7.1 Large bore, high integrity piping

Large bore piping was inspected at Moss Landing, San Jose and Antioch. The

piping systems at San Jose and Antioch did not have any obvious or reported damage. The gas regulator lines and the main steam lines at Moss Landing showed signs of distress.

The gas regulator station at Moss Landing is shown on Plate 6.19. The piping is large bore (in excess of 300 mm) with valves and is supported on concrete pads on a landfill site. The large ground motions on this site caused relative displacement of the pipes and their supports. This in turn failed the support clamps without significantly damaging the pressure integrity of the gas pipes. Indeed, the damaged supports had been packed with timber to support the pipes after the event, but some of these packs had fallen out without any apparent sagging of the pipe.

The main steam lines at Moss Landing ran from approximately 50 m vertically down to approximately 10 m above ground level. The pipes were laterally supported at two positions over this run and the higher restraint showed permanent displacements in excess of 150 mm for minor axis bending of the restraining beams. Again, there were no reports of the pressure boundary of the piping being damaged, although with an immediate hot shutdown occurring, this could not be accurately checked until the plant is recommissioned.

The only other reported incident noted was on a small co-generation plant at Gilroy which indicated that a steam valve seal leaked after the event, but this leak was so trivial that the plant stayed on-line for four hours before being requested to shut down at which time the seal was repacked.

6.7.1 Small Bore Piping

There were numerous small bore pipes at all of the sites visited and there were no reports of damage. Indeed some of the stainless steel piping at the Gonzales winery was so thin walled that it was reported to have been significantly damaged due to water hammer from a valve closure in the past, and yet no damage was reported following the earthquake. Similarly, no nozzle damage was reported or noted during the team's inspection, other than the boiler tubing at Moss Landing PG&E.

6.8 SERVICES

The serviceability of lifelines was not specifically inspected other than for electricity supplies. However, it was noted that the water supplies and the sewerage lines which ran underground in the Marina and other soft site districts in San Francisco were quite badly damaged. The source of this damage was typically the large ground movements which could not be accommodated by concrete and cast iron pipes. Elsewhere water and sewerage distribution systems performed well although numerous small leaks and breaks were reported. Near the epicentre, people were advised of possible contamination to water supplies. In the counties of Santa Cruz and Monterey damage to lines caused raw sewage to be discharged to sea and many beaches were closed to the public. At Moss Landing the settlement of bridge embankments damaged pipelines.

The telephone system in San Francisco and other Bay Areas was reported to have survived the earthquake satisfactorily. It was understood that a reduced service was maintained but that most problems in communications were due to the system being inundated with calls, or because office switchboards which required electrical power supplies, failed due to the loss of power.

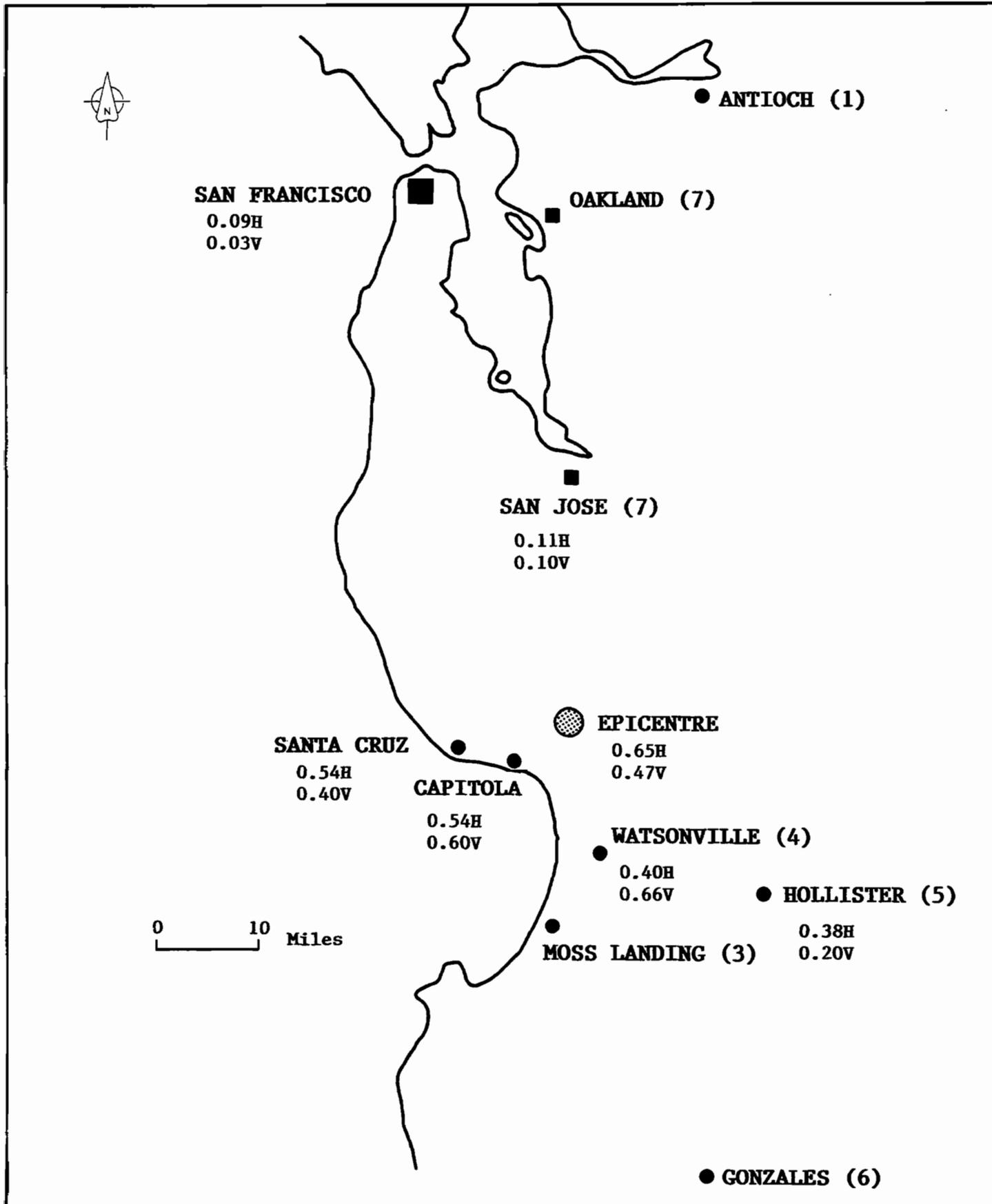


Figure 6.1 : Location plan and acceleration levels (g)

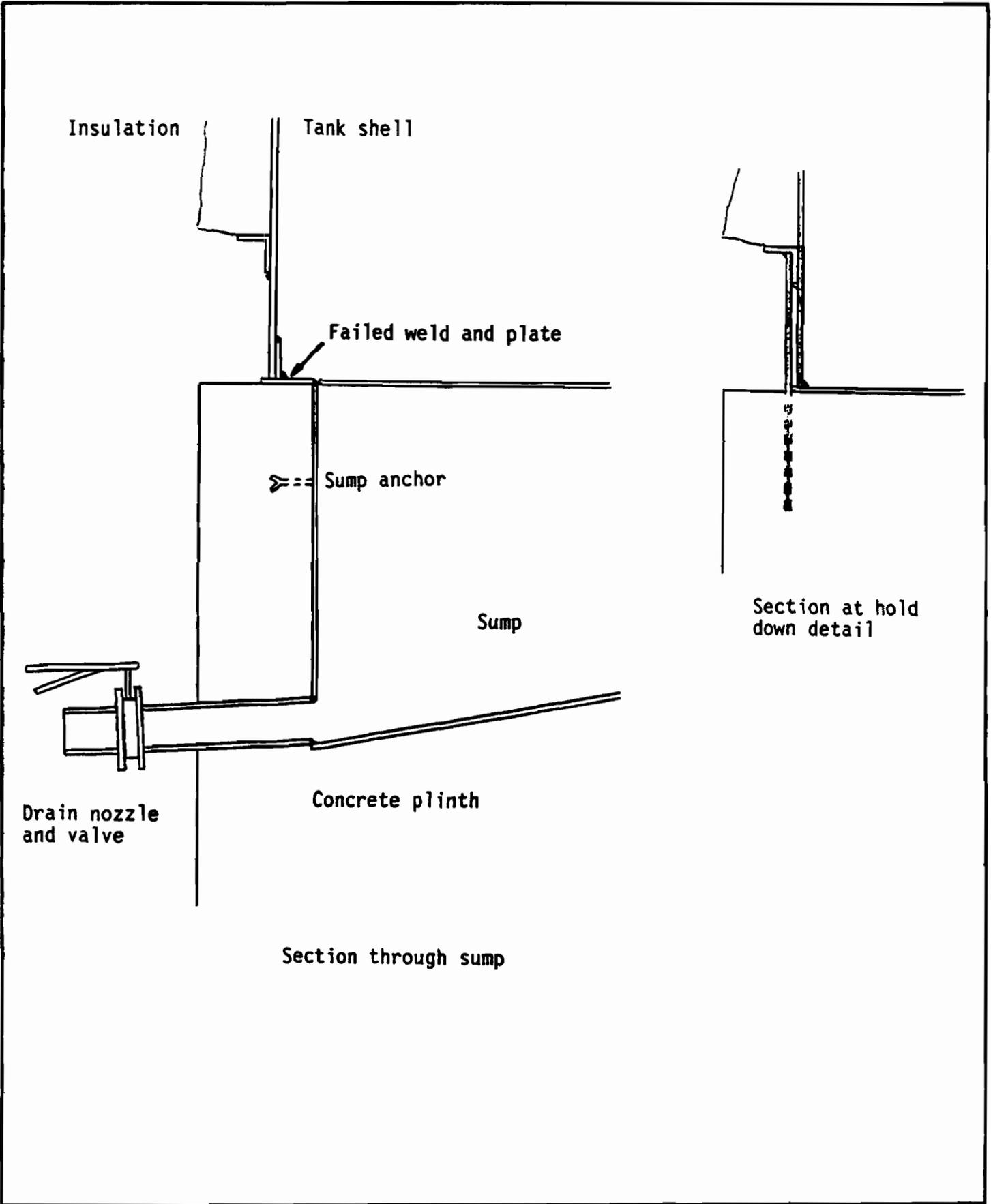


Figure 6.2 : International Vintners, Gonzales - Wine storage tank failure



Plate 6.1 : Heavy Bracing Antioch



Plate 6.2 : PG and E Overall Plan

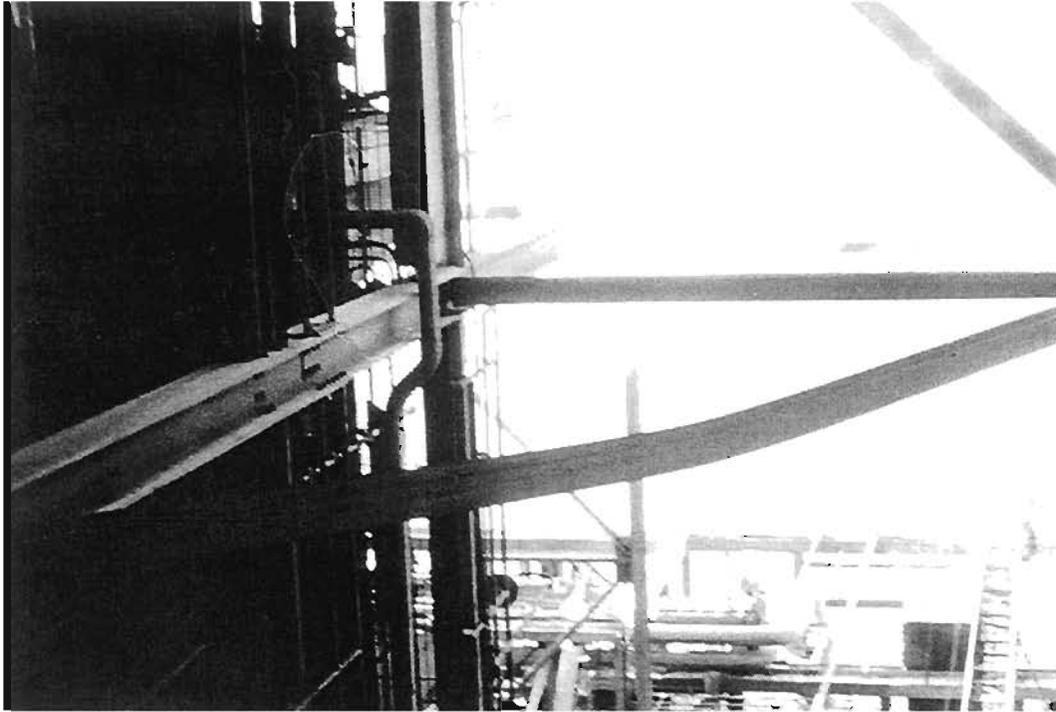


Plate 6.3 : PG and E Buckled Brace

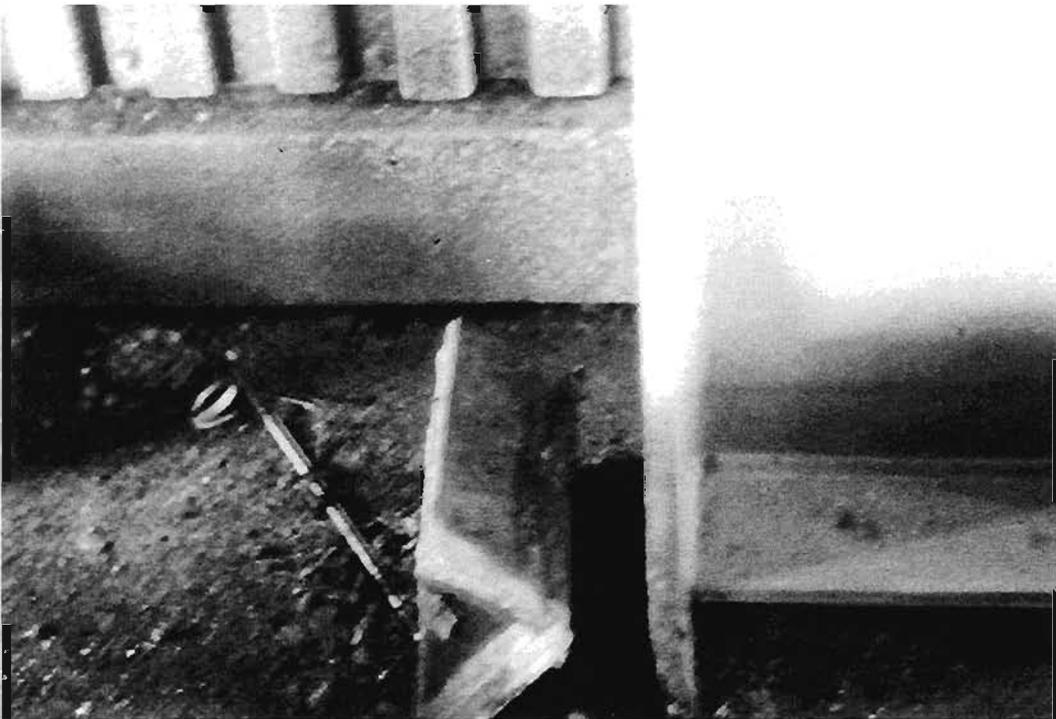


Plate 6.4 : PG and E Shear Block Failure



Plate 6.5 : PG and E Replacement Steelwork Guide

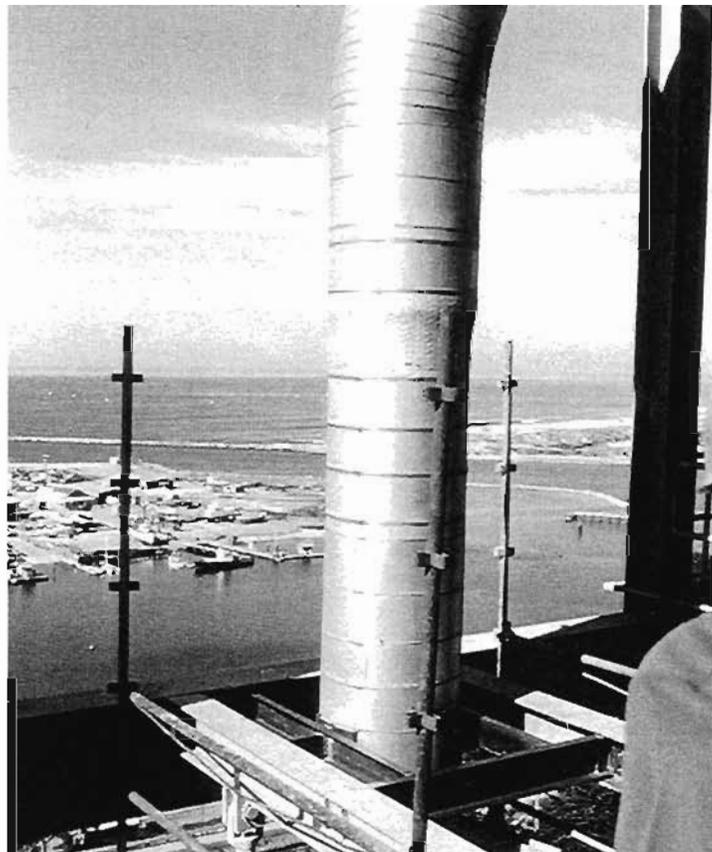


Plate 6.6 : PG and E Chimneys

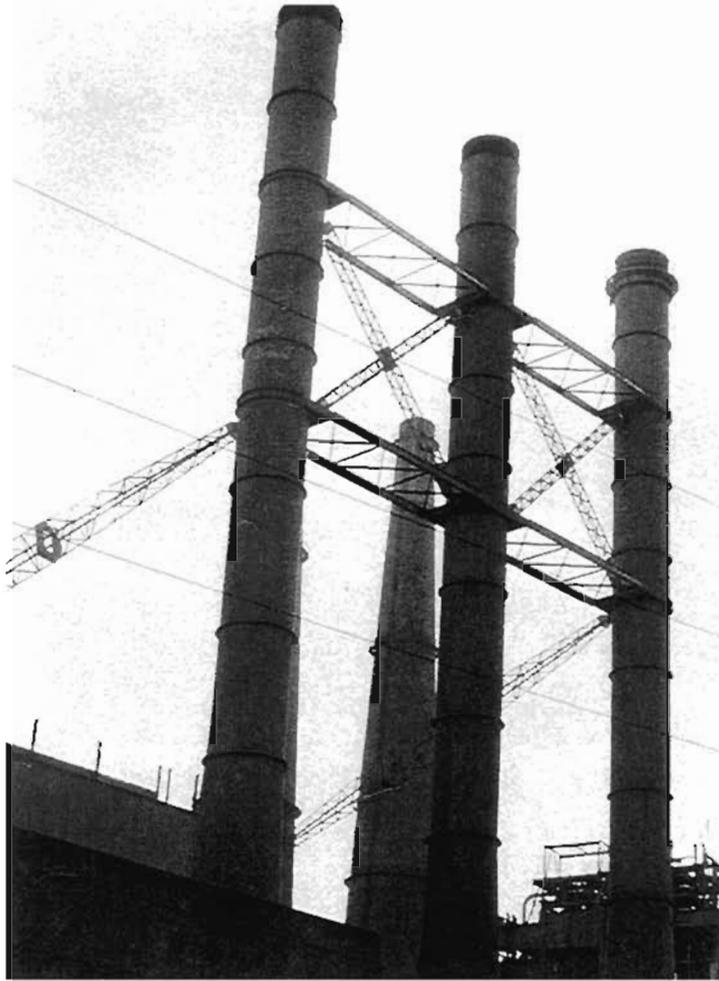


Plate 6.7 : PG and E Braced Chimneys

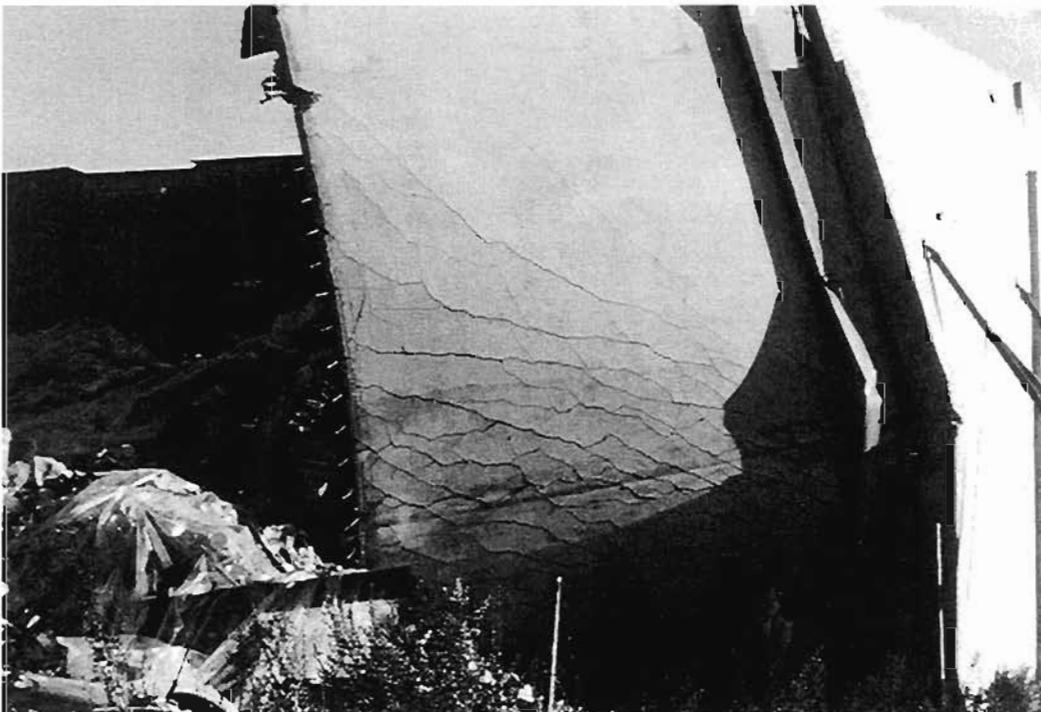


Plate 6.8 : Hollister - Tilt-up Structure

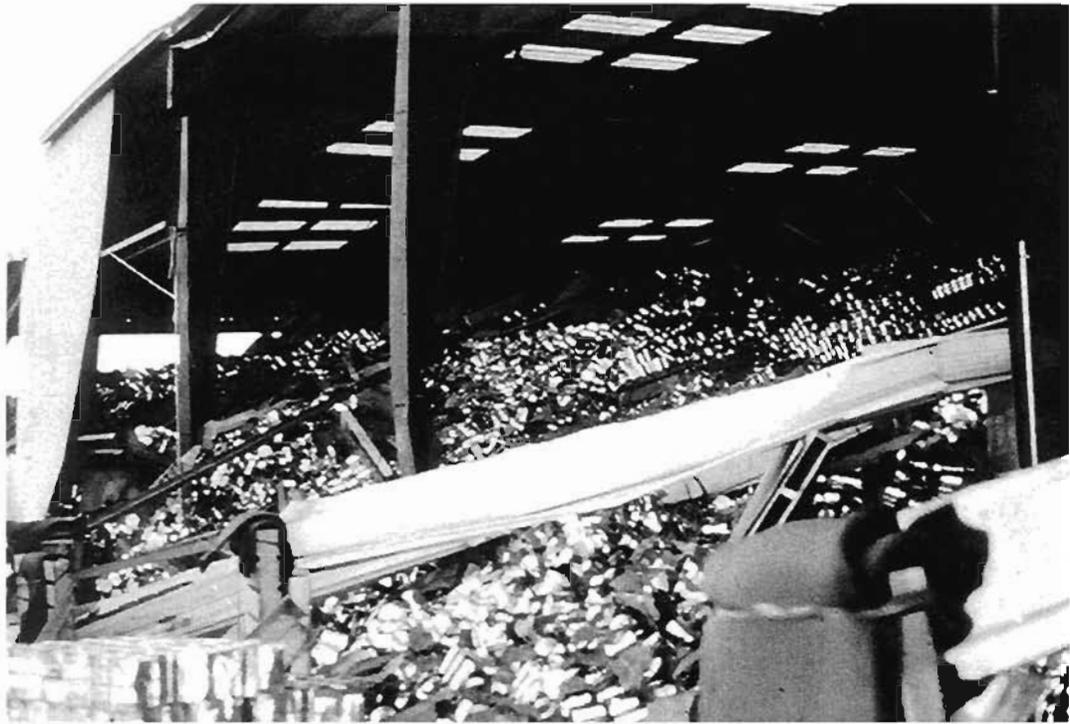


Plate 6.9 : Hollister - Metalclad Warehouse



Plate 6.10 : Hollister - Twisted Column on Metalclad Warehouse



Plate 6.11 : Hollister - Small Tins (Shear/Crush)

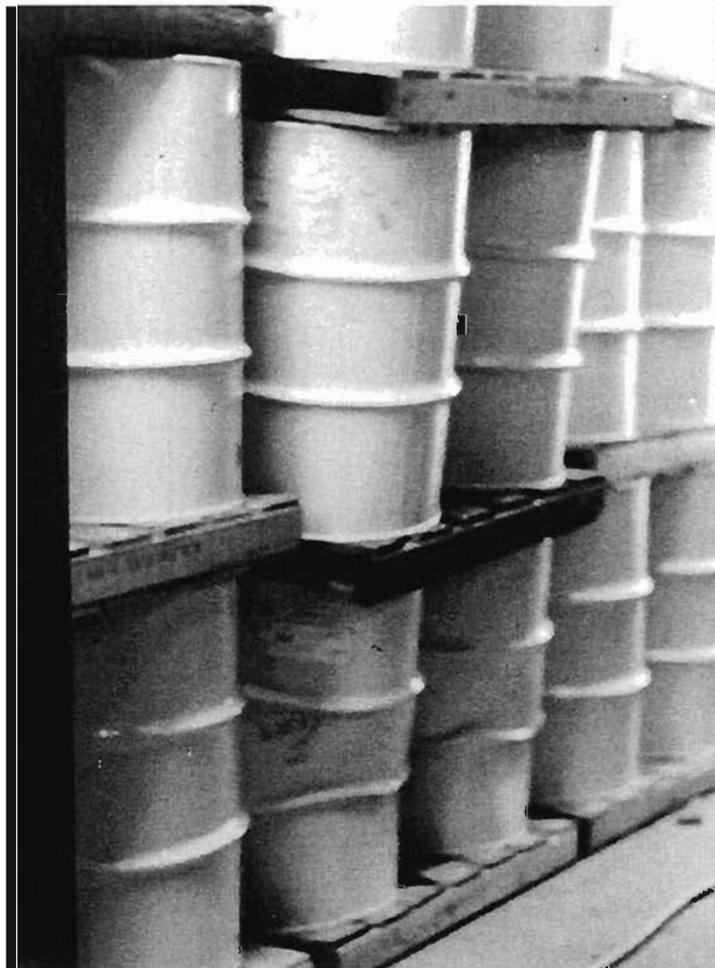


Plate 6.12 : Hollister - Drums (Buckle/Nearly Topple)

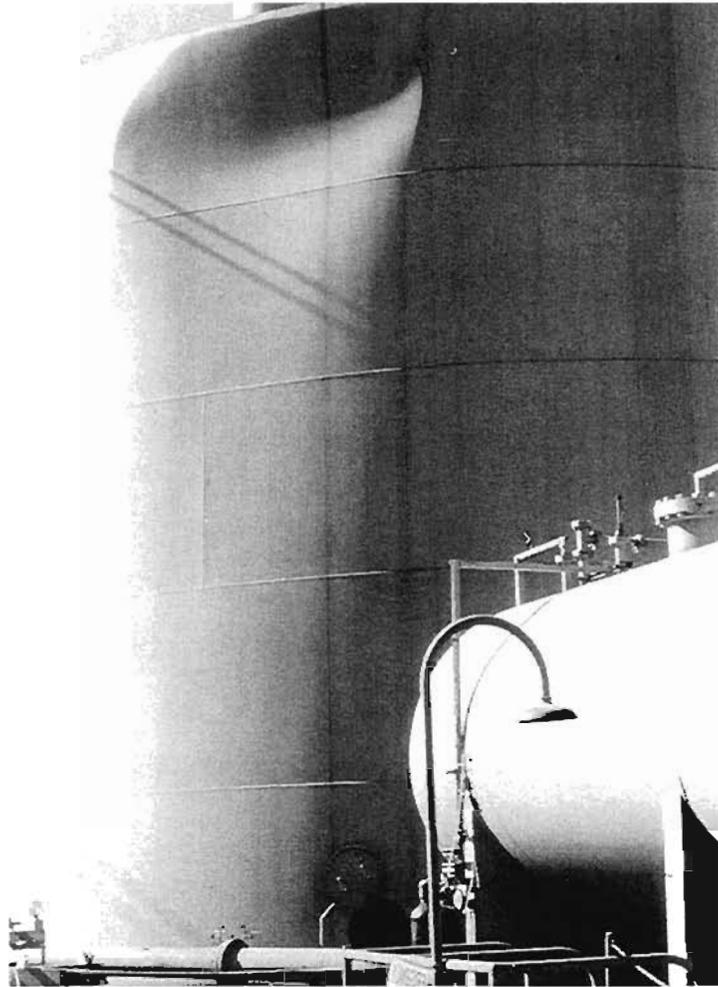


Plate 6.13 : PG and E - Tank Failure

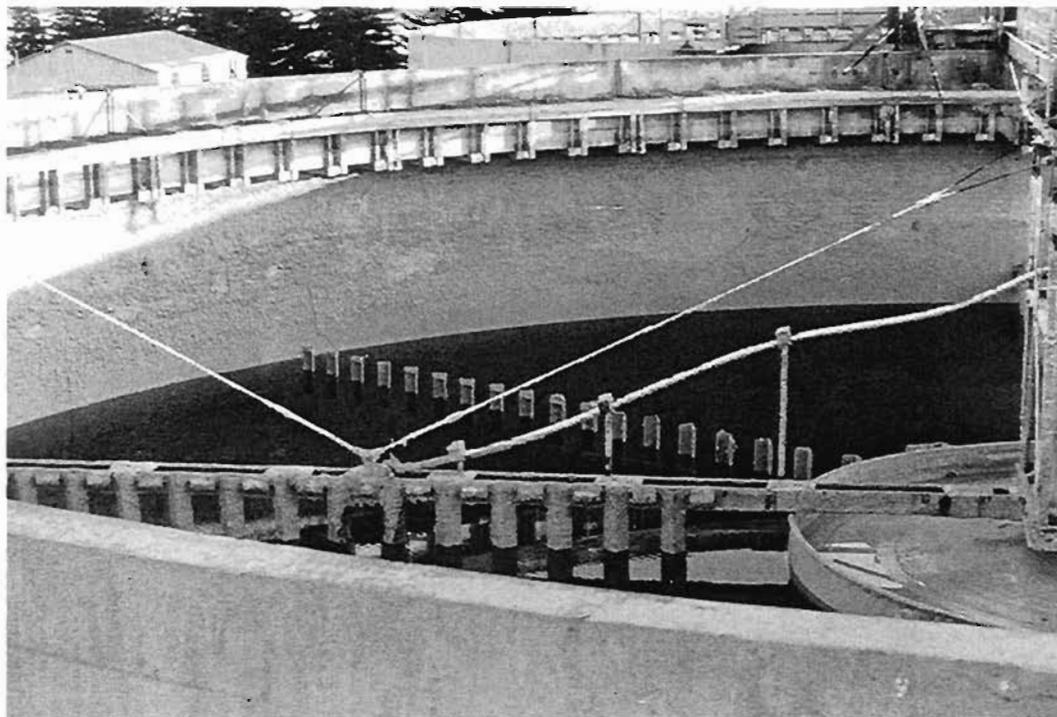


Plate 6.14 : Magnesium Plant, Moss Landing, Jammed Paddle

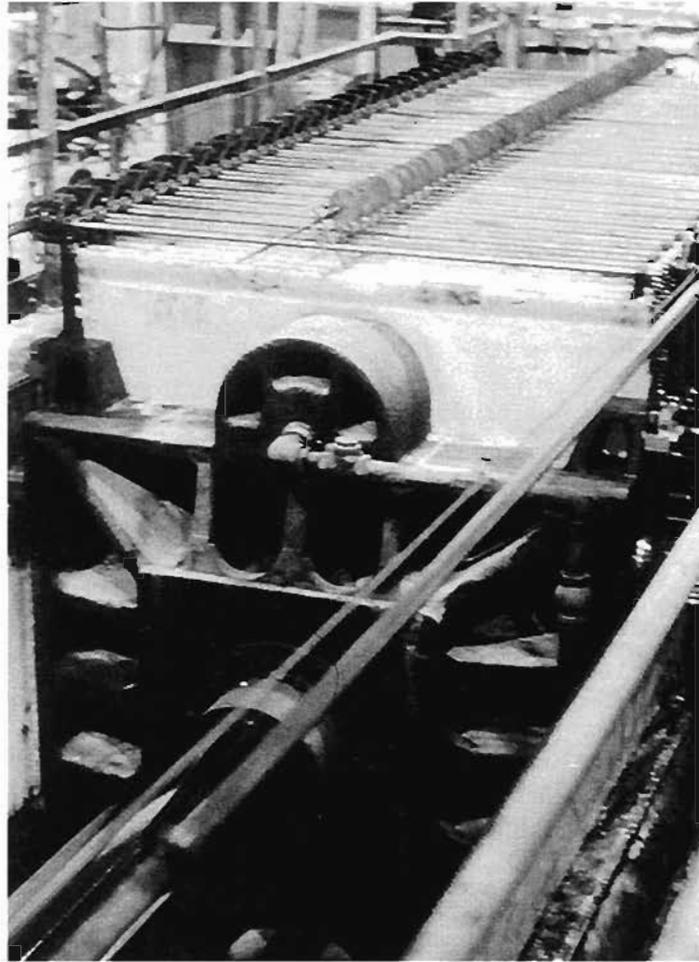


Plate 6.15 : Dewater Press

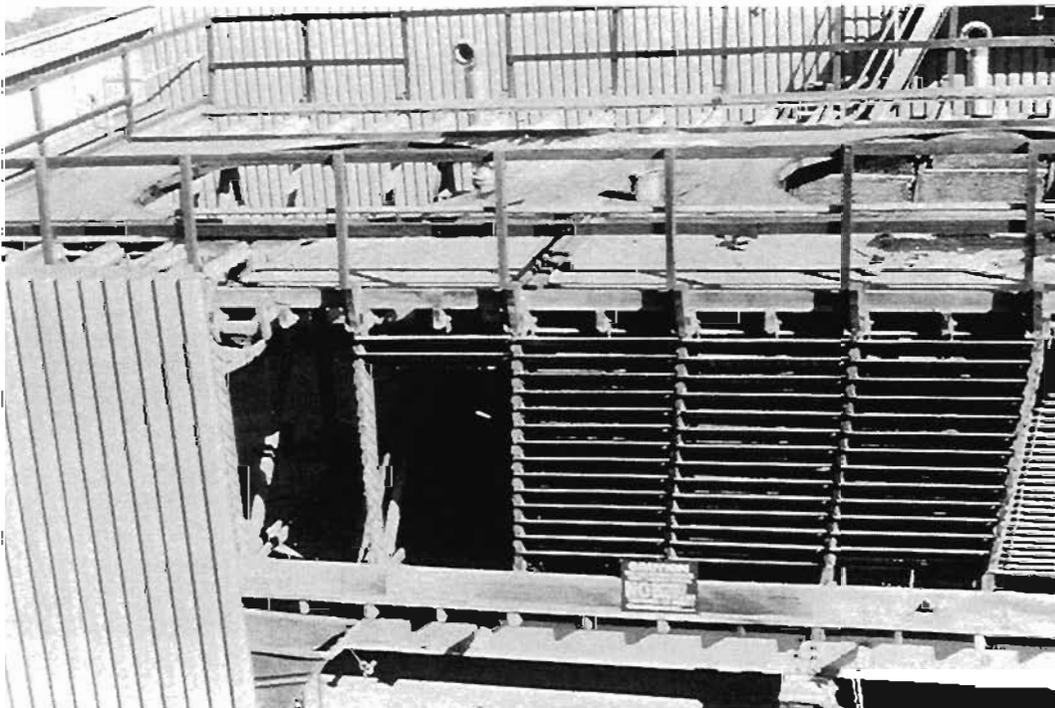


Plate 6.16 : Oakland - Fan Cooling Tower

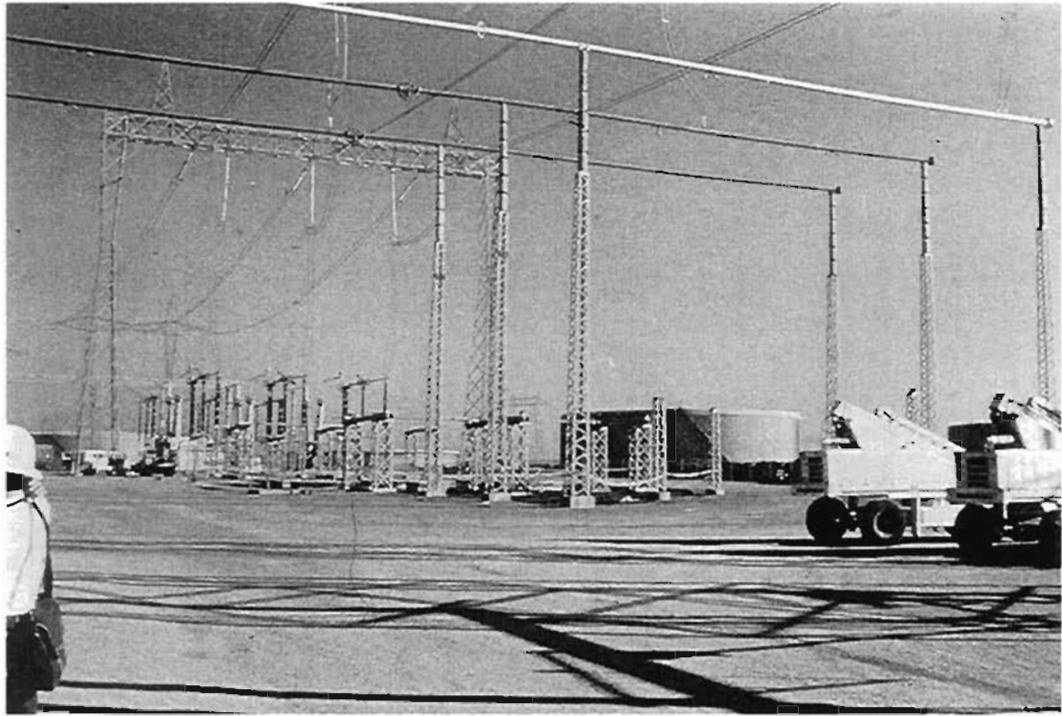


Plate 6.17 : PG and E Tall Bus-bars



Plate 6.18 : PG and E Transformers

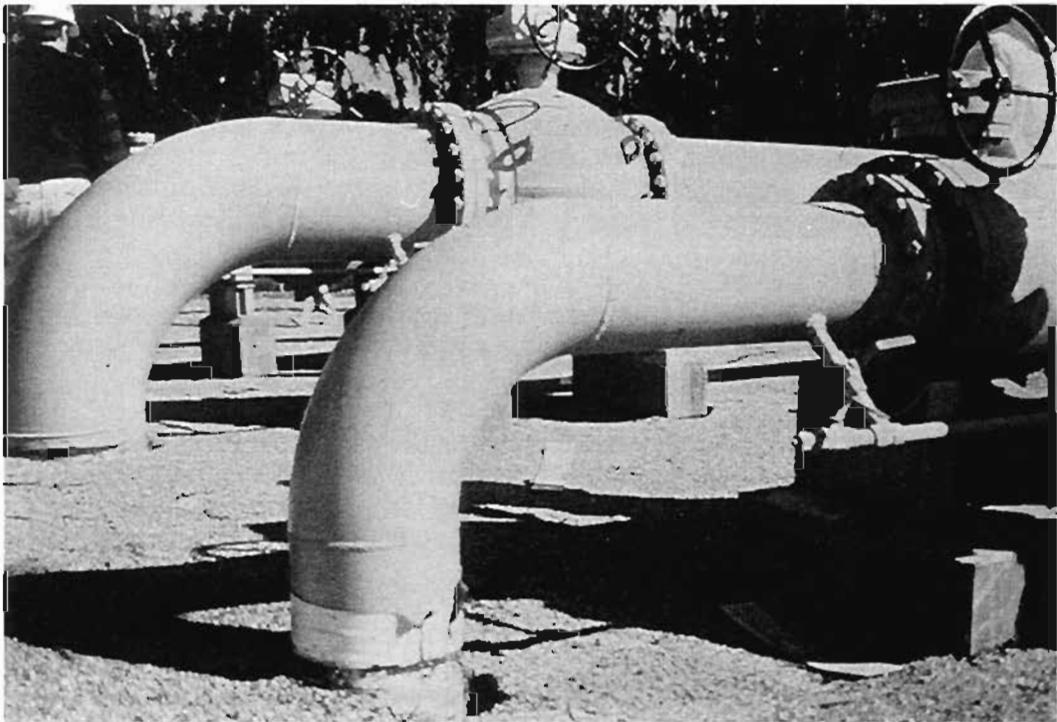


Plate 6.19 : PG and E Gas Regulator Station

GEOTECHNICAL ASPECTS

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7.1 SUMMARY

The Magnitude 7.1 earthquake which struck Northern California on October 17 1989 caused extensive damage in the epicentral region with liquefaction, landslides and lateral spreading causing failures of dykes and bridge abutments and blockages of many roads particularly in the Santa Cruz mountains. A number of dams in this region were damaged, the most severe of which was the Austrian Dam impounding Lake Elsin Reservoir.

Ground settlement and lateral spreading caused the failure of flood control dykes in Santa Cruz and the dramatic collapse of a causeway at Moss Landing with extensive longitudinal cracking along the shoulders.

Some distance to the north there was also substantial damage in the Bay Area, principally on areas of low-lying filled ground or on soft deposits. Such localisation of damage at a relatively large epicentral distance reinforces the importance of the local ground conditions in determining the modification of incoming waves and the consequent soil-structure system response.

In San Francisco itself damage to structures and lifeline facilities was closely linked to areas of filled ground, particularly in the Marina district but also at the Embarcadero at the end of Market Street and along the former creek beds in the area to the south of Market. The vast areas of hydraulic fill that make up the Port of Oakland and including Oakland Airport and the Alameda Naval Air Station showed substantial settlement, causing severe cracking of the Naval runways and part of the Oakland Airport runway where settlement accompanied lateral spreading. Many of the wharf structures at the Port of Oakland were severely damaged where the lateral spreading underneath the concrete deck of the wharf failed the supporting piles. Crane rails lying on the hydraulic fill settled with the ground up to 400 mm, putting several of the dock cranes out of action. The differential settlement between the wharf and the filled ground burst many of the water supply services to the wharf.

In the Marina district there was clear evidence of permanent ground movement. The buildings are predominantly residential timber frame structures on shallow strip footings and fit a clear pattern of three storey buildings in the middle of a block with a four storey structure at each corner. With few exceptions, the three storey buildings were undamaged whereas the four storey structures were clearly more vulnerable with the worst damage occurring to buildings which had a soft first storey.

In the filled areas around San Francisco Bay ground softening accompanied by pore pressure rise similarly amplified low frequency motions prior to the onset of liquefaction. The Nimitz freeway which collapsed in Oakland was a double deck structure supported on piles in very soft clay and loose sands and silts, with stiff clay layers at depth. It appears that the freeway collapse was a transverse failure caused by very large lateral accelerations at the top level of the structure. Although no evidence of foundation failure has been found, foundation compliance may have contributed to the amplification of motion.

The evidence of permanent ground movement and soft ground amplification has attracted international geotechnical attention both in relation to the damage in the epicentral region and to the large focal distance low frequency shaking in the Bay Area.

7.2 INTRODUCTION

Ground failures in Northern California caused by earthquakes have historically been concentrated in a narrow coastal strip, extending up to 70 miles inland, from Humbolt Bay in the North to the Sabinas River in Monterey County to the South. This zone lies almost entirely within the Coastal Ranges province which broadly comprises a series of mountain ranges aligned North-West South-East with valleys infilled with deep alluvial deposits. The Loma Prieta earthquake of 17 October 1989 had its epicentre in the Santa Cruz mountains towards the Southern end of this historically vulnerable zone. The EEFIT team investigated a number of specific ground failures that occurred as a result of the 17 October earthquake but did not attempt to carry out a rigorous survey of all such failures.

Geotechnical events could be broadly classified into two main categories; firstly the response of sloping ground including earth dams and landslides, and secondly the response of lowlying flat alluvial or reclaimed ground. In the mountainous epicentral region there were widespread landslides and rockfalls which blocked vital road links such as Highway 17, and ground fissures which resulted from these downslope movements. Ground motions showed high peak accelerations both horizontally and vertically. Failures of this type are discussed in Section 8 of this Report.

In this section, attention is focussed on the ground response in low-lying saturated soft soils in the Bay Area, where soft soil deposits are widespread (as they are also along the alluvial plains of the San Lorenzo and Pajaro rivers in the epicentral area and the Salinas river to the South).

7.3 THE GEOTECHNICAL BACKGROUND OF THE BAY AREA

Following the retreat of the glaciers around 15,000 years ago sea levels began to rise. At that time the coastline of what is now Northern California was as much as 30 miles West of the current position and sea level around 100 m lower, Reference 1. The Golden Gate formed a narrow gorge through which a great river drained from the Central Valley, with tributaries from the Santa Clara Valley. At the end of the ice age throughout the world sea levels rose rapidly passing through the Golden Gate and reaching as far South as the site of the present Dumbarton Bridge around 8,000 years ago. The young soft bay muds were then deposited over the Pleistocene alluvium which surrounds the Bay today. The developed areas around the Bay are largely sited on these lowlying areas between the mountains and the sea which comprise broadly parallel zones of upland soils, older alluvial deposits, younger softer alluvial deposits and tidal mudflats.

Under pressure to develop the area, particularly in the city of San Francisco and at the Port of Oakland, extensive reclamation of the tidal marshes has been ongoing over the

past 100 years. Techniques of filling have developed over this period as understanding of the vulnerability of soft soils to settlements has grown but much of the early infill, and indeed the hydraulic fill placed during the 1920's, 1930's and post-war, is extremely susceptible to settlement and lateral spreading.

The influence of soft ground on incoming seismic ground motions is to selectively amplify low frequency motion. Indeed in the San Francisco Bay Area seismic waves with frequencies between 1 - 1.5 Hz are amplified the most, based on recordings of underground nuclear test blasts in Nevada, Reference 1. The bay mud has a low seismic impedance, defined as the product of shear wave velocity V_s and bulk density ρ , and typical values of V_s are in the range 90 - 130 m/s and density 1300 - 1700 kg/m³.

Assuming a uniform shear modulus G with depth H and using $H = \lambda/4$, where λ is the wavelength, amplification of a 'shear beam' in the range 1 - 1.5 Hz would be consistent with depths of bay mud in the range 15 - 32.5 m, which is common. A depth of bay mud of 40 m might be expected to amplify motions in the range 0.6 - 0.8 Hz. The recent Holocene alluvial deposits are denser and stiffer, with typical values of $\rho = 1900 \text{ kg/m}^3$ and V_s in the range 200 - 300 m/s. Depths of alluvium of the order of 33 - 75 m would be consistent with amplification in the range 1 - 1.5 Hz.

An approximate solution to estimating the natural frequency of a shear beam with a varying shear modulus is to use the value of G at mid-depth, Reference 2. Assuming G is proportional to the square root of the effective confining pressure then a column of loose saturated sand 20 m deep with a water table near the surface will have a natural frequency of the order of 2.2 Hz. Even ignoring strain softening effects, with excess pore pressure generation equal to only 50% of the insitu vertical effective stress this frequency value would fall to below 1.9 Hz.

Clearly amplification of motion is to be expected on soft soil sites at frequencies of between 1 - 2 Hz, depending on the site profile, in the Bay Area. The evidence of damage from the Loma Prieta earthquake outside the epicentral area points almost exclusively to a geotechnical connection as damage was localised on soft soil sites.

7.4 DAMAGE AND GROUND MOVEMENT IN THE BAY AREA

7.4.1 Port of Oakland

A plan of the Oakland Harbour area with surface deposits is shown in Figure 7.1, based on Reference 3. The Western peninsula extension to the port includes the Matson and 7th St. PCT Terminals and was constructed during the early 1960's at the same time as the Bay Area Rapid Transit (BART) which runs under the site. The 7th Street and Matson Terminals were extensively damaged in the earthquake by settlement and lateral spreading. In Figure 7.2 a cross-section through Berth 35 shows hydraulic sand fill of the order of 10 m deep overlying a thin layer of soft Bay mud with further dense sands and alluvial deposits at depth. The wharf structure itself is supported on short piles which are founded in the natural sand deposits beneath the soft Bay mud. The hydraulic fill is 'contained' within a dyke structure which surrounds the filled area.

Settlement in this area was the main cause of damage and is shown in Plate 7.1. Vertical settlement of the hydraulic fill by up to 0.5 m was observed, shearing water supply lines to the wharf and at Berths 35 - 37 provoking settlement of the rear crane rail, putting the facility out of action. Lateral spreading of the embankment under the wharf imposed high lateral forces on the piles shearing off piles at deck level in extreme cases, Plate 7.2. New designs of wharf now use

vertical instead of raking piles to support the deck although these were also found to be damaged by lateral spreading (at Berth 32, for example). In each case the innermost line of piles was the most heavily damaged.

Vertical settlement and lateral spreading also caused extensive damage to the Alameda Naval Air Station and to Oakland International Airport. In the case of Oakland Airport the most heavily damaged section of the 3000m runway was the 1000m extension completed in the early 1970's to carry freight traffic. The runway runs parallel to the coastal dyke, at a distance of around 150 - 250m from it and at an elevation of around +3m with a shallow lagoon between them. The coastal dyke spread seawards and the runway spread 0.3 - 0.6m laterally towards the lagoon causing extensive cracking of the surface which rendered the runway inoperable. The loss of income from freight traffic was a severe threat to the financial viability of the Airport.

7.4.2 Marina District

The Marina district of San Francisco is an exclusive residential area on the shore of the Bay. The pattern of construction is three storey structures along a street with a four storey structure marking the corners of a block, Plate 7.3. Foundations are typically shallow strip or pad footings. Figure 7.3 shows a map of the Marina district with the former shoreline marked and areas of former marshland identified. The fill material extends to some depth and typical SPT data is shown in Figure 7.4.

Large sand boils were noted in Marina Park by the waterfront and in the built-up area between Marina Boulevard and Lombard. There were several cases of sand boils erupting inside buildings and sand pouring out of ground floor garages into the street. The gas mains and water supplies were badly disrupted by ground movement and cracking along the streets showed clearly how one block had moved relative to another. Where holes had been dug to repair services substantial differential movement could be seen between the asphalt surface and the underlying soil, as if the surface structures had moved on a raft, Plate 7.4.

A striking feature of the damage was that, almost without exception, four storey buildings were damaged and three storey structures were not. There were cases where three storey buildings had been damaged by impact from an adjacent four storey building but there were few if any examples of a badly damaged three storey structure. Four storey buildings were on the corners of a block, and many had a soft storey at ground floor level to provide off-street parking. Although the pattern of cracking and failure of services is simply explained by the widespread liquefaction, the selective damage to building structures is not. Widespread liquefaction would isolate surface structures from the ground motion but tear apart tall and short buildings alike by lateral spreading. An alternative explanation is that as pore pressures rose in the ground increasing amplification through the softening ground provoked large amplitude shaking firstly in the lower frequency structures which had fundamental frequencies closer to the earthquake driving frequency. Three storey structures would be naturally stiffer, particularly as they are wedged together along the block with little freedom of movement.

7.4.3 Embarcadero/Market

The main water supply to the downtown area runs in underground pipeline Northwards under Valencia, South Van Ness and Harrison Street as it did in 1906. Recent research has mapped the locations of water main breaks onto subsurface topographical features such as areas of marsh and estuary and filled

ground to show the close correlation between damage to underground services in the 1906 earthquake and the distribution of soft and filled ground, Reference 4.

During the 1989 earthquake there was again evidence of widespread differential settlements in the area South of Market with some damage to underground pipelines, see for example Plate 7.5. However, it was not always clear whether settlements were new or old, as parts of the area are run down and repair work has not been carried out. Access ramps to the overhead James Lick Skyway were closed at several locations including 4th Street for inspection. The depth of fill at this site is around 2.4m, overlying 5m of very soft silty clay, Reference 5. From 4th Street to 6th Street the freeway passes over large areas which are known to have liquefied in 1906 as shown in Figure 7.5. At specific sites such as 6th and Townsend there was clear evidence of liquefaction and large ground settlements, which is a site on the fringe of the identified 1906 zone. Further North along 6th Street, between Tehama and Clementina, there were extensive settlements on either side of the street with the centreline of the road apparently supported by a large diameter underground pipeline. This pattern of uneven differential settlements was marked throughout the area and probably reflected both non-uniformity in the soil as well as stiff underground inclusions such as services.

The Embarcadero freeway which crosses the end of Market in front of the Ferry building is also constructed over filled ground. The nature of the ground can be seen in a cross-section down Market Street in Figure 7.6, after Reference 5. There was clear evidence of liquefaction around the piers of the overhead structure, Plate 7.6, with settlements of several centimetres throughout the car park which is located under the freeway.

7.4.5 Nimitz freeway

The most dramatic collapse caused by the earthquake was the failure of around 1 mile of elevated freeway in Oakland. The I-880 freeway runs broadly east-west along the shore of the bay, passing on an elevated structure to the south of the centre of Oakland. At Oakland harbour the road turns sharply northwards before turning again westwards to join the approach to the Bay Bridge, Figure 7.1.

The structural form and failure of the elevated section is described elsewhere in this Report and in this section discussion is limited to the behaviour of the ground and its foundations.

The freeway collapse was limited to the north-south section described above and which may be seen in Figure 7.1. The foundations of the piers are piled with a substantial pile cap at the foot of each pier. Detailed drawings of the foundations are not available, but a sketch indicating the pile cap and piles is shown in Figure 7.7. Plate 7.7 shows an excavated pile cap.

The ground conditions along the collapsed section are poor, but not in general filled ground as has been reported elsewhere. A typical borehole, Figure 7.8, shows layers of alluvial deposits, including loose saturated silty sands and soft clays. Conditions only improve at depth. This profile is in broad agreement with the evidence of Figure 7.1 which shows the freeway following the original shoreline, with reclaimed land to the west and downtown Oakland to the east.

In the downtown area most of the CBD is founded on Merritt Sand, a beach or near-shore deposit of slightly clayey, silty sand, regarded as a good foundation material.

Strong motion records in Oakland showed remarkable coherence on firm ground, typified by the CSMIP Oakland 2-storey building strong motion instrument station 58224 which showed peak horizontal accelerations of 0.23g and a peak vertical acceleration of 0.16g, Table 2.1.

The time histories of ground motion on the soft soil sites in Oakland show clearly a strong amplification of low frequency motions. This characteristic feature of soft soil sites is caused by the low shear modulus of the ground leading to a natural frequency within the range of the incoming base shaking. Certain soils, such as soft clays, have an inherently low shear modulus. Other soils, including saturated silty sands, will lose their initially high shear modulus and degrade with excess pore pressure generation during shaking. As the amplitude of shaking builds up strain softening will further degrade the soil.

7.5 CONCLUSIONS

The Loma Prieta earthquake generated all the major geotechnical phenomena which are associated with ground shaking. The major lessons to be learnt arise from the heavy damage caused by large amplitude low frequency shaking up to large distances from the epicentre. For San Francisco and Oakland this was a relatively small event because of their epicentral distances. The area susceptible to ground softening and amplification during a larger event would be very widespread.

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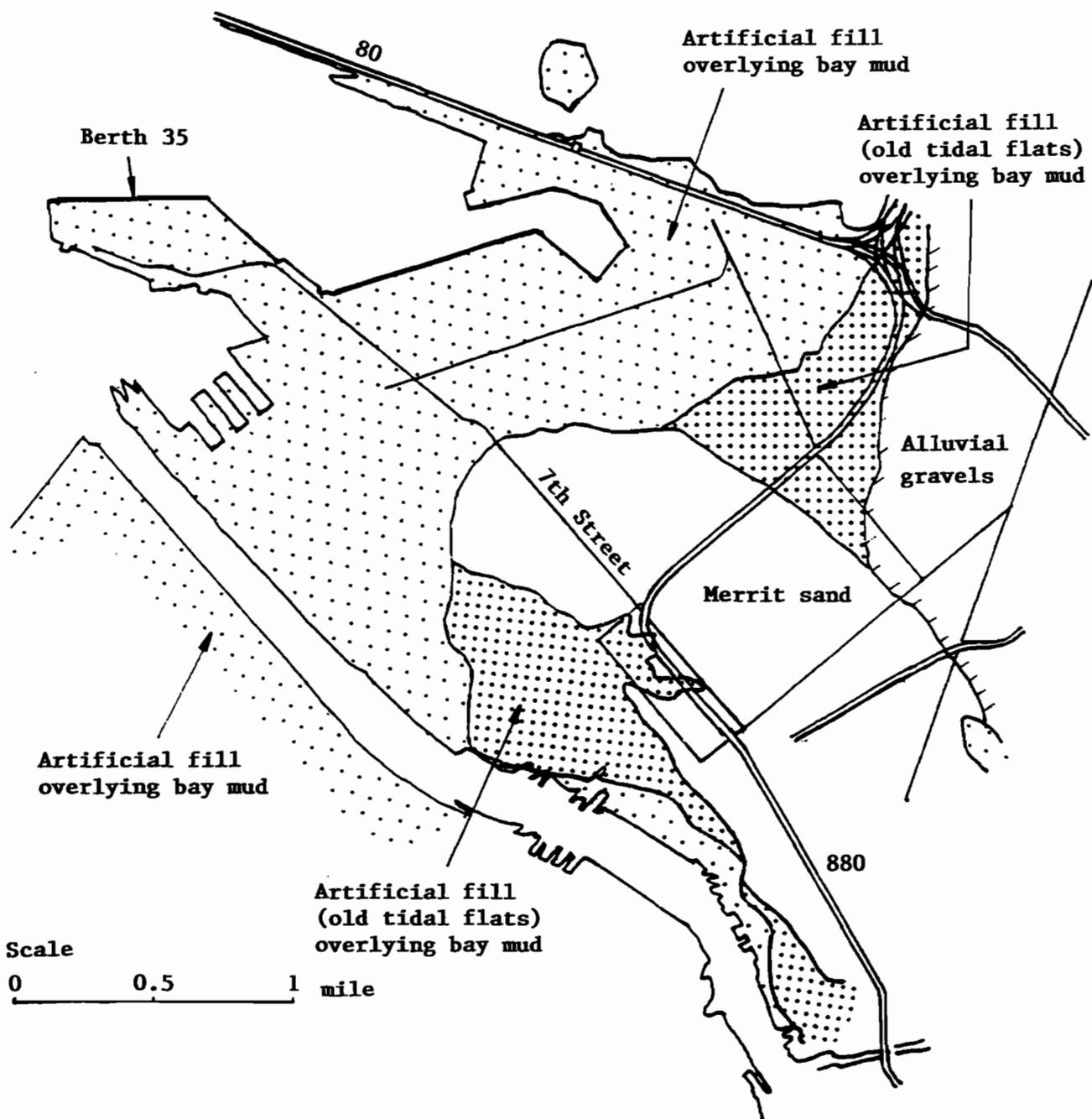


Figure 7.1 : Plan of Oakland showing superficial deposits
(after Radbruch, 1957)

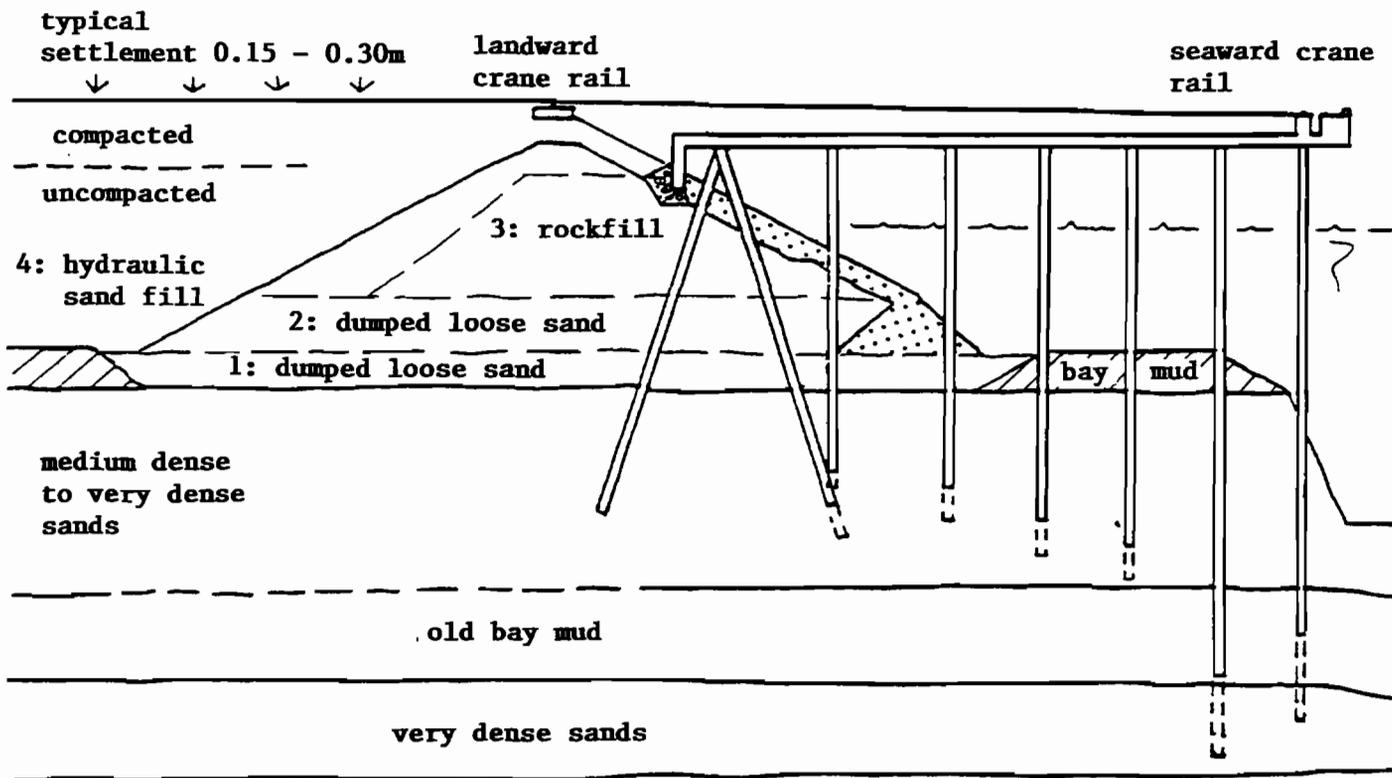
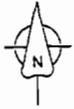


Figure 7.2 : Cross-section through Berth 35 (after the Port of Oakland and Geomatrix Consultants)



SAN FRANCISCO BAY

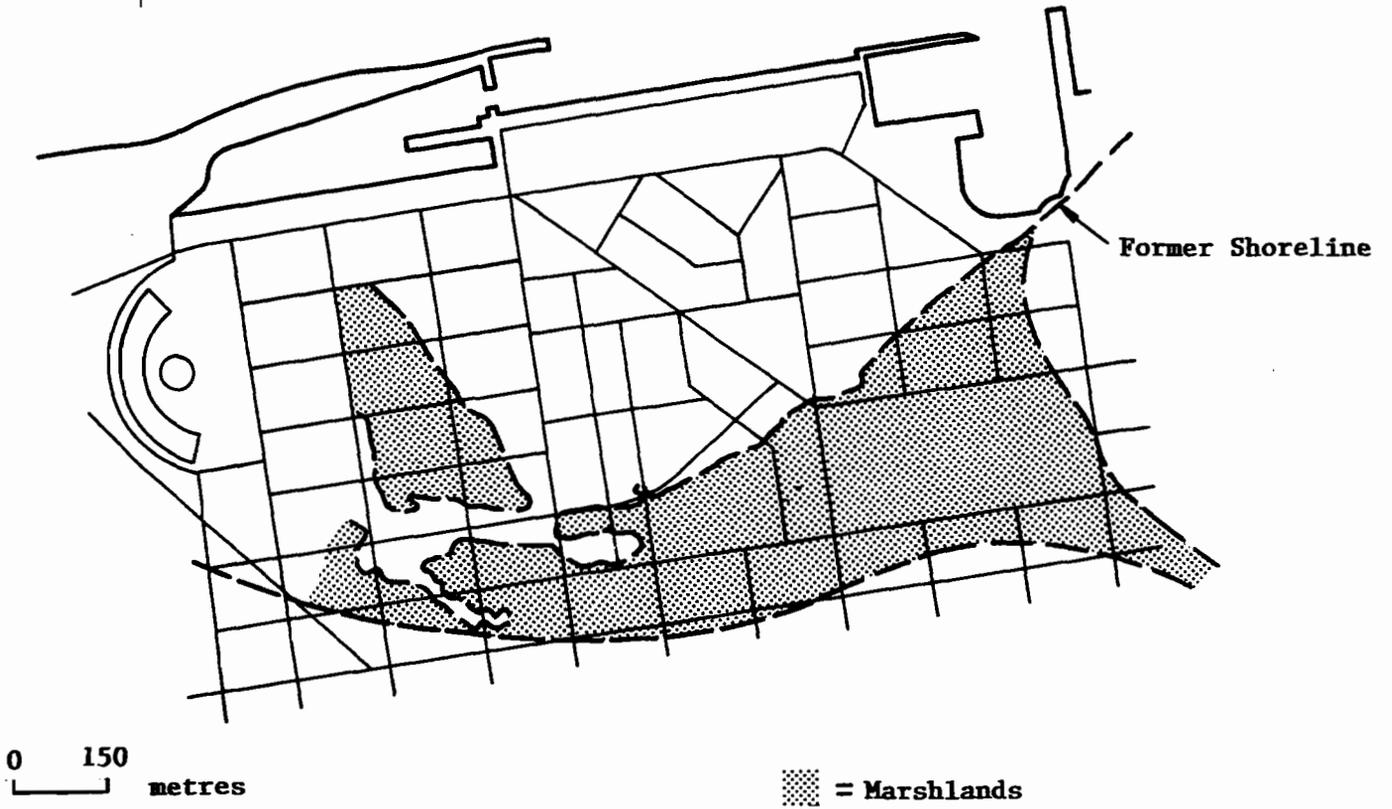


Figure 7.3 : Map of the old shoreline in the Marina District
(after Miller Pacific)

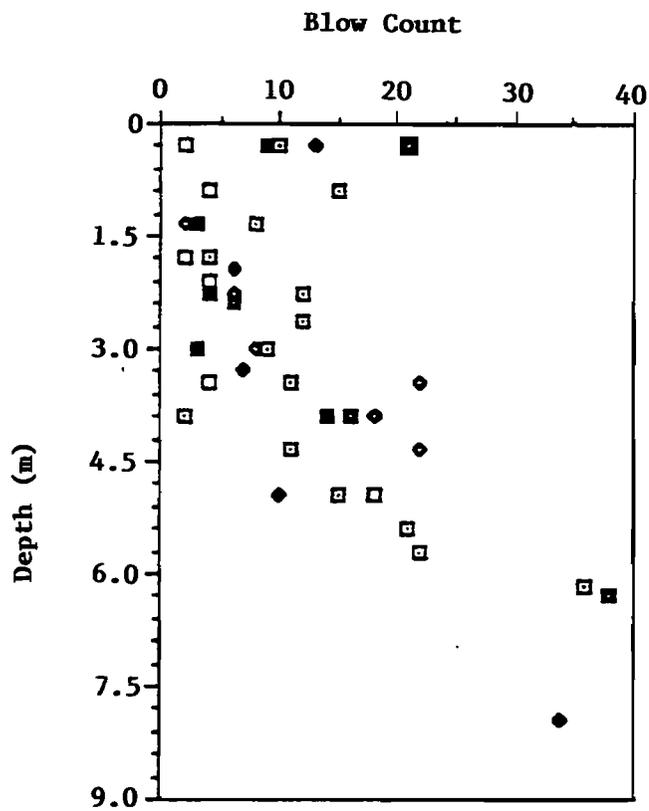


Figure 7.4 : Typical SPT Data from North Point, Marina District (after Miller Pacific)

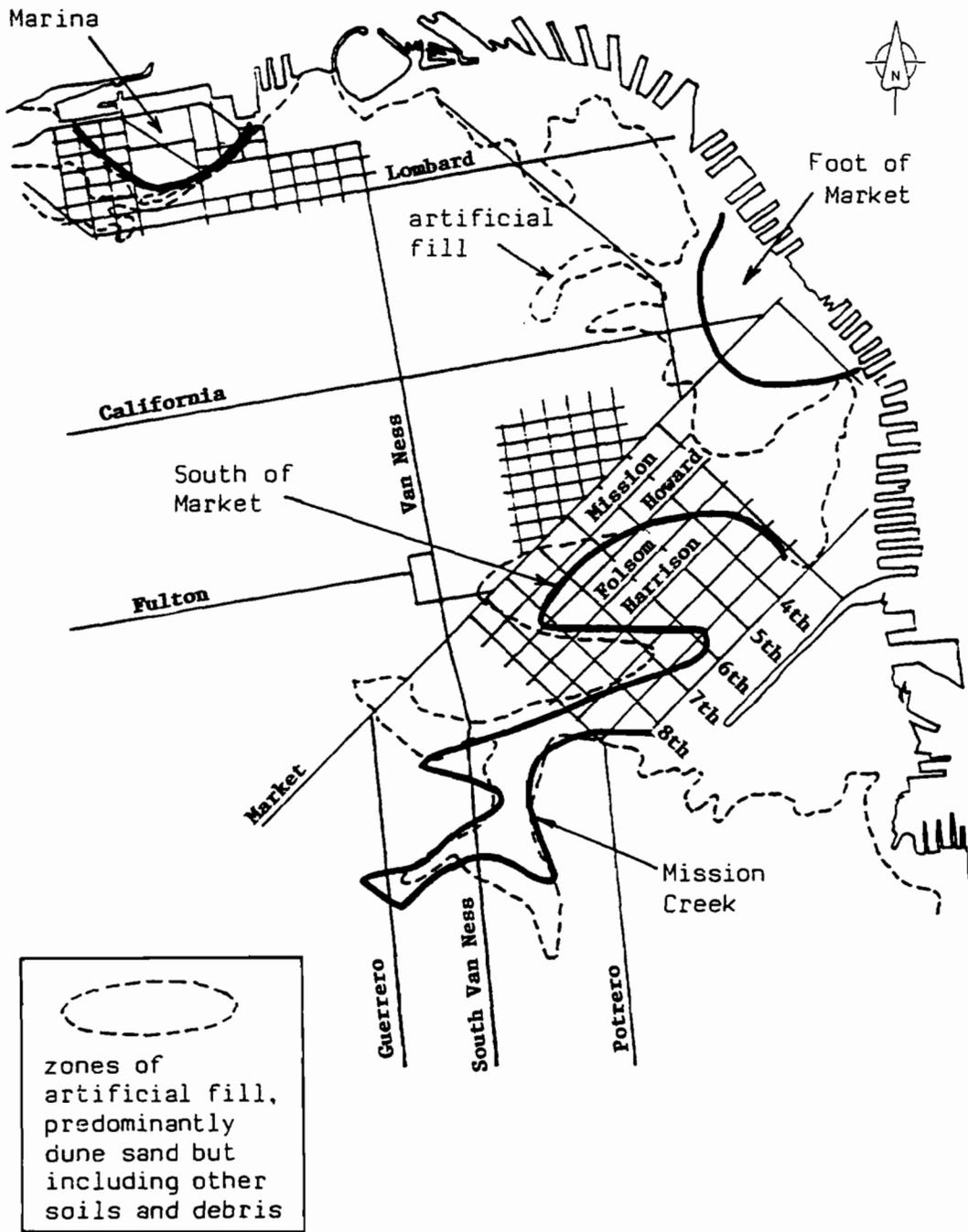


Figure 7.5 : Areas of San Francisco which liquefied in 1906, superimposed on zones of fill (after Youd and Hoose, 1978)

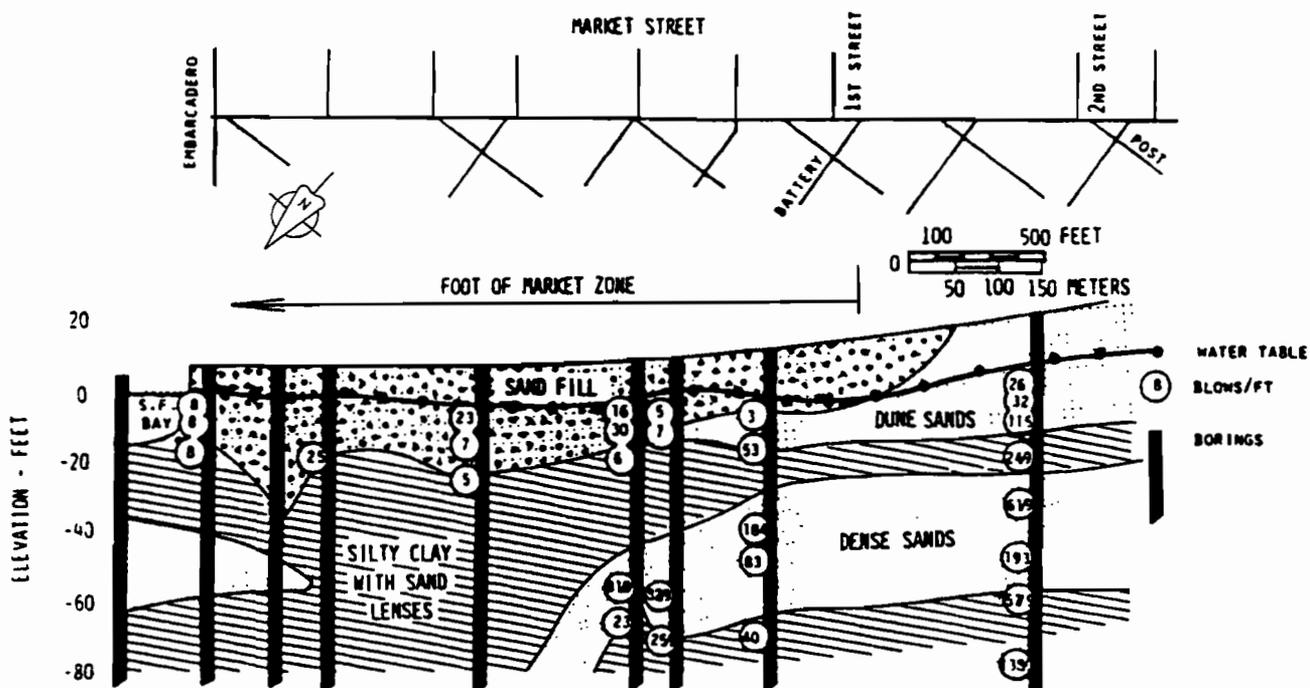
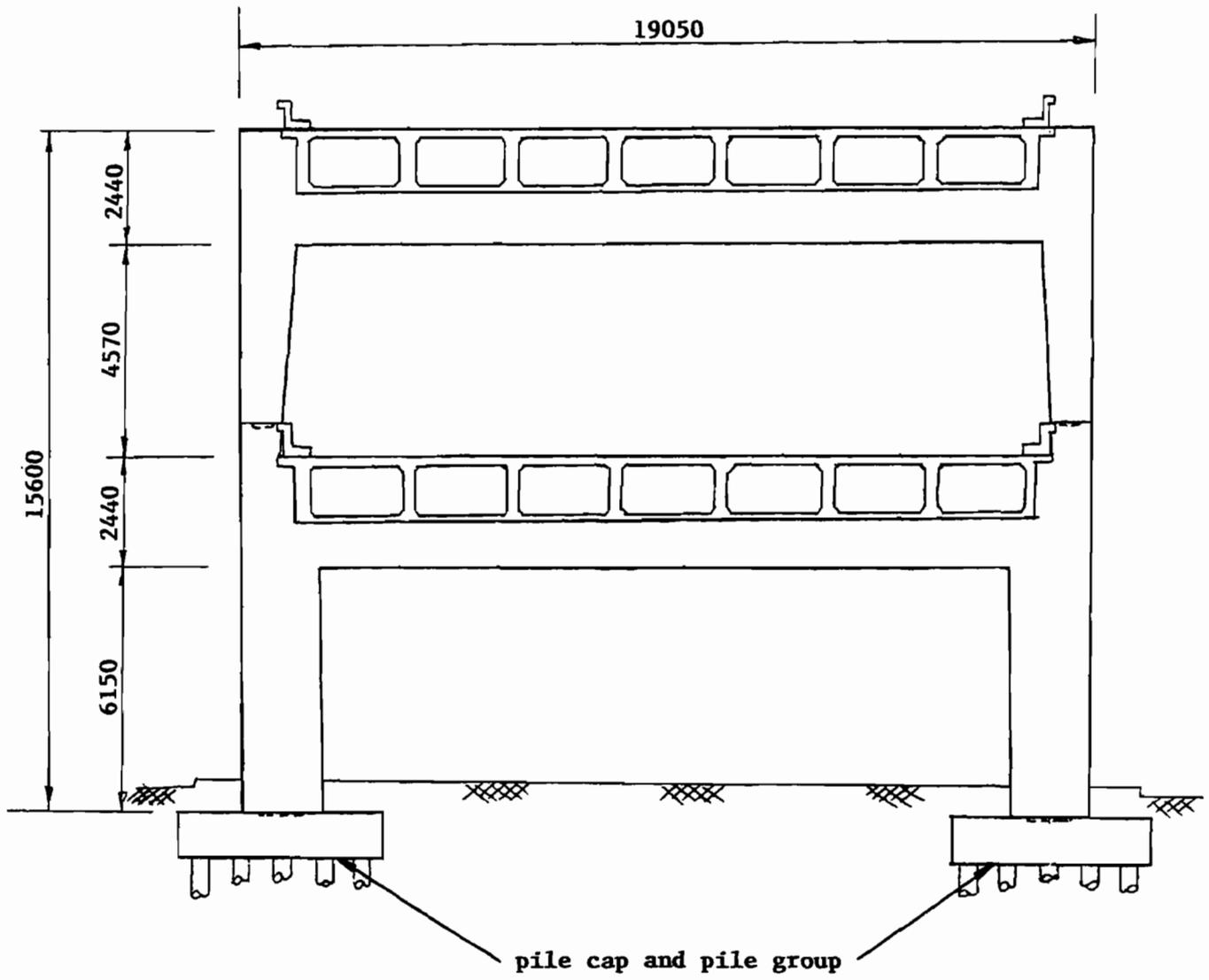


Figure 7.6 : Geotechnical cross-section through Market Street (Youd and Hoose, 1978)



Pile group : typical arrangement 5 x 4 grid at 3' spacing,
 1' diameter spiral welded pipe piles. Average length
 length around 50' (e.g. near 22 street), design
 pile loading 45 tons

Figure 7.7 : Elevation through typical Bent (type B1) showing as-built foundation

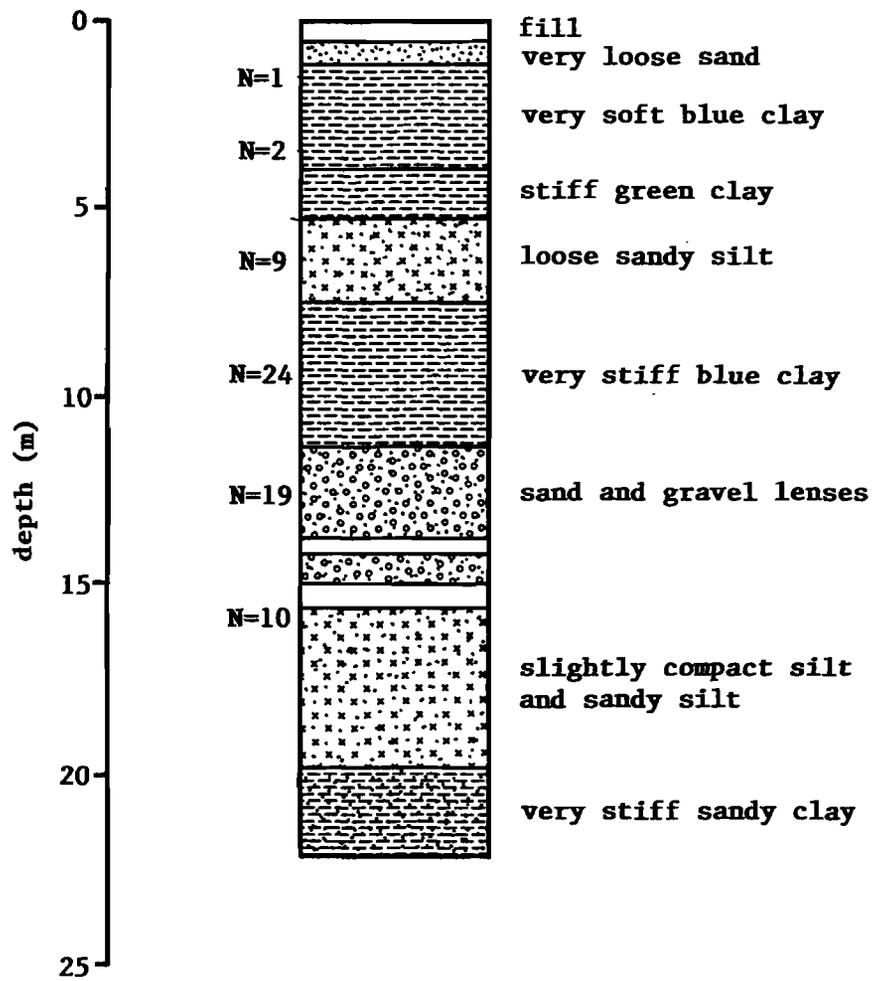


Figure 7.8 : Borehole at 22nd Street, Oakland (water table at 0.6 - 1.2m depth)

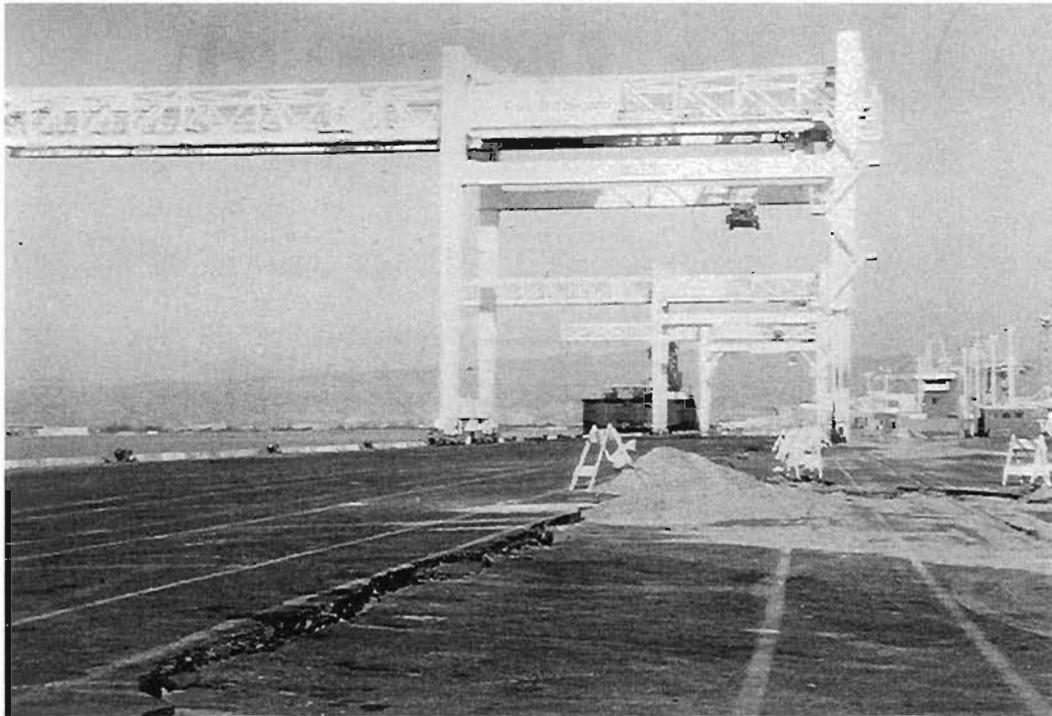


Plate 7.1 : Settlement behind wharves at Oakland harbour

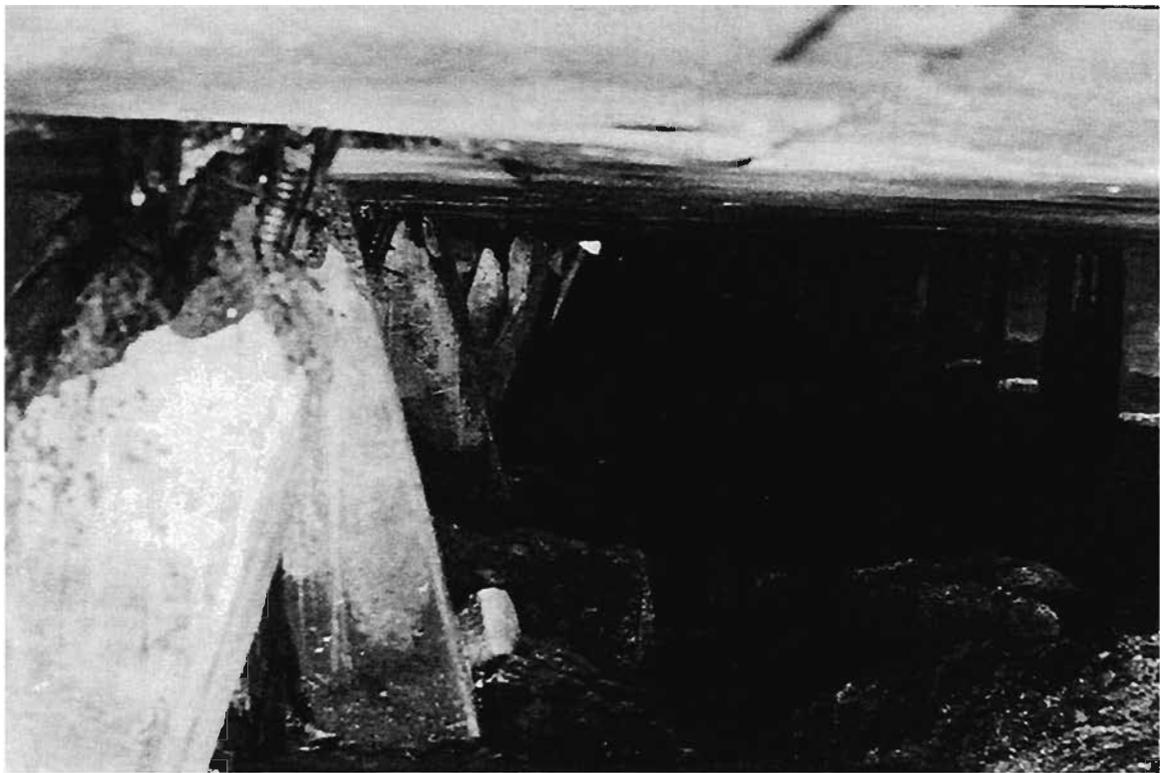


Plate 7.2 : Lateral spreading under wharf



Plate 7.3 : Typical buildings in the Marina District

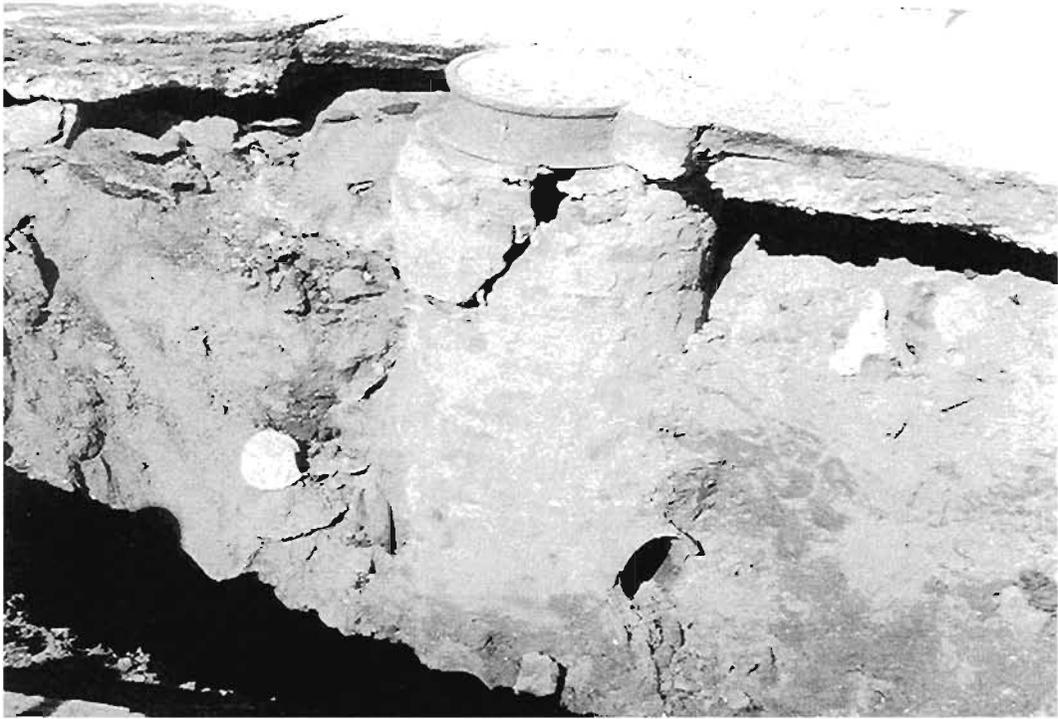


Plate 7.4 : Differential movement between the pavement and underlying soils



Plate 7.5 : Differential settlement south of Market Street



Plate 7.6 : Liquefaction around pile cap on the Embarcadero



Plate 7.7 : Excavated pile cap on the Nimitz freeway

GEOTECHNICAL PHENOMENA IN THE EPICENTRAL AREA

A Coatsworth,
Principia Mechanica

8.1 INTRODUCTION

Media attention on damage from the Loma Prieta earthquake concentrated on the San Francisco area, partly due to the presence of TV outside broadcast units for the World Series baseball match at Candlestick Park but also because communications in the epicentral area were poor, in part due to geotechnical failures.

EEFIT studied geotechnical aspects of failures in the Santa Cruz Mountains, around the towns and communities of Los Gatos, Santa Cruz and Watsonville, and Moss Landing harbour. Damage in these areas was strongly related to geotechnical conditions, as in the San Francisco area.

8.2 GEOMORPHOLOGY

Los Gatos is on high ground north of the Santa Cruz Mountains, in part within the Los Gatos Creek.

Santa Cruz lies on the coast at the mouth of the San Lorenzo River, Figure 6.1. The river has been trained to a trapezoidal channel for 4000m through the town centre, which is itself founded largely on alluvium.

Watsonville is some three miles inland on the floodplain of the Pajaro River, the mouth of which is currently at the south end of a spit. The names of the backwaters to the north of the river (Watsonville Slough, Struve Slough, Harkins Slough) reflect the marshy nature of the ground. Floods in 1982 killed 25 people in Watsonville.

Moss Landing Harbour lies within a bar to the Elkorn Slough eight miles south of Watsonville. The development around the harbour has taken place on mudflats.

8.3 GROUND FRACTURES

There were a number of major extensional fractures along the ridge close to Summit in the Santa Cruz Mountains. These fractures were orientated close to the line of the San Andreas Fault, with sinistral displacements of up to three feet. Identical structures were photographed at this same location following the 1906 earthquake. The movement of the underlying San Andreas Fault is dextral, and was not manifest in surface fracturing. The surface fractures were superficial, not connecting with the main fault rupture, that from the location of aftershocks appears to have not ruptured much shallower than three miles.

The most spectacular of these cracks could be followed for about 500m, severing all services along a track to some houses and fracturing the corner of a swimming pool.

Around the upper Corralitos Creek north of Watsonville, the road was severely fractured parallel to the neighbouring river valley, which follows the line of the San Andreas Fault.

8.4 LANDSLIDES

The Santa Cruz Mountains comprise Sanat Margareta Sandstone and Santa Cruz Mudstone rising to about 120m. The mountains are traversed north-west to south-east by Summit Road, cut north-south by Highway 17, and contain numerous mountainous roads.

Landslides were common in the Santa Cruz Mountains. Two of the largest blocked much of Highway 17. A slide at Summit had a volume of 70,000 m³, and that to the south at Laurel Canyon had a volume of about 150,000 m³. These took place on steep (circa 60 degree) slopes of soil and weathered rock vegetated with pines. Highway 17 was originally severed, and even 18 days after the earthquake was open to uni-directional traffic only with a police escort. The substantial reinforced concrete central barrier of the two lane carriageway was destroyed in several places, but played a role in protecting one carriageway from debris. The landslides occurred at a time of drought; clearance of the debris and regrading of the slopes was a race against the arrival of the autumnal rain, which would threaten mudslides on slopes stripped of vegetation.

Hundreds of minor landslides occurred on lesser roads in the Santa Cruz Mountains.

In the Upper Corralitos Creek, almost in the line of the San Andreas Fault, three landslides blocked the river creating temporary lakes 6 to 10m deep. These threatened to burst and flood Eureka Canyon Road and the village of Corralitos, the closest community to the epicentre. These lakes were drained within two weeks with the aid of funds provided by the US Soil Conservation Service.

In a lowland area the Green Valley Road north of Watsonville suffered landsliding in the areas of its higher embankments.

8.5 DAMS

The Santa Cruz Mountains host three major reservoirs: Lakes Elsman and Lexington, and Loch Lomond. Damage to Lake Elsman was reported by the New Civil Engineer magazine (NCE, 2 November 1989) but access was not possible.

Lexington Dam is further from the epicentre (16 miles) than Lake Elsman, but suffered similar damage. The earthfill structure is 60m high, with a crest length of 250m and it was built in 1952 to retain a reservoir of 25000 acre feet. It is a well instrumented dam with more than 30 piezometers, 2 settlement tubes and 2 strong motion seismographs. The motion on the south west abutment was 15%g vertical, with horizontal components of 45% and 41%g. Crest motion was 20%g vertical and 40%g horizontal.

Lexington Dam experienced transverse cracking close to both the south west and north east rock contacts. The cracks ran most of the height on the downstream face, and at least to the water's edge on the upstream face. The cracks were considered to be through-going. Longitudinal cracks may have been due to dessication, not seismic movement. Aggregate lateral movement of the centre of the crest was estimated as about 300mm. The concrete spillway appeared undamaged, but movement of the north-east abutment to the bridge probably rendered the bridge a write-off. After successive years of drought, the reservoir was very low. Damage to the dam, and potential consequences to the township of Los Gatos would otherwise have been much greater.

Newell Dam, which impounds Loch Lomond 10 miles north of Santa Cruz, experienced cracking to a depth of 2.5m. These cracks were filled and tamped.

8.6 LEVEES

Both the San Lorenzo River and the Pajaro River have been the subject of levee raising and construction as part of flood control measures. Neither flood protection project considered seismic risk. Earthquake design was initially a requirement for structures. Only with the 1960 Chile earthquake were the full consequences of seismically induced settlement appreciated, and with the 1964 Alaska and Niigata earthquakes exhibiting liquefaction. Levee reconstruction was required following the Loma Prieta earthquake. Data for the original construction and remedial work are as follows:

	Pajaro River	San Lorenzo River
Built	1949	1959
Length of levee	20.7 miles	3.2 miles
Catchment area	1275 square miles	137 square miles
Population at risk	27,000	47,000
Riverside slope	1V:2.5H	1V:3H
Landside slope	1V:2H	1V:3H
Height of levees	0.6 - 3.6m	0 - 3m

A further 4.2 miles of levee were built on the Lower Corralitos Creek immediately east of Watsonville.

The levees to the San Lorenzo and Pajaro Rivers suffered cracking and settlement due to liquefaction of the underlying alluvium. Sand boils were observed both on the riverside and on the landside, where they extended hundreds of feet into open farmland. Lateral deformation by several feet towards the river was experienced by the levees, thus damaging bridge abutments.

Spreading of levees resulted in longitudinal cracks and settlement by about 1 foot (0.3 m). Transverse cracks occurred, particularly at bends.

The US Army Corps of Engineers carried out emergency reinstatement to the as-built condition of 4530 feet of levee in Santa Cruz mainly around the mouth of the San Lorenzo River, and 7000 feet around Watsonville. The work was a race against the autumn rains because of the likelihood that flooding would occur due to:

- direct overtopping
- direct flow through transverse cracks
- piping and erosion through shortened seepage paths.

More damage occurred at the downstream end of the Pajari River, probably due to a higher groundwater table there.

The performance of these non-seismically designed levees on soft alluvium may be compared with the good performance of the dykes around Foster City, built on engineered hydraulic fill to the south east of San Francisco.

8.7 BRIDGES

The damage to levees caused secondary damage to bridges. The abutments of the Riverside bridge in Santa Cruz moved inwards and settled resulting in closure of the bridge. It was to be replaced. Resurfacing of the abutment area of the nearby Broadway Bridge was required.

The collapse of the Thruwachter Bridge across the Pajaro River illustrates the caution required of earthquake field investigators. The bridge connected two counties and had received little maintenance from either. It was condemned prior to the earthquake, which caused inward movement of its abutments and the adjoining levees. The EEFIT team saw the bridge after it had finally been demolished by the USAE.

Bridges across the sloughs west of Watsonville and the bridge carrying Highway 1 over Elkorn Slough required resurfacing of their abutments due to settlement. Worse abutment failure occurred to a bridge on the mountain pass between Watsonville and Gilroy.

Settlement and inward movement of the abutments to the bridge across Moss Landing Harbour displaced it from its supporting piles.

8.8 ROADS

Highway 17, the Summit Road and minor roads in the Santa Cruz Mountains experienced:

- cracks to the sub-base, particularly on bends;
- landslides and rockfalls on the cut side;
- settlement on the fill side.

The road around Moss Landing Harbour settled a minimum of 150 - 200mm and cracked. Sand on an adjacent parking lot confirmed that liquefaction had occurred. Jetty Road, which crosses the upper reaches of the harbour on a causeway, was impassable. Edge failure with a lateral movement of up to 2m occurred. Longitudinal movement of about 1m buckled the tarmac and settlement by a similar amount took place. The damage was worst where the causeway crossed an Armco culvert.; the poor state of this pipe suggests that there may have been pre-existing erosion damage to the causeway. In the nearby mudflats there were sandboils of 1 - 2m in diameter, and fissures up to 150mm wide containing silty fine sand.

8.9 SERVICES AND TANKS

Services ruptured due to ground displacement close to the failed river levees in Santa Cruz. Similar damage occurred in the areas of Watsonville worst effected by structural damage. Services were also damaged in downtown Los Gatos.

Fuel oil tanks at Moss Landing Harbour settled and appeared to have leaked at pipework connections.

8.10 CONCLUSIONS

Damage due to dams and the potential consequences of failure would have been greater, but for the drought conditions.

Speed of repair to landslide stricken roads and to flood protection levees was dictated by the anticipated autumnal rains.

No loss of life due to geotechnical failures was reported, but the potential for secondary damage due to flooding was considerable.

The structural damage experienced in Watsonville is attributable to the construction on a flood plain, where the weak soils accentuated the damaging low frequency motions. Likewise, damage in Santa Cruz was on alluvium adjacent to the river mouth.

Provision of seismic resistance in development on deep alluvial soils may be severely limited by the economic value of the facility.

The vulnerability of the soft soil locations was known to Californian earthquake engineers before the earthquake. However this known deficiency of the UBC (1988) could not be resolved within even a Californian, let alone national, application of the Code. This demonstrates the dangers of the averaging process used in deriving seismic codes and the requirement for micro-zonation of seismic hazard.

8.11 ACKNOWLEDGEMENT

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