National Research Council Canada
Conseil national de recherches Canada

Institute for Research in Construction
Institut de recherche en construction

The 1989 Loma Prieta (San Francisco Area) Earthquake: Site Visit Report

by Alexander M. Jablonski, K. Tim Law, David T. Lau, Jean-Robert Pierre and James H.K. Tang, with the collaboration of J. Hans Rainer and David E. Allen

Internal Report No. 594

Date of issue: May 1990

Canada

THE 1989 LOMA PRIETA (SAN FRANCISCO AREA) EARTHQUAKE

- SITE VISIT REPORT

by

Alexander M. Jablonski[1]
K. Tim Law[1]
David T. Lau[2]
Jean-Robert Pierre[3]
James H.K. Tang[4]

with the collaboration of

J. Hans Rainer[1]
David E. Allen[1]

1. Structures Section
Institute for Research in Construction
National Research Council of Canada
Ottawa, Ontario

2. Department of Civil Engineering
Carleton University
Ottawa, Ontario

3. Services Etudes et Normalisation
Hydro-Québec
Montréal, Québec

4. Civil Engineering and Architecture Dept.
Ontario Hydro
Toronto, Ontario

Institute for Research in Construction
National Research Council of Canada
Ottawa, May 1990

ABSTRACT

The 7.1-magnitude earthquake that shook the San Francisco Bay area in the late afternoon of October 17, 1989, was the largest earthquake in northern California since the 8.3-magnitude San Francisco earthquake in 1906. It resulted in an economic loss of nearly $10 billion, 66 deaths and more than 3200 injuries. Eight days after the main shock, a team consisting of researchers from the Institute for Research in Construction, National Research Council of Canada, and industrial specialists was dispatched to assess the amount of structural damage in the San Francisco Bay area. This report contains background information on the Loma Prieta earthquake, its impact in the affected areas and reconnaissance team observations related to geotechnical aspects, performance of buildings, performance of transportation structures, damage to power generating and transmission facilities and other industrial facilities. While a number of cities, including San Francisco and Oakland, suffered varying levels of damage, severe damage was limited to areas associated with soft sub-soil layers, especially fill areas. The earthquake caused ground failures (including soil liquefaction, landslides, soil shifts and ground cracks) throughout the San Francisco Bay area and along the Pacific coast. Failure to retrofit older seismically vulnerable non-engineered structures was a major factor causing damage. Lessons learned from the earthquake from the Canadian perspective are also discussed.

ACKNOWLEDGEMENTS

The authors wish to express their appreciation to numerous individuals in Canada and California, who assisted the team during the reconnaissance visit after the Loma Prieta earthquake.

The authors are grateful to Dr. J.H. Rainer, Head of the Structures Section, for arranging the visit to the San Francisco Bay Area. This visit would not have been possible without funding for the members of IRC provided by Canada Mortgage and Housing Corporation, Ottawa; the other members were supported by their respective employers, Hydro-Quebec, Montreal; Ontario Hydro, Toronto and Carleton University, Ottawa.

During the visit the authors visited many institutions including universities, county and municipal offices and engineering companies. Their officials were very helpful and contributed to this task; only some of them could be listed below.

The access to areas affected by the earthquake was made possible by the help, guidance and sponsorship of the Earthquake Engineering Research Institute, El Cerrito, California, which included the team in its post-disaster reconnaissance efforts.

The assistance of the following individuals from various institutions and companies is gratefully acknowledged:

<u>Earthquake Engineering Research Institute (EERI)</u>, El Cerrito, California

Robert D. Hanson
Susan K. Tubbesing
Lee Benuska
Elizabeth Dembinska - Arscott

<u>Pacific Gas and Electric (PG&E)</u>, San Francisco, California

Bill Grey
Stephen E. Hartz
Gerald C. Laguens
Edward N. Matsuda

<u>Electrical Power Research Institute(EPRI)</u>, Palo Alto, California

Dr. Robert P. Kassawara
Dr. John F. Schneider

<u>Stanford University</u>, Palo Alto, California

Dr. H. Shah
Dr. A.J. Schiff

U.S. Geological Survey (USGS), Menlo Park, California

Dr. Kenneth R. Lajoie
Dr. Erdal Safak

EQE Engineering, San Francisco, California

Brian J. Benda
Tom K. Chan
Greg S. Hardy

City of San Francisco, California

Frank Lew

City of Watsonville, California

Neil England

Many thanks are also due to numerous individuals from the areas affected by the earthquake for their assistance and description of their feelings and experience during these difficult days.

Thanks are also expressed to engineers and scientists from other national teams, especially those from Britain, France and Italy, whom the IRC team met during the visit and with whom discussions were held regarding the damaged sites.

The financial contribution of Canada Mortgage and Housing Corporation to the IRC team members, under a contract concerning earthquake effects on housing in Canada, is gratefully acknowledged.

CONTENTS

Chapter 1

SYNOPSIS OF THE LOMA PRIETA EARTHQUAKE AND ITS IMPACT IN THE SAN FRANCISCO BAY AREA

by

K.T. Law and A.M. Jablonski

1.1 INTRODUCTION

On October 17, 1989, 17:04 Pacific Daylight Time, a strong earthquake, called the Loma Prieta Earthquake, of Richter magnitude $M_L = 7.0$ originated from the San Andreas fault and shook the entire San Francisco Bay area. The average surface wave magnitude (M_S) was estimated by the U.S. Geological Survey as 7.1. The epicentre was located approximately 16 km northeast of the city of Santa Cruz in the Forest Nisene Marks State Park in the Santa Cruz Mountains (Fig. 1.1). The felt area stretched from Las Vegas, Nevada to Los Angeles, California. The hypocentral depth was about 18.5 km, which is relatively deep for most events associated with the San Andreas fault. The fault movement includes not only typical horizontal component of slip, but also a significant thrusting of the southwest side up and over the northeast side. The rupture zone was 40 km long and the rupture mechanism in the main shock was completed in about 8 seconds [1,2]. The overall felt duration of the strong shaking in some areas reached 20 seconds.

Within a period of twelve days after the mainshock, 80 aftershocks of Magnitude 3.0 and larger were recorded. The largest aftershock, Magnitude 5.2, occurred 37 minutes after the main one. The second largest aftershock, Magnitude 5.0, occurred Thursday, November 2, 1989. Seismologists advised that additional aftershocks, although generally becoming smaller and smaller, could be expected in the next following weeks to months after the earthquake [2,3,4].

Shortly after the earthquake, the Institute for Research in Construction of the National Research Council of Canada (IRC/NRCC) dispatched a team of five geotechnical and structural engineers to visit the San Francisco Bay area. The team was composed of Tim Law (team leader) and Alex Jablonski of NRCC, Jim Tang of Ontario Hydro, Jean-Robert Pierre of Hydro Québec and David Lau of Carleton University. The purpose of the visit was to determine the nature of damage by on-site inspection and to draw lessons for Canadian application. The visit was partly supported by the Canada Mortgage and Housing Corporation. On-site assistance was arranged by Earthquake Engineering Research Institute, El Cerrito, California. This assistance was indispensable for gaining access to the disaster areas.

1.2 INFORMATION ON GROUND MOTION SEISMOGRAPH DATA

The Loma Prieta mainshock had triggered almost all seismographic stations in the San Francisco Bay Area. Two major networks exist there, one installed and monitored by the Division of Mines and Geology, State of California (DMGSC), under the California Strong Motion Instrumentation Program (CSMIP) and the other by the U.S. Geological Survey in Menlo Park. Some measurements were recorded by other networks, e.g., Seismographic Stations of the University of California at Berkeley and Stanford University [5,6]. A summary of the main strong-motion measurements of ground motion stations is presented in Table 1-1.

CSMIP Network

Some 73 strong-motion accelerographs in the CSMIP network were triggered during the Loma Prieta earthquake. They included both ground-response stations and the structure response records. The closest accelerographs were located in Santa Cruz and in Watsonville and they recorded highest horizontal and vertical peak accelerations: Santa Cruz - 0.64g H, 0.47g V; Watsonville - 0.39g H, 0.66g V. They showed over 10 seconds of very strong shaking which could be related to an area of extensive damage. They indicated also a very high amplitude vertical motion which was seen at nearby stations. The records from San Francisco stations showed differences depending on the type of soil deposits and other geologic features. At Telegraph Hill and Rincon Hill - two stations in the eastern part of San Francisco - indicated relatively low accelerations, 0.09g H and 0.09g V. The station at Presidio, on rock, about 2.5 km southwest of the heavily damaged Marina district, recorded 0.21g H and 0.06g V, while the station at Pacific Heights, on firm ground, only 0.06g H and 0.03g V [5].

Several structures were instrumented. Among them was the station at Lexington Dam located 26 km from the epicenter (0.45g H and 0.15g V measured at abutment) and stations in the City of Oakland in the Lake Merrit district near the collapsed I-880 freeway section with 0.25 - 0.29g, H range and 0.07 - 0.16 g, V range, respectively [5].

USGS Network

Strong motion data was also collected from the USGS network. It included 38 stations (21 ground stations, 13 large buildings, 5 hospitals, 2 dams and 2 bridge abutments). The closest USGS Station was located at Anderson Dam, east of Morgan Hill at a distance of 27 km from the epicenter. The peak accelerations at the downstream located accelerograph reached 0.26 g. In Hollister, approximately 45 km southeast of the epicenter, two accelerographs recorded relatively high-amplitude (0.20g and 0.29g) horizontal motions. These stations were in the reported direction of the mainshock rupture propagation (about 10 km of the San Andreas fault zone). In the Palo Alto Veteran's Administration Hospital (6-storey building) peak accelerations were 0.38g in the basement and 1.09g on the roof. Several buildings in downtown San Francisco and on the east side of the Bay were instrumented. The Transamerica Pyramid Building on Montgomery Street in San Francisco (22 channels of acceleration data) reported peak horizontal accelerations of 0.10g at the foundation and 0.31g at the 49th floor. A 30-storey structure on Christie Avenue in Emeryville, about 100 km north of the epicenter and less than 2 km north of the collapsed section of the I-880 freeway in Oakland, reported peak accelerations of 0.26g, 0.32g, 0.24g and 0.39g at the ground, 13th, 21st and 31st (roof) levels [6]. Figure 1.2 presents the measured peak horizontal acceleration vs. epicentral distance. The attenuation tendency is also shown.

1.3 IMPACT

This earthquake was one of the most serious natural disasters in U.S. history. The estimated direct cost of property damage was as high as $6 billion. At least 62 people were confirmed dead and 3,757 people were injured. More than 12,000 people were displaced from their homes by the earthquake [2]. Indirect losses could, however, reach $10 billion. There were different estimates of losses reported by various agencies. The above figures are based on the estimates by the Governor's Office of Emergency Services, State of California, as appeared in Ref. 2.

The Loma Prieta earthquake caused severe damage to a number of engineering structures: the Cypress Street viaduct of the Interstate Highway 880, also called the Nimitz Freeway, collapsed, killing dozens of motorists; the section over one of the piers of the San Francisco-Oakland Bay Bridge on the East Bay crossing collapsed - the earthquake-caused displacement was more than 7 inches (18 cm); a number of bridges and highways were heavily damaged (among them San Lorenzo River Bridge on Highway 9 and Struve Slough Bridge on Highway 1 near Watsonville). The nine counties of San Francisco, Alameda, San Mateo, Santa Cruz, Santa Clara, San Benito, Monterey, Contra Costa and Marin were affected and reported damage in millions of dollars. A number of cities were hit, including San Francisco and Oakland, but the major areas of destruction were limited to several pockets associated with soft soil deposits, especially fill areas. Many buildings including old custom-made wood-framed (Marina district, San Francisco) and unreinforced masonry buildings especially from the turn of the century were seriously damaged in many areas (Los Gatos, Santa Cruz, Watsonville). Modern houses and high-rise buildings experienced heavy swaying, but stood up very well with some architectural and superficial damage reported in San Francisco. However, a few high-rise buildings on the east side of the Bay were damaged and were evacuated. Some single-family wood-framed buildings were knocked off foundations and collapsing "cripple" stud walls between the ground floor and the foundation collapsed, making the buildings unsafe for occupancy (Watsonville, Santa Cruz).

California has a special elevator code for use in tall buildings in earthquake zones. The performance of many elevators built according to this code, however, was poor. Many problems were encountered, the most common one being that the counter weight jumped off the guide rail, rendering the elevator unusable. It appears that the code needs revising.

Of the 1,500 highway bridges in the area, three had one or more spans that collapsed. Ten others were closed due to structural damage; 10 required shoring to be safely used; and 73 others sustained less severe damage. In many places, subsidence of the bridge approaches close to abutments were filled with asphalt to reduce large bumps which appeared after the earthquake. Some roads in the epicentral area (e.g., Highway 17 between San Jose and Santa Cruz) were damaged by landslides caused by the earthquake. Road barriers cracked severely and broke. Deformed and cracked road surfaces required immediate repair [7].

Damage to the control tower at the San Francisco international airport closed the facility for 13 hours. Liquefaction caused damage to one of the runways, partially closing the Oakland airport. Damage to the roadbed of Caltrain temporarily disrupted services between San Jose and San Francisco. However, the BART rapid transit system performed well with only temporary disruptions of service.

Water and sewage systems were damaged in some communities from the epicentral area to San Francisco. However, most disruptions to water supplies could be attributed to other causes. Local water lines were broken in Los Gatos, Santa Cruz, and in the Marina district of San Francisco. For instance, in the Marina district there were 72 significant pipe failures and 25 breaks outside that area (with 10 of these in the neighbourhood of Market Street). The 12-in. high-pressure (firefighting) and regular water lines did not break in the Marina district. But another 12-in. high-pressure line south of Market Street broke and depleted a 750,000-gal tank used for firefighting. In the epicentral area were more broken water mains (100 in Hollister and 60 in Santa Cruz). In Santa Clara County, one of two 66-in. raw water lines failed where it crosses the San Andreas fault. Water treatment plants and pumping stations performed quite well. The Lexington Pumping Station and Reservoir, which are located approx. 24 km from the epicenter, reported complete failure of 30-in. welded steel pipe at the top of the dam. The reservoir has an earth dam, which was observed to have some cracks. The East Bay Municipal District reported failure of welds in a 60-in. concrete-covered, spiral-welded steel line. On the East Bay side, over 140 mains broke due to the earthquake-induced displacements [7,8].

Initial power outages affected 1.4 million customers. Within 48 hours, though, service to almost all communities had been restored. The most severe damage was reported in many substations (ceramic members of circuit breakers and oil leaks to transformers) with major damage in two key substations in San Jose and San Mateo. The 500-kV switchyards at the Metcalf substation and at the Moss Landing Power Generation Plant were damaged (interruption of services in the upper peninsula and in Santa Cruz and Watsonville). A key element was quick replacement of damaged equipment. New equipment was flown in from the east coast with help of the U.S. Air Force and some others came from utilities in southern California through mutual-aid agreements [7].

Two major natural gas transmission routes supply the San Francisco Bay Area system. From the south through the Santa Clara Valley, they are two large-diameter pipes; and from the east, three lines are used for supply. Other local lines are also used to feed outlying areas. There was no reported damage to the major natural gas transmission system. There were only three breaks reported in local lines: in a 20-in. semi-high pressure welded steel distribution line in Oakland, a 12-in. line in Hollister and an 8-in. line in Santa Cruz. In the Marina district in San Francisco, about 10 miles of gas lines were replaced after the earthquake. These gas lines were made of iron and were very brittle. New gas lines conformed to ASTM 2513 specifications (flexible plastic pipes) [7,8].

Post-earthquake fires were limited but there was one spectacular fire in the Marina district in San Francisco and another in Watsonville. Both were caused by broken gas lines.

Communication systems and computer networks were also affected. As in most earthquakes, an increase in telephone traffic after the event overloaded the system. Service announcements on the radio on the night after the earthquake requested that only emergency calls be made. It probably resulted in the system's overall good performance. Long distance calls were delayed to give first priority to the local calls.

Many industrial facilities reported damage and they included storage facilities, tanks, food processing factories and wineries. The "Silicon Valley" electronic and computer industry survived to a great extent and reported limited architectural damage and damage to equipment.

Schools, colleges and universities escaped major structural damage. Minor structural damage and architectural damage was reported at Stanford University in Palo Alto.

1.4 NOTE ON EMERGENCY PREPAREDNESS

California has made extensive preparations for post-disaster emergencies, and these preparations have paid off very well in real situations. Most California cities and communities have annual drills for earthquake emergencies. These drills are costly and inconvenient but their value was demonstrated. Shortly after the earthquake, emergency control centers were set up at the major disaster areas and were staffed by police, firefighters, rescue workers, building inspectors and authorized volunteers. As one example, and as part of the emergency plan, volunteers from as far away as Los Angeles were on their way to the San Francisco Bay area within minutes after the earthquake.

In the epicentral area, tent shelters were erected in public parks and food was provided by the Red Cross and other organizations. The Federal Management Emergency Agency provided immediate funds. Structural engineers who had previously been trained in earthquake damage inspection categorized buildings into 3 groups: safe, unsafe, and limited access. Access was completely denied to "unsafe" buildings; entry into buildings declared "limited access" was permitted under supervision and for short periods only.

Communication emergency systems performed well. The use of cellular phones was singled out as the best performer at the emergency control centers located in specific disaster areas. CB radios were completely jammed because of the overwhelming usage after the earthquake. Other telephone lines were in operation except those that went through sophisticated private switching units. Some switching units failed because of power outages.

REFERENCES

[1] A. Astanech, V.V. Bertero, B.A. Bolt, S.A. Mahin, J.P. Moehle, R.B. Seed - "Preliminary Report on the Seismological and Engineering Aspects of the October 17, 1989 Santa Cruz (Loma Prieta) Earthquake," Report No. UCB/EERC-89/14, Earthquake Engineering Research Center, College of Engineering, University of California at Berkeley, October 1989.

[2] G. Plafker, J.P. Galloway (editors) - "Lessons Learned from the Loma Prieta, California, Earthquake of October 17, 1989," U.S. Department of Interior, Geological Survey Circular 1045, November 1989.

[3] October 17, Loma Prieta Earthquake, Preliminary Observations, U.S. Department of the Interior, Geological Survey, Western Region, Menlo Park, California 94025, Public Affairs Office Release of October 21, 1989.

[4] October 29, The Loma Prieta, California, Earthquake of October 17, 1989, Aftershock Sequence Observations and Forecast, U.S. Department of Interior, Geological Survey, Western Region, Menlo Park, California 94025, Public Affairs Office Release of October 29, 1989.

[5] A Shakal, M. Huang, M. Reichle, C. Ventura, T. Cao, R. Sherbune, M. Savage, R. Darragh, C. Petersen - "CSMIP Strong-Motion Records from the Santa Cruz Mountains (Loma Prieta), California Earthquake of 17 October 1989," Report No. OSMS 89-06, California Strong Motion Instrumentation Program, California Department of Conservation, Division of Mines and Geology, Office of Strong Motion Studies, Sacramento, California, November 1989.

[6] R. Maley, A. Acosta, F. Ellis, E. Etheredge, L. Foote, D. Johnson, R. Porcella, M. Salsman, J. Switzer - "U.S. Geological Survey Strong-Motion Records from the Northern California (Loma Prieta) Earthquake of October 17, 1989," U.S. Geological Survey Open-File Report 89-568, October 1989.

[7] Lifeline Response to Loma Prieta Earthquake, ASCE News, December 1989, Vol. 14, No. 12, p. 12.

[8] The October 17, 1989 Loma Prieta Earthquake, EQE Quick Report, San Francisco, California, October 1989.

TABLE 1-1

Measured values of peak ground accelerations from the Loma Prieta Earthquake [5,6]

Station	Estimated Distance from Epicentre (km)	Peak Acceleration, %g	
		Horizontal	Vertical
Corralitos (landslide deposits)	5	0.64	0.47
Watsonville (fill on alluvium)	11	0.39	0.66
Capitola	14	0.54	0.60
Santa Cruz	23	0.47	0.40
San Juan Bautista - 101 Overpass	25	0.15	0.10
Anderson Dam	27	0.39	0.19
San Jose Interchange 101/280/680	34	0.18	0.08
Halls Valley	38	0.13	0.06
Hollister Airport	45	0.29	0.16
Monterey - City Hall	46	0.07	0.03
Palo Alto, VA Hospital, Bldg 2	47	0.38	0.20
Palo Alto - 2-storey Office Bldg	57	0.21	0.09
Fremont - Mission San Jose	60	0.13	0.09
Calaveras Array Sunol Fire Station	63	0.10	0.03
Redwood City Canada College Campus Bldg	65	0.09	0.04
Foster City Menhaden Court	66	0.12	0.09
Livermore, VA Hospital Bldg. 62	67	0.06	0.03
Upper Crystal Springs Res.	70	0.16	0.06
Hayward City Hall	74	0.06	0.03
Hayward Muir School	77	0.18	0.10
Bear Valley Station 10 Webb Residence	86	0.13	0.05
San Bruno - 6-storey office Bldg	89	0.03	0.02
San Francisco Thornton Hall	03	0.14	0.04
San Francisco 575 Market St.	96	0.11	0.06
Oakland - 2-storey building	99	0.26	0.16
San Francisco - Rincon Hill	102	0.09	0.03
San Francisco - Pacific Heights	104	0.06	0.03
San Francisco - Telegraph Hill	104	0.08	0.03
San Francisco - Presidio	105	0.21	0.06
San Francisco - Cliff House	107	0.11	0.06
Martinez, VA Hospital	109	0.07	0.03
Larkspur, Ferry Terminal	115	0.14	0.06

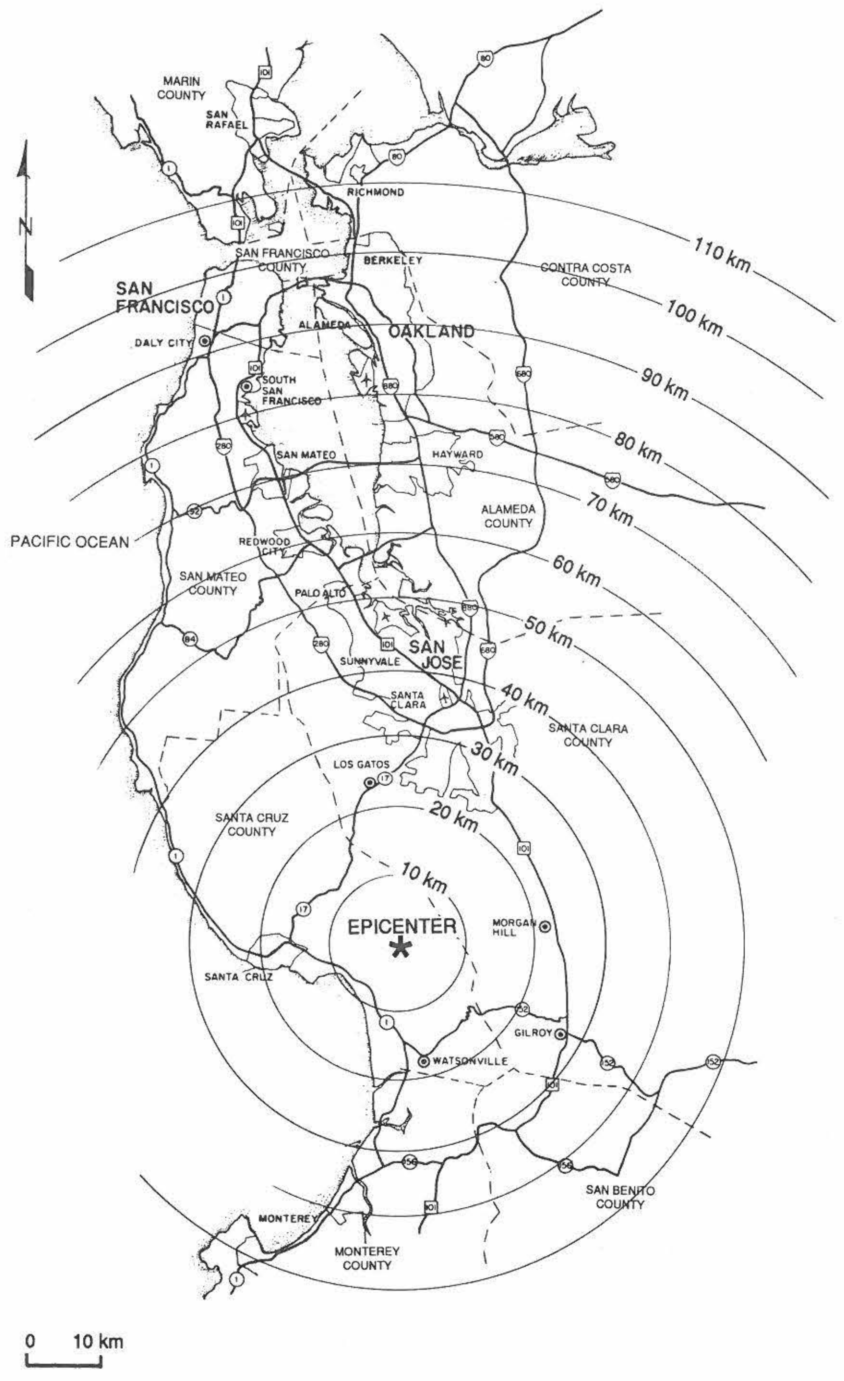


Fig. 1.1 The geographical areas affected by the Loma Prieta Earthquake

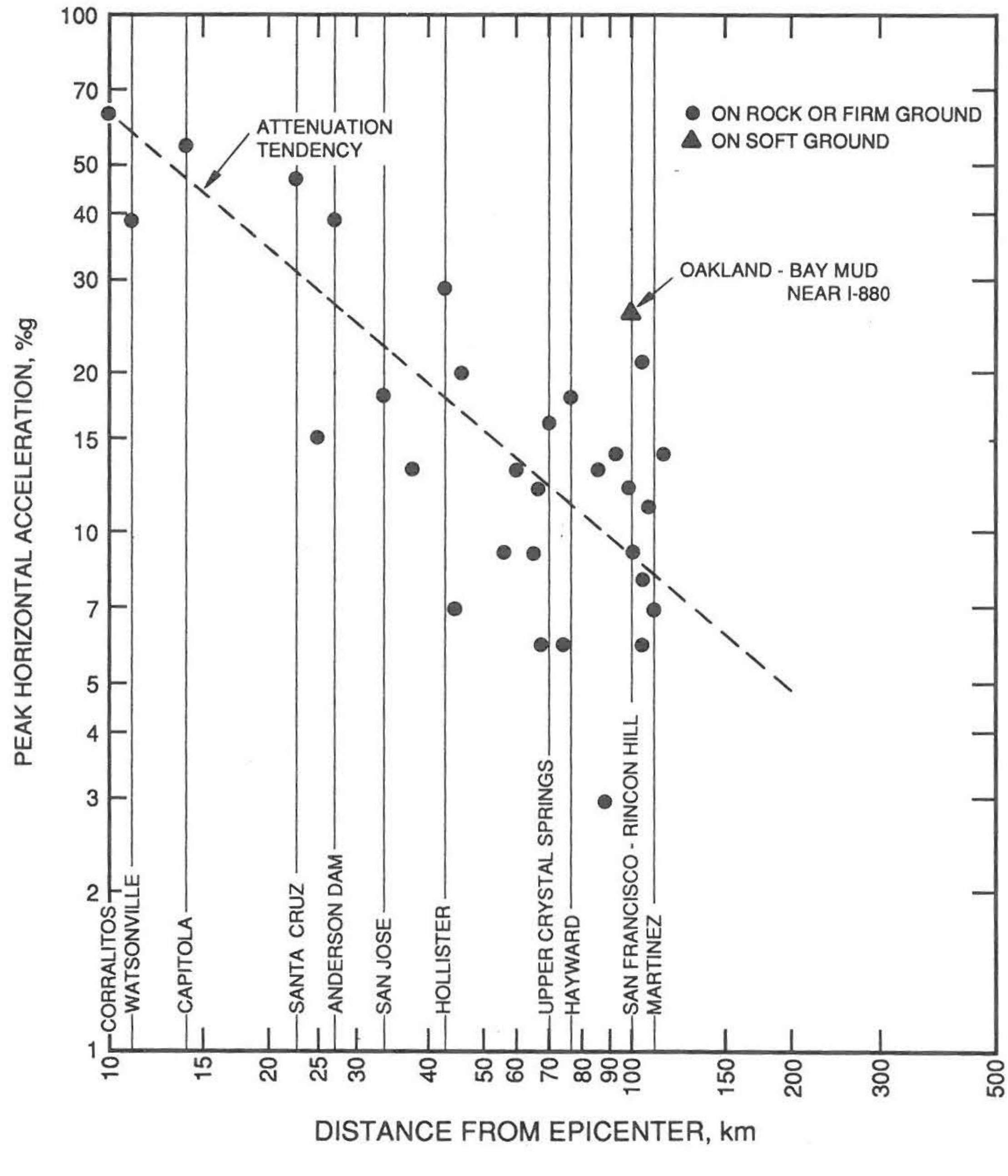


Fig. 1.2 Measured peak horizontal acceleration vs epicentral distance, Loma Prieta Earthquake

Chapter 2

GEOTECHNICAL ASPECTS

by

K.T. Law

2.1 INTRODUCTION

The Loma Prieta Earthquake of October 17, 1989 underlines the fact that the adverse soil conditions and inadequate structural integrity in buildings are the two most important contributors to seismic hazards. In this chapter, the role played by soil conditions are reviewed under the following headings: 1) amplification of motions, 2) liquefaction failure, and 3) other ground problems.

2.2 AMPLIFICATION OF MOTIONS

Widespread earthquake damage to structures and buildings is generally a direct result of the intensity and type of ground shaking. Local ground conditions can change the characteristics of earthquake motions that exist at the bedrock. In particular, thick deposits of compressible soils can raise or amplify the intensity of motions in a certain frequency range leading to severe damage. The amplification of earthquake motions is a complex process and is dependent on soil properties, thickness, frequency content of motions and local geological settings. For a given earthquake and geological setting, the amplification increases with the increase of soil compressibility and with soil thickness. Soil compressibility is sometimes expressed in terms of the shear wave velocity. The lower the shear wave velocity, the higher is the compressibility.

In the San Francisco Bay region three types of soil deposits are common: fills, Bay mud and alluvium. Fills are man-made materials normally loose in nature and much thinner than the natural deposits of Bay mud and alluvium. Fills are found around the waterfront of San Francisco and Oakland. For example, the Marina District, located on the northern coast of the City of San Francisco, is situated on top of a hydraulically placed fill underlain by a thick layer of Bay mud. The fill is very loose, uniform sand with sea shells. The Bay mud consists mostly of recent deposits (8,000 years and younger) of soft plastic carbonaceous clay, silt and minor sand inclusions [1]. It is loose, with a moisture content normally exceeding 50% and may be as thick as 40 m. Shear wave velocities in this deposit range from 90 to 130 m/s. The alluvium, with thickness reaching 600 m, corresponds to an older Bay sediment unit. It consists mostly of silty clay, silty clayey sand, sand and gravel. Generally the moisture content is less than 40%. The shear wave velocities in this deposit increase with depth and at the surface the value is about 200 m/s. Since soil compressibility is inversely proportional to the shear wave velocity, amplification of motions is highest for thick deposits of Bay mud. During an aftershock of magnitude of 4.8, Professor Shah of Stanford University recorded that the acceleration at a certain frequency at the surface of a thick deposit of Bay mud was 20 times higher than that of a rock site equidistant from the epicentre [2].

Structures founded on these compressible deposits have been subject to high horizontal excitation during the earthquake. The major damages in the cities of San Francisco and Oakland, about 100 km from the epicentre, were located on these deposits. Measurements showed that these deposits amplified the maximum peak horizontal accelerations by a factor of two to three. Many residential houses in the Marina District sustained severe damage and some even collapsed mainly because of the high amplification due to the Bay mud and localized liquefaction of the hydraulic fill. Houses similar to the collapsed ones just outside the Marina District suffered from slight damage because of lower excitation. The pier supporting the collapsed section of the Bay Bridge and the collapsed Cypress section of the Nimitz Freeway (Fig. 2.1) were also founded on the Bay mud, while the other non-collapsed section was founded on the alluvium. This observation supports the contention that Bay mud yields a higher amplification than the alluvium. Structural damages were found in a number of multi-storey steel frame buildings and reinforced concrete buildings in downtown Oakland where alluvium prevails. In downtown Santa Cruz where land was reclaimed with man-made fills, 85% of of the unreinforced masonry buildings were damaged.

2.3 LIQUEFACTION FAILURE

Soil liquefaction is the phenomenon in which loose saturated granular deposit such as sand and silt is being transformed to a liquid as a result of earthquake shaking or by other dynamic disturbance. During this earthquake soil liquefaction failures were found over an extensive area ranging from very near the epicentre to more than 100 km away as shown in Fig. 2.2 [3]. Many of these areas are known to have experienced similar failures in the 1906 San Francisco earthquake [4]. This observation is consistent with the Chinese and Japanese experience and supports the notion that a loose deposit liquefied during an earthquake does not necessarily improve its dynamic strength and may liquefy again in subsequent earthquakes.

Liquefaction caused damage in the form of sand boils, ground cracks, ground heave, building collapse and differential settlement. Some examples are shown in Figs. 2.3 to 2.6. Near the epicentre such as Santa Cruz and Moss Landing, numerous sand boils were found. These sand boils had a typical conical shape with a crater in the centre. Outside Watsonville along the Pajaro River, 1.5 km of levee was damaged by soil liquefaction.

In the Marina District, liquefaction failure was widespread. More than 20 sand boils were noted by the visiting team. These sand boils did not possess the typical conical shape with a crater in the middle. Instead, they were simply patches of sand lying on the lawn (Fig. 2.4) or on the pavement. The sand was dark grey in colour, uniformly graded with occasional sea shells indicating that the hydraulic fill placed on the site had liquefied during the earthquake. Soil liquefaction resulted in the collapse of a three storey house and damage to a number of other houses by differential settlement (Fig. 2.5). Many sidewalks were buckled (Fig. 2.6) and ground cracks were found in several places. The large ground movements wrecked buried utilities including gas lines that led to a spectacular fire. The whole area was evacuated and public access was restricted.

An attempt is made here to assess the liquefaction potential under the Loma Prieta earthquake using a recently developed method by Law et al [5]. This method is based on the total seismic energy travelling through and dissipated in a soil media. As such, this method more adequately accounts for the complete spectrum of ground motions that give rise to liquefaction failure. For clean sand, this method can be reduced to the following condition for a liquefaction failure to occur:

$$\frac{T(M,R)}{\eta_l(N_1)} \geq 1.0$$

where

$$T(M, R) = 10^{1.5M} / R^{4.3}$$

M = magnitude of earthquake on the Richter scale

R = hypocentral distance in km

$$\eta_l(N_1) = 2.28 \times N_1^{11.5} \times 10^{-10}$$

and N_1 = Standard Penetration Resistance after correction to a vertical effective pressure of 100 kPa and energy efficiency of 60%.

This expression can be applied in different ways. One way is by substituting the earthquake magnitude, M, and the hypocentral distance from a site, R, into the expression to obtain the Standard Penetration Resistance, N_1, below which liquefaction will occur. The results of such an application to the San Francisco area is represented by the contour lines of N_1 shown in Fig. 2.2. Take Marina district as an example. The condition of liquefaction failure is when N_1 is less than 10. Such a value is not surprising for the liquefied material there, which, as mentioned earlier, is composed of the hydraulic fill.

2.4 OTHER GROUND PROBLEMS

Loose granular deposits, both saturated or unsaturated, may densify leading to considerable settlement even without the phenomenon of liquefaction. A good example is found at Embarcadero Freeway on the northeastern coast of San Francisco. Here fills were placed on top of Bay mud. The structure of the freeway was damaged to the point of near collapse again due to ground motion amplification. The paved ground surface at one location showed a differential settlement of about 15 cm because of densification of the fill (Fig. 2.7).

A large number of landslides and rockfalls were reported in the Santa Cruz Mountains near the fault rupture zone. Many of these landslides (Fig. 2.8) were partly caused by rain that came after the earthquake. Highway 17, one of the two main highways from the north to Santa Cruz, was partially closed. A number of single family houses were destroyed.

Signs of distress in a number of dams were reported. The Lexington earthdam, the abutment of the Elsman dam and about 1.5 km of the San Lorenzo levee in Santa Cruz suffered from cracks.

REFERENCES

[1] R.D. Borcherdt - "Study for seismic zonation of the San Francisco Bay Region." U.S. Geological Survey Professional Paper 941-A, 1975.

[2] S.C. Shah - Personal communication, 1989.

[3] A. Astaneh, V.V. Bertero, B.A. Bolt, S.A. Mahin, J.P. Moehle, R.B. Seed - "Preliminary Report on the Seismological and Engineering Aspects of the October 17, 1989 Santa Cruz (Loma Prieta) Earthquake," Report No. UCB/EERC-89/14, Earthquake Engineering Research Center, College of Engineering, University of California at Berkeley, October 1989.

[4] G. Plafker and J.P. Galloway - "Lessons learned from the Loma Prieta, California, Earthquake." U.S. Geological Survey Circular 1045, 1989.

[5] K.T. Law, Y.L. Cao, and G.N. He - "An energy approach for assessing seismic liquefaction potential." Canadian Geotechnical Journal, Vol. 26, No. 3, 1990, pp. 320-329.

Fig. 2.1 The collapse of the Nimitz Freeway in Oakland

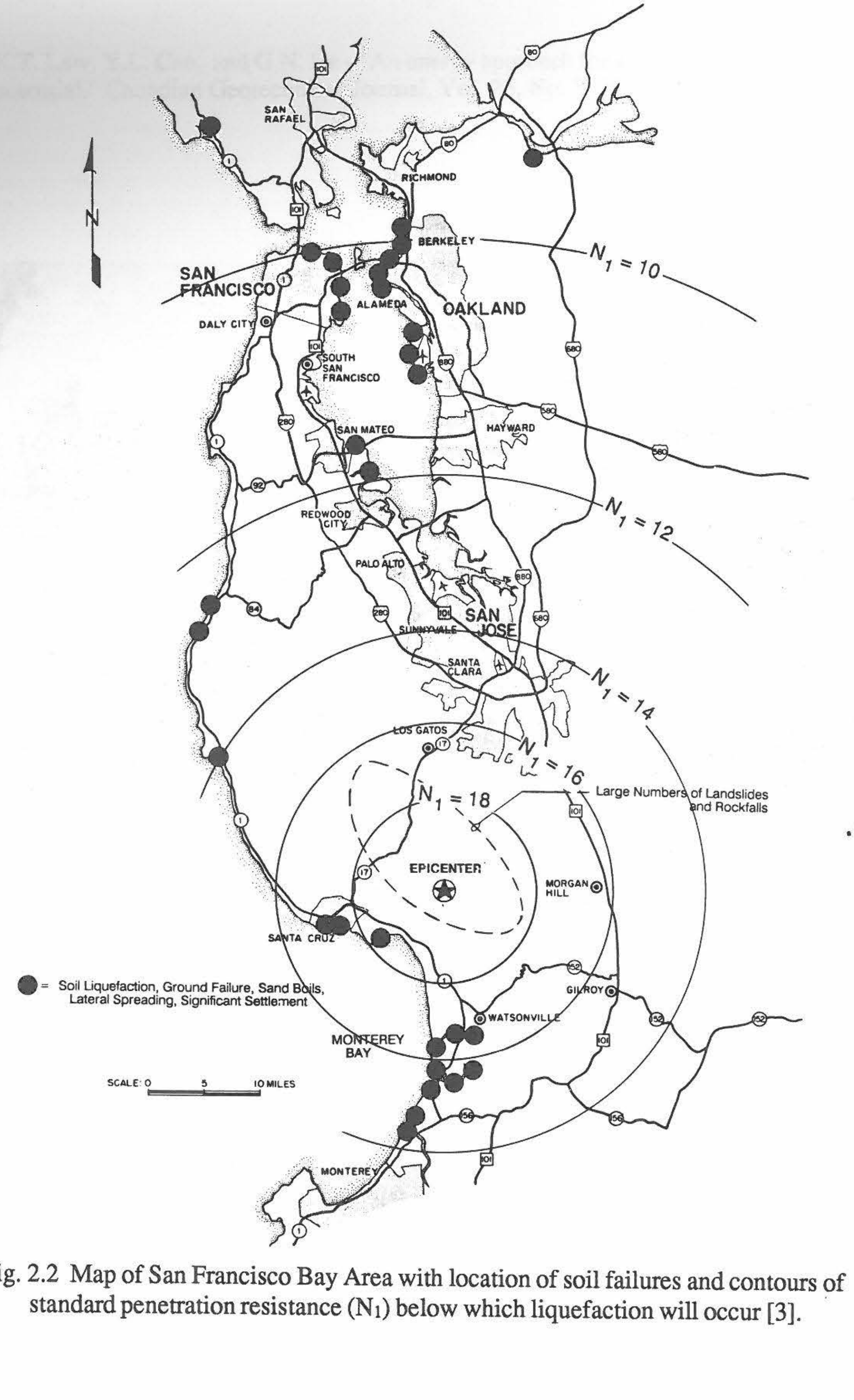


Fig. 2.2 Map of San Francisco Bay Area with location of soil failures and contours of standard penetration resistance (N_1) below which liquefaction will occur [3].

Fig. 2.3 Partially collapsed house due to liquefaction failure in Aptos, 9.5 km from epicenter

Fig. 2.4 Sand boil in Marina District, San Francisco

Fig. 2.5 House damaged by differential settlement due to soil liquefaction in Marina District, San Francisco

Fig. 2.6 Buckled sidewalk due to soil liquefaction in Marina District

Fig. 2.7 15-cm differential settlement under the Embarcadero Freeway

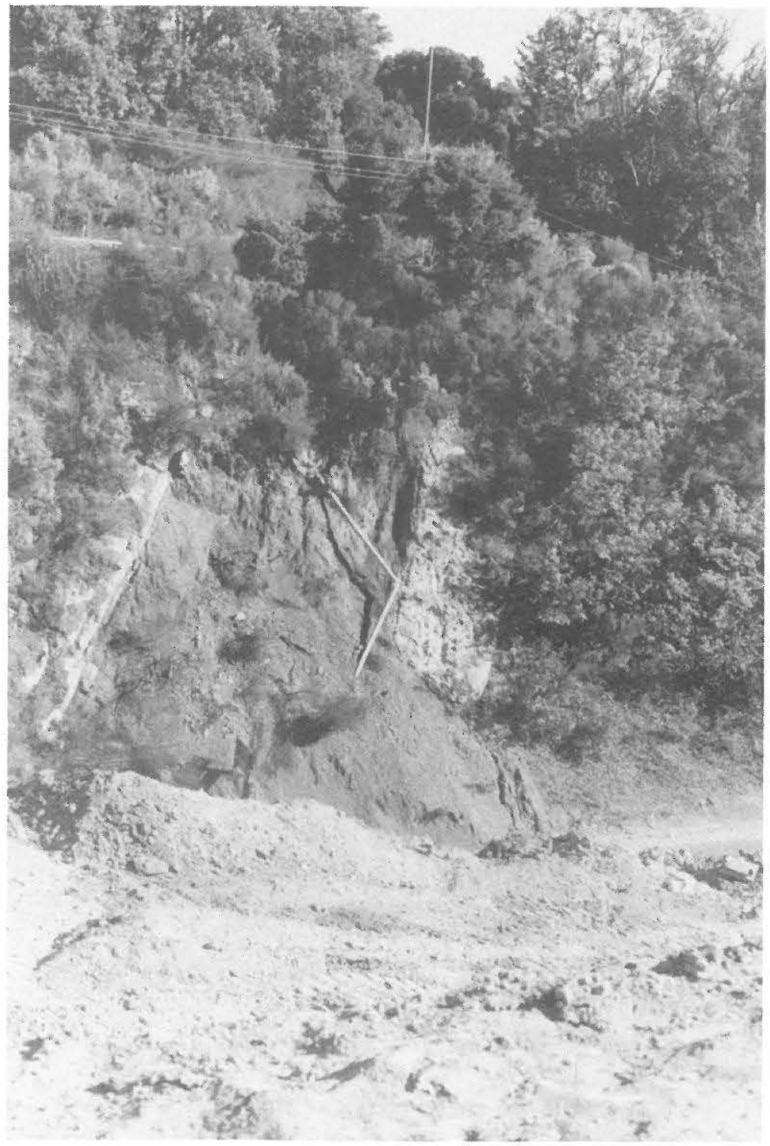


Fig. 2.8 Landslide on Highway 17

Chapter 3

PERFORMANCE OF BUILDINGS

by

A.M. Jablonski

3.1 INTRODUCTION

The impact of major earthquakes is usually evaluated based on the data analyzed from past events. Generally, the widespread damage to buildings situated as far as San Francisco or Oakland, about 100 km from the epicenter, is quite unusual for an earthquake of magnitude 7.1. The Loma Prieta earthquake provided an extraordinary opportunity to test the behaviour of various types of building structures, from single-family houses and medium-size buildings in San Francisco and Santa Cruz to high-buildings in San Francisco and Oakland.

In general, the performance of building structures during an earthquake depends on a large number of factors including type of structure, year of construction, lateral resistance and local ground effects like soil liquefaction, densification of soil, differential settlement, lateral spread and uplift. The actual fault rupture zone is usually the most prone to structural damage. During the site visit, which had started about 8 days after main shock, only a visual assessment of damage was carried out. Careful examination of building interiors was not possible. Many affected areas were visited in the following cities and communities: San Francisco, Oakland, Emeryville, Berkeley, Los Gatos, Santa Cruz, Watsonville, Palo Alto, Menlo Park, Los Altos Hills, San Juan Bautista and San Martin.

This chapter covers briefly the performance of various types of buildings in the earthquake-affected areas. Reference is made to epicentral distance and closest recorded peak horizontal and vertical accelerations [1,2]. Structural damage was widespread and often of a complicated nature. Aftershocks had augmented damage in several buildings. It had not been possible to visit all affected areas, but a brief description of representative damage patterns is presented below.

3.2 WOOD-FRAME HOUSING

Two types of wood-frame houses sustained heavy damage. The first type could be classified as single-family dwellings of various forms. They are located in the epicentral area (of approximately 50-km radius). The second type is associated with 3-4 storey wood-frame apartment houses and also 2-3 storey townhouses in the Marina District in San Francisco about 100 km from the epicentre.

Epicentral Area

"Cripple" foundation walls, also called "pony" walls, failed in many old wood-frame single-family houses in the epicentral area (e.g., in Watsonville and in Los Gatos) causing serious damage. Cripple walls are stud type walls which connect the concrete or masonry foundations and the first floor framing. They are a typical feature in older buildings of 50 years or more in California. They are usually short (less than 14" (35.6 cm) high), but in some recently built modern wood houses could reach a height of one full storey. In many instances, the stud walls appeared not to have enough lateral resistance to withstand severe shaking. They were also poorly anchored to the foundations and sometimes were simply not connected at all.

Improper bracing (sheathing with plywood) and inadequate or missing connections to the foundations as well as to the first floor framing had caused them to be moved laterally off their foundations. In some cases, the cripple walls were lying flat on their side. The building frame lost its integrity and the house became unsafe for occupants. Examples of the cripple wall failure are presented in Figs. 3.1, 3.2, 3.3 and 3.4. These houses are in the city of Watsonville and about 12 km from the epicenter. This area experienced high frequency shaking with the horizontal acceleration reaching 0.33 g and the vertical one 0.66 g. The duration of strong shaking was approximately 16 seconds. A residence, approximately 60 years old, at Brennan Street in Watsonville is shown in Fig. 3.1. Failing cripple walls shifted the overall wood-frame structure off the foundations and caused partial loss of integrity. Although these walls were sometimes sheathed or sealed with horizontal boards, nailing was sparser than required by the present code. The panels could not provide the lateral resistance to prevent rocking and in some instances this led to the complete failure of cripple stud walls. Figure 3.2 presents this type of complete failure and also a destroyed entrance porch in an old residence at 130 Brennan Street in Watsonville. Another cripple wall failure is shown in Fig. 3.3 where the house at 113 Sudden Street in Watsonville was shifted by more than 1 ft (32 cm). Also, large openings (e.g., porches) collapsed due to lack of lateral resistance. Some examples of collapsed porches are shown in Figs. 3.4, 3.5 and 3.7. Figure 3.6 also shows a collapse of the cripple wall in the house as well as of the porch. The small staircase leading to the porch was destroyed and later removed. This is an old residence with a cripple wall on its perimeter. The cripple wall was moved about 18 in. (45 cm) laterally due to lack of anchorage to the foundation (Fig. 3.6). The collapsing porch also caused extensive damage to the ceilings (Figs. 3.5 and 3.7).

Modern wood houses with and without cripple walls performed well unless they were situated on ground fissures [3]. There was also no damage reported in the modern residences located on long posts on the sides of hills. Most of these buildings were built according to recent codes. However, some houses with large openings like garage doors situated under the living area or with other irregularities sustained substantial damage. Figures 3.8 and 3.9 present a modern residence in Los Altos Hills, near Palo Alto, about 50 km from the epicenter and where the horizontal acceleration reached about 0.38 g. The large opening of the garage doors under the living space created lack of lateral resistance. In some cases the perimeter band joist was anchored to the foundation using hold-down anchors with lag screws into the band joist. The lag screws failed in withdrawal from the joist due to severe shaking [3]. In general poor connection between elements and/or lack of continuity between parts of the building were the prime reasons for damage in wood-frame houses.

Marina District, San Francisco

The largest area with substantial structural damage outside the epicentral region was the Marina District in San Francisco. This area, along with some portions of downtown around the Market Street and also in the neighborhood of Mission Street, are situated on deep soft deposits of fill and Bay mud. The layers of sand liquefied during severe shaking, resulting in upheaving of streets and sidewalks and settlements. The shaking also amplified, as happened during the Mexico City earthquake of 1985. The highest horizontal acceleration, 0.21 g, was recorded in Presidio, on rock, a few blocks northwest from Marina and 105 km from the epicenter.

This part of the city was constructed in the early 1920's. Primarily, there are two main types of custom-made wood-frame houses: 2-3 storey townhouses with garages on the first level, and 3 -5 storey large apartment houses with the first floor occupied also by garages. There is no separation space between buildings. A large percentage of these buildings suffered during the Loma Prieta earthquake and experienced different levels of damage. In some cases, especially, 3-5 storey apartment houses partially or completely collapsed. A 4-storey wood-frame apartment building with its first floor completely levelled is shown in Fig. 3.10. The garage floors in these buildings had acted as a "soft storey" with no lateral resistance. These floors had only very limited or no bracing, or had sheathed walls constructed with horizontal boards nailed to posts. The exterior walls are made of stucco, fake stone, brick veneer or some combinations thereof. Stucco walls prevail. The posts had acted as double-hinged columns, which were unable to withstand the amount of shift caused by the amplified shaking. This led to the large deformations and collapse in some cases of one or two stories. The exterior walls also sustained heavy damage. Many 2- or 3-storey townhouses within blocks showed some damage over garage doors and in walls. Damage to interiors could not be assessed without entering a building.

Figures 3.11 - 3.14 show examples of the "soft storey" effect. An almost-collapsed 4-storey apartment building at the corner of Beach and Broderick Streets is shown in Fig. 3.11. The corner bracing is inadequate to prevent the shift of about 50 cm in one horizontal direction. Extensive shoring is required to prevent total collapse. A close-up of the first "soft storey" effect is presented in Fig. 3.12. Hinges were created and a very extensive racking took place. Another corner building with the first storey entirely taken by garages is shown in Fig. 3.13. A detail of the post connection to the second floor framing is visible in Fig. 3.14. On some buildings situated at the block corners, but with more sheathed walls (horizontal boards), the amount of damage was much smaller (Fig. 3.15) while at other locations end buildings suffered more damage than interior ones. In some cases brick walls collapsed onto sidewalks, creating some danger to pedestrians (Fig. 3.16).

Buildings (2-3 storey) within street blocks survived the earthquake with some minor structural and architectural damage as shown in Fig. 3.17. In general, upper floors sustained little or no structural damage although in the case of buildings at the corners, the entire ground floor was shifted.

3.3 UNREINFORCED MASONRY BUILDINGS

Many unreinforced masonry buildings suffered severe damage and in some instances complete collapse. They were examples of out-of plane collapse of masonry bearing walls (San Francisco, Watsonville and Santa Cruz). All types of masonry walls experienced damage including collapsed brick veneer (Fig. 3.16), bearing walls, parapets, and in-fill masonry walls. The most severely damaged masonry buildings were reported in Santa Cruz (the case of the Pacific Garden Mall in the center of the city), Los Gatos and in Watsonville. Old structures from the turn of the century suffered most. Examples of damage to unreinforced masonry buildings are presented in Figs. 3.18, 3.19 and 3.20.

In Fig. 3.18 is shown an old unreinforced masonry building from the end of last century which experienced substantial damage to its parapet walls and the upper portion of its bearing walls. Falling debris presented danger to pedestrians. The measured horizontal acceleration reached 0.39 g and the vertical acceleration was 0.66 g in the city of Watsonville. The heavily damaged unreinforced masonry buildings in downtown Watsonville were quickly demolished (Fig. 3.19). Another example of damage to the unreinforced masonry storage building in Oakland near the collapse of the Nimitz freeway is shown in Fig. 3.20. The heavy shaking at the roof level and the different type of response of the roof structure from the bearing walls resulted in damage to the walls. The part of the roof collapsed into the top floor. The nearby horizontal acceleration was in the range of 0.29 g as reported in Ref. 2.

3.4 HIGH-RISE BUILDINGS

Most of the high-rise buildings in downtown San Francisco and Oakland rode out the earthquake without serious damage to their structural frame or loss of functionality. However, there were some high-rise buildings in Oakland with more serious damage; the steel-frame building at Franklin and 18th Street, erected in the early 1960's, had significant shear cracking of the shear walls and some damage to first floor columns (Figs. 3.21 and 3.22). Also the City Hall building in Oakland, under heritage protection, was evacuated due to structural damage one week later.

Pounding of structures caused significant damage in several locations in San Francisco and Oakland. In Fig. 3.23 the effect of pounding between two adjacent structures built in the 1920's is shown.

Somewhat non-typical damage was observed in case of the Amfac Hotel at the San Francisco International Airport. A penthouse together with a heavy water tank fell and inflicted heavy damage to the surrounding reinforced concrete structure (Fig. 3.24).

3.5 COMMENTS ON RETROFITTED BUILDINGS

There were a number of retrofitted buildings which performed well at the various epicentral distances. Some examples are presented in Figs. 3.25, 3.26 and 3.27. The upgraded institutional stone masonry buildings at Stanford University in Palo Alto escaped with only minor damage in a few locations, in contrast with unreinforced masonry and old style reinforced-concrete buildings, which suffered damage estimated at $160 million.

An old building of mixed construction was retrofitted with use of the internal steel frame. This building is situated in South Market Street in San Francisco (Fig. 3.25). The U.S. Geological Survey buildings were upgraded after the 1971 San Fernando earthquake. Two types of different external steel frames were used to allow some transfer of an extensive horizontal force which may occur during an earthquake motion (Figs. 3.26 and 3.27). These structures were subjected to peak horizontal accelerations in the range of 0.21 g to 0.38 g, measured in nearby Menlo Park.

REFERENCES

[1] A. Shakal, M. Huang, M. Reichle, C. Ventura, T. Cao, R. Sherbune, M. Savage, R. Darragh, C. Petersen - "CSMIP Strong-Motion Records from the Santa Cruz Mountains (Loma Prieta), California Earthquake of 17 October 1989," Report No. OSMS 89-06, California Strong Motion Instrumentation Program, California Department of Conservation, Division of Mines and Geology, Office of Strong Motion Studies, Sacramento, California, November 1989.

[2] R. Moley, A. Acosta, F. Ellis, E. Etheredge, L. Foote, D. Johnson, R. Porcella, M. Salsman, J. Switzer - "U.S. Geological Survey Strong-Motion Records from the Northern California (Loma Prieta) Earthquake of October 17, 1989," U.S. Geological Survey Open-File Report 89-568, October 1989.

[3] W.A. Baher, D.H. Brown, J.R. Tissel - "Loma Prieta Earthquake (San Francisco Bay Area) October 17, 1989," APA Technical Report Number T89-28, Technical Services Division, American Plywood Association, Washington 98411, November 9, 1989.

Fig. 3.1 An old residence with cripple wall failure affecting integrity - 114 Brennan Street in Watsonville

Fig. 3.2 Complete collapse of cripple wall (approx. 2′ (64 cm) shift) and destroyed entrance porch due to foundation settlement - 130 Brennan Street in Watsonville

Fig. 3.3 The collapsed cripple wall shifted this house by more than 1′ (32 cm) at 119 Sudden Street in Watsonville

Fig. 3.4 Two houses on Sudden Street, one with cripple wall failure and collapsed entrance porch, and the other with no damage - City of Watsonville

Fig. 3.5 Collapsed cripple wall and porch in an old residence at Main Street in Los Gatos

Fig. 3.6 House shifted from the foundation by approximately 18″ (45 cm) and the small staircase to the porch destroyed and removed in a residence on Main Street in Los Gatos

Fig. 3.7 Collapsed porch in an old residence on Main Street in Los Gatos

Fig. 3.8 Damage to a modern wood-frame house in Los Altos Hills

Fig. 3.9 Lack of lateral resistance and creation of two hinges in garage posts contributed to damage (note destroyed brick veneer wall)

Fig. 3.10. A 4-storey wood-frame multi-apartment house reduced to 3-storey building in the Marina District, San Francisco

Fig. 3.11. Almost collapsed 4-storey wood-frame apartment building due to "soft storey" effect at the corner of Beach and Broderick, Marina District, San Francisco

Fig. 3.12. Close-up of the first "soft storey", Beach/Broderick Streets, Marina District, San Francisco

Fig. 3.13. Another example of "soft storey" effect on Jefferson St., Marina District, San Francisco (note horizontal boards on posts and over the garage doors)

Fig. 3.14. Inadequate connection detail between post and floor framing in the corner 4-storey apartment building in Marina District, San Francisco

Fig. 3.15. 4-storey apartment building with more sheathed walls (horizontal boards) experienced less damage and much less horizontal shift - Marina District, San Francisco

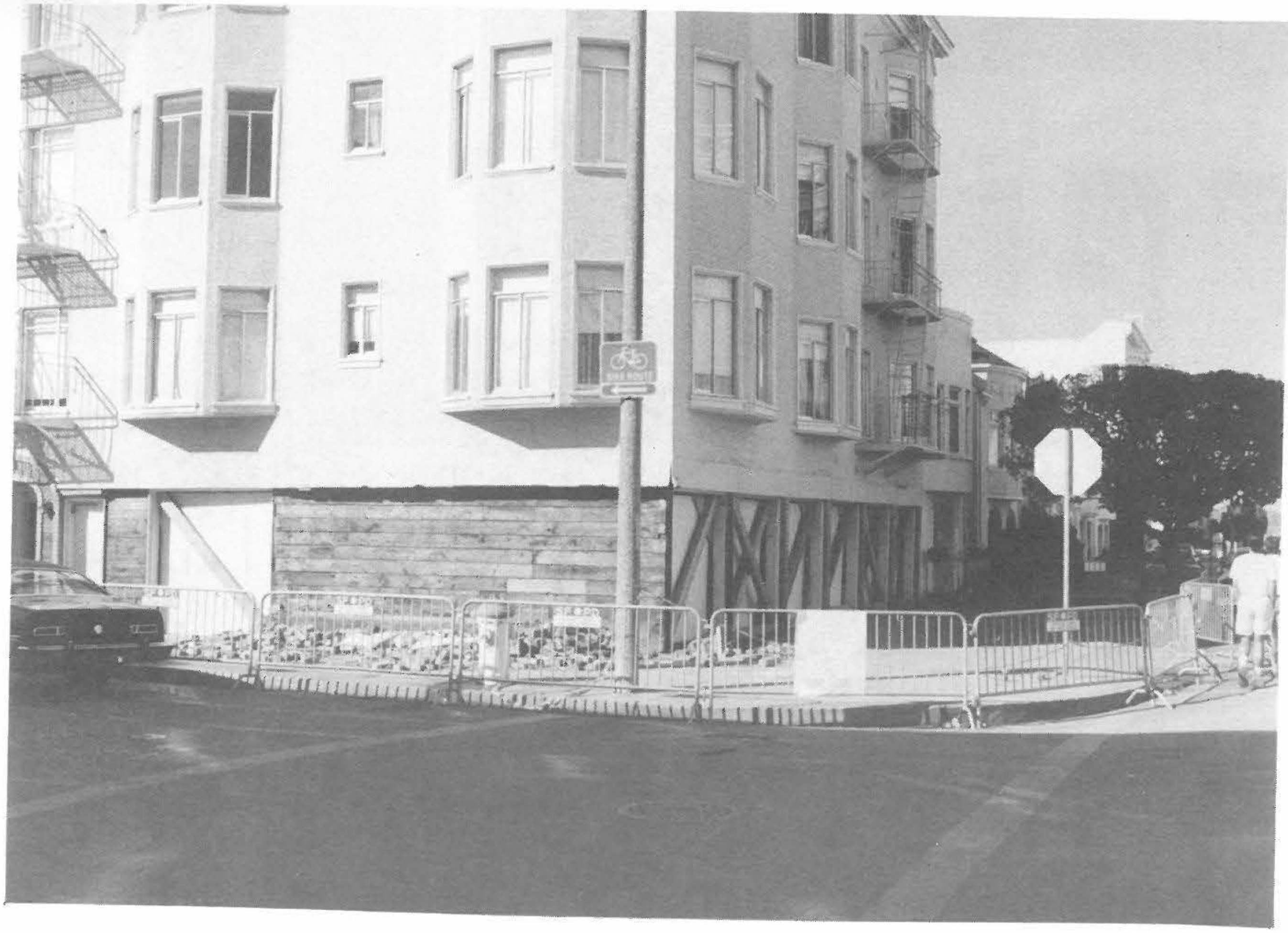


Fig. 3.16. 4-storey apartment building with destroyed brick wall - Marina District, San Francisco

Fig. 3.17. An example of 3-storey building covered with stucco which experienced minor damage over openings at the first floor level - Marina District, San Francisco

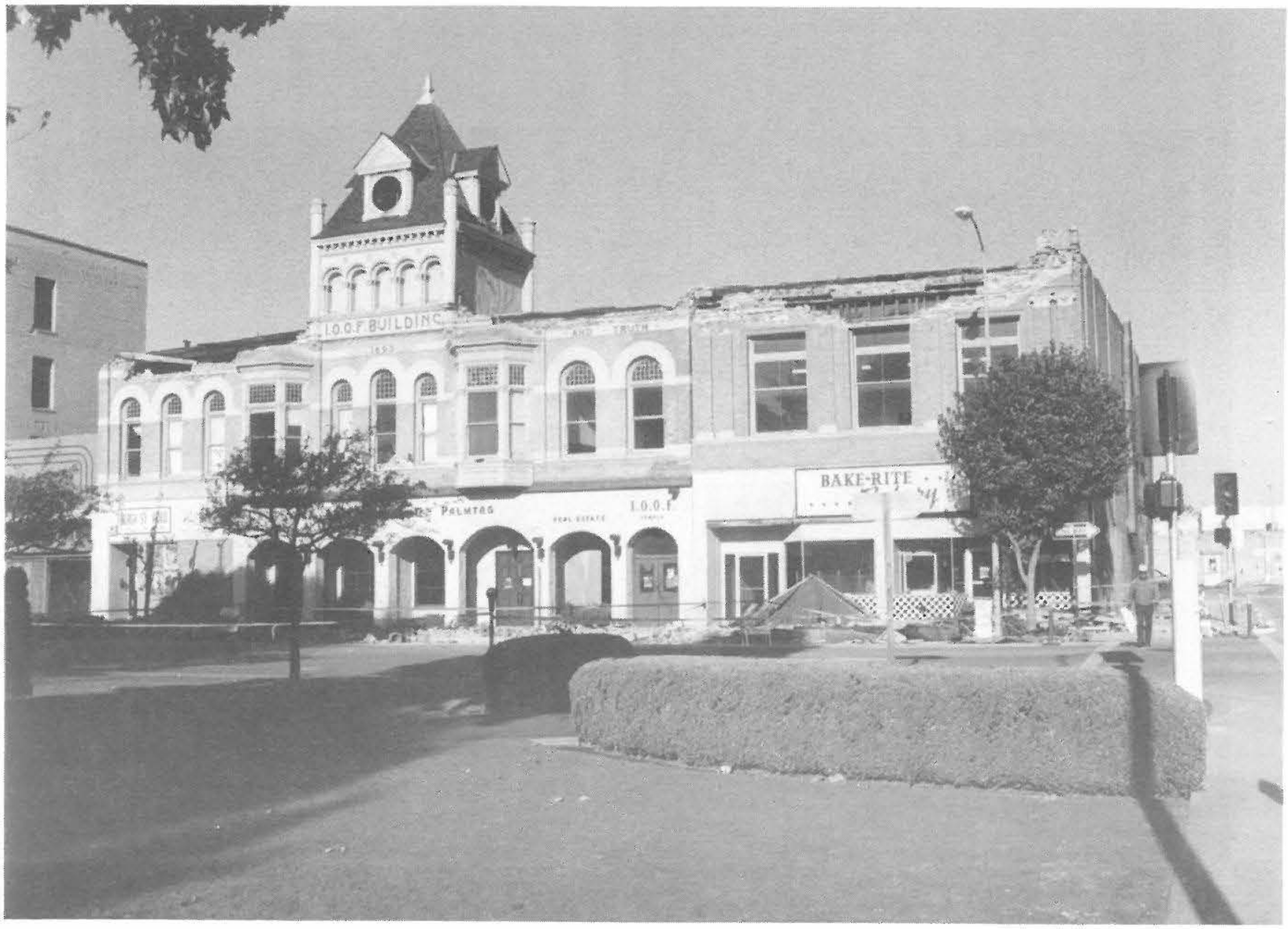


Fig. 3.18. Damage to an old unreinforced masonry building structure - I.O.O.F. Building built in 1893 on East Beach Street in Watsonville

Fig. 3.19. Demolishing process of the heavily damaged unreinforced masonry building on Main Street in downtown Watsonville

Fig. 3.20. A 4-storey storage building on Campbell Street in Oakland with damage to the parapet walls and collapsed roof

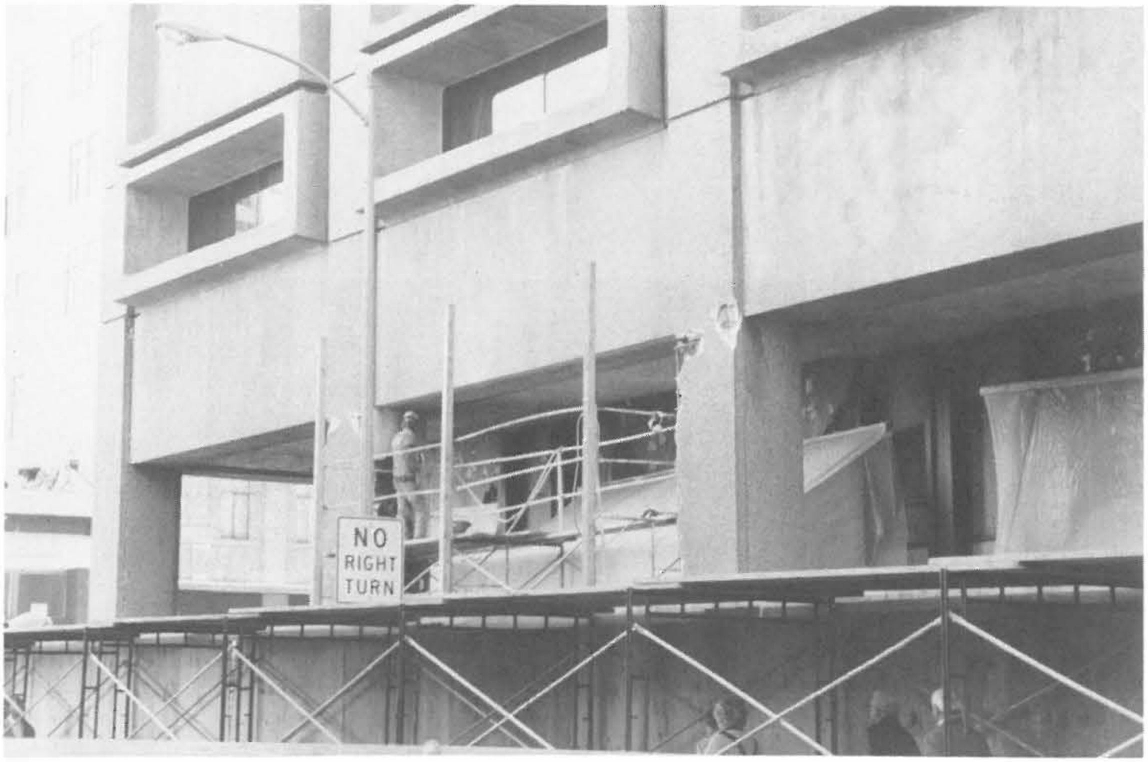


Fig. 3.21. Damage to first-level concrete columns and shear walls in high-rise building in downtown Oakland

Fig. 3.22. Damage to column in high-rise building in downtown Oakland

Fig. 3.23. An example of pounding between two adjacent buildings, Mission Street, San Francisco

Fig. 3.24. Collapsing water tank destroyed a penthouse at the top of Amfac Hotel at the San Francisco Airport

Fig. 3.25. An old structure of the corner building upgraded after 1971 San Fernando earthquake performed well

Fig. 3.26. Seismically upgraded concrete-frame building of the U.S. Geological Survey in Menlo Park, California

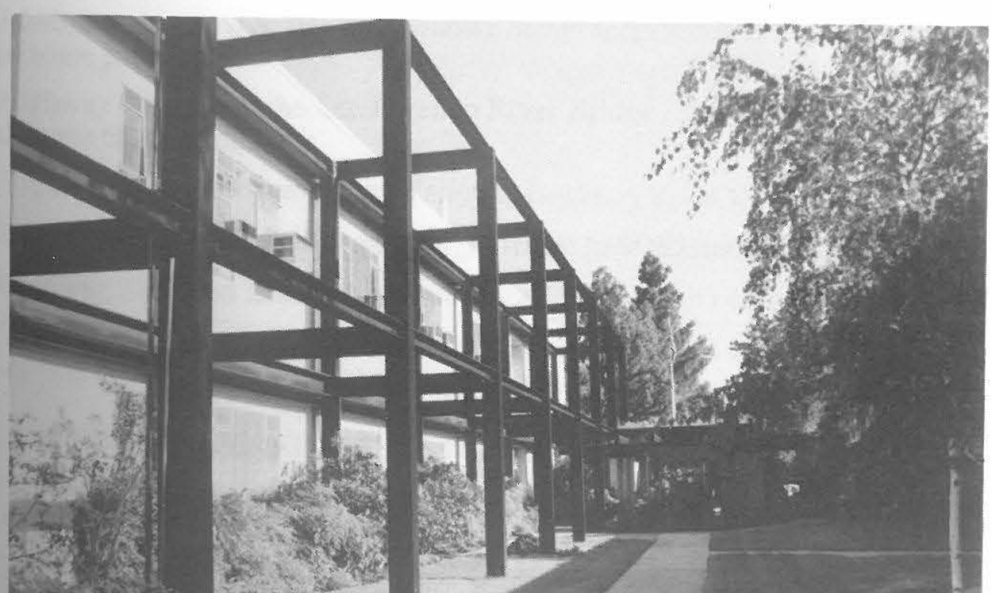


Fig. 3.27 Steel frame was used to strengthen another concrete-frame building with in-fill walls, U.S. Geological Survey, Menlo Park, California

well

Chapter 4

PERFORMANCE OF TRANSPORTATION STRUCTURES

by

D.T. Lau and A.M. Jablonski

4.1 INTRODUCTION

The Loma Prieta earthquake had a major effect on transportation routes in the San Francisco Bay Area. Several highways, overpasses and bridges were damaged and subsequently closed for a short time, with some probably for more than a year. The strong shaking associated with this quake was about 10 seconds. Had it lasted much longer, damage to transportation structures would have been much more severe. During this site visit only a few damaged transportation structures were visually assessed and photographic documentation gathered. Figure 4.1 presents a summary of affected transportation routes including the most severely damaged structures like the I-880 viaduct in Oakland, the Oakland Bay Bridge and the Struve Slough Bridge on Highway 1 near Watsonville. The following road closures were still listed several days after the earthquake during our site visit:

- I-80 and I-880 closure between Berkeley and I-980 in Oakland due to collapse of Cypress viaduct of I-880.
- The Oakland Bay Bridge closure owing to collapse of a road section over one of its piers (closed for one month).
- The Embarcadero Freeway in San Francisco closed due to earthquake damage.
- I-280 closed from U.S. 101 to downtown San Francisco due to severely damaged columns in some places.
- Highway 9 closed at the San Lorenzo River Bridge (near Santa Cruz) due to structural damage to the bridge.
- Highway 17 closed from Scotts Valley to Highway 9 due to landslides.
- Highway 1 closed at the Struve Slough Bridge near Watsonville due to collapsed bridge.
- Highway 25 closed from U.S. 101 to about 20 km south of Hollister due to roadslides.

This chapter covers the performance of transportation structures visited including some secondary effects for the case of small bridges in Santa Cruz.

4.2 COLLAPSE OF INTERSTATE I-880 CYPRESS STREET VIADUCT

For the October 17 earthquake, the single most devastating damage occurred along the Interstate I-880 freeway in Oakland. More than a mile of the double deck elevated roadway collapsed during the earthquake, causing 49 deaths and many more injuries.

The collapsed freeway is located a short distance south of the east side entrance intersection of the San Francisco - Oakland Bay Bridge, approximately 100 km from the epicenter. The double deck freeway is 3 km long and is bounded by 7th Street on the south and 34th Street on the north in Oakland. The location of the collapsed section of I-880 is indicated in Fig. 4.1.

Two acceleration records of the ground motion in the vicinity of the site were obtained. The station located 2 km north in Emeryville recorded a peak horizontal ground acceleration of 0.26 g [1], whereas the other record obtained at the Oakland Outer Harbor Wharf, 2 km to the west, had a peak horizontal acceleration of 0.29 g [2]. It is believed that the Cypress structure was subjected to similar intensity ground shaking during the earthquake.

Before the damage, the Cypress Viaduct was a major truck traffic artery in the East Bay, with a daily traffic volume of more than 195,000 vehicles. The freeway structure was designed in 1954 according to the American Association of State Highway Officials (AASHO) Standard Specification for Highway Bridges applicable at the time. At the time of its completion in 1957, it was one of the first continuous double deck freeways in the U.S. The following is a brief description of the Cypress Viaduct structure.

Both the lower and upper decks are approximately 53 ft wide, allowing four lanes of traffic on each level. The north bound traffic traveled on the lower deck. The deck is a 7-cell reinforced concrete box girder structure supported by reinforced concrete girders, with a typical span of about 80 ft. in the longitudinal direction. The elevations of typical bents showing the structural configurations are presented in Fig. 4.2. The lower columns have uniform cross-section, while the upper columns taper off slightly at the lower ends.

There are two common types of design layouts, different mainly in the placements of the pin connections in the upper frame structure [3]. The first layout (Fig. 4.2 (a)) has hinge connections at the bottom of the upper columns where the upper frame is connected to the lower frame structure, whereas the joints at the tops of the upper columns to the upper girder are capable of resisting moment. This design layout of the upper structure has only one degree of redundancy. In the second layout shown in Fig. 4.2 (b) the upper structure has three hinge joints and only one moment connection at the lower end of the east side column, rendering the upper frame a statically determinate structure. The lateral load resisting capability of both design layouts is rather limited, and is concentrated to only one or two moment resisting connections. According to the designer of the viaduct, the flexible joints were put in place because engineers at the time could not precisely predict the stresses that heavy trucks and temperature variations might impose on the structure. The joints provided them with a margin of error to make sure the columns would not fail if the freeway was subjected to unusual load distribution [4]. Moreover, the seismic force induced by the ground motion during an earthquake was not as well understood at the time. It is now suspected that the design layout is one of the major contributing factors to the failure of the roadway.

As the first phase of a retrofit program intended to strengthen the highway structure for better seismic resistance, the longitudinal box girder decks were tied together by U-shaped cables at the expansion joints in 1982. The retrofit program was initiated earlier as a consequence of the lessons learned from the experience of the 1971 San Fernando Earthquake in Southern California, during which numerous highway structures were damaged. The planned second phase of the program concerned strengthening of the columns and the joint regions, and was scheduled to be carried out in 1992.

The observed damage of the Cypress Viaduct is described next. A photo of the collapsed freeway is presented in Fig. 4.3. For a total distance of 1.2 km, the columns of the upper bent structures failed, causing the collapse of more than 50 spans of the upper deck; they dropped vertically onto the lower deck. In almost all cases, the lower bent structures and the lower decks were capable of withstanding the tremendous impact, and were largely intact after the earthquake.

It is significant that the lower bent was designed as a rigid moment resisting frame, different from that of the upper bent, which was much more flexible. The cross section of the northernmost collapsed section of the box girder deck is shown in Fig. 4.4.

Failure modes of the upper frames along the 1.2-km distance are quite consistent. For most of the collapsed bents of the type with layout 1, the columns failed at the hinge joint regions due to shear force from the lateral load induced by the ground motion. The columns were thrown out transverse to the deck, as shown in Fig. 4.3. From the wreckage, evidence showed that typically only four small dowel bars and a drain pipe at the center extended continuously from the lower column - lower girder joint to the upper column (Fig. 4.5). Obviously these reinforcements were not sufficient to resist the lateral shear force imposed on the upper column by the earthquake. In Fig. 4.6 it can be seen that the failure plane coincides with the curved surface defined by the lower girder longitudinal reinforcement bending downward into the lower column [5]. The dragging of the sheared-off upper columns when the upper deck collapsed caused further damage on the lower column, pulling some of the longitudinal reinforcement bars away from the column. Also from Fig. 4.6 it seems that the retrofitted expansion joint performed satisfactorily during the earthquake, holding the two spans of the deck together in the longitudinal direction.

About six bents of the type with layout 2 failed with partial collapse of the upper deck as shown in Fig. 4.7. The upper deck rotated about the hinge joints of the remaining standing upper columns on the west side. The east side of the upper deck collapsed onto the lower deck after the collapse of the east side columns, which had the only moment resisting connections for the upper frames.

The observed damage of another similar double-deck freeway structure, the Embarcadero Freeway in San Francisco, seems to support the Cypress observation that the lower girder - lower column joint region experienced significant lateral shear force during the earthquake. As shown in Fig. 4.8, the joint region suffered significant numbers of diagonal shear cracks. Thus it is plausible that this freeway structure might have suffered the same devastating failure as the Cypress Viaduct, had the shaking of the ground been more intense or of longer duration.

4.3 THE OAKLAND BAY BRIDGE

The Oakland Bay Bridge is a 13.4-km long structure connecting San Francisco and Oakland via the Yerba Buena Island. This structure has two major portions: the West Bay Crossing and East Bay Crossing. The West Bay Crossing consists of two suspended sections with a central anchoring block. The East Bay Crossing consists of several cantilever and simply supported trusses. The Oakland Bay Bridge was erected almost simultaneously with the Golden Gate Bridge in the late 1930's.

The bridge has two 20-m wide concrete decks situated one on top of the other. These decks are supported by a system of longitudinal beams which rest on the transverse plate girders. Two rows of trusses each, on one side, support them and transfer the forces to the principal structural elements, which are different at each crossing. During the Loma Prieta earthquake 13 trusses between Pier E9 and Pier E23 on the Oakland side were moved in the east direction by about 7 inches (18 cm). The connectors were not designed to withstand such displacement and failed, leading to failure of the upper deck section onto the lower deck section. The failed sections were single spans that linked the cantilever and incline sections of the East Bay Crossing. They were located over the truss structure of Pier E9. The locations of damaged elements in the East Bay

Crossing are presented in Fig. 4.9. An overview of the missing portions of the Bay Bridge after they were removed is shown in Fig. 4.10. After the earthquake the collapsed decks were resting on a transformer station, which probably stopped them from crashing down onto the truss structure of Pier E9. It is speculated that this in fact saved Pier E9 from total collapse [3].

A view of the lower transverse girder, with remaining parts of connectors that were partially destroyed during the earthquake, is presented in Fig. 4.11. The type of grid structure under the upper deck in the neighborhood of the collapsed section is shown in Fig. 4.12 and a close-up of the connector which failed on the other side of the transverse girder is presented in Fig. 4.13.

Four seismograph stations are located in the neighborhood of the Bay Bridge. The highest ground horizontal acceleration, 0.26 g (260°), was recorded by the USGS station in the basement of the 30-storey structure on Christic Avenue in Emeryville, nearly 100 km north of the epicenter and less than 2 km north of the collapsed I-880 freeway in Oakland. The two other accelerations were 0.22 g (350°) horizontal, and 0.06 g vertical [1]. Three other stations belonging to the CSMIP network had the following readings: Rincon Point at the San Francisco side of the Bay Bridge - 0.09 g (90°), 0.08 g (360°) and 0.03 g (up); Yerba Beuna Island - 0.06 g (90°), 0.03 g (360°) and 0.03 g (up); 24-storey residential building in Oakland, south of the I-880 freeway - 0.18 g (305°), 0.14 g (35°) and 0.04 g (up) [6]. Thus, substantially higher ground accelerations were recorded on the Oakland side in the Bay Bridge vicinity. Divers were employed to inspect the pier foundations underwater and found the concrete foundations of Pier E9 and Pier E23 to be cracked.

The two collapsed sections were later removed by a large crane and replaced in a record time of one month. Subsequently, regular traffic through the bridge was restored.

4.4 STRUVE SLOUGH BRIDGE

The Struve Slough Bridge is a part of the Cabrillo Highway, which is part of Highway 1. The bridge is located near the city of Watsonville between Harkins Slough Road and Beach Road. This is, in fact, a pair of bridges carrying the northbound and southbound lanes separately. The bridge was erected in 1965 using Raymond tapered-shell piles driven an average of 80 ft (25.0 m) [7,8].

Based on the site visit observations, the bridge collapsed in a very unusual way. Probably, at first, soft soil layers were densified due to shaking, and subsequently the whole area of the old river bed shifted laterally. The amount of vertical displacement was estimated to be about one foot (30 cm). An overview of the Struve Slough Bridge from the northern abutment is presented in Fig. 4.14. The amount of lateral shift is clearly visible in Fig. 4.15. A column was completely sheared off and displaced from the external bridge beam. Some columns punched through the deck of the southern lane (Figs. 4.16 and 4.17). During the earthquake a pick-up truck was just entering a lane when it collapsed. The driver and truck barely escaped a collision with one of the protruding columns. The collapsed portion of the deck (with the punched columns) did not disintegrate (Fig. 4.18).

The closest CSMIP strong-motion seismograph station is located in the downtown of Watsonville, about 3 km eastward from the collapsed bridge. The station was installed in the 4-storey office building (concrete structure) and consists of 13 sensors. The measured maximum ground accelerations were: 0.28 g (45°), 0.39 g (135°) and 0.66 g (up). The building was founded on fill over alluvium. The recorded horizontal acceleration reached 1.24 g (the maximum recorded for the Loma Prieta earthquake) [6].

4.5 OTHER TRANSPORTATION STRUCTURES INCLUDING ROADS

Highway 17

Highway 17 between Los Gatos and Santa Cruz (see location on Fig. 4.1) was heavily damaged due to several landslides in the Santa Cruz Mountains (Figs. 4.19 and 4.20). Heavy rain which followed the earthquake added to the poor stability of the road embankments. The large horizontal displacements resulted in heavy cracking and in some locations complete breakage of the road barriers located in the middle of the road at the sides. Examples of the damaged barriers are presented in Figs. 4.21 and 4.22.

Modern Overpasses

Some modern overpasses like the one on I-280 and G3 were slightly damaged. The cracks on the columns near the ground levels were immediately repaired by epoxy and covered (Figs. 4.23 and 4.24).

Secondary Type of Damage

An example of secondary type of damage is presented in Fig. 4.25. The piles were washed around from the ground at one of the piers of the bridge on Eaton Street in Santa Cruz. A ground fissure caused by the earthquake broke a water pipe and the leaked water was the prime reason for this damage (Fig. 4.26).

REFERENCES

[1] R. Maley, A. Acosta, F. Ellis, E. Etheredge, L. Foote, D. Johnson, R. Porcella, M. Salsman and J. Switzer, - "U.S. Geological Survey Strong-Motion Records From the Northern California (Loma Prieta) Earthquake of October 17, 1989," Department of the Interior, U.S. Geological Survey, Open-File Report 89-568, October 1989.

[2] "Second Quick Report on CSMIP Strong-Motion Records from the October 17, 1989 Earthquake in the Santa Cruz Mountains," Department of Conservation, Division of Mines and Geology, Office of Strong Motion Studies, Sacramento, California, October 25, 1989.

[3] A. Astaneh, V. Bertero, B. Bolt, S.A. Mahin, J. Moehle and R. Seed - "Preliminary Report on the Seismological and Engineering Aspects of the October 17, 1989 Santa Cruz (Loma Prieta) Earthquake," UCB/EERC-89/14, Earthquake Engineering Research Center, University of California, Berkeley, October 1989.

[4] Oakland Tribune, October 26, 1989.

[5] D.K. Nims, E. Miranda, I.D. Aiken, A.S. Whittaker and V. Bertero - "Collapse of the Cypress Street Viaduct as a Result of the Loma Prieta Earthquake," UCB/EERC-89/16, Earthquake Engineering Research Center, University of California, Berkeley, November 1989.

[6] A. Shakal, M. Huang, M. Reichle, V. Ventura, T. Cao, R. Sherburne, M. Savage, R. Darragh, C. Petersen - "CSMIP Strong-Motion Records from the Santa Cruz Mountains (Loma Prieta), California Earthquake of 17 October 1989," Report No. OSMS 89-06, California, Department of Conservation, Division of Mines and Geology, Sacramento, California, November 1989.

[7] "The October 17, 1989 Loma Prieta Earthquake," EQE, San Francisco, October 1989.

[8] J. Khanna - "The Loma Prieta (Northern California) Earthquake of 17 October 1989 - Some Observations and Reflections," the BC Professional Engineer, January 1990, pp. 23-27.

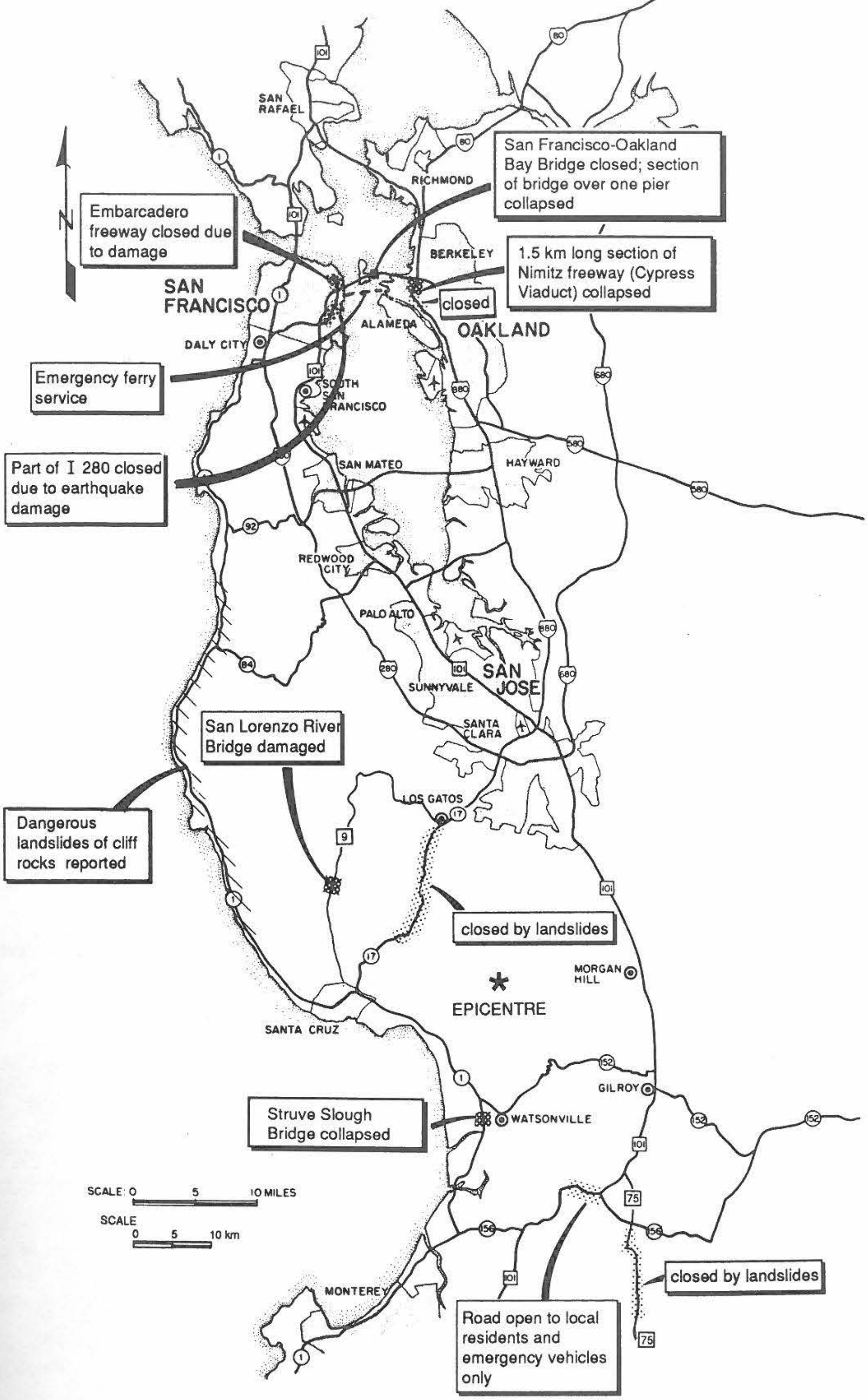
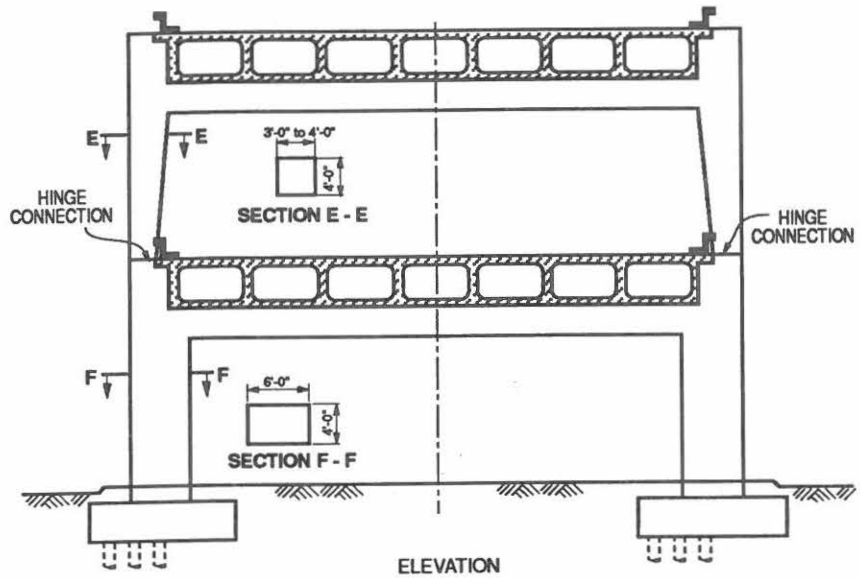
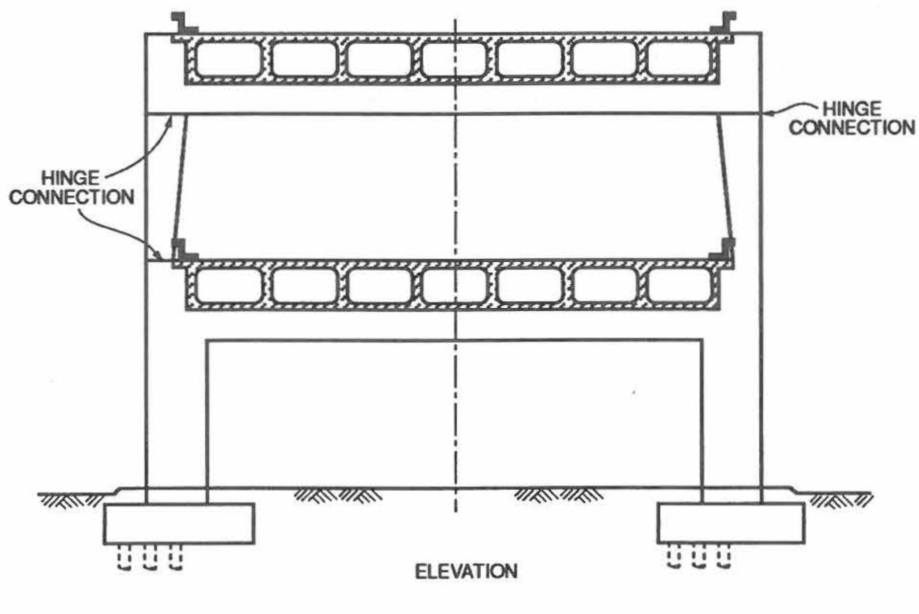


Fig. 4.1 Locations of damaged transportation facilities and road closures in the Bay Area after the Loma Prieta Earthquake of October 17, 1989.



(a) Layout 1



(b) Layout 2

Fig. 4.2 Typical design layout of bent structure

Fig. 4.3 Collapsed Cypress Street viaduct

Fig. 4.4 Northernmost collapsed section of box girder upper deck

Fig. 4.5 Detail of upper column failure

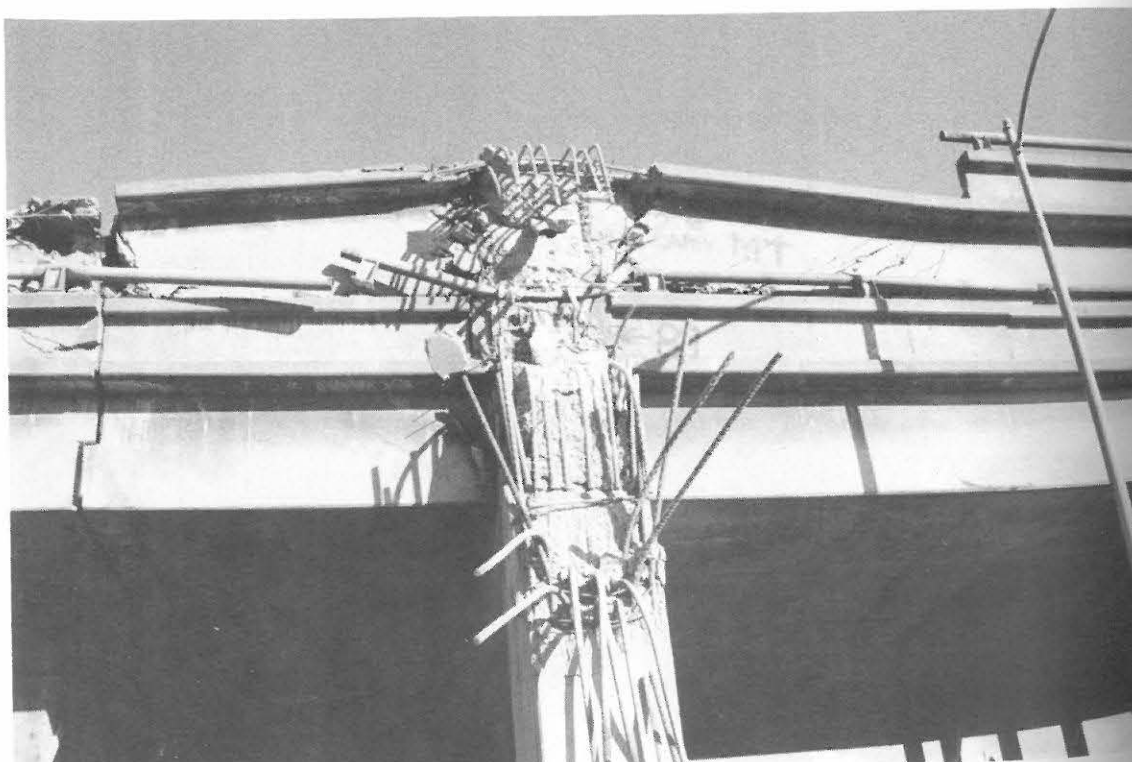


Fig. 4.6 Detail of lower column damage

Fig. 4.7 Partial collapse of upper deck

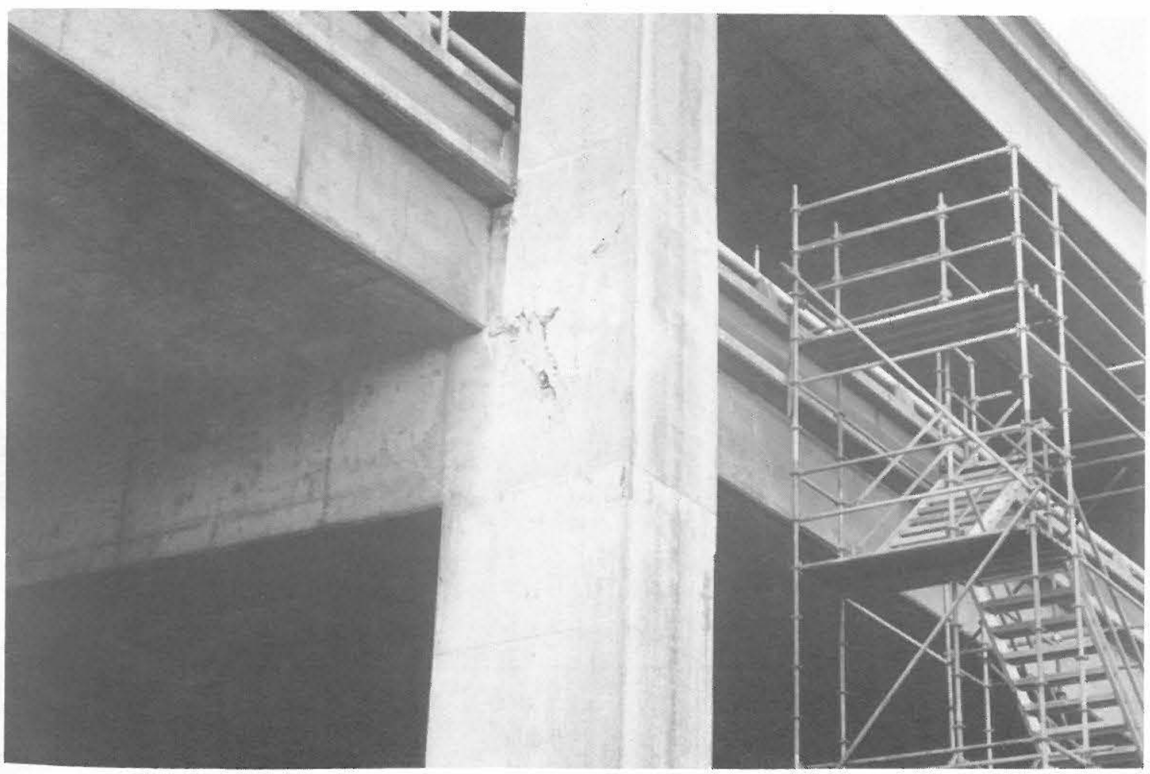


Fig. 4.8 Diagonal shear cracks in Embarcadero Freeway

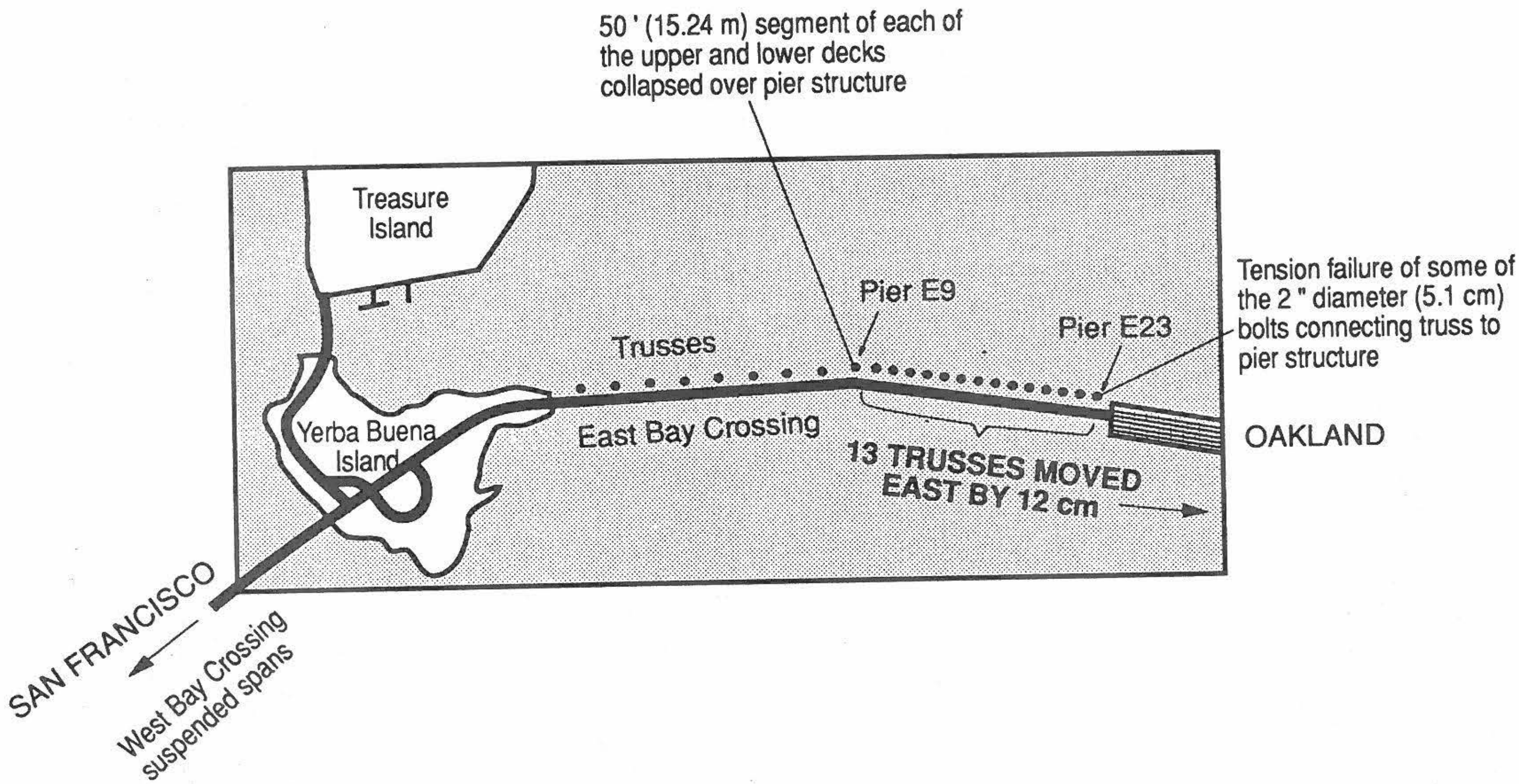


Fig. 4.9 Locations of damaged elements on the San Francisco-Oakland Bay Bridge during the Loma Prieta earthquake

Fig. 4.10 The Bay Bridge under repair - the two collapsed sections are already removed

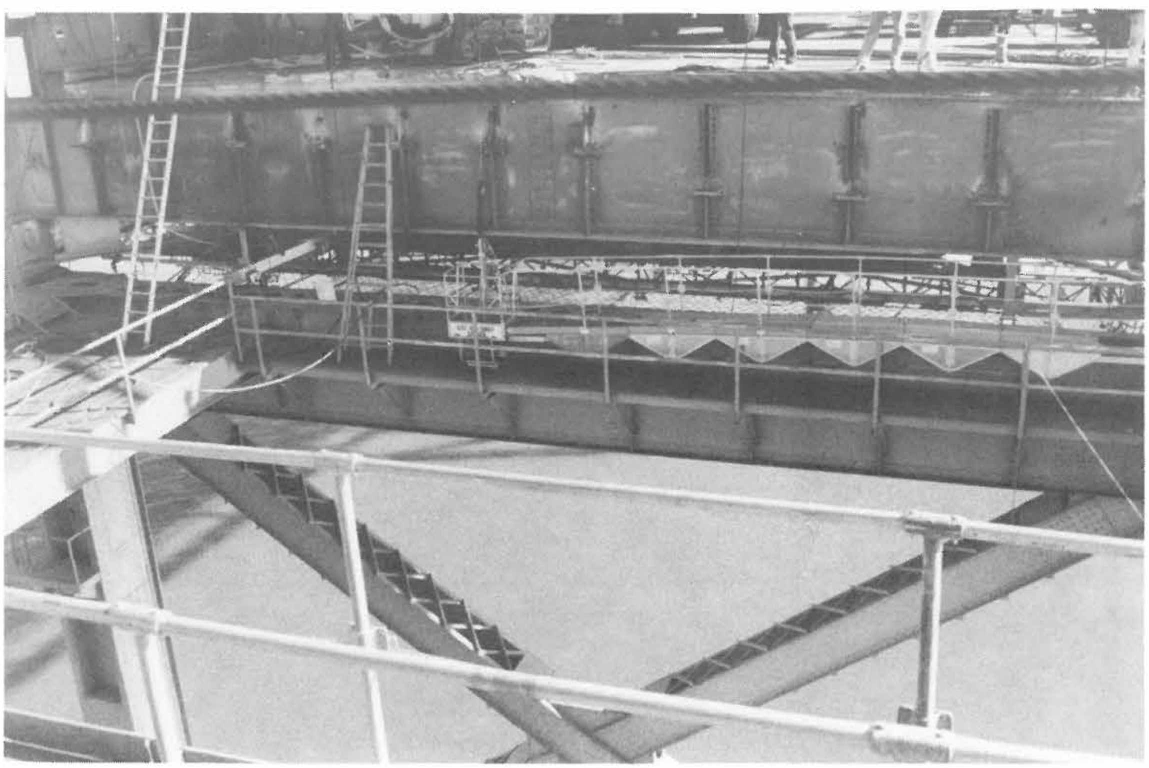


Fig. 4.11 The lower transverse girder with remaining parts of connectors which were destroyed during the earthquake

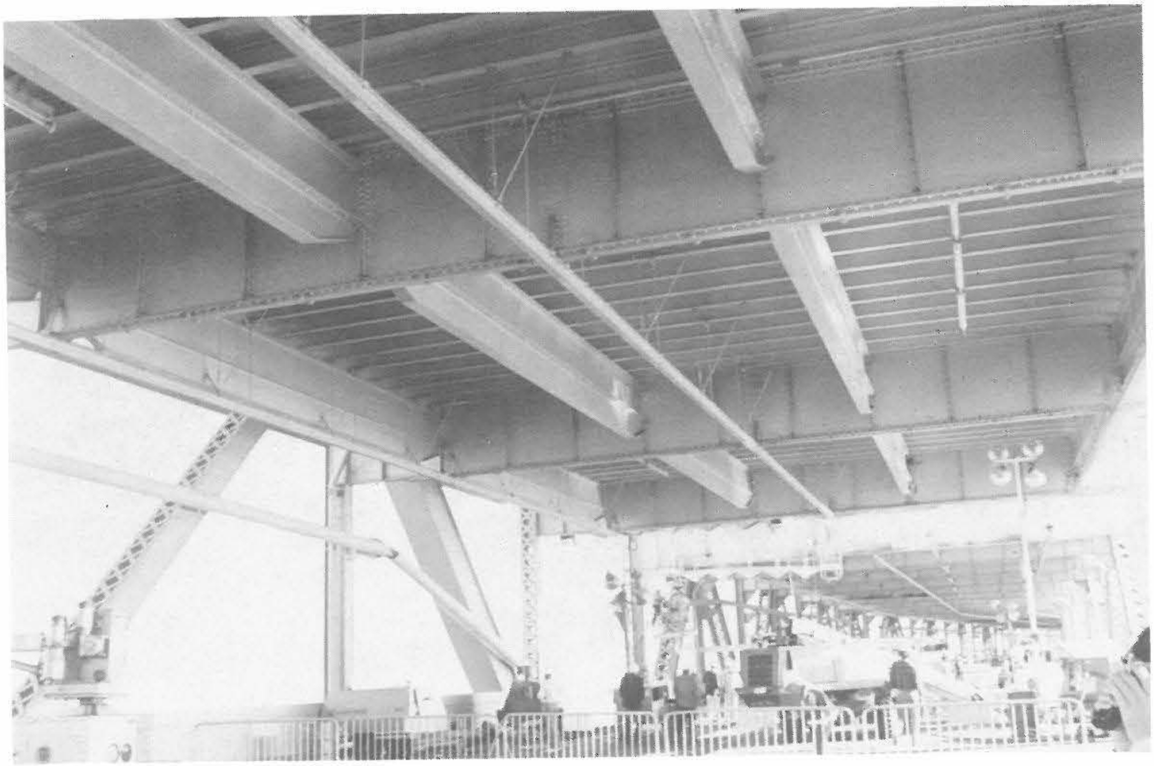


Fig. 4.12 The steel grid structure which carries the upper deck of the Bay Bridge

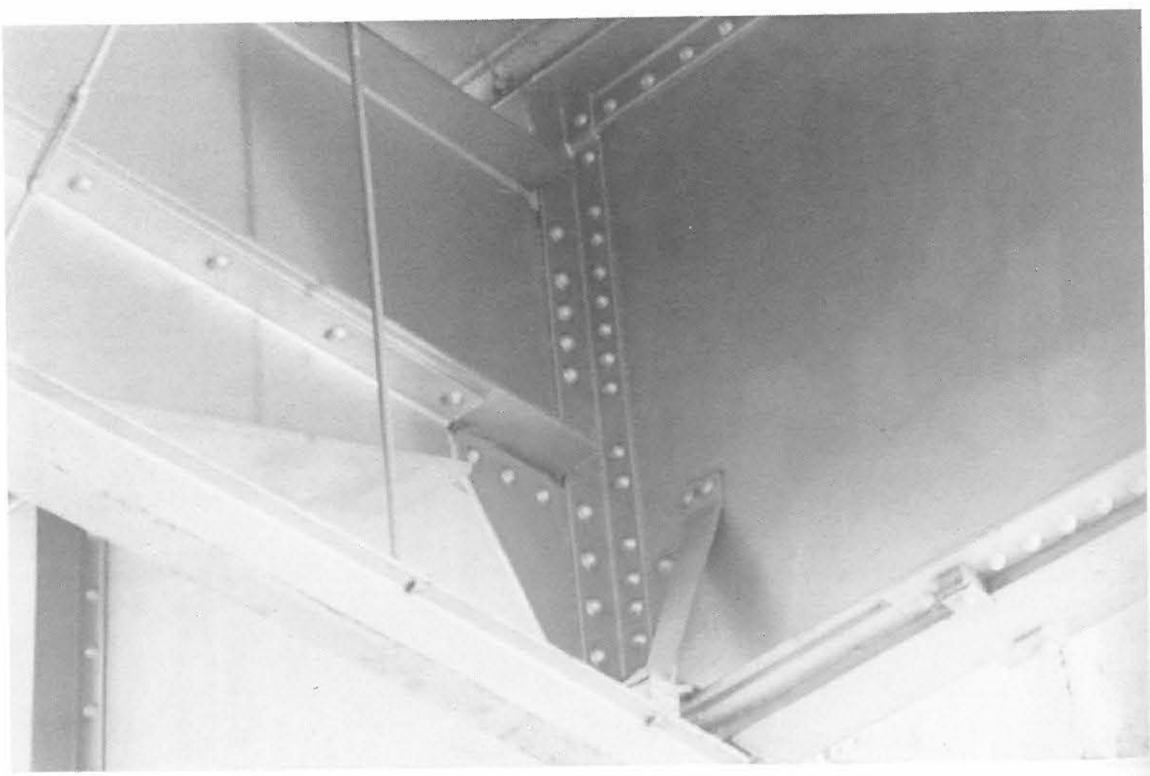


Fig. 4.13 Close-up of the detail similar to that which failed on the other side of the transverse girder

Fig. 4.14 The two heavily damaged lanes of the Struve Slough Bridge on Highway 1 near Watsonville

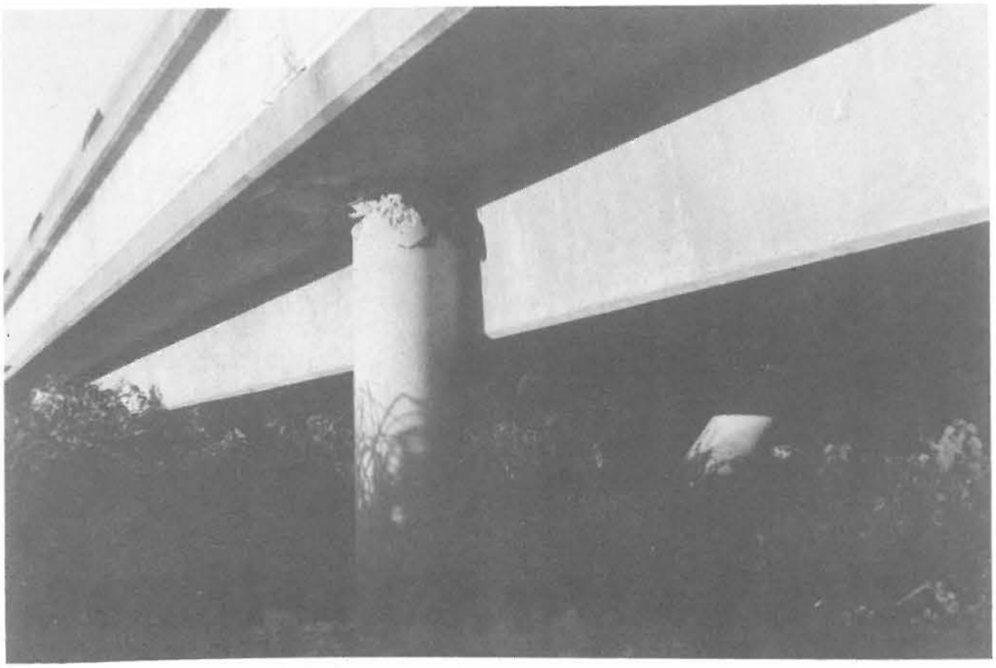


Fig. 4.15 A displaced column of the Struve Slough Bridge near Watsonville

Fig. 4.16 The southern lane collapsed and some displaced columns punched through the deck of the Struve Slough Bridge near Watsonville

Fig. 4.17 The difference between levels before and after collapse - the Struve Slough Bridge

Fig. 4.18 The collapsed portion of the deck of the Struve Slough Bridge did not disintegrate during the earthquake

Fig. 4.19 A landslide triggered by the Loma Prieta earthquake on Highway 17 in the Santa Cruz Mountains

4.20 A dangerous landslide being removed on Highway 17 in the Santa Cruz Mountains

Fig. 4.21 A destroyed road barrier on Highway 17 near Los Gatos

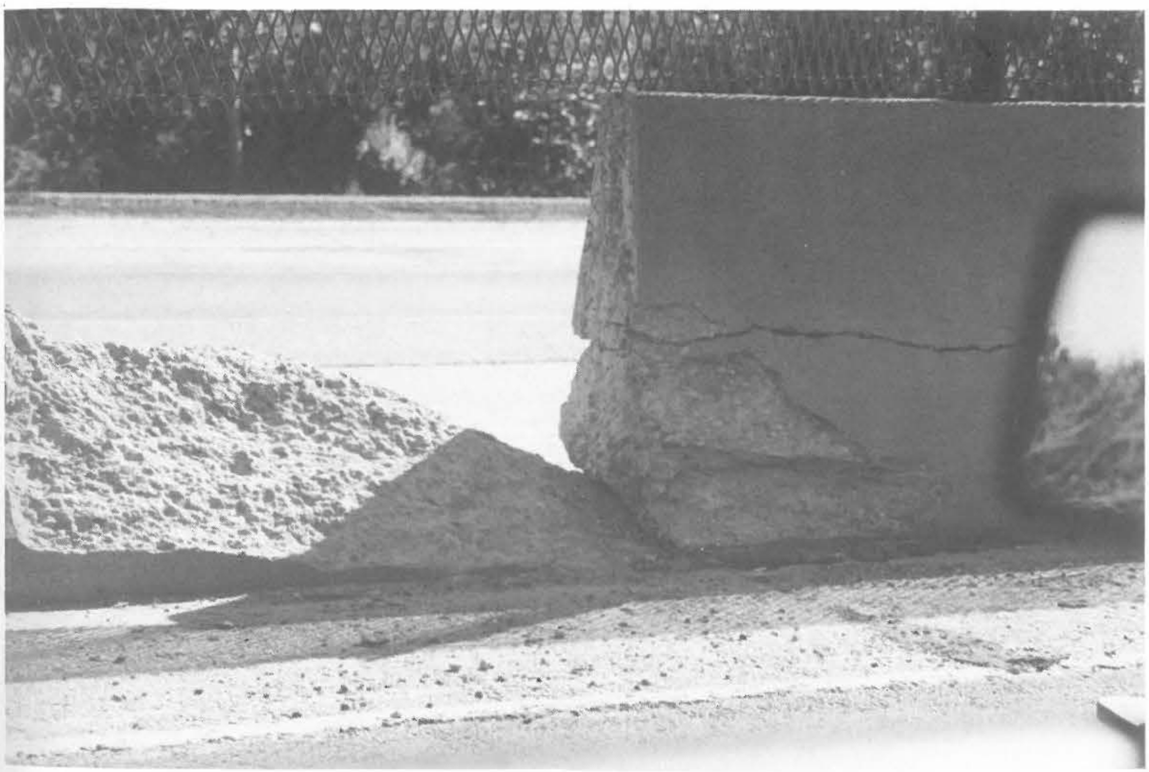


Fig. 4.22 A large horizontal displacement caused damage to the road barriers on Highway 17

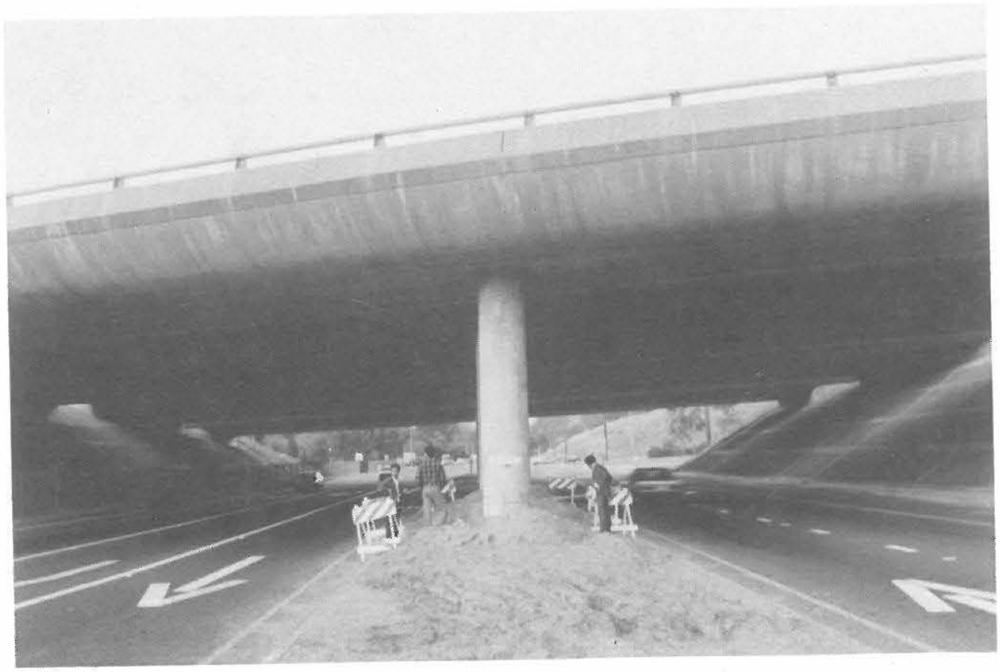


Fig. 4.23 The IRC team inspects a damaged column of the modern overpass (I-280 and G3) near Palo Alto

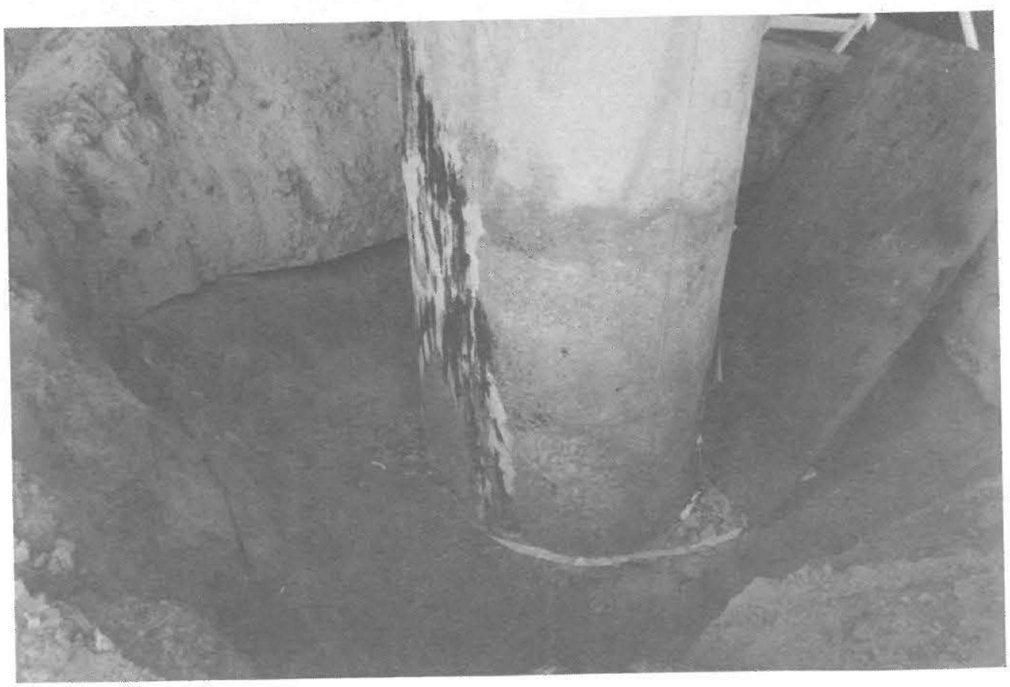


Fig. 4.24 A repaired crack showed the amount of distress experienced by the column (I-280 and G3 intersection)

Fig. 4.25 Water from a broken pipe caused damage to the pier of the bridge on Eaton Street in Santa Cruz

Fig. 4.26 Filled fissure which caused a break of the water pipe - bridge on Eaton Street in Santa Cruz

Chapter 5

PERFORMANCE OF POWER GENERATING AND TRANSMISSION FACILITIES

by

J.-R. Pierre and J.H.K. Tang

5.1 ELECTRIC POWER SYSTEMS

The Loma Prieta earthquake disrupted the electric power systems to about 1.4 million customers in a large area from Santa Cruz near the earthquake's epicenter to San Francisco in the north. The electric power facilities are operated by the Pacific Gas and Electric Company. Figure 5.1 shows the main power plants and transmission routes in the earthquake-affected area.

The most severe damage occurred at high voltage substations (230 and 500 kV), which lost a lot of electrical equipment and experienced oil leaks in transformers. Pockets of damage were also reported in the local power distribution systems. However, PG&E was able to restore bulk power to the Bay Area almost immediately by routing power transmission around damaged substations. Within 48 hours, full service to most areas was restored except for about 26,000 customers.

The estimated replacement cost of the equipment alone in three substations (San Mateo, Metcalf, and Moss Landing) was about $15 million. The major problem in restoring electrical supply after the event was to acquire the necessary equipment within a very short time. In this case, due to the extreme urgency, the U.S. Air Force flew in the new equipment through some mutual aid agreements with other utilities in the east coast and southern California.

5.2 NUCLEAR POWER PLANTS

The earthquake caused no damage to the five active nuclear reactors in California. The five are in Southern California: two at Diablo Canyon, which is located approximately 145 miles (230 km) from the earthquake's epicenter, and three at San Onofre, which is more than 350 miles (560 km) from the epicenter.

Diablo Canyon is operated by the Pacific Gas and Electric Company. Unit 2 of Diablo Canyon remained at 100% power during the event; unit 1 was in a refuelling outage. Ground motion at the plant was about 0.01 g. The Unit 1 slab felt 0.0044 g. No damage was reported at the plant. However, an Unusual Event, the lowest level of alert, was declared by the station. When the Unusual Event was terminated after about two and a half hours, there was no effect on power generation or operation of either unit. The San Onofre nuclear plant is owned by the Southern California Edison Company. One of the three reactors was closed for normal refuelling. None of the units was affected by the earthquake. The two that were operating at the time continued to run normally after the event.

Rancho Seco, a Sacramento Municipal Utility District nuclear power plant located about 85 miles (137 km) from the earthquake's epicenter, was closed earlier in 1989 by a public referendum requiring the plant to shut down permanently. The plant was therefore in cold shutdown during the earthquake. In any case, no damage was reported.

5.3 THERMAL POWER PLANTS

The Potrero and Hunter's Point Power Plants in San Francisco are about 60 miles (95 km) from the epicenter. These two plants had only superficial damage. One of the two 106 MW units at Hunter's Point went off-line. The third unit came off-line as load dropped within an hour after the earthquake. The Hunter's Point plant was back on line two days later. The 217-MW unit at Potrero Power Plant sustained some damage in the seismic restraints on the boiler. It went off-line mainly due to relay chatter.

Located about 30 miles (50 km) south of the epicenter is the Moss Landing Power Plant, which was damaged by the quake. This seven-unit plant uses primarily natural gas, but it can also burn oil. Units 1 to 5, built in the 1950's, range from 110 to 122 megawatts in size. Units 6 and 7 were built in 1967-68 and are 750 megawatts each (Fig. 5.2). Only unit 6 was operating at the time of the earthquake. Unit 7 had been removed from service for routine maintenance before the earthquake.

Due to the close proximity of the Moss Landing Power Plant to the earthquake, a special tour of the plant was made to observe more closely the extent of the damage. The plant tour, mainly concentrated at unit 6 and 7 as well as other supporting facilities in the station, is summarized as follows:

Structural

The power plant structures for units 6 and 7 are separated from units 1-5 by more than 200 ft (61 m). The superstructure, consisting of a 200 ft (61 m) high braced-steel frame supported on the ground floor, generally performed very well. Only two bracing members buckled (Fig. 5.3). There was no crack in the foundation slab, although some ground settlement was observed outside. The underground concrete service water tunnel, 10 ft x 10 ft (3 m x 3 m), was not damaged.

Piping & Supports

No damage to piping was observed. A lot of insulation on piping came loose after the quake. These were asbestos insulations and were immediately cleaned up and replaced. A vertical strut, supporting the horizontal snubbers for the super heater by-pass between the primary heater and secondary heater, yielded in bending. It was obvious that the 18 in. pipe had experienced large displacement during the earthquake (Fig. 5.4). Two beams, used as horizontal lateral restraints to the main-steam piping that feeds into the turbine, were severely bent as a result of the large movement of the piping (Fig. 5.5). The underground gas pipes had no indication of damage.

Mechanical Equipment

The horizontal flow heat exchangers fell off the support rockers because there were no stops or anchors. Several boiler tubes were found cracked. After the earthquake, all bearings of the turbine at unit 6 were taken out and inspected. A no. 3 bearing at the low pressure shaft appeared to be affected. It was repaired and replaced.

Electrical Components

All cable trays performed well. Only one coupling connecting a flexible conduit to a rigid conduit was torn. Batteries were tied down and were not damaged.

Concrete Stacks

Each unit has one reinforced concrete stack 500 ft (152.4 m) in height with steel liners. The stack has a pile foundation. The stacks did not have any damage. A 750,000-gallon distilled water storage tank occupies the lower portion of each stack. The only damage was the cracking of the body of the first valve leading to the water tank (Fig. 5.6).

Tanks

A 1950 vintage well water storage tank (Fig. 5.7), used for fire water supply, was damaged during the earthquake. The tank has a capacity of 750,000 gallons, a diameter of 57 ft., 6 in. (17.5 m) and a height of 40 ft. (12.2 m). The bottom plate - tank wall connection ruptured as a result of uplift caused by the sloshing of the water. The bottom plate was torn off approximately 1/3 of the circumference around the tank base, as shown in Fig. 5.8. Upon further inspection it was noticed that the bottom seam was heavily corroded and thus probably contributed to the failure. The sudden loss of contents through the ruptured bottom opening created a vacuum inside the tank. As a result, the conical tank roof caved in and the top of the tank shell buckled (Fig. 5.9).

More than a dozen large empty fuel storage tanks were not used at the time of the earthquake; these were not damaged (Fig. 5.10).

Miscellaneous

Jars in the chemical laboratory fell off the shelves. Some suspended ceilings suffered minor damage.

Summary

In spite of the damage mentioned above, four hours after the earthquake in-house power was brought back and the plant was on-line again within 24 hours.

References 1, 2, 3 and 4 were used during preparation of this part of the report.

5.4 HYDROPOWER PLANTS

San Luis Hydro and O'Neill Hydropower plants reported only very minor damage, and stayed on-line during and after the earthquake.

5.5 SUBSTATION DAMAGE

Three substations suffered considerable damage. These are: Moss Landing, Metcalf and San Mateo at epicentral distances of 45, 30 and 75 km, respectively (Fig. 5.1). Accelerations recorded in surrounding areas give an idea of the intensity of the tremor at Metcalf, San Mateo and Moss Landing substations: accelerations were between 0.26 g and 0.45 g near Metcalf, between 0.12 g and 0.39 g near Moss Landing and between 0.12 g and 0.29 g near San Mateo.

San Mateo Substation

At the 230-kV high voltage San Mateo Substation, four GE air circuit breakers ruptured at the porcelain column base.

Metcalf Substation

At the Metcalf Substation, which has a voltage of 500 kV, three Westinghouse circuit breakers were damaged. Oil leaks were detected at the radiator-tank connection of a GE power transformer and also in a GE current transformer.

Moss Landing Substation

At the Moss Landing Substation, Westinghouse 500-kV gas circuit breakers failed and were overturned, as illustrated in Fig. 5.11. The anchoring devices did not work and the steel supports were dragged down by the falling circuit breakers. Several Westinghouse 500-kV voltage transformers were also damaged (Fig. 5.12). Some aluminum conductors (busbars) broke (Figs. 5.13 and 5.14). Several current transformers and disconnectors (Figs. 5.15, 5.16, 5.17, 5.18, 5.19 and 5.20), as well as wave-traps, did not withstand the tremor. An oil leak was detected at the base of an Edison power transformer (Figs. 5.21 and 5.22).

For the low voltage structures shown in Fig. 5.23, conductors linking disconnectors to busbars broke during the tremor since they could not withstand the movements of the main structure.

The Hitachi circuit breakers at Moss Landing and San Mateo substations were not damaged.

The Westinghouse 500-kV SF6 gas circuit breakers were manufactured in 1970 for a design acceleration of 0.20 g. They were later reinforced with fibre-glass cables in 1975 to withstand an acceleration of 0.30 g (Photos 5.11 and 5.12). It appears now [5] that the 0.30 g acceleration was used as a static acceleration, which is transmitted from the ground to the centre of gravity of the apparatus with no amplification. In reality, this type of equipment behaves dynamically, with an amplification factor of 2 to 3.5 of ground acceleration. In addition, the substation is located on an alluvial deposit (Figs. 5.24 and 5.25). This fact certainly did not help, since it probably increased the intensity and the duration of the vibrations by approximately 1.5 times. Therefore, a 0.10-g bedrock acceleration can easily be multiplied by more than five times at the centre of gravity of the apparatus, which gives: $0.10 \times 3.5 \times 1.5 = 0.525$ g. It can thus be concluded that the reinforcement was ineffective. Perhaps it would have been more effective to increase the damping capacity of the apparatus.

The aluminum busbars that broke seem to be quite flexible because of the span of the conductor, shown in Fig. 5.26, as well as by the damping devices used (Figs. 5.12 and 5.27). Surprisingly, the aluminum conductor fractured, before the supporting insulator. According to PG&E analysis [5], the aluminum bar was quite weak. It could only withstand an acceleration of about 0.20 g compared to 0.30 g for the porcelain.

The circuit breaker support anchors that broke were designed in 1970 (Figs. 5.28, 5.29 and 5.30). A second -generation model performed well (Fig. 5.31). The new concept is now of the type indicated in Figs. 5.32, 5.33, 5.34 and 5.35.

No power transformers overturned or slid from their bases. As shown in Fig. 5.36, they were well anchored. Capacitor banks are usually somewhat vulnerable to earthquakes; however, none of the PG&E system was damaged.

The series compensation platforms nearest to the epicentre (some 80 km), at Tesla (ASEA and GE) and Los Banos (GE) were not damaged. These facilities are of the same type as those now used on the Hydro-Quebec 315-kV system at Kamouraska and those to be installed on the 735-kV system.

At present, PG&E requires their equipment to withstand a minimum acceleration of 0.40 to 0.50 g, using ten 5-sinusoidal beat cycles or seismic response spectrum.

5.6 CONTROL EQUIPMENT

Control buildings behaved well. They are usually built with prefabricated reinforced concrete or reinforced masonry. Equipment in the buildings (control panel, DC batteries, etc.) performed well. It seems that the gas or sudden-pressure relays did not trip during the quake [6]. It should be remembered that nearly 2000 MW, or 70% of the load loss due to the Saguenay quake in Quebec on November 25, 1988, was due to seismic relays vibration [7]. At present, PG&E uses the following gas and pressure relay models: Qualitrol 900-1, series 900 and 910 and GE 900-1, series 900.

5.7 LOAD LOSSES

Total load loss due to this 7.1-magnitude Loma Prieta earthquake was estimated at approximately 4300 MW, the overall PG&E system capacity being about 18,000 MW [6]. The Saguenay earthquake (M = 6) caused a load loss of 3,000 MW on the Hydro-Quebec system, which has a capacity of 30,226 MW [7].

REFERENCES

[1] "Loma Prieta Earthquake October 17, 1989," Preliminary Reconnaissance Report, Earthquake Engineering Research Institute, November 1989.

[2] A. Astaneh, V.V. Bertero, B.A. Bolt, S.A. Mahin, J.P. Moehle, R.B. Seed - "Preliminary Report on the Seismological and Engineering Aspects of the October 17, 1989 Santa Cruz (Loma Prieta) Earthquake," Earthquake Engineering Research Center, University of California at Berkeley, October 1989.

[3] "The October 17, 1989 Loma Prieta Earthquake," EQE Engineering, October 1989.

[4] Extracts of Design Requirements of Moss Landing Power Plant, Pacific Gas and Electric Company.

[5] E. Matsudo, PG&E, private communication.

[6] B. Gray and Hertz, PG&E, private communication.

[7] J.-R. Pierre - "Comportement des postes et expertise des dommages causés par le séisme du Saguenay," Hydro-Quebec, Service Etudes et Normalisation, mars 1989.

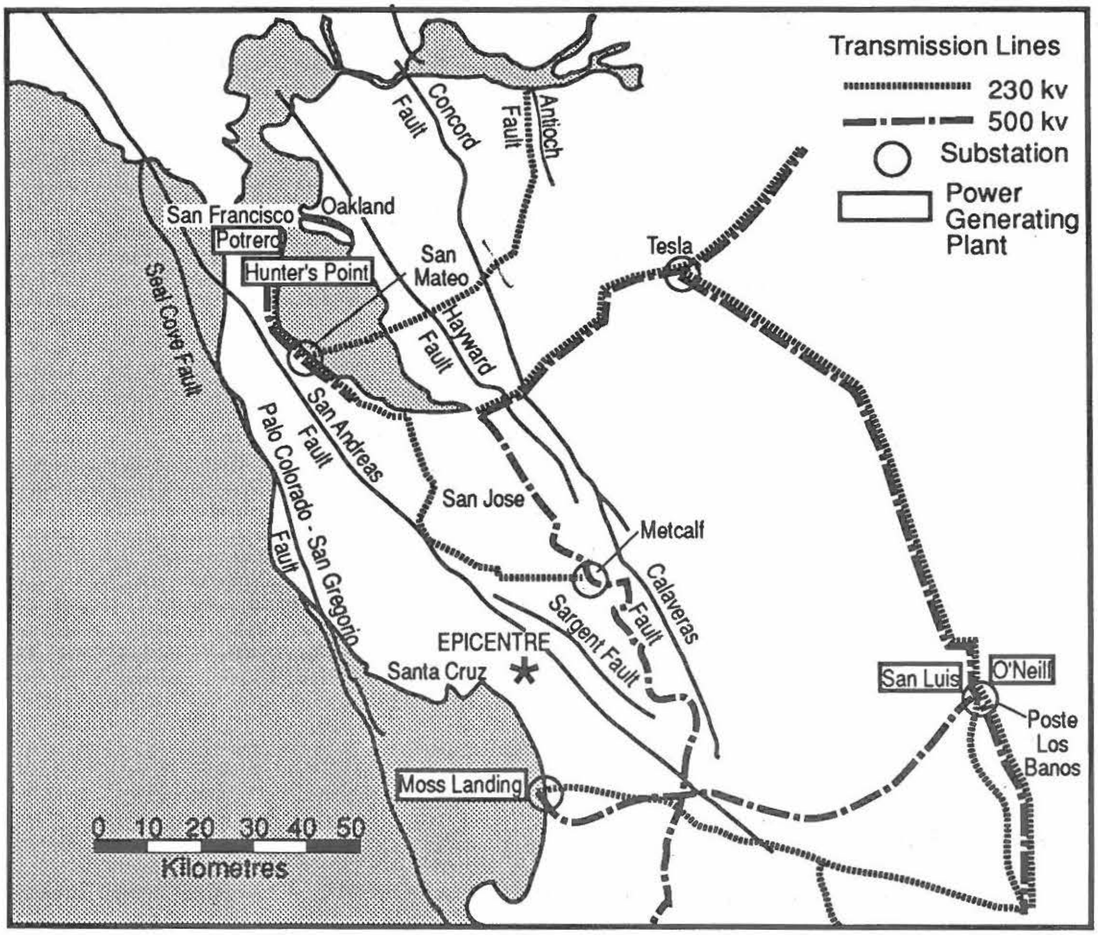


Fig. 5.1 Electric power systems in the earthquake-affected area

Fig. 5.2 Moss Landing Power Plant

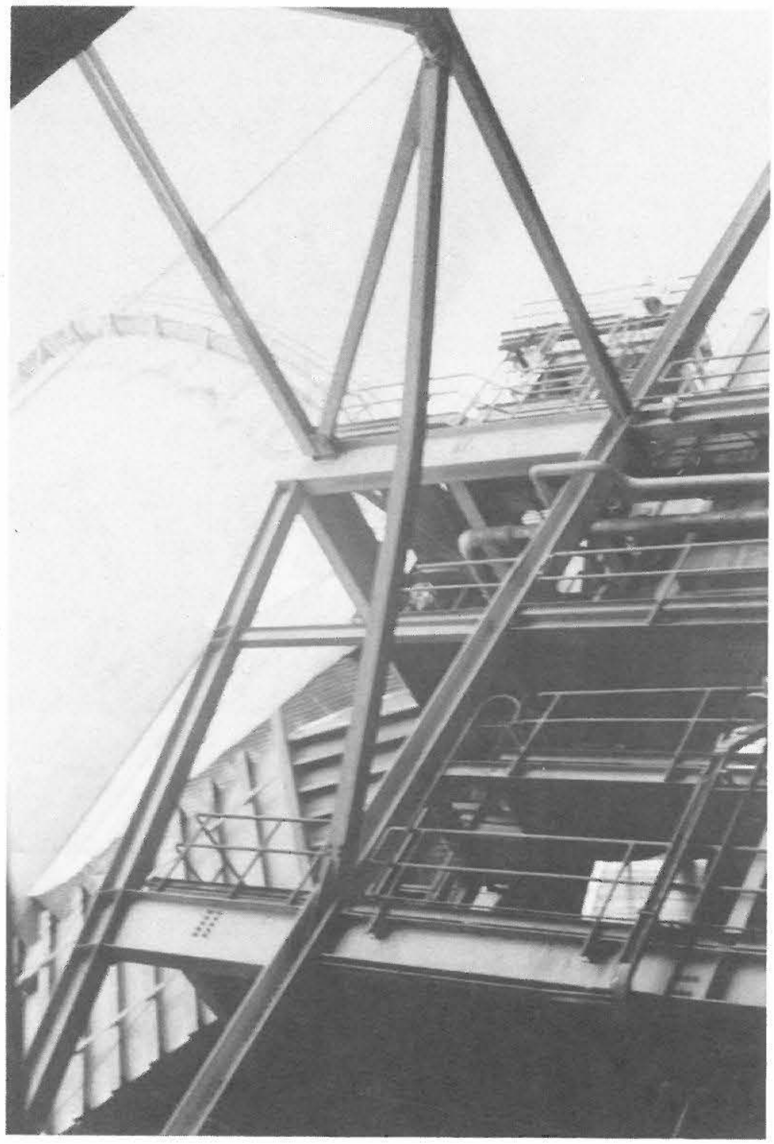


Fig. 5.3 Damage to two bracing members

Fig. 5.4 Damage to a vertical strut supporting a horizontal snubber

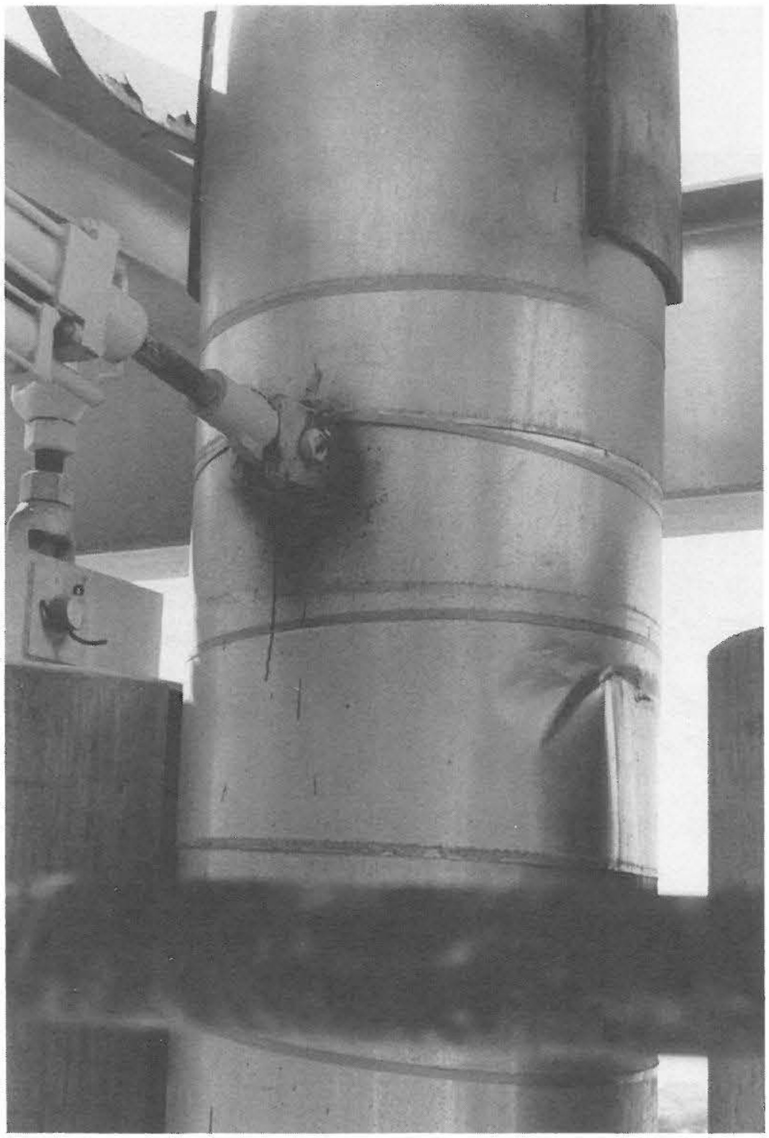


Fig. 5.5 Damage to horizontal lateral restraints

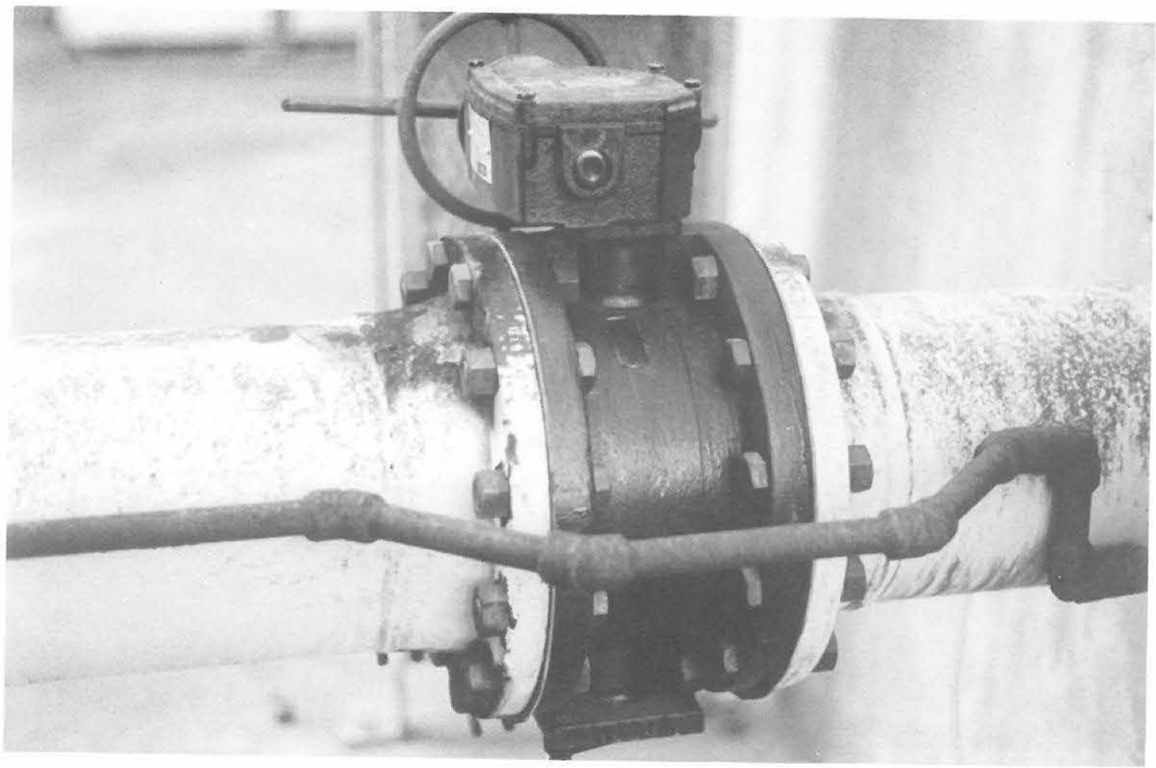


Fig. 5.6 Replacement of cracked valve

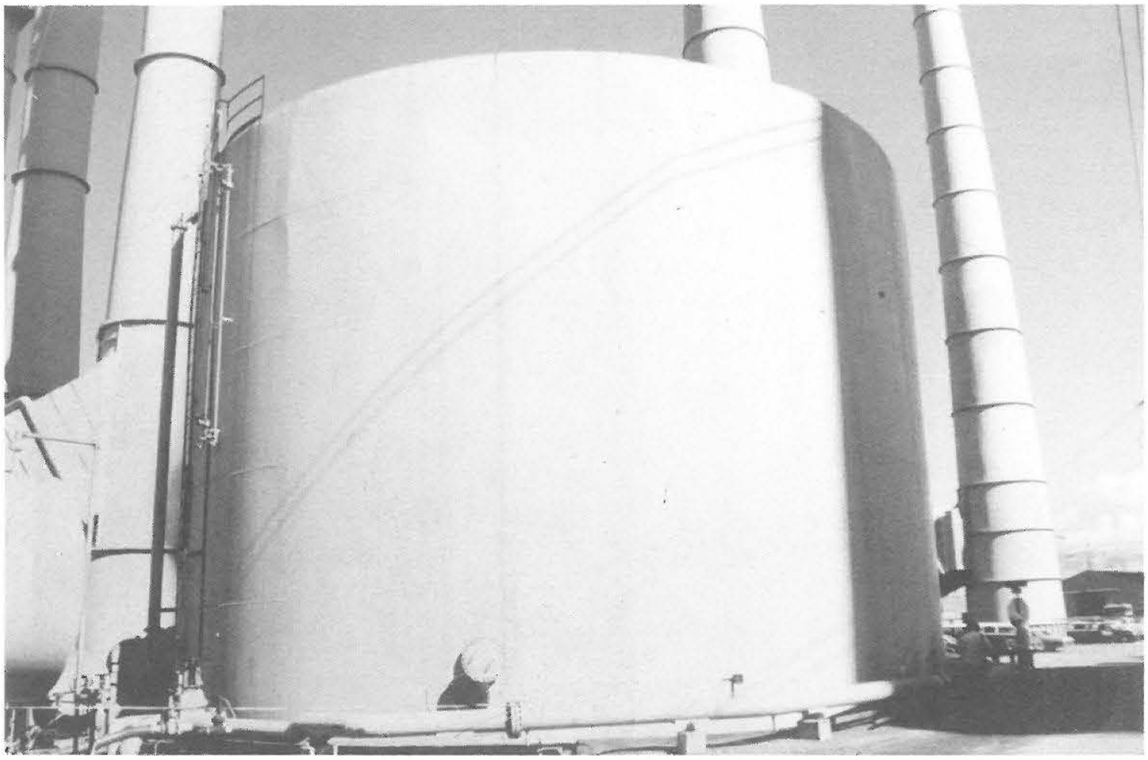


Fig. 5.7 The well water storage tank in Moss Landing Power Plant

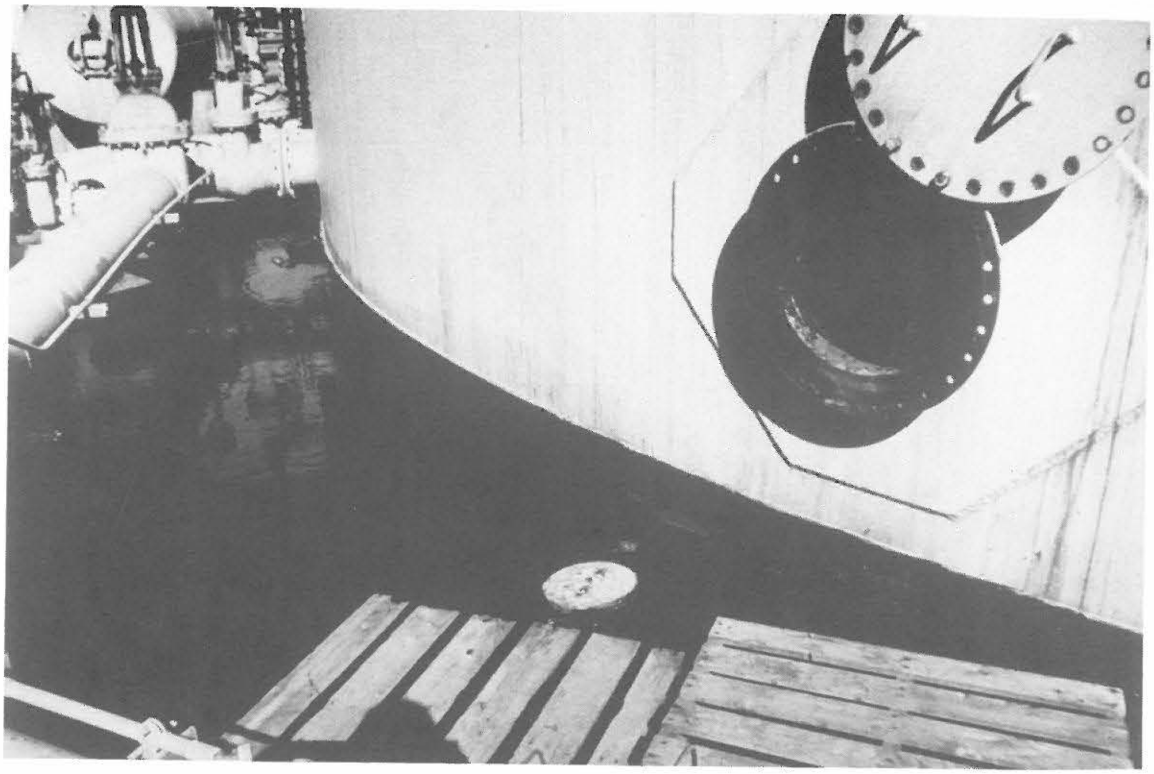


Fig. 5.8 Rupture of tank bottom

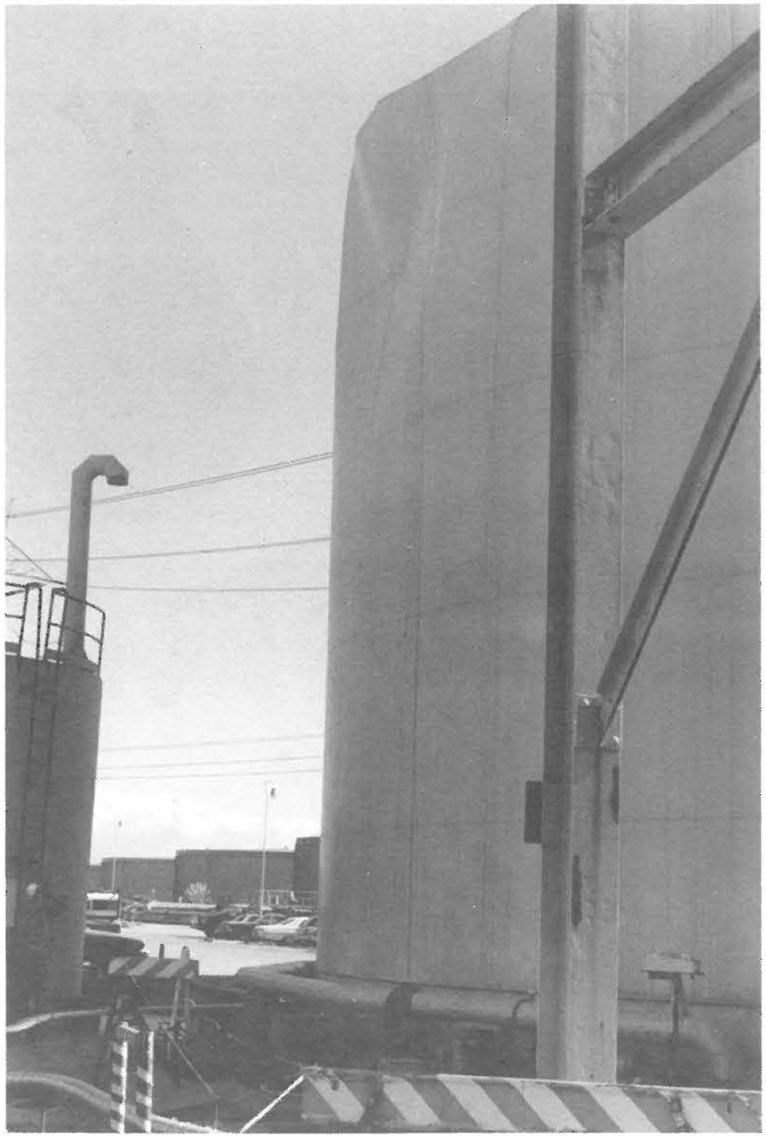


Fig. 5.9 A buckled tank shell - Moss Landing

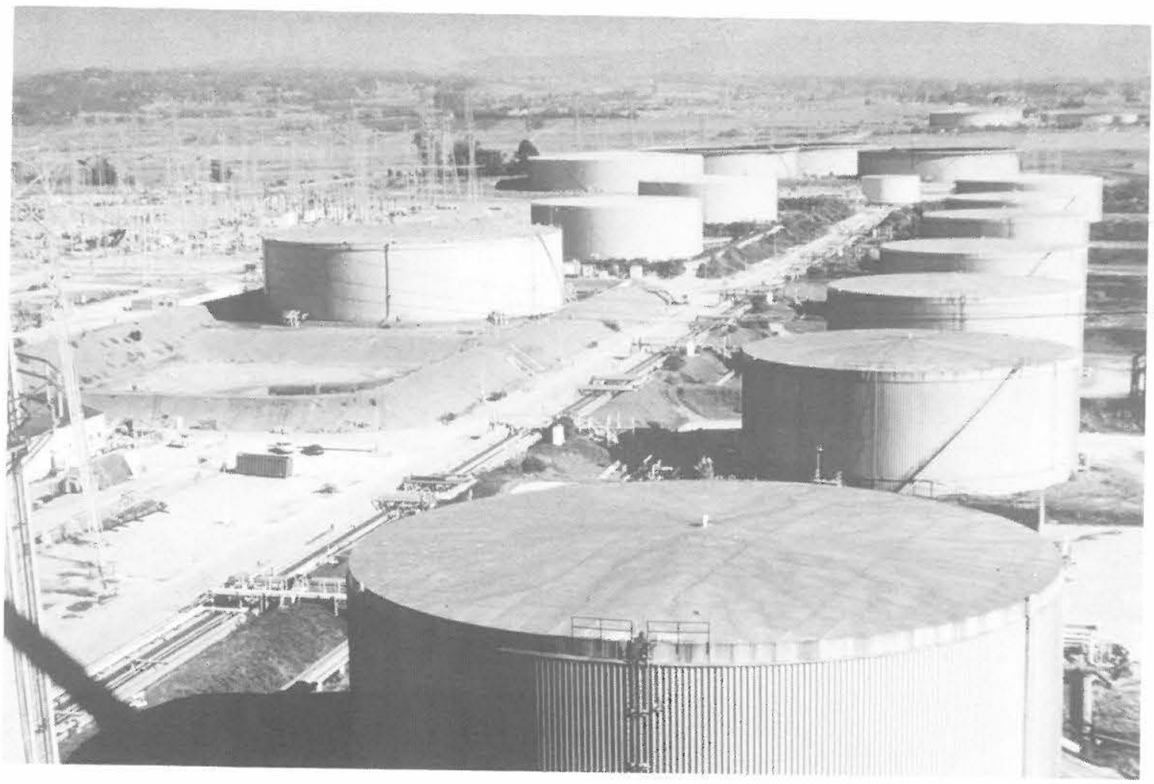


Fig. 5.10 Fuel tanks in Moss Landing Power Plant

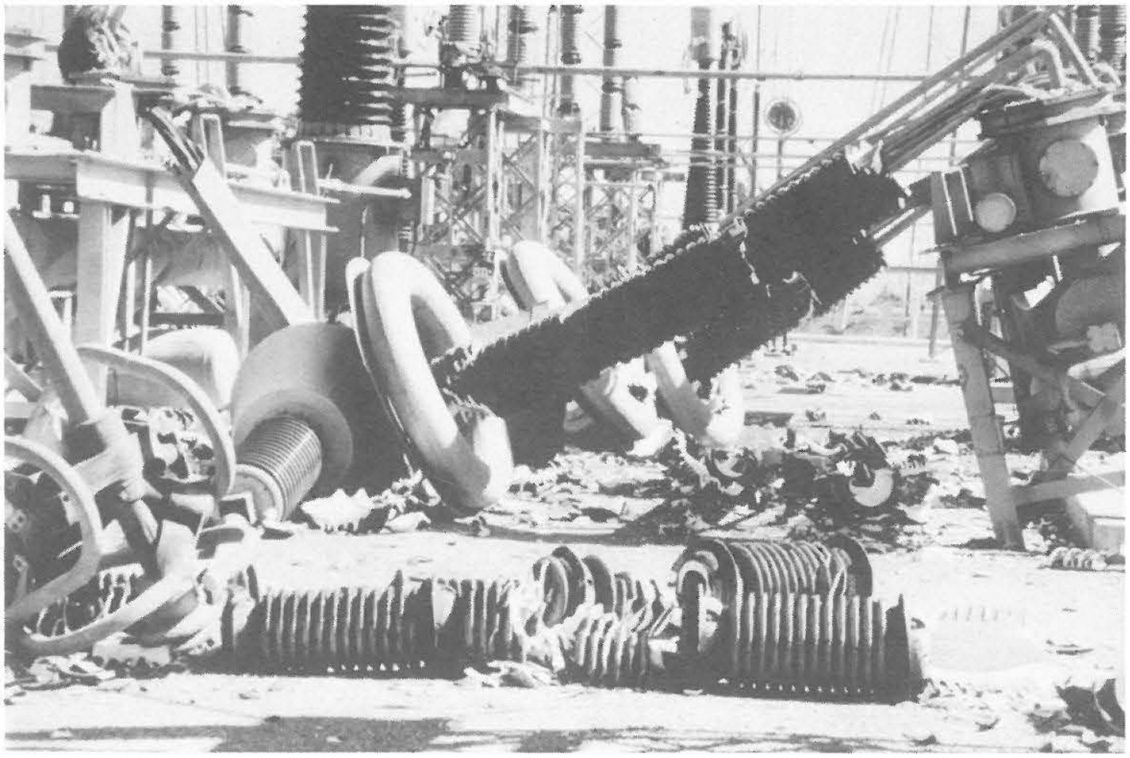


Fig. 5.11 Collapse of several 500-kV SF6 circuit breakers (courtesy of J.W. Lafferty, PG&E)

Fig. 5.12 Damaged 500-kV tension transformers at Moss Landing (courtesy of J.W. Lafferty, PG&E)

Fig. 5.13 Broken aluminum conductor - Moss Landing Power Plant

Fig. 5.14 Another broken aluminum conductor - Moss Landing Power Plant

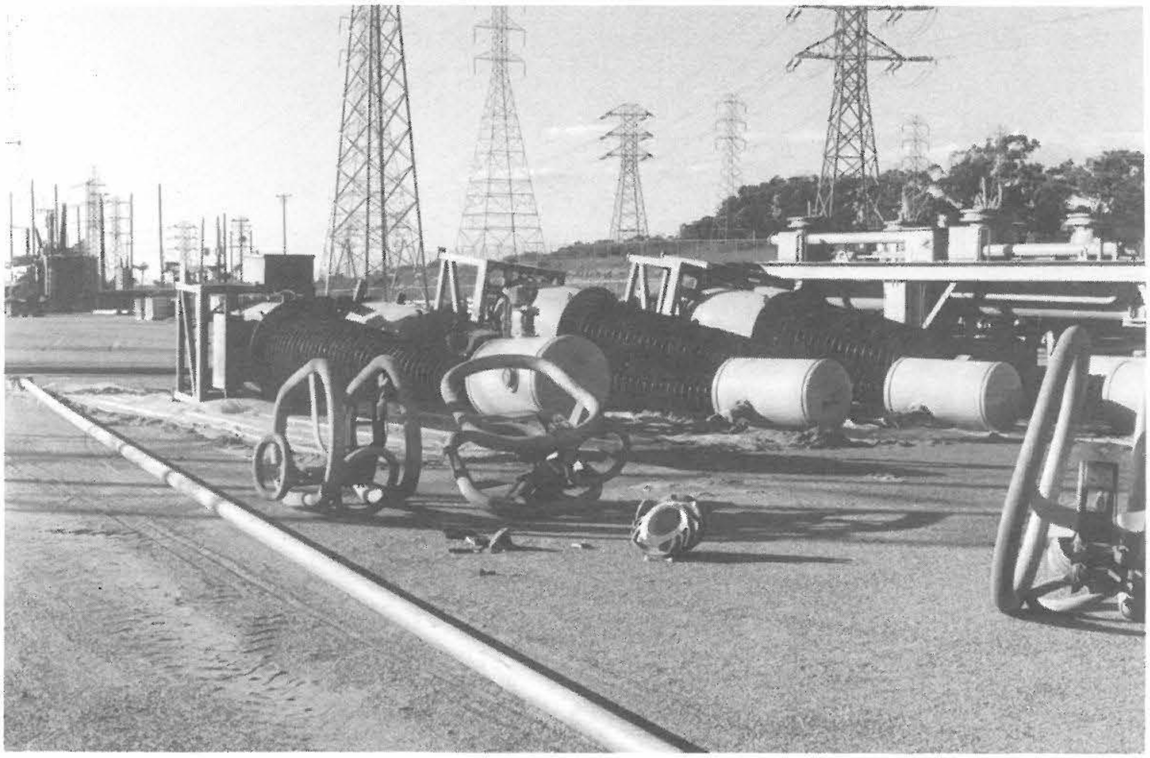


Fig. 5.15 Current transformers in the substation yard (one week after the earthquake)

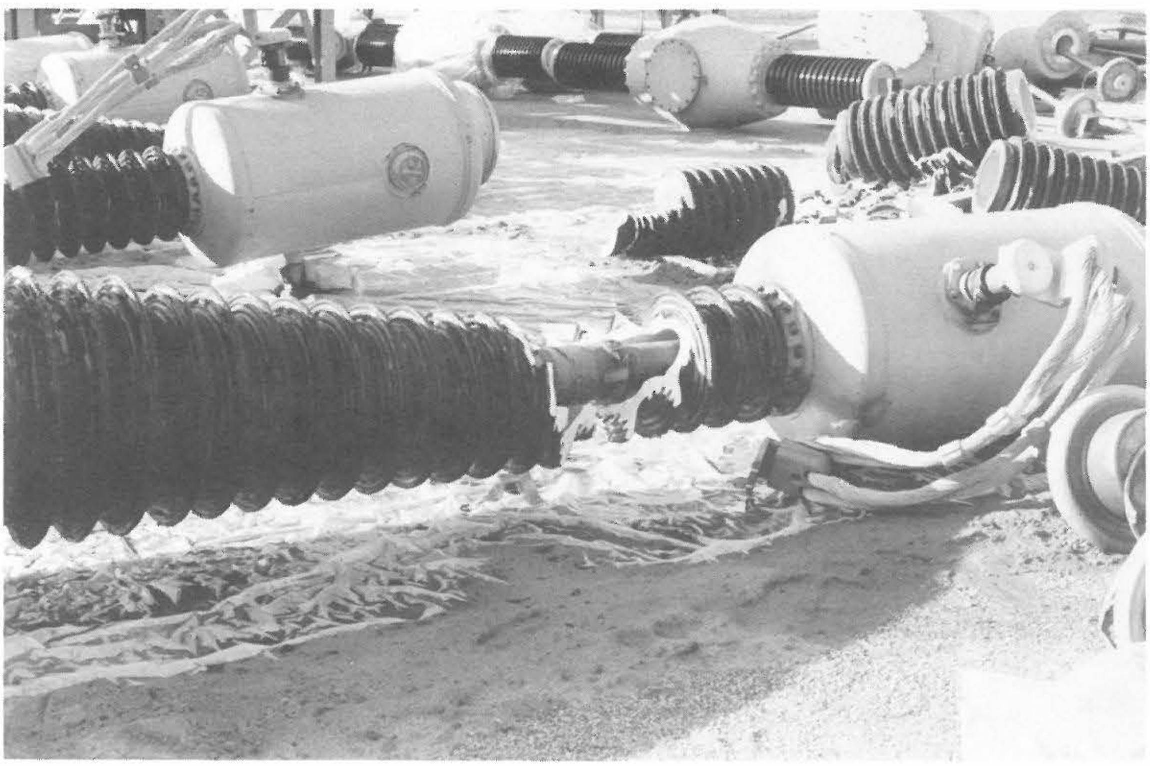


Fig. 5.16 Close-up of broken current transformers in the substation yard

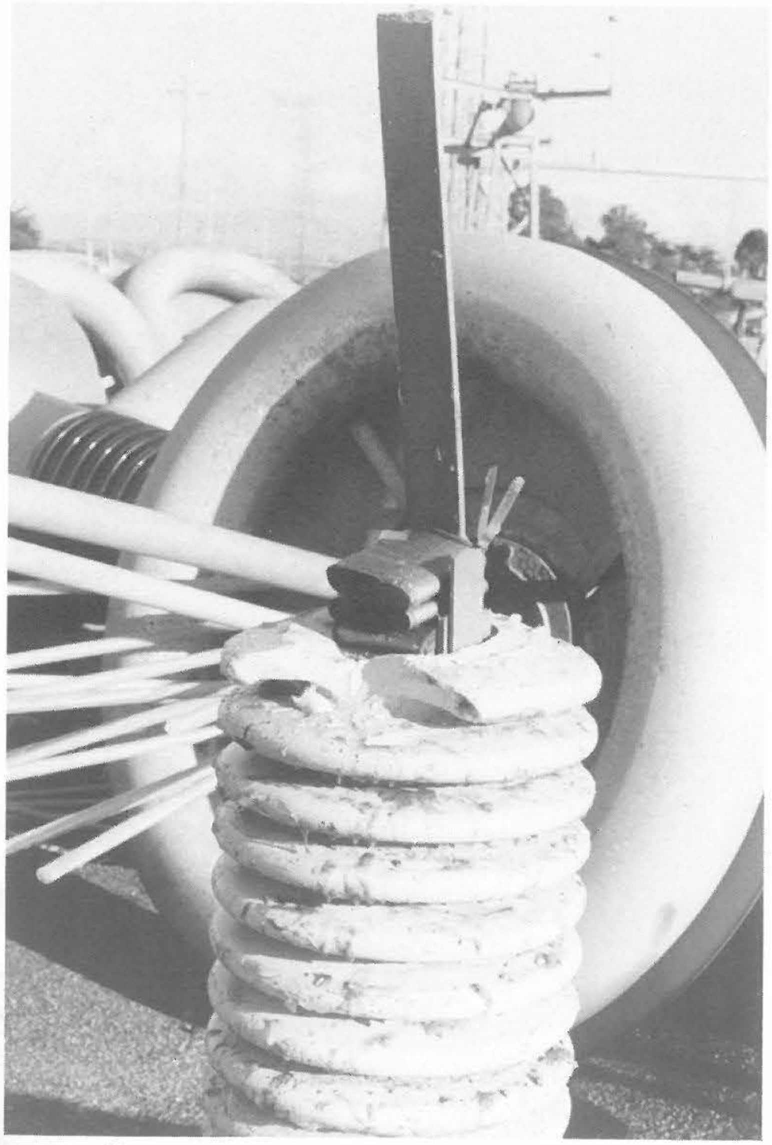


Fig. 5.17 Remains of a destroyed 500-kV tension transformer

Fig. 5.18 Remains of a destroyed 500-kV disconnector. Failure of a flanged cap

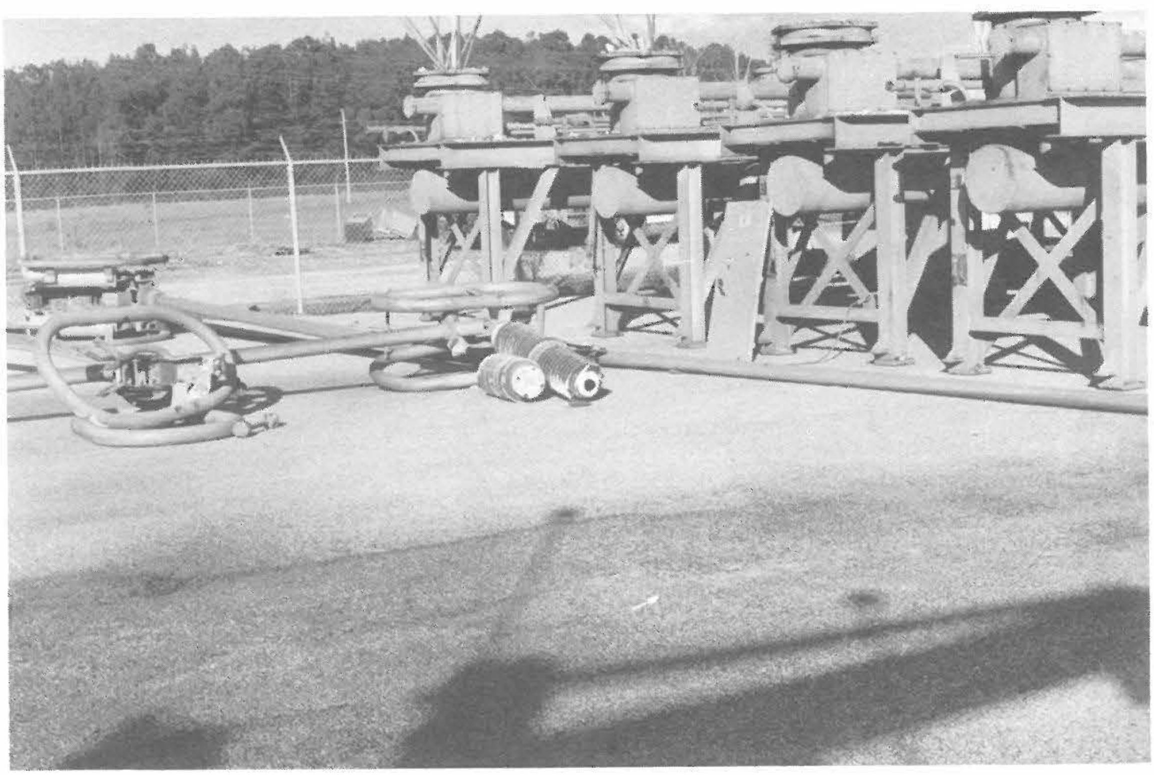


Fig. 5.19 Remains of 500-kV SF6 circuit breakers and disconnectors (one week after the earthquake)

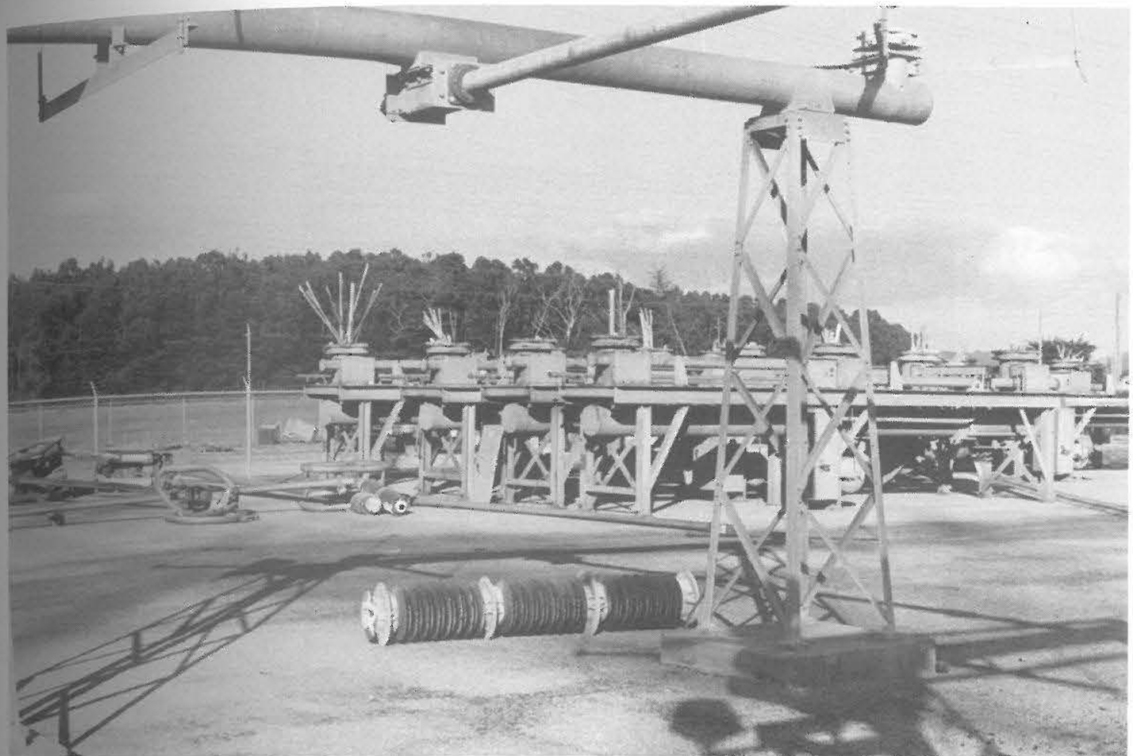


Fig. 5.20 Remains of 500-kV SF6 circuit breakers and disconnector - Moss Landing Power Plant

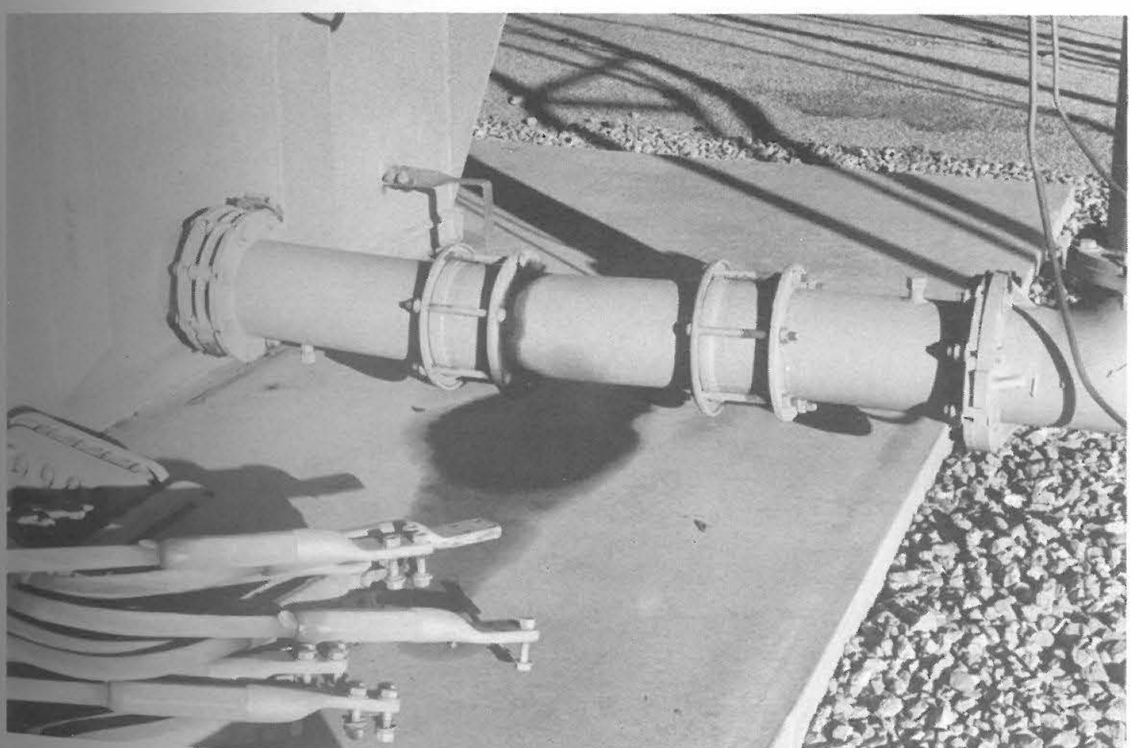


Fig. 5.21 Oil leak at the base of a 500-115-kV power transformer

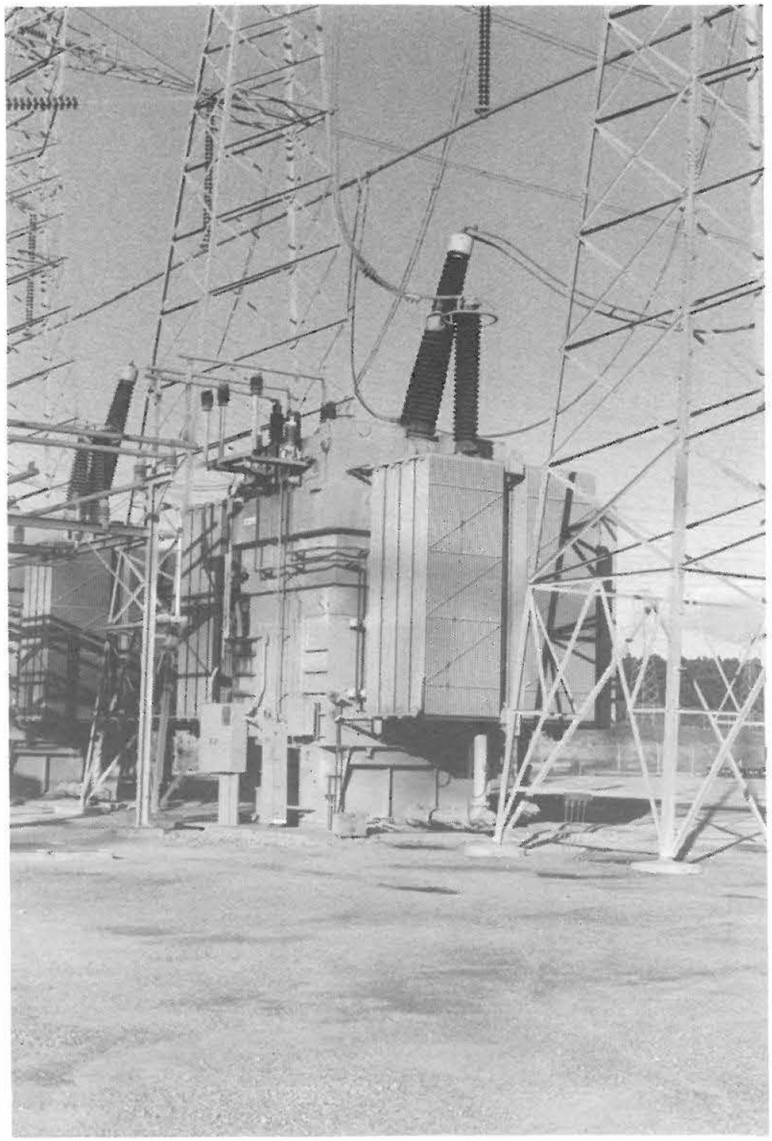


Fig. 5.22 Edison power transformer which leaked during the earthquake

Fig. 5.23 115-kV substation structure at Moss Landing Power Plant

Fig. 5.24 Entrance to Moss Landing Power Plant

Fig. 5.25 View of the substation and power generating plant sites at Moss Landing Power Plant

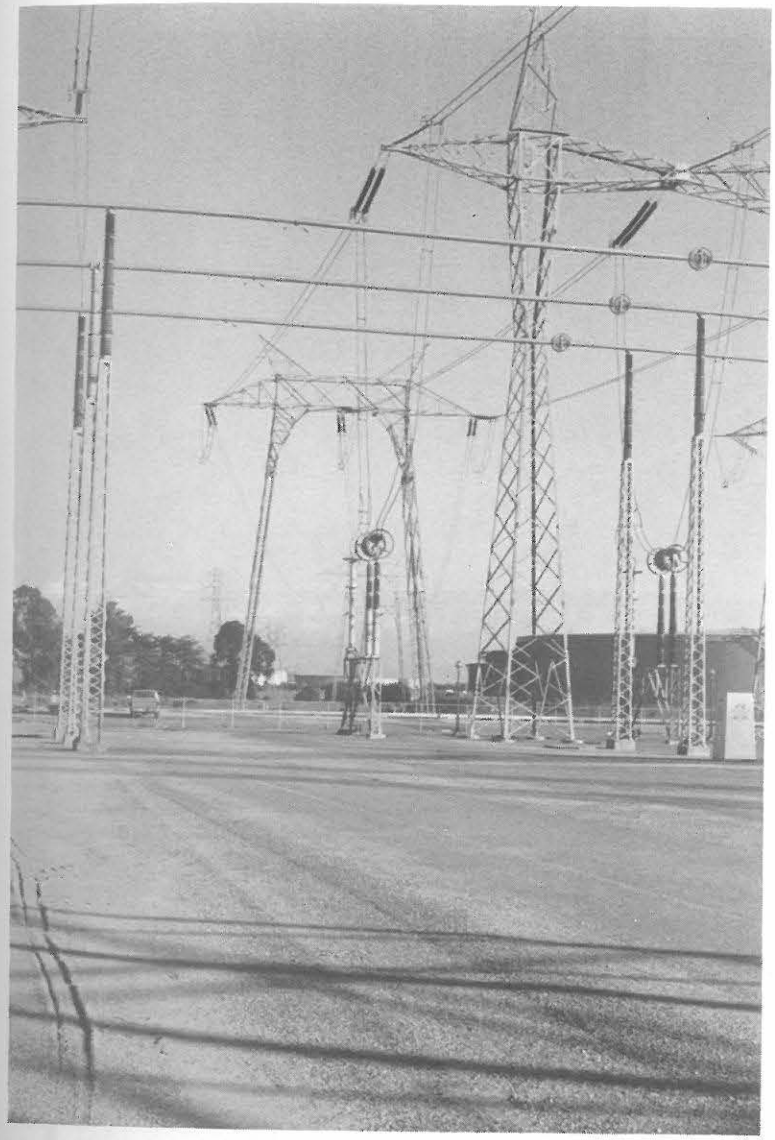


Fig. 5.26 Rigid busbar broken during the earthquake (photograph taken a day after repair)

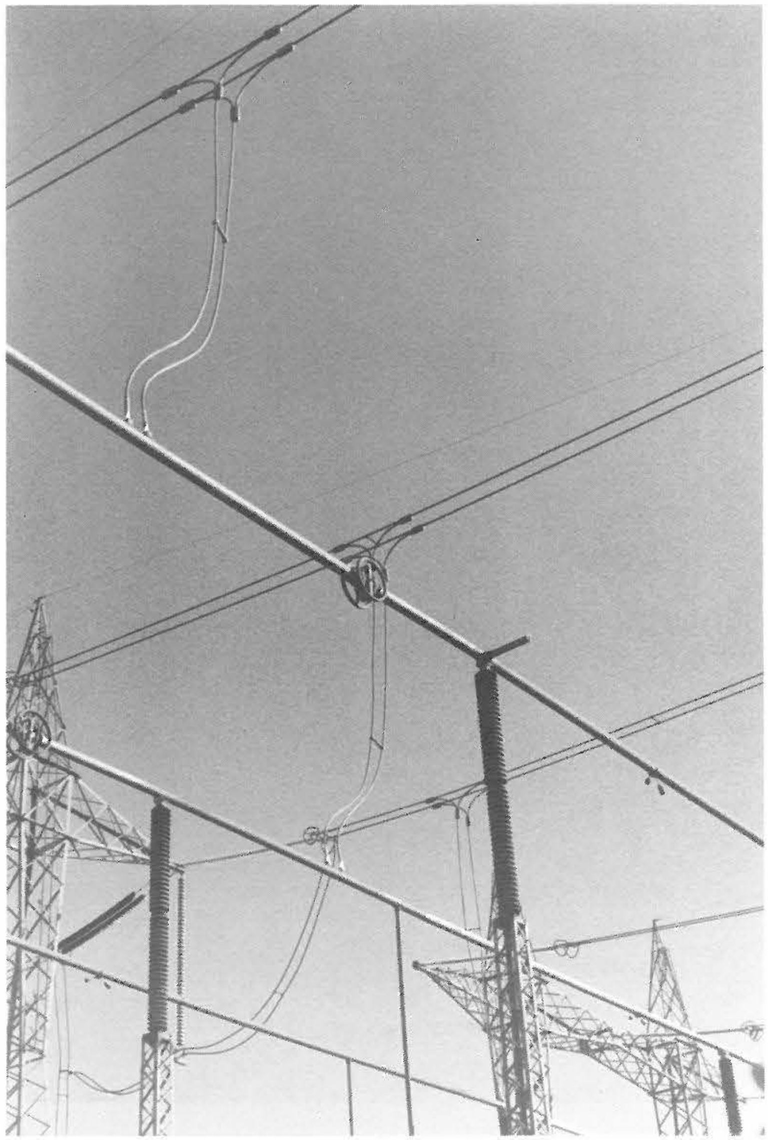


Fig. 5.27 Close-up of the rigid aluminum busbar

Fig. 5.28 Anchoring plate of the 500-kV SF6 circuit breaker support

Fig. 5.29 Anchor bolts of the 500-kV SF6 circuit breaker support

Fig. 5.30 Location of the base plate and of the anchored bolts (an old concept)

Fig. 5.31 Second generation concept of anchoring device

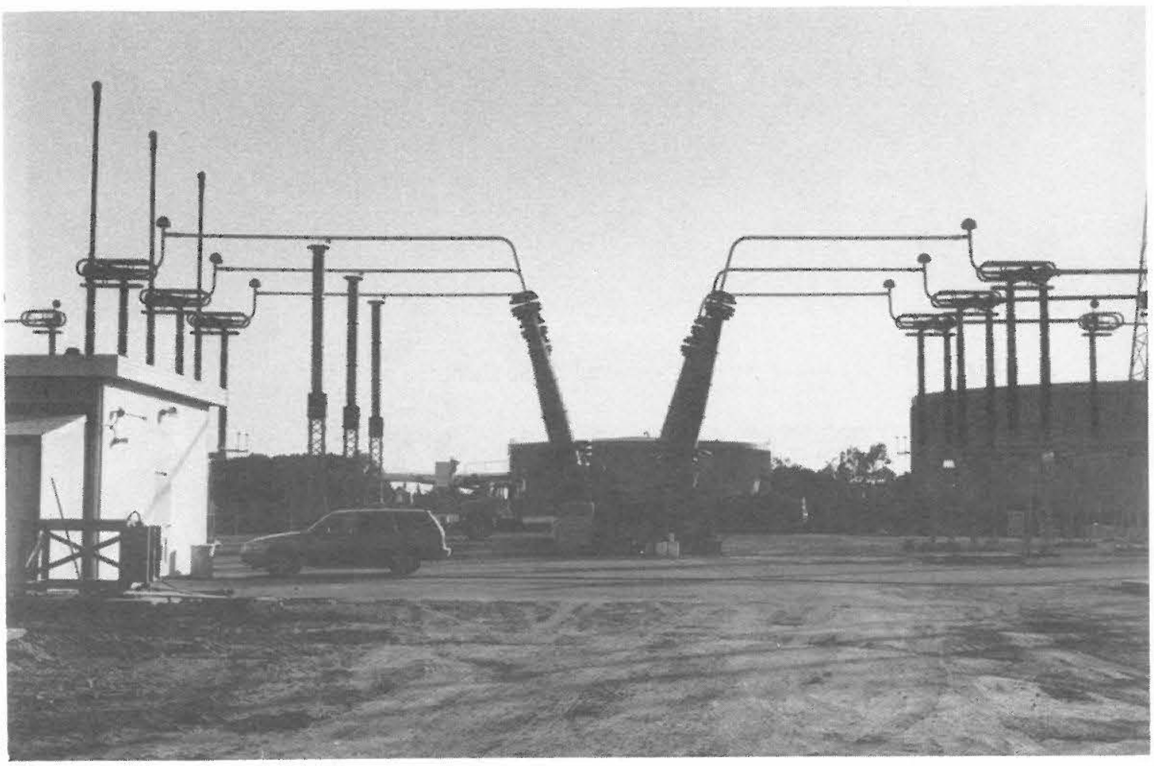


Fig. 5.32 New 500-kV equipment in Moss Landing substation

Fig. 5.33 New circuit breaker support

Fig. 5.34 New anchoring device concept of the 500-kV SF6 circuit breaker support

Fig. 5.35 Installation instructions for the anchors

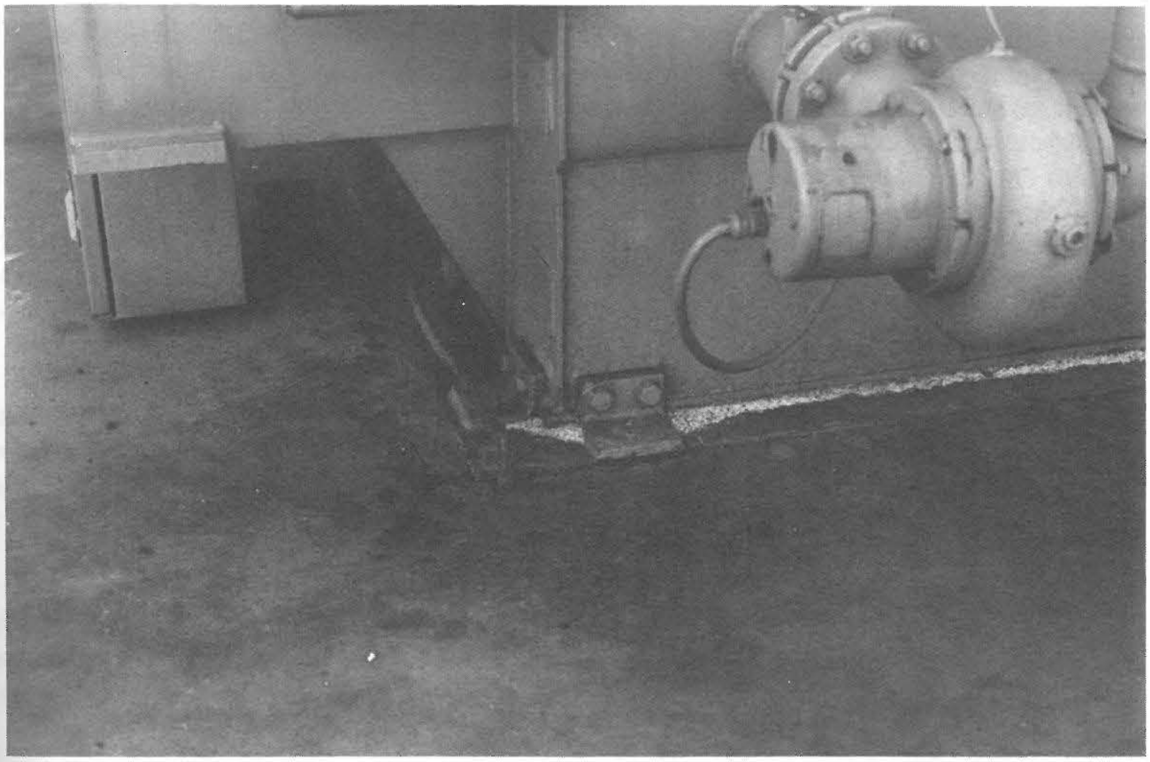


Fig. 5.36 Typical transformer anchorage

Chapter 6

INDUSTRIAL FACILITIES

by

D.T. Lau

6.1 INTRODUCTION

The San Francisco Bay Area is a major commercial and industrial center on the west coast of North America. Most notably is the famous "Silicon Valley" located in the South Bay regions of Santa Clara and San Jose, where many of the design and manufacturing companies in the computer and electronic industries are concentrated. Other important manufacturing and research facilities for various types of light and heavy industries and educational institutions, such as petrochemical, biotechnology and food processing, can be found throughout the San Francisco Bay Area. The epicenter of the October 17, 1989 earthquake was located 16 km northeast of Santa Cruz and 30 km south of San Jose, or approximately 110 km south of the city of San Francisco, in the Santa Cruz Mountains. Many industrial facilities throughout the region experienced strong ground motions from the main shock and the many aftershocks.

In general, most industrial facilities in the Silicon Valley area survived the earthquake with only minor or no property damage. However, work stoppage ranging from 24 hours to almost a week was widespread because of power outage, fallen shelves, and dislocation of equipment that was not properly secured. For many of these companies, the suspension of production accounted for most of the total financial loss related to the earthquake.

In Richmond, north of Oakland on the east side of the San Francisco Bay, three oil tanks with a capacity of 400,000 gallons ruptured at a petroleum refinery. But no oil spilled into the bay. The leaked oil was contained by a dike built around the tanks. The reconnaissance team was unable to arrange site inspection of these damaged facilities.

In areas closer to the epicenter (Santa Cruz, Watsonville, and other nearby towns), the industrial facilities are usually light industries associated with agriculture or food processing. In general, these facilities suffered notably more severe damage than the industrial facilities located further north in the San Francisco Bay Area.

Due to lack of time and other limitations, only specific cases of seismic damage to industrial facilities were investigated first hand during the visit. The following is a summary of the findings from those site visits.

6.2 WINERY FACILITIES

Several wine production facilities are located near the epicentral region. One winery in the town of San Martin, approximately 20 km east of the epicenter, suffered significant earthquake damage. It is estimated that this particular site probably experienced about 0.5-g peak horizontal ground acceleration from the main shock of the October earthquake. This estimate is based on several strong ground motion records obtained approximately 8 km south of the winery in the town of Gilroy [1,2]. The production facilities in the winery consist mainly of a number of industrial buildings and numerous stainless steel tanks of different sizes, located both indoor and outdoor.

The bottling and hospitality building is an old reinforced brick structure with a wooden truss roof, built in the 1930s (Fig. 6.1). This structure was extensively damaged during the 1984 Morgan Hill earthquake which had a magnitude 6.2 [3]. After that earthquake, the damage was repaired and the structure underwent a seismic retrofit program. The retrofit program primarily concerned the use of steel plate and long bolts to hold the bricks of the pilasters together, as shown in Fig. 6.2. The roof beam was also tied down to the steel plate of the pilaster, preventing the loss of support for the roof beam during earthquakes. The upgraded system performed well during the Loma Prieta earthquake. A 1/2-in. (1.2-cm) gap between the nuts and the steel plate around some pilasters was observed. The roof of the structure suffered some damage. Particularly, the ceiling tiles at the corner were cracked, and many of them had broken loose. Other damage inside the bottling building involved mostly broken or dislocated pipes and shifting of equipment.

Other important facilities of the production plant include a cold-room building and the adjacent barrel building. The cold-room building is a modern, single-storey, tilt-up structure built in 1979 and 1982. Inside the cold-room building, several dozen thin-walled stainless steel fermentation tanks are located. Many of them were damaged during the earthquake. Inside the barrel building, large number of wooden barrels are stacked on shelves. During the 1984 Morgan Hill earthquake, many of the barrels fell to the floor, presenting a major seismic hazard to the workers. As part of the retrofit program after that earthquake, the barrels were tied down, and the shelves braced with tie rods. The system performed also well during this earthquake and thus significantly reduced the hazard.

Many of the thin-walled, stainless steel tanks located outdoors just outside the cold-room building were damaged during the earthquake. The following is a more detailed description of the tank damage.

Outdoor Thin-Walled Stainless Steel Tanks

The outdoor thin-walled, stainless steel tanks are shown in Fig. 6.3. These tanks have slightly sloped bottoms for better drainage, and are anchored to the concrete foundation pad 2 to 4 feet (0.6 to 1.2 m) above the ground by anchor bolts. The storage tanks have cooling jackets around part of the cylindrical shells about 2 feet (0.6 m) above the base. Coolant is circulated through the jacket to maintain the liquid content at the proper temperature.

Many of the outdoor tanks suffered earthquake damage. The severity of the damage depended on the liquid content level at the time of the earthquake. In general, empty tanks suffered little or no damage. In other cases, the anchor bolts of many tanks were pulled off the concrete pad of the foundation (Fig. 6.4) as the tanks rocked in response to the sloshing liquid during the strong ground motions. In some cases, the anchored nuts were broken and the tanks "walked away" for a significant distance from the original positions (Fig. 6.5). Following the failure of the anchorage system, the high compressive axial stress from the rocking motion of the tanks buckled the tank shells (Fig. 6.4). It is significant to note that in most cases the presence of the cooling jacket confined the buckling to the region between the cooling jacket and the tank base.

Cold-Room Building Storage Tanks

Inside the cold-room building are several dozen anchored, thin-walled, stainless steel fermentation tanks of different capacities. At least two 25,000-gallon tanks suffered the commonly known "elephant foot" type of buckling, of which the characteristic is that the buckling is generally located just above the tank base and spreads around the entire or most of the circumference (Fig. 6.6).

Many of the older 19,000-gallon tanks nearby suffered the diamond-shaped type of buckling shown in Fig. 6.7. The locations of buckling were observed to be mostly near the bottom part of the second course of the cylindrical shells. Many of these tanks had been damaged during the 1984 Morgan Hill earthquake. Consequently, the bottom course of the damaged tanks were replaced by thicker ones. Probably because of these repairs, the Loma Prieta earthquake damage to these tanks has now shifted to the next weakest section at the bottom of the original second metal course. Several other 50,000-gallon tanks suffered a type of damage similar to that suffered by the 19,000-gallon tanks.

All the damaged tanks were filled to capacity at the time of the earthquake. The sloshing of the liquid induced an overturning moment at the tank base that was resisted by axial compressive stress in the tank shell. It was this high axial compressive stress that buckled the tank wall.

Inside the cold-room building, all the stainless tanks are anchored to concrete foundation pads which are about 12 inches (0.3 m) thick. Many of the anchorage systems failed, in that the anchor bars were pulled out of the foundation and the tanks ruptured (Fig. 6.8). Without the anchorage protection, evidence of rocking of the tank during the earthquake could be found in the crushing and spalling of some of the concrete foundation pads shown in Fig. 6.9. In one instance, the rocking of the tank was so severe that it caused some damage to the building because of the pounding of the tank against the building wall.

The catwalks supported from the tops of some of the tanks were severely damaged by the earthquake. Many of the supports broke loose and the entire walkways were on the verge of falling down. Signs of pounding between the catwalks and the tanks were observed.

Much of the PVC piping in the facility was broken or dislocated during the earthquake because of the motions or shifting of the connecting equipment.

6.3 FOOD PROCESSING FACILITIES

There are many food processing plants in the Watsonville area. An apple cider vinegar plant is located a few city blocks from downtown Watsonville, 11 km from the epicentre. A nearby strong motion record at the Watsonville Telephone Building showed that the site probably experienced a peak horizontal ground acceleration of about 0.39 g [1,2].

The main processing plant is an old single-storey structure, which suffered only little or no significant damage. However, much of the equipment and many containers inside the structure were tipped over, and many of the pipes were broken. On the outside, two fibreglass-reinforced plastic tanks were overturned. The tank bottoms were ripped open as shown in Fig. 6.10, while the tanks were completely sheared off the anchorage. In the plant, there were also four outdoor stainless-steel tanks storing vinegar. The observed damage of these tanks are presented in the following section.

Stainless-Steel Vinegar Storage Tanks

Fig. 6.11 shows the four thin-walled, stainless steel tanks. All four tanks are anchored to a continuous concrete foundation pad. Each tank is 28 ft (8.5 m) in diameter and 28 ft (8.5 m) high, and has a capacity of 130,000 gallons. The shell construction of all four tanks is similar. The bottom five courses of the cylindrical shell are constructed from 12-gauge steel plate, whereas the top two courses and the bottom plate consist of 14-gauge steel. Each steel course is 4 ft (1.2 m) high. All the tanks have a fixed conical roof.

One of the four tanks, constructed in 1982, was ruptured during the earthquake, losing its contents. The damaged tank suffered severe "elephant foot" buckling at the tank base all around the circumference (Fig. 6.12). The buckled region was confined within the bottom 30 cm of the tank wall. For approximately one third of the circumference, the buckled region had completely collapsed into a 15-cm wide flap. A second elephant foot buckling evidently occurred above the closed flap at these locations in Fig. 6.13. The damaged tank was full at the time of the earthquake. The other three tanks were 1/4 to 1/2 full at the time; they were not damaged. As a result of the uneven shortening of the buckled region, the damaged tank suffered significant permanent tilt after the earthquake.

A 1-1/2 in. (3.8-cm) angle ring is welded to the base of the tank. Six angle plates, each about 3-1/4 inches (8.2 cm) wide, anchored the damaged tank to the foundation, with one leg welded to the angle ring on the tank wall, and the other leg welded to a steel plate embedded in the concrete foundation. At least one of the welded angle plates was torn off the tank wall (Fig. 6.14), rupturing the tank shell and causing loss of the liquid contents.

The undamaged tanks have slightly different anchorage arrangement. Six angle pieces are welded to the tank wall that is reinforced by additional steel plates. The other side of the angle piece is welded to the embedded anchorage plate in the concrete foundation as shown in Fig. 6.15. This arrangement distributes the anchorage forces more evenly than the design of damaged tank. For the survival of anchored tanks in earthquake, it is important to design the details of the anchorage system properly.

The reinforced gusset plate above the drain pipe at the bottom of the damaged tank buckled as a result of the rocking of the tank shell.

6.4 WATER DISTRIBUTION SYSTEM

In general, the water distribution systems throughout the San Francisco Bay Area suffered only minor damage, mostly water pipe breaks due to ground movements. In the city of San Francisco alone, there were reports of about 30 breaks in addition to approximately another 50 to 70 breaks in the Marina District. The cost of the damage to the water lines was estimated at $0.5 to 1 million U.S.

6.5 WATER TREATMENT PLANT AND SEWAGE SYSTEMS

In Santa Clara County, the Rinconada Water Treatment Plant in the city of Los Gatos, about 30 km north of the epicenter, reported that one of its two 66-inch input mains were damaged, and was subsequently shut down. The other input main was undamaged and kept operating after the earthquake. The damage to the input main was properly due to the ground deformations as the failure occurred at the location where the water line crossed over the San Andreas fault line.

In hard-hit Santa Cruz County, 5 million gallons of raw sewage a day were dumped into the ocean because of broken pipes in the sewage system. The sewage system in Monterey County was seriously damaged. Consequently, water supply to large areas may have been contaminated.

6.6 WATER STORAGE RESERVOIR

In the northeastern part of the town of Watsonville, about 11 km south of the epicenter, the Amesti Reservoir, shown in Fig. 6.16, is an unanchored steel tank with a capacity of 1 million gallons. It supplies drinking water to area residents. The diameter of the reservoir is 80 ft (24.4 m); its height, 25 ft (7.6 m). The reservoir has a knuckled, conical, fixed roof supported by a centre column. The bottom plate is 1/4 in. (6.3 mm) thick. The tank shell consists of three courses of steel plate. The bottom course has a thickness of 0.292 in. (7.4 mm) and the top two courses have a thickness of 1/4 in. (6.3 mm). The design and construction of the reservoir conformed to the America Water Works Association Standard AWWA D100 specifications at the time. The reservoir was erected in 1971.

The foundation has an 8-in. (20-cm) gravel base on top of 12 in. (30 cm) of compacted sub-base. Asphalt paving was then applied on top of the gravel base.

At the time of the earthquake, the water level was at 23 ft (7 m), 2 ft (0.6 m) below the top of the tank wall. Apparently, the waves generated during the earthquake from sloshing of the water inside the reservoir damaged the reservoir roof because of the impact. The hydrodynamic force also caused uplift and rocking of the reservoir. As a result of the pounding of the tank wall onto the foundation, as much as 3 in. (7.6 cm) of settlement was observed at some locations around the tank base (Fig. 6.17). The movement of the reservoir also sheared off a small connecting pipe near the base. The tank shell survived the earthquake without any sign of buckling or distress. However, the roof access ladder attached to the side of the tank wall fell off but was easily repaired. Cracks in the asphalt paving were observed.

REFERENCES

[1] "Second Quick Report on CSMIP Strong-Motion Records from the October 17, 1989 Earthquake in the Santa Cruz Mountains," Department of Conservation, Division of Mines and Geology, Office of Strong Motion Studies, Sacramento, California, October 25, 1989.

[2] R. Maley, A. Acosta, F. Ellis, E. Etheredge, L. Foote, D. Johnson, R. Porcella, M. Salsman and J. Switzer - "U.S. Geological Survey Strong-Motion Records from the Northern California (Loma Prieta) Earthquake of October 17, 1989", Department of the Interior, U.S. Geological Survey, Open-File Report 89-568, October 1989.

[3] "Preliminary Report on the Effects of the April 24, 1984, Morgan Hill, California, Earthquake", EQE Incorporated, San Francisco, California, June 1984.

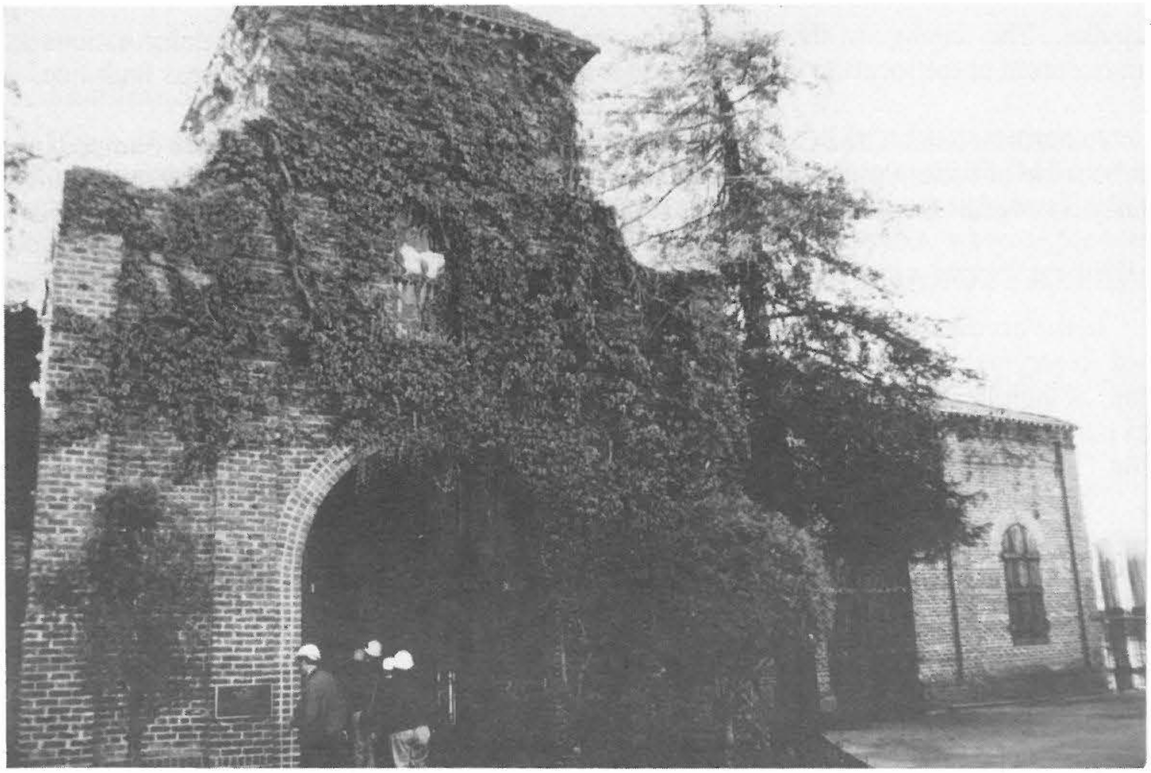


Fig. 6.1 The winery bottling and hospitality building

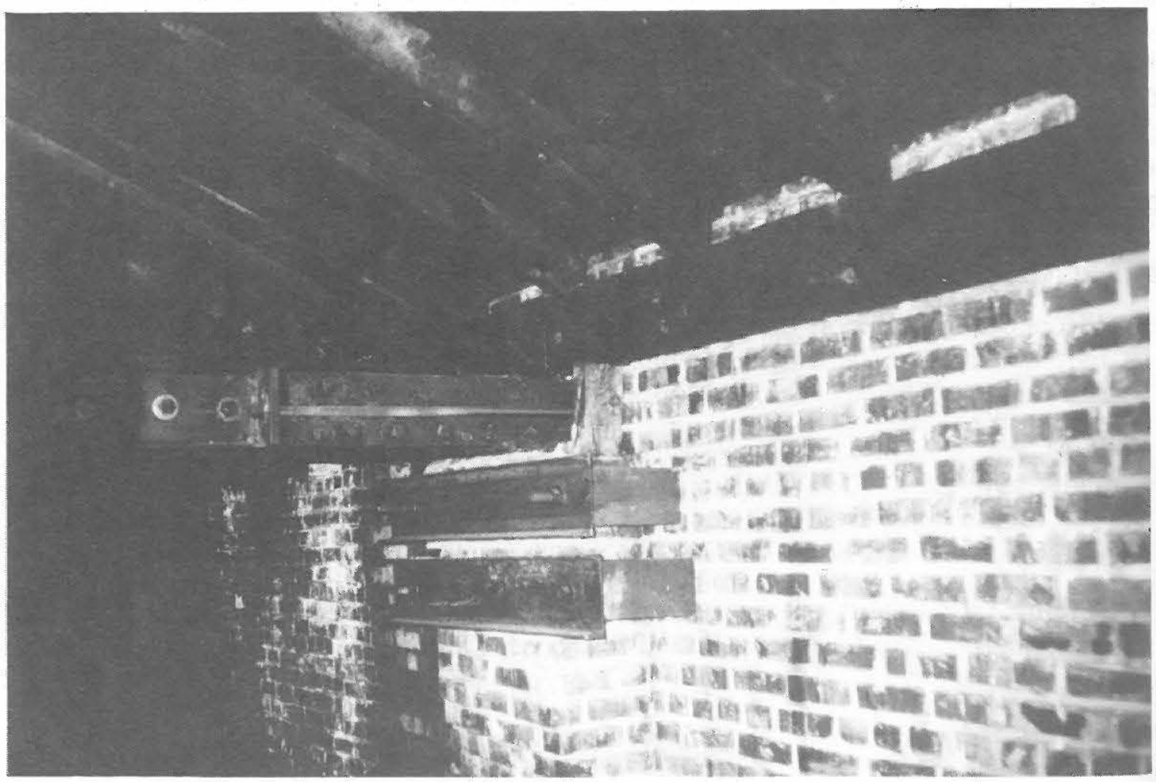


Fig. 6.2 The retrofitted roof beam - pilaster connection

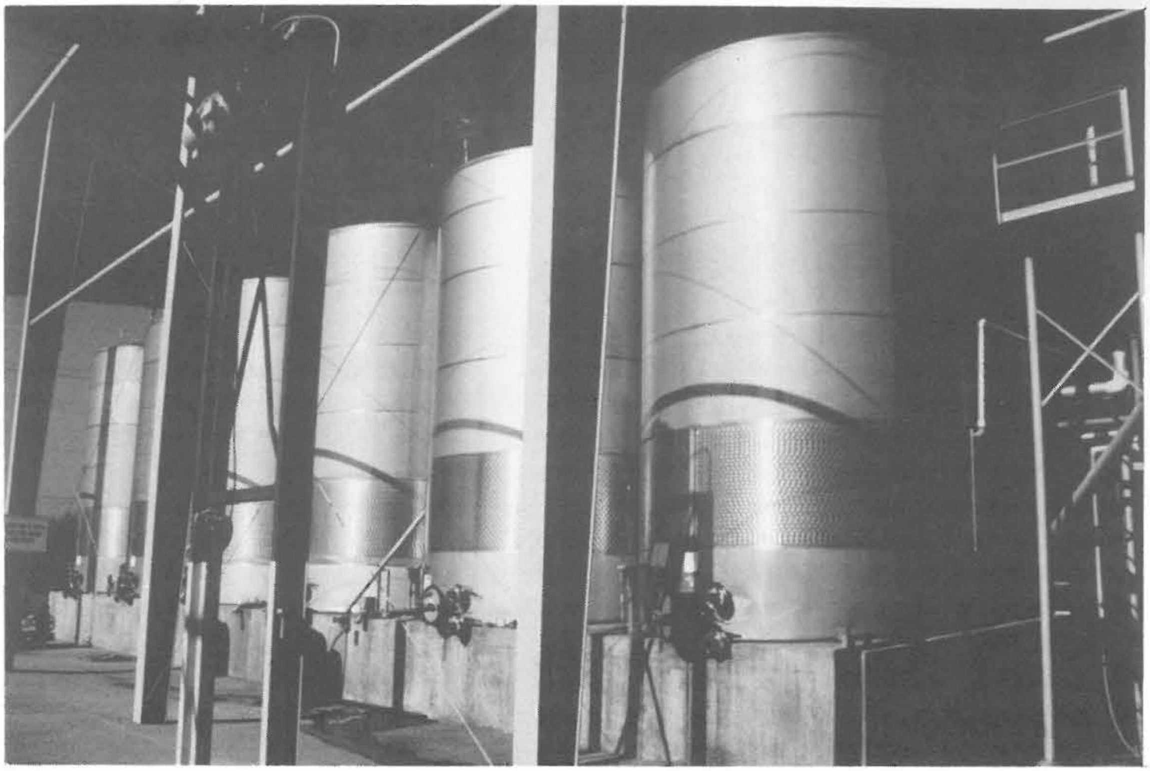


Fig. 6.3 The outdoor stainless steel tanks with cooling jackets

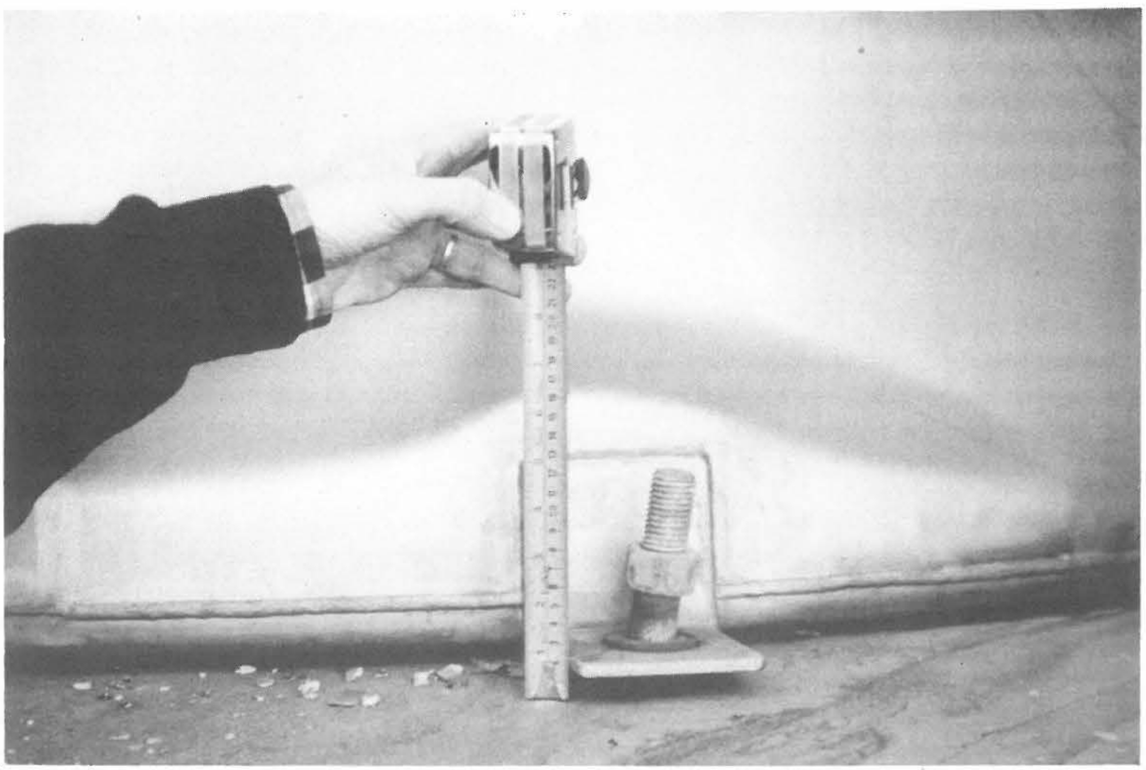


Fig. 6.4 Pulling out of the anchored bolt

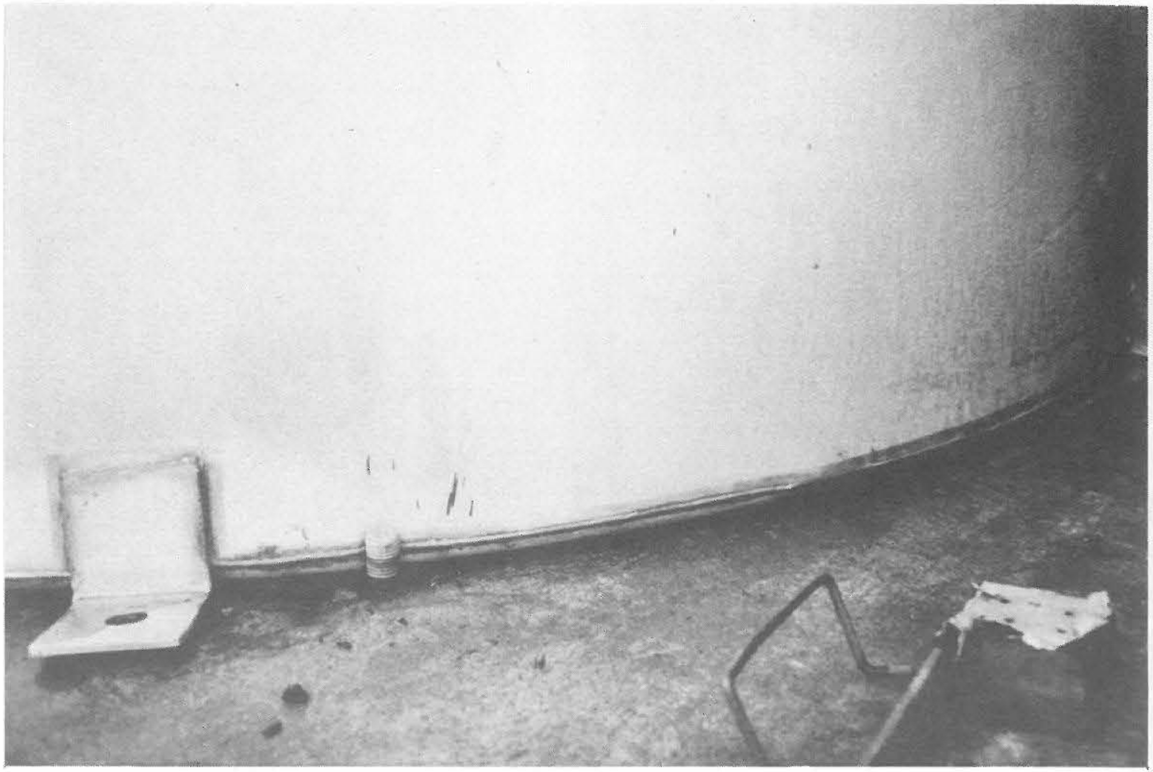


Fig. 6.5 Walk off of the tank

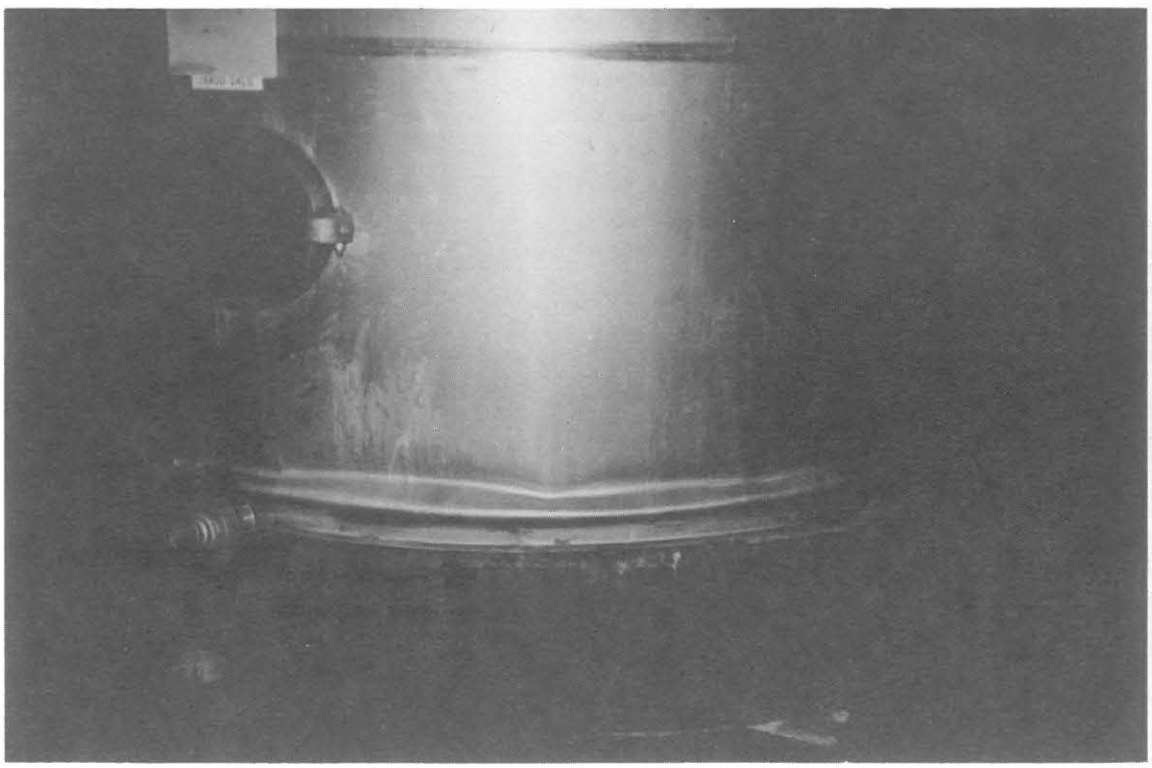


Fig. 6.6 "Elephant-foot" buckling of tank

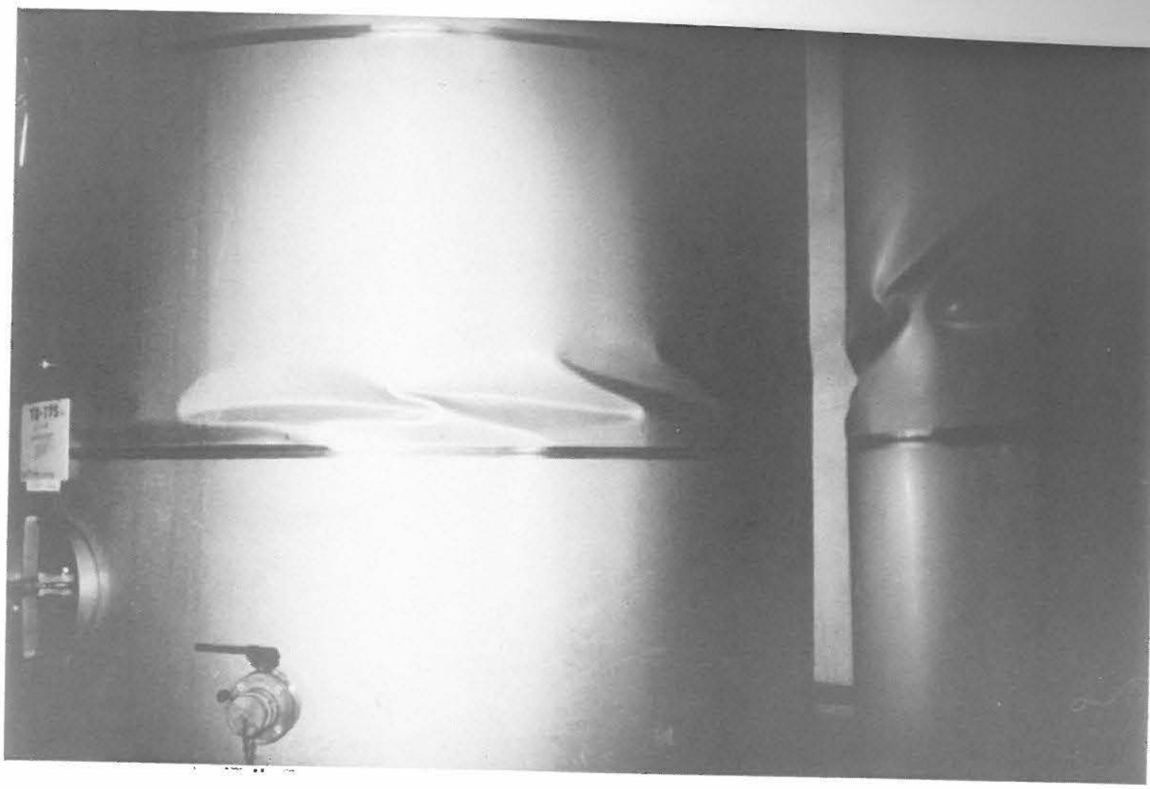


Fig. 6.7 Diamond shaped buckling of tank

Fig. 6.8 Failure of anchor bars and rupturing of the tank

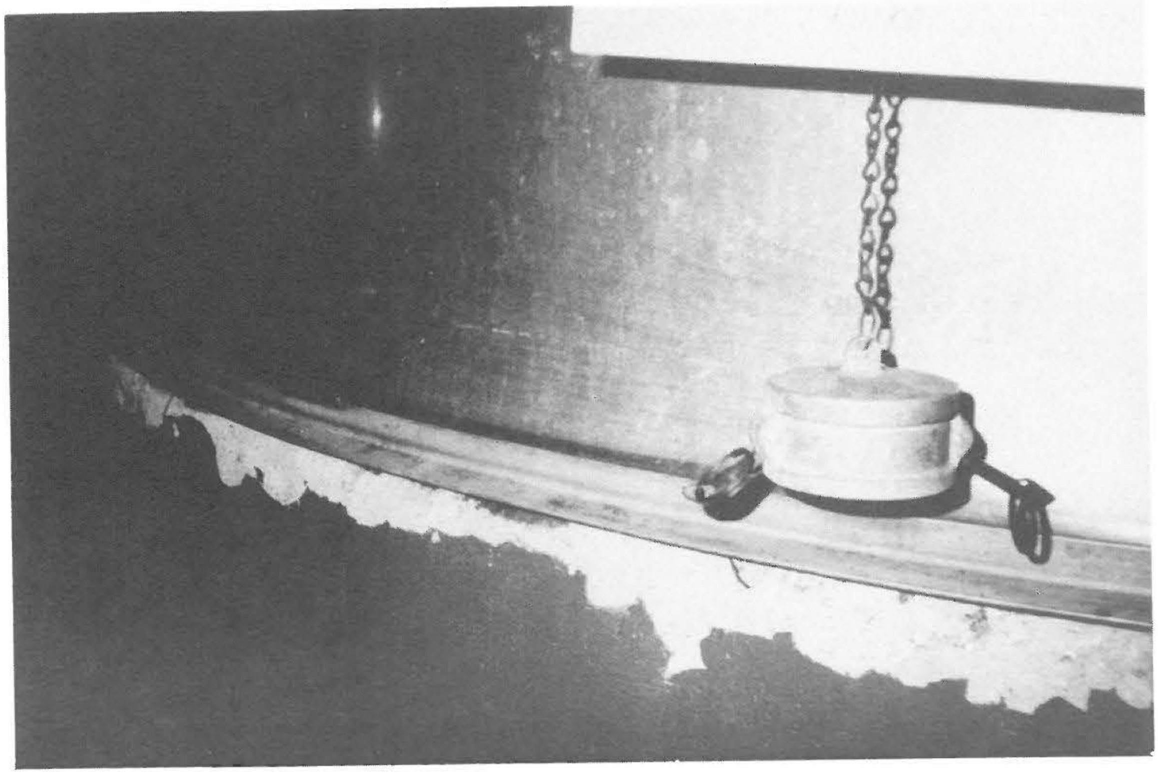


Fig. 6.9 Spalling of the concrete foundation pad due to rocking of tank

Fig. 6.10 Damaged fibreglass-reinforced plastic tanks

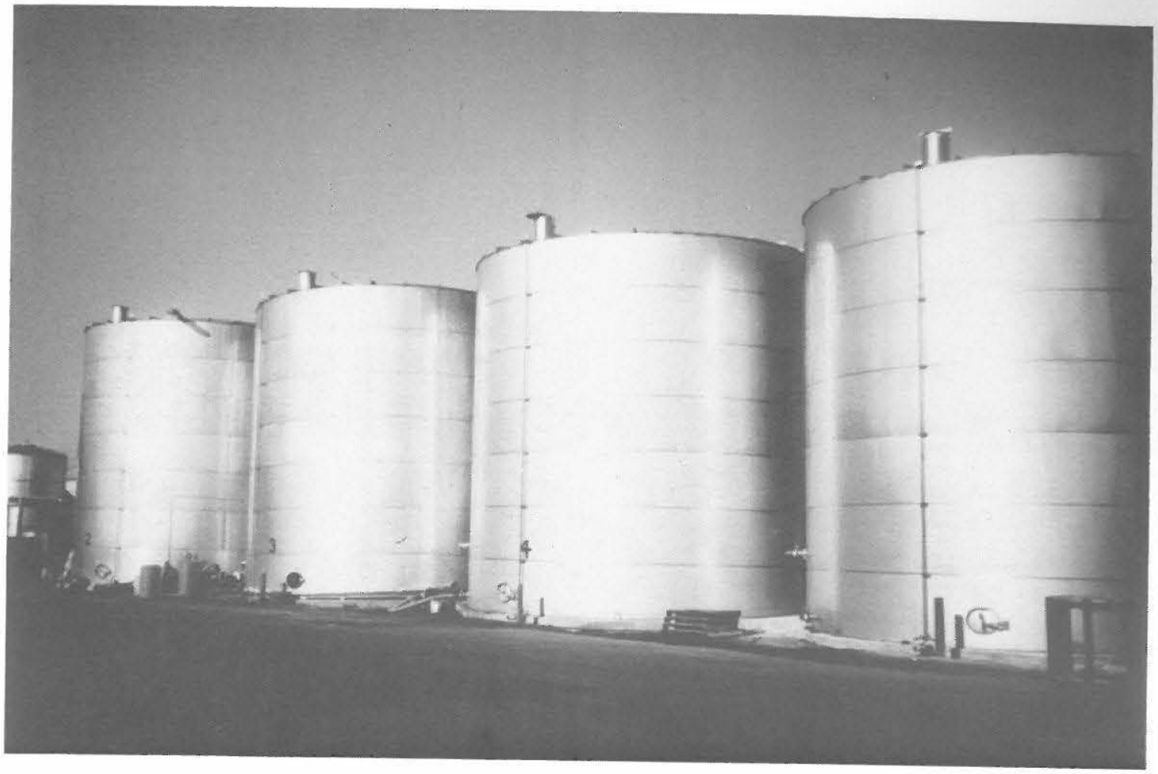


Fig. 6.11 Stainless steel tanks storing vinegar at food processing plant

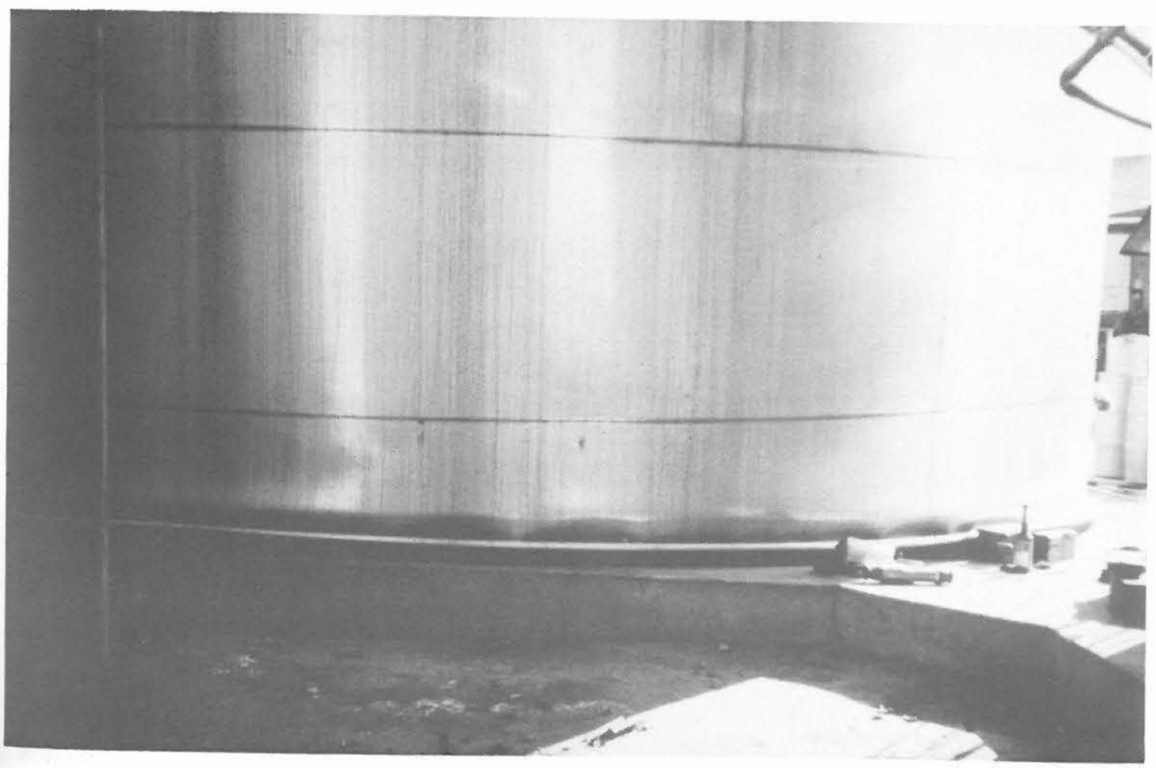


Fig. 6.12 "Elephant-foot" buckling of tank

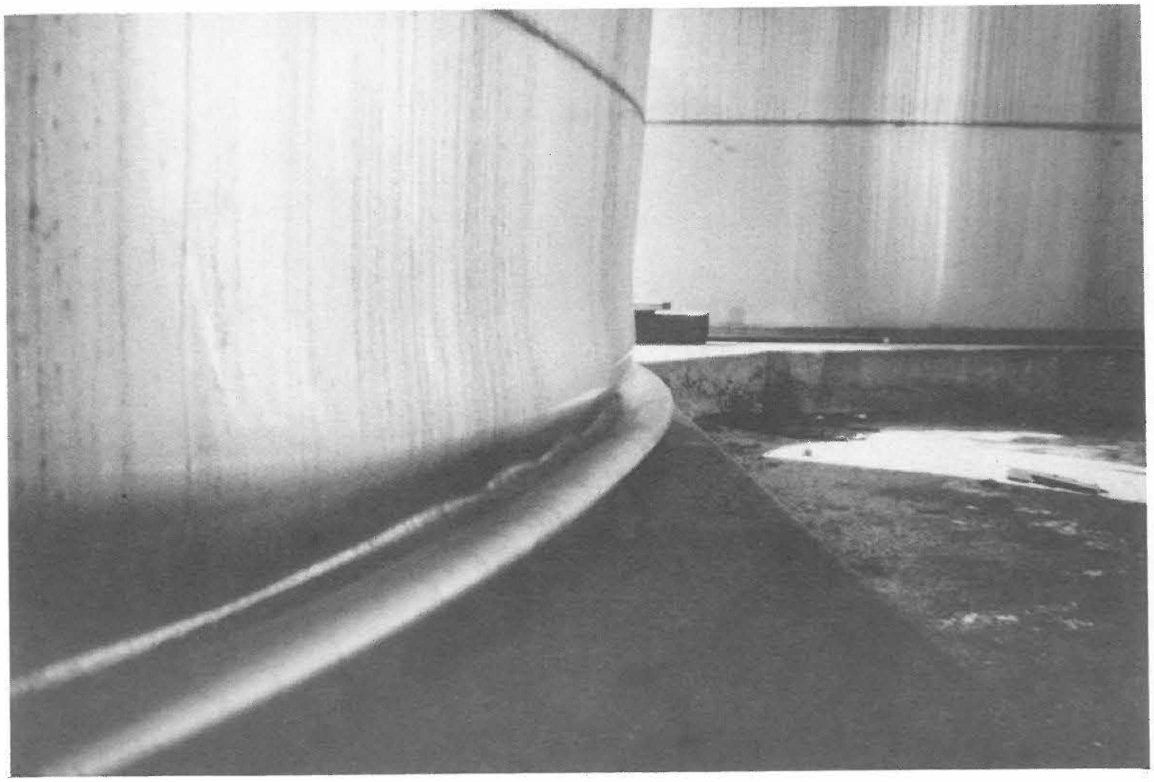


Fig. 6.13 "Second elephant-foot" buckling of tank

Fig. 6.14 Failure of welded angle plate

Fig. 6.15 Anchorage of undamaged tank

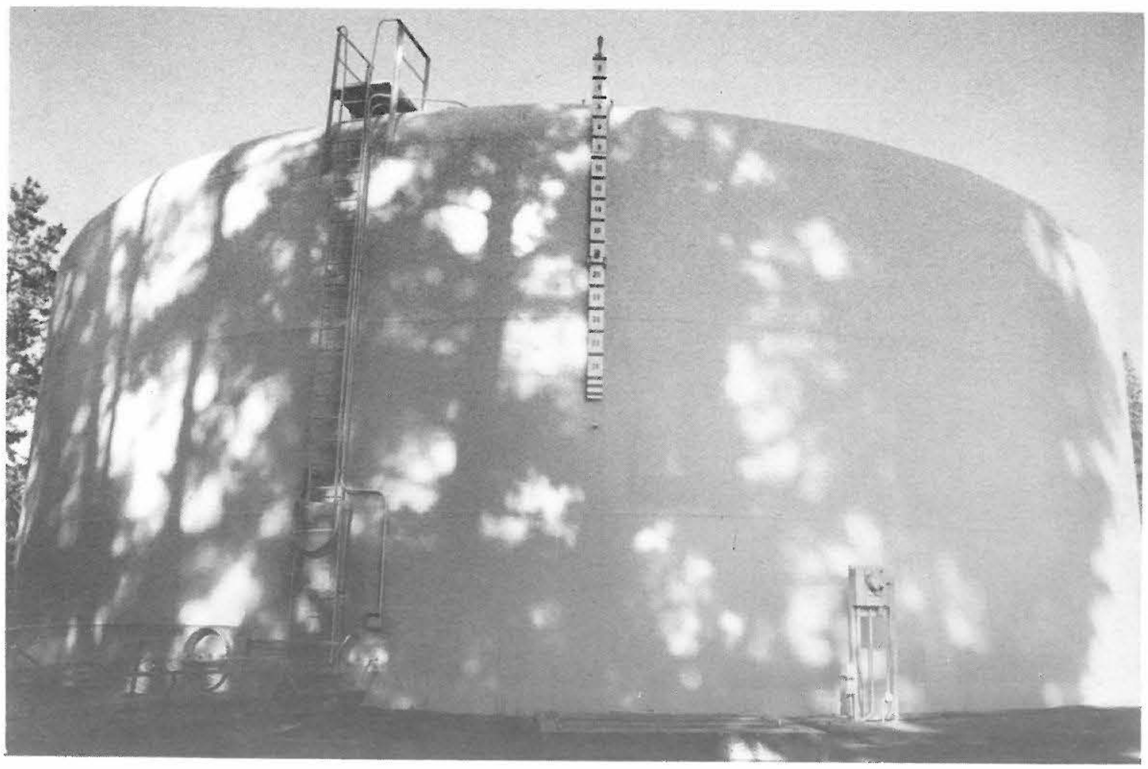


Fig. 6.16 The Amesti Reservoir in Watsonville

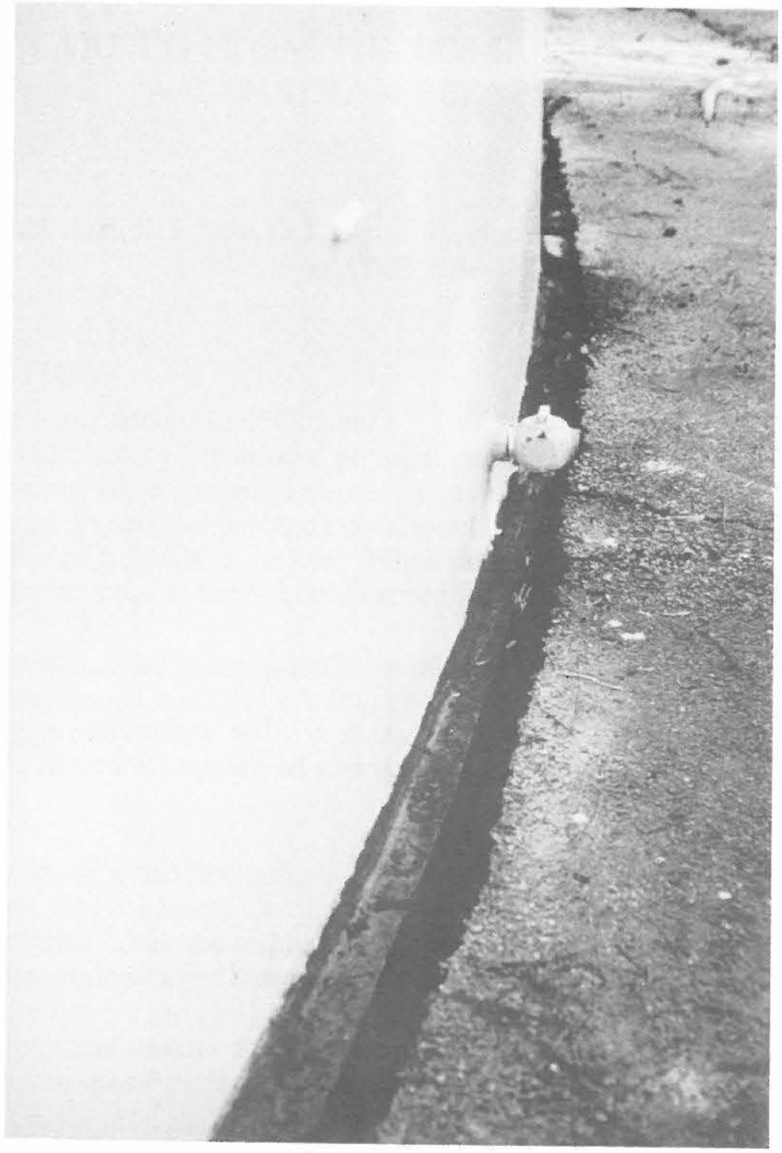


Fig. 6.17 Evidence of rocking of reservoir and support settlement

Chapter 7

LESSONS LEARNED FROM THE LOMA PRIETA EARTHQUAKE - A CANADIAN PERSPECTIVE

by

A.M. Jablonski, K.T. Law, D.T. Lau, J.-R. Pierre, J.H.K. Tang, J.H. Rainer and D.E. Allen

7.1 INTRODUCTION

The Loma Prieta earthquake of October 17, 1989, has already been evaluated as the largest natural disaster in U.S. history. Estimated physical damage (replacement of structures that have been damaged, repair of some structural elements) has exceeded the $8 billion in losses resulting from Hurricane Hugo. Functional damage may also include psychological stress and disruption of the routine in the life of millions of people. When these types of loss are added to the physical damage, the earthquake-induced losses could exceed $10 billion.

Earthquake engineering is a new multidisciplinary branch of engineering which encompasses also major environmental aspects, including human behaviour during disaster conditions. This earthquake has triggered research tasks in many areas which have played a significant role in reducing of seismic hazard of engineered structures as that of well as non-engineered, residential housing.

An earthquake of similar magnitude can happen in both highly active seismic regions in Canada, near Quebec City in Eastern Canada and on the West Coast, including the highly populated Greater Vancouver Area. Thus, the results of this site reconnaissance study should be of particular interests to Canadian engineers and researchers involved in earthquake engineering studies.

In this chapter the several findings are discussed with respect to Canadian scene and Canadian codes. Some specific items will require more detailed studies.

The most important characteristics of the effects of the Loma Prieta earthquake can be summarized as follows:

1. the significant effect of soil conditions leading to a very variable pattern of ground motions, including two sudden increases in amplitudes reported by stations in Presidio and near the Golden Gate Bridge as well as in downtown Oakland, about 100 km from the epicenter;
2. the damage (and in some cases complete collapse) of many non-engineered, residential structures, especially in the Marina District in San Francisco caused by a "soft-storey" effect and liquefaction of soft soil deposits;
3. the dramatic collapse of the reinforced-concrete Cypress Street viaduct of Interstate 880 in Oakland, a double-deck structure, and the damage observed in other similar structures in San Francisco, and

4. many examples of liquefaction and liquefaction-induced damage on fills around the San Francisco Bay Area and some on the Pacific Coast in the epicentral area.

7.2 BUILDINGS

Two major types of affected structures can be recognized: a) engineering structures and b) residential, non-engineered housing. For the purpose of comparisons, the 1985 versions of the National Building Code of Canada (NBC) and the Uniform Building Code (UBC) will be used. Some potential deficiencies of NBC are noted.

Engineering Structures (Part 4, NBC)

For the San Francisco area, the 1985 UBC design base shear (V) for low-level buildings results in a base shear of V = 0.12 W, where W = weight of building. For a comparable building in Vancouver, the NBC (1985) prescribes a base shear V = 0.088 W. For tall buildings (example period T = 2s), V_{UBC} = 0.047 W, and V_{NBC} = 0.031 W. Since both codes use a load factor of 1.5, it can be seen that the ratio of design forces for Vancouver to those of San Francisco is about 2/3. Thus, the design earthquake forces stated in the NBC (1985) for Vancouver would be slightly less than those for a Zone 3 requirement stated in the UBC, which in turn is 0.75 that of San Francisco. Also, it should be pointed out that neither the NBC nor the UBC deals with the problems related to liquefaction directly, but both codes consider it an area deserving special attention [1, 2].

Residential, Non-Engineered Housing (Part 9, NBC)

Non-engineered housing includes single-family houses and multi-unit dwellings up to and including 3 or 4 storeys in height and less than 600 m^2 in floor area.

The following table compares the requirements for residential buildings given in Part 9 of the NBC (1985) and to the requirements in the UBC (1985). The requirements are similar, except that some earthquake requirements included in the UBC are missing from the NBC.

Table: Comparisons of Earthquake Code Requirements for Residential Construction

Requirement	UBC	NBC, Part 9
Wall Bracing (in-plane)	2517(g)3	not covered except for post-and- beam construction Part 4 via 9.24.1.5
Cripple Stud Foundation Walls	2517(g)4	Part 4 via 9.15.1.5
Anchorage to Foundations	2907(f)	9.23.6
Beam Splice Ties over Supports	2517(c)	not covered
Lateral Support of Masonry Walls	2407(e)	9.20.10 and 11
Anchorage of Masonry Veneer	3006	9.20.9.9
Reinforcing of Masonry	2407(h)4B	9.20.17
Anchorage and Reinforcing of Masonry Chimneys	3704(c)	not covered
Stability of Masonry Parapets	2312 Table 23-J	9.20.6.7

The experience of the Loma Prieta earthquake indicates that the most serious deficiency in Part 9 of the NBC (1985) is the lack of any requirements for wall bracing in wood-frame construction. Ground floors of 2- or 3-storey residential buildings containing large openings like garages are vulnerable. Other potential deficiencies in Part 9 of the NBC (1985) include the need for tying ends of beams over supports and anchorage of masonry chimney to the roof and floors. Collapses due to these deficiencies have occurred in this and other earthquakes.

Lateral collapse of foundation cripple-stud walls (weak in racking resistance) was also a serious failure mode, especially in the epicentral area. This is covered by Part 4 in the NBC lateral force requirements via Subsection 9.15.15 (for wood structures) and Article 9.4.1.1 (for other construction not specified in Part 9). Many designers or builders, however, may not be fully aware of this.

Structural failures that occurred in residential buildings in the San Francisco or in the epicentral area were due to deficiencies that are prohibited by the UBC. Although the earthquake intensity was less than the design earthquake, except near the epicenter, experience indicates that the present UBC requirements appear satisfactory. Since Part 9 of the NBC has a similar requirement, the detailed causes for the failures should be investigated.

The main problems in Canada, as in other countries, concern the safety of existing older buildings with serious deficiencies, such as unreinforced masonry, non-ductile concrete structures and wood frame structures with weak ground stories. Future investigations of case studies of damaged reinforced concrete structures and some of retrofitted older buildings could calibrate respective requirements. The seismic evaluation and possible upgrading of existing Canadian building inventory in seismic areas needs attention.

7.3 SOIL CONDITIONS

The 1989 Loma Prieta earthquake strongly demonstrated again that three conditions occurring simultaneously can create a major seismic disaster: a significant earthquake, a thick deposit of compressible soil, and a high population density. Inadequate structural design, of course, will add to the disaster.

The ground shaking during this earthquake in the cities of San Francisco and Oakland was considered light to moderate because of the considerable distance from the epicentre. Except at the surface of thick compressible soil layers, the maximum peak acceleration was about 0.1 g. Such an acceleration is possible in Canada, with a return period of 125 years and 170 years for Vancouver and Quebec City, respectively. Accelerations with these return periods are more common than the ones (return period of 475 years) used in the seismic zoning map in the National Building Code of Canada, 1985. While structures designed according to this code may survive an earthquake with an intensity similar to the one experienced in San Francisco, structures built to requirements below the current code may suffer damage. The fact that San Francisco suffered severe damage during this earthquake, in spite of an existing stringent local code, should be of concern for Canada.

Thick deposits of compressible soils are also found in seismic regions in Canada. Some are located in major population centres. The most prominent examples are the Fraser River Delta in British Columbia and Quebec City in the province of Quebec. Both are situated on thick alluvial deposits which may amplify ground motions and may potentially liquefy during an earthquake. Richmond is situated on the Fraser Delta where the maximum thickness of alluvial deposit may reach 600 m. The lower town of Quebec City is situated on the St. Charles River Basin with the alluvial deposits reaching a depth exceeding 50 m. These two cities, therefore, satisfy all three conditions for an earthquake disaster. While the seismic risk in Richmond is well recognized, the seismic risk in Quebec City is not as well appreciated, but was underlined by the 1988 Saguenay earthquake of magnitude 6.0. Significant damage in Quebec City, 150 km from the epicentre, was confined to the lower town area. This earthquake has also caused extensive liquefaction failure leading to significant damages in Ferland and Boileau, Quebec [3].

Seismic hazards, therefore, are real both in the western and eastern regions of Canada. With the extensive records obtained through monitoring the 1989 Loma Prieta event and using available interpretative skills, earthquake engineers will be able to learn important lessons applicable to Canadian conditions, ultimately to reduce the hazard potential of future earthquakes.

7.4 POWER GENERATING FACILITIES

Damage to power facilities due to the earthquake was typical of those that have been observed in other major earthquakes. The earthquake-induced failures are mainly limited to some high voltage electrical equipment. Most engineered structures, piping and industrial equipment performed fairly well during the earthquake with very little permanent damage.

The Moss Landing Power Plant had probably experienced twice its seismic design level and survived with minimal damage, demonstrating the inherent margins of similar installations.

It was again shown that conventional power transmission would be susceptible to earthquake-induced failures. The need for safeguarding power loss due to a major earthquake would have to be considered based on economics and ability of the existing transmission network to restore power to the community, if required, under emergency situations similar to those in San Francisco.

7.5 TRANSMISSION FACILITIES

The information obtained from the Loma Prieta earthquake (M=7.1) is very useful and instructive since the major tremor predicted for the Charlevoix-Kamouraska area could reach a magnitude of 7 to 7.5. At present, the electrical apparatus bought for this area is required to withstand local seismic acceleration values from 0.23 g in Quebec City to 0.70 g in the centre of the Charlevoix area. These acceleration values prove to be very realistic, since the maximum acceleration recorded near the Loma Prieta quake epicentre was 0.64 g.

The Loma Prieta earthquake proved once again the vulnerability of ordinary equipment to seismic forces. EHV equipment suffered most damage. It should be remembered that no 735-kV substations were affected by the Saguenay earthquake. These new lessons prompt us to pay special attention to this equipment and especially to the seismic resistance of rigid aluminum busbars, particularly in 735-kV substations designed between 1965 and 1970, such as Lévis, Laurentides, Jacques-Cartier which have been designed for a short-circuit value of 20 KA which seems more critical compared to the seismic loads. This point should however be studied seriously for all new 735-kV substations which will be built in the Charlevoix-Kamouraska area.

It is quite remarkable to note how accurately the high-seismic risk areas are identified and how the probability of earthquake occurrence can be predicted in California. It is true that the seismological context of the northeastern North America intraplate earthquake does not lend itself to such accuracy. Nonetheless, considering the high stakes involved, efforts should be made to try to locate relatively accurately the epicenters of quakes of magnitude 5.6 to 6.5 that occurred in the Montreal region in 1732, 1816, 1897. Did these tremors originate from Montreal Island itself or from the surrounding areas? Some answers would greatly help to take this aspect into account for the planning and siting of electrical installations.

It should be stressed, however, that the probability of a Hydro-Quebec electrical installation being hit twice by significant quakes during its useful life span (50 years) is low, except in the Charlevoix-Kamouraska region. From this standpoint, there is no comparison with California. Nevertheless, it is quite probable that the overall power system will be hit in different places over a relatively short period of time. Over a 20-year period, i.e., 1924 to 1944, Quebec was shaken by five earthquakes of magnitudes 5.8 to 7: 1924 and 1925 in Charlevoix, 1935 in Temiscamingue; 1939 in Charlevoix; and 1944 in Cornwall-Massena and St-Regis area [4].

REFERENCES

[1] Uniform Building Code, International Conference of Building Officials, Whittier, California, 1985.

[2] National Building Code of Canada (NBC), National Research Council of Canada, Ottawa, Ontario, 1985.

[3] M. Tuttle, K.T. Law, L. Seaber, and K. Jacob, "Liquefaction and ground failure in Ferland, Quebec triggered by the 1988 Saguenay Earthquake." Accepted for publication in the Canadian Geotechnical Journal, Vol. 27, No. 5, 1990.

[4] J.-R. Pierre -"General aspects of the damage caused to civil works and electrical transmission systems by the Loma Prieta (Santa Cruz) earthquake relative to Eastern Canada's and Quebec's seismic risk," Hydro-Quebec, Montreal, February 1990.