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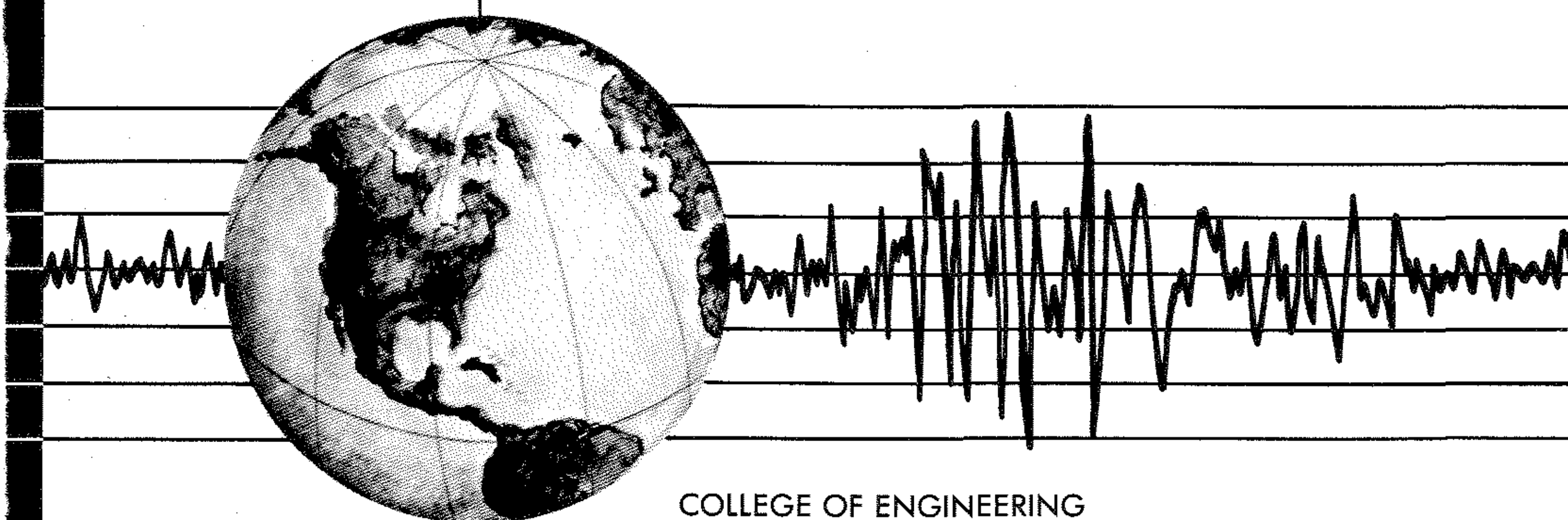
EARTHQUAKE ENGINEERING RESEARCH CENTER

PRELIMINARY REPORT ON THE PRINCIPAL GEOTECHNICAL ASPECTS OF THE OCTOBER 17, 1989 LOMA PRIETA EARTHQUAKE

Preliminary findings from field investigations and associated studies performed by teams from the University of California at Berkeley, as well as by a number of cooperating organizations, researchers and local professional engineers, immediately following the earthquake.



COLLEGE OF ENGINEERING

UNIVERSITY OF CALIFORNIA AT BERKELEY

PRELIMINARY REPORT ON

THE PRINCIPAL GEOTECHNICAL ASPECTS OF

THE OCTOBER 17, 1989 LOMA PRIETA EARTHQUAKE

by

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Report No. UCB/EERC-90/05

Earthquake Engineering Research Center
College of Engineering
University of California at Berkeley

April 1990

Dedication:

This report is dedicated to the memory of

Professor Harry Bolton Seed

who would have loved to have been involved

in these studies . . . and who was.

PREFACE

This report has been prepared in response to the widespread interest in the Loma Prieta Earthquake of October 17, 1989. It contains preliminary information and observations regarding the principal geotechnical engineering aspects of the earthquake. Field investigations are still in progress and additional laboratory and analytical studies have also been initiated. Thus, more definitive conclusions regarding the earthquake and its effects will be developed during the months ahead. These results and observations will continue to be reported in subsequent publications.

It is clear from the data available at present that the Loma Prieta Earthquake represents the most costly natural disaster in U.S. history. Damage includes (a) widespread landslides in the epicentral region, (b) liquefaction and other soil failures in numerous regions surrounding the San Francisco Bay and elsewhere, (c) structural distress and failures in commercial and residential buildings, transportation and other lifeline facilities, and industrial structures, (d) damage to nonstructural elements and contents of structures, and (e) widespread disruption of utilities. In addition, 62 deaths have been directly attributed to the earthquake.

Geotechnical factors exerted a major influence on damage patterns and loss of life in this catastrophic event. The vast majority of damage to structures and other facilities occurred on sites underlain by deep soil deposits which amplified shaking levels at these locations. This includes the sites of the collapsed San Francisco/Oakland Bay Bridge section, the collapsed Cypress/Interstate 880 elevated highway, and the heavily damaged Pacific Garden Shopping Mall in Santa Cruz, as well as other heavily damaged regions in San Francisco, Oakland, Alameda and elsewhere. More than 50 of the 62 deaths attributed to the earthquake occurred on sites underlain by deep, and typically cohesive, soil deposits. Away from the immediate vicinity of the sparsely populated fault rupture zone, damage to structures and facilities on sites underlain by rock or stiff, shallow soil deposits, even in densely populated areas, was relatively light.

Additional significant geotechnical features of this earthquake included: (a) widespread liquefaction which resulted in significant damage to bayshore areas on both sides of the bay in the densely populated central San Francisco Bay Area, as well as farther south along the Pacific coast in the Santa Cruz/Monterey Bay region, and (b) widespread landslides and rockfalls which closed roads and caused considerable damage in the region near to the zone of fault rupture in the relatively sparsely populated Santa Cruz Mountains. Slides and rockfalls also occurred along the Pacific Coast, and small slides and rockfalls were observed at distances of up to 70 miles from the epicentral region. In addition, a number of major earth and rockfill dams exist in close proximity to the fault rupture region and so were strongly shaken. A few of these sustained minor to moderate damage, but none showed any signs of potential instability such as might precipitate a reservoir release. In general, the performance of major earth and rockfill dams, in the face of strong levels of shaking, was very good.

Finally, it should be noted that the fault rupture events of October 17, 1989 occurred on a segment of the San Andreas Fault system fortuitously located in the relatively sparsely populated Santa Cruz Mountains, well to the south of the densely populated greater San Francisco Bay Area. A number of structures and sites in the immediate vicinity of the main fault rupture were simply overwhelmed by strong inertial forces, but the levels of shaking which propagated to the more distant regions of concentrated population were considerably less severe and did not represent a true test of the ability of this region to survive a major earthquake. Larger and considerably more damaging earthquakes continue to remain likely to occur in the future, in closer proximity to the populous Bay Area, along both the San Andreas and Hayward Fault systems. Continued efforts to develop and implement engineering solutions to mitigate the likely consequences of such future events are thus of utmost importance.

The damage and destruction wrought by the earthquake of October 17, 1989 should serve as a stark warning of the need to pursue, with increased diligence, ongoing efforts to improve the safety of the public, and of their buildings and infrastructure. There is much to be learned from the study of this earthquake, and it must be hoped that these lessons can be implemented in time to save lives and prevent higher levels of damage in future, and ultimately inevitable, larger seismic events.

Raymond B. Seed
March 17, 1989

ACKNOWLEDGEMENTS

This report presents the results of studies performed without conventional funding or support. As such, the information gathered and presented herein represents a tribute to the earthquake engineering professional community at large, both in the San Francisco Bay Area and around the world, whose rapid response to this tragic earthquake made this report possible. It is not possible to individually cite all of the engineers, seismologists, geologists, public officials, etc. who contributed to the information summarized herein, but their rapid professional response and their laudible dedication to public safety have not passed unnoticed or unappreciated.

The authors also wish to give special thanks to a number of people who provided special assistance in gathering data for this report. This includes a number of faculty, researchers, graduate students and visiting scholars from the University of California at Berkeley including R. W. Boulanger, R. P. Shilling, L. S. Meier, P. G. Nicholson, G. R. Schmertmann, T. Masood, S. A. Anderson, Prof. S. A. Mahin, Prof. J. P. Moehle and Prof. B. A. Bolt. Special thanks are also extended to: Dr. Marshall Lew of LeRoy Crandall and Associates; Dr. T. L. Holzer, Bill Brown and Dr. A. G. Brady of the U.S. Geological Survey, Tom Sharp of the City of Santa Cruz Public Works Department; Roger Dehaven and Bob Swanstrom of the City of San Francisco Public Works Department; Bernard A. Vallerga of B. A. Vallerga, Inc.; David T. Schrier of Harlan Tait; Professors G. Wayne Clough and James R. Martin, II of Virginia Polytechnic Institute and State University; J. Richard Farris, John A. deBecker and Eldon M. Jemtrud of the U.S. Navy Western Division Naval Facilities Engineering Command; Dr. J. David Rogers of Rogers/Pacific; Bob Darragh, Dr. Robert Semple, Demetrious Koutsoftas and Cindy Shaw of Dames and Moore; Bill Cotton of William Cotton and Associates; Frank Rollo of Treadwell and Associates, Inc.; Anne Becker of the U.C. Berkeley Dept. of Geology and Geophysics; Nancy Hardaker of the State of California Office of Emergency Services; Jim McFarlane of the Port of San Francisco; Irene Stachura of the J. Porter Shaw Library, S.F. Maritime National Historic Park; John Bowman, State of California Department of Transportation; Richard Pardini of the San Jose Water Company; K. Rodda of W. A. Wahler & Associates, R. L. Volpe of R. L. Volpe & Associates, Robert E. Tepel of the Santa Clara Valley Water District; Anthony F. Shakal of the California Division of Mines and Geology (CDMG/SMIP); Rick Haltenhoff of Associated Terra Consultants (and the County of Santa Clara); Prof. Tatsuo Omachi and Takumi Toshinawa of the Tokyo Institute of Technology; Dr. Yutaka Nakamura of the Japanese National Railway Technical Research Institute; Prof. W. D. Liam Finn of the University of British Columbia; W. Pennington of the U.S. Bureau of Reclamation and B. Heyenbruch of the U.S. Army Corps of Engineers.

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Chapter One: INTRODUCTION

The Loma Prieta Earthquake of October 17, 1989 was the most costly single natural disaster in U.S. history. Damages resulting from this earthquake included widespread landslides in the epicentral region, liquefaction and other soil failures in numerous regions surrounding the San Francisco Bay and elsewhere, structural distress and failures in commercial and residential buildings, damage to nonstructural elements and contents of structures, damage to critical transportation systems, and widespread disruption of utilities and other lifeline facilities.

At least 62 fatalities have resulted from the earthquake, 38 of these from the collapse of the elevated Cypress section of the Interstate 880 freeway in Oakland. More than 2,400 individuals suffered injuries requiring medical treatment. While these numbers are tragic, it must be recognized that the number of deaths and injuries would almost certainly have been much higher if it were not for the fact that many people had gone home from work early on the day of the earthquake to watch the "World Series" of baseball.

Table 1.1 presents a summary of preliminary data currently available regarding the approximate distribution of deaths, injuries and damages in the counties most significantly affected by the earthquake. Economic losses cited in this table do not include costs associated with interruption of services, loss of revenue, business disruption, loss of personal income, etc. All estimates of damages are preliminary, and are likely to change as further data becomes available.

Possibly the most vivid examples of damage associated with the earthquake are the catastrophic collapse of the Cypress section of the Interstate 880 freeway in Oakland, the partial collapse of a section of the San Francisco Bay Bridge, the structural damage and fires which occurred in the Marina District in San Francisco, and the collapse of structures in the Pacific Garden Mall in Santa Cruz. However, the earthquake's effects extend far beyond these few examples. It is estimated by the Federal Emergency Management Agency (FEMA), the California Office of Emergency Services and others that more than 105,000 homes, 500 apartment buildings, and 3,500 businesses have been damaged by the earthquake. It is expected that more than 1,000 structures will have to be condemned and demolished. In total, more than 8,000 people were displaced from their homes by the earthquake. It is expected that damages directly attributable to the earthquake will amount to between 6 and 10 billion dollars. Costs associated with loss of business revenue and personal income will substantially increase this amount.

This report presents a preliminary overview of the principal geotechnical aspects of this damaging earthquake. Figure 1.1 presents a map of the region affected by this event, as well as an overview of the principal regions affected by ground failure (e.g. landslides and soil liquefaction). These two general types of ground failure, landslides and soil liquefaction, occurred at sites distributed over a

Table 1.1: Approximate Distribution of Deaths, Injuries and Damages Associated with the Loma Prieta Earthquake of October 17, 1989

County	Deaths	Injuries	Damage* ($ Billions)	Private Buildings (Damaged/Destroyed)	Businesses (Damaged/Destroyed)
Alameda	42	481	1.476	2,763/17	414/17
Contra Costa	0	22	0.025	485/0	124/0
Marin	0	0	0.002	24/0	20/0
Monterey	1	14	0.118	341/19	48/11
San Benito	0	110	0.102	174/62	35/22
San Francisco	13	700	2.759	382/11	134/NA
San Mateo	0	451	0.294	782/1	793/1
Santa Clara	1	1,305	0.728†	5,124/131†	364/6†
Santa Cruz	5	671	0.433	13,329/774	1,615/310
Solano	0	3	0.004	2/0	0/0

* Approximate values based on public and private sources.

† Santa Clara County figures presented do not include San Jose damages.

[Source: California State Office of Emergency Services]

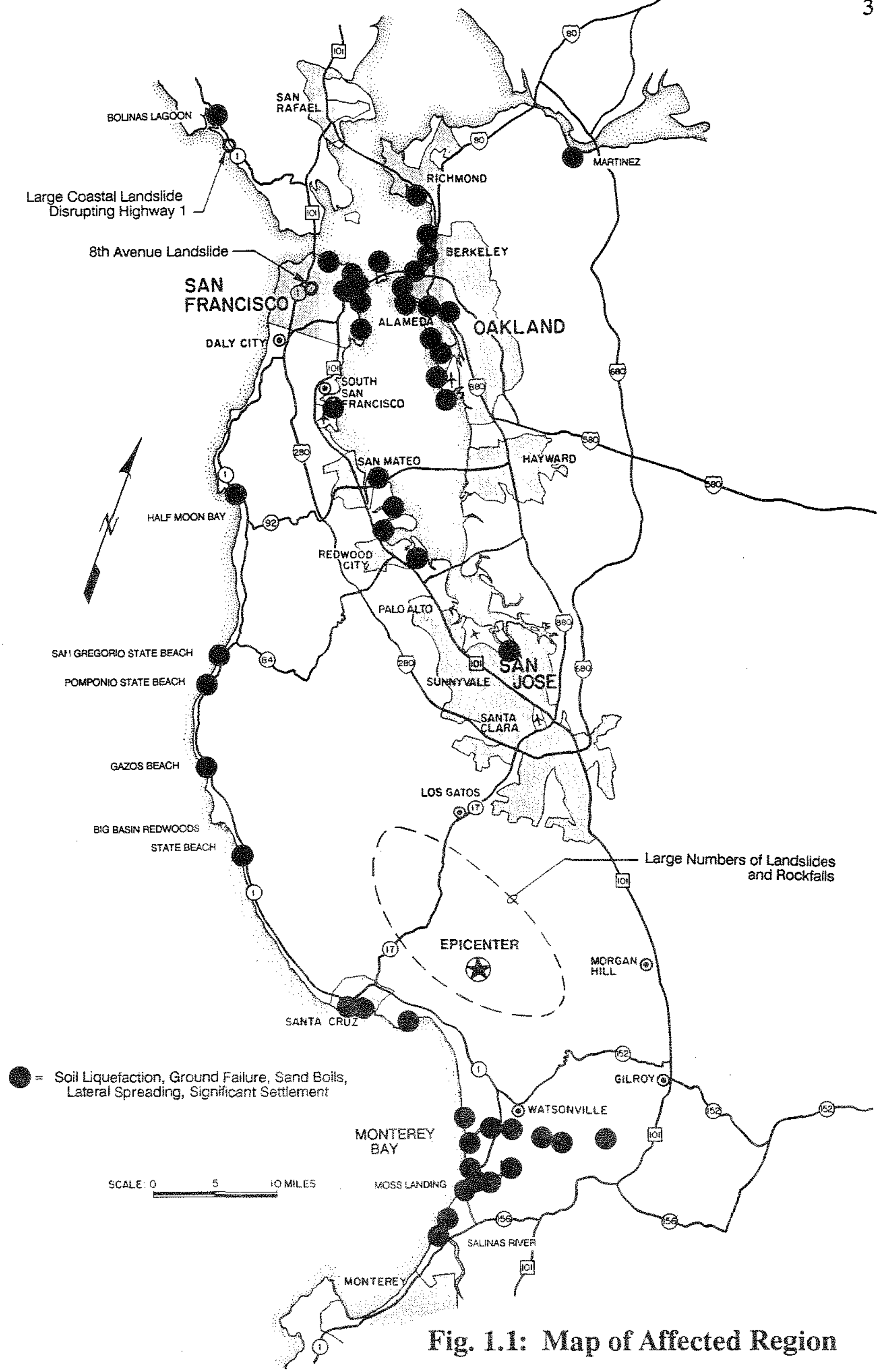


Fig. 1.1: Map of Affected Region

large area surrounding the epicentral region, and resulted in significant damages. It was, however, a third geotechnical feature which represented the unifying "theme" of this damaging earthquake. Local site effects exerted a strong influence on the severity of ground shaking throughout the affected region. The result of this was massive concentration of damages (and loss of life) at sites comprising significantly less than one percent of the affected region.

These sites share a single, vitally important characteristic: all are underlain by "poor" soil conditions (typically deep, clayey alluvium) which amplified the levels of shaking produced at the ground surface. As a result of this site-specific amplification of shaking levels, and especially of preferential amplification of long-period motions at these sites, well over half of the economic damages and more than 80 percent of the loss of life appears to have occurred on considerably less than one percent of the land area within approximately 50 miles of the fault rupture zone. This is despite the fact that most of these sites of heavily-concentrated damage occur near the outer extremes of this area of approximately 50 miles radius. This tremendously pronounced influence of "site effects" on local ground shaking, and on global damage patterns, is the most striking and important feature of the Loma Prieta Earthquake. To understand these effects, however, requires an understanding of the seismology and geotechnics of this event, as well as an understanding of ground failures and structural failures, and of their relative contributions to the overall damage patterns.

Chapter Two of this report provides a brief overview of the seismology of the Loma Prieta Earthquake. Chapter 3 describes sites damaged by soil liquefaction, and provides some historical context for this damage. The fourth chapter presents a brief overview of the pronounced influence of local site conditions on both ground shaking and resulting damage. This includes structural damage at many of the most heavily damaged sites, as well as much of the soil liquefaction described in Chapter 3. Chapter 5 presents a brief overview of landsliding triggered by this event: more than 1,000 landslides occurred in the Santa Cruz Mountains in and near the epicentral region, and additional slides occurred at sites as far as 70 miles from the epicenter. Chapter 6 presents a brief overview of the performance of dams during this event. A number of major earth and rockfill dams were strongly shaken by this earthquake. Several of these were damaged, but performance was generally good: no dams suffered sufficient damage as to represent serious risk of release of their reservoirs.

Finally, Chapter 7 presents a summary of the geotechnical features of the Loma Prieta Earthquake, and some of the principal lessons to be learned from them. These are locally significant, inasmuch as this event represents an opportunity to develop a better understanding of local geology and site conditions, and of their tremendous impact on seismic risk exposure. This earthquake is also expected to be globally significant, as these effects have already been unusually well documented by the local and international research and professional communities, and studies currently in progress can be expected to lead to significant improvement of both the state of the art as well as the "state of practice" in dealing with site effects and their important influence on seismic risk and damage.

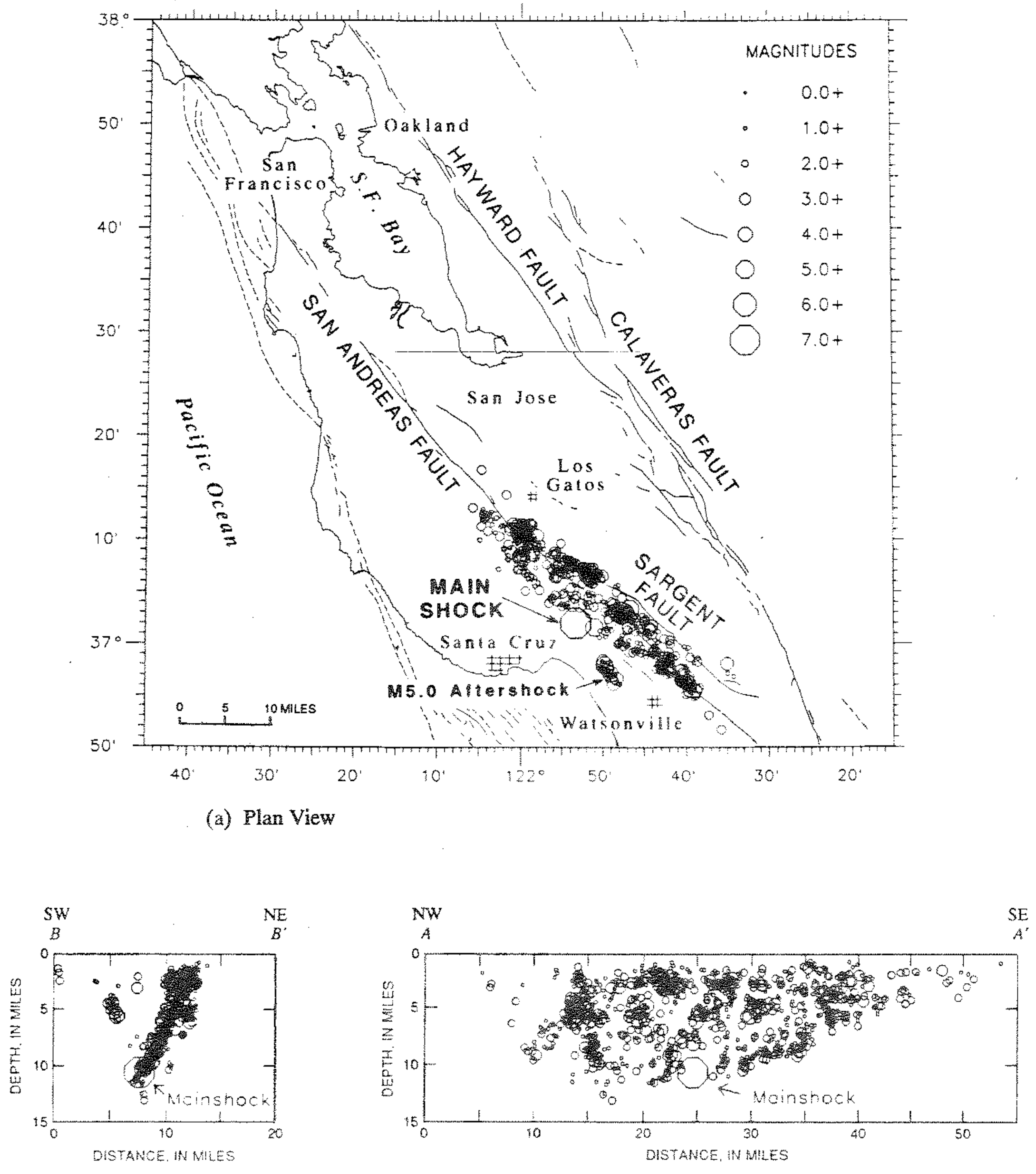
Chapter Two: SEISMOLOGY

The Loma Prieta Earthquake of October 17, 1989 occurred at 5:04 p.m. local time, when a segment of the San Andreas fault northeast of Santa Cruz, California ruptured over a length of approximately 28 miles (45 km). This rupture produced an earthquake with Richter Magnitude $M_L = 7.0$ (as assessed by the Seismographic Stations at the University of California at Berkeley), and average surface wave magnitude $M_S = 7.1$ (as assessed by the U.S. Geological Survey). The epicenter was located approximately 10 miles (16 km) northeast of Santa Cruz and approximately 20 miles (30 km) south of San Jose, at 37.037° N. latitude and 121.883° W. longitude.

Figure 2.1(a) shows the location of the fault rupture, as clearly delineated by the main event and associated aftershock sequence epicenters from October 17-20, 1989. Figure 2.1(b) shows a cross-section through the fault region from southwest to northeast. As indicated in this figure, the fault plane in the vicinity of the rupture segment dips to the southwest at an angle of approximately 70°. Figure 2.1(c) shows a cross-section along the fault axis from northwest to southeast. It should be noted that although the epicenter plots of Figure 2.1 indicate a total fault rupture over approximately 200 square miles of the buried fault surface, and over a segment of the fault almost 50 miles (80 km) in length, the initial rupture associated with the main event of October 17 was somewhat less extensive.

The main rupture began at fifteen seconds after 5:04 p.m. on October 17, at a depth of approximately 11 miles (18 km) below the Earth's surface, and near the center of the eventual rupture plane. Over the course of the next 7 to 10 seconds the rupture spread approximately 12 miles (20 km) to the north and 12 miles (20 km) to the south along the fault. This unusual medial location of the hypocenter within the ruptured area resulted in the unusually short duration of this earthquake of $M_S = 7.1$. The roughly 8 to 10 seconds of strong shaking induced by this rupture is considerably less than the duration normally associated with an event of this magnitude, and this unusually short duration (or limited number of loading cycles) is likely to have had a beneficial effect on the performance of structures, sites and facilities impacted by this earthquake. The rupture also propagated towards the Earth's surface, but appears to have stopped at a depth of approximately 3 to 4 miles (5 to 7 km) during the main event. Neither the main event, nor the numerous aftershocks, appear to have resulted in extension of the main rupture to the Earth's surface.

The segment of the fault ruptured by this event represents the southern-most segment of the much longer segment of the San Andreas Fault which ruptured in 1906 producing the catastrophic 1906 San Francisco Earthquake of magnitude 8+. The Loma Prieta rupture zone occurs at a point where the San Andreas Fault bends, resulting in a localized compression zone. It has been hypothesized that this compressional feature may account for the nearly equal amounts of right-lateral and reverse slip which occurred on this steeply dipping fault plane. As shown schematically in Figure 2.2, the right lateral offset was approximately 6.2 feet and the



(b) Vertical Cross-Section Across the Fault Plane

(c) Vertical Cross-Section Parallel to the Fault Trace

Fig. 2.1: Locations of Main Event and Aftershock Epicenters for the Loma Prieta Earthquake Sequence; October 17-20, 1989

[Source: U.S. Geological Survey]

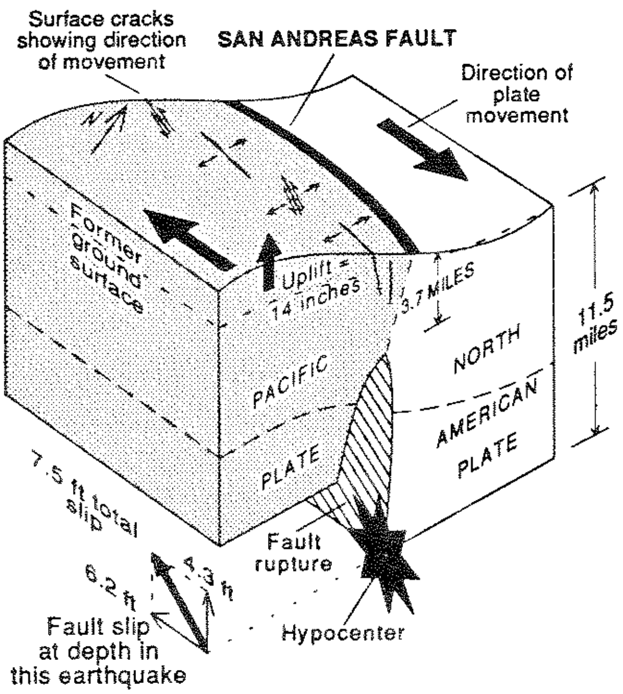


Fig. 2.2: Schematic Illustration of Estimated Fault Movements; Loma Prieta Earthquake of October 17, 1989
[Source: U.S. Geological Survey]

vertical (reverse) offset approximately 4.3 feet. This was locally significant, as it produced unusually high peak vertical ground surface accelerations, which were approximately as high as (and at one recording station significantly higher than) the peak horizontal ground surface accelerations on the upthrown block in close proximity to the rupture region (immediately to the southwest of the rupture plane). These unusually high local vertical accelerations do not appear to have propagated to great distances from the epicentral region.

The aftershock sequence which followed the main event appears to have been fairly typical of the aftershock activity normally associated with major California earthquakes, as aftershock frequency declined rapidly over the three weeks following the main event. A total of 51 aftershocks of magnitude 3.0 or larger occurred within the first 24 hours following the earthquake, and 16 more occurred the following day. Only 20 more aftershocks of magnitude 3.0 or larger occurred during the next 3 weeks. The two largest aftershocks occurred within the first 33 hours after the main shock, and both had magnitudes of approximately 5.0 to 5.2. The first of these occurred shortly after the main event, and was centered near the epicenter of the main event on the main rupture plane. The second major aftershock occurred approximately 33 hours after the main event on a parallel fault approximately 10 km to the southwest of the main rupture zone.

Finally, it should be noted that the Loma Prieta Earthquake of October 17, 1989 in several ways represents a series of circumstances which, together, combined to limit both injuries and damages to structures and facilities. The first of these was the location of the ruptured fault segment. South of San Jose and north of Santa Cruz, this rupture occurred in a sparsely populated region in the Santa Cruz mountains, not in close proximity to a major metropolitan area as would have been the case with an earthquake of similar magnitude occurring along a more northern segment of either the San Andreas or Hayward Faults. This, along with the unusually short duration which resulted from the centrally located hypocenter and symmetrically spreading fault rupture propagation mechanism, combined to limit both damages as well as deaths and injuries to levels considerably lower than those likely to result in the event of similar or larger slippages on these more northern major fault segments.

Chapter 3: SOIL LIQUEFACTION

3.1 Introduction:

Soil liquefaction affected sites over a widespread area, as shown previously in Figure 1.1, including sites as far as 70 miles from the epicenter of the main event. Liquefaction caused considerable damage at a number of sites throughout the central San Francisco Bay area, as well as farther south in the Santa Cruz and east Monterey Bay regions. Soil liquefaction, and considerable associated damage to structures and facilities, occurred in areas of northern and eastern San Francisco, at Treasure Island in the center of San Francisco Bay, along the east San Francisco bayshore from Richmond south to Alameda, at Santa Cruz, and in the east Monterey Bay region. This event does not appear to have induced any significant liquefaction of soils south of the east Monterey Bay Region. As indicated in Figure 1.1, the two most northern sites found to show evidence of liquefaction (to date) are: (a) a sand boil adjacent to a pile supporting a pier on the south shore of Suisun Bay at Martinez, and (b) a series of sand boils observed and photographed in a lagoon at Bolinas, on the western edge of the Marin Peninsula, with associated lateral spreading of the adjacent beach. There was no resulting damage to structures and facilities at either of these locations, and they are of interest only as the sites farthest from the fault rupture zone known to have suffered soil liquefaction. This chapter will provide a brief overview of liquefaction-related phenomena associated with this earthquake, and will also provide some historical context for these observations, as well as a brief discussion of their importance.

3.2 The San Francisco Peninsula:

3.2.1 The Marina District, San Francisco

Widespread liquefaction caused extensive damage to the Marina District, centrally located on the Northern coast of the City of San Francisco. Loose, fine sandy fill liquefied and resulted in sand boils, lateral spreading, settlement, partial bearing failures, structural distress, pavement damage, and damage to pipes and other buried utilities. This region also suffered considerable damage to structures as a result of strong ground shaking. A number of buildings were destroyed or badly damaged; much of the area was evacuated and public access was restricted immediately following the earthquake.

Much of the liquefaction-related damage in the Marina District is an indirect legacy of the 1906 San Francisco earthquake, as much of the liquefaction occurred in hydraulic fill which was placed to create new land in order to provide a site upon which to host a World Fair: the 1915 Panama Pacific Exposition. Major factors in San Francisco's decision to host this World Fair were a desire to celebrate the successful rebuilding of the city in the wake of the catastrophic 1906 earthquake and fire, which had destroyed major portions of the city, and a desire to demonstrate to the world that the city had been successfully resurrected.

Figure 3.1 is a map of the Marina District as it existed at the time of the Loma Prieta Earthquake of October 17, 1989. Super-imposed on this is the old 1869 shoreline and the associated marshy deposits occurring at the south-west limit of the small embayment which existed at that time. Much of the existing Marina District consists of landfill placed since 1869, both to reclaim the marshes and to infill the small baylet. This fill, which was placed in two general stages or periods, is composed primarily of uncompacted fine sands and silty sands. It was primarily within these loose, saturated cohesionless soils that widespread liquefaction occurred.

The first stage of fill placement occurred between about 1870 and the end of the 19th century, and consisted primarily of placement of loosely dumped fill around the perimeter of the small Marina bay and in the perimeter marshes. Most of this fill was dune sand taken from onshore dune deposits occurring adjacent to the southeast edge of the Marina District. A seawall was also constructed to provide a protected harbor. The heavy dashed line in Figure 3.1 shows the resulting coastline and seawall as they existed at the end of the 19th century.

After the 1906 San Francisco Earthquake and fire, the Marina District was selected as the site for the 1915 Panama Pacific Exposition (and World's Fair). To create sufficient land, the harbor area enclosed by the 1899 seawall was infilled with hydraulic fill. Dredged material, consisting primarily of fine, silty sand, was pumped in hydraulic suspension into the enclosed harbor and allowed to settle. This hydraulic fill process typically results in a loose, saturated fill which is vulnerable to potential soil liquefaction during earthquake shaking, and the Marina fill was no exception to this. Figure 3.2 is a photograph of the Marina District viewed from Fort Mason, immediately to the east. Taken in January of 1910, this photograph shows conditions as they existed at an early stage of the hydraulic fill placement. Note the seawall at the right of the photograph, and the small barge and floating pipeline in the bay depositing hydraulic fill. Figure 3.3, taken looking north across Fort Mason and the Marina District in October of 1910, shows a more advanced stage of fill placement as the hydraulic fill has just risen above the surface of the bay. It is interesting to note the glistening sheen of the fill surface at this stage.

Soil liquefaction during the Loma Prieta Earthquake of October 17, 1989 occurred in both the hydraulically placed fill and the earlier, uncompacted fills around the perimeter of the District. Figure 3.4 shows a typical sand boil on the Marina Green at the northern edge of the Marina District. Numerous sand boils occurred throughout the District, both on open ground and along cracks and joints in pavements, gutters, and around the edges of structures. Sand intrusions also occurred in basements and ground floors of buildings. Changes in both color and gradation of the extruded boil materials were readily apparent at various locations across the District, and these could be readily correlated with the origins of the fill materials and their placement history.

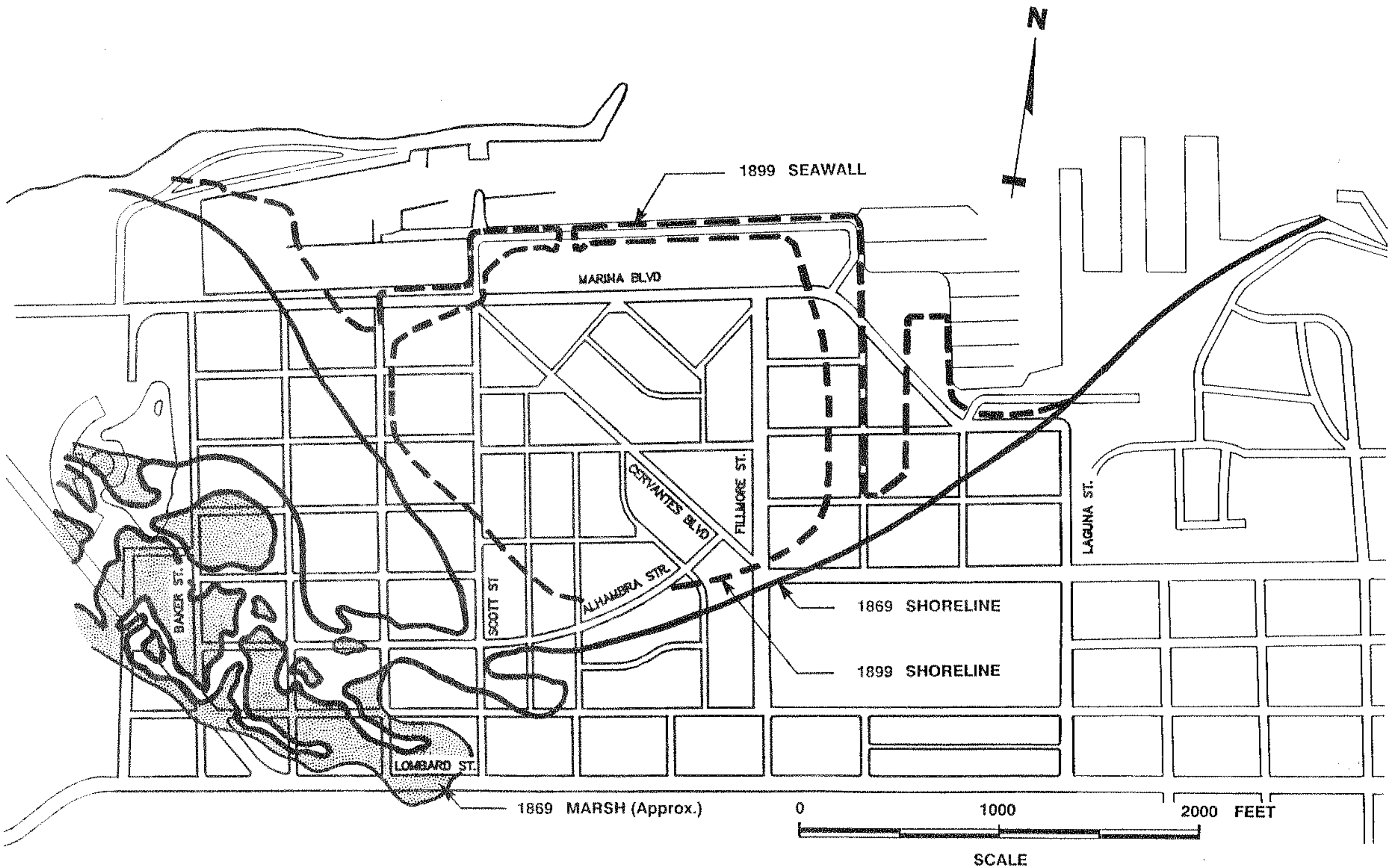


Fig. 3.1: Map of the Marina District in San Francisco Showing the Approximate Locations of the Earlier Coastlines and Marshes of 1869 and 1899

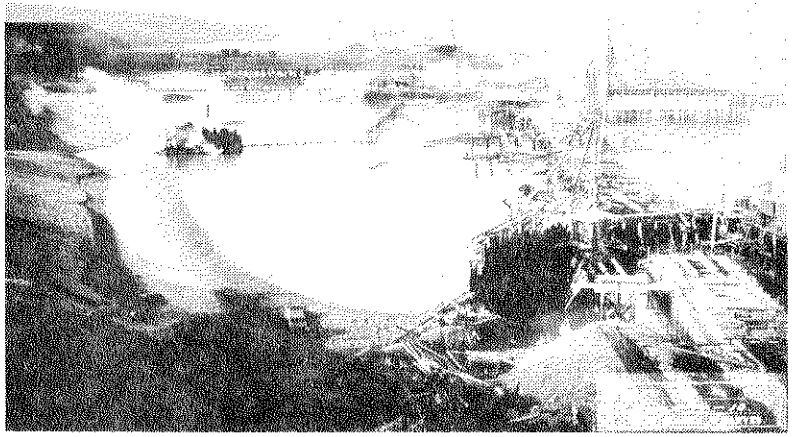


Fig. 3.2: View of the Marina District Looking West from Fort Mason in January of 1910. [Photo courtesy of the San Francisco Maritime Nat'l. Historic Park]

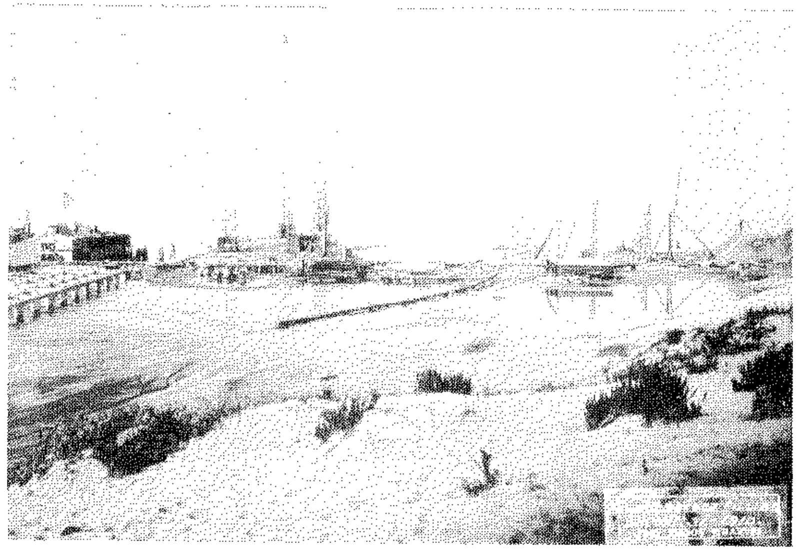


Fig. 3.3: View of the Marina District Looking North in October of 1910. [Photo courtesy of the San Francisco Maritime Nat'l. Historic Park]

Fig. 3.4: Sand Boil on Marina Green, San Francisco

Fig. 3.5: Lateral Spreading, Settlement and Damage to the Seawall at the Marina Yacht Harbor, San Francisco

Figure 3.5 shows damage to the sea wall at the edge of the small yacht harbor at the north end of the District. This damage was the result of lateral spreading associated with liquefaction of the fill materials. Figure 3.6 illustrates conditions near the center of the Marina District (farther inland) immediately after the earthquake. In addition to the collapsed structure, this figure clearly shows buckling of the sidewalk at two locations. This buckling is not the result of settlement, but rather of lateral compression of the pavement due to lateral spreading associated with liquefaction of the underlying fill in this area. Similar evidence of lateral spreading, including extension and/or compression of pavements, occurred throughout much of the District.

Figure 3.7 shows an example of a building which suffered a partial bearing failure as a result of liquefaction-induced loss of strength of its foundation soils. The right-hand side of this building settled several inches into the fill. Figure 3.8 shows sand extruded under the garage door of a building in the Marina District, and Figure 3.9 presents an example of differential settlement of interior footings in another structure near the center of the Marina District. This settlement, resulting from softening of the underlying sandy fill, resulted in wracking and damage to the structure. These and other examples of liquefaction-related damages occurred throughout much of the Marina District.

Figure 3.10 shows the approximate zone of apparent soil liquefaction during the Loma Prieta Earthquake of October 17, 1989, as evidenced by sand boils, settlement, lateral spreading or compression, pavement damage, and/or foundation displacements. A comparison with Figure 3.1 shows that the zone of apparent liquefaction encompasses essentially the entire hydraulically filled central region, as well as portions of the earlier fill around the perimeter of the District and overlying the coastal marshes at the western end of the District.

Figure 3.11 shows the locations of breaks in water pipes greater than 4-inches in diameter, as well as breaks in main sewer lines. Considerable damage to buried utilities occurred as a result of soil liquefaction in the Marina District during the earthquake. In addition to water and sewer breaks, numerous breaks in natural gas pipelines also occurred. It was, however, the water main outages which were most nearly catastrophic.

Several fires occurred in the Marina District immediately after the earthquake, and water outages prevented rapid extinguishing of these. In fact, the largest fire, which occurred at the corner of Beach Street and Divisadero Street was only contained when the city's fireboat was brought to the edge of the Marina Harbor and hoses were run from the fireboat to the fire. The massive pumping capacity of the fireboat was then used to pump water from the Bay, supplementing the capacity of other portable pumps already on the scene, to contain the fire. It is also interesting to note that shortages of equipment forced the Fire Department to remove two fire trucks from San Francisco's Fire Museum to assist in fighting fires during this earthquake. Apparent shortages of equipment, and widespread loss of water pressure in Districts throughout much of the City following this relatively moderate

Fig. 3.6: Pavement Buckling Indicative of Lateral Compression; Marina District, San Francisco

Fig. 3.7: Settlement and Partial Bearing Failure; Marina District, San Francisco

Fig. 3.8: Sand Extruded Beneath A Garage Door; Marina District, San Francisco

Fig. 3.9: Differential Settlement of Interior Footings; Marina District, San Francisco

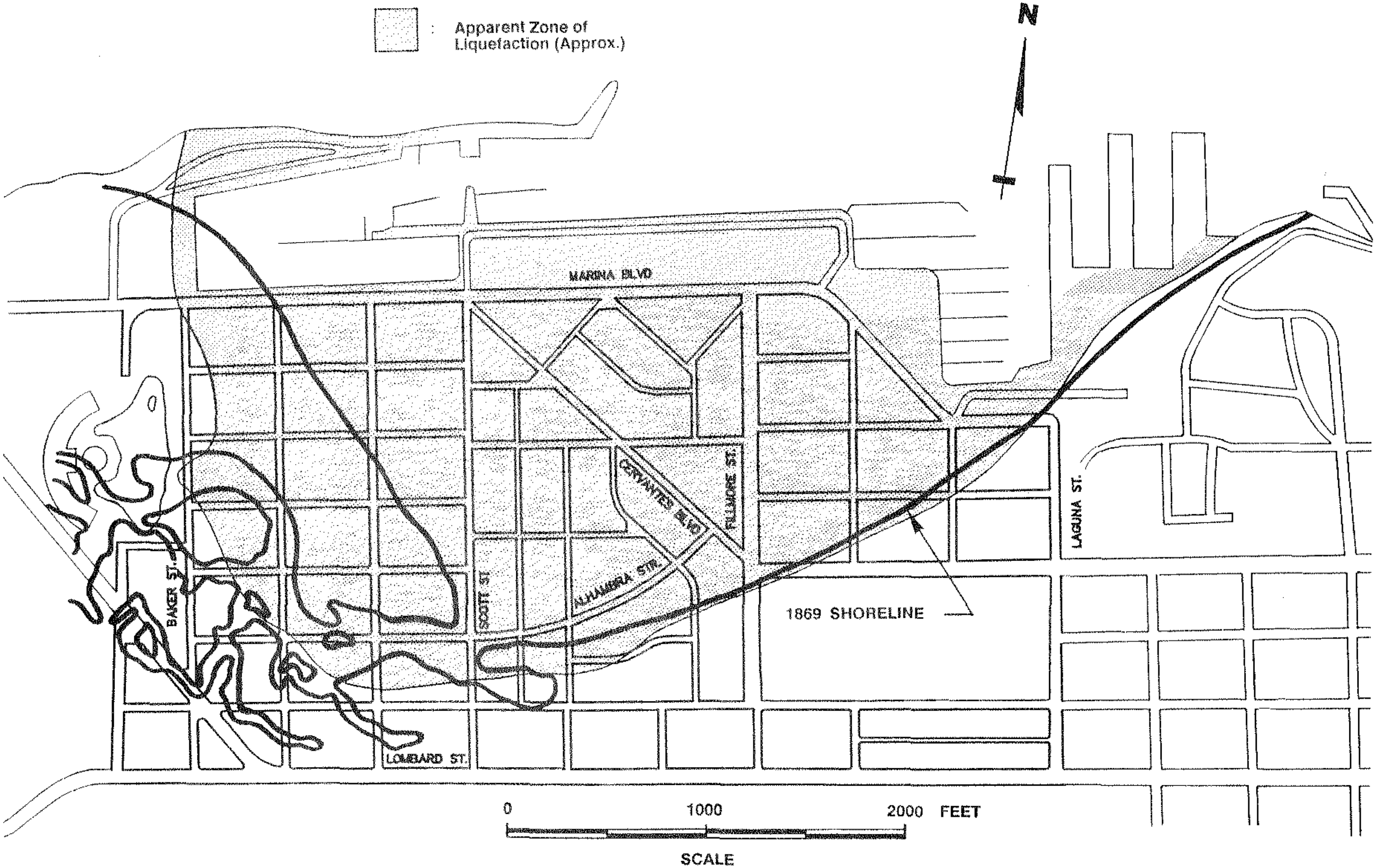


Fig. 3.10: Apparent Zone of Soil Liquefaction in the San Francisco Marina District on October 17, 1989

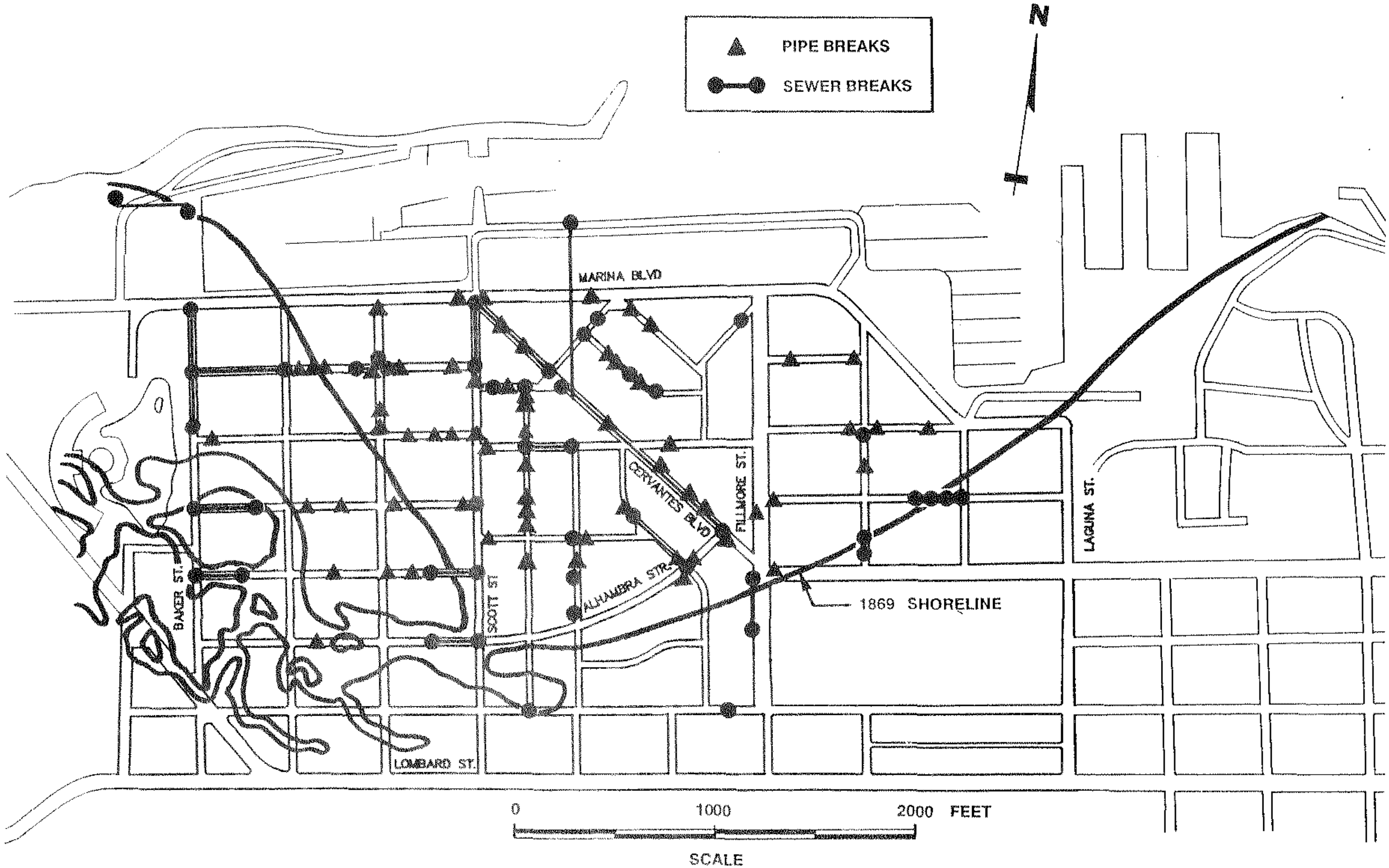


Fig. 3.11 Locations of Major Water Pipe and Sewer Breaks; Marina District, San Francisco

earthquake centered more than 40 miles to the south, appear to raise some question as to San Francisco's ability to deal adequately with a larger number of fires and similar losses of emergency water supplies and pressure in hydrants in the wake of a larger or more near-field seismic event which might occur on either the San Andreas or Hayward Faults.

It should also be noted that the City of San Francisco is taking steps to reduce this apparent exposure. The City has already purchased a second fireboat since the earthquake, providing increased pumping capacity and thus improved fire protection for bayshore regions. In addition, the City is continuing to improve the "survivability" of a citywide auxiliary high-pressure water system which was created specifically to provide fire protection after earthquakes, and is purchasing additional portable fire hydrants and special fire trucks capable of utilizing the pumping capacity of the City's two fireboats at sites some distance inland from the bayshore. The City also maintains a system of approximately 150 underground cisterns throughout the downtown financial district to provide water for fighting post-earthquake fires in this region.

Figure 3.12 shows the locations of heavily damaged structures, as indicated by post-earthquake inspection "tags", roughly one month after the earthquake. City inspectors placed tags of different colors on buildings; these tags indicated their perceived level of safety and controlled access to these buildings in the wake of the earthquake. Structures with red tags were considered unsafe for occupancy, while yellow tags indicated structures to which limited access might be allowed. Damaged structures were periodically re-inspected, and tags were changed or removed as these inspections progressed. As illustrated in Figure 3.12, a majority of the structural damage occurred near the heart of the Marina District. Much of this ground is underlain by the loose hydraulic fill placed in 1910-1912, and much of the rest is underlain either by fill placed to reclaim the perimeter marshes or by naturally deposited loose to medium dense beach and dune sands which occur at the edges of the region.

This does not mean, however, that this concentration of structural damages is due primarily to soil liquefaction. Instead, a majority of the damage to structures in the Marina District on October 17, 1989 was caused by strong shaking. The fill in much of the region of heaviest structural damage is underlain by relatively soft and compressible recent clayey estuarine deposits. These are underlain, in turn, by deeper, and much stiffer pleistocene units, also consisting primarily of clayey soils. These cohesive soils strongly amplified the relatively modest levels of shaking produced in the bedrock underlying the Marina District by the fault rupture near Santa Cruz (more than 40 miles to the south), and resulted in considerably stronger (amplified) levels of shaking at the ground surface. These local soil conditions also altered the frequency characteristics of the accelerations propagating from the rock to the ground surface. This amplification of accelerations, and the especially pronounced amplification of long period motions, appears to have been the primary cause of structural damage in this region. It is also interesting to note that much of the structural damage was associated with the collapse of "weak" ground floors consisting primarily of garages with few walls and thus little structural capacity for carrying lateral shear forces at the ground-floor levels of two to four-story apartment structures.

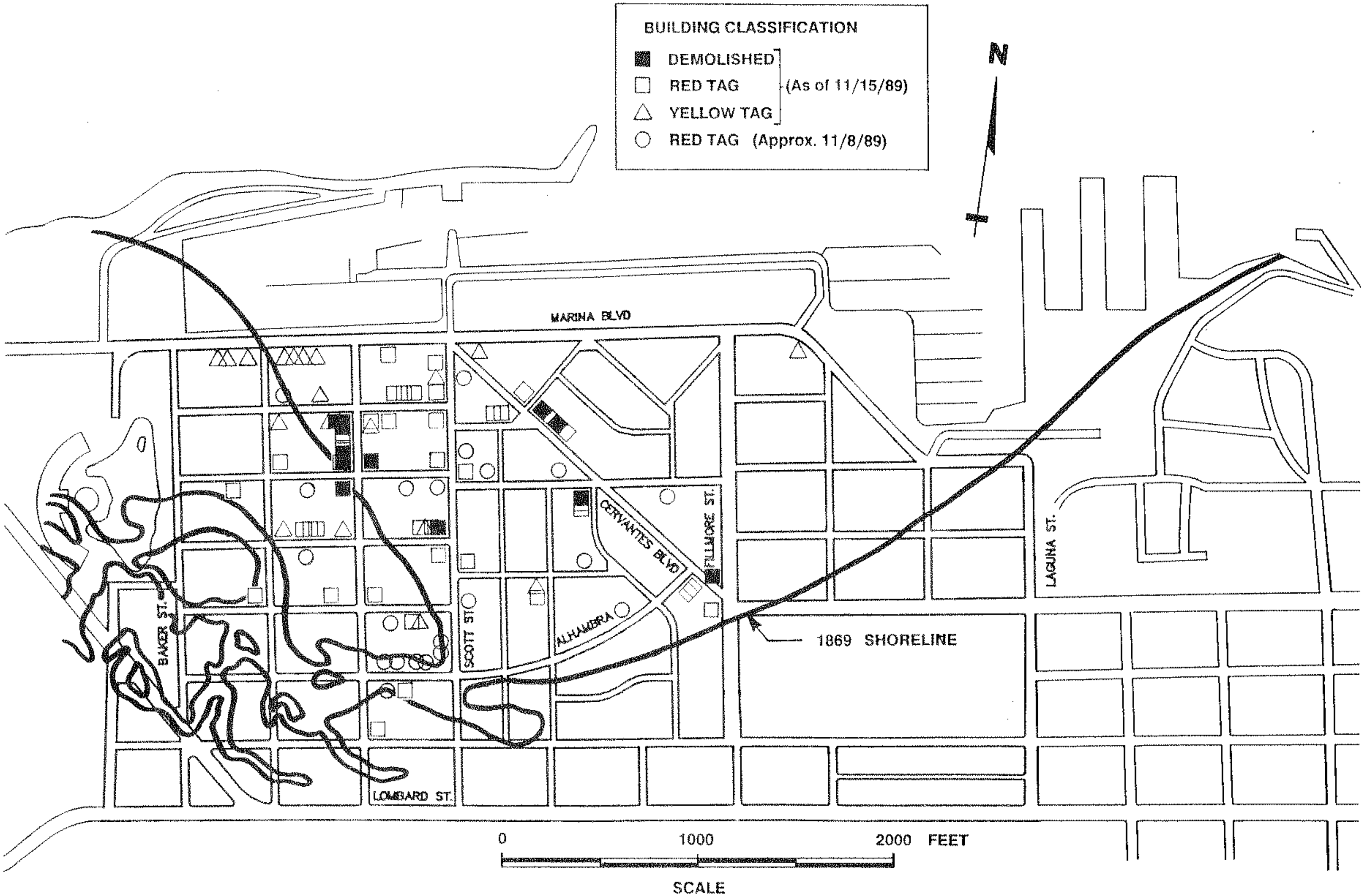


Fig. 3.12: Locations of Demolished Structures, and Structures Marked with Red and Yellow Post-Earthquake Inspection Tags as of Mid-November, 1989: Marina District, San Franciso

It has been noted by other researchers that much of the worst structural damage, and most collapses of buildings with "soft" ground floors, occurred at corners of blocks. This does not appear to have been due to shaking being passed along a line of adjacent structures in such a manner that the end structure "fell over". Rather, it appears that corner lots were typically larger than lots within the interiors of the blocks, and so more typically had three or four-story apartment structures with garages dominating their ground floors built upon them. Interior lots were more commonly smaller and had smaller two to three-story structures, typically with their ground floors less completely dominated by garages. The larger, taller structures typically had more upper floor mass with which to load weak ground stories with shear forces, and typically had "weak" ground stories. They may also have had longer natural periods which were more nearly resonant with the long period ground motions produced by the underlying soil conditions.

It should also be noted that structures similar to those which collapsed or suffered major damage in the Marina District, including three and four-story apartment buildings with "soft" ground floors (garages) also occur in the neighborhoods to the south and east of the Marina District. These essentially identical buildings were typically undamaged in these neighborhoods which are founded on considerably stiffer, shallower soils and/or rock.

Finally, it should be noted that although a majority of the damage to structures in the Marina District was the result of strong ground shaking, amplified by the clayey soils underlying the cohesionless surface fills, this does not mean that potential liquefaction of the surface fills in this region does not represent a significant potential hazard in future earthquakes. Although liquefaction occurred over a widespread area of the Marina District on October 17, 1989, all of the evidence available suggests that the "degree" of liquefaction was relatively moderate over much of this region. This was due to the relatively moderate level and duration of strong shaking produced by the fault rupture which occurred more than 40 miles to the south. Larger or more near-field future earthquakes, producing stronger levels of shaking and/or a longer duration of shaking, can be expected to induce significantly more serious and widespread liquefaction of the Marina fills, and the resulting loss of strength, settlement and lateral spreading of the foundation soils can be expected to represent a more serious threat to structures in this area.

Soil conditions in the Marina District, and the performance of these soils and the structures and facilities built upon them, represent a good model of both the site conditions and performance of numerous other sites distributed throughout the central Bay Area. These additional sites, which also suffered as a result of soil liquefaction, collectively encompass many times the total land area of the Marina District. A consistently repeated pattern throughout bayshore regions of San Francisco as well as Oakland, Alameda and Emeryville on the east side of the bay, and Treasure Island at the center of the bay, was the liquefaction of loose, sandy fills underlain by cohesive soils. Much of the liquefaction in all of these regions occurred in loosely dumped sandy fills and/or hydraulic fills underlain by relatively soft clay, known locally as San Francisco Bay Mud, underlain in turn by very deep, stiff, overconsolidated older cohesive soil units. The deep clays, and the relatively soft Bay Mud (when present), acted to amplify accelerations at these sites, producing cyclic loading of sufficient severity to cause liquefaction of the loose sandy fills.

3.2.2 The Old Mission Bay and Embarcadero Regions

Although much attention has been focussed on the soil liquefaction which occurred in the Marina District, considerably larger areas of San Francisco also showed evidence of liquefaction during the Loma Prieta earthquake. This includes large portions of the bay shore Embarcadero region along the north-eastern edge of San Francisco, and major portions of the fill deposits south of Market Street encompassing the old Mission Bay region.

(a) The Old Mission Bay Region

In the great San Francisco earthquake of 1906, the heaviest and most intensely concentrated damages (prior to the post-earthquake fire which levelled much of the city) occurred in three well-defined regions: two of these were areas south of Market Street and the third was the Embarcadero bayshore region. Figure 3.13 is a map of the eastern portion of San Francisco as it exists today. The zones in this figure enclosed with heavy dashed lines are the three zones which were judged to have suffered the most severe shaking and damages (commensurate with a Rossi-Forrel Intensity rating of IX to X, the highest intensities on this scale) during the 1906 San Francisco earthquake prior to the great post-earthquake fire (Wood, 1908). Also shown on this figure is the old shoreline as it existed in 1852, along with the extensive marsh deposits which extend well inland from what was the old Mission Bay. As shown in this figure, the three regions which were judged to have suffered most heavily during the 1906 earthquake coincided with regions consisting of fill underlain by the soft clay deposits of these marshy regions and the old edges of San Francisco and Mission Bays.

Fill was placed to reclaim the marshes and extend the coastline out from the old Mission Bay shoreline between approximately 1850 and the end of the 19th century. Most of this fill was loosely dumped, and much of the fill consisted of locally available dune sands and silty sands though considerable quantities of rubble and debris from several large fires and the 1868 earthquake, as well as gravels, hay bales, garbage and other materials were also randomly placed. This resulted in conditions similar to those previously described for the Marina District in which an uncompacted loose, and primarily sandy fill, typically 20 to 30 feet in depth, is underlain by soft compressible clay (Bay Mud). The Bay Mud is underlain, in turn, by older and much stiffer cohesive deposits.

A colorful description of the exceptionally poor soil conditions underlying the loosely dumped fill in the old Mission Bay region is provided by Hittell (1878) who wrote:

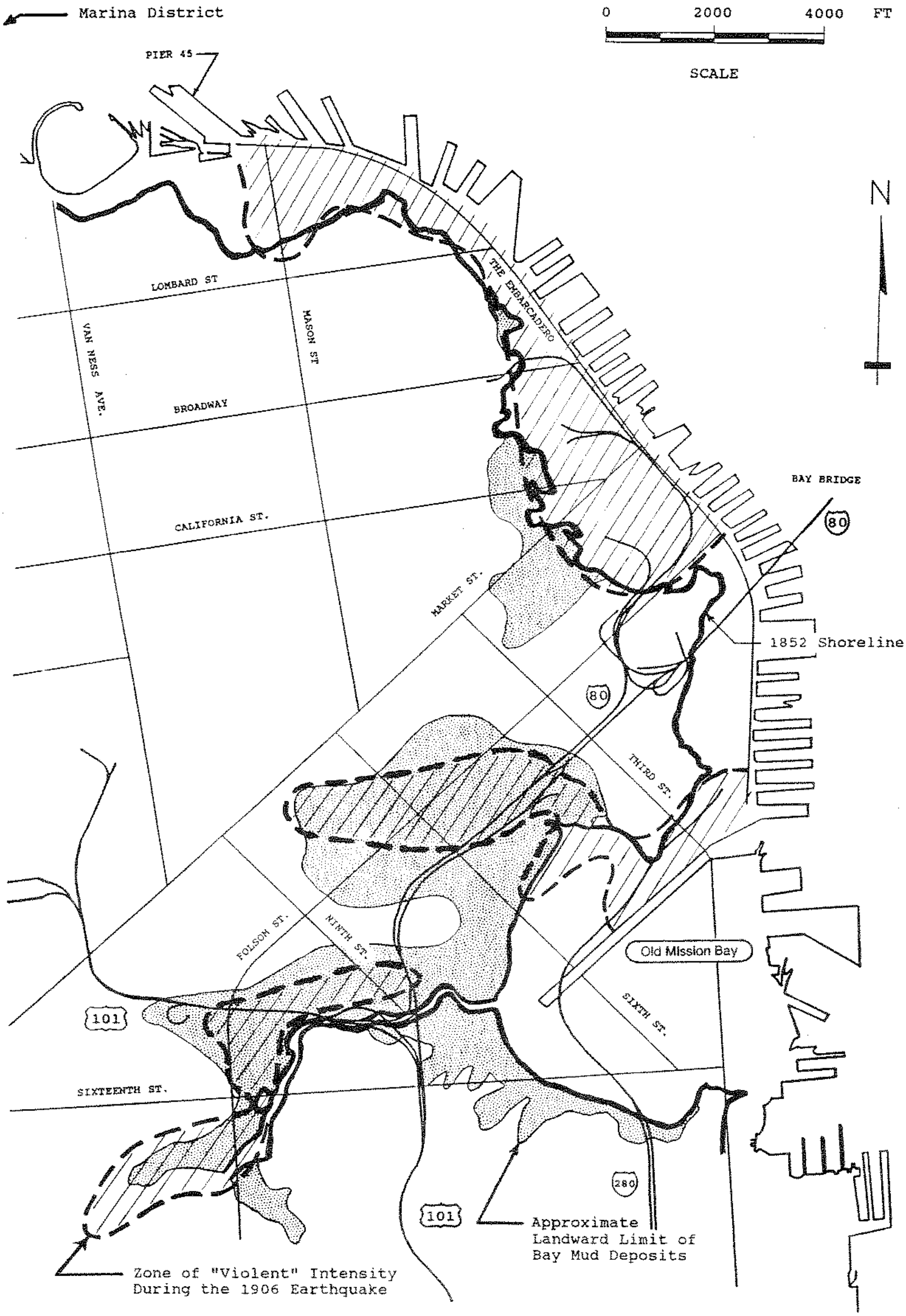


Fig. 3.13: Map of Eastern San Francisco Showing the Regions Most Intensively Damaged During the 1906 Earthquake (Before the Post-Earthquake Fire), and the Historic Coastline and Marshes of 1852

> "The peat in the marshes that had their heads near the site of the new city hall was strong enough to sustain a small house or a loaded wagon, though a man, by swinging himself from side to side, or by jumping upon it, could give it a perceptible shiver. There were weak places in it, however, and a cow which in searching for sweet pasture undertook to jump from one hard spot to what appeared to be another, made a mistake, for it gave way under her and a gentleman hunting nearby was surprised to see her go down and still more to observe that she did not come up again.
> . . . Many ludicrous scenes occurred in filling up the swamps. When streets were first made the weight of the sand pressed the peat down, so that the water stood where the surface was dry before. Sometimes the sand broke through, carrying down the peat under it, leaving nothing but water or thin mud near the surface. More than once a contractor had put on enough sand to raise the street to the official grade, and gave notice to the city engineer to inspect the work, but in the lapse of a day between the notice and the inspection, the sand had sunk down six or eight feet; and when at last a permanent bottom had been reached, the heavy sand had crowded under the light peat at the sides of the street and lifted it up eight or ten feet above its original level. . . so that houses . . . were carried away from their original position and tilted . . ."

Figure 3.14 shows the locations of heavily damaged structures as indicated by red and yellow post-earthquake inspection tags as of mid-November, 1989. It is interesting to note that there is again some evidence of concentration of damage in the regions indicated in the previous figure as having suffered most heavily in 1906.

It should also be noted that much of the extensive damage which occurred in these regions in 1906 appears to have been directly attributable to massive and widespread liquefaction of the sandy fills in these regions. Figure 3.15 shows regions in which evidence of soil liquefaction was observed following the Loma Prieta earthquake of October 17, 1989. Once again, the regions south of Market Street showing evidence of liquefaction in the recent earthquake are largely coincident with the regions which suffered heavy damage in 1906.

This does not mean, however, that massive and widespread damage occurred during the Loma Prieta earthquake as a result of liquefaction of soils in these regions. The significantly smaller magnitude Loma Prieta event, centered more than 45 miles to the south and having an unusually short duration for a magnitude 7.1 event (as described previously in Section 2.0), did not produce massive and catastrophic liquefaction throughout the filled regions. Instead, it appears that the Loma Prieta earthquake shaking produced minor liquefaction and/or pore pressure generation and associated ground softening representing incipient or near-liquefaction over much of this region.

Nonetheless, the fact that widespread evidence of liquefaction and/or incipient liquefaction was produced by the relatively moderate magnitude and duration of shaking generated at these sites by the Loma Prieta earthquake serves as a warning of the potentially catastrophic consequences of soil liquefaction which may be expected to occur throughout this region in the event of occurrence of a larger magnitude or more near-field event on either the San Andreas or Hayward faults.

Figure 3.16 illustrates conditions near the corner of what are now 6th and Howard Streets immediately after the great earthquake of 1906. The structure shown

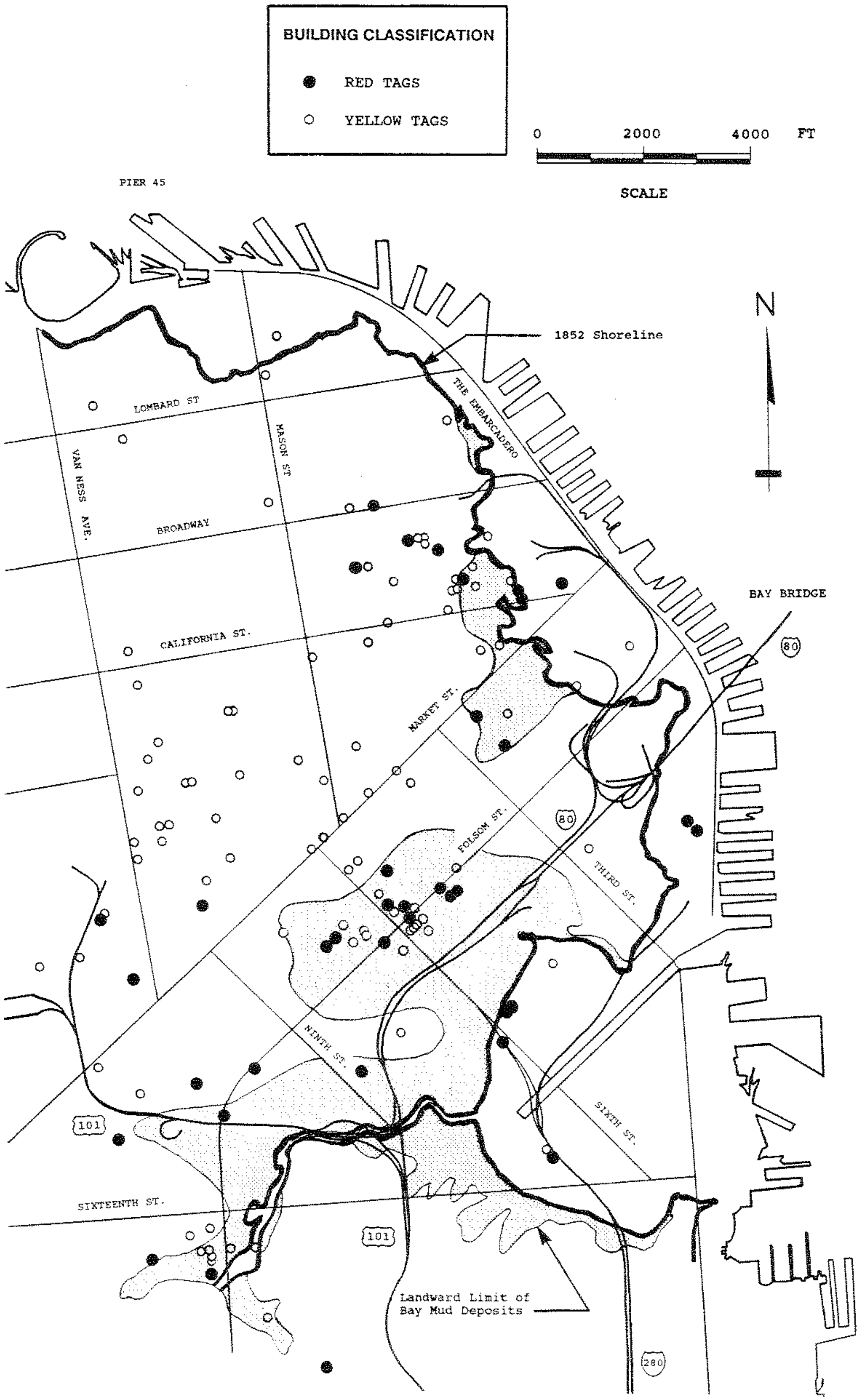


Fig. 3.14: Locations of Demolished and/or Heavily Damaged Structures in East San Francisco Based on Post-Earthquake Inspection Tags as of Mid-November, 1989

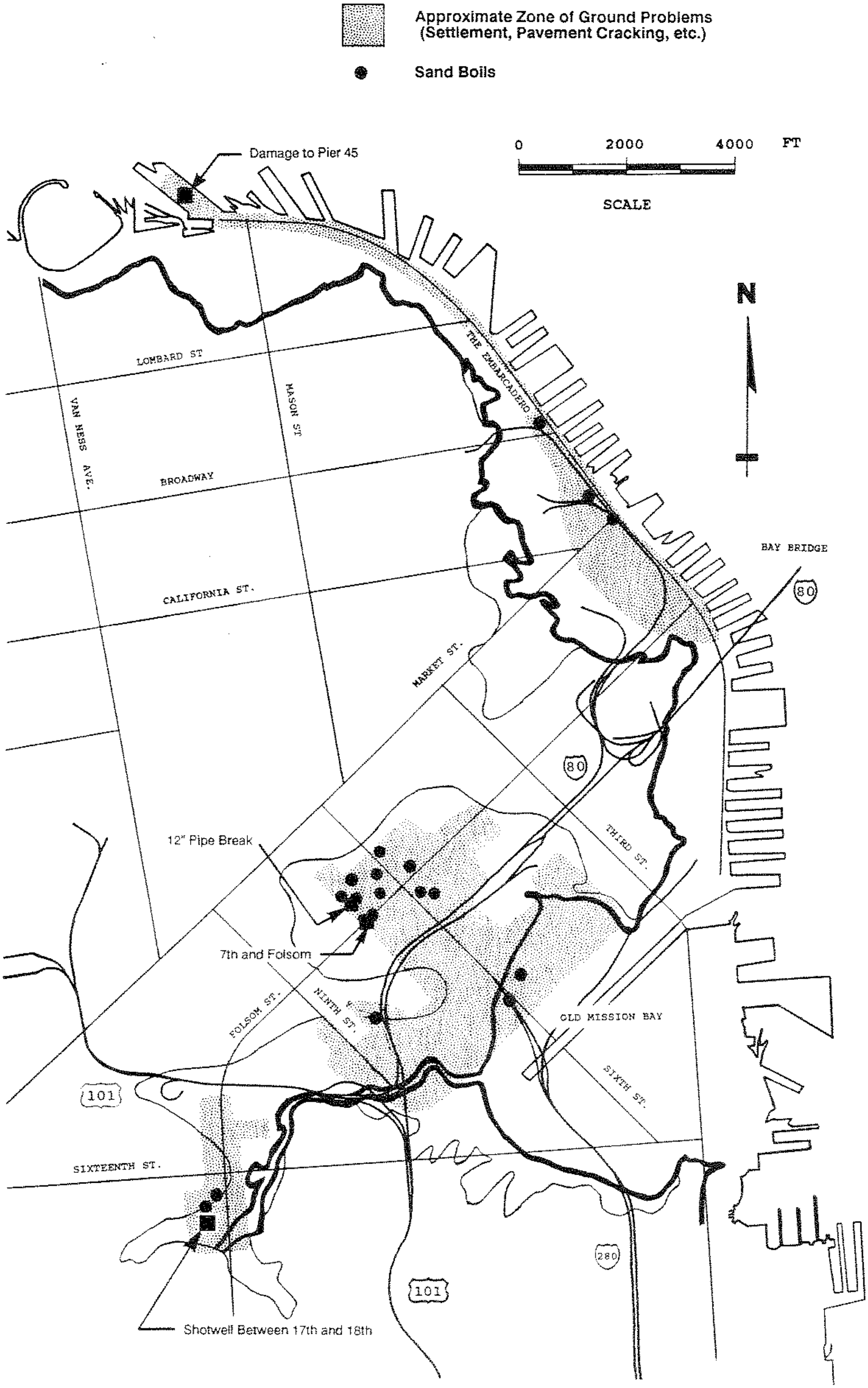


Fig. 3.15: Apparent Extent of Soil Liquefaction in San Francisco's Embarcadero and Old Mission Bay Regions on October 17, 1989

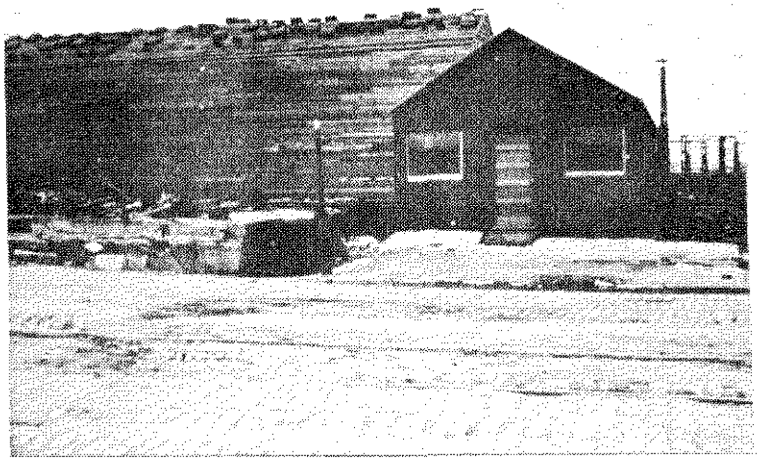


Fig. 3.16: Conditions Near 6th and Howard Streets After the 1906 San Francisco Earthquake [Lawson et al., 1908]

Fig. 3.17: Demolition of a Damaged Structure Near 6th and Howard Streets After the Loma Prieta Earthquake

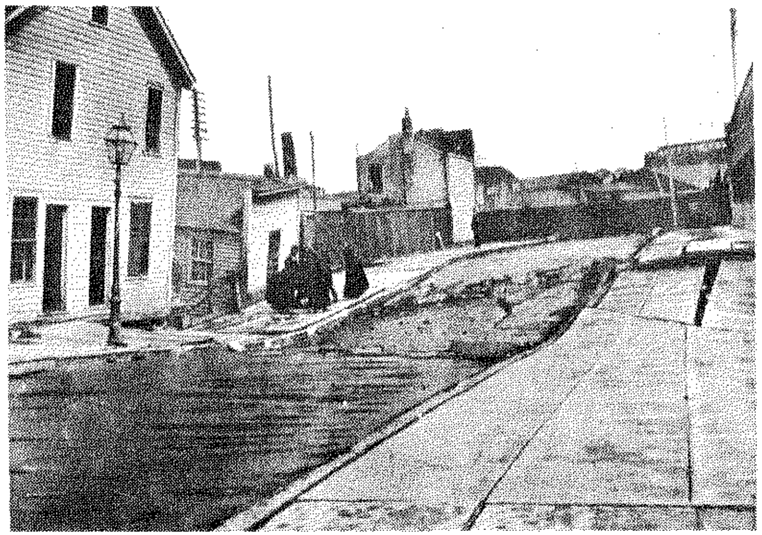


Fig. 3.18: Conditions on Dore Street Near Brannan After the 1906 San Francisco Earthquake [Lawson et al., 1908]

Fig. 3.19: Conditions on Dore Street Between Brannan and Bryant After the 1906 San Francisco Earthquake [Lawson et al., 1908]

is supported on piles, and the surrounding ground has settled nearly 3 feet as a result of liquefaction. Figure 3.17 shows demolition of a 5-story apartment building at virtually this same location in November of 1989. This building sustained extensive structural damage due to liquefaction and differential settlement of the foundation soils during the Loma Prieta earthquake of October 17, 1989.

Figures 3.18 and 3.19 illustrate the considerable ground deformations which resulted from liquefaction in 1906 in the vicinity of what are now Dore and Brannan Streets in the heart of the old Mission Bay region. The ground shown in Figure 3.18 had been level prior to the earthquake; liquefaction and resulting ground deformations produced permanent undulations or "waves" with amplitudes of up to 5 feet in this area. Figure 3.19 illustrates the effects of lateral spreading and bearing failures in this region during the 1906 earthquake. The building at the center of the photograph displaced more than 8 feet to the left as a result of lateral spreading associated with loss of strength of the underlying foundation soils. Figure 3.20 shows sand boils, differential settlement and resulting damages to a structure on 6th Street south of Howard Street (in the vicinity of Figs. 3.18 and 3.19) after the 1989 Loma Prieta earthquake. The basement of the building was filled with intruded sand, and sufficient structural damage occurred as a result of differential settlements that the building was condemned. Figure 3.21 shows a building one block farther to the North. The basement of this structure also experienced considerable sand intrusions during the Loma Prieta Earthquake, and foundation settlements resulted in sufficient damage that this structure was condemned as well.

Figure 3.22 shows conditions in the vicinity of 19th Street and Shotwell in the wake of the 1906 San Francisco earthquake. Note the foundation bearing failures and differential settlements, the tilted structures and the uneven ground (which had been essentially level prior to the earthquake). Figures 3.23 and 3.24 show damage to several structures also at Shotwell at 18th Street in the wake of the 1989 Loma Prieta earthquake. Four of these buildings suffered differential settlements and/or partial bearing failures, and significant sand intrusions occurred in the basements of three of the buildings. All four buildings were heavily damaged by differential foundation displacements.

Sand boils were observed at a number of locations in the Old Mission Bay Region in the wake of the 1989 earthquake, including Russ Street, Moss Street, Clementina between 5th and 6th Streets, 6th and 7th Streets near Howard, Natoma near 7th Street, Clara near 6th Street, Bluxome and Townsend Streets (near 6th Street), and Folsom and Shotwell Streets (between 17th and 18th Streets), as shown in Figure 3.15. In addition, sand intrusions into the basements of buildings occurred at a number of sites including three buildings on Shotwell Street (between 17th and 18th Streets), and two buildings on Howard Street near 6th and 7th Street. The building at 7th and Howard (Figure 3.21) was filled to a depth of 2 feet with sand inflow material. There have been other reports of sand boils, but these have not been confirmed by the authors, and boils reported herein do not include vented sands which may have been associated with ruptured buried pipelines.

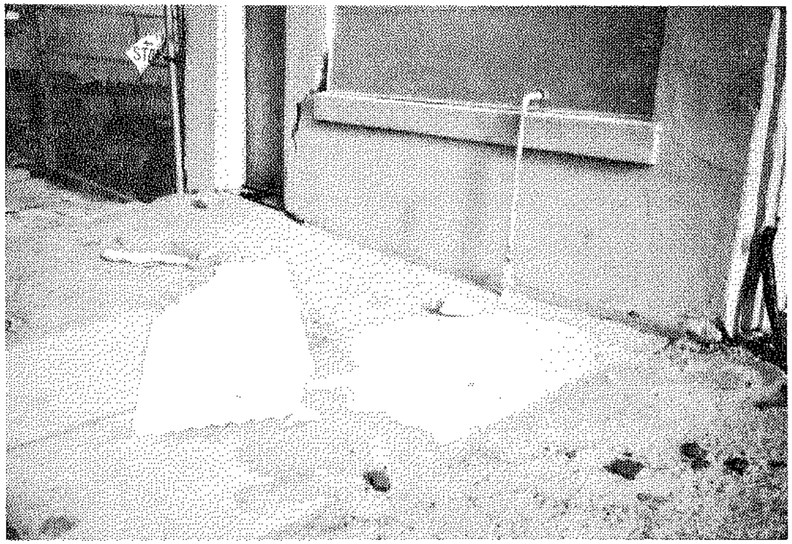


Fig. 3.20: Sand Boils, Foundation Movement and Structural Damage on 6th Street South of Howard After the Loma Prieta Earthquake [Courtesy of Dr. Marshall Lew]

Fig. 3.21: Structure Heavily Damaged by Liquefaction-Induced Foundation Movements at 7th and Howard Streets in the Loma Prieta Earthquake

Fig. 3.22: Conditions at 18th and Shotwell Streets After the 1906 San Francisco Earthquake [Lawson et al., 1908]

Fig. 3.23: Four Buildings on Shotwell near 18th Street Damaged by Liquefaction-Induced Foundation Displacements in the Loma Prieta Earthquake

Fig. 3.24: Closer View of Structures from Fig. 3.23 Showing Settlements and Structural Cracking

Fig. 3.25: Example of Centerline Cracking of a Roadway Pavement in the Old Mission Bay Region: 6th Street at Tehama [Courtesy of Dr. Marshall Lew]

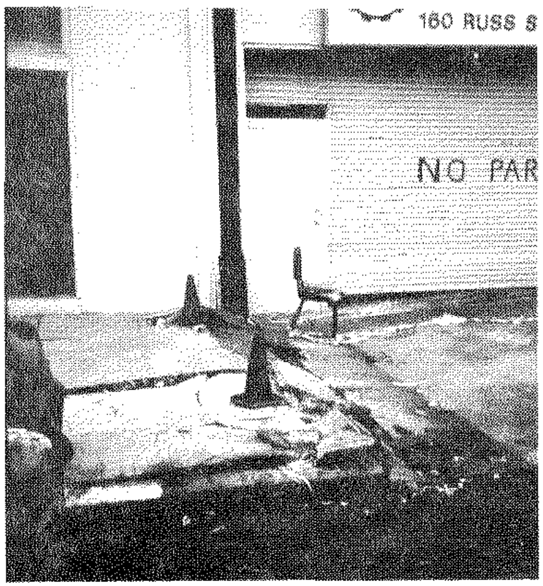


Fig. 3.26: Pavement Damage and Differential Settlement on Russ Street Near Howard Street [Courtesy of Dr. Marshall Lew]

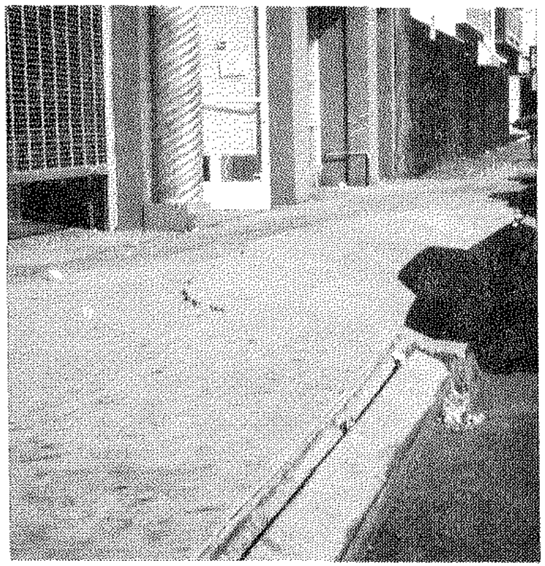


Fig. 3.27: Settlement Adjacent to Storm Sewer Alignment on Bryant Street at Dore Street

A number of pipe breaks occurred in this region, and the most significant of these was a break in a 12-inch diameter high pressure buried pipeline which occurred on 7th Street between Mission and Howard Streets. This was a major pipeline in the Auxiliary Water Supply System, operated by the San Francisco Fire Department specifically for fighting post-earthquake fires, and its rupture (along with other, smaller breaks in the system) reduced water pressures significantly throughout much of the system.

Much of the liquefaction-induced damage in the old Mission Bay region during the 1989 earthquake consisted of minor settlements and pavement damage. Figure 3.25 shows typical pavement damage in this region. Numerous roads cracked along their centerlines, directly above the centrally located main sewer conduits which are common to this region of the city. Figures 3.26 and 3.27 show further examples of pavement damage in this region following the 1989 earthquake.

Although liquefaction occurred over large portions of the old Mission Bay region in the 1989 Loma Prieta earthquake, it was, over most of this region, considerably less severe than that which appears to have occurred in the same area as a result of the significantly more severe shaking produced by the 1906 earthquake. A number of structures in this region were seriously damaged by foundation movements associated with liquefaction in the 1989 event, but these were, fortunately, limited in number.

This does not mean that soil liquefaction in the old Mission Bay region of San Francisco during the Loma Prieta Earthquake was not significant. The widespread evidence of liquefaction and/or incipient liquefaction over large areas of this region as a result of the relatively moderate levels of shaking produced by a magnitude 7.1 event of unusually short duration centered more than 40 miles to the south, together with the catastrophic history of liquefaction-induced ground failures in this region during the 1906 San Francisco Earthquake, serve as a clear warning of the high likelihood of widespread and massively damaging liquefaction occurring in this region as a result of larger or more near-field future seismic events. The overall hazard, in terms of potential loss of life, may in fact be appreciably higher in the old Mission Bay region than in San Francisco's Marina District.

The potentially liquefiable fills in the Mission Bay region cover more than three times the land area of those in the Marina District, and both regions are densely populated. In addition, the relatively light, ductile wood-frame structures of the Marina District (typically with exterior stucco cladding), are better able to withstand moderate foundation movements than are the structures occurring throughout most of the old Mission Bay region. Most of the buildings in the old Mission Bay region are two to four story masonry and/or concrete buildings, and unreinforced masonry buildings are not uncommon. Many of the buildings in this area are more than 60 years old, and many have already been distressed by ongoing foundation settlements as a result of the clayey soils underlying the surface fills. The generally poor structural types and conditions, coupled with the history of poor performance of the liquefiable surface fills, suggest that significant liquefaction-related hazard exists throughout much of this region. It must be hoped that these conditions can be improved prior to the occurrence of larger and/or more near-field future earthquakes.

(b) The Embarcadero Region

As illustrated previously in Figures 3.13 through 3.15, much of the Embarcadero district forming the northeast coast of San Francisco also consists of fill placed to reclaim tideland marshes and to extend the coastline seaward. As with the old Mission Bay region to the south, the fill is typically on the order of 20 to 30 feet thick and consists primarily of loosely dumped or hydraulically placed sands and silty sands, along with gravels, rubble and debris. The fill overlies soft clays which are, in turn, underlain by deeper and much stiffer cohesive deposits.

Most of the fill in this region was placed in the latter half of the 19th century. Figure 3.28 is an old lithograph showing the shoreline at what is now North Beach as it existed in 1846, at which time the small baylet at this location was known as Yerba Buena Cove. Figure 3.29 shows workers and observers pausing during construction of the seawall in this area in 1881. The region behind this seawall was subsequently infilled with non-engineered loose sandy fill.

Liquefaction occurred in the Embarcadero district during the 1906 San Francisco earthquake. Major settlements, significant lateral spreading and loss of foundation bearing capacity caused considerable damage during that earthquake. H. O. Wood, in his contribution to the landmark post-earthquake report edited by Lawson (1908), provides the following description of conditions in this region following the 1906 San Francisco Earthquake:

> "In spots the street sank bodily, certainly as much as 2 feet, probably more . . . The surface of the ground was deformed into waves and small open fissures were formed, especially close to the wharves. Buildings on the water side, along East Street, generally slumped seaward, in some cases as much as 2 feet . . . These phenomena seem to suggest that the materials used in filling were shaken together so as to occupy less space with the accompanying development of waves, fissures and structural damage".

Lawson himself, in the report, provides the following clear description of sand boils in this area (long before engineers understood soil liquefaction):

> "In many places the made land settled. At the junction of Market and Front Streets, the ground sank a foot or two, and there was evidence that the tide had risen in the adjoining lot at the same time, for a pond of water collected and remained until low tide . . . At the corner of First and Market Streets, the ground opened in a fissure several inches wide. At other places, the ground opened and water was forced above the surface. At Fremont and Mission Streets the ground opened in many places".

Derleth (1906) provides further insight into the magnitude of ground deformations in this area, in writing:

Fig. 3.28: Yerba Buena Cove as it Appeared in 1846

Fig. 3.29: Construction of the Embarcadero Seawall in 1881. [Photo courtesy of the San Francisco Maritime Nat'l. Historic Park]

Fig. 3.30: "Centerline" Cracking of Pavement on Mission Street Near Beale Street After the Loma Prieta Earthquake

> "All of the made ground between the Market St. water front and the region east of Montgomery St. has been decidely moved and deformed. Wave-like effects are common along lower Market St. and the water front. Wave-like depressions and crests amounting to four and five feet are found throughout this region".

Soil liquefaction also occurred in the Embarcadero district during the Loma Prieta earthquake of October 17, 1989. This liquefaction was relatively minor in severity, however, and does not appear to have encompassed the entire filled region. This is illustrated in Figure 3.15, in which the shaded region delineates the apparent zones of soil liquefaction during the Loma Prieta earthquake.

Liquefaction caused relatively little damage in this region during the 1989 event. Most evidence of liquefaction consisted of relatively minor settlement and/or cracking of pavements, though sand boils occurred at several locations including a number of boils at three sites on the west side of the Embarcadero (a) across from the Ferry Building (at the foot of Market Street), (b) beneath the elevated highway offramp between Washington and Clay Streets, and (c) between Broadway and Vallejo Streets. Most of the structures in this region are major, multi-story buildings and are supported on deep piles extending well below the potential depth of influence of soil liquefaction. Lateral spreading does not appear to have caused significant ground movements in this area during the Loma Prieta event. In general terms, "liquefaction" in this region appears to have been relatively moderate, and in most areas appears to have consisted mainly of partial pore pressure generation and softening and related settlements and/or lateral displacements on the order of one to several inches, rather than more severe loss of ground strength.

As described previously for the old Mission Bay district, numerous streets exhibited cracked pavements along their center-lines, overlying the centrally located major sewer conduits typical of this coastal area of the city. Figure 3.30 shows a typical example of centerline pavement cracking in this region. Figure 3.31 presents an example of pavement cracking associated with liquefaction-induced settlements near the foot of Market Street, and Figure 3.32 illustrates settlement of the curb and pavement adjacent to a pile-supported pier carrying a portion of the elevated Embarcadero freeway.

Settlements throughout much of the district could be relatively easily observed by comparison between settled pavement surfaces and marks left by the original pavement at the bases of the walls of pile-supported structures in this area. Settlements of the extreme edge of the coastal fill at the ends of the pile-supported piers along the waterfront varied between a minimum of approximately 1 inch at several piers to a maximum of approximately 5 to 6 inches near Piers 15 and 17.

A single notable exception to the general observation that liquefaction caused little serious damage in this region during the Loma Prieta Earthquake was the damage which occurred at Pier 45. Figure 3.33 shows liquefaction-induced settlement and cracking of the pavement at the entrance to this pier, and Figure 3.34 shows a crack running along the pier inside one of the four large warehouses on this pier. Liquefaction-induced damages to Pier 45 and its warehouse structures caused closure and/or partial closure of several of the warehouses and temporary relocation

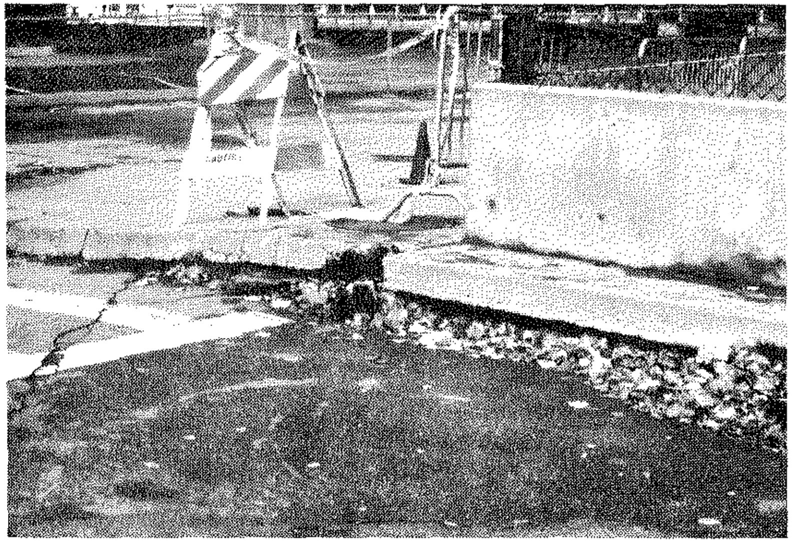


Fig. 3.31: Settlement and Pavement Damage on the West Side of Embarcadero near Market Street [Courtesy of Dr. Marshall Lew]

Fig. 3.32: Settlement Adjacent to Pile-Supported Bent of the Embarcadero Viaduct

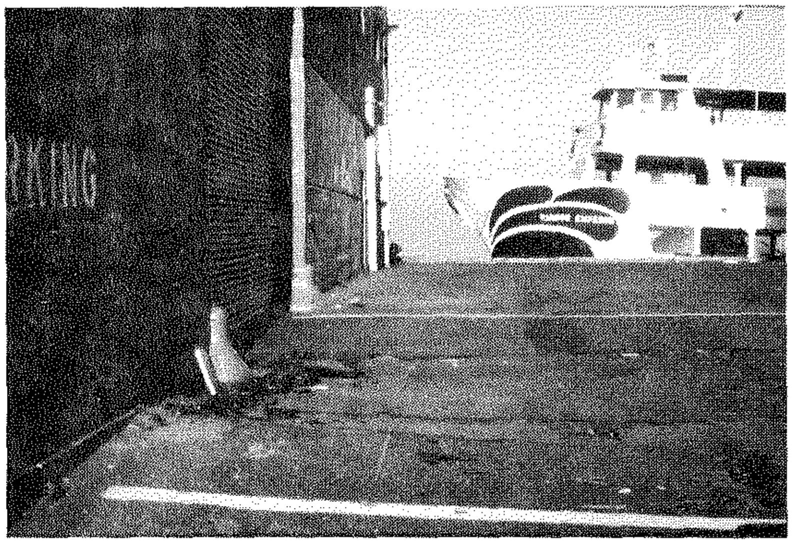


Fig. 3.33: Settlement at the South End of Pier 45, San Francisco

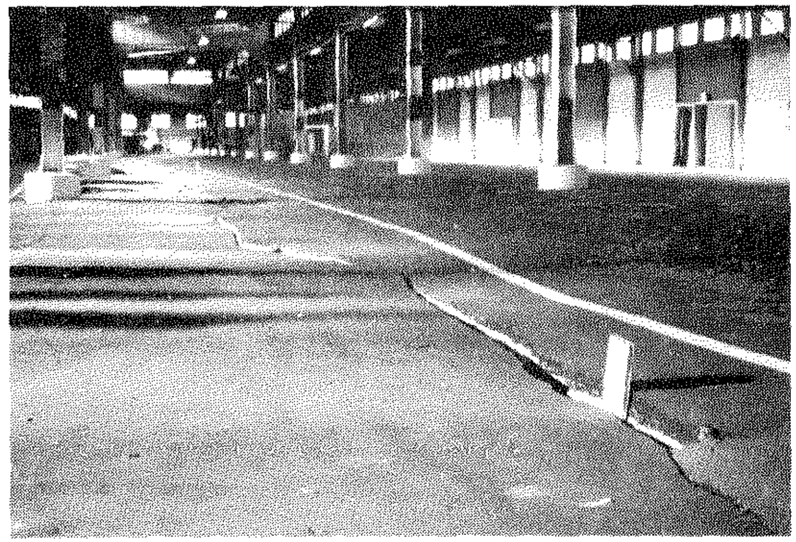


Fig. 3.34: Cracking Along Pier 45, within Warehouse, San Francisco Embarcadero Seafront

of the seafood processing operations previously located in several of these structures. This, in turn, had a significant impact on the local fishing industry, as facilities located at Pier 45 process a significant portion of the area's fishing catch.

3.3 The West San Francisco Bay Shoreline:

Scattered evidence of minor settlements apparently related to liquefaction or at least partial pore pressure generation and subsequent densification occurred south of the old Mission Bay region in the vicinity of Islais Creek channel and in northwest Hunter's Point. This caused minor pavement cracking, but no significant structural damage. Immediately to the south, soil liquefaction also occurred at Hunter's Point Naval Station. Liquefaction at this site occurred in loose sandy fill placed within sheet pile "cells" to form a pier area. Significant settlements of up to 6 inches occurred in the pier fill, and the outlines of sheet pile cells could be seen. A large sink hole also occurred at this site and silt boils covered an area of approximately 200 square feet. No apparent damage to the sheet pile cell walls or to structures or other facilities occurred at this location, and the pier pavement was simply repaired.

South of San Francisco, liquefaction occurred on undeveloped land at the Bay shore immediately north of San Francisco International Airport, as evidenced by several sand boils. It is important to note that this land, owned by the Airport, contained no structures, buried utilities or other facilities and that as a result this minor liquefaction caused no damage. There does not appear to have been any damage to the runways or taxiways at the Airport during the Loma Prieta earthquake, and no evidence of liquefaction was observed in close proximity to buildings or other facilities.

As illustrated previously in Figure 1.1, evidence of minor soil liquefaction was also observed at several sites farther to the south along the west Bay shoreline. Sand boils and lateral spreading were observed at the perimeter of a sanitary landfill immediately north of Foster City (and immediately north of the San Mateo Bridge). This liquefaction does not appear to have damaged the landfill.

Sand boils were also observed and photographed on a beach at the southern edge of Foster City. These were removed by tidal action during the first few days following the earthquake. No liquefaction appears to have occurred in the engineered fill upon which Foster City itself is constructed, and no foundation-related damage or settlements appear to have occurred in Foster City.

South of Foster City, evidence of minor liquefaction was observed at a concrete-processing plant located near the Port of Redwood City. This plant suffered minor cracking and settlement of an aggregate storage silo, which appears to have been the result of foundation settlements. In addition, several sand boils were observed at an undeveloped bayshore site just south of Redwood City, approximately 1.5 miles north of the Dumbarton Bridge.

3.4 Treasure Island:

Treasure Island, a man-made island in the center of San Francisco Bay, was constructed in 1936-37 to create a site to host the 1939 Golden Gate Exposition (and World Fair.) Figure 3.37 shows an oblique aerial view of this low-lying island immediately north of the Bay Bridge. The island was constructed above a sand bar and Bay Mud (soft silt and clay sediments) immediately north of Yerba Buena Island (a rocky outcrop in the center of the bay.) Access to the island is achieved via the San Francisco-Oakland Bay Bridge, which is anchored at Yerba Buena Island, and then by means of a short causeway from Yerba Buena Island to Treasure Island.

The fact that the island was created to host a major international fair is only the first of several major similarities between this site and San Francisco's Marina District. The island was constructed on a shoal north of the rocky outcrop which forms Yerba Buena Island, and was created by hydraulically placing fill dredged from several sites on the bay floor, as shown in Figure 3.35. The hydraulic fill, which consisted primarily of sands and silty sands, was retained by a series of perimeter rock dikes. As a result of the variable depth of water over the site, and differing levels of exposure to potential wave erosion, the configuration of the rock dikes varied at different sections around the perimeter of the island, as shown in Figure 3.36. At all stations, however, the rock dikes were built using the "upstream" construction method in which successive dike stages were constructed inboard of the lower dike and so were founded on hydraulic fill from the previous stage of construction.

Soil conditions underlying the fill are highly variable. Portions of the fill are underlain by relatively soft, normally consolidated San Francisco Bay Mud. Other portions rest on sandier shoal material. Underlying the shallow and relatively recent deposits are older and much stiffer pleistocene deposits consisting primarily of overconsolidated clayey soils. The depth to bedrock varies, and generally increases with increased distance to the north from Yerba Buena Island.

These fill conditions, and the underlying foundation soil conditions, are similar in many ways to the conditions described previously in the Marina District of San Francisco. Most of the island consists of a surficial layer comprised of loose, sandy hydraulic fill (with varying fines content), with a water table near the surface. This fill overlies generally sandy and clayey sediments to varying depths, and these served to amplify the levels of shaking which propagated up to the overlying hydraulic fill. A single instrumental recording on the Treasure Island fill showed a peak horizontal acceleration of 0.16 g, as contrasted with a nearby instrument on (rocky) Yerba Buena Island which recorded a peak acceleration of only 0.067 g.

The loose hydraulic fill was vulnerable to liquefaction at moderate levels of shaking, and evidence of liquefaction was pervasive over most of the island in the wake of the earthquake. Several notable exceptions included zones where site improvement techniques had been used to mitigate liquefaction susceptibility for specific facilities: these were successful in all cases.

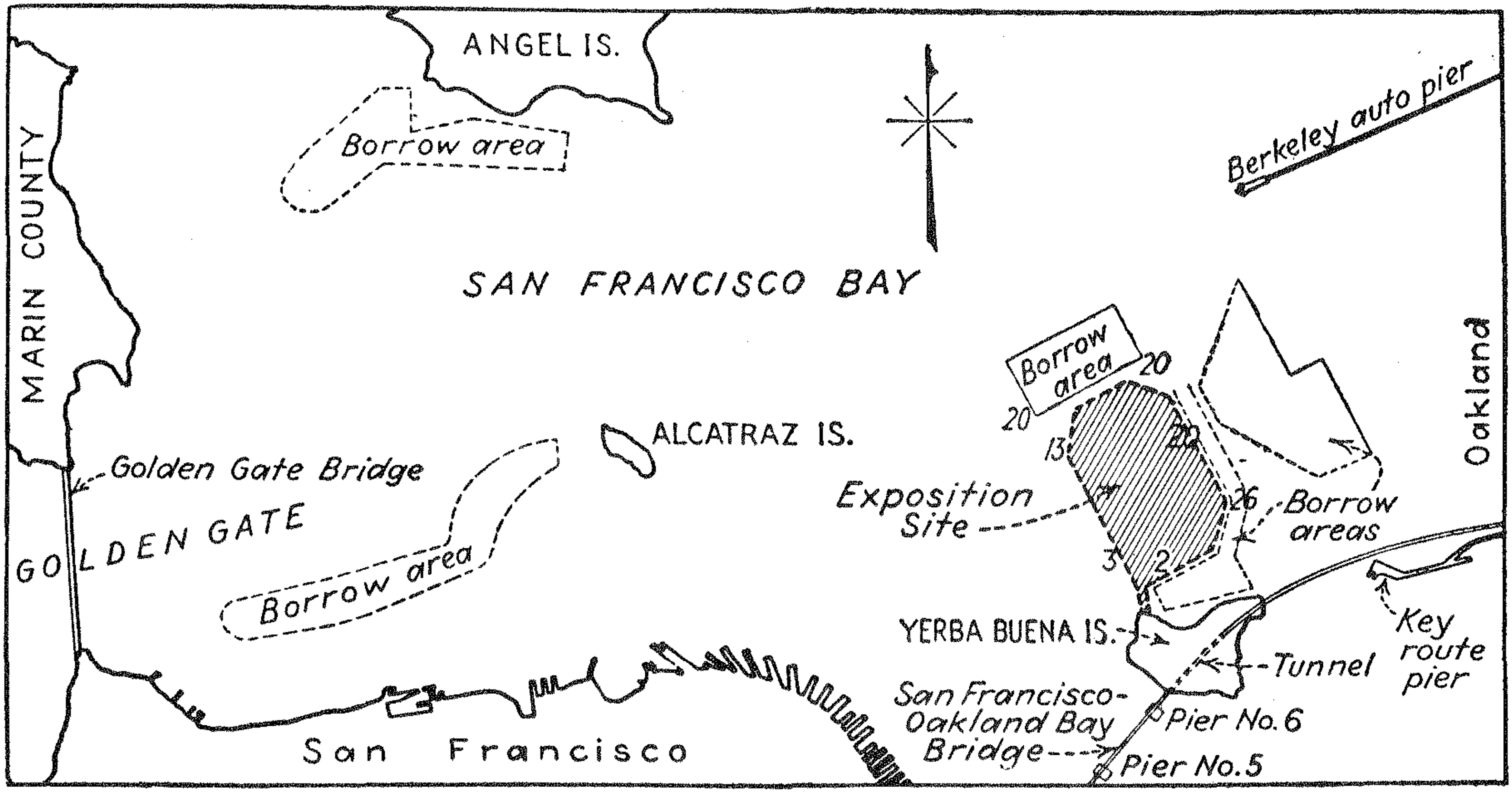


Fig. 3.35: Map Showing the Location of Treasure Island ("Exposition Site"), Yerba Buena Island, and Dredge Borrow Areas from Which Fill was Obtained [U.S. Army Corps of Engineers, 1937]

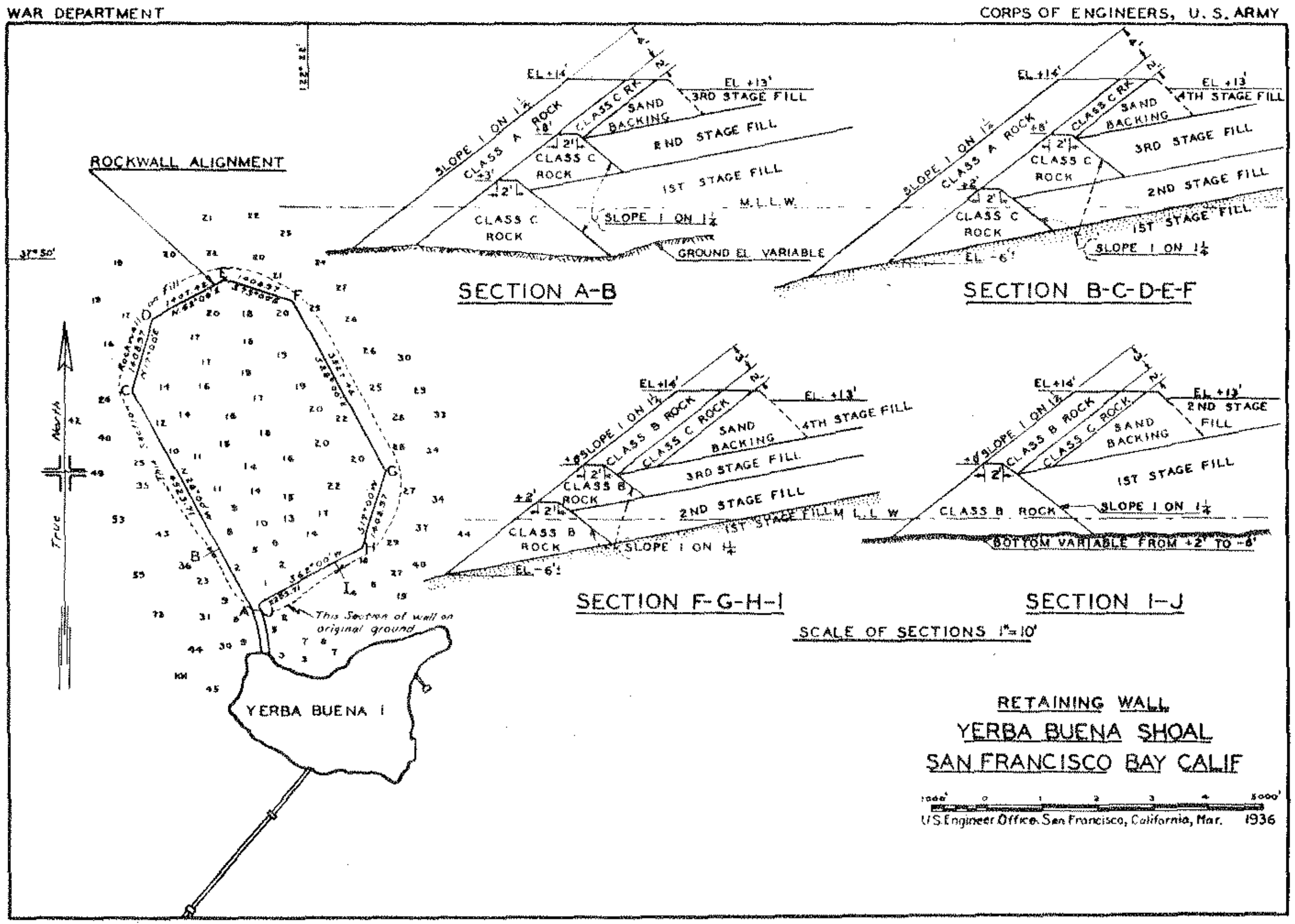


Fig. 3.36: Configurations of Perimeter Rock Dikes; Treasure Island [U.S. Army Corps of Engineers, 1937]

Sand boils occurred at numerous locations, as did surface settlements of up to 12 inches. Numerous pipe breaks occurred and most of the island was without water service for three days following the earthquake. Lateral spreading and settlement of the crests of the levees surrounding the island occurred in a number of locations. The maximum levee crest settlement appears to have been a settlement of nearly 2 feet at the northern end of the island. Numerous small cracks and fissures also suggest some lateral ground movements, especially near the edges of the island. Particularly noteworthy is a crack near the east side of the island which runs parallel to the edge levee and appears to be essentially continuous over a distance of approximately 2500 feet. This crack or fissure passes through Building No. 7, and this structure achieved some notoriety in post-earthquake press reports as the floor slab cracked and a major sand inflow occurred, filling the ground floor of the building with sand to a depth of as much as 6 inches in one area and causing the building's occupants to undertake a hasty exit.

Figure 3.38 is an aerial view of the northern end of the Island, taken on the day after the Loma Prieta Earthquake. Numerous light spots in this photograph (many of which are reflecting sunlight) are sand boils which occurred during the Loma Prieta Earthquake. Sand boils and settlements occurred over most of the Island, with the few exceptions being localized zones in which ground improvement techniques had been applied. Many of the sand boils were quite large. Figures 3.39 and 3.40 show examples of large boils near the central western edge and southwest corners of the island, respectively. Sand boils of up to 20 feet in diameter occurred in numerous locations, and some of these exuded "clay balls" as well as sandy and silty boil material. Figures 3.41 and 3.42 present examples of two areas in which sand boils, settlements and associated expulsion of water created ponds of fair size. Figure 3.41 shows boil material and ponded water at the schoolyard on 12th Street, near the center of the island, and Figure 3.42 shows similar conditions on 9th Street several blocks farther south.

Figure 3.43 shows the area at the head of a wharf at the south-eastern corner of the Island. Soils adjacent to the area immediately inboard of the wharf (at the heads of a nearby group of wharfs) had been densified by vibro-flotation and showed no signs of liquefaction, settlement, or lateral spreading. Adjacent to the improved zone, however, major sand boils occurred in the fill. These can be seen in the upper right-hand corner of Figure 3.43, and a closer view of one of these boils, as well as work to repair ruptured buried utilities, is presented in Figure 3.44.

This good performance of the densified hydraulic fill at the head of several wharves was repeated in several other locations where different ground improvement techniques had been employed to mitigate liquefaction potential for the fill underlying a limited number of specific structures and facilities. Ground improvement techniques used at these sites included vibroflotation, compaction piles, and gravel columns. All appear to have successfully prevented liquefaction of the hydraulic fill in the areas in which they were employed.

Fig. 3.37: Oblique Aerial View of Treasure Island Looking North Across the Bay Bridge

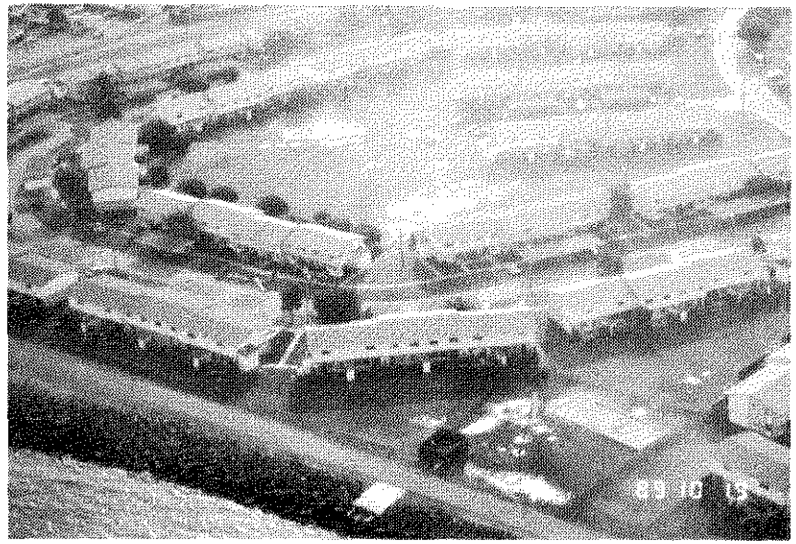


Fig. 3.38: Oblique Aerial View of the Northern End of Treasure Island on October 18, 1989

Fig. 3.39: Large Sand Boils near the Western Edge of Treasure Island [Photo courtesy of David T. Schrier]

Fig. 3.40: Large Sand Boils Near the Southwest Corner of Treasure Island [Photo Courtesy of David T. Schrier]

Fig. 3.41: Sand Boils and Ponded Water at Schoolyard on 12th Street, Treasure Island, on October 18, 1989. [Photo courtesy of David T. Schrier]

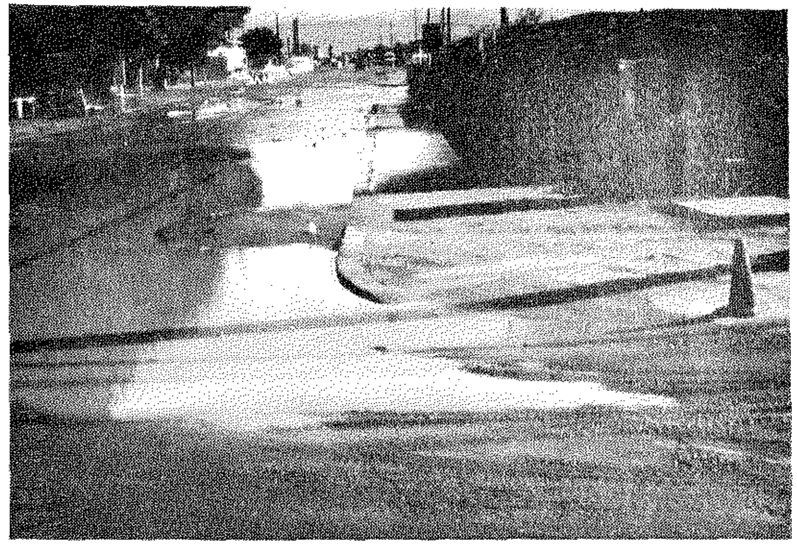


Fig. 3.42: Sand Boils and Ponded Water on 9th Street, Treasure Island, on October 18, 1989. [Photo courtesy of David T. Schrier]

Fig. 3.43: Aerial View of Southeast Corner of Treasure Island on October 19, 1989

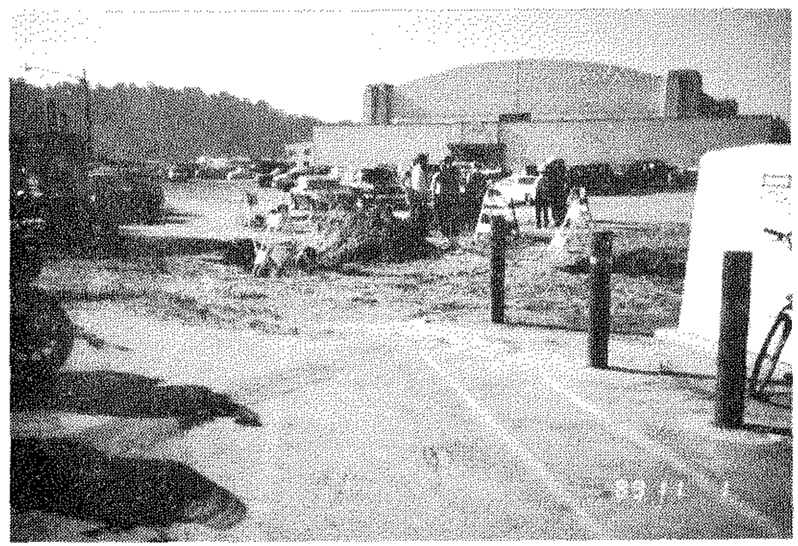


Fig. 3.44: Closer View of the Area in the Upper Right-Hand Corner of Figure 3.43

Although liquefaction appears to have occurred over most of the island, it appears to have resulted in relatively little serious damage to structures. In addition to Building No. 7, which was significantly damaged, several other structures of 1930's vintage suffered cracking and/or separation of their base slabs, and minor sand inflows occurred in at least two of these structures. This relative lack of serious structural damage is almost certainly the result of the relatively moderate levels and duration of ground shaking produced by the Loma Prieta Earthquake, which was centered more than 50 miles to the south. Although structural damage was light, it should be noted that this relatively moderate shaking did produce general subsidence on the order of 3 to 12 inches over large parts of the island, and levee settlements, cracks and other clear evidence of limited lateral spreading occurred near the edges of the island on portions of all sides. Studies are currently underway to evaluate the likely severity of liquefaction, and the depth to which it might extend, in the event of a larger and/or more near-field earthquake producing stronger ground shaking and/or similar levels of shaking but of greater duration. In addition to providing a basis for evaluation of the foundation performance of individual structures, these studies will also address the likelihood of major settlements of the island, major movements associated with lateral spreading, and potential stability failures at the fill edges.

3.5 The East San Francisco Bayshore Region:

As shown in Figure 3.45, soil liquefaction also occurred at a number of sites in the east San Francisco Bay Area along the east bay shoreline. Liquefaction occurred as far north as the harbor at Richmond, and caused minor damage at sites progressively farther south along the shoreline in Berkeley and Emeryville. Liquefaction also caused considerable damage to the San Francisco-Oakland Bay Bridge approach fill in this area. Immediately south of the Bay Bridge, soil liquefaction caused significant damage to a number of facilities at the Port of Oakland and at Alameda Naval Air Station. Additional liquefaction, which resulted in little damage, occurred farther south along the Alameda shoreline and at Bay Farm Island. Finally, soil liquefaction caused significant damage to the north end of the main runway at Oakland International Airport. This section will briefly describe liquefaction-related damages at these sites throughout the east San Francisco Bayshore region.

3.5.1 Richmond Harbor

As indicated in Figure 3.45, soil liquefaction occurred at a site at the western end of Richmond Inner Harbor. This site, approximately 55 miles north of the fault rupture, represents the most distant site from the zone of energy release to suffer significant soil liquefaction in the Loma Prieta Earthquake.

Much of the land at the edge of Richmond's Inner Harbor was created by placement of sandy hydraulic fill. As with the coastal regions of San Francisco and

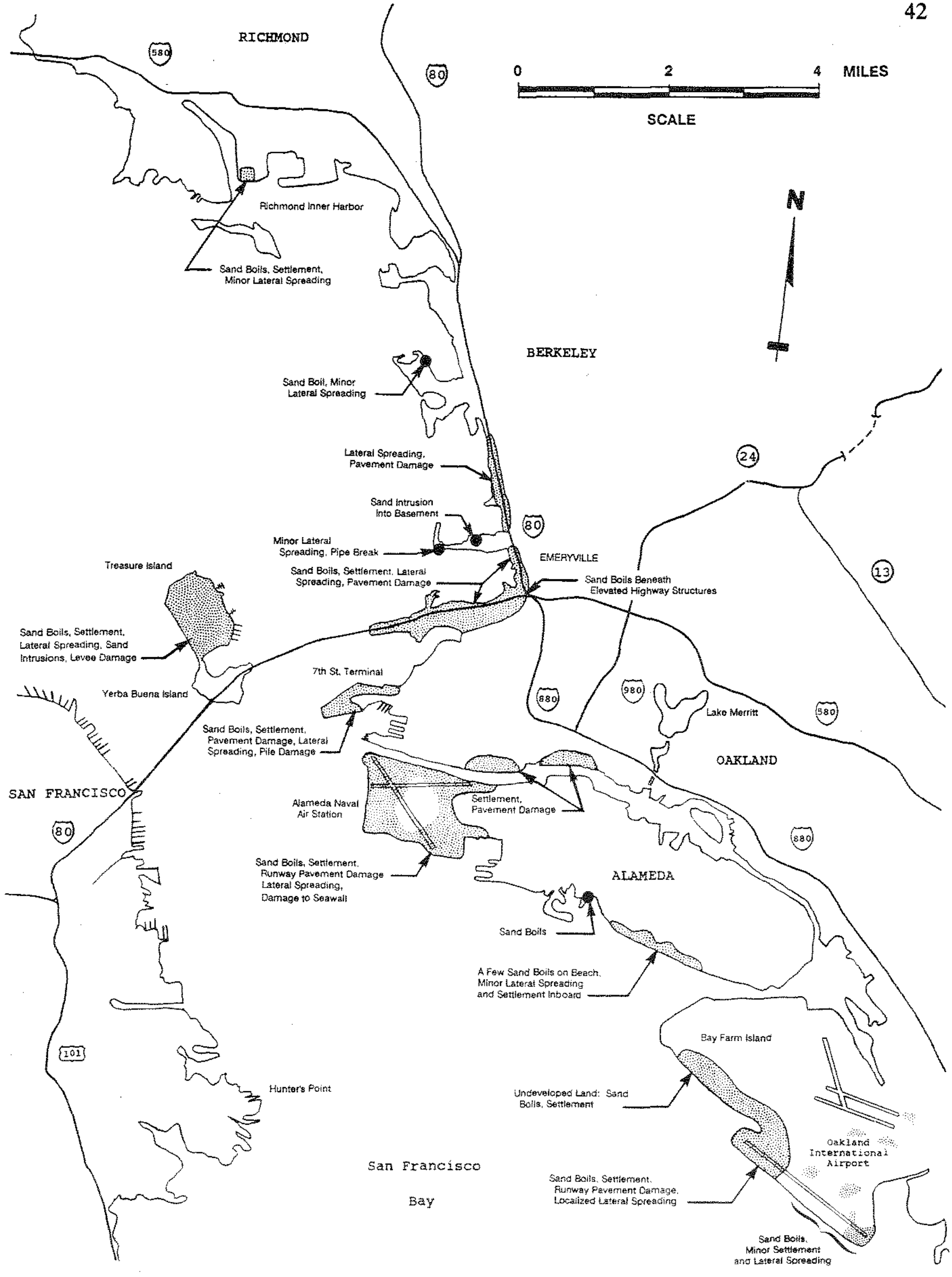


Fig. 3.45: Map of the East San Francisco Bay Shore Region Showing Sites Which Experienced Liquefaction During the Loma Prieta Earthquake of October 17, 1989

Treasure Island described in the previous sections, this fill, in most places, overlies deposits of normally consolidated clay known locally as San Francisco Bay Mud, which are underlain, in turn, by stiffer cohesive soils. The area which liquefied on October 17, 1989 is a zone approximately 250 ft wide and 1,000 ft long at the edge of the harbor at the foot of 10th Street. This site lies at the extreme western end of the Richmond Inner Harbor, and overlies a historic stream channel so that the underlying cohesive soils at this site are considerably deeper than those underlying the rest of the Inner Harbor fill. It appears likely that these deeper cohesive deposits amplified the levels of acceleration at this site and so contributed to the observed soil liquefaction.

The site in question is mainly open, undeveloped ground with no structures, utilities, or other facilities. Major harbor facilities, including various liquid storage tanks as well as major warehouses, docks, etc., all occur farther to the east along the shore of the inner harbor.

At the site in question, four large sand boils and a dozen smaller boils vented fine sands and silty sands to the surface. In addition, minor settlements and lateral spreading occurred at the edge of the harbor adjacent to a small pile-supported dock. This dock and an adjacent small structure are the only developed facilities in the apparent zone of liquefaction. Neither of these was seriously damaged by what appear to have been settlements of on the order of approximately 1 to 3 inches, and lateral displacements of similar magnitude as a result of lateral spreading.

Figures 3.46 and 3.47 show sand boils near the middle of the area in which liquefaction occurred. Figure 3.48 shows the small dock and the adjacent structure which was lightly damaged by the liquefaction-induced ground displacements. Figure 3.49 shows a warehouse adjacent to the area in which liquefaction occurred. This structure suffered considerable damage during the earthquake, but this damage does not appear to have been caused by liquefaction of the foundation soils. Instead, this building appears to have suffered damage as a result of strong shaking. The absence of damage to a number of similar masonry warehouse structures at other sites in the Richmond Inner Harbor provides support for the hypothesis that the unusually deep clays at the extreme western edge of the Inner Harbor, overlying the old channel, amplified the levels of ground shaking produced in this area.

3.5.2 The Berkeley/Emeryville Bayshore Region

Soil liquefaction occurred at a number of sites along the Berkeley/Emeryville bay shoreline. As shown in Figure 3.45, a single sand boil, and minor lateral spreading, was observed at the eastern edge of the landfill immediately south of the Municipal waste dump at the foot of University Avenue in Berkeley.

South of University Avenue, lateral spreading and minor settlements damaged the pavements of both the Interstate 80 coastal highway and the frontage road outboard of the highway. Pavement cracks parallel to the coastline occurred in both

Fig. 3.46: Sand Boils on Undeveloped Land at Richmond Inner Harbor

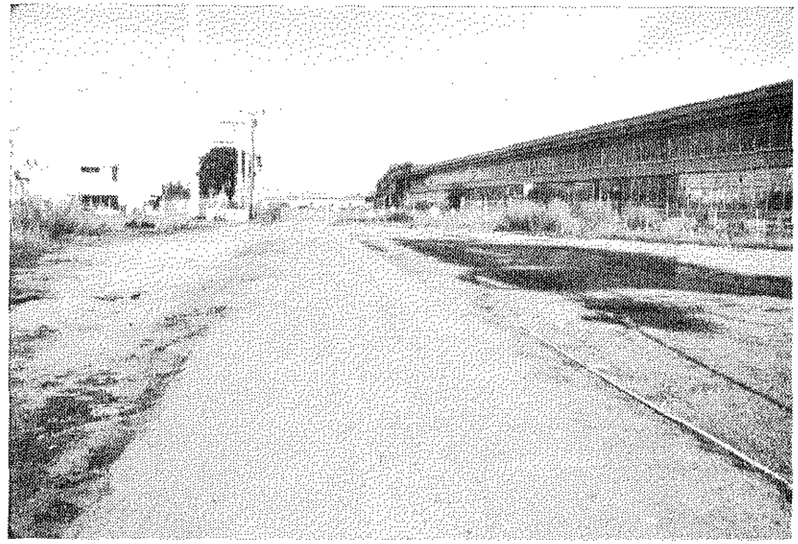


Fig. 3.47: Sand Boils on Undeveloped Land at Richmond Inner Harbor

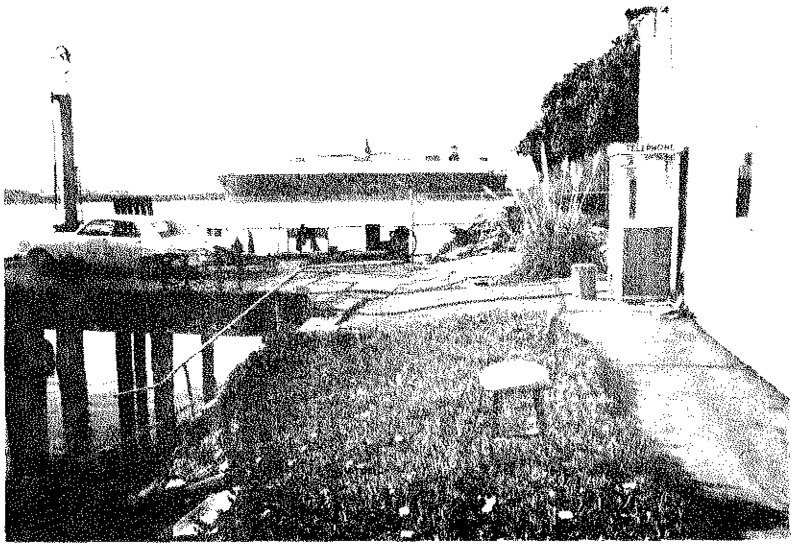


Fig. 3.48: Small Dock and Adjacent Structure at Edge of Apparent Zone of Liquefaction at Richmond Inner Harbor

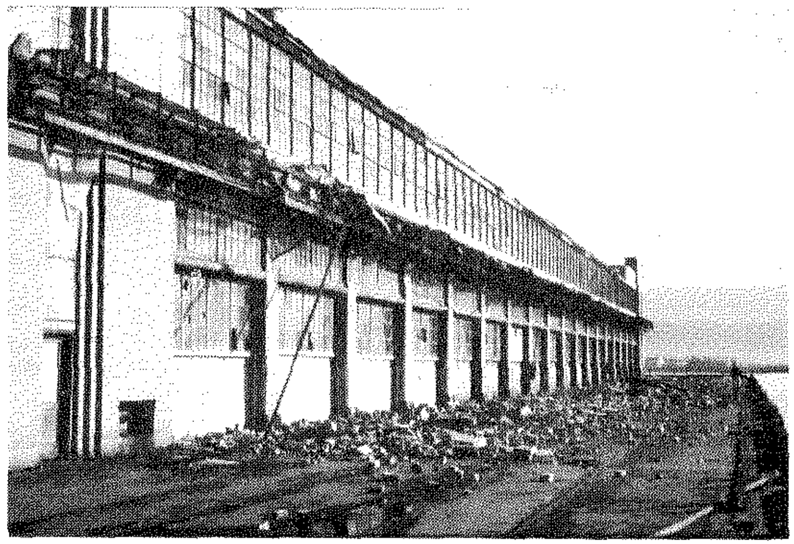


Fig. 3.49: Damage to Warehouse Structure at Richmond Inner Harbor

of these roadways south of Ashby Avenue and north of Emeryville's Watergate peninsula fill. Most cracks were less than an inch in width, though some cracks were continuous for well over 100 feet. Little serious damage was done to the pavements in these areas, and these pavements were repaired by simply repaving within two days after the earthquake.

As indicated in Figure 3.45, a minor sand intrusion occurred within a basement parking structure at the Watergate peninsula fill. No serious damage to the building resulted. In addition, minor lateral spreading cracked the road pavement and caused a pipe break at the southwest corner of this peninsula fill. It could not be ascertained with certainty whether this apparent lateral spreading was the result of soil liquefaction, or of near slope stability failure of the edge of the fill which rests upon soft, weak clay (San Francisco Bay Mud).

Along the southern Emeryville coast, from the Watergate peninsula fill south to the Bay Bridge mole (approach fill peninsula), both the highway and frontage road pavements were again cracked and fissured as a result of lateral spreading towards the Bay. Lateral spreading appeared to be a bit more pronounced in this region, with total lateral movements of on the order of 1 to 4 inches. In addition, several of the fissures exuded sands and silty sands, and several additional sand boils occurred outboard of the frontage road. Figure 3.50 shows an example of an open fissure which exuded sandy material in this area. The exuded boil materials were fine sands and silty sands, and were typical of the hydraulic fill materials which were placed to create much of the bayfront areas of Emeryville, as well as portions of west Oakland and Alameda.

The hydraulic fills at the Emeryville shoreline extend inland (east) across the highway, but no liquefaction was observed on the inboard side of the highway. This may have been due, in part, to shallower deposits of Bay Mud and/or of the stiffer, older cohesive sedimentary units generating lesser levels of acceleration inboard of the highway. It is also due, in large part, to recent re-development of much of this area. Major portions of the inboard fill have been densified by vibroflotation techniques as part of foundation preparation for new structures built since 1970 in this area.

3.5.3 The Bay Bridge Mole

As shown in Figure 3.45, the Bay Bridge mole (or peninsula approach fill) immediately south of Emeryville, was extensively damaged by soil liquefaction. Appreciable settlements occurred over most of the peninsula fill, and settlements of up to 16 inches occurred in several locations. In some cases, differential settlements produced an uneven, "hummocky" pavement surface with permanent "waves" of up to 6 inches in amplitude. Lateral spreading was also significant along most of the fill, and produced numerous longitudinal fissures in the road pavement parallel to the fill edges. Many of these were of considerable length (one was over 300 feet long), and open fissure widths of 1 to 3 inches were not uncommon. Many of these fissures

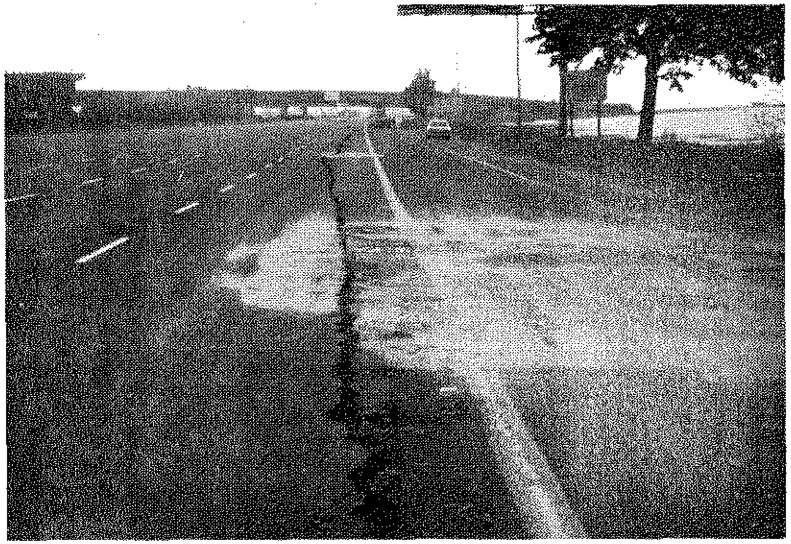


Fig. 3.50: Example of Fissure in Bayshore Highway in Southern Emeryville caused by Lateral Spreading Due to Liquefaction

Fig. 3.51: Example of Pavement Fissure Due to Lateral Spreading and Settlement at the Bay Bridge Approach Fill

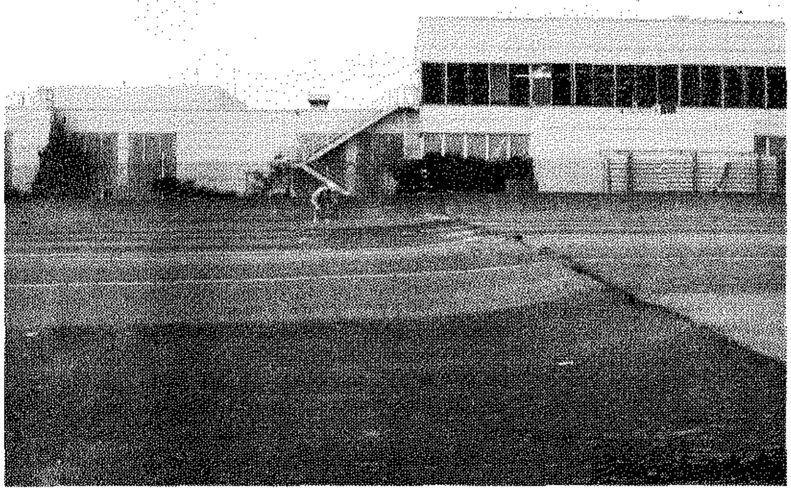


Fig. 3.52: Pavement Settlement Adjacent to the Operations Building on the Bay Bridge Approach Fill

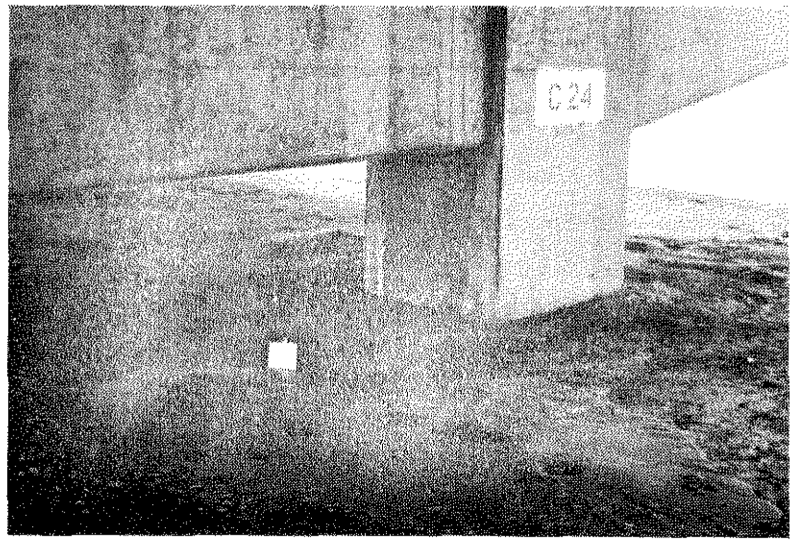


Fig. 3.53: Example of Sand Boils Beneath the Elevated Distribution Structure Inland of the Bay Bridge Approach Fill in Oakland

exuded fine sands and silty sands, and numerous additional sand boils occurred off the shoulders of the roadway in open, undeveloped lands at the edge of the bay.

Figure 3.51 shows an example of a fairly "typical" fissure, infilled with exuded boil materials, near the southwest edge of the approach fill. This particular fissure opened to a maximum width of approximately 3 inches, and exhibited a maximum differential vertical offset of 1.5 inches. Larger fissures, and more pronounced offsets, were not uncommon. Figure 3.52 shows liquefaction-induced settlement of the pavement adjacent to the south side of the operations and maintenance building at the toll plaza, farther out along the peninsula fill. The pavement at the right of this picture did not settle because it is supported by an underground access tunnel which crosses under the highway at this location. The building itself is pile-supported, and was not significantly damaged by the liquefaction, settlement and lateral spreading of the surrounding fill.

Minor liquefaction, as evidenced by small sand boils, also occurred beneath several elevated sections of the highway "distribution structure" immediately inland of the Bay Bridge approach fill. Figure 3.53 shows several of these boils adjacent to one of the elevated support bents in this area. This minor liquefaction does not appear to have resulted in any significant damage to the distribution structure in this event, and no evidence of similar liquefaction has been found at the southeast end of the distribution structure where it joins the elevated Cypress viaduct section of Interstate 880.

3.5.4 The Port of Oakland

Immediately south of the Bay Bridge, soil liquefaction caused considerable damage to container terminal facilities at several locations in the Port of Oakland. Much of the extreme western region of Oakland, south of the Bay Bridge approach fill, is filled land underlain at shallow depths by relatively soft normally consolidated clay (Bay Mud) and at greater depths by older, stiffer (though still primarily cohesive) deposits. These older, stiffer soil units are underlain by bedrock at a depth of several hundred feet throughout much of the area. Much of the fill is loosely dumped and/or hydraulically placed sandy fill. As a result, a number of sites in this area have conditions similar to those described in virtually all of the previous sections of this chapter: saturated, loose, cohesionless and thus potentially liquefiable surface fills underlain by deep, cohesive soil deposits which amplified the local accelerations produced by the Loma Prieta Earthquake.

As shown in Figure 3.45, three general areas encompassing four major container terminals owned by the Port of Oakland were significantly damaged by soil liquefaction during the earthquake. The four terminals damaged were: (a) the 7th Street Terminal (peninsula fill), (b) the Matson Terminal (at the juncture of the northeast corner of the 7th Street Terminal and the southern end of the Outer Harbor terminals), (c) the APL Terminal at Middle Harbor, and (d) the Howard Terminal (farther east along the Inner Harbor, north of Alameda Island).

All of these terminals have pile supported wharves at the edges of the terminal fills. In most areas, these piles extend through perimeter sand and rock dikes which serve as containment for the hydraulic fill which forms the terminal land inboard of the wharves. The hydraulic fill consists primarily of fine dredged sands and silty sands.

The most severe damage to Port facilities occurred at the 7th Street Terminal. Liquefaction of the hydraulic fill resulted in settlement, lateral spreading, and cracking of the pavement over large areas of the terminal. Maximum settlements of the paved container yards inboard of the wharves were on the order of one foot. A number of large cranes operate along the edges of the fill, and these traverse laterally along the wharves on railroad tracks. The outboard rail was pile supported, on the concrete wharf, and suffered no appreciable settlement. The inboard rail, however, was supported on the fill throughout much of this terminal. As a result, differential settlements of the inboard rail, as well as pavement damage due to general settlement and lateral spreading of the fill, rendered a number of these major cranes immobile immediately after the earthquake. The Matson docks, however, were able to continue limited operations as both the front and rear crane rails were founded on the pile-supported concrete wharf deck in the Matson dock area.

Figure 3.54 shows two of the large cranes operating along the northern side of the west end of the 7th Street Terminal, and Figure 3.55 shows a closeup view of damage to the pavement beneath the crane nearest to the camera in Figure 3.54. Similar cracks and fissures occurred along much of the terminal. Figure 3.56 shows a typical sand boil in the paved container yards in this area.

In addition to settlement and lateral spreading, and associated pavement damage and related mobility problems for the large terminal cranes, damage occurred at the tops of a number of piles supporting the wharves in this area. Figure 3.58 is a cross-section through the northern edge of the 7th Street Terminal showing the rock dike, the pile-supported concrete wharf, and the hydraulic fill. Damage to piles occurred at the tops of the inboard, battered piles and consisted primarily of tensile failures, though some piles also appeared to have been damaged in shear and compression. Figure 3.57 shows typical damage to the top of one of the concrete piles at this location. The vertical piles farther outboard were largely undamaged (a few isolated piles sustained minor damage). It has not yet been conclusively established whether this pile damage was caused by (a) liquefaction of the hydraulic fill and associated increased lateral thrust, spreading and settlement, or (b) by oscillatory ground movements associated with strong ground shaking during the earthquake. This pile damage appears to represent an example of the damage that can occur as a result of employment of battered piles to resist lateral movements in relatively soft, compliant soil foundations. These piles represent a "stiff" inclusion in an otherwise relatively flexible surrounding foundation and structural system. The result is large stress concentrations, alternately tensile and compressive, during earthquake shaking. The mode of failure observed, however, was predominantly tensile failure driven by outboard thrust of the fill, suggesting that liquefaction and associated lateral spreading were important factors.

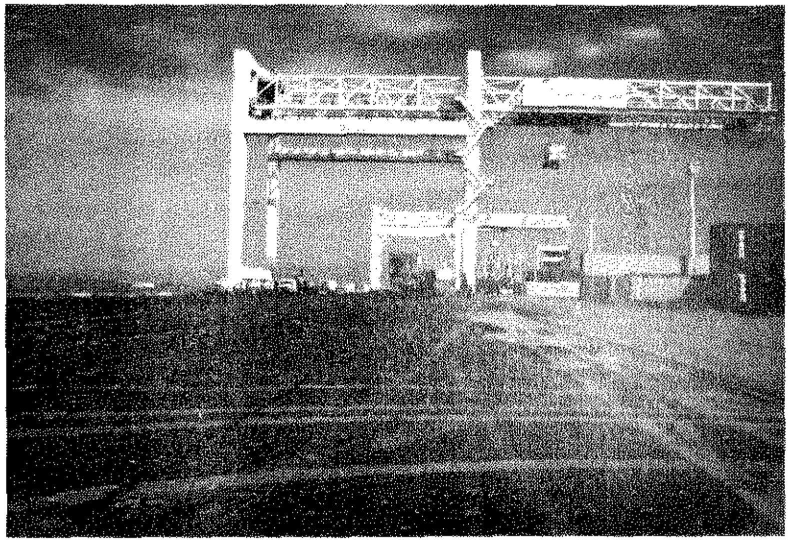


Fig. 3.54: View Looking East Along the 7th Street Terminal Showing Two of the Large Cranes at this Terminal and Pavement Damage

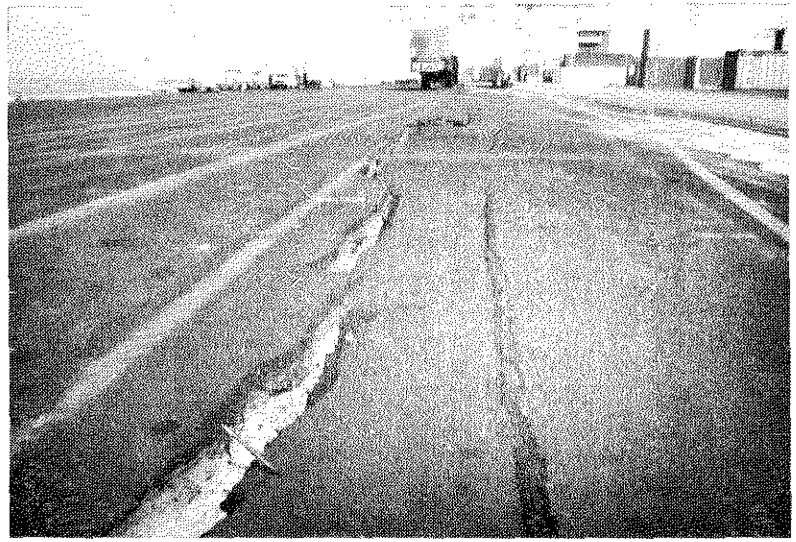


Fig. 3.55: Closeup View of Pavement Damage from Figure 3.54, Showing Open Fissures Exhibiting Both Vertical and Lateral Differential Movements

Fig. 3.56: Sand Boil at the 7th Street Terminal

Fig. 3.57: Tensile Failures at Top of Battered Piles at the 7th Street Terminal [Photo courtesy of the U.S. Army Corps of Engineers, South Pacific Division]

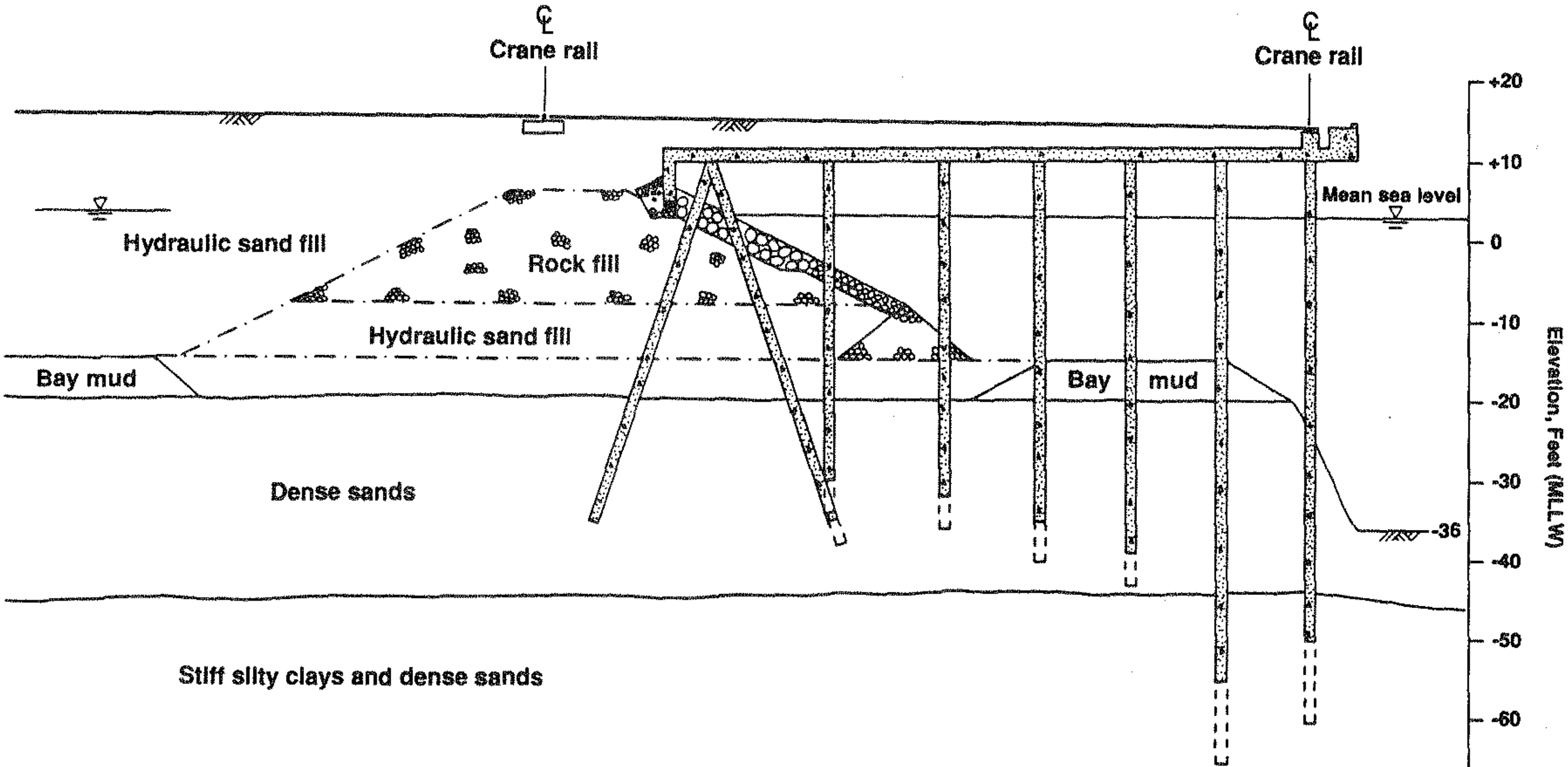


Fig. 3.58: Cross-Section Through the Edge of the Fill, Dike and Wharf at the North Side of the 7th Street Terminal, Port of Oakland [Courtesy of EERI Post-Earthquake Investigation Report, in progress]

As a result of widespread damage to the battered piles, the Port of Oakland is replacing the battered piles with vertical piles designed to resist lateral forces, but providing limited lateral flexibility during earthquake loading that will be more compatible with the surrounding foundation soils and structural systems. In addition, the pile-supported wharf deck will be extended inboard, with additional piles, to provide improved support for the inboard crane rails in this area.

An additional liquefaction-related feature occurred on the southern shore of the 7th Street Terminal peninsula fill. A small public recreational park, located at the eastern end of the apparent zone of liquefaction on this southern shoreline, sufferred lateral spreading with movements towards the bay of several feet. There are no significant structures or developed facilities at this open park site, so no serious damages appear to have occurred. It is, however, interesting to note that the nature and magnitude of the observed shoreline deformations are strongly suggestive of near or incipient stability failure of this shoreline.

The Port of Oakland's APL and Howard Terminals are largely similar to the 7th Street Terminal except that: (a) the Howard fill containment dikes are comprised entirely of rock, and the APL containment dikes entirely of sand, (b) both the front and rear crane rails are pile supported, and (c) all piles supporting the wharves and the crane rails at the Howard Terminal are vertical or nearly vertical (max. batter = 1:12). Liquefaction of the hydraulic fill caused appreciable settlements over large areas at both the APL and Howard Terminals, with maximum settlements of up to 12 inches. This damaged pavements at the edges of the wharves and in the inboard container yards, but there was no apparent damage to piles, and there were no significant adverse movements of the crane rails. It appears likely that the pavement damage at these Terminals can be relatively simply repaired.

3.5.5 Alameda Naval Air Station

As shown in Figure 3.45, soil liquefaction occurred over large areas of Alameda Naval Air Station, immediately south of the Port of Oakland. Numerous sand boils, some of them very large, and significant settlements and lateral spreading occurred over a large area at the western end of the Station. The airfield runways that occur in this area were significantly damaged. Figure 3.59 shows a large sand boil (and sinkhole) adjacent to the intersection of the two runways, and Figure 3.60 shows damage to the runway shoulder pavement nearby. Both runways, and two taxiways, were significantly damaged and were rendered inoperational immediately after the earthquake. Damage to the pavements consisted of heaving and/or settlement, minor spreading, and resulting uneven surfaces as well as cracking and separation at joints. One large area at the runway intersection appeared to have heaved as much as four inches, and considerable cracking of the asphalt pavement slabs occurred in this area. Maximum crack and/or joint openings were on the order of four inches, though most were considerably narrower. Vertical offsets across joints and cracks ranged from zero to approximately two inches.

Fig. 3.59: Large Sand Boil and Sinkhole Adjacent to Runway at Alameda Naval Air Station

Fig. 3.60: Example of Damage to Taxiway Pavements; Alameda Naval Air Station

Fig. 3.61: Minor Settlement and Buckling of Pavement at the Southwest Edge of Alameda

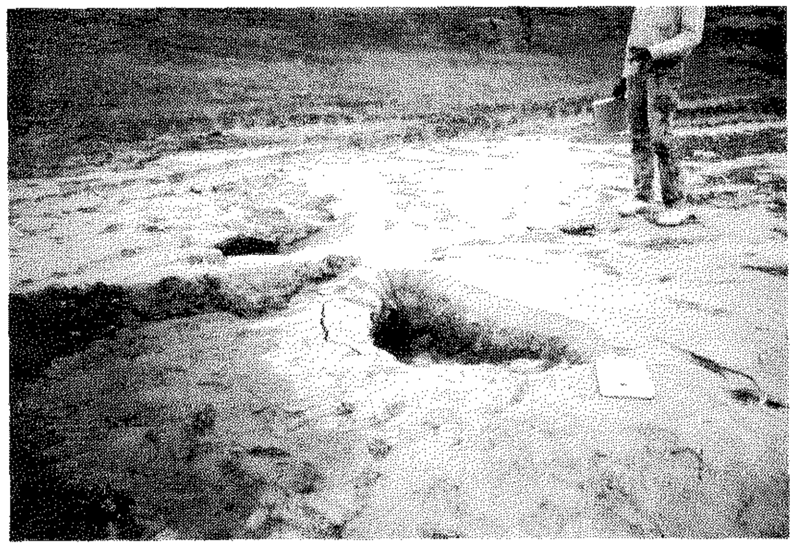


Fig. 3.62: Sand Boils on Undeveloped Land at the Northwest Corner of Bay Farm Island

As with many of the sites discussed previously, the western portion of Alameda Naval Air Station is built upon sandy hydraulic fill, underlain at shallow depths in many areas by soft Bay Mud, and at greater depths by older, stiffer cohesive soil units. Settlements on the order of several inches to a foot occurred over much of the western end of the Station, in the runway areas, but relatively little liquefaction occurred to the east in the area occupied by most of the buildings and other related base facilities. A few structures were lightly damaged in this area as a result of relatively modest foundation movements, but most buildings on the base were undamaged. Minor settlements and ground displacements in some areas resulted in separation of exterior steps and cracking of concrete sidewalks. In addition, several sewer and waterline breaks occurred in this area.

3.5.6 Alameda and Bay Farm Island

As shown in Figure 3.45, scattered evidence of liquefaction, as evinced by sand boils, minor settlements and minor lateral spreading, occurred at several locations along the west coast of Alameda. These ground deformations caused minor cracking of pavements and ruptured a number of pipelines, but caused no serious damage to structures. Figure 3.61 shows a typical example of the very minor ground deformations in this area.

South of Alameda, and immediately north of the Oakland International Airport, considerable liquefaction occurred at the northwest corner and at points along the western edge of Bay Farm Island. Numerous sand boils, many of them relatively large (e.g. Figure 3.62) occurred in this area. Most of Bay Farm Island consists of sandy hydraulic fill, underlain by Bay Mud and deeper, stiffer alluvium. The soil liquefaction, however, was largely confined to undeveloped, open lands in the areas indicated in Figure 3.45. Most of the rest of the island has been developed for residential housing and light commercial use, and the fill in the developed areas was densified by vibroflotation prior to construction of buildings. This densification, in all areas, successfully prevented soil liquefaction from damaging structures, though light damage to roadway and parking lot pavements occurred at the edge of one developed area.

3.5.7 Oakland International Airport

Immediately south of Bay Farm Island, soil liquefaction caused considerable damage to the main jet runway (Runway No. 11-29) at Oakland International Airport. Additional evidence of liquefaction, including sand boils, settlement and lateral spreading, occurred over wide areas of the airport fill to the north and south and east of the damaged runway section. As shown in Figure 3.45, the main runway is located at the southwestern edge of the Airport. Much of the runway and inboard taxiway area is loose, sandy fill underlain at shallow depths by soft clay (Bay Mud), and at greater depths by older and much stiffer estuarine deposits. The perimeters of the airport fill are lined with dikes to prevent inundation during unusually high tides and storms.

Extensive soil liquefaction occurred at the western section of the airport fill, and damaged the westernmost 3,000 ft of the 10,000 ft long main runway. In addition, the adjacent taxiway pavement was also heavily damaged. Figure 3.63 is an oblique aerial view showing the western end of the main runway (at the left of the photo), and the main taxiway inboard of this runway. Also shown to the west is a portion of the perimeter dike system. In this photograph, which was taken the morning after the earthquake, one can see light patches and lines which are exuded sandy material emitted from boils and cracks in the darker pavement surface. Figure 3.64 shows a closer view of some of these cracks and boils, and Figure 3.65 shows a much closer view of these taken from ground level.

The western 3,000 ft of the main runway was cracked and damaged by settlement, heaving and lateral spreading. Of this 3,000 foot long damaged section, the easternmost 2,000 ft were less heavily damaged than the extreme western portion, and were repaired within 4 weeks. As a result, the airport was able to resume essentially full operations with an only slightly shortened operational runway 9,000 ft in length on November 20 (after having operated at somewhat reduced capacity with 6,500 feet of operational runway up to that time.).

In addition to fill materials vented through cracks and joint separations in the runway and taxiway pavements, additional sand boils occurred on the open lands around and between these paved surfaces. Some of the largest sand boils observed in the earthquake occurred at this site. Figure 3.66 shows the central vent of one of the largest boils found. As shown by the hardhat and 12" extended steel tape measure, the central inner funnel of this sand boil is more than 4 ft in diameter. The overall sand boil cone was nearly perfectly round and symmetric, with an approximate height of 2 ft and an outer diameter of approximately 25 ft.

Figure 3.67 shows a crack in the pavement of the taxiway adjacent to the main runway, and Figure 3.68 shows cracking of the main runway. Cracks in the main runway and the adjacent taxiway had maximum widths of up to approximately 12 inches, and vertical offsets of up to 8 inches were observed. The fill in the area of the western end of the main runway underwent settlements of on the order of 6 to 12 inches, and evidence of lateral spreading also occurred over large parts of this fill zone.

In addition to liquefaction, settlement, and lateral spreading of the main fill in and around the west end of the main runway, the surrounding perimeter dikes at this end of the runway fill also suffered from settlement and lateral spreading in several places. Figure 3.69 shows cracking along the crest of a levee section near the northwest corner of the runway fill which suffered as a result of both settlement and lateral spreading. The maximum observed levee settlement in this area was on the order of 2 to 3 feet, and lateral deformations associated with spreading were generally similar in magnitude. Extensive liquefaction also damaged the fill to the north and west of the main runway. Figure 3.70 shows sand boils and major fissures caused by lateral spreading in this area. There are no pavements or structures in this area.

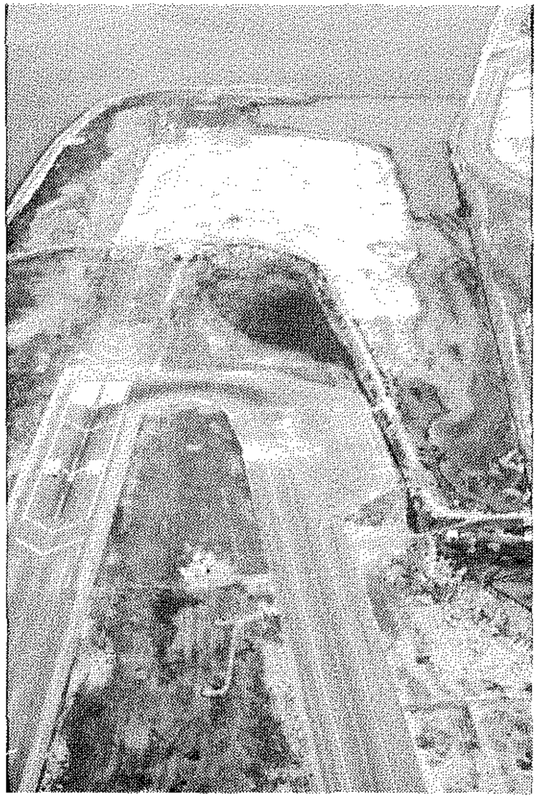


Fig. 3.63: Aerial View of the Western End of the Main Runway and Adjacent Taxiway at the Oakland International Airport on October 18, 1989

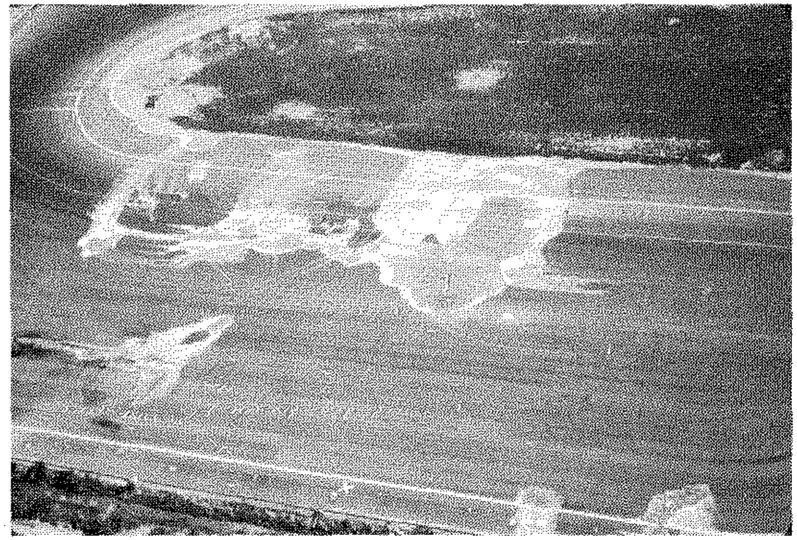


Fig. 3.64: Aerial View of the Main Taxiway Adjacent to the West End of the Main Runway, Oakland International Airport

Fig. 3.65: Fissures and Exuded Boil Materials, Main Runway, Oakland International Airport

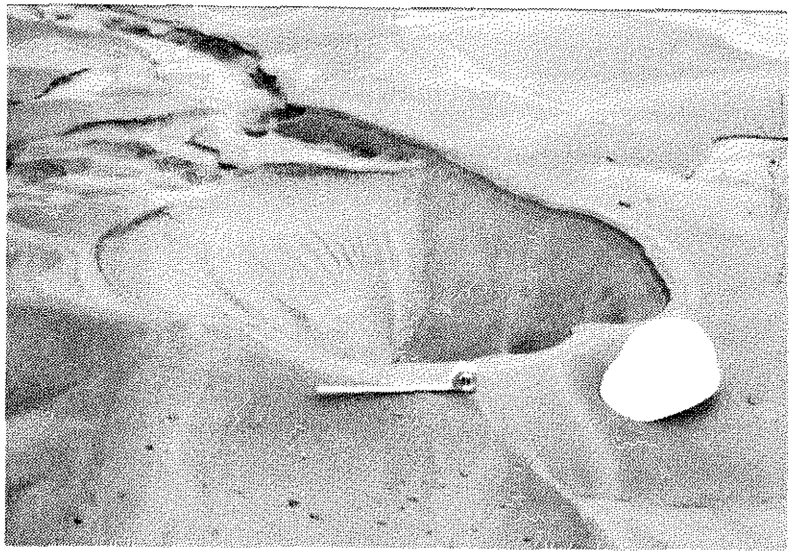


Fig. 3.66: Center of Large Sand Boil Near the North End of the Main Runway, Oakland International Airport

Fig. 3.67: Crack in the Pavement of the Taxiway Adjacent to the North End of the Main Runway, Oakland International Airport

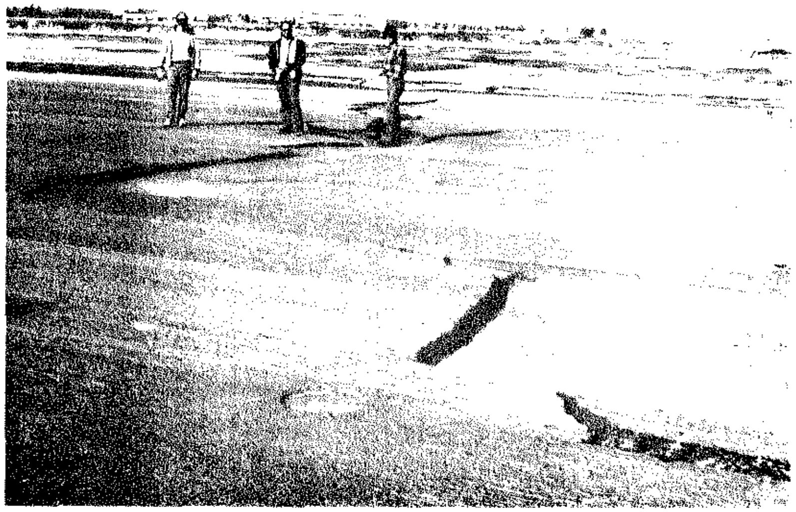


Fig. 3.68: Cracking of the Pavement of the Main Runway, Oakland International Airport [Photo courtesy of B. A. Vallerga]

Fig. 3.69: Cracking and Settlement of Perimeter Levee Near the Northwest Corner of the Oakland International Airport Runway Fill

Fig. 3.70: Major Fissures, Grabens and Sand Boils Near the Northwest End of the Main Runway, Oakland International Airport [Photo courtesy of B. A. Vallerga]

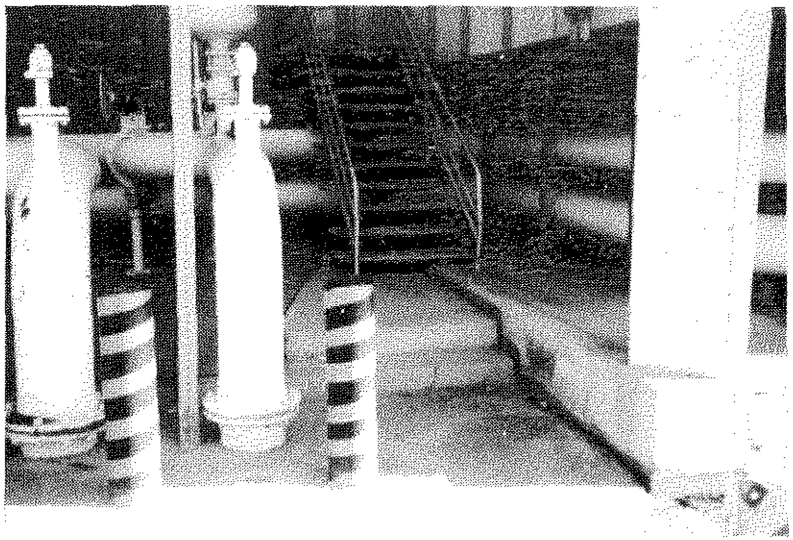


Fig. 3.71: Settlement Adjacent to Main Terminal Building, Oakland International Airport [Photo courtesy of B. A. Vallerga]

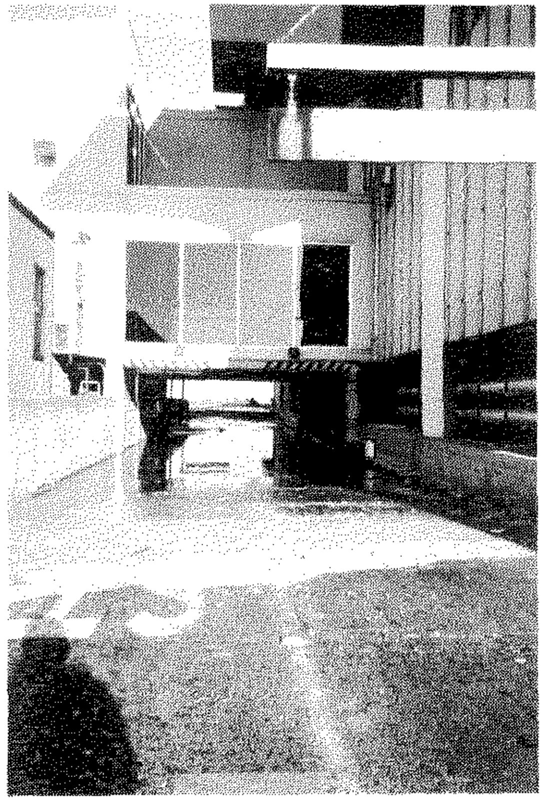


Fig. 3.72: Below-Grade Tramway Filled with Sand; Main Terminal Buildings, Oakland International Airport [Photo courtesy of B. A. Vallerga]

Liquefaction does not appear to have damaged the eastern 7,000 ft of the main runway, though clear evidence of soil liquefaction was also observed in this eastern zone. Minor settlement and/or lateral spreading of the perimeter levees occurred at several points in this eastern zone, and sand boils, settlement and lateral spreading caused relatively minor damage to the lightly paved perimeter overrun apron at the edge of the main runway in this area, though the main runway pavement was not damaged. This, along with general similarity of fill materials and placement conditions in the eastern fill zone with those of the liquefied western section, suggest that stronger shaking, or similar levels of shaking with longer duration, might pose considerable danger with respect to liquefaction of the eastern fill zones.

In addition to the liquefaction which occurred in the fill zones at and around the main runway and the adjacent parallel taxiway, additional liquefaction occurred over large areas of the airport fill farther to the north and east. These additional zones of liquefaction included areas between the main runway and the main terminal buildings, and some minor damage to taxiway pavements and aprons resulted. Liquefaction also occurred at the location of the main terminal buildings. Figure 3.71 shows settlement of the fill (and the steps) adjacent to one of the two main terminal buildings. The buildings themselves are supported on deep foundations and did not suffer any significant damage, but settlements of up to 3 inches were observed in the surrounding soils at several locations. In addition, a below ground tramway, which allows service vehicles carrying passengers' luggage to pass under a portion of one of the main terminal buildings, filled to a depth of approximately six feet with exuded sands (and water), as shown in Figure 3.72. Further evidence of liquefaction (boils, settlement and lateral spreading) was also observed to the north of the main terminal buildings near the access road to the main cargo terminal, though no damage to structures or other developed facilities occurred in this area.

3.5.8 East Bayshore South of the Oakland Airport

No evidence of soil liquefaction was observed on the east San Francisco Bay shoreline to the south of the Oakland International Airport, nor along the edges of the sloughs and stream channels inland of the bay shorelines in this region.

3.6 San Jose and the South San Francisco Bay Region:

Very little evidence of soil liquefaction was found in the San Jose area at the southern end of San Francisco Bay. This lack of liquefaction was important to researchers, as there was considerable liquefaction in the alluvial deposits in and around the south bay shoreline and farther south along the sloughs and stream channels during the great San Francisco Earthquake of 1906. It is clear that the magnitude and duration of shaking in this region caused extensive liquefaction in 1906, and virtually no liquefaction in the Loma Prieta event. Many of the soil deposits in this area thus represent potential "banded" field data points for development and/or further refinement of methodologies for evaluation of in situ liquefaction resistance.

Fairly thorough inspections of the south Bay perimeter shoreline and the areas around the edges of all sloughs and stream channels extending inland from the bay shore were performed within the first three days after the earthquake. The only evidence of probable soil liquefaction found in this area occurred at two sites. The first of these was at an electrical power station near the Guadaloupe River approximately one mile northwest of San Jose Municipal Airport. Minor settlement of a tower foundation at this site was suggestive of liquefaction-related ground softening, but this could not be confirmed. No significant damage resulted.

The second site at which evidence of probable liquefaction was observed was on the east bank of the Guadaloupe River, across the river from the southeast corner of San Jose Municipal Airport. Minor lateral spreading and settlement caused minor cracking in the pavement of the airport frontage road in this location. This was quickly and simply repaired, and no damage to airport lands or facilities was observed.

3.7 The Pacific Coast:

3.7.1 Introduction

As shown in Figure 1.1, soil liquefaction occurred at a number of beaches along the Pacific coast south of San Francisco and to the north of Santa Cruz, including beaches at Half Moon Bay, Pomponio State Beach, Gazos Beach, and Big Basin Redwoods State Beach. At all of these sites, sand boils and lateral spreading occurred as a result of liquefaction of dune sands at the edges of impounded lagoons at the mouths of streams, inboard of the surf zones of the beaches. The absence of any evidence of liquefaction on the sections of the beaches nearer the ocean suggest that these sands, densified somewhat by surf activity, were sufficiently dense as to resist liquefaction under the levels and durations of shaking produced at these sites, while the inboard dune sands were not. No damage to structures or other developed facilities occurred at any of these sites.

Farther south along the Pacific coast, in close proximity to the fault rupture region, soil liquefaction occurred over a considerable portion of central Santa Cruz. This liquefaction, as evinced by sand boils, settlement, cracking and buckling of pavements, lateral spreading, etc., occurred in the City of Santa Cruz over an area roughly one kilometer wide and extending at least 1.5 kilometers inland at the mouth of the San Lorenzo River. Considerable structural damage also occurred in this area, including the collapse of a major shopping mall, but it appears that most of the structural damages in this area were the result of strong shaking, with relatively little contribution from foundation displacements due to liquefaction. Additional lateral spreading and sand boils occurred, but caused little damage, immediately to the south of this central section of Santa Cruz at the edge of the Santa Cruz small craft harbor.

South of Santa Cruz, widespread liquefaction (lateral spreading, settlement and sand boils) occurred along the east coast of Monterey Bay at Moss Landing and at the mouths of Watsonville Slough and the Pajaro River. Liquefaction extended well inland (more than six miles) along both the Watsonville Slough and the Pajaro and Salinas Rivers, and resulted in lateral spreading which damaged thousands of feet of levees along these two channels. Sand boils were also observed in cultivated fields in this area, and many of these can be traced to old stream chanel alignments. Minor, non-structural damage occurred at a major power plant sited on the coast at Moss Landing, and considerable damage occurred as a direct result of soil liquefaction at a Marine Research Facility at Moss Landing.

No evidence of significant liquefaction has been reported (to date) in the Monterey area or farther south.

3.7.2 Santa Cruz

A City of Santa Cruz seismic hazard study, published in 1976, delineates a large region of central Santa Cruz, at the mouth of the San Lorenzo River, as being likely to suffer to some extent as a result of liquefaction in a major earthquake (City of Santa Cruz, 1976). This region, essentially encompassing the main alluvial deposits at the river mouth, is approximately 7,000 feet wide and extends more than 8,000 feet inland, as shown in Figure 3.73. This region includes the well-known main beach and Boardwalk areas, as well as nearby light commercial and residential zones. The unshaded region immediately inboard of the Municipal Wharf in Figure 3.73 represents a cemented sandstone outcrop.

The shaded zone delineating potential liquefaction hazard shown in Figure 3.73 was established using SPT-based techniques for evaluation of the in situ liquefaction resistance of sandy soils, as proposed by Seed and Idriss (1971). Figure 3.74 presents a mapping of features clearly associated with soil liquefaction during the Loma Prieta Earthquake of October 17, 1989, including sand boils, pipe breaks, significant lateral spreading, cracking, fissures and significant pavement damage. Additional minor pavement damages, as well as zones of minor ground settlement and/or lateral spreading are not mapped. Also included on this figure is the outline of the shaded region of predicted potential liquefaction hazard from Figure 3.73. As shown in Figure 3.74, clear evidence of liquefaction was observed throughout much of the previously identified zone of potential liquefaction, but little evidence of liquefaction was observed beyond the boundaries of the identified zone of potential liquefaction. (It should be noted that the undeveloped land in the southwest corner of the zone of potential liquefaction, marked with question marks in Figure 3.74, was not investigated prior to the rains which occurred several days after the earthquake. As a result, no evaluation of the occurrence or non-occurrence of liquefaction in this area could be made.) In general, the 1976 hazard studies appear to have correctly defined the areas of principal liquefaction risk, and thus provide a strong endorsement for the use of in situ testing-based methods (e.g. SPT) as a basis for evaluation of liquefaction potential.

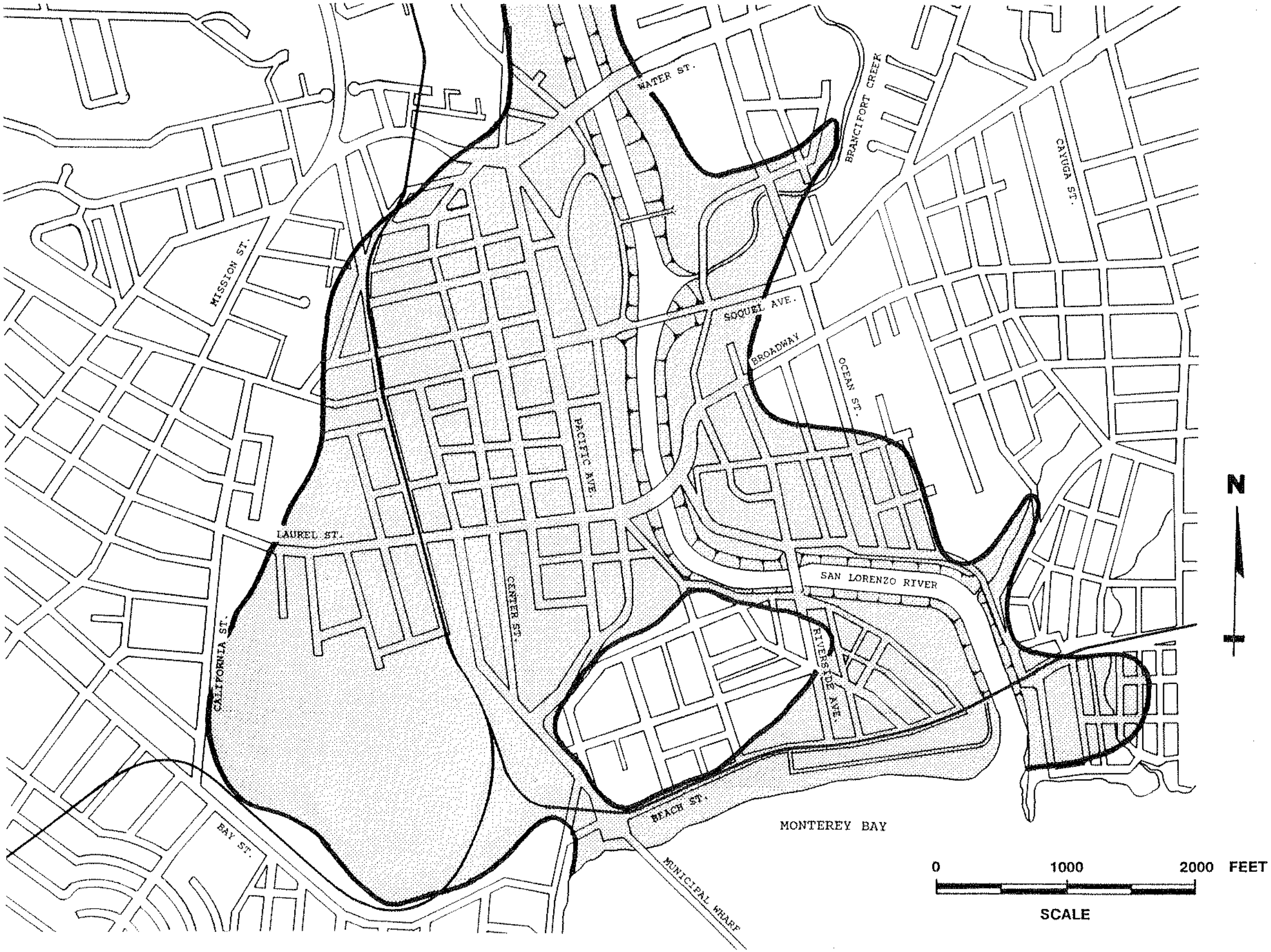


Fig. 3.73: Map of Central Santa Cruz Showing the Zone of Highest Risk of Soil Liquefaction

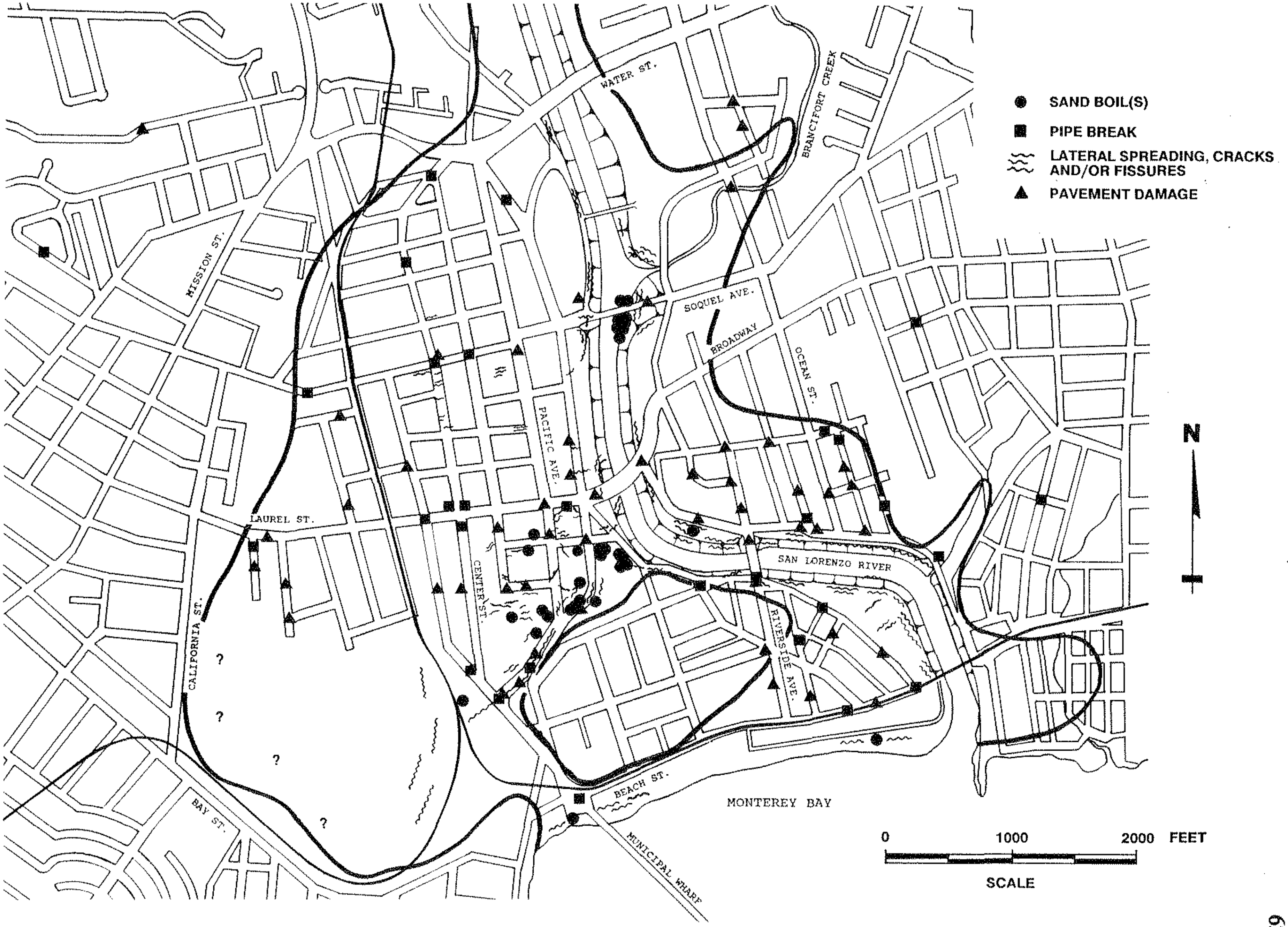


Fig. 3.74: Map of Central Santa Cruz Showing Major Liquefaction-Related Features After the Loma Prieta Earthquake of October 17, 1989

Soil liquefaction and associated settlement and lateral spreading produced boils and fissures at several points along the main beach, but these were fairly quickly erased by rainfall and tidal action. Liquefaction also caused settlement and lateral spreading which resulted in considerable damage to the levees lining the San Lorenzo River throughout much of this area, as shown in Figure 3.74. Figure 3.75 shows a fairly typical example of damage to one of these levee sections. Fissures, with widths of up to 6 inches and depths of more than 5 feet, were relatively common along several thousand feet of levees in this area.

Figure 3.76 shows an example of pavement buckled by compression as a result of lateral spreading near the center of the zone of apparent liquefaction, and Figure 3.77 shows the corner of a building which both settled and moved laterally several inches as a result of ground softening in this area. Although considerable evidence of settlement and minor lateral movements was observed throughout this region of the city, it does not appear that these liquefaction-related ground displacements contributed significantly to most of the serious structural damages sufferred in this region.

Figure 3.78 shows the locations of destroyed and heavily damaged structures, including those of the Pacific Garden Mall. Although these damages are concentrated within the zone of potential (and apparent) liquefaction, it does not appear that liquefaction was a major factor in most of the damages observed. It is too early to make a definitive statement regarding the potential contribution of liquefaction to these damages, but it appears at this time that: (a) most of the severely damaged structures in the Pacific Garden Mall were damaged primarily as a result of their inability to adequately resist the strong shaking to which they were subjected, and (b) most of the damages to residences located southwest of the Pacific Garden Mall were the result of collapse of structurally inadequate cripple walls and/or unbolted foundation connections, so that many of these residences were displaced off of their foundations by strong shaking.

Although the displacements and ground softening associated with liquefaction do not appear to have been a major factor in most of the damage to structures in this region in this short duration event (approximately 8 to 10 seconds of strong shaking), the occurrence of liquefaction over much of this area (as correctly predicted by the 1976 seismic hazard studies) suggests that liquefaction continues to represent a potentially significant hazard in future events of longer duration.

Finally, it should be noted that the 1976 City of Santa Cruz hazard studies also identified two other nearby areas as potentially vulnerable to liquefaction. Both of these are very much smaller than the zone shown in Figures 3.72 and 3.73. The first of these is an area encompassing several city blocks approximately 1,000 feet south along the coast from the southern end of the zone shown in Figure 3.73, at the northern edge of the Santa Cruz small craft harbor. This prediction, too, was well-supported by observed performance as liquefaction produced several small sand boils and lateral spreading caused minor damage to the pavement of the road at the edge

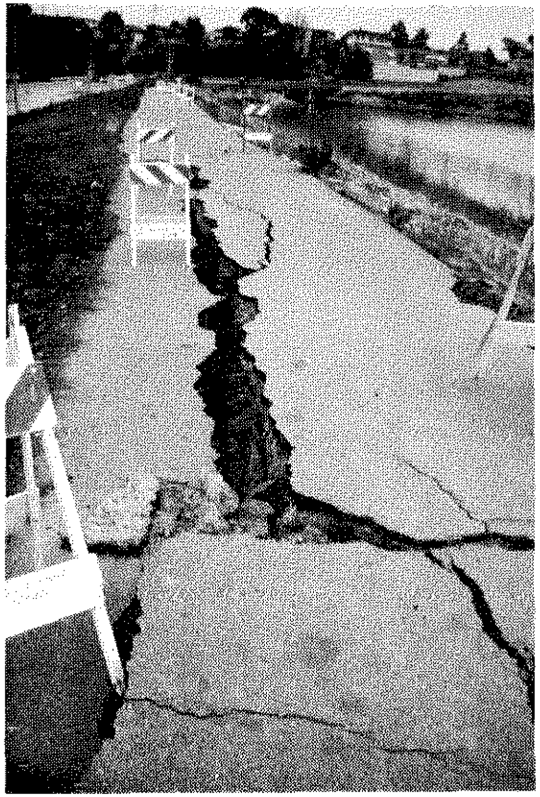


Fig. 3.75: Settlement and Lateral Spreading of Levee Adjacent to the San Lorenzo River; Central Santa Cruz

Fig. 3.76: Compression-Induced Pavement Buckling in Central Santa Cruz

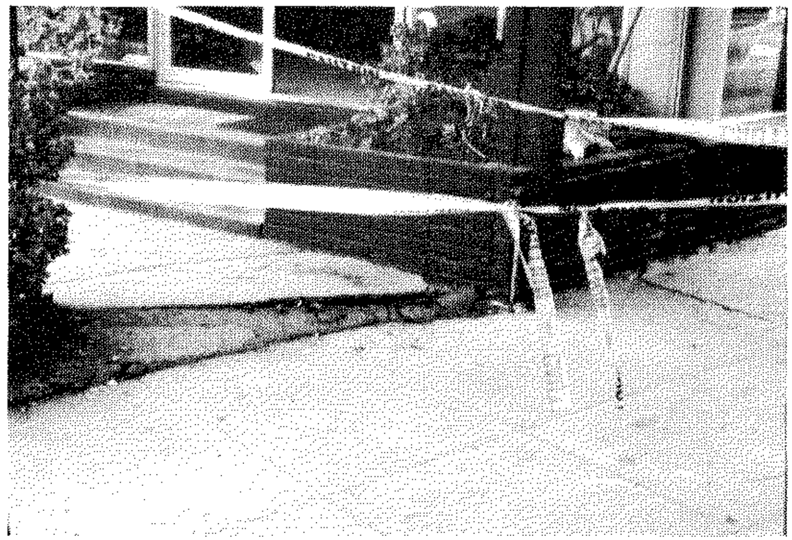


Fig. 3.77: Settlement and Lateral Displacement of the Corner of a Building in Central Santa Cruz

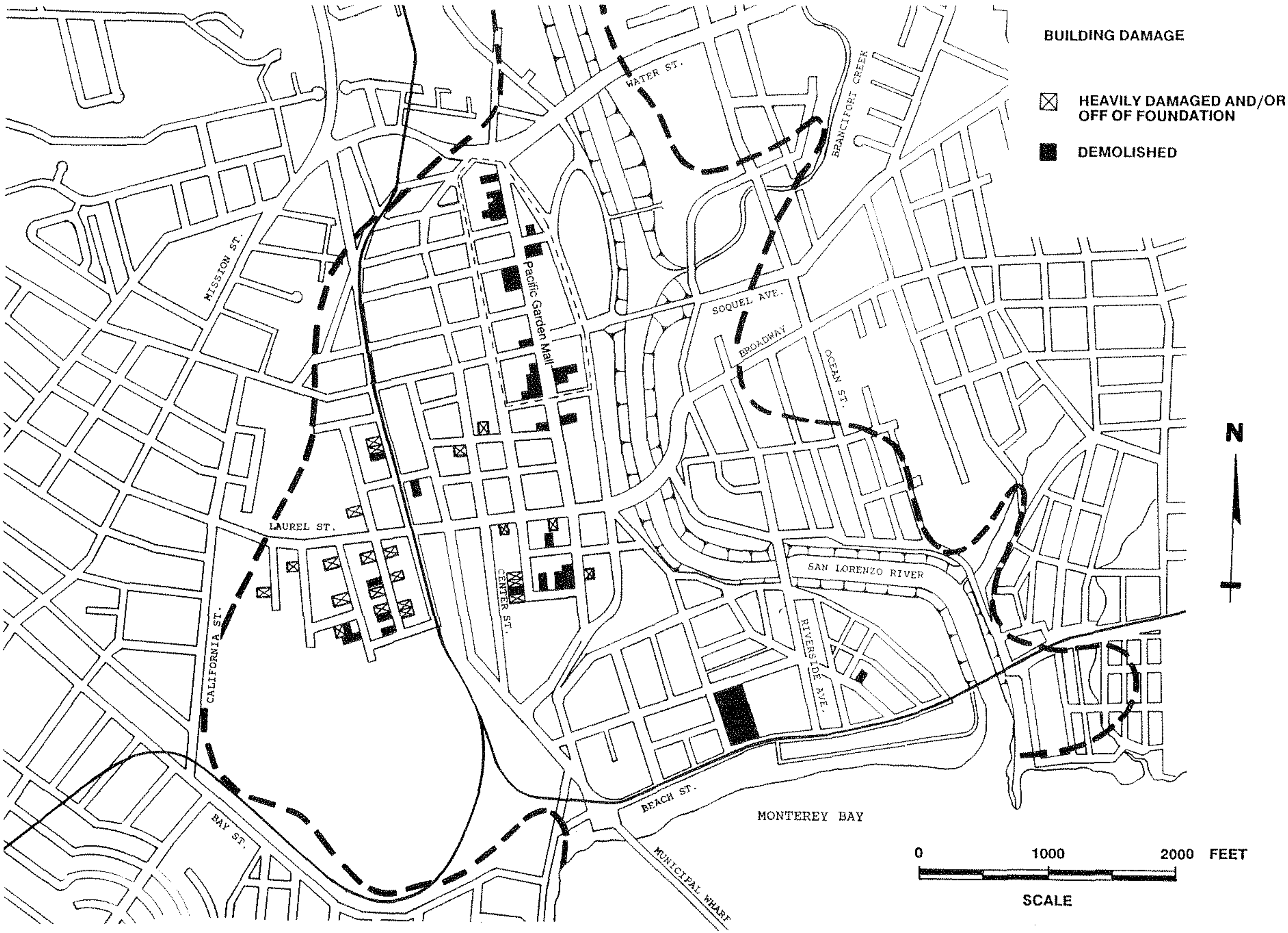


Fig. 3.78: Map of Central Santa Cruz Showing Locations of Demolished and Heavily Damaged Structures

of the harbor. This damage was quickly and simply repaired, and there was no apparent damage to structures in this area.

The other area identified as potentially vulnerable to liquefaction is a small public park on the coast, approximately 11,000 feet north of the zone shown in Figure 3.73. This small park was not inspected prior to the rains which occurred several days after the earthquake. As a result of these rains, and the lack of pavements and structures within the park, no firm conclusions regarding the occurrence or non-occurrence of liquefaction at this park site have been drawn to date.

3.7.3 The East Monterey Bay/Watsonville Region

South of Santa Cruz, widespread liquefaction occurred over large areas along the east shoreline of Monterey Bay, and also at sites extending well inland along several of the stream channels in this region, as shown previously in Figure 1.1. Liquefaction produced spectacular sand boils and lateral spreading fissures at the mouths of the Watsonville Slough and the Salinas and Pajaro Rivers, and extended inland more than six miles along these channels. This produced lateral spreading which damaged thousands of feet of levees lining the two channels, and may also have contributed to the damage which closed two major bridges across these channels. Liquefaction was directly responsible for the destruction of a Marine Research Facility at Moss Landing, and damaged other structures and facilities at this location. Numerous sand boils were observed in cultivated fields immediately east of Monterey Bay, and in most cases these could easily be correlated with known historic stream channel locations.

Figure 3.79 shows one of hundreds of sand boils which occurred on the beach at the mouth of the Pajaro River, and Figure 3.80 shows fissures caused by lateral spreading adjacent to the river at this location. Liquefaction caused only minor damage to a number of structures at this location, including wracking of stairwells and decks, and the rupture of a water main. An eyewitness provides an interesting view of the sequence and timing of events at this site. The eyewitness, terrified by the strong shaking at this site, raced out of her wood-frame residence (clutching her young nephew under one arm) and ran to the "safety" of the open beach. Arriving at the beach some seconds after the shaking had subsided, she was initially relieved. She was then frightened anew when, after some additional brief interval of time elapsed, sand boils began to erupt from the beach and fissures appeared "more or less all at once" over an area of several acres as the beach softened and "spread" towards the ocean. [She ran back into her residence at this point.]

Clear evidence of soil liquefaction (sand boils, settlement and lateral spreading) extended well inland along both the Watsonville Slough and Salinas River channels, and damaged thousands of feet of levees along the sides of these channels. Sand boils were also observed at several locations along Struve Slough. In addition to liquefaction along these current channels, numerous sand boils occurred in open, cultivated agricultural fields inland of the east Monterey Bay coastline. Figure 3.81

Fig. 3.79: Sand Boil and Lateral Spreading on the Beach at the Mouth of the Pajaro River

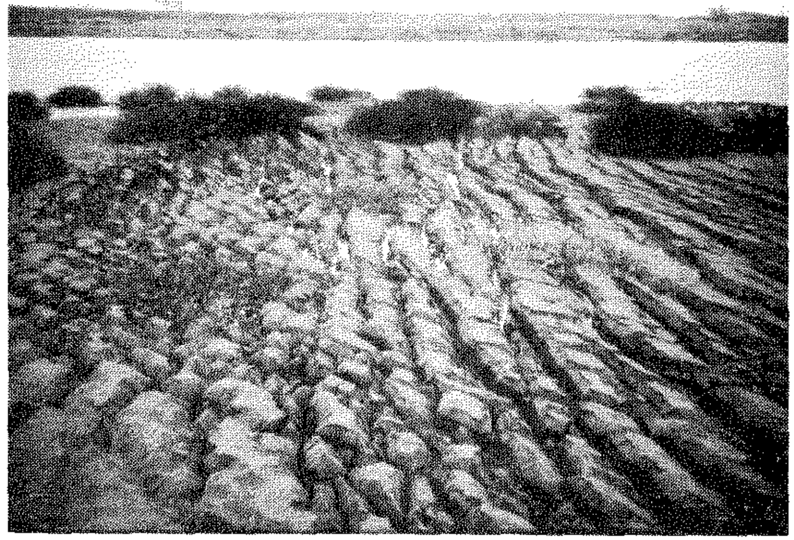


Fig. 3.80: Lateral Spreading at the Mouth of the Pajaro River [Photo courtesy of James R. Martin, II]

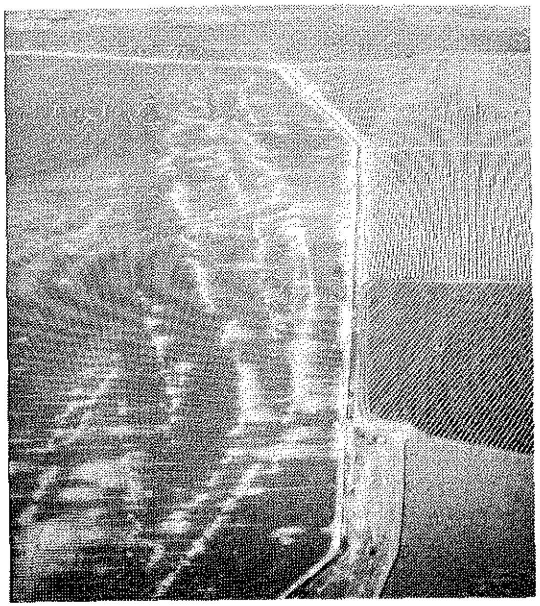


Fig. 3.81: Aerial View of Sand Boils in Cultivated Fields South of Moss Landing, Along the Old Salinas River Channel

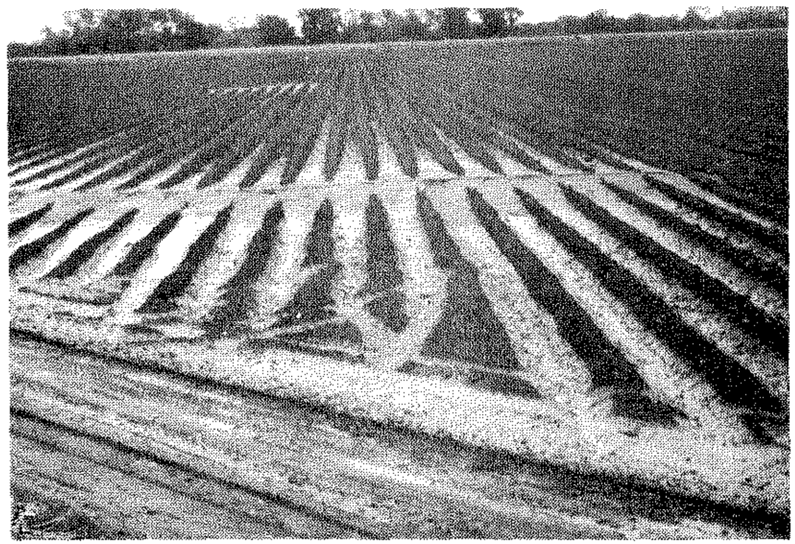


Fig. 3.82: Ground View of Sand Boils in a Cultivated Field East of Moss Landing

shows numerous boils in open fields just south of Moss Landing along what had once been a northern leg of the Salinas River Channel, and Figure 3.82 shows similar boils in an open field farther to the east. These boils tended to form largely linear features across the open fields, and appeared to correlate with the locations of historic stream channel deposits.

It should also be noted that the widespread liquefaction of alluvial channel deposits in the east Monterey Bay region is not without historic precedent. As an example, Figure 3.83 shows damage to a roadway caused by massive lateral spreading and settlement adjacent to the Salinas River, approximately five miles inland from the coast, which occurred during the 1906 San Francisco Earthquake. There is considerable, well-documented evidence of widespread liquefaction of channel deposits throughout this general region during the 1906 earthquake.

Figure 3.84 shows the approach road embankment at Moss Landing State Beach as it appeared immediately after the Loma Prieta Earthquake of October 17, 1989. This embankment is of special interest, as it appears to represent the only case of complete flow failure as a result of soil liquefaction during this event. As shown in Figure 3.84, the center of this embankment slumped as much as 5 feet. This was accompanied by large lateral movements of both the embankment and foundation materials at both sides of the embankment. A corrugated metal culvert which had passed transversely through the center of the embankment (beneath the roadway) separated at a joint near the center of the embankment, and the two halves of the culvert were carried by soils out beyond the original toes of the embankment. Numerous sand boils occurred in close proximity to the failed roadway embankment section.

One of the clearest examples of the structural damage that can result from soil liquefaction was the destruction of the Marine Research Facility at Moss Landing. As shown in Figures 3.85 through 3.87, this facility was a group of low, modern 1 and 2-story structures founded on concrete slabs. The structures were grouped together to provide a series of classrooms and laboratories surrounding a central courtyard. The buildings do not appear to have been significantly damaged by shaking during the Loma Prieta Earthquake. This facility was, however, destroyed beyond repair by foundation displacements (settlement and lateral spreading) as a result of liquefaction of the foundation soils. The inboard roadway adjacent to this structure settled several feet, and lateral spreading deformations of the foundation soils stretched the facility by 6 feet, literally pulling it apart. Figures 3.85 and 3.86 present exterior views of this facility, clearly showing the massive damage caused by these foundation movements. Figure 3.87 presents an interior view, showing a section of the main foundation slab pulled apart by these movements.

The site for the Marine Research Facility is located on a sandy peninsula between the Pacific Ocean and the old trace of the Salinas River. To the immediate south of the building cluster, fissures and a sand boil were found in a volleyball court. Within a few hundred feet of the facility, an approach fill to a timber pile supported bridge across the old Salinas River was found to have slumped approximately 4 to 5

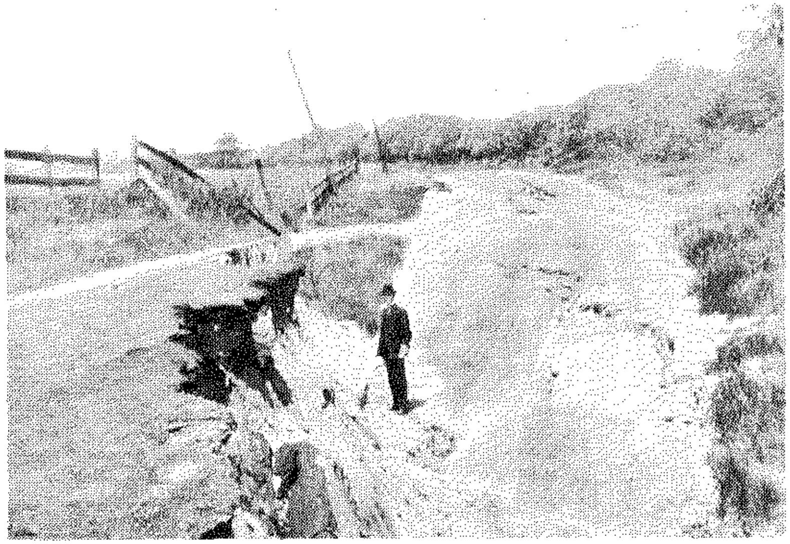


Fig. 3.83: Apparent Massive Lateral Spreading and Settlement at the Edge of the Salinas River, Near Spreckels, After the 1906 San Francisco Earthquake [Lawson et al., 1908]

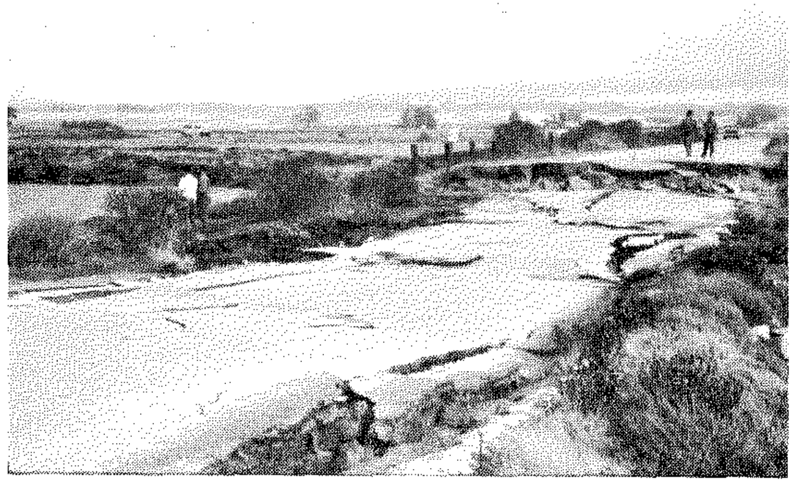


Fig. 3.84: Flow Failure of the Moss Landing Approach Road Embankment

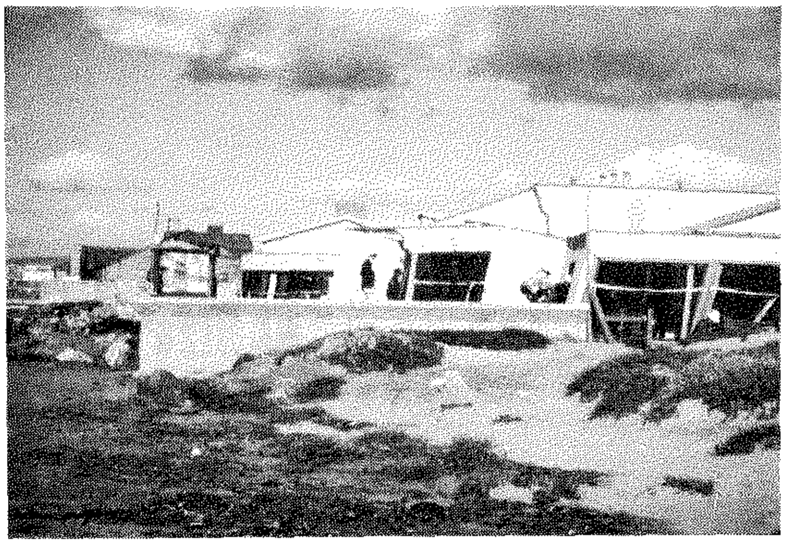


Fig. 3.85: Damage to the Moss Landing Marine Research Facility as a Result of Settlement and Lateral Spreading

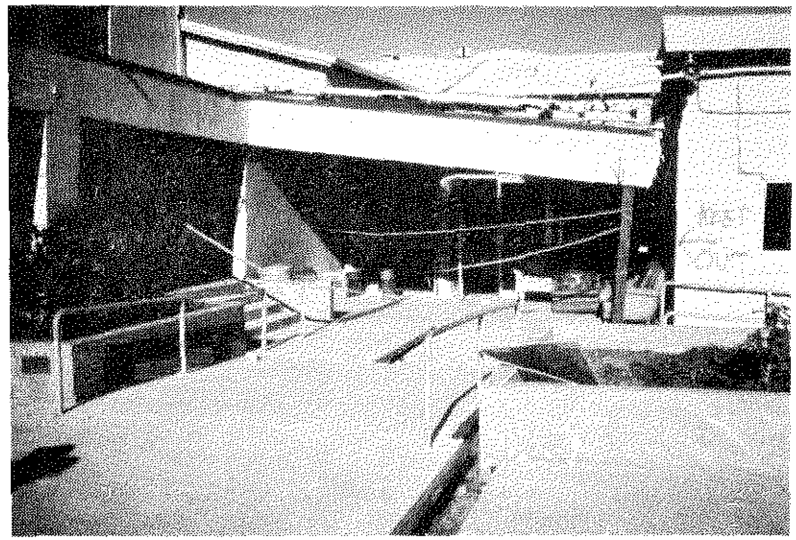


Fig. 3.86: Damage to the Moss Landing Marine Research Facility as a Result of Settlement and Lateral Spreading

Fig. 3.87: Damage to the Moss Landing Marine Research Facility as a Result of Settlement and Lateral Spreading

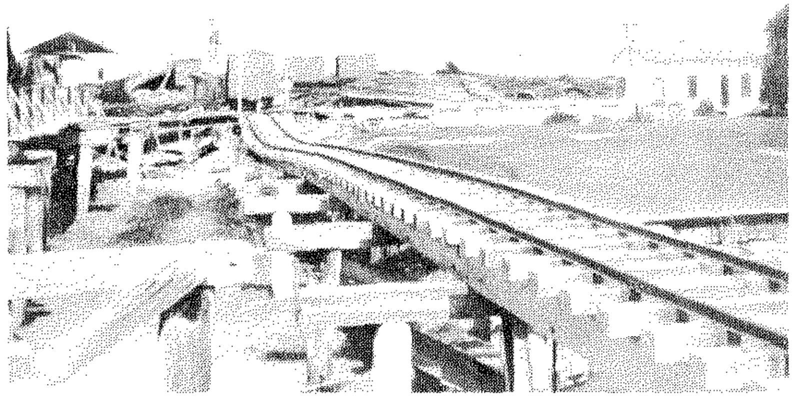


Fig. 3.88: Warehouse at Moss Landing Destroyed by Lateral Spreading in the 1906 San Francisco Earthquake [Lawson et al., 1908]

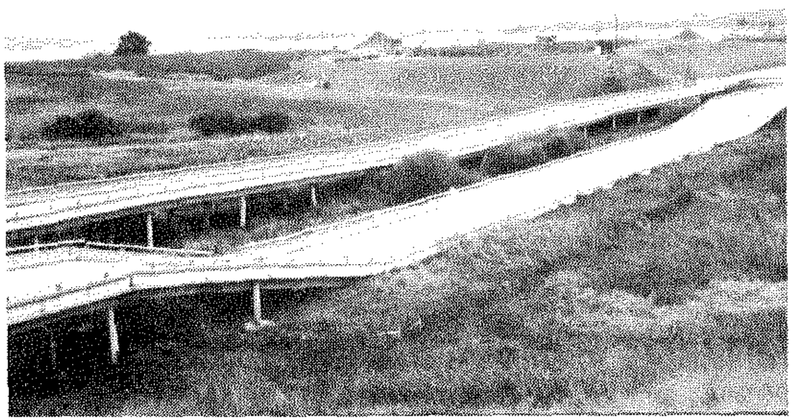


Fig. 3.89: Collapse of the Elevated Highway 1 Crossing over the Struve Slough Near Watsonville

Fig. 3.90: Punching of Piles Through the Collapsed Roadway at the Struve Slough Crossing

feet, severing water and/or sewer pipe lines running across the bridge. Further north along the peninsula, several other structures were damaged due to lateral spreading, although not nearly to the same degree as the Marine Research Facility. The most common damage appeared to be cracking, settlement, and spreading of concrete slabs, although there was also damage to buried utility pipes.

Once again, it is interesting to note that soil liquefaction in this area is not without historic precedent. Figure 3.88 shows several structures at Moss Landing as they appeared shortly after the 1906 San Francisco Earthquake. The pile of rubble at the center of this photograph (between the two white buildings) was a large warehouse which collapsed as a result of 12 feet of lateral spreading at this site, not far from the site of the Marine Research Facility where lateral spreading of on the order of 6 feet occurred during the Loma Prieta Earthquake.

Although widespread liquefaction occurred in the east Monterey Bay region during the Loma Prieta Earthquake of October 17, 1989, damages to structures and other facilities were relatively limited in this sparsely populated region. In addition to the extensive damage done to the Marine Research Facility at Moss Landing, ground softening and minor deformations caused limited damage to several other buildings at and near the main commercial fisheries and the marina at Moss Landing. Several fuel storage tanks (eight feet in diameter and approximately 20 feet high) suffered partial bearing failures and tilted, but these tanks did not rupture.

A large (500 kV) power generation plant owned and operated by PG&E near Moss Landing was also slightly damaged by soil liquefaction. Several water tanks at this facility were damaged, apparently as a result of foundation softening and displacements, and one of these ruptured. Settlements of several inches were noted at a number of locations at and near the plant, and resulting damages (including breaks in utilities) reduced the operating capacity of the plant for a short period immediately after the earthquake, but the main structures and appurtenant facilities at the plant (e.g. the large smokestacks) are all pile-supported, and these do not appear to have been damaged.

In addition to damaging levees, roads, and the relatively few structures described above, soil liquefaction has also been cited by a number of investigators as possibly having contributed to the damage to a pair of parallel major highway bridges carrying Highway 1 across Struve Slough near Watsonville. Figure 3.89 shows the collapsed section of this elevated highway across Struve Slough. The south-bound section collapsed completely, and several of the piles supporting this structure were thrust through the collapsed roadway, as shown in Figure 3.90. The parallel north-bound section was also damaged, but did not collapse. The mode of failure appears to have been excessive lateral deflections of the vertical, reinforced concrete piles supporting the structure. This, in turn, resulted in failures of the connections of the tops of the piles to the bases of the support bents under the combination of flexural and shear forces produced by these movements. Figures 3.91 and 3.92 show examples of typical damage at the pile/bent connections.

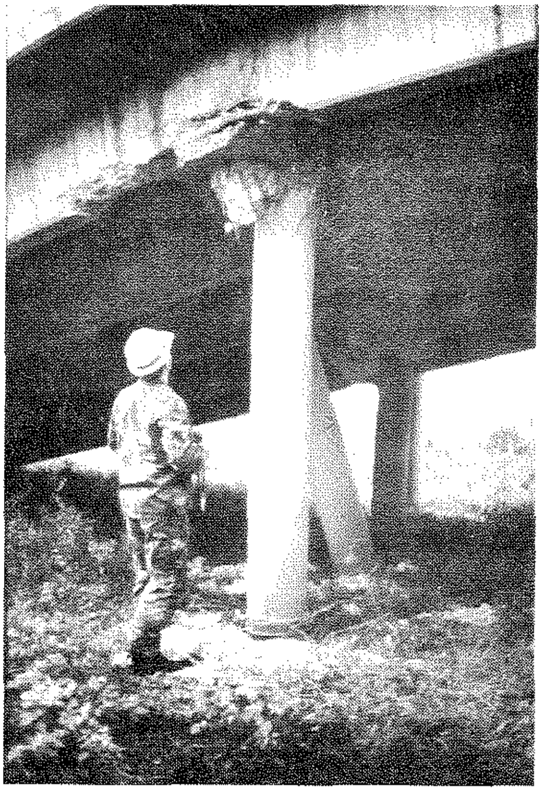


Fig. 3.91: Damage to Pile/Bridge Connection; Struve Slough Crossing

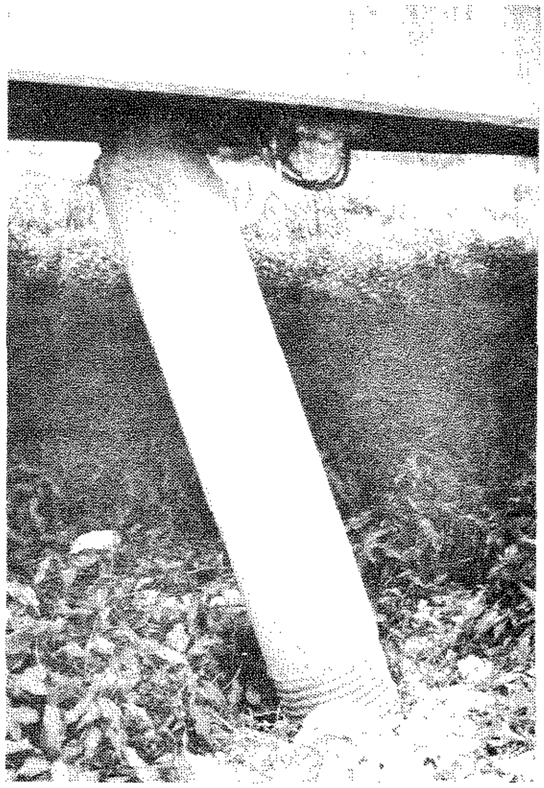


Fig. 3.92: Damage to Pile/Bridge Connection; Struve Slough Crossing

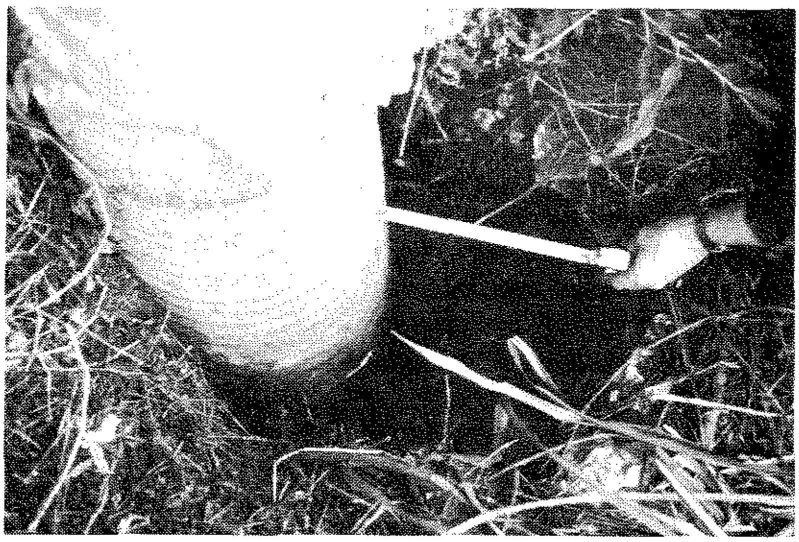


Fig. 3.93: Typical Gap Adjacent to One of the Piles Supporting the Collapsed Struve Slough Crossing

The excessive lateral pile deflections may have been the result of inadequate lateral support provided by the soils surrounding the upper portions of the piles. All of the piles beneath the collapsed section showed "gaps" between the piles and the surrounding soil indicating that significant movements had occurred. A typical example of one such gap is shown in Figure 3.93. The poor lateral support of the foundation soils appears to have been due primarily to the soft, weak saturated clays and organics which comprise much of the upper foundation. Alluvial sands are also present in these foundation soils, and sand boils were noted at several locations indicating that some liquefaction occurred at this site. These boils were not extensive, however, and it does not appear that ground softening as a result of liquefaction was a major factor in the lack of lateral support provided by the upper foundation soils. In addition, most of the piles showed no signs of vertical settlement, so that partial bearing failures as a result of liquefaction do not appear to have been a significant factor in this failure.

3.8 Summary:

Soil liquefaction occurred over a widespread area as a result of the Loma Prieta Earthquake. Limited liquefaction occurred, but caused no damage, at sites as far as 70 miles north of the epicenter in Marin and Martinez, and extended south as far as the southern end of the east Monterey Bay region.

Liquefaction caused considerable damage to structures and facilities in the central San Francisco Bay Area throughout bayshore regions of San Francisco, Oakland, Alameda and Emeryville, as well as at Treasure Island. These damages were, however, relatively "moderate" as a result of (a) the attenuation of strong shaking from the zone of fault rupture located more than 40 miles to the south, and (b) the unusually short duration (8 to 10 seconds) of the strong shaking produced by this event with M_S = 7.1. Indeed, the fact that this relatively moderate level and short duration of shaking did induce some degree of liquefaction over large areas, and the previous (and well-documented) performance histories of many of these areas (which suffered considerably as a result of soil liquefaction in the 1906 San Francisco Earthquake), serves as a stark warning of the increased levels of damage likely to occur at these sites in the event of occurrence of a larger and/or more near-field future earthquakes producing stronger levels of shaking of longer duration.

Farther to the south, and nearer to the epicentral region, widespread soil liquefaction also occurred in alluvial deposits throughout the Santa Cruz and east Monterey Bay regions. Levels of strong shaking were higher in these areas, but it appears likely that liquefaction-induced damages were again reduced by the unusually short duration of strong shaking produced by this earthquake. Damage to structures and facilities attributable to soil liquefaction was also minimized by the relatively sparse population and development of this region.

In addition to serving as a powerful warning of the hazard associated with potential soil liquefaction in future earthquakes, the behavior of potentially liquefiable sites provides an invaluable opportunity for researchers to develop, refine and/or verify methodologies for evaluation and mitigation of liquefaction potential and its adverse consequences. In general, liquefaction behavior during the Loma Prieta Earthquake can be categorized by dividing the region of northern California affected by the Loma Prieta Earthquake into four zones as follow:

(1) The Central San Francisco Bay Area: Widespread liquefaction occurred in bayshore fills in San Francisco, Oakland, Emeryville and Alameda, as well as at Treasure Island. Site conditions were generally similar at most of the affected sites, and typically consisted of 10 to 30 feet of loose, sandy fill underlain by deep cohesive deposits. At most sites, the upper cohesive deposits were soft to medium stiff estuarine clays known locally as San Francisco Bay Mud. The deeper deposits were typically stiffer, overconsolidated clays and sandy clays, with sandy and gravelly layers. Bedrock depths extend from relatively shallow depths well inshore to depths of several hundred feet at the outboard edges of many of these fill zones. Peak horizontal bedrock accelerations were on the order of 0.06 g to 0.12 g throughout this region. These levels of shaking were amplified by factors of 2 to 3 by the deep, cohesive foundation soils, producing peak surface accelerations on the order of 0.16 g to 0.33 g at the bayshore sites in question. These amplified levels of shaking were sufficient to induce liquefaction within the surficial loose sand and silty sand fills in many of these areas, and especially within many dredged hydraulic fills. The extent of this liquefaction, however, and the resulting adverse consequences appear to have been limited by the relatively short duration of strong shaking.

(2) The South San Francisco Bay Shoreline and San Jose: Very little evidence of soil liquefaction, and no serious resulting damages, were apparent in the southern Bay Area. This is interesting, in part, because fairly widespread liquefaction of alluvial channel deposits occurred in this region during the 1906 San Francisco Earthquake, which would have produced somewhat stronger levels of shaking of considerably longer duration in this region than those produced by the Loma Prieta Earthquake. It is also of interest because this region is closer to the epicentral region than Zone 1 described above, which did experience considerable liquefaction in the Loma Prieta event. It thus appears that: (a) levels of shaking during the Loma Prieta event which were sufficient to induce liquefaction in uncompacted hydraulic fills in the central Bay Area, were generally not sufficient to produce liquefaction in the natural alluvial soils of the southern Bay region, and (b) the somewhat stronger and much longer-duration levels of shaking produced by the 1906 San Francisco Earthquake had been sufficient to cause considerable liquefaction in these alluvial deposits. This relatively well-defined pattern of behavior offers some significant opportunities for geotechnical earthquake engineering researchers.

(3) The Santa Cruz/East Monterey Bay Region: The strong levels of shaking in this region produced widespread liquefaction within natural alluvial and coastal beach and dune deposits. Fortunately, damages associated with soil liquefaction were somewhat limited by the relatively sparse population and development in this region.

(4) Other Areas: In addition to the three main "zones" described above, soil liquefaction occurred at a number of sites along the Pacific Coast north of Santa Cruz, and at Martinez to the east of Suisun Bay. Liquefaction in these areas caused no serious damage to structures or facilities.

The relatively well-defined patterns of behavior with respect to soil liquefaction during the Loma Prieta Earthquake thus appear to consist of: (a) widespread liquefaction of relatively "moderate" severity in loose sandy fills (especially hydraulic fills) underlain by deep cohesive deposits in the central San Francisco Bay Area, (b) non-liquefaction (for the most part) of alluvial deposits in the south San Francisco Bay Area, despite the fact that many of these deposits had liquefied in 1906, and (c) widespread liquefaction of alluvial deposits nearer to the epicentral region in the Santa Cruz and east Monterey Bay areas.

In addition to noting these relatively well-defined patterns of behavior, a number of important additional observations can be made at this time, as follow:

1. Much (if not virtually all) of the liquefaction in the central San Francisco Bay Area, as well as at Santa Cruz and the east Monterey Bay area, had been correctly predicted as likely to occur during moderate to severe earthquake shaking. Moreover, many sites at which liquefaction occurred during the Loma Prieta Earthquake had been documented by researchers as sites at which liquefaction had occurred during the 1906 San Francisco Earthquake. Examples include the liquefaction observed along the Embarcadero shoreline and in the south of Market areas of San Francisco, at Moss Landing, and along the Pajaro and Salinas Rivers. [It should also be noted, however, that many of the hydraulic fills which liquefied during the Loma Prieta Earthquake post-date the 1906 earthquake]. In retrospect, there appear to be few surprises in terms of sites at which liquefaction occurred.
2. The relatively moderate levels and short duration of shaking, generated in the San Francisco Bay Area by the M_S = 7.1 Loma Prieta Earthquake which was centered well to the south near Santa Cruz, represent a poor "test" of the ability of the Bay Area to withstand the stronger levels and longer durations of shaking likely to be produced by larger and/or more near-field future seismic events. Accordingly, the widespread occurrence of slight to "moderate" liquefaction over large shoreline areas in this region, as well as the previous poor performance of many of these areas in the 1906 earthquake, serves as a stark warning of the ongoing hazard exposure associated with potential liquefaction in future events. Large, densely populated areas, as well as important harbor facilities and airports likely to be in demand for emergency transport after a major earthquake, appear to face considerable liquefaction hazard exposure.
3. Although a majority of the liquefaction-induced damage during the Loma Prieta Earthquake occurred in bayshore fills, this does not mean that

"fills" represent an intrinsically hazardous condition. Although liquefaction was fairly widespread in loose, uncompacted sandy bayshore fills, in many areas similar fills which had been compacted prior to the earthquake performed well and experienced no liquefaction. Sites where densified hydraulic fills performed well, while adjacent undensified fills liquefied, include Foster City and parts of Treasure Island, Emeryville, Alameda and Bay Farm Island.

In summary, if there is a single overall lesson to be learned from the occurrence of soil liquefaction during the Loma Prieta Earthquake, it is: (a) that considerable liquefaction-related risk to the population and infrastructure of the San Francisco Bay Area continues to exist, (b) this risk can be quanitified, and the liquefaction hazard at any given site can be correctly and reliably evaluated, and (c) once potentially liquefiable sites have been identified, the associated hazard can either be avoided or mitigated, though at some cost. It must be hoped that the lessons learned from the Loma Prieta Earthquake will spur local policy makers to undertake the difficult actions necessary to begin to remedy the considerable risk to the population and infrastructure of the Bay Area associated with current conditions at many of the sites discussed in this chapter. Preliminary indications are hopeful in this regard at many of these sites, but much more remains to be done.

Chapter Four: EFFECTS OF LOCAL SITE CONDITIONS ON GROUND MOTIONS

4.1 Introduction:

Geotechnical factors exerted a major influence on the nature and severity of ground shaking during the Loma Prieta Earthquake. Indeed, the major influence of geologic conditions or "local site effects" on both strong shaking characteristics and resulting damage patterns was one of the most striking features of this event.

A majority of the damage to structures and other facilities, and more than 50 of the 62 deaths attributed to the earthquake, occurred at sites underlain by soil deposits which served to amplify shaking intensities at these sites. This included the sites of the collapsed San Francisco/Oakland Bay Bridge section, the collapsed Cypress/Interstate 880 elevated highway viaduct, and the heavily damaged Pacific Garden Shopping Mall in Santa Cruz, as well as other heavily damaged regions in San Francisco, Oakland, Alameda, Watsonville and elsewhere.

This concentration of damage on a few relatively distinct sites comprising less than one percent of the "strongly" shaken region was due primarily to the local soil conditions at these sites. These concentrated damages occurred at sites underlain by deep, and in most cases primarily cohesive, soil deposits which served to amplify the relatively moderate levels of "bedrock" shaking generated at these sites by the earthquake, producing significantly stronger levels of surface shaking. This amplification was especially pronounced at sites underlain by soft to medium stiff marine estuarine clays and silty clays. Peak accelerations on rock in the central San Francisco Bay region (the San Francisco, Treasure Island, Oakland, Alameda and Emeryville region) appear to have been on the order of 0.06 g to 0.12 g. Instrumental recordings, as well as dynamic response analyses, show that many of the bayshore soil deposits in this region amplified these levels of shaking by factors of about 2 to 3, producing peak ground surface accelerations at deep alluvial bayshore sites on the order of 0.16 g to 0.33 g in this region. In addition, amplification of the longer period components of shaking was especially pronounced, so that the resulting surface motions were particularly damaging to taller, longer period structures.

This type of pronounced, site specific amplification (and spectral amplification, or resonant soil-structure interaction) of ground motions was not a surprise to the earthquake engineering community. Similar site-specific amplification has been noted as an important factor strongly influencing damage patterns in numerous previous major earthquakes over the past 30 years, and these effects had been widely predicted for many of the San Francisco Bayshore sites which suffered particularly heavily during the Loma Prieta Earthquake (e.g.: Idriss & Seed, 1968; Borcherdt, 1970; Borcherdt et al., 1975; Seed & Sun, 1989; etc.).

This chapter presents a preliminary overview of the influence of geologic factors, or "local site effects", on strong ground shaking characteristics during the Loma Prieta Earthquake. This includes an overview of regional geology and the regional distributions of ground motions of varying characteristics, as well as a discussion of individual sites and damage patterns.

4.2 Regional Geology and Observed Ground Motion Characteristics

San Francisco Bay is a northwest trending depression, bounded on the west side by the San Andreas Fault and on the east side by the Hayward and Calaveras Fault systems. The bay basin is largely infilled with alluvial deposits, as well as some aeolian sands. This alluvium is very deep in many areas, with depths to bedrock of as much as 600 feet in some areas along the eastern margins of the bay. Most of the deeper alluvial deposits are primarily clays and silty and sandy clays, though layers and lenses of sandy and gravelly soils are not uncommon. These deeper alluvial deposits are of Pleistocene age, and are very stiff, strong materials as a result of overconsolidation due to both ageing effects and sustained periods of consolidation due to global sea level lowering during glacial ice ages. The upper unit of the bay sediment sequence is a more recent unit which post-dates the last glacial sea level drawdown, and which is continuing to be deposited. This material, known locally as Bay Mud, is a dark gray, marine estuarine clay or silty clay. Extending to maximum depths of up to approximately 100 feet in some areas, this material is a normally consolidated, soft to medium stiff, and highly compressible soil.

For the purposes of obtaining an overview of seismic site response characteristics, the major geologic units of the greater San Francisco Bay Area can be categorized broadly as (1) bedrock and stiff, shallow soils, (2) alluvium, and (3) sites surrounding the edges of San Francisco Bay underlain by deposits of Bay Mud. Figure 4.1 shows these major geologic features of the Bay Region, and divides site conditions into these three broad categories (after Borcherdt, et al., 1975).

Superimposed on this generalized geologic map are the peak horizontal ground surface accelerations recorded at selected sites during the Loma Prieta Earthquake by instruments operated by the U.S. Geological Survey (USGS) and by the California Division of Mines and Geology Strong Motion Instrumentation Program (CSMIP), (Maley, et al., 1989; Shakal, et al., 1989).

It is immediately clear from this figure that high peak horizontal ground surface accelerations, on the order of 0.45 g to 0.64 g, occurred at and near the apparent zone of fault rupture. An interesting feature of this event was the occurrence of unusually high peak vertical accelerations on the (western) upthrown block in close proximity to the rupture zone. As shown in Figure 4.1, two stations in this region recorded large peak vertical accelerations of 0.60 g and 0.66 g. These were larger than the peak horizontal accelerations of 0.54 g and 0.39 g, respectively, recorded at these stations. These unusually high vertical accelerations do not appear to have propagated to great distances from the fault rupture region, as peak vertical

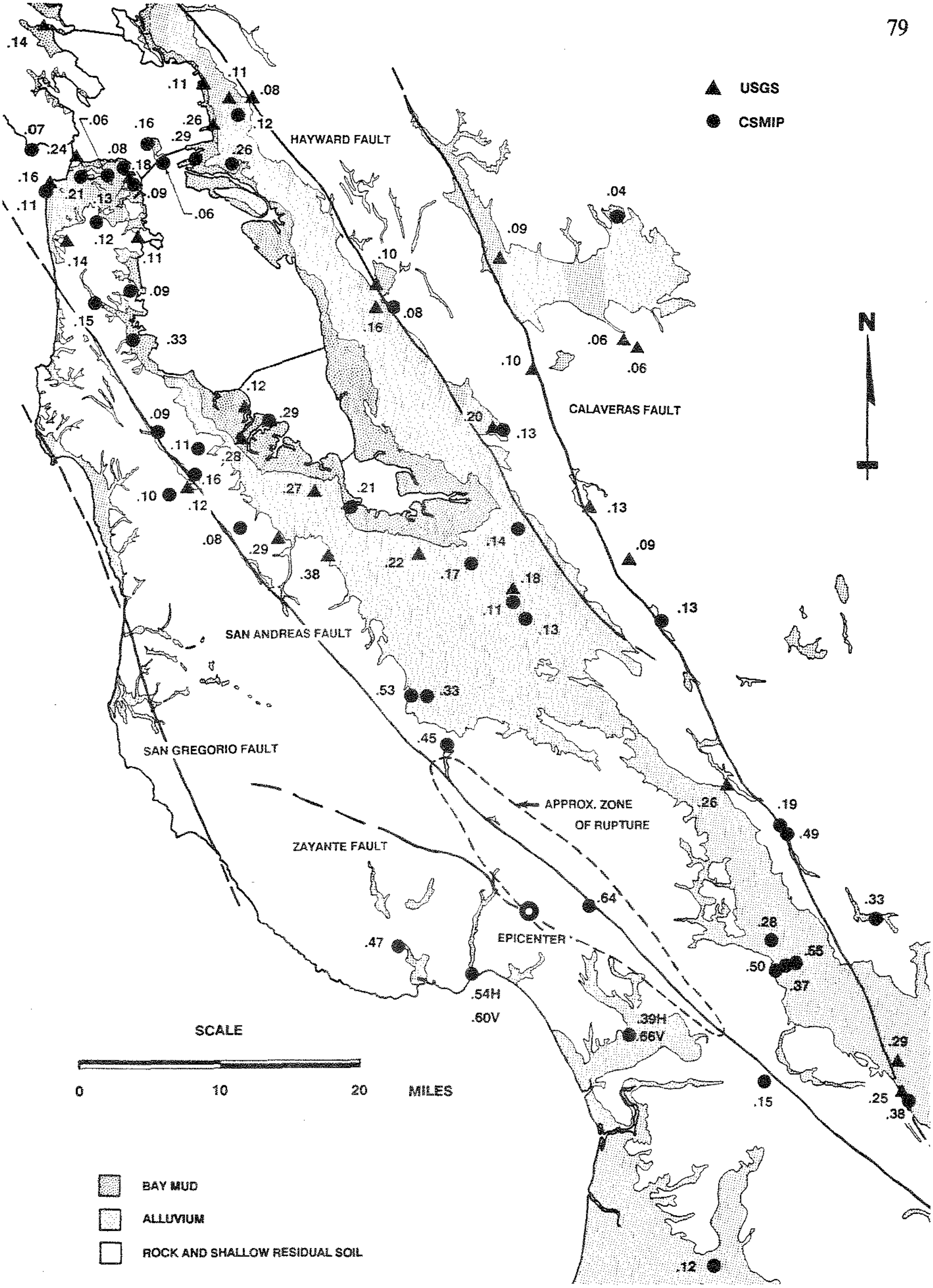


Fig. 4.1: Overview of Regional Geology and Recorded Peak Horizontal Ground Surface Accelerations During the Loma Prieta Earthquake of October 17, 1989

accelerations at more distant recording stations were generally significantly lower than horizontal peak accelerations.

Figure 4.2 presents a plot of recorded peak ground surface accelerations as a function of distance from the fault rupture surface. It must be emphasized that in this and all other, similar figures presented herein, the unit of measure is the distance from the site to the nearest point on the actual (apparent) fault rupture surface. This is not the same as "epicentral distance", and provides a significantly better measure of the effects of attenuation than can be achieved using epicentral distance. In this figure, subsurface conditions at the recording stations are subdivided into four general categories. The first two, "Rock" and "Stiff soils" correspond to the rock and shallow, stiff soils of Figure 4.1. The "Deep soils" are generally "alluvium", and the "Soft soils" are bayshore sites indicated as "Bay Mud" sites in Figure 4.1.

As shown in Figure 4.2, there is a general decrease in peak accelerations with distance from the rupture zone, but this is remarkably less pronounced for "soft soil" sites than for all other site conditions. In addition, it has been widely noted that peak ground surface accelerations at "soft soil" sites in the central San Francisco Bay Area, at distances of 30 to 70 miles (50 to 100 km) north of the fault rupture, greatly exceed those predicted by most conventional ground motion attenuation relationships. This, in turn, has led to some speculation regarding potential "northward focusing" of seismic energy from the Loma Prieta rupture.

This, however, appears unlikely. Instead, these "relatively" high levels of acceleration on soft soil sites in the central Bay Area appear to be the result of localized amplification of relatively moderate bedrock accelerations during propagation towards the ground surface through the overlying cohesive soils. Moreover, the general distribution of magnitudes of accelerations on rock sites, and on soil sites other than "soft soils" (or Bay Mud), appear to be fairly typical of those produced by western U.S. fault ruptures of this size, and so show no pronounced "focusing" effects.

This is illustrated in Figures 4.3 through 4.5. Figure 4.3 shows peak horizontal accelerations recorded on "rock" and "stiff soil" sites only (and for $M_S = 7.1$ events), plotted vs. a mean and mean ± two standard deviation attenuation relationship for "rock" sites proposed by Seed and Idriss (1982). Figure 4.4 shows this same data plotted vs. a similar attenuation relationship (also for rock sites, and for $M_S = 7.1$) proposed by Idriss (1985). As shown in these figures, peak accelerations on rock and stiff soil sites conform well to both of these attenuation relationships: the mean predicted values follow the central trends of the recorded data, and most of the recorded data falls within the ± 2 Std. Dev. bands. Accordingly, it must be concluded that the attenuation of peak ground surface accelerations recorded at "rock" and "stiff soil" sites during the Loma Prieta Earthquake conforms well with that predicted based on previous western U.S. earthquakes, and that no sign of pronounced "northward focusing" is exhibited by the observed data.

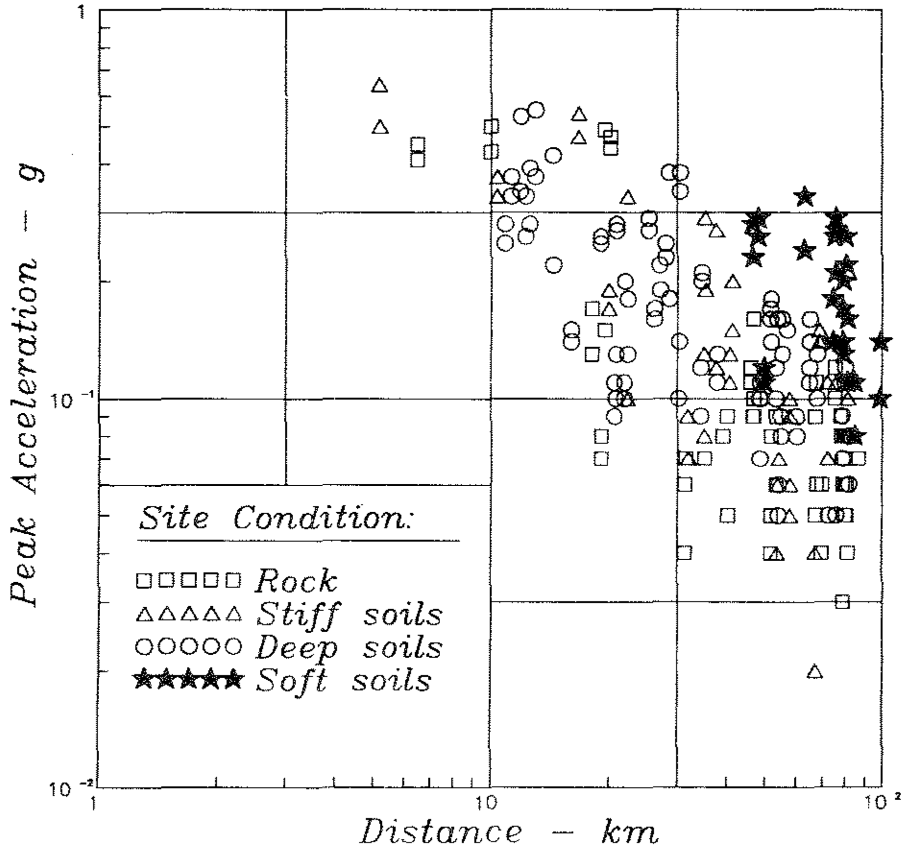


Fig. 4.2: Recorded Peak Horizontal Ground Surface Accelerations vs. Distance from the Fault Rupture Surface

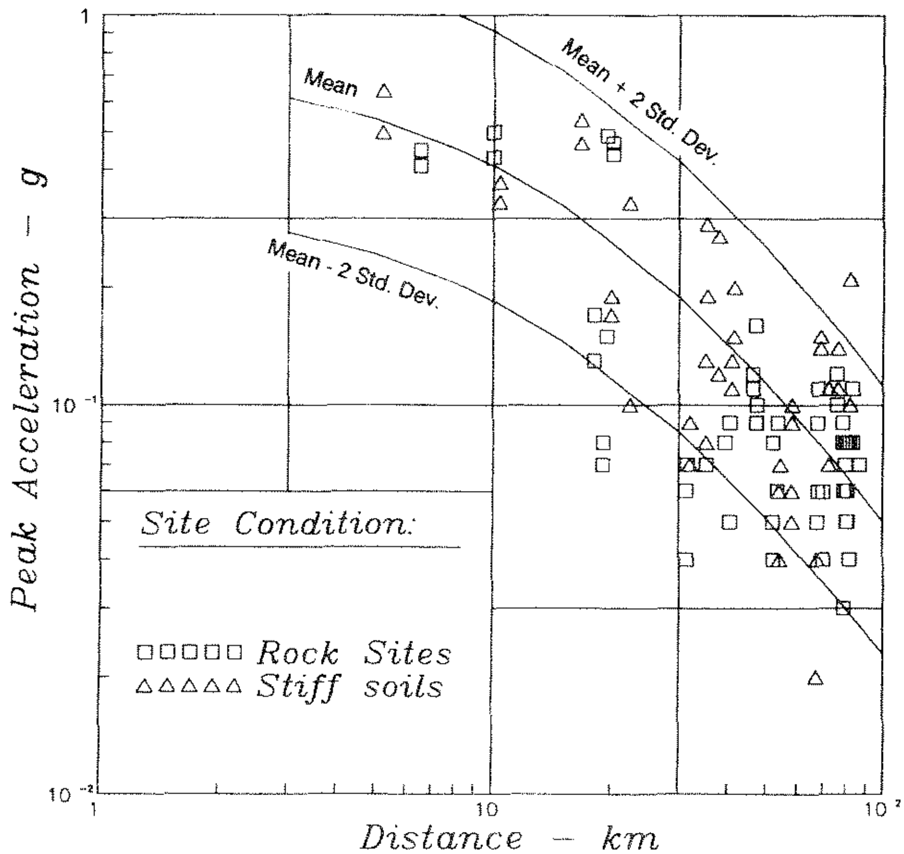


Fig. 4.3: Recorded Peak Horizontal Ground Surface Accelerations on Rock and Stiff Soil Sites, and Attenuation Relationship Proposed by Seed and Idriss (1982)

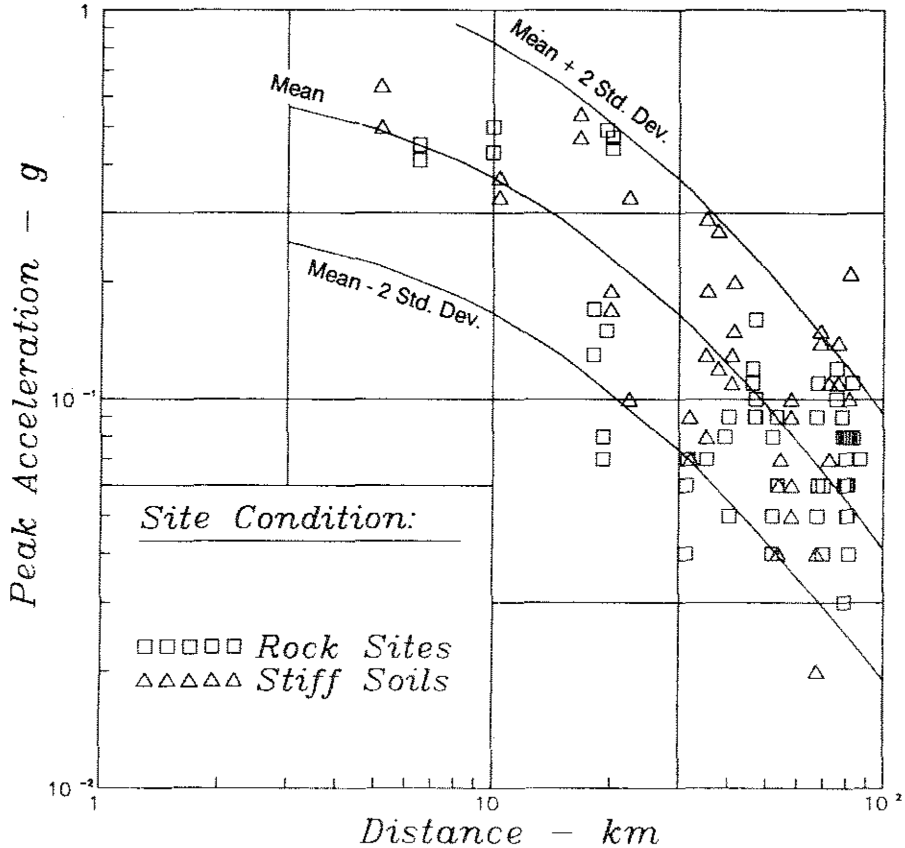


Fig. 4.4: Recorded Peak Horizontal Ground Surface Accelerations on Rock and Stiff Soil Sites, and Attenuation Relationship Proposed by Idriss (1985)

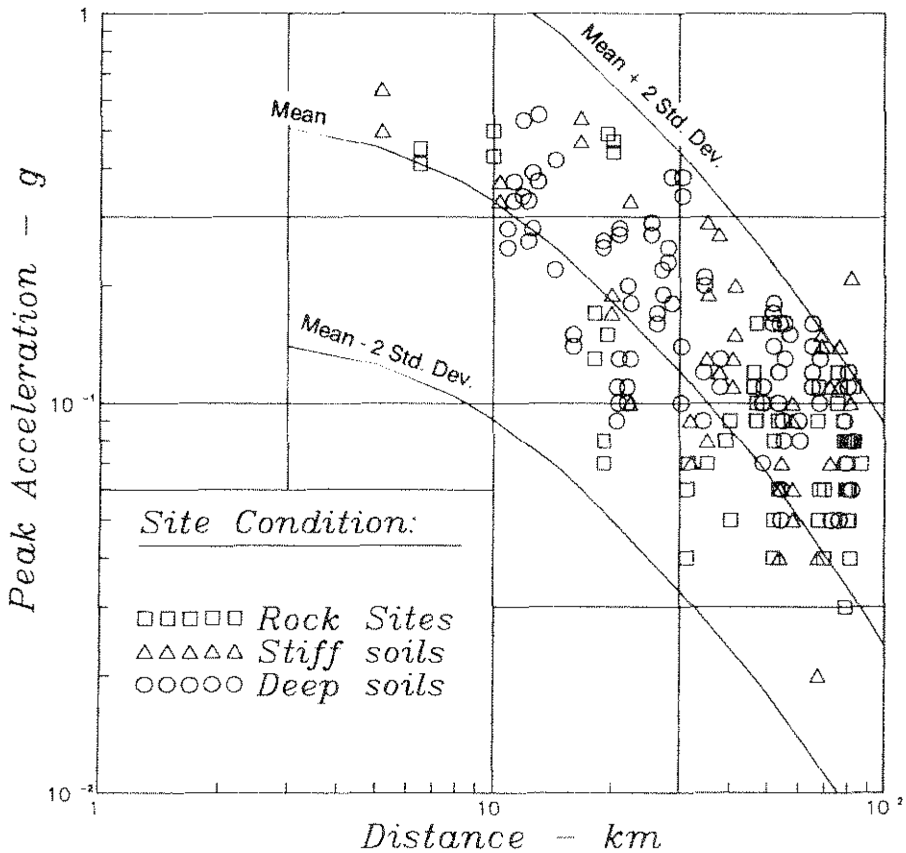


Fig. 4.5: Recorded Peak Horizontal Ground Surface Acclerations for All Site Conditions Except "Soft Soils", and Attenuation Relationship Proposed by Joyner and Boore (1988)

Figure 4.5 presents a plot of recorded peak horizontal ground surface accelerations vs. distance for "rock", "stiff soil" and "deep soil" sites (alluvium). Superimposed on this is a mean and mean ± two std. deviations attenuation relationship proposed by Joyner and Boore (1988). It can be quickly seen that virtually all of the data falls within the ± 2 std. dev. bands, but that the mean of the recorded data falls slightly above the predicted mean. On closer inspection, it can be observed that the "deep soil" sites show an especially pronounced tendency to exceed the predicted values. This is probably due in part to site amplification effects at deep alluvial sites. This effect is not very pronounced, however, and it may be concluded that the attenuation relationship selected does a reasonably good job of fitting the observed data.

The data plotted in Figure 4.5 represents data for all site conditions except "soft soils" or Bay Mud. Figures 4.6 and 4.7 present plots of recorded peak ground surface acceleration vs. distance for these "soft soil" or Bay Mud sites. Superimposed on these are the attenuation relationships proposed by Joyner and Boore (1988) and by Idriss (1985). As shown in these figures, the peak ground surface accelerations recorded at these "soft" bayshore sites greatly exceeded the values predicted by these types of attenuation relationships.

This exceedance of the "predicted" peak accelerations was the result of site-specific amplification of the relatively modest levels of bedrock acceleration as they propagated up through the overlying cohesive soils to the surfaces of these "soft" bayshore sites, and this was a major factor in the heavy concentration of damages and loss of life that occurred on "soft" bayshore sites in the central San Francisco Bay region.

It was, however, only one of two important factors which led to the observed concentration of damage at these sites. The deep cohesive soils at these sites also tended to cause especially pronounced amplification of the long period (or low frequency) components of the bedrock shaking, so that the resulting amplified ground surface motions had a larger concentration of shaking energy in longer period ranges. These long period motions were especially damaging to long period, major structures, which were largely resonant with these long period motions, and also to structures which rapidly softened with the onset of damage and so responded strongly at longer periods.

This double effect; amplification of peak ground surface accelerations, and a shift in concentration of energy to longer period ranges (or especially-pronounced amplification of long-period motions), led to a clear concentration of damages on "soft" bayshore sites on both sides of the central San Francisco Bay region, affecting bayshore regions in San Francisco, Richmond, Emeryville, Oakland and Alameda, as well as at Treasure Island and south of San Francisco along the west bayshore.

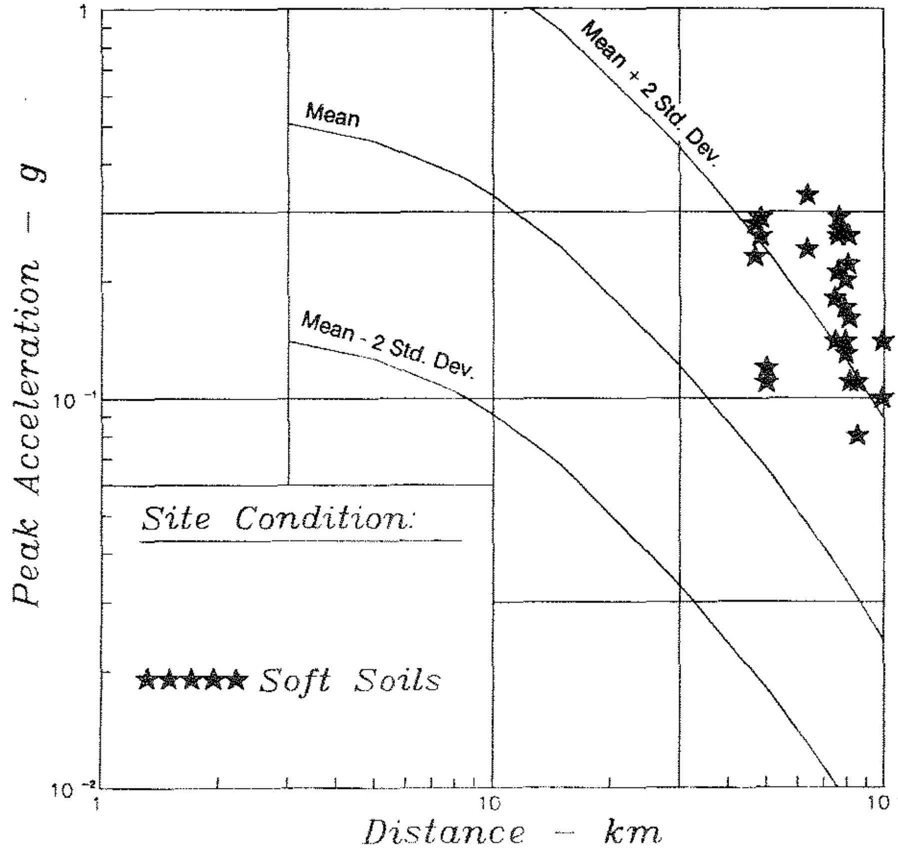


Fig. 4.6: Recorded Peak Horizontal Ground Surface Acclerations on Soft Bayshore Sites, and Attenuation Relationship Proposed by Joyner and Boore (1988)

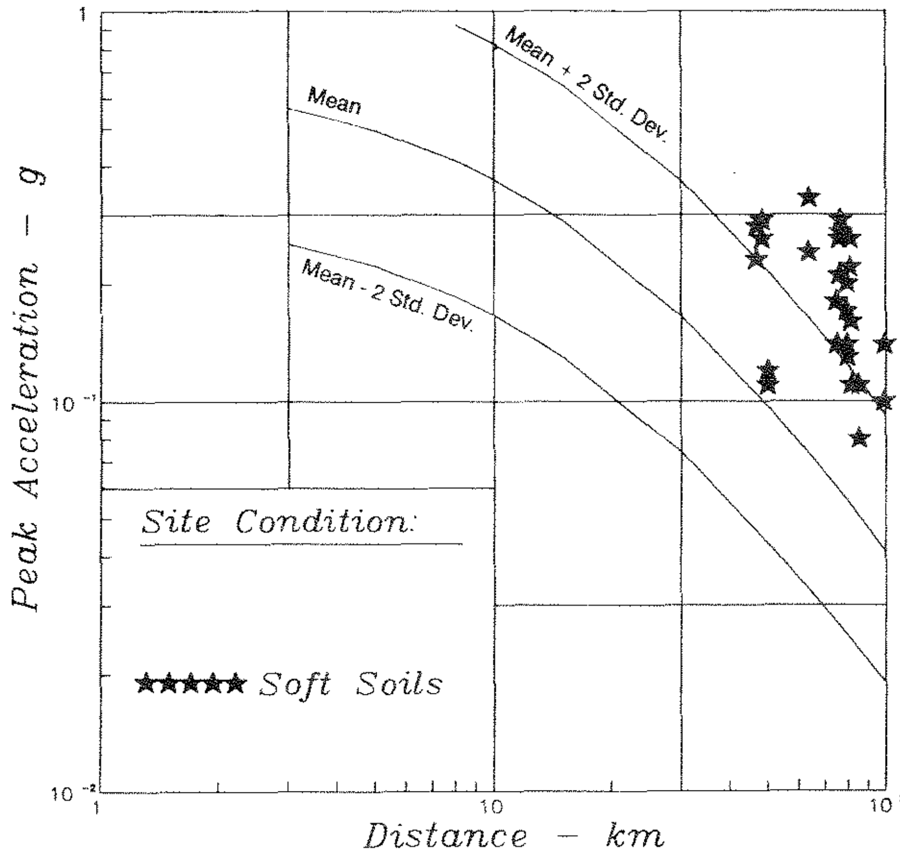


Fig. 4.7: Recorded Peak Horizontal Ground Surface Accelerations on Soft Bayshore Sites, and Attenuation Relationship Proposed by Idriss (1985)

4.3 Local Site Effects and Damage

A good example of this was the catastrophic collapse of an elevated viaduct carrying Interstate 880 through west Oakland. The elevated I-880 Cypress viaduct is a double-decked highway structure, approximately 1.5 miles long, running from north to south through west Oakland, just south of the Bay Bridge approach and distribution structure. The catastrophic collapse of the northern end of the Cypress viaduct (Figures 4.8 through 4.10) was the single most devastating failure during the Loma Prieta Earthquake, resulting in the loss of 38 lives, as well as massive and indefinite disruption of eastern Bay Area transportation networks.

As shown in the simplified geologic map presented in Figure 4.11 (after Hough et al., 1989), the northern end of this viaduct, which collapsed catastrophically, overlies somewhat different soil conditions than the southern end, which was also damaged but which did not collapse. Much of the collapsed northern end of the structure was underlain by an engineered surface fill (compacted, primarily cohesive soils), approximately 15 to 25 feet in depth, underlain along much of this section by soft to medium stiff recent estuarine deposits of silty marine clay known locally as Bay Mud. These Bay Mud deposits, ranging in thickness from 0 to approximately 25 feet, are in turn underlain by very deep deposits of older (Quaternary), and much stiffer (primarily cohesive) sediments. These deeper sediments are primarily stiff, overconsolidated clays and sandy clays, but also include layers and lenses of both sandy and gravelly soils. The upper portion of these deep Quaternary deposits are referred to as the Temescal Formation, and are underlain by deeper, stiffer units referred to locally as the Alameda Formation. These deep deposits are overconsolidated as a result of extended periods of glacial drawdown of sea levels, as well as by ageing effects, and represent strong, competent foundation materials. Most major structures in this area (including the Cypress viaduct) are founded on piles supported in these stiff, deep alluvial soils. As a result, the depth to bedrock is poorly defined throughout this region as few borings have been drilled down to bedrock. A single deep boring, performed by CALTRANS in December and January of 1989-90 along the alignment of the collapsed northern end of the Cypress viaduct, encountered bedrock of the Franciscan Formation at a depth of 570 feet.

The southern section of the Cypress viaduct is also founded on deep alluvium, and a second boring performed by CALTRANS during this same period encountered bedrock at a depth of 535 feet at one point along this southern section's alignment. This southern section is not underlain by any Bay Mud, and the similar shallow, engineered surface fill which is present along much of this section appears to be underlain by somewhat stiffer, stronger and sandier alluvium of the Merrit Formation, underlain in turn by deep deposits of the Alameda Formation. Accordingly, although the near-surface soils appear to be somewhat stiffer, the site conditions may still be generally characterized as consisting primarily of deep, stiff alluvium with both cohesive and cohesionless layers and lenses.

There are no strong motion recordings of the main Loma Prieta event from the Cypress site, but based on strong motion records from nearby sites, aftershock

Fig. 4.8: The Collapsed Interstate 880 Cypress Viaduct

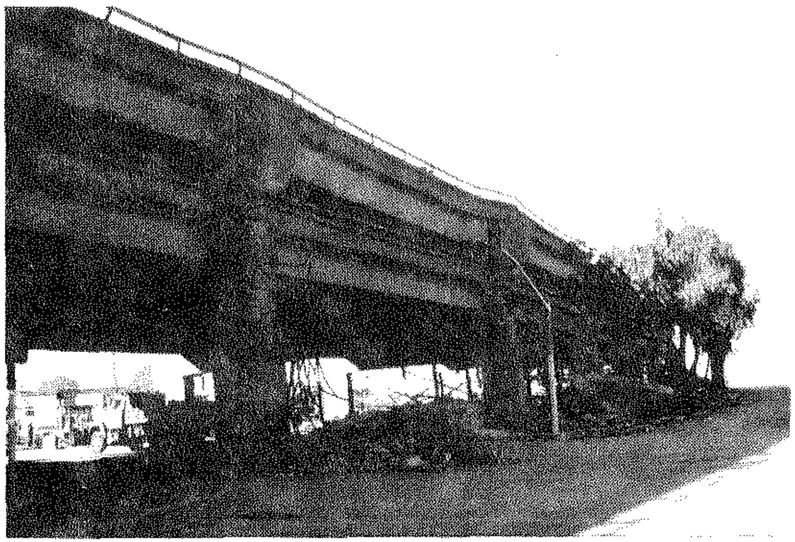


Fig. 4.9: The Collapsed Interstate 880 Cypress Viaduct

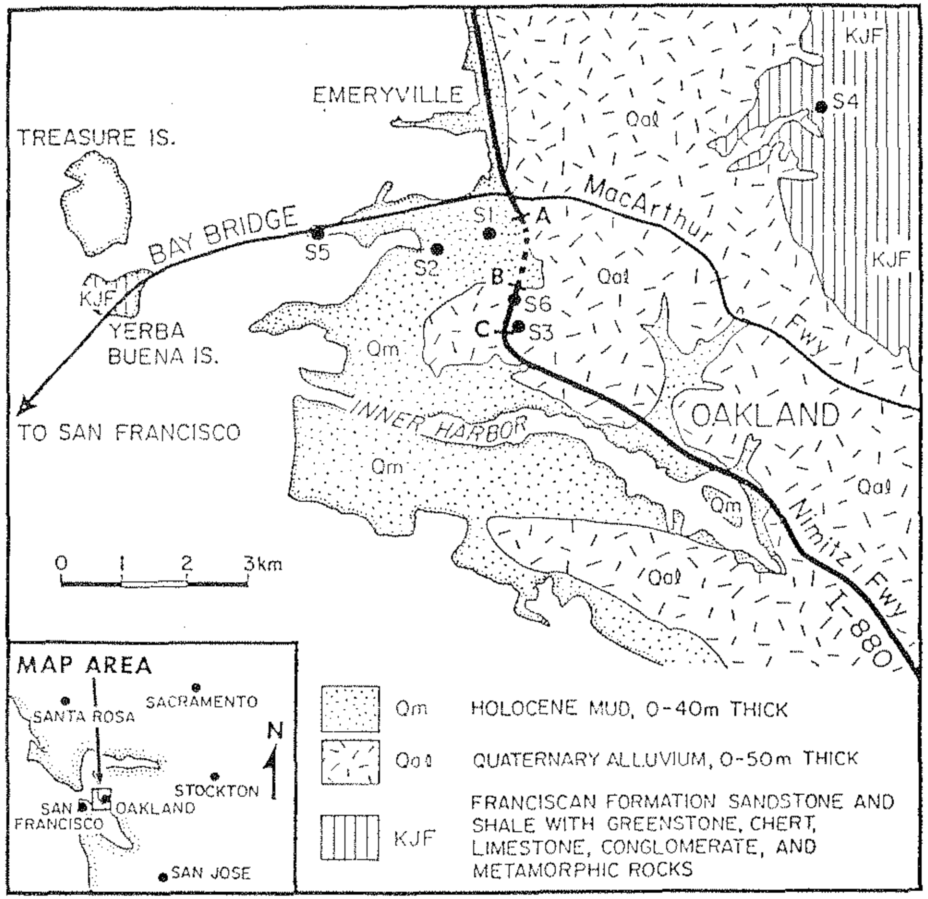


Fig. 4.10: Simplified Geologic Map Showing Alignment of the I-880 Cypress Elevated Viaduct (Viaduct collapsed from A to B, and was damaged from B to C). [After Hough, et al., 1989]

recordings, surface geophysical measurements and preliminary site response analyses, it appears that peak horizontal ground surface accelerations at the collapsed northern end of the Cypress viaduct were on the order of 0.25 g to 0.33 g. Equally importantly, the predominant site periods along much of this section, for these levels of shaking, appear to be on the order of 1.2 to 2 seconds. The Cypress viaduct was a complex structure, and the predominant period of the structure under these levels of shaking has not yet been conclusively determined, but it appears likely that the structure would have responded strongly to these long-period motions, especially as damage began to "soften" the structure's response stiffness. Similar preliminary analyses, aftershock recordings and surface geophysical measurements suggest that the southern portion of the Cypress viaduct alignment, with its somewhat stiffer near-surface alluvial soils, may have experienced similar to slightly lower levels of peak ground surface acceleration, and with similar to slightly lower predominant periods.

In summary, it appears likely that local site conditions strongly amplified peak ground surface accelerations, and especially the long-period components of shaking, and that this was a significant factor in the observed collapse of the northern end of the Cypress viaduct, and the damage to the southern end of this structure. It must be emphasized, however, that local site effects were certainly not the only important factor in this collapse. Any structure, including the Cypress viaduct, is vulnerable to damage or failure when the intensity of shaking exceeds the capacity or resistance of the structure. At the Cypress site, and other sites of heavily concentrated damages, local site effects (amplification and long-period resonant soil/structure interaction) acted to increase the intensities of shaking. At all of these sites, however, it was the less-resistant structures which were most heavily damaged. Thus the interplay of site characteristics, shaking levels, and structural design details must be understood as contributing jointly to the observed damages at these sites.

The Cypress structure was by no means a solitary example of local site effects resulting in increased surface shaking intensity; such effects were evident over "soft" bayshore sites throughout the central San Francisco Bay Region. Another prominent example was the Marina District on the northern coast of San Francisco. As described previously in Section 3.2.1, much of the Marina District rests upon shallow, sandy fill, extending to depths of typically 15 to 30 feet. This is underlain, throughout much of the District, by between 10 to 60 feet of soft to medium stiff Bay Mud. The Bay Mud is underlain by stiffer, denser sediments, including stiff cohesive soils and medium dense to dense sandy soils. The depth to bedrock beneath the District is not well-defined, but limited available data and extrapolation of onshore surface contours suggest that the bedrock forms a half bowl, opening to the north towards the bay. A single deep boring, performed by the USGS in December of 1989, near the center of the District near the intersection of Beach and Divisadero Streets, encountered rock at a depth of 260 feet (Kayen et al., 1990). A second deep boring near Buchanan and Bay Streets shows the depth to rock to be similar at this location (Whitworth, et al., 1932).

Figure 3.12 shows the locations of heavily damaged structures, as indicated by post-earthquake inspection "tags", approximately one month after the earthquake.

City inspectors placed tags of different colors on buildings; these tags indicated their perceived level of safety and controlled access to these buildings in the wake of the earthquake. Structures with red tags were considered unsafe for occupancy, while yellow tags indicated structures to which limited access might be allowed. Damaged structures were periodically re-inspected, and tags were changed or removed as these inspections progressed. Most of these structures were two to four-story apartment buildings and, as illustrated in Figure 3.12, were located near the heart of the Marina District. Much of this ground is underlain by the loose hydraulic fill placed in 1910-1912, and much of the rest is underlain either by fill placed to reclaim the perimeter marshes or by naturally deposited loose beach and dune sands which occur at the edges of the region.

This does not mean, however, that this concentration of structural damages was due primarily to liquefaction of these saturated sandy soils. Instead, a majority of the damage to structures in the Marina District on October 17, 1989 was caused by strong shaking, as the cohesive soils underlying the fill strongly amplified the relatively modest levels of shaking produced in the bedrock underlying the Marina District. These local soil conditions also altered the frequency characteristics of the accelerations propagating from the rock to the ground surface. This amplification of accelerations, and the especially pronounced amplification of long period motions, appears to have been the primary cause of the observed heavy concentration of structural damage in this region. It is also interesting to note that much of the structural damage was associated with the collapse of "weak" ground floors consisting primarily of garages with few walls and thus little structural capacity for carrying lateral shear forces. Two examples of this are shown in Figures 4.11 and 4.12.

It has been noted by other researchers that much of the worst structural damage, and most collapses of buildings with "soft" ground floors, occurred at corners of blocks. This does not appear to have been due to shaking being passed along a line of adjacent structures in such a manner that the end structure "fell over". Rather, it appears that corner lots were typically larger than lots within the interiors of the blocks, and so more typically had three or four-story apartment structures with garages dominating their ground floors built upon them. Interior lots were more commonly smaller and had smaller two to three-story structures, typically with their ground floors less completely dominated by garages. The larger, taller structures typically had more upper floor mass with which to load weak ground stories with shear forces, and typically had "weak" ground stories. They may also have had longer natural periods which were more nearly resonant with the long period ground motions produced by the underlying soil conditions.

It should also be noted that structures similar to those which collapsed or suffered major damage in the Marina District, including three and four-story apartment buildings with "soft" ground floors (garages) also occur in the neighborhoods to the south and east of the Marina District. These essentially identical buildings were typically undamaged in these neighborhoods which are founded on considerably stiffer, shallower soils and/or rock.

Fig. 4.11: Examples of Collapse of Two Structures in San Francisco's Marina District Due to "Soft" Ground Floors

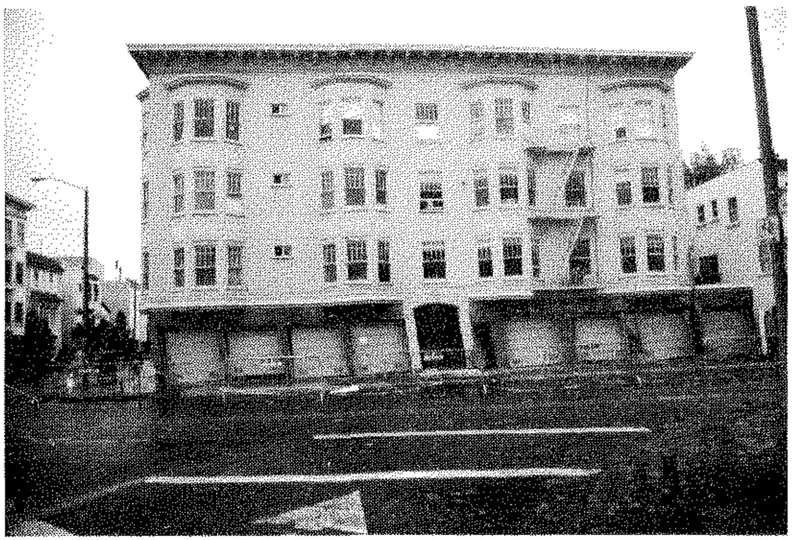


Fig. 4.12: Partial Collapse of a Structure in San Francisco's Marina District With a "Soft" Ground Floor

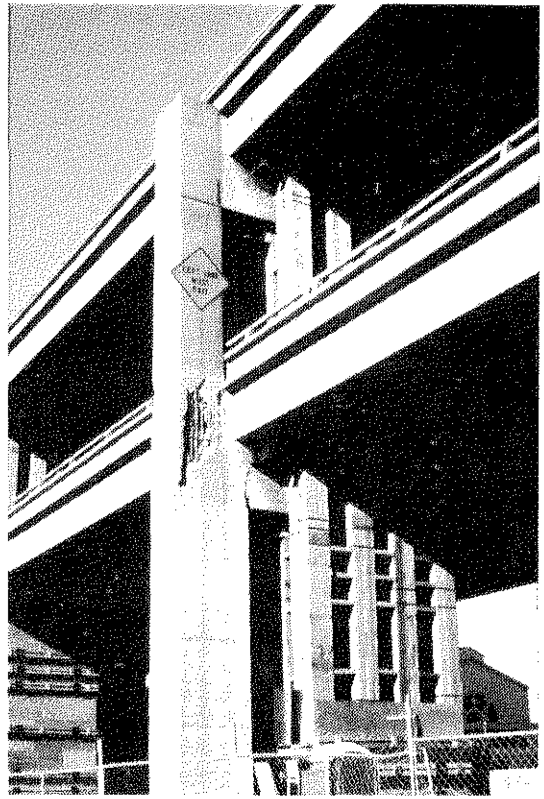


Fig. 4.13: Damage to the Embarcadero Highway Elevated Viaduct in San Francisco

The pattern of soil conditions present throughout much of the Marina District is repeated at numerous other sites throughout the central Bay region. Examples of additional locations with shallow surface fills, underlain by soft to medium stiff recent estuarine clays and silty clays (Bay Mud), underlain in turn by deeper, stiffer, overconsolidated Pleistocene sediments, also include the following sites:

- Along the San Francisco Embarcadero coastline, and at the foot of Market Street; areas where limited liquefaction occurred. In addition to liquefaction, a number of structures were damaged by strong shaking, including, most prominently, the double-decked elevated Embarcadero highway viaduct. This important structure was damaged (e.g. Figure 4.13), and remains out of service, resulting in significant traffic disruption and major ongoing adverse economic impacts for businesses in northern and central San Francisco.

- In the historic old Mission Bay marsh region in San Francisco. The region surrounding Sixth and Folsom in San Francisco was an area of particularly concentrated structural damages, as shown in Figure 3.14, including the catastrophic failure of the unreinforced masonry structure near 6th and Townsend shown in Figure 4.14. A major portion of one of the exterior walls of this building fell onto the sidewalk, killing 5 people. A smaller, similar pocket of both concentrated structural damage and liquefaction occurred on similar marsh deposits near Shotwell at Folsom and 17th Streets. Another elevated highway viaduct, the Interstate 280 viaduct, was also damaged and remains out of service in the Old Mission Bay region.

- South of San Francisco, along the bay shoreline, a number of structures appear to have been strongly shaken and damaged due, at least in part, to underlying soft, cohesive bay sediments. This included light damage to terminal buildings at San Francisco International Airport, and significant damage to the Amfax Hotel in Burlingam, shown in Figure 4.15. This hotel was damaged when a rooftop water tank was shaken off, carrying away the top of the centrally located elevator tower as shown in closeup view in Figure 4.15.

- In the central San Francisco Bay and at Treasure Island. As described previously in Section 3.4, amplification of shaking levels by the underlying alluvial soils is suspected to have been a significant factor in the widespread liquefaction which occurred at Treasure Island. The soft Bay Mud, and underlying deeper, stiffer alluvium also appear to have been important factors in the collapse of a section of the Bay Bridge between Yerba Buena Island and Oakland (see Figure 4.16). The two deck sections (upper and lower decks) at the collapse point were simply supported on 5-inch long support brackets, and appear to have slipped off of these supports as a result of more than 5 inches of (transient) differential seismic displacement of adjoining bridge sections during shaking (A. Astaneh, 1989). The depth of the alluvial deposits at and near the collapsed section

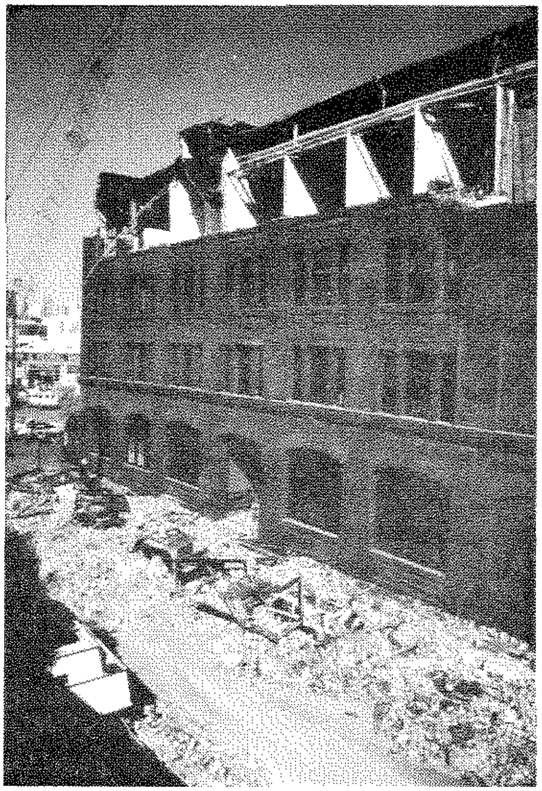


Fig. 4.14: Collapse of Exterior Wall of Unreinforced Masonry Structure Near 6th & Townsend, San Francisco

Fig. 4.15: Damaged Roof and Top of Elevator Tower, Amfax Hotel, Burlingam [Photo courtesy of Prof. S. A. Mahin]

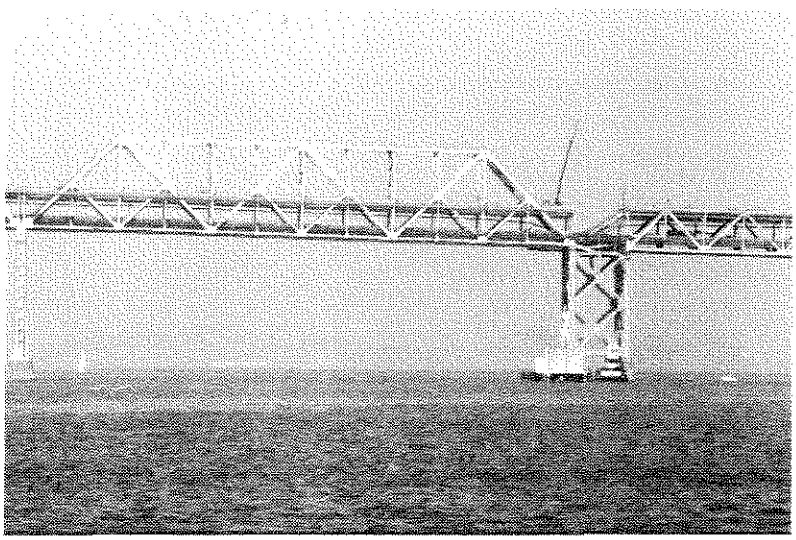


Fig. 4.16: Collapsed Section of San Francisco-Oakland Bay Bridge Prior to Removal of Fallen Deck Sections

is not known, but is known to be in excess of 350 feet. These deep alluvial deposits, with a fairly thick surficial layer of soft to medium stiff Bay Mud, appear likely to have produced amplified long period strong shaking, resulting in large transient displacements of both the bridge foundation elements and the towers and deck sections. Additional, less dramatic damage to this bridge also appears likely to have been influenced by the amplified long-period motions at this site.

- At Richmond Harbor, amplified levels of long period shaking on fill overlying Bay Mud and deeper, stiffer alluvium damaged several petroleum storage tanks and caused minor damage to appurtenent piping (Dames & Moore, 1989), and damaged an unreinforced masonry warehouse, as shown previously in the photograph in Figure 3.49. This damage occurred, despite the unusually short duration of this M_S = 7.1 event, at a distance of more than 60 miles from the northern end of the fault rupture, and more than 70 miles north of the epicenter. Extrapolation of recorded data suggests that peak horizontal accelerations on rock at this site would have been on the order of only 0.03 to 0.08 g. The damage to the warehouse shown in Figure 3.49 appears to represent an especially clear demonstration of the influence of foundation soils on strong shaking as this structure, which sustained considerable damage, was sited above a historic channel and so was underlain by unusually deep deposits of Bay Mud (beneath the surface fill, and underlain in turn by older, stiffer alluvium), while similar nearby structures, founded on more favorable soil conditions, appear undamaged.

- Along the Emeryville and Port of Oakland shorelines, at Alameda and at the Oakland International Airport, amplification of shaking appears again to have been a significant factor in the observed liquefaction of loose fills underlain by Bay Mud and deeper, stiffer alluvial deposits (as described previously in Sections 3.5.2 through 3.5.7). In addition, the amplified long-period motions produced in these areas resulted in strong shaking of tall, multi-story and high rise structures in the Emeryville and West Oakland areas. An example of the dangers associated with large, heavy objects being thrown about by these strong shaking levels is presented in Figure 4.17. Fortunately, the high rise buildings in the Emeryville bayshore region are modern (post-1970) structures, built to withstand some level of seismic loading, and none of these high rise buildings appear to have been seriously damaged by the levels (and short duration) of shaking produced by this event.

- A large number of structures were heavily damaged by amplified levels of long period shaking in large parts of West Oakland. Much of downtown and west Oakland is founded on fills underlain by Bay Mud and older, stiffer alluvium, and considerable structural damage occurred in these areas. Indeed, widespread and serious damage to major structures in these areas was one of the clearest examples of structural damage influenced by

Fig. 4.17: Displaced Office Furniture in Upper Floor of a High-Rise Building in Emeryville/West Oakland Area

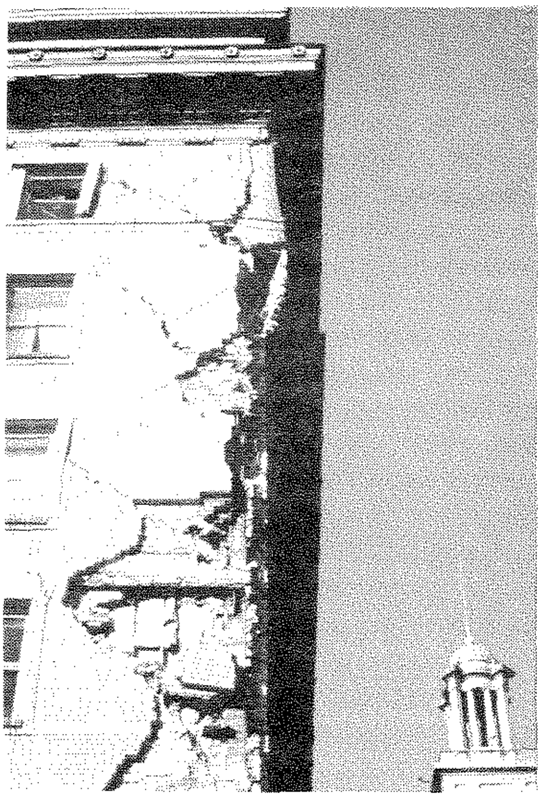


Fig. 4.18: Damage to an Upper Corner of the Oakland Hotel [Photo courtesy of Prof. S. A. Mahin]

Fig. 4.19: Near Failure in Shear Due to Pounding Between Two Buildings on Franklin Near 21st Street, Oakland [Photo courtesy of Prof. S. A. Mahin]

local soil conditions. Many of the heavily damaged multi-story and high rise office buildings, retail operations, hotels and apartment complexes in this area appear to have been either masonry or reinforced concrete structures built between 1930 and 1970 (e.g. the Oakland Hotel, Figure 4.18), during which time site effects and resonant soil/structure interaction were poorly understood. Most of the damage appears to have been the result of strong shaking (strong structural response to the amplified, long period ground surface motions), but a number of cases of "pounding" between adjacent multi-story and/or high rise structures (e.g. Figure 4.19) were also noted. In addition to large economic losses and disruption of business, another very serious consequence of the damage to tall structures in this area was the forced closure of the damaged City Hall, which disrupted city operations at a particularly unfortunate time. (City Hall remains closed, pending analysis and retrofit.) Although most of the major structures damaged in this area were built prior to 1970, damage to tall structures also included at least one prominent building built during the 1970's. The seismic codes of this era, though providing some general improvement in resistance, thus appear to have underestimated site amplification and spectral amplification (or resonant site/structure interaction) effects, as this relatively distant $M_S = 7.1$ event (of unusually short duration) represents a considerably less severe level of seismic shaking than that more typically associated with a "design level" event in this area.

In addition to these examples of apparent "site effects" at San Francisco bayshore sites, it appears likely that site effects also played a role in the structural damage that occurred in both Santa Cruz and Watsonville. Both central Santa Cruz, and the city of Watsonville are underlain by deep alluvial deposits. These deep soil profiles are not likely to have significantly amplified the already high peak horizontal accelerations on rock at these two sites near to the fault rupture zone, but they are likely to have increased the concentration of ground surface shaking energy in longer period ranges, and this may have adversely impacted the behavior of some structures. It is interesting to note, in this respect, that the heaviest concentration of structural damage in the City of Santa Cruz (including the devastating damage to structures of the Pacific Garden Shopping Mall) occurred on sites underlain by alluvial deposits at the mouth of the San Lorenzo River, as shown previously in Figure 3.78.

On the other hand, much of the structural damage in both Santa Cruz and Watsonville occurred to either: (a) older, "historic" buildings (often unreinforced masonry) with apparently inadequate seismic resistance, or (b) single-family houses which were rocked or toppled off their foundations due to collapse of "cripple walls" between the foundation and ground floor. It appears likely that many of these structures would have been overwhelmed by strong levels of shaking at these sites near the rupture zone even if they had been founded on shallower, stiffer soils or on rock.

Finally, it is worth noting that our society has a propensity for constructing major population centers on "level" sites underlain by alluvial deposits. As many of our cities and towns are underlain by fairly "deep" soils, it is of vital importance that: (a) we come to fully understand the important influence of these soil conditions on site response characteristics during earthquakes, (b) this understanding is meaningfully and adequately reflected in design practice and in seismic building code provisions, and (c) these effects are fully and correctly accounted for in seismic re-evaluation and retrofit (as necessary) of existing structures on "deep" soil sites.

4.4 Treasure Island and Yerba Buena Island:

An excellent example of the influence of local soil conditions on ground shaking characteristics is provided by the sets of strong motion recordings obtained at two stations: (1) on Yerba Buena Island, and (2) on Treasure Island. Both islands are located at the center of San Francisco Bay, approximately 45 miles north of the fault rupture surface, and the strong motions recorded at these two stations differ significantly as a result of different foundation conditions.

Yerba Buena Island is a large, rocky outcrop near the center of the bay, and anchors the Bay Bridge as shown previously in Figure 3.35. Treasure Island, as described previously in Section 3.4, is a man-made island comprised primarily of loose, dredged hydraulic fill underlain by the natural bay sediments. The strong motion recordings at the Treasure Island and Yerba Buena Island stations thus represent a pair of recordings at nearly the same location (and distance from the fault rupture), but for a "rock" and a "deep, soft soil" site.

Figure 4.20 presents a schematic illustration of the soil column underlying the Treasure Island recording station. The upper 30 feet of soil consists of loose, dredged hydraulic fill, primarily sand and silty sand. This is underlain at the recording station by approximately 15 feet of loose silty sand which may represent either additional hydraulic fill or part of the sand bar upon which parts of the island fill were placed. This is underlain by approximately 55 feet of soft to medium stiff, normally consolidated silty clay (Bay Mud). The Bay Mud is underlain by approximately 40 feet of alternating layers of dense, fine sand and silty sand, and layers of stiff, overconsolidated sandy clay. Beneath this, stiff to hard, overconsolidated silty clays and clays, with occasional seams and lenses of sandy and gravelly soils, extend down to bedrock, which occurs at a depth of approximately 285 feet.

Also shown in Figure 4.20 are the N-S component strong motion records from the Yerba Buena "rock" site (shown at the base of the profile) and from the Treasure Island station (shown at the ground surface). It is clear from inspection that the Treasure Island ground surface record has a significantly higher (amplified) peak ground surface acceleration, and a longer predominant period.

These site effects are also evident upon examination of the peak ground accelerations of all three directional components of the motions recorded at these

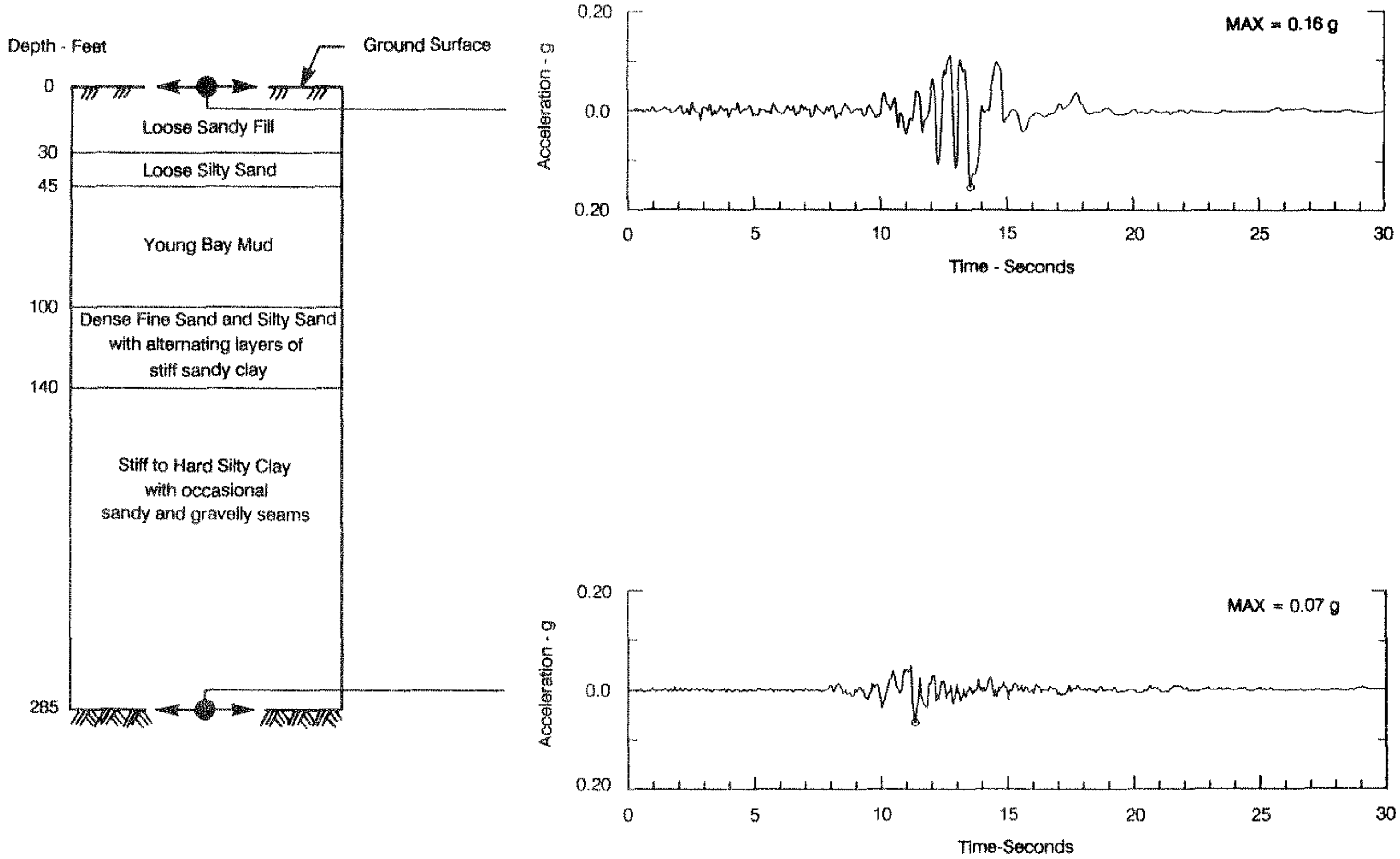


Fig. 4.20: Schematic Soil Profile and Site Response at the Treasure Island Station

two stations, as well as upon examination of the associated response spectra. The peak ground surface accelerations recorded at the Treasure Island and Yerba Buena Island stations were as follow:

	N-S Component	E-W Component	Vertical Component
Treasure Island	$A_{max} = 0.10$ g	$A_{max} = 0.16$ g	$A_{max} = 0.02$ g
Yerba Buena Island	$A_{max} = 0.03$ g	$A_{max} = 0.07$ g	$A_{max} = 0.03$ g

Figures 4.21 and 4.22 show the response spectra (5% damping) for the E-W and N-S component records, respectively, at these two stations. In addition to amplification of the peak ground surface accelerations, it can be seen in this figure that the deep, soft cohesive soil profile at the Treasure Island station also caused especially pronounced amplification of the long-period components of motion.

Although this preliminary report has, thus far, avoided presentation of detailed analyses, it is of interest to examine a fairly simple, preliminary analysis of these response records and they are illustrative of the important effects associated with these site conditions.

The soil profile shown in Figure 4.20 was modelled, using the program SHAKE90 (Schnabel, et al., 1990), in a one-dimensional dynamic site response analysis based on vertical propagation of shear waves. The E-W Yerba Buena Island record was taken as a basis for development of the "bedrock" input motion applied at the base of the soil column, and the resulting ground surface motions calculated were compared with the E-W component of the motions recorded at the Treasure Island station. It should be noted that the Yerba Buena Island record was an "outcrop" motion, and so was slightly modified (accelerations were decreased by almost 10% and the predominant period was slightly increased, as described by Schnabel et al., 1972) to generate a more representative "bedrock" motion at a depth of 285 feet.

The program SHAKE90 is a slightly modified version of the well-known program SHAKE (Schnabel, et al., 1972), and uses the "equivalent linear" method to model nonlinear dynamic soil moduli and damping as a function of shear strain. Nonlinear soil properties for the cohesionless strata were modelled using the dynamic modulus degredation vs. shear strain (G vs. γ) and damping ratio vs. shear strain (β vs. γ) relationships for cohesionless soils proposed by Seed et al., 1984. Maximum (small strain) shear wave velocities in the upper, loose sandy fill were modelled as $v_s \approx 500$ to 600 ft/sec, increasing with depth, and velocities of $v_s \approx 550$ to 650 ft/sec were used to model the loose silty sand overlying the Bay Mud.

Dynamic properties for Bay Mud are fairly well-established, based both on extensive research by numerous investigators as well as on investigations (typically proprietary) performed for civil projects. A good summary of published data is presented by Sun et al. (1988) and Seed and Sun (1989). Figure 4.23 shows the shear

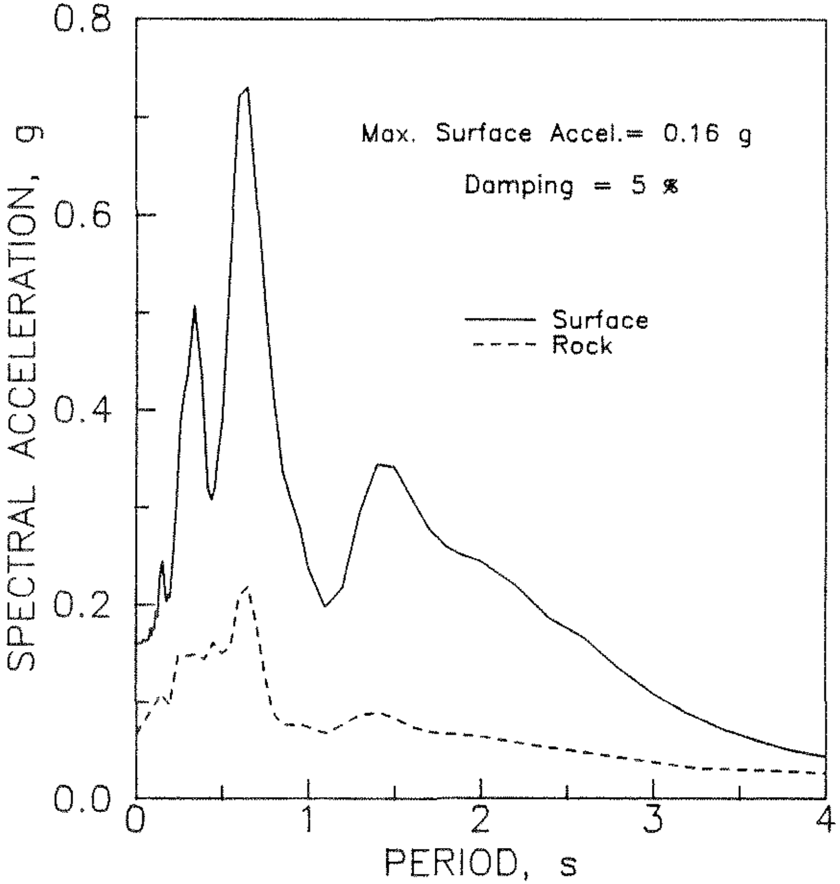


Fig. 4.21: Response Spectra for Treasure Island and Yerba Buena Island Station Records (90 degree components)

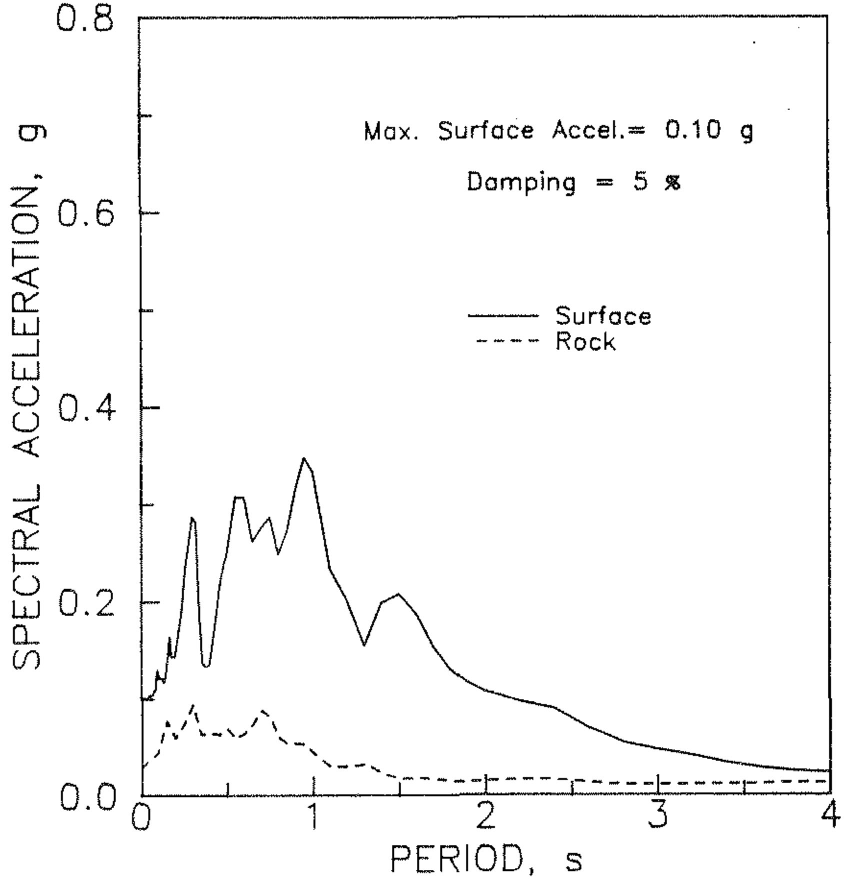


Fig. 4.22: Response Spectra for Treasure Island and Yerba Buena Island Station Records (0 degree components)

strain dependent dynamic modulus degredation and damping relationships used in these analyses to model strain-dependent behavior of Bay Mud. Based on published data, as well as local experience, shear wave velocities within the Bay Mud were modelled as $v_s \approx 500$ to 700 ft/sec, increasing with depth.

The dense, sandy strata underlying the Bay Mud were again modelled using the relationships proposed by Seed et al. (1984), with $v_s \approx 1100$ ft/sec. The stiff, underlying alluvium was modelled using strain dependent modulus degredation and damping relationships proposed by Seed et al. (1988) for cohesive soils of intermediate plasticity, with $v_s \approx 1100$ to 1400 ft/sec, again increasing with depth. It should be noted that these analyses are not very sensitive with respect to the modelling of the strain-dependent nonlinear behavior of the materials underlying the Bay Mud, as these stiff soils do not behave in a very nonlinear fashion at the relatively moderate levels of excitation caused by this earthquake. Nonlinear behavior of the upper (and softer) soils is important, however. Finally, the "bedrock" was modelled as an elastic half-space with $v_s \approx 3{,}500$ ft/sec.

Figure 4.24 shows the results of these one-dimensional ("columnar") dynamic response analyses. The lower dashed line represents the response spectrum of the input "rock" motion, and the upper dashed line represents the response spectrum for the resulting, calculated ground surface motion. Also shown for comparison, with a solid line, is the actual recorded E-W ground surface motion at the Treasure Island site.

As shown in this figure, these relatively simple, one-dimensional response analyses performed using the "equivalent linear" method to model nonlinear soil behavior provide good agreement with the observed surface response. The calculated maximum horizontal ground surface acceleration of $a_{max} = 0.18$ g agrees well with the recorded value of $a_{max} = 0.16$ g. The calculated motion also provides a reasonably good "fit" for the recorded motion's response spectrum, with a strong spectral peak at a period of $T_s \approx 0.6$ seconds, and a secondary peak at $T_s \approx 1.3$ seconds.

It is interesting to note that the spectral peak at $T_s \approx 0.6$ seconds does not represent the predominant period of this deep, soft site. Instead, this is the result of the second mode of the site's response being strongly exited by an input rock motion with a concentration of energy at $T_s \approx 0.6$ seconds. The predominant natural period of the site is approximately $T_s \approx 1.3$ seconds, at this level of shaking, and upon close inspection it can be seen that <u>spectral</u> amplification (the ratio of $a_{max,spect,surface}$ vs. $a_{max,spect,rock}$) is approximately 4 to 5 near this period range, and only approximately 2.5 to 3 at and near $T_s = 0.6$ seconds.

Finally, although these simple dynamic response analyses do a good job of predicting the recorded Treasure Island ground surface motions, the agreement between the calculated and the recorded motions is not perfect. The failure of the calculated motions to pick up the full spectral content (or spectral peak) of the recorded motions at $T_s \approx 0.35$ seconds (see Figure 4.24) is probably due in large part

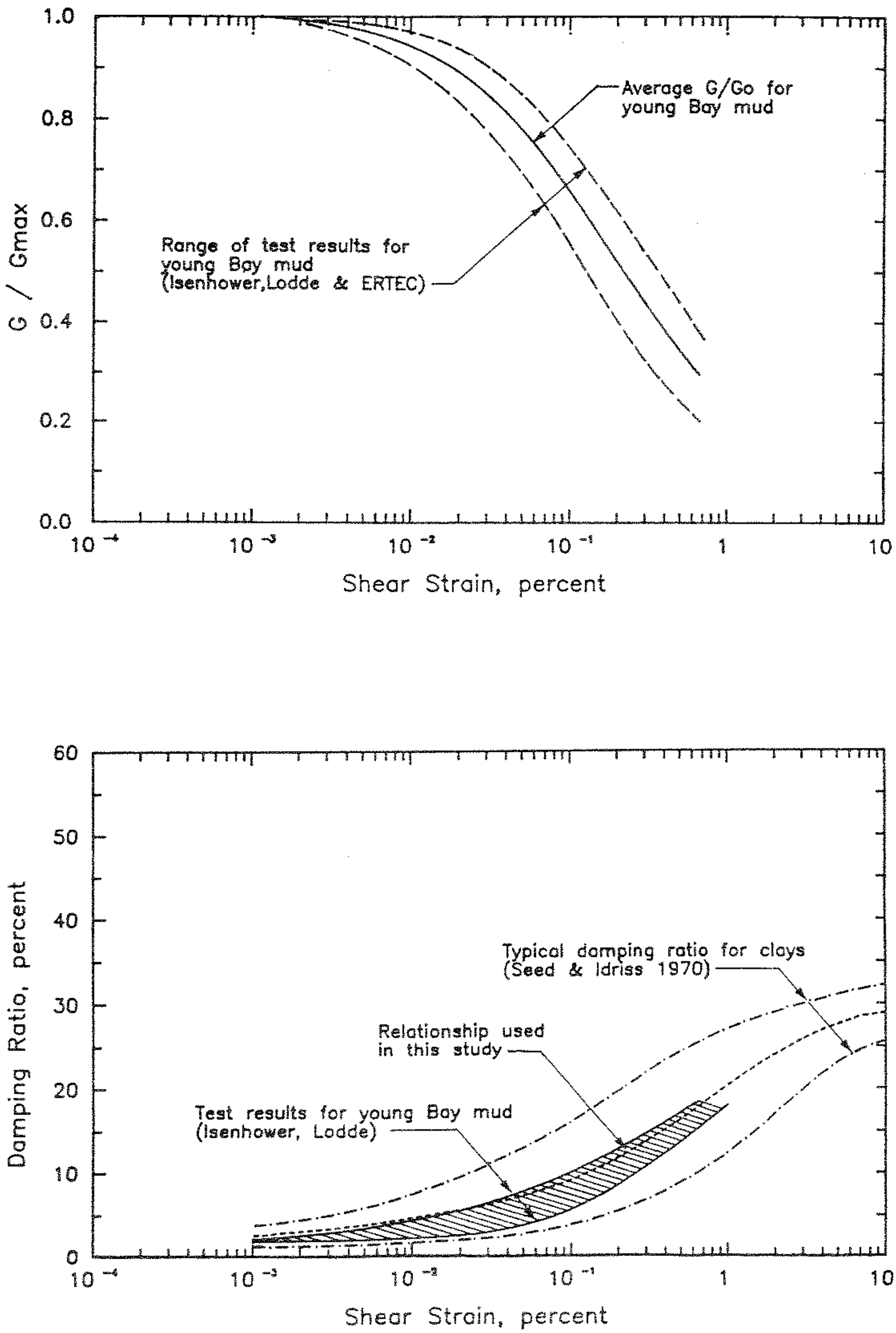


Fig. 4.23: Variation of Dynamic Shear Modulus and Damping Ratio with Shear Strain for Young San Francisco Bay Mud (Seed and Sun, 1989)

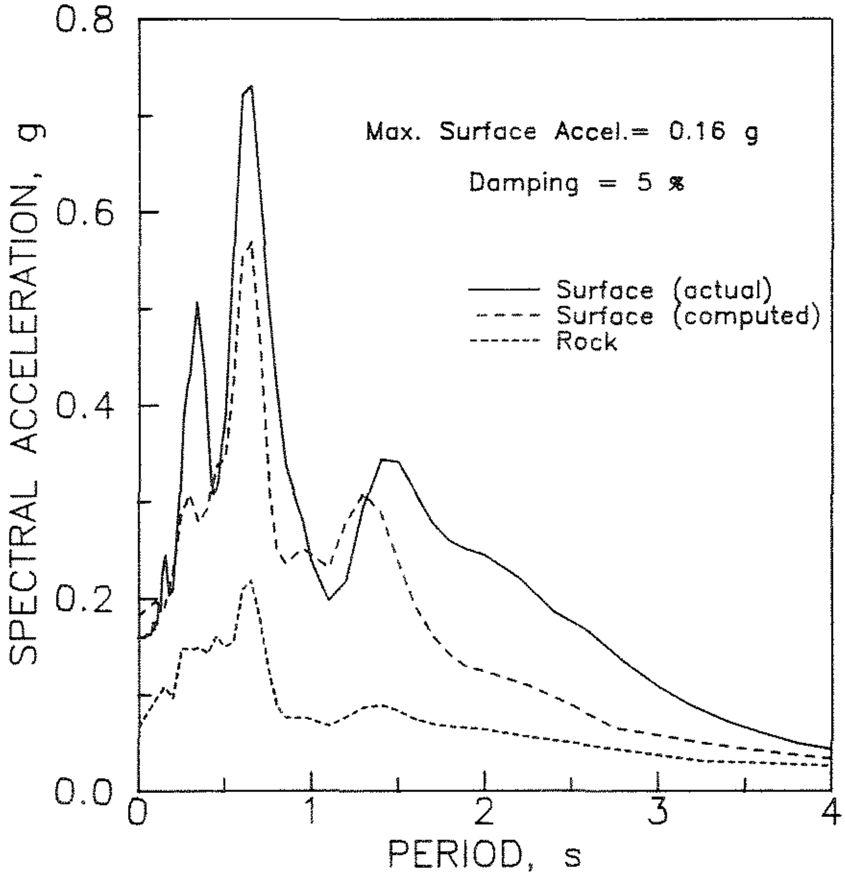


Fig. 4.24: Comparison Between Calculated and Recorded Response Spectra - Treasure Island Fire Station (90 degree comp.)

of the tendency of the "equivalent linear" method of modelling nonlinear soil response to result in overdamping of higher frequencies. This can suppress high frequency spectral peaks to some extent. Fully nonlinear analyses, performed in the time domain, have also been performed for this site, and these preliminary analyses are well able to model this low period spectral peak (which appears to correspond to the third mode of the site's response).

An additional shortcoming of the analyses presented in Figure 4.24 is the underestimation of the long period spectral response at $T_s \gtrsim 1.5$ seconds. It is initially tempting to attribute this to pore pressure induced softening (or liquefaction) of the upper fill, and indeed the Treasure Island record shows distinct evidence of softening after approximately 14 seconds of shaking, as shown in Figure 4.20. Analyses, however, suggest that this apparent softening towards the end of the event, which cannot be modelled using the "equivalent linear method", does not well explain this underestimation of long-period motions. A possible alternate hypothesis might be that these long period motions may result, in large part, from surface waves. These might be locally generated by the dipping of the Yerba Buena rock outcrop beneath the alluvium and fill of Treasure Island, and would not be amenable to simple, one-dimensional columnar dynamic response analyses.

4.5 Summary & Conclusions:

A number of geotechnical factors exerted a tremendous influence on damage patterns and loss of life during the Loma Prieta Earthquake. Near to the zone of fault rupture, many structures were simply overwhelmed by high inertial forces. More than half of the damages, however, and more than 80 percent of the loss of life, occurred at sites in the north-central San Francisco Bay Area, far from the epicentral region. This concentration of damage on a few relatively distinct sites comprising less than one percent of the "strongly" shaken region was due primarily to the local soil conditions at these sites. These concentrated damages occurred at sites underlain by deep, and primarily cohesive, soil deposits which served to amplify the relatively moderate levels of "bedrock" shaking generated by the earthquake in this region, producing significantly stronger levels of surface shaking. Peak accelerations on rock in the San Francisco, Treasure Island, Oakland, Alameda and Emeryville region appear to have been on the order of 0.06 to 0.12 g. Instrumental recordings, as well as dynamic response analyses, show that many of the bayshore soil deposits in this region amplified these levels of shaking by factors of about 2 to 3, producing peak ground surface accelerations at deep alluvial sites on the order of 0.16 to 0.33 g in this region. In addition, amplification of the longer period components of shaking was especially pronounced, so that the resulting surface motions were particularly damaging to taller, longer period structures.

This type of pronounced, site specific amplification (and spectral amplification, or resonant soil-structure interaction) of ground motions was not a surprise to the earthquake engineering community. Similar site-specific amplification has been noted as an important factor controlling damage patterns in numerous previous major earthquakes over the past 30 years. In addition, the important impact of local site conditions on strong shaking characteristics and resulting damage patterns at Bay Area sites had been well-documented by numerous investigators in the wake of the 1906 San Francisco Earthquake, and a number of engineers and researchers had performed site-specific response studies resulting in microzonation mapping which well-predicted the observed zones of high intensity shaking at "soft" bayshore sites during the Loma Prieta Earthquake. The map shown in Figure 4.25 (Borcherdt, et al., 1975) is but one of a number of such maps.

Building code provisions dealing with these "site" effects have gradually evolved over the past 20 years, and a particularly important improvement in these provisions occurred in 1988 as a result of the clearly overwhelming influence of local site effects on the catastrophic damages suffered by major buildings on deep clay sites during the 1985 Mexico City Earthquake (1988 Uniform Building Code). It may be anticipated that further improvements in the ways that the effects of local geotechnical site conditions are dealt with in seismic building codes will result from the lessons learned yet again in this regard during the Loma Prieta Earthquake.

Finally, it must be noted that site-specific "amplification" is a nonlinear effect. Relatively modest levels of peak horizontal acceleration on rock throughout the central Bay Area on the order of 0.06 g to 0.12 g were "amplified" to produce peak horizontal ground surface accelerations approximately 2 to 3 times higher on soft, bayshore sites. Amplification of peak ground surface accelerations was even more pronounced (factors of 4 to 8 for soft bayshore sites relative to rock sites) for low level aftershock recordings. At stronger levels of bedrock shaking in future, larger or more near-field events, amplification of peak ground surface accelerations will be less pronounced; indeed, for levels of $a_{max,rock} > 0.4$ g, the peak ground surface accelerations on soft clay sites may be slightly less than those on rock.

This does not mean that site effects will not have a potentially important adverse impact on structural performance on soft and deep clay sites for strong levels of bedrock shaking. Although peak ground surface amplification becomes less pronounced for stronger levels of shaking at such sites, <u>spectral</u> amplification, or preferential amplification of longer period components of motion, will still result in concentration of shaking energy in long period ranges, and will thus produce potentially highly damaging ground motions at these sites.

In summary, amplification of accelerations, as well as the especially pronounced amplification of long period motions, at sites underlain by soft and/or deep soil deposits, appears to have been a major factor in controlling damage patterns during the Loma Prieta Earthquake. This was no surprise to the engineering community, and these "site effects" are likely to continue to be important factors in future seismic events in this region.

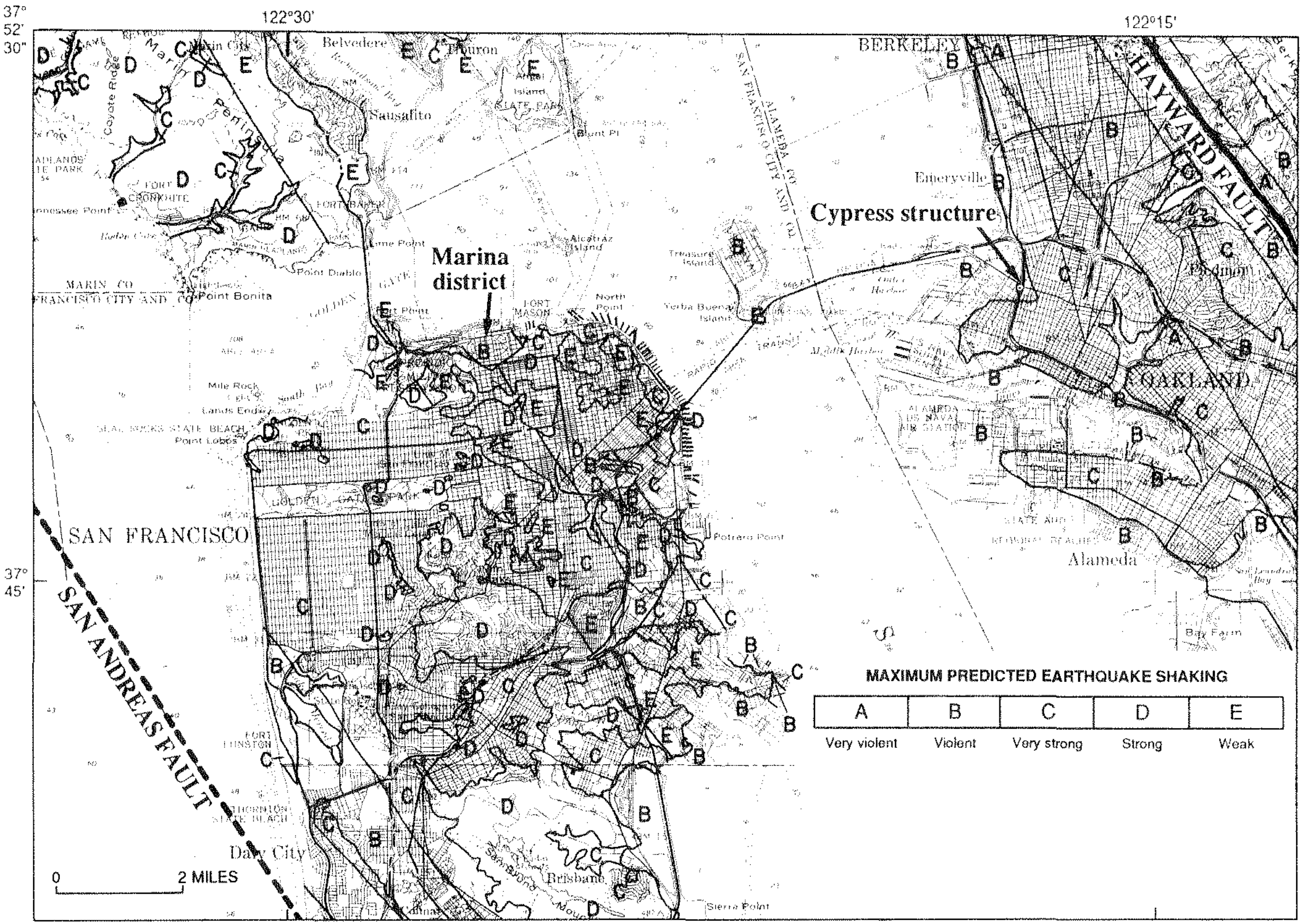


Fig. 4.25: Map Showing Predicted Maximum Intensities of Shaking for Central San Francisco Bay Area Sites [After Borcherdt, et al., 1975, as modified by the U.S. Geological Survey, 1989]

Chapter Five: SLOPE STABILITY PROBLEMS

The Loma Prieta Earthquake caused numerous landslides and rockfalls over a large portion of northern and central California. Extensive and widespread slope stability failures occurred in the Santa Cruz Mountains near the zone of fault rupture, and stability failures and rockfalls also occurred at a number of points along the Pacific coastal bluffs from Rio Del Mar on the east coast of the Monterey Bay to Daly City. In addition, small slides and rockfalls were noted as far as 70 miles north of the epicenter, as far as 30 miles east of the epicenter, and as far as 30 miles to the south of the epicenter. These small slides and rockfalls were of little consequence, but two major slides with serious adverse consequences also occurred far from the epicentral region in central San Francisco, and on the west (Pacific) coast of Marin.

During the Loma Prieta Earthquake of October 17, 1989, a large number of landslides occurred in the Santa Cruz Mountains near the fault rupture region. More than one thousand slides and rockfalls of varying size occurred in this area, most of them within the zone indicated with a dashed line in Figure 5.1, and exploration of this wooded, mountainous region is continuing. Most of the slides and rockfalls in this region were relatively small and shallow, extending to depths of less than 10 to 20 feet. In addition, extensive slumping and cracking occurred in fills along most of the mountain roads in the region, and an example of this is shown in Figure 5.2.

While most slides and rockfalls were relatively small and shallow, a number of larger, deeper slides occurred with widths of up to several hundred feet and lengths of up to one half mile and more. Two relatively large failures in interbedded Tertiary sandstones and shales caused the temporary closure of Highway 17 from San Jose to Santa Cruz at the locations shown in Figure 5.1. The photograph in Figure 5.3 shows one of these two slides on Highway 17 as it appeared shortly after the road had been reopened. Figure 5.4 shows the partially developed head scarp of another slide damaging another road in this area, and Figure 5.5 shows the toe of another slide closing a road in this region. It was difficult to photograph most of the slides in this region due to the steep topography and the densely wooded terrain.

The principal impacts of the widespread sliding in this mountainous region were two-fold: (1) the main highway (Highway 17) and many of the secondary roads in this region were at least temporarily closed, disrupting communications and partially isolating Santa Cruz from the southern San Francisco Bay Area at a time when the City of Santa Cruz had urgent need of emergency assistance and supplies, and (2) a number of single family residences were destroyed or damaged by landslides in this region. This mountainous region is subject to relatively frequent landslides and flowslides during wet winters, and considerable historic local experience with problems associated with slope instability has led to a general avoidance of residential construction on the most obviously unsafe slopes. This, along with the sparse population of this fairly isolated mountainous region, served to reduce to some extent the number of residences and other buildings destroyed and

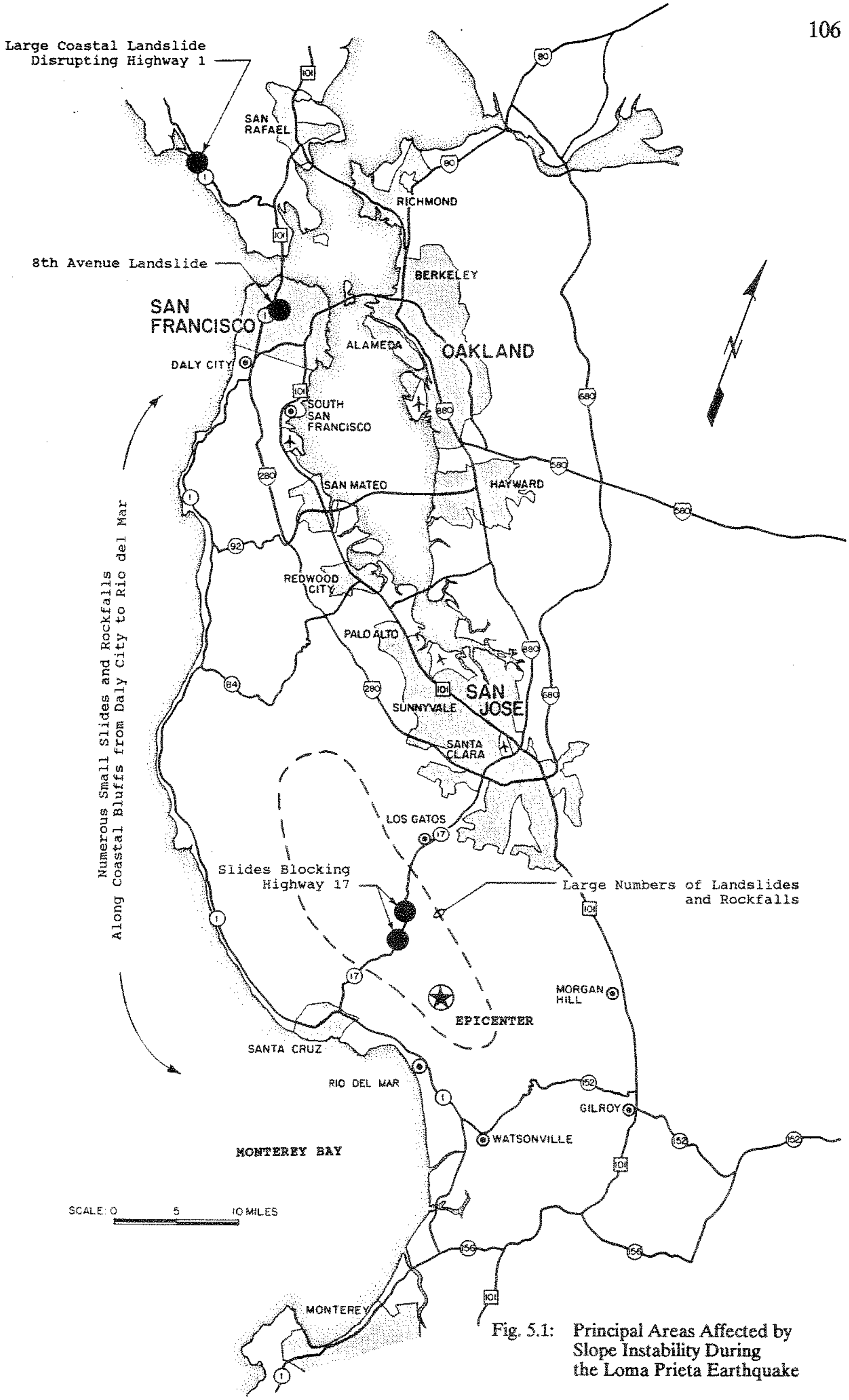


Fig. 5.1: Principal Areas Affected by Slope Instability During the Loma Prieta Earthquake

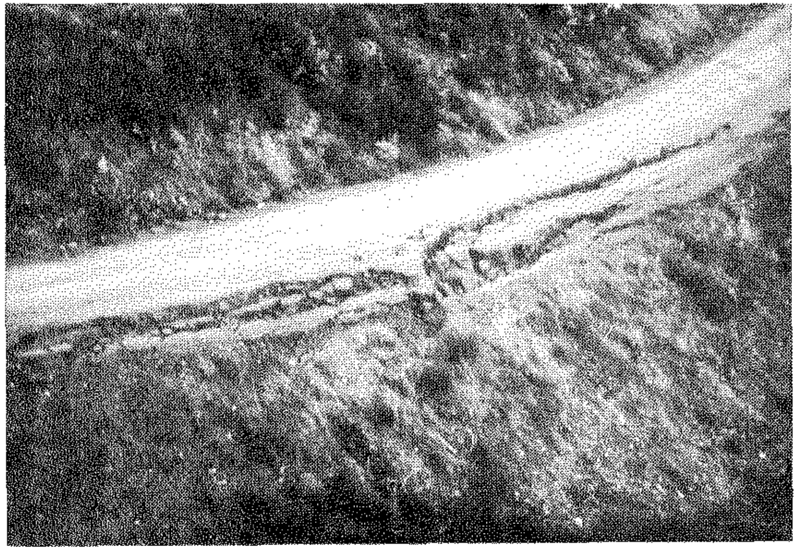


Fig. 5.2: Slumping of Fill Along Summit Road in the Santa Cruz Mountains [Photo courtesy of the U.S. Army Corps of Engineers Waterways Experiment Station.]

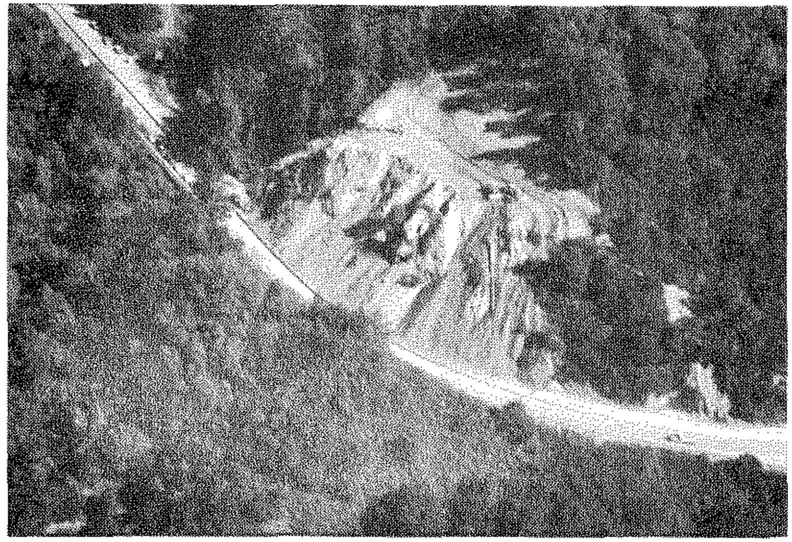


Fig. 5.3: Repair of Landslide on Highway 17 in the Santa Cruz Mountains

Fig. 5.4: Head Scarp of Partially Developed Slide on Summit Road in the Santa Cruz Mountains

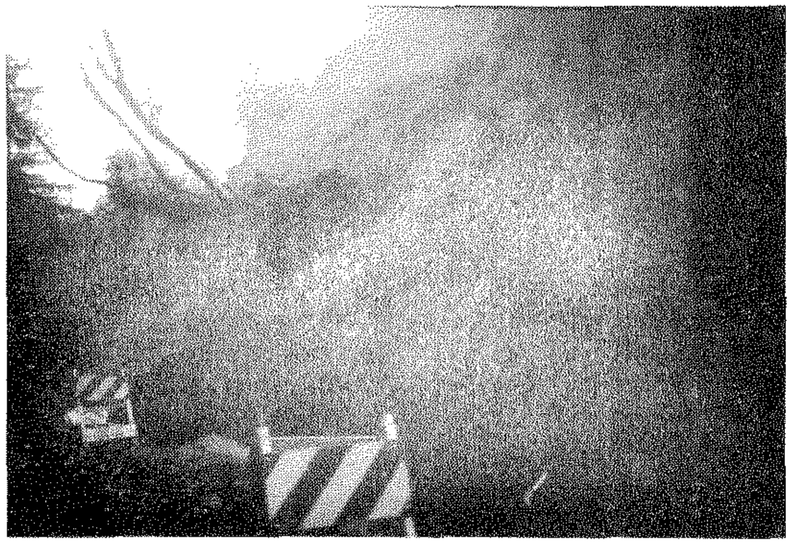


Fig. 5.5: Toe of a Landslide Encroaching on Soquel-San Jose Roadway in the Santa Cruz Mountains [Photo courtesy of James R. Martin II]

limit, to some extent, the number of structures damaged by sliding and rockfalls. In all, approximately 500 to 800 structures, mostly single family residences, were heavily damaged or destroyed as a result of sliding and general stability problems in this region; roughly one-half of these were damaged by conventional landslides, and the remainder by slumping and settlement of minor fills and/or by ground fissures apparently associated (in most cases) with near instability of slopes. No lives were lost as a direct result of landslides in this area.

The most damaging single slide in this region was the Goebel Court or Redwood Estates landslide. This is a reactivated, ancient landslide with a surface area of approximately 25 acres. The slide plane, located by means of borings and inclinometers, occurs at an average depth of approximately 50 feet, with a maximum depth of 96 feet at one location. A total of 19 homes have been damaged or significantly adversely affected by this slide. The slide occurs largely within interbedded Oligocene and Miocene sandstones and shales of the Vaqueros formation. Although post-earthquake investigations revealed clear evidence of historical slide movements, there is no record of movements of this slide mass having occurred during the severe winters of 1982 and 1983.

In addition to the widespread occurrence of landslides and rock falls in the Santa Cruz Mountains, numerous slope failures occurred along the Pacific coastal bluffs from about Rio Del Mar, just southeast of Capitola, to as far north as Daly City (as indicated in Figure 5.1). The materials forming these cliffs range from relatively well indurated siltstones and sandstones to rather weakly indurated cemented marine terrace sands and gravels. As a result, the character of the failures and the extent of damage was quite variable.

The more indurated rocks generally belong to the Purissima Formation and typically form very steep cliffs as high as 150 feet. These cliffs are found along most of the coastline from Capitola northward to Half Moon Bay. The failures which occurred during the earthquake along this section of the coast were typically rock falls and topples involving relatively small volumes of material. Figure 5.6 shows one such failure, in this case a small rock fall which further undermined an already exposed foundation of an apartment building in Capitola. It is important to note that this foundation has been partially exposed for some time and that previous failures had occurred here during the winter storms of 1982 and 1983.

Several larger falls involving a few tens of cubic yards also occurred. One of these larger failures killed one person on a beach just north of Santa Cruz. Evidence of failures in the form of small talus cones at the bases of the cliffs was ubiquitous along almost the entire length of the coastline between Santa Cruz and Half Moon Bay. In this respect, while the character of the failures was the same as is typical of failures caused along this same stretch of the coast by winter rains or wave erosion, the failures due to the Loma Prieta Earthquake were much more pervasive than those which typically occur during winter storm seasons, and were more pervasive than those which occurred during the unusually severe winters of 1982 and 1983. Also, newly opened tension cracks have been reported at numerous locations along

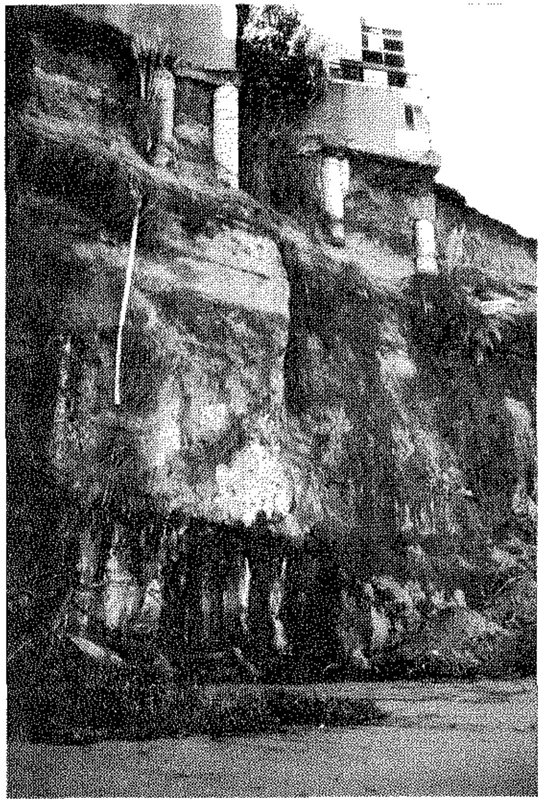


Fig. 5.6: **Apartment Building Partially Undermined by a Small Coastal Bluff Failure Near Capitola**

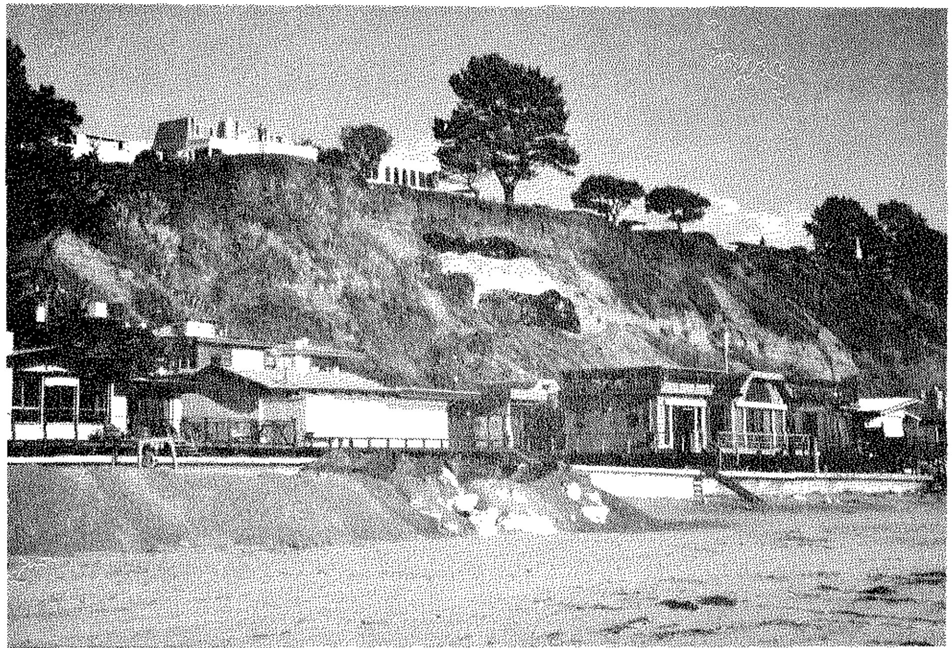


Fig. 5.7: Typical Development Along the Crests and Bases of Coastal Bluffs Between Capitola and Rio Del Mar

Fig. 5.8: Coastal Bluff Failure at Daly City

these coastal bluffs, suggesting heightened risk of occasional rock falls and increased likelihood of further failures during future winter rains.

The weakly indurated Tertiary sandstones and siltstones, and the Quaternary terrace deposits, are exposed along the bluffs in Daly City and Pacifica at the far north end of the affected coastline, and east of Capitola, between Capitola and Rio Del Mar. The failures which occurred in these materials were typically shallow slides, involving mainly the weathered horizon along the slope face, and occasional small block falls and topples. The coastline between Capitola and Rio Del Mar is quite heavily developed with structures both at the crests and at the bases of the slopes (e.g. Figure 5.7). Although widespread evidence of isolated failures of various aerial extent was present throughout much of this region, the damage to individual structures was relatively limited. In most cases the damage occurred to the structures at the base of the slopes as the falling soil overtopped garden walls, impacted walls, and partially buried some of the properties. The damage at the top of the slopes consisted mainly of loss of ground and tensile cracking, in some cases as far back as 30 feet from the crest of the slope (Plant and Griggs, 1990). In several locations the bluffs formed narrow ridges which appear to have amplified the ground motion and which were shattered; 3 houses located on such ridges were sufficiently heavily damaged as to require removal (Plant and Griggs, 1990).

The largest failure in these coastal bluffs occurred in Daly City, just north of the San Andreas Fault (Figure 5.8). The bluffs along this section of the coast vary in height from about 300 to 500 feet, and failures have previously occurred in this area during the 1906 and 1957 earthquakes. This failure is also notable because it occurred at a significant distance from the epicenter, about 55 miles, and the estimated peak horizontal accelerations were relatively low, on the order of 0.10 to 0.14 g. The implications regarding the potential for failures in future earthquakes along this section of the coast are clear, and deserve careful consideration in future planning.

In addition to sliding and rockfalls in the epicentral region and along the Pacific coast between Rio Del Mar and Daly City, isolated slides and rockfalls occurred over large surrounding areas. Many of these were very small, but two of the most damaging slope stability problems caused by the Loma Prieta Earthquake occurred (a) in central San Francisco, and (b) even farther north on the west coast of the Marin Peninsula.

The first of these problems was the 8th Avenue landslide, which occurred near the center of the City of San Francisco, as shown in Figure 5.1. Eighth Avenue in San Francisco runs in a north to south direction along the top of a very steep east-facing slope, with a face slope that varies from 1:1 to 2:1 (horizontal:vertical). Figure 5.9 shows an oblique view down this steep slope from the rear porch of a home on the crest of the slope. The slope height increases from about 70 ft near the intersection of Eighth Avenue and Moraga Street to about 110 ft near the intersection of Eighth Avenue and Ortega Street. Eighth Avenue is aligned with the eastern crest of a large sand dune, and the slope which underlies the roads and houses in this area is

Fig. 5.9: View Downslope Showing the Steepness of the Slope Face at Eighth Avenue near Noriega Street, San Francisco

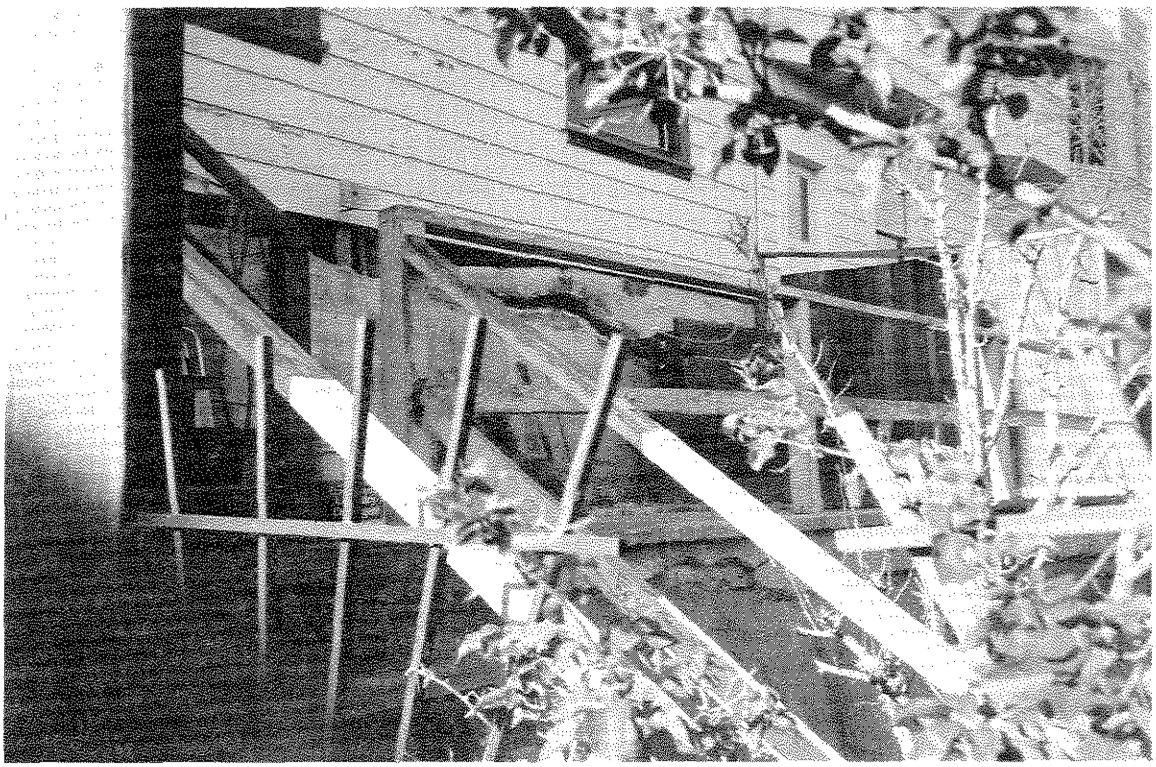


Fig. 5.10: Home With Foundation Sheared-Through by Eighth Avenue Slide

comprised of loose, and in some zones weakly cemented, dune sands. Evidences of slope instability have been reported since houses were first constructed along the east side of the street about 50 years ago.

The ground shaking caused by the Loma Prieta Earthquake induced a sudden slip within the sand slope that resulted in large differential settlements between the front and rear of the houses on the east side of the street. More than thirty houses were destroyed or will require major repairs. An example is the house shown in Figure 5.10 whose foundation was sheared through by the slope displacements. There is no evidence that soil liquefaction played a part in this failure, or that any liquefaction occurred at this site. Instead, the failure appears to have occurred within unsaturated sandy soils, and to have been simply the result of application of modest dynamic (seismic) forces to a slope that had been only marginally stable under static conditions. Observations have shown that there have been no significant additional movements since the earthquake. Nonetheless, strengthening of this large and marginally stable slope against further movements in the future will be needed.

The second major slope stability problem which occurred far from the Loma Prieta Earthquake fault rupture zone was the apparent reactivation (or at least acceleration) of a major slide on the western coast of the Marin Peninsula. As shown in Figure 5.1, this large landslide is located along State Highway 1 between Muir Beach and Stinson State Beach, approximately eight miles northwest of the Golden Gate Bridge, roughly 70 miles north of the epicenter and more than 55 miles north of the nearest point on the fault rupture surface. The highway at this location skirts the coastline in an area of high relief and is benched into highly weathered and sheared sandstone and shale of the Franciscan formation. The slide mass covers an area extending roughly 1000 feet horizontally and 1000 feet in vertical relief. The slide plane, located with slope indicators, is roughly 100 feet beneath the surface. This slide has experienced various levels of activity for many years, and resumed movement at an accelerated rate following the Loma Prieta Earthquake. The recent episode of movement has severely damaged a portion of Highway 1, resulting in closure for an indefinite period of time.

The marginal stability of the Highway 1 slide has resulted in two fairly recent episodes of excessive displacements; movement following the severe storms in the winter of 1982 and the present activity subsequent to the October 17 earthquake. The 1982 storms reactivated portions of the slide mass and resulted in the closing of the highway for several months. At that time three distinct sections of the slide mass were distinguished; a northern, central, and southern section. The northern section experienced the most displacement following the 1982 storms. Remedial measures consisting of shallow cutting and grading, and the installation of horizontal drains were required to ensure adequate stability for the repaired roadway.

In the period between 1982 and 1989 the slide exhibited continuous movement at "manageable" rates on the order of one foot or less per year. The rate of movement increased substantially immediately after the October 17 event. A suite of slope indicators was installed in the weeks following the earthquake to monitor subsequent movement. Approximately two to three weeks after the earthquake a 400 to 500 foot section of the highway was declared unuseable and closed pending repairs. The central portion of the slide mass has reactivated and is continuing to move; various portions of the slide mass have moved between four to eight feet in the eight week period between mid-January and mid-March, 1990. The marginal initial post-earthquake stability and resulting displacements may have been exacerbated by several small to moderate storms which have resulted in total precipitation of approximately 15 inches. CALTRANS engineers have estimated that approximately 600,000 cu yds of material will have to be excavated in order to reroute the highway to a more stable bench above the present roadway, and it is expected that the highway will remain closed until Autumn of 1990 as this excavation and re-routing of the highway proceed.

In summary, landslides and rock falls occurred over a large area during the Loma Prieta Earthquake. More than 1,000 slides and rock falls occurred in the mountainous epicentral region, and these disrupted roads and utilities and destroyed or heavily damaged approximately 500 to 800 homes and small businesses. Additional slides and rockfalls occurred along the Pacific coast, including the apparent reactivation of a major coastal slide mass resulting in the ongoing closure of a section of Highway 1 on the west coast of Marin. Additional slides and rockfalls occurred to the north, south and east of the epicentral region, and the most damaging of these was the 8th Avenue slide which damaged more than 30 homes in central San Francisco.

Overall, the principal impacts of slope instability were two-fold: (1) slides and rockfalls damaged and destroyed homes and businesses, and (2) slides disrupted major transportation arteries. Although an unfortunately large number of structures were destroyed or damaged by sliding, this number was limited to some degree by the sparse development and population of the mountainous epicentral region. Similarly, although the temporary closure of Highway 17 and other access routes from the south San Francisco Bay Area to Santa Cruz occurred at a critical time (immediately after the earthquake when emergency services and supplies were urgently needed), and despite the significant costs and potential damage to local economies occasioned by the reactivation of a major slide disrupting Highway 1 on the west coast of Marin, significantly more disruption of transportation systems was caused by the closures of the San Francisco-Oakland Bay Bridge and of major highway segments in both San Francisco and Oakland, than by slope instability.

This does not, however, mean that earthquake-induced slope instability does not pose a major threat to the greater San Francisco Bay Area. The Bay Area basin and the Pacific coast remain areas of considerable topographic relief, with innumerable structures and facilities located on hillsides and coastal bluffs, and the major fault systems responsible for much of the topographic relief remain capable of producing large earthquakes likely to cause widespread slope instability and resulting damage and destruction of structures and facilities, as well as closure of roads and disruption of vital utilities. Relatively few of the well over 1,000 landslides and rockfalls induced by the Loma Prieta Earthquake resulted in significant damage to structures or disruption of major roads and utilities. This would not have been the case if the earthquake had occurred on any of the several major Bay Area fault segments centered in or near to the major urban areas located to the north on the San Francisco Peninsula and in the eastern San Francisco Bay Area.

Chapter Six: PERFORMANCE OF EARTH AND ROCKFILL DAMS

6.1 Introduction:

A large number of earth and rockfill dams were strongly shaken by the Loma Prieta earthquake of October 17, 1989. In fact, more than 100 dams of various compositions and geometries were located within 60 miles of the epicenter of the main shock. Many of these were relatively small embankments, but a number of major dams were also strongly shaken. No dams failed, and no dams demonstrated signs of potential major instability such as might precipitate a reservoir release. In addition, the risk to the public was further reduced by the occurrence of the earthquake at a time when most of the reservoirs were at unusually low levels as a result of local drought the previous year and the late Autumn timing of the earthquake. The behavior of earth and rockfill dams in this earthquake, as documented by field observations of damage and by strong motion recordings, provides an excellent opportunity to investigate the response of earth dams to strong ground shaking.

The behavior of five earth and rockfill dams are of special interest. The locations of these five dams (Austrian Dam, Lexington Dam, Guadalupe Dam, Anderson Dam, and San Justo Dam) are shown on the map of the South San Francisco Bay Area in Figure 6.1. These five dams are of particular interest because of the relative extent of damage observed at these dams and/or because of the earthquake engineering research value of the dams' performances and strong motion instrument recordings made at these dams during the 1989 Loma Prieta earthquake.

6.2 Austrian Dam:

Figure 6.2 shows a plan view and maximum height cross section of Austrian Dam, which impounds Lake Elsman. Austrian Dam is a 185-foot high rolled earth fill dam built around 1950. The dam has a concrete spillway channel located near its right abutment. Austrian Dam is located approximately 7½ miles from the epicenter of the main shock of the 1989 Loma Prieta earthquake, but is in very close proximity to the northern section of the Loma Prieta Earthquake fault rupture zone.

Austrian Dam is situated between the San Andreas Fault System and the Sargent Fault. The dam is only about 4,000 feet south of the main intersection of these fault zones, with the trace of the 1906 movement on the San Andreas Fault located only 1700 feet west of the dam and the Sargent Fault located less than 700 feet east of the dam. It would appear reasonable to expect that a major earthquake might produce some amount of tectonic movement on subsidiary fault features or on other planes of weakness in the foundation rock materials. In fact, the damage to the upper spillway section and the earth fissures along the right side of the reservoir appear to form a linear feature across the dam site (see Figure 6.3). The earth fissures appear to resemble a landslide head scarp feature nearer to the embankment

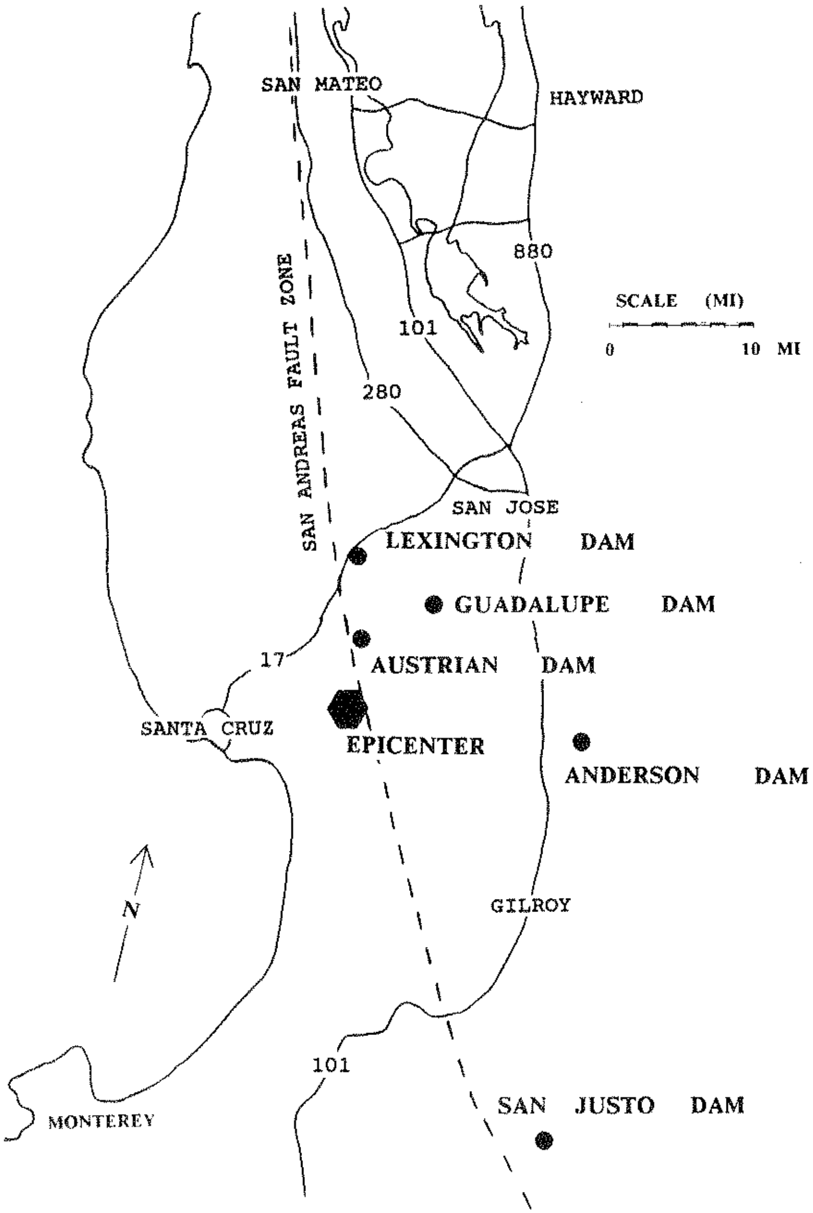
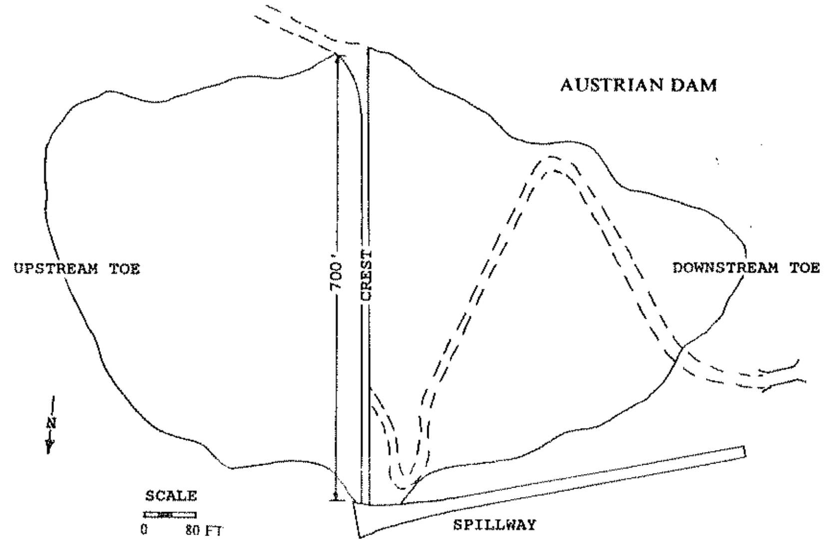
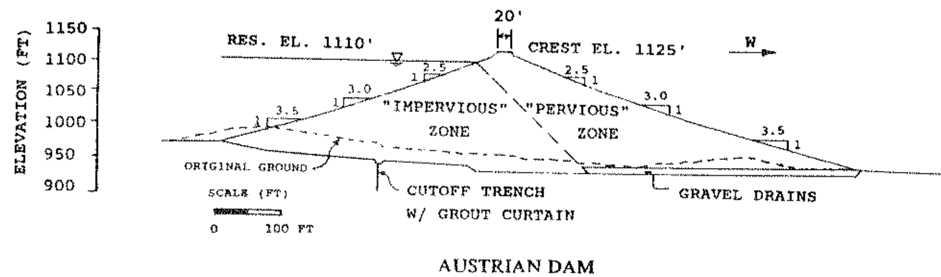


Fig. 6.1: Map Showing Locations of the Five Dams of Principal Interest



(a) Plan View



(b) Maximum Cross Section

Fig. 6.2: Plan View and Cross Section of Austrian Dam

where the reservoir side slopes are relatively steep (See Figure 6.4), but the movement is expressed as a 25 to 50 foot wide zone of multiple ground fissures in less steep ground further from the embankment. Although there is considerable variation along these features, the earth fissures near the embankment appear to display primarily a vertical component of movement with a minor left-lateral horizontal displacement component.

The dam owner's geotechnical consultant has studied this feature in detail and has concluded that the ground movements near the spillway are the result of a large landslide movement in the downstream right abutment, and that the ground fissures along the right side of the reservoir are a result of local slumping in relatively incompetent surficial fill materials. It was thus concluded that the earth fissures and spillway cracking which initially appeared to form a relatively linear feature across the dam site are not the result of an underlying primary fault feature. One of the surprising characteristics of the M_S = 7.1 Loma Prieta earthquake was that the zone of rupture was relatively deep for a strike-slip fault movement in this region of California. In fact, direct evidence of surface faulting as a result of the earthquake has not yet been conclusively identified.

The behavior of the earth embankment itself is also of considerable interest. The maximum cross section through Austrian Dam is shown in Figure 6.2(b). At the time of the earthquake, the water level was roughly 90 feet below the maximum reservoir elevation. The embankment was constructed by selective borrowing near the dam site, and sampling of embankment materials during an earlier study of the dam suggests that there is not an appreciable difference between the upstream "impervious" zone material and the downstream "pervious" zone material. Piezometer readings also indicate that the permeability of the earth materials across the dam cross section does not vary significantly, and that the gravel strip drains are not fully effective. Hence, the dam appears to be nearly homogeneous, being composed of primarily gravelly to sandy clay material.

Strong motion attenuation relationships and strong motion recordings at nearby rock sites suggest that peak horizontal ground accelerations of rock at the Austrian Dam site were probably on the order of 0.5 g. The survey monuments along the 700 foot long crest of the dam were surveyed the day before the earthquake, and a resurvey of these crest monuments just two days after the main shock revealed that relatively significant deformation of the dam's crest occurred (See Figure 6.5). The right crest section appeared to move downstream horizontally 1.5 feet relative to the left crest section. Maximum vertical settlements along the crest occurred near the right abutment and were measured to be on the order of 2.5 to 3 feet.

The strong shaking and ground movements produced extensive longitudinal and transverse cracking along the crest, especially near the spillway section at the right abutment (See Figure 6.6). The transverse earth fissures near the spillway were up to 1 foot wide and up to 10 to 25 feet deep. Considerable fissuring also occurred near the left abutment (See Figure 6.7). This abutment fissuring generally followed an access road which connected the left abutment to the intake structure, which was

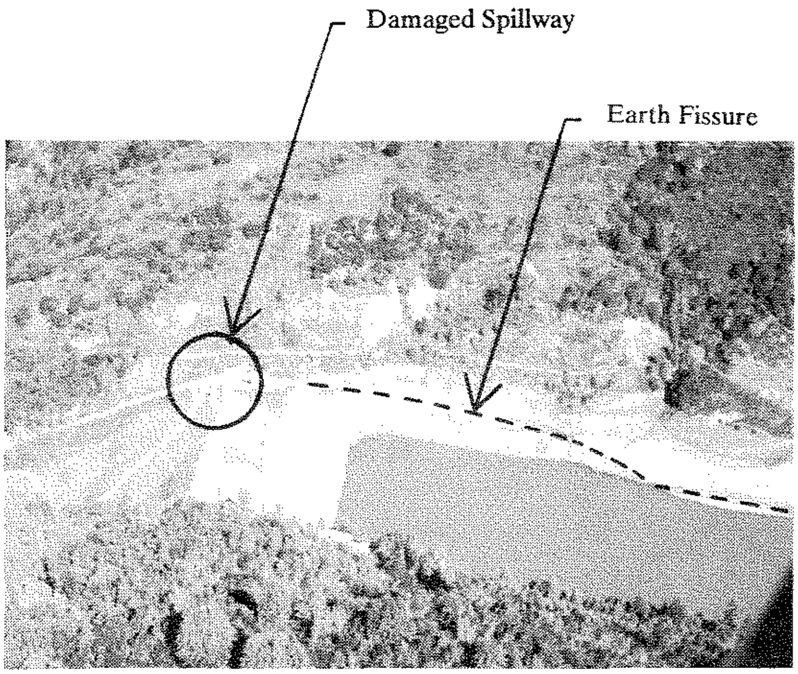
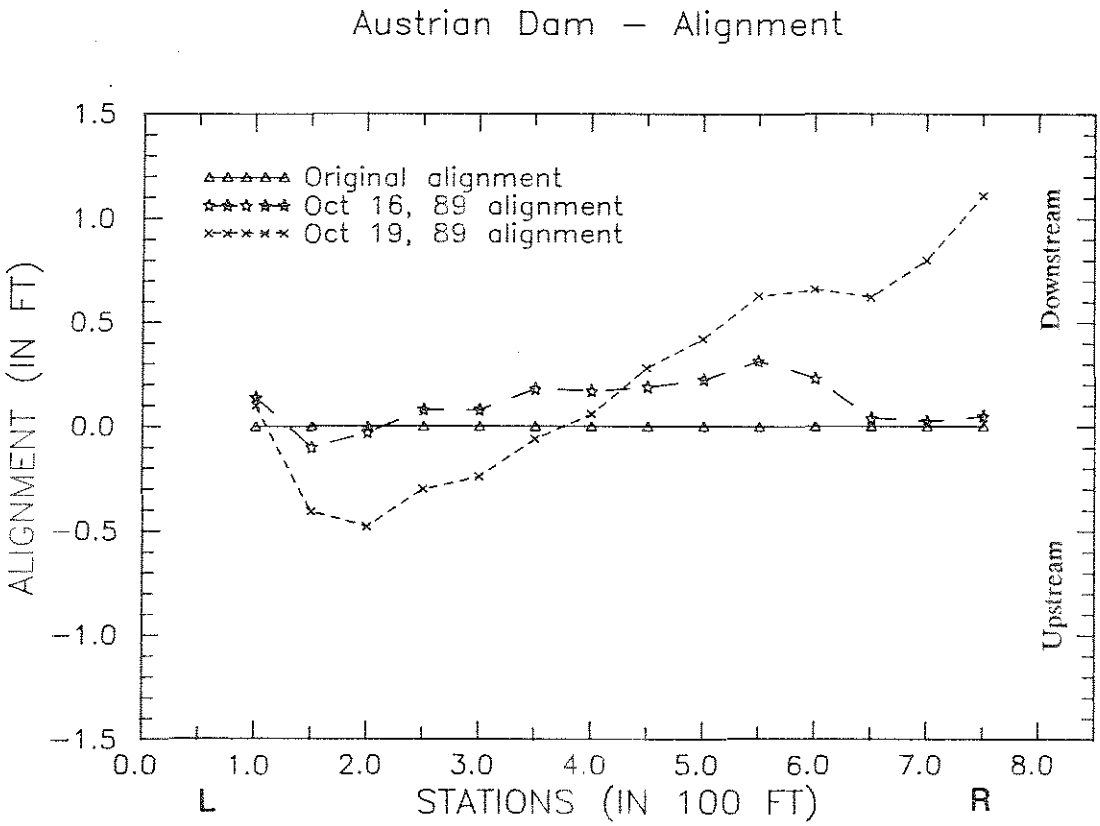
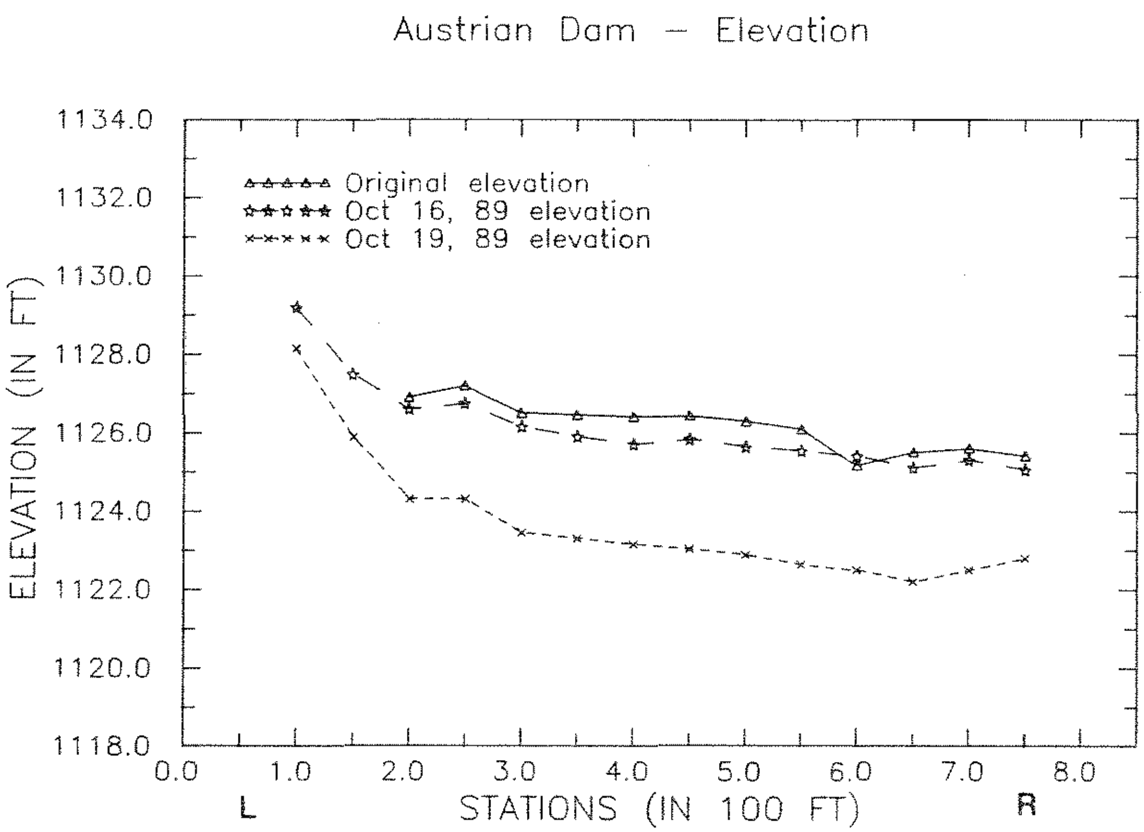


Fig. 6.3: Photograph of Austrian Dam and Lake Elsman Reservoir

Fig. 6.4: Fissure in Right Abutment at Austrian Dam



(a) Alignment Surveys



(b) Elevation Surveys

Fig. 6.5: Crest Movement at Austrian Dam

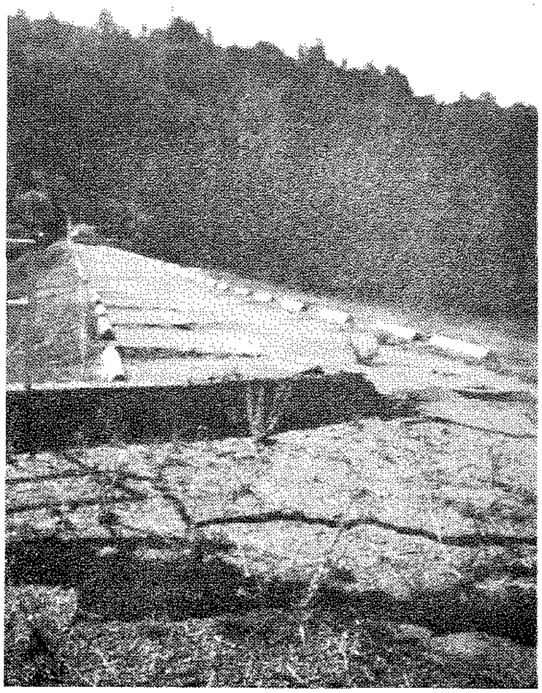


Fig. 6.6: Fissures and Damage Along the Crest of Austrian Dam

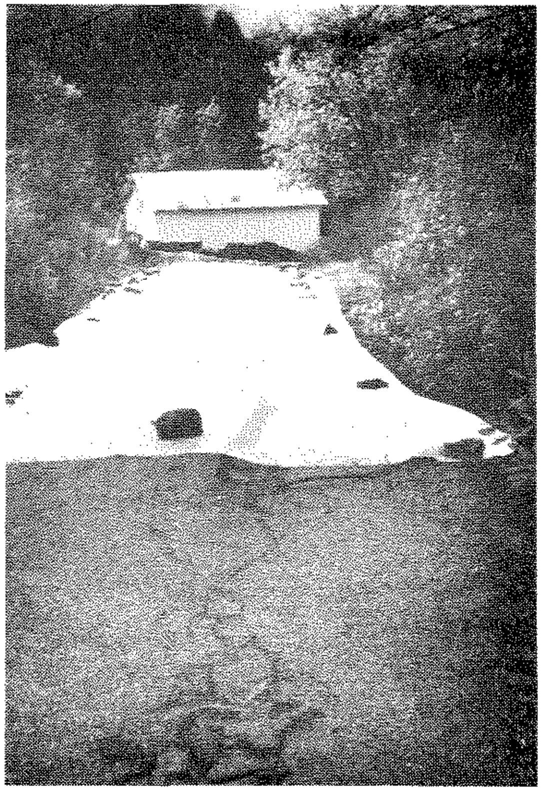


Fig. 6.7: Fissures near Left Abutment at Austrian Dam

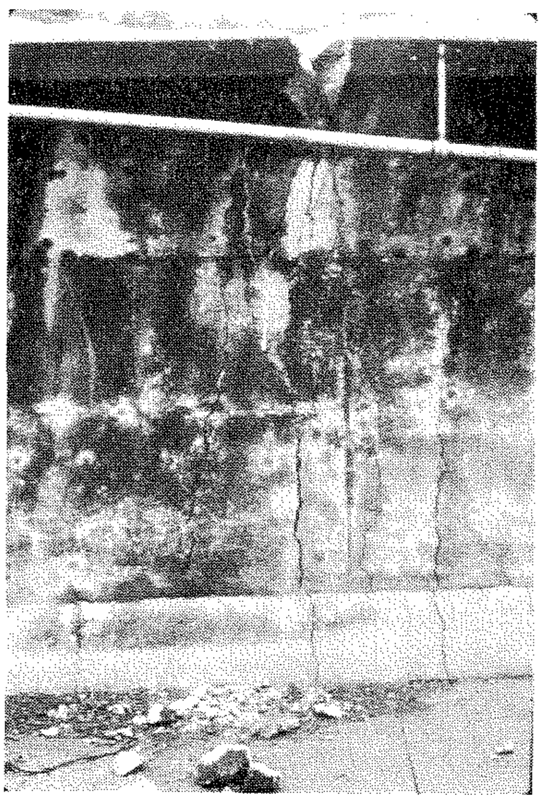


Fig. 6.8: Cracking in Concrete Spillway at Austrian Dam

located on the left side of the reservoir. None of the observed fissures posed any significant threat of reservoir release, as the reservoir level was low.

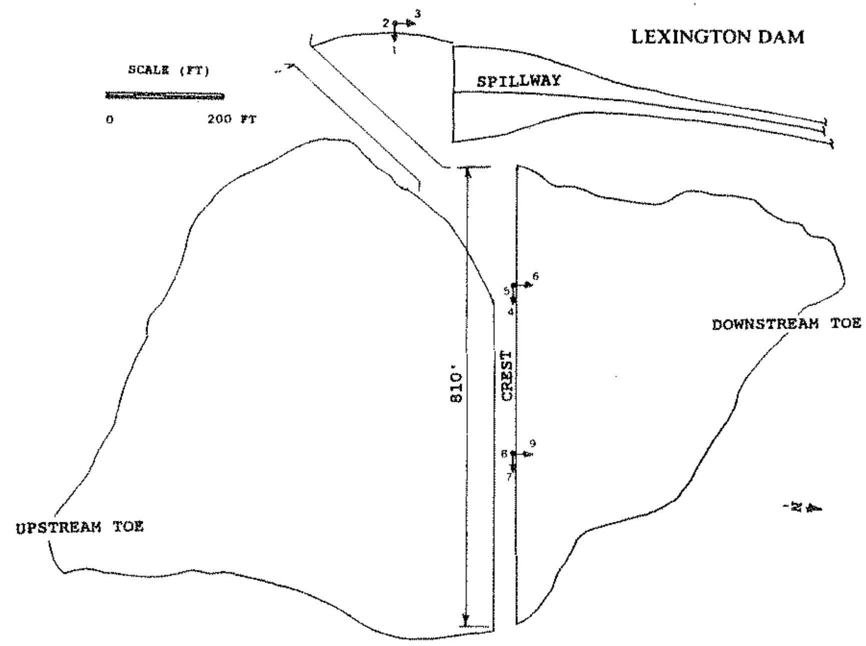
The majority of the localized ground failure and cracking occurred near the spillway section, and these earth movements produced extensive cracking of the upper quarter of the concrete spillway channel, as shown in Figure 6.8. The cracks were oriented primarily perpendicular to the centerline of the spillway section. The cracks were typically 0.25 to 0.5 inches in width, with a fairly regular spacing of 2 to 4 feet. The earth material in front of the mouth of the spillway settled 6 to 12 inches relative to the spillway's inlet elevation, and large ground fissures were observed along the contact between the concrete spillway section and the ground surrounding it. In addition, differential movements between the concrete spillway walls and the surrounding soil caused several of the concrete cutoff walls (positioned to prevent seepage along the exterior walls of the concrete spillway along the plane of concrete/soil contact) to be sheared off.

In addition to the damage to the crest and spillway, a series of roughly parallel longitudinal cracks formed in the upstream and downstream faces of the dam. Four longitudinal cracks which were approximately 5 to 15 feet deep and 1 to 4 inches wide formed in the upper 50 feet of the upstream face. A number of longitudinal cracks which were 3 to 8 feet deep and 2 to 6 inches wide formed in the downstream face. The majority of the longitudinal cracks in the downstream face were located near the crest, although some limited cracking also occurred near the toe of the dam. In addition, there was some minor bulging of the downstream toe. The cracks did not appear to result from slope instability, but rather from settlement and rearrangement of the earth embankment. These cracks continued to open up and other cracks became better defined over a period of 7 to 10 days after the earthquake's main shock. This phenomenon could have resulted from major aftershocks, rainfall during this period, soil creep, and/or pore pressure dissipation and consolidation.

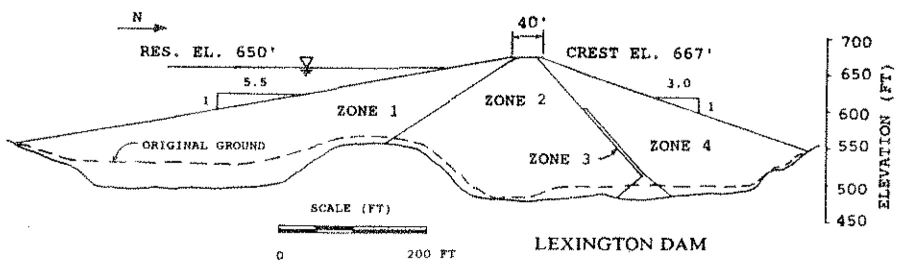
The ground fissures and spillway damage did not appear, however, to pose a significant hazard as the reservoir level was low, and the owner immediately performed temporary repairs to the damaged sections of the dam, and also quickly repaired the spillway. Additional geotechnical studies of the dam are currently in progress to ensure its continued safe performance in future seismic events.

6.3 Lexington Dam:

Lexington Dam is located approximately 6 miles downstream of Austrian Dam on Los Gatos Creek. Lexington Dam (See Figure 6.9) is a 195 foot high rolled earth fill dam built in 1953. Its crest length is 810 feet and it has a concrete spillway over its left abutment. The embankment's relatively thick sandy gravelly clay core is bordered by upstream and downstream shells composed of random materials consisting primarily of clayey sands and silts. The upstream slope is 5.5:1 (H:V) and the downstream slope is 3:1 (H:V). A chimney drain composed of free-draining rock lies between the core and the downstream shell. The reservoir level was approximately 100 feet below the maximum reservoir level at the time of the earthquake.

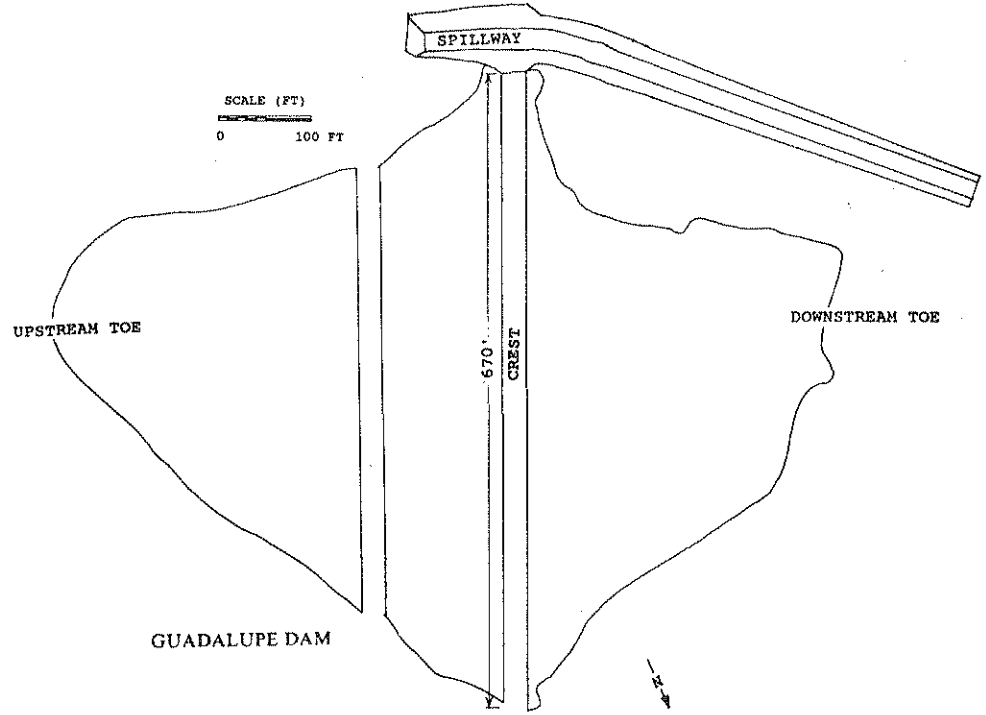


(a) Plan View

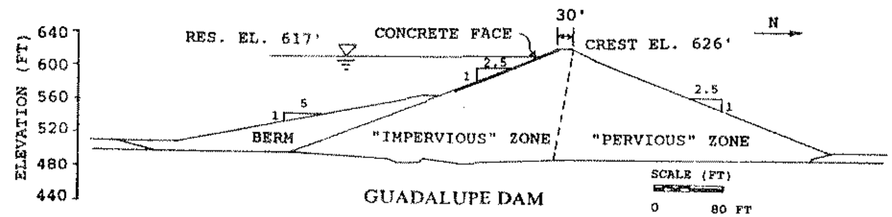


(b) Maximum Cross Section

Fig. 6.9: Plan View and Cross Section of Lexington Dam



(a) Plan View



(b) Maximum Cross Section

Fig. 6.10: Plan View and Cross Section of Guadalupe Dam

As shown in Figure 6.9(a), the dam was instrumented with strong motion sensors at its left abutment, left crest and right crest. These accelerographs recorded transverse (upstream/downstream) horizontal peak ground accelerations on the order of 0.45 g, 0.39 g and 0.45 g, respectively. The left abutment or "bedrock" peak acceleration is within the range predicted by appropriate strong motion attenuation relationships for a site approximately 2 miles from the nearest point on the fault rupture surface for a M_S = 7.1 event, but is a bit lower than the mean or expected value based on such relationships. In addition, there appears to be some spectral acceleration amplification at lower frequencies (0.8 to 1.2 Hz) which may indicate that the recorded "bedrock" motion may have been affected by local topographic or geologic conditions.

The strong ground shaking produced transverse cracking across the crest as well as in the upstream and downstream faces at the right and left abutments. The transverse cracking was fairly isolated and produced narrow cracks typically 3 to 5 feet deep. Minor longitudinal cracking occurred across the downstream face near the crest. The maximum earthquake-induced crest deformations were approximately 0.9 feet of vertical settlement, and 0.25 feet of lateral displacement in the downstream direction. The earthquake shaking and ground movements produced extensive cracking in the bridge abutment at the left abutment of the dam and ruptured a water line which crosses the dam near the crest. There were no indications of potential instability, and the minor cracking will be simply repaired.

6.4 Guadalupe Dam:

Guadalupe Dam is located approximately 11.5 miles from the earthquake's epicenter, and approximately 7 miles from the fault rupture surface, and probably experienced peak ground accelerations on the order of 0.4 g to 0.5 g. The embankment (see Figure 6.10) is 142 feet high with a crest length of 700 feet. The original embankment, which was built in 1935, is a rolled earth fill dam with upstream concrete facing. Similar to Austrian Dam, the embankment is nearly homogeneous as the selective borrowing technique employed to construct the dam did not appear to produce distinct "impervious" and "pervious" zones as designed. The upstream stabilizing berm was constructed in 1972 to improve the embankment's static and seismic slope stability.

The dam suffered minor transverse cracking at each abutment, minor longitudinal cracking along the crest, and a maximum crest settlement of approximately 0.6 feet. Two interesting aspects of the behavior of this dam were the behavior of the concrete facing and the performance of the upstream stabilizing berm. The concrete facing sections appeared to pound together and ride up over each other at joints, producing cracking and spalling of the concrete. In addition, numerous longitudinal cracks were found in the embankment itself near the top of the upstream buttress, and these may have resulted from dynamic stress concentrations resulting from the transition in dam geometry at this location. Alternatively, these cracks may have been caused by past settlements caused by placement of the 1972 buttress fill, which may have produced differential settlements resulting in cracks which surfaced only after the strong shaking produced by the Loma Prieta Earthquake. The berm may have been instrumental, however, in limiting the damage to the crest of the dam. Overall, the performance of this dam appears to have been good.

6.5 Anderson Dam:

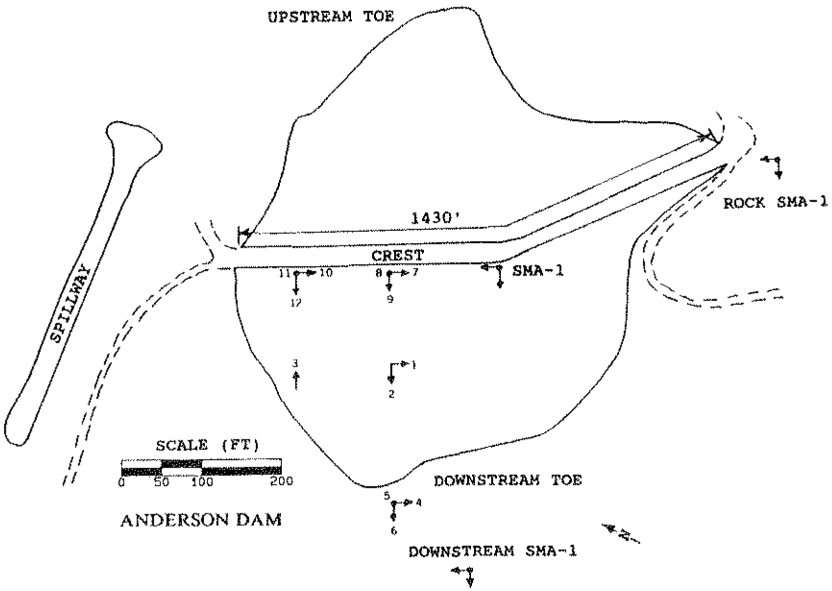
Anderson Dam (see Figure 6.11) is a 240 foot high, 1,430 foot long dam with a central compacted gravelly, clayey sand core, and dumped and then sluiced rockfill shells. Both dam faces slope at 2.5:1 (H:V). The dam is well-instrumented as shown in Figure 6.11, and offers an exceptional opportunity for study because it was excited by the 1984 Morgan Hill earthquake as well as the 1989 Loma Prieta earthquake. The dam's epicentral distances in these two events were approximately 10 miles and 16 miles, respectively. The 1984 Morgan Hill earthquake generated transverse peak ground accelerations at the crest and near the downstream toe of 0.63 g and 0.41 g, respectively. On the other hand, the 1989 Loma Prieta earthquake generated peak transverse ground accelerations of 0.43 g at the crest and 0.23 g near the downstream toe. The downstream sensor is located on an alluvial foundation. The left abutment "bedrock" sensor recorded a transverse peak ground acceleration of 0.07 g during the 1989 Loma Prieta earthquake. In each event, the duration of strong shaking was on the order of 8 to 12 seconds.

Both the 1984 and 1989 earthquakes produced similar patterns of distress in the embankment. Extensive, but shallow, longitudinal cracks formed in the compacted fill along the crest approximately overlying the contacts between the shells and the core of the dam. Between 1984 and 1989, the embankment crest was raised about 3 to 5 feet and a small sliver fill was placed in a localized area on the downstream edge of the crest. This sliver fill was placed to provide space for an instrumentation vault. Many of the larger cracks which developed following the 1989 Loma Prieta event were concentrated around this instrumentation vault. Another set of cracks ran longitudinally along the edges of both sides of the crest at the base of the guard rail posts.

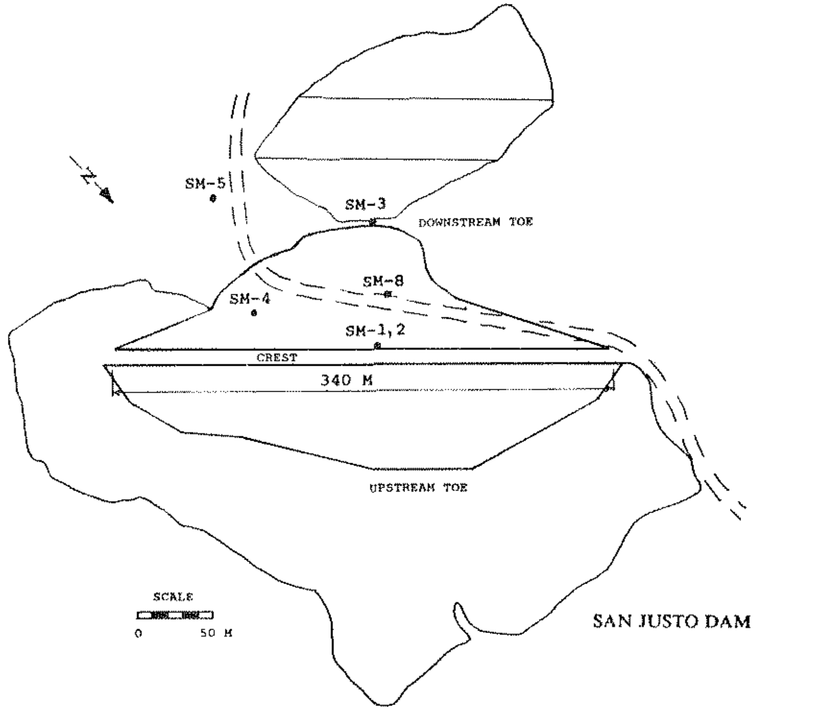
The maximum crest settlement following the Loma Prieta earthquake was about 0.5 inches on the upstream edge and about 1.8 inches on the downstream edge. The maximum width of the cracks were approximately 1.5 inches along the downstream edge of the crest and about 0.75 inches near the instrumentation vault. These cracks appear to pose no threat to the stability of the embankment, and the overall performance of the dam thus appears to have been very good.

6.6 San Justo Dam:

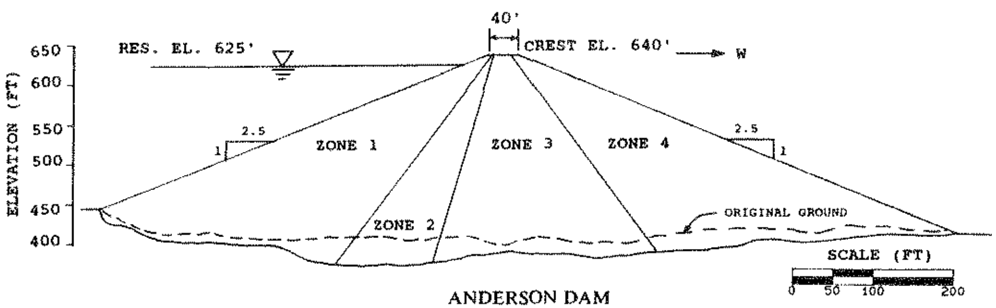
San Justo Dam is a relatively new dam completed in 1986. It is a zoned earth and rockfill dam of modern design (See Figure 6.12). The dam is 135 feet high with a crest length of 1,115 feet. It is located approximately 30 miles from the epicenter of the main shock.



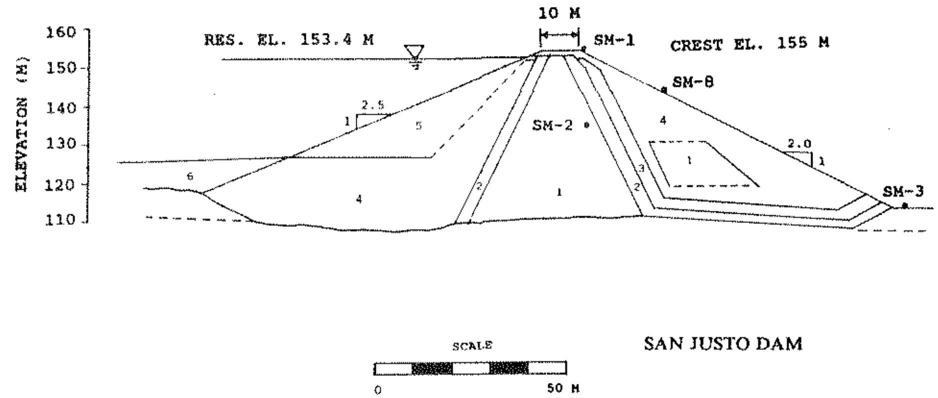
(a) Plan View



(a) Plan View



(b) Maximum Cross Section



(b) Maximum Cross Section

Fig. 6.11: Plan View and Cross Section of Anderson Dam

Fig. 6.12: Plan View and Cross Section of San Justo Dam

The dam showed no indications of significant distress, but it affords an excellent opportunity for improving our current level of understanding of the performance of earth dams excited by major earthquake events because it is well-instrumented. As shown in Figure 6.12(b), the San Justo Dam was built with a strong motion sensor embedded within the embankment at a depth of 62 feet below the crest (SM-2). Sensors are also located on the mid-downstream face, as well as at the crest and downstream toe. The downstream toe sensor is sited on unindurated, compacted sediments. In addition, a subsidiary dike, which is about half the height of the main dam (approximately 70 feet high), is also instrumented with strong motion sensors at its crest and left abutment.

The transverse peak ground acceleration recorded near the downstream toe of the main dam was on the order of 0.26 g; whereas, the peak accelerations at the crest and mid-downstream face were 0.50 g and 0.35 g, respectively. This variation in recorded transverse peak ground accelerations represents an amplification factor between the mid-downstream face and the downstream toe of 1.5 to 1, and an amplification factor between the crest and the downstream toe of 2 to 1. The transverse peak ground acceleration within the embankment at a point about 62 feet below the crest elevation was on the order of 0.27 g, and so represents approximately one-half of the observed peak crest acceleration. The peak acceleration recorded at the crest of the subsidiary dike was 0.29 g. The strong motion sensors sited on the left abutments of the main dam and the subsidiary dike did not operate.

6.7 Other Dams:

A number of other earth and rockfill dams were strongly shaken by the 1989 Loma Prieta earthquake, and a few additional cases will be discussed briefly in this section.

Newell Dam, which impounds Loch Lomond Lake, is a 182 foot high zoned earth fill dam built in 1960. The dam is located approximately 8 miles north of Santa Cruz with an epicentral distance of 12½ miles. The dam's crest length is 750 feet. There is a concrete-lined spillway on the left abutment. The earthquake shaking produced a 300 foot long longitudinal crack in the upstream face of Newell Dam. The crack was roughly 10 to 15 feet below the crest of the dam and varied from 1 to 4 inches in width and 1 to 3 feet in depth. A number of zones of earth fissures and shallow landslides formed in the reservoir sides.

Chesbro Dam is a 79 foot high, 720 foot long earthfill dam located about 13 miles from the epicenter of the main shock. It was built in 1955. Longitudinal cracking was observed along the upstream edge of the crest near the location of a cut previously made to repair the upstream slope. The main crack was about 200 feet long, with a maximum width of 4 inches and a maximum vertical offset of 4 inches. The maximum crest settlement was nearly 0.4 feet.

Vasona Dam is a 34 foot high, 1,000 foot long earth embankment dam located on Los Gatos Creek. Its epicentral distance was approximately 16 miles. Extensive, but shallow, longitudinal cracking occurred along the crest of the embankment at locations similar to those observed after a M_L = 5.1 earthquake (probably a foreshock of the main event) on August 8, 1989.

Calaveras Dam is located approximately 29 miles from the epicenter of the main shock, northeast of the city of San Jose. Horizontal peak ground accelerations on the order of 0.08 to 0.13 g were recorded at the dam site. Calaveras Dam is of interest because it is a hydraulic fill dam. The dam, however, exhibited no signs of significant distress.

6.8 Summary:

A number of earth and rockfill dams were strongly shaken by the 1989 Loma Prieta earthquake, and, in general, the dams performed satisfactorily. No dams failed, and no dams demonstrated signs of potential major instability such as might precipitate an uncontrolled reservoir release. In addition, risk to the public was further reduced by the occurrence of the earthquake at a time when reservoirs were low.

Although no dams failed, very strong levels of shaking near the zone of fault rupture did induce minor damage in a number of dams, including Lexington Dam, Guadaloupe Dam, Anderson Dam, Newell Dam (at Loch Lomond Reservoir), Chesbro Dam, and Vasona Dam, and moderate damage in Austrian Dam (at Lake Elsman). This "damage" typically consisted of relatively shallow longitudinal cracks along the crests and/or upper portions of the dam faces, shallow transverse cracking at or near the abutments, and in some cases minor crest subsidence. Preliminary studies suggest that this damage poses no significant risk to the overall stability of these embankments.

Engineers and the public should be encouraged by the general good performance of major dams in the areas where strong earthquake shaking was induced. These encouraging results, however, should be tempered by the recognition that most of the reservoirs near the center of strong shaking were very low at the time of the earthquake. Consequently, major portions of the dam embankments were not as saturated as they would have been during full reservoir conditions. Thus, the 1989 Loma Prieta Earthquake may not have been the full test of these structures that might first be inferred.

The five earth and rockfill dams highlighted in this chapter experienced relatively high peak ground accelerations (e.g. $(a_{max})_{crest} \approx 0.4$ g - 0.5 g). For the instrumented dams studied, the amplification of the transverse horizontal peak ground acceleration through the embankment sections was on the order of 1 to 2. Overall, the documented performance of the earth and rockfill dams excited by the 1989 Loma Prieta earthquake provides a valuable opportunity to improve our current level of understanding of the behavior of earth dams during earthquake strong shaking, and to refine and verify the analytical techniques used to predict and evaluate the dynamic response and performance of earth and rockfill dams.

Chapter Seven: SUMMARY AND CONCLUSIONS

7.1 General:

The Loma Prieta Earthquake of October 17, 1989 was the single most costly natural disaster in U.S. history. Current estimates of damages directly attributable to this earthquake are on the order of \$6 billion to \$10 billion dollars. Sixty two fatalities have been attributed to this earthquake, and thousands of people were either injured or made homeless.

Although these numbers do represent a major tragedy, it is important to recognize that a number of important factors combined to limit both deaths and injuries, as well as economic losses, so that the Loma Prieta Earthquake did not represent a "test" of the ability of the greater San Francisco Bay Area's population and infrastructure to withstand a major near-field seismic event. The most significant of these factors were:

(1) The location of the fault rupture zone: Located in the sparsely populated Santa Cruz Mountains, well to the south of the much more heavily populated San Francisco Bay area, this fault rupture produced considerably lower levels of shaking throughout the populous Bay Area than would have been produced by a rupture of similar or larger magnitude on a more northernly segment of the San Andreas or Hayward Faults.

(2) The unusually short duration of this M_S = 7.1 event: As a result of the medial location of the epicenter within the fault rupture zone, and the resulting symmetrically bifurcated fault rupture mechanism, this event produced only approximately 8 to 10 seconds of strong shaking throughout most of the affected regions, rather than the roughly 15 to 30 seconds of strong shaking more typically associated with events of this magnitude. This short duration, or limited number of loading cycles, greatly reduced the structural damage potential as well as the degree or severity of soil liquefaction and other ground failures.

Several major fault systems (e.g. the San Andreas, Hayward and Calaveras fault systems) are potentially capable of producing seismic events of similar or larger magnitude in closer proximity to major population centers in the San Francisco Bay Area. Such events would produce considerably stronger levels of shaking, and shaking of longer duration, in these areas, and would result in considerably larger loss of life as well as increased injuries and financial losses.

Indeed, the relatively large levels of damage wrought by the Loma Prieta Earthquake serve to demonstrate the apparent vulnerability of portions of the Bay

Area's population and infrastructure to major seismic events. Steps have been taken to reduce this vulnerability over the past 25 years, including progressive improvements in seismic provisions of building codes, improvements in microzonation and land use planning, and seismic retrofit of a limited number of structures and facilities. It is clear from the performance of structures and facilities during the Loma Prieta Earthquake, however, that much remains to be done.

7.2 Site Effects:

A number of geotechnical factors exerted a strong influence on damage patterns and loss of life during the Loma Prieta Earthquake. Near to the zone of fault rupture, many structures were simply overwhelmed by high inertial forces. Well over half of the damages, however, and more than 80 percent of the loss of life, occurred at sites in the north-central San Francisco Bay Area, far from the epicentral region. This concentration of damage on a few relatively distinct sites comprising less than one percent of the "strongly" shaken region was due primarily to the local soil conditions at these sites. These concentrated damages occurred at sites underlain by deep, and primarily cohesive, soil deposits which served to amplify the relatively moderate levels of "bedrock" shaking generated by the earthquake in this region, producing significantly stronger levels of surface shaking. Peak accelerations on rock in the San Francisco, Treasure Island, Oakland, Alameda and Emeryville region appear to have been on the order of 0.06 to 0.12 g. Instrumental recordings, as well as dynamic response analyses, show that many of the bayshore soil deposits in this region amplified these levels of shaking by factors of about 2 to 3, producing peak ground surface accelerations at deep alluvial sites on the order of 0.16 to 0.33 g in this region. In addition, amplification of the longer period components of shaking was especially pronounced, so that the resulting surface motions were particularly damaging to taller, longer period structures.

This type of pronounced, site specific amplification (and spectral amplification, or resonant soil-structure interaction) of ground motions was not a surprise to the earthquake engineering community. Similar site-specific amplification has been noted as an important factor controlling damage patterns in numerous previous major earthquakes over the past 30 years. Building code provisions dealing with these effects have gradually evolved over the past 20 years, and a particularly important improvement in these provisions occurred in 1988 as a result of the clearly overwhelming influence of local site effects on the catastrophic damages suffered by major buildings on deep clay sites during the 1985 Mexico City Earthquake (1988 Uniform Building Code). It may be anticipated that further improvements in the ways that the effects local geotechnical site conditions are dealt with in seismic building codes will result from the lessons learned yet again in this regard during the Loma Prieta Earthquake.

It is important to note, however, that the resulting improved codes are likely to apply only to new structures and facilities, and so will do little to improve the levels of safety of existing facilities and structures, and of their occupants. Among the important lessons to be learned from the Loma Prieta Earthquake must be the recognition of the unacceptably high level of vulnerability associated with many existing structures and facilities. Proper structural evaluation, with appropriate consideration of the influence of local ground conditions on the nature and severity of potential shaking in future events, can correctly identify the structures and facilities at greatest risk. As a society, however, we have a poor history with respect to implementing the often difficult measures necessary to reduce this risk to existing structures. In addition, as a profession we have devoted unfortunately little of our earthquake engineering research efforts (to date) to issues associated with seismic retrofit of existing structures and facilities. The importance of improving our ability to perform efficient and reliable seismic re-evaluation and retrofit, and the need for policy makers to mandate the often financially and politically difficult programs necessary to implement such retrofit, are among the most important lessons to be learned from the Loma Prieta Earthquake.

7.3 Soil Liquefaction:

In addition to the pronounced influence of local site effects on ground shaking characteristics and associated damages, another important geotechnical feature of the Loma Prieta earthquake was the widespread occurrence of soil liquefaction which resulted in significant damage to bayshore areas throughout much of the densely populated north-central San Francisco Bay Area, as well as farther south in the Santa Cruz/East Monterey Bay region. Significant liquefaction-induced damages occurred at many sites in these areas, but again it must be noted that the relatively moderate levels and short duration of shaking generated in the populous San Francisco Bay Area by the M_S = 7.1 Loma Prieta Earthquake which was centered well to the south near Santa Cruz, represents a poor "test" of the ability of the Bay Area to withstand the stronger levels and longer durations of shaking likely to occur in future events. Accordingly, the widespread occurrence of generally "slight to moderate" liquefaction over large shoreline areas in this region, as well as the previous poor performance of many of these areas (which suffered massive damages as a result of "severe" liquefaction in the 1906 earthquake), serves as a stark warning of the ongoing hazard exposure associated with potential liquefaction in future events. Large, densely populated areas, as well as important harbor facilities and airports likely to be in demand for emergency transport after a major earthquake, appear to face considerable liquefaction hazard exposure in future seismic events.

Much (if not virtually all) of the liquefaction in the central San Francisco Bay Area, as well as at Santa Cruz and the east Monterey Bay area, had been correctly predicted as likely to occur during moderate to severe earthquake shaking. Moreover, many of the sites at which liquefaction occurred during the Loma Prieta Earthquake had been documented by researchers as sites at which liquefaction had occurred during the 1906 San Francisco Earthquake. [Two notable exceptions were the central San Francisco Marina District hydraulic fill and the 1936 Treasure Island hydraulic fill, both of which post-date the 1906 earthquake]. In retrospect, there appear to be few surprises for the engineering community in terms of sites at which liquefaction occurred. The same cannot be said, however, for the general public who even at this point, five months after the Loma Prieta Earthquake, appear to have little collective understanding of the extent of potential liquefaction likely to occur in future seismic events, and of its potential ramifications.

If there is a single overall lesson to be learned from the occurrence of soil liquefaction during the Loma Prieta Earthquake, it is: (a) that considerable liquefaction-related risk to the population and infrastructure of the San Francisco Bay Area continues to exist, (b) this risk can be quantified, and the liquefaction hazard at any given site can be correctly and reliably evaluated, and (c) once potentially liquefiable sites have been identified, the associated hazard can either be avoided or mitigated, though at considerable cost in some instances. It must be hoped that the lessons learned from the Loma Prieta Earthquake will spur local policy makers, and society as a whole, to undertake the difficult actions necessary to begin to remedy the considerable risk to the population and infrastructure of the Bay Area associated with current conditions at many of the sites discussed in Chapter 3. Preliminary indications are hopeful in this regard at many of these sites, but much more remains to be done.

In addition, it should be noted that although a majority of the liquefaction-induced damages during the Loma Prieta Earthquake occurred in bayshore fills, this does not mean that "fills" represent an intrinsically hazardous condition. Although liquefaction was fairly widespread in loose, uncompacted sandy bayshore fills, in many areas similar fills which had been compacted prior to the earthquake performed well and experienced no liquefaction. Sites where densified hydraulic fills performed well, while adjacent undensified fills liquefied, include Foster City and parts of Treasure Island, Emeryville, Alameda and Bay Farm Island.

7.4 Slope Stability:

In addition to the pronounced influence of local site effects on ground shaking characteristics, and the damages wrought by soil liquefaction, a third important geotechnical feature of the Loma Prieta Earthquake was the widespread occurrence of problems associated with slope instability. More than one thousand slides occurred in the Santa Cruz Mountains, in and around the fault rupture region, and additional slides occurred farther afield, especially along the Pacific coastal bluffs to the north of the epicentral region.

The principal impacts of slope instability were two-fold: (1) slides and rockfalls damaged and destroyed homes and businesses, and (2) slides disrupted major transportation arteries. Although a large number of structures were destroyed or damaged by sliding, this number was limited to some degree by the sparse development and population of the mountainous epicentral region. Similarly, although the temporary closure of Highway 17 and other access routes from the south San Francisco Bay Area to Santa Cruz occurred at a critical time (immediately after the earthquake when emergency services and supplies were urgently needed), and despite the significant costs and potential damage to local economies occasioned by the reactivation of a major slide disrupting Highway 1 on the west coast of Marin, significantly more disruption of transportation systems was caused by the closures of the San Francisco-Oakland Bay Bridge and of major highway segments in both San Francisco and Oakland, than by slope instability. This does not, however, mean that earthquake-induced slope instability does not pose a major threat to the greater San Francisco Bay Area. The Bay Area basin and the Pacific coast remain areas of considerable topographic relief, with innumerable structures and facilities located on hillsides and coastal bluffs, and the major fault systems responsible for much of the topographic relief remain capable of producing large earthquakes likely to cause widespread slope instability and resulting damage and destruction of structures and facilities, as well as closure of roads and disruption of vital utilities.

7.6 Earth and Rockfill Dams:

A number of earth and rockfill dams were strongly shaken by the 1989 Loma Prieta earthquake, and, in general, the dams performed satisfactorily. No dams failed, and no dams demonstrated signs of potential major instability such as might precipitate an uncontrolled reservoir release. In addition, risk to the public was further reduced by the occurrence of the earthquake at a time when reservoirs were low.

Although no dams failed, very strong levels of shaking near the zone of fault rupture did induce minor damage in a number of dams. This "damage" typically consisted of relatively shallow longitudinal cracks along the crests and/or upper portions of the dam faces, shallow transverse cracking at or near the abutments, and in some cases minor crest subsidence. Preliminary studies suggest that this damage poses no significant risk to the overall stability of these embankments.

Engineers and the public should be encouraged by the general good performance of major dams in the areas where strong earthquake shaking was induced. These encouraging results, however, should be tempered by the recognition that most of the reservoirs near the center of strong shaking were very low at the time of the earthquake. Consequently, major portions of the dam embankments were not as saturated as they would have been during full reservoir conditions. Thus, the 1989 Loma Prieta Earthquake may not have been the full test of these structures that might first be inferred.

7.6 Research Opportunities:

In addition to these general lessons to be learned regarding the impact of geological and geotechnical features and phenomena on damage patterns, and on the future seismic vulnerability of the greater San Francisco Bay Area's population and infrastructure, the Loma Prieta Earthquake also provides some unusually outstanding opportunities for research in a number of general areas. As a result, the lessons likely to be learned from studies of the Loma Prieta event and its consequences are likely to have a significant impact on future practice, and thus on the seismic safety of people and facilities around the world in future seismic events.

The Loma Prieta Earthquake provides unusually good opportunities for important research due to a number of factors, including:

(1) The strongly shaken region was unusually well-instrumented, and an unusually large number of strong motion recordings were obtained. These are well distributed spatially, and reflect a wide variety of geologic conditions. In addition, arrays on individual dams and buildings provide especially valuable opportunities for detailed response analysis studies of major structures and dams, and

(2) The complex geology of the shaken region provided a wide array of site conditions, and these had a clearly important influence on both strong shaking and damage patterns. In addition, damages were well-documented by a local professional community with an unusually high concentration of seismic expertise in both the research and general consulting communities.

Important opportunities for research include studies involving improvement, verification and/or calibration of methods for: (a) evaluation of dynamic site response, and of associated damage potential, (b) evaluation and mitigation of soil liquefaction potential and associated hazard for structures and facilities, (c) evaluation of seismic slope stability and identification of potentially seismically unstable slopes and coastal bluffs, and (d) evaluation of dynamic response and performance of earth and rockfill dams and embankments. In these and other areas, research opportunities generated by the tragic Loma Prieta event are likely to result in significant advances, and may be expected to contribute to substantial improvements in the levels of safety provided for society and its infrastructure in future seismic events.

7.7 Conclusions:

In summary, the Loma Prieta Earthquake of October 17, 1989 was a major tragedy, and one which provides the engineering profession with a number of valuable lessons and opportunities for research. This devastating event serves as a reminder of the unacceptably high level of risk or seismic exposure associated with the likely occurrence of larger and considerably more damaging future earthquakes both in the greater San Francisco Bay Area and around the world.

There is an urgent need to pursue the research opportunities provided by the Loma Prieta Earthquake, and to rapidly transfer the benefits of such research into the mainstream of professional practice. In addition, there is also an urgent need to educate both policy makers and the general public, and to motivate them to undertake the difficult actions necessary to begin to remediate the levels of seismic hazard exposure associated with existing conditions.

In addition to having been a major tragedy, the Loma Prieta Earthquake also represents a major opportunity for future improvement of the level of seismic safety provided for society and for its infrastructure. This must be resolutely pursued at all levels, both professional and political, as such improved safety is too precious a goal to command less than our utmost efforts.

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