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SYNOPSIS: This paper provides an overview of areas in San Francisco which were affected by soil liquefaction and significant ground deformation as a result of the Loma Prieta earthquake. The distribution of pipeline system damage is examined, and comparisons are made between 1989 and 1906 patterns of water supply damage. Special attention is given to the Marina to illustrate how the natural site conditions and artificial fills contributed to soil liquefaction and buried pipeline damage of both the water and gas distribution networks. Finally, the liquefaction potentials of natural beach and sand bar deposits, land-tipped fill, and hydraulic fill are evaluated and compared.

INTRODUCTION

The geotechnical and lifeline aspects of the 1989 Loma Prieta earthquake provide a unique opportunity for improving our understanding of earthquake effects. Not only has this earthquake provided substantial data pertinent to ground deformation and lifeline response, but case history information from Loma Prieta also can be compared with reports of the 1906 earthquake which include detailed studies of ground movement and pipeline system damage. Comparing and contrasting the Loma Prieta and 1906 earthquakes in San Francisco allows us to establish a range of earthquake responses, marked at one end by a major earthquake with locally severe and site-dependent effects and, on the other end, by a great earthquake with widespread and catastrophic consequences.

The performance of water supply systems is a concern for cities in areas of potential seismic activity. This concern is coupled closely with the problems of fire following earthquakes. For San Francisco, fire is a critical issue because a substantial part of the city burned after the 1906 earthquake and because of the many closely spaced timber buildings there. The problems of earthquake fire in San Francisco are linked with the occurrence of soil liquefaction, extent of ground deformation, and response of buried piping. A case history analysis of the Loma Prieta earthquake allows us to probe complex interactions among lifeline performance, geotechnical characteristics, and emergency response.

This paper begins with a general overview of areas in San Francisco that were affected by soil liquefaction and significant ground deformation as a result of the Loma Prieta earthquake. The distribution of pipeline system damage is examined next, and comparisons are made between 1989 and 1906 patterns of water supply damage. Special attention is given to the Marina to illustrate how the natural site conditions and artificial fills contributed to soil liquefaction and buried pipeline damage of both the water and gas distribution networks. Finally, the liquefaction potentials of natural beach and sand bar deposits, land-tipped fill, and hydraulic fill are evaluated and compared on the basis of an extensive database from soil explorations undertaken in the Marina.

LIQUEFACTION AND GROUND DEFORMATION

Figure 1 shows four areas within San Francisco for which there is historical evidence of soil liquefaction and large ground deformations during the 1906 earthquake. These areas, which are bounded by dashed lines, include the Mission Creek, South of Market, Foot of Market, and Marina districts. Each of these areas was investigated after the 1989 earthquake by one or more of the coauthors within the approximate boundaries shown by the solid lines.

Brief accounts of the visible geotechnical effects in the areas identified in Figure 1 are provided under the headings which follow. These accounts are taken from more comprehensive descriptions provided by O'Rourke, et al. (1990).

Mission Creek

East of Mission and Capp Sts., soil liquefaction occurred in the same places it had been observed after the 1906 earthquake. The most prominent damage caused by soil liquefaction occurred as differential settlement, racking, and tilting of Victorian two to four-story timber frame buildings on South Van Ness, Shotwell, and Folsom Sts. between 17th and 18th Sts. Sand boils were observed at all these locations. The most severe damage was observed at the middle west side of Shotwell St., where maximum building settlements on the order of 0.2 to 0.4 m occurred.

In contrast to this damage, the area west of Mission St. apparently was unaffected by soil liquefaction, even though lateral spreading and subsidence of 1.5 to 2.0 m were observed at Valencia and Guerrero Sts. in 1906. It is not known currently what influence the construction of the Bay Area Rapid Transit System, along Mission St., may have had on the subsurface conditions and potential for liquefaction in this vicinity.

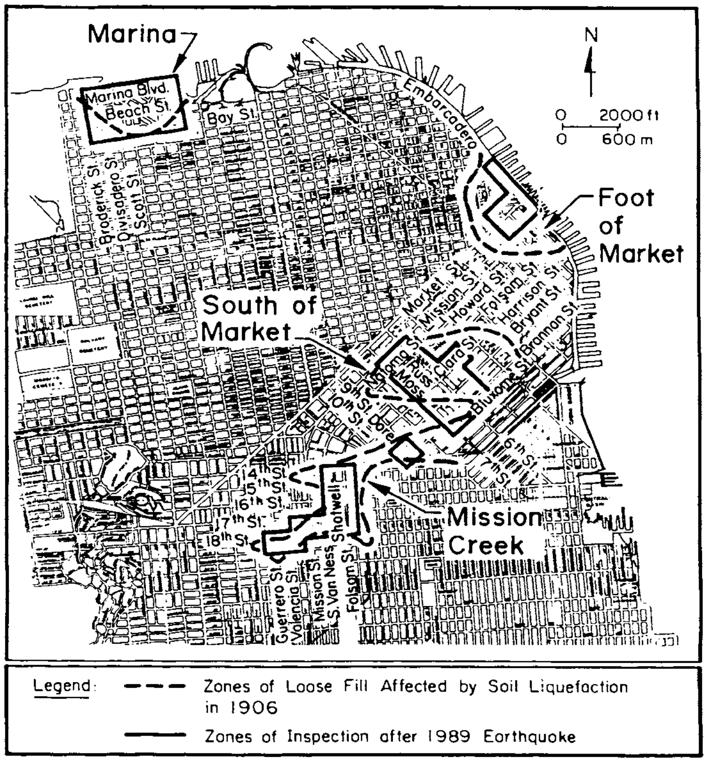

Figure 1. Zones of Soil Liquefaction in San Francisco Caused by 1989 Loma Prieta Earthquake (after O'Rourke, et al., 1990)

South of Market

South of Mission St., sand boils were observed along curb and building lines at various locations on 7th, 6th, Natoma, Russ, Moss, Clara, Bluxome, and Townsend Sts. From Mission to Folsom St., 10 to 30-mm-wide cracks were observed down the centerline of 7th St., with differential settlement to the east and west of the cracks. Sand flowed into the basement of a building at the corner of Howard and 7th Sts., filling it with approximately 0.6 m of material. Differential settlements and cracks were apparent on 6th St. between Folsom and Harrison Sts. Compression ridges in the form of buckled street pavements and sidewalks were observed along Russ St., approximately 30 to 60 m north of Folsom St.

At the intersection of 6th St. with Bluxome and Townsend Sts., there was substantial differential settlement. Beneath the western curb line of 6th St. at this location, there is a 2-m-diameter concrete sewer supported on piles. The ground settled sharply adjacent to each side of the sewer, with settlements of roughly 0.4 to 0.5 m at the northeast corner of 6th and Townsend Sts. relative to the sewer centerline. Differential settlements of 150 to 250 mm were observed adjacent to the pile-supported columns of the Rt. 280 highway ramp at this location. Sand boils were observed beneath the Rt. 280 highway ramp.

Foot of Market

Differential settlements and lateral displacements were observed along the Embarcadero from Howard St. to just north of the Ferry Building. Subsidence of approximately 0.3 m was observed immediately north of the intersection of Market St. and the Embarcadero. Sand boils were observed along the Embarcadero between the Ferry Building and Pier 1. A prominent 25-mm-wide crack was open beneath the Embarcadero Skyway, running parallel to the seawall for the full distance between Howard and Mission Sts. The crack showed lateral movement toward the bay at a distance of about 20 m behind the current seawall. Differential settlements of 25 to 100 mm were observed adjacent to the pile-supported columns of the Embarcadero Skyway.

Marina

The Marina was the site of some of the most devastating and well publicized damage caused by the earthquake. The damage occurred in two to four-story timber frame structures with concrete and masonry bearing wall foundations. The worst damage was concentrated at apartment buildings with multiple garages at ground level. These structures lacked sufficient strength and stiffness to resist shear distortion caused by seismic shaking. Where buildings with soft bottom stories were located at street corners or adjacent to open spaces, the absence of support from neighboring structures resulted in severe racking and, occasionally, structural collapse.

Although ground shaking was the principal cause of building damage, permanent ground movements also contributed to structural distortion. Permanent ground deformation was evident on virtually all streets. Buckled and fractured sidewalks and street pavements were apparent throughout the Marina district. Sand boils were observed along curb lines, sidewalks, foundation bearing walls, and in fields and gardens. In many instances, sand erupted into garages, forming deposits 0.3 to 0.6 m thick. A more detailed description of liquefaction effects in the Marina is given in a forthcoming section of this paper.

WATER SUPPLY PIPELINE DAMAGE

After the 1906 earthquake, more than 10.6 km^2 of the city burned, destroying 490 blocks and causing partial destruction of 32 additional blocks (Gilbert, et al., 1907). This conflagration represents the largest single fire loss in U.S. history. Fire spreading was abetted by loss of water from pipeline ruptures in zones of soil liquefaction. Scawthorn and O'Rourke (1989) have shown that liquefaction-induced ground movements caused multiple failures in the pipeline trunk systems from the College Hill and University Mound Reservoirs, thereby cutting off 56% of the total stored water in San Francisco to the Mission and central business districts of the city. Lateral spreading and subsidence along Valencia St. ruptured 400 and 550-mm-diameter pipelines, which led directly to the loss of 43,000 m^3 of water from the College Hill Reservoir in less than 24 hours (Schussler, 1906). The College Hill Reservoir still provides water for San Francisco, and currently stores about 51,000 m^3 for distribution in the city's municipal supply network.

Figure 2 shows a plan view of the location of breaks in the water distribution system of San

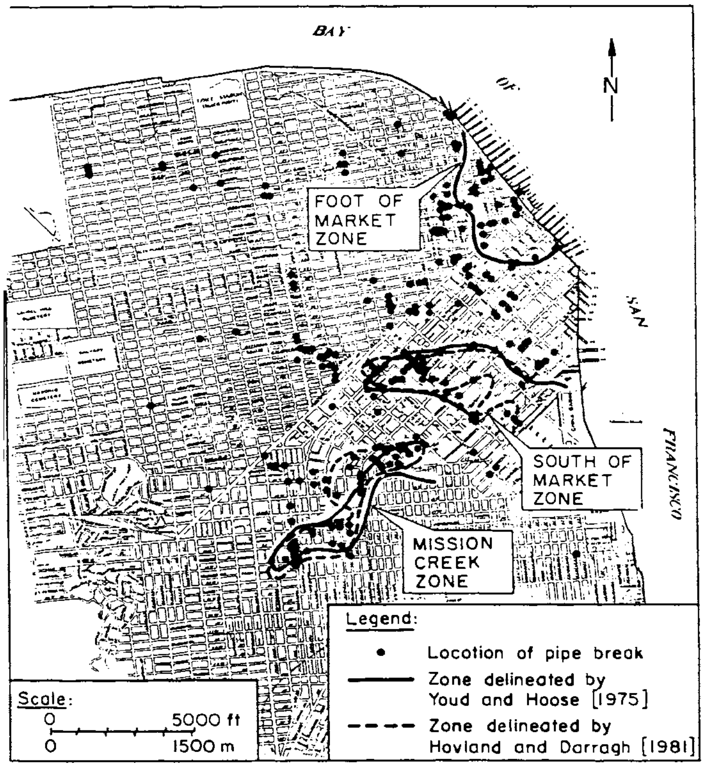

Figure 2. Water Supply Pipeline Breaks and Zones of Soil Liquefaction Caused by 1906 San Francisco Earthquake (after O'Rourke, et al., 1985)

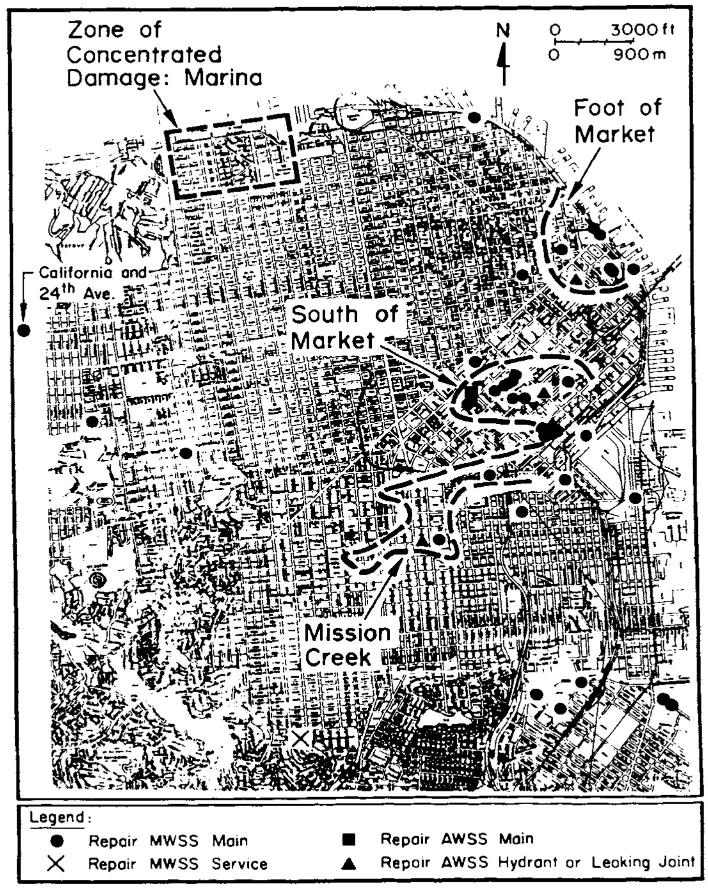

Figure 3. Water Supply Pipeline Breaks and Zones of Soil Liquefaction Caused by 1989 Loma Prieta Earthquake

Francisco after the 1906 earthquake (O'Rourke, et al., 1985). This plot of pipeline breaks was developed on the basis of maps by Schussler (1906) and Manson (1908), which show the locations of principal water main ruptures. Superimposed on the plot are the boundaries in which lateral spreading from soil liquefaction was observed during the 1906 earthquake. The boundaries of lateral spreading are plotted on the basis of zones delineated by Youd and Hoose (1975) and Hovland and Darragh (1981). The agreement among the zone boundaries from the different studies is good. Although the zones of lateral spreading account for only 5% of the built-up area of 1906 affected by strong ground shaking, approximately 52% of all pipeline breaks occurred inside or within one city block of these zones.

Figure 3 shows the locations of water pipeline system repairs after the 1989 Loma Prieta earthquake. Zones of potential soil liquefaction, based on maps by Youd and Hoose (1975) and Hovland and Darragh (1981) also are shown. Repairs to both the current Municipal Water Supply System (MWSS) and Auxiliary Water Supply System (AWSS) are indicated by the appropriate symbols. The MWSS supplies potable water for domestic and commercial uses, as well as for fire fighting via hydrant and sprinkler systems. After the 1906 earthquake, the AWSS was constructed to provide emergency fire protection. This system is intended to augment the city's existing fire fighting capacity by providing a supplementary network that works independently of, but in parallel with, the MWSS. The site of the Marina is shown, but because of the extensive MWSS damage at this location, repair plots are not indicated. Separate plots and descriptions of Marina pipeline damage are provided in the next section of this paper.

By comparing Figures 2 and 3, it is clear that pipeline breaks in 1989 and 1906 were clustered at similar locations in liquefaction-prone areas. The consistency in damage patterns, combined with the facts that: 1) pipelines tend to deform as the ground deforms, and 2) a pipeline network covers a broad area, provides us with the opportunity to use the pipeline system as an observational tool which reflects local earthquake severity. At the same time, the damage observations give us an empirical framework from which to estimate the potential levels of pipeline repairs relative to local geologic and geotechnical site characteristics.

Table 1 summarizes the number of water main repairs, total linear distance of main, and repairs per distance for several areas of San Francisco. Also included in the table are the Modified Mercalli Intensities (MMI) associated with each area, as estimated from field observations by the authors, as well as MMI estimates reported by Plafker and Galloway (1989).

TABLE I. Summary of MWSS Pipeline Repairs for Various Sites in San Francisco[a]

Site	Repairs	Length of Pipeline, km	Repairs per km	Modified Mercalli Intensity MMI
Marina	69	11.3	6.11	IX
South of Market	13	17.1	0.76	VIII
Foot of Market	6	13.8	0.43	VII
Mission Creek	2	15.1	0.13	VII
Remainder of System	15	1740	0.01	VI

a - Repairs to services not included

Table 1 shows substantial differences in pipeline repair rates for different sectors of the city. In effect, the repair rate is an index of seismic intensity. In Figure 4, the repair statistics for Loma Prieta are combined with repair statistics for other earthquakes to explore further the relationship between pipeline damage and MMI. All data in the plot represent systems composed entirely or predominantly of cast iron lines.

As illustrated in the figure, there is a clear linear trend of repairs/km with MMI. Moreover, the trend in the Loma Prieta data is consistent with that of the other western U.S. earthquakes. Such empirical evidence implies that cast iron pipeline damage varies exponentially with MMI. Repair rates increase by roughly an order of magnitude for each single digit advance in the MMI scale.

PIPELINE DAMAGE IN THE MARINA

Water supply damage in the Marina has been reported elsewhere (O'Rourke, et al., 1990; O'Rourke and Roth, 1990), and only the salient features of the damage are reported herein.

Figure 5 shows a plan view of the MWSS pipelines and repairs relative to the current street system, 1899 shoreline (Sanborn Ferris Map Co., 1899) and 1857 shoreline (U.S. Coast Survey, 1857). Most repairs were concentrated in the area of hydraulic fill within the lagoon bounded by the 1899 seawall, or along the eastern margins of the seawall and 1857 shoreline.

There were about 123 repairs in the Marina, more than three times the number of repairs in the entire MWSS outside the Marina. Repairs were made at locations of sheared or disengaged service connections with mains, flexural round cracks in mains, and longitudinally split sections of main. In some cases, damage was concentrated at or near gate valves. These devices tend to anchor the pipelines, and therefore may contribute to locally pronounced deformation and stresses. The figure shows the locations of repairs to: a) services, b) mains, and c) sections of line at or near gate valves. The MWSS pipelines were composed predominantly of cast iron,

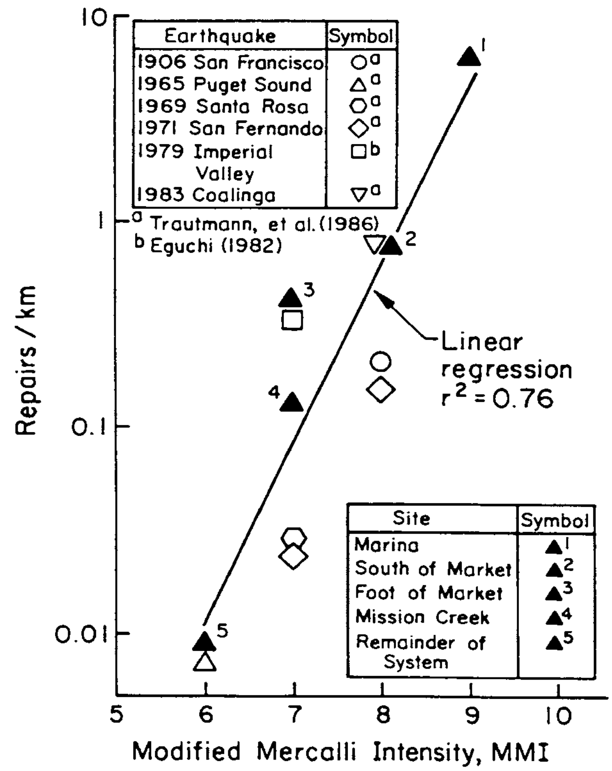

Figure 4. Repair Rate of Cast Iron Pipeline Systems versus Modified Mercalli Intensity

with cement-caulked joints and nominal diameters of 100, 150, 200, and 300 mm.

Figure 6 shows the locations of pipeline repairs to the AWSS and high pressure gas distribution network in the Marina. It should be noted that the term "high pressure" is intended to distinguish pipelines conveying gas at approximately 200 kPa from low pressure distribution lines at approximately 2 kPa.

Even though substantial damage was sustained by the MWSS in the Marina, there was only one instance of AWSS repair in this area. This occurred at a leaking joint at Scott and Beach Sts. Pipelines of the AWSS are equipped with sleeved joints, which are restrained against pullout by longitudinal bolts. Cast iron pipelines of 250 and 300-mm diameters are used in the Marina with joint-to-joint lengths of 3.7 m. The relatively large diameter-to-length ratio, in conjunction with joints which are able to rotate and are axially restrained, allows the pipelines to accommodate differential ground movement.

Only one location of damage was reported for the high pressure gas distribution lines at a miter joint near the boundary of hydraulic fill and 1857 shoreline. The high pressure mains were constructed mostly of Grade B steel with electric arc girth welds.

In contrast to the high pressure system, there was substantial damage of the low pressure gas distribution mains. Because of the extensive damage, the low pressure system was isolated in

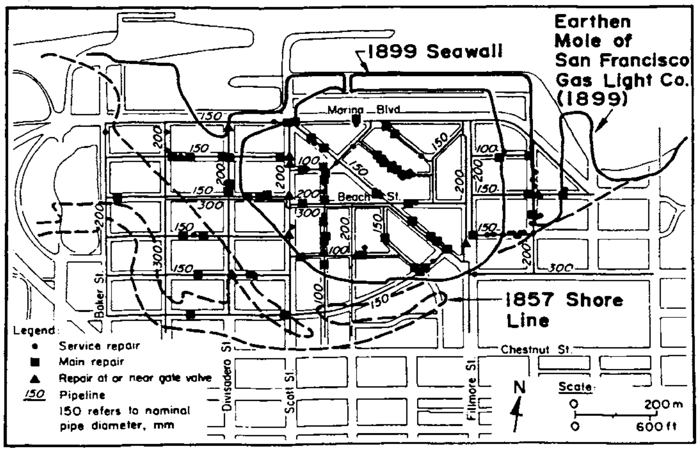

Figure 5. MWSS Repairs in the Marina (after O'Rourke and Roth, 1990)

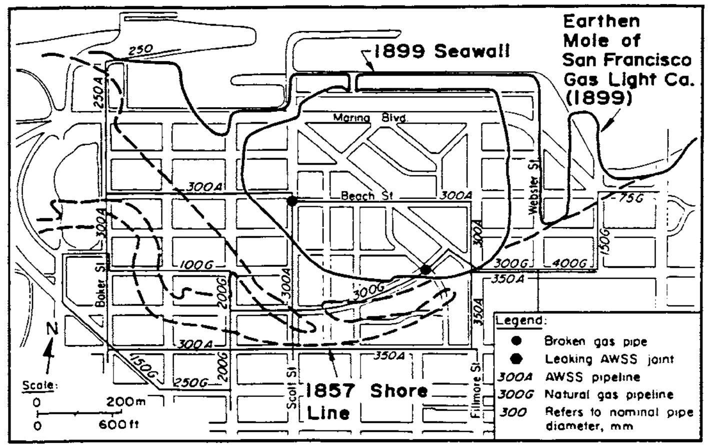

Figure 6. AWSS and High Pressure Gas Distribution Repairs in the Marina

the Marina by closing shutdown valves. Approximately 13.6 km of steel and cast iron mains, ranging from 100 to 300 mm in diameter, were replaced within the area bounded by Laguna, Lombard, Lyon St., and the bay (Phillips and Virostek, 1990). A little over half this number were replaced with medium density polyethylene (MDPE) piping inserted within existing steel and cast iron pipes. The remainder was replaced by direct burial of MDPE piping. The nominal diameters of replacement pipes were 50 to 150 mm, with about 90% of the piping having a 50 mm diameter.

PERFORMANCE OF AWSS

The AWSS is shown in Figure 7. It is separated into an upper and lower zone, each of which operates nominally at a pressure of about 1 MPa. There are approximately 200 km of buried pipe, with nominal diameters ranging from 250 - 500 mm. Nearly 160 km of the system is cast iron, to which about 40 km of ductile iron pipe have been added during the past several decades. The AWSS has no building connections or service lines; only fire hydrants can draw from the system.

The system is supplied by the Twin Peaks Reservoir, Ashbury Tank, and Jones St. Tank, which hold 38, 1.9, and 2.8 million liters, respectively. Two pump stations can pump salt water from San Francisco Bay into the system to augment the water supply. The stations have four diesel pumps, each of which can pump 9500 liters/ min. at 2 MPa into the lower zone. The city's fireboat, "Phoenix," can be connected to each of five manifolds to inject an additional 38,000 liters/ min. at 1 MPa into the lower zone. Additional water for fire fighting is stored in a series of underground cisterns, each of which holds an average of about 284,000 liters.

The most serious damage was the break of a 300-mm-diameter cast iron main on 7th St. between Mission and Howard Sts. Water flow through this break, supplemented by losses at broken hydrants, emptied the Jones St. Tank. Loss of this supply led to loss of water and pressure throughout the lower zone of the AWSS. This resulted in an especially sensitive condition in the Marina, where damage in the MWSS had cut off alternative sources of pipeline water.

When fire broke out at the corner of Divisadero and Beach Sts., water to fight the fire was drafted and relayed from the lagoon in front of the Palace of Fine Arts, approximately three blocks away. The fireboat, "Phoenix," and special hose tenders were dispatched to the site. Approximately one and a half hours after the main shock, water was being pumped from the fireboat and conveyed by means of 125-mm-diameter hosing, which had been brought to site by the hose tenders. Eventually, the supply of water to the fire was about 23,000 liters/min. The fire was brought under control within about three hours after the earthquake.

The special hose tenders and large-diameter hoses belong to the Fire Department's Potable Water Supply System (PWSS), which can move throughout the city and connect with the fireboat, underground cisterns, the underground pipeline network, and other sources of water to provide an additional measure of flexibility under emergency circumstances. The system had been implemented only two years before the earthquake.

AWSS SIMULATION STUDIES

To understand better the performance of the AWSS, computer simulations of the network were performed with the program GISALLE (Graphical Interactive Serviceability Analysis for LifeLine Engineering). This program was developed to represent the AWSS as part of a special demonstration project to develop advanced techniques of computer graphics for lifeline systems and to prove the feasibility of applying these techniques to a real system (Khater, et al., 1989; Grigoriu and O'Rourke, 1989). GISALLE has been checked successfully against special fire flow tests run by the San Francisco Fire Department. The computer model is built around a hydraulic-pipeline-network program that has been modified so as to allow the simulation of post-earthquake damage states. A special code has been developed for GISALLE to model accurately the hydraulic performance of damaged systems with many pipeline breaks.

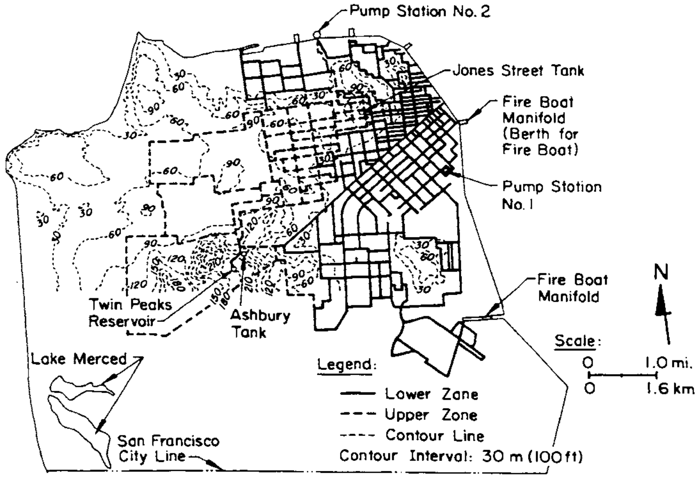

Figure 7. Plan View of AWSS

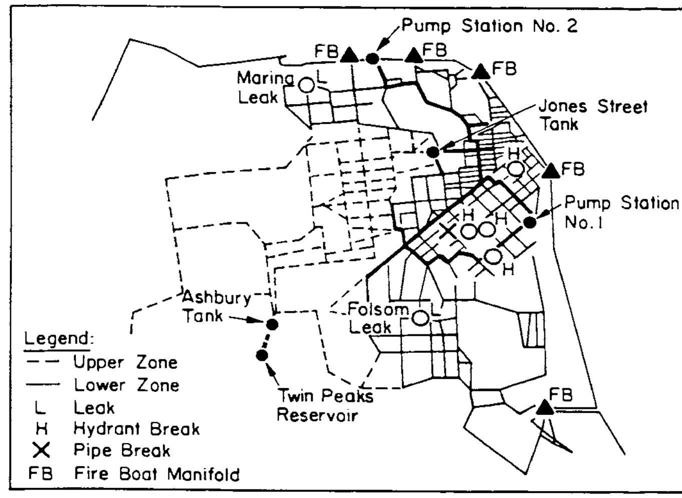

Figure 8. Conditions Used in Computer Simulation of AWSS Performance During the Loma Prieta Earthquake

Figure 8 shows a plan view of the system that was simulated to reproduce the conditions on the night of the earthquake. Water in the lower zone was supplied by the Jones St. Tank. The lower and upper zones were isolated from one another with closed gate valves. Pump Stations 1 and 2 were not included in the simulation to replicate the system conditions immediately following the earthquake.

Damage of the AWSS occurred as a broken 300-mm diameter main on 7th St., four broken hydrants, and two leaking joints. The approximate locations of these damaged components are illustrated in the figure. Hydrants were the most vulnerable parts of the system, with damage being concentrated at elbows. Typical construction involves a 200-mm diameter cast iron elbow affixed to a concrete thrust pad beneath the street surface hydrant. Damage at hydrant elbows occurred as 45° fractures centered on the elbows.

Two models of the system were analyzed by a) neglecting water losses from leaking joints, and b) treating the leaking joints as open hydrants. By analyzing both situations, pressures and flows consistent with the system performance were bounded.

Figures 9 and 10 show the results of the analyses in graphical format. In each figure, open arrows denote water egress either from the Jones St. Tank or damaged components. The solid arrows denote internal flow. Zones of potential soil liquefaction in the South of Market and Foot of Market areas also are shown.

The South of Market area had been recognized as a zone of potentially unstable ground, called an infirm area, and had been isolated from adjoining portions of the network by closed gate valves. Only one open gate valve was provided for this zone at the intersection of Market and 6th Sts., as illustrated in the figures. This gate valve was designed to be operated remotely with utility-supplied electric power. Because of electric power loss at the time of the earthquake, the valve could not be closed remotely. Consequently, water flow through this gate valve in each figure equals the sum of water losses from the broken main and two broken hydrants in this particular infirm area of the system.

In Figures 9 and 10, the total flow rates from the Jones St. Tank are 65,000 and 78,000 liters/min., respectively. The increase in flow rate for the second case represents the additional draw on the system from modeling leaks as equivalent to open hydrants. Given that the normal operating capacity of the Jones St. Tank is approximately 2.72 million liters of a maximum 2.84 million liters, the time required to empty the Jones St. Tank would have been between 42 and 35 minutes, pertaining to the conditions in Figures 9 and 10, respectively.

This estimated time to loss of tank agrees with observations during the earthquake. Scawthorn and Blackburn (1990) report that, when the first engine arrived at the Marina fire approximately 45 minutes after the earthquake, it could not draw water from the AWSS hydrants. This time for engine arrival exceeds that analyzed for loss of the Jones St. Tank. Moreover, it was estimated that loss of water from a height of 10.6 to 5.8 m in the Jones St. Tank took approximately 15 to 20 minutes. This decrease in water level involves a loss of approximately 1.2 million liters, resulting in an outflow of 60,000 to 80,000 liters/min. over the estimated time span, which is consistent with the analytical results.

The AWSS performance and computer simulations emphasize how rapidly water can be lost and how important automatic control of isolation gate valves can be. The simulations also underscore the importance of hydrant breaks. In both Figures 9 and 10, the combined flow from two broken hydrants in the South of Market infirm area equals the flow from the ruptured 300-mm main.

GEOTECHNICAL CHARACTERISTICS OF THE MARINA

The Marina was the most heavily damaged neighborhood of San Francisco, and is a particularly interesting subject for case study analysis.

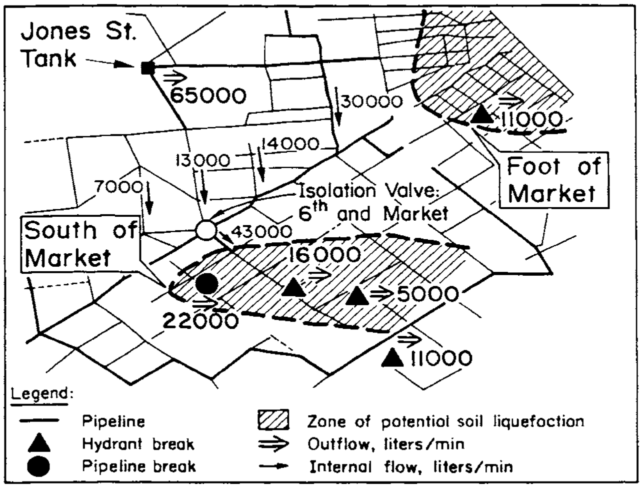

Figure 9. Results of AWSS Simulation with Negligible Water Loss from Leaking Joints

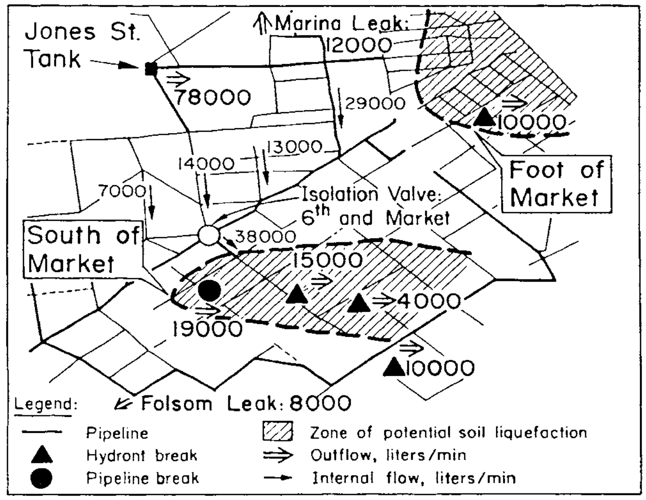

Figure 10. Results of AWSS Simulation with Leaking Joints Modeled

Several reports already have been written about the Marina, most notably studies by USGS (1990) and Mitchell, et al. (1990). Additional investigations are underway, including work reported in this conference (Bardet, 1991). Space limitations allow only a brief treatment in this paper. For more detailed evaluation of the geotechnical characteristics, the reader is referred to USGS (1990).

Figure 11 shows a plan view of the northern part of San Francisco which surrounds the Marina district. An area of concentrated study is illustrated, within which the most severe effects of the earthquake were experienced. The figure shows outcrops and soundings to bedrock mapped by Schlocker (1974). Only data which were confirmed as pertaining to Franciscan bedrock are shown. In addition, the locations of several other borings to bedrock are indicated.

On the basis of the boreholes and outcrops illustrated in Figure 11, contours of bedrock elevation were plotted and are displayed in Figure 12. The contours were developed with the computer program "Surfer" (Golden Software, Inc., 1985) using a procedure referred to as kriging, in which the contours are developed with minimal estimation variance from a statistical evaluation of the input data (e.g., Ripley, 1981). All elevations are referenced to the San Francisco zero datum.

Figure 12 indicates that the Marina is underlain by an oblong bedrock basin, with its long axis oriented in a northwest direction. This direction appears to be structurally controlled, being subparallel to anticlinal and synclinal axes mapped by Schlocker and coworkers in San Francisco (1974). The long axis also is subparallel to a zone of sheared rocks trending northwest beneath the Marina (Schlocker, 1974). The basin is over 75 m deep below Marina Blvd. adjacent to Marina Green and the yacht harbor.

Figure 13 shows a plan view of selected soil borings performed for engineering projects in the Marina that preceded the Loma Prieta earthquake, in addition to borings performed to clarify foundation conditions beneath individual buildings during reconstruction and repair after the earthquake. There are over 125 borings shown in the figure, which were reviewed as part of the site characterization reported in this paper. The boring locations are superimposed on the existing street plan and outlines of the 1857 and 1899 shorelines.

The historical shoreline deserves comment. In the 1890s, a seawall was constructed in the Marina (Olmsted, et al., 1977) by dumping rock, which had been hauled to the site on barges, and backfilling behind the rock embankment with sand taken primarily from dunes. Similar construction was performed by the San Francisco Gas Light Company to establish an earthen mole. This configuration of seawall, embankment, and artificial fill remained essentially unchanged until 1912, when construction on site was started for the 1915 Panama Pacific International Exposition.

The lagoon enclosed by the seawall was filled with dredged soil pumped from depths of 10 to 15 m at distances of 180 to 600 m offshore. Relatively strict control of the fill material was exercised. The opening along the northern line of the seawall was used to sluice out fine grained and organic materials during hydraulic filling. It was estimated that 70% of the fill placed in this way was sand (Olmsted, 1977).

The Marina, therefore, is a composite of natural alluvium and clay on top of which fill was placed by: a) hauling and tipping into the bay, and b) hydraulic pumping into the lagoon enclosed by the old seawall. Surficial sands and fills in the Marina are underlain by Recent Bay Mud.

Figures 14 and 15 show elevation contours of the top and bottom of Recent Bay Mud, respectively. These contours were developed in a manner similar to those in Figure 12, with the exception that substantially more borehole data were available for comprehensive mapping of the Bay Mud surfaces.

Figure 14 shows that the top of mud, which forms

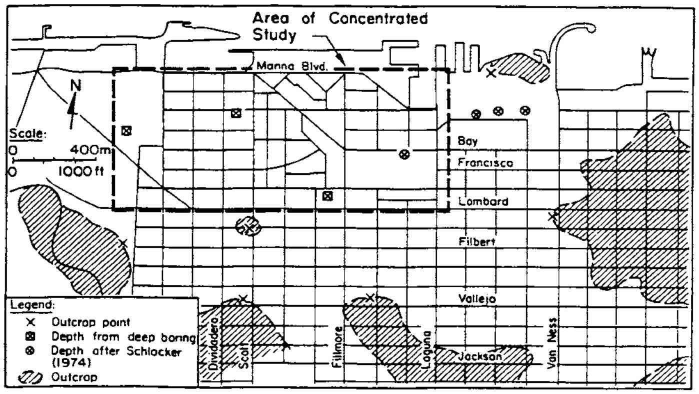

Figure 11. Plan View of Marina and Surrounding Neighborhoods

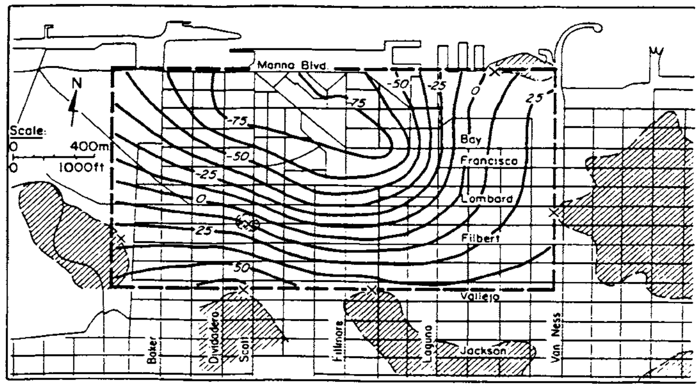

Figure 12. Bedrock Contours Beneath the Marina

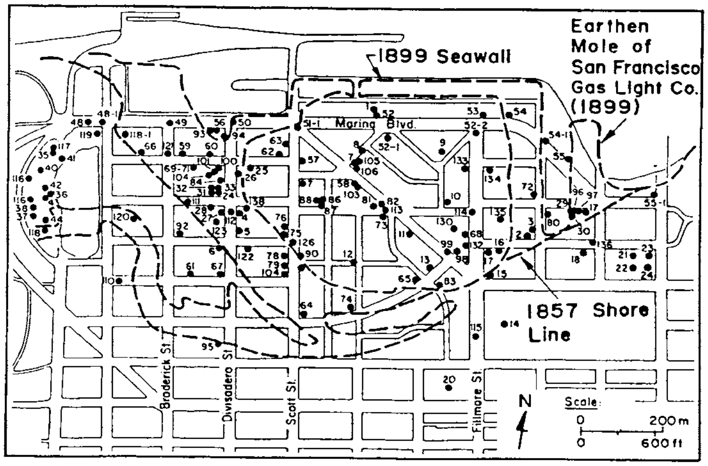

Figure 13. Soil Borings in the Marina

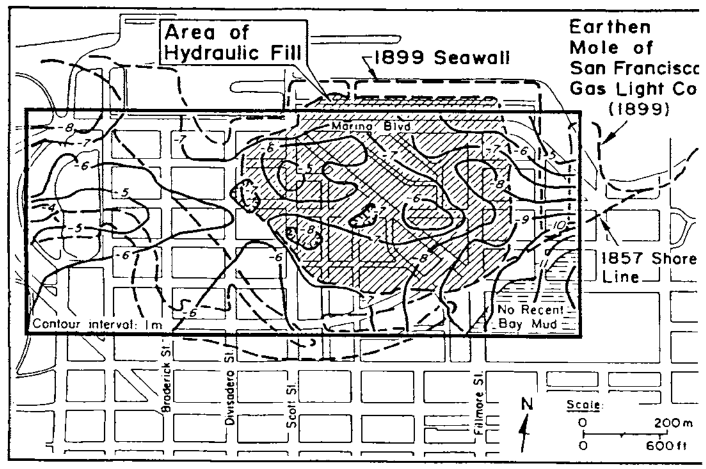

Figure 14. Contours of Equal Elevation for Top of Recent Bay Mud in the Marina

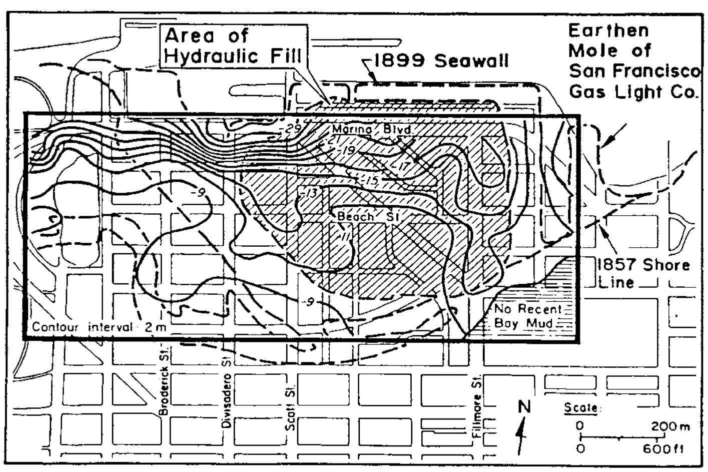

Figure 15. Contours of Equal Elevation for Bottom of Recent Bay Mud in the Marina

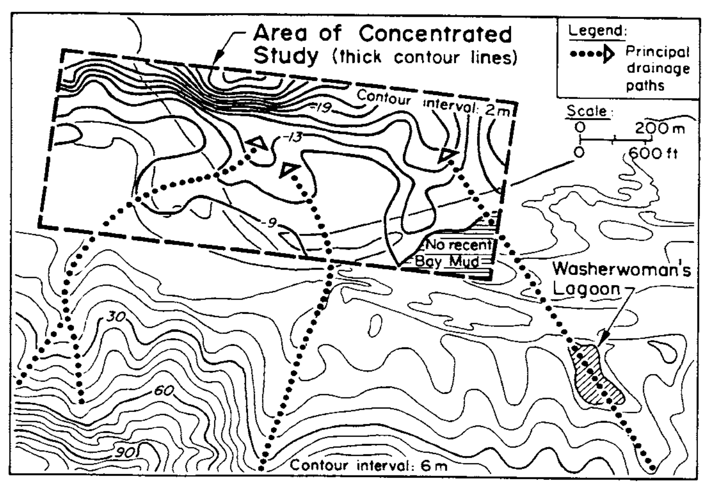

Figure 16. Erosional Surface Beneath Recent Bay Mud in Relation to San Francisco Topography

the underlying surface of the fills, is irregular. This irregularity is especially noticeable beneath the hydraulic fill, and takes the form of a hummocky surface with mounds and depressions. Such an irregular surface is consistent with the method of filling whereby sands were pumped into various portions of the lagoon, causing local bearing failures, with associated mounding of displaced fine grained sediments.

Figure 15 shows that the Recent Bay Mud thickens rapidly towards the bay, with its steepest bottom surface from Broderick to Scott Sts. between Marina Blvd. and Jefferson. Slopes of the bottom surface in this region are locally as steep as 10°. The configuration of contours apparently is related to surface drainage features established before the deposition of mud at the end of the Pleistocene.

Figure 16 shows the contours of the bottom of Recent Bay Mud superimposed to scale on a contour map of northern San Francisco. The San Francisco contours are based on one of the original coastal surveys of this area (U.S. Coast Survey, 1857). The basal contours of the mud are consistent with drainage features converging on the Marina from higher elevations to the south. A prominent drainage line crosses the old Washerwoman's Lagoon and intersects the eastern sector of the erosional surface beneath the mud. Similar to the long axis of the bedrock basin, this drainage line is oriented in a northwest direction, subparallel to major structural features which have been mapped in San Francisco.

A cross-section of the natural sands, fills, and Recent Bay Mud along Marina Blvd. (section A-A') is shown in Figure 17. This cross-section was developed primarily from borings performed for the North Shore and Channel Outfalls Consolidation Project (Dames and Moore, 1977), and has been published elsewhere (O'Rourke, et al., 1990). Data from various boreholes are summarized in the form of uncorrected Standard Penetration Test (SPT) values, equivalent SPT, and Torvane undrained shear strengths, as explained by the borehole legend in the figure. All uncorrected SPT measurements were consistent with ASTM specifications (ASTM, 1989). Equivalent SPT readings were estimated from blow count measurements performed with nonstandard equipment according to the recommendations of Roth and Kavazanjian (1984).

Figure 17 shows loose fill with a maximum depth of about 9 m extending along Marina Blvd. from approximately Baker to Buchanan Sts. This distance correlates well with the distance between locations of the 1857 shoreline shown in Figure 12. The depth to water table is between 1.5 and 2 m. Underlying the loose fills and natural sand deposits is Recent Bay Mud, which varies in thickness along Marina Blvd. from 9 to 23 m and extends to a maximum depth of 32 m. Underlying the Recent Bay Mud are dense sands and stiff to hard clays.

Figure 18 shows contours of soil settlement and sand boil locations in the Marina, as reported by Bennett (1990). It should be noted that the contours in Figure 18 represent the incremental settlements between 1974 and 1989, and thus involve some component of movement which may be attributed to consolidation of the Recent Bay

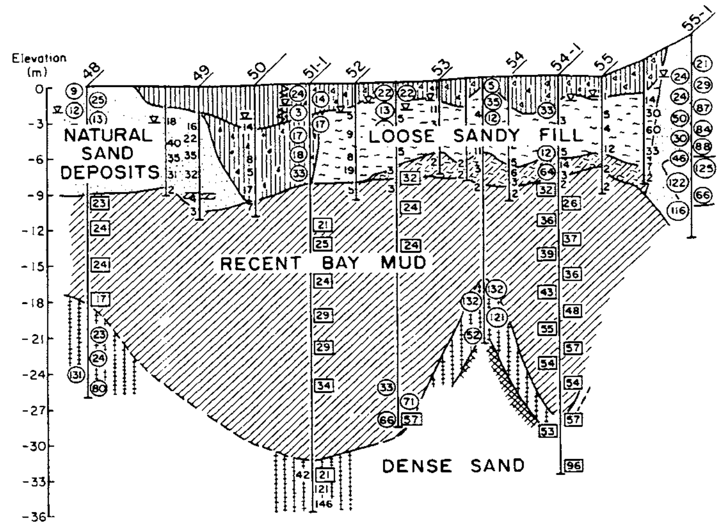

Figure 17. Cross-Section of Subsurface Soils in the Marina

Mud. Judging from the incremental settlements from 1961 to 1974, which also were reported by Bennett (1990), such time-dependent displacement would represent only a small percentage (10 - 20%) of the vertical movements shown in the area of hydraulic fill. It also should be pointed out that no distinction is made in this figure between sand boils arising from soil liquefaction as opposed to those potentially caused by leaking water pipes, as was done by Bennett.

To represent the distribution of MWSS damage, the Marina was divided into a grid of approximately 40 cells, and the number of repairs per length of pipeline in each cell was counted. Each repair rate then was normalized with respect to a reference length of 300 m to provide a consistent basis for evaluation. Contours of equal repairs per 300 m of pipeline were drawn and superimposed on the street system and previous shorelines, as illustrated in Figure 19. The contours of pipeline repair rates are related closely to the settlement contours, hydraulic fill, and 1857 shoreline. High concentrations of pipeline repair fall within the area of hydraulic fill, and the heaviest repair concentration occurs at the junction of the hydraulic fill, 1899 seawall, and 1857 shoreline. Over 80% of MWSS main repairs were for round cracks, which implies that bending and longitudinal tension were the predominant modes of deformation.

LIQUEFACTION POTENTIAL

Because the history of development in the Marina is well understood, it is possible to identify the different types of granular deposits in this district with reasonable accuracy. At least three different types of granular soil can be identified: 1) hydraulic fill, 2) fill tipped from shore or barge, herein referred to as land-tipped fill, and 3) natural beach and sand bar deposits.

Liquefaction potential analyses were performed for each soil type using the empirical relationship between cyclic stress ratio and corrected

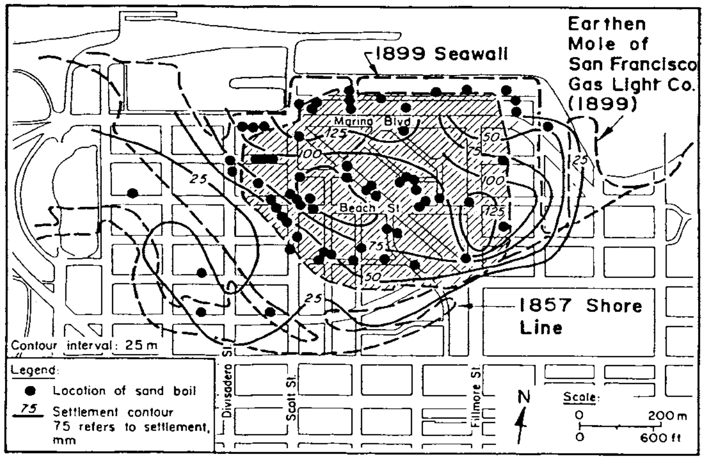

Figure 18. Sand Boil Locations and Contours of Equal Settlement in the Marina

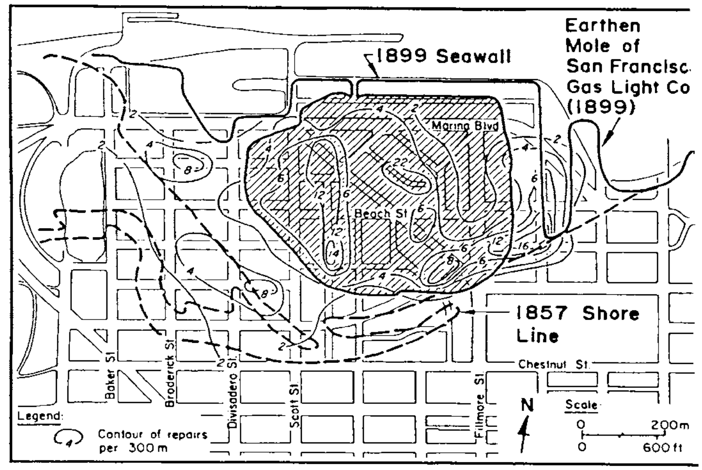

Figure 19. Contours of Equal Intensity of MWSS Pipeline Repair in the Marina

SPT values developed by Seed, et al. (1983). The SPT values were obtained in accordance with ASTM specifications (ASTM, 1989) and corrected for factors such as in-situ confining stress and energy losses, following the recommendations of Seed, et al. (1983). The cyclic stress ratios for various depths were calculated assuming a peak acceleration of 0.2 g, which is consistent with the peak horizontal component of acceleration recorded at the nearby Presidio.

Figure 20 shows the cyclic stress ratio plots for the three types of soil. In each figure, the empirical dividing line between liquefiable (upper left) and nonliquefiable soils is plotted for a magnitude 7.1 earthquake. Curves for both clean sand and sand with 10% fines were drawn in accordance with the recommendations of Seed and DeAlba (1988) and plotted in the figures.

The most striking feature of the plots is the relatively high susceptibility to liquefaction displayed by the hydraulic fill. As illustrated in Figures 18 and 19, concentrations of settlement, sand boils, and MWSS pipeline damage are associated with this highly susceptible material. The corrected SPT values for the majority of measurements in the sample are less than 5. This implies that the hydraulic fill would have a very low undrained residual shear strength, perhaps lower than 5 kPa, based on empirical relationships suggested by Seed (1987). Such low strength makes the hydraulic fill vulnerable to flow failure. Accordingly, the stability of the seawall in the Marina is especially critical because it provides lateral confinement of soil, which might otherwise be subject to retrogressive sliding in the event of a major earthquake.

In contrast, both the land-tipped fill and natural soils show increasing resistance to liquefaction. These soils also show a successively larger range of in-situ densities, reflected by the SPT values. Roughly half the land-based fill data plot to the right of the 10% silt curve. The distribution of settlement and pipeline damage within the land-tipped fills seems consistent with the behavior implied by the plot. The majority of the natural soil data plot to the right of the empirical dividing lines.

The wide range of in-situ densities for the land-tipped fills and natural soils implies a variability in liquefaction response. Such variations in density can affect the extent of liquefaction, as well as the magnitude and pattern of ground deformation once high pore pressures develop in the looser materials. Data such as those in Figure 20 raise questions about the role of soil variability during seismic shaking and its relationship with the loss of shear strength and ground deformation.

CONCLUSIONS

Soil liquefaction and ground deformation caused by the Loma Prieta earthquake occurred at the same locations in San Francisco as those during the 1906 earthquake. Damage patterns in the water supply pipeline networks are remarkably similar for both earthquakes. Pipeline repair following earthquakes is an index of seismic severity, and repair records help to quantify the vulnerability of various parts of the city. There is a clear correlation between pipeline repairs per km and MMI, as developed on the basis of damage statistics and observations associated with the Loma Prieta and other western U.S. earthquakes.

Computer simulations of AWSS performance during the earthquake are consistent with observations in the field. The computer simulations indicate that water was lost from the lower zone reservoir of the system in about 35 to 40 minutes. Moreover, the computer simulations emphasize the importance of an independent power supply for isolation valves and the substantial effect that hydrant breaks have on water lost from the system. Computer analyses have shown that two hydrant breaks can result in water leakage equivalent to the break of a 300-mm main at locations such as the South of Market area.

The events of the earthquake show that flexibility provided by the Portable Water Supply System (PWSS) was of critical importance in controlling and suppressing the fire which erupted in the Marina. The ability to operate with portable hosing and draft from a variety of water sources,

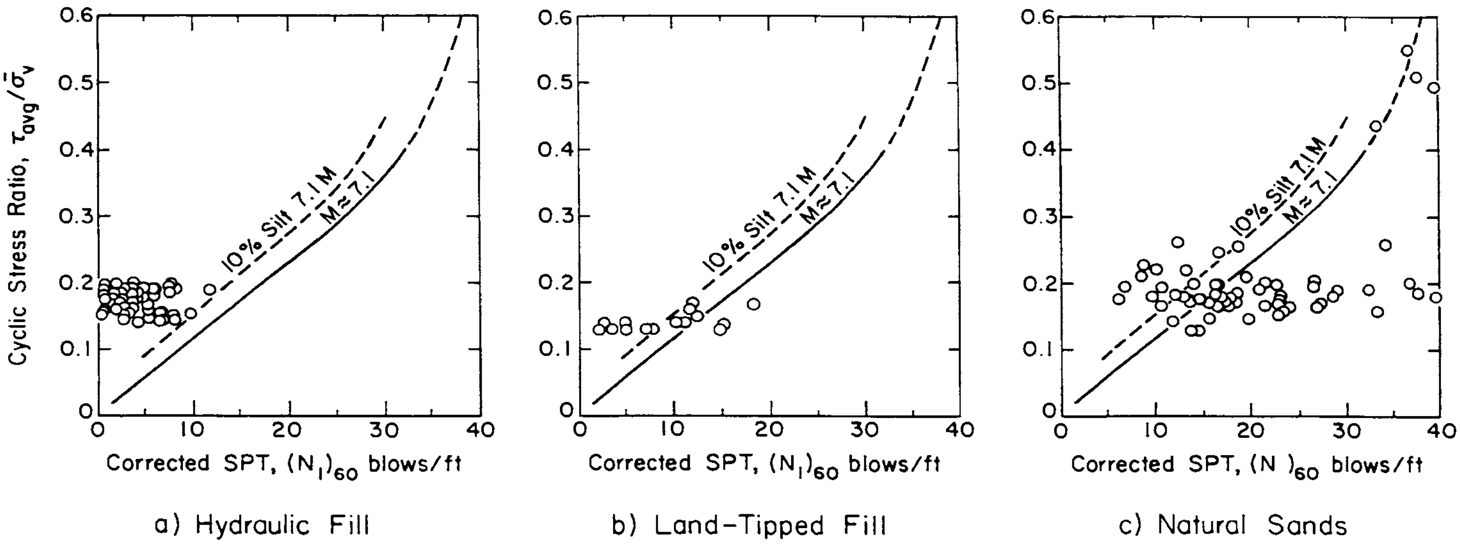

Figure 20. Liquefaction Susceptibility of Soils in the Marina for the 1989 Loma Prieta Earthquake

including underground cisterns and fireboats, provides a valuable extra dimension in the city's emergency response.

The Marina provides an excellent case history of how site amplification and soil liquefaction affect surface structure and buried lifeline performance. An account is given in this paper of the subsurface conditions in the Marina and their relationship with various geologic features in San Francisco. Damage to the water and gas distribution pipeline networks in the Marina is described and related to site characteristics. Liquefaction susceptibility analyses show the relative vulnerability of hydraulic fill, land- and barge-tipped fill, and natural beach and sand bar deposits to seismic shaking. Of the three soils, the hydraulic fill shows the smallest range of in-situ density and highest susceptibility to liquefaction. The possibility of flow failure in the hydraulic fill underscores the importance of the seawall in providing lateral restraint and encourages measures to promote its seismic stability.

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