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Lessons Learned from Liquefaction and Lifeline Performance During San Francisco Earthquakes

T. D. O'Rourke
Cornell University, Ithaca, New York

J. W. Pease
Cornell University, Ithaca, New York

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T.D. O'Rourke
Professor, Cornell University, Ithaca, New York

J.W. Pease
Graduate Research Assistant, Cornell University, Ithaca, New York

SYNOPSIS: This paper presents information about subsurface conditions, liquefaction-induced ground movements, and lifeline performance during the 1906 and 1989 earthquakes in San Francisco. Three sites of soil liquefaction and pipeline damage during both earthquakes are evaluated, including the Marina, South of Market, and Mission Creek areas. Important lessons are summarized about the effects of transient lateral shear strains on pipeline performance, post liquefaction consolidation, use of submerged fill thickness as a microzonation technique for predicting liquefaction severity and potential pipeline damage, the relationship between surface manifestations of liquefaction and subsurface geometry of deposits, and factors affecting the magnitude of lateral spread.

INTRODUCTION

The City of San Francisco provides an excellent case history source for evaluating the site conditions which contribute to soil liquefaction, the mechanisms of large ground deformation which result from this phenomenon, and the influence of such deformation on buried lifeline systems. Liquefaction and large ground deformation have been observed and measured in San Francisco during two earthquakes with significantly different magnitudes, durations, and proximity of fault rupture to sites of interest. The local geologic and fill conditions are representative not only of other areas in the San Francisco Bay region, but have characteristics similar to soft soil and liquefaction-prone areas in many parts of the world. Past earthquake-induced ground failures are illustrative of urban seismic hazards, wherein ground deformation has important repercussions on the infrastructure and lifeline systems.

During both the 1906 San Francisco and 1989 Loma Prieta earthquakes, liquefaction in the city occurred in virtually the same locations with similar effects on buried pipeline systems. If considered strictly for the recurrence of liquefaction, a detailed study of San Francisco sites would be important to clarify subsurface conditions and soil properties related to the different modes of ground deformation observed after both earthquakes. When considered also in the light of lifeline performance, a detailed study of San Francisco sites becomes a critically important exercise in characterizing urban hazards, and has repercussions with respect to emergency response and city planning.

In 1906, the failure of water supply pipelines in zones of liquefaction-induced ground movement seriously affected the fire fighting capabilities in the city. Approximately 500 city blocks burned to the ground, with an additional 35 partially damaged (Gilbert, et al., 1906). This conflagration represents the single worst fire loss in U.S. history. In 1989, the city was again dangerously close to widespread fire loss because of pipeline failure from liquefaction-induced ground movement. When fire erupted in the Marina, the in-ground water distribution systems lacked sufficient pressure to control the blaze. If it had not been for the foresight of fire department personnel who had implemented a Portable Water Supply System (Scawthorn, et al., 1992), fire spread through the closely spaced timber frame buildings of the Marina could have resulted in extensive damage and loss of life.

Figure 1 shows the areas of San Francisco in which soil liquefaction was observed in 1906 and 1989. This paper focuses on the Marina, Mission Creek, and South of Market areas. Subsurface data have been compiled and interpreted for the Marina (O'Rourke, et al., 1992) and the Mission Creek and South of Market areas (Pease and O'Rourke, 1993). The subsurface soil and groundwater conditions are described for each site, with special attention to the thickness of submerged fill and its relationship with liquefaction severity and buried pipeline damage. Large transient lateral shear strains are shown to be an important consequence of soil liquefaction. It is likely that lateral ground displacements associated with these strains are the primary cause of the extensive damage to the water distribution system in the Marina. The influence of surface gradient and thickness of liquefiable layer on the magnitude of soil displacement is evaluated on the basis of subsurface data and observations after the 1906 earthquake. The surface manifestation of liquefaction effects is evaluated with respect to the ratio of the non-liquefiable upper layer

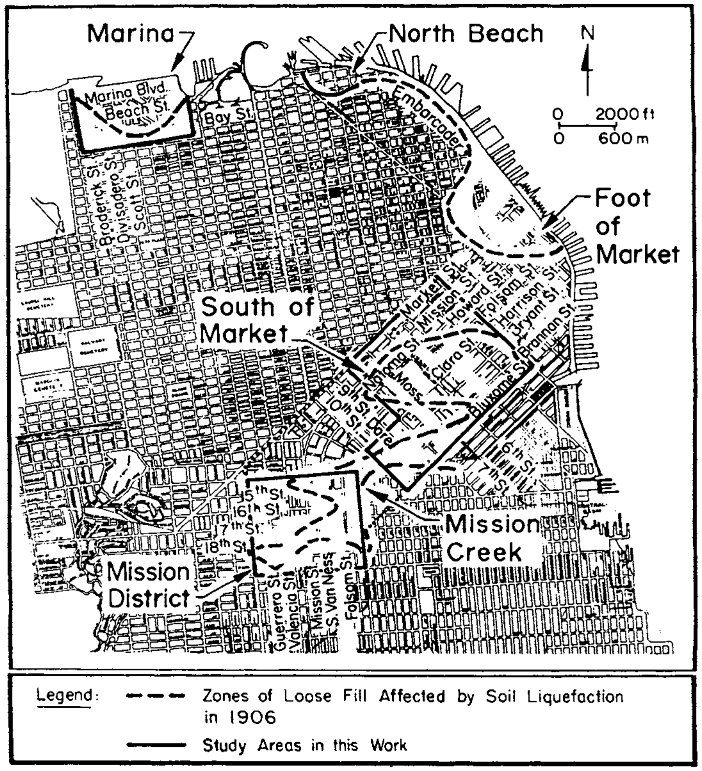

Figure 1. Principal Areas of Soil Liquefaction in the 1906 and 1989 Earthquakes, showing the Marina, Mission District, and South of Market Study Areas

thickness to the thickness of the underlying liquefiable layer. The thickness of liquefiable fill is evaluated for several San Francisco sites and shown to be an excellent index for predicting the severity of liquefaction.

MARINA SOIL CONDITIONS

Soil conditions and liquefaction in the Marina have been discussed extensively by several researchers, including O'Rourke, et al. (1992), Bonilla (1992), and Bardet, et al. (1992). In this paper, the findings of O'Rourke, et al. (1992), Pease, et al. (1992), and Pease and O'Rourke (1995) are summarized to provide a general overview of subsurface conditions.

The Marina District study area defined in this work consists of 1.0 km^2 bounded by Marina Boulevard, Laguna, and Lombard Sts., and the Presidio, and to the north, east, south, and west, respectively. As discussed by Bardet, et al.(1992), soft soil conditions, including recent bay deposits and over 90 m thickness of Pleistocene deposits in the Marina basin, affected site response and ground accelerations. The district is developed largely on fill either dumped or hydraulically placed over natural deposits at shallow depth. Saturated fill in former bays and marshes is the major source of liquefaction in the study area. As pointed out by O'Rourke, et al. (1992), liquefaction characteristics and field performance can be studied for natural, dumped fill, and hydraulic fill deposits. Because the Marina was not well developed in 1906 and only sparse information exists for the effects of the 1906 earthquake at this location, emphasis is placed on liquefaction and lifeline behavior during and after the 1989 Loma Prieta earthquake.

Soil conditions in the Marina District were mapped in three dimensions using a combination of subsurface and historic records. More than 180 borehole records and 15 CPT soundings were assembled from engineering projects before and after the 1989 earthquake. The locations of boreholes and soundings used to assess subsurface conditions are shown in Figure 2. This map also shows the outlines of the 1857 and 1906 shorelines, features which are repeated in all subsequent maps of the Marina in this paper. Detailed descriptions of the historic development of the Marina are provided by Bonilla (1992) and O'Rourke, et al. (1992).

Dividing the land area by the number of borings, there was one exploration per 4900 m^2, or the equivalent of one boring every 70 m on a rectangular spacing. Subsurface records were located and mapped with respect to a rectangular coordinate system based on the street grid. Elevations were interpreted for key subsurface features, such as the water table, fill, Holocene bay mud, and bedrock. Elevations are referenced to the San Francisco City Datum (SFCD). Mean sea level is El. -2.7 m with respect to this datum. The work presented in this paper draws on maps previously developed by O'Rourke, et al. (1992) and Pease, et al. (1992). Contour plots of surface features were generated using the computer program "Surfer" (1987). The program uses a procedure, known as kriging, to perform a statistical evaluation of randomly spaced data and develop an evenly spaced data grid with minimal estimation variance (Ripley, 1987). Surfaces are stored in the computer and generated from rectangular grids of data. Contour lines are plotted to represent the surfaces. Data from two grids can also be manipulated mathematically to produce a third grid, thus providing for the superposition of different surfaces.

Cross-section A-A' in Figure 3 shows the soil profile along Marina Blvd. from approximately Baker St. to Buchanan St. Loose fills extend to a maximum depth of about 9 m. The depth to water table is approximately 2.5 m. Underlying the loose fills and natural sand deposits is Holocene bay mud, which in the cross-section varies from 9 to 32 m thickness. Underlying the mud are dense sand and stiff to hard clay.

The submerged fill thickness was computed by subtracting the water table elevation from the elevation of the base of fill. Elevations of both the water table and the base of fill were mapped using the records from the boreholes shown in Figure 2. The most probable boundary

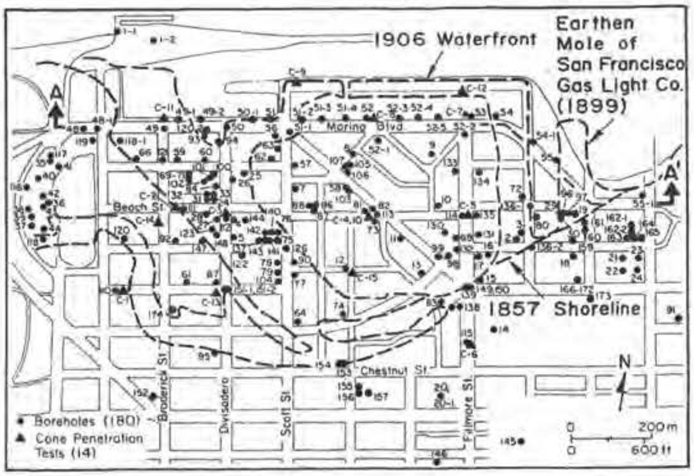

Figure 2. Locations of Conventional Boreholes, Cone Penetration Soundings, and Cross-Section A-A' in the Marina District

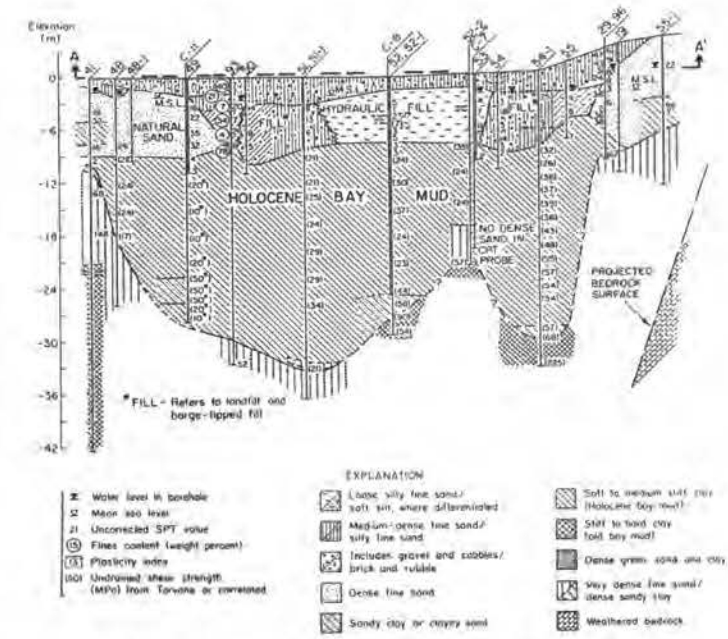

Figure 3. Cross-Section A-A' along Marina Boulevard. Elevation refers to San Francisco City Datum, which is 2.7 m above Mean Sea Level (M.S.L.)

between fill and underlying natural soil was evaluated from changes in soil type, color, penetration resistance, and presence of debris. While penetration resistance in fill varies due to changes in fill density and increased resistance from gravel and rubble, a consistent contrast in density was observed between dumped fill and underlying natural sands. Contrast between grey hydraulic fill and underlying bay mud was difficult to interpret in some locations, although the fill was more heterogeneous and often included some debris at this base. Fill depth in the former marsh is evaluated primarily from historic topography, due to the absence of subsurface records.

The water table is uniformly 2 to 3 m below the ground surface throughout the area, except southeast of Bay and Fillmore Sts., where the ground surface rises rapidly and the water depth is greater than 6 m. A map of the depth of groundwater has been published by Bonilla (1992).

The submerged fill thickness plotted in Figure 4 may be regarded as a map of potentially liquefiable deposits, in which the increasing thickness reflects the potential for increasing liquefaction severity. In most land-tipped fill areas, there is less than 2 m thickness of submerged fill.

MARINA SETTLEMENT

Settlement was calculated from level surveys performed by the City of San Francisco, the U.S. Geological Survey (Bennett, 1990), and Cornell researchers. City surveys provide the baseline of pre-earthquake elevations for all settlements in this work. Data from three successive surveys in 1961, 1974, and October 1990 were obtained from the City of San Francisco Department of Public Works. City survey markers are located only at intersections, so that settlement data for the Marina District are separated at 100 to 150 m horizontal spacings. At each intersection, there are typically 2 to 10 survey marks which consist of permanent, covered monuments and semi-permanent marks on curbs, storm-water catch basin frames, fire hydrants, and structures.

U.S. Geological Survey researchers performed an optical level survey on Divisadero St. and a district-wide survey three and four weeks, respectively, after the earthquake. Optical leveling equipment was used. Typically one survey marker was resurveyed at each intersection using City surveys as pre-earthquake data. Some intersections were not resurveyed. Bennett (1990) identified settlements from 1974 to the November 1989 survey. As noted by Bennett, those field data reflect cumulative settlements between 1974 and 1989.

Survey records were collected from the City for October 1990, and surveys were conducted by Cornell researchers in July 1992 and March 1993. Measurements were obtained at intersections not already surveyed after the earthquake. Additional data were obtained for areas of lesser damage in the area of Strawberry Island Marsh and the southern end of Marina Cove. These locations are of interest because they overlie shallow deposits of submerged fill.

Following the work of O'Rourke, et al. (1992) and Pease, et al. (1992), it is assumed that the survey of November 1989 includes the effects of post-liquefaction consolidation. To

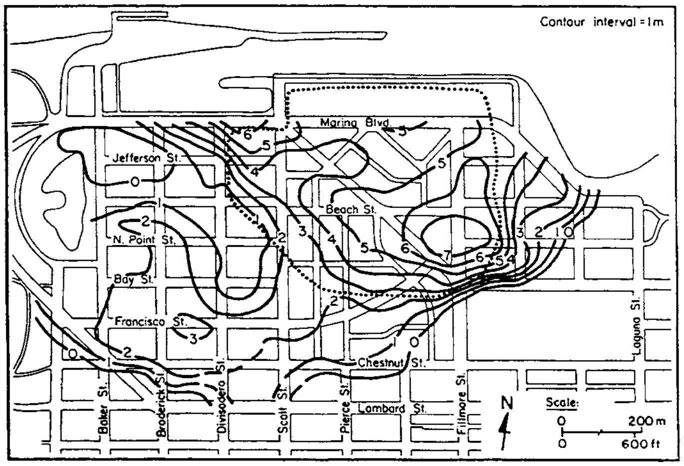

Figure 4. Thickness of Submerged Fill Deposits in the Marina District

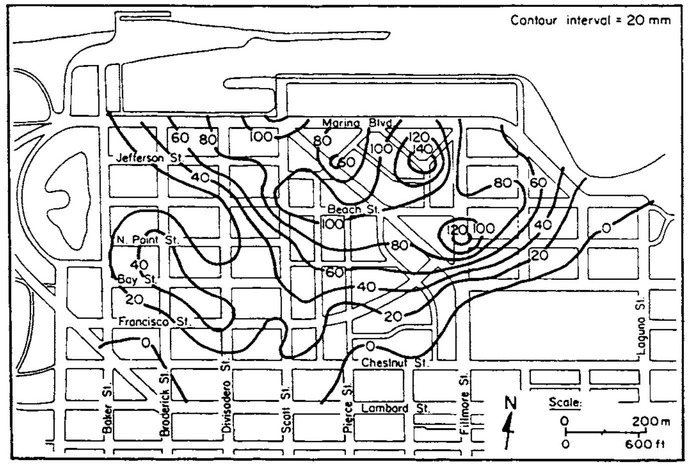

Figure 5. Contours of Corrected Settlement in mm in the Marina District

evaluate the liquefaction-related settlement, it is necessary to subtract movement associated with time-dependent consolidation of Holocene bay deposits from the survey measurements.

Secondary settlements for the survey period including the earthquake, 1974 to 1989, were calculated according to procedures explained by O'Rourke, et al. (1992), and at each intersection were subtracted from the unadjusted settlements to yield "corrected" settlements. These corrected settlements, in Figure 5, represent the settlement caused by post-liquefaction consolidation after the Loma Prieta earthquake.

The pattern of liquefaction settlement strongly reflects the distribution and thickness of submerged fill in Figure 4. Magnitude of settlement was less than 5 mm for streets southeast of Marina Cove overlying former sand dunes, and was from 0 to 10 mm in locations overlying the highest portions of former Strawberry Island. Settlement of land-tipped fill exceeds 80 mm between Divisadero and Scott Sts. where dumped fill thickness increases rapidly toward San Francisco Bay. Significant settlements also occurred inshore of Strawberry Island to the southwest, in the former marsh. From 40 to 60 mm of settlement occur overlying the former tidal channel in the marsh under Broderick and Divisadero Sts., which represents the location of locally thickest fill. Settlement is largest in areas underlain by hydraulic fill, where corrected survey measurements identify from 40 to 170 mm settlement. Uneven settlement occurred in the hydraulic fill, possibly due to large lateral variations in soil consistency and in thickness of non-liquefiable lenses in the fills (O'Rourke, et al., 1992).

PIPELINE PERFORMANCE IN THE MARINA

Pipeline performance in the Marina has been described in detail by O'Rourke, et al. (1991, 1992). In this work, the earthquake response of water supply piping is covered by first summarizing the principal findings of previous investigations and then relating pipeline damage to earthquake-induced ground deformation. Whereas previous studies have shown a strong correlation between pipeline damage and permanent ground movement related to post-liquefaction consolidation (O'Rourke, et al. 1992), this work focuses on the effects of transient motion, primarily in the form of ground oscillation. As described by Youd (1984), ground oscillation results from the surface layer overlying the liquefied deposit vibrating in a different mode from adjacent firm ground, causing dynamic opening and closing of fissures among surface blocks and firm ground. This type of movement is shown to be a primary source of pipeline deformation and a more plausible cause of damage than the differential settlement arising from post-liquefaction consolidation.

Water to the Marina District is supplied by two systems of pipelines: the Municipal Water Supply System (MWSS) and the Auxiliary Water Supply System (AWSS). The MWSS supplies potable water for domestic and commercial uses, as well as for firefighting via hydrants and sprinkler systems; the AWSS supplies water exclusively for firefighting purposes. Within the area bounded by the 1857 shoreline on the south and the current shoreline, there are approximately 11.3 km of pipeline belonging to the MWSS and 2.3 km of pipeline belonging to the AWSS. The MWSS mains are predominantly 100, 150, 200, and 300 mm in diameter, whereas the AWSS mains are predominantly 250 and 300 mm in diameter. The pipelines in both systems are composed of pit-cast iron, and many were installed between late 1924 and 1925. The MWSS pipelines were built predominantly with cement-caulked, bell-and-spigot couplings, whereas the AWSS pipelines were built with special couplings to allow rotational and axial flexibility. All pipes were buried at nominal depths to top of pipe between 0.9 and 1.2 m.

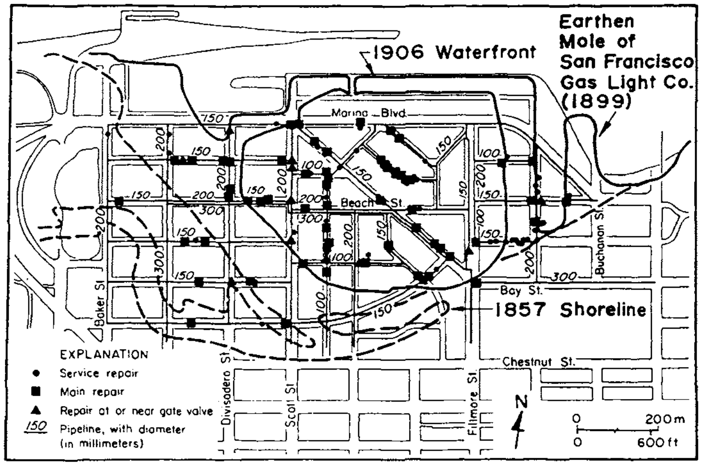

Figure 7. Contours of MWSS Repair Rates per 300 m in the Marina District, including Damage to Mains, Service Lines, and Sections Near Gate Valves

The locations of MWSS pipelines and repairs relative to the street system, 1906 waterfront, and 1857 shoreline, are shown in Figure 6. Repairs were made at points of sheared or disengaged service connections with mains, flexural round cracks in mains, and longitudinally split sections of mains. In some places, damage was concentrated at or near gate valves, which tend to anchor the pipelines and therefore may contribute to locally pronounced deformations and stresses.

A total of 123 repairs were made to the MWSS mains and services in the Marina District, more than three times the number in the entire MWSS elsewhere. A total of 69 repairs were made to mains, including those at or near gate valves; more than 80 percent of these repairs were attributed to round cracks. In contrast, only one leaking joint was found in a 300-mm-diameter AWSS pipeline out of 2.3 km of 250-and 300-mm-diameter pipeline within the same area described above, resulting in a repair rate of only 0.43 per km. This repair was at the intersection of Beach and Scott Sts., at a pipeline junction within one half block of the boundary of hydraulic fill.

To represent the distribution of MWSS damage, the Marina District was divided into a grid of approximately 40 cells and the number of repairs within each cell was counted (O'Rourke, et al., 1992). The repairs were then normalized with respect to the reference length of 300 m to provide a consistent basis for evaluation. Contours of equal repair rates were drawn and superimposed on the street system and previous shorelines, as shown in Figure 7. Inspection of Figures 5 and 7 shows that the closely spaced settlement contours, indicating the largest local settlement slopes correspond to the highest repair-rate contours. High concentrations of pipeline repair fall within the

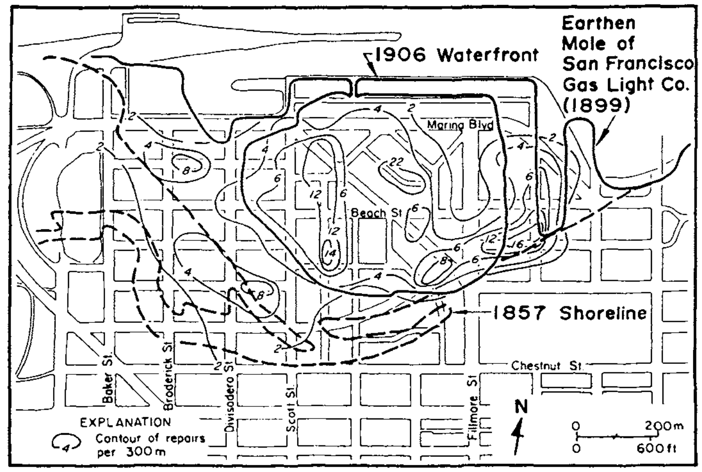

Figure 6. Repairs to MWSS Mains, Service Lines, and Sections Near Gate Valves in the Marina District

area of hydraulic fill. The heaviest repair concentration occurs at the junction of the hydraulic fill, seawall, and 1857 shoreline, except for an isolated area on Rico Way, where unusual constraints occurred as a result of pipeline construction along the curved street.

To explore further the relation between pipeline damage and settlement, O'Rourke, et al. (1992) correlated MWSS repair rates with both settlement magnitude and slope of the local settlement profile. The MWSS pipeline repairs within a half-block of each intersection in all street directions were divided by the total length of pipe within this area and correlated with the settlement measured at each intersection. In addition, the MWSS pipeline repairs along each block were divided by the total length of pipeline and correlated with the local settlement slope. O'Rourke, et al. (1992) refer to local settlement slope as angular distortion, and this term is retained in this paper.

Only a weak correlation was found between repair rate and the magnitude of settlement. Relatively good correlations were found between repair rate and angular distortion. Regressions were developed for each diameter of main, and the resulting relations between MWSS repair rate and the angular distortion are plotted in Figure 8. An equation and coefficient of determination, r^2, are given for each regression curve in Figure 8. The slopes of these plots increase in inverse proportion to the nominal pipe diameter. For 200 mm diameter mains, a bilinear plot (dashed curve, Fig. 8) also conforms with the data.

The damage mapped in Figures 6 and 7 and its close relation to the pattern of settlement mapped in Figure 5 indicate a strong link between pipeline damage and differential settlement that is corroborated further by correlations plotted in Figure 8.

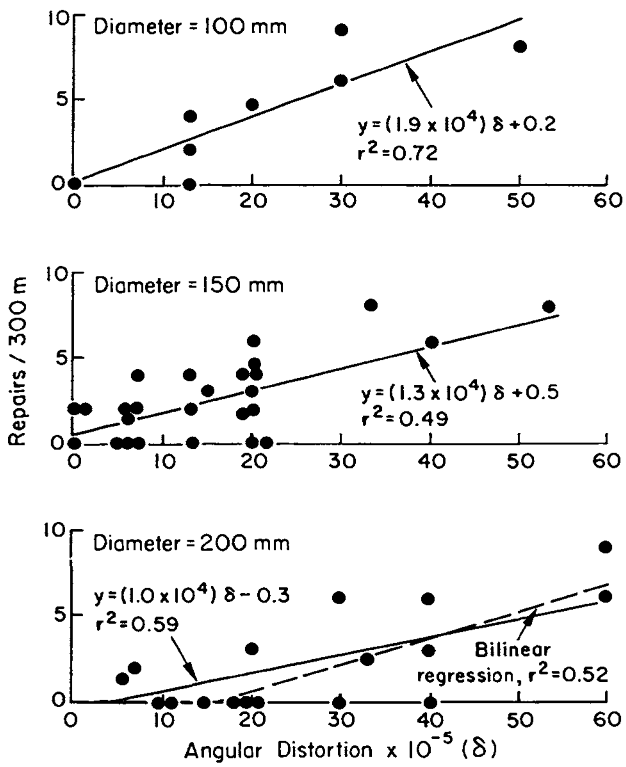

Figure 8. Linear Regressions of MWSS Repair Rate Versus Angular Distortion for Water Mains 100, 150, and 200 mm in Diameter

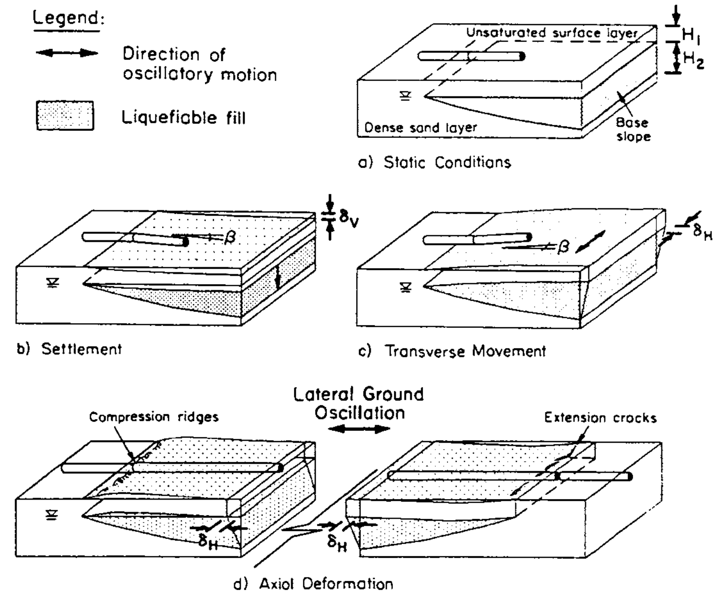

Figure 9. Schematic Comparison (a) of Pipeline Deformations Above a Liquefied Deposit (b) Due to Settlement, and (c, d) Due to Transverse and Parallel Ground Oscillation

Settlement causes damage to small diameter pipelines by means of longitudinal curvature and consequent bending strains which result from interaction between the ground and pipe. If it is assumed that the pipeline is relatively flexible (as would be appropriate for 100, 150, and 200 mm diameter pipe lines) and deforms as the ground deforms, the maximum longitudinal bending strain in the pipe, ε_b, is given by:

$$\varepsilon_b = \frac{\kappa D}{2} \qquad (1)$$

in which κ is the pipeline or ground curvature and D is the outside pipe diameter. Curvature is equal to the second derivative of the settlement profile.

Given the contours of settlement in Figure 5, it is a relatively simple matter to compute the second derivative of settlement and substitute it into Equation 1 to develop a spatial distribution of maximum bending strain sustained by pipelines with different diameters. Calculations performed in this manner, however, show a very low level of bending strain, on the order of 1 to 10 $\mu\varepsilon$, which is roughly two orders of magnitude below the level necessary for tensile failure of pit cast iron.

The inconsistency between bending strains determined from differential settlement and the strain level required for failure means that an alternative source of pipeline distortion needs to be identified. The level of this distortion must be compatible with the failure strain of pit cast iron. Moreover, since pipeline repair rates correlate well with angular distortion, the deformation mechanisms must also be able to explain why damage is so closely correlated with differential settlement.

Measurements at instrumented sites of liquefaction have shown relatively large transient shear strains in liquefied soils which are compatible with the type of deformation associated with strong ground oscillation. For example, Zeghal and Elgamal (1994) have used strong motion recordings and pore pressure measurements at the Wildlife Test Site to assess shear strain, stress, and pore pressure response during the 1987 Superstition Hills earthquake. Their analysis of the records indicates that lateral shear strains as high as 1.5% were sustained in the liquefiable sands. Iai, et al. (1994) presents strong motion displacement data near quay walls affected by the 1993 Kushiro earthquake. The records show transient surface displacement of 200 mm and suggests that transient lateral strains of approximately 2 to 3% were experienced. Analyses of recorded and simulated strong motion records at Treasure Island described by Pease and O'Rourke (1995) suggest that lateral shear strains of approximately 2% were experienced by the liquefiable fill at this site.

When integrated over the thickness of liquefiable fill, transient lateral shear strains can have a strong influence on the lateral deformation imposed on buried pipelines. Figure 9 illustrates how buried pipelines are affected by ground deformation arising from settlement caused by post-liquefaction consolidation (Figure 9b) and by lateral shear strains (Figure 9c and d). As an approximation, surficial soil and pipeline lateral movements are equal to the product of the average lateral shear strain and the thickness of soil subjected to liquefaction. The magnitude of lateral displacement, therefore, will vary in direct proportion to the thickness of the liquefiable fill. As illustrated in Figure 9, variations in lateral displacement are largest where variations in submerged fill thickness are largest, which frequently occurs along the margins of the fill.

Figure 9d shows that lateral ground strains can result in axial compressive and tensile strains in a buried pipeline. If it is assumed that the pipeline deforms axially as the ground deforms (as would be appropriate for relatively thin wall pipe anchored in the ground by multiple service connections and tees), the maximum axial strain in the pipe, ε_a, is given by:

$$\varepsilon_a = \Delta\delta_H/L \tag{2}$$

in which $\Delta\delta_H$ is the differential lateral displacement in the upper, non-liquefiable layer over a horizontal distance, L. Given the pattern of transient lateral shear strains and the contours of submerged fill thickness in Figure 4, it is a relatively simple matter to determine the areal distribution of lateral displacement and take the first derivative of this displacement pattern in a direction parallel to buried piping to develop a spatial distribution of maximum axial strain sustained in the pipelines.

Evaluation of microstructure, casting practices, and laboratory tests on cast iron pipe specimens have been performed as part of a detailed study of cast iron pipeline response to permanent ground deformation (Taki and O'Rourke, 1984; O'Rourke and Harris, 1983). These investigations have shown that the threshold for acceptable tensile strain is between 500 to 600 $\mu\varepsilon$ for cast iron pipe. Strains imposed by ground deformation above this level are not advisable because of increased risk with respect to tensile failure of the pipe. Limiting tensile strain, ε_a, also can be evaluated in relation to the pull-out capacity of cast iron joints by means of the following expression:

$$\varepsilon_a = \frac{fd}{Et} \tag{3}$$

in which f is the adhesive shear strength mobilized between the caulking material and cast iron joint surface, d is depth of caulking, E is the Young's modulus of cast iron, and t is the pipe wall thickness.

The great majority of MWSS pipelines in the Marina were constructed with cement caulked joints (O'Rourke, et al., 1992). Given typical values of E = 69 GPa (Taki and O'Rourke, 1984), d = 38 mm (O'Rourke and Trautmann, 1980), and f = 3.5 to 5.0 MPa (Committee on Cast Iron Pipe Joints, 1915), the tensile strain consistent with joint pull-out for a nominal 150-mm-diameter pipeline with 11-mm wall thickness is between 170 and 250 $\mu\varepsilon$.

As mentioned previously, the maximum transient lateral shear strain in submerged fill at Treasure Island has been estimated to be approximately 2% in the north-south direction, which corresponds to the azimuth of strongest recorded ground acceleration. Similarly, a lateral shear strain of 1.25% in the east-west direction has been estimated from a ratio of 1.6 between north-south and east-west components of maximum ground motion. Using these strains at the Marina, in conjunction with the submerged fill thickness in Figure 4, computer analyses were used to determine lateral displacement patterns for both the north-south and east-west alignments. Taking the first derivatives of these functions at various locations, the maximum lateral ground strains affecting north-south and east-west oriented pipelines were calculated to delineate zones of maximum lateral strain affecting buried pipelines in Figure 10. These zones are compared with areas which experienced the highest concentrations of pipeline damage of more than 6 repairs/300 m.

Of particular significance is that lateral ground strains varying from 200 to 900 $\mu\varepsilon$ are predicted over a significant portion of the submerged fill. These levels of lateral strain, unlike the bending strains calculated from differential settlement, are fully compatible with levels sufficient to cause damage in cast iron pipelines. Moreover, the lateral ground strains are similar in magnitude to the angular distortion calculated from the post-liquefaction settlement contours. This similarity arises because average vertical strains caused by post-liquefaction consolidation are approximately 2% on average (O'Rourke, et al., 1992), which is approximately equal to the best estimate of transient lateral shear strain for the Marina. The use of lateral ground strain caused by oscillatory motion of the liquefied fill provides, therefore, an alternative source of pipeline deformation which not only is compatible with the failure threshold of cast iron pipelines, but also is able to explain why pipeline damage is so closely correlated with differential settlement.

There is good agreement between the areas of maximum lateral ground strain in Figure 10 and those with the highest concentration of pipeline damage. In particular, there is close agreement along the southeastern and western margins of the fill.

Although pipeline damage correlates well with the location of maximum lateral ground strain, it should be recognized that the actual causes of pipeline damage are related to complex interactions which involve large axial tensile

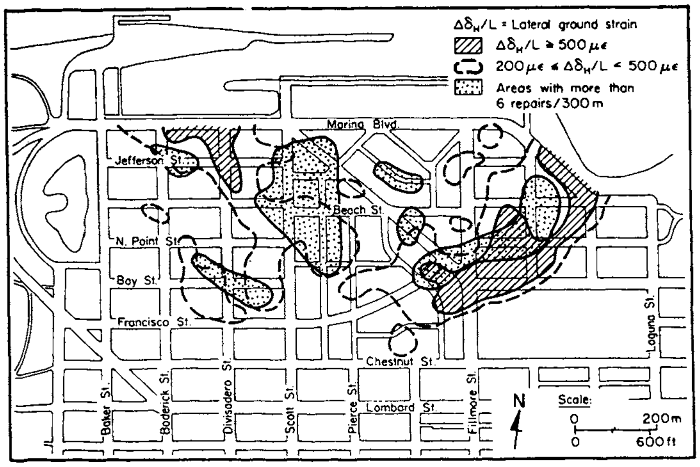

Figure 10. Maximum Lateral Ground Strains Resulting from Ground Oscillation with 1.25 to 2.0% Shear Strain, and Regions with Pipe Damage Exceeding 6 repairs/300 m

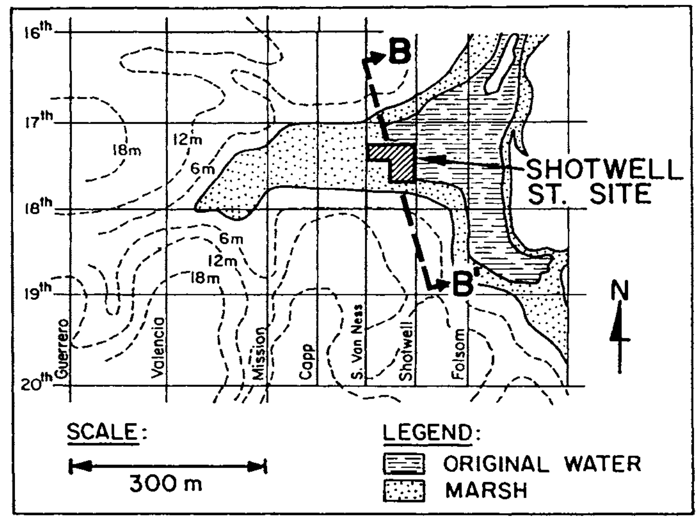

Figure 11. 1857 Topography, Marsh and Bay Regions, and Location of Section B-B' in the Mission District

strains, joint pull-out failures, abrupt vertical settlement, concentrated compression at buckled and heaved surfaces, excessive bending at locations of local pipeline restraint, and abrupt lateral offsets in the soil. Rather than viewing lateral ground strain as a cause of axial failure, it is more appropriate to regard the strain as an index of local deformation imposed during the earthquake. In this way, lateral ground strain is a measure of the severity of ground oscillation as well as the associated multiple effects of liquefaction on buried pipeline performance.

MISSION CREEK SOIL AND SITE CONDITIONS

Mission Creek refers to a former north-south estuary in the Mission District between Folsom and Harrison Sts. from 15th to 22nd Sts., which connected to Mission Bay though the South of Market area. A major branch of Mission Creek flowed from the west between 17th and 19th Sts., where it joined the estuary. Consequently, in this paper the estuary and valley to the east of Shotwell St. are referred to as Lower Mission Creek, and the stream and ravine to the west of Shotwell St. are referred to as Upper Mission Creek. The region was urbanized between 1860 and 1890. Ravines and estuaries were filled during this period resulting in deposits of loose, cohesionless fine sand which are susceptible to liquefaction.

The 1.2 km^2 area, referred to as the Mission District in this work, is shown in Figure 11. The original water and marsh areas are shown in the figure, together with topographic contours mapped in 1853 (U.S. Coast Survey, 1853). Subsurface investigations were performed by Cornell researchers for USGS throughout this area, with one of the locations of concentrated exploration between Shotwell St. and South Van Ness Ave. as shown on the map.

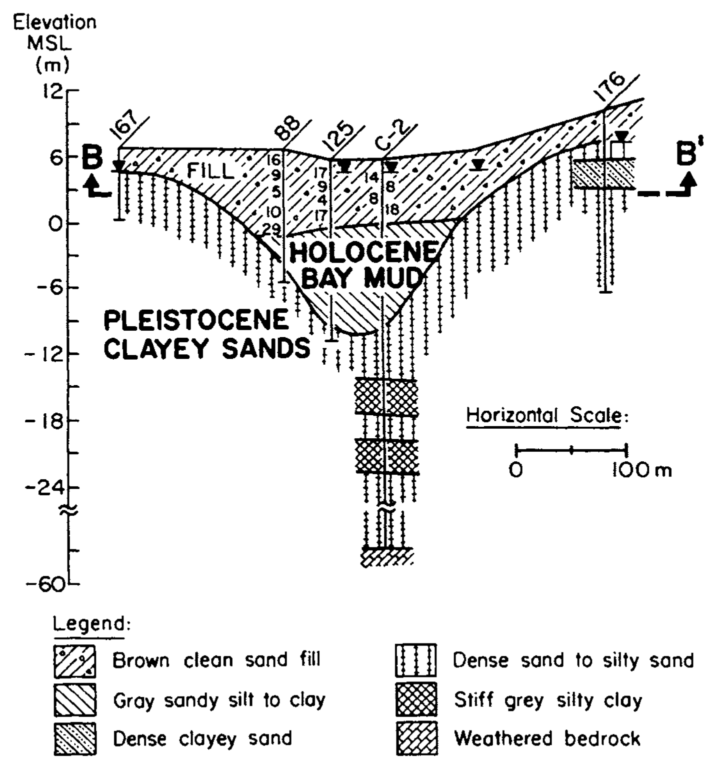

Figure 12. Cross-Section B-B' in the Mission District

Cross-section B-B' in Figure 12 shows the subsurface conditions encountered in the transition area between Upper and Lower Mission Creek. The soil profile was drawn on the basis of conventional borings and CPT soundings performed by Cornell researchers, as well as conventional borings collected for various construction projects. A database of 146 boreholes, soundings, and surface excavations was collected to evaluate subsurface conditions. A detailed description of the data collection and mapping procedures is given by Pease and O'Rourke (1993).

The soil profile consists of loose, relatively clean sand fill to a maximum depth of approximately 7 m near the center of the old Mission Creek ravine. The water table is approximately 1 m below ground surface near the ravine center. Underlying the fill is Holocene bay mud with maximum thickness of approximately 9 m. Dense sands and stiff clay extend from the base of the mud to a depth of approximately 58 m where Franciscan bedrock is encountered.

Thicknesses of submerged fill were obtained by subtracting elevation surfaces for the base of fill from the elevation of the water table. Submerged fill thickness is mapped in Figure 13. Contours indicate fill thickness below the water table in meters. The zero contour indicates where the water table is at the level of the base of fill, and represents a theoretical boundary between regions where liquefaction can potentially occur and where it cannot occur. As a practical matter, a dashed upper bound contour has been drawn to represent a margin to accommodate uncertainties in elevations of the fill and the groundwater table. The upper bound contour represents locations where groundwater levels are within 2 m of the base of the fill deposit. This line is proposed to indicate reasonable limits on the extent of liquefaction in the case of non-uniform changes in fill thickness and variations in the water level.

Severity of liquefaction is likely to be greatest for thick submerged fill layers. In Figure 13, 4 to 8 m of submerged fill are observed at both Valencia and Shotwell Sts. in former Upper Mission Creek. Only 2 to 3 m of submerged fill are observed under Mission near 18th St. As much as 4 m of submerged fill are observed in former estuarine areas of Lower Mission Creek.

Since liquefiable thickness is determined on the basis of submerged deposits, it follows that the depth to groundwater represents the thickness of overlying non-liquefiable soils. In Upper Mission Creek in the Mission District, unsaturated surface fills may be between 2 and 6 m in thickness. In Lower Mission Creek, potentially liquefiable zones are capped with only 1 to 3 m of unsaturated cover. Increased thickness of non-liquefiable deposits increases the effective confining stresses in the liquefiable zone and may mitigate the effects of liquefaction on surface distortion.

Current surface contours are illustrated in Figure 14 for the Mission District. This map is based on survey data at major street intersections by the City of San Francisco Department of Public Works in 1973 and 1991. The contours are based on data at roughly 190 m spacings. Contours intervals are 2 m with respect to San Francisco City Datum. Average surface slopes in percent between intersection are shown in italics for selected streets.

The Mission District has relatively gradual surface gradients dipping toward the east with slopes of 2.5 percent or less. Steeper slopes occur at bedrock spurs at 20th and Dolores and 14th and Dolores Sts. Between 17th and 19th Sts., the former Upper Mission Creek ravine is shown by the looping of contours between Dolores and Valencia; below Valencia St. the ravine is not apparent. Along 18th St. from Valencia St. to South Van Ness Ave., typical surface slope of the filled ravine is 1.1 percent. Uphill, from Valencia to Guerrero Sts., the gradient of 18th St. decreases to 0.6 percent. Lower Mission Creek in the vicinity of Folsom between 14th and 18th Sts. has little or no surface gradient.

1906 LIQUEFACTION IN MISSION CREEK

Figure 15 is a map of ground deformations and liquefaction features observed after the 1906 earthquake in the Mission District. The deformation patterns were previously mapped by O'Rourke and Lane(1989), and have been superimposed over contours of submerged fill thickness. Hatchured lines indicate settlement, with the hatchures pointing toward the area of settlement.

This symbol may indicate either gradual subsidence or an abrupt scarp. Sub-parallel lines indicate lateral spreads, the magnitude of which is shown by the separation between lines. Paired arrows indicate the location of abrupt compressional, extensional, or lateral offsets of pavement or streetcar tracks. Lack of historic data in a given area may indicate that relatively minor damage occurred in that area compared to areas that were documented.

A striking feature of Figure 15 is the occurrence of lateral spread in areas with roughly 2 m or more of submerged fill. A major lateral spread is centered on Valencia St. with over 6 m of submerged fill. The ground under Valencia St. spread east and slightly northward down the center of the former ravine, with maximum lateral displacements of 1.8 m to 2.4 m and settlement of 1.5 m. A second area of lateral spread occurred between Capp St. and South Van Ness Ave, where maximum lateral movement of 1.2 m occurred. The thickness of submerged fill is greater than 2 m in this area, and increases towards the east. Maximum lateral movement and settlement of 0.3 m occurred over a width of 120 m across Mission Street between these two sites. Given the relatively small movement on Mission St., it is possible that lateral spread in Upper Mission Creek may not have been a single continuous feature, but may have consisted of two or more discontinuous features occurring in locations where submerged fill is thickest.

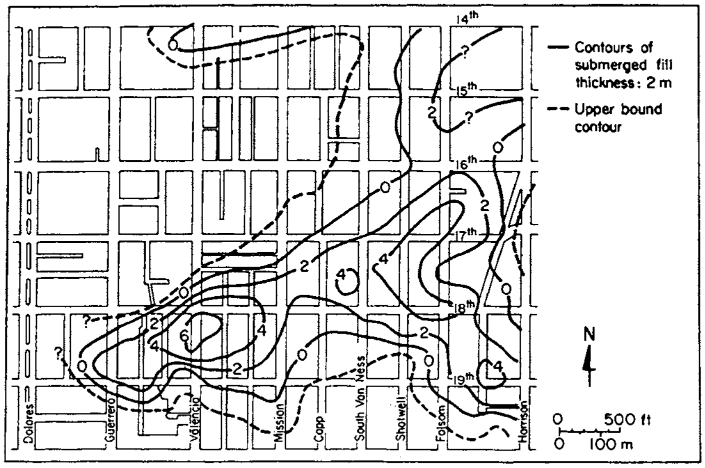

Figure 13. Thickness of Submerged Fill Deposits in the Mission District

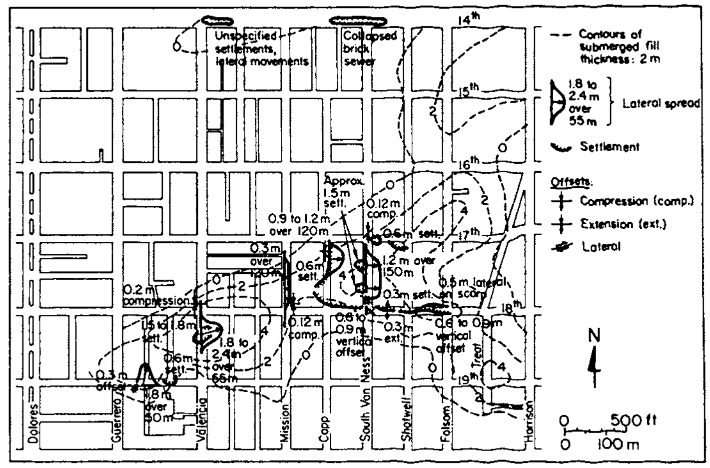

Figure 15. Ground Displacement and Liquefaction Features after the 1906 Earthquake in the Mission District

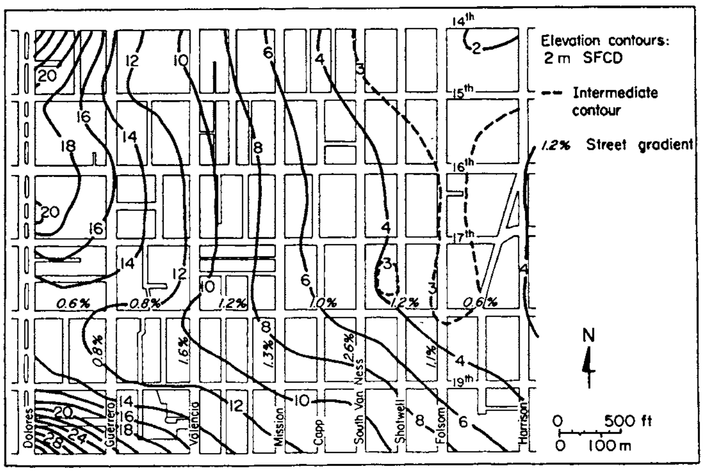

Figure 14. Current Surface Elevations and Street Gradients in the Mission District

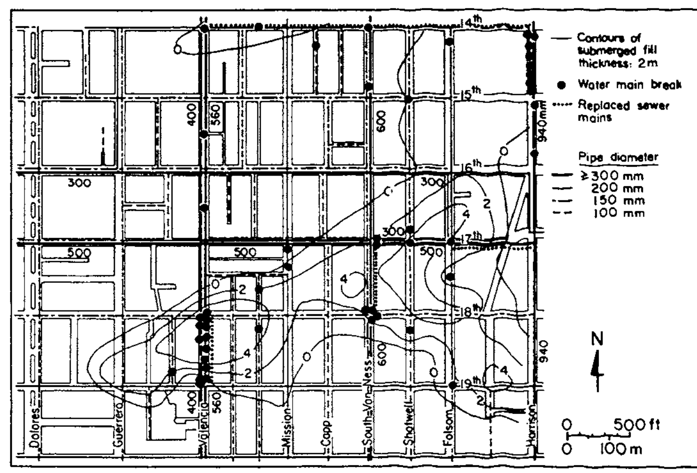

Figure 16. Water Supply Pipeline and Sewer Damage in 1906 in the Mission District

Nearly all liquefaction features were confined within zones of submerged fill. Compressional, extensional, or lateral offsets were prominent at the edges of submerged fill where fill is between 1 and 4 m thick. Buckled curbs, major extension cracks, and distortion of pavements were prominent on 18th St between Mission and Folsom Sts. on the south side of the liquefaction zone. Cracks were widest near Folsom St., where submerged fill thickness is as deep as 4 m.

Settlements, lateral movements, and damage to water mains and sewers occurred on 14th St. between Valencia St. and South Van Ness Ave where there is also a layer of submerged fill (Schussler, 1906, Derleth, 1906). Extensive damage to building cripple walls and foundation cracking occurred on Folsom and Treat Sts. for two or three blocks south of 18th St. (Lawson, et al., 1908). This area coincides roughly with the limits of submerged fill in the southern branch of Mission Creek.

Figure 16 shows damage after the 1906 earthquake to the water supply and sewer system in the Mission District superimposed on the contours of liquefiable thickness. The locations and sizes of water mains are based on 1912 maps of the water distribution system (Edward Denny and Co., 1912). Water main breaks, shown by dark circles, refer to breaks reported by the Spring Valley Water Company (Schussler, 1906, Manson, 1908). In some cases where pipelines cross or run parallel, it is uncertain which pipes were broken. More than one break occurred at locations indicated by Manson on South Van Ness at 17th and 18th, and multiple breaks may have occurred at other locations as well. Sewer damage is based on repairs reported by the City of San Francisco (Schussler, 1906).

Fifty water main breaks, denoted by solid dots, were reported in the Mission District. Eighty percent of these breaks occurred in the zone overlying submerged fill. Two major areas of breakage occurred on major trunk lines across the submerged fill zones at Valencia St. and Harrison St. Between Valencia and Harrison Sts., breaks were less concentrated and tend to be located toward the margins, rather than the center of the submerged fill. In many cases they correspond to the locations of scarps and offsets in Figure 15.

1989 LIQUEFACTION IN THE MISSION DISTRICT

In the 1989 Loma Prieta earthquake, liquefaction in the Mission District resulted in light to moderate damage associated with sand boils, settlement, pavement cracking, and strong ground shaking, which was limited to Lower Mission Creek. The most significant damage occurred between South Van Ness, Shotwell, 17th, and 18th Sts. Damage on both sides of Shotwell included sand boils, building settlement, tilting, and structural damage. Liquefaction damage in this area extended to Folsom St. between 17th and 18th Sts., where settlement and sand boils were observed.

Two pipeline breaks occurred. A 150-mm-diameter MWSS pipe broke near Shotwell and 18th Sts. A hydrant connection for the AWSS was broken at 18th and Folsom Sts.

No liquefaction features were observed west of South Van Ness Ave., suggesting that liquefaction was absent in Upper Mission Creek. A possible explanation may be that liquefaction in 1989 was limited to areas overlying Holocene bay mud which produced site amplification effects. However, soft Holocene bay mud as thick as 11 m extends as much as one block west of South Van Ness Ave. in Upper Mission Creek. O'Rourke, et al.(1992) suggest that Holocene Bay mud at Shotwell St. and Holocene alluvial clay at Valencia St., in combination with deep soil profiles, produced similar levels of site amplification, despite liquefaction having occurred at only the Shotwell site. They point out that the deeper groundwater levels in Upper Mission Creek would have resulted in greater confining stresses, thereby increasing the threshold level of ground motions required for liquefaction. It is likely that greater confining stresses, in combination with a larger thickness of non-liquefiable soil near the surface, contributed to greater resistance against liquefaction as well as reduced opportunities for its expression at the ground surface.

SOUTH OF MARKET SOIL AND SITE CONDITIONS

A study area of 2.2 km^2, shown in Figure 17, is located in the South of Market, in a region bounded by Market and Townsend Sts. between 3rd and 8th Sts., and by Harrison and Division Sts. between 8th and 11th Sts. It includes two areas of major lateral spread in 1906: the former Sullivan Marsh and the channel of Lower Mission Creek near Dore St. Both these areas were tidal lands, artificially filled with soil from adjacent sand dunes. Liquefaction-induced damage was concentrated in the same zones after the 1989 Loma Prieta earthquake.

Cross-section C-C' in Figure 18 shows the subsurface conditions encountered along the western margin of buried marsh land, once referred to as Sullivan's Marsh. The soil profile was drawn on the basis of conventional borings and CPT soundings performed by Cornell researchers, as well as conventional borings collected for various construction projects. A database of 306 boreholes, soundings and surface excavations was collected to evaluate subsurface conditions. A detailed description of the data collection and mapping procedures is given by Pease and O'Rourke (1993).

The soil profile consists of relatively loose sands with variable fines content to a depth of approximately 6 to 9 m across much of the site. The water table is at a depth of 3 to 4 m below ground surface. The fill is underlain by a relatively thin layer of peat. Under the central portion of the site is a buried valley filled with Holocene bay mud with maximum thickness of approximately 24 m. Dense sands and stiff clays underlie the bay mud to depths in the range of 62 m below ground surface where bedrock is encountered.

Extent and thickness of liquefiable deposits were evaluated by the same methodologies as described for the Marina District. Submerged fill thickness is mapped in Figure 19. Contours indicate fill thickness below the water table in meters. The zero contour indicates where the water table is at the level of the base of fill, and represents a theoretical boundary between regions where liquefaction can potentially occur and where it cannot occur. As a practical matter, a dashed upper bound contour has been drawn to represent a margin to accommodate uncertainties in elevations of the fill and the groundwater table. The upper bound contour represents locations where groundwater levels are within 2 m of the base of the fill deposit.

In Figure 19, approximately 6 m of saturated, loose fill occurs at 7th St. from Mission to Howard Sts. Submerged fill also exceeds 4 m along Mission Creek at Dore and Brannan Sts., 5th and Harrison Sts., and 6th and Townsend Sts. These areas overlie narrow buried ravines

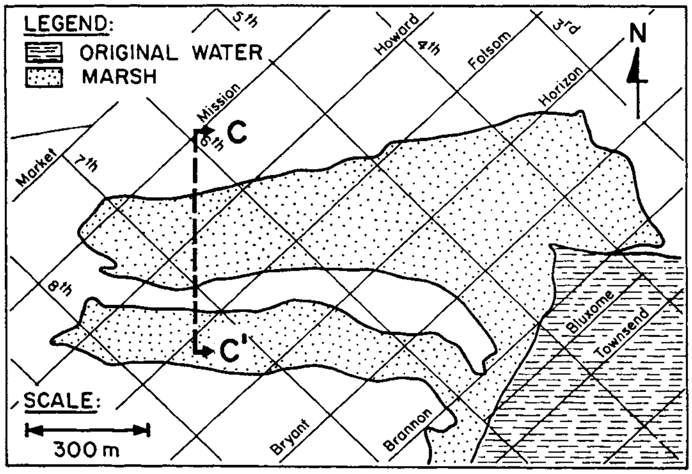

Figure 17. Boundaries of Marsh, Mission Bay, and Location of Section C-C' in the South of Market

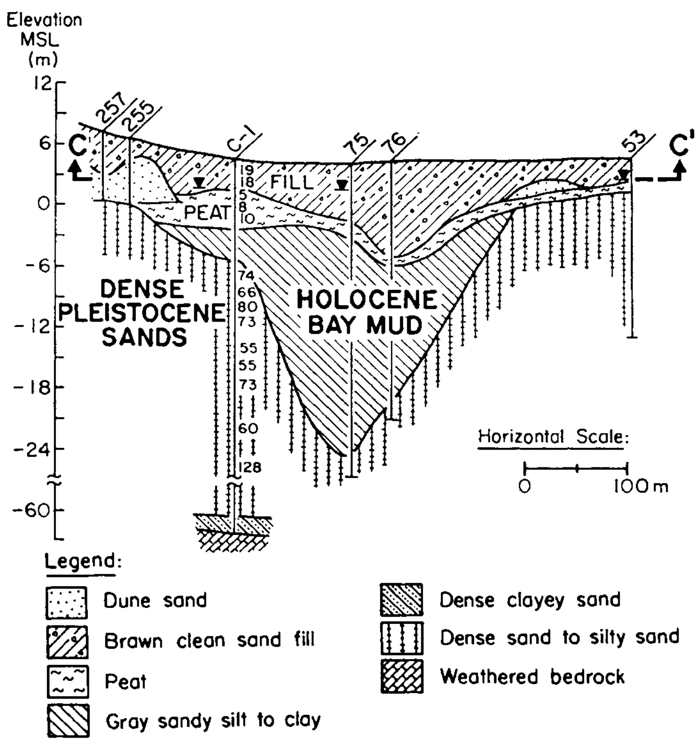

Figure 18. Cross-Section C-C' in the South of Market

filled with Holocene bay mud as thick as 30 m. Submerged fill thickness is 2 m or greater throughout the former marsh area.

The depth of groundwater represents the thickness of non-liquefiable soils overlying a liquefiable deposit. In the South of Market, Sullivan Marsh and Lower Mission Creek submerged fills are overlain by less than 2 m of overlying soil. Near Mission and Howard Sts., the surface layer becomes 4 to 6 m in thickness. Isolated areas of loose saturated fill, which may exist in the former dune fields near Market St., are overlain by 4 to 8 m of non-saturated soils.

Current surface contours are illustrated in Figure 20. The map is based on survey data acquired at major street intersections by the City of San Francisco Department of Public Works, between 1985 and 1991. The contours are based on data at roughly 200 to 250 m spacings. Contours intervals are 2 m with respect to San Francisco City Datum. Average surface slopes in percent between intersections are shown in italics for selected streets.

Contours indicate that South of Market has shallow grades of 1 percent on average from northwest to southeast. A relatively steep gradient of 2.3 percent occurs between Mission and Howard on 7th St. and a slope of 3.2 percent between Mission and Howard on 6th St. From 3rd to 5th Sts., similar gradients occur on the north side of the former marsh, but they lie outside of the submerged fill zone. Between Howard and Folsom Sts., some gradients are on the order of 1 percent Between Folsom and Townsend Sts. in Sullivan Marsh, surface gradients are negligible.

1906 LIQUEFACTION IN THE SOUTH OF MARKET

Figure 21 is a map of ground deformations and liquefaction features observed after the 1906 earthquake in the South of Market. The deformation patterns were mapped previously by O'Rourke and Lane (1989), and are superimposed over contours of submerged fill thickness. Similar symbols are used to describe liquefaction ground deformations. A heavy dotted line shows the outline of the major zones of subsidence as reported by Schussler (1906). Schussler indicates that lateral spread, offsets, and wavelike deformation were common throughout the area. Lawson, et al. (1908) reported settlements of amounts varying from a few millimeters to 900 mm or more throughout the former Sullivan Marsh.

Lateral displacement of 1.5 to 2.4 m was observed roughly parallel to 7th St. between Mission and Howard Sts. (Reynolds, 1906). The northern extent of ground failure was near the U.S Post Office at 7th and Mission, and was well documented by contemporary photographs. Maximum eastward displacement of 1.5 m and settlement of 1.5 m were noted at that corner. Submerged fill is as thick as 6 m, and a street gradient of 2.3 percent are present under this block. Lateral displacement of streets 0.9 to 1.8 m eastward was typical further east in Sullivan Marsh.

In several places, both compressional and extensional offsets were superimposed. Kurtz (1906) describes a streetcar track at 4th and Bryant where rails were buckled due to 60 to 150 mm of shortening in compression, but then

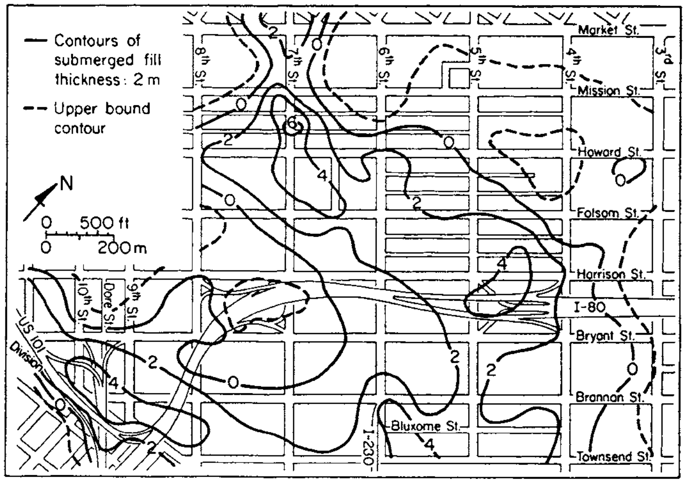

Figure 19. Thickness of Submerged Fill Deposits in the South of Market

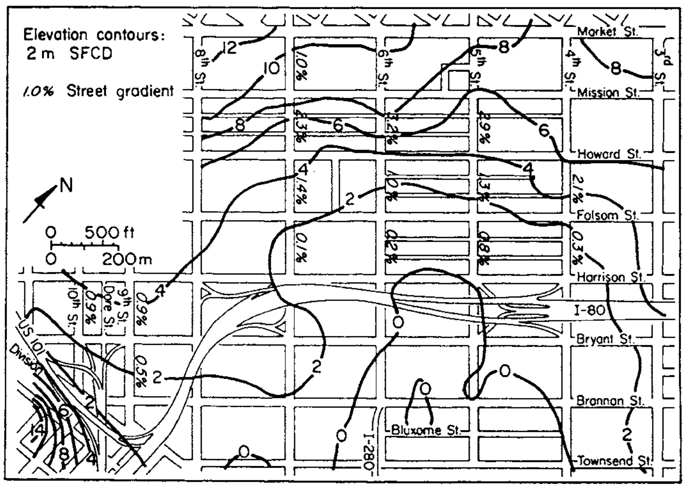

Figure 20. Current Surface Elevations and Street Gradients in the South of Market

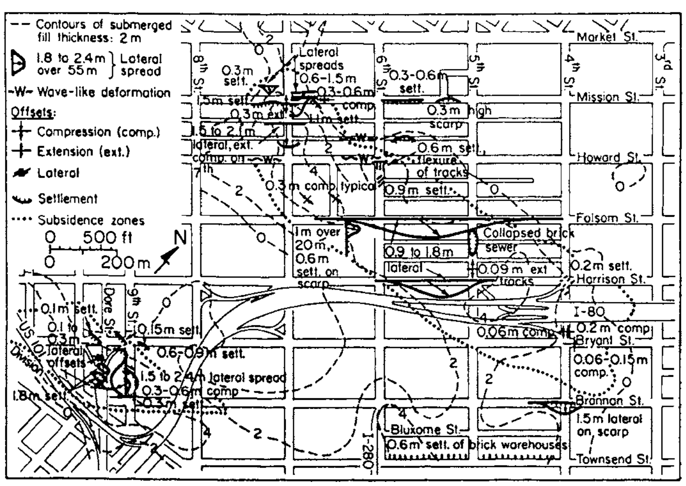

Figure 21. Ground Displacement and Liquefaction Features after the 1906 Earthquake in the South of Market

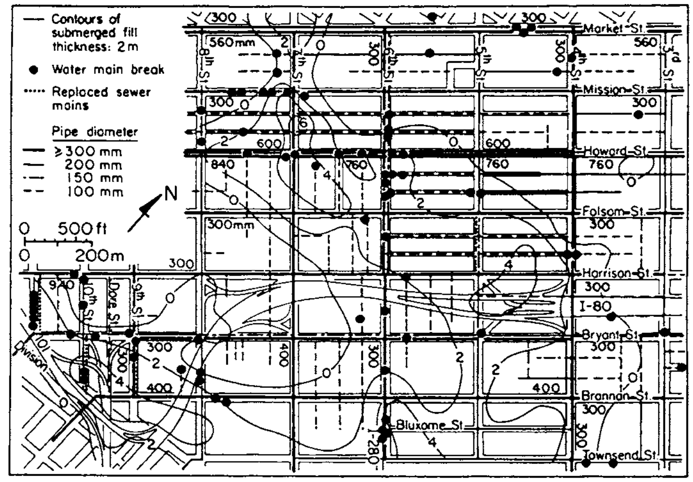

Figure 22. Water Supply Pipeline and Sewer Damage in 1906 in the South of Market

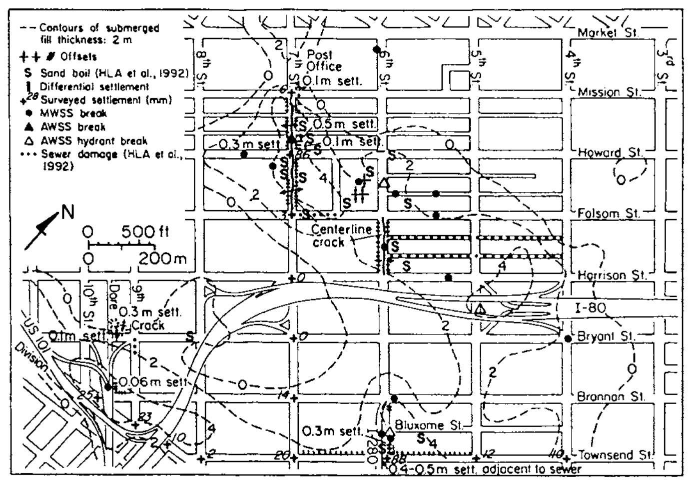

Figure 23. Ground Displacements, Liquefaction Features, and Pipeline Repairs after the 1989 Earthquake in the South of Market

translated back to their original position, leaving a 150 mm gap in the joints at the same location. In front of the Post Office at Seventh and Mission Sts., a 1 m high compression ridge occurred adjacent to settlement of similar magnitude, and parallel to cracks indicating over 1 m of lateral extension. At Brannan and 9th Sts., a 0.3 m high compression ridge marked the edge of the Dore St. subsidence zone, but settlement of 300 mm occurred immediately adjacent to the buckled pavement. The existence of compressional features adjacent to zones of permanent extensional displacement can occur only if the compression is related to transient effects, such as would occur during ground oscillation.

Figure 22 shows damage to the water supply and sewage systems in the South of Market superimposed over contours of liquefiable thickness, similar to the map for the Mission District. Water main breaks, shown by filled circles, were reported by Schussler (1906) and Manson (1908). More than one break may have occurred at marked locations. Sewer damage is also shown by dotted lines, based on damage reported by Schussler (1906).

Seventy-nine pipe breaks, denoted by solid dots, were reported in the South of Market study area. Of these breaks, 85 percent occurred in areas of submerged fill or immediately adjacent to the zero fill thickness contour. In numerous places, reported pipeline breaks are outside the zone of predicted submerged fill but within the dashed line indicating the upper bound contour. Breakage was extensive on pipelines crossing Sullivan Marsh on Mission and Howard Sts. In contrast, breaks in Sullivan Marsh southeast of Howard St. tend to be concentrated at the margins, rather than in the center of the lateral spread zone. Except on 11th St., pipeline breaks in Lower Mission Creek also are concentrated along the edges of the lateral spread zone.

1989 LIQUEFACTION IN THE SOUTH OF MARKET

During the 1989 Loma Prieta earthquake, liquefaction resulted in moderate to severe damage associated with sand boils, settlement, pavement damage, strong ground shaking, and pipeline damage. Figure 23 summarizes evidence of liquefaction in the South of Market and associated ground movements and utility damage. Damage features are superimposed on contours of submerged fill thickness from Figure 19. Sand boils are indicated by open dots; hatchures indicate areas of observed settlement, and compressional and extensional offsets are indicated by paired arrows. Settlements between surveys in 1985 and 1990 are shown along 7th, Townsend, and Division Sts. Damage to the water supply systems and sewer lines are also summarized in the figure.

Cracks, 10 to 30 mm wide, and differential settlement were observed down the centerline of 7th between Mission and Folsom Sts. Settlement was noted as far north as the corner of the Post Office at Mission St., where large movements had occurred in 1906. Large differential settlements between 300 to 500 mm were observed adjacent to buildings on 7th just north of Howard St. A 300-mm diameter water main rupture may have contributed to damage in this area. Extensional cracks were also observed in 6th St. between Folsom and Harrison. Multiple compression ridges buckled street pavements and sidewalks along Russ St. approximately 30 to 60 m north of Folsom St. Beneath the west curb of 6th St. at Townsend, ground settled sharply 400 to 500 mm adjacent to a 2 m diameter pile-supported concrete sewer.

Accurate settlements were obtained from elevation surveys performed by the City of San Francisco Department of Public Works between 1981 and 1985, and repeated between 1990 and 1992. Typical survey monuments are provided at major intersections on curbs, sidewalks, sewer catch basins, and fire hydrants. Some minor settlement is likely to have occurred due to secondary settlement of Holocene Bay mud, but no correction is made in the figure. The trend in settlements along these streets corresponds closely to changes in liquefiable thickness. Settlements of over 80 mm were measured at 7th and Howard, and at 6th and Townsend Sts. where submerged fill thickness exceeds 4 m. Settlements of 10 to 20 mm are seen in areas of 0 to 2 m submerged fill, including Division St. No settlement occurred on 7th at Harrison and Bryant Sts. where the fill boundary is higher than the groundwater table.

In the South of Market, 14 repairs in the MWSS were reported. With the exception of the break on 6th near Market St., all these breaks lie within the submerged fill zone as developed from subsurface mapping.

Including a hydrant break at 6th and Bluxome Sts. which resulted from falling masonry, all four breaks in the AWSS in the South of Market occurred in the submerged fill zones. The most serious damage to this system was due to a 300-mm-diameter main break on 7th St. between Mission and Howard Sts. Water flow through this break and nearby hydrant breaks helped to empty the AWSS reservoir supplying the central business district in approximately 30 to 40 minutes (O'Rourke, et al., 1991).

Liquefaction in the South of Market recurred in 1989 in the same locations as in 1906. Damage in 1989 was generally in areas underlain by more than 2 m of submerged fill. In 1989, as in 1906, the most severe damage occurred in the vicinity of 7th St. between Mission and Howard. Similar centers of pipeline damage occurred on 6th near Market St., 6th near Bluxome St., 8th and Bryant Sts., and Brannan and Dore Sts. Each of these five locations is in an area of deep submerged fill. The close agreement of type of damage and relative severity in both earthquakes suggests that these sites are especially prone to liquefaction and associated damage.

MAGNITUDE OF LATERAL DISPLACEMENT

Correlations between the magnitude of horizontal surface displacement associated with lateral spread and various topographical, geographical, and soil factors have been investigated (e.g., Hamada, et al., 1986; Bartlett and Youd, 1992). Hamada, et al. (1986) correlated horizontal movement with several different parameters, including thickness and depth of liquefiable layer, gradient of ground surface, gradient of base of liquefiable layer, and soil factors indicating relative susceptibility to liquefaction. They found that the best correlation involved the thickness of the liquefiable layer. Subsequent publications by Hamada (1992a and 1992b) have substantiated these initial assessments, and have indicated that, for data associated with the 1964 Niigata and 1983 Nihonkai-Chubu earthquakes, the thickness of the liquefiable layer is the most significant parameter which correlates with magnitude of lateral movement. Hamada reported that correlations involving surface slope and gradient of the base of the liquefiable layer do not result in a statistically meaningful basis for empirical prediction of lateral spread displacement.

Bartlett and Youd (1992) studied lateral spread displacements for various U.S. and Japanese earthquakes, with the majority of data related to the 1964 Niigata and 1983 Nihonkai-Chubu earthquakes. They performed multiple linear regression analyses on the assembled database, in which a stepwise procedure was employed to search for the parameters with highest degree of correlation with magnitude of lateral movement. In contrast to the findings of Hamada and coworkers, they found that geometric parameters related to the ground slope and free face conditions provided the highest degree of correlation.

Table 1 summarizes the observed lateral movement, surface slope, estimated slope of base of submerged fill, submerged fill thickness, and surface layer thickness for 15 locations in the two study areas in 1906. Where a range of values is presented in Figures 11 and 20, the table indicates the average maximum displacement. Subsurface parameters were determined at the location of maximum displacement of each lateral spread. Surface slopes and base gradients are average slopes evaluated from computerized surfaces over a 60 to 90 m horizontal distance.

As shown in Figure 24a, a reasonable fit exists for the plot of lateral ground displacement versus submerged fill thickness. Roughly half the variation in displacement data can be explained by thickness of liquefiable soils ($r^2 = 0.50$). Lateral displacement is roughly 30 percent of the thickness of submerged fill, and most data are bounded by ratios corresponding to 15 to 45 percent of submerged fill thickness.

Figure 24b shows the relationship between surface slope and permanent lateral displacement. Consistent with observations reported by the National Research Council (1985), surface slopes of lateral spread zones are between 0.5 and 2.5 percent. The best linear fit of the data has an $r^2 = 0.25$, which represents a poor correlation for explaining the variability of observations. Single variable and multiple linear regressions involving the surface layer thickness did not improve the coefficient of determination achieved by regression with only the thickness of liquefiable fill.

Table 1. Summary of Lateral Spreads, Surface Gradients, Submerged Fill Thickness, and Documented Lateral Deformation During the 1906 Earthquake

Location	Lateral Movement, m	Surface Slope, %	Slope of Base of Fill, %	H_2 [a] m	H_1 [b] %
19th St. at Mission Playground	1.8	1.8	5.0	3.0	5.5
Valencia St. at 18th to 19th Sts.	2.1	1.8	1.8	6.8	3.0
Mission St. at 17th to 18th Sts.	0.3	1.1	0.5	2.0	3.5
Capp St. at 17th to 18th Sts.	1.0	1.0	0.8	3.5	4.3
South Van Ness Ave. at 17th to 18th Sts.	1.2	1.2	1.2	4.0	3.2
18th St. at South Van Ness Ave. to Shotwell St.	0.3	1.3	6.0	2.5	2.5
18th St. at Shotwell to Folsom Sts.	0.5	0.6	3.0	3.2	2.5
9th St. at Bryant to Brannan Sts.	1.9	0.5	1.8	3.0	2.5
7th St. at Market to Mission Sts.	0.6	1.0	6.0	1.0	4.5
Mission & 7th Sts.	1.5	1.7	6.0	3.0	4.0
7th St. at Mission to Howard Sts.	2.1	2.3	6.0	6.0	3.7
Columbia St. at Folsom St.	1.0	1.0	1.5	2.3	2.0
Columbia St. at Harrison St.	0.0	0.5	1.5	1.0	2.0
Folsom St. at 5th to 6th Sts	1.4	0.8	1.8	2.5	2.0
Harrison St. at 5th to 6th Sts	1.4	0.4	0.5	3.0	1.5

a - Thickness of upper nonliquefiable layer
b - Thickness of underlying liquefiable layer

Data reported by Hamada (1992a and 1992b) indicate that Japanese lateral spreads are, on average, 125 percent of liquefied thickness, roughly four times greater than the percentage displacement for San Francisco sites in Figure 25a. Bartlett and Youd (1992) included Mission District and South of Market data in their study, and found that their models significantly overpredicted lateral displacement in 1906 by a factor of five to ten.

It is unclear which conditions in San Francisco have contributed to significantly smaller magnitude of lateral spread than observed at other sites. Most of the sites evaluated by Bartlett and Youd and Hamada and coworkers are primarily in alluvium or other natural deposits. In contrast, San Francisco liquefaction occurred in sandy fills which may have soil characteristics or geometric features that differ from natural deposits. Bartlett and Youd (1992) have suggested that the three-dimensional geometry of filled channels may have been a factor in reducing displacements. Deep liquefiable deposits in the Mission District and South of Market are a block or

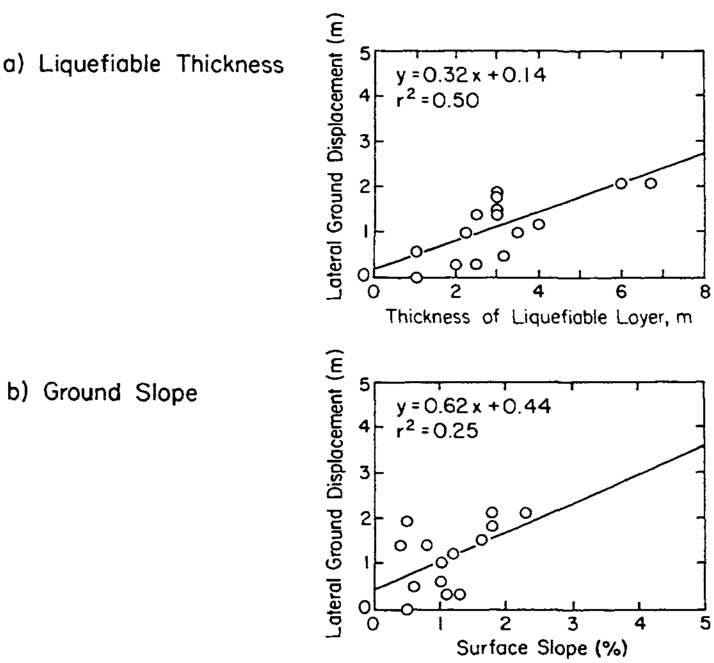

Figure 24. Linear Regression of 1906 Lateral Ground Displacement for Liquefiable Thickness and Ground Slope

less in width, which may have limited displacement due to lateral resistance along the boundaries of filled channels, and which may have resulted in non-uniform and somewhat discontinuous patterns of both horizontal and vertical movement. In comparison, liquefiable soils at sites in Niigata, Japan (e.g., Hamada, 1992a) show wider spatial distribution due to their alluvial origin, which may explain both more persistent liquefaction features and greater magnitude of deformation. Lastly, as case studies represent unique seismic events, it is possible that characteristics of the 1906 ground motions in San Francisco, which are not well documented, may have affected site response for lateral spreads in this study.

SURFACE MANIFESTATION OF LIQUEFACTION

Ishihara (1985) investigated the effect which the thickness of liquefiable soil relative to that of the non-liquefiable surface layer has on the occurrence of liquefaction damage. He used observations from areas affected by the 1983 Nihonkai-Chubu earthquake, with approximate peak ground acceleration 0.2 g, and the 1976 Tangshan earthquake, with approximate peak ground acceleration of 0.4 to 05. g. Because the magnitudes and approximate acceleration levels associated with these two earthquakes are consistent with those for the 1906 and 1989 earthquakes in San Francisco, it is especially interesting to see if similar trends are evident in the data collected for this study.

Consistent with Ishihara's approach, subsurface conditions in the Mission District and South of Market areas were interpreted as illustrated in Figure 25. The capping or surface layer thickness, H_1, denotes the thickness of nonliquefied soil which overlies a liquefied layer of thickness, H_2. In the top two profiles, the liquefiable thickness, H_2, is defined by the zone of saturation due to the groundwater table. In the lower two profiles, a clayey layer which has a non-liquefiable consistency occurs near the ground surface. In the bottom profile, the clay layer extends below the water table reducing the liquefiable thickness, H_2, and correspondingly increasing the thickness of surface layer, H_1.

Figure 26 provides a plot of observed ground deformation in the Mission District and South of Market areas on axes corresponding to liquefiable thickness (ordinate) and surface layer thickness (abscissa). Open circles represent sites where photographs or accounts indicate no ground deformation. Other symbols indicate observations of ground deformation, including lateral spreads, pavement offsets, and sand boils. Offsets include tension racks, compression ridges, sharp lateral displacements, and scarps associated with lateral spreads. Observable surface distortions associated with wave-like deformation or differential settlement are categorized as ground offsets. Relatively uniform settlements are indicated by a separate symbol. Subsurface parameters were evaluated as closely as possible to each reported or photographed feature; where areas of interest extended over a length of greater than 30 to 50 m, more than one datum is used to define different portions of the feature.

Figure 26 indicates that for the 1906 earthquake, extensive damage was limited to areas with liquefiable thickness greater than 1 m. Lateral spreads, ground deformation, and offsets may be observed over a similar range of surface and liquefiable thicknesses. The 1906 data are not entirely consistent with the threshold proposed by Ishihara (1985) for liquefaction damage resulting from strong ground shaking (0.4 to 0.5 g) in the 1976 Tangshan earthquake. In particular, there is evidence of surface damage in areas of fill at previous dune sites where the liquefiable thickness is relatively small (Pease and O'Rourke, 1993). Because the surface layer thickness does not exceed 6 m in the areas in this work, there is no evidence to compare with Ishihara's threshold curve for surface thickness greater than 6 m.

Figure 26 indicates that for the 1989 earthquake, ground deformation was observed where both the submerged fill thickness is greater than or equal to approximately 2 m, and the surface layer is 3 m or less in thickness. Distribution of sand boils and surface deformations with respect to subsurface parameters are similar. Peak ground accelerations at soil sites in San Francisco

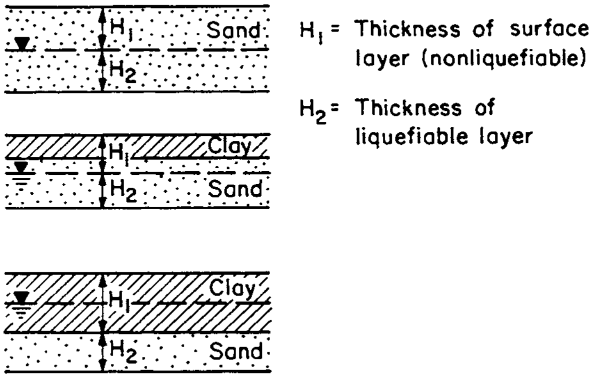

Figure 25. Subsurface Thicknesses for Liquefaction Assessment (after Ishihara, 1985)

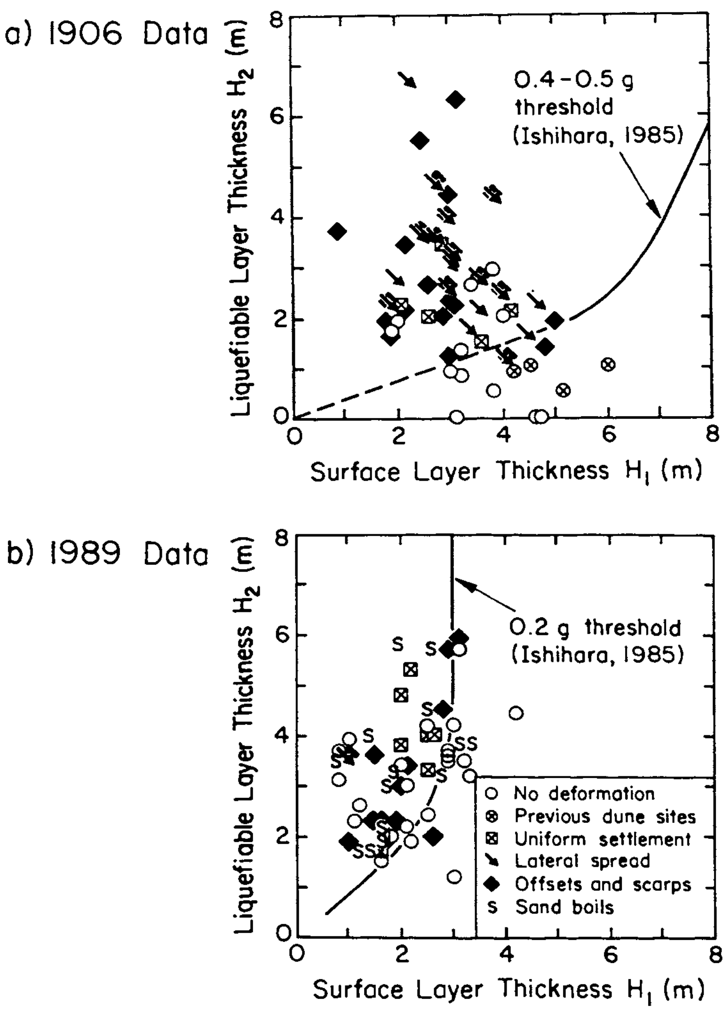

Figure 26. Ground Deformation in the South of Market and Mission District Plotted with Respect to Subsurface Geometry for the 1906 and 1989 earthquakes

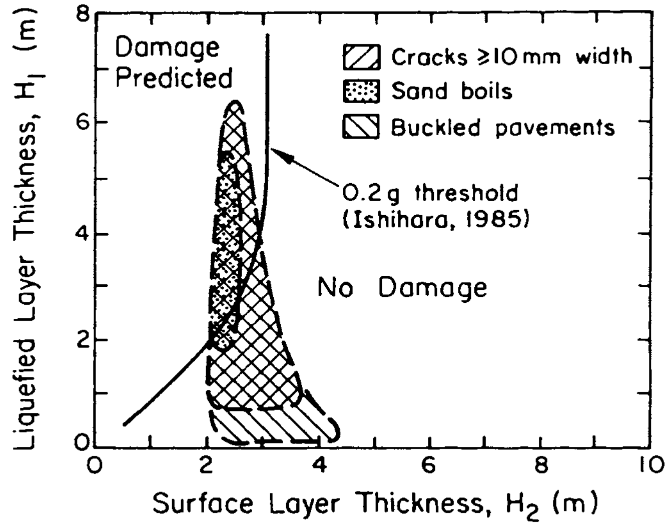

Figure 27. Ground Deformation Plotted with Respect to Subsurface Geometry for the Mission District in the 1989 earthquake

during the Loma Prieta earthquake have been assessed as 0.15 to 0.25 g (e.g. O'Rourke, et al., 1992; Bardet, et al., 1992). The distribution of observed damage in the Mission District and South of Market areas in 1989 is in agreement with the bounding curve for liquefaction damage presented by Ishihara for moderate ground shaking (0.2 g) developed from the 1983 Nihonkai-Chubu earthquake. It should be recognized that the most severe surface effects were centered on the thickest pockets of liquefiable fill, and therefore somewhat limited in areal extent. Accordingly, the liquefaction characteristics in the Mission District and South of Market area differ from those in the Marina where thick deposits of liquefiable fill are concentrated over a larger surface area.

Consistent with Ishihara's approach, subsurface conditions in the Marina also were interpreted as shown in Figure 27. Given the density of plotted data, the distribution of observed features is indicated by shaded areas rather than individual points.

Sand boils occurred within the bounds suggested by Ishihara. However, liquefaction effects extended to and slightly beyond the bounds of submerged fill. Ishihara's proposed relationship among surface damage, H_1 and H_2, is based on a predominantly one-dimensional model of liquefaction in that site response is correlated primarily with regard to the vertical soil profile. The Marina response shows that the three-dimensional characteristics of the liquefiable deposit can play an important role. Extensional and compressional features are concentrated along the margins of the hydraulic fill due to ground oscillation, which appears to have conveyed deformations outside the zones of H_1 and H_2 combinations identified by Ishihara (1985).

Youd and Garris (1994) have evaluated Ishihara's empirical criteria for the relative thicknesses of liquefiable and non-liquefiable layers for 15 different earthquakes, and concluded that the thickness bounds proposed by Ishihara appear to be valid for sites not susceptible to ground oscillation or lateral spread. For sites susceptible to ground oscillation or lateral spread, they also concluded that the bounds suggested by Ishihara are not sufficient for predicting surface disruption. Data from the Marina, Mission Creek, and South of Market areas are consistent with the findings of Youd and Garris. Detailed investigation of surface disturbance in the Marina in 1989 shows that the three-dimensional characteristics of ground oscillation are responsible for heaved and fractured pavements and damaged pipelines near and slightly beyond the margins of liquefiable soils. Ishihara's empirical criteria cannot account for such effects.

HAZARD MAPS

There is remarkably consistent spatial correlation between locations of thickest liquefiable fill and areas of most severe damage in the 1906 and 1989 earthquakes. Recognizing the close correlation between the thickness of liquefiable fill and the potential severity of liquefaction, hazard maps have been developed for the Mission District and South of Market areas in Figure 28 and 29, respectively. In each of these maps, the thickness of submerged fill is used as the primary index for identifying areas of potential liquefaction deformation. For a scenario earthquake equivalent to the 1906 event, the legend in each map indicates the range of maximum lateral displacement which is possible for areas with different thicknesses of liquefiable fill.

Shaded areas in Figure 28 and 29 denote regions with greater than 2 m of liquefiable soils, which are associated with high levels of lateral displacement in the 1906 earthquake. As indicated by the key in each figure, liquefiable thickness provides a means to predict magnitude of permanent lateral displacement. Manifestation of other liquefaction features including subsidence, sand boils, ground deformations, and pavement offsets are also likely to be found in these areas in proportion to the severity of permanent lateral deformation, and are also therefore related to liquefiable thickness.

The maximum lateral extent for the occurrence of liquefaction in Figure 28 and 29 is indicated by a hatchured line referred to as the upper bound contour. Areas potentially subject to liquefaction, on the hatchured side of the contour, represent areas where the base of fill occurs within 2 m above mapped groundwater levels to areas with 2 m thickness of submerged fill. The upper bound contour encompasses the region of uncertain saturation of fills due to fluctuations of groundwater, and also appears to bound the historic occurrence of damage. The limits of liquefaction in the Mission District in Figure 28 are roughly in agreement with the delineations of liquefaction zones in previous studies, (e.g., Youd and Hoose, 1978; O'Rourke and Lane, 1989). In contrast, the upper bound of liquefaction in the South of Market in Figure 29 includes dune depression fills which were identified by Pease and O'Rourke (1993). These fills have not been identified in previous works and may increase the extent of the potentially hazardous areas.

A dashed line indicates the boundary between areas where surface layer thickness overlying saturated deposits exceeds 3 m, and where the surface layer thickness is less than 3 m. Damage in the 1989 Loma Prieta earthquake was not observed in areas with greater than 3 m of surface fill. The thickness of surface deposits does not appear to have an influence on liquefaction in a great magnitude event, but may predict the absence of liquefaction damage for an earthquake of lesser magnitude.

In the course of this work, other factors were identified which reduce the potential for liquefaction in the areas of submerged fill deposits. Specifically, liquefaction hazard assessment must include consideration of the fines content and plasticity of fill materials. In particular, it is noted in Figure 28 that severe liquefaction is not likely in the vicinity of 19th St. between Folsom and Harrison Sts., despite the presence of submerged fill thickness exceeding 2 to 4 m. For more information about the hazard maps, reference should be made to Pease and O'Rourke (1993).

CONCLUDING REMARKS

The 1906 and 1989 earthquakes in San Francisco provide clear and unmistakable evidence for a strong relationship between ground deformation associated with liquefaction and the seismic performance of buried lifelines. The detailed information collected for both earthquakes provides important lessons which should be understood by those responsible for engineering, emergency response, and city planning in areas with loose fill and natural sand deposits. The principal lessons are summarized as follows:

- The thickness of submerged loose fill and loose natural sand deposits is one of the most significant factors affecting the severity of liquefaction. Investigations of liquefaction in the Marina, Mission Creek, and South of Market areas of San Francisco show that the thickness of liquefiable fill can be correlated with magnitude of lateral spread, settlement associated with post-liquefaction con-

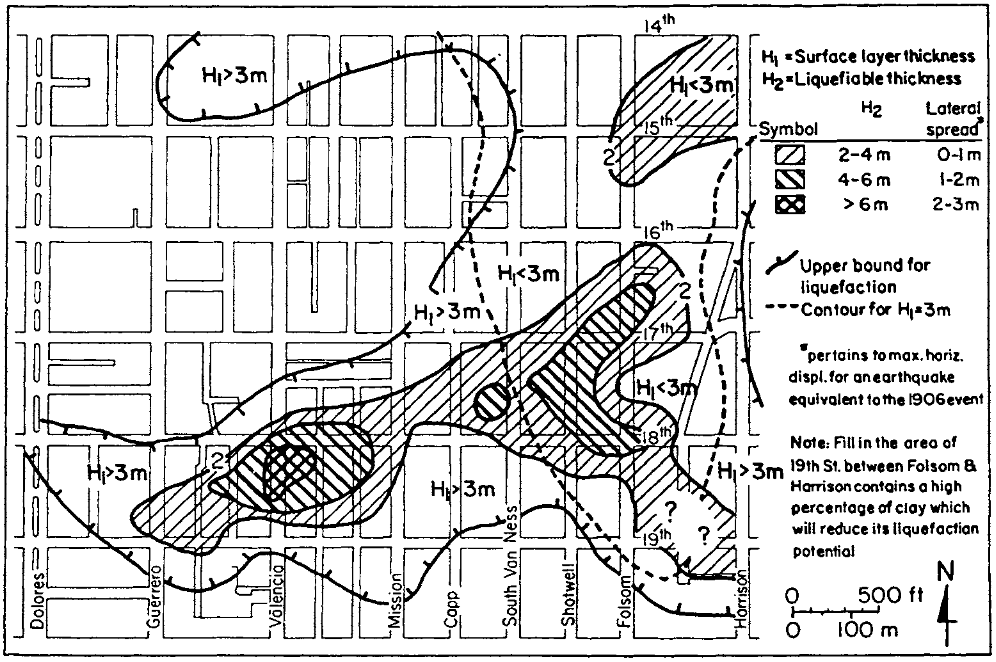

Figure 28. Liquefaction Hazard Map of the Mission District, Showing Zones of Highest Potential Damage and Lateral Movement for an Earthquake of Similar Magnitude to the 1906 San Francisco Earthquake. [Note: A dashed contour (H_1 = 3 m) indicates the regions where surface layer thickness is less than 3 m, for which liquefaction damage is more likely to be observed during lower magnitude earthquakes, such as the 1989 Loma Prieta earthquake.]

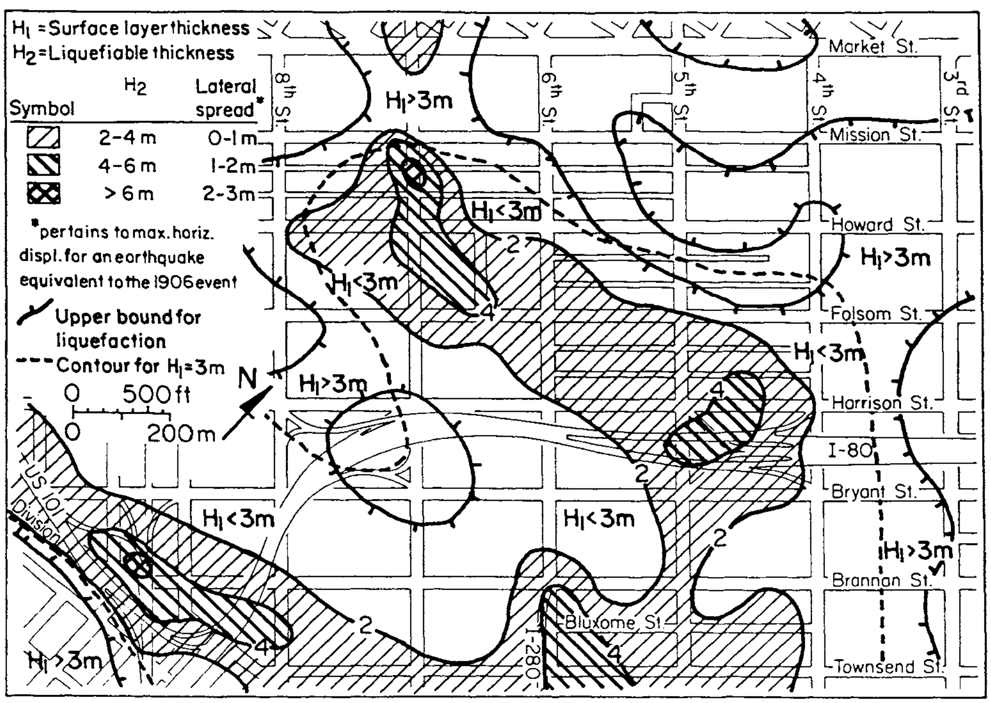

Figure 29. Liquefaction Hazard Map of the South of Market, Showing Zones of Highest Potential Damage and Lateral Movement for an Earthquake of Similar Magnitude to the 1906 San Francisco Earthquake [Note: A dashed contour (H_1 = 3 m) indicates the regions where surface layer thickness is less than 3 m, where liquefaction damage is more likely to be observed during lower magnitude earthquakes, such as the 1989 Loma Prieta earthquake.]

solidation, and magnitude of horizontal surface movement generated by transient lateral shear strain. These deformations contribute directly to pipeline damage, the disruptions of streets and sidewalks, and disturbance of related surface and subsurface structures. Buildings are damaged by these types of deformation and also by loss of bearing of shallow strip and spread footings founded on liquefiable deposits.

- Mapping the thickness of liquefiable deposits provides an excellent means of locating areas of potentially severe liquefaction and showing their relationship with underground utilities, buildings, and transportation facilities. The thickness of a liquefiable fill or natural sand deposit is easily adapted to Geographical Information Systems (GIS), and thus can provide an effective vehicle for assessing urban hazards, microzoning for seismic hazard reduction, and planning for optimal lifeline performance during an earthquake.

- Transient lateral shear strains in liquefied soils are a prime cause of horizontal displacements and seismic damage to buried lifeline systems. Although ground oscillation has been recognized as a source of significant deformation in previous studies, recent earthquake measurements at saturated sand sites have provided a quantitative basis for estimating the magnitude of transient lateral shear strains. Using lateral shear strain levels of 1 to 2 percent, which are consistent with earthquake measurements, it is shown that transient horizontal ground deformation in the Marina was of sufficient magnitude to damage cast iron water mains and that the areal distribution of most severe transient deformation coincides with that of the most intense damage to the pipeline system. This finding is highly significant because it shows for the first time how transient horizontal deformation at liquefaction sites affects the performance of buried utilities. It also shows how the three-dimensional characteristics of liquefiable soils control the pattern of transient lateral displacements at the ground surface.

- Analytical results indicate that maximum horizontal ground strains generated by transient lateral shear deformation in the Marina were on the order of 500 to 900 $\mu\varepsilon$. Such strains are consistent with failure levels in brittle pipelines, such as cast iron, asbestos cement, and vitrified clay pipe. These strain levels may also damage steel distribution piping at service connections and locations of local weakness and deterioration, but are not likely to damage steel and polyethylene piping in good repair.

- Data from the Marina, Mission Creek, and South of Market areas for both the 1906 and 1989 earthquakes are consistent with the findings of Youd and Garris (1994) who show that Ishihara's empirical criteria (1985) for the relationship between surface disturbance and relative thicknesses of liquefiable and non-liquefiable layers are not sufficient for predicting surface disruption at sites of ground oscillation and lateral spread.

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