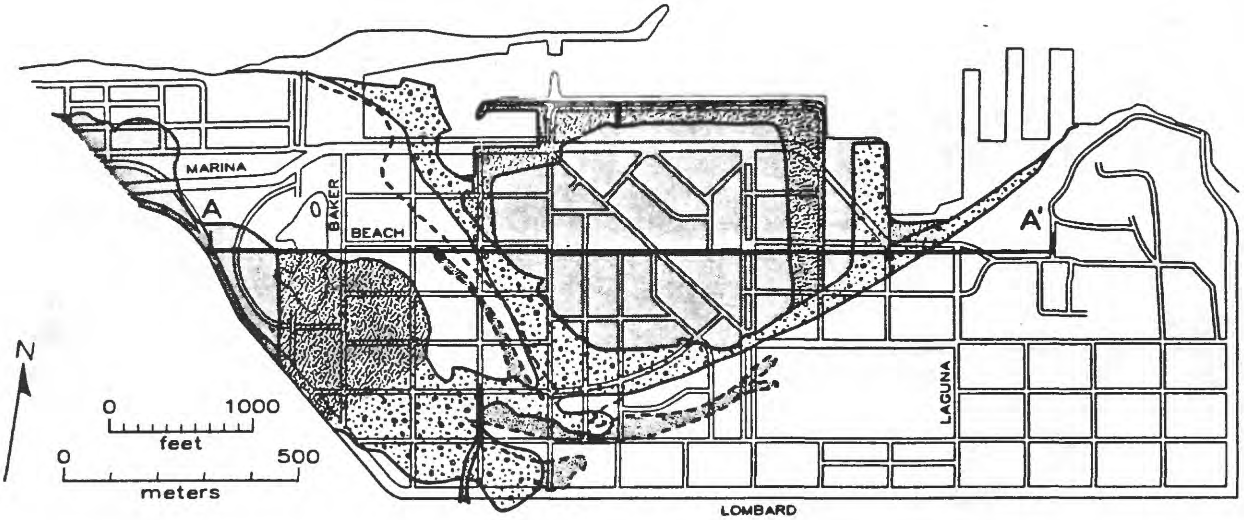
Department of the Interior
U.S. Geological Survey

EFFECTS OF THE LOMA PRIETA EARTHQUAKE ON THE MARINA DISTRICT SAN FRANCISCO, CALIFORNIA



ARTIFICIAL FILLS

1906–1917 (principally 1912)

1895–1906

1869–1895

1851–1869

OPEN-FILE REPORT 90-253

Menlo Park. California

April 20, 1990

EFFECTS OF THE LOMA PRIETA EARTHQUAKE ON THE MARINA DISTRICT SAN FRANCISCO, CALIFORNIA

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SUMMARY AND CONCLUSIONS

Thomas L. Holzer and Thomas D. O'Rourke

Introduction

Immediately following the October 17, 1989, Loma Prieta earthquake, U. S. Geological Survey scientists investigated the effects of the earthquake on structures and ground in the Marina District of San Francisco. They were assisted by researchers from Cornell University supported by the National Center for Earthquake Engineering Research. The deployment was part of a larger effort to investigate effects from the earthquake throughout northern California. This open-file report is a preliminary summary of their observations in the Marina District. It was prepared because these observations are critical for decisions about reconstruction and mitigation of the earthquake hazard in the Marina District. This section summarizes the observations in the attached reports and describes the earthquake hazard in the Marina District.

Two earthquake effects in the Marina District were recognized by the investigators - liquefaction and amplification of earthquake shaking. Liquefaction implies that parts of the thick sequence of sedimentary deposits that underlie the district experienced high water pressures during earthquake shaking which caused temporary loss of strength and a behavior much like quick sand. The parts of the deposit vulnerable to liquefaction are sands that are below the water table and have soil structures that densify when vibrated. Amplification implies that the district shook more strongly during the earthquake than the immediate surrounding area. The amplification was caused by the geologically young sedimentary deposits on which the district is built. These deposits modified the earthquake waves as they passed upward from underlying bedrock. The hazard from these two effects was compounded by the design of many structures in the district. Most of the district was built before the implementation of modern building codes, and few structures were designed to resist the large lateral loads from strong earthquake shaking. In addition, liquefaction was not widely recognized as a hazard and few foundations were designed to accommodate the loss of soil strength and differential settlements associated with liquefaction of the sands.

Understanding the effects of earthquakes in the district is a serious matter because the effects can be expected to repeat in future earthquakes. The U.S. Geological Survey estimated in 1988 that there was a 1 in 2 chance of a magnitude 7 earthquake in the next 30 years on fault segments that are closer to the district than was Loma Prieta and which would shake the district even more strongly.

Damage

Three principal types of damage to surface structures in the district were observed - racked first stories, settlements, and pounding (Celebi, "Types of Structural Damage"). Distorted or racked first stories were common in structures with large openings for garages. These structures generally had insufficient stiffness and strength at ground level to resist the lateral load imposed by earthquake shaking. Building settlement was common in areas where liquefaction occurred. Some buildings settled and deformed as the sandy soil beneath the structure liquefied. In other cases, damage resulted from buildings in close proximity hitting each other in a phenomenon known as pounding.

To identify general areal trends of structural damage, we referred to the evaluations that were made for the City of San Francisco during the building inspections following the earthquake (Seekins, Lew, and Kornfield: "Areal Distribution of Damage to Surface Structures", Figs. 1 and 2). The maps show that damage was not restricted to the area underlain by artificial fill where evidence for liquefaction was strong. Substantial structural damage, therefore, was related to earthquake shaking independent of liquefaction.

Liquefaction

The reason for the liquefaction hazard is evident in the geology of the district described by M.G. Bonilla, "Natural and Artificial Deposits in the Marina District." In fact, much of the liquefaction hazard in the district was created by the manner in which ground beneath the district was artificially built up. Prior to 1851 most of the district area consisted of either open water or marsh (Bonilla, Fig. 1). The primary land areas were a beach spit known as Strawberry Island which extended northwestward from Francisco Street across Divisadero, Broderick, and Baker Streets, and an area of dune sands approximately east of Webster and south of Bay Streets (for locations of streets, see Fig. 1 in this section). From 1851 to 1912, both the marsh and open water were filled piecemeal. A significant infilling occurred in 1912 to establish new ground for the 1915 Panama-Pacific International Exposition. This fill (Bonilla, Fig. 6), which underlies the area approximately bounded by Divisadero, Capra, and Webster Streets, and Marina Boulevard, consists of sand that was dredged and pumped into a cove by hydraulic filling. The fill was placed to allow silt and clay in the dredged material to flow back into the bay. Sands in hydraulic fills may have sensitive soil structures that tend to densify when shaken by earthquakes. Thus, the combination of method of filling and high water table promoted soil conditions susceptible to liquefaction. Unfortunately, earthquake-induced liquefaction was not a widely recognized hazard in 1912, and no effort was expended to compact or otherwise stabilize the fill as would be possible with placement and site-improvement techniques that are often followed today.

The post-earthquake investigation conducted and described by M.J. Bennett, "Geotechnical Characteristics of Unconsolidated Deposits, Ground Effects, Leveling Survey, and Liquefaction Analysis" demonstrates that most of the liquefaction in the Marina District occurred within the area underlain by hydraulic fill. Sand boils that erupted on the land surface and filled many garages and basements match best with samples from borings in the artificial fill on the basis of both grain size and color. In addition, most of the sand boils that were clearly caused by liquefaction erupted in the area underlain by fill (Bennett, Fig. 6). The few sand boils outside of the fill area coincided with water main breaks and may not have been caused directly by liquefaction. Ground cracking, which is often associated with liquefaction-induced ground movement was widespread in the area underlain by hydraulic fill.

The inferred area of liquefaction is supported by a study of pipeline breakage within the Municipal Water Supply System by T.D. O'Rourke and B. L. Roth, "Performance of Pipeline Systems in the Marina." Most of the repairs were concentrated within the area of hydraulic fill (O'Rourke and Roth, Fig. 1), the area where most liquefaction-induced ground deformation occurred.

The susceptibility of the hydraulic fill and other deposits to liquefaction was evaluated by standard engineering methods at six locations where boreholes were drilled in January 1990 (Bennett, Fig. 1). The results confirmed the vulnerability of the hydraulic fill to liquefaction at modest levels of earthquake shaking. Although no recordings of the mainshock were collected in the Marina District, the fill is predicted to liquefy at the level of shaking that we estimated for the Loma Prieta earthquake (Bennett, Figs. 17 and 18). Only a few of the tests in natural sand deposits indicated that liquefaction may have occurred within parts of the natural deposits.

Amplification

"Ground Motion Amplification in the Marina" by Boatwright, Seekins, and Mueller describes and analyzes aftershock recordings which indicate that earthquake motions in the district are significantly amplified relative to bedrock motions. The amplification occurs as the seismic waves pass into and through the sediments underneath the district. Recordings of aftershocks were made at five sites with portable instrumentation deployed in the district immediately following the Loma Prieta earthquake. Comparison of these recordings with recordings at bedrock stations at Fort Mason, Pacific Heights, and Nob Hill indicates that ground motions, caused by aftershocks ranging in earthquake magnitude from M_L 2.1 to 5.0, were amplified throughout much of the district by a factor ranging from 6 to 10 (Boatwright and others, Fig. 3). Amplification at frequencies of greatest relevance to structures in the district, 2 to 4 Hertz (cycles per second), ranged from 3 to 4. The seismic amplification was observed throughout the area bounded on the east, west, south, and north by Buchanan, Divisadero, North Point Streets, and Marina Boulevard, respectively. The effect presumably extends beyond this area but it was not documented with portable field instruments.

An important consideration is that amplification factors for stronger earthquakes such as the Loma Prieta mainshock may be smaller than those determined from these aftershocks. The soft sediments beneath the district are unlikely to amplify shaking as much when the intensity of shaking is stronger. This effect is known as nonlinearity. Despite the caveat that the aftershock amplification factors cannot be used explicitly to estimate the strength of shaking for higher earthquake intensities, the observations demonstrate that the district tends to shake more strongly than the surrounding region. A critical issue is how much stronger? This issue must be resolved by evaluating the district soil response to earthquake shaking as a function of peak ground motion, duration of shaking, and potential frequency content of incoming waves.

The amplification effect, as was the liquefaction effect, can be anticipated on the basis of the geology of the district (M.G. Bonilla, "Natural and Artificial Deposits in the Marina District"). The district sits atop a thick sequence of geologically young sedimentary deposits. Deep borings near Buchanan and Bay Streets and Beach and Divisadero Streets indicate that the sediments are more than 250 feet thick. Moreover, most of the district is underlain by a deposit of soft, bay mud, which is as thick as 6? feet beneath Marina Boulevard. The properties of these sediments are similar to sediments elsewhere in which amplification effects have been observed ("Engineering and Seismic Properties of the Soil Column at Winfield Scott School, San Francisco," by Kayen, Liu, Fumal, Westerlund, Warrick, and Lee). Although subsurface information is sufficient to document the presence of a wedge-shaped sedimentary basin that thickens to the north, its precise configuration is not known. Despite our inability to draw precise boundaries of the margin of the area affected by amplification, there appears to be a strong correlation between severely shaken areas and locations underlain by bay mud.

Conclusions

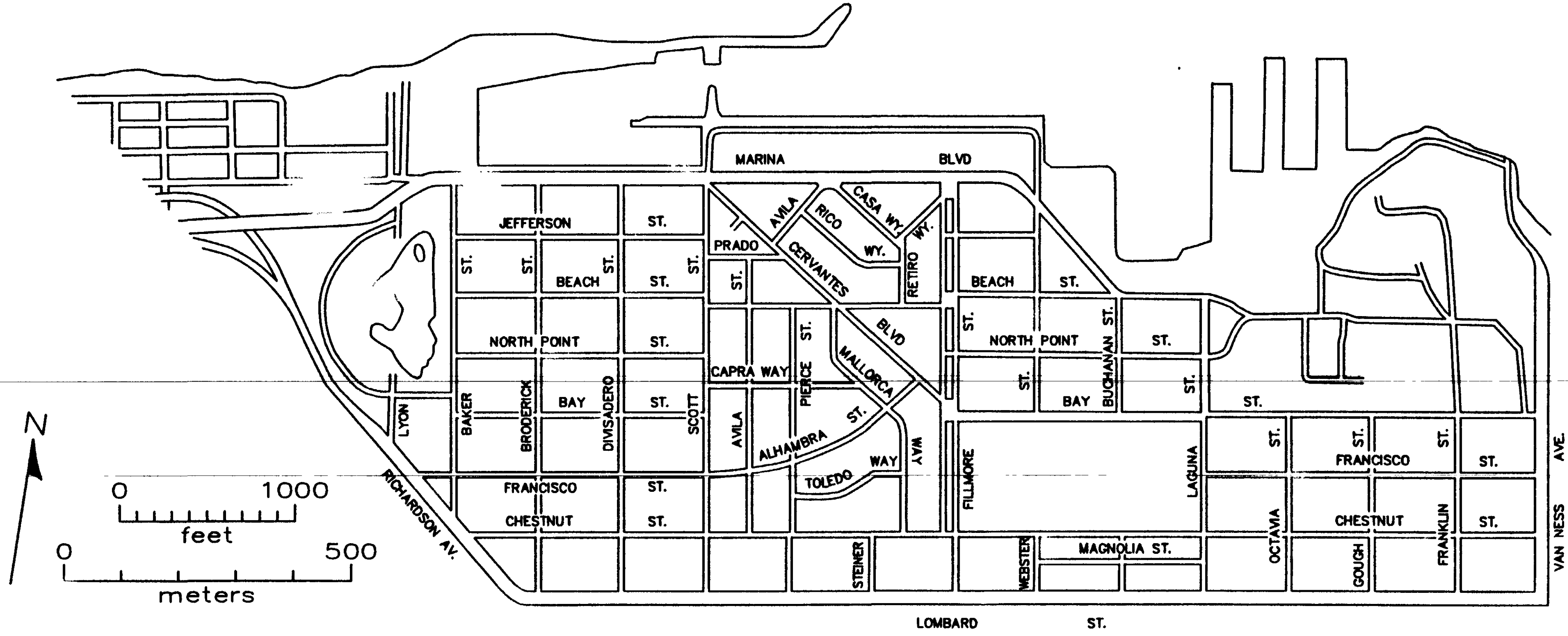
Two earthquake effects - liquefaction and amplification of earthquake shaking - damaged structures in the Marina District during the Loma Prieta earthquake and can be expected to repeat in future earthquakes.

Liquefaction potential is particularly high in the area approximately bounded by Divisadero, Capra, and Webster Streets, and Marina Boulevard. This area is underlain by sand which was pumped into a former cove in 1912 by hydraulic filling. This created a deposit that is vulnerable to earthquake shaking. The sand liquefied during the Loma Prieta earthquake and retains its susceptibility to liquefaction in future earthquakes. Although deposits beneath the surrounding area in the district are more resistant to liquefaction, tests in these deposits indicate that portions of them may also liquefy.

Liquefaction caused many cracks to open, or displace, the ground in the district during the Loma Prieta earthquake, but amounts of opening were modest, typically only a few inches. Nevertheless, the potential for larger horizontal ground displacements and settlements exists in the event of a stronger earthquake. Investigations in San Francisco have shown that areas of liquefaction during the 1906 earthquake reliquefied during the 1989 earthquake, although the magnitude and extent of ground displacements were smaller in 1989. The recurrence of liquefaction underscores the vulnerable nature of loose waterfront fill and suggests that the effects in the district of a stronger earthquake would include larger horizontal and vertical soil movements. Such displacements could be comparable to the displacements of several feet that were observed in similar fills during the 1906 earthquake.

Amplification of earthquake shaking occurred over a broad area in the district. It extended beyond the area bounded on the east, west, south, and north by Buchanan, Divisadero, North Point Streets, and Marina Boulevard, respectively. The amplification was caused by the passage of seismic waves into and through a thick sedimentary basin beneath the district, and correlates with locations underlain by soft bay mud. Aftershock recordings revealed maximum amplifications of six- to ten-fold. The precise magnitude of amplification for large earthquakes is uncertain because measurements from small aftershocks cannot be extrapolated directly to higher levels of earthquake shaking.

Our preliminary investigation reveals that both liquefaction and shaking caused damage to structures in the district. Structures were damaged on both natural deposits and artificial fills. Further investigation is required to define the relative significance of each of these hazards. Damage from earthquake shaking was abetted by inadequate stiffness of bottom stories and, in some cases, by the deterioration of timber or lack of proper anchoring of timber frames to their concrete strip foundations. Underground pipelines appear to have been damaged most severely by permanent differential ground displacements caused by liquefaction and consolidation of loose fills.



Base from U.S. Geological Survey San Francisco North 7.5' quadrangle, photorevised in 1973

Figure 1. Map of Marina District with street names.

INTRODUCTION

Thomas L. Holzer

Heavy and widespread damage to structures in the Marina District of San Francisco, California, was caused by the M_s 7.1 Loma Prieta earthquake of October 17, 1989. Field evidence indicated that both liquefaction and locally intense strong ground shaking occurred in the district. The purpose of this technical report is to summarize the results of the post-earthquake field investigation that was conducted by the U. S. Geological Survey with assistance from Cornell University researchers to understand and clarify the causes of damage to structures in the Marina District. Although the findings are preliminary, the early release of these findings is prompted by the need to guide reconstruction and future planning in the Marina District.

The results, summarized in this report, of the post-earthquake investigation in the Marina District are based on several activities. Immediately following the earthquake, geologic effects, including ground deformation, ground cracks, and sand boils, were mapped and sampled. Simultaneously, portable seismic recording instruments were deployed to monitor aftershocks. Later, subsurface conditions beneath the district were delineated on the basis of historical documents which describe the history of development and filling of the Marina, a compilation of borehole data, and a modest drilling program conducted by the USGS. In addition to depicting subsurface conditions, the areal distribution of damaged structures was compiled.

This report summarizes the geology and engineering properties of soils beneath the Marina District, the areal distribution of geologic effects and damaged structures, and the implications of the aftershock recordings that were collected in the district. The observations suggest that structural damage from the Loma Prieta earthquake was caused by both liquefaction and earthquake shaking which was more severe in the district because of amplification by underlying geologic deposits.

NATURAL AND ARTIFICIAL DEPOSITS IN THE MARINA DISTRICT

M.G. Bonilla

Introduction

The purpose of this section is to describe the geology and history of emplacement of artificial fills in the Marina District. The scope of the section is limited both by its purpose and the fact that it was prepared with a time deadline. Because of the restraints in scope and time, not all available information was assembled or analyzed. This report is thus preliminary and subject to future modification.

Certain drill hole logs (*i.e.*, descriptions of materials encountered) were made available to us with the understanding that they not be published. Some of the information whose source is not given in the following text is based on such proprietary information.

Acknowledgements

Substantial help in finding sources of information was provided by Jackie Freeberg of the U.S.Geological Survey Library in Menlo Park, Roger Bonilla of the Palo Alto City Library, Nicole Bouche of the Bancroft Library in Berkeley, and Gladys Hanson and Patricia Akre of San Francisco Archives, Main Library, San Francisco. Jane Wegge of the State Lands Commission, Sacramento, expedited the selection and duplication of out-of-print maps. F. R. Rollo of Treadwell Associates, Inc., San Francisco, provided access to some unpublished data. Julius Schlocker identified the rock in drill hole USGS WSS. B.L. Roth and T.D. O'Rourke prepared very useful three-dimensional diagrams of the surfaces of the bay mud.

History of the Marina District

Historical Development

Human modification of the natural environment of the Marina District has had a profound effect on subsurface conditions there. The original, natural conditions will be described, and then the modifications that occurred during settlement of the area. The earliest accurate map of the shoreline in the Marina District, dated 1851 (fig. 1), shows a small embayment west of the hill now occupied by Fort Mason. For convenience, this embayment is referred to in the following text as Marina cove. The map also shows a meandering tidal slough draining a marsh that extended westward into the Presidio from the vicinity of the present Scott Street. North of this principal slough was a broad sandy area of beach sand, probably with a thin discontinuous cover of dune sand, referred to as Strawberry Island (Dow, 1973). The north edge of Strawberry Island is labelled Sand Point on the 1851 map. A narrow waterway extended northwest of the mouth of the principal slough, just reaching the present position of Beach Street east of Broderick Street. Another small waterway, trending northeast, lay east of the principal slough. A narrow strip of beach sand was to the north and a broad area of dune sand was to the east of this waterway. The features shown on an 1857 U.S Coast Survey map, designated T687, are almost the same, except for shortening of the narrow northwest-trending waterway and an eastward shift in the positions of the mouth of the principal tidal slough and associated sand spits at the south end of the Marina cove. These changes were very likely natural, as no roads or structures are shown near the shores.

By 1869 (fig. 2) the mouth of the principal slough had shifted westward, probably by natural processes, and the northwest- and northeast-trending narrow waterways mentioned above no longer exist. Probably both of the narrow waterways were artifically filled, at least in part, as roads are shown crossing their former sites. A roadway, undoubtedly on fill, is shown partially crossing the principal slough along the present position of Divisadero Street at Francisco Street. The Fillmore Street wharf, built in 1863 and 400 ft long (Dow, 1973, p. 95), is shown extending into Marina cove north of the present position of Bay Street at Fillmore Street; presumably the wharf was built on piling. East of the Fillmore Street wharf is an artificial fill, perhaps 100 ft long, along the east side of the present position of Webster Street and south of the present position of North Point Street. The symbol used on the 1869 map suggests that this fill was of sand.

In the 1860s a hotel, shooting gallery, and other structures were built north of the present site of the Palace of Fine Arts. The Santa Cruz Power Co. had a small wharf in the same vicinity (Dow, 1973), probably one of the two wharves shown on figure 2 northwest of Marina cove at a site north of the present-day Marina Blvd near Yacht Road. The Phelps Manufacturing plant, which made bolts, heavy forgings, railroad cars, and cable was built in 1882 in a triangular area bounded by present-day Fillmore, Bay, and Buchanan streets (Dow, 1973, p. 95).

In 1891 the San Francisco Gas Light Company built a pier extending 1,000 feet north of Bay Street at its property east of the Phelps plant. Based on the pier's effect on the shoreline after several years, Dow (1973, p. 97) suggests that the pier was not built on piling but was constructed of fill. This pier is shown in figure 3.

By 1894, a sea wall had been built around property owned by J.G. Fair. The sea wall was probably built of rock from nearby hills (Dow, 1973, p. 96). According to Dow (1973, p. 101), this is the sea wall that retained the hydraulic fill placed for the 1915 Panama-Pacific International Exposition, and was at or near the present sea wall north of the Marina Green. However, the sea wall shown on the 1895 map (fig. 3) only partly coincides with the present sea wall, and does not reach the east or west shores of the lagoon.

There was a dump at the foot of Webster and Bay streets in the 1900s (Khorsand, 1973, p. 35). From the context of the description, this was most probably before the 1906 earthquake. How much debris, if any, from the 1906 earthquake was encorporated in fills in the Marina District is unknown and, as described in a following section, 1906 debris would be difficult to distinguish from the Panama-Pacific International Exposition debris. Two historical accounts that cover the Marina District (Dow, 1973; Khorsand, 1973) make no mention of any dumping of 1906 debris at Harbor View (present-day Marina District). Two general reports on the 1906 earthquake state that debris from the main part of San Francisco was dumped in Mission Bay and some was hauled by barges to the vicinity of Mile Rock, west of the Golden Gate (Bronson, 1959, p. 170; Sutherland, 1959, p. 197). Probably a small amount of 1906 debris is in the fills. Other dumps, related to the Panama-Pacific International Exposition, are mentioned in a following section of this report.

In the post-1906 period, the largest changes in the Marina District were made in connection with the 1915 Panama-Pacific International Exposition. These changes are described in detail in the section on artificial fills. In 1912 large hydraulic fills were placed in the central part of the Marina, and in adjacent parts of the Presidio. Smaller hydraulic and other fills were placed through 1917, during restoration of the site of the Exposition.

After restoration of the Exposition site, the land was unused until 1924, when sale of the land to developers quickly led to residential construction (Dow, 1973, p. 103-108). Various modifications were made in the yacht harbor area, north of Marina Boulevard. These included enlargement of the harbor, changes in breakwaters and sea walls, and the addition of some small fills.

Effects of pre-1989 earthquakes

The 1868 earthquake produced minor effects and the 1906 earthquake substantial effects in the Marina District. In the 1868 earthquake, which originated on the Hayward fault, a fissure opened on the beach at the foot of Webster Street below the high water mark (Lawson and others, 1908, p. 438). Based on this description and the 1869 map, this fissure was about half way between Bay Street and North Point Street. Shaking intensity in the Marina District during the 1906 earthquake was in the second highest category of the intensity scale used by Lawson and others (1908, Map 9). Buildings were not numerous in the Marina District however, and the map of Lawson and others shows that assignment of intensity rating was equivocal for part of the area. Some frame buildings were tilted and some foundation walls were cracked. The Baker Street sewer north of North Point Street was broken and "frail frame buildings were thrown out of the vertical" (Lawson and others, 1908, p. 232). Damage to the San Francisco Gas Light Co. buildings (shown on fig. 3) was more severe. Humphrey's (1907) description of the damage to those buildings gives information on ground deformation, quality of construction and apparent direction of shaking:

> "...none of the buildings escaped damage. The collapse of the stack wrecked the light slate-covered iron roof of the power house and started the fire that destroyed the roof of the boiler house. The ground settled very considerably under the vibrations of the earthquake, and further destruction was caused by the unequal settling of the building. The main shock appeared to come from the north, and the north walls received the greatest damage. The end wall of the retort house was pushed out 1 foot at the center, but was saved from collapse by the tie-rods which held it to the roof truss. The walls were cracked at the northwest and northeast corners. The scrubber and gas-tar holder houses were wrecked, the heavy wooden roof truss collapsing. Nearly every wall was moved slightly, but the brickwork was generally very good, and apparently had cement in it. The exhaust house had three intermediate walls, 18 inches thick at the top. The north wall and the next one fell into the building, the side walls being pushed out 6 inches. The building had wooden roof trusses and the north truss cracked at the center mortise. The floor settled badly around the condensers. The gas holder collapsed from the sudden release of the gas due to a break in the mains. The trestle pier extending into the bay also collapsed." (Humphrey, 1907, p. 27-28).

The damage described above is not clearly related to areas of artificial fill. The locations of the individual buildings mentioned in the quotation are unknown; however the group of buildings labeled "San Francisco Gas Light Co." on the 1895 map straddle various materials, including dune sand, beach sand, and artificial fill. The damage was thus not confined to artificial fill. The distribution of areas of various intensities shown in the 1906 report in other parts of the Marina District is also not clearly related to areas of artificial fill (Lawson and others, 1908, Maps 17 and 19).

Geology

Bedrock and unconsolidated natural deposits

Bedrock

The bedrock underlying the Marina consists of the Franciscan Formation and serpentine. Nearby outcrops (*i.e.*, where the rock reaches the ground surface) consist of sandstone and shale, except to the west where serpentine is also exposed (Schlocker, 1974). Shale bedrock was found in a drill hole near the south end of the Palace of Fine Arts, but sheared bedrock in a U.S.G.S. drill hole southeast of the intersection of Divisadero and Beach streets consists mostly of serpentine.

The configuration of the bedrock surface under the Marina District is very poorly known, but the gross shape is of a half basin deepening northward. The bedrock surface is probably irregular and cut by erosional valleys, as it is in other parts of the San Francisco Peninsula (Bonilla, 1964; Schlocker, 1974, pl.3). A likely lower limit of depth to bedrock in this area is the elevation of the bedrock sill at the Golden Gate, which is about 400 ft below mean sea level (Carlson and McCulloch, 1970). The nearest outcrops of bedrock are in Fort Mason to the east and at the intersection of Scott and Greenwich streets to the south (Schlocker, 1974). To the northwest, at Anita Rock in San Francisco Bay, bedrock nearly reaches the water surface (Carlson and McCulloch, 1970). Drill holes encountered bedrock in the Marina at the following depths below mean sea level: 147 ft at Lombard Street west of Fillmore Street; 256 ft on Buchanan Street south of Bay Street, 252 ft on the south side of Beach Street east of Divisadero Street; and about 75 ft at Lyon Street north of Bay Street (Bartell, 1913; Whitworth, 1932; U.S.G.S. hole WSS; and unpublished data).

Unconsolidated natural deposits

The Franciscan Formation in the Marina District is buried by a complex sequence of unconsolidated deposits. The term "unconsolidated" is used here in the geologic sense (*i.e.*, not hard rock) rather than the geotechnical sense. The complexity of these deposits can be partially understood by reviewing the recent geologic history of the San Francisco Bay estuary system. During the last million years at least four periods of estuarine (bay) deposition occurred in San Francisco Bay, separated by periods in which the level of the ocean was lowered because ocean water was incorporated in glaciers (Atwater, 1979). The lower sea levels during glacial periods resulted in erosion of valleys in the then-existing deposits. Exposure of the deposits resulted in near-surface dessication and oxidation which made the deposits more firm and produced the brown colors commonly reported in drill holes. This complex geologic history produced a variety of unconsolidated geologic units in the Marina District and surounding area, including bay, marsh, beach, and dune deposits.

Some of the oldest unconsolidated deposits in the vicinity of the Marina are part of the Colma Formation (Schlocker, 1974, plate 1), commonly a weathered sand thought to have originated primarily as a beach deposit. This formation is estimated to be 500,000 or more years old (Helley and Lajoie, 1979). Clayey sand encountered in a drill hole in Fort Mason at an elevation of 42 ft below mean sea level may be part of the Colma Formation (Schlocker, 1974, p. 70).

During the last interglacial time (Sangamon Interglaciation, 75,000-125,000 years ago) an extensive estuarine deposit formed in San Francisco Bay (Atwater and others, 1977; Atwater, 1979, fig. 3). This is sometimes called older bay mud. The thick clay encountered at a depth of 76 ft in U. S. Geological Survey drill hole WSS, south of Beach Street and east of Divisadero Street, is probably the 100,000-year-old estuarine deposit, rather than the still older estuarine deposits that formed in San Francisco Bay.

Sea level during the last (Wisconsin) glaciation was 300-400 feet (90-120 m) lower than it is now and the ocean shoreline was probably near the Farallon Islands. Holocene estuarine deposits (bay mud) accumulated during the sea level rise that followed the last glaciation. The rising sea is estimated to have entered the Golden Gate 10,000-11,000 years ago (Helley and Lajoie, 1979, p. 18). The bay mud, generally a clay or silty clay, formed the bottom of the small shallow embayment that occupied much of the area where the Marina District was built, and underlies most of the District. Some of the geotechnical properties of the bay mud in the Marina District are treated elsewhere in this report, and general descriptions of bay mud are given by Schlocker (1974) and by Helley and Lajoie (1979).

Tidal marsh deposits, now covered by artificial fill, underlie a narrow band in the southwest part of the Marina District and continue into the Presidio. The marsh deposits consist of clay and silt containing small quantities of marsh vegetation. The marsh deposits join the bay mud that underlies the Marina District, and in places interfinger with beach sand. General descriptions of marsh deposits around San Francisco Bay are given by Helley and Lajoie (1979) and Atwater and others (1979).

Modern beach sand underlies the northwest part of Marina District and forms a narrow strip in the southeast part of the District. Modern dune sand (wind-deposited sand) underlies the eastern and southeastern part of the District. The bulk of both of these deposits consists of clean, well-sorted (*i.e.*, the grains are essentially the same size) sand. The areal distribution and a detailed description of the beach and dune sands are given by Schlocker (1974).

<u>Artificial deposits</u>

The historical development of the area, summarized above, provided information on where old sea walls, piling, and perhaps other artificial materials may be present in the Marina District. Because the Panama-Pacific International Exposition had such a large effect on the environment of the Marina, additional details concerning it are given below. The positions of the shoreline and associated features shown on old topographic and planimetric maps were used to document the age and areal extent of artificial fills in the Marina District, including fills related to the Exposition.

Panama-Pacific International Exposition

Pre-existing conditions

Prior to modifications for the Panama-Pacific International Exposition detailed maps were made showing existing conditions. One map had a one-foot contour interval and another had soundings in the Marina cove (Todd, 1921, v. 1, p. 299). Unfortunately we have been unable to find copies of those maps. However, the general conditions before construction of the Panama-Pacific International Exposition can be inferred from a map showing artificial fills in 1906 (Lawson and others, 1908, Map No. 17), verbal descriptions, and a photograph taken in 1912.

By 1906 the Marina cove was enclosed except for a narrow opening to the north, and had a rim of artificial fill around it (fig. 4). Little historical information is at hand as to the method of placement or nature of this fill, but segments of the sea walls shown on the 1895 map must have been incorporated in it. Two borings made through the fill in 1975 encountered sand containing some rock fragments, brick, and other rubble (Dames and Moore, 1976). Artificial fill had also been placed over the eastern part of the principal slough as far west as the present position of Lyon Street. A photograph (fig. 5), probably taken in April of 1912, shows conditions similar to those shown on the 1908 map. A notable difference is that the photograph shows a broader area of fill on the east side of the cove than is shown on the 1908 map.

Prior to changes related to the Panama-Pacific International Exposition, the cove was "...12 feet in depth to the mud at mean high tide, formed by a sea-wall running east and west along the line of what became the northern boundary of the grounds." (Markwart, 1915a, p. 63). These water depths suggest some shallowing of the cove occurred after 1896, as the U.S. Coast and Geodetic Survey map H2254, surveyed in 1895-1896, shows similar depths measured from a lower datum, mean low water.

Artificial fills related to Panama-Pacific International Exposition

Hydraulic filling of what remained of the Marina cove was done from April 13 to September 7, 1912. A suction dredge was positioned about 300 ft offshore, and generally moved parallel to the shore. If the discharge had too much soft material, the dredge was moved to get a larger proportion of sand. The Marina cove that was being filled contained semi-fluid sludge. A gate was left in the old sea wall so that the sand, discharged on the landward side, would displace as much as possible of the soft material into the Bay. The fill was about 70 percent sand and 30 percent mud (Todd, 1921, v.1, p. 300).

The process of filling was also described by Markwart (1915a, p. 64-65): "...whenever the discharge contained too high a percentage of mud and silt, the dredgers were moved to positions in the Bay where the banks contained more sand." To help remove mud from the bottom of the original basin, "...at times water was pumped instead of sand and this carried out considerable mud in solution through the waste gate."

Hydraulic fill was also used west of Lyon Street in a low-lying area along an old tidal channel. The rougher water offshore from this area required a sea-going dredge (Todd, 1921, v.1, p. 300). This fill did not exceed 6 ft in depth. It was "...mostly sand with a slight percentage of mud and frequently large boulders..." (Markwart, 1915b).

"Six or eight acres, on part of which lay the eastern half of the Court of the Four Seasons, had to be filled by scrapers to bring it up to grade..." (Todd, 1921, v.1, p. 162). The center of this court was southeast of the intersection of Beach and Broderick, on the old sand spit formerly called Strawberry Island. The northwest-trending waterway mentioned above in connection with the 1851 map was in this area and may account for the need for a special fill. The east half of this court would cover only about one acre. This is probably the same fill described by Dow (1973, p.101) as covering twelve acres. Dow infers that the source of this fill was dune sand from the undeveloped land at the east end of the Exposition grounds (Dow, 1973, p.101- 102).

The method of placement of the fill in a band one-half block wide between the 1895-1906 fill and the 1891 San Francisco Gas Light Company pier is uncertain. The 1906 map shows a sand pattern without a definite boundary to the north, which suggests that natural sedimentation was taking place there. This strip was filled by the time of the Panama-Pacific International Exposition, and no doubt was filled for the exposition, but the information at hand does not discuss the method of filling.

Piling

A large amount of wooden piling from the Panama-Pacific International Exposition probably still exists in the Marina District. The piling may have affected the damage that occurred in the 1989 earthquake. Driving of the piling must have caused local densification of the hydraulic fill, and the piling would provide resistance to both vertical and horizontal ground movements. Thus the piling could affect both long-term differential settlement and earthquake-generated ground displacements. Todd (1921, v. 5, p. 247) says "By permission of the owners the piles driven at various points remained." Specifications for dismantling of the exposition include the statement that piles: "...shall be cut off two (2) feet below the surface of the ground as it existed at the time the site was taken over" (Todd, 1921, vol. 5, Appendix p. 134). Unfortunately the position of the ground surface referred to by Todd is unknown to us. The exposition structures were designed for a life span of only a few years and the piles were probably not treated with creosote. Thus, the parts of the piles above the water table may have deteriorated because of decay and termite action; however wooden piles that are submerged, *i.e.*, below the water table, last a very long time.

An understanding of the number and spacing of the piles can be gained from the specifications for the Mines Building and Varied Industries Building, which call for piles to be in clusters ranging from two to ten piles, the clusters to be about 28 ft apart from north to south and 82 ft apart from east to west. The estimated quantity of piling for these two buildings was 3,000 piles totaling 122,000 lineal feet (Markwart, 1915a, appendix B). The total amount of piling for the entire exposition was about 500,000 lineal feet (Panama-Pacific International Exposition Company, 1913).

Piling was used for the structural frame of eleven of the twelve large buildings and for the floor substructure of five of the large buildings (Markwart, 1913, table 1). Piling was also used for the columns supporting reinforced concrete fire walls (Markwart, 1915b).

Piles varied in length from 16 to 75 ft, and they were to be driven into a layer of green sand and clay that underlies the site. One of the reasons for using piles was for greater safety in case of earthquakes (Markwart, 1913, p. 902).

Post-Exposition changes

Demolition of Exposition buildings

After closure of the Exposition, dynamite was used to bring down buildings and other structures. Almost all of the Exposition structures were of wood. Wood that could not be economically salvaged was burned on the site on a daily basis by the fire department (Todd, 1921, v. 5, p. 246-247). Reinforced concrete firewalls, foundations, and transformer vaults were dynamited and broken up by a pile hammer (Todd, 1921, v. 5, p. 246)

Restoration of Exposition site

As previously stated, piling was not removed but foundation obstructions were removed to some unknown depth: "By permission of the owners the piles driven at various points remained. Foundation obstructions had to be removed and the streets and sidewalks brought back where they belonged." (Todd, 1921, v. 5, p. 247).

Post-exposition filling was also done, and was described as follows: "Some of the lands had not been filled up to the terms of the leases when they were built upon, and it was now necessary to carry out this part of the Exposition's obligations. They were filled partly by the public dump method, but by September, 1916, a suction dredge went to work pumping mud over them, and finished by January, 1917" (Todd, 1921, v. 5, p. 247). One public dump was at Lobos Square,which is now the site of the Marina Jr. High School and the Moscone Recreation Center, which are southeast of the intersection of Bay and Webster streets. The location of other dumps is not given. The dredger fills in the post-Exposition period required construction of retaining levees, but their thickness and areal extent are unknown.

Changes after 1917

The land on which the Exposition stood was unused until 1924, when residential construction began (Dow, 1973, p. 103-108). Any fills related to residential construction are probably very small. Some modifications have also been made in the yacht harbor area, including enlargement of the harbor, changes in breakwaters and sea walls, addition of some small fills, and construction of a major sewer line under Marina Boulevard in the early 1980s.

General distribution and age of artifical fills

The areas of artificial fills of various ages are shown on figure 6. This figure is based on superimposing, on a 1973 map, the maps of 1851, 1869, 1895, and 1908, supplemented by descriptions of the Panama-Pacific International Exposition fills. The dotted line in the southwest part of the map separating the 1869-1895 fill from the 1895-1906 fill is taken from the 1899 edition of a topographic map that was surveyed 1892-1894 (Lawson, 1914, Topography, San Francisco Quadrangle). The map (fig. 6) shows the major time spans during which the fills were emplaced, but each of the outlined areas may contain small fills younger than the designated ages. Small fills probably exist beyond the areas of fills shown on figure 6.

Part of the shoreline in 1851 is shown by a dashed, curved line that trends generally northwest. The area between this line and the edge of the 1869-1895 fill probably grew by natural sedimentation. The narrow northwest- and northeast-trending 1851-1869 fills may include naturally-deposited material along waterways, as mentioned previously.

The 1869-1895 fill probably consists mostly of sand, which was locally available from beach and dune deposits. In places it may contain riprap (large blocks of stone) placed for protection from wave action. The rectangular, north-trending area in the northeast part of the 1869-1895 fill (fig. 6) is the site of the 1891 Pacific Gas Light Co. pier. Examination of the 1895 map (fig. 3) suggests that it had a rim of riprap, and, as mentioned previously, it probably was a solid fill rather than a pier constructed on piling. Here and there, debris from factories and other sources probably is contained in the fill also.

The source and method of emplacement of the 1895-1906 fill are are largely unknown. The fill no doubt contains remnants of sea walls shown on the 1895 map, and J.G. Fair's sea wall.

The 1906-1917 fills were principally emplaced in 1912 for the Panama-Pacific International Exposition, using the hydraulic fill methods previously described. As noted before, this area also includes post-Exposition hydraulic fill of unknown dimensions, an area of scraper fill, and public dumps, all related to the Exposition.

In general, the fills in the Marina District are principally sand obtained from nearby sites on land or offshore. The varied history of development of the area, both cultural and physical, implies that a great variety of materials are encorporated in the fills. The following two excerpts from drillers' logs illustrate the point. "Heavy timbers in bottom—moved over 3 feet. Dug down 10 feet. Bored down 5 feet to elevation -11. Rocks too big. Could not get a 6-inch auger through them. Moved 20 feet away. Dug down 8 feet to big rocks. Abandoned." (Whitworth, 1932, Hole 19, dated 1/24/23, near Beach and Webster streets). "Made two attempts through sand filling to depth of 22 feet. Encountered large rocks. Abandoned." (Whitworth, 1932, Hole 22, dated 1/24/23, near southwest end of San Francisco Gas Light Co. pier).

A cross-sectional view of the Marina District

An east-west section of the Marina District is shown in figure 7. This figure utilizes the historical data discussed above, and logs of holes, drilled from 1912 to 1990, that are within about one-half block of Beach Street. Most of the drill holes are represented by rectangles, and the materials found in them are shown by patterns. The patterns between holes represent an interpretation and correlation of the geologic units encountered in the holes. In making the correlations, interpretations different than shown on certain drill logs were made. Usually this involved artificial fill, which can be very difficult to distinguish from natural materials unless exotic materials such as bricks are found. The locations of proprietary drill hole logs that were also used in constructing the section are shown by a special line symbol. The locations of street centerlines are marked by short vertical lines and old shoreline positions are indicated by arrows. The vertical scale of the section is 6.7 times larger than the horizontal scale, and therefore the slopes in the section are greater than in the ground. Following are some additional comments regarding the section and its interpretation. Because the youngest deposits are of most practical interest, they are discussed first.

Artificial fill

The principal area of hydraulic fill is labeled on the section (fig. 6). This is probably the maximum limit, because the indicated boundaries are based on a 1906 map, and non-hydraulic fill could have been added between 1906 and the commencement of hydraulic filling on April 13, 1912. For example, a photograph (fig. 5) shows probable non-hydraulic fill on the northeast side of the cove in 1912, prior to placement of the hydraulic fill.

Three drill holes in the vicinity of Retiro Street support the historical data indicating that fill was placed after the 1912 hydraulic fill. The ground surface was about 5 ft lower than it is today when holes labelled ASCE 11, 10A, and 11A (near 3500 ft on the horizontal scale of fig. 7) were drilled. These holes were drilled in 1912, after placement of the main hydraulic fill. Official street grades in that vicinity were between 3 and 4 feet (Anonymous, 1900), and this increment of fill was probably placed during restoration of the 1915 Exposition site. The vertical position of hole ASCE 5A (near 1100 ft on the horizontal scale of fig. 7) also suggests post-1915 fill; this is explained by the fact that the drill hole is about half a block south of the line of the section (*i.e.*, fig.7), in a former marsh area.

The bottom of the artificial fill as revealed by drill holes is very close to the bottom of the cove as indicated by soundings made in 1895-1896. This implies that the hydraulic filling process in 1912 neither vertically displaced or eroded the mud at the bottom of the cove to any great extent, at least along the present line of Beach Street. As noted in the discussion of the Exposition, sometimes clear water was pumped in to remove the softest bay mud, but evidence of that is not recognizable in the section.

A small body of artificial fill in the western part of the section was placed on a marsh area near the Palace of Fine Arts. Some of this was probably placed by hydraulic methods in 1912 for the Exposition.

Natural deposits

Two units of sand are shown flanking the main body of artificial fill. That on the west is known to have been part of the sand spit called Strawberry Island. This beach sand apparently had a thin cover of dune sand over it, and the western sand unit probably contains some interbedded dune sand. The eastern sand unit probably is part of an extensive dune field that existed in the eastern part of the Marina. The eastern unit probably includes beach sand, as the old maps show a narrow strip of beach sand on the southeast side of Marina cove.

Underlying the main body of artificial fill is the bay mud. This Holocene estuarine deposit extends westward under the beach and dune sand to the vicinity of the Palace of Fine Arts. The top of the bay mud is rather even, but the bottom descends eastward and the bay mud thickens greatly east of Fillmore Street, and then lenses out eastward near Buchanan Street. The valley-like shape of the bottom of the bay mud extends northward and is well shown in the three-dimensional view (fig 8) prepared by Bruce L. Roth and T. D. O'Rourke of Cornell University.

A layer of green sand and clay, often described as hard, was found to underlie the bay mud in 80 percent of the holes drilled for the 1915 Exposition. As previously noted, the Exposition piles were founded in this layer. About 90 percent of the Exposition holes reached a "yellow hardpan" beneath the hard green sand and clay. Although descriptions vary, several 1989 and 1990 drill holes found a similar zone containing hard or firm layers which produced high peaks on the cone penetrometer test records. Although not reported in all of the drill holes shown in figure 7, the hard zone seems to extend from the Palace of Fine Arts at least as far east as Fillmore Street, and is shown with a special symbol in figure 7. The zone has an irregular surface not only in the section along Beach Street but in other parts of the Marina as well. An interpretation of the hard zone is that the "yellow hardpan" is the top of an erosion surface that formed during the the low sea level of the last glaciation, and the green layer (sand and clay) formed in the early stages of estuarine deposition. Placement of the green layer with the estuarine deposits is supported by the fact that it is locally interbedded with the bay mud (Whitworth, 1932, borings 2, 11, and probably 14A). If the interpretation is correct, the hard zone (*i.e.* the "hardpan" and the hard sand and clay) is near the boundary between the Holocene and Pleistocene, and is about 10,000 years old. Samples collected above and below this zone in drill hole USGS WSS (at the Winfield Scott School) proved difficult to process for an age determination by the radiocarbon method (Steven Robinson, U. S. Geological Survey, personal communication, 1990) and may not clarify this matter.

In nearly all the Exposition holes (Whitworth, 1932) in which bay mud can be identified, the green sand and clay is directly below the bay mud, and the "hardpan" is six feet or less below the bay mud. All five of the USGS drill holes and at least two proprietary drill holes shown in figure 7 encountered a dense or hard zone within a few feet of the bay mud. Thus the bottom of the bay mud as shown on figure 8 approximates the top of the hard zone and the probable Holocene-Pleistocene boundary.

Sand and silty sand encountered below the hard zone in drill hole USGS WSS is probably not the Colma Formation, because the Colma Formation is probably 500,000 years old or older. The sand and silty sand are probably less than 100,000 years old, because they overlie the thick clay (older bay mud), thought to be about 100,000 years old, found in the drill hole.

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Figure captions

Figure 1. Part of U.S. Coast Survey chart No. 314, dated 1851. The original map is at 1/10,000 scale and distinguishes marsh and sand areas by line pattern. "Pt San Jose" was later called Black Point. Bracketed labels are not on original map.

Figure 2. Part of U.S. Coast Survey map No. 3055, dated 1869. The original map is at 1/40,000 scale and has a 20-foot contour interval. Surveys for this map were done in 1850-1857 and 1867-1868.

Figure 3. Part of U.S. Coast and Geodetic Survey Register No. 2205, surveyed in 1895. The intersection of North Point and Buchanan streets is in the center of the San Francisco Gas Light Co. group of buildings. The building in the southeast corner of the intersection still exists, and is called the Pacific Union Company building. The "Cal. Pressed Brick Works" is northeast of the intersection of Jefferson and Broderick streets. The boundary of the Presidio in 1895 is shown as a dash-dot line. Bracketed labels are not on original map, which is at 1/10,000 scale.

Figure 4. Part of the geologic map published in the report on the 1906 earthquake (Lawson and others, 1908, Map No. 17, 1/40,000 scale). Bracketed label is not on original map, which identified by color the area of artificial fill surrounding Marina cove.

Figure 5. Photograph of the Marina area taken in April (?) 1912. The cove that was filled for the Panama-Pacific International Exposition is in the right middle ground. Piling from the angled pier shown on the 1895 map (figure 3) is in the lower right side of the cove. The pond in the left part of the photo is now part of the lagoon at the Palace of Fine Arts. Next to the pond is Baker Street, and then Broderick Street. Photo courtesy of Archives, San Francisco Main Library.

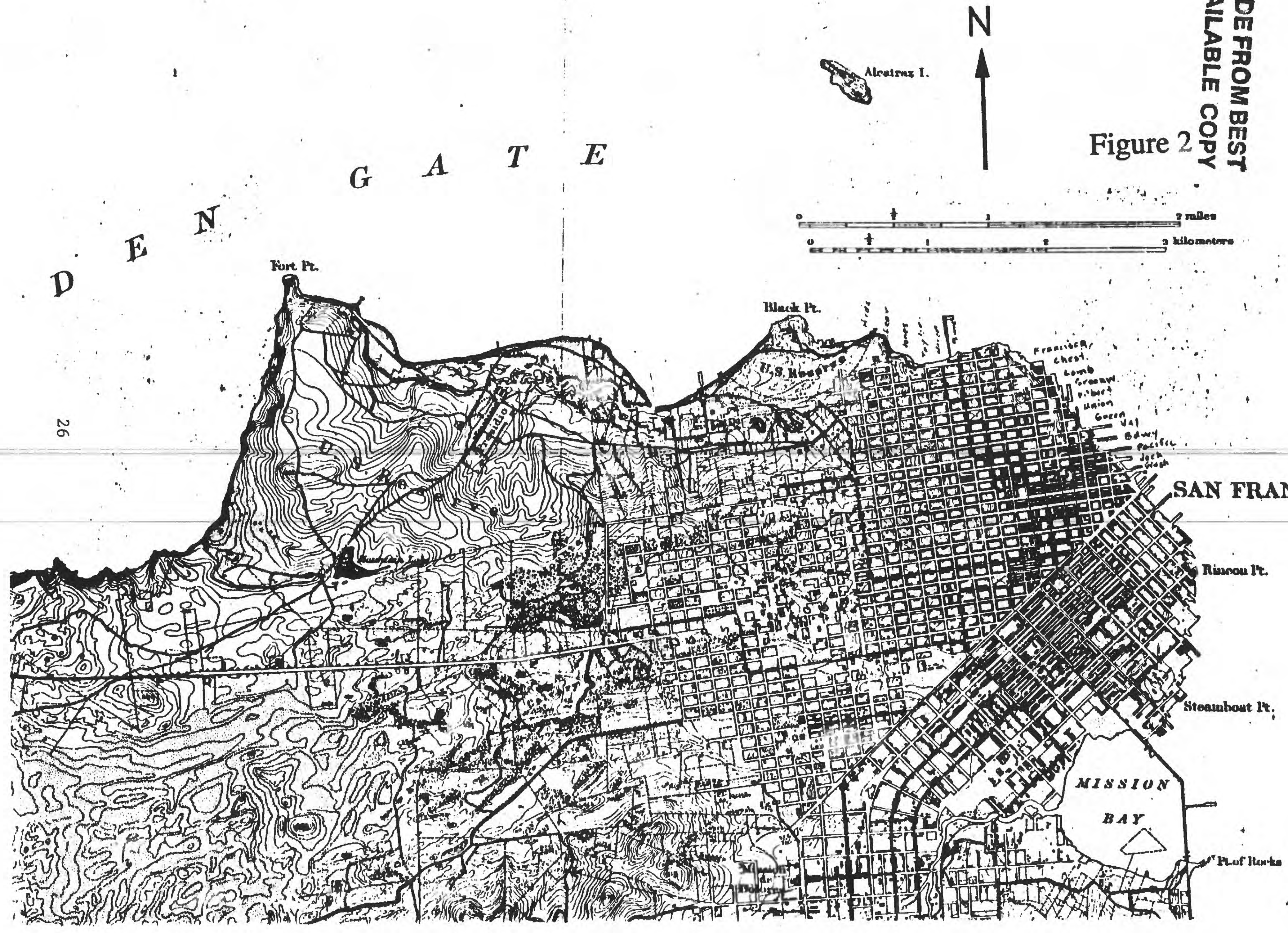
Figure 6. Map showing artificial fills of various ages. Curved northwest-trending dashed line in left part of map represents part of the 1851 shoreline. Shorelines and other features on the old maps cannot be precisely related to modern maps because the positions of many natural and cultural landmarks are shown differently on the old and new maps, and a best-fit compromise must be made by superimposing the maps at a common scale. Thus, the fill boundaries and other features shown may be in error by 100 ft or more. The fills near the Fort Mason docks and the Marina yacht harbor are not delineated. The Fort Mason fill was placed between 1895 and 1909, and the fills north of the yacht harbor are post-1914.

Figure 7. East-west cross-section of the Marina District along Beach Street, from the Presidio to Fort Mason. The location of the cross section is shown on figure 6 by the line labeled A-A.' Horizontal position of fill boundaries and shorelines may be in error by 100 ft or more. Sources of drill hole logs are identified as follows: ASCE, Whitworth (1932); DM, Dames and Moore (1976, 1977); USGS, U.S. Geological Survey, unpublished data, 1990). The U.S. Geological Survey holes were logged by T.E. Fumal and M.J.Bennett. The supplementary drill holes are proprietary information from various sources.

Figure 8. Three-dimensional view of the bottom of the bay mud. As discussed in text, the bottom of the bay mud has nearly the same position and shape as the top of a hard zone, and is close to the Holocene-Pleistocene boundary, about 10,000 years old. The diagram also shows the overlying ground surface, key streets, and locations of some of the borings used in constructing the figure. Figure prepared by B.L. Roth and T.D. O'Rourke, Cornell University.


[Fort Mason]
[Marina cove]
[Strawberry Island]
N
1 MILE
7000 FEET
1 KILOMETER

Figure 1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D E N G A T E
Fort Pt.
Black Pt.
Presidio
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Francisco
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Filbert
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Wash
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Rincon Pt.
Steamboat Pt.
MISSION
BAY
Pt. of Rocks

Figure 2

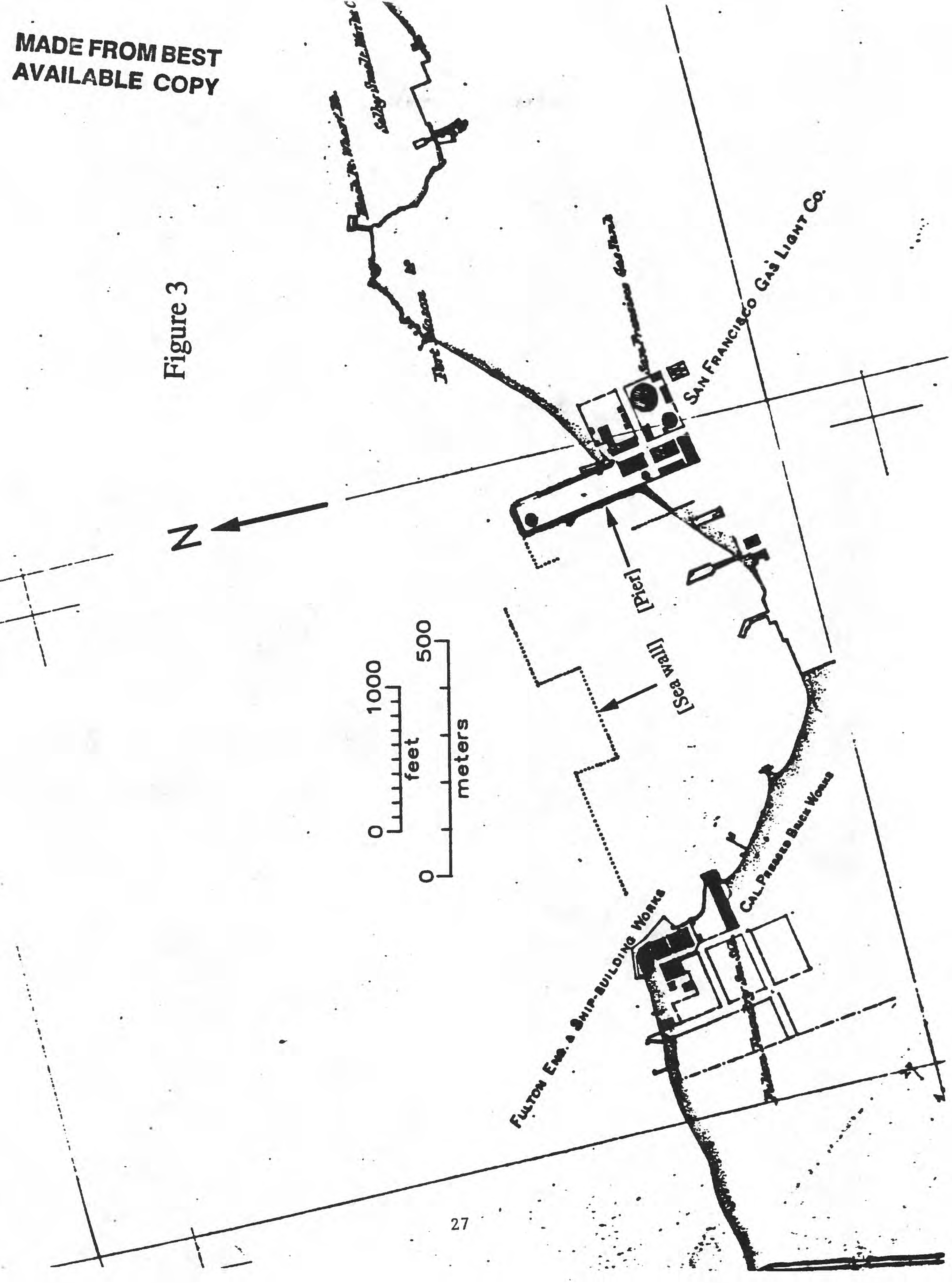
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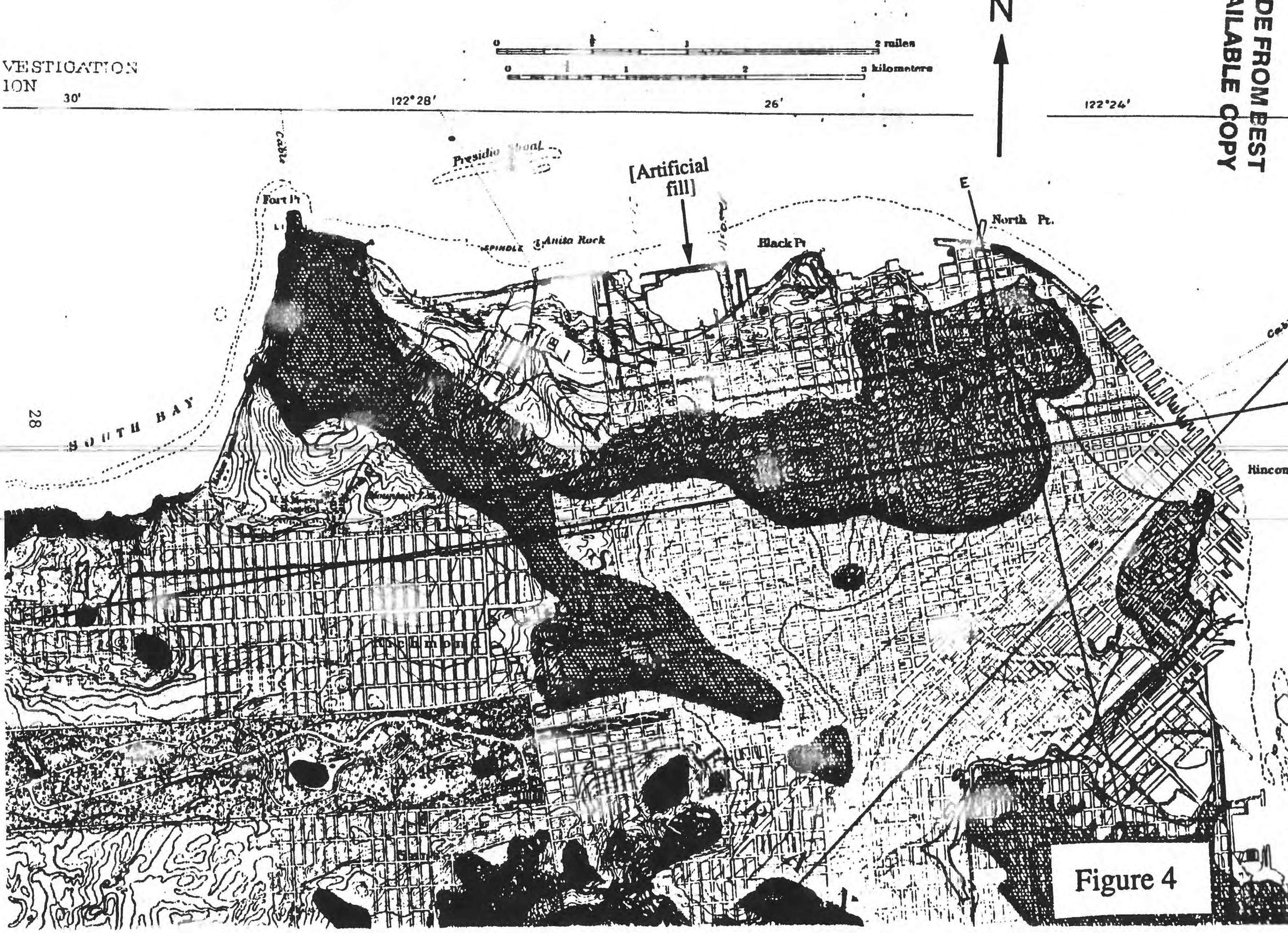
Figure 3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North Pt.
SOUTH BAY
Rincon I

Figure 4

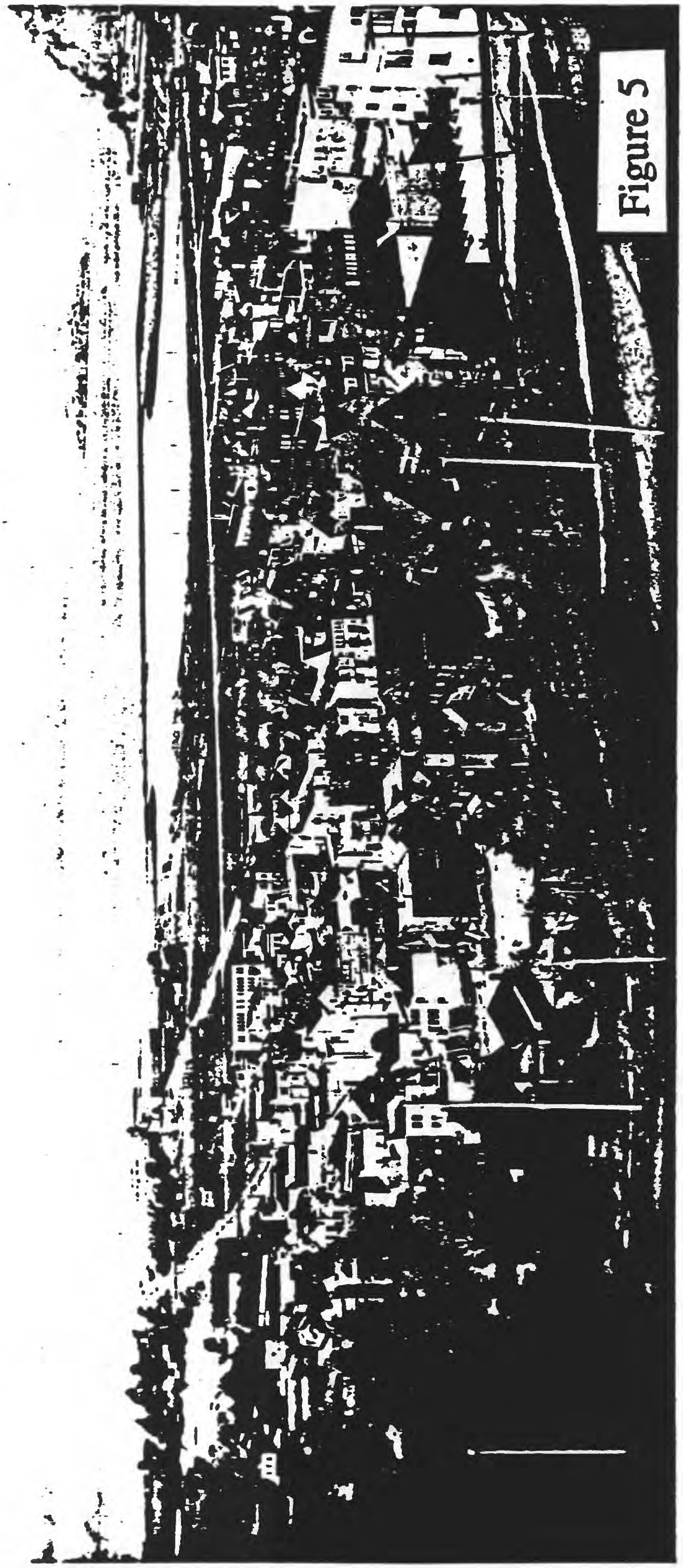
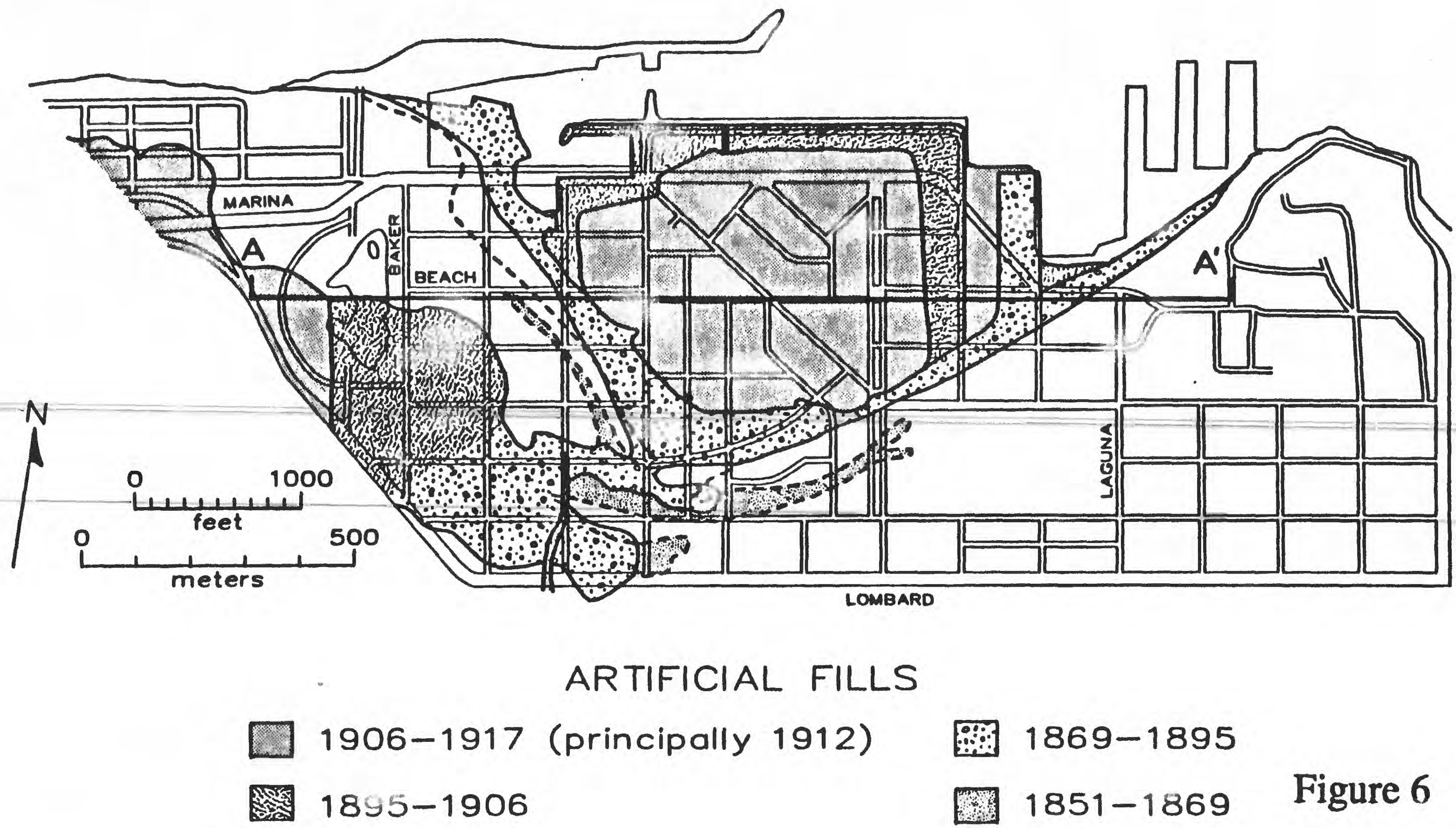


Figure 5


MARINA
A
BAKER
BEACH
A'
LAGUNA
LOMBARD
N
0
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feet
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meters
ARTIFICIAL FILLS
1906–1917 (principally 1912)
1895–1906
1869–1895
1851–1869

Figure 6

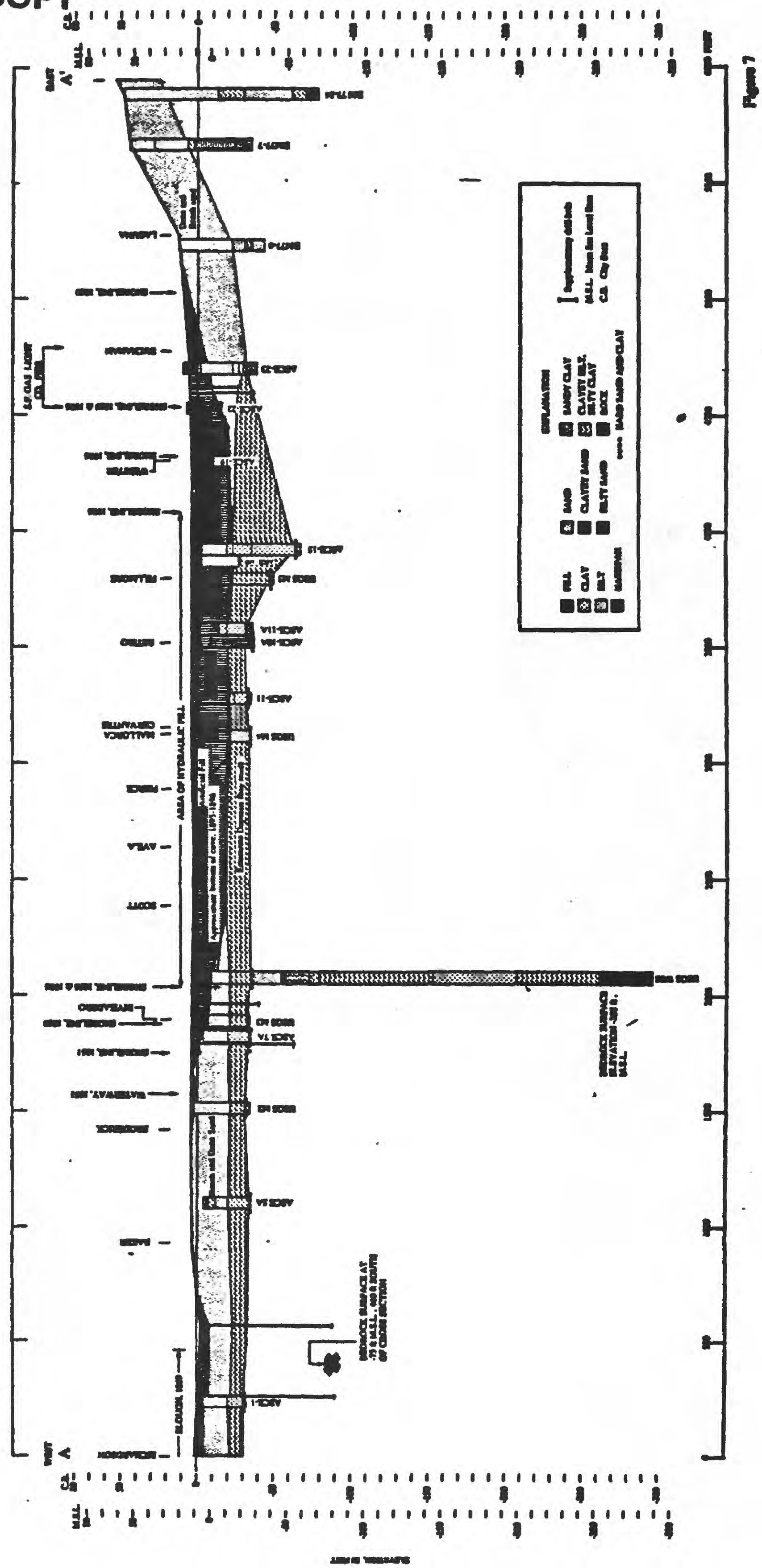
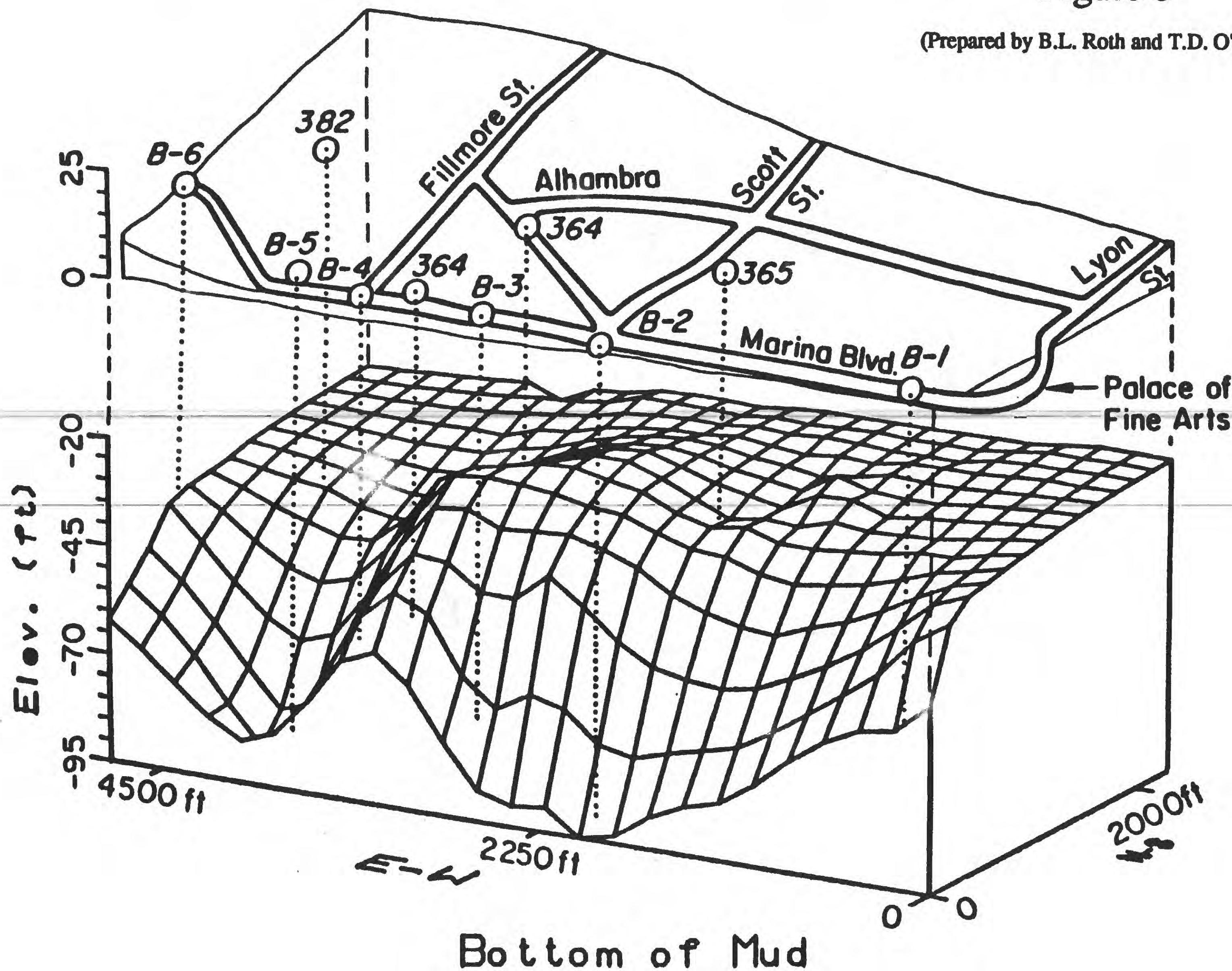


Figure 7

Figure 8

(Prepared by B.L. Roth and T.D. O'Rourke)



TYPES OF STRUCTURAL DAMAGE

M. Çelebi

Introduction

The majority of the buildings in the Marina District of San Francisco are at least 50 years old, are 2 to 4 stories in height, and with the exception of only a couple of new, reinforced concrete buildings, are constructed of timber frames. In general, then, the buildings can be classified in one or more of the following categories:

(a) timber frame with stucco exterior;

(b) timber frame with brick veneer exterior;

(c) timber frame with timber exterior;

(d) masonry (stone and brick);

(e) reinforced concrete frame (few newer buildings);

(f) mixture of all above.

The foundation systems of most of the older buildings are continuous plain (unreinforced) concrete spread footing around the perimeter of each building supplemented by individual plain concrete rectangular block or pedestal footings not much larger than the 6″ × 6″ timber or steel support columns distributed throughout the ground floor. In a couple of exceptional cases, the footings are of masonry. To our current knowledge, no mat or raft foundation exists in the residential buildings of the Marina district.

Damage to the buildings in the Marina was caused by several factors. However, in general, the damage resulted from two main causes:

(a) structural deficiences including those of foundations;

(b) soil-related phenomena.

In many buildings, however, both causes contributed to the damage.

In the following paragraphs, only those factors related to structural performance will be elaborated. Those related to soil behavior are described only in general terms. Detailed description of the soil-related phenomena is presented in other parts of this report.

Types of Damage

Soft First Stories

The ground levels of most buildings are used as garages and semi-living and/or appliance and storage areas. In general, the walls on the ground levels, as well as the rest of the building, are built-up by lath and plaster covering the timber framing system. These walls effectively provide stiffness in their planes. However, the large openings on the ground level to accommodate garage doors decrease the stiffness and strength at that level as compared to the stiffer floors above. This common design problem, known by the engineering community as a "soft first story", is particularly acute in many buildings at street intersections which have garage entrances on two sides and do not have the benefit of the stiffness and strength provided by the adjacent buildings on the two street sides (Figure 1). For many Marina District buildings there was the tendency to provide decorative designs to fit the beauty of the area without providing proportional stiffness and strength in the front facades of the buildings particularly at the ground levels. Thus, the "failure mechanism" formed by the sway of the soft first story, with the more rigid upper floors virtually intact, was common.

Pounding Effects

Due to architectural necessity and lack of buildable land, the owners and developers have been forced to construct adjacent buildings with practically no space between them. In some places, this caused one building to pound against another and possibly adversely affected the fate of some. This behaviour was particularly true for the fate of some corner-of-the-block buildings. In such buildings, the pounding was made more severe by twisting of the buildings because of the eccentricity caused by large openings on both sides of corner buildings.

Differential Settlement

Within the Marina area, there were many examples of the effect of differential settlement of the foundations. An acute example is an elongate zone of differential settlement above an 8 ft. diameter sewer pipe. Damage to structures occurred where they were built above this sewer line (Figure 2). Normally, sewer pipeline systems are laid within the street boundaries with hookups from buildings.

Other Contributory Factors to Damage

The buildings in the Marina District suffered also due to the following conditions which affected their integrity:

(1) *Deterioration:* It was observed that deterioration occurred due to age and/or environmental conditions of the area. In many cases, plain concrete footings were partially disintegrated. Termite and/or dry rot of the timber members underneath the stucco or veneer exterior also was noted (Figure 3).

(2) *Pre-earthquake condition:* The whole Marina District in general was experiencing uniform or differential settlement of some magnitude. Prior to this earthquake, it is likely that a good percentage of these buildings were out of plumb. Evidence of this exist in the driveways, the sloping plain concrete mat in the garages and the structures themselves. Many buildings already had sagging lintels above the garage doors prior to the earthquake.

(3) *Deficient Building to Foundation Anchorages:* Several buildings were not anchored to their foundations. In others, the anchorages were not sufficient (Figure 4). Stairs and steps in many cases were not integrally anchored to the building.

(4) Insufficient or lack of proper connection at joints both at foundation to timber (or steel) column joints, or column top to girder joints of the ceiling at the gound levels.

(5) Lack of tie-beams both at foundation level and at ceilings of ground levels.

Figure Captions

Figure 1. This building, like many others in the Marina District, was at the corner of the block and had a soft first story. The upper floors are stiffer than the first floor.

Figure 2. (a) The building was built on land where an $8'$ sewer pipe crossed. During the earthquake, differential settlement in the vicinity of the pipe adversely affected the building. (b) the differential settlement of the pipe caused a $1''$/foot slope on the sidewalk and consequently considerable damage to the building.

Figure 3. The timber in many buildings was deteriorated as in this one.

Figure 4. (a) Some buildings were not at all or insufficiently bolted to the foundation. This building on Jefferson Street of the Marina District suffered from both the pounding effect from the adjacent buildings; and (b) the lack of sufficient anchorage of the building to the foundation.

Figure 1

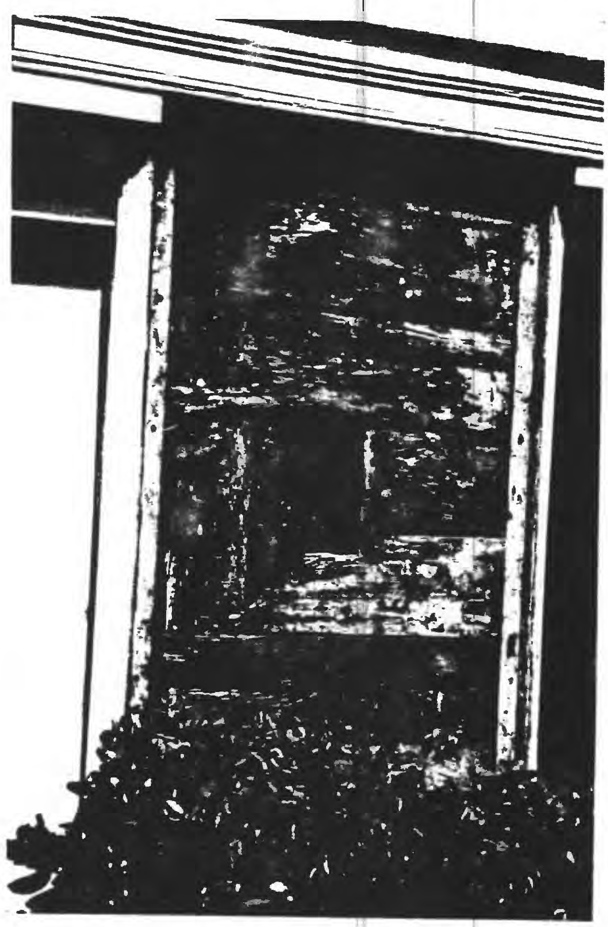


Figure 3

Figure 2A

Figure 2B

Figure 4A

Figure 4B

AREAL DISTRIBUTION OF DAMAGE TO SURFACE STRUCTURES

Linda Seekins, Frank Lew[1], and Lawrence Kornfield[2]

Introduction

In the week following the Loma Prieta earthquake the City of San Francisco Building Department conducted an extensive damage survey in the Marina District. We have plotted the results of this survey as a detailed map in order to investigate the areal distribution of structural damage and to look for correlations between damage patterns and the local geology.

Acknowledgements: The authors wish to thank Lorraine Hollis for the fine job she did of creating figures 1 and 2.

The City Survey

The city assembled a team of more than 500 people to inspect structures for damage. The team consisted of four categories: 1) city staff members, including Building and Electrical inspectors as well as city employees from the Bureau of Engineers; 2) engineers from the Structural Engineers Association of California; 3) inspectors and engineers from other cities; and 4) local volunteer engineers and contractors. In general, buildings were inspected as a result of owner and/or tenant requests, or because a drive-by survey indicated the need for an inspector to return. The Building Department maintains a list of all the unreinforced masonry buildings in San Francisco, and sent inspectors to all (approximately 2,000) of them. Because of the widespread damage in the Marina district, every building in that area was inspected. The criteria used are described in building safety evaluation manual ATC 20 (Applied Technology Council, 1989). Buildings were given either a red (extreme hazard, unsafe for occupancy or entry), yellow (dangerous, no usage on continuous basis, no entry by public) or green (inspected, no restriction on use or occupancy) tag. The red and yellow tagged buildings, as of the end of November 1989, are plotted in figure 1.

The list of red and yellow tagged structures provides us with an excellent data base of the location of damage in San Francisco. It should be noted that damage is not confined to red- and yellow-tagged buildings. Many green-tagged structures had non-structural damage, such as minor cracks in walls or

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1. City of San Francisco Department of Public Works

2. City of San Francisco Building Inspection Bureau

veneers. But the red and yellow tags reflect the most serious destruction and we consider it appropriate to use them to show the damage distribution. There were a few problems in the original files given to us by the city, such as truncated addresses (in one or two cases this seemed to be the result of an inspector entering a partial street number of a collapsed building where the address couldn't be read). The buildings at the northwest corner of Beach and Divisidero which burned down in the fire that followed the earthquake do not appear on the list. The task of editing the city files to accommodate these and other similar sorts of problems was considered to formidable for the scope of this report, so for the most part they were plotted as is. The exceptions are a few demolished buildings that can be seen on aerial photographs taken shortly after the earthquake, but which were not included in the list of red tags. Some buildings received more than one color tag. In the case of a red tag changed to a yellow tag, or vice versa, both are shown. Red or yellow tags that were upgraded to green tags are shown only as the original tag. Finally, as described below, these inspections were performed by people with a wide variability in their backgrounds.

Areal Distribution of Damage

The mapped distribution of tagged structures shown in figure 1 should be used only to indicate general areal trends, as there may be some inaccuracies for individual buildings because of the reasons discussed in the previous section. Most of the red- or yellow-tagged structures lie in a rectangle bounded by Marina Boulevard on the north, Fillmore on the east, Chestnut on the south and Baker on the west. Within this rectangle there is a large C-shaped zone where the concentration of damaged buildings is especially dense. This zone is bounded by Marina Boulevard on the north, Avila on the east. Francisco to the south and Broderick on the west.

Figure 2 shows undifferentiated red and yellow tags superimposed on a map of miscellaneous and hydraulic fills and natural ground (from figure 6, M. Bonilla, this volume). Damage within the Marina district does not appear to correlate with detailed ground conditions. It is no worse on either the miscellaneous fills or the hydraulic fill than on the natural ground. Strawberry Island, the spit of natural land in the western part of the Marina, underlies much of the C-shaped zone where the damage is especially concentrated. While the northwest part of the 1912 hydraulic fill performed predictably poorly, buildings in the north-central and northeast part of the hydraulic fill fared relatively well. The structural damage in the Marina district does not show any simple correlation between the ground surface that was originally the island, and land that was once marsh or lagoon that has since been filled in.

Conclusions

Many initial media reports about the Marina stated that the damage in the district was the result of differential settlement caused by the liquefaction of the underlying fill. A preliminary examination of the areal distribution of damage to structures suggests that this is not always the case. The lack of correspondence between structural damage and fill boundaries has already been discussed. The distribution of liquefaction, as represented by sand boils, is shown in figure 6 (Bennett, this volume). While there is some overlap between areas undergoing liquefaction and areas which suffered systematic damage, as expected, there is also considerable damage outside the zone of liquefaction. Most of the C-shaped zone containing the highest concentration of red- and yellow-tagged buildings lie to the west and southwest of the liquefaction area. The implication is that while some of the destruction in the Marina was undoubtedly due to liquefaction, or a combination of liquefaction and shaking, a large part of it was probably due to shaking alone. We note, however, that this is a very preliminary result and further study is recommended. Building construction, for example, was not considered, and must be carefully examined to make sure that it is not a controlling factor in the damage distribution. The nature of the damage does not appear in this map, and can undoubtedly provide information on the cause. But as of this writing, the damage to structures in the Marina District of San Francisco can not be attributed to liquefaction alone.

References Cited

Applied Technology Council, 1989, Procedures for post-earthquake safety evaluation of buildings: ATC-20, Redwood City, CA.

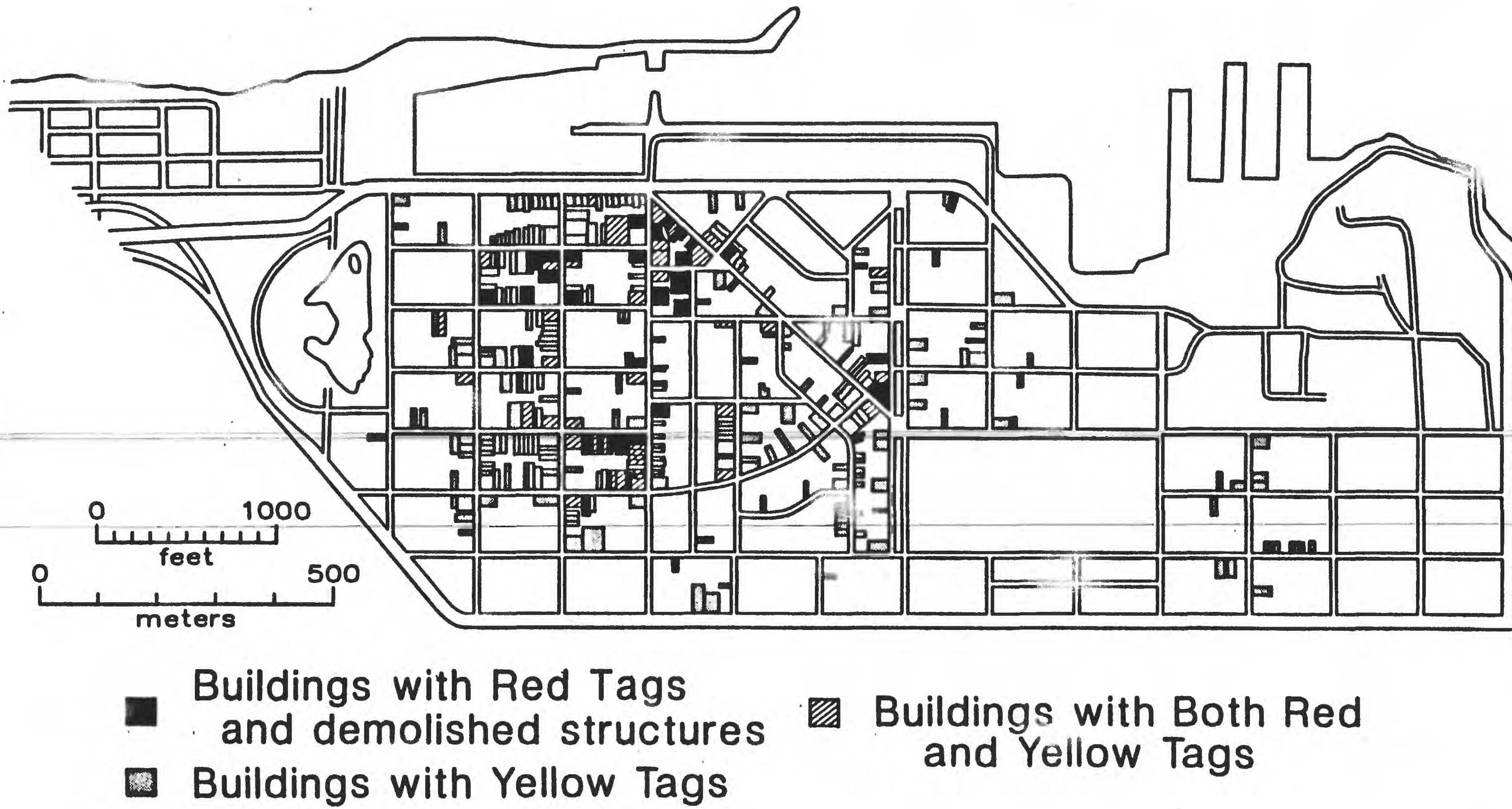

Figure 1. Buildings in the Marina District with red or yellow tags from the City of San Francisco Building Department following the October 17, 1989 Loma Prieta earthquake.

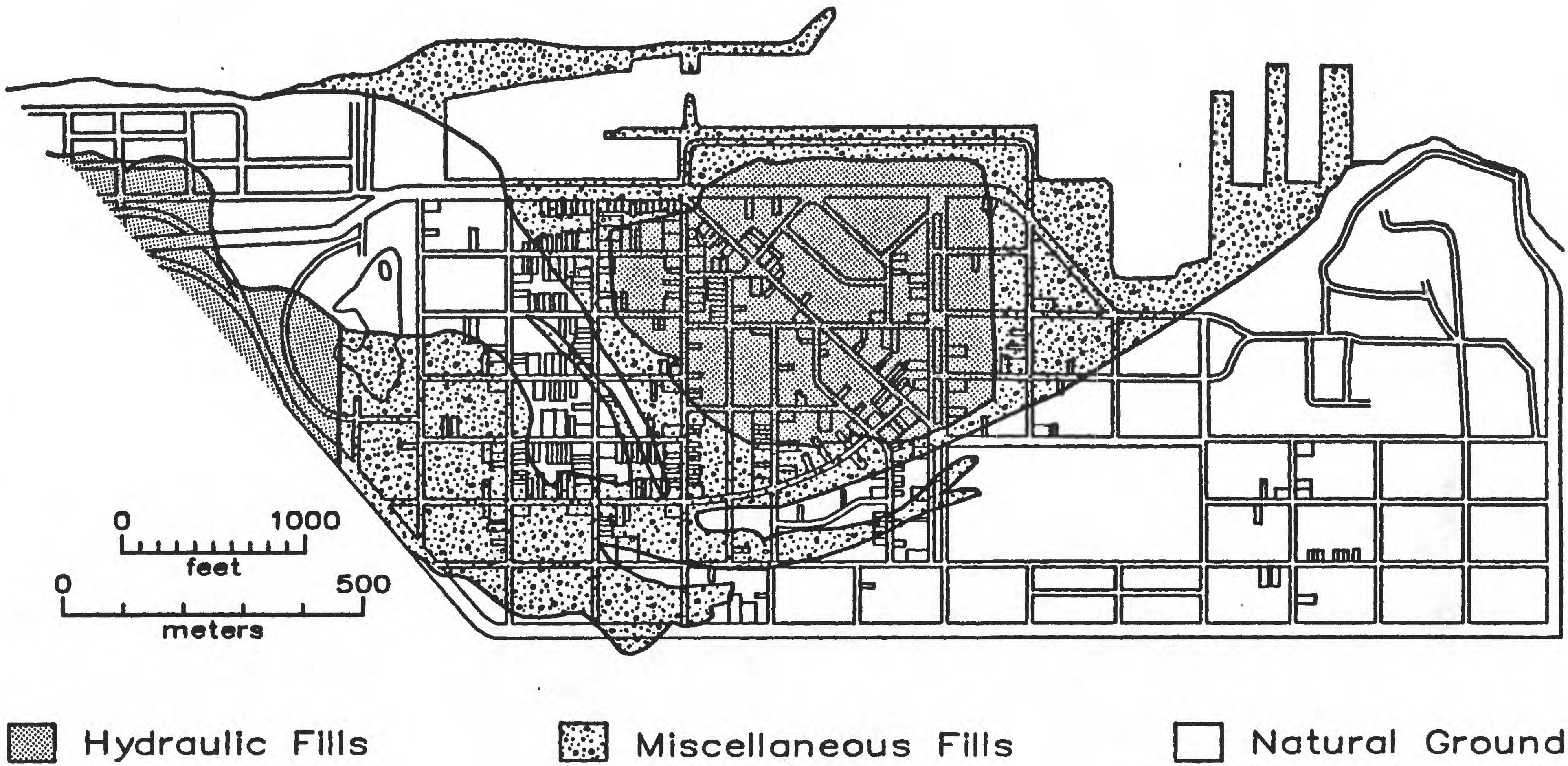

Figure 2. Ground type and red and yellow tagged structures in the Marina District.

GROUND DEFORMATION AND LIQUEFACTION OF SOIL IN THE MARINA DISTRICT

Michael J. Bennett

Introduction

A reconnaissance survey of effects from the Loma Prieta earthquake was conducted in the Marina district to assess ground deformation patterns and liquefaction effects that resulted from the 17 October 1989 Loma Prieta earthquake. The survey began on 19 October 1989 and included mapping the location of sand boils, ground cracks and other features that would indicate what effect liquefaction had on the Marina district. Between November 3, 1989 and November 10, 1989, a leveling survey was conducted to determine the vertical settlements caused by the earthquake and any post-earthquake settlement. Based on the results of the reconnaissance and leveling surveys six sites were selected for detailed subsurface study. At each site, between 25 January 1990 and 20 February 1990, a cone penetration test and standard penetration test was performed to define the soil layering and obtain soil samples for laboratory index tests. Using data from the penetration tests a liquefaction analysis was performed to determine the liquefaction resistance of the different subsurface layers. Field penetration tests indicate that the hydraulic fill has a low liquefaction resistance during earthquake shaking.

Acknowledgements: The leveling survey was led by A. Okamura; the field crew included M. Sako, A. Gartner, M. Beeson, R. Collier, O. Guracar, P. Okubo, W. Keith, and P. Bruggman. I thank Shinji Yao and Joseph Grech of the city and county of San Francisco, Department of Public Works, Bureau of Engineering, for their cooperation in providing the survey data needed to determine settlement in the Marina district. I thank Michael Lane, Bureau of Engineering, for his cooperation in providing permits to conduct geotechnical investigations in the Marina. Coyn Criley assisted in the field with penetration sounding and sampling; and conducted laboratory index tests. Thomas Fumal logged samples in the field. S. Walker shared photographic evidence of backyard sandboils. Many people provided support in the field and in conversation, I thank: Manuel Bonilla, Kenneth Lajoie, John Berrill, and Thomas O'Rourke. Robert Kayen supplied valuable review comments. Thanks is also given to all the Marina residents for their patience and cooperation.

Geotechnical characteristics

Cone Penetration Test (CPT)

The subsurface investigation was initiated with six CPT soundings in both artificial fill and unconsolidated natural deposits as shown in figure 1. The procedures and equipment used are consistent with the requirements of ASTM D3441-79 (American Society for Testing and Materials (ASTM), 1983). The CPT measures the penetration resistance at the tip and friction resistance along the side of an electric cone penetrometer, 3.4 cm in diameter. At the tip of the penetrometer a 10cm^2 60^o cone measures tip resistance (qc); behind the cone is a sleeve, with an area of 150 cm^2, that measures side friction (fs). The type of soil and its density or consistency can be interpreted from tip resistance, side friction and the ratio (Rf, percent) between qc and fs.

Standard Penetration Test (SPT)

Following completion of the CPT soundings, a hollow-stem auger boring was made within 1 m of each of the CPT soundings. Standard penetration tests were conducted in each boring to obtain samples and penetration data for a liquefaction analysis. The SPT procedure followed the guidelines outlined in ASTM D1586-67 (ASTM, 1983); modifications for use with the hollow stem auger are described in Youd and Bennett (1983). Holes five and six were drilled using a 6-in outside diameter, 2.5-in inside diameter hollow-stem auger; holes one through four were drilled with a 10-in outside and 4-in inside hollow stem auger. The Mobile "ADO standard penetration sampler" is used with sample liners (inside diameter 1.38 in). The hammer used to drive the sampler is a Mobile "In-hole sampling hammer"; the hammer weighs 140 pounds and is dropped 30 in using the Mobile "Safe-T-Driver" hoist. The drop-efficiency of the hammer is approximately 68 percent (Douglas and Strutynsky, 1984). For each test the sampler is seated 6 in into the soil, the number of hammer blows needed to advance the sampler the next 12 in is the field blow count (N).

Index Tests

Samples collected during the SPT's were examined in the field for texture, layering, and color (Munsell, 1975). Water content (ASTM, D2216-80) and grain size (ASTM, D422-63) were measured in the laboratory. Sediment was classified using the Unified Soil Classification (Howard, 1987).

Subsurface Samples

Grain size characteristics and descriptions of samples are listed in Appendix A. Median grain size (d50) is the grain size diameter at which half the sample is finer and half is coarser. Median grain size and frequency are shown in fig. 2A. Three groups are defined in this figure, the largest group, composed of hydraulic fill samples, has a median size that ranges between 0.150 - 0.200 mm; the second group, composed of dune sand, of median grain sizes range between 0.225-0.275 mm; and the third group, composed of beach sands, of median grain sizes range between 0.250-0.375 mm. The grain-size characteristics of each of the subsurface units are show in Table 1.

Table 1. Grain size and environment of subsurface units.

Geologic unit and classification	Grain size characteristics avg d_{50} (mm),	grain size, mm min and max	fines content (%) avg, min and max
Hydraulic fill (SP, SP-SM)	0.177,	0.152 to 0.197	10, 3-21
Dune deposit (SP-SM, SP)	0.249,	0.231 to 0.268	6, 3-9
Beach deposit (SP)	0.309,	0.272 to 0.361	3, 3-4

The coefficient of uniformity (Cu) is a rough measure of grain size sorting (the size range over which the sand grains occur in the sample), it is calculated from the ratio of grain size at the 60^{th} percentile to grain size at the 10^{th} percentile (d60/d10 = Cu). The three main subsurface units listed in Table 1 have similar coefficients (2) and are well sorted. Although individual samples are well sorted the beach deposit shows the widest range in median grain size. The CPT tip resistance, blow/count (N), and sediment type is shown for each of the borings in figure 3.

Samples were taken from excavations to a maximum depth of about 5 ft. Excavation samples have the smallest median grain size (0.046 mm) and the most fines of any sandy soil.

A representative size curve for each of the major subsurface units and the excavation samples is shown in figure 4. Logs for the six SPT borings are shown in Appendix B. A cross section of the Marina, using the CPT and index tests is shown in figure 5.

Geologic effects

Distribution of sand boils

Sand boils provided the most direct evidence of liquefaction in the Marina. In order to liquefy, sand must be below the water table and be loose enough for the ground to settle or densify. As the sand structure densifies during an earthquake the water between the sand grains is squeezed. If the water pressure rises to equal the weight of the overlying deposit, liquefaction occurs. Effects of liquefaction include: sand boils, lateral spreading, and settlement. Factors that influence water pressure build up include: earthquake magnitude, ground acceleration, distance from the seismic energy source, duration of shaking, grain size characteristics, and sand density.

Most of the sand boils occurred between Divisadero and Webster streets and Bay street and Marina Green (fig. 6). This area is underlain by hydraulic fill that was emplaced for the Panama-Pacific exhibition. Sand boils were found in backyards, frontyards, garages, in streets, and alongside house foundations. Because access to backyards and structures was restricted, it is possible that the map is incomplete. Nevertheless, it is believed to fairly represent the areal distribution of sand boils.

Most sand boils are fine grained gray sand, commonly containing shells. A few sand boils are fine to medium grained brown sand. One group of brown sand boils at the eastern end of the Marina Green is of special interest. This group of sand boils consists of six individual sand deposits that were erupted onto the ground surface not in classic "volcano" form but as a lateral eruption through the grass turf. Shells, mud balls, and charred wood were also erupted with the brown sand. At several of the gray sand boils on the Marina Green the sand bowed up the grassy turf up to 5 cm, and then was ejected onto the surface through "tears" in the turf. Large amounts of gray sand were erupted onto the Marina Green parking lot, some was associated with a long east-west crack parallel to the sea wall.

Although there is some overlap in grain size the brown sand boils are coarser grained than the gray sand boils (fig. 4). The brown sand ejected as sand boils is coarser and contains fewer fines than from the brown sand that immediately underlies the street to a depth of approximately 5 ft. The grain size characteristics of the sand boils are shown in Table 2.

Table 2. Grain size characteristics of sand boils

Sand boil color and classification	Grain size characteristics: grain size (mm) avg d_{50},	min and max	fines content (%) avg, min and max
Brown (SP)	0.235,	0.184-0.305	4, 1-11
Gray (SP, SP-SM)	0.168,	0.145-0.230	9, 2-18

The grain size characteristics of the gray sand boils and hydraulic fill are similar (figs. 2A & 2B); based on this similarity the gray sand boils are interpreted to have originated from the hydraulic fill. Also, the grain size characteristics of the brown sand boils and dune deposit are similar (figs. 2A & 2B); based on this similarity the brown sand boils are interpreted to have originated from dune sand that was used as fill. The dune sand often served as the source for filling in the Marina (Bonilla, this volume). Similar dune sands are reported to have been used as fill in Yerba Buena Cove (Roth and Kavazanjian, 1984).

Pavement cracks and damaged sidewalks

Cracks in streets and damaged sidewalks were a common consequence of ground shaking and ground deformation in the Marina district. Most of these effects were limited to the area bounded by Broderick and Webster, and Francisco and Marina Green (fig. 7). Cracks in the street are generally oriented north-south and east-west and are likely controlled by the street pattern. These cracks show various combinations of compression, extension, and shear that were caused by horizontal displacement and vertical settlement.

Sidewalks thrust up into tent-like forms were the result of compression. Thrusting of sidewalks was most common in the north-south direction. Curb thrusting was also common where long straight sections and curved sections of sidewalk were thrust outward over curbs with the effect that curbs appear to tip into the street. In general, the cracking pattern within the residential part of the Marina was restricted to the areas that had been filled, although some small cracks are associated with the beach deposit. No large scale cracks were found that would indicate there was one major failure zone. Most cracks indicated settlement and/or lateral movement was less than 100 mm. The most prominent exception occurred in the Winfield Scott school playground, where a series of north-south and east- west cracks displayed up to 230 mm of east-west compression and 150 mm of north-south movement. Although the cracks are oriented north and west, the system of cracks trend parallel to the contact between the 1912 hydraulic fill and the older fill and beach deposits.

Demolished and Badly Tilted Buildings

Buildings that collapsed and were demolished and buildings that are seriously tilted are shown in figure 8. Some buildings simply collapsed, while others toppled over. First stories of some buildings were tilted, but the building stayed up and was later demolished. Other buildings with tilted first stores were later righted. All of the buildings that collapsed, or caught fire and collapsed, or were so badly damaged that they were torn down occur in the areas of fill. Most of the buildings that were badly tilted occur in the same area. Of the natural deposits, only the area underlain by the beach deposit contained buildings that were seriously tilted.

Settlement

Introduction

A second order, class 1 leveling survey was conducted, by a US Geological Survey team, in the Marina to measure the vertical settlements caused by the earthquake. The location of the leveling stations is shown in figure 9. The intersection of Lombard and Laguna was assumed to be stable for computational purposes. Vertical settlement is the difference in elevation from a survey conducted in 1974 and the present survey. For the purpose of this report the settlement that occurred during 1974-1989 is referred to as "post-earthquake" settlement, the settlement that occurred during this time interval is not solely a result of the Loma Prieta earthquake. Settlement may be caused by consolidation of the fill and/or secondary compression of the bay mud.

Methods

The following is the written communication of Arnold Okamura, survey chief, describing the methods used to evaluate elevation and misclosure: "Collimation of the level instrument was checked daily before each leveling session. Backward and forward sight lengths were balanced to within 1 m per setup and per section. The maximum sighting length was 50 m with an average length of 30 m. Wild turning plates were used as turning points. The bottom 0.5 m of the rod was not read, and the leveling procedure was double-simultaneous. On the first day, the error was 1.7 mm for the 2.7 km closed rectangular loop. Subsequent levellings created many loops, and these were inverted to derive the standard error of the entire survey (1.512 mm times the square root of the distance leveled). This observed error was used in reducing the data from the second run on Divisadero Street. Because of the nondescript measuring points of many of the momuments, reoccupation of the same point as earlier surveys was uncertain, especially with the wider base of the rods that we employed" (written communication, Arnold Okamura, 1989).

District wide settlement

The change in elevation between 1974 and November 1989 was compared with elevation change between 1961 and 1974 (fig. 10) to evaluate settlement associated with Loma Prieta earthquake. However, not all of the points that were surveyed in 1989 and 1974 were surveyed in 1961. For example, the monument at the SE corner of Lombard and Broderick did not exist in 1961; instead another point at the SE corner that was measured in 1961 and 1974 was used. It is assumed that different points on the same intersection would have similar settlements with time. Three points on two different corners (#55) between 1961 and 1974 have elevation changes that range from -2 mm to +1 mm. The changes in elevation between 1961 and 1989 are listed in Appendix C.

Magnitudes of settlement within the district correlate well with the geologic units (fig. 11). The average settlement for the different deposits before and after 1974 is shown in figure 11. The ratio between settlement in 1974-1989 and 1961-1974 is shown in figure 12 and Table 3. The natural deposits (dune, beach, and older alluvium) typically settled between 3 and 7 mm prior to 1974. After 1974 the range in average settlement of the natural deposits was 2 to 18 mm. In terms of settlement the fill is divided into three groups; a central fill (1869-1895), a southwestern fill (1895-1906), and the post-1906 Marina Cove fill. Except for the dune and older alluvial deposits all units showed greater settlement in the period 1974-1989 than the period 1961-1974. The difference as shown by the ratio of post 1974 settlement to pre 1974 settlement indicates that the increase of settlement was not uniform. The post-1906 fill showed almost 9 times more settlement after 1974 than before 1974 whereas the western fill showed less than twice as much settlement. Profiles along Divisadero (fig. 13) and Beach Streets (fig. 14) show that different magnitudes of settlement are associated with different geologic units. The hydraulic fill of the Marina cove is associated with the greatest magnitude of settlement.

Two leveling surveys along Divisadero were made one week apart (November, 10-17). Holding the Lombard station as constant, settlement increases uniformly northward, at Marina Blvd the one week difference is 4 mm (fig. 15). Although the changes are small they are not random (written communication, A. Okamura, 1989).

Table 3. Relation between settlement before and after 1974.
* The numbers in column 6 were calculated before rounding off the numbers in columns 3 and 5.

Environment	1961-1974 Settlement		1974-1989 Settlement		(5)/(3)
	mm	mm/yr	mm	mm/yr	
(1)	(2)	(3)	(4)	(5)	(6)*
Older Alluvial	4.8	0.4	2.7	0.2	0.5
Dune	3.0	0.2	1.5	0.1	0.4
Beach	6.5	0.5	18.4	1.2	2.5
Southwestern Fill	7.3	0.6	14.8	1.0	1.8
Central Fill	17.0	1.3	64.8	4.3	3.3
Post 1906 Fill	9.7	0.7	95.9	6.4	8.6

Simply comparing the total settlement between the two surveys presents a problem. First, different time spans are involved, 13 years versus 15 years. This problem is resolved by comparing the annual rate of settlement (settlement mm/number of years) between the two surveys. Second, and more complex, is how much of the settlement can be attributed directly to the earthquake? One can assume that the settlement that occurred during 1961-1974 also occurred during 1974-1989. Thus, the settlement created by the earthquake would be equal to the 1974-1989 settlement minus the 1961-1974 settlement. Because of the many uncertainties the uncorrected settlement between 1974-1989 is used to describe the "post-earthquake" settlement.

The precise leveling survey in the Marina measured settlement over a relatively large area, some of the areas that experienced over 100 mm of settlement did not outwardly display evidence of settlement, others dramatically displayed evidence of settlement. The area near Beach St. and Fillmore experienced over 100 mm of settlement, but displayed relatively less observable damage than the area near Prado and Avila that also experienced more than 100 mm of settlement. A map showing the relation between settlement and damage, as measured by the red/yellow tag damage survey (Seekins, Lew and Kornfield, this volume), is shown in figure 16. It can be seen that 73 percent of the red tag buildings are located in the areas that have been filled, and except for the red-tag buildings on the beach deposit, ninety one percent of the red-tag buildings are located where settlement was at least 25 mm. Also, when the figure 16 is compared to the location of sand boils (fig. 6) it can be seen that sand boils generally occur where there has been at least 50 mm of settlement.

Local evidence of settlement

Some of the largest and best defined evidence of local settlement occurred along Marina Blvd. (fig. 8). For example, on Marina Blvd. between Scott and Broderick, settlement between 20 and 150 mm occurred along the front and sides of some houses. Effects associated with the Marina Blvd settlement include sand boils, buckled rain spouts on the front of houses, cracked and rotated driveway pavement and an inability to open garage doors. Between 40 and 50 mm of local differential settlement also occurred on Webster between Jefferson and North Point. Most of the settlement occurs at the joint between the uplifted sidewalk concrete and the downdropped driveway concrete (residence). At one residence on the same block a drain pipe connecting the house to a sub-sidewalk drain was buckled when the residence settled. The west curb along Webster (near Jefferson) settled 100 mm and is level with the street. A complex pattern of settlement and lateral movement occurs on North Point between Fillmore and Webster. Some of the north-south and east-west cracks define a zone that has settled at least 75 mm and moved northward (50 mm (?)). Directly north of this settlement area a gray sand boil was found in a back yard. On the street, brown sand was likely associated with a water line breakage.

Local differential settlement was also seen at engineered works where a contrast exists between improved ground associated with the engineered works and the unimproved ground. For example, differential settlement of approximately 150 mm occurred along a 2.4 m diameter storm drain outfall at the Marina seawall. Where the sea wall and outfall intersect there is also approximately 40 mm of northward separation between the wall and the sidewalk. The grassy area of the Marina Green has settled differentially over the outfall approximately 60 mm, the west side of the Marina Green moved down relative to the Marina Green on the east side of the outfall. Sand boils occur parallel to and on the westside of the outfall. Where the south curb of Marina Boulevard intersects the underlying outfall, the curb shows 70 mm of settlement on the west side. The outfall passes underneath the eastern side of the house immediately south of the disrupted curb. The house now tilts 1-2.5 degrees to the west owing to differential settlement, that is, the portion of the house not on the outfall settled more than the portion on the outfall. The outfall also passes beneath the adjacent walls of two houses on Cervantes. Both houses show settlement up to 70 mm on the ends of the houses farthest from the outfall. The outfall passes underneath a garage on Beach St. causing differential settlement. This section of Beach Street settled an average of 5 mm between 1961 and 1974; between 1974 and 1989 the average settlement was 108 mm, taking into account the average settlement before 1974, this section of Beach settled approximately 100 mm.

Other forms of local differential settlement are shown by: building supports in garages breaking through the concrete floor and settling 120 mm, and sewer structures displaying 75 mm differential settlement at Jefferson and Broderick. Near the intersection of Divisadero and Jefferson one building was found to have settled approximately 250 mm to the north. Associated with this settlement was a badly tilted building on the corner and sand boil deposits. On the other hand the area identified as SW fill experienced few damaging ground effects and settlement overall was low. This area contains no hydraulic fill and sand below the surface is medium dense to dense.

Liquefaction Analysis

The liquefaction resistance of the artificial fill and unconsolidated natural deposits was determined using the simplified procedure (Seed and others, 1983; Seed and others, 1985). The procedure is based on the empirical relation between corrected blow counts from the SPT and the average cyclic shear stress ratio induced in the soil by the earthquake. The blow count (N) is corrected, $(N_1)_{60}$, to a standard overburden pressure of one ton per square foot and a hammer efficiency of 60 percent. The average induced cyclic stress ratio (CSR) is a function of the soil density, soil depth, elevation of the water table, and peak earthquake acceleration. The equation for CSR is given by:

$$CSR = 0.65(a/g) * \frac{\text{total stress}}{\text{effective stress}} * (r_d) \qquad (1)$$

where;
a = maximum acceleration,
g = acceleration due to gravity,
total stress = total weight of overlying soil and water,
effective stress = initial vertical effective stress,
r_d = stress reduction factor that varies from 1 at the ground surface to approximately 0.9 at 30 ft.

Two ground accelerations that are believed to be upper and lower bounds for the mainshock were used in the analysis, a minimum acceleration of 0.16 g and a maximum acceleration of 0.32 g. The minimum acceleration was measured at Treasure Island, a site that experienced liquefaction, has a similar artificial fill, and is approximately the same distance from the seismic source. The maximum acceleration is based on correlations of aftershock recordings in the Marina district and with main shock recordings elsewhere in San Francisco (Boatwright, Seekins, and Mueller, this volume). Other variables needed to compute cyclic stress ratio include depth to water table; depth to water table for the USGS borings are listed in Appendix B, depth to water table for tests along Marina Blvd. is approximately 8 ft. For tests conducted in the eastern part of the Marina in natural deposits the approximate water table depth is 13.5 ft. The assumed density of the hydraulic fill is 120 lbs/ft^3 (likely range for density is 110 to 120 lbs/ft^3), the assumed density of the natural deposits is 130 lbs/ft^3.

Grain size measurements from more than 30 sandboils and subsurface samples indicate the fines content of the fill and natural deposits ranges from 3 to 10 percent.

A liquefaction analysis of the fill and the natural deposits was conducted using SPT data collected by the USGS and also by Dames and Moore (Dames and Moore, 1976; Dames and Moore, 1977). The results of the analysis generally agree, data from the USGS and Dames and Moore are shown in figure 17, data from the USGS is shown in figure 18. The minimum acceleration, 0.16g, is sufficient to liquefy most of the fill, at 0.32 g all of the tests in the fill indicate liquefaction would occur. At the minimum acceleration, only 2 tests in the natural deposits indicate the sediment would liquefy, most of the tests indicate a high resistance to liquefaction. At the maximum acceleration, approximately 25 percent of the tests in the natural deposits indicate the soil has a low resistance to liquefaction.

Conclusions

Geotechnical Investigation

1. The three main subsurface units in the upper 16 m are; 1) hydraulic fill, very low to low penetration resistance and irregular bedding; 2) dune sand, very dense; and 3) beach sand, very dense.

2. Only a "hard pan" layer at approximately 11.5 m was found in all borings.

3. The Bay mud deposit along the Beach St profile line is very loose sandy silt and silty sand.

4. The three main units are fine grained well sorted (poorly graded) sand with less than 10 percent fines. The range in median grain size between the three main units is distinctly different.

Geologic Effects

5. Sand boils were found primarily in the area underlain by the 1912 hydraulic fill, and where settlement since 1974 has been at least 50 mm.

6. The two sand boil types are characterized by: fine gray sand similar in grain size characteristics to the hydraulic fill and coarser brown sand similar in size characteristics to the dune sand.

7. Sand boils were found on the Marina Green, in front yards, back yards, in garages, and next to foundations.

8. Cracks are generally oriented north-south and east-west with the street pattern, no coherent cracking pattern was found; cracks show shear, extension, and compression.

9. Some of the most dramatic cracking is associated with engineered underground features that experienced little settlement relative to the surrounding soil.

10. Settlement measured after 1974 was up to an order of magnitude greater than before 1974. Most of the settlement after 1974 can be attributed to the earthquake.

11. The Marina Cove area showed the most settlement, average settlement between 1974 and 1989 is about 96 mm, about 8.6 times more than before 1974.

12. The most visible signs of settlement on Marina Blvd. and Webster Street are not in the 1912 hydraulic fill; some of the worst damage (Beach and Divisadero, and Jefferson and Divisadero) is not in the hydraulic fill.

13. The dune deposit and the older alluvial deposits showed the least settlement.

14. Settlement continued after the earthquake, in a one week period in November settlement along Divisadero ranged from 0 mm on Lombard to 4 mm on Marina Blvd.

15. Over 79 percent of the buildings that were red tagged are located in areas that were filled. Excepting red tagged buildings located on the beach deposit, over 90 percent of the buildings that were red tagged are located in areas that experienced at least 25 mm of settlement between 1974 and 1989.

Liquefaction Analysis

15. At a lower bound acceleration of 0.16 g most of the hydraulic fill has a very low resistance to liquefaction, whereas the natural deposits have a moderate to very high resistance; at 0.32 g all of the fill has a very low resistance to liquefaction and approximately 25 percent of the natural deposits have a low resistance to liquefaction.

16. The hydraulic fill placed in Marina Cove has a very low resistance to liquefaction at any of the likely accelerations that were felt in the Marina district.

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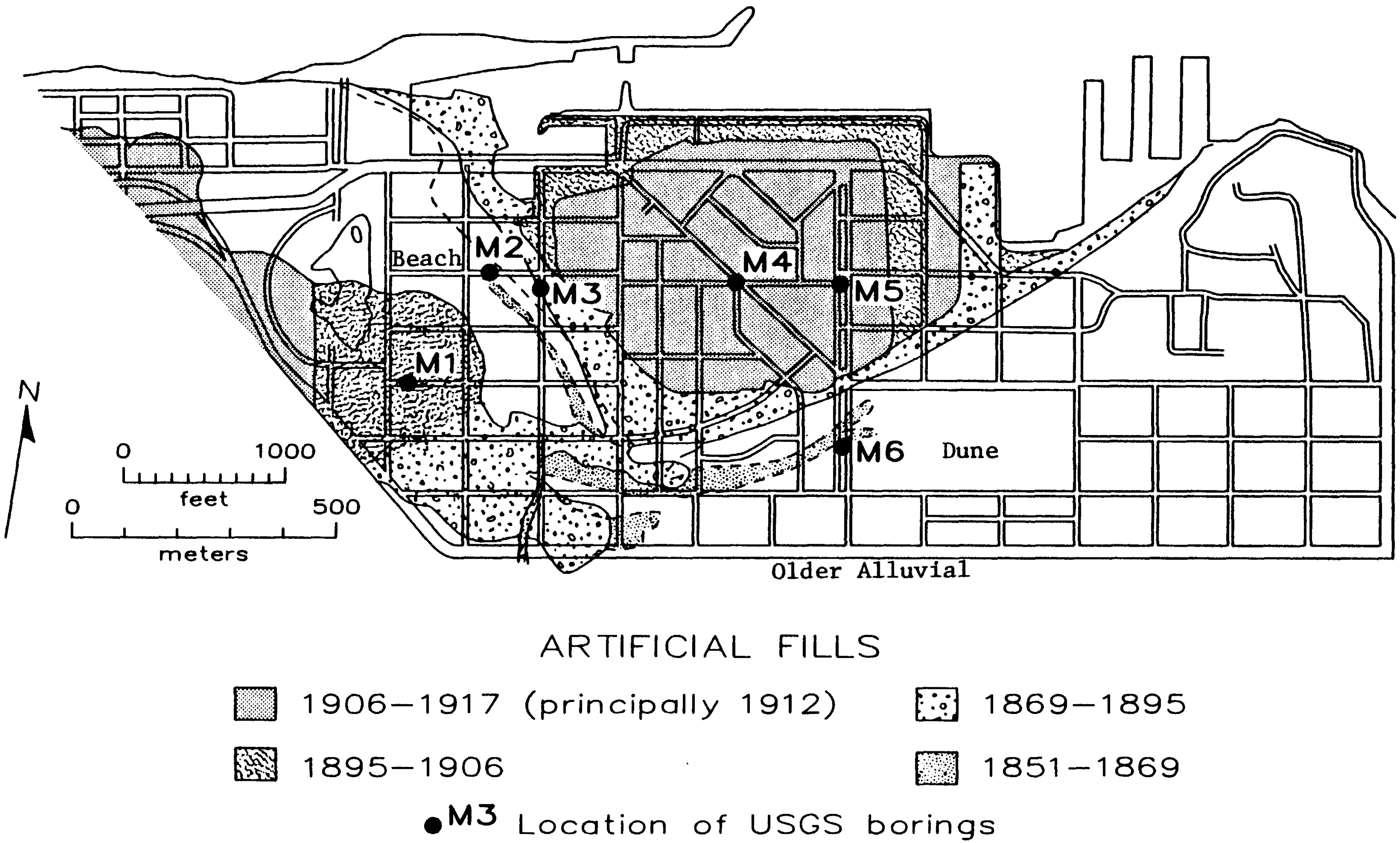
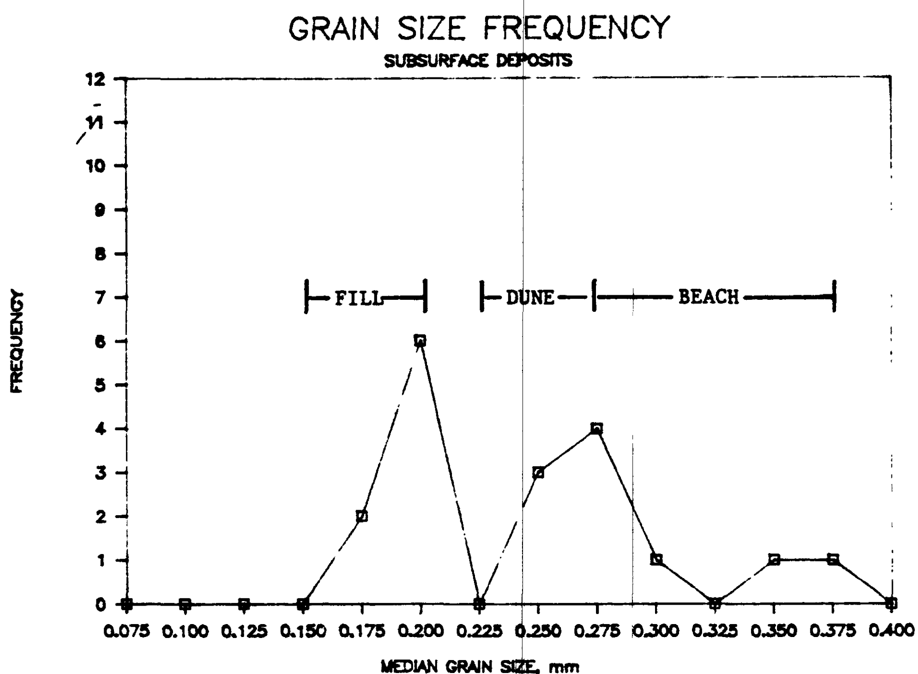
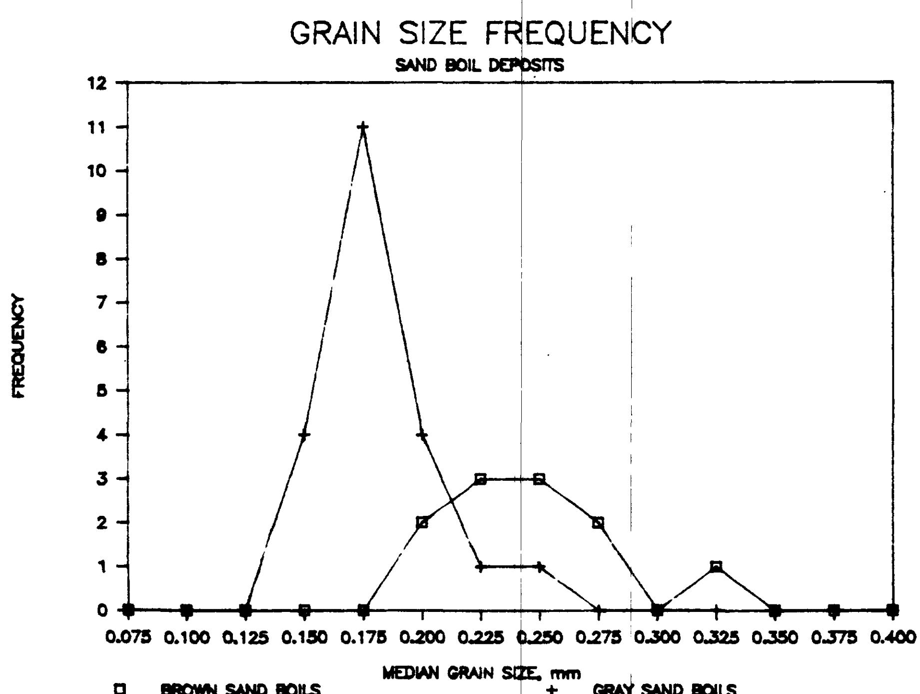


Figure 1. Map shows the general locations of the geologic units (the older alluvial deposits are generally south of Lombard St) and the location of the different fills used to reclaim the Marina district. The central fill area is referred to as Marina Cove.

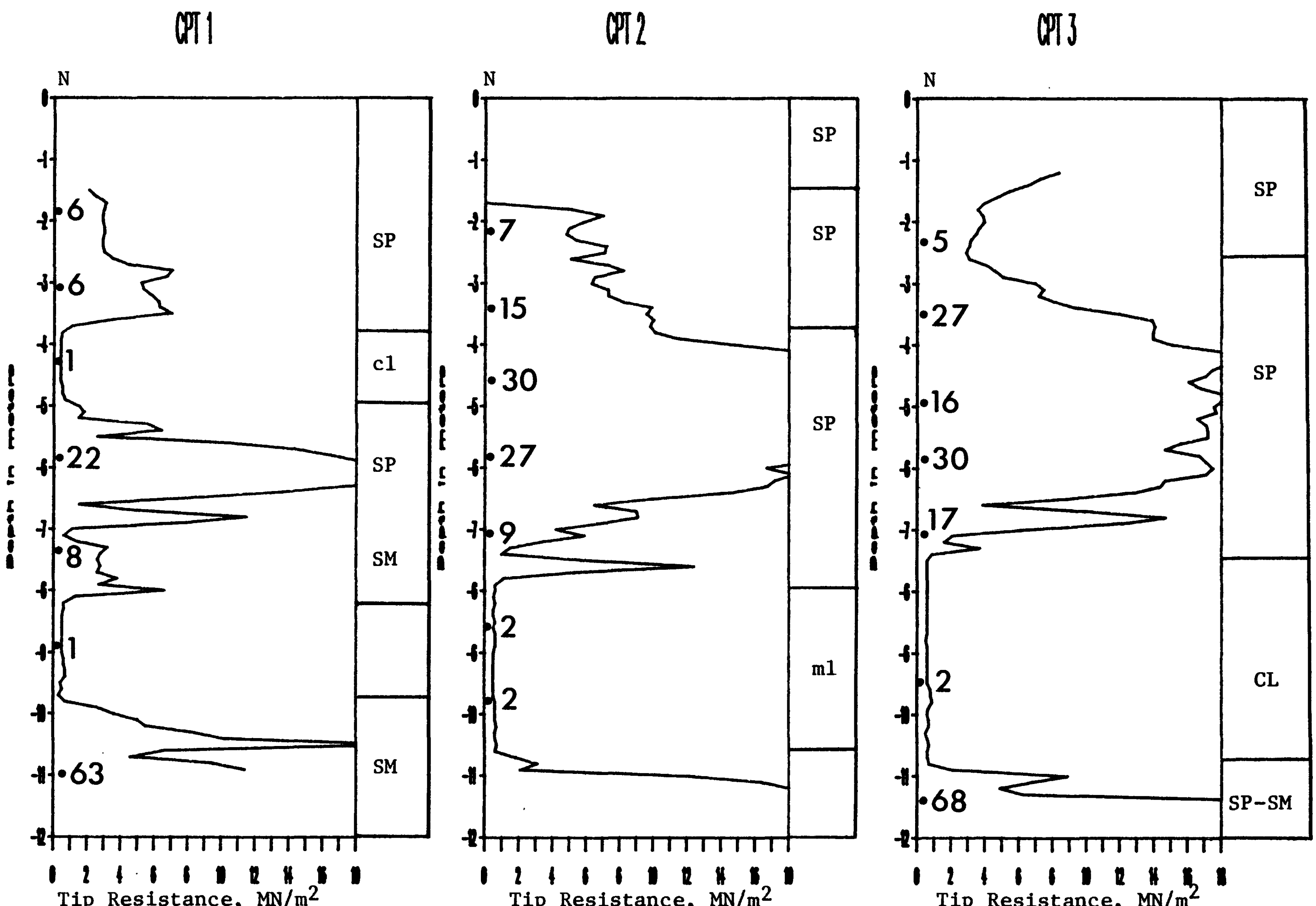


2A.



2B.

Figure 2A. Grain size frequency of subsurface units. In 2A the peak at 0.200 mm indicates that 6 samples have median grain sizes between 0.175 mm and 0.200 mm. The median grain sizes of the different subsurface units generally do not overlap. 2B. Grain size frequency of sand boils. Gray sand boils have a finer median grain size than the brown sand boils.



3A.

Figure 3A and 3B. Soil profiles defined by the cone penetration test. Blow counts (N) are uncorrected. Lower case classification symbols indicate uncertainty in the designation. Depth is in meters. Note change in vertical scale for CPT 5.

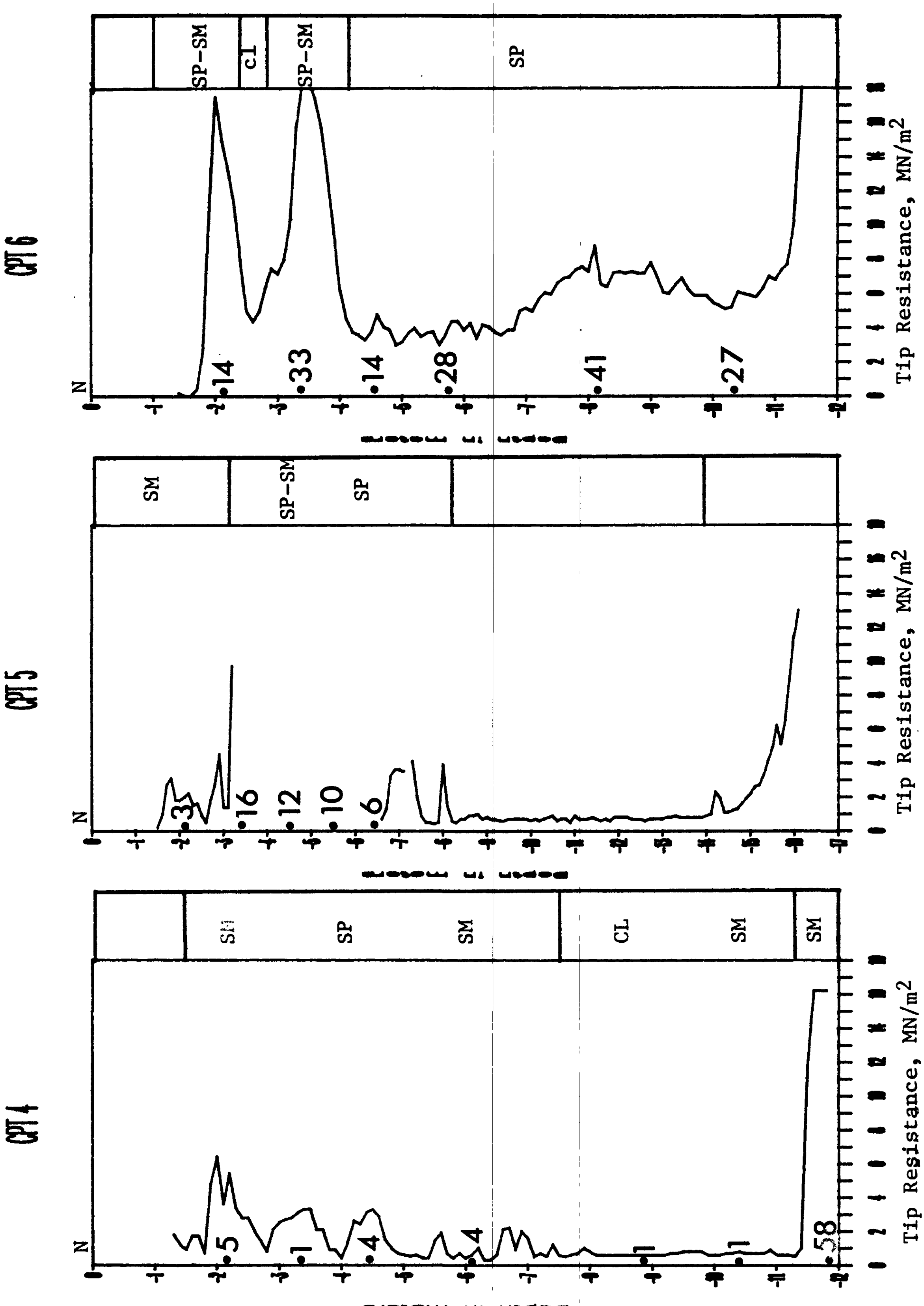

CPT 4
CPT 5
CPT 6
SM
SP
SM
CL
SM
SM
SP-SM
SP
SP-SM
cL
SP-SM
SP
N
Tip Resistance, MN/m2
•5
•1
•4
•4
•58
•3
•16
•12
•10
•6
•14
•33
•14
•28
•41
•27

Figure 3B

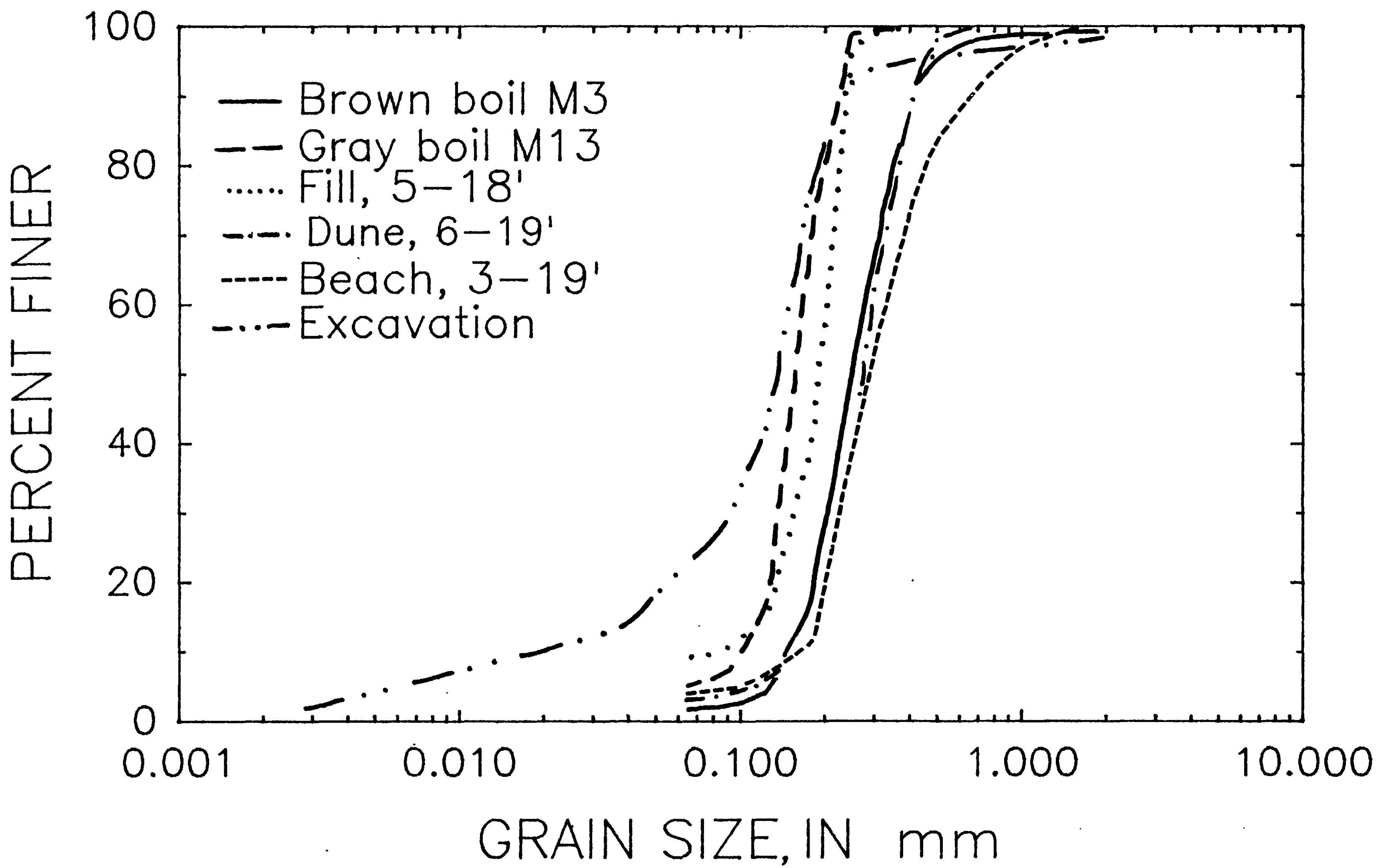
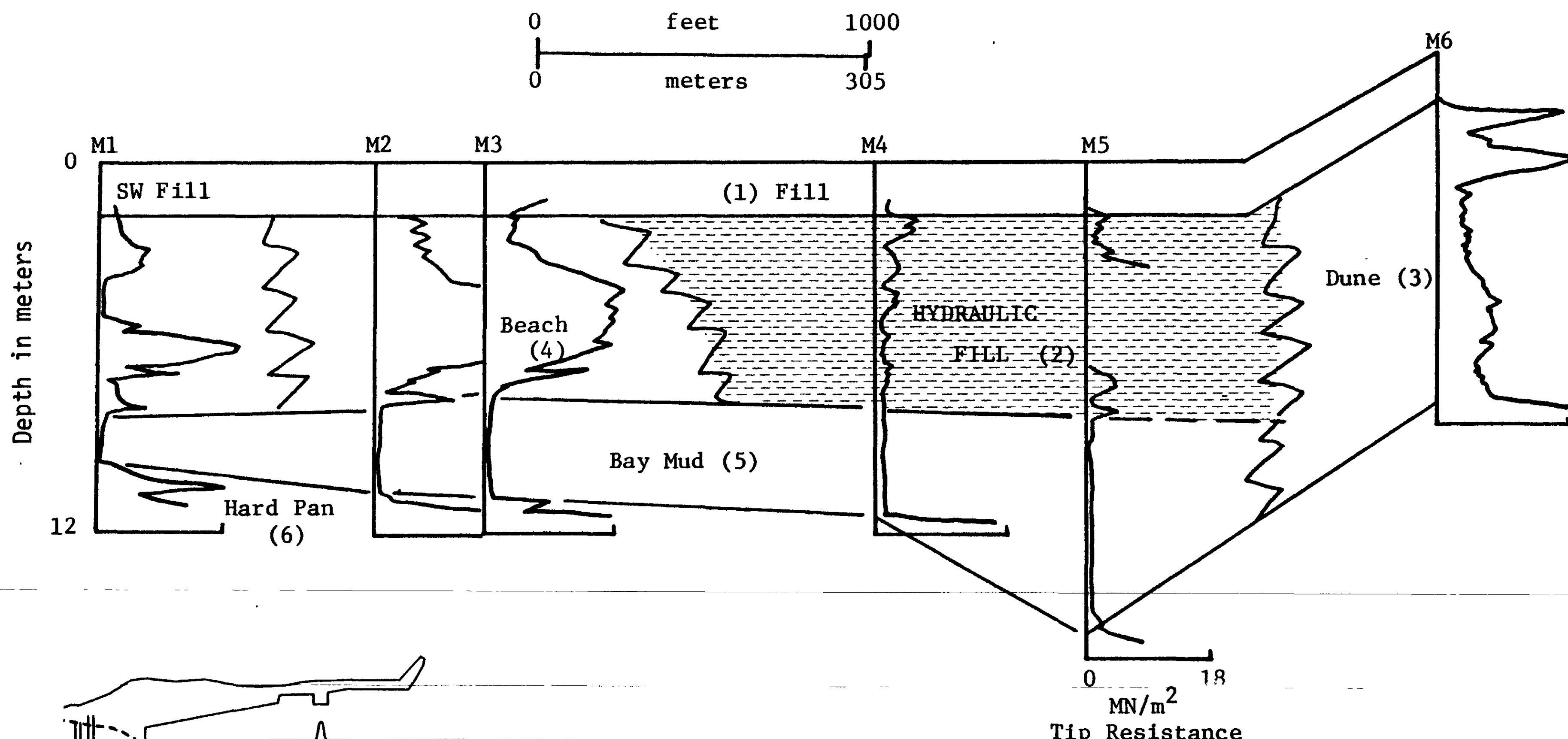
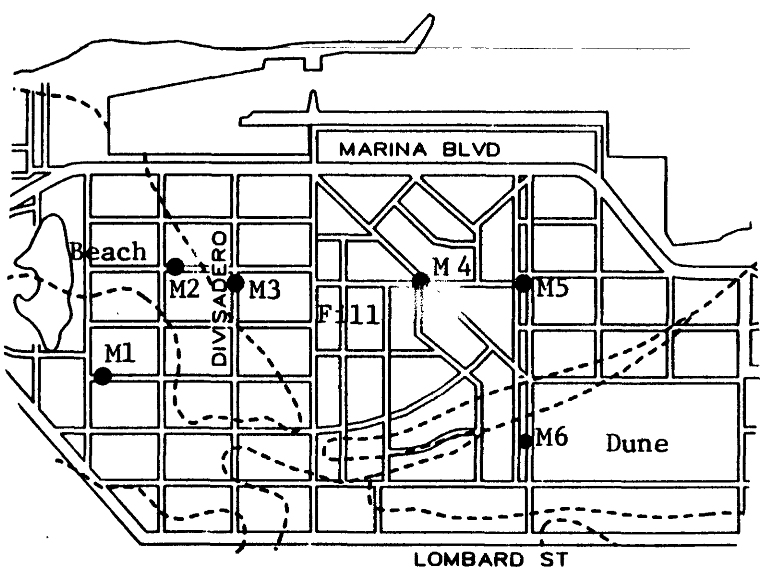


Figure 4. Grain size curves for subsurface and sand boil samples.

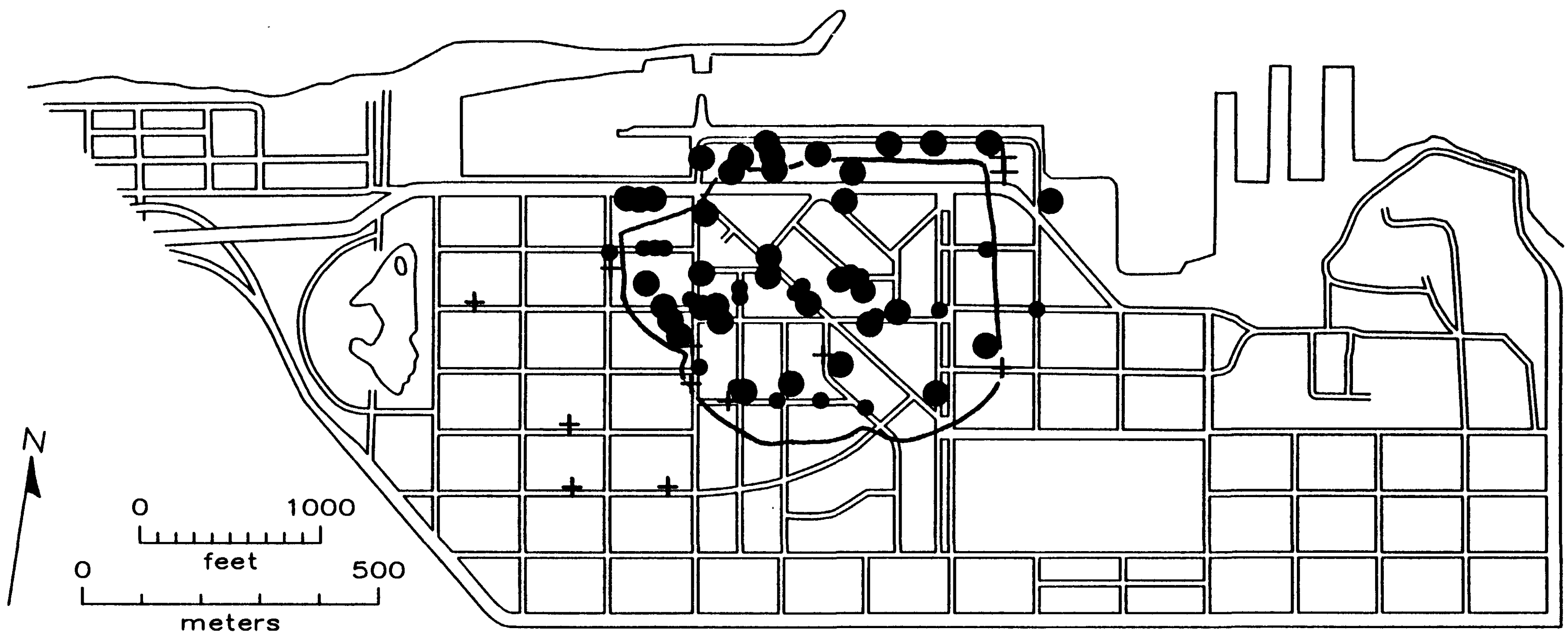




Fill	(1)	loose brown sand (SP)
Hydraulic Fill	(2)	loose gray sand (SP) and silty sand (SM)
Dune	(3)	dense brown sand (SP-SM)
Beach	(4)	dense dark gray sand (SP
Bay Mud	(5)	loose greenish gray to black sandy silt (ML) and silty sand (SM)
Hard Pan	(6)	very dense olive gray sand (SM)

Figure 5. Generalized cross section based on the cone penetration tests. The profile line is not straight. Note the contrast in tip resistance between the hydraulic fill (unit 2), beach deposit (unit 4), and dune deposit (unit 3).

LOCATION OF SAND BOILS



EXPLANATION

+ Brown sand boils, definite

+ Brown sand boils, probably from pipe breakage

● Gray sand boils, definite

• Gray sand boils, probably from pipe breakage

Figure 6. Location of sand boils. Sand boils are found primarily in the area

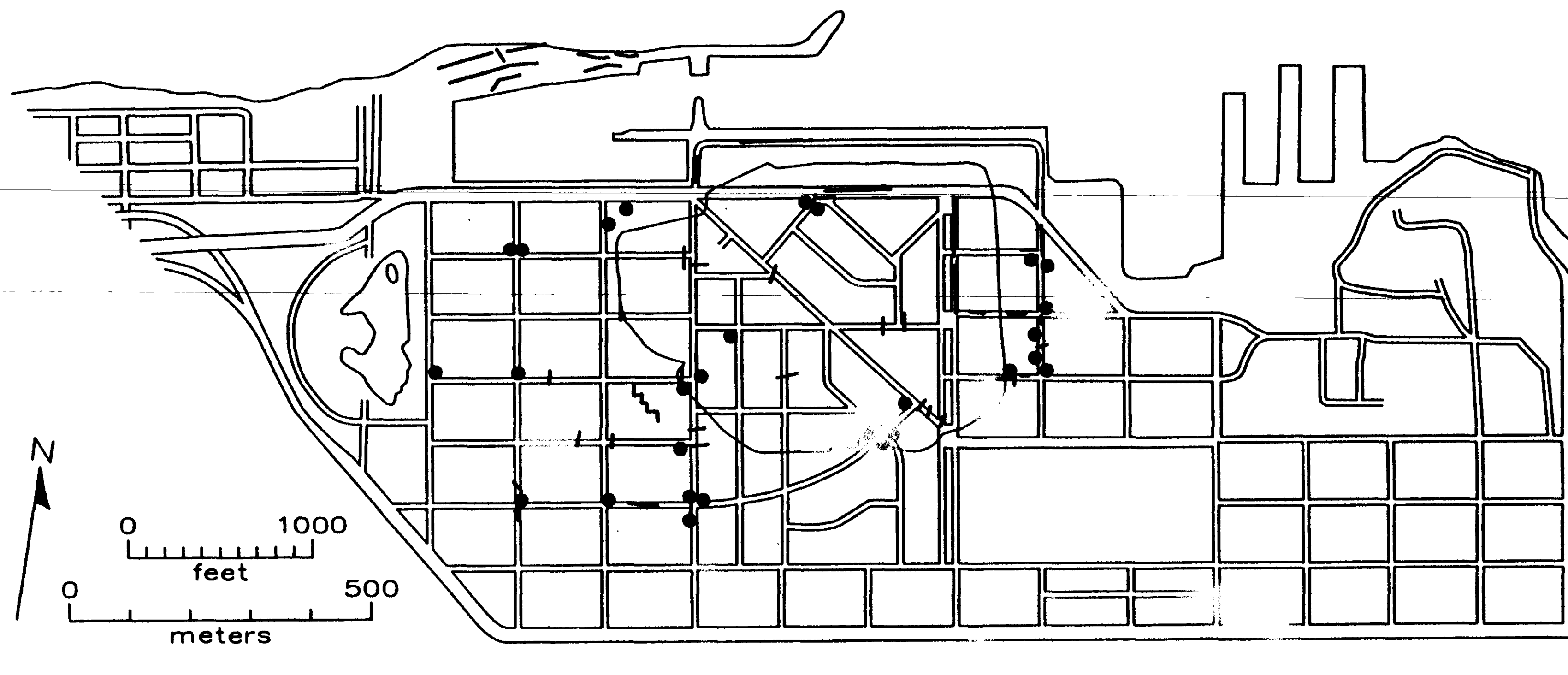


Figure 7. Distribution of cracks and curb damage.

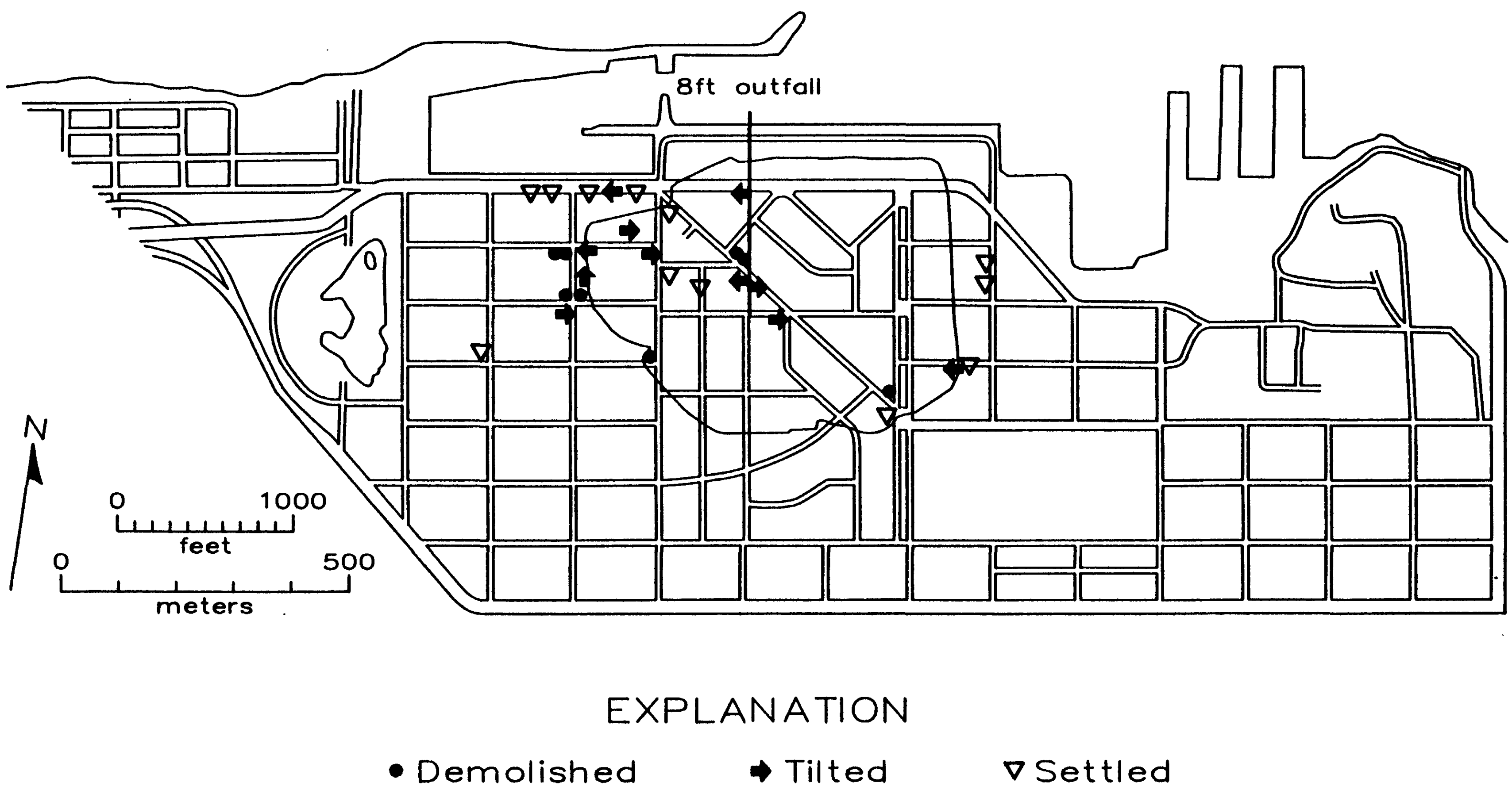


Figure 8. Buildings that were demolished, badly tilted, or displaying large settlements are generally confined to the areas that were filled.

LOCATION OF SURVEY STATIONS

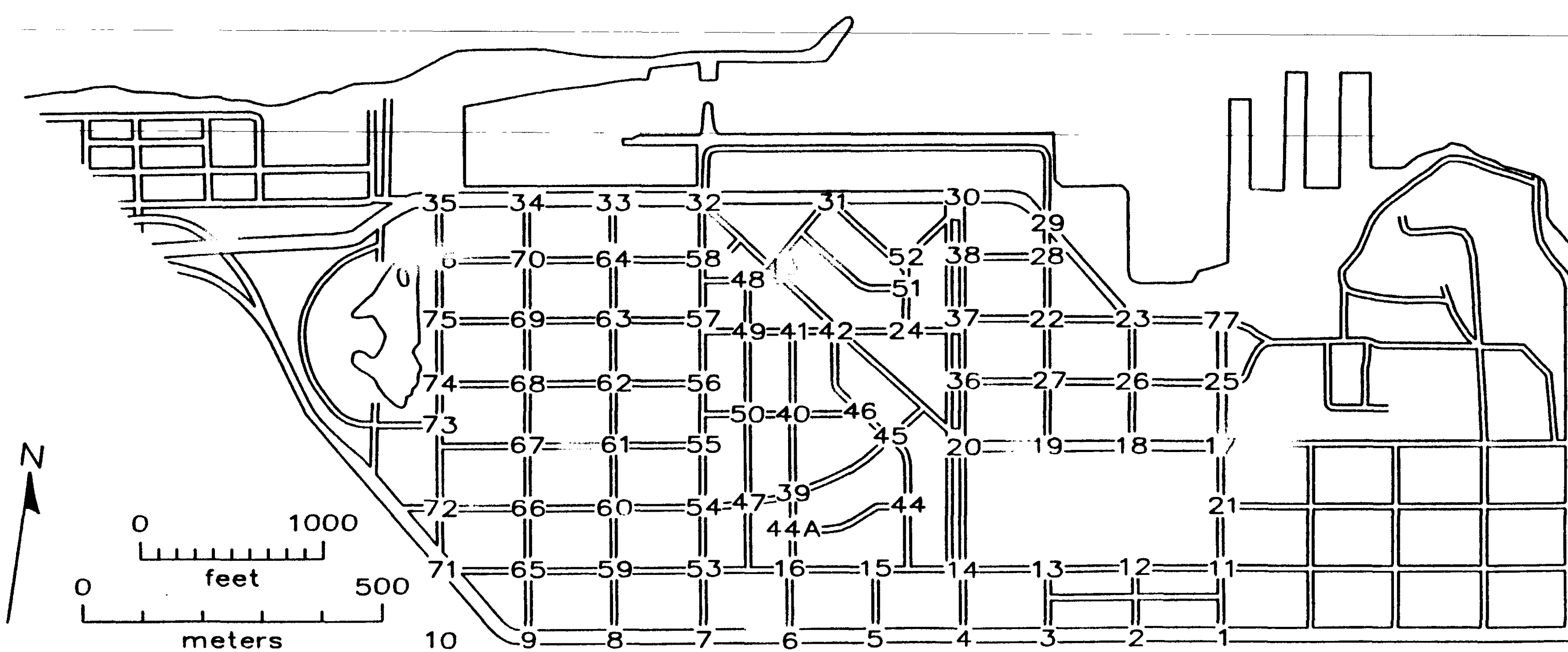


Figure 9. Location of survey stations. A description of settlement at the stations is given in Appendix C.

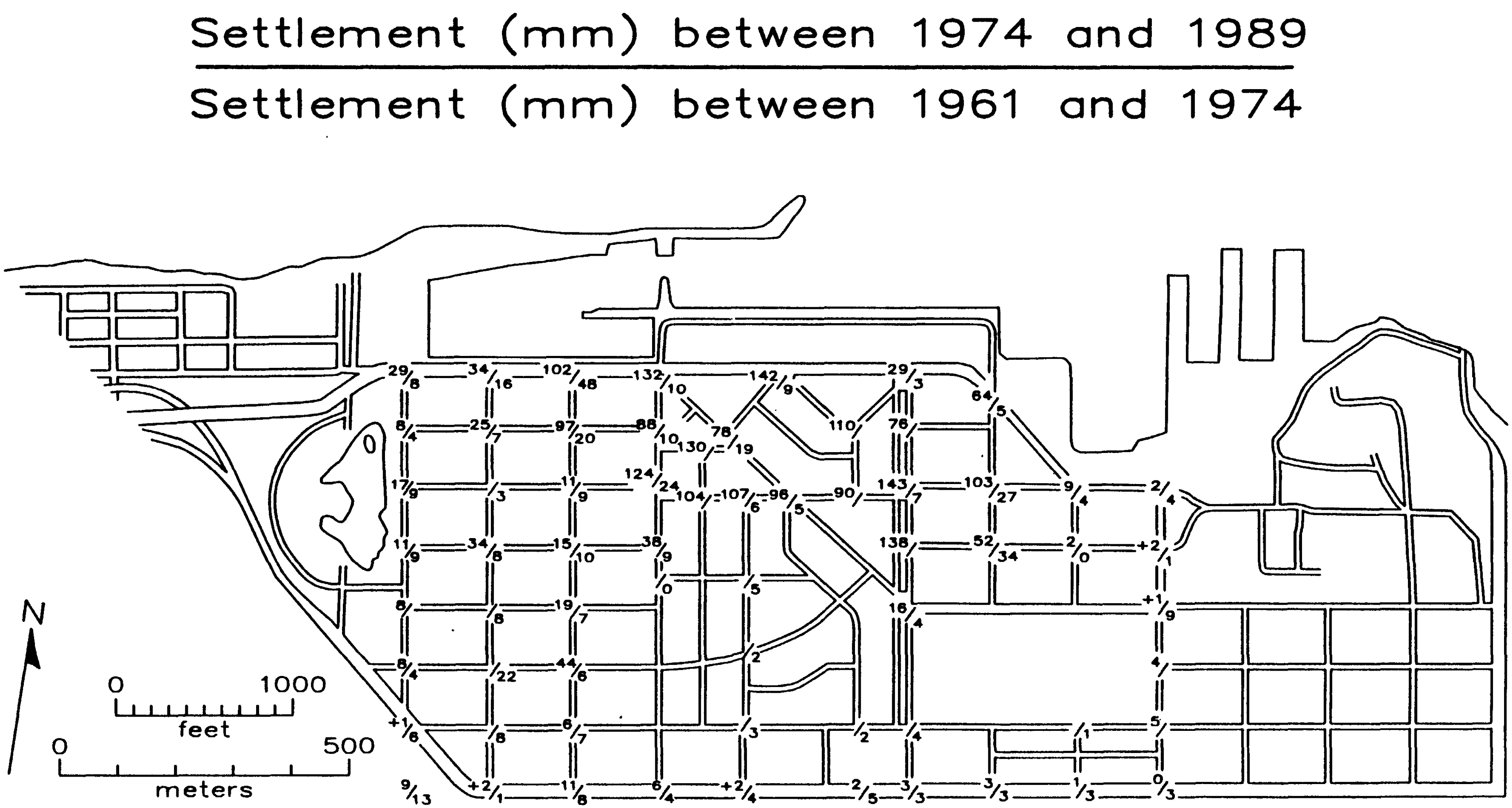


Figure 10. Settlement between the years 1961-1974 and 1974-1989.

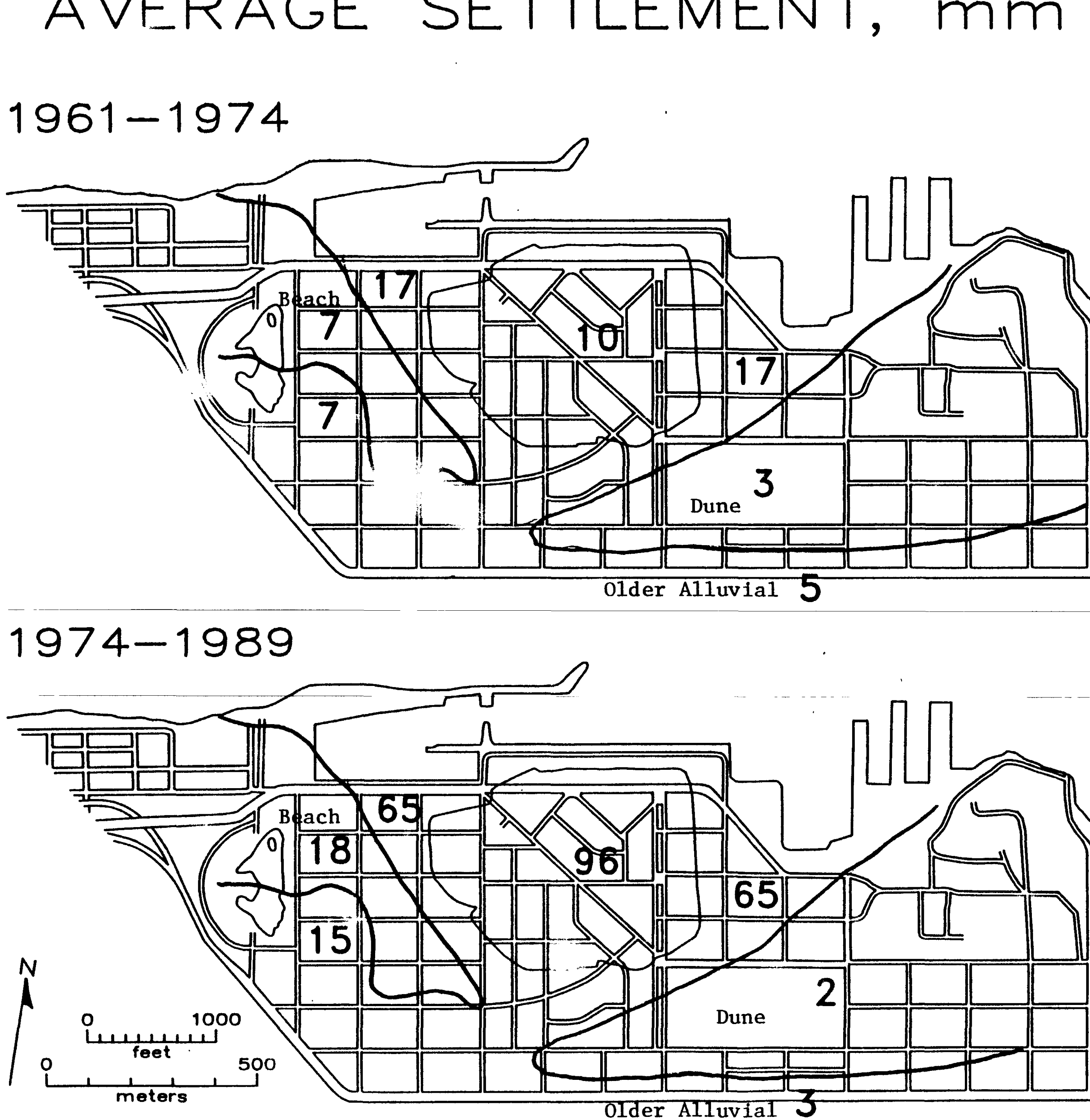


Figure 11. Average Settlement between the years 1961-1974 and 1974-1989.

RATIO $\frac{\text{YEARLY SETTLEMENT 1974-1989}}{\text{YEARLY SETTLEMENT 1961-1974}}$

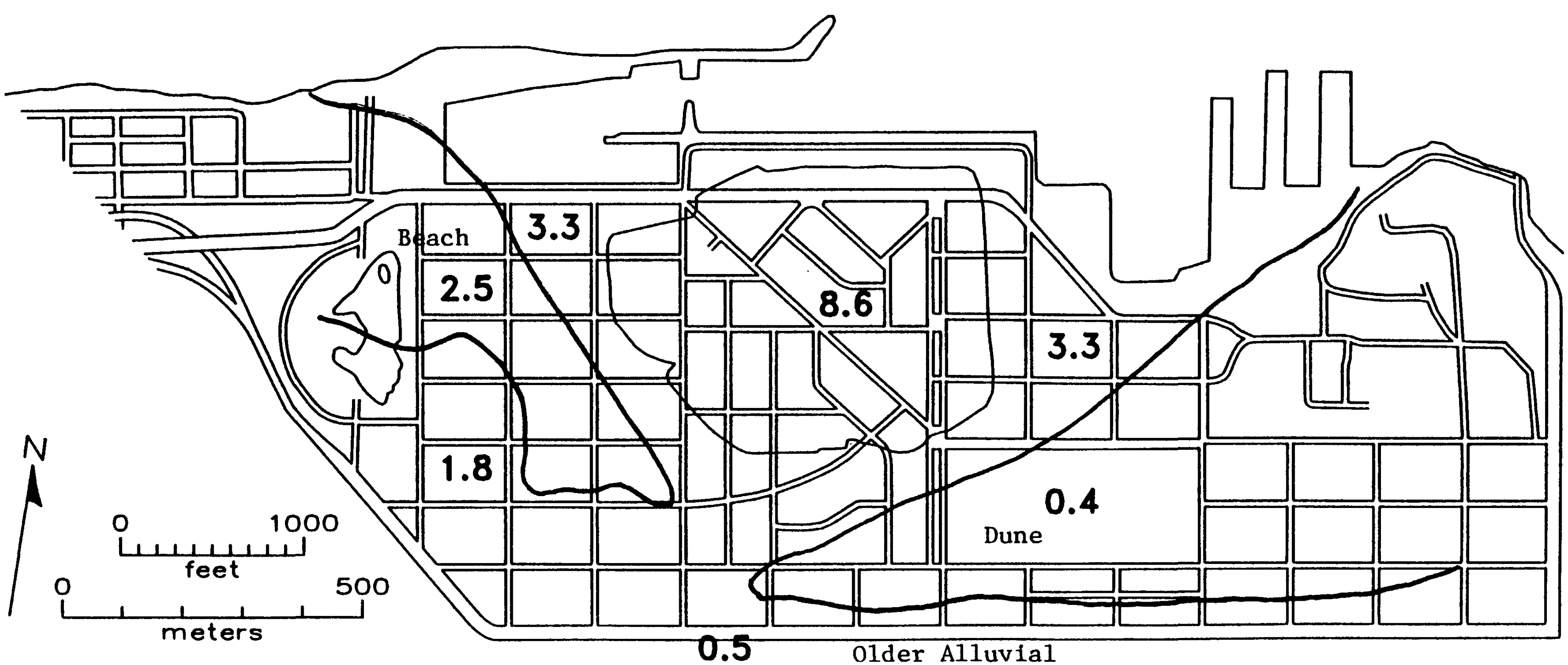


Figure 12. Ratio of 1974-1989 settlement to 1961-1974 settlement. Ratios less than one indicate more settlement took place before 1974 than after 1974.

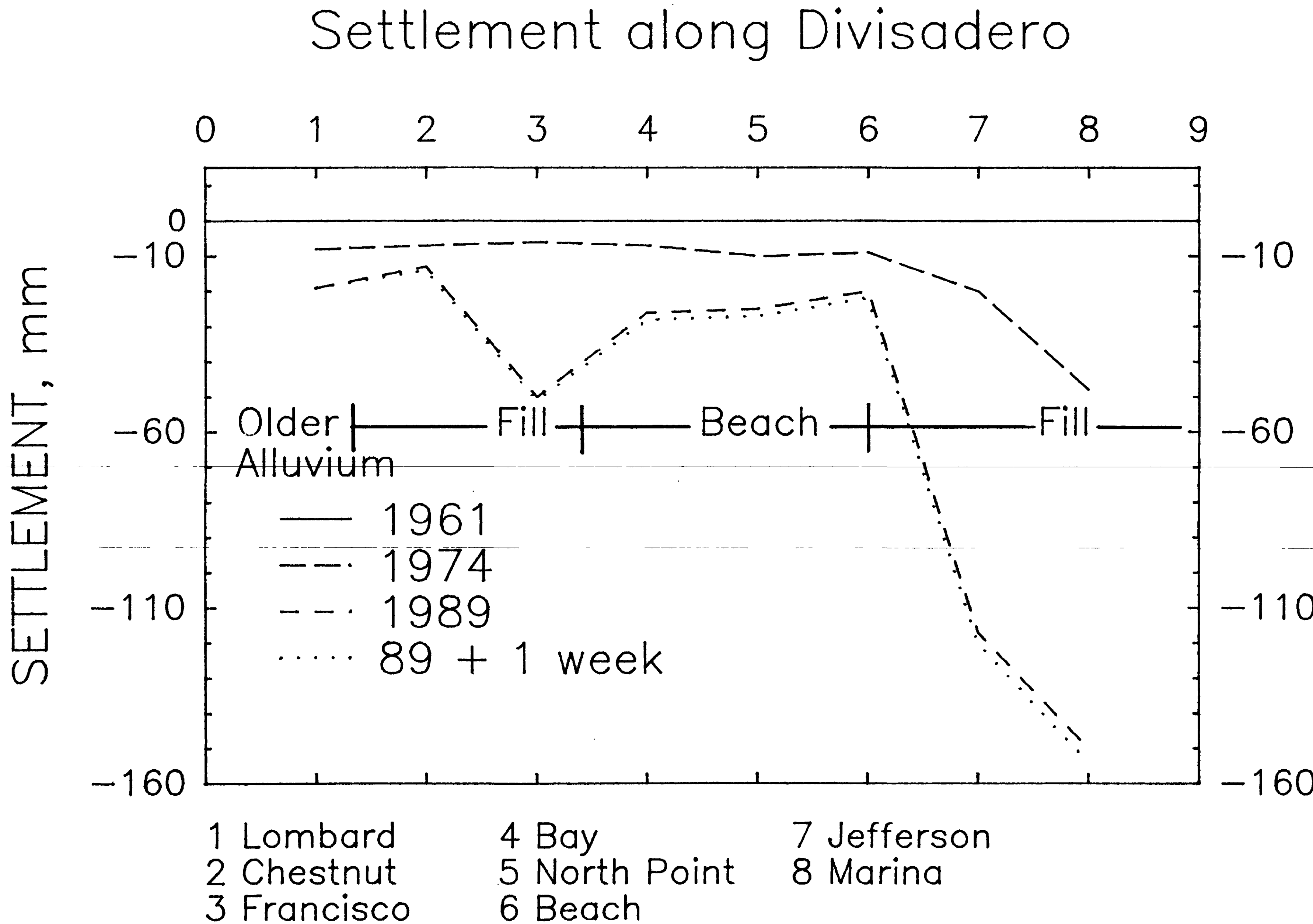


Figure 13. Settlement along Divisadero for three time intervals, 1961-1974, 1974-1989, and a one week period in November 1989.

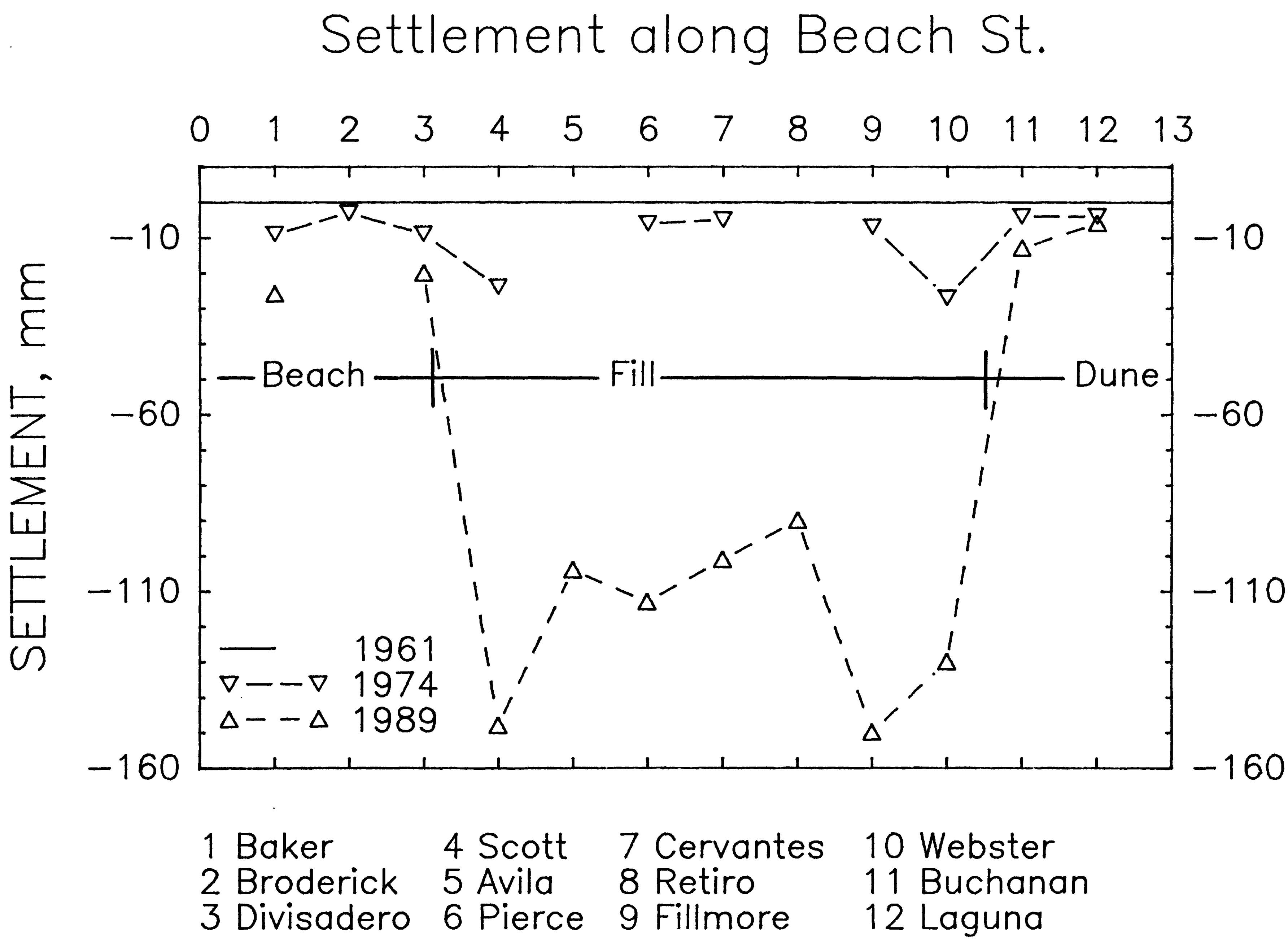


Figure 14. Settlement along Beach St. for the time intervals, 1961-1974, and 1974-1989. Avila and Retiro were not surveyed in 1961.

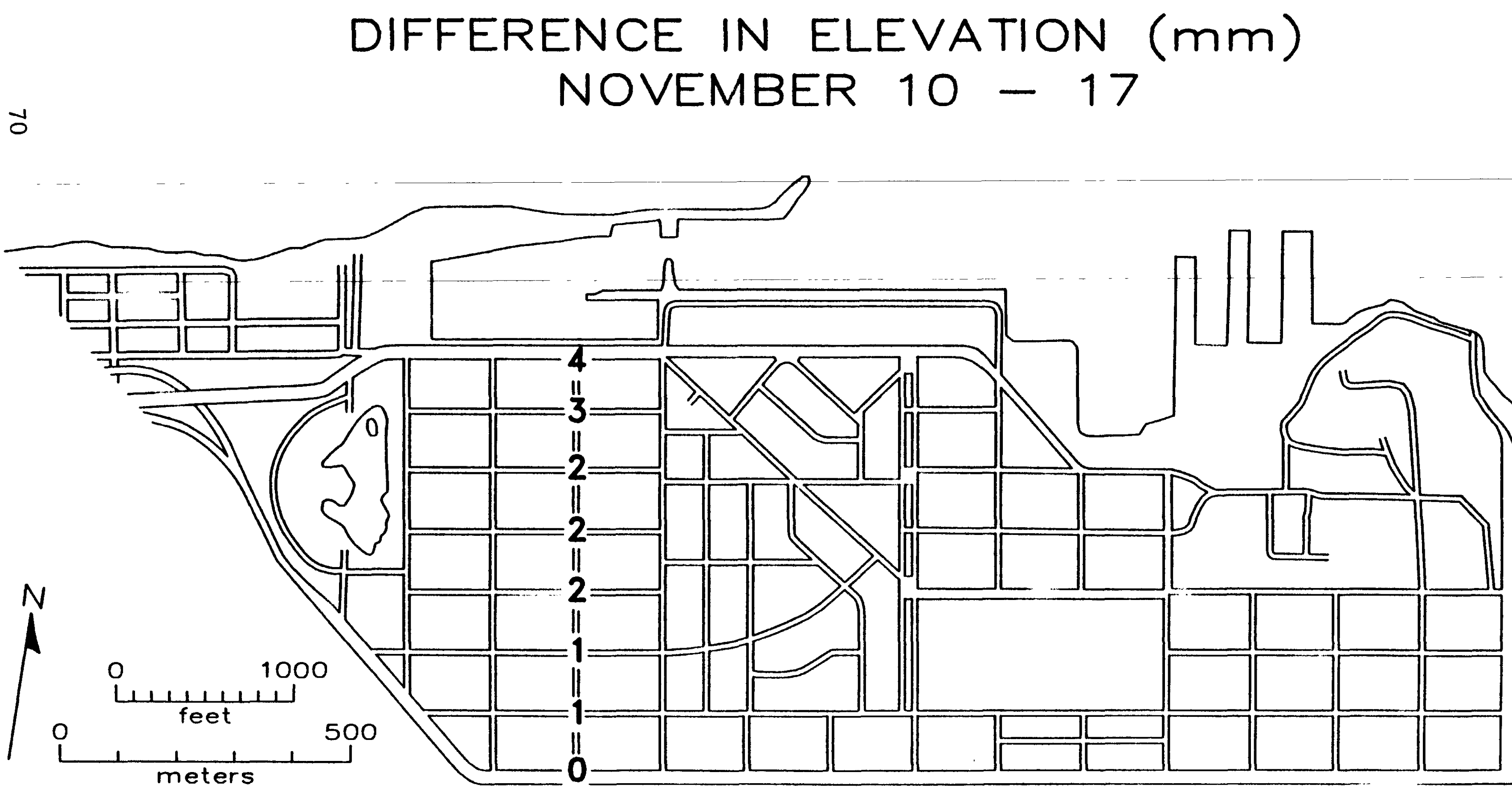


Figure 15. Difference in elevation along Divisadero between November 10-17, 1989.

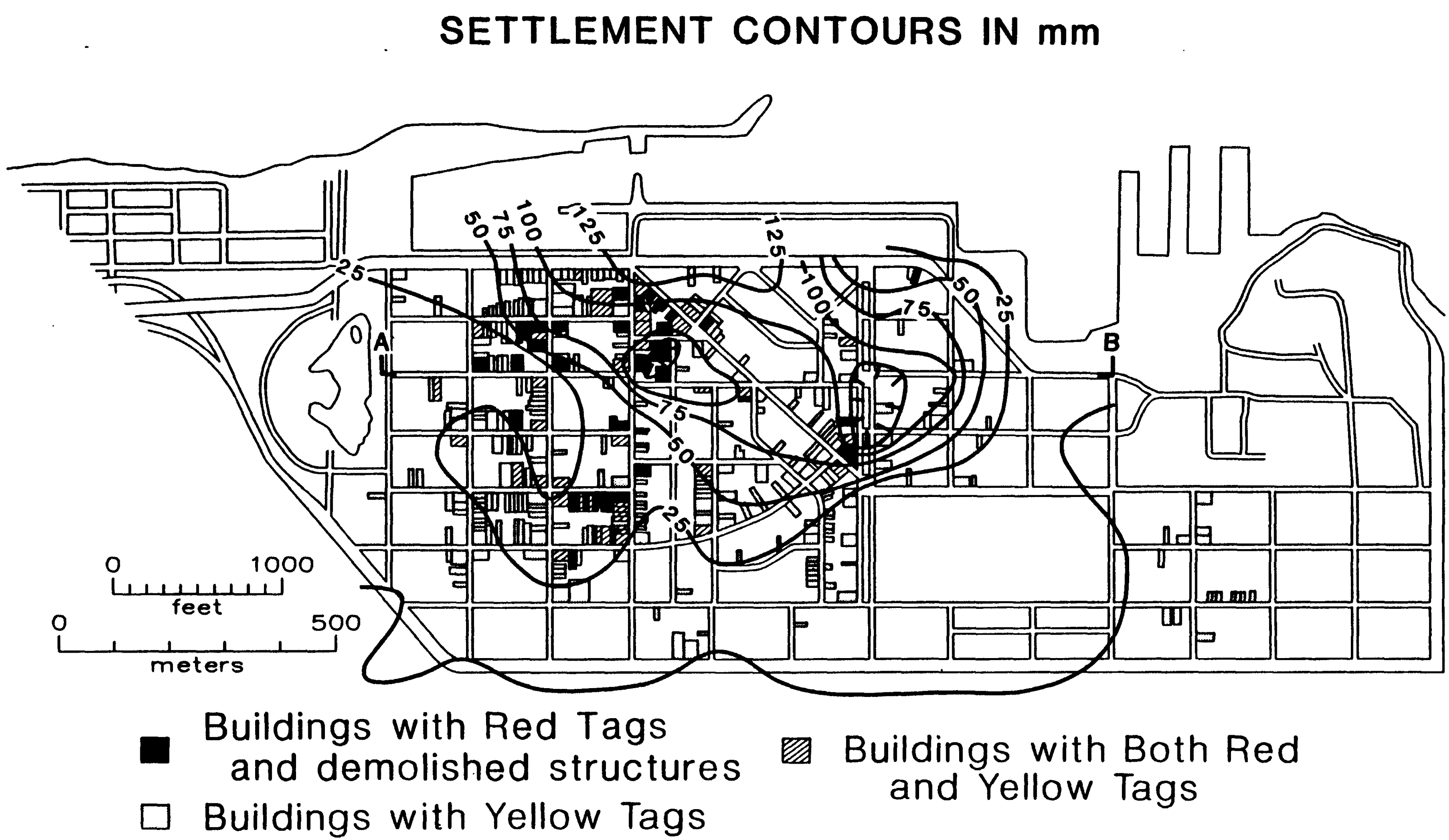


Figure 16. Red and yellow tag damage map (Seekins, this volume) is superimposed with 1974-1989 settlement contours. No points were surveyed east of Laguna or west of Baker.

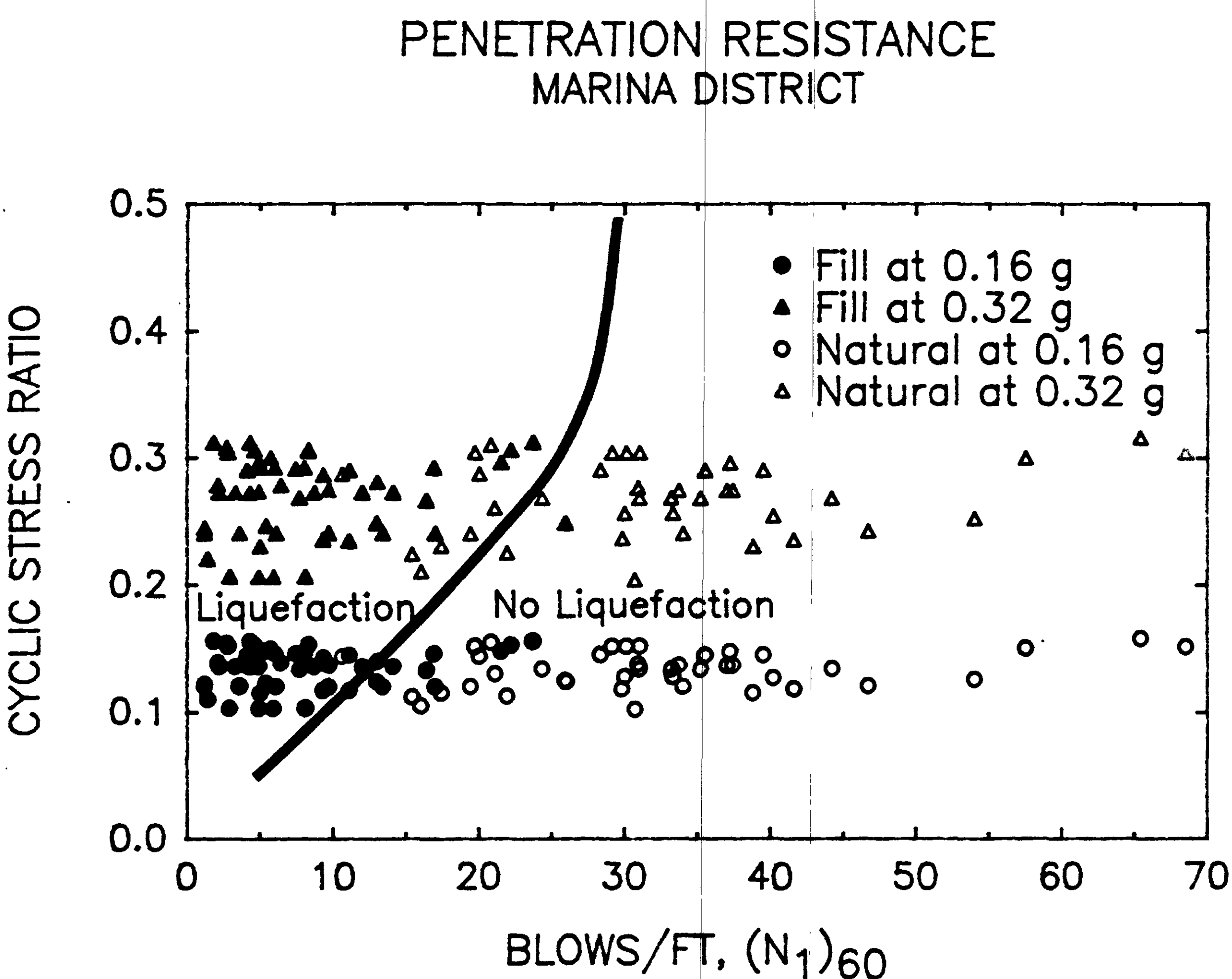
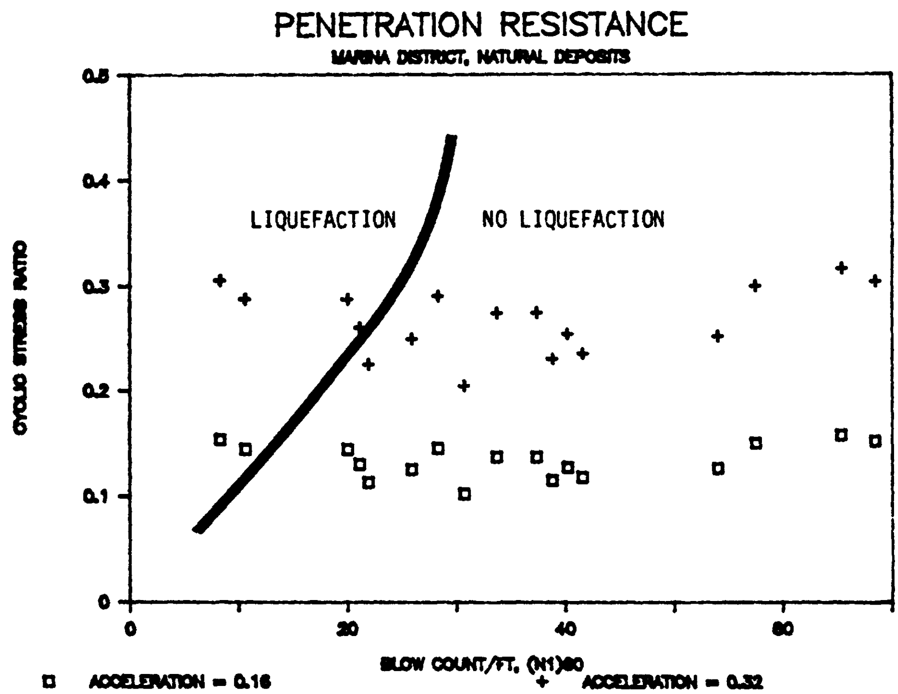


Figure 17. Liquefaction resistance chart using data from Dames and Moore (1976 and 1977) and USGS (this report). The fill, mostly hydraulic fill from Marina Cove, has a very low resistance to liquefaction at accelerations between 0.16 and 0.32 g. The natural deposits have a high resistance to liquefaction at an acceleration of 0.16 g, at the higher acceleration of 0.32 g the liquefaction resistance of some of the natural deposits is low. The line represents the boundary between liquefaction and no liquefaction for a M 7.1 earthquake.



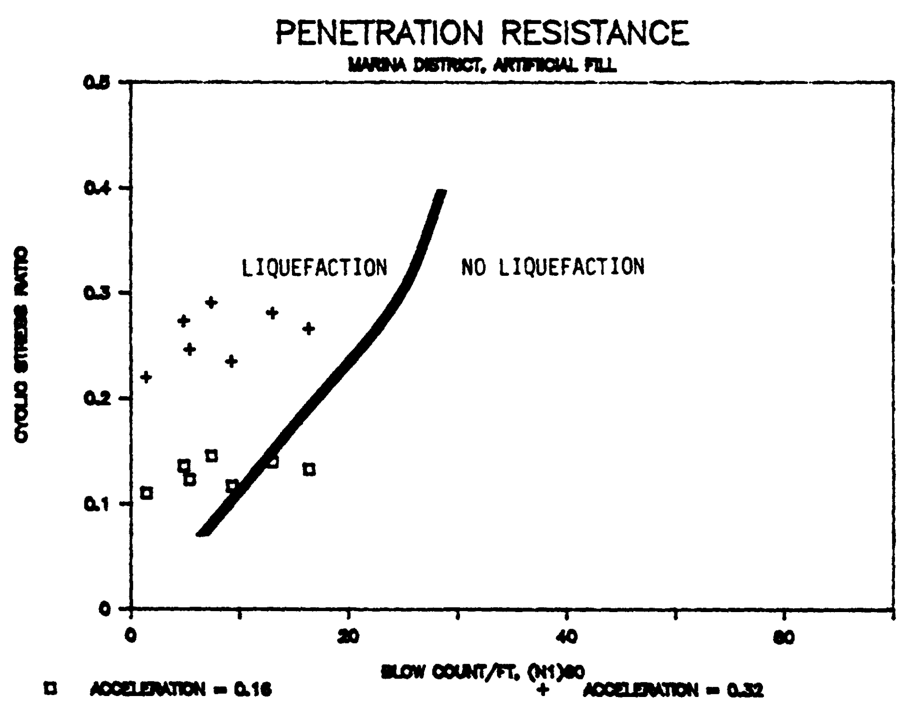


Figure 18. Liquefaction resistance chart using data from the USGS. This data confirms the Dames and Moore (1976 and 1977) data that show the fill has a very low resistance to liquefaction, whereas the natural deposits have a moderate to high resistance to liquefaction.

APPENDIX A. Grain size characteristics

Sand boil samples

Sample number	Depth m	ft	G	S	M	C	d50	Cu	USC	DESCRIPTION
M-1	surface		0	98	2	-	0.212	1.5	SP	pg SAND
M2-1	surface		0	98	2	-	0.305	1.8	SP	pg SAND
M2-2	surface		0	98	2	-	0.215	1.5	SP	pg SAND
M2-4	surface		0	97	3	-	0.240	1.7	SP	pg SAND
M2-5	surface		0	89	11	-	0.184	2.9	SP-SM	pg SAND w/M
M3-1	surface		0	98	2	-	0.240	1.7	SP	pg SAND
M4-1	surface		0	99	1	-	0.275	1.6	SP	pg SAND
M5-1	surface		0	99	1	-	0.197	1.6	SP	pg SAND
M7	surface		0	84	15	1	0.157	3.5	SM	silty SAND
M8	surface		0	94	6	-	0.173	1.6	SP-SM	pg SAND w/M
M9	surface		0	97	3	-	0.180	1.6	SP	pg SAND
M10	surface		0	98	2	-	0.180	1.3	SP	pg SAND
M11	surface		0	98	2	-	0.170	1.3	SP	pg SAND
M12	surface		0	91	9	-	0.145	1.9	SP-SM	pg SAND w/M
M13	surface		0	94	6	-	0.160	1.6	SP-SM	pg SAND w/M
M14	surface		0	95	5	-	0.170	1.7	SP	pg SAND
M15a	surface		0	85	15	-	0.147		SM	silty SAND
M15b	surface		0	95	5	-	0.230	1.8	SP	pg SAND
M16-1	surface		0	88	12	-	0.170	3.2	SP-SM	pg SAND w/M
M16-2	surface		0	89	11	-	0.149	2.5	SP-SM	pg SAND w/M
M17	surface		0	85	15	-	0.154		SM	silty SAND
M18	surface		0	88	12	-	0.178	3	SP-SM	pg SAND w/M
M20	surface		0	95	5	-	0.195	1.8	SP	pg SAND
M21	surface		0	74	22	4	0.140	7.6	SM	silty SAND
M22	surface		0	96	4	-	0.202	2.1	SP	pg SAND
M30	surface		0	87	13	-	0.153	3.1	SP-SM	pg SAND w/M
M31	surface		0	97	3	-	0.251	1.8	SP	pg SAND
M32	surface		0	93	7	-	0.221	2.4	SP-SM	pg SAND w/M
M33	surface		0	94	6	-	0.147	1.6	SP-SM	pg SAND w/M
M40	surface		0	42						
M41	surface		0	87	13	-	0.163	3.2	SP-SM	pg SAND w/M
M42	surface		0	35	47	18	0.046		ml	SILT w/S
M45	surface		0	93	7	-	0.160	1.7	SP-SM	pg SAND w/M
M46	surface		0	88	12	-	0.155	2.7	SP-SM	pg SAND w/M
M50	surface		0	76	24	-	0.232		SM	silty SAND
M51	surface		0	41	53	6	0.058	4.6	ML	SILT w/S
M52	surface		0	82	18	-	0.152		SM	silty SAND
M53	surface		0	94	6	-	0.240	1.9	SP-SM	pg SAND w/M

Subsurface samples

Sample number	Depth m	ft	G	S	M	C	d50	Cu	USC	DESCRIPTION
Schol	30.3	99.5	0	6	68	26	0.016		cl	lean CLAY
MAR1	0.9	3	0	96	4	-	0.321	1.8	SP	pg SAND
	1.8	6	0	97	3	-	0.309	1.7	SP	pg SAND

	m	ft.	G	S	M	C	d_{50}	C_u	USC	DESCRIPTION
	3.0	10	0	98	2	-	0.298	1.6	SP	pg SAND
	4.3	14	0	9	48	43	0.008			
	5.8	19	0	95	5	-	0.303	1.2	SP	pg SAND
	7.3	24	0	89	11	-	0.271	5	SP-SM	pg SAND w/M
	11.0	36	0	81			0.200		SM	silty SAND
MAR2	0.6	2	0	97	3	-	0.405	2.6	SP	pg SAND
	1.2	4	0	98	2	-	0.280	1.6	SP	pg SAND
	2.1	7	0	97	3	-	0.270	1.6	SP	pg SAND
	3.4	11	0	97	3	-	0.239	1.8	SP	pg SAND
	5.8	19	0	98	2	-	0.253	1.5	SP	pg SAND
	7.0	23	0	96	4	-	0.298	1.9	SP	pg SAND
	8.5	28	0	19	81	-				
MAR3	1.5	5.0	0	97	3	-	0.275	1.6	SP	pg SAND
	3.8	12.5	0	97	3	-	0.272	1.8	SP	pg SAND
	4.9	16.0	0	97	3	-	0.361	2.1	SP	pg SAND
	3.0	10.0	0	96	4	-	0.288	1.9	SP	pg SAND
	6.9	22.5	0	96	4	-	0.350	2.1	SP	pg SAND
	9.4	31	0	36	46	20	0.045			
MAR4	2.0	6.7	0	95	5	-	0.178	1.6	SP	pg SAND
	2.3	7.4	0	83	17	-	0.152		SM	silty SAND
	3.4	11.0	0	95	5	-	0.178	1.6	SP	pg SAND
	6.1	20.0	0	79	12	8	0.160		SM	silty SAND
	8.8	29.0	0	34	41	24	0.041		ml	
	10.4	34.0	0	54	32	14	0.094	45	SM	silty SAND
	11.9	39.0	0	82	11	7	0.181		SM	silty SAND
	12.0	39.5	0	85	15	-	0.182		SM	silty SAND
MAR5	1.9	6.3	0	70	30	-	0.130		SM	silty SAND
	2.1	7.0	0	80	20	-	0.171		SM	silty SAND
	2.3	7.5	16	62	22	-	0.210		SP-SM	pg SAND w/M&G
	3.1	10.3	0	92	8	-	0.178	2	SP-SM	pg SAND w/M
	3.3	10.8	36	55	9	-	1.250	34	SP-SM	pg SAND w/M&G
	3.4	11.3	0	88	12	-	0.178	3.1	SP-SM	pg SAND w/M
	4.4	14.3	0	93	7	-	0.185	1.9	SP-SM	pg SAND w/M
	5.5	18.0	0	90	10	-	0.185	2.3	SP-SM	pg SAND w/M
	6.4	21.0	0	97	3	-	0.197	1.4	SP	pg SAND
MAR6	2.1	7.0	0	91	9	-	0.231	2.7	SP-SM	pg SAND w/M
	3.4	11.0	0	94	6	-	0.261	2	SP-SM	pg SAND w/M
	4.6	15.0	0	97	3	-	0.240	1.5	SP	pg SAND
	5.8	19.0	0	96	4	-	0.268	1.7	SP	pg SAND
	7.0	23	0	94	6	-	0.244	1.9	SP-SM	pg SAND w/M

gravel (G) = greater than 4.75 mm
sand (S) = 0.075-4.75 mm
silt (M) = 0.005-0.75 mm
clay (C) = less than 0.005 mm
this symbol (-) indicates no hydrometer test was made
pg = poorly graded (well sorted)
pg SAND w/M&G = poorly graded sand with silt and gravel

Appendix B.

SPT BORING LOGS

Site	Depth, ft	Description
Marina 1, elevation 13.3 ft., water table 7.5 ft.		
	1.6- 2.5	dark brown (10YR3/3) sand, N=6
	8.2-12.1	very dark gray (N/3) sand, N=6
	12.1-16.1	soft dark-greenish gray silty clay, N=1
	16.1-26.2	very dark-greenish gray (5BG3/1) medium dense to dense sand, some clay, N=8,22
	26.2-32.2	soft greenish-gray (5BG3/1,5GY4/1) clayey silt, <u>Bay Mud</u>
	32.2-36.1	very dense strong brown and grayish brown (7.5YR4/6 and 2.5Y5/2) sand, "hard pan", N=63
Marina 2, elevation 12.6 ft., water table 9.0 ft.		
	1.6- 9.8	loose yellowish brown (10YR5/4) sand, N=7
	9.8-26.6	dense dark-greenish gray (5BG3/1) sand, N=24, <u>Beach</u>
	25.6-35.1	soft dark gray (5BG3/1) clayey silt, N=2, <u>Bay Mud</u>
	35.1-36.4	very dense sand, "hard pan"
Marina 3, elevation 12.0 ft., water table 9.0 ft.		
	1.6- 8.5	loose brown (10YR4/3) poorly graded SAND (SP), N=5
	8.5-24.3	dense dark gray (5Y3/1) poorly graded SAND (SP), N=23, <u>Beach</u>
	24.3-35.4	soft dark greenish gray (5BG4/1) sandy SILT (ML) N=2, <u>Bay Mud</u>
	35.4-38.1	very dense dark olive brown (2.5Y3/4) poorly graded SAND with silt (SP-SM) "hard pan", N=68
Marina 4, elevation 12.3 ft., water table 9.5 ft.		
	1.6- 5.9	loose grayish brown (2.5Y3/2) silty sand
	5.9-26.2	loose bluish gray (5B3/1) poorly graded SAND (SP) and silty SAND (SM), creosote smell, alternating sand and silt, N=3.5

26.2-36.7	loose black (5BG2.5/1) silty SAND (SM) to sandy SILT (ML), N=1, Bay Mud
36.7-37.7	dark greenish gray* (5G3/1) silty sand on top of; very dense olive brown (2.5Y4/4) silty SAND (SM), N=58, "hard pan", (*, described as green sand in many boring logs)

Marina 5, elevation 12.1 ft., water table 8 ft assumed

0 - 3.3	concrete, railroad tie and gravel
3.3- 9.8	loose olive brown (2.5Y3/4) to yellowish brown (10YR4/6) to grayish brown (2.5Y4/2) silty SAND (SM) to poorly graded SAND (SP-SM), N=3
9.8	boulder, couldn't get past it, began new hole 3' south
9.8-26.9	loose greenish gray (5BG3/1) to black (5Y2.5/2) poorly graded SAND with silt (SP-SM), gravel at top, N=11, N decreases with depth, Fill
26.9-52.5	soft clayey silt, Bay Mud
52.5	very dense sand, "hard pan"

Marina 6, elevation 26.0 ft., water table 18.0 ft.

0 - 3.3	concrete, railroad tie, and gravel
3.3-13.1	dense yellowish brown (2.5Y4/4 and 10YR4/4) poorly graded SAND with silt (SP-SM), with 1.5- ft thick clayey interbed, N=24
13.1-37.1	medium dense dark brown (10YR4/3) poorly graded SAND (SP), N=28
37.1-37.7	very dense sand, "hard pan"

Appendix C. Settlement between 1961-1974 and 1974-1989

		Elevation in feet		Change in mm	
Monument	1961	1974	1989	[61-74]	[74-89]
SM01SE	32.495	32.484	32.485	-3	0
SM02SE	32.498	32.488	32.486	-3	-1
SM03SE	29.612	29.601	29.591	-3	-3
SM04SE	23.513	23.504	23.495	-3	-3
SM05SE	23.604	23.588	23.584	-5	-1
SM06SE	16.658	16.646	16.654	-4	3
SM07SE	17.410	17.396	17.3[illegible]	-4	-6
SM08SE	19.655	19.630	19.5[illegible]	-8	-11
MH09SE	21.616	21.614		-1	
SM09SE		21.497	21.504		2
FH10NE	30.908	30.865	30.838	-13	-8
FH11SW		37.166		-7	
SWI11SW	33.415	33.4		-5	
MSTEP11NE		35.180	35.166		-4
SWI12SW		27.468		-1	
SM14SE		20.609	20.598		-3
FH15SW		20.317		-2	
FH16SE		12.435		-1	
SWI16SW	8.493	8.478		-5	
SWI17SW	29.378	29.348	29.352	-9	1
CCC18SE	14.903	14.883		-6	
CCC19CC	15.046	15.03		-5	
FH20SE	15.692	15.680	15.627	-4	-16
FH21NE		36.584	36.573		-3
FH22NE	4.549	4.460	4.121	-27	-103
SM23SE	5.835	5.821	5.784	-4	-11
FH24NE		5.643	5.348		-90
MH25SW	17.118	17.114	17.120	-1	2
FH26SE	14.663	14.663	14.656	0	-2
FH27NE	6.938	6.828	6.657	-34	-52
FH29SE	2.251	2.235	2.024	-5	-64
SM30SE	-0.838	-0.849	-0.945	-3	-29
FH31SW	0.864	0.834	0.367	-9	-142
SM31.3N			-0.715		
FH32SE	1.215	1.183	0.751	-10	-132
SWI33SE	-1.045	-1.201		-48	
SM33SE		-1.082	-1.417		-102
FH34SE	1.060	1.007	0.896	-16	-34
SM35SE	-0.495	-0.521	-0.617	-8	-29
FH36SE		8.855	8.403		-138
FH37NW	5.093	5.071	4.603	-7	-143
FH38NE		2.527	2.274		-77
MH39SW	6.704	6.696		-2	
ASW40SE	4.52	4.504		-5	
SWI41SW	2.265	2.246		-6	
FH41SE		4.430	4.078		-107
SWI42SE	3.000	2.982	2.667	-5	-96
FH43NE	2.587	2.525	2.270	-19	-78

Monument	Elevation in feet 1961	1974	1978	Change in mm (61-74)	(74-89)
SMH48CC		1.434	1.007		-130
FH49NE		3.555	3.215		-104
SMH51CC		2.393	2.254		-42
SMH52CC		2.145	1.785		-110
FH55NW	7.797	7.797		0	
FH55NE	6.352	6.344		-2	
SWI55NW	4.052	4.055		1	
FH56NE	4.913	4.885	4.761	-9	-38
FH57NE	3.534	3.456	3.048	-24	-124
FH58NE		2.595	2.307		-88
SWI58NW	0.102	0.068	-0.108	-10	-54
SM59SE	8.250	8.227	8.209	-7	-5
FH60NE	7.949	7.930	7.786	-6	-44
SM61SE	4.488	4.465	4.403	-7	-19
FH62NE	5.418	5.384	5.335	-10	-15
SM63SE		2.586	2.549		-11
step63SE	3.743	3.715		-9	
FH64NE	2.846	2.782	2.464	-20	-97
FH65NW	17.079	17.052		-8	
MSTEP66NW	7.81	7.739		-22	
FH67SE	7.042	7.016		-8	
FH68SE		5.889	5.778		-34
CCC68NW	4.924	4.899		-8	
FH69NE	6.657	6.648		-3	
SWI69NE	2.831	2.927		29	
SWI70NW	1.73	1.708		-7	
FH70SE		3.902	3.819		-25
SWI72NE	5.794	5.78		-4	
MH72CC	6.63	6.748		36	
FH72NE		7.773	7.753		-6
SM73NE	4.068	4.049	4.023	-6	-8
FH74SE	6.658	6.628		-9	
FH74NE		5.153	5.118		-11
BSTEPNE75	3.581	3.537		-13	
FH75SE	5.296	5.279		-5	
SWI75NE	1.781	1.925	1.870	44	-17
SM76SE	1.341	1.328	1.303	-4	-8
SM77SW	11.722	11.709	11.703	-4	-2

Naming survey monuments

The convention used in naming the monuments was monument type, followed by intersection number (from monument map) and corner direction. For example, SM63SE is a survey monument on the SE corner of intersection 63 (Divisadero and Beach); SMH48CC is a sewer manhole in the center of intersection 48 (Prado and Avila); FH27NE is a fire hydrant on the northeast corner of intersection 27 (North Point and Webster); SWI17SW is a surface water intake on the southwest corner of intersection 17 (Bay and Laguna) (written communication, A Okamura, 1989).

PERFORMANCE OF PIPELINE SYSTEMS IN THE MARINA

Thomas D. O'Rourke[1] and Bruce L. Roth[1]

Water to the Marina 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 fire fighting via hydrant and sprinkler systems. The AWSS supplies water exclusively for fire fighting purposes.

The AWSS was built to provide an extra level of fire protection as a result of experience gained from the 1906 earthquake. It comprises approximately 200 km of buried pipe, with nominal diameters ranging from 250-500 mm. The pipelines are located primarily throughout the northeastern portion of the City of San Francisco (O'Rourke and others, 1990). 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.

Within the Marina, in an area bounded by the 1857 shoreline on the south (U.S. Coast Survey, 1857) and the current shoreline on the north, there are approximately 11,300 m of pipelines belonging to the MWSS and 2290 m of pipelines belonging to the AWSS. The MWSS water mains are 100, 150, 200, and 300 mm in diameter, whereas the AWSS water mains are predominantly 250 and 300 mm in diameter. The pipelines in both systems are composed of pit cast iron and most were installed in the Marina between late 1924 and 1925. The MWSS pipelines were built with cement caulked, bell-and-spigot couplings, whereas the AWSS pipelines were built with special couplings, as described in the discussion of pipeline damage which follows. All pipelines were buried at nominal depths to top of pipe between 0.9 to 1.2 m.

Because of their relatively small diameters and full embedment, water distribution pipelines tend to deform in conjunction with the ground. Locations of repair, therefore, become a direct reflection of the intensity of combined transient and permanent ground deformations.

Figure 1 shows a plan view of the MWSS pipelines and repairs relative to the current street system, 1899 shoreline (Sanborn

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Ferris Map Co., 1899) and 1857 shoreline (U.S. Coast Survey, 1857). The shorelines shown in this figure are consistent with the map of natural soils and fills presented by M.G. Bonilla, "Natural and Artificial Deposits in the Marina District" (Figure 6). 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 shore-line. A few pipelines were repaired in an area underlain by native soil, formerly known as Strawberry Island and described by Bonilla.

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.

To represent the distribution of 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 2. The contours of pipeline repair rates are related closely to the 1857 shoreline, with the great majority of pipeline damage occurring within artificial fills. 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 embankment, and 1857 shoreline.

Table 1 summarizes the MWSS pipeline repairs in the Marina according to pipe diameter, main repair, damage at or near gate valves, and repair rate. The repair rate is defined as the number of repairs, including mains and main sections adjacent to gate valves but excluding services, per 300 m of line. By normalizing the number of repairs relative to the length of a specifically sized pipe, it is possible to check for a relationship between damage and diameter of line. As can be seen in the table, the repair rate declines in inverse proportion to the diameter of pipe. Over 80 per cent of main repairs were for round cracks which implies that bending and longitudinal tension were the prominent modes of deformation.

Figure 3 shows a plan view of the AWSS pipelines and repairs relative to the current street system, 1899 shoreline, and 1857 shoreline. In contrast to the MWSS performance, there was only one repair in the AWSS. This occurred at a leaking joint near the intersection of Scott and Beach Streets.

As indicated in Table 1, there was one repair in the 300-mm-diameter mains of the MWSS, which was performed at a round crack. Although there was nearly twice the linear distance of 250 and 300-mm-diameter mains of the AWSS in the Marina, no pipe ruptures were observed and the only repair was for a leaking joint. Pipelines of the AWSS are equipped with sleeve joints, which are restrained against pullout by longitudinal bolts. The 250 and 300-mm diameter pipelines have 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, apparently was successful in allowing the pipelines to accommodate differential ground movement.

Table 1. Summary of Pipeline Damage in MWSS in the Marina*

Pipeline Diameter mm	Main Repairs	Repairs at or near Gate Valves	Repair Rate Repairs per 300 m
100	16	2	4.5
150	33	8	2.2
200	7	2	0.8
300	1	-	0.2

* Service repairs, which total 54, are not included in the table.

The most serious damage to the AWSS occurred outside of the Marina in an area of soil liquefaction on 7th Street between Mission and Howard Streets. A 300-mm-diameter cast iron main broke at this location. Water flow through this break, supplemented by losses at broken hydrants, emptied the Jones St. Tank of its entire storage of 2.8 million liters in approximately 20 to 30 minutes. The Jones St. Tank is the reservoir which provides water directly into the lower zone of AWSS, from which the Marina is supplied (O'Rourke and others, 1990). Loss of this reservoir 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 Streets, water to fight the fire was pumped 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 the 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 Portable 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.

Because of damage sustained by the gas distribution system in the Marina, a decision was made to replace substantial lengths of gas mains. In an area bounded by Marina Boulevard, Buchanan, Chestnut, and Lyons Streets, approximately 13,400 m of mains were replaced either by direct burial or the insertion of medium density polyethylene piping into existing mains. The damaged piping consisted predominantly of 100, 150, and 200-mm-diameter cast iron and steel pipelines. Because of the widespread replacement, it was not necessary to identify specific locations needing repair. Accordingly, records showing specific type and location of gas pipeline damage, comparable to those for the water distribution system, were not acquired.

In summary, pipeline repairs in the MWSS and replacements in the gas distribution system show that pipe damage was concentrated in the Marina primarily in areas of artificial fill. The detailed record of MWSS repairs shows a high concentration of damage in areas underlain by hydraulic fill, with the heaviest concentration of damage at the junction of the hydraulic fill, 1899 seawall embankment, and 1857 shoreline. Damage in the MWSS pipelines was inversely proportional to pipe diameter. The AWSS pipelines, which were equipped with flexible and horizontally restrained joints, experienced very little damage in the Marina, with only one leaking joint needing repair. The water supplied by the AWSS was lost because of damage sustained outside the Marina near the intersection of 7th and Mission Streets. The ability to fight the fire which erupted in the Marina was provided by a special system of portable hosing. The vulnerable nature of buried pipelines to ground deformations both inside and external to the Marina underscores the importance of a flexible water supply system which can pump water from the Bay in times of emergency.

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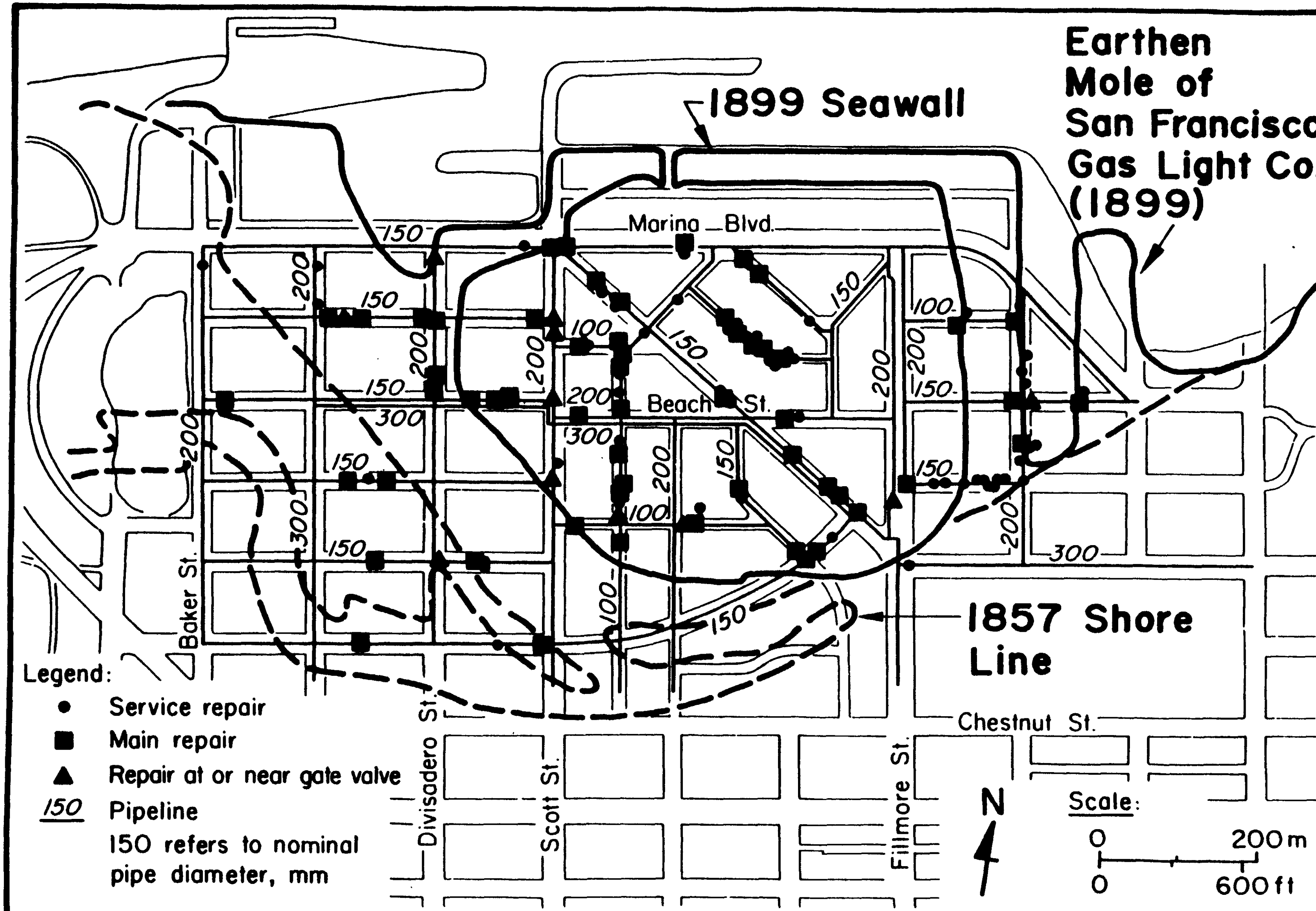

Figure 1. Map view of repairs to Municipal Water Supply System in the Marina District.

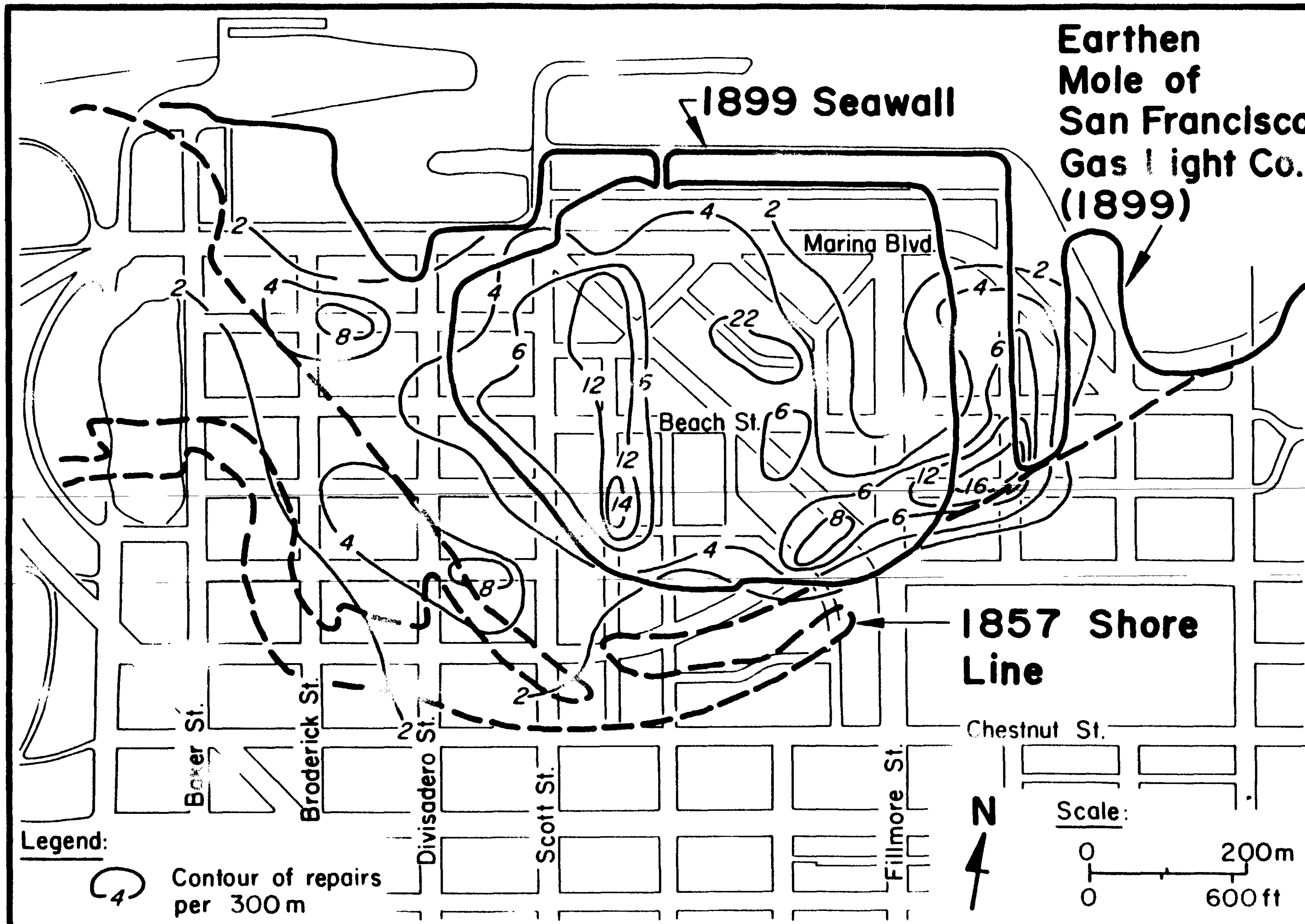

Figure 2. Contours of repairs to the Municipal Water Supply System per 300 m of pipeline.

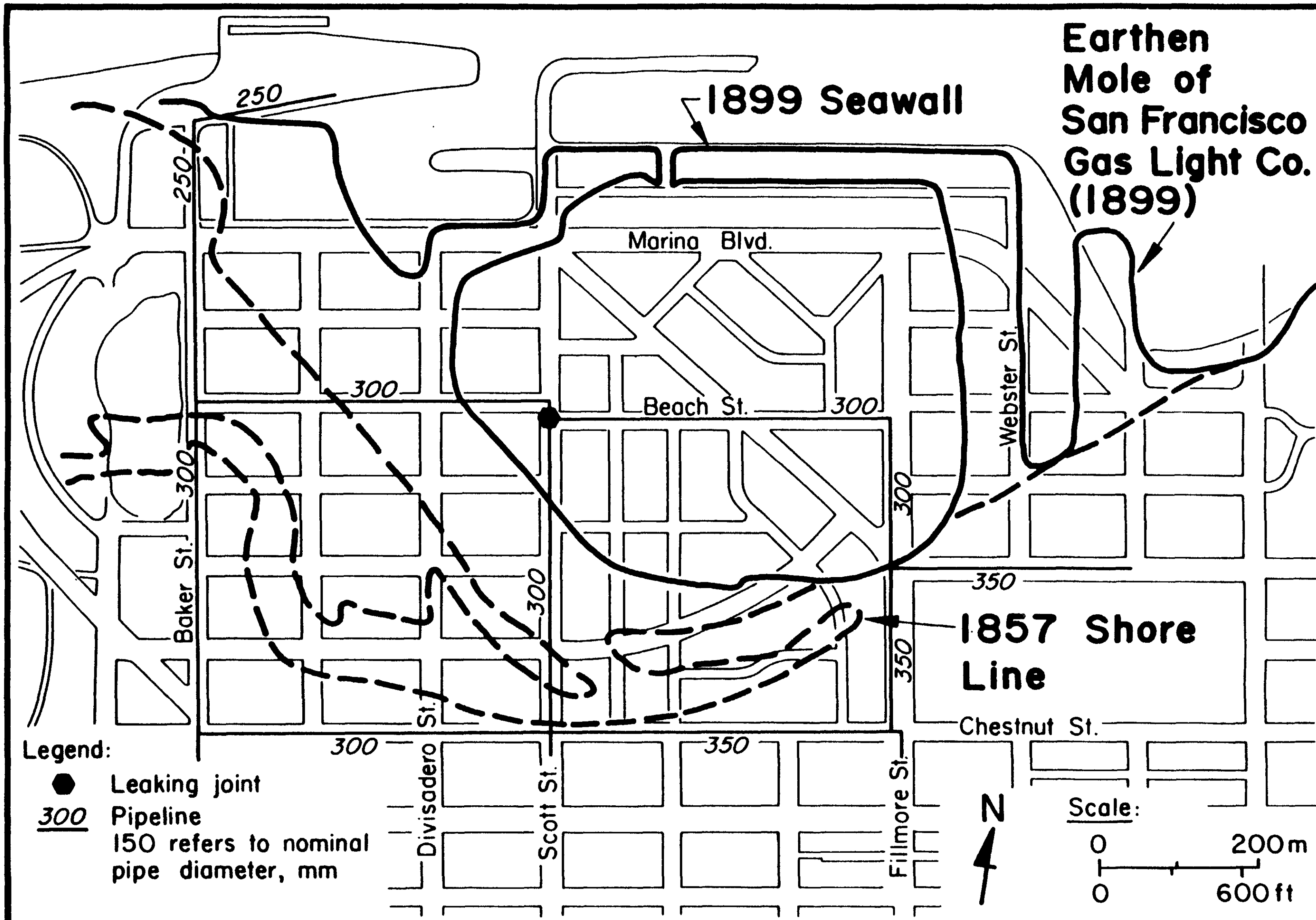

Figure 3. Map view of repairs to Auxiliary Water Supply System in the Marina District.

GROUND MOTION AMPLIFICATION IN THE MARINA DISTRICT

John Boatwright, Linda C. Seekins, and Charles S. Mueller

Introduction

The concentration of damage from the Loma Prieta earthquake in the Marina District suggests that there was a significant amplification of ground motion in the Marina relative to nearby undamaged areas such as Pacific Heights or Russian Hill. To investigate this amplification, the US Geological Survey, Branch of Engineering Seismology and Geology, deployed triggered seismographs with GEOS recorders (Borcherdt et al., 1985) at nine sites inside and outside of the Marina District for three weeks following the main shock. Despite the large epicentral distances (≈ 100 km) and the relatively noisy seismic environment, these instruments were able to record 16 aftershocks ranging in size from $M_L = 2$ to 5.

This paper analyzes these aftershock recordings to determine the relative site response or amplification as a function of frequency in the Marina District. By suitably combining the spectra of the recorded shear waves, we estimate the seismic amplification for five sites in the Marina relative to a site at Fort Mason. The results indicate that the ground motions within the Marina District are amplified by factors ranging from 6 to 10 for periods around one second; the amplification decreases gradually to a factor of 3 for periods around 0.3 s and to a factor of 2 for periods around 0.2 s. We note that this amplification spans the approximate range for the fundamental periods of 3-4 story wood-frame structures (0.3 to 0.5 s), suggesting that seismic amplification contributed significantly to the earthquake damage in the Marina.

Because no accelerographs are permanently sited in the Marina District, the ground motions from the Loma Prieta main shock were not recorded there. The extensive building damage and ground failure in the Marina, however, impels estimates (or extrapolations) of the main shock ground motions. Conditional estimates of the ground motion can be made using aftershock recordings at sites within the Marina together with aftershock recordings at sites in San Francisco which recorded the main shock. The closest accelerograph which recorded the main shock and which we re-occupied to record aftershocks was located at a fire station on Pacific Heights, approximately 1.5 km south of the Marina.

The seismic amplifications determined from the aftershock recordings are explicitly appropriate only for weak levels of ground motion. Using these amplifications to estimate the strong ground motion which the Marina experienced during the Loma Prieta main shock is problematic. The ground failure and liquefaction of hydraulic fill which occurred in part of the Marina (see Bennett, this volume) conclusively indicate that the ground behaved non-linearly and that the aftershock amplifications cannot be used to estimate strong ground motions in these areas. One of the instrumented sites in the Marina, however, lies outside the area of ground failure and may be more reasonably assumed to have behaved linearly. Extrapolating the main shock ground motion for this site yields spectral accelerations which slightly exceed the spectral accelerations recorded at the Outer Harbor Wharf in Oakland and significantly exceed the spectral accelerations recorded at all other sites in San Francisco.

Instrument Locations

Figure 1a shows the locations of the five instruments deployed within the Marina. In general, the site names are abbreviations for the street or building where the instruments were located; for example, stations NPT and BEA were located on North Point and Beach Streets, while station PUC was located at the Pacific Union Company building, also known as the "Gas Light Building" (see Bonilla, this volume). Figure 1a also shows the location of station MAS, deployed on the knoll at Fort Mason.

Figure 1b shows the locations of all but one of the instruments whose recordings are analyzed in this paper. The stations CAL and RIN, on Pacific Heights and Rincon Hill, were co-located with strong motion accelerographs which recorded the main shock. A third station, DIA, was co-located with a strong motion accelerograph on Diamond Heights, but plots to the south of the area shown. The station LEA is located on Nob Hill. These four stations, together with the station MAS at Fort Mason, constitute a set of "hard-rock" sites located in areas which suffered little or no damage during the main shock.

The aftershocks recorded by the various stations are tabulated in Table 1. The station MAS recorded the largest number of aftershocks, probably as a result of its relative isolation from vehicular traffic and its hard-rock site characteristics: the instrument was deployed in a concrete ammunition bunker poured onto Franciscan sandstone. Station MAS recorded almost all the aftershocks which were recorded in San Francisco and provides a crucial lynchpin for comparing site amplifications throughout the city. The instruments which were deployed within the Marina District had to be retrieved about two weeks after the earthquake because the work of replacing the gas and water mains made it impossible to record aftershocks.

The station CAL, co-located with a strong motion accelerograph on Pacific Heights, is the most important station for estimating the main shock ground motions in the Marina. Although there was an accelerograph located at the Letterman Hospital in the Presidio, we were unable to co-locate a triggered seismograph at this site. The eleven aftershocks which were recorded by both stations CAL and MAS insure that the relative amplification of these two sites is very well determined.

Aftershock Recordings

The EW horizontal components of ground velocity recorded at five of the stations during a $M_L = 3.4$ aftershock are plotted in Figure 2. This aftershock occurred 8 days after the main shock. Despite its relatively small size, the earthquake was the most well recorded event (in the Marina) of the aftershock sequence. A cursory glance at the ground velocities shows the severity of the ground amplification problem for the Marina District. Stations NPT, BEA, and LMS are located within the Marina: the peak velocities for these stations range from 0.04 to 0.05 cm/s. Stations MAS and CAL are located at relatively "hard-rock" sites: the peak velocities for these stations are 0.02 and 0.01 cm/s, respectively. The hypocentral distances to these stations vary only from 97 to 99 km, so that we can reasonably assume that the incident wavefield is nearly identical for all the stations.

The spectral amplitudes of the shear waves are plotted in Figure 3 as a function of logarithmic frequency. The seismic amplification apparent in Figure 2 is clearly delineated in the frequency domain. In particular, the spectral amplitudes of the shear waves recorded at CAL are consistently the smallest among the five stations. Station MAS exhibits spectra amplitudes which are intermediate to those from station CAL on Pacific Heights and those from the stations in the Marina. Station BEA has a marked spectral peak at 2.2 Hz, while stations NPT and LMS have spectral peaks near 4.0 Hz, which is approximately the fundamental frequency for a 2-story building with a wood frame.

In general, the relative spectral amplitudes recorded at these stations vary between events, depending on the component of ground motion, and on the hypocentral distance, the hypocentral depth and the focal mechanism of the aftershock. To insure that we obtain the least biased estimate of the seismic amplification, we combine all of the aftershock recordings together. The next section briefiy describes the analytical background for this procedure.

Decomposition into Source and Site Spectra

Following Andrews (1986), we assume that each record spectrum is the product of a site response spectrum and a source spectrum, and may be written as

$$r_k R_k(f) = SR_i(f) ES_j(f) \tag{1}$$

where the subscripts k, i, and j refer to the recording, to the station, and to the earthquake, respectively. $SR_i(f)$ is the site response spectrum for the ith station, while $ES_j(f)$ is the source spectrum for the jth earthquake. The geometrical spreading factors $r_k = x_{ij}/2$ are set equal to half the hypocentral distance between the station and the earthquake: the factor of 2 accounts for the amplification of the free surface. The record spectra, $R_k^2(f)$, are determined by summing the square of the spectral amplitudes of the two horizontal components of the shear wave. The spectral amplitudes have been resampled logarithmically to save space, where the frequencies from 0.1 to 100 Hz are divided into 40 frequency bands which each span a factor of $2^{1/4} = 1.189$ in frequency.

This system of equations can be linearized by taking logarithms and solved by minimizing the error in the k equations

$$\left(\ln R_k(f) + \ln r_k - \ln SR_i(f) - \ln ES_j(f)\right)/\sigma_k^2(f) = 0\,. \tag{2}$$

The variances $\sigma_k^2(f)$ are determined by taking small samples of the P-wave coda before the shear wave arrivals. Summing the square of the noise spectra from the two horizontal components of ground motion yields the noise functions $N_k^2(f)$. The variances are constrained following Andrews (1986) as

$$\sigma_k(f) = \max\left(N_k(f)/R_k(f), 0.5\right) \tag{3}$$

which is equivalent to saying that the signal to noise ratio of the data cannot exceed a factor of 2. This conditioning is necessary because equation (1) represents a relatively inexact decomposition and because the noise samples are imperfect estimates of the actual noise or uncertainty in the data.

For K recordings, I stations, and J earthquakes, equation (2) describes a system of K equations to determine $I + J$ unknowns. There is one undetermined degree of freedom associated with this system of equations. Physically, this undetermined degree of freedom means that we can estimate "relative" but not "absolute" site response or source spectra without adducing further constraints.

As a first constraint for the inversion, we set the site response for one station identically equal to one so that the site response for the $i - 1$ other stations are determined relative to this station (Mueller and Bonamassa, written communication, 1989). We use station MAS as the reference station both because it recorded the largest number of earthquakes and because it has the most average response of the hard-rock stations. The station constraint is simply written as $\ln SR_{MAS}(f) = 0$. Inverting the consequent system of equations (2) determines the site and source spectra,

$$\frac{SR_i(f)}{SR_{MAS}(f)} \quad \text{for } i = 1, I \quad \text{and} \quad SR_{MAS}(f) ES_j(f) \quad \text{for } j = 1, J\,. \tag{4}$$

Dividing the constrained source spectra by the appropriate geometrical spreading factors (that is, $x_{MAS\,j}/2$) yields spectral estimates of the ground motion recorded at station MAS from the jth earthquake.

Figure 4 shows the source spectra for event 2990901, whose waveforms and shear wave spectra are plotted in Figures 2 and 3. The shaded area spans ± one standard deviation and represents 85% confidence limits. Note that the range has been increased relative to that of Figure 3 to include the spectral amplitudes for frequencies above 10 Hz, and that the seismometer response has been corrected, amplifying the low frequency spectral amplitudes. At frequencies below 1.0 Hz, the recorded spectra are contaminated by additive noise from the GEOS recorders. Because the data from each of the GEOS recorders are similarly contaminated, however, this low frequency noise is generally projected onto the source spectra rather than the site spectra.

Relative Site Amplifications

Figure 5 shows the site response of station NPT relative to station MAS; station NPT is located near the corner of North Point and Divisadero in an area which suffered significant damage during the main shock. The relative site response is well determined within the frequency band from 0.6 Hz (1.6 s) to 15 Hz. The site response is amplified by a factor of 7 near 1 Hz and decreases gradually with increasing frequency up to 15 Hz. At the lowest frequencies, the amplification is approximately a factor of 2.

The relative site response for all five stations located in the Marina are plotted together in Figure 6. Although there is some variation between stations, the overall behavior is remarkably similar: a rapid increase to a peak amplification of 6-10 near 1 Hz and a gradual decrease with increasing frequency up to 15 Hz. The frequency band from 0.7 Hz to 3.0 Hz is significantly amplified for all the Marina stations relative to station MAS. Note that the logarithmic resampling does not smooth the input record spectra, so that the apparent lack of sharp resonance peaks is a real characteristic of these site response spectra.

There are some variations between the stations which are of interest, but which are not easy to disentangle from Figure 6. In particular, station PUC, located near the corner of North Point and Buchanan, is more strongly amplified at high frequency than the rest of the Marina stations. The amplification at station PUC relative to station MAS is approximately a factor of 3 over the frequency band from 2.5 to 25 Hz. We note that this site is located to the east of the hydraulic fill: it is possible that the hydraulic fill itself attenuates the high frequency motion for stations BEA and LMS.

The site response for the hard-rock stations, plotted in Figure 7, indicates the significance of the seismic amplification in the Marina. On average, these stations show no amplification on the frequency band from 0.3 to 20 Hz. At frequencies from 5.0 to 15 Hz, the amplification at station LEA increases to a factor of 2, while the amplification at station CAL is approimately half that of station MAS at frequencies from 2.0 to 20 Hz. The increase at low frequency of the relative site response for station DIA is the result of a malfunctioning recorder at this site; the low frequency recorder noise is much stronger for this station than for the other stations and is therefore projected onto the site response.

Extrapolating Main Shock Ground Motions

If we include the accelerograph recordings of the main shock in the record set to be decomposed into source and site spectra, it is possible to reconstrain equation (2) to yield linear extrapolations of the main shock ground motions at stations which only recorded aftershocks. Setting the source spectra of the main shock identically equal to one, (that is $\ln ES_o(f) = 0$, where the subscript o indicates the main shock) is mathematically equivalent to constraining one of the site spectra. Inverting the consequent set of equations determines the site and source spectra

$$SR_i(f)ES_o(f) \quad \text{for } i = 1, I \quad \text{and} \quad \frac{ES_j(f)}{ES_o(f)} \quad \text{for } j = 1, J\,. \tag{5}$$

Dividing the constrained site spectra by the appropriate geometrical spreading factors $(x_{io}/2)$ yields spectral estimates of the main shock ground motions at the I stations which recorded aftershocks.

As a test of this procedure, Figure 8 shows the recorded and extrapolated acceleration spectra for the main shock at station DIA. The excessive low frequency noise from the GEOS recorder deployed at station DIA precluded using the main shock accelerograms recorded at this station in the inversion as the inversion which results in the decomposition of equation (5) assumes that the recording characteristics are the same for each event. We used the accelerograms recorded at stations CAL and RIN. The fit of the extrapolated spectrum to the recorded spectrum is adequate, if not exemplary, for frequencies between 1.5 and 15 Hz.

To extrapolate the ground motions in the Marina for the main shock, we consider station NPT, located near the corner of North Point and Divisadero. Bonilla (this volume) indicates that this site is underlain by beach sands rather than any of the various man-made fills. In particular, the limit of the 1912 hydraulic fill lies a block to the east near Scott (see Bonilla, Figure 6). Moreover, the settlement in this area was minimal, about 15 mm (see Bennett, this volume). While these considerations do not insure that the ground below station NPT behaved linearly, they suggest that the main shock ground motion at NPT may be more reasonably estimated assuming linearity than at the stations located within the hyraulic fill.

The extrapolated acceleration spectrum for station NPT is plotted in Figure 9 using 85% confidence limits. As plotted in Figure 5, the amplification for station NPT has a broad peak at 1 Hz and a "side lobe" at 2.3 Hz; these peaks are clearly evident in the extrapolated acceleration spectrum. The acceleration spectra from three accelerographs which recorded the main shock are also plotted in Figure 9. The spectrum labelled PRS was obtained from an accelerogram recorded ≈ 1.5 km to the southwest of the Marina in a small building on the Presidio Golf Course. The spectrum labelled EMT was obtained from free-field accelerograph in Emeryville. Finally, the spectrum labelled OHW was obtained from an accelerograph at the Outer Harbor Wharf in Oakland which recorded the largest accelerations of any site located north of the San Francisco Airport.

Station PRS recorded a peak ground acceleration of 21% g, which was the largest free-field acceleration recorded in San Francisco. The spectrum for station PRS is a factor of 4 smaller than the extrapolated spectrum for station NPT around 1 Hz and a factor of 2 smaller at most other frequencies. Similarly, station EMT recorded a peak ground acceleration of 25% g; the spectrum for station EMT is about a factor of two smaller than the extrapolated spectrum for station NPT at frequencies above 0.8 Hz. Finally, the peak ground acceleration recorded at station OHW was 29% g; the spectrum for station OHW is remarkably similar to the extrapolated spectrum for station NPT at frequencies above 4 Hz; the extrapolated spectrum for station NPT exceeds the spectrum for station OHW at 1 Hz and 2-3 Hz.

This extrapolation indicates that the ground below station NPT has the potential to strongly amplify the main shock ground motion. Although the spectral decomposition does not incorporate the phase information necessary for synthesizing acceleration time histories, this comparison of spectral amplitudes suggests a range of 25% - 35% g for the extrapolated peak acceleration at station NPT. We note that this range exceeds all the free-field peak accelerations recorded in San Francisco. These estimates for peak acceleration are also commensurate with intensity levels VIII and IX on the Modified Mercalli Intensity Scale (Evernden and Thompson, 1985), which was the range of intensity assigned to the Marina District. Thus, the extrapolated accelerations appear to be reasonable estimates rather than extremal upper bounds for the ground motions in the main shock.

In contrast, extrapolating main shock ground motions in areas which suffered ground failure appears to overestimate the actual ground motions. Other than the blocks between Marina Boulevard and Beach, to the east of Scott (which suffered large settlements, see Bennett, this volume), the area underlain by the 1912 hydraulic fill was less severely damaged than the areas to the west and south (see Seekins et al., this volume). This damage pattern contradicts the variation of relative amplifications plotted in Figure 6, which show all the Marina stations to be similarly amplified on the frequency band from 1 to 5 Hz, and suggests that the ground motions in the main shock were smaller inside the area underlain by the 1912 hydraulic fill because the ground behaved non-linearly.

Figure 10 shows the extrapolated spectra for the main shock at two sites located on the 1912 hydraulic fill in the Marina, stations BEA and LMS. As in Figures 8 and 9, these extrapolations are plotted using 85% confidence intervals. The spectrum labelled TRI was obtained from the accelerogram recorded on Treasure Island which had a peak ground acceleration of 16% g. Hanks (oral communication, 1990) has suggested using this accelerogram as an analog for the ground motion in the Marina. The thick dashed line is an extrapolation for the main shock ground motions at Treasure Island, obtained from the relative amplification of the station TRI to the Yerba Buena Island site (station YBI) determined from recordings of seven aftershocks by Jarpe et al. (1990) and the spectrum of the main shock recorded at station YBI.

The extrapolated spectrum for station TRI is very similar to the extrapolated spectra for stations BEA and LMS, particularly for frequencies above 2 Hz. The extrapolation spectrum for station TRI overestimates the recorded spectrum of the main shock ground motions, however, by a factor of 3 on the frequency band from 1 to 10 Hz. The distribution of damage suggests that it is reasonable to use the accelerogram recorded at TRI to approximate the ground motions within the area underlain by the 1912 hydraulic fill, where a peak acceleration of 16% g appears more appropriate than a range of peak acceleration from 25% to 35% g. Similarly, it is reasonable to use the overestimate (of a factor of 3) as a measure of the uncertainty inherent in extrapolating strong ground motions for the Marina sites where ground failure and liquefaction indicate that the ground behaved non-linearly.

Conclusions

Recordings of aftershocks of the Loma Prieta earthquake indicate that weak ground motions in the Marina are significantly amplified relative to sites in Fort Mason and Pacific Heights over the frequency band from 1 to 5 Hz. This frequency band also spans the approximate range of the fundamental frequencies for 2-4 story wood-frame buildings, where the relative amplification is approximately a factor of 3-4. The extensive "shaking" damage in the Marina, that is, damage not directly associated with ground failure and liquefaction, suggests that the strong ground motions in parts of the Marina District were similarly amplified during the main shock.

Acknowledgements

We are indebted to Bob Bowen, Joseph Machi, Dave Katz, Al De Martini, the Pacific Union Company, to Jayne Shaeffer of the National Park Service, and to Captain John Herrington, Captain Jim Groshong, and Captain Doug Goodin of the San Francisco Fire Department for allowing us to deploy GEOS recorders at the sites necessary to carry out this study. Chris Deitel, Gene Sembera, Jim Gibbs, Tom Hanks, Tom Noce, and Roger Borcherdt installed and maintained the instruments, while Gary Glassmoyer was responsible for data playbacks and the consequent organization of the Loma Prieta data set. The accelerograms from stations CAL, RIN, PRS, DIA, TRI, YBI, and OHW were gathered, processed, and distributed by the California Strong Motion Instrumentation Program, Division of Mines and Geology. The accelerogram from station EMT was gathered and processed by the National Strong Motion Network in Menlo Park. The station designations used in this paper for the accelerograph sites may differ from designations used by these agencies in their reports. We thank Bill Joyner and Bob Brown for their thoughtful reviews of this paper.

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Table

EVENT	MAG	NPT	BEA	LMS	DEM	PUC	CAL	MAS	LEA	RIN	DIA
2921014	5.0	Q								Q	
2930018	3.9		L					L			
2930813	3.7	B	B					B	B		
2940049	4.3		T	T		T		T	T		
2940832	2.8		K					K	K		
2942215	4.6			D		D		D	D		
2951424	3.8	R	R					R	R		
2980127	4.5	N		N				N			
2981301	3.7			A	A			A			
29909[illegible]	3.4	O	O	O			O	O			
30108[illegible]	2.1	F	F				F	F			
3021311	3.0	D	D					D			
3031117	3.6	J		J			J	J			
3040835	3.1		C					B			
[illegible]3	3.4						K	K			
[illegible]550	4.4	I					I	I		I	
3071048	3.1						E	E			
3080716	3.6						C	C		C	
3090130	3.8						S	S			
3091337	4.1						P	P			
3102337	3.0							I			I
3112342	4.2						P	Q			P
3121345	2.1						I	I			I

Aftershocks recorded at the Marina. Arrival times (in Julian days, Greenwich Mean Time hours, and minutes) are entered in the leftmost column. Note that Julian day 291 corresponds to October 18 and Julian day 312 to November 8, and that GMT is eight hours ahead of PST and seven hours ahead of PDT. The Loma Prieta main shock, which occurred at 5:04 P.M. PDT on October 17, would be identified as event 2910004. The magnitudes given in the seond column were taken from the US Geological Survey preliminary listing. The letters printed in each station column refer to the second when the instrument was triggered (A for 0-2 s, B for 3-5 s, C for 6-8 s, etc...).

Figure Captions

Figure 1a. Station locations within the Marina. Note that stations BEA, LMS, and DEM lie within the area of the 1912 hydraulic fill (see Bonilla, this volume) while stations NPT and PUC lie outside it. Station MAS is sited on an outcrop of Franciscan sandstone at Fort Mason. Figure 1b. Station locations within the northeastern section of San Francisco. Stations CAL and RIN were co-sited with SMA-1 instruments which recorded the main shock, on Pacific Heights and Rincon Hill, respectively. Station LEA was sited at a fire station on Nob Hill. The unlabelled circles show other seismograph locations within the city whose recordings are not analyzed in this paper.

Figure 2. EW ground velocity for a $M_L \approx 3.4$ aftershock (event 2990901). This earthquake probably was able to trigger the Marina stations because it occurred at 2 A.M. on a Wednesday morning. Stations NPT, BEA, and LMS are in the Marina District. The S-waves are strongest on the EW component, while the P-waves are the weakest. The spectral amplitudes plotted in Figure 3 are determined from 20 s samples which start ≈ 2 s before the S-wave arrival.

Figure 3. Spectral amplitudes of the shear waves plotted in Figure 2. Note that the spectra of the Marina stations (NPT, BEA, LMS) are similar in overall amplitude, as the relative amplitudes of the seismograms suggest. For this event and this component of ground motion, station MAS is amplified by a factor of 3 relative to station CAL at 3 Hz.

Figure 4. Derived source spectrum for event 2990901. The "site" constraint used to obtain the spectral decomposition described by equation (4) constrains this source spectrum to be the record spectrum expected at station MAS for this event. The shaded area shows the 85% confidence interval for the expected spectrum. To estimate this spectrum, the seismogram spectra plotted in Figure 3 have been corrected to actual ground velocity (amplifying the low frequencies) and the two horizontal components have been combined together.

Figure 5. Site response spectrum for station NPT, relative to station MAS. The vertical bars show the 85% confidence interval for the spectral estimates. The relative site response is well determined from 0.7 to 15 Hz. At low frequencies, the relative amplification is slightly larger than a factor of 2. The amplification peaks around 1 Hz and decreases gradually with increasing frequency until 15 Hz, where the amplification ≈ 1.

Figure 6. Site response spectra for the five Marina stations, relative to station MAS. Although there is some variation between stations, the overall amplification is remarkably consistent, comprising a peak amplification of a factor of 6-10 near

1.0 Hz; this amplification gradually decreases with increasing frequency. Station PUC has the greatest amplification at high frequencies, while station LMS has the least amplification at high frequencies. Note that these site response spectra do not exhibit sharp peaks which can be interpreted as evidence for site resonance.

Figure 7. Site response spectra for the five hard-rock stations, relative to station MAS. The "site" constraint for station MAS yields a relative amplification equal to 1. The amplification at station MAS represents an approximate average for the hard-rock stations. Station CAL has the least amplification, while stations RIN and LEA have the greatest, reaching relative amplifications of 2. The apparent amplification at low frequencies for station DIA is due to a malfunctioning recorder deployed at this site.

Figure 8. Comparison of the extrapolated acceleration spectrum with the recorded acceleration spectrum for the main shock at station DIA. The extrapolated spectrum, indicated by the shaded region, slightly overestimates the recorded spectrum on the frequency band from 1 to 10 Hz. The mismatch at frequencies below 1 Hz is caused by the malfunctioning recorder. Note that only three recordings were used to determine this extrapolation (see Table), of which only one was within the aftershock zone; the other two events were small earthquakes which occurred near Daly City.

Figure 9. Comparison of the extrapolated acceleration spectrum for the main shock at station NPT with the recorded acceleration spectra from three accelerograph sites. The extrapolated spectrum is plotted as a shaded region. Station PRS is located on the Presidio Golf Course, Station EMT is in Emeryville, and station OHW is at the Outer Harbor Wharf in Oakland. These three accelerographs recorded peak accelerations of 21%, 25% and 29% g, respectively.

Figure 10. Comparison of the extrapolated acceleration spectra for two sites within the 1912 hydraulic fill with an extrapolated acceleration spectrum for station TRI (heavy dashed line), located on Treasure Island, as determined from the relative amplification of the NS component of motion for seven aftershocks, and the main shock recording at station YBI on Yerba Buena Island. The recorded acceleration spectrum for station TRI is about a factor of three less than the extrapolated spectra.

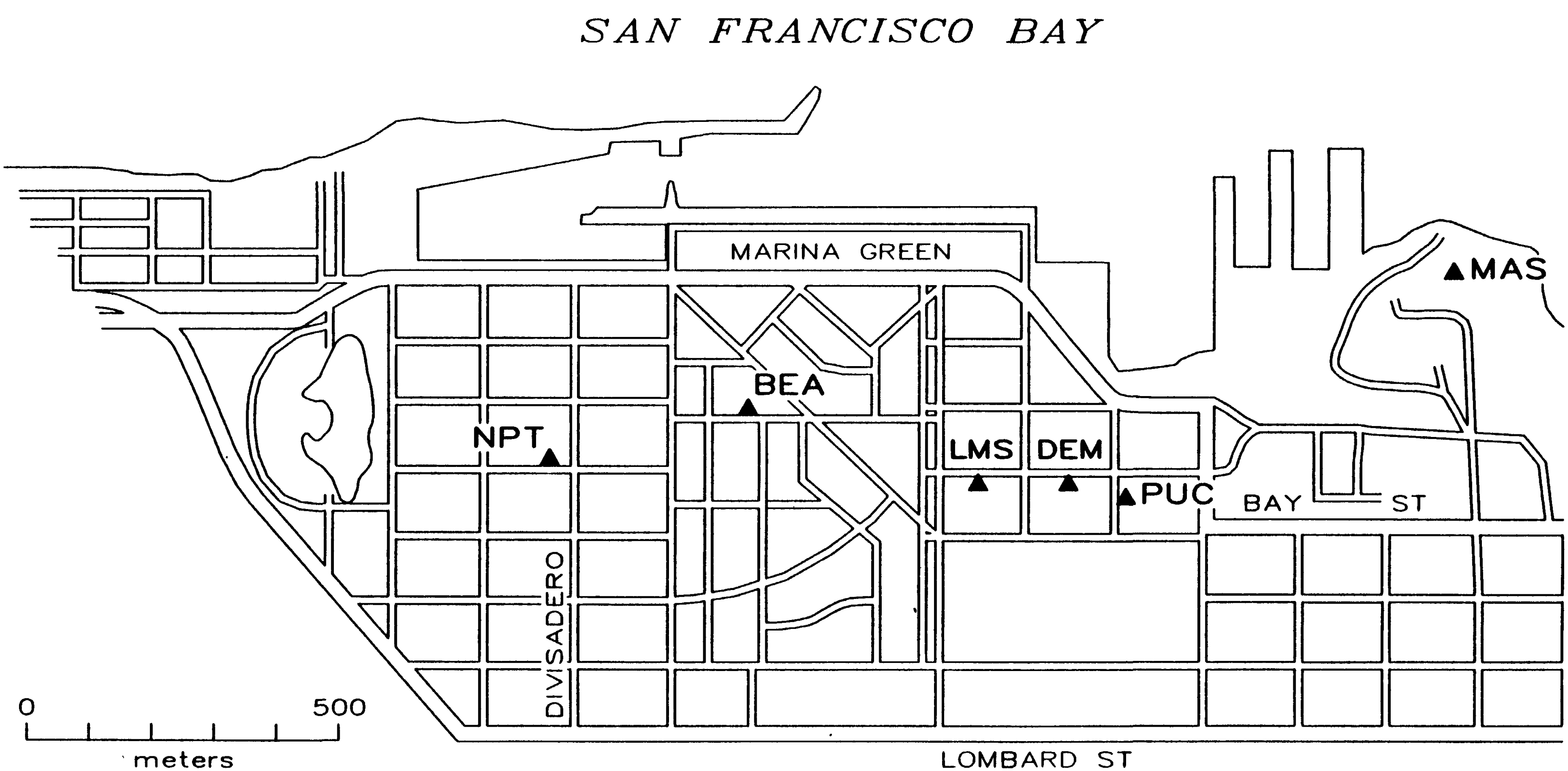

SAN FRANCISCO BAY
MARINA GREEN
MAS
BEA
NPT
LMS
DEM
PUC
BAY ST
DIVISADERO
LOMBARD ST
0
500
meters

Figure 1A

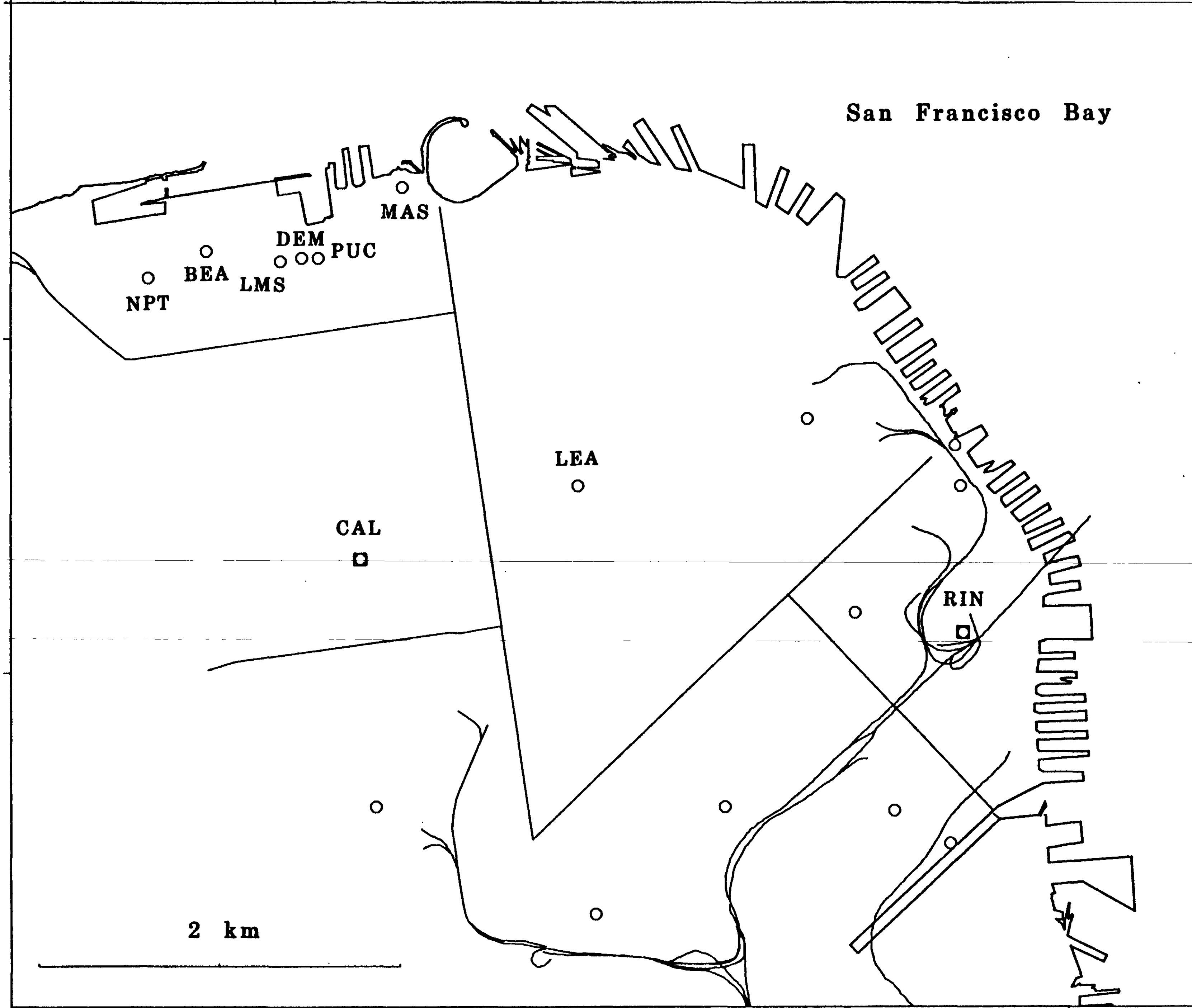
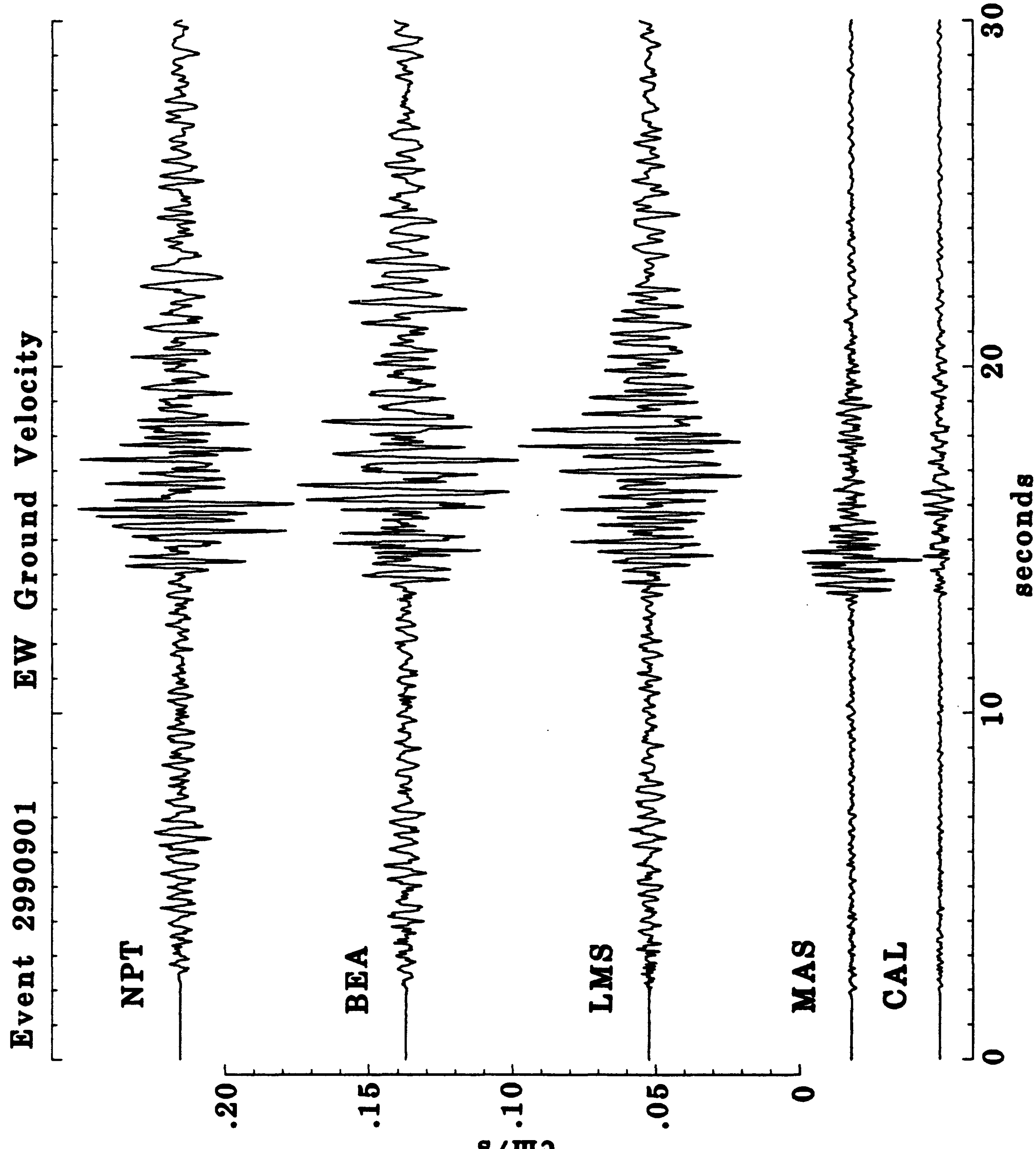


Figure 1B



Event 2990901 EW Ground Velocity
NPT
BEA
LMS
MAS
CAL
.20
.15
.10
.05
0
cm/s
0
10
20
30
seconds

Figure 2

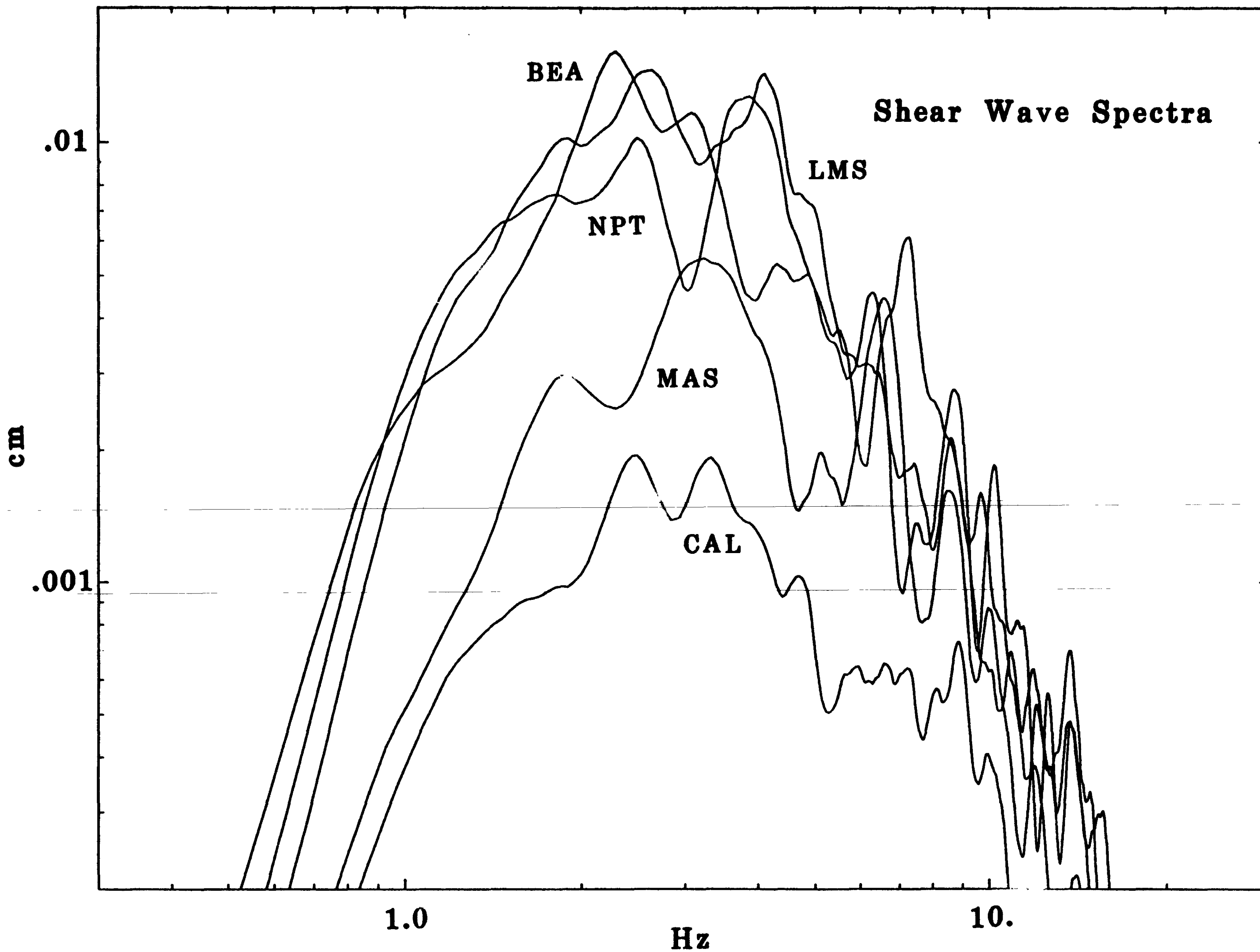


Figure 3

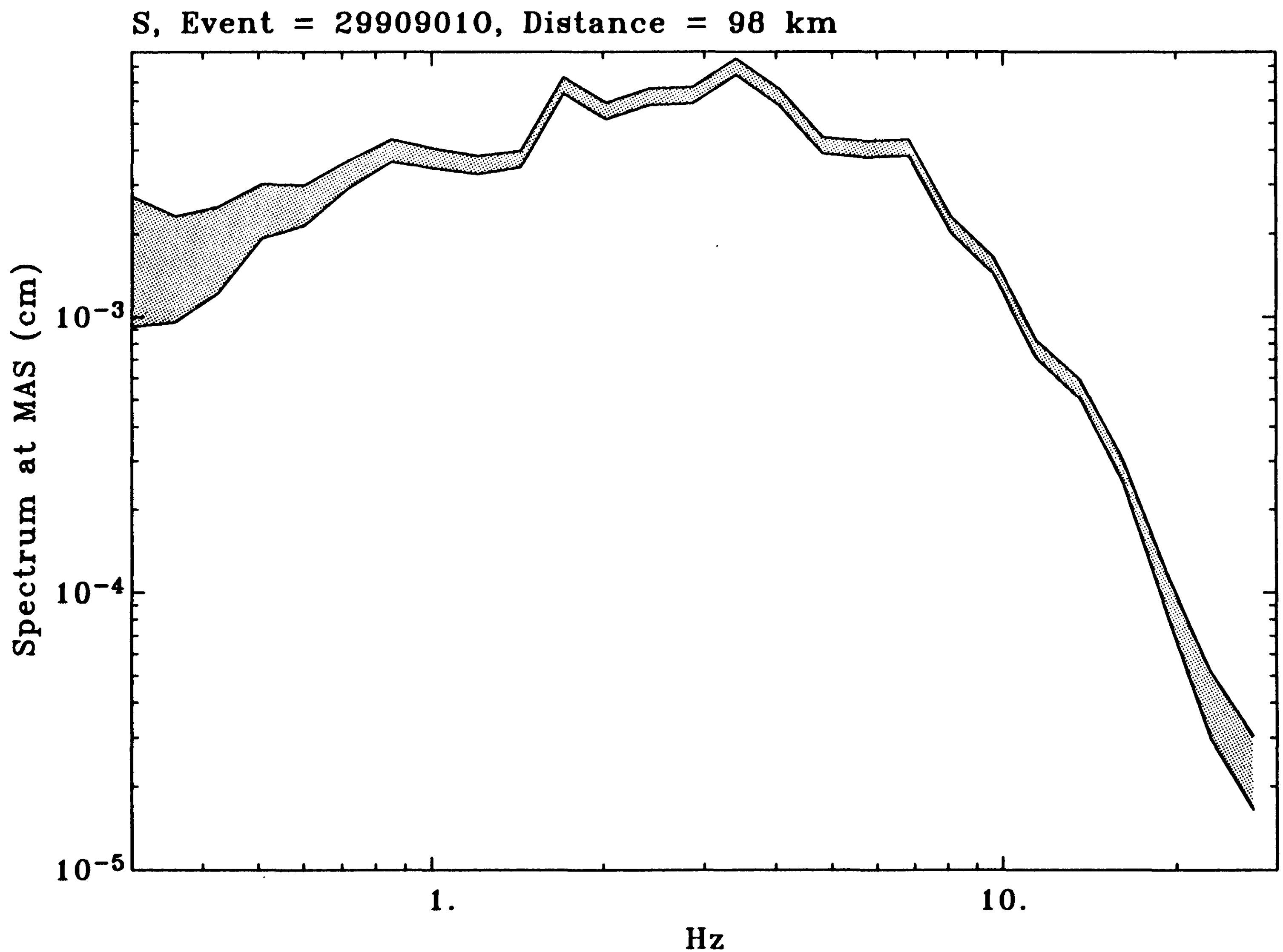
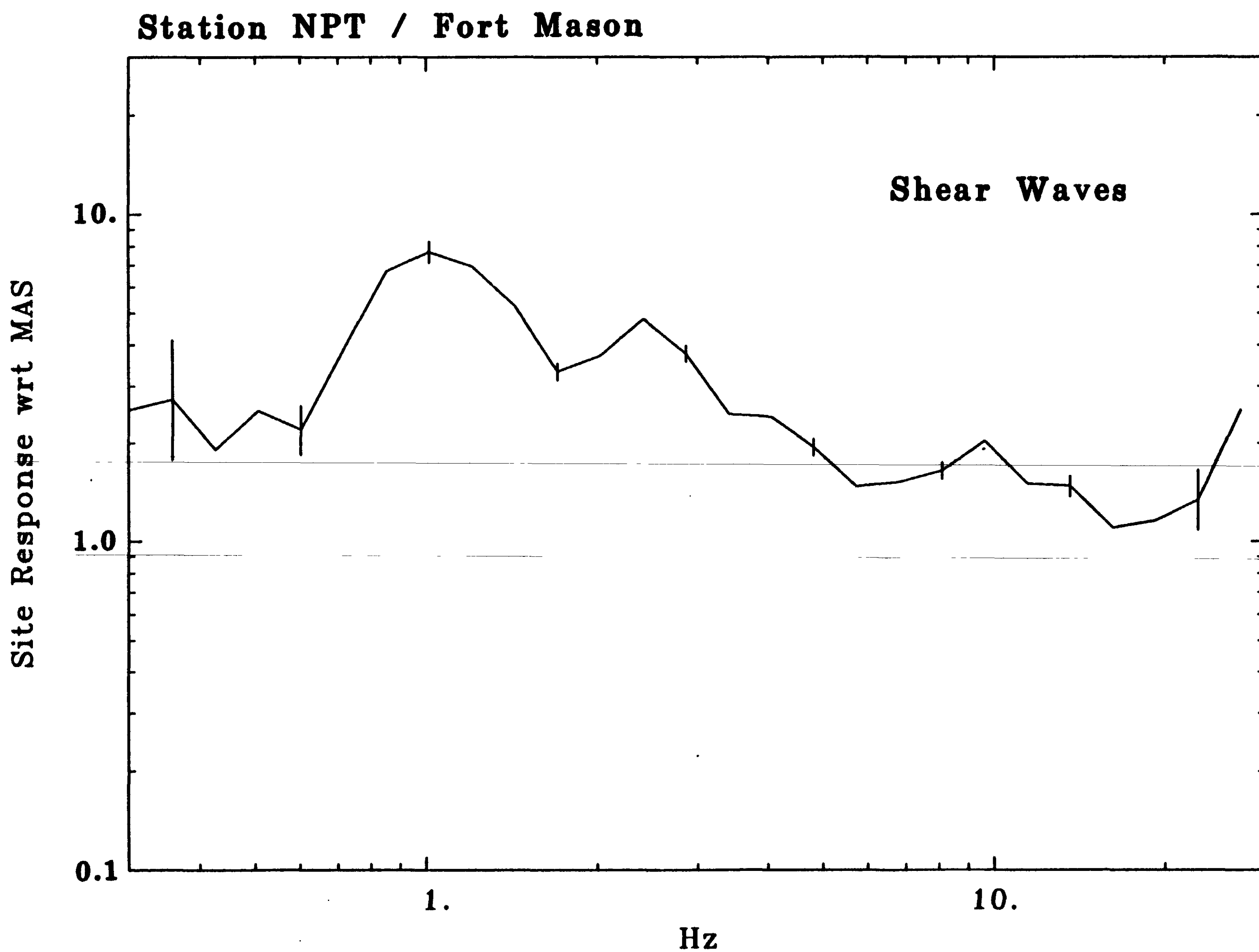
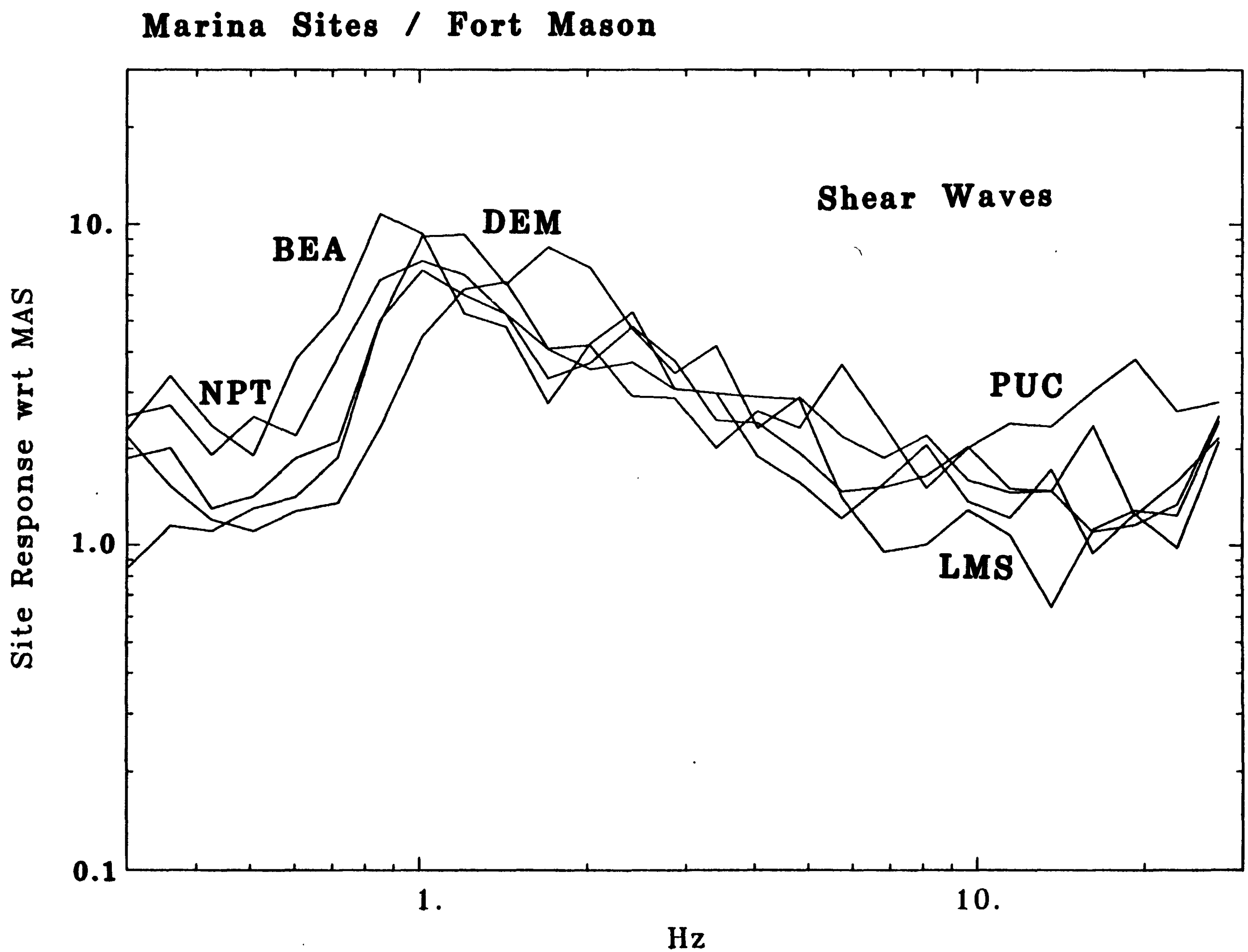

S, Event = 29909010, Distance = 98 km
Spectrum at MAS (cm)
10−3
10−4
10−5
1.
10.
Hz

Figure 4



Station NPT / Fort Mason
Shear Waves
Site Response wrt MAS
10.
1.0
0.1
1.
10.
Hz

Figure 5



Marina Sites / Fort Mason
Shear Waves
BEA
DEM
NPT
PUC
LMS
10.
1.0
0.1
1.
10.
Site Response wrt MAS
Hz

Figure 6

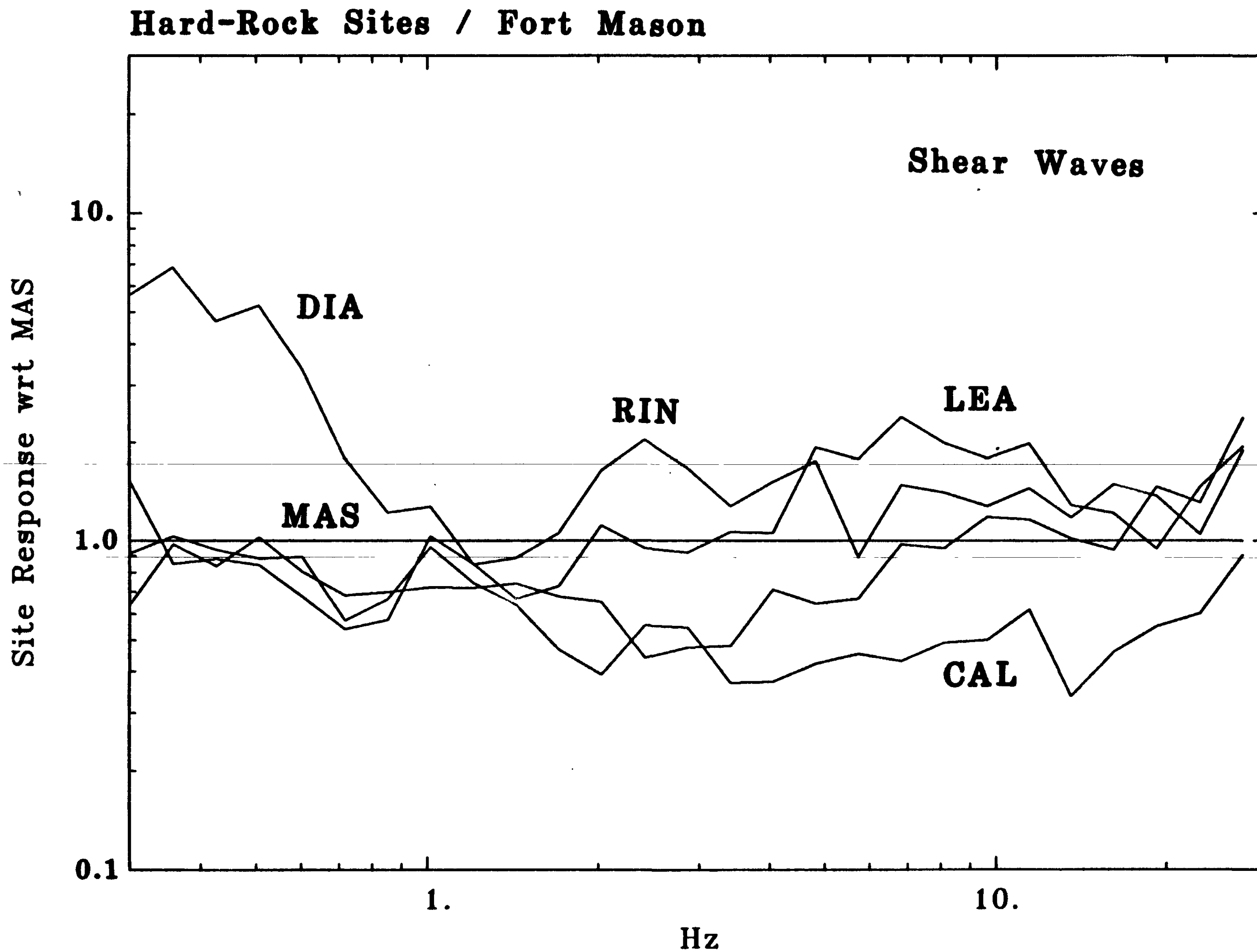

Hard-Rock Sites / Fort Mason
Shear Waves
DIA
RIN
LEA
MAS
CAL
10.
1.0
0.1
Site Response wrt MAS
1.
10.
Hz

Figure 7

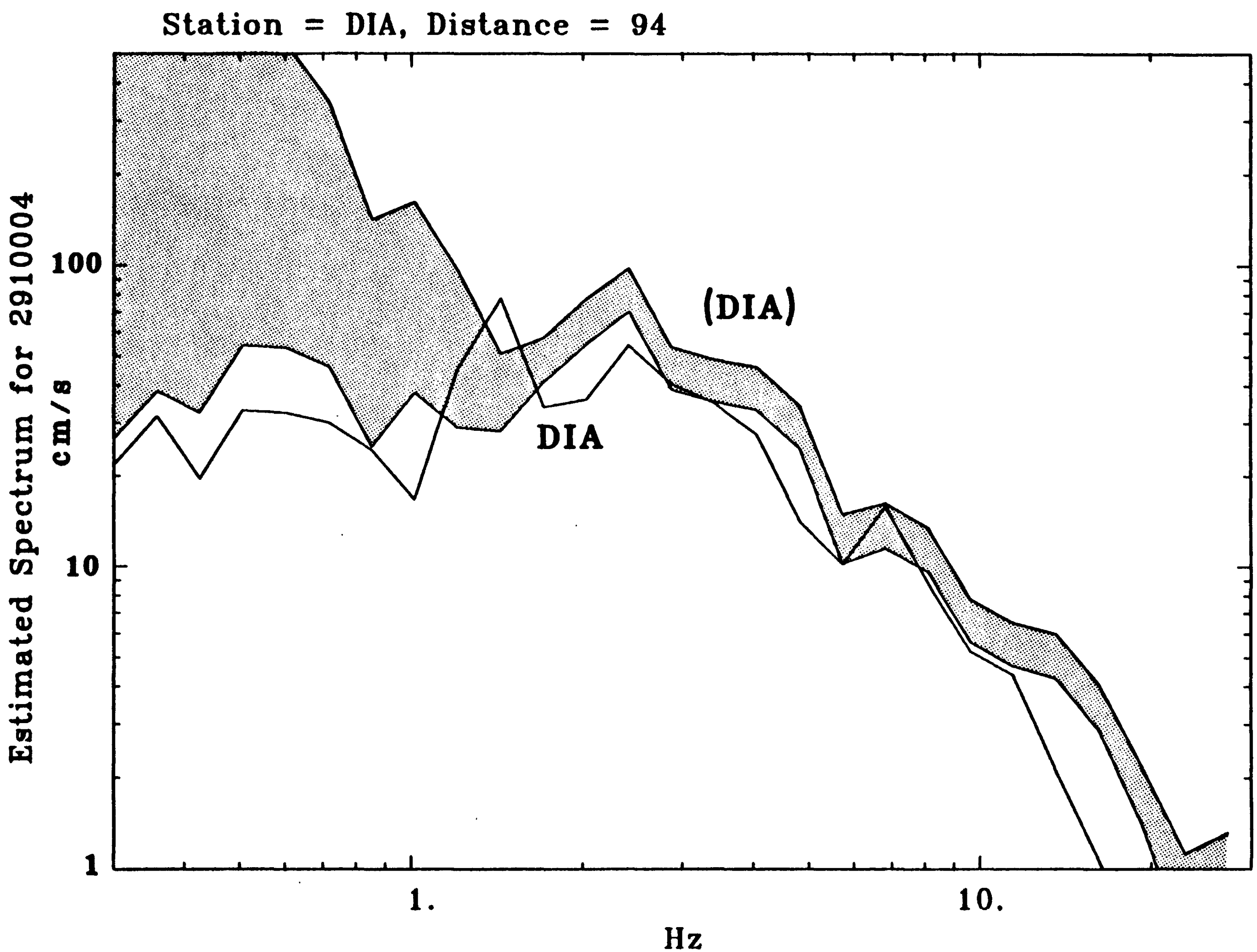
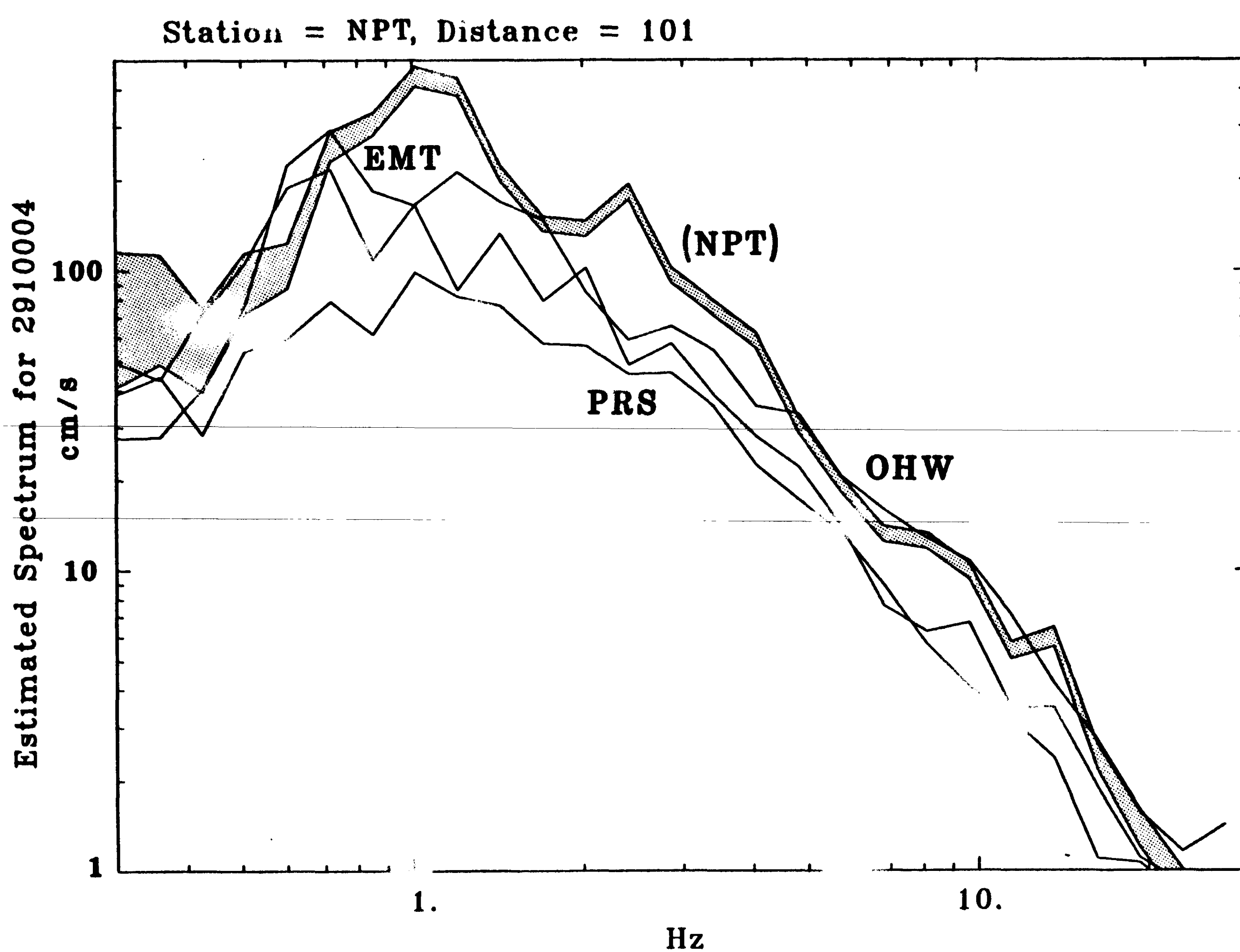

Station = DIA, Distance = 94
Estimated Spectrum for 2910004
cm/s
100
10
1
1.
10.
Hz
(DIA)
DIA

Figure 8



Station = NPT, Distance = 101
EMT
(NPT)
PRS
OHW
100
10
1
Estimated Spectrum for 2910004
cm/s
1.
10.
Hz

Figure 9

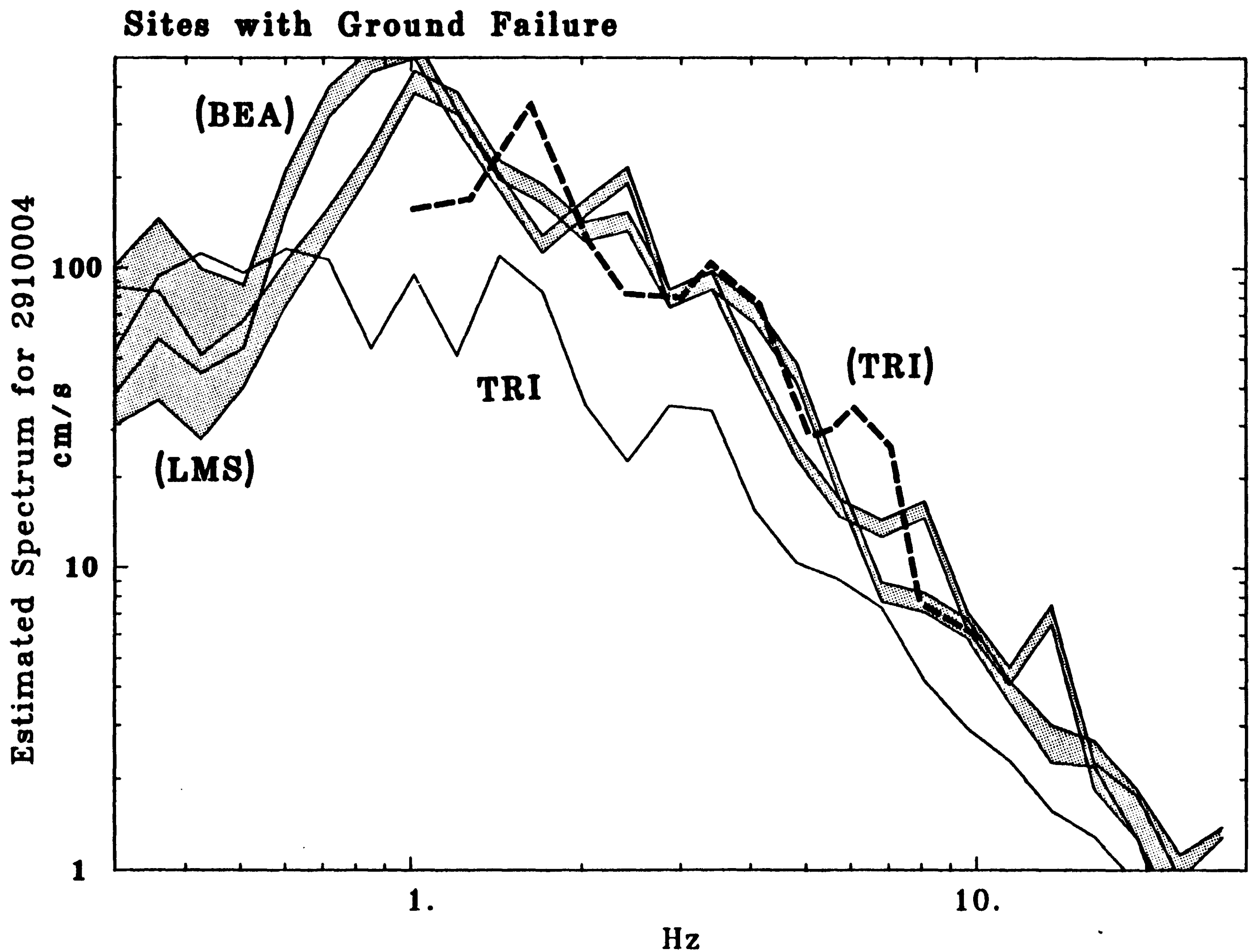

Sites with Ground Failure
Estimated Spectrum for 2910004
cm/s
100
10
1
1.
10.
Hz
(BEA)
(LMS)
TRI
(TRI)

Figure 10

ENGINEERING AND SEISMIC PROPERTIES OF THE SOIL COLUMN AT WINFIELD SCOTT SCHOOL, SAN FRANCISCO

Kayen, R.E., Liu, H.-P., Fumal, T.E., Westerlund, R.E., Warrick, R.E., Gibbs, J.F., Lee, H.J.

Introduction

The U.S.G.S. took a soil boring to a total depth of 91 meters at a site on the Winfield Scott School property at Beach and Divisadero Streets to investigate the effect of soil conditions on strong ground motion and liquefaction in the Marina District (Figure 1). The site was chosen because the locally heavy damage sustained to structures, pavement, and public works near the school. This chapter presents preliminary stratigraphic, soil engineering, physical property, and seismic velocity data, as determined from soil samples recovered from this boring as well as data from *in situ* geophysical logs. The hole itself will be used to establish a downhole accelerometer to investigate the effect of the soil column on seismic waves as they propagate to the earth's surface.

Borings were made near this site in 1912 during the planning stage of the Panama Pacific International Exhibition. These holes bottomed at depths between 7.9 and 10.4 meters in what was referred to as "Yellow hardpan" (ITTE, 1950). This layer was incorrectly interpreted to indicate bedrock (Schlocker, 1974). Our boring, however, shows that depth to bedrock at this site is 79.5 meters, approximately 8-to-10 times greater than previously thought. A consequence of the deep soil conditions at this site is an amplification of earthquake motions propagating upwards from bedrock to the ground surface.

Methods & Results

A descriptive log of the soil column at Winfield Scott School made during the continuous boring to bedrock was augmented by thirteen soil samples. Contract drillers began by coring through a patch of surficial

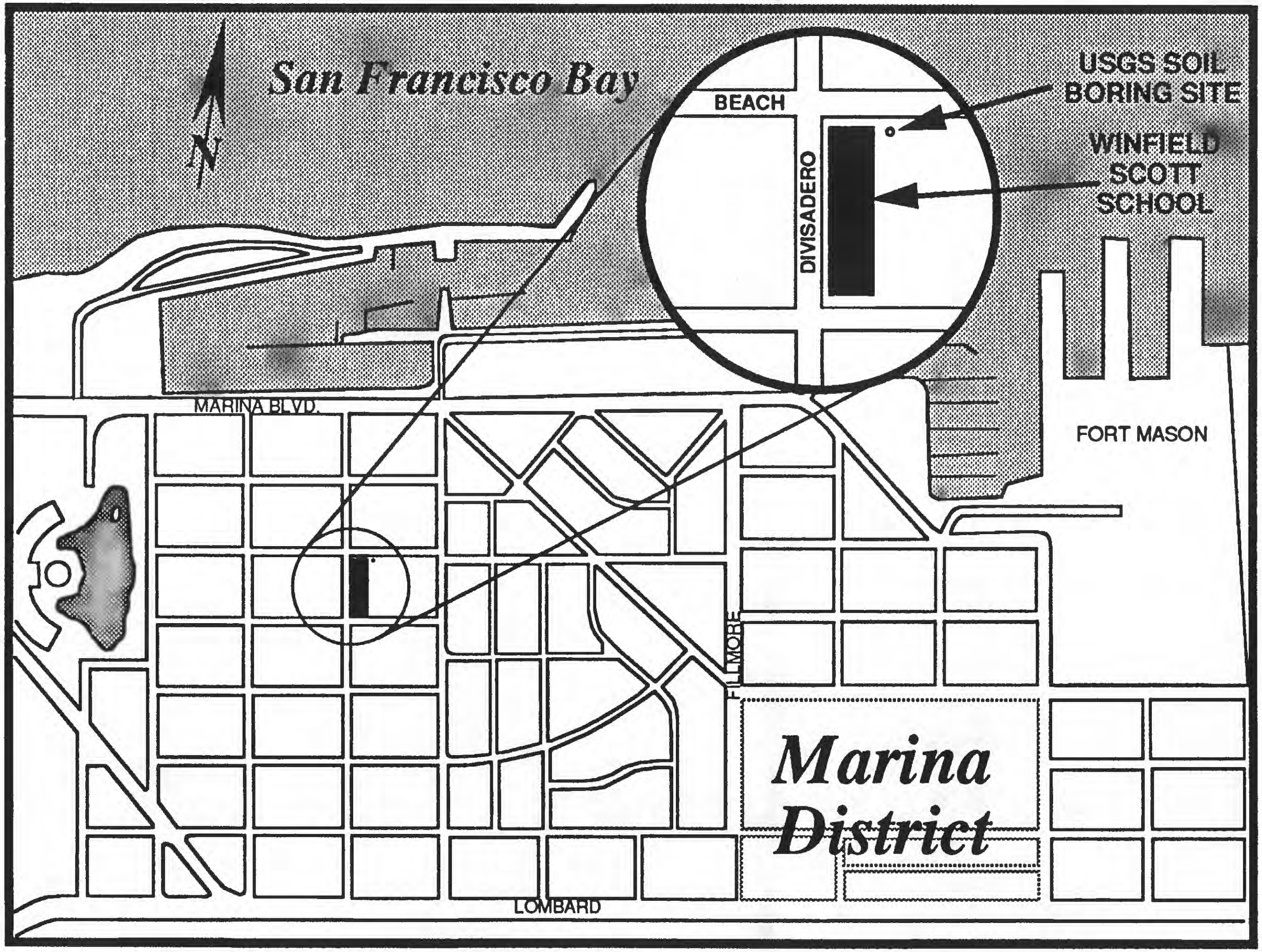


Figure 1. Location mapof Winfield Scott Scool and the USGS soil boring site.

asphalt on the playground near the Beach St. fence, twenty meters from the school building (Figure 1). Drilling continued through the upper layer of fill and reached bedrock (serpentine) at 79.5 meters. The hole was continued to a depth of 91 meters in order to place an accelerometer well within the bedrock. Samples were taken above 11.6 meters with unlined Shelby tubes which were pushed into the soil. Below this depth a Pitcher sampler was used.

The stratigraphic sequence found at the Winfield Scott School site is presented as Figure 2; descriptive detail is given in Table 1. Stratigraphic units were defined from field observations of soil cuttings combined with samples listed in Table 1.

The soil column at the USGS boring site consists of 4.3 meters of filled sand overlying another 3.5 meters of (natural?) sand deposits. Below is an interbedded sequence of clayey-sand and clay to a depth of 11.6 meters interpreted to be the base of Holocene Bay Mud. The "Yellow hardpan", noted in the 1912 boring logs, is actually a layer of dense sand lying between 11.6 and 22.9 meters and is characterized by a distinct yellow-brown color and high penetration resistance. The lower 57.9 meters of the soil column consists of stiff dark greenish gray to olive-gray Pleistocene Bay Mud which overlies sheared serpentine at 79.5 meters.

In order to preserve the samples for geotechnical study the ends were capped and taped to maintain the in-situ moisture condition. The samples were then stored at 4°C to preserve physical properties and minimize biological growth.

Laboratory testing of the samples included measurements of water content, bulk density, grain size, Atterberg limits, and vane shear strength. Field measurements of compressional wave and shear wave velocity were made using a downhole configuration (Warrick, R.E., 1974, Liu, *et al.*, 1988).

Water Content and Bulk Density

Water content was determined for samples below the fill and underlying sand deposits (7.6m) and are assumed to be 100% saturated (below water table) (Figure 2, Table 2). Water content, w, is the weight of water divided by the weight of the soil particles.

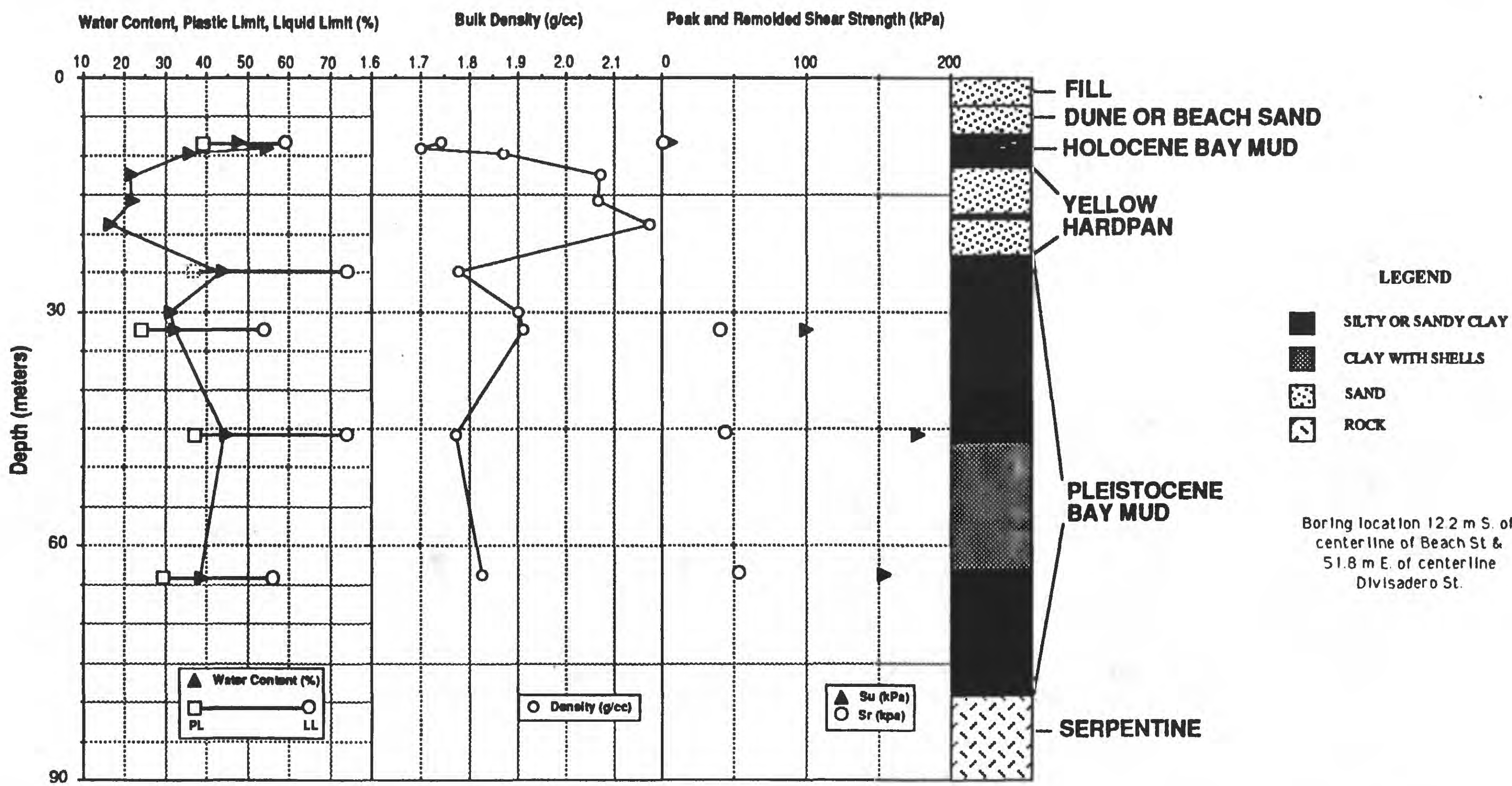


Figure 2. Stratigraphy, and physical- and engineering-properties of the soil column at Winfield Scott School.

UNIT	UNIT DEPTH (m)	DESCRIPTION	SUB-UNIT	SUB-UNIT DEPTH (m)	SUB-UNIT DESCRIPTION	SAMPLE	WATER CONT. (%)	SAMPLE DEPTH (m)
Unit 1	**0 - 4.3m**	**Hydraulic or Dumped Fill**	1.1	0-1.2	Dark grayish brown sand. Color: 10YR4/2			
			1.2	1.2-3	Olive gray fine sand with shells. Color 5Y3/2	WSS-1		1.5-2.5
			1.3	3-4	Olive gray fine sand, no shells. Color 5Y3/2	WSS-2		3.0-4.0
			1.4	4-4.3	Black fine sand, organic rich with wood fragments interpreted to be the base of the fill.			
Unit 2	**4.3 - 7.6**	**Natural sand deposits, possibly dune or beach.**	2.1	4.3-6.1	Dark yellowish brown sand, medium to coarse with some gravel at 5.5m Color: [illegible]/2	WSS-3		4.7-5.7
			2.2	6.1-7.6	Dark gray fine-to-medium sand. Color: 5Y4/1	WSS-4		6.1-7.1
Unit 3	**7.6 - 11.6**	**Interbedded sequence of fine sand and clay interpreted to be Holocene Bay Mud.**	3.1	7.6-8.5	Olive gray sandy clay with silt and fine sand lenses. Color: 5Y3/2.	WSS-5	47.7	7.6-8.6
			3.2	8.5-11.6	Dark greenish gray sandy clay . Color: 5GY4/1	WSS-6	54.7/35.6	9.1-10.1
Unit 4	**11.6 - 22.9**	**"Hardpan" layer, fine yellow-brown sand with interbeds of sandy clay .**	4.1	11.6-15.2	Yellow-brown sand, well sorted. Color: 10YR4/2	WSS-7	21.7	12.2-13.2
			4.2	15.2-16.1	Olive-brown silty sand. Color: 2.5YR4/4.	WSS-8	22.0	15.2-16.2
			4.3	16.1-18.3	Dark yellowish brown silty sand and sandy clay. Color: 10YR4/2.			
			4.4	18.3-22.9	Olive gray to grayish brown sand and clayey sand. Color 5Y4/1 to 2.5Y5/2	WSS-9	16.6	18.3-19.3
Unit 5	**22.9 - 79.5**	**Greenish gray Pleistocene Bay Mud . Occasional shell layers .**	5.1	22.9-24.4	Greenish-gray sandy clay 5G4/1			
			5.2	24.4-80.5	Stiff dark greenish-gray silty-clay with occasional layers of abundant shells. 5GY4/1	WSS-10 WSS-11 WSS-12 WSS-13	43.8 32.2 44.6 39.1	24.4-25.4 32.0-33.0 45.7-46.7 64.0-65.
Bedrock	**79.5**	**Olive gray Franciscan Serpentine**				WSS-14		82.3-82.7

Table 1. Soil and Sample Log for Winfield Scott School (Beach and Divisadero), Marina District of San Francisco

Sample	Unit	Depth (m)	Sand (%)	Silt (%)	Clay (%)	Water content (%)	Density (g/cc)	Peak vane shear strength, s_u (kPa)	Remolded vane shear strength, s_r (kpa)	Sensitivity	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
WSS-1	Fill	2.3	92	6	2								
WSS-2	Fill	3.8	98	2	0								
WSS-3	Natural sand	5.3	99	1	0								
WSS-4	Natural sand	6.9	95	4	1								
WSS-5	Holocene Bay Mud	8.4	0	67	33	47.7	1.74	6.4	1.0	6.4	59	39	30
WSS-6	Holocene Bay Mud	9.1				54.7	1.70						
WSS-6	Holocene Bay Mud	9.9	67	21	12	35.6	1.87						
WSS-7	Dense Sand	13.1	96	3	1	21.7	2.07						
WSS-8	Dense Sand	16.0	74	20	6	22.0	2.07						
WSS-9	Dense Sand	19.1	99	1	0	16.6	2.17						
WSS-10	Pleistocene Bay Mud	25.2	1	52	47	43.8	1.78				74	37	37
WSS-11	Pleistocene Bay Mud	32.8	1	48	51	32.2	1.91	100.0	40.0	2.5	54	24	30
WSS-12	Pleistocene Bay Mud	46.5	1	41	58	44.6	1.77	177.0	44.0	4.0	75	37	38
WSS-13	Pleistocene Bay Mud	64.8	0	53	47	39.1	1.83	154.0	53.0	2.9	56	29	27
WSS-14	Serpentine	82.3											

Table 2. Soil Properties of the USGS soil boring, Winfield Scott School.

At the top of the Holocene mud (Unit 3), the silty clay layer between 7.6 and 8.5 meters has a natural water content of approximately 48%. The sand and silty clay layers which comprise the remainder of the Holocene Bay Mud, however, have water contents ranging from 35.6%-to-54.7%. The hardpan layer (between 11.6 and 22.9 meters) is particularly dense, as shown by the very low water contents (16.6%-22%). Water content in the Pleistocene Bay Mud between 22.9 and 79.5 meters ranges from 32%-to-44%, characteristic of stiff fine grained soils..

Bulk density (g), is determined both from the known weights and volumes of the soil boring sub-samples, as well from water content data assuming 100% saturation (Figure 2). The measured bulk densities ranged from 1.69 g/cc, typical of the Holocene Bay Mud to 2.17 g/cc' typical of the dense "Hardpan" layer.

Grain Size distribution

Grain size distributions (Figure 3) were determined as percent sand, silt, clay using wet sieve and pipette methods (Carver,1971). Both the fill which was emplaced prior to the Panama Pacific International Exhibition and the underlying natural sand deposits have the highest sand content, typically in excess of 90% (Figure 3, Table 2, and Bennett, this volume). The Holocene Bay Mud consists of interbedded units of clay, silt, and fine sand. The hardpan layer consists of interbedded units of clay, sand, to silty-sand. Below, the Pleistocene Bay Mud is comprised almost entirely of silt-to-clay sized particles.

Atterberg Limits

Soils typically exist within three possible states: semi-solid (brittle), plastic, and liquid (Lambe and Whitman, 1969). Atterberg limits are standard though arbitrary boundaries defining these states, and are expressed in terms of percent water content. The liquid limit (LL), the boundary between the liquid and solid states, was determined by the Casagrande drop-cup method using an ASTM groove tool (ASTM Standard: D4318-84, 1987). The plastic limit (PL), the boundary between the plastic and

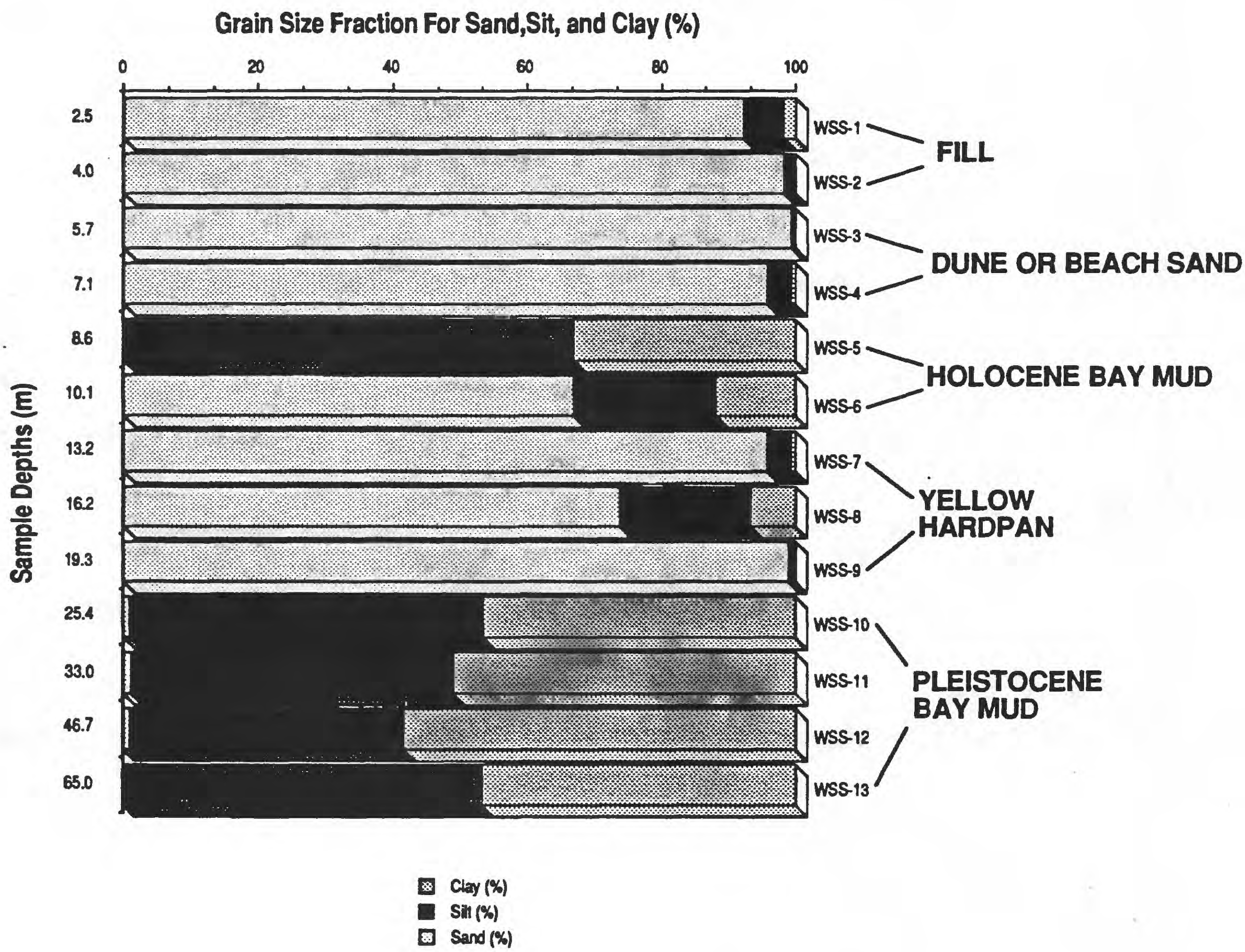


Figure 3. Grain size distribution at sampled intervals

semi-solid states, was determined by the rolled soil thread method (ASTM Standard: D4318-84, 1987). The span of water contents through which soil behaves plastically is defined as the Plasticity Index (PI), the difference between the Liquid Limit and the Plastic Limit.

The Atterberg limits for cohesive soils are shown both in Figure 2 with respect to *in situ* water content, and Figure 4. Both the cohesive Holocene Bay Mud at 8.3 meters and the entire 57.6 meter-thick sequence of Pleistocene Bay Mud are highly plastic soils, positioned mostly below or on the "A" line. This line divides inorganic silts and organic clays from inorganic clays. The soils are designated as inorganic silt mixed with inorganic clay (MH to CH) according to the Unified Soil Classification System. Atterberg limits are presented in Table 2.

Shear Strength

A laboratory vane shear apparatus was used on the boring samples to estimate the in-situ undrained shear strength of the cohesive soil layers. We inserted a four-bladed laboratory vane (1.27 by 1.27 cm), approximately 1.5 cm beneath the soil surface. By rotating a spring of known stiffness which is attached to the vane at 90°/minute, shear stresses were imparted on the sediment until large strain deformation was achieved. Peak torque applied to the spring was measured and used to calculate the undrained shear strength, s_u (ASTM standard: D2573-72, 1987, Figure 2). After initially shearing the sample, the vane was rotated five revolutions and a second measure was made of the residual strength mobilized at high strains.

Dividing the peak shear strength by the effective overburden pressure gives normalized strength values of approximately 0.4 to 0.6 for the Pleistocene Bay Mud. Such low normalized values indicate that these muds are normally-consolidated to slightly over-consolidated. Sensitivities for both Pleistocene and Holocene Bay Mud are moderate, and are typical of San Francisco Bay Mud.

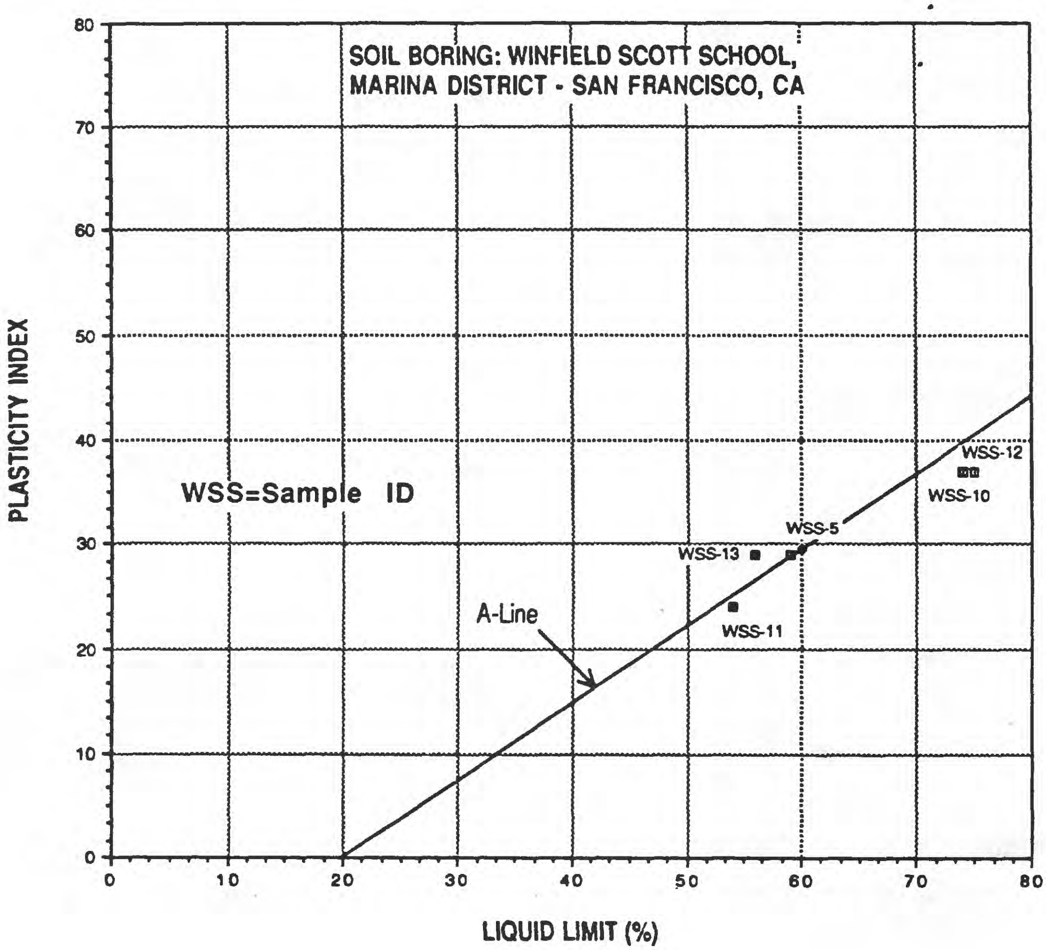


Figure 4. Atterberg Limits determination for cohesive samples.

Seismic Wave Velocities

The seismic P- and S-wave velocities were determined using the downhole method. An air-powered, impulsive, and horizontally polarized (SH)-wave source (Liu et *al.*, 1988) placed at the playground surface generated the shear waves, whereas the vertical impact of a sledge hammer on a steel plate generated the compressional waves. The source offset was 1.88 m and 2.23 m from the center of the hole for the SH- and P-waves, respectively. A Mark Products L-28LT-3DS 8-Hz three component geophone, mounted on a borehole locking device, was used as the downhole sensor.

Data were taken starting at a depth of 1.66 m and then at 0.91 m intervals to a maximum depth of 28.18 m. An ES&G ES-1200 digital seismograph, triggered by an impact switch, was used to record the waveforms at a sampling rate of 1024 samples/channel/s. Travel times were determined for the first arrival and the first downward velocity maximum of the compressional waveforms; travel times were determined for the first S-wave arrival and the following two extrema in the recorded horizontal-component motions. The results are shown in Figure 5. The velocities, inferred from the travel time data, are shown in Figure 6.

The p-wave velocity has a constant value through the Pleistocene Bay Mud at about 1740 m/sec and increases to 3000 m/sec in the serpentine bedrock. The *in situ* shear-wave velocities of the artificially filled sand, natural sand deposits, and Holocene Bay Mud are 130 m/s, 175 m/s, and 145 m/s, respectively. The shear-wave velocity of the dense sand varies between 290 m/s and 455 m/s; the shear-wave velocity determined for the Pleistocene Bay Mud is 265 m/s. Shear wave arrivals travelling through the deposits below 28 meters were masked by the high amplitude arrivals travelling down the casing or borehole. We will conduct further experiments to try and determine the shear wave velocity below this depth.

Discussion

The USGS soil boring at Winfield Scott School discovered dramatically different soil conditions from those interpreted from the pre-Panama Pacific

WINFIELD SCOTT SCHOOL

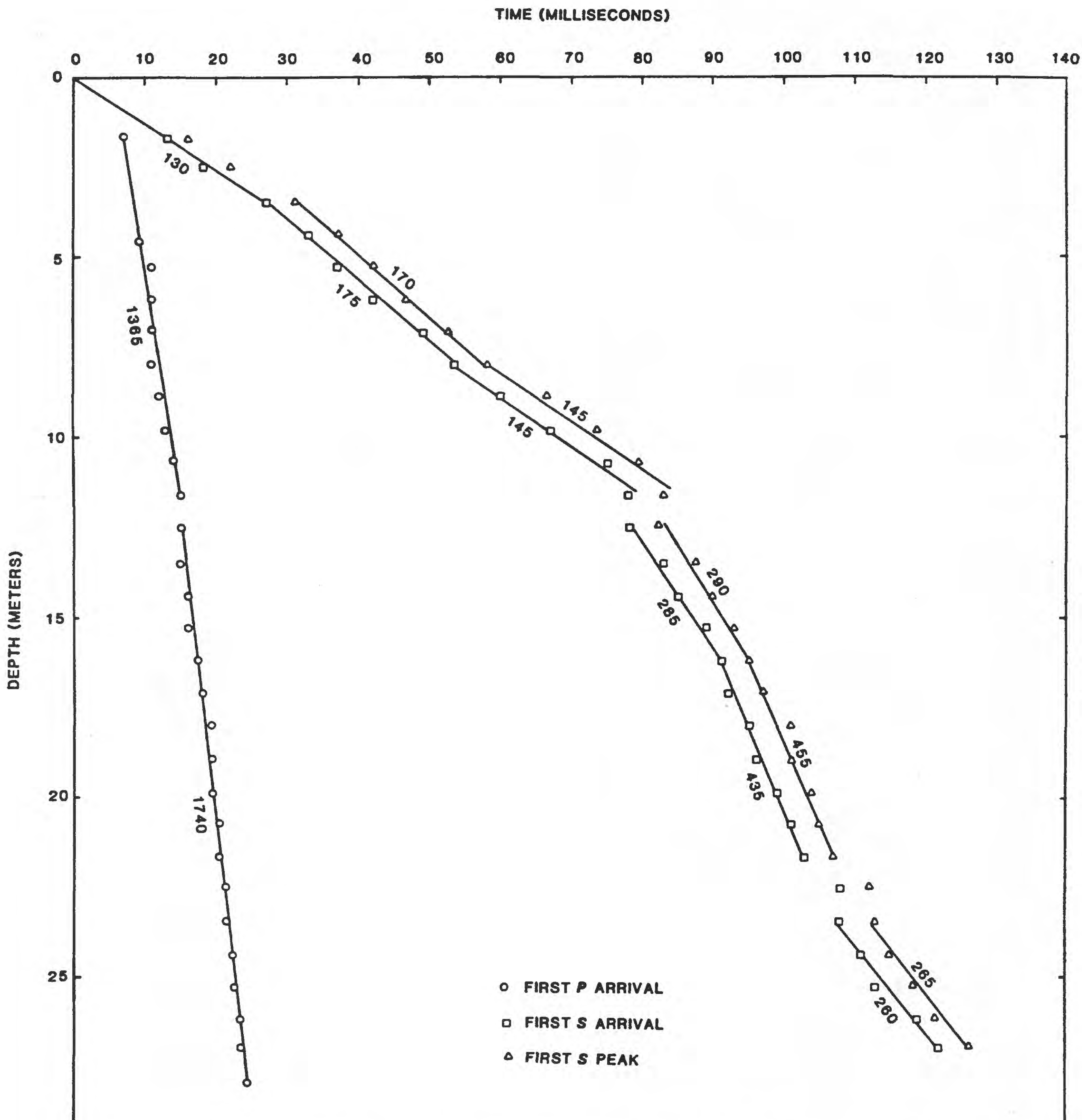


Figure 5. Seismic shear wave and compression wave travel times versus depth.

WINFIELD SCOTT SCHOOL

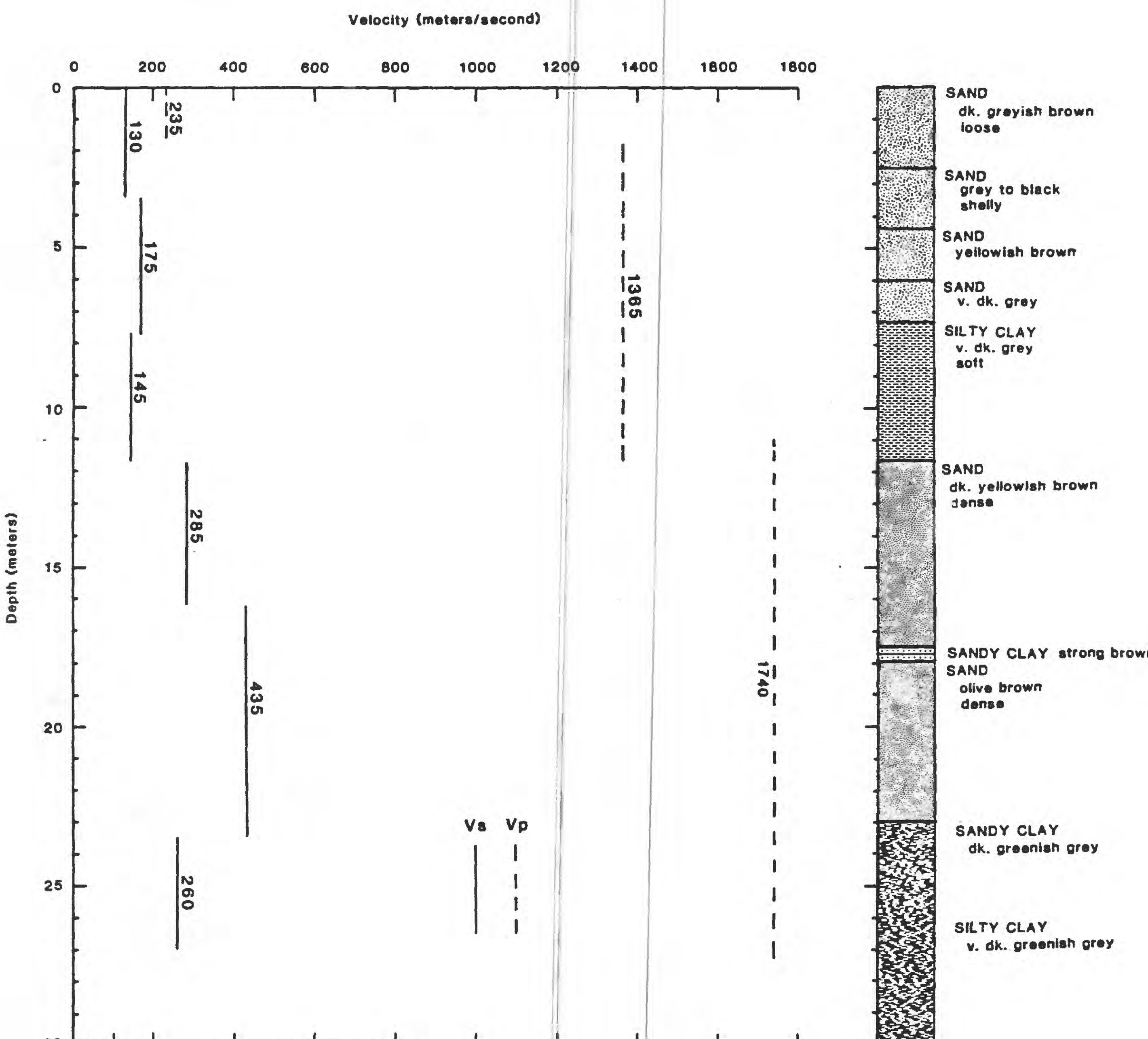


Figure 6. Shear wave and compression wave velocity versus depth (meters/sec).

Exhibition soil logs (Schlocker, 1974). Unfortunately, the 1912 soil logs led to the mistaken belief that the depth to bedrock ("Yellow hardpan") in the Marina District west of Fillmore St. was typically less than 12 meters. In contrast,this post-earthquake USGS soil boring identified a 57.6 meter-thick section of Pleistocene Bay Mud beneath the "Hardpan" layer.

The existence of a thick sequence of soft soils above bedrock has an impact on the site response to earthquake motions. For example, Seed and Idriss (1982) present curves for peak velocity versus distance from the zone of energy release for soil and rock sites for M=6.5 earthquakes (Figure 7). Their data shows that peak velocity above deep soil is typically amplified by a factor of two or three compared with rock sites. Velocity measurements recorded during after-shocks indicate that similar velocity amplifications occurred in the Marina district (see Boatwright, *et al.*, this volume, Figure 2).

A second measure of site response that is influenced by a thick section of soft soil is the acceleration response spectrum, the peak acceleration measured at the roof of single-degree-of-freedom structures for a continuum of natural resonant periods. When acceleration response spectra for deep soil sites are compared with spectra for nearby rock sites it can be seen there is often a dramatically increased acceleration for structures with dominant periods greater than 0.3 seconds (Figure 8). That is, on deep soil sites accelerations are likely to be amplified in multi-story structures whose dominant resonant periods are greater than 0.3 seconds. It is possible that the deep soil conditions amplified lower frequency motions in the Marina District which adversely effected the taller 3 and 4 storey structures.

Conclusions

- The depth to bedrock at Beach and Divisadero Streets is approximately 79.5 meters, eight-to-ten times the depth reported in soil boring logs of the Panama Pacific International Exhibition.

- The soil column below Winfield Scott School is composed of 5 primary units; 1) fill, placed prior to the Panama Pacific International

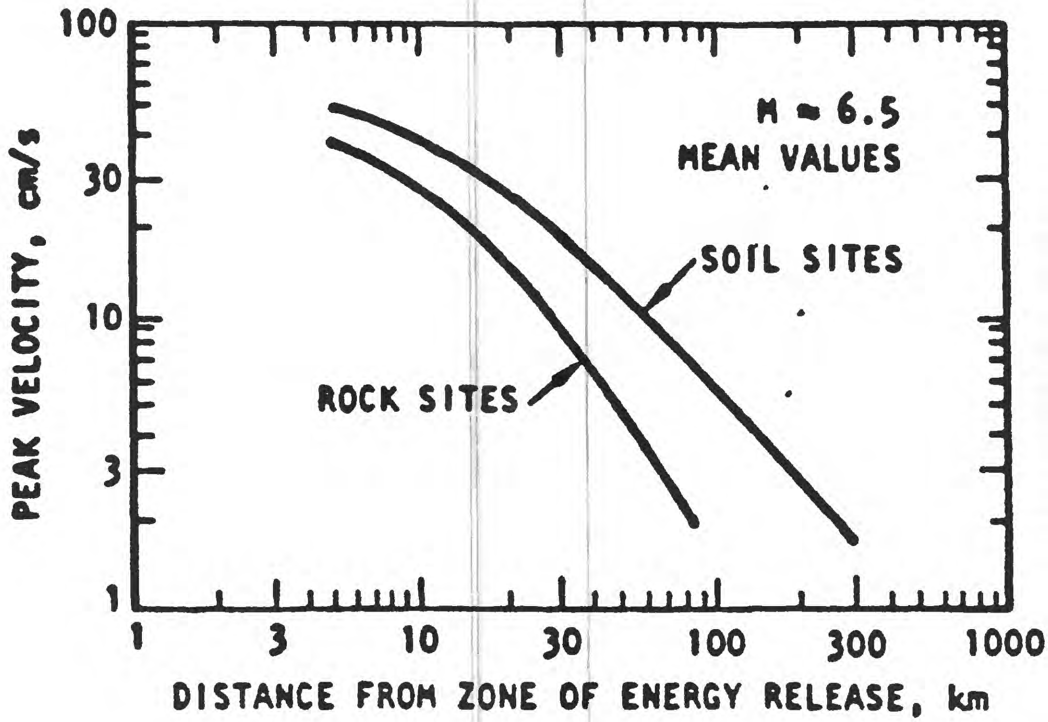


Figure 7. Peak velocity versus distance from zone of energy release (from Seed and Idriss, 1982).

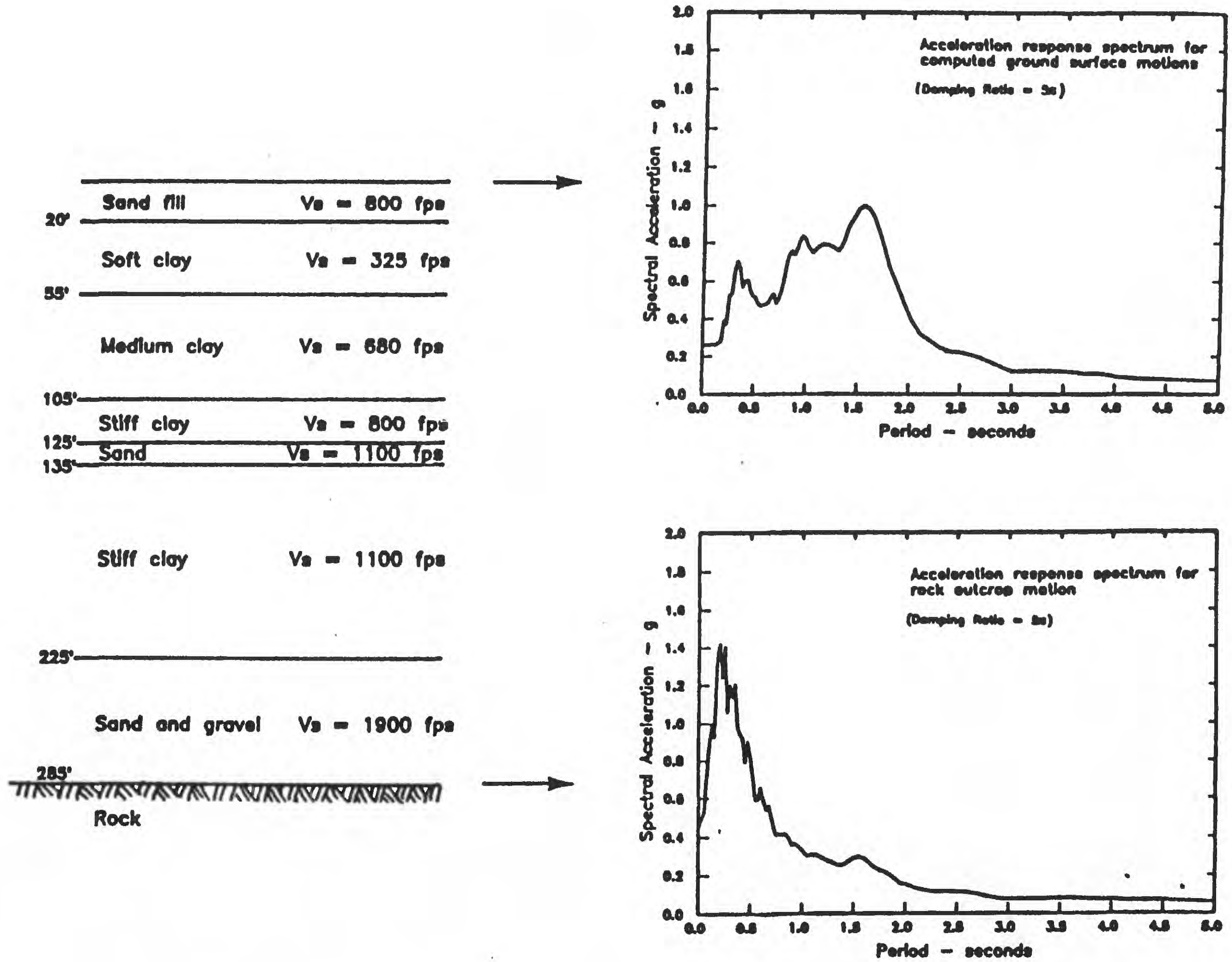


Figure 8. Predicted acceleration response spectra for the Southern Pacific Building (foot of Market Street) for a magnitude 7-1/4 earthquakewithsoildepth similar to the Marina District (from Seed and Sun, 1989). The incident rock motion entering at the base is filtered and amplified through the soil for natural periods of vibration in excess of 0.3 seconds. As a consequence of amplification through soil, structures with dominant periods in excess of 0.3 seconds will likely experience greater accelerations during earthquakes than nearby structures of similar design founded on rock.

Exhibition, between 0-4.3m; 2) natural(?) dune or beach sand between 4.3-7.6m; 3) Holocene Bay Mud and sand between 7.6-11.6m; 4) dense yellow "hardpan" between 11.6-22.9m; and 5) Pleistocene Bay Mud between 22.9-79.5m.

- Seismic shear wave and compression wave velocities as well as densities for the soil section at the USGS site are significantly lower than for the underlying Franciscan formation. The strong impedance contrast that exists between relatively dense, rigid bedrock and the softer sediment above typically results in amplification of ground motions at the surface.

Acknowledgments

We thank Mr. Thomas Sammon of the San Francisco Unified School District, who facilitated our drilling and logging operations at the Winfield Scott School; Brian Edwards and Tahir Masood who reviewed the manuscipt; and Willie McArthur who did part of the geotechnical laboratory work.

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