FINAL REPORT
GEOTECHNICAL INVESTIGATION
Marina Boulevard Seawall
San Francisco, California

City and County of San Francisco
San Francisco, California

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Project No. 1526.02

Treadwell&Rollo
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1.0 INTRODUCTION

This report presents the results of the investigation performed by Treadwell & Rollo, Inc. (T&R) to evaluate the potential for liquefaction[1] and lateral spreading[2] to occur behind the Marina Boulevard seawall in San Francisco, California. The seawall runs along the northern side of Marina Boulevard between Scott and Baker Streets, as shown on Figure 1. T&R previously performed a geotechnical study for the seawall, the results of which were presented in a draft report dated 26 January 1994.

Our services were provided in accordance with the City and County of San Francisco, Department of Public Works (DPW) Order No. 169,333 (approved 20 February 1996) and our proposal dated 18 January 1996. Our services for this specific assignment were authorized in a DPW Service Order dated 15 February 1996.

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[1] Liquefaction is a phenomenon in which saturated, cohesionless soil experiences a temporary loss of strength because of the buildup of excess pore water pressure, especially during cyclic loading such as that induced by earthquakes. Soil most susceptible to liquefaction is loose, clean, saturated, uniformly graded, fine-grained sand or silt of low plasticity that is relatively free of clay. Sandy gravel of low relative density is also susceptible to liquefaction.

[2] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

2.0 BACKGROUND

Constructed around 1934, the seawall is approximately 1,100 feet long and consists of a nine-foot-high cantilever concrete wall with basalt rock facing. The wall is supported on vertical and battered, composite concrete and wood piles that reportedly extend 48 to 62 feet below the pile cap. At the base of the seawall extending towards the yacht harbor, there is a gentle slope. The Marina Boulevard box sewer parallels the seawall near the northern curbline of Marina Boulevard. The box sewer is a high capacity, gravity flow, consolidating sewer designed to collect the dry and wet weather flows from all the existing outfall sewers in the northshore area. The plan dimensions of the box sewer increase from west to east, varying from 11 feet wide by 12 feet high at Baker Street to approximately 21 feet wide by 23 feet high at Buchanan Street.

A report entitled "Final Report, Liquefaction Study, Marina District and Sullivan Marsh Area, San Francisco, California" dated August 1991, by Harding Lawson Associates, et. al., was prepared for the City and County of San Francisco, Department of Public Works to evaluate and recommend measures to minimize damage to public facilities during a Richter magnitude 8.3 earthquake. The report concludes the Marina Boulevard seawall may move significantly toward the yacht harbor during this magnitude earthquake and the ground surface between the seawall and sewer may settle because of lateral spreading. In the authors' judgment, the box sewer would be able to retain the soil south of Marina Boulevard; however, movement of the box sewer could occur. Therefore, it was recommended that the seawall be strengthened and the ground improved between the seawall and the box sewer. Preliminary cost estimates for the installation of tensile piles along the seawall and the installation of stone columns between the seawall and box sewer were presented in the report.

A committee was appointed by the City and County of San Francisco Board of Supervisors to review the above-referenced Liquefaction Study report. In a report entitled "Final Report, Marina District Liquefaction Study Task Force, San Francisco, California" dated May 1992, the committee recommended the Marina Boulevard seawall be strengthened and the ground between the seawall and the box sewer be improved in accordance with the recommendations provided in the Liquefaction Study report. The committee estimated the cost of strengthening the seawall to be between $300,000 and $350,000 and the cost of improving the ground by installing stone columns to be $250,000.

The DPW subsequently performed a detailed cost analysis of the proposed seawall strengthening and found the estimated costs to be considerably higher than those presented in both the Liquefaction Study and task force reports. DPW decided to re-evaluate the risk of earthquake-induced damage to the box sewer and seawall and the mitigation measures proposed by the Liquefaction Study. We subsequently performed a geotechnical study using subsurface data from previous investigations by others. In the study, we concluded there is potential for several feet of horizontal ground movement to occur behind the seawall during a repeat of the 1906 Earthquake, and both the seawall and box sewer behind the seawall would likely experience damage from the lateral spreading.

3.0 SCOPE OF SERVICES

The objectives of this study were to better characterize the liquefiable deposit in the seawall vicinity and to confirm the conclusions in the previous studies. As outlined in our proposal dated 18 January 1996, our scope of services included drilling seven test borings, performing compression and shear wave velocity measurements in three of the test borings, performing laboratory tests on selected soil samples obtained from the test borings, and performing five cone penetrometer tests (CPTs). On the basis of the field and laboratory test results and engineering analyses, we developed conclusions regarding:

- subsurface conditions along the length of the seawall using data from previous investigations, as well as data from this investigation
- the potential for liquefaction and lateral spreading of the fill behind the seawall (on both sides of the box sewer), including the lateral extent of the liquefiable soil to the west and an estimate of the potential amount of lateral spreading
- the potential for deep-seated rotational failure of the outboard slope through the Bay Mud below the liquefiable soil
- the presence of the former Fair's seawall and its effect on lateral spreading
- the potential risk to the box sewer and to the seawall due to liquefaction and lateral spreading

This report also presents alternative mitigation measures to reduce the potential for this lateral spreading.

4.0 HISTORICAL DEVELOPMENT

Much of the Marina District is composed of land that was once submerged and was reclaimed from the Bay by filling between 1851 and 1917. The areas underlain by artificial fill are shown on Figure 2. These areas were determined by superimposing historic maps prepared by the U.S. Coast and Geodetic Survey and the State Harbor Commission. These maps show the Bay's shoreline extending roughly from Blackpoint at Fort Mason southwest to what is now the intersection of Lombard and Divisadero Streets, then turning northwest and following the present alignment of Richardson Avenue. Along the western boundary of the shoreline was a broad area of beach sand, known as Strawberry Island. The small bay formed by the historic shoreline is referred to as Marina Cove.

To retain the fill placed in Marina Cove, Fair's seawall was built in the 1890s. The seawall, which was completed in 1894, was built by dumping rock from a pile-supported trestle. The approximate alignment of Fair's seawall is shown on Figures 1 and 2. Dune sand, excavated and transported from outside the western boundaries of the Marina District, was dumped adjacent to the seawall; however, filling of the cove was not completed. By 1906, Marina Cove was enclosed, except for a narrow opening to the north.

In 1911, the area of the Marina District was chosen as the site of the 1915 Panama-Pacific International Exposition. To create developable land for the Exposition, the filling of Marina Cove resumed. This fill was largely "hydraulic fill" composed primarily of sand and silty sand. The hydraulic fill was dredged from the Bay and pumped into place without any attempts to densify the material.

Many wood piles were driven to support the Exposition buildings. The pile lengths reportedly varied between 13 and 63 feet. After the Exposition, most of the structures were demolished with dynamite. Records indicate many of the wood piles were cut off a few feet below grade and left in place. Following demolition of the Exposition buildings, low areas were filled by public dumping or mud pumping. In 1924, the land was subdivided into individual lots. The Marina District was subsequently developed during the 1930s.

5.0 FIELD INVESTIGATION

To investigate and sample the soil beneath the seawall alignment, a subsurface investigation was performed on 1 through 4 and 17 and 19 April 1996. The field investigation included drilling test borings, performing cone penetrometer tests and measuring compression and shear wave velocities of the fill soil. Details of the field investigation are discussed in the remainder of this section of the report.

5.1 Test Borings

For this investigation, we drilled seven test borings to depths ranging from about 20 to 40 feet below the ground surface at the locations shown on Figure 3. The test borings were advanced using rotary-wash drilling equipment. Boring locations were chosen to supplement information from previous investigations. Our field engineer logged the soil encountered and obtained samples for laboratory testing. Logs of the borings are presented in Appendix A as Figures A-1 through A-12.

Soil samples were obtained using three samplers:

- Standard Penetration Test (SPT) split-barrel sampler without liners (1.375-inch inside diameter [ID] and 2-inch outside diameter [OD])
- Sprague & Henwood (S&H) split-barrel sampler (2.43-inch ID, 3-inch OD with liners)
- Shelby (thin-walled tubes, 2.8-inch OD, 0.065-inch wall thickness).

The split-barrel samplers were driven with a 140-pound, above-ground safety hammer falling 30 inches. The blow counts required to drive the S&H sampler the final 12 inches of an 18-inch drive were converted to SPT N-values and are presented on the boring logs. Where the SPT sampler was used, the actual blow counts are shown on the logs. Samples of soft soil were obtained by hydraulically pushing 30-inch-long Shelby tubes into the soil. The soil encountered in our borings is classified in accordance with the soil classification system described on Figure A-13.

All borings, except B-4, B-6 and B-7 were backfilled with cement-bentonite grout as required by the City and County of San Francisco, Bureau of Environmental Health Management. Solid, Schedule 40 polyvinyl chloride (PVC) casing, three inches in diameter (ID), was placed in borings B-4 , B-6 and B-7 to depths varying from about 25 to 30 feet below the existing ground surface. Both the bottom and top of the casing were capped. The annulus around the casing was filled with Monterey No. 3 sand. The cased holes were subsequently used for geophysical testing. After the completion of the geophysical testing, the inside of the PVC casing was filled with cement-bentonite grout.

5.2 Cone Penetrometer Tests

The CPTs were performed by hydraulically pushing a 1.4-inch diameter, cone-tipped probe into the ground. The cone on the end of the probe measures tip resistance, and a sleeve behind the cone tip measures frictional resistance. A small, porous stone between the cone and the friction sleeve monitors pore pressures in the soil during penetration. Electrical gauges within the cone measure soil parameters continuously during the entire depth of each probing. Soil data, including tip resistance, frictional resistance, pore water pressure, probe inclination, and surrounding temperature were transferred to a computer during each test. Accumulated data was processed by computer to provide engineering information, such as the types and approximate strength characteristics of the soils encountered.

The CPTs were advanced to depths of about 8 to 40 feet below the ground surface. The CPT logs, which show tip resistance, local friction and friction ratios, as well as an interpreted soil profile, are presented in Appendix B.

5.3 Geophysical Study

A geophysical study was also performed to measure the compression wave (P-wave) and shear wave (S-wave) velocities of the fill materials behind the seawall. The downhole seismic surveys were conducted at test borings B-4, B-6 and B-7. The seismic velocity data was collected, as described and illustrated in the report prepared by NORCAL Geophysical Consultants, Inc., dated 14 May 1996. NORCAL's report is attached as Appendix C. A summary of the interpreted P- and S-wave velocities are summarized in Table 1.

TABLE 1 - SUMMARY OF DOWNHOLE SEISMIC SURVEYS

Test Boring	Depth (feet)	P-wave velocity (feet/second)	S-wave velocity (feet/second)
TB-4	0 - 23	1,650	550
TB-6	0 - 5	950	450
	5 - 15	2,300	650
	15 -23	2,300	850
TB-7	0 - 5	1,600	750
	5 - 25	3,000	750

6.0 LABORATORY TESTING

Representative soil samples were selected for laboratory testing. Samples were tested to determine moisture content, dry density, grain size distribution, and triaxial shear strengths. The laboratory test results are presented on the boring logs at the appropriate sample depth, and the results of the grain size distribution are presented graphically in Appendix D.

7.0 SUBSURFACE CONDITIONS

To illustrate subsurface conditions, five idealized subsurface profiles were prepared and are presented on Figures 4 through 8. Three of the profiles are perpendicular to the seawall alignment and two are parallel to the alignment. Because lateral spreading during a major earthquake can occur several hundred feet behind a slope face, the profiles extend to Francisco Street, five blocks south of the seawall. The profiles are based on test boring data gathered during this investigation as well as previous investigations. More detailed information concerning subsurface conditions immediately adjacent to the seawall are shown on Figures 9 and 10.

Our test borings, CPTs and other subsurface data indicate the area behind the Marina Boulevard seawall is underlain by approximately 10 to 30 feet of fill. The fill thickness increases toward the seawall. The fill is thinnest west of Broderick Street in the area of the former Strawberry Island. In the seawall vicinity, there are two types of fill:

- land-tipped fill which consists of Dune sand (medium- to fine-grained sand) and rock fill that was end dumped, and

- hydraulic fill which consists of fine-grained sand and silty sand that was dredged and pumped into place.

The rock fill, which was encountered between Divisadero and Scott Streets, was probably placed during construction of Fair's seawall. The fill materials, including the rock fill, are generally loose to medium dense.

West of approximately Divisadero Street, the fill is underlain by a natural sand deposit, which extends to a depth of approximately 35 feet below street grade. The natural sand is generally medium dense to dense. The natural sand deposit is underlain by weak, compressible clay (Bay Mud). East of Divisadero Street, the Bay Mud deposit is directly beneath the fill. The thickness of the Bay Mud layer varies from approximately 45 feet at Baker Street to 75 feet at Scott Street. Below the Bay Mud are dense sand and stiff clay that extend to bedrock. The depth to bedrock along the seawall alignment is estimated to be between 200 and 300 feet below the ground surface.

Because of the proximity to the Bay and the high permeability of the fill materials, groundwater levels in the vicinity of the seawall will fluctuate with tidal changes. Groundwater was encountered at depths ranging from 7 to 9 feet below the ground surface during drilling of previous borings near the seawall.

8.0 SEISMIC SETTING AND PRIOR EARTHQUAKE BEHAVIOR

The seismic setting of San Francisco, including locations of active faults and the probability of future earthquakes, is presented below. In addition, behavior of the Marina district during the 1906 San Francisco and 1989 Loma Prieta Earthquakes is discussed.

8.1 Seismic Setting

The City and County of San Francisco lies within the seismically active California Coast Ranges geomorphic province, which is characterized by a series of northwest-trending, subparallel, and generally linear mountain belts and valleys underlain by a complex series of faulted and folded rocks. Several active northwest-trending, strike-slip faults, which are all part of the San Andreas Fault System, are present in the vicinity of San Francisco. The major active faults in the area and their distances from the site are as follows:

Major Active Fault	Closest Distance (miles)
San Andreas	7 SW
Hayward	12 NE
San Gregorio	15 SW
Calaveras	30 NE

The location of the Marina District relative to these faults is shown on Figure 11.

The San Andreas and Hayward Faults have been the sources of large historical earthquakes. The largest earthquake to affect the Bay Area was the San Francisco earthquake of 18 April 1906, which had a Richter magnitude of 8.3 (moment magnitude 7.9). This earthquake occurred when a 270-mile-long segment of the San Andreas Fault ruptured between Cape Mendocino and San Juan Bautista, including the portion of the fault closest to the Marina District. The Loma Prieta earthquake of 17 October 1989 occurred when a segment of the San Andreas Fault northeast of Santa Cruz ruptured over a length of approximately 28 miles. The epicenter of the earthquake was approximately 60 miles southeast of the Marina District. The earthquake was assigned a Richter magnitude of 7.1 (moment magnitude 6.9) by the U.S. Geological Survey. The fault rupture was bilateral (in two directions), resulting in only about 10 seconds of strong ground shaking. Its duration was about one-half the duration normally associated with an event of this magnitude. Two other large earthquakes are believed to have occurred on the portion of the San Andreas Fault south of San Francisco in 1838 and 1865. Large earthquakes on the Hayward Fault last occurred in 1836 and 1868. These earthquakes were approximately magnitude 7.

The Working Group on California Earthquake Probabilities (U.S. Geological Survey Circular 1053, 1990) predicts a 67 percent probability of one or more magnitude 7 earthquakes occurring on any one of the major faults within the San Francisco Bay region by the year 2020. The probability of a repeat of the 1906 earthquake, which is considered to be the maximum credible earthquake on the northern portion of the San Andreas Fault, is estimated to be about two percent by the year 2020.

During the Loma Prieta earthquake, the estimated peak ground acceleration in the Marina District was about 0.2 times gravity (g) and the duration of strong ground shaking was about 10 seconds. We estimate the peak ground acceleration due to moment magnitude 7.0 and 7.9 earthquakes occurring on a nearby segment of the San Andreas would be approximately 0.4 and 0.5 times gravity (g), respectively. For the magnitude 7.9 event (a repeat of the 1906 earthquake), it is estimated the duration of ground shaking would be 45 to 60 seconds.

8.2 Performance During Previous Earthquakes

Because the Marina District was relatively undeveloped at the time, little data is available regarding the effects of the 1906 Earthquake on the Marina District; however, the shaking intensity in the Marina during the 1906 earthquake was in the second highest category on the intensity scale used by Lawson, et. al. (1908). It was reported that between 2 and 3 feet of settlement took place on Bay Street between Webster and Laguna Streets. Evidence of ground displacement was also seen in the Baker Street sewer, north of Northpoint Street.

Ground deformation and soil liquefaction occurred throughout the Marina District as a result of the 1989 Loma Prieta earthquake. Soil liquefaction, as evidenced by the presence of sand boils on the ground surface, occurred on most streets where there was building damage. Most of the sand boils were found at the location of the former Marina Cove, where hydraulic fill was placed. Types of ground deformation observed in the Marina District included differential settlements and lateral displacements. Local differential settlements and lateral displacements of up to four inches were observed at the northwest corner of Beach and Divisadero Streets, at the southwest corner of Divisadero and Jefferson Streets, and on Northpoint Street approximately 200 feet west of Webster. Lateral displacement of seven inches northward over 100 feet was measured in the Winfield School playground, approximately 130 feet east of the intersection of Beach and 1Divisadero. Along the Marina Boulevard seawall, seismic-induced ground settlement varied from about one inch at the west end of the seawall to six inches near the east end. No visible damage or movement of the Marina Boulevard seawall or box sewer was observed for the segment between Scott and Baker Streets. However, east of Scott Street along the Marina Green there was evidence the concrete seawall moved several inches. This portion of the seawall is not pile supported; it is supported by a continuous strip footing.

9.0 LIQUEFACTION AND LATERAL SPREADING ANALYSIS

9.1 Liquefaction Analysis

Soil liquefaction is a phenomenon where loose, saturated, cohesionless soil experiences a temporary reduction of strength during strong cyclic loading such as that produced by earthquakes. Typically, liquefaction potential increases with increased duration and magnitude of the cyclic loading. Soils most susceptible to liquefaction are loose, clean, saturated, uniformly graded, fine-grained sand, and cohesionless silt.

Data from our investigation, as well as previous investigations, indicates there are four distinct types of granular soil in the vicinity of the Marina Boulevard seawall that may be susceptible to liquefaction. These soil types are:

- the land-tipped sand fill just below the water table
- the hydraulic fill
- the gravel and rock fill comprising Fair's seawall
- the native sand.

On the basis of our field observations and laboratory tests, we conclude that the submerged land-tipped sand fill (below a depth of about seven feet) and most of the hydraulic fill will liquefy during a major earthquake. Because of the high permeability of the gravel and rock fill comprising Fair's seawall, we judge that sufficient pore pressures to cause liquefaction would not develop in the rock fill and therefore the potential for liquefaction is low. Finally, we judge that isolated pockets within the native sand may liquefy; however, because these pockets would be non-continuous, the movement associated with liquefaction would be minor (less than a few inches).

9.2 Lateral Spreading Analysis

Lateral spreading is generally the most pervasive and damaging type of liquefaction-induced ground failure generated by earthquakes. During lateral spreading, surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial soil is transported downslope or in the direction of a free face, such as a channel slope, by earthquake and gravitational forces. Horizontal ground displacement of several feet can occur on gentle slopes (0.3 to 5 percent grade). Ground displacements as large as 15 to 20 feet on a 0.2 percent slope occurred during the 1964 Niigata, Japan, earthquake and displacements as large as three feet occurred on 0.05 to 0.10 percent slopes during the 1964 Alaska earthquake. Horizontal ground movements ranging from 2 to 5 feet occurred in the south of Market Street area of San Francisco during the 1906 earthquake, resulting in damage to many buildings, bridges, roads, and pipelines.

The Hyogo-Ken Nanbu Earthquake, which occurred on 17 January 1995, caused extensive damage to waterfront facilities in Kobe, Japan. Liquefaction-induced settlement and lateral spreading of several feet occurred during the magnitude 6.8 earthquake.

We evaluated the potential for lateral spreading of the soil behind the Marina Boulevard seawall using an empirical relationship developed by Bartlett and Youd (1992). This relationship incorporates the thickness of the liquefiable layer, the fines content and mean grain-size diameter of the liquefiable soil, the magnitude and distance of the earthquake from the site, the slope of the ground surface, and boundary conditions, such as a free face, to estimate the horizontal ground movement.

Using the available soil and topography data, we compared the lateral spreading predicted by the Bartlett and Youd method with the actual deformations observed during the Loma Prieta earthquake. Magnitude of the earthquake and distance from the causative fault were modified to correct for amplification of the ground motion and duration of shaking. For our analysis, we used a ground slope surface of 0.4 percent and a free face ratio of about 2 percent. Mean grain sizes of 0.2 and 0.3 millimeters (mm) were used for the hydraulic and land-tipped fills, respectively; the fines content of the fill was varied from 5 to 30 percent. Using these parameters, the empirical relationship predicts a few inches of lateral spreading would have occurred during the Loma Prieta earthquake. As discussed above, lateral spreading of 4 to 7 inches occurred only in localized areas of the Marina District during the earthquake.

To model a repeat of the 1906 Earthquake, we used the same soil data, but modified the earthquake magnitude and distance to the fault. The empirical relationship predicts that lateral spreading of 4 to 6 feet toward the yacht harbor will occur east of Divisadero Street during such an earthquake. Significant lateral spreading is not expected to occur west of Divisadero Street, where the soil below the water table consists primarily of medium-dense to dense natural sand (instead of fill). We also analyzed the potential for lateral spreading to occur during a magnitude 7 earthquake on the portion of the San Andreas Fault closest to the site. For this earthquake magnitude, the relationship predicts lateral spreading of several feet east of Divisadero Street.

Bartlett and Youd used this same empirical method to predict several meters of horizontal movement in the South of Market area for the 1906 Earthquake. The actual lateral spread movement, which occurred on gentle slopes of 0.6 to 0.8 percent, was about five feet. They attribute the overprediction to the poor quality of the subsurface data, as well as boundary effects.

There are several conditions in the Marina District that would likely reduce (but not eliminate) the potential lateral spreading that may occur behind the Marina Boulevard seawall. These conditions include: 1) the presence of rock fill along the alignment of Fair's seawall, 2) the reported presence of numerous wood piles that extend through the fill, 3) the presence of vertical and battered wood piles beneath the Marina Boulevard seawall, and 4) the presence of a one-foot-thick layer of gravel beneath the Marina Boulevard box sewer. The gravel reduces uplift pressure that may develop below the box sewer if the underlying fills liquefy during an earthquake. We judge these factors could reduce the amount of lateral spreading south of the box sewer to about three feet.

9.3 Slope Failure Analysis

We checked the stability of the slope outboard of the seawall during a moment magnitude 7 and 7.9 earthquake using two different analyses. For the first analysis we assumed the following:

- the sliding mass (Fair's seawall and the soil above it) is a rigid wedge
- a linear, relatively horizontal failure plane along the interface between Fair's seawall and the Bay Mud
- a lateral load imposed on Fair's seawall by the liquefied hydraulic fill.

Under these conditions, we calculated that a yield acceleration of 0.2g would reduce the safety factor against sliding of the wedge to 1.0. Therefore, if the peak ground acceleration during an earthquake exceeds 0.2g, some movement of Fair's seawall would be expected. Because the anticipated peak ground acceleration during both a magnitude 7 and a magnitude 7.9 earthquake is greater than 0.2g, we performed a deformation analysis of Fair's seawall, using the simplified method developed by Makdisi and Seed (1978). Using this method, we estimate that during a repeat of the 1906 earthquake there could be up to three feet of lateral movement; for a magnitude 7 on the nearby San Andreas or Hayward Fault, we estimate the lateral movement would be about six inches. This movement would be limited to the section of Fair's seawall that retains the hydraulic fill (east of Divisadero Street).

We also performed a slope stability analysis assuming a circular failure plane. Soil strengths used in the analysis were estimated based on laboratory tests and borings and CPT data. The computer program TSTAB and Janbu's method was used to compute the factor of safety of various circular slip surfaces. The analysis indicates that under static conditions the minimum factor of safety against sliding is about 2.0. To model the effects of an earthquake on a potential slide mass, an equivalent static horizontal force was applied to the mass. The force is the product of the seismic coefficient, which is some fraction of gravity, and the weight of the potential slide mass. This method assumes the sliding mass behaves as a rigid body. Our analysis indicates the seismic coefficient that reduces the factor of safety of the slope to 1.0 is 0.28g for the slip surface with the lowest static factor of safety. Because the yield acceleration for the critical circular slip surface is greater than the yield acceleration computed for Fair's seawall, the potential amount of slope movement during an earthquake would be less than estimated above. Using the Makdisi and Seed method, we estimate the movement of the circular slip surface would be on the order of 1 to 2 feet during an earthquake of similar magnitude to the 1906 Earthquake and a few inches during a magnitude 7 event. These earthquakes have a probability of 2 and 67 percent, respectively, of occurring by the year 2020.

10.0 CONCLUSIONS AND RECOMMENDATIONS

On the basis of this investigation, we conclude there are two distinct seismic stability issues to be addressed in the analysis of the Marina Boulevard seawall. They are Local and Area-wide Stability. Local Stability refers to the Marina Boulevard seawall, its foundations, the backfill immediately behind the seawall, and the sidewalk/jogging path and utilities supported by the backfill. Area-wide Stability refers to both the Marina and adjacent Fair's seawall, the box sewer, Marina Boulevard, and all of the area south of Marina Boulevard that was reclaimed by hydraulic filling.

10.1 Area-wide Stability

Considering the large mass of ground that is susceptible to movement (see Figure 2 for limits of hydraulic fill), and the amount of movement that is anticipated during a major earthquake, it may not be economically feasible to address Area-wide Stability. Our analysis indicates that Fair's seawall, and the ground behind it, may move as much as three feet toward the Bay during a repeat of the 1906 earthquake (moment magnitude 7.9), which is estimated to have a probability of occurrence of 2 percent by the year 2020. The estimated movement during a moment magnitude 7.0 earthquake on either the San Andreas or Hayward Fault, a more likely event, is six inches. Vertical movement up to one foot should also be anticipated. This movement is expected to occur primarily east of Divisadero, where Fair's seawall retains very loose hydraulic fill. Improvements to the Marina Boulevard seawall and/or the ground between the seawall and box sewer will not significantly reduce the amount of area-wide lateral spreading that is expected to occur.

10.2 Local Stability

Local stability, i.e., the area between the seawall and the box sewer, is governed by: the adequacy of the wall to resist earthquake forces; the presence of potentially liquefiable material behind the wall and the resulting lateral pressures.

Local Stability can be addressed by:

1. strengthening the seawall to resist the lateral forces due to seismic loading, **or**
2. reducing the lateral forces by improving the soil between the wall and the box sewer, **or**
3. doing nothing and repairing the seawall, utilities, and the sidewalk/jogging path behind the seawall after an earthquake.

However, neither strengthening the seawall (Method 1) nor reducing the lateral forces (Method 2) will protect the box sewer and the area south of the box sewer which could move 1/2 to 3 feet during a magnitude 7.0 and 7.9 earthquake, respectively. Further, even if Method 1 or 2 is implemented, the seawall could be irreparably damaged by Area-wide lateral spreading during a magnitude 7.9 earthquake.

10.2.1 Method 1 - Strengthening the Seawall

The seawall stability analysis performed for the Liquefaction Study of August 1991 only considered the static and dynamic lateral earth pressures imposed on the seawall by nonliquefied fill. Based on this analysis, it was concluded the connection between the concrete and wood portions of the piles supporting the seawall would fail in tension due to seismic loading during a major earthquake. Because of this potential failure mode, it was recommended that a row of 20-foot-long concrete piles be installed along the rear of the seawall to resist seismic overturning forces.

Our analysis indicates the submerged land-tipped fill behind the seawall will liquefy during a major earthquake. This layer, which generally is limited to about 3 to 5 feet in thickness, appears to be continuous along the alignment of the seawall and for several blocks southward of the seawall. Liquefaction of this layer would result in a larger lateral load on the wall than was estimated for the 1991 Liquefaction Study.

We concur with the Liquefaction Study report that the addition of new piles at the rear of the seawall is one solution to resist the seismic overturning forces on the seawall. If this solution is used, we recommend the tensile piles be designed to resist the lateral earth pressures shown on Figure 12.

We judge that prestressed, precast concrete piles would be the most economical pile type. For design of the piles, we recommend using an allowable skin friction of 300 pounds per square foot (psf) in the Bay Mud and 500 psf in the rockfill above the Bay Mud. These values, which are for temporary loading, include a factor of safety of at about 1.5. We anticipate it may be difficult to drive the piles through the rockfill comprising Fair's seawall and predrilling may be required to achieve the required pile embedment. We expect the excavation for the new pile cap will extend several feet below the groundwater table. Therefore, extensive dewatering should be anticipated.

10.2.2 Method 2 - Soil Improvements Behind the Seawall

In the 1991 Liquefaction Study, the installation of stone columns was recommended between the box sewer and the seawall to improve the overall stability of the area; however, our investigation indicates the liquefaction potential of the fill immediately behind the seawall, other than the submerged land-tipped fill, is low. Because of the limited depth of soil to be improved (about 7 to 12 feet below the ground surface), we judge that other improvement techniques would be less costly than stone columns.

One solution that may be more economical than the tension piles (Method 1) would consist of strengthening the land-tipped fill behind the seawall to reduce the lateral earth pressures imposed on the seawall. Strengthening the land-tipped fill would consist of two soil improvement measures used in conjunction with one another: 1) compaction grouting the submerged land-tipped fill between the seawall and the box sewer, and 2) installing three layers of geogrids in the backfill behind the wall.

Compaction grouting consists of driving a small-diameter pipe into the loose soil to be improved and injecting a cementitious grout under pressure. The grout displaces the soil and forces it into a denser mass. Compaction grouting is generally performed on a grid pattern with injection points spaced approximately 4 to 6 feet on center; however, the contractor should establish the injection point spacing. The zone of soil improvement should extend between depths of about 7 and 12 feet and may be modified depending upon the field conditions encountered during installation of the injection point. The injection points can readily be moved in the field if a utility line is encountered. To reduce the liquefaction potential of the submerged land-tipped fill, we recommend the "post-grout" CPT tip resistance of this material be improved to at least 100 tons per square foot (tsf).

Geogrids (soil reinforcement) may be installed behind the seawall to minimize static and dynamic lateral earth pressures pressure imposed on the seawall. We compute that three layers of geogrid (Tensar UX1500 or equivalent) placed at depths of 3, 5, and 7 feet below the existing grade, as shown on Figure 13, will be adequate to reduce the lateral earth pressures such that tension piles would not be required. Each geogrid layer should extend southward at least 15 feet from the seawall. The northern edge of the geogrid layers should be wrapped, encapsulating that end of the fill. Each geogrid layer should be prestressed by stretching prior to being covered with fill. The soil excavated to provide space for the geogrid installment may be reused as engineered fill. The soil, which will consist primarily of sand, should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction[3] below a depth of three feet (as measured from the top of pavement) and 95 percent relative compaction above a depth of three feet.

11.0 FUTURE ACTION

Based on the findings in this report, we recommend the following action be taken:

1. Be prepared to repair the box sewer and utilities along Marina Boulevard in the event of an earthquake.
2. Perform an analysis to compare the cost of the measures discussed above for reducing the potential for damage to the Marina Boulevard seawall and adjacent improvements during an earthquake to the cost of repairing the seawall and these improvements after an earthquake (cost/benefit analysis). The analysis should take into account the probabilities of earthquake occurrence.

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[3] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-91 laboratory compaction procedure.

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