T&R / RYCG
A Joint Venture

19 September 2011
Project 730509401

Ms. Carol Bach
Port of San Francisco
Pier 1, The Embarcadero
San Francisco, California 94111

Subject: Geotechnical Investigation
Amador Street Sanitary Pump Station Improvements
San Francisco, California

Dear Ms. Bach:

T&R/RYCG, A Joint Venture, is pleased to present our geotechnical investigation report for the planned new Amador Street Sanitary Pump Station in San Francisco, California. Our services were performed in accordance with our proposal for the Pier 94 Backlands Improvement and Amador Street Sanitary Pump Station, dated 25 January 2011. This report presents our geotechnical conclusions and recommendations for the Amador Street Sanitary Pump Station only; recommendations for the Pier 94 Backlands Improvement will be submitted in a separate report.

The Amador Street Sanitary Pump Station project consists of sanitary sewer infrastructure improvements along Amador Street and will provide provisions for increased sewer capacity for the Pier 94 Backlands Area, approximately 47 acres of property between Piers 94 and 96, as shown on Figure 1. The existing pump station is east of the 700 block of Amador Street, as shown on Figure 2. The ground surface elevation at the pump station site and vicinity ranges from approximately 1 to 2 feet.[1]

Plans are to partially or completely demolish the existing pump station sump/sanitary collection structure and forcemain, and construct a new reinforced concrete pump station and ductile iron forcemain at the site. The new pump station will have dimensions of approximately 8-1/2 feet by 17 feet and will extend approximately 20 feet below the existing ground surface (bgs). The anticipated pump station loads were not known at the time this report was prepared.

SCOPE OF SERVICES

Our scope of services, outlined in our proposal dated 25 January 2011, consisted of: drilling one boring, performing laboratory testing, evaluating the subsurface conditions at the site, and performing engineering analyses to develop conclusions and recommendations for the new pump station and forcemain, including foundations, retaining walls, and below-grade walls.

SUBSURFACE INVESTIGATION

Our subsurface investigation for the new pump station was performed in conjunction with our investigation for the Pier 94 Backlands project. On 24 May 2011, subsurface conditions were explored at the pump station site by drilling one boring, designated B-14. The approximate location of the boring is presented on Figure 2. Prior to performing our field investigation, we performed the following tasks:

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[1] Elevations in this report refer to San Francisco City Datum (SFCD) in feet unless otherwise indicated.

- prepared an environmental health and safety plan
- obtained a drilling permit from the Monitoring Wells Section of the San Francisco Department of Public Health (SFDPH)
- obtained an encroachment permit from the Port of San Francisco
- notified Underground Service Alert (USA)
- checked the boring locations for utilities using an independent private utility locator.

Boring B-14 was drilled to a depth of about 77-1/2 feet bgs using a truck-mounted rotary wash drill rig operated by Pitcher Drilling of East Palo Alto, California. During drilling, our field engineer logged the boring and obtained representative samples of the soil encountered for classification. The log of the boring is presented in Appendix A on Figure A-1. The soil encountered in the boring was classified in accordance with the classification system presented on Figure A-2.

Soil samples were obtained using three different types of samplers: two driven split-barrel samplers and one pushed thin-walled sampler. The sampler types are as follows:

- Sprague & Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel or brass tubes with an inside diameter of 2.43 inches.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter, without liners.
- Dames & Moore Piston Sampler (D&M) with a 2.5-inch outside diameter and a 2.43-inch inside diameter.

The sampler types were chosen on the basis of the soil type being sampled and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the relative density of sandy soil. The D&M sampler was used to obtain relatively undisturbed samples of the soft to medium stiff cohesive soil.

The SPT and S&H samplers were driven with a 140-pound, above-ground, automatic safety hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy and are shown on the boring logs. The blow counts used for this conversion were: 1) the last two blow counts if the sampler was driven more than 12 inches, 2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and 3) the only blow count if the sampler was driven six inches or less. The D&M sampler is pushed hydraulically into the soil; the piston pressure required to advance the sampler is shown on the logs, measured in pounds per square inch (psi).

Upon completion, the borehole was backfilled with grout consisting of cement, bentonite, and water in accordance with the requirements of the SFDPH. The grouting was completed under the intermittent observation of a SFDPH inspector. The soil cuttings from the boring were collected in 55-gallon drums which were stored temporarily at the site, tested, and eventually transported off-site for proper disposal.

LABORATORY TESTING

We re-examined the soil samples obtained from our boring to confirm the field classifications and select representative samples for geotechnical laboratory testing. The laboratory testing program was designed to correlate and evaluate engineering properties of the soil at the site. Soil samples from boring B-14 were tested to measure moisture content, dry density, fines content, strength, and consolidation characteristics. The laboratory test results are presented on the boring log and in Appendix B.

SUBSURFACE CONDITIONS

Our investigation indicates the pump station site is underlain by approximately 14 feet of heterogeneous fill consisting of silty sand with gravel, clayey sand with variable gravel content, gravel with sand, and clay with variable sand content. The sand and gravel layers are very loose to medium dense, and the clay layers are stiff. The fill is underlain by approximately 61 feet of soft to stiff, highly compressible clay, known locally as Bay Mud. Results of laboratory tests performed on the Bay Mud indicate it is still consolidating under the existing fill load. The Bay Mud is underlain by very dense sand to the maximum explored depth of approximately 77-1/2 feet bgs.

Groundwater was encountered at a depth of approximately seven feet bgs during drilling (corresponding to approximate Elevation -6 feet); however, this measurement was obtained before the groundwater level was allowed to stabilize. We expect the groundwater level at the site to fluctuate based on seasonal variations in rainfall. The groundwater level is also likely to be influenced by changes in sea level and fluctuations of tides.

REGIONAL SEISMICITY

The major active faults in the area are the San Andreas, Hayward, and San Gregorio faults. These and other faults of the region are shown on Figure 3. For each of the active faults within 100 kilometers (km) of the site, the distance from the site and estimated mean characteristic Moment magnitude[2] event [2007 Working Group on California Earthquake Probabilities (WGCEP 2008) and Cao et al. (2003)] are summarized in Table 1.

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[2] Moment magnitude is an energy-based scale that provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

TABLE 1

Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Maximum Magnitude
N. San Andreas - Peninsula	12	West	7.2
N. San Andreas (1906 event)	12	West	8.1
Total Hayward	17	East	7.0
Total Hayward-Rodgers Creek	17	East	7.3
N. San Andreas - North Coast	18	West	7.5
San Gregorio Connected	19	West	7.5
Mount Diablo Thrust	33	East	6.7
Total Calaveras	34	East	7.0
Monte Vista-Shannon	36	Southeast	6.5
Rodgers Creek	38	North	7.1
Green Valley Connected	38	East	6.8
Point Reyes	45	West	6.9
West Napa	48	Northeast	6.7
Greenville Connected	51	East	7.0
Great Valley 5, Pittsburg Kirby Hills	56	East	6.7
Great Valley 4b, Gordon Valley	71	Northeast	6.8
N. San Andreas - Santa Cruz	72	Southeast	7.1
Great Valley 7	75	East	6.9
Hunting Creek-Berryessa	80	North	7.1
Zayante-Vergeles	82	Southeast	7.0
Great Valley 4a, Trout Creek	94	Northeast	6.6
Monterey Bay-Tularcitos	95	Southeast	7.3
Maacama-Garberville	96	North	7.4

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through January 1996. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay on the San Andreas Fault.[3] The estimated Moment magnitude, M_w, for this earthquake is about 6-1/4. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7-1/2. The San Francisco

3 Toppozada, T.R. and Borchardt G., 1998, Re-Evaluation of the 1836 "Hayward Fault" and the 1838 San Andreas Fault earthquakes, Bulletin of Seismological Society of America, 88(1), 140-159.

Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 430 kilometers long. It had a maximum intensity of XI (MM), a M_w of about 7.9 and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent major earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with a M_w of 6.9. The epicenter of the earthquake was in the Santa Cruz Mountains, approximately 90 km from the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (M_w = 6.2).

The 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 30-year probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area to be about 63 percent. More specific estimates of the probabilities for different faults in the Bay Area are shown in Table 2.

TABLE 2

WGCEP (2008) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward – Rodgers Creek	31
North San Andreas	21
Calaveras	7
San Gregorio Connected	6
Concord-Green Valley	3
Greenville	3
Mount Diablo Thrust	1

CONCLUSIONS AND RECOMMENDATIONS

We conclude that from a geotechnical standpoint, the pump station and associated improvements can be constructed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and are implemented during construction. The primary geotechnical issues to be addressed for the project are the presence of liquefiable fill and construction of the pump station structure below the groundwater level and in soft Bay Mud. Our conclusions and recommendations regarding these and other issues are discussed in the remainder of this section.

Seismic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction,[4] lateral spreading,[5] and cyclic densification.[6]

The site is in a liquefaction seismic hazard zone as defined by the California Division of Mines and Geology (CDMG) map entitled *State of California Seismic Hazard Zones, City and County of San Francisco, Official Map*, dated 17 November 2001 (see Figure 5). This map was prepared in accordance with the Seismic Hazards Mapping Act of 1990. We evaluated the potential for liquefaction to occur at the site in accordance with Special Publication 117A, *Guidelines for Evaluating and Mitigating Seismic Hazards Zones in California*, dated 11 September 2008.

The level of ground shaking that may occur at the site during future earthquakes is uncertain because the location, recurrence interval, and magnitude of future earthquakes are not known. A peak ground acceleration (PGA) of 0.36 times gravity (0.36g) was used in our liquefaction analysis. This PGA was calculated using the procedures specified in the 2010 San Francisco Building Code for the Design Earthquake and Site Category E. We assumed an earthquake magnitude of 8.1, which was the maximum Moment magnitude for the 1906 earthquake on the San Andreas Fault as shown in Table 1. A groundwater level at approximately seven feet bgs was used in our liquefaction analyses.

The liquefaction analyses were performed in accordance with the methodology presented in *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, prepared by the National Center for Earthquake Engineering Research (NCEER), dated 31 December 1997, and in Youd et al. (2001). In boring B-14, an approximately four-foot-thick layer of saturated, very loose to medium dense clayey sand with gravel was encountered within the fill layer at a depth of about 10 feet bgs. We conclude that the clayey sand encountered in the fill below groundwater may liquefy during a major

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4 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

5 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.

6 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing differential settlement.

earthquake. However, because the fill at the site is variable and this layer was not encountered in the nearby boring B-13 drilled for the Pier 94 Backlands study, we conclude the potentially-liquefiable fill at the site is isolated and discontinuous. We estimate the amount of ground surface settlement associated with post-liquefaction reconsolidation using the methodologies developed by Tokimatsu and Seed (1984) should be on the order of 1-1/4 inches. Because the pump station will extend below the fill to a depth of about 20 feet bgs, we conclude it will not be adversely affected by the liquefaction-induced settlement; however, utilities connected to the new pump station should be designed to accommodate 1-1/4 inches of liquefaction-induced settlement between the new pump station and the adjacent ground.

The potential for liquefaction-induced ground rupture and sand boils to occur at the site depends on the thickness of the liquefiable soil layer relative to the thickness of the overlying non-liquefiable material. Youd et al. (1995) presented an empirical relationship that provides criteria that can be used to evaluate whether liquefaction-induced surface ruptures and sand boils would be expected to occur under a given level of shaking for a liquefiable layer overlain by a non-liquefiable surficial layer. The potentially liquefiable soil layer encountered at the site is about four feet thick and is below a depth of about seven feet bgs. Therefore, we conclude the potential for surface manifestations of liquefaction to be low.

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as an open cut, or in the direction of a regional slope or gradient. Because the potentially-liquefiable soil at the site is discontinuous, we conclude the potential for lateral spreading at the site is low.

Cyclic densification of non-saturated sand (sand above the groundwater table) caused by earthquake vibrations may result in settlement. Subsurface conditions in boring B-14 indicate that the fill above groundwater contains layers of sand and gravel. We estimate that surface improvements bearing within the non-saturated granular fill may settle up to approximately 1/4 inch as a result of strong shaking from a large earthquake.

Pump Station Foundation Design and Settlement

Based on the results of our investigation, we anticipate the soil below the bottom of the planned pump station will consist of soft Bay Mud. The soil at this level has low strength and high compressibility. As previously discussed, our laboratory tests indicate the Bay Mud is still consolidating under the existing fill. We estimate that up to approximately one foot of additional consolidation settlement will occur in the site vicinity as a result of the existing fill loads. The remaining consolidation settlement is expected to occur over a time frame on the order of 50 years.

Beneath the pump station itself, if the weight of the new structure exceeds the weight of the soil it displaces, additional consolidation settlement of the underlying soft Bay Mud will occur. Conversely, if the weight of the structure is less than that of the displaced soil, the underlying soft Bay Mud will heave as it rebounds under the reduced load. Closely matching the weight of the structure to the weight of the displaced soil can reduce the magnitude of the additional soil settlement or heave beneath the pump station. Alternatively, the pump station and associated pipe connections can be designed to accommodate the anticipated soil movement.

We conclude the pump station may be supported on native soil provided the subgrade is prepared in accordance with the recommendations in a subsequent section of this report. To limit the magnitude of additional consolidation settlement, the pump station should be designed using a net bearing pressure of 200 pounds per square foot (psf) for dead plus live loads.

For a net pressure increase of 200 psf, we calculate that up to approximately 3/4-inch of consolidation settlement could occur locally beneath the structure; this settlement is in addition to the remaining consolidation settlement in the area resulting from the existing fill. We calculate the localized consolidation settlement will occur over a time frame of about 40 to 50 years.

To design the pump station foundation using the modulus of subgrade reaction method, we recommend a modulus of subgrade reaction of 3.2 kips per cubic foot (kcf). The modulus value is representative of the anticipated settlement under the net bearing pressure of 200 psf. After the foundation design is completed, we should review the computed settlement and bearing pressure profiles to check that the modulus value is appropriate.

Lateral loads can be resisted by a combination of passive pressure acting on the face of the pump station foundation and friction along the bottom of the foundation. For computing passive pressure resistance on the foundation, we recommend using a uniform allowable pressure of 750 psf. Frictional resistance should be computed using a friction factor of 0.20 (assumes the bottom of the foundation will be waterproofed). These values include a factor of safety of at least 1.5.

Design Groundwater and Hydrostatic Uplift

As previously discussed, groundwater was encountered at a depth of approximately seven feet bgs (corresponding to approximate Elevation -6 feet) during our subsurface investigation. We conclude a design groundwater elevation of Elevation -4 feet is appropriate for the pump station site.

Because the pump station will extend below the groundwater level, it will need to be waterproofed and designed to resist hydrostatic uplift loads associated with a design groundwater level. We recommend a factor of safety of at least 2.0 be used for permanent uplift. Uplift loads may be resisted by the weight of the structure and any overlying soil. We recommend the soil weight be calculated using unit weights of 50 pounds per cubic foot (pcf) for soil below the design groundwater table (γ_b) and 115 pcf for soil above the design groundwater table (γ_T). If additional uplift resistance is needed, the pump station foundation can be extended around the outside perimeter of the structure and soil backfill placed above the "lip" to add weight. Alternatively, tiedown anchors may be used. We can provide recommendations for tiedowns should it be determined they are needed.

Retaining Walls

Retaining walls less than three feet in height should be designed for at-rest earth pressures using an equivalent fluid weight of 60 pcf. A backdrain should be provided to prevent the buildup of hydrostatic pressure. Where retaining walls are not backdrained, the walls should be designed for at-rest earth plus hysdrostatic pressures using an equivalent fluid weight of 94 pcf.

Retaining walls may be supported on continuous footings at least 18 inches wide. To limit total foundation settlement to less than 1-1/2 inch, we recommend the retaining wall footings be designed using an allowable bearing pressure of 500 psf. Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of footings and the supporting soil. For passive resistance we recommend using an allowable uniform pressure of 750 psf (rectangular distribution). Frictional resistance should be computed using a base friction coefficient of 0.3. These values include a factor of safety of 1.5.

Below-Grade Walls

Below-grade walls should be designed as retaining walls restrained from rotation using an at-rest equivalent fluid weight of 94 pcf, plus a traffic increment where the wall is within 10 feet of adjacent roadways. The traffic increment consists of a uniform (rectangular distribution) lateral pressure of 100 psf, applied to the portion of the wall within 10 feet of the ground surface.

If the below-grade walls will be greater than 12 feet in height, the walls should be designed for the governing condition of the at-rest earth pressure or active earth pressure plus seismic pressures (Lew et al. 2010). We evaluated seismic earth pressures using the method presented in Lew et al. (2010). Considering that the design PGA for the site is about 0.36g and the wall backfill will likely consist of onsite soil containing clay, we conclude that the seismic earth pressure will be small and the at-rest earth pressure provided above will govern the design.

To protect against moisture migration, the pump station walls should be waterproofed and water stops placed at all construction joints. The waterproofing should be placed directly against the backside of the pump station walls. During placement of backfill behind the below-grade walls, the walls should be braced, or hand compaction equipment should be used, to reduce compaction-induced pressures on the walls (as determined by the structural engineer).

Subgrade Preparation

We conclude the pump station may bear on native soil. The subgrade should be cut into undisturbed soil. Regardless of whether the site is actively or passively dewatered, the soil at the bottom of the excavation will be soft and near saturation. To protect the subgrade, we recommend heavy construction equipment not be allowed within two feet of subgrade and that final excavation be made with an excavator equipped with a smooth bucket. Construction traffic should not be allowed on the subgrade to prevent disturbance to the subgrade. Any soft or pumping areas in the subgrade should be repaired by overexcavating and replacing it with a lean concrete rat slab or a layer of reinforcement geotextile and crushed rock. If a significant portion of the excavation bottom is unstable it may be prudent to overexcavate one to two feet of soil and replace it with reinforcing fabric sandwiched between crushed rock to construct a working pad.

Fill Placement

We anticipate fill placement at the site will consist primarily of backfill around the new pump station. Soil excavated during construction will generally be acceptable for use as general fill and backfill, provided it is free of organic material, free of Bay Mud, and contains no rocks or lumps larger than three inches in greatest dimension. Soil excavated from below the groundwater level will have a high moisture content and may require drying before it can be re-used as backfill. Alternatively, where space around the perimeter of the pump station is limited, backfill may consist of crushed rock or controlled density fill. If crushed rock is used as backfill, it should be tamped in place.

Where onsite soil is used as fill, the onsite soil should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to at least two percent above optimum moisture content, and compacted at least 90 percent relative compaction.[7] Clean sand or gravel (defined as soil with less than 10 percent fines by weight) used as fill should be compacted to at least 95 percent relative compaction. Jetting should not be permitted.

All fill material, including onsite fill, should be submitted to the Geotechnical Engineer for approval at least three working days before it is used on site. For imported fill, the grading subcontractor should provide analytical test results or other suitable environmental documentation indicating the proposed fill material is free of hazardous materials at least three days before use at the site. If these data are not provided, up to two weeks may be required to perform any required analytical testing on proposed import soil. Bulk samples of all soil materials should be provided to the Geotechnical Engineer at least three working days before use at the site so a compaction curve and/or gradation analysis can be obtained.

Temporary Cut Slopes and Shoring

We anticipate an excavation approximately 20 to 22 feet deep will be needed to install the pump station. The soil to be excavated consists primarily of sand, gravel, and clay, which can be excavated using conventional earth-moving equipment such as loaders and backhoes. Excavations that will be deeper than five feet and will be entered by workers should be sloped or shored in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). We judge that temporary cuts in native soil inclined no steeper than 2:1 (horizontal to vertical) will be stable provided that they are above groundwater and are not surcharged by equipment or building material. The safety of workers and equipment in or near excavations is the responsibility of the contractor. The contractor should be familiar with applicable local, state, and federal regulations for temporary shoring, including the current OSHA Excavation and Trench Safety Standards.

Considering the depth of excavation (20 to 22 feet) and the relatively shallow groundwater level, we conclude that temporary shoring of the excavation will be required. We judge the most economical shoring system for the project will consist of interlocking sheet piles. Where excavation depths exceed about 12 feet, tiebacks or internal bracing will likely be required. We anticipate space limitations within the excavation will likely preclude the use of tiebacks. Interlocking sheet pile shoring has an advantage of being capable of minimizing the lateral flow of groundwater into the excavation, thus significantly reducing the amount of water to be pumped during construction. Sheetpiles can be used in conjunction with a passive dewatering system, provided the sheetpiles are embedded sufficiently deep to reduce potential for bottom heave. Figure 6 presents the lateral earth pressures we recommend for design of an internally braced sheetpile wall with a passive dewatering system.

If traffic is within a distance equal to the shoring depth, a uniform surcharge load of 100 psf acting on the upper 10 feet should be used in the design. In addition, an increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials will be within a distance equal to the shoring depth. The increase in pressure should be determined after the surcharge loads are known.

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[7] 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 D1557-09 laboratory compaction procedure.

Vertical loads on the shoring can be resisted by skin friction along the sides of the sheetpiles below the bottom of the excavation. To compute vertical capacities for the shoring, we recommend using an allowable skin friction value of 500 psf; this value includes a factor of safety of 1.5. End bearing should be neglected.

The anticipated deflections of the shoring system should be estimated to check whether they are acceptable. The shoring system should be sufficiently rigid to prevent detrimental movement of the temporary shoring and possible damage to existing improvements, including adjacent structures and underground utilities. In our experience, the deflection of a properly designed shoring system should generally be within one inch.

The shoring system should be installed by an experienced shoring contractor. The contractor should be solely responsible for the design of temporary shoring. We should review the final shoring plans to check that they are consistent with the recommendations presented in this report. In addition, we recommend that a representative from our office observe installation of the temporary shoring system.

Temporary Dewatering

An excavation below groundwater will be required to install the pump station, and we conclude temporary dewatering will be required during construction. A passive dewatering system should be used in conjunction with sheet pile shoring system. A passive dewatering system consists of a series of gravel-filled trenches, at least two feet deep.[8] The trenches would need to be installed on the perimeter of the excavation, as well as across it at a spacing of no more than 15 feet, center-to-center. The trenches would be sloped to drain to collection points where submersible pumps ("trash" pumps) could be installed inside drums ("drum sumps").

ADDITIONAL GEOTECHNICAL SERVICES

Before construction, T&R/RYCG should review the project plans and specifications to check their conformance with the intent of our recommendations. During construction, we should observe site preparation, temporary shoring installation, excavation, subgrade preparation, and placement and compaction of fill. These observations will allow us to compare the actual with the anticipated soil conditions and to check that the contractor's work conforms with the geotechnical aspects of the plans and specifications.

LIMITATIONS

The conclusions and recommendations presented in this report result from limited engineering studies based on our interpretation of the geotechnical conditions existing at the time of the investigation. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, T&R/RYCG should be notified to make supplemental recommendations, if necessary.

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8 This is the minimum depth required, and is provided as a way to minimize the cost of the dewatering system. Use of deeper trenches may result in improved groundwater collection and more stable (less saturated) excavation subgrade, and can be used at the contractor's option.

We appreciate the opportunity to provide our services to you. If you have any questions, please call.

Sincerely yours,
T&R/RYCG, A Joint Venture

Linda H.J. Liang, G.E.
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Attachments: References
Figure 1 – Site Location Map
Figure 2 – Site Plan
Figure 3 – Regional Seismic Hazard Zones Map
Figure 4 – Modified Mercalli Intensity Scale
Figure 5 – Regional Seismic Hazard Zones Map
Figure 6 – Typical Lateral Earth Pressures for Temporary Shoring with Internal Bracing and Passive Dewatering
Appendix A – Log of Boring
Appendix B – Laboratory Test Results

QUALITY CONTROL

Robert Y Chew, G.E.
Geotechnical Engineer

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FIGURES

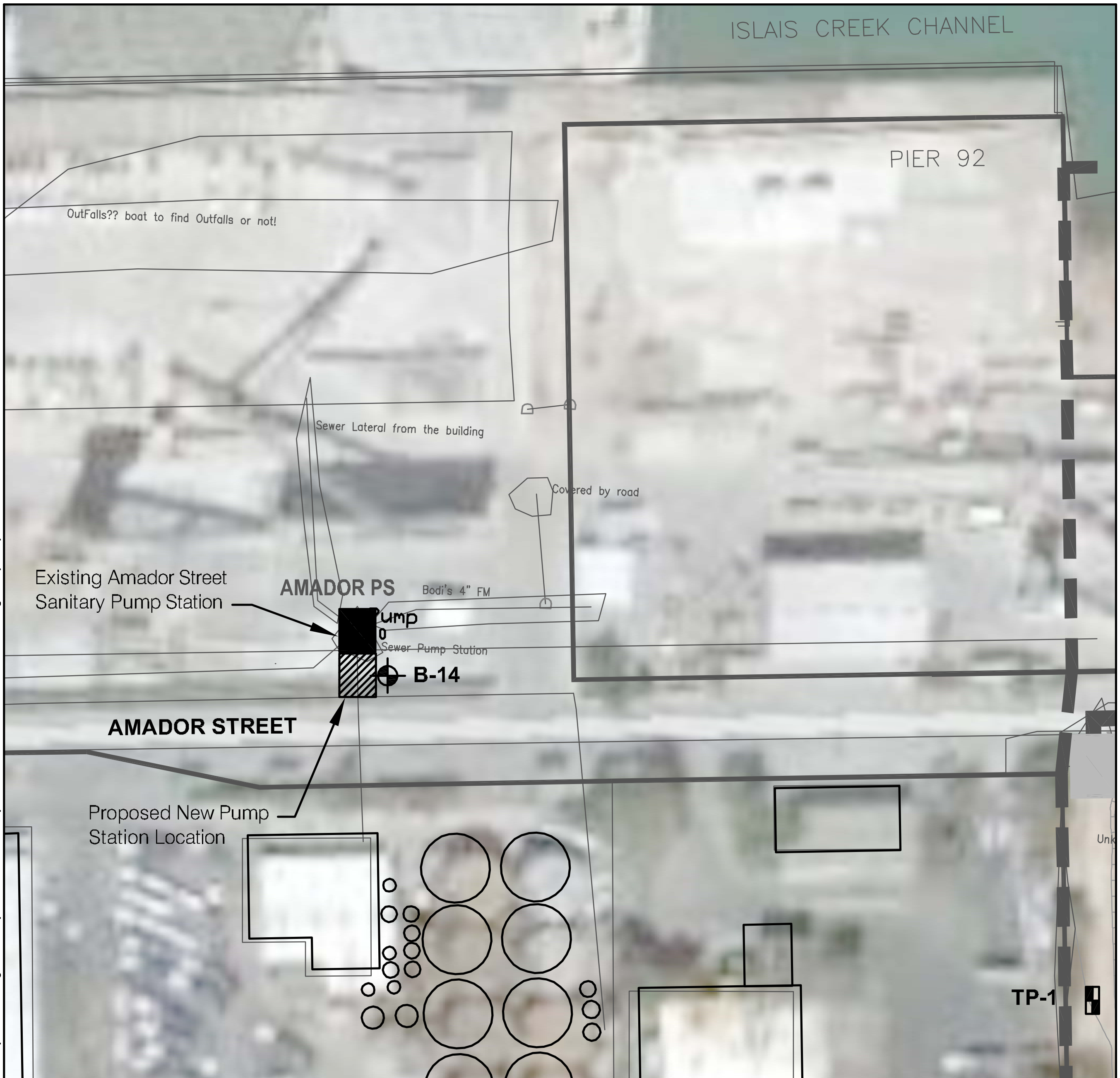
Base map: The Thomas Guide
San Francisco County
2002

0 2000 Feet

Approximate scale

N

AMADOR STREET **SANITARY PUMP STATION IMPROVEMENTS** San Francisco, California	**SITE LOCATION MAP**
T&R / RYCG A Joint Venture	Date 07/20/11 | Project No. 730509401 | Figure 1



EXPLANATION

B-1 Approximate location of boring by T&R/RYCG, June 2011

TP-1 Approximate location of test pit by T&R/RYCG, June 2011

0 100 Feet

Approximate scale

Reference: Base map from a drawing and electronic file provided by the Port of San Francisco, delivered 04/08/11 and Google Earth Pro, 2011.

AMADOR STREET **SANITARY PUMP STATION IMPROVEMENTS** San Francisco, California	**SITE PLAN**
T&R / RYCG A Joint Venture	Date 07/20/11 | Project No. 730509401 | Figure 2

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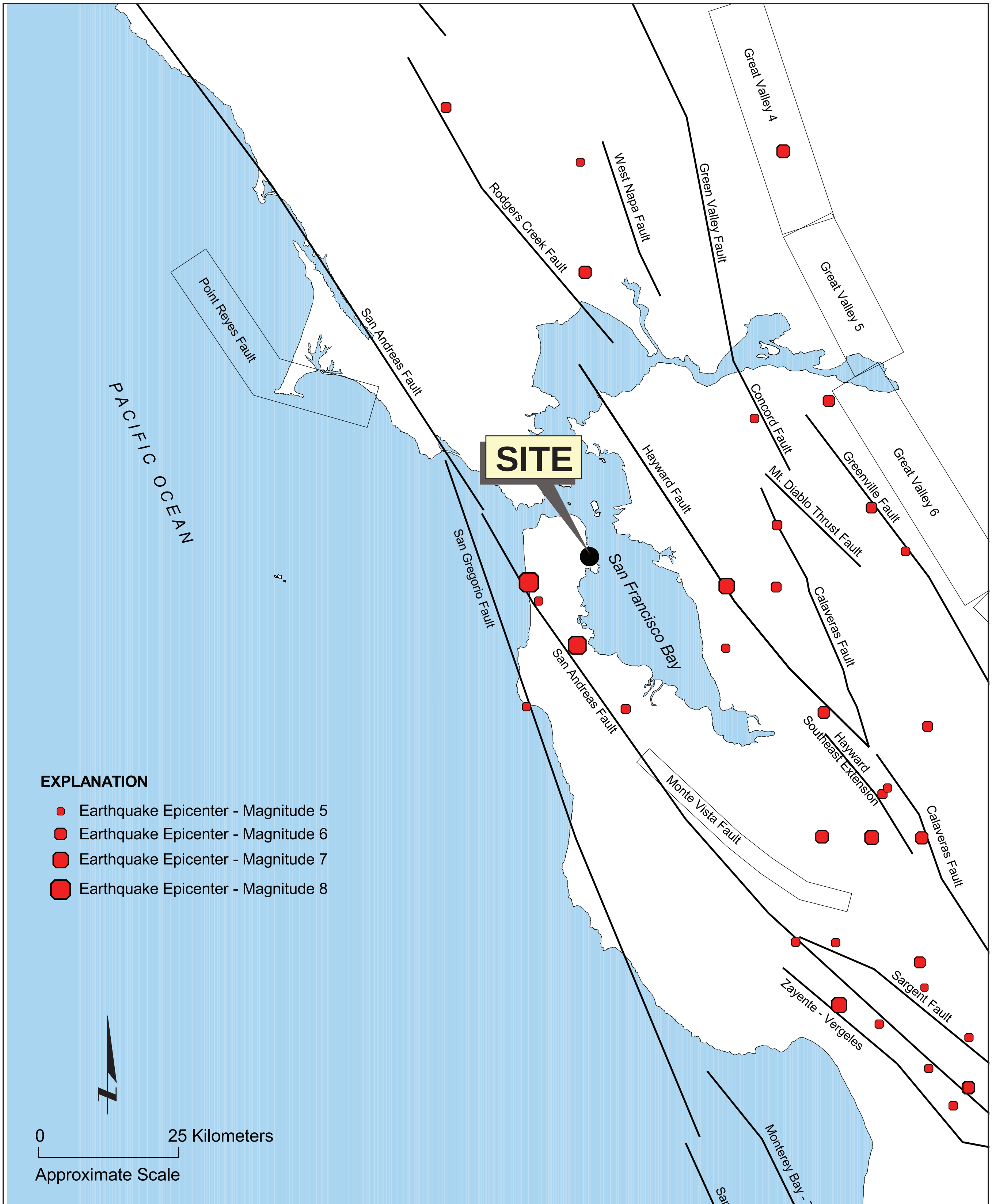

NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

AMADOR STREET
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San Francisco, California

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MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

Date 07/20/11	Project No. 730509401	Figure 3

I **Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.**
Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II **Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.**
As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III **Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.**
Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV **Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.**
Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V **Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.**
Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI **Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.**
Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII **Frightens everyone. General alarm, and everyone runs outdoors.**
People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII **General fright, and alarm approaches panic.**
Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

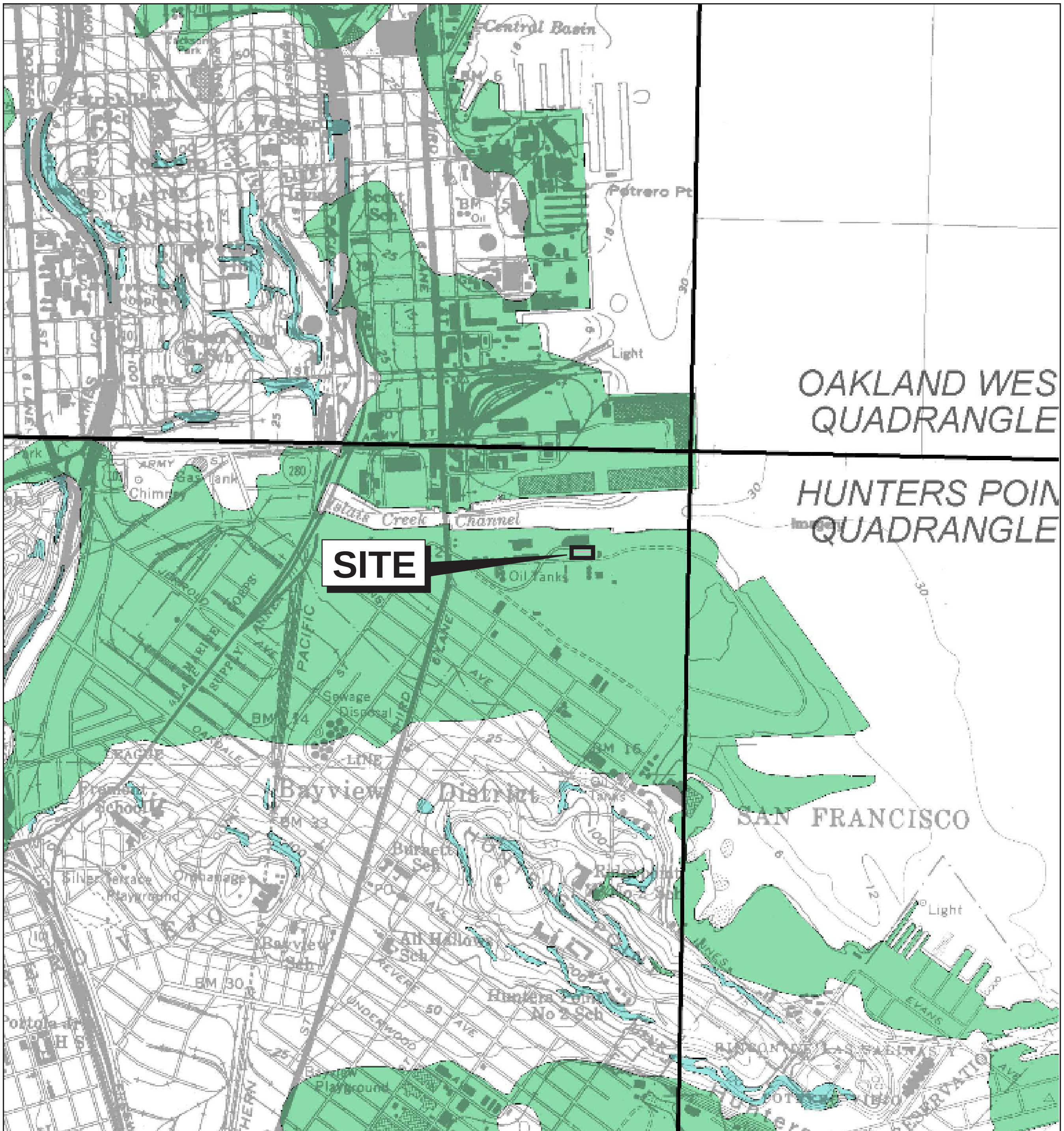
IX **Panic is general.**
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X **Panic is general.**
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI **Panic is general.**
Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII **Panic is general.**
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

AMADOR STREET **SANITARY PUMP STATION IMPROVEMENTS** San Francisco, California	**MODIFIED MERCALLI INTENSITY SCALE**
T&R / RYCG A Joint Venture	Date 07/14/11 | Project No. 730509401 | Figure 4



EXPLANATION

Zone of Liquefaction

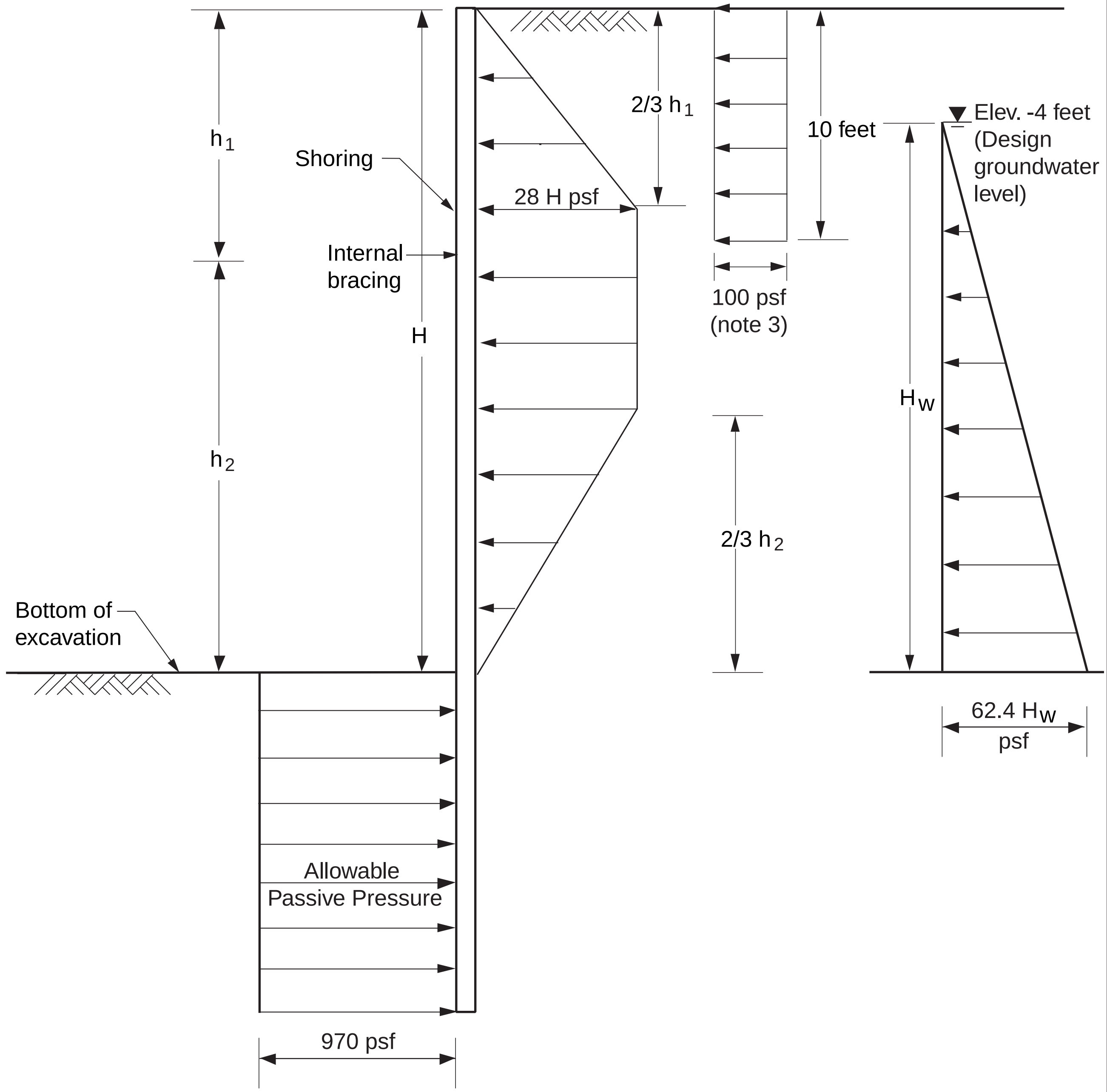
Earthquake-induced Landslides

0 2000 Feet

Approximate scale

Reference:
State of California "Seismic Hazard Zones" City and County of San Francisco Released on November 17, 2001

AMADOR STREET **SANITARY PUMP STATION IMPROVEMENTS** San Francisco, California	**REGIONAL SEISMIC HAZARD ZONES MAP**
T&R / RYCG A Joint Venture	Date 07/20/11 | Project No. 730509401 | Figure 5



Notes:
1. The above pressure diagram assumes that the shoring walls consist of impervious sheetpile shoring system.
2. Passive pressure values include a factor of safety of about 1.5.
3. Pressure due to vehicle surcharge along streets (heavy equipment should come no closer than 5 feet to face of excavation).
4. Assumes a passive dewatering system is used at the base of the excavation and groundwater is retained behind the shoring system.

AMADOR STREET **SANITARY PUMP STATION IMPROVEMENTS** San Francisco, California	**TYPICAL LATERAL EARTH PRESSURES** **FOR TEMPORARY SHORING WITH** **INTERNAL BRACING AND PASSIVE DEWATERING**
T&R / RYCG A Joint Venture	Date 07/20/11 | Project No. 730509401 | Figure 6

APPENDIX A
Log of Boring

PROJECT: **AMADOR STREET SANITARY PUMP STATION IMPROVEMENTS**
San Francisco, California

Log of Boring B-14

Boring location:	See Site Plan, Figure 2
Date started:	5/24/11
Date finished:	5/24/11
Drilling method:	Rotary Wash
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Automatic
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)
Logged by:	T. Shu

Ground Surface Elevation: 1.5 feet[2]

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–2					10 inches concrete over 14 inches cement-treated aggregate base (FILL)						
2–4.5	GRAB			SM	SILTY SAND with GRAVEL (SM) reddish-brown, moist, fine to coarse gravel (FILL)						
4.5–6	S&H	10, 7, 11	12	GP	GRAVEL with SAND (GP) black and olive-green, medium dense, moist, with angular serpentinite fragments (FILL)						
6–6.5				CL	SANDY CLAY (CL) gray and olive-brown, stiff, moist, trace gravel (05/24/11, 1:05 p.m.) (FILL)						
6.5–7	SPT	2, 3, 8	13	SC	CLAYEY SAND (SC) gray and olive-brown, medium dense, moist, trace gravel (FILL)				42.9	15.4	117
7–7.5				CL	SANDY CLAY (CL) dark gray and brown, stiff, wet, trace gravel (FILL)						
7.5–10				CL-CH	CLAY with SAND (CL-CH) bluish-gray, stiff, wet, trace gravel (FILL)						
10–14	S&H	2, 5, 13	13	SC	CLAYEY SAND with GRAVEL (SC) gray, very loose to medium dense, wet (FILL)						
12	SPT	3, 2, 1	4	SC					20.1		
13.5–14.5	D&M		400 psi								
14–30				CH	CLAY (CH) gray, medium stiff, wet, high plasticity, trace shell fragments (BAY MUD)						
18–19.5	D&M		50 psi	CH		TV		800			
25–26.5	D&M		350 psi	CH		TV		800			

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PROJECT:	AMADOR STREET **SANITARY PUMP STATION IMPROVEMENTS** San Francisco, California	**Log of Boring B-14** PAGE 2 OF 3

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAY (CH) (continued)						
31												
32												
33												
34												
35						no shell fragments						
36	D&M			350 psi		Triaxial Test, see Figure B-1	TxUU TV	3,500	600 800		56.5	67
37						Consolidation Test, see Figure B-2					63.2	62
38												
39												
40												
41												
42												
43												
44												
45					CH	grades with shell fragments						
46	D&M			300 psi		BAY MUD						
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

PROJECT:	AMADOR STREET SANITARY PUMP STATION IMPROVEMENTS San Francisco, California	Log of Boring B-14

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	D&M			400 psi	CH	CLAY (CH) (continued) grades stiff, no shell fragments	TV		1,000			
62												
63												
64												
65												
66												
67						BAY MUD						
68												
69												
70												
71												
72												
73						grades sandy at 73 feet						
74												
75			10		SP	SAND (SP) gray, very dense, wet, fine-grained						
76	S&H		26 50/ 5"	35/ 5"								
77	SPT		18 50/ 5"	60/ 5"								
78												
79												
80												
81												
82												
83												
84												
85												
86												
87												
88												
89												
90												

Boring terminated at a depth of 77.4 feet below ground surface.
Boring backfilled with cement grout.
Groundwater measured at 7.4 feet during drilling.
TV = torvane

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on San Francisco City Datum.

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Project No.: 730509401

Figure: A-1c

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	**Grain Size in Millimeters**
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample, hand auger

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push sampler

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

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SANITARY PUMP STATION IMPROVEMENTS
San Francisco, California

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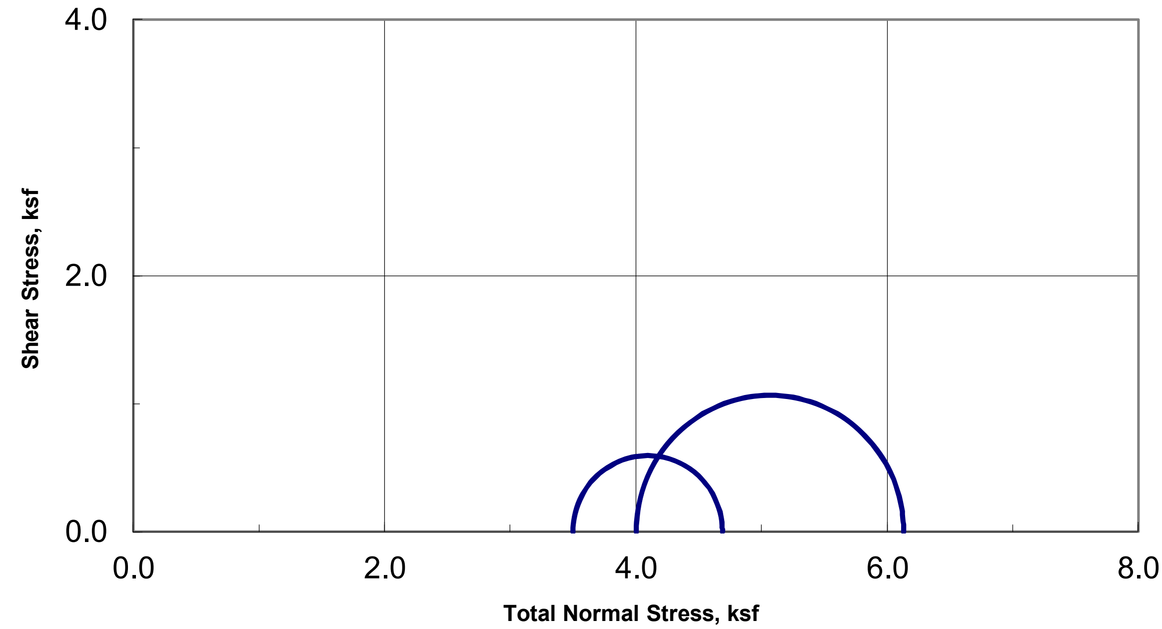
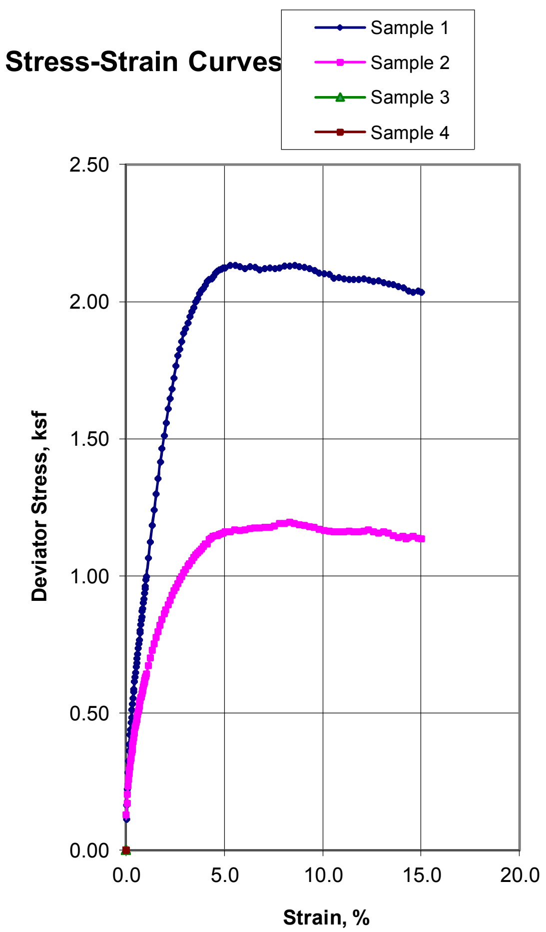
CLASSIFICATION CHART

Date 07/20/11 | Project No. 730509401 | Figure A-2

APPENDIX B
Laboratory Test Results

Unconsolidated-Undrained Triaxial Test
ASTM D-2850





Sample Data				
	1	2	3	4
Moisture %	47.4	56.5		
Dry Den,pcf	73.8	66.5		
Void Ratio	1.284	1.536		
Saturation %	99.7	99.3		
Height in	5.02	5.01		
Diameter in	2.40	2.40		
Cell psi	27.8	24.3		
Strain %	5.30	8.30		
Deviator, ksf	2.133	1.196		
Rate %/min	1.00	1.00		
in/min	0.050	0.050		
Job No.:	**092-005**			
Client:	**T&R/RYCG**			
Project:	**Pier 94 Backlands/ 730509401**			
Boring:	B-11	B-14		
Sample:	11	8		
Depth ft:	40(Tip-4")	35(Tip-4")		

Visual Soil Description	
Sample #	
1	Gr CLAY, trace brn org & Sa pockets (Bay Mud)
2	Gray CLAY, trace shell fragments (Bay Mud)
3	
4	

Remarks:

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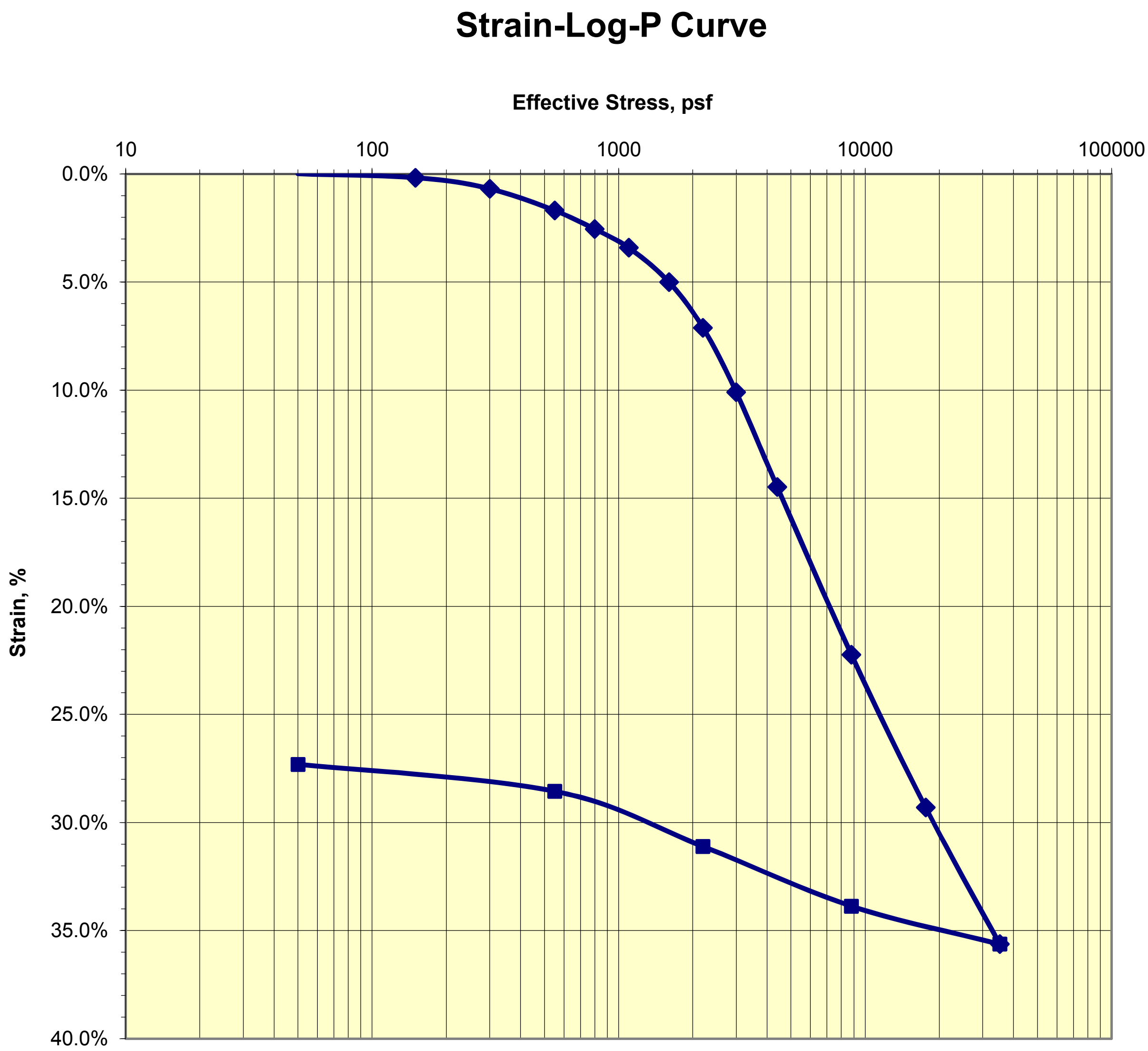
Figure B-1

Consolidation Test
ASTM D2435

Job No.:	092-005	**Boring:**	B-14	**Run By:**	MD
Client:	Robert Y Chew Geotechnical	**Sample:**	8	**Reduced:**	PJ
Project:	Pier 34 Backlands - 2011-003	**Depth, ft.:**	35(Tip-3")	**Checked:**	PJ/DC
Soil Type:	Gray CLAY (Bay Mud)			**Date:**	7/6/2011



Ass. Gs = 2.7	Initial	Final
Moisture %:	63.2	42.0
Dry Density, pcf:	61.6	79.0
Void Ratio:	1.734	1.133
Saturation:	98.3%	100%

Remarks:

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Figure B-2