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Geotechnical Engineering • Geology • Hydrogeology

GEOTECHNICAL DESIGN REPORT
BRANNAN STREET WHARF
SAN FRANCISCO, CA

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Prepared for:

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INTRODUCTION

The Brannan Street Wharf project entails the construction of a proposed bayfront park along The Embarcadero between Piers 30-32 and Pier 38 in San Francisco, California. The project includes the partial or complete demolition of Pier 36 wharf and structures, possible strengthening of the existing seawall along The Embarcadero within the limits of the project, and pile-supported wharf construction for the proposed park. The wharf would occupy approximately 800 feet along The Embarcadero waterfront, and vary in width from 30 feet to about 140 feet. The location of the study area is indicated on Plate 1 – Site Location Map.

This geotechnical report presents the findings, conclusions, and recommendations of our geotechnical study of the Brannan Street Wharf project. This report also includes a summary of the geologic conditions at the site and potential geologic and seismic hazards.

WORK PERFORMED

In accordance with our professional consulting services agreement dated March 5, 2009 and subsequent conversations with the project design team, we have completed the scope of work described below:

1. **Exploratory Drilling and Testing.** We explored subsurface conditions by means of drilling four rotary wash borings (B-1 through B-4) and two cone penetration test (CPT) soundings (CPT-1 and CPT-2). The boring and CPT locations are shown on Plate 2 – Site Plan and Boring Location Map. Exploration number, date of exploration, surface elevation and depth are shown on Table 1 – Summary of Geotechnical Explorations. Surface elevations were roughly estimated from previous plans of the project area, and are approximate. All elevations on Table 1, and referred to throughout this report, are with respect to North American Vertical Datum of 1988 (NAVD 88). San Francisco City Datum (SFCD) is 11.34 feet above NAVD 88.

   We visually classified the soil during drilling. We recovered split-spoon (Standard Penetration Test) samples, relatively undisturbed 2 ½ inch diameter sleeve samples using a split-barrel sampler, and undisturbed 3 inch diameter Shelby tube samples. Selected samples were transferred to the laboratory for testing. Boring logs are presented in Appendix A – Boring Logs. CPT soundings are presented in Appendix B – Cone Penetration Tests.

TABLE 1 – SUMMARY OF GEOTECHNICAL EXPLORATIONS

Boring	Date Drilled	Approximate Surface Elevation (feet, NAVD 88)	Depth (feet)
B-1	4/21/09-4/22/09	+11	121.5
B-2	4/21/09	+11	8.0
B-3	4/23/09-4/24/09	+13	140.9
B-4	4/22/09-4/23/09	+13	100.9
CPT-1	4/20/09	+11	137.6
CPT-2	4/20/09	+11	139.9

2. **Vane Shear Testing.** We performed vane shear testing in the young bay mud stratum in boring B-3. Vane shear tests were performed at 3-foot depth increments from the mudline to a depth of approximately 25 feet below the mudline. The results from the vane shear testing are presented in Appendix C – Vane Shear Test Results.

3. **Laboratory Testing.** We performed moisture, density, grain size analysis, Atterberg limits, unconfined compressive strength, unconsolidated undrained triaxial shear strength, and consolidation tests on selected soil samples to measure pertinent index and engineering properties. The laboratory test results are presented on figures in Appendix D – Laboratory Test Results and on the boring logs on Plates A-1.1 through A-1.6.

4. **Engineering Analysis.** We analyzed subsurface conditions and field and laboratory test results, prepared soil profiles, and reviewed regional and local geology and seismicity. Additionally, we analyzed the following geotechnical design issues:

    - Seismic hazards evaluation including strong ground shaking, fault rupture, liquefaction, seismic settlement, lateral spread, seismically-induced landslides and inundation by tsunamis;
    - Acceleration response spectra in accordance with the 2007 CBC as well as probabilistic earthquakes with a 10 percent and 50 percent probability of exceedance in 50 years;
    - Static and pseudo-static stability of the shoreline including estimates of shoreline displacement for various levels of seismic shaking;
    - Static stability of proposed riprap revetment mitigation for concrete seawall stability;
    - Lateral earth pressures for the existing concrete seawall;
    - Axial capacity of the existing Pier 36 caissons;

- Axial compressive and tensile capacity of proposed new 24” octagonal precast, prestressed driven piles and 24” and 36” diameter steel pipe piles, including the effects of drag loads on piles;
- Axial load deflection behavior of new piles (t-z and q-z springs);
- LPILE parameters for lateral analysis of existing and proposed new piles;
- Effect of lateral spreading ground on proposed wharf piles; and
- Consolidation-related settlement from the addition of riprap revetment.

5. **Report.** We prepared this report presenting our geotechnical findings, conclusions and recommendations regarding the proposed Brannan Street Wharf project.

FINDINGS

SITE CONDITIONS

The proposed Brannan Street Wharf is on the San Francisco Bay waterfront along The Embarcadero between Piers 30-32 and Pier 38 (Plate 1 – Site Location Map). The existing Pier 36 is at the southern end of the proposed project (Plate 2 – Site Plan and Exploration Location Map). Pier 36 is a reinforced concrete structure supported on reinforced concrete belled caissons. An approximately 34,500 square foot warehouse building sits on Pier 36. Pier 34 was demolished in about 2001, and formerly extended out from The Embarcadero in approximately the middle of the project. An approximately 25- to 50-foot wide marginal wharf is located along the Embarcadero extending out from the seawall.

The original shoreline was located about 1,000 feet west of the Brannan Street Wharf area, close to the intersection of Bryant and Fremont Streets. In the 1860s, piers extended out from the shoreline to approximately where the Embarcadero is now located. According to previous reports (HLA et al., 1992), seawall sections 11 and 12 were built from the foot of Main Street to the foot of King Street between about 1907 and 1914, establishing the current shoreline. The early portions of the seawall (prior to the 1906 Earthquake) were reportedly constructed by dredging the soft mud to a depth varying from 20 to 35 feet below low water over a width of approximately 30 feet at the trench bottom, and filling the trench with rock. Additional rock was placed forming a wide base, so that the seawall was 100 feet wide at a point 20 feet below low tide. Modifications to this general approach were made in subsequent portions of the seawall. A concrete seawall supported on timber piles was constructed within the rock fill seawall for the portions along the project limits. Selected plans from the Port of San Francisco are provided in Appendix F – Historic Sea Wall Plans.

SEISMICITY

The San Francisco Bay Area is in a seismically active region near the boundary between two major tectonic plates, the Pacific Plate to the southwest and the North American Plate to the northeast. Since approximately 23 million years ago, about 200 miles of right-lateral slip has occurred along the San Andreas Fault Zone to accommodate the relative movement between these two plates. The relative movement between the Pacific Plate and the North American Plate generally occurs across a 50-mile zone extending from the San Gregorio fault in the southwest to the Great Valley Thrust Belt to the northeast. In addition to the right lateral slip movement between tectonic plates, a compressional component of relative movement has developed between the Pacific Plate and a smaller segment of the North American Plate at the latitude of San Francisco Bay during the last 3.5 million years. Strain produced by the relative motions of these plates is relieved by right lateral strike slip faulting on the San Andreas and related faults (San Gregorio, Calaveras, Hayward), and by vertical reverse slip displacement on the Great Valley and other thrust faults in the central California area.

The San Francisco Bay Area contains several active faults that could cause strong ground shaking at the project site. Plate 3 – Regional Active Fault Map shows faults in the vicinity of the proposed Brannan Street Wharf. The San Andreas Fault is approximately 8.9 miles southwest of the site, while the Hayward Fault is approximately 9.6 miles northeast of the site. The San Andreas, Hayward, Calaveras and Rodgers Creek faults have produced measurable historic ground motion and movement. The San Andreas Fault is capable of producing an earthquake of an estimated maximum magnitude of 7.9. The recurrence interval of a 7.9 magnitude seismic event on this segment of the San Andreas Fault is estimated on the order of 200 years. A summary of nearby faults is presented in Table 2 – Active and Potentially Active Faults.

TABLE 2 – ACTIVE AND POTENTIALLY ACTIVE FAULTS

Fault	Distance [1] (miles)	Estimated Maximum Earthquake Magnitude	Historic Earthquakes	
			Year	Magnitude
San Andreas (1906 rupture) [2]	8.9 [2]	7.9 [2]	1906	7.9
San Andreas (Peninsula)	8.9	7.1	1838 1898 1989	6.8 6.2 7.1
Hayward	9.6	7.1	1868	6.8
San Andreas (North Coast)	11.4	7.6	NA	NA
San Gregorio-Seal Cove	12.1	7.3	NA	NA
Calaveras	20.1	6.8	1861 1955 1979 1984 2007	5.3 5.5 5.9 6.1 5.4
Rodgers Creek	21.3	7.0	1898	6.4
Concord-Green Valley	23.2	6.9	NA	NA
Monte Vista-Shannon	25.1	6.8	NA	NA
Point Reyes	26.7	6.8	NA	NA

(1) Data determined from EQFAULT (Blake, 2000).
(2) 1906 rupture event assumes rupture of North Coast, Peninsula, and Santa Cruz Mtns. segments to San Juan Bautista. Maximum magnitude based on 1906 average 5 m displacement (WGCEP, 2003; Petersen et al., 1996).

GEOLOGY

Regional Geology. The site is located in California's Coast Ranges province, which consists of 500 miles of northwest-trending ridges and valleys. San Francisco Bay occupies a depression or basin between the San Andreas Fault to the west and the Hayward Fault to the east where the Earth's crust has been down-warped between these geological structures. Late Pleistocene and Holocene sediments (less than 1.0 million years old) were deposited in the basin as it subsided. In the project vicinity, these sediments comprise deposits of younger bay mud, undifferentiated sedimentary deposits, older bay mud, and alluvial/colluvial deposits, all of which rest on a variety of bedrock types associated with the Franciscan Complex. The Franciscan Complex makes up much of the basement rock of the Coast Ranges and consists of an assemblage of deformed and metamorphosed rock units, including sandstone, shale, serpentinite and greenstone. These units formed as a subduction complex associated with continuous east-dipping subduction at the margin of the North American and Pacific plates. These two plates move relative to each other, with the San Andreas Fault Zone at the junction. The Pacific Plate, on the west side of the fault zone, is moving north relative to the North American Plate on the east.

Local Geology. The Brannan Street Wharf area is located east of Rincon Hill, which is composed of rocks of the Franciscan Complex. In general, basement units of the Franciscan Complex in the wharf area are covered by Quaternary sands and clays, bay mud deposits, and artificial fill. The units are described from youngest to oldest, which approximates their vertical distribution from the top to the deeper units. Table 3 presents general descriptions of the geologic units. The geographic distribution of these units at the ground surface is depicted on Plate 4 – Geologic Map.

TABLE 3 – SUMMARY OF GEOLOGIC UNITS IN THE BRANNAN STREET WHARF AREA

Geologic Unit	Age	Lithology
Artificial Fill (af)	Historic (0-200 years old)	Mixture of sand, gravel, and some clay. Abundant debris including wood, glass, and brick.
Bay Mud Deposits (Qbm)	Holocene to Pleistocene (0-1.8 million years old)	Highly compressible clay with minor layers of silt and clayey sand. Some shell fragments.
Upper Layered Sediments (Qul)	Holocene to Pleistocene (0-1.8 million years old)	Undifferentiated alluvial and marine deposits, consisting of interbedded sands and clays.
Franciscan Complex (Kfs, Kfss)	Cretaceous to Jurassic (65 to 165 million years old)	Mixed assemblage of distinct bedrock types, including sandstone, shale, serpentinite.

Source: Blake et al., 2000.

EARTH MATERIALS

Our subsurface exploration consisted of three geotechnical borings to depths ranging from approximately 100 to 140 feet, one geotechnical boring that hit drilling refusal at 8 feet, and two cone penetration tests (CPTs) to depths of approximately 140 feet. We also reviewed geotechnical borings and cross sections from past geotechnical studies and construction drawings provided to us by the Port of San Francisco and obtained at the San Francisco Department of Building Inspection. The logs of the drill holes and the CPT soundings are included in Appendices A and B, respectively. Idealized subsurface profiles are included on Plates 5 and 6 – Geologic Cross Sections. The locations of the cross sections are delineated on Plate 2.

The subsurface conditions consist of approximately 25 feet of young bay mud underlain by Quaternary-age alluvial/marine deposits. The land to the west of the seawall was reclaimed from San Francisco Bay. Most of the young bay mud was dredged out, and artificial fill was placed to attain site grades.

Artificial Fill and Rock Dike. Artificial fill and rock dike materials were encountered to a depth of approximately 36.5 to 42 feet in the borings and CPTs behind the concrete seawall (B-1, CPT-1 and CPT-2). The upper approximately 12 feet of fill consists of a mixture of clay, sand, gravel, cobbles and construction debris. Soil types included sandy gravel, silty gravel, clayey gravel, gravelly sand, clayey sand, sandy silt, gravelly clay, and sandy clay. The fill was generally loose. Below approximately 12 feet, the fill became coarser and was indicative of material types expected for the early 1900s seawall construction. The boring logs and cross sections, as well as later sections of this report, label this material as "rock dike". Based on laboratory test results and drilling behavior, we expect this material is a mixture of sand, gravel, cobbles and boulders. As illustrated on some of the historic sections of the rock dike seawall, we expect that larger rock was used at the base and at the top of the rock dike section. The middle section appears to be predominantly sandy gravel or gravelly sand, generally loose to medium dense, although portions of the drilling were difficult indicating either larger sized cobbles and boulders and/or wood debris.

Young Bay Mud. The young bay mud is a soft, normally consolidated, plastic clay that is characterized by low shear strength and high compressibility. Borings B-3 and B-4 encountered approximately 25 feet of young bay mud. The bay mud was typically dark gray to black, wet, very soft, fat (i.e. highly plastic) clay. A thin layer (approximately 2 to 8 feet thick) of young bay mud, sometimes intermixed with gravel fill, was encountered in each of the borings and CPTs underlying the rock dike at a depth of approximately 35 to 40 feet below ground surface.

Upper Layered Sediments. As encountered in our explorations, upper layered sediments are approximately 60 feet thick, and underlie the young bay mud. These alluvial/marine deposits consist of interbedded sands and clays. The sand layers are typically dense to very dense and the clay layers are typically stiff to hard. We encountered a layer of dense sand at the bottom of the upper layered sediments in each of our five explorations. The top of this sand layer ranges from elevation -67 to -80 feet and the bottom of the sand layer (top of old bay mud) ranges from elevation -87 to -93 feet. The thickness of the dense sand layer ranges from 10 to 20 feet.

Old Bay Mud. An approximately 30-foot thick old bay mud stratum, a marine unit deposited during an interglacial period prior to the Wisconsin Glaciation, was encountered starting at about 90 feet below NAVD 88 datum.

The older bay mud encountered in our explorations predominantly consists of dark gray to very dark greenish gray, wet, stiff to very stiff, fat clay. A sequence of lean clay, and occasional lenses of silty sand and clayey sand were also encountered within the old bay mud stratum.

Lower Layered Sediments. A sequence of interbedded alluvial and marine sediments was encountered in explorations that extended below old bay mud (B-3, CPT-1, and CPT-2). We have labeled this unit "Lower Layered Sediments". Based on boring B-3, this unit consists of dark greenish gray, wet, hard, gravelly clay with sand and reddish brown and yellowish brown, wet, very dense, clayey sand to silty sand.

GROUNDWATER

The groundwater level behind the concrete seawall will experience some tidal influence from the adjacent San Francisco Bay and will fluctuate relative to daily high and low tide levels. Groundwater was estimated to be at about 7 to 9 feet deep at the time of drilling at boring B-1, and in the predrill holes for CPT-1 and CPT-2. These groundwater depths correspond to a groundwater elevation of approximately +2 to +4 feet (NAVD88).

CONCLUSIONS AND RECOMMENDATIONS

1.0 GENERAL

The following sections provide our conclusions and recommendations for evaluation and design of Brannan Street Wharf. The key geotechnical issues for the project are considered to be:

- The selection, design and constructability of pile foundations to support the proposed wharf;
- Evaluation of the global shoreline stability and concrete seawall stability during a seismic event;
- Mitigation of lateral spread potential and/or evaluation of the impacts of lateral spread on the proposed improvements; and
- Improvement of seawall stability to resist seismic forces.

We consider the proposed improvements to be geotechnically feasible, provided that our geotechnical recommendations are incorporated into design and construction documents.

2.0 SEISMIC DESIGN CONSIDERATIONS

2.1 **General.** The main geologic and seismic hazards at the site are expected to be strong ground shaking, seismic settlement, lateral spread and inundation by tsunamis. Our evaluation of site class, fault rupture, ground shaking, liquefaction and seismic settlement, lateral spread, seismically-induced landslides, and inundation by tsunamis are provided in the following sections.

2.2 **Site Class.** We evaluated the Site Class of the project site in accordance with Section 1613 of the 2007 California Building Code (CBC). Laboratory testing and vane shear test results indicate the shear strength of the approximately 30-foot thick layer of young bay mud ranges from approximately 100 pounds per square foot (psf) to approximately 300 psf. The laboratory test results of the plasticity index of the younger bay mud ranged from 58 to 65. The water content of the younger bay mud was found to range from 77.6 to 117 percent. Therefore, the site east of the existing seawall should be categorized as Site Class E since there is at least 10 feet of soil having the following characteristics: 1) plasticity index (PI) greater than 20; 2) natural water content greater than or equal to 40 percent; and 3) shear strength less than 500 psf. The land portion of the project is located within Site Class D as the bay mud was dredged and removed during seawall construction. As indicated later in Section 2.9, the site response between bayward and landward sides of the seawall will likely be gradual rather than abrupt since the thickness of the bay mud gradually increases for a horizontal distance of approximately 25 feet away from the seawall.

2.3 **Fault Rupture.** No active or potentially active faults are known to cross the site. Consequently, the hazard posed by ground rupture due to fault offset is considered to be negligible.

2.4 **Ground Shaking**. Strong ground shaking will occur at the site as a result of a moderate to large earthquake occurring on one of the active regional faults. The San Andreas Fault is closest to the site (8.9 miles), and therefore has the capability of producing the strongest ground shaking in the event of a large earthquake.

The California Geological Survey (CGS, formerly known as California Division of Mines and Geology) and United States Geological Survey (USGS) completed probabilistic seismic hazard maps in 1996 (Petersen et al., 1996), and subsequently updated fault parameters and revised the maps in 2002 (Cao et al., 2003). USGS provides a web-based program to evaluate the USGS Probabilistic Uniform Hazard Response Spectra (http://earthquake.usgs.gov/research/hazmaps/design). Based on this data, the peak ground acceleration (PGA) at the site is estimated to be 0.47g for an earthquake having a 10 percent probability of exceedance in 50 years. The PGA is approximately 0.70g for an earthquake having a 2 percent probability of exceedance in 50 years.

2.5 Liquefaction and Seismic Settlement. Liquefaction is a phenomenon wherein a temporary, partial loss of shear strength occurs in a soil due to increases in pore pressure that result from cyclic loading during earthquakes. Saturated, loose to medium dense sands and silty sands are most susceptible to liquefaction. Consequences of liquefaction can include ground settlements, foundation failure, sand boils, and lateral spreading.

The site is mapped within a liquefaction hazard zone by the California Geological Survey (CDMG, 2000) as is all of the man-made land around the bay margin. The liquefaction hazard zones in relation to the Brannan Street Wharf project are shown on Plate 7 – Liquefaction Hazard Map. HLA et al. (1992), in a study of liquefaction of the North Beach, Embarcadero Waterfront, South Beach and Upper Mission Creek areas, indicate that liquefaction-induced settlement along The Embarcadero in the project vicinity may be on the order of 3 to 12 inches.

The subsurface conditions encountered in the predrill holes for CPT-1 and CPT-2 and in boring B-1 indicate that the fill and rock dike, extending to depths of up to approximately 42 feet, is a mixture of clay, sand, gravel, cobbles, boulders and construction debris. The fill was likely dumped with not much compactive effort, and therefore is generally loose to medium dense. Because the fill has significant gravel and larger sized rocks, we do not expect liquefaction or a significant strength loss in the material during seismic shaking. However, due to the loose nature of some of the fill soils, ground surface settlement of the area behind the seawall may occur as the earthquake shaking causes rearrangement of soil particles into a denser state. We estimate that seismic settlement may be on the order of 3 to 6 inches.

Liquefaction of a significant portion of the soils underlying The Embarcadero and land to the west has the potential to impact the project site due to increased lateral earth pressures and lateral spread. Therefore, to further evaluate the liquefaction potential of the site vicinity landward of the seawall, we reviewed boring logs provided by the Port of San Francisco (1961) and geotechnical reports at the San Francisco Department of Building Inspection. We reviewed the report for Bayside Village (Harding Lawson Associates, 1986) bounded by Brannan, Beale, Bryant and Delancey Streets. We were not able to obtain the report for the Delancey Street Foundation development bounded by Brannan Street, The Embarcadero and Delancey Street. Based on our review of the additional plans and reports, the fill underlying The Embarcadero and Bayside Village extends to a depth of approximately 10 to 20 feet and is a heterogeneous mix of sand, gravel, silt and clay with boulders, cobbles and construction debris. Portions of the fill appear to be subject to liquefaction during strong ground shaking whereas other portions appear to be non-liquefiable.

Based on this additional review, and review of our liquefaction assessment of soils from our exploratory borings and cone penetration tests, it is our opinion that though liquefaction-related displacement and strength loss may occur within isolated zones of the fill behind the seawall, the majority of the fill will not experience a significant strength loss.

2.6 **Lateral Spread.** Lateral spreading is a seismically induced ground deformation failure in which near surface soil layers typically break into blocks that progressively move downslope or toward a nearby free face such as a stream channel, river embankment, or a shoreline. Underground facilities and structural elements (e.g., pipelines, spread footings, pile foundations, etc.) that extend through or across a zone of lateral spreading may be pulled apart or sheared.

The seawall sections between Taylor and Mission Streets and from King to Berry Streets had been completed prior to the 1906 San Francisco Earthquake (HLA et al., 1992). Because there were no reports of movements of the seawall, HLA et al. concluded that the wall performed well in the earthquake and sections of the seawall built after the earthquake will be stable. Another possible explanation for no reported movement is that the seawall movement was much less than the devastation caused by the earthquake and fire in other parts of the City. HLA et al. indicate that the lateral displacement due to liquefaction along The Embarcadero in the project vicinity may be on the order of 3 to 12 inches.

As presented in the subsurface profiles on Plates 5 and 6, we encountered a layer of bay mud underlying the seawall at a depth of approximately 40 feet. The bay mud was sometimes intermixed with gravelly fill (e.g. boring B-1.) This layer may provide a plane of weakness that can strain during a seismic event and cause lateral spread of the shoreline. From our analysis, we do not anticipate large movements, but lateral deformations of up to approximately 20 inches appear possible. The free-field soil displacements were calculated to be 2.4 inches, 6.8 inches, 10.3 inches and 19.3 inches for seismic events corresponding to 50% in 50 year probability of exceedance, California Building Code (CBC) Design Level Earthquake, 10% in 50 year probability of exceedance, and CBC Maximum Considered Earthquake, respectively. This hazard is addressed more fully in Section 5 of this report.

2.7 **Seismically-Induced Landslides.** The site is not within an area subject to seismically-induced landslides due to the level grades.

2.8 **Inundation by Tsunamis.** Tsunamis are long period waves caused by underwater seismic disturbances, volcanic eruptions, or submerged landslides. The disturbance can occur thousands of miles from the San Francisco area, and generate a tsunami wave that affects the site. As tsunami waves approach the coast, they may increase in height to tens of feet.

According to published data (URS Blume, 1974; Borrero et al., 2006), the maximum recorded runup (tsunami wave) at the Presidio occurred after the 1964 Alaskan earthquake. The wave measured 7.5 feet at the Golden Gate, with strong surges and high water observed in Sausalito and other Marin County locations on San Francisco Bay, Berkeley, Richmond and Oakland. Damage within San Francisco Bay included boats being torn from moorings, and damaged docks and piers with damage estimates of approximately $1 million (Borrero et al., 2006).

Ritter and Dupre (1972) mapped the site within an area of potential inundation by tsunamis. The mapping is based on a runup of 20 feet at the Golden Gate Bridge and tsunami attenuation in San Francisco Bay based on the May 1960 and March 1964 tsunamis. Based on this data, the Ritter and Dupre mapping indicates the runup at the site may be on the order of 12 feet (60 percent of that at the Golden Gate). Subsequent research using numerical modeling (Garcia and Houston, 1975; Borrero et al., 2006) has concluded that the estimates by Ritter and Dupre are probably too high. Garcia and Houston (1975) made 100 year return period and 500 year return period tsunami predictions for San Francisco Bay for a flood insurance study. For sites in San Francisco south of the Bay Bridge, Garcia and Houston predict tsunami runup of 4.5 to 5.0 feet for a 100 year return period and 7.5 to 8.4 feet for a 500 year return period. These values have been adopted in the Marine Oil Terminals chapter (Chapter 31F) of the 2007 CBC, and therefore, appear to be the most relevant for the design of the project. Borrero et al., based on deterministic studies of potential seismic and landslide sources, indicated that the event that generated the largest tsunami runup within San Francisco Bay was a M_w 9.2 earthquake on the Alaska Peninsula segment of the Alaska-Aleutian subduction zone. This event resulted in a maximum tsunami runup at the Potrero district in San Francisco of approximately 5.9 feet accounting for a 150 percent factor of safety on model runs.

2.9 **Acceleration Response Spectra.** We evaluated the acceleration response spectra for several earthquake hazard levels at the request of Winzler & Kelly / Structus, Inc. Joint Venture (JV). The hazard levels evaluated consist of the maximum considered earthquake (MCE) in accordance with the 2007 CBC, and probabilistic events having a 10 percent probability of exceedance in 50 years and 50 percent probability of exceedance in 50 years.

The 2007 CBC uses MCE ground motion maps developed by the USGS (Frankel et al., 1996). For regions of high seismicity near known fault sources with short return periods (like in much of the San Francisco Bay Area), the MCE ground motions are determined from the USGS deterministic hazard maps developed by using ground motion attenuation functions on the median estimate increased by 50 percent.

As discussed in Section 2.2, the site lies straddling two different Site Classes corresponding to the bayward and landward sides of the seawall. The seismic parameters in Table 4 – Seismic Design Parameters may be used in developing the site response. We computed the MCE response spectra and design response spectra in accordance with the 2007 CBC, and provide the results in graphical and tabular format in Appendix E – Acceleration Response Spectra. The transition in site response between bayward and landward sides of the seawall will likely be gradual rather than abrupt since the thickness of the bay mud gradually increases for a horizontal distance of approximately 25 feet away from the seawall. Two possible approaches seem reasonable for capturing this 25-foot wide transition zone. One approach would be to consider both response spectra for Site Class D and Site Class E, and design the improvements within this zone to perform adequately for both spectral accelerations. The second approach would be to use linear interpolation to calculate spectral accelerations within the transition zone where the spectral acceleration at the seawall would be that selected from the Site Class D spectrum and the spectral acceleration 25 feet seaward of the seawall would be that selected from the Site Class E spectrum.

TABLE 4 – SEISMIC DESIGN PARAMETERS

Site Class	Spectral Acceleration for Short Periods, S_s	Spectral Acceleration for 1 Second Period, S_1	Site Coefficients Fa	Site Coefficients Fv
D (Landward of Seawall)	1.5g	0.604g	1.0	1.5
E (25 Feet Bayward of Seawall)	1.5g	0.604g	0.9	2.4

Chapter 31F of the 2007 CBC establishes minimum engineering, inspection and maintenance criteria for marine oil terminals in order to prevent oil spills and to protect public health, safety and the environment. Chapter 31F provides a simplified method to evaluate spectral response based on peak ground acceleration and spectral acceleration values from the USGS probabilistic maps and site amplification effects in accordance with Section 31F.4.2.4. We adopted this simplified method to provide an estimate of the spectral accelerations at two probabilistic events: 10 percent probability of exceedance in 50 years (475 year return period) and 50 percent probability of exceedance in 50 years (72 year return period). The response spectra from this analysis are provided for Site Classes D and E in Appendix E.

3.0 GROUNDWATER

The groundwater level behind the concrete seawall will experience some tidal influence from the adjacent San Francisco Bay and will fluctuate relative to daily high and low tide levels. Groundwater was estimated to be at about 7 to 9 feet deep at the time of drilling at boring B-1, and in the predrill holes for CPT-1 and CPT-2. We assumed a design groundwater elevation of +3 feet (NAVD88), corresponding to a depth of approximately 8 feet, for geotechnical analyses including lateral loads on the existing seawall and slope stability analyses. The groundwater level may vary above or below this level during construction due to tidal influence, stormwater runoff, and other factors.

4.0 SOIL PROPERTIES

The subsurface conditions consist of approximately 25 feet of young bay mud underlain by Quaternary-age alluvial/marine deposits. Franciscan Complex bedrock underlies the site at depth. The land to the west of the seawall was reclaimed from San Francisco Bay. Most of the young bay mud was dredged out, and artificial fill was placed to attain site grades.

The subsurface conditions were idealized to group soils with similar strength and deformation characteristics for engineering analysis purposes. Geologic profiles are presented on Plates 5 and 6. The location of geologic profile lines are shown on Plate 2.

The man-made fills were subdivided into two units: artificial fill and rock dike. Artificial fill was found to be highly variable and was encountered at the boring and CPT locations behind the seawall within the upper approximately 12 feet. The rock dike is comprised of material placed during construction of the seawall construction in the early 1900s, and as encountered in the borings includes sand, gravel, cobbles and boulders. The soil properties used in our geotechnical analyses for these units are included in Table 5 – Soil Properties for Design.

A young bay mud unit has been deposited within the marine environment of San Francisco Bay in recent geologic time (last approximately 9,000 years). The bay mud at the boring locations and as indicated on plans of Pier 36 is approximately 25 to 30 feet thick. The elevation of the bottom of the bay mud appears to be fairly consistent within the project limits based on available subsurface data. The unit weight and strength of the bay mud increases with depth as a result of the increase in pressure with depth. Laboratory triaxial shear strength tests and a consolidation test of this stratum is provided in Appendix D, and in-situ vane shear strength tests from boring B-3 are provided in Appendix C. Strength tests indicate that the shear strength generally increases from approximately 100 psf near the top of the stratum to 300 psf at the bottom of the normally consolidated unit. A lower, semi-consolidated unit of young bay mud lies between the normally consolidated unit and the Upper Layered Sediments. A vane shear test of this unit indicates a shear strength of approximately 500 psf. A thin layer (approximately 2 to 8 feet thick) of young bay mud, sometimes intermixed with gravel fill, was encountered in each of the borings and CPTs underlying the rock dike at a depth of approximately 35 to 40 feet below ground surface. Because of the consolidation of this unit from the overlying fill, the intermixing with gravelly fill materials, and the likely discontinuous nature of this stratum, it is difficult to predict the strength of this layer with much precision. To account for these factors, we have estimated the shear strength to be approximately 1,200 psf. A summary of the soil properties used in our geotechnical analyses for the young bay mud units is included in Table 5.

The young bay mud is underlain by a layered deposit of sands and clays that is referred to as Upper Layered Sediments. We have grouped the sediments into predominantly clay units and predominantly sand units for design purposes, although soil types include sand, silty sand, clayey sand, clay, and sandy clay. The stratification of these deposits is more thinly bedded than indicated in the idealized geologic cross sections on Plates 5 and 6. The sand layers are typically dense to very dense and the clay layers are typically stiff to hard. The existing caissons for Pier 36 and the existing rock dike derive their support within the upper part of the Upper Layered Sediments, indicating good bearing characteristics of this stratum. A layer of dense sand is present at the bottom of the Upper Layered Sediments in each of our five explorations and in many of the historic borings. The thickness of the dense sand layer ranges from 10 to 20 feet. A summary of the soil properties used in our geotechnical analyses for the sand units and clay units within the Upper Layered Sediments is included in Table 5.

A thicker sequence of clay underlies the Upper Layered Sediments. This unit is commonly referred to as "older bay mud." This unit was presumably deposited during an interglacial period when the sea level was higher (similar to today). The older bay mud is overconsolidated, and therefore has much better bearing characteristics than the young bay mud. Laboratory triaxial shear strength tests and unconfined compression tests of this stratum are provided in Appendix D. Field torvane shear strength tests and pocket penetrometer compressive strength tests are included on the logs in Appendix A. Strength tests indicate that the shear strength is on the order of 1,100 psf to 4,000 psf. A summary of the soil properties used in our geotechnical analyses for the older bay mud is included in Table 5.

Another sequence of layered deposits was encountered in boring B-3 and cone penetration tests CPT-1 and CPT-2 underlying the older bay mud. This unit is referred to as Lower Layered Sediments. This unit includes interbedded sands and clays with gravelly layers. The CPT probes were unable to penetrate this unit very deeply and encountered refusal at approximately 140 feet below ground surface. Also, blow counts within this unit exceeded 50 for 6 inches of penetration with a Modified California sampler. A summary of the soil properties used in our geotechnical analyses for this unit is included in Table 5.

TABLE 5 – SOIL PROPERTIES FOR DESIGN

Layer	Total Unit Weight, γ (pcf)	Cohesion, c (psf)	Friction Angle, ϕ (degrees)
FILL (af)			
Artificial Fill – Sand	125	0	28
Artificial Fill – Clay	125	800	0
Rock Dike	130	0	32 to 36
Proposed Riprap Revetment	132	0	42
YOUNG BAY MUD (Qybm)			
Normally Consolidated Unit	90	Increases with depth from 100 to 300 psf	0
Semi-Consolidated Unit	95	500	0
Unit Below Fill Behind Rock Dike	110	800	0
Unit Below Rock Dike	110	1,200	0
UPPER LAYERED SEDIMENTS (Qul)			
Sandy Units	135	0	35 to 38
Clayey Units	124	1,500	0
OLD BAY MUD (Qobm)			
Clay	115	1,600	0
LOWER LAYERED SEDIMENTS (Qll)			
Sand	138	0	38

5.0 SHORELINE GLOBAL STABILITY

5.1 General. Due to the presence of a potentially weak layer at the bottom of the rock dike seawall, we evaluated the slope stability and lateral spread potential of the existing shoreline. The generalized cross section for the slope stability analysis is based on the section shown on Plate 6. In our analysis, we generally followed the procedures outlined in the Guidelines for Analyzing and Mitigating Landslide Hazards in California (SCEC, 2002).

5.2 Generalized Soil Properties. Soil strengths were characterized using total stress strength parameters. Undrained shear strengths were used for clayey soils. Drained strength parameters were used for sandy soils. Since we anticipate rapid dissipation of pore pressures in the rock dike material during a seismic event, we did not consider a reduction in the effective stresses due to pore pressure generation. Shear strength parameters and unit weights used in the slope stability analyses are presented in Table 6 – Geotechnical Parameters for Slope Stability Analysis.

TABLE 6 – GEOTECHNICAL PARAMETERS FOR SLOPE STABILITY ANALYSIS

Layer	Total Unit Weight, γ (pcf)	Cohesion, c (psf)	Friction Angle, ϕ (degrees)
FILL (af)			
Artificial Fill	125	800	0
Rock Dike	130	0	36
YOUNG BAY MUD (Qybm)			
Normally Consolidated Unit	90	200	0
Semi-Consolidated Unit	95	500	0
Unit Below Fill Behind Rock Dike	110	800	0
Unit Below Rock Dike	110	1,200	0
UPPER LAYERED SEDIMENTS (Qul)			
Sandy Units	135	0	35
Clayey Units	124	1,500	0

5.3 **Static Stability Analysis.** We used computer program SLOPE/W to perform limit equilibrium analysis of the slope stability section. Morgenstern and Price's method of analysis was utilized to compute static factors of safety of circular slip surfaces. This method of analysis satisfies all three equations of equilibrium: horizontal force, vertical force and moment. Generally, a factor of safety of 1.5 is desirable to limit the risk of slope instability due to uncertainties in the soil strength parameters. Our analysis indicated a factor of safety of approximately 2.1. The computer output of the critical failure surface and the factor of safety is provided on Plate 8 – Static Slope Stability Section.

5.4 **Seismic Stability Analysis.** A pseudo-static representation of seismic loading was used in SLOPE/W to evaluate the factor of safety during a seismic event. Four different levels of seismic loading were considered for our analysis: the maximum considered earthquake (MCE) and design earthquake (DE) in accordance with the 2007 CBC, and probabilistic events having a 10 percent probability of exceedance in 50 years and 50 percent probability of exceedance in 50 years. Since the factor of safety was evaluated to be less than one for all four levels of seismic loading, we evaluated the estimated displacement based on a simplified semi-empirical predictive model proposed by Bray and Travasarou (2007).

Unlike Newmark models where the sliding mass is assumed to be a rigid block, the model proposed by Bray and Travasarou accounts for the deformability of the sliding mass. The sliding soil mass is characterized by its strength as represented by the yield coefficient, $\mathbf{k_y}$, and its dynamic stiffness as represented by its initial fundamental period, $\mathbf{T_s}$.

The model requires computing $\mathbf{k_y}$ using a limit equilibrium slope stability procedure. The coefficient at which the factor of safety for stability equals unity is defined as $\mathbf{k_y}$. The calculated $\mathbf{k_y}$ for the section analyzed is approximately 0.22g. The computer output of the critical failure surface under seismic loading and the yield coefficient is provided on Plate 9 – Seismic Slope Stability Section.

The fundamental period of the sliding mass, $\mathbf{T_s}$, can be taken as:

$$T_s = \frac{4H}{V_S}$$

where H = maximum vertical distance between the ground surface and slip surface used to determine k_y, and

V_S = representative small strain shear wave velocity of materials within the sliding mass

Based on the geometry of the slide mass and the materials within the slide mass, we estimated $\mathbf{T_s}$ to be 0.26 second.

Using these properties, the estimated median displacement by the Bray and Travasarou predictive model was 6.8 inches and 19.3 inches for the DE and MCE, respectively. The estimated median displacement was 2.4 inches and 10.3 inches for the probabilistic events having a 10 percent probability of exceedance in 50 years and 50 percent probability of exceedance in 50 years, respectively. Bray and Travasarou indicate that the seismic displacement range for 16 to 84% exceedance levels (plus or minus one standard deviation from the mean) can be taken as half to twice the median estimate, respectively. Therefore, the lateral displacement during the DE, for example, may be expected to range from approximately 3 ½ to 14 inches.

The strength parameters with the highest degree of uncertainty are also the most critical in estimating lateral spread displacement. These parameters are the friction angle of the rock dike material and the average shear strength along the weak layer underlying the rock dike. Another uncertainty is the stabilizing effect of existing and proposed piles that are installed through the rock dike and weak stratum into the underlying Upper Layered Sediments. Additional geotechnical explorations can be performed if a more refined analysis of lateral spread is desirable.

5.5 Shoreline Stability Improvement Measures. The most effective means to eliminate lateral spread potential is to improve the soils underlying the rock dike or to extend a secant pile wall or sheet pile wall through the rock dike and weak stratum to resist lateral movement. Based on preliminary cost estimates of these strategies, we understand these mitigation measures will not be incorporated in the current project due to budget limitations. Without such mitigation measures, the need to repair some existing and proposed new facilities should be anticipated after a significant seismic event.

6.0 SEAWALL STABILITY

6.1 Lateral Earth Pressures. We evaluated lateral earth pressures on the concrete seawall for the existing conditions depicted on plans from the Board of State Harbor Commissioners (1909b and 1912). The lateral earth pressure diagrams are included on Plate 10 – Lateral Earth Pressures.

Active earth pressures are imposed by the soil on walls that are unrestrained so that the top of the wall is free to translate or rotate at least 0.004H, where H is the height of the wall. The active earth pressure varies with effective stress in the soil and the soil type. Our recommended distribution of active earth pressure with depth is provided on Plate 10.

Unbalanced hydrostatic pressures should be accounted for in the stability evaluation of the wall. We estimate that the maximum unbalanced hydrostatic level between the tide level in the bay and the groundwater level behind the wall is approximately 5 feet. The pressure distribution caused by this unbalanced hydrostatic pressure is provided on Plate 10.

The seawall may experience additional lateral loads imposed by traffic, construction, storage of materials, or other at-grade applied vertical surcharge loads. The lateral distribution of surcharge loads depends on the load magnitude, location, and surface dimensions of the load footprint (e.g., point loads, areal loads). For uniform areal loads over a large footprint, we recommend that a uniform lateral pressure increment of **0.35 x q** be applied to the seawall, where **q** is the applied surface pressure. This load should be applied in a rectangular pressure distribution for the upper 13 feet of the seawall as shown on Plate 10.

In addition to the active earth pressures, the seawall should be designed to consider additional earth pressures due to earthquake loading. We considered the potential influence of liquefaction and resulting increased soil pressures, but concluded that though liquefaction-related displacement and strength loss may occur within isolated zones of the fill behind the seawall, the majority of the fill will not experience a significant strength loss. Therefore, we evaluated the seismic earth pressure using the Mononobe-Okabe (M-O) seismic coefficient analysis (Seed and Whitman, 1970). We used a total stress analysis to evaluate the seismic earth pressures of the retained fill on the bulkhead retaining wall. In accordance with recommendations to use seismic coefficients corresponding to $^{1}/_{3}$ to ½ of the peak acceleration of the design earthquake (Whitman, 1990), we used a horizontal seismic coefficient, $\mathbf{k_h}$, of one-half of the peak ground acceleration of the MCE event. The distribution of the earth pressure due to seismic loading for the MCE event is provided on Plate 10. To evaluate the seawall stability for various other earthquake hazard levels, the equivalent fluid pressures presented in Table 7 – Seismic Increment Equivalent Fluid Pressures may be used.

TABLE 7 – SEISMIC INCREMENT EQUIVALENT FLUID PRESSURES

Earthquake Ground Motion Level	Seismic Increment Equivalent Fluid Pressure (pcf)	
	Artificial Fill	Rock Dike
MCE per 2007 CBC	31	29
2% in 50 years	39	36
10% in 50 years	22	21
50% in 50 years	9	8

Lateral loads on the seawall can be resisted by a combination of passive pressures and either the shear and bending moment resistance of the timber piles or shear resistance along the base. Since the marginal wharf is going to be demolished as part of the proposed project and there will likely be a seismic joint between the proposed Brannan Street Wharf and the seawall, the lateral support provided by these structures were not considered in seawall stability.

While shear resistance is mobilized with small lateral movements, the passive pressure depends on more significant lateral displacement of the wall. Soft clays and loose sands generally require more displacement to mobilize the ultimate passive resistance than stiff clays, dense sands and gravels. For the material types existing at the face of the embedded portion of the seawall, we estimate that the displacement to achieve ultimate passive pressure resistance is approximately 5 percent of the depth of embedment of the seawall, **Z**. The distribution of passive earth pressure with depth is provided on Plate 10. As presented, we recommend a conservative ultimate passive pressure resistance since we are unsure of the current condition of the bulkhead retaining wall in the rock dike, and the potential steep angle of the rock dike down and away from the toe of the wall. The ultimate passive earth pressures presented are based on the unit weight and shear strength profile of bay mud. These are Rankine earth pressures and neglect wall friction. Oftentimes, the displacement to achieve ultimate passive earth pressures exceeds the allowable displacement of the structure. We estimate that approximately 85 percent of the ultimate passive resistance will be mobilized with a displacement of 2.5 percent of **Z**, and 50 percent of the ultimate passive resistance will be mobilized with a displacement of 0.5 percent of **Z**.

The displacement of the seawall is also resisted by the lateral resistance of the existing timber piles. A lateral load versus displacement analysis (e.g. LPILE analysis) can be performed to evaluate the contribution of the timber piles to seawall stability if the timber piles are judged to be in sound condition. The LPILE parameters provided in Section 7 of this report may be used to perform such an analysis. In the event that the existing timber piles fail because of exceeding shear or bending moment capacities, the friction along the base of the wall can be utilized to assess seawall stability. A coefficient of friction of 0.35 may be used for estimating the resistance due to base friction. The coefficient should be multiplied by the dead load only, and should account for the buoyant weight of the seawall for the submerged portion.

6.2 **Seawall Stability Improvement Measures.** Several strategies were explored for resisting the seismic loads that were determined by the structural designers to exceed the lateral load resistance capabilities of the existing seawall. Mitigation measures considered included tieback anchors, vertical or battered piles including micropiles, H-piles and drilled shafts, and riprap revetment. More detailed descriptions of these mitigation measures are included in the Conceptual Structure Alternatives report prepared by Winzler & Kelly / Structus, Inc. JV (2009a). An additional advantage of conceptual design alternatives for a secant pile seawall replacement (Concept Alternative SW4) and dredging and placement of a riprap buttress fill (Concept Alternative SW6) is the mitigation of the lateral spread potential addressed in Section 5 of this report. These alternatives, however, are quite costly. We understand that mitigation of the seismic instability of the seawall will likely be limited to riprap placement at the 15-foot high seawall at Piers 32, 34 due to budget limitations. Without more extensive mitigation measures, the need to repair some existing and proposed new facilities should be anticipated after a significant seismic event.

6.3 **Riprap Revetment.** Riprap revetment may be used along portions of the seawall to improve seawall stability. At this time, we understand that this mitigation measure will be limited to the 15-foot high seawall at Piers 32, 34. We evaluated the increased passive resistance, surface settlement due to consolidation of young bay mud, and stability of the riprap revetment. A schematic of the riprap placement is included on Plate 11 – Proposed Riprap Revetment for Seawall Stability. Dredging will be required to construct the proposed revetment; most notably to construct the keyway section. The keyway is important to provide stability during riprap placement. The keyway should be constructed first.

The lateral resistance due to passive earth pressures from the revetment is provided on Plate 12 – Passive Pressure of Riprap Revetment. The earth pressures are based on Rankine earth pressure theory and neglect wall friction. Riprap revetment was estimated to have a unit weight of 132 pcf and a friction angle of 42 degrees. To achieve 100 percent of the ultimate passive resistance, the bench width extending from the concrete seawall at constant elevation should be at least two times the riprap height, **D**. A reduced ultimate passive resistance is also provided on Plate 12 for riprap that is sloped at 3 to 1 (horizontal to vertical) starting at the face of the seawall. For riprap, we estimate that the displacement to achieve ultimate passive pressure resistance is approximately 2 percent of the depth of embedment of the seawall below the top of riprap, **Z**. Oftentimes, the displacement to achieve ultimate passive earth pressures exceeds the allowable displacement of the structure. We estimate that approximately 85 percent of the ultimate passive resistance will be mobilized with a displacement of 1 percent of **Z**, and 50 percent of the ultimate passive resistance will be mobilized with a displacement of 0.3 percent of **Z**.

The addition of riprap will initiate consolidation of the young bay mud, and settlement of the riprap surface over time. Based on historic plans of the seawall construction, we assumed that rock dike was placed outboard of the seawall at an approximate downward slope of 1 to 1. Over time, bay mud has settled on top of the rock dike. The idealized geologic section used in our consolidation analyses is included on Plate 6, with the assumed riprap revetment cross section based on the 95 percent design drawings as depicted on Plate 13 – Estimated Settlement of Riprap Revetment. We evaluated the settlement based on one-dimensional consolidation of normally consolidated bay mud. The coefficient of consolidation, Cc, was considered to be approximately 1.0 based on a consolidation test of the young bay mud and empirical correlations with water content, liquid limit and initial void ratio. Based on our analyses, we estimated the settlement of the surface of the riprap revetment after 100% consolidation of the bay mud, which is provided on Plate 13. Due to the limited thickness of bay mud expected at the seawall and pile supports from the demolished marginal wharf, we anticipate very limited settlement close to the seawall. With distance away from the seawall, where the bay mud thickens, we estimate that the riprap will settle up to approximately 25 percent of its placed height. Based on these estimates, we recommend placing the riprap at a flatter angle for the 12 feet closest to the seawall so that the post-consolidation slope is maintained at approximately 3 to 1 or flatter. An initial riprap revetment slope of 8 to 1 would allow for approximately 2.5 feet of settlement, which would represent a conservative, upper bound estimate. The bay mud will consolidate slowly, with about 90 percent of the expected settlement to be complete after 20 years. For riprap quantity estimates, some displacement of the bay mud should be anticipated during placement so that more riprap will be required than the existing bathymetric contours would suggest to reach the design grades.

Static and pseudo-static limit-equilibrium slope stability analyses were performed for the proposed riprap revetment. We evaluated the riprap revetment cross section based on the 95 percent design drawings as depicted on Plate 13. The results of the static stability analysis with the critical slip surface are shown on Plate 14 – Static Stability of Proposed Riprap Revetment. The factor of safety was calculated to be 1.0 (i.e. marginally stable). Since this does not account for the strengthening influence of existing and proposed new piles, the post-construction long term factor of safety is likely higher. If an earthquake were to occur soon after construction, the increased weight of the revetment would likely cause a bearing capacity failure and outward rotation of the toe of the riprap. Over time, the underlying bay mud will consolidate and gain strength, increasing the factor of safety against such a failure.

7.0 WHARF STRUCTURE

7.1 **General.** The proposed Brannan Street Wharf will be a pile-supported structure extending out over San Francisco Bay from the existing Embarcadero promenade. The wharf would occupy approximately 800 feet along The Embarcadero waterfront, and vary in width from 30 feet to about 140 feet. The foundation plans from the 95 percent design drawings provided by Winzler & Kelly / Structus Inc. JV are included in Appendix H – Preliminary Foundation Plans for Brannan Street Wharf. Prior to construction of the wharf, the proposed project includes extensive demolition of a portion or all of the existing wharf structures. The existing Pier 36 is at the southern end of the proposed project. Pier 36 is a reinforced concrete structure supported on reinforced concrete belled caissons. An approximately 34,500 square foot warehouse building sits on Pier 36. Pier 34 was demolished in about 2001, and formerly extended out from The Embarcadero in approximately the middle of the project. An approximately 25- to 50-foot wide marginal wharf is located along the Embarcadero extending out from the seawall.

New piles will be required to support a portion or perhaps all of the Brannan Street Wharf structure depending on the decision to re-use existing Pier 36 caissons. Pile types that are suitable include driven precast prestressed piles, steel H-piles, steel pipe piles and drilled shafts. There are trade offs with each of these pile types relating to drivability, flexibility in changing pile lengths if variable subsurface conditions are encountered, corrosion resistance, and lateral load resisting capacity. We understand that 24-inch octagonal precast, prestressed driven piles and 24-inch diameter driven steel pipe piles have been selected by Winzler & Kelly / Structus, Inc. JV for support of the wharf.

The axial capacity of existing Pier 36 caissons is addressed in Section 7.3. The axial and lateral capacities of new piles are provided in Sections 7.4 and 7.5, respectively. We also have evaluated the effect of lateral spreading ground on the proposed new piles, and included these results in Section 7.6. Lastly, corrosion considerations and construction considerations regarding pile installation are provided in Sections 7.7 and 7.8, respectively.

7.2 Demolition of Existing Piles. In general, the piles that are not incorporated into the new wharf structure should be demolished to a depth of approximately 2 feet below mudline rather than completely extracting them. Complete extraction of large quantities of piles could result in reduction of the strength characteristics of the soils. Close to the shoreline, this could lead to destabilizing of the existing seawall and cause lateral deformations. If selected piles require extraction due to conflicts with the proposed improvements, we should be consulted to assess the potential geotechnical impacts.

7.3 Axial Capacity of Pier 36 Caissons. We reviewed foundation plans of Pier 36 provided by the Port of San Francisco, which are attached in Appendix G – Historic Pier No. 36 Plans. Plans are dated March 1909. The foundations for Pier 36 within the first approximately 440 feet extending out from the seawall consist of belled caissons with a shaft diameter of 42 inches and bell diameters (excluding the cast iron bell) of between 63 and 75 inches. The majority of the caissons have a bell diameter of 63 inches. The caisson cylinder lengths gradually increase from about 41 feet near the seawall to about 50 feet at the western edge of the 440-foot long steel and concrete pier. The subsurface profile shown on the plans indicate the caissons are founded in the “Hard Bottom” which would correspond to the Upper Layered Sediments of this study. The plans suggest that the caissons were hand dug which should result in a fairly clean base at the caisson bottom.

The ultimate bearing capacity of the existing caissons was estimated based on procedures outlined in the Federal Highway Administration’s drilled shaft manual (O’Neill and Reese, 1999). Borings, laboratory tests and blow count data indicate that the upper portion of the Upper Layered Sediments is comprised of dense to very dense, poorly graded sand with silt and silty sand. Grain size and Atterberg limits tests indicate this upper sand deposit contains approximately 14 to 22 percent non-plastic silty fines. Blow count correlations, where average $(N_1)_{60}$ is approximately 35, indicate that the average friction angle is 37 degrees. Based on these soil properties, we estimate an ultimate bearing capacity of 21 tons per square foot based on the following equation provided in the FHWA manual:

$$q_{max} = 0.60\ N_{SPT}$$

where q_{max} = net ultimate unit base resistance in tons per square foot, and

N_{SPT} = average SPT blow count in blows per foot

Standard bearing capacity equations (e.g. Meyerhof, Hansen) indicate bearing capacities may be quite a bit higher, though these higher capacities are not supported by compression load tests for reasonable amounts of settlement. The FHWA equation is based on observations from compression loading tests on drilled shafts on clean bases at settlements of 5 percent of the base diameter. Therefore, we anticipate approximately 3 inches of settlement to obtain the ultimate compressive capacity. It is common to use factors of safety in the range of 2.5 to 3.0 to evaluate allowable bearing capacities of end-bearing belled caissons. At allowable bearing capacities using these factors of safety, the settlement of the caisson tops should be less than about ¾ inch.

If adjacent caissons at the existing spacing are used to support the Brannan Street Wharf deck, we recommend using a group efficiency factor of 0.9 to account for axial group effects. For caissons spaced greater than three times the bell diameter, 100 percent of the load carrying capacities may be used in the structural analysis.

7.4 Axial Capacity of New Piles. Based on preliminary information provided by Winzler & Kelly / Structus, Inc. JV, we understand that the required axial compressive capacity of the wharf piles may be on the order of 200 kips. We evaluated the axial compressive and axial tensile capacities of 24-inch octagonal precast, prestressed driven piles, 24” diameter driven steel pipe piles, and 36” diameter driven steel pipe piles. The piles will gain their resistance in both side resistance along the length of the pile and in end bearing. The allowable axial capacities of the driven piles were evaluated using computer program APILE Plus Version 5.0 (Ensoft, 2007). We evaluated pile capacities by two analysis methods: American Petroleum Institute Recommended Practice 2A (RP2A) and Federal Highway Administration (FHWA) methods. The analysis results were similar with the values obtained by the RP2A method slightly less for the soil conditions at the site.

The allowable axial capacities for the 24-inch octagonal piles are presented in Plate 15 – Axial Capacities of 24” Octagonal Piles. The allowable axial capacities for the 24” diameter and 36” diameter steel pipe piles are presented in Plates 16 and 17, respectively. We recommend that driven piles be installed a minimum of 20 feet below the base of the young bay mud into the Upper Layered Sediments primarily to ensure long-pile behavior for lateral loading. The allowable capacities are based on the RP2A method. A factor of safety of 2.0 was used for skin friction resistance and 3.0 for end bearing resistance. Plates 15 through 17 do not include the uplift resistance afforded by the buoyant weight of the piles, but can be added at the discretion of the structural engineer. Generally, lower factors of safety are allowed when considering short duration seismic loads, and if this is the case, the capacities provided on Plates 15 through 17 may be increased by one-third.

Two compression capacity curves are provided in each of Plates 15 through 17. The higher capacity curve corresponds to areas where no new fill will be placed. In areas of riprap placement, the riprap and underlying young bay mud will settle and result in downward dragloads on the piles. Dragloads of approximately 35 kips were calculated for 24” piles and approximately 50 kips were calculated for 36” piles. The skin friction resistance of the piles was decreased by the dragload, and the allowable capacities of piles in areas of new fill placement is presented as the lower of the compression capacity curves on the plates.

In evaluating the capacities of piles, the frictional resistance through fill and rock dike in the near-shore area were ignored since there will likely be disturbance due to pre-drilling at some locations. Also, the skin friction resistance in the younger bay mud stratum was not included for compressive capacities since these materials may consolidate after pile installation and create nominal dragloads on the pile as opposed to additional pile support. For axial tensile capacities, frictional resistance within the young bay mud was included, however.

The soils at the anticipated design tip elevations of the piles (elevation -70 to -95 feet based on the 95 percent design drawings) vary from very stiff sandy clay to very dense sand within the Upper Layered Sediments to stiff, fat clay within the old bay mud stratum. Significant end bearing resistance is possible within some of the thicker units of sand but is a small component of the total capacity if the piles terminate within or close to the top of a clay layer. Therefore, due to the stratification of sands and clays in the Upper Layered Sediments, we used very conservative estimates of soil shear strength for end bearing resistance estimation. A fairly consistent dense sand layer sits atop the old bay mud. From our five exploration points, the top of this sand layer ranges from elevation -67 to -80 feet and the bottom of the sand layer, corresponding to the top of old bay mud, ranges from elevation -87 to -93 feet. The thickness ranges from 10 to 20 feet. As indicated on the pile capacity plates, this sand layer may be a potential end-bearing stratum for higher-capacity driven piles. If higher capacities at shorter lengths are desirable, a pile indicator program may be beneficial to evaluate the consistency of this stratum. If piles are driven below an elevation of approximately elevation -126 feet (NAVD88) into the Lower Layered Sediments, higher allowable capacities can be achieved, if required, due to a more reliable and homogenous end-bearing stratum.

We estimate that the movement of the pile top due to imposing the allowable capacities in compression will be on the order of ½ inch for dead plus normal duration live loads. The axial deflection may increase to approximately 1 inch if these capacities are increased by one-third when evaluating additional short term seismic loads.

Piles should be spaced at least three pile diameters center to center. Axial group reduction factors for allowable capacities can be provided for piles that are spaced more closely, upon request.

7.5 **Axial Load Deflection Behavior of New Piles.** We evaluated the load deflection behavior of the piles using APILE Plus Version 5.0 (Ensoft, 2007). As part of the axial capacity calculations, axial load transfer (t-z) and tip load displacement (q-z) curves are generated for the idealized subsurface profile. These curves, or soil springs, that were used in our analyses are provided in Tables 8 through 14.

The t-z curves are provided at various depth increments along the pile shaft in Tables 8 through 11. The skin friction mobilizes linearly from zero at no pile displacement to the specified maximum unit skin friction capacity at the specified pile movement. The skin friction remains constant at larger displacements.

The q-z curves are provided in Tables 12 through 14. The non-linear load displacement curve is provided for various pile tip elevations. The elevation at which the evaluation was performed is presented on the axial capacity graphs on Plates 15, 16 and 17, although the curve is representative of the elevation ranges provided in the tables.

Table 8
Compression Springs for 24" Octagonal Piles and 24" Steel Piles

Soil Layer	Point #	Elevation (NAVD 88) (feet)	Depth below Mudline (inches)	Depth below Mudline (feet)	Maximum Unit Skin Friction Capacity, t_{max} (psi)	Pile Movement, z at t_{max} (inches)
Young Bay Mud -Normally Consolidated	1	-8.5	6	0.5	0 *	0 *
	2	-9.5	18	1.5		
	3	-10.5	30	2.5		
	4	-11.5	42	3.5		
	5	-12.5	54	4.5		
	6	-13.5	66	5.5		
	7	-14.5	78	6.5		
	8	-15.5	90	7.5		
	9	-16.5	102	8.5		
	10	-17.5	114	9.5		
	11	-18.5	126	10.5		
	12	-20.5	150	12.5		
	13	-22.5	174	14.5		
	14	-24.5	198	16.5		
	15	-26.5	222	18.5		
	16	-27.5	234	19.5		
YBM -Semi-Consolidated	17	-28.5	246	20.5	0 *	0 *
	18	-32.5	294	24.5		
	19	-34.5	318	26.5		
Upper Layered Sediments -Silty Sand	20	-35.5	330	27.5	2.794	0.1
	21	-36.5	342	28.5	3.565	0.1
	22	-40.5	390	32.5	4.729	0.1
	23	-44.5	438	36.5	5.893	0.1
	24	-48.5	486	40.5	7.058	0.1
	25	-52.5	534	44.5	8.222	0.1
ULS -Sandy Clay	26	-56.5	582	48.5	7.558	0.2
	27	-60.5	630	52.5	7.101	0.2
	28	-64.5	678	56.5	7.432	0.2
	29	-68.5	726	60.5	7.749	0.2
	30	-72.5	774	64.5	8.054	0.2
ULS -Sand	31	-76.5	822	68.5	13.89	0.1
	32	-80.5	870	72.5	13.89	0.1
	33	-84.5	918	76.5	13.89	0.1
	34	-88.5	966	80.5	20.15	0.1
Old Bay Mud	35	-92.5	1014	84.5	10.04	0.2
	36	-96.5	1062	88.5	10.26	0.2
	37	-100.5	1110	92.5	10.48	0.2
	38	-104.5	1158	96.5	10.69	0.2
	39	-108.5	1206	100.5	10.90	0.2
	40	-112.5	1254	104.5	11.11	0.2
	41	-116.5	1302	108.5	11.31	0.2
	42	-120.5	1350	112.5	11.51	0.2
	43	-124.5	1398	116.5	11.70	0.2
Lower Layered Sediments	44	-128.5	1446	120.5	13.89	0.1
	45	-132.5	1494	124.5	13.89	0.1
	46	-136.5	1542	128.5	13.89	0.1
	47	-140.5	1590	132.5	13.89	0.1
	48	-144.5	1638	136.5	13.89	0.1
	49	-148.5	1686	140.5	13.89	0.1
	50	-152.5	1734	144.5	13.89	0.1
	51	-156.5	1782	148.5	13.89	0.1
	52	-160.5	1830	152.5	13.89	0.1
	53	-164.5	1878	156.5	13.89	0.1

* Compression springs assumed to be zero to account for downdrag loads.

Table 9
Tension Springs for 24" Octagonal Piles and 24" Steel Piles

Soil Layer	Point	Elevation (NAVD 88)	Depth below Mudline		Maximum Unit Skin Friction Capacity, t_{max}	Pile Movement, z at t_{max}
	#	(feet)	(inches)	(feet)	(psi)	(inches)
Young Bay Mud -Normally Consolidated	1	-8.5	6	0.5	0.2730	0.2
	2	-9.5	18	1.5	0.2988	0.2
	3	-10.5	30	2.5	0.3419	0.2
	4	-11.5	42	3.5	0.3727	0.2
	5	-12.5	54	4.5	0.3971	0.2
	6	-13.5	66	5.5	0.4177	0.2
	7	-14.5	78	6.5	0.4356	0.2
	8	-15.5	90	7.5	0.4573	0.2
	9	-16.5	102	8.5	0.4849	0.2
	10	-17.5	114	9.5	0.5126	0.2
	11	-18.5	126	10.5	0.5390	0.2
	12	-20.5	150	12.5	0.5881	0.2
	13	-22.5	174	14.5	0.6335	0.2
	14	-24.5	198	16.5	0.6758	0.2
	15	-26.5	222	18.5	0.7156	0.2
	16	-27.5	234	19.5	0.7346	0.2
YBM -Semi-Consolidated	17	-28.5	246	20.5	0.9774	0.2
	18	-32.5	294	24.5	1.324	0.2
	19	-34.5	318	26.5	1.384	0.2
Upper Layered Sediments -Silty Sand	20	-35.5	330	27.5	2.002	0.1
	21	-36.5	342	28.5	2.554	0.1
	22	-40.5	390	32.5	3.389	0.1
	23	-44.5	438	36.5	4.223	0.1
	24	-48.5	486	40.5	5.058	0.1
	25	-52.5	534	44.5	5.892	0.1
ULS -Sandy Clay	26	-56.5	582	48.5	5.416	0.2
	27	-60.5	630	52.5	5.089	0.2
	28	-64.5	678	56.5	5.326	0.2
	29	-68.5	726	60.5	5.553	0.2
	30	-72.5	774	64.5	5.771	0.2
ULS -Sand	31	-76.5	822	68.5	11.59	0.1
	32	-80.5	870	72.5	12.54	0.1
	33	-84.5	918	76.5	13.49	0.1
	34	-88.5	966	80.5	14.44	0.1
Old Bay Mud	35	-92.5	1014	84.5	7.19	0.2
	36	-96.5	1062	88.5	7.35	0.2
	37	-100.5	1110	92.5	7.509	0.2
	38	-104.5	1158	96.5	7.662	0.2
	39	-108.5	1206	100.5	7.812	0.2
	40	-112.5	1254	104.5	7.959	0.2
	41	-116.5	1302	108.5	8.103	0.2
	42	-120.5	1350	112.5	8.245	0.2
	43	-124.5	1398	116.5	8.385	0.2
Lower Layered Sediments	44	-128.5	1446	120.5	13.89	0.1
	45	-132.5	1494	124.5	13.89	0.1
	46	-136.5	1542	128.5	13.89	0.1
	47	-140.5	1590	132.5	13.89	0.1
	48	-144.5	1638	136.5	13.89	0.1
	49	-148.5	1686	140.5	13.89	0.1
	50	-152.5	1734	144.5	13.89	0.1
	51	-156.5	1782	148.5	13.89	0.1
	52	-160.5	1830	152.5	13.89	0.1
	53	-164.5	1878	156.5	13.89	0.1

Table 10
Compression Springs for 36" Steel Piles

Soil Layer	Point #	Elevation (NAVD 88) (feet)	Depth below Mudline (inches)	Depth below Mudline (feet)	Maximum Unit Skin Friction Capacity, t_{max} (psi)	Pile Movement, z at t_{max} (inches)
Young Bay Mud -Normally Consolidated	1	-8.5	6	0.5	0 *	0 *
	2	-9.5	18	1.5		
	3	-10.5	30	2.5		
	4	-11.5	42	3.5		
	5	-12.5	54	4.5		
	6	-13.5	66	5.5		
	7	-14.5	78	6.5		
	8	-15.5	90	7.5		
	9	-16.5	102	8.5		
	10	-17.5	114	9.5		
	11	-18.5	126	10.5		
	12	-20.5	150	12.5		
	13	-22.5	174	14.5		
	14	-24.5	198	16.5		
	15	-26.5	222	18.5		
	16	-27.5	234	19.5		
YBM -Semi-Consolidated	17	-28.5	246	20.5	0 *	0 *
	18	-32.5	294	24.5		
	19	-34.5	318	26.5		
Upper Layered Sediments -Silty Sand	20	-35.5	330	27.5	2.794	0.1
	21	-36.5	342	28.5	3.565	0.1
	22	-40.5	390	32.5	4.729	0.1
	23	-44.5	438	36.5	5.893	0.1
	24	-48.5	486	40.5	7.058	0.1
	25	-52.5	534	44.5	8.222	0.1
ULS -Sandy Clay	26	-56.5	582	48.5	7.558	0.3
	27	-60.5	630	52.5	7.101	0.3
	28	-64.5	678	56.5	7.432	0.3
	29	-68.5	726	60.5	7.749	0.3
	30	-72.5	774	64.5	8.054	0.3
ULS -Sand	31	-76.5	822	68.5	13.89	0.1
	32	-80.5	870	72.5	13.89	0.1
	33	-84.5	918	76.5	13.89	0.1
	34	-88.5	966	80.5	20.15	0.1
Old Bay Mud	35	-92.5	1014	84.5	10.04	0.3
	36	-96.5	1062	88.5	10.26	0.3
	37	-100.5	1110	92.5	10.480	0.3
	38	-104.5	1158	96.5	10.690	0.3
	39	-108.5	1206	100.5	10.900	0.3
	40	-112.5	1254	104.5	11.110	0.3
	41	-116.5	1302	108.5	11.310	0.3
	42	-120.5	1350	112.5	11.510	0.3
	43	-124.5	1398	116.5	11.70	0.3
Lower Layered Sediments	44	-128.5	1446	120.5	13.89	0.1
	45	-132.5	1494	124.5	13.89	0.1
	46	-136.5	1542	128.5	13.89	0.1
	47	-140.5	1590	132.5	13.89	0.1
	48	-144.5	1638	136.5	13.89	0.1
	49	-148.5	1686	140.5	13.89	0.1
	50	-152.5	1734	144.5	13.89	0.1
	51	-156.5	1782	148.5	13.89	0.1
	52	-160.5	1830	152.5	13.89	0.1
	53	-164.5	1878	156.5	13.89	0.1

* Compression springs assumed to be zero to account for downdrag loads.

Table 11
Tension Springs for 36" Steel Piles

Soil Layer	Point #	Elevation (NAVD 88) (feet)	Depth below Mudline (inches)	Depth below Mudline (feet)	Maximum Unit Skin Friction Capacity, t_{max} (psi)	Pile Movement, z at t_{max} (inches)
Young Bay Mud -Normally Consolidated	1	-8.5	6	0.5	0.2730	0.3
	2	-9.5	18	1.5	0.2988	0.3
	3	-10.5	30	2.5	0.3419	0.3
	4	-11.5	42	3.5	0.3727	0.3
	5	-12.5	54	4.5	0.3971	0.3
	6	-13.5	66	5.5	0.4177	0.3
	7	-14.5	78	6.5	0.4356	0.3
	8	-15.5	90	7.5	0.4573	0.3
	9	-16.5	102	8.5	0.4849	0.3
	10	-17.5	114	9.5	0.5126	0.3
	11	-18.5	126	10.5	0.5390	0.3
	12	-20.5	150	12.5	0.5881	0.3
	13	-22.5	174	14.5	0.6335	0.3
	14	-24.5	198	16.5	0.6758	0.3
	15	-26.5	222	18.5	0.7156	0.3
	16	-27.5	234	19.5	0.7346	0.3
YBM -Semi-Consolidated	17	-28.5	246	20.5	0.9774	0.3
	18	-32.5	294	24.5	1.324	0.3
	19	-34.5	318	26.5	1.384	0.3
Upper Layered Sediments -Silty Sand	20	-35.5	330	27.5	2.002	0.1
	21	-36.5	342	28.5	2.554	0.1
	22	-40.5	390	32.5	3.389	0.1
	23	-44.5	438	36.5	4.223	0.1
	24	-48.5	486	40.5	5.058	0.1
	25	-52.5	534	44.5	5.892	0.1
ULS -Sandy Clay	26	-56.5	582	48.5	5.416	0.3
	27	-60.5	630	52.5	5.089	0.3
	28	-64.5	678	56.5	5.326	0.3
	29	-68.5	726	60.5	5.553	0.3
	30	-72.5	774	64.5	5.771	0.3
ULS -Sand	31	-76.5	822	68.5	11.59	0.1
	32	-80.5	870	72.5	12.54	0.1
	33	-84.5	918	76.5	13.49	0.1
	34	-88.5	966	80.5	14.44	0.1
Old Bay Mud	35	-92.5	1014	84.5	7.19	0.3
	36	-96.5	1062	88.5	7.35	0.3
	37	-100.5	1110	92.5	7.509	0.3
	38	-104.5	1158	96.5	7.662	0.3
	39	-108.5	1206	100.5	7.812	0.3
	40	-112.5	1254	104.5	7.959	0.3
	41	-116.5	1302	108.5	8.103	0.3
	42	-120.5	1350	112.5	8.245	0.3
	43	-124.5	1398	116.5	8.385	0.3
Lower Layered Sediments	44	-128.5	1446	120.5	13.89	0.1
	45	-132.5	1494	124.5	13.89	0.1
	46	-136.5	1542	128.5	13.89	0.1
	47	-140.5	1590	132.5	13.89	0.1
	48	-144.5	1638	136.5	13.89	0.1
	49	-148.5	1686	140.5	13.89	0.1
	50	-152.5	1734	144.5	13.89	0.1
	51	-156.5	1782	148.5	13.89	0.1
	52	-160.5	1830	152.5	13.89	0.1
	53	-164.5	1878	156.5	13.89	0.1

Table 12
End Bearing Springs for 24" Octagonal Piles

Soil Layer	Tip Elevation of Pile (NAVD 88) (feet)	Depth below Mudline (inches)	(feet)	Mobilized End Bearing, q (lbs)	Pile Movement, z (inches)
Upper Layered Sediments	-55 to -89	564 to 972	47 to 81	0	0.00
				14,900	0.05
				29,800	0.31
				44,700	1.01
				53,600	1.75
				59,600	2.40
				59,600	6.00
Old Bay Mud	-89 to -126	972 to 1416	81 to 118	0	0.00
				16,400	0.05
				32,800	0.31
				49,200	1.01
				59,000	1.75
				65,600	2.40
				65,600	6.00
Lower Layered Sediments	-126 to -138	1416 to 1560	118 to 130	0	0.00
				165,500	0.05
				331,000	0.31
				496,500	1.01
				595,800	1.75
				662,000	2.40
				662,000	6.00

Table 13
End Bearing Springs for 24" Steel Piles

Soil Layer	Tip Elevation of Pile (NAVD 88) (feet)	Depth below Mudline (inches)	Depth below Mudline (feet)	Mobilized End Bearing, q (lbs)	Pile Movement, z (inches)
Upper Layered Sediments	-55 to -89	564 to 972	47 to 81	0	0.00
				14,100	0.05
				28,300	0.31
				42,400	1.01
				50,900	1.75
				56,500	2.40
				56,500	6.00
Old Bay Mud	-89 to -126	972 to 1416	81 to 118	0	0.00
				15,600	0.05
				31,100	0.31
				46,700	1.01
				56,000	1.75
				62,200	2.40
				62,200	6.00
Lower Layered Sediments	-126 to -138	1416 to 1560	118 to 130	0	0.00
				157,000	0.05
				314,000	0.31
				471,000	1.01
				565,200	1.75
				628,000	2.40
				628,000	6.00

Table 14
End Bearing Springs for 36" Steel Piles

Soil Layer	Tip Elevation of Pile (NAVD 88) (feet)	Depth below Mudline (inches)	Depth below Mudline (feet)	Mobilized End Bearing, q (lbs)	Pile Movement, z (inches)
Upper Layered Sediments	-55 to -89	564 to 972	47 to 81	0	0.00
				31,800	0.07
				63,600	0.47
				95,400	1.51
				114,500	2.63
				127,200	3.60
				127,200	6.00
Old Bay Mud	-89 to -126	972 to 1416	81 to 118	0	0.00
				35,000	0.07
				70,000	0.47
				105,000	1.51
				126,000	2.63
				140,000	3.60
				140,000	6.00
Lower Layered Sediments	-126 to -138	1416 to 1560	118 to 130	0	0.00
				255,825	0.07
				511,650	0.47
				767,475	1.51
				920,970	2.63
				1,023,300	3.60
				1,023,300	6.00

7.6 Lateral Load Capacities. Resistance to lateral loading will be provided by passive resistance of the soil against the pile. In lateral load analyses, non-linear soil springs are applied at each depth increment, and are represented by soil resistance, **p**, at a lateral deflection, **y**. Soil parameters to generate "p-y" springs in the computer program LPILE are provided for two idealized soil profiles in Tables 15 and 16 – LPILE Parameters for Lateral Analysis. These springs can then be used to evaluate the load-deflection response of individual piles. Table 15 provides LPILE parameters for lateral analysis of piles or drilled shafts for the wharf structure, while Table 16 provides these parameters for the existing seawall.

The "p-y" springs generated using the soil parameters provided in Tables 15 and 16 are applicable for individual piles only. Lateral response of piles in a group is affected by pile spacing, pile orientation, and direction of loading. If the spacing of individual piles is at least five pile diameters, the piles may be assumed to develop their full capacity. If the piles are spaced three pile diameters, there will be overlapping of shear failure planes, and the ultimate resistance of the piles in a group will be less than the summation of the ultimate resistance of individual piles. At a spacing of three pile diameters in the direction of loading, we recommend the ultimate soil resistance, $\mathbf{p_{ult}}$, should be reduced to 90 percent of $\mathbf{p_{ult}}$ of an individual pile for the leading row of piles in the group and to 70 percent of $\mathbf{p_{ult}}$ for the trailing piles in a group. If the side-to-side spacing is also at three pile diameters, an additional reduction factor should be applied so that the leading piles are 85 percent of $\mathbf{p_{ult}}$ and the trailing piles are 65 percent of $\mathbf{p_{ult}}$. For spacing between three and five pile diameters, the ultimate soil resistance reduction factors may be obtained by interpolation. Some manipulation of the soil strength parameters or manually inputting "p-y" springs will be necessary to correctly model piles in a group using the LPILE program. Alternatively, computer program GROUP can be used to model a segment of the wharf structure, which will internally generate group efficiency factors.

TABLE 15 –LPILE PARAMETERS FOR LATERAL ANALYSIS OF PILES/DRILLED SHAFTS FOR WHARF STRUCTURE

Modeled Soil/Rock Unit	Depth Below Existing Pier Deck (feet)	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Soil Strength		Lateral Pile Capacity Assessment (e.g. "LPILE", "GROUP")		
				c (psf)	**ϕ** (deg.)	**Soil Type for p-y curve**	**k** (pci)	**ε_{50}**
Young Bay Mud - Normally Consolidated (Qybm)	19 to 43	90	28	100 to 300	-	"Soft Clay"	-	0.04 to 0.02
Young Bay Mud - Semi-Consolidated (Qybm)	43 to 48	95	33	500	-	"Soft Clay"	-	0.015
Upper Layered Sediments -Silty Sand (Qul)	48 to 70	135	73	-	35	"Sand"	90	-
Upper Layered Sediments -Sandy Clay (Qul)	70 to 84	124	61	1500	-	"Stiff Clay w/o Free Water"	-	0.007
Upper Layered Sediments -Sand (Qul)	84 to 100	136	74	-	38	"Sand"	125	-
Old Bay Mud (Qobm)	100 to 130	115	53	1600	-	"Soft Clay"	-	0.005

NOTES:

1. *c = cohesion, ϕ = friction angle, k = modulus of horizontal subgrade reaction for LPILE and GROUP analysis, and ε_{50} = value of strain at 50% maximum stress for clays in LPILE and GROUP analysis.*

TABLE 16 – LPILE PARAMETERS FOR LATERAL ANALYSIS OF PILES/DRILLED SHAFTS FOR SEAWALL

Modeled Soil/Rock Unit	Depth Below Existing Sidewalk	Total Unit Weight	Effective Unit Weight	Soil Strength		Lateral Pile Capacity Assessment (e.g. "LPILE", "GROUP")		
				c	ϕ	Soil Type for p-y curve	k	ε_{50}
	(feet)	(pcf)	(pcf)	(psf)	(deg.)		(pci)	
Artificial Fill, above GWT (af)[2]	1 to 8	125	125	800	-	"Stiff Clay w/o Free Water"	-	0.01
Artificial Fill, below GWT (af)[2]	8 to 13	125	63	800	-	"Stiff Clay w/o Free Water"	-	0.01
Rock Dike (af)	13 to 38	130	68	0	32	"Sand"	60	
Young Bay Mud - Below Rock Dike (Qybm)	38 to 43	110	48	1200	-	"Soft Clay"	-	0.01
Upper Layered Sediments -Silty Sand (Qul)	43 to 62	135	73	-	35	"Sand"	90	-
Upper Layered Sediments -Sandy Clay (Qul)	62 to 80	124	61	1500	-	"Stiff Clay w/o Free Water"	-	0.007
Upper Layered Sediments - Sand (Qul)	80 to 100	136	74	-	38	"Sand"	125	-
Old Bay Mud (Qobm)	100 to 130	115	53	1600	-	"Soft Clay"	-	0.005

NOTES:

1. *c = cohesion, ϕ = friction angle, k = modulus of horizontal subgrade reaction for LPILE and GROUP analysis, and ε_{50} = value of strain at 50% maximum stress for clays in LPILE and GROUP analysis.*
2. *Discount artificial fill layer for analysis of piles supporting seawall.*

7.7 Effect of Lateral Spreading Ground on Pile Foundations. Extensive damage has occurred to pile foundations from lateral spreading ground during past earthquakes, especially when a non-liquefied crust moves laterally over a liquefiable layer. This geotechnical topic has therefore been researched fairly extensively in recent years (Dobry et al., 2003; Juirnarongrit and Ashford, 2006; Brandenberg et al., 2007a and 2007b; White et al., 2008).

In Section 5, we provided our evaluation of the shoreline global stability analysis, and concluded that the lateral spread deformation at Brannan Street Wharf may range up to approximately 1 to 2 feet during the MCE seismic event. Less displacement would occur for smaller seismic events. The greatest strains caused by the lateral spread will be within the young bay mud stratum. The rock dike and the seawall will likely strain less. Nevertheless, a lateral translation of the entire soil profile above the slide plane, assumed to be at the young bay mud / Upper Layered Sediments contact, would provide a conservative evaluation of the effect of lateral spreading ground on pile foundations. The translation may be assumed to taper from its maximum value at mid-depth of the bay mud layer below the rock dike to zero at the top of the Upper Layered Sediments (i.e. a taper depth of 2.5 feet).

Based on the slope stability analysis results (Plate 9), piles within approximately 75 feet of the seawall will be loaded laterally by the lateral spreading ground to some degree, with the greatest effect to piles that are installed through existing rock dike or proposed new riprap revetment. The wharf piles within the existing rock dike and proposed new riprap should be designed to account for the additional lateral loads.

Quantifying the effect of lateral spreading ground on the pile foundations for Brannan Street Wharf (Winzler & Kelly / Structus, Inc. JV, 2010) is complicated. The effects on a single row of widely-spaced piles and the effects on a continuous wall are fairly well understood, but the multiple rows of relatively closely-spaced piles at Brannan Street Wharf introduce a level of complexity. The foundations as presently proposed will, to some extent, reinforce the shoreline and limit the amount of lateral spread. On the other hand, the stabilizing influence of the piles will attract more load as the soil is less able to fail around the piles.

Several methods are available to evaluate the slope behavior, the lateral loading on the piles, and the deflections, shears and moments along their lengths. These include limit equilibrium methods (e.g. SLOPE/W) where the stabilizing piles are modeled as reinforcing elements that increase the factor of safety for slope stability. Deformations can then be computed for the slope that includes the pile foundations. Another approach is provided in the LPILE Technical Manual (Ensoft, Inc., 2004) and has been advanced by recent research. Stress development along the pile is a function of the relative movement between the soil and the pile. The load imposed by the lateral spreading ground is applied to the pile, and the deflected shape, shears and moments can be computed. A third approach would be to use finite element models (e.g. FLAC, PLAXIS) to better capture the complexity of the soil structure interaction between the laterally spreading ground and the pile foundations.

After discussion with structural engineers with Winzler & Kelly / Structus, Inc. JV, we understand an LPILE-type lateral loading will be implemented in the SAP2000 structural engineering software to evaluate the effect of lateral spreading ground on the wharf. This requires offsetting the "p-y" springs by the amount of free-field soil displacement, $\mathbf{y_s}$. If $\mathbf{y_s}$ is greater than the movement of the pile, the soil will be loading the pile. Soil resistance is only available if the pile movement is greater than $\mathbf{y_s}$. The "p-y" springs generated using the soil parameters provided in Tables 15 and 16 may be used in evaluating the effect of lateral spread displacement on the wharf piles.

7.8 **Corrosion Considerations.** Structures in a marine environment are exposed to severely corrosive soils and seawater. Unprotected steel elements will corrode, and protection of foundations and other structural elements will be required.

We recommend that pile foundations be designed, at a minimum, in accordance with Caltrans Corrosion Guidelines (Caltrans, 2003a) and Article 8.22 of the Caltrans Bridge Design Specifications (BDS) (Caltrans, 2003b). In accordance with Caltrans recommendations, the corrosion allowance for unprotected steel piling should be 0.004 inches per year for the immersed zone of the pile and 0.001 inches per year for the portion of the pile in young bay mud. Steel H piles have two exposed faces, so the corrosion allowance should be doubled for this pile type. Alternatively, concrete encasement or cathodic protection can be used to protect steel piles. Minimum concrete cover, epoxy coating of prestressing steel and other recommendations for reinforced concrete piles are included in the Caltrans BDS. It is important to provide durable concrete with a low water to cementitious materials ratio.

7.9 **Construction Considerations.** Pile driving to the desired tip elevations within the Upper Layered Sediments and old bay mud is expected to be possible provided the appropriate pile hammer is selected for the project. A hammer that can deliver enough energy to the tip of the piles to drive them efficiently and without damage should be selected. We recommend that the foundation contractor submit the specifications for the pile driving equipment for review at least a week before the start of pile installation.

Piles will experience hard driving within some of the sand units within the Upper Layered Sediments. A review of pile driving records of nearby piers (Port of San Francisco, 1961) indicates that this stratum was penetrated with a drop hammer with approximately 38,000 to 57,000 foot pounds of energy. Although the plan sheet does not indicate the pile type, Lee and Praszker (1987) indicate piles at Pier 30/32 that penetrated to 80 to 130 feet were 18- to 20-inch square reinforced concrete piles. Larger pile hammers of up to about 100,000 foot pounds may be required to drive larger diameter piles through the sand units of the Upper Layered Sediments. It is possible that some piles may encounter driving refusal in the dense sand layer at approximately elevation -67 to -93 feet (NAVD88). In this event, dowel tube lengths and pile reinforcement should allow for cutoff of the pile if it hits refusal within this sand unit. If driving to design pile tip is essential, it may be prudent to install jetting tubes in the precast, prestressed concrete piles. Also, difficult driving conditions may be encountered close to the shoreline where the rock dike seawall was constructed. Based on our subsurface exploration program, the upper and bottom parts of the rock dike may contain large boulders that could be difficult to penetrate. Also, wood debris may be encountered within the artificial fill and rock dike. Predrilling through the rock dike or spudding with a short, stout section of pile to break up hard material may be required at some pile locations.

We recommend that an indicator pile program be performed prior to casting of production piles. The indicator pile program would provide estimates of pile lengths to supplement the data obtained from the subsurface borings. Indicator piles can be driven at production pile locations. Further, we recommend that a Pile Driving Analyzer (PDA) and CAPWAP program be used for a subset of the indicator piles to evaluate the design, appropriate size of hammer, and to establish pile driving acceptance criteria for the project. Performing PDA testing during a re-strike after the pile is allowed to set will likely also be required to demonstrate that the allowable axial capacities have been attained. At least six piles, and at least two piles of each different pile type, should be tested. The locations of the piles for PDA testing should be widely spread across the site to adequately evaluate the variability in subsurface conditions. Although we do not consider that a static axial load test is required if PDA testing is performed, it may be considered as another alternative to confirm pile design and installation procedures. The production piles should be installed with the same pile driving hammer and fuel setting as the indicator piles.

Driving piles creates ground vibrations and noise during installation that may be disturbing to residents and tenants. The effect of pile driving operations on structures within approximately 50 feet of the site should be evaluated. It is prudent to perform preconstruction surveys of the buildings within approximately 100 feet or more of the site to discern between pre-construction distress and distress that may be caused by pile driving activities. Vibration and noise monitoring should be performed during the indicator pile program and during installation of production piles to evaluate peak particle velocities at adjacent structures. We should review these data as they are obtained.

8.0 CLOSURE

The conclusions and recommendations presented herein are professional opinions based on geotechnical and geologic data and the project as described. A review by this office of any foundation, excavation, grading plans and specifications, or other work product that relies on the content of this report, together with the opportunity to make supplemental recommendations is considered an integral part of this study. Should unanticipated conditions come to light during project development or should the project change from that described, we should be given the opportunity to review our recommendations.

The findings and professional opinions presented in this report are presented within the limits prescribed by the client, in accordance with generally accepted professional engineering and geologic practices. There is no other warranty, either express or implied.

Submitted by:
GEOTECHNICAL CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
DERON VAN HOFF
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Exp. 12/31/10
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STATE OF CALIFORNIA

Deron J. van Hoff 6/25/10

Deron J. van Hoff, P.E., G.E.
Geotechnical Engineer, GE 2575

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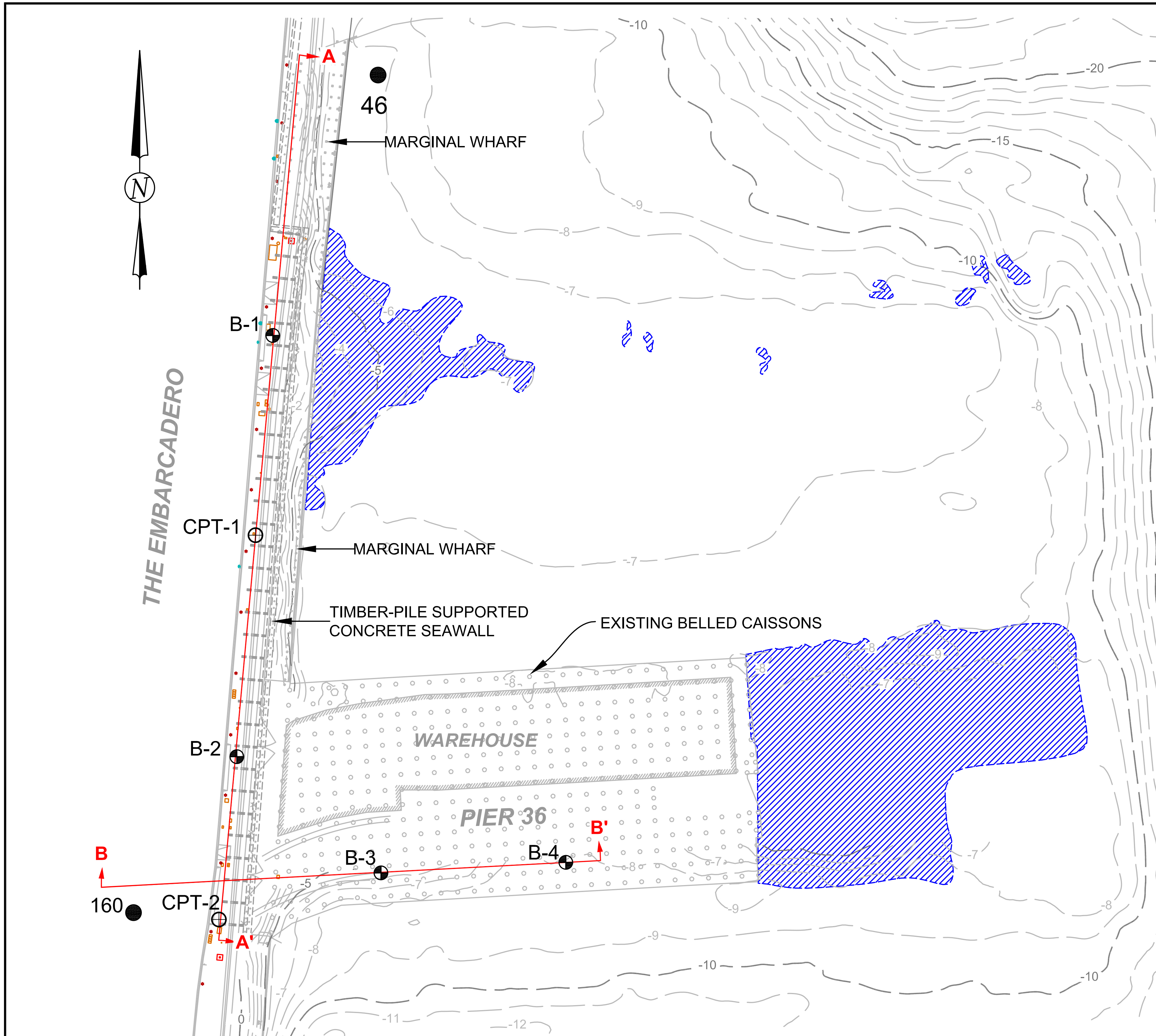
SITE LOCATION MAP	PLATE 1
BRANNAN STREET WHARF	
JUNE 2010	SF09005



LEGEND

B-3 Exploratory Boring (GTC, April 2009)

CPT-1 Cone Penetration Test (GTC, April 2009)

46 Exploratory Boring (San Francisco Port, July 1961)

A A' Geologic Cross Section Lines. See Plates 5 and 6 for Geologic Cross Sections.

Dock Ruins and Debris

GRAPHIC SCALE

50 0 25 50 100 200

(IN FEET)

1 inch = 100 ft.

Notes:

Exploration locations are approximate and based on available information provided in referenced reports.

Base map provided by Port of San Francisco, 2009. Elevations are based on NAVD 88.

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SITE PLAN AND EXPLORATION LOCATION MAP	PLATE 2
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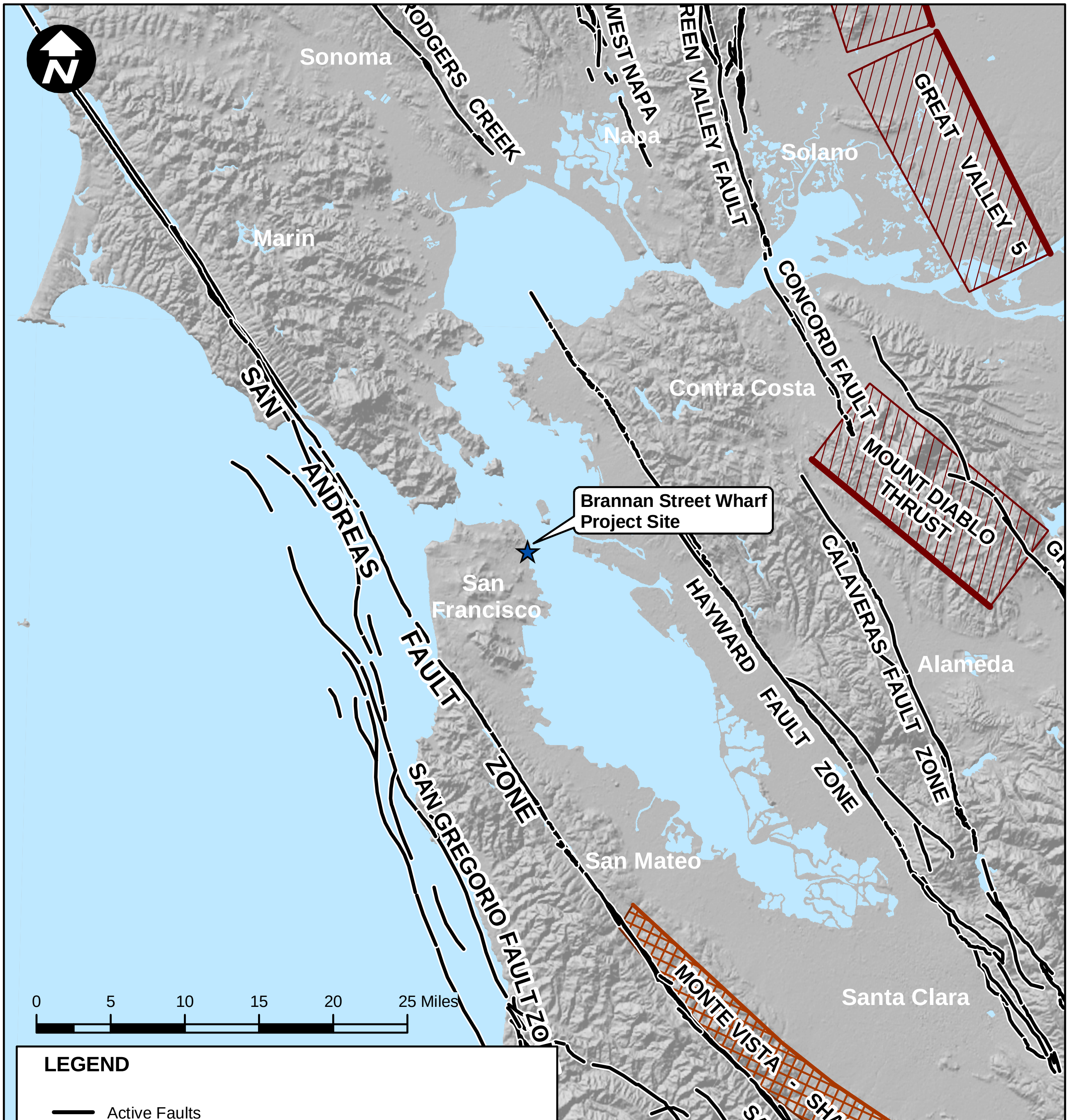

LEGEND

Active Faults

Reverse Fault (rectangle represents projection of the fault plane to the surface)

Blind Thrust Faults (faults do not intersect the surface, mapped trace represents projection of upper edge of the fault to surface; rectangle represents projection of the fault plane to the surface)

Fault Data Sources: CGS 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0, Bryant, W. A. (compiler) and Cao, et. al., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, Appendix A - 2002 California Fault Parameters

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REGIONAL ACTIVE FAULT MAP	PLATE 3
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Legend

Surficial Deposits
- Qaf - Artificial Fill
- Qd - Dune Sand
- Qu - Undifferentiated Surficial Deposits

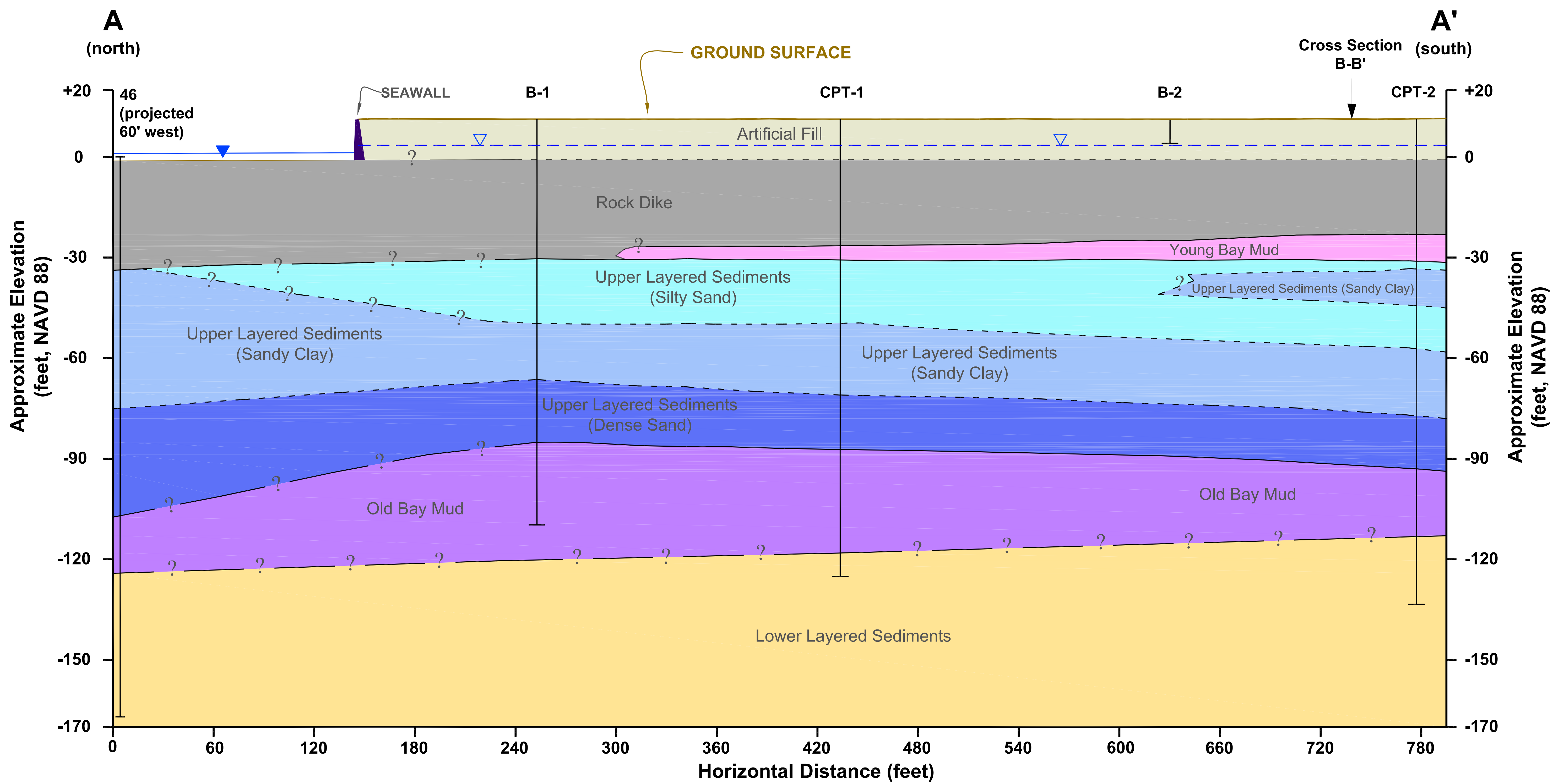
Franciscan Complex
- Kfs, KJs - Sandstone and Shale
- Kfss - Massive sandstone
- Kfsh - Thin bedded sandstone and shale

Geologic Contacts
- contact, approx. located
- bedrock fault (inactive), approx. located

Source: USGS MF-2337, Geologic map and map database of parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California: Digital database Version 1.0

0 500 1,000 2,000 Feet

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome St., Suite 402 San Francisco, CA 94111	GEOLOGIC MAP	PLATE 4
	BRANNAN STREET WHARF	
	JUNE 2010	SF09005



LEGEND

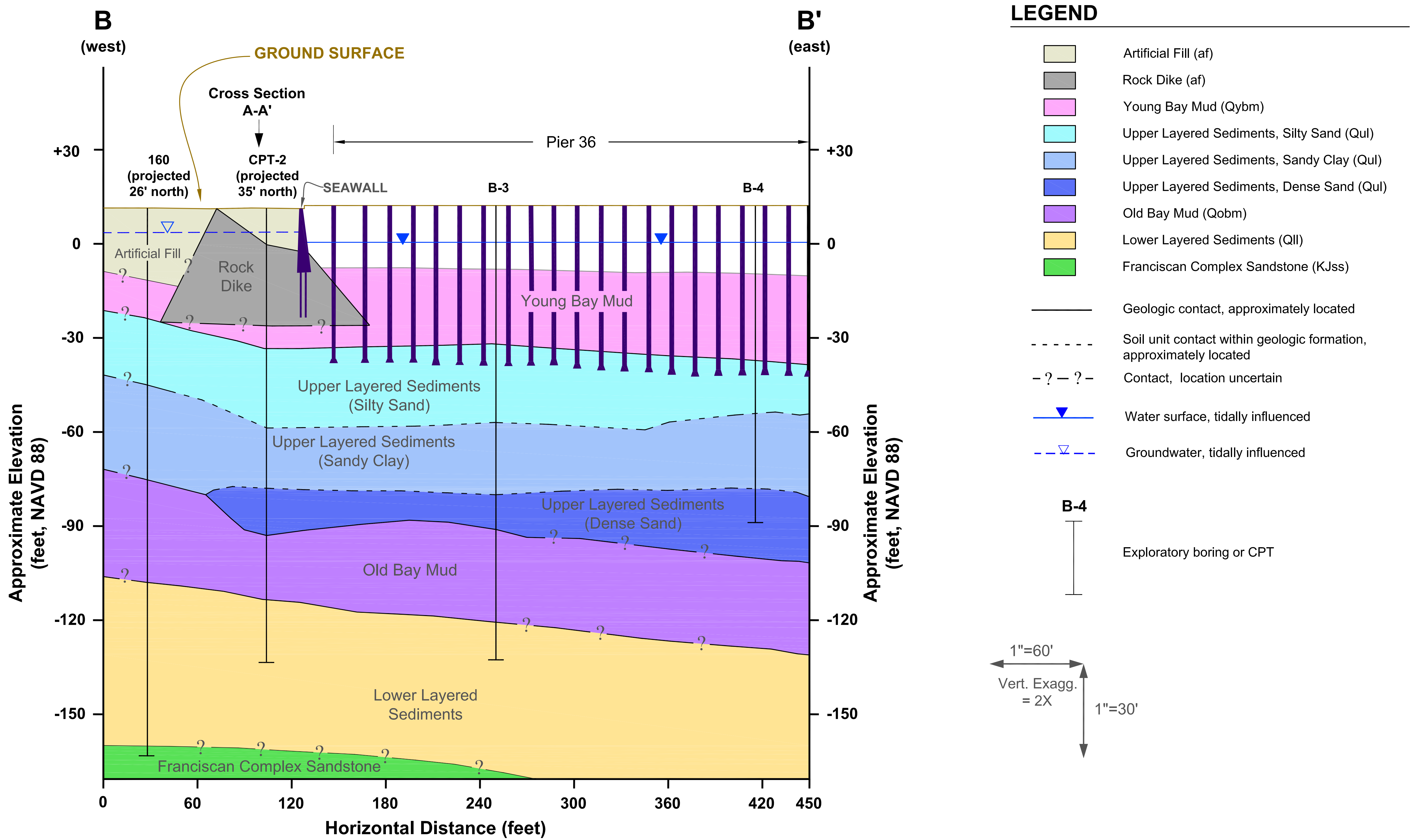
- Artificial Fill (af)
- Rock Dike (af)
- Young Bay Mud (Qybm)
- Upper Layered Sediments, Silty Sand (Qul)
- Upper Layered Sediments, Sandy Clay (Qul)
- Upper Layered Sediments, Dense Sand (Qul)
- Old Bay Mud (Qobm)
- Lower Layered Sediments (Qll)

- Geologic contact, approximately located
- Soil unit contact within geologic formation, approximately located
- Contact, location uncertain
- Water surface, tidally influenced
- Groundwater, tidally influenced

B-1 Exploratory boring or CPT

1"=60'
Vert. Exagg. = 2X
1"=30'

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome Street, Suite 402 San Francisco, CA 94111	GEOLOGIC CROSS SECTION A-A'	PLATE 5
	BRANNAN STREET WHARF	
	JUNE 2010	SF09005



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GEOLOGIC CROSS SECTION B-B'	PLATE 6
BRANNAN STREET WHARF	
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Source: CGS, 2001. Seismic Hazard Mapping Program,
Official Map of Seismic Hazard Zones, City of San Francisco.

0 500 1,000 2,000 3,000 Feet

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LIQUEFACTION HAZARD MAP	PLATE 7
BRANNAN STREET WHARF	
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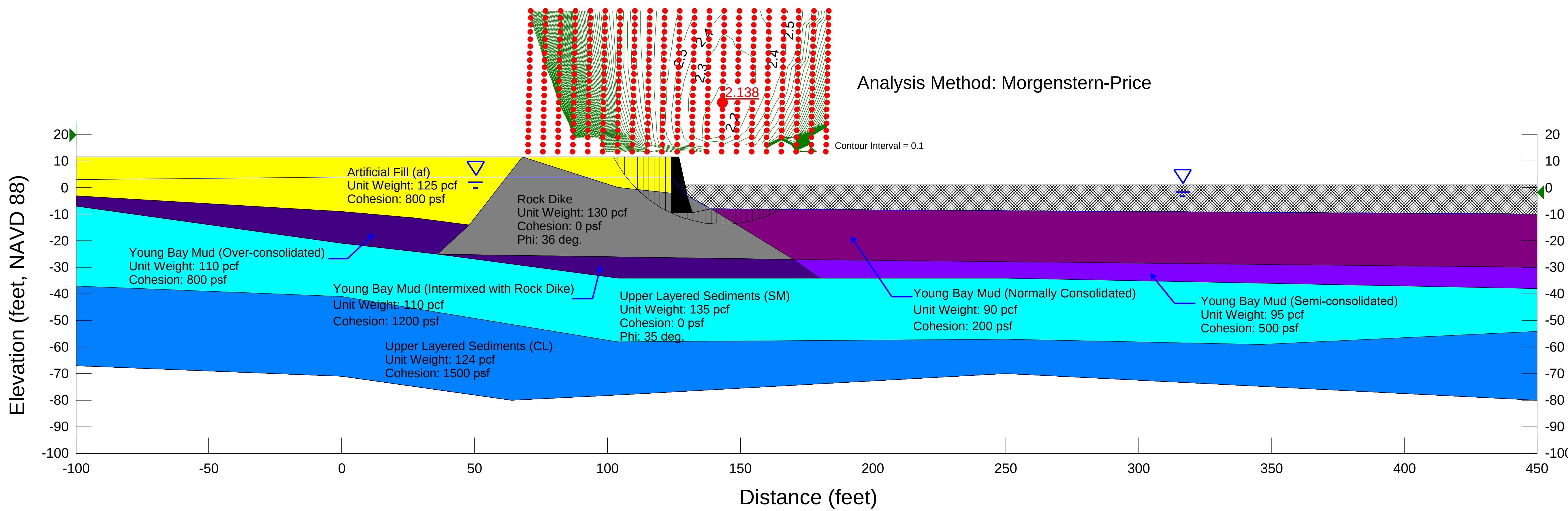

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STATIC SLOPE STABILITY SECTION

PLATE 8

BRANNAN STREET WHARF

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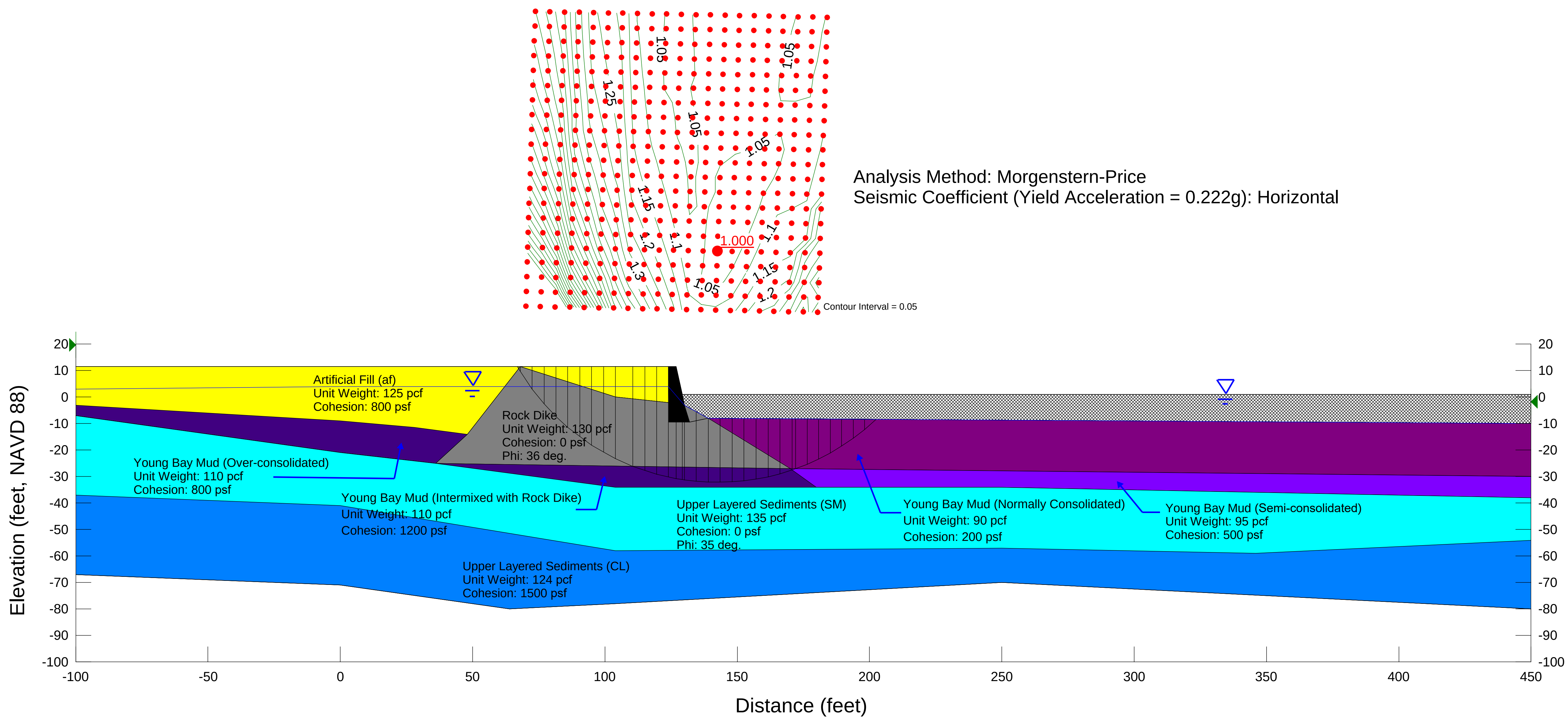

Analysis Method: Morgenstern-Price
Seismic Coefficient (Yield Acceleration = 0.222g): Horizontal

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SEISMIC SLOPE STABILITY SECTION

PLATE 9

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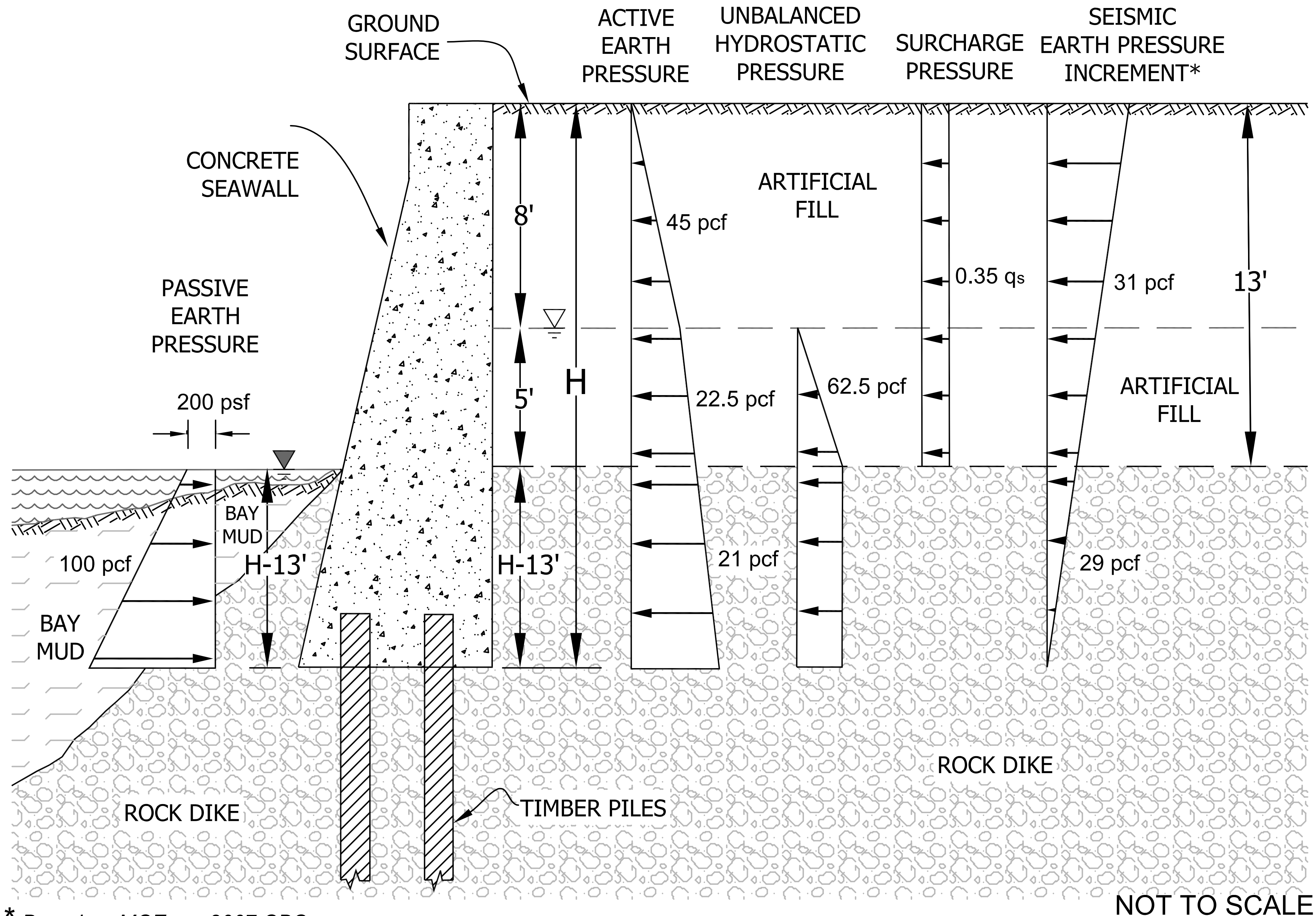

* *Based on MCE per 2007 CBC.*

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LATERAL EARTH PRESSURES	PLATE 10
BRANNAN STREET WHARF	
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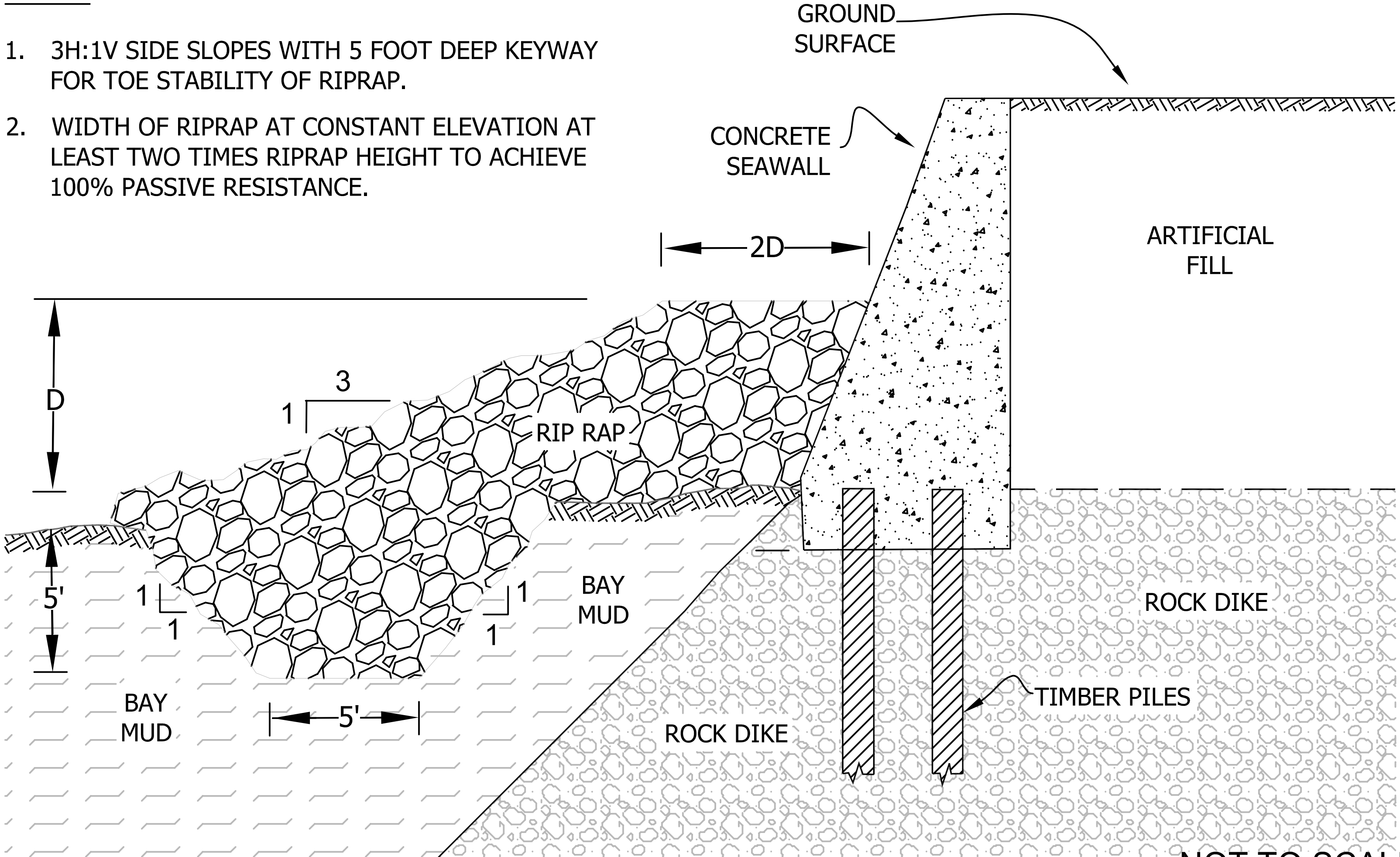
NOTES:

1. 3H:1V SIDE SLOPES WITH 5 FOOT DEEP KEYWAY FOR TOE STABILITY OF RIPRAP.
2. WIDTH OF RIPRAP AT CONSTANT ELEVATION AT LEAST TWO TIMES RIPRAP HEIGHT TO ACHIEVE 100% PASSIVE RESISTANCE.

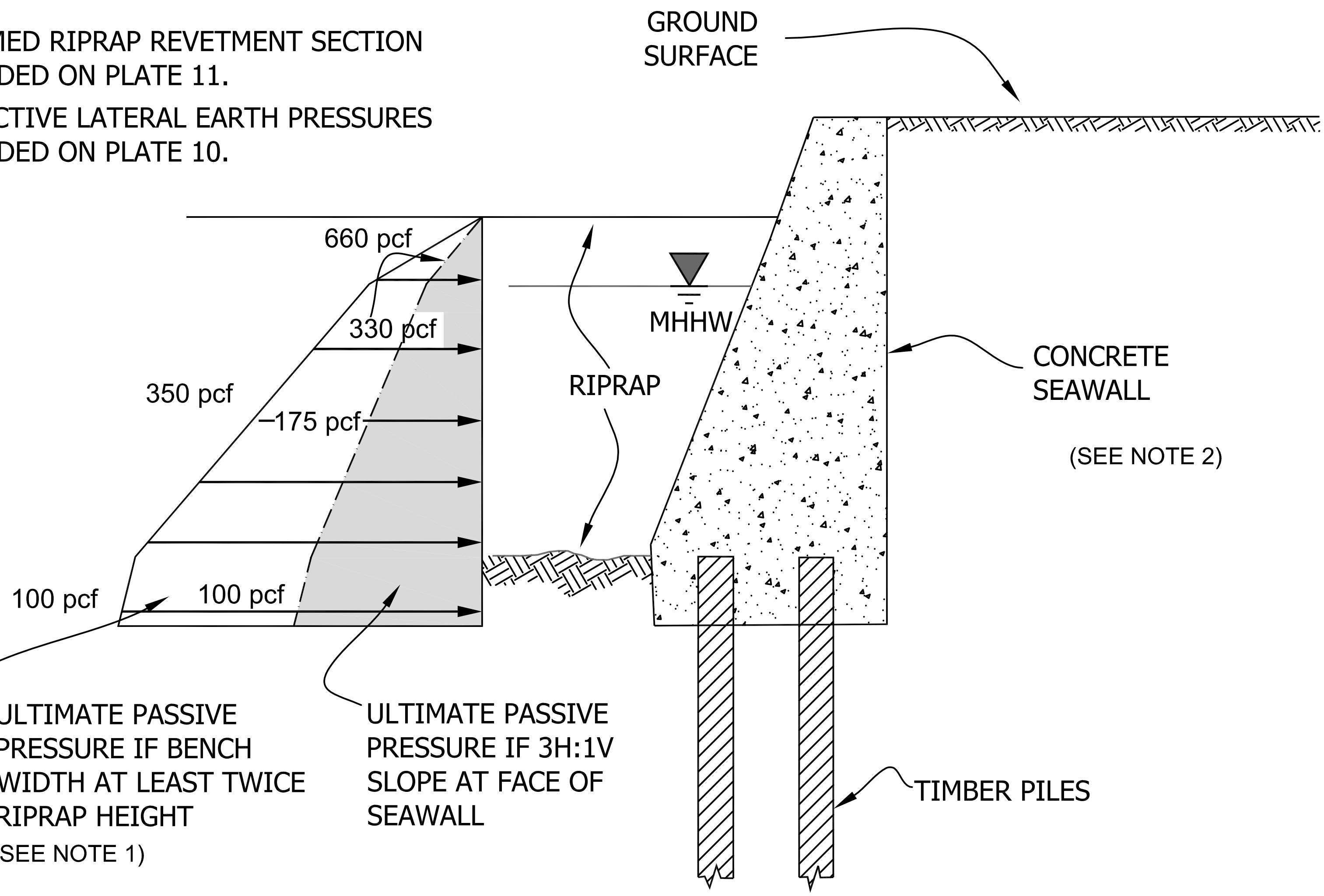


NOT TO SCALE

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome Street, Suite 402 San Francisco, CA 94111	PROPOSED RIPRAP REVETMENT FOR SEAWALL STABILITY	PLATE 11
	BRANNAN STREET WHARF	
	JUNE 2010	SF09005

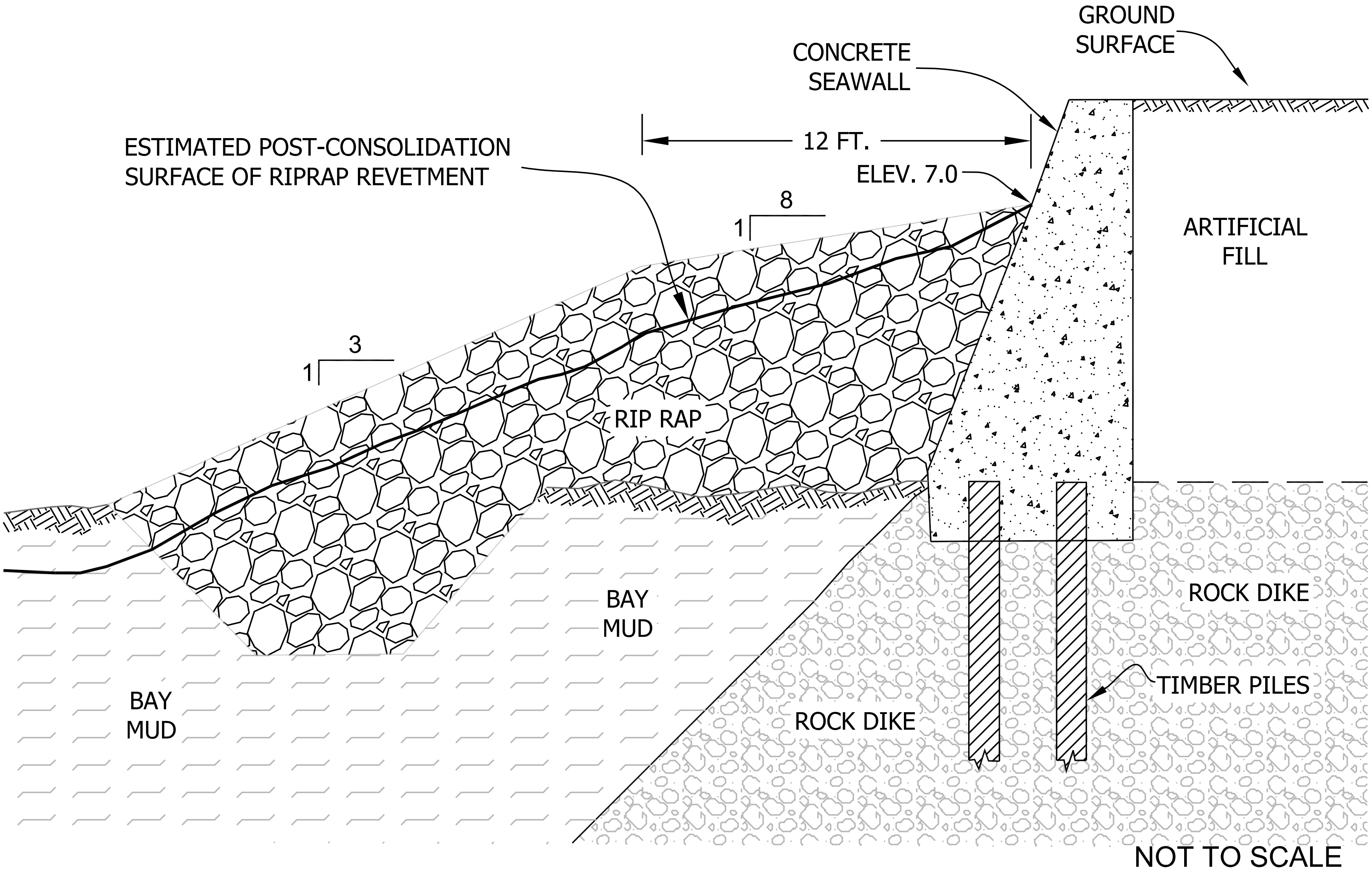
NOTES:

1. ASSUMED RIPRAP REVETMENT SECTION PROVIDED ON PLATE 11.
2. USE ACTIVE LATERAL EARTH PRESSURES PROVIDED ON PLATE 10.



NOT TO SCALE

GEOTECHNICAL CONSULTANTS, INC. 500 Sansome Street, Suite 402 San Francisco, CA 94111	PASSIVE EARTH PRESSURE OF RIPRAP REVETMENT	PLATE 12
	BRANNAN STREET WHARF	
	JUNE 2010	SF09005



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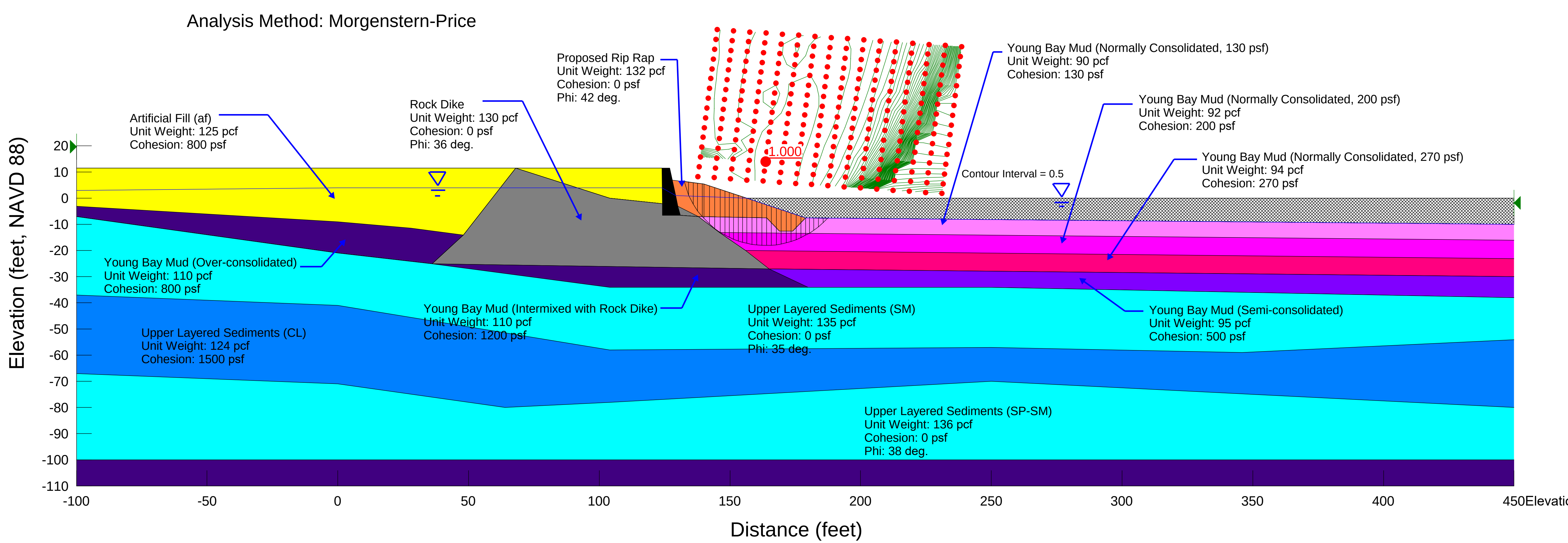
ESTIMATED SETTLEMENT OF RIPRAP REVETMENT	PLATE 13
BRANNAN STREET WHARF	
JUNE 2010	SF09005



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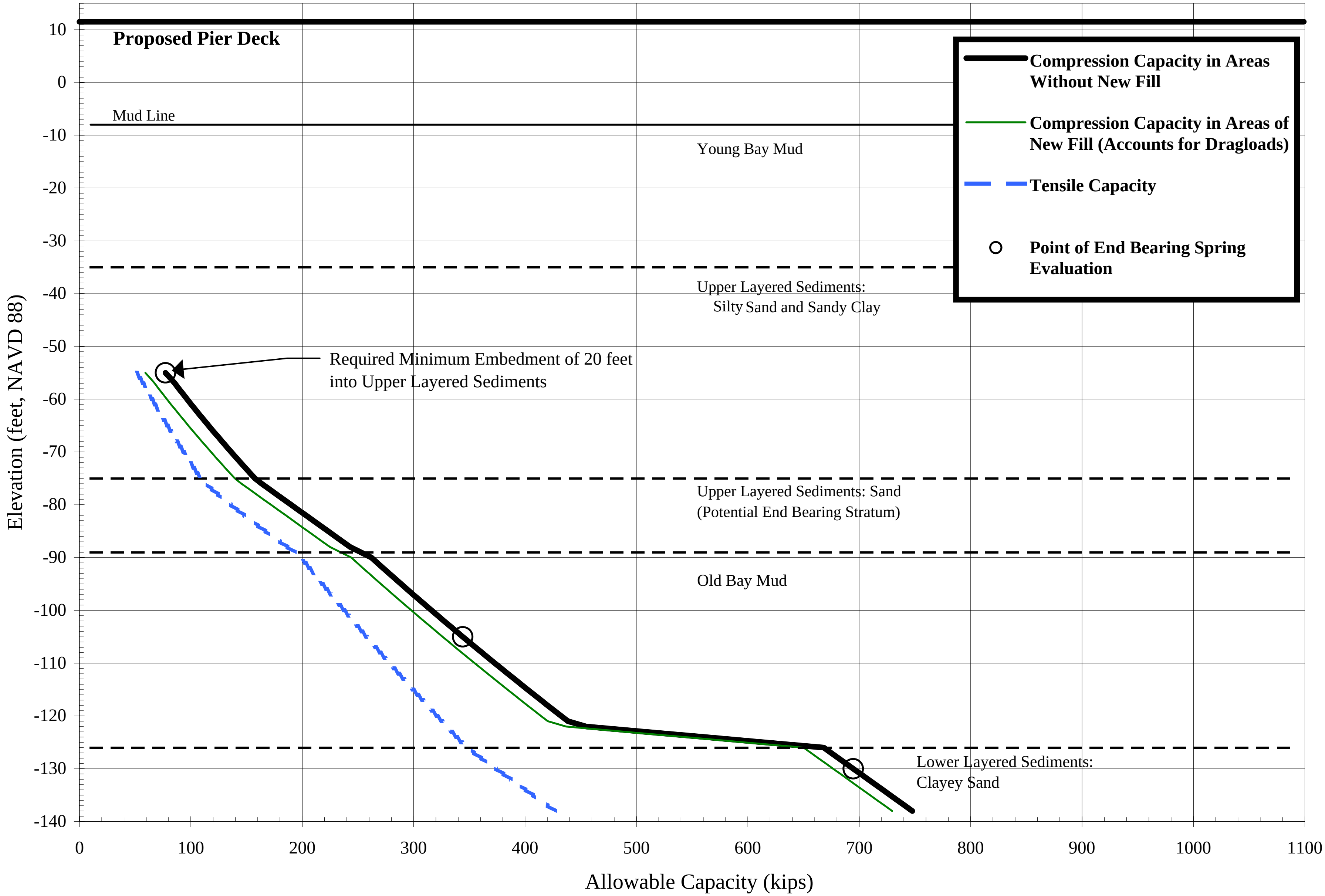
STATIC STABILITY OF PROPOSED RIPRAP REVETMENT	PLATE 14
BRANNAN STREET WHARF	
JUNE 2010	SF09005



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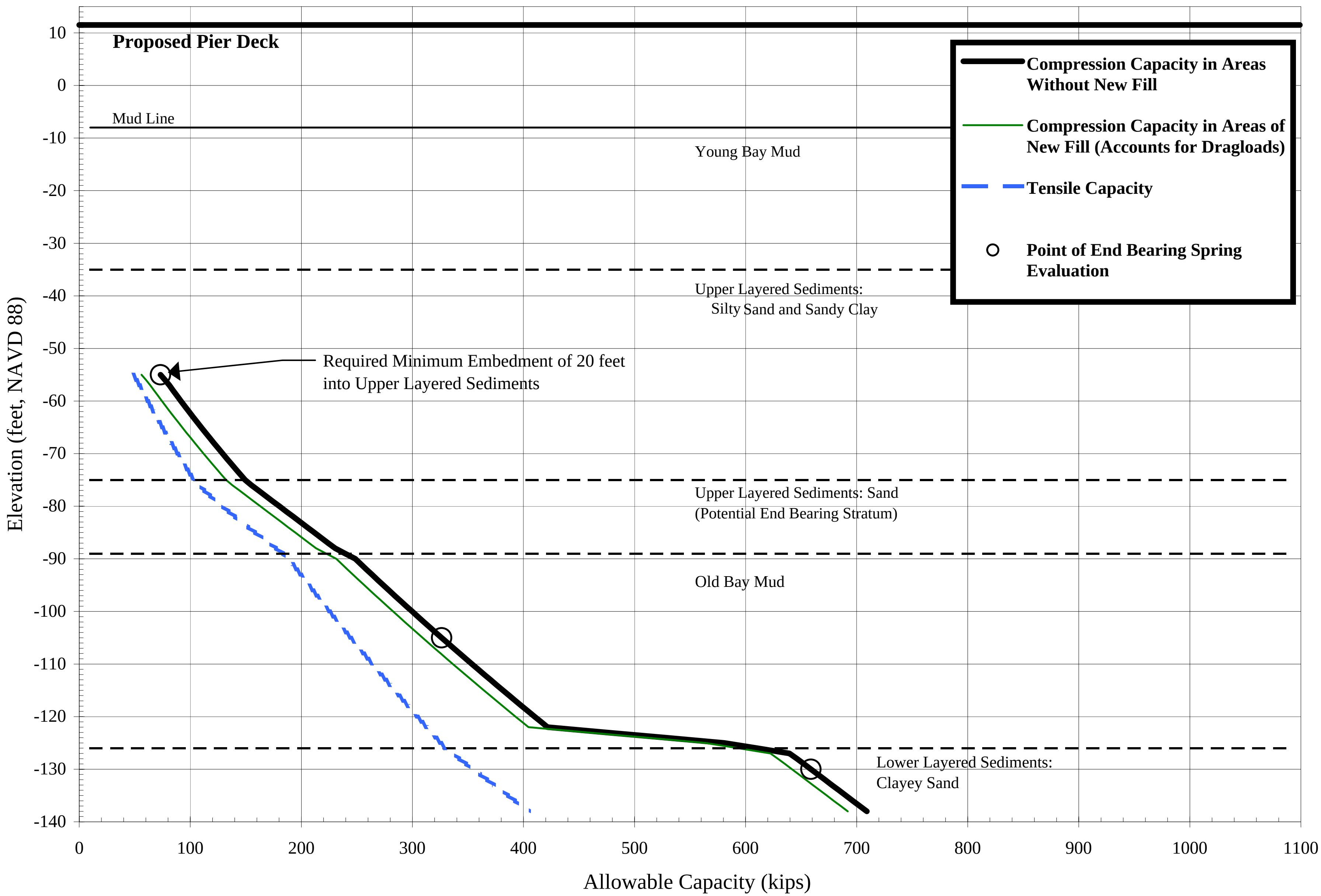
AXIAL CAPACITIES OF 24" OCTAGONAL PILES	PLATE 15
BRANNAN STREET WHARF	
JUNE 2010	SF09005



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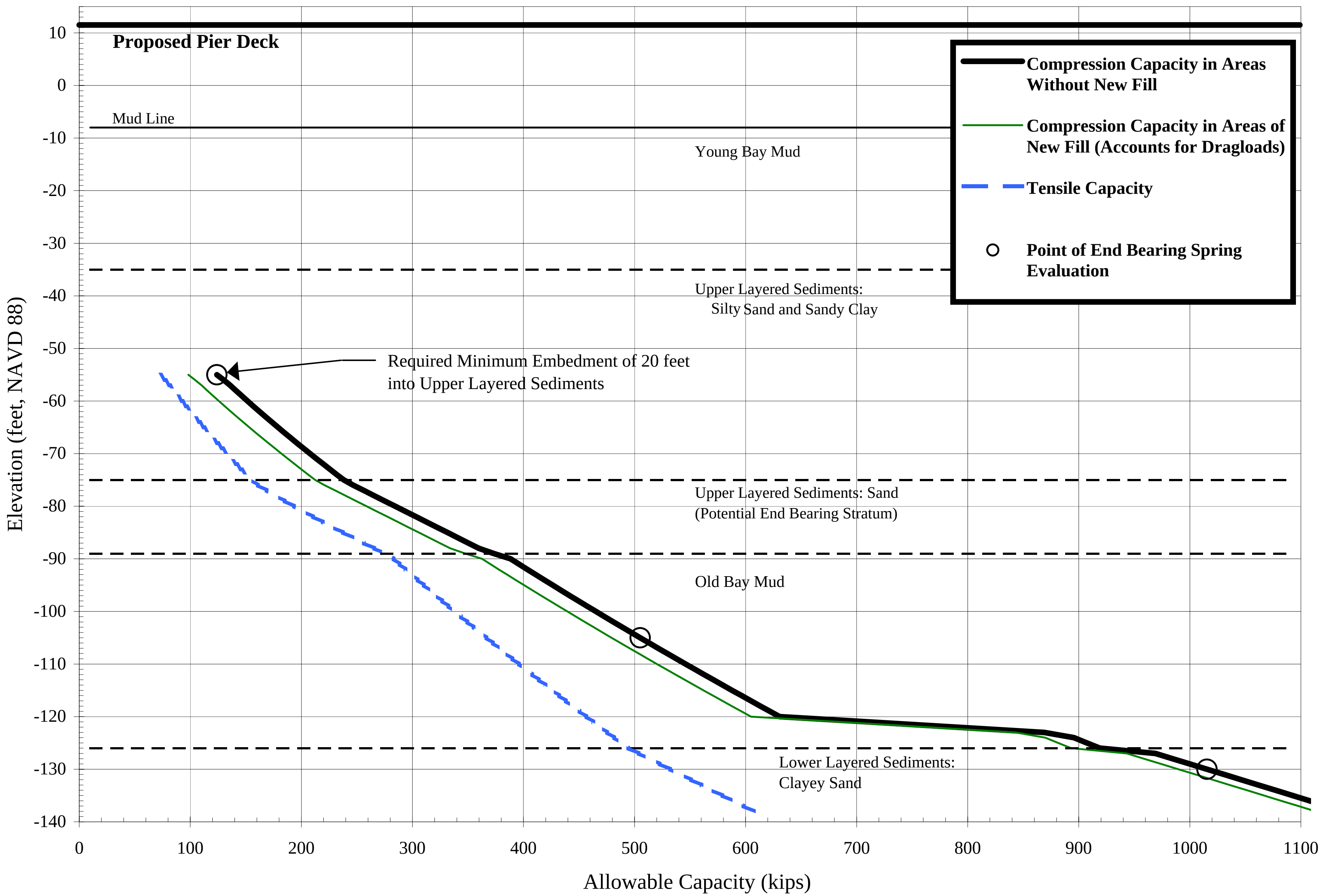
AXIAL CAPACITIES OF 24" STEEL PIPE PILES	PLATE 16
BRANNAN STREET WHARF	
JUNE 2010	SF09005



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AXIAL CAPACITIES OF 36" STEEL PIPE PILES	PLATE 17
BRANNAN STREET WHARF	
JUNE 2010	SF09005

APPENDIX A
BORING LOGS

APPENDIX A
BORING LOGS

SUBSURFACE EXPLORATION

Subsurface exploration for our geotechnical study of the Brannan Street Wharf took place between April 20 and April 24, 2009. The subsurface exploration consisted of drilling four mud rotary borings (B-1 through B-4) and conducting two cone penetration test (CPT) soundings (CPT-1 and CPT-2). The borings and CPTs were backfilled with cement grout. The following table shows the depth and approximate elevation of the borings.

TABLE A-1 – SUMMARY OF GEOTECHNICAL EXPLORATIONS

Exploration	Date Drilled	Approximate Surface Elevation (feet, NAVD 88)	Depth (feet)
B-1	4/21/09-4/22/09	+11	121.5
B-2	4/21/09	+11	8.0
B-3	4/23/09-4/24/09	+13	140.9
B-4	4/22/09-4/23/09	+13	100.9
CPT-1	4/20/09	+11	137.6
CPT-2	4/20/09	+11	139.9

Locations of the subsurface explorations are shown on Plate 2. Logs of the borings are presented as Plate A-1.1 through Plate A-1.6. CPT test data are provided in Appendix B.

The stratification lines shown on the boring logs represent the approximate boundaries between soil types; the actual transition may be gradual. The boring locations were estimated in the field by measuring from site features. Surface elevations of the borings were roughly estimated. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

SOIL SAMPLING METHODS

Soil sampling methods used during the exploration program were Standard Penetration Tests (SPTs), a 2.5-inch diameter split barrel sampler, and a Shelby tube sampler.

SPTs were performed using a 2-inch outside diameter, 1.5-inch inside diameter steel sampler without liners. The sampler was driven by repeatedly dropping a 140-pound safety hammer approximately 30 inches onto the sampling rod to which the sampler was attached. The number of blows required to drive the sampler the last 12 inches of a total 18-inch drive is referred to as the standard penetration test blow count or N-value, and is recorded on the drill hole logs. Blow counts were recorded for the purpose of estimating relative soil densities.

A split barrel sampler was driven a total of 18 inches or until refusal per ASTM D1586. The sampler is 3 inches outside diameter and 2.5 inches inside diameter lined with three six-inch long brass tubes with an inside diameter of 2.42 inches. The sampler was driven by repeatedly dropping a 140-pound safety hammer approximately 30 inches on the drill rod to which the sampler was attached. The number of blows required to drive the sampler the last 12 inches of a total of 18-inch interval is referred to as the blow count and is recorded on the boring logs.

Samples were collected within the young bay mud and old bay mud by using thin walled Shelby tubes measuring 3 inches in diameter and 3 feet in length. The piston pressure required to advance the Shelby tube is indicated on the boring logs.

LOG OF DRILL HOLE

JOB NO.: SF09005
PROJECT: Brannan Street Wharf
LOCATION: Brannan Street and The Embarcadero, San Francisco, California
DRILLING METHOD: Trash barrel (0-12 feet); Tri-cone roller bit with casing (12-41.5 feet)

LOGGED BY: D. van Hoff
CHECKED BY: J. Thurber

DRILL HOLE NO.: CPT-1 (Predrill)
DRILLING DATE: April 20, 2009
ELEVATION: 11 feet ±
DATUM: NAVD 88

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
						Portland Cement Concrete, approximately 6 3/8" thick, reinforced with #4 rebar approximately 2 7/8" from top.						
						Aggregate Base, approximately 6 inches thick. SANDY GRAVEL (GP), grayish brown, damp, gravel clasts up to approximately 3/4" dia.						
5						"ARTIFICIAL FILL (af)" SILTY GRAVEL (GM), grayish brown, damp, gravel clasts up to 3/4" dia.						
						SANDY SILT (ML), dark yellowish brown, moist, with gravel and brick debris, layers of silty gravel, gravel clasts up to approximately 3 inches dia.						
10						GRAVELLY CLAY (CL), dark brown, wet, gravel clasts up to approximately 3 inch dia.						
						"ROCK DIKE" Switched to casing, no sampling, no return of soil cuttings from 12 to 35 feet.						
15						Hard drilling from 15 to about 22 feet, driller indicated possible wood debris.						
20												
25												
30						Harder drilling from about 30 to about 33 feet.						
						POORLY GRADED SAND with SILT (SP-SM), grayish brown, wet, very dense, fine to medium grained sand.						
35	◣	53				Minor coarse grained sand.						
						COBBLES and/or BOULDERS in fill, hard sandstone.						
40	◣	13				"YOUNG BAY MUD (Qybm)" FAT CLAY (CH), black, wet, soft, grades to dark gray.						
						SILTY SAND (SM), dark greenish gray, wet, medium dense, fine to medium grained sand.						
45												
50												

NOTES:
1) Bottom of boring at 41.5 feet; set casing to 41.5 feet below ground surface for CPT Sounding.
2) Groundwater not measured.
3) Boring backfilled with cement grout after CPT sounding on 4/21/09.
4) Hammer efficiency of automatic hammer assumed to be 75 percent (C_E=1.25).

LOG_DRILL_HOLE_APPROX ELEVATION SF09005 BRANSON STREET WHARF.GPJ GTC.GDT 7/15/09

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF09005
PROJECT: Brannan Street Wharf
LOCATION: Brannan Street and The Embarcadero, San Francisco, California
DRILLING METHOD: Trash barrel (0-12 feet); Tri-cone roller bit with casing (12-38.5 feet)

LOGGED BY: N. Kumar
CHECKED BY: D. van Hoff

DRILL HOLE NO.: CPT-2 (Predrill)
DRILLING DATE: April 20, 2009
ELEVATION: 11 feet ±
DATUM: NAVD 88

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
						Portland Cement Concrete, approximately 8 1/4" thick, reinforced with #4 rebar approximately 5 1/4" from top.						
						"ARTIFICIAL FILL (af)" GRAVELLY SAND (SP), brown, damp.						
						CLAYEY SAND (SC), brown, damp.						
5						SANDY CLAY (CL), brown, moist, with brick debris. 6 inches of concrete.						
						CLAYEY GRAVEL/GRAVELLY CLAY (GC/CL), grayish brown, moist. Wet.						
10						Concrete debris, larger than 6 inches diameter, cut with trash barrel, hard, strong, grayish brown, wet, coated with clayey sand.						
15						"ROCK DIKE" Switched to casing installation.						
20						Rubble continues.						
25						Driller reports lots of wood.						
30												
35		18				SILTY SANDY GRAVEL (GM), brownish gray, wet, medium dense, angular to subangular pieces of rock.						
		2				"YOUNG BAY MUD (Qybm)" FAT CLAY (CH), black, wet, soft, with minor shell fragments, slight organic/hydrocarbon odor.						
40						NOTES: 1) Bottom of boring at 38.5 feet; set casing to 38.5 feet below ground surface for CPT Sounding. 2) Groundwater not measured. 3) Boring backfilled with cement grout after CPT sounding on 4/20/09. 4) Hammer efficiency of automatic hammer assumed to be 75 percent (C_E=1.25).						
45												
50												

LOG OF DRILL HOLE

JOB NO.: SF09005
PROJECT: Brannan Street Wharf
LOCATION: Brannan Street and The Embarcadero, San Francisco, California
DRILLING METHOD: Trash barrel (0-10 feet); Rotary Wash, 4 7/8" dia., Automatic hammer (10-121.5 feet)

LOGGED BY: D. Agnew
CHECKED BY: D. van Hoff

DRILL HOLE NO.: B-1
DRILLING DATE: April 21-22, 2009
ELEVATION: 11 feet ±
DATUM: NAVD 88

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
						Concrete sidewalk, 10 inches thick.						
						"ARTIFICIAL FILL (af)" CLAYEY SAND (SC) with GRAVEL, brown, moist, loose, angular gravel, gravel clasts up to 2 inch dia.						
5		7										
10		6				SANDY GRAVEL with CLAY (GP-GC), very dark gray and brown, wet, loose, brick debris. Slow drilling at 11 feet.						GS
						"ROCK DIKE" SANDY GRAVEL (GP), dark gray, wet, loose, sandstone gravel and cobbles, fine to coarse grained sand.						
15		8										
20		28										GS
25		9										
30		19				Medium dense.						
35						Encountered obstruction. Driller reports probable wood. Punched through obstruction with a drag bit, casing is difficult to advance.						
		29				POORLY GRADED SAND with SILT (SP-SM), gray, wet, dense, fine to medium grained sand.						GS
						GRAVELLY CLAY (CH), dark gray, wet, soft.						
40		50/4.5"				CLAYEY GRAVEL (GC), dark gray, wet, very dense.						
						"UPPER LAYERED SEDIMENTS (Qul)" POORLY GRADED SAND with SILT (SP-SM), dark yellowish brown, wet, very dense, fine to medium grained sand.						
45		41										
						SILTY SAND (SM), olive brown, wet, dense, fine grained sand.						
50		25										

LOG_DRILL_HOLE_APPROX ELEVATION SF09005 BRANSON STREET WHARF.GPJ GTC.GDT 7/15/09

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF09005
PROJECT: Brannan Street Wharf
LOCATION: Brannan Street and The Embarcadero, San Francisco, California
DRILLING METHOD: Trash barrel (0-10 feet); Rotary Wash, 4 7/8" dia., Automatic hammer (10-121.5 feet)

LOGGED BY: D. Agnew
CHECKED BY: D. van Hoff

DRILL HOLE NO.: B-1
DRILLING DATE: April 21-22, 2009
ELEVATION: 11 feet ±
DATUM: NAVD 88

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
						Dark yellowish brown.						
60		59					116.0	16.8				GS
						Interbedded POORLY GRADED SAND with SILT (SP-SM) and CLAY (CL), reddish brown sand with gray clay interlayers, wet, dense/hard, minor gravel in sand layers.						
65												
70		42		3.5			110.2	22.8				
75												
						SILTY SAND (SM), brown to reddish brown, wet, very dense, fine grained sand.						
80		90/11"					113.8	17.5				GS
						POORLY GRADED SAND with SILT (SP-SM), interbedded reddish brown and dark yellowish brown, wet, very dense, fine grained sand.						
85												
90		87/11.5"										GS
95												
						"OLDER BAY MUD (Qobm)" FAT CLAY (CH), very dark greenish gray, wet, stiff, minor fine grained sand.						
100		15		1.25 1.5 1.75			92.3	31.8	51	21		GS, TUU
105												

LOG_DRILL_HOLE_APPROX ELEVATION SF09005 BRANSON STREET WHARF.GPJ GTC.GDT 7/15/09

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF09005
PROJECT: Brannan Street Wharf
LOCATION: Brannan Street and The Embarcadero, San Francisco, California
DRILLING METHOD: Trash barrel (0-10 feet); Rotary Wash, 4 7/8" dia., Automatic hammer (10-121.5 feet)

LOGGED BY: D. Agnew
CHECKED BY: D. van Hoff

DRILL HOLE NO.: B-1
DRILLING DATE: April 21-22, 2009
ELEVATION: 11 feet ±
DATUM: NAVD 88

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
	■	31		2.25		Interbedded SILTY SAND to CLAYEY SAND (SM/SC) and FAT CLAY (CH), very dark greenish gray, wet, medium dense/very stiff, minor gravel.			99	34		GS
115						SANDY CLAY (CL), very dark greenish gray, wet, hard, fine grained sand, minor coarse sand and gravel.						
120	■	64		4.3			121.2	15.4			4,140	
125						NOTES: 1) Bottom of boring at 121.5 feet. 2) Groundwater estimated at 8 feet during drilling. 3) Boring backfilled with cement grout on 4/22/2009. 4) Hammer efficiency of automatic hammer assumed to be 75 percent (C_E=1.25).						
130												
135												
140												
145												
150												
155												
160												

LOG_DRILL_HOLE_APPROX ELEVATION SF09005 BRANSON STREET WHARF.GPJ GTC.GDT 7/15/09

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF09005
PROJECT: Brannan Street Wharf
LOCATION: Brannan Street and The Embarcadero, San Francisco, California
DRILLING METHOD: Trash barrel (0-6 feet); Tri-cone roller bit with casing (6-8 feet)

LOGGED BY: D. Agnew
CHECKED BY: D. van Hoff

DRILL HOLE NO.: B-2
DRILLING DATE: April 21, 2009
ELEVATION: 11 feet ±
DATUM: NAVD 88

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
						Concrete sidewalk, 10 inches thick						
						Approximately 2 feet of crushed, loose asphalt rubble.						
5		6				"ARTIFICIAL FILL (af)" SILTY SAND (SM), mottled brown, gray, light brown, moist, loose, fine to medium grained sand, some gravel, trace clay, red brick in sampler shoe and middle of sample. At 6 feet very slow drilling, ~1 ft/30 minutes, with tri-cone bit and casing. Drilling refusal at 8 feet.						
10						NOTES: 1) Bottom of boring at 8 feet. 2) Groundwater not encountered. 3) Boring backfilled with cement grout on 4/21/2009. 4) Hammer efficiency of automatic hammer assumed to be 75 percent (C_E=1.25).						
15												
20												
25												
30												
35												
40												
45												
50												

LOG_DRILL_HOLE_APPROX ELEVATION SF09005 BRANSON STREET WHARF.GPJ GTC.GDT 7/15/09

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF09005
PROJECT: Brannan Street Wharf
LOCATION: Brannan Street and The Embarcadero, San Francisco, California
DRILLING METHOD: Vane shear test (19-44.5 feet); Rotary Wash, 4 7/8" dia., Automatic hammer (44.5-140.9 feet)

LOGGED BY: D. Agnew
CHECKED BY: D. van Hoff

DRILL HOLE NO.: B-3
DRILLING DATE: April 23-24, 2009
ELEVATION: 13 feet ±
DATUM: NAVD 88

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
0						PIER 36 CONCRETE DECK						
10						Water at 10 feet, subject to tides.						
19						"YOUNG BAY MUD (Qybm)" Vane shear testing in young bay mud from 19 feet to 43.5 feet.						
43.5						"UPPER LAYERED SEDIMENTS (Qul)"						
45		46				SILTY SAND (SM), olive gray, wet, dense, fine grained sand, minor clay.	116.4	16.4				GS
						Brown to dark yellowish brown.						
50		29				Fine to medium grained sand, trace clay.			NV	NP		GS

LOG_DRILL_HOLE_APPROX ELEVATION SF09005 BRANSON STREET WHARF.GPJ GTC.GDT 7/15/09

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF09005
PROJECT: Brannan Street Wharf
LOCATION: Brannan Street and The Embarcadero, San Francisco, California
DRILLING METHOD: Vane shear test (19-44.5 feet); Rotary Wash, 4 7/8" dia., Automatic hammer (44.5-140.9 feet)

LOGGED BY: D. Agnew
CHECKED BY: D. van Hoff

DRILL HOLE NO.: B-3
DRILLING DATE: April 23-24, 2009
ELEVATION: 13 feet ±
DATUM: NAVD 88

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
		47				Very dense, slightly more clay.						
60		47				Olive brown, less clay.						
65		45										
						FAT CLAY (CH), olive gray, some brown, wet, very stiff, minor sand, minor peat rootlets.						
70		17	1,680	2.5			78.9	45.5	77	31	1,970	
						CLAY (CL), olive gray, wet, stiff, silty sand interbeds.						
75		14	1,300	1.0 1.9			93.3	29.3	45	20		GS
						SANDY CLAY to CLAYEY SAND (CL/SC), brown with reddish brown and yellowish brown mottling, wet, hard/dense, fine grained sand, minor gravel, minor silt.						
80		54	1,660	1.5								
						CLAYEY SAND (SC), yellowish brown with dark reddish brown, mottling, wet, dense, fine grained sand.						
85		61	2,700				118.0	15.3	31	18		GS
90		31				Light yellowish brown, less clay.						
						POORLY GRADED SAND with SILT (SP-SM), reddish brown, wet, very dense, fine grained sand.						
95		79										GS
						SILTY SAND (SM), olive gray, wet, very dense, fine grained sand.						
100		90/11.5"										
						"OLDER BAY MUD (Qobm)" FAT CLAY (CH), dark gray, wet, stiff to very stiff, with silty clay layers.						
105		18	1,160	2.25			89.5	35.0	59	22	2,170	

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF09005
PROJECT: Brannan Street Wharf
LOCATION: Brannan Street and The Embarcadero, San Francisco, California
DRILLING METHOD: Vane shear test (19-44.5 feet); Rotary Wash, 4 7/8" dia., Automatic hammer (44.5-140.9 feet)

LOGGED BY: D. Agnew
CHECKED BY: D. van Hoff

DRILL HOLE NO.: B-3
DRILLING DATE: April 23-24, 2009
ELEVATION: 13 feet ±
DATUM: NAVD 88

DEPTH (FEET)	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
	17	1,580	2.2		78.6	44.8	84	29	2,180	
115	100 psi			Shelby tube, 32"/32" recovery.	72.6	51.2	90	33		TUU
120	17	1,380	2.5		74.1	49.3	92	33	2,240	
125	11	2,300	3.0 3.75	Minor peat.						
130	31	1,800	4.0	Very dark gray, very stiff, minor gravel, minor coarse sand.						
				SANDY CLAY (CL), olive gray, wet, very stiff, fine grained sand.						
				"LOWER LAYERED SEDIMENTS (Qll)" GRAVELLY CLAY (CL) with SAND, dark greenish gray, wet, hard.						
135	50/6"									
				CLAYEY SAND to SILTY SAND (SC/SM), reddish brown and yellowish brown, wet, very dense, fine grained sand.						
140	50/5"									
145										
150										
155										
160										

NOTES:
1) Bottom of boring at 140.9 feet.
2) Depths measured below existing Pier 36 deck.
3) Boring backfilled with cement grout on 4/24/2009.
4) Hammer efficiency of automatic hammer assumed to be 75 percent (C_E=1.25).

LOG_DRILL_HOLE_APPROX ELEVATION SF09005 BRANSON STREET WHARF.GPJ GTC.GDT 7/15/09

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF09005
PROJECT: Brannan Street Wharf
LOCATION: Brannan Street and The Embarcadero, San Francisco, California
DRILLING METHOD: Rotary Wash, 4 7/8" dia., Automatic hammer

LOGGED BY: D. Agnew
CHECKED BY: D. van Hoff

DRILL HOLE NO.: B-4
DRILLING DATE: April 22-23, 2009
ELEVATION: 13 feet ±
DATUM: NAVD 88

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
0						PIER 36 CONCRETE DECK						
5												
10						Water at 10 feet, subject to tides.						
15												
20						Mud line at approximately 21 feet.						
						"YOUNG BAY MUD (Qybm)" FAT CLAY (CH), dark gray to black, wet, very soft.						
25												
	Shelby tube	50 psi				Shelby tube recovery 23"/30".	42.4	107.0	93	35		TUU
30	Shelby tube	50 psi	100			Shelby tube recovery 30"/30".	47.4	88.4	96	36		TUU
35	Shelby tube	50 psi	120			Shelby tube recovery 24.5"/30".	47.9	87.4	99	38		C, TUU
40	Shelby tube	50 psi	140			Shelby tube recovery 27.5"/30".						
45	Shelby tube	50 psi	280			Soft, shelby tube recovery 34"/34".	52.4	77.6	101	36		TUU
						"UPPER LAYERED SEDIMENTS (Qul)" SILTY SAND (SM), brown to dark yellowish brown, wet, dense, fine grained sand.						
50	Sample	49					115.1	17.1				GS

LOG_DRILL_HOLE_APPROX ELEVATION SF09005 BRANSON STREET WHARF.GPJ GTC.GDT 7/15/09

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF09005
PROJECT: Brannan Street Wharf
LOCATION: Brannan Street and The Embarcadero, San Francisco, California
DRILLING METHOD: Rotary Wash, 4 7/8" dia., Automatic hammer

LOGGED BY: D. Agnew
CHECKED BY: D. van Hoff

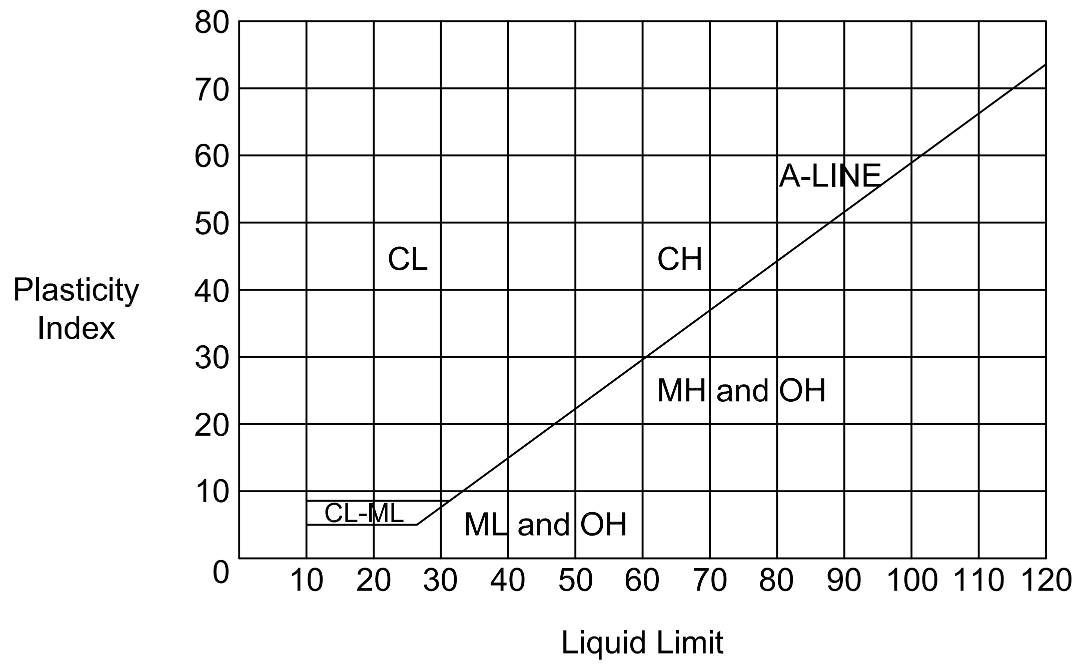
DRILL HOLE NO.: B-4
DRILLING DATE: April 22-23, 2009
ELEVATION: 13 feet ±
DATUM: NAVD 88

DEPTH (FEET)	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
60	31			Trace clay.						GS
65				SANDY CLAY (CL), olive brown, wet, hard, minor peat rootlets.						
70	42			SILTY CLAY (CL), olive gray, wet, very stiff.	106.0	23.6			4,010	
75				Interbedded SILTY SAND (SM) and SILTY CLAY (CL), dark greenish gray, wet, very stiff.						
80	20	1,200			89.2	31.6				GS
85										
90	73			SILTY SAND (SM), brown, wet, very dense, dark reddish brown mottling.	119.4	15.4				GS
95										
100	50/5"			Olive brown.						
105										

NOTES:
1) Bottom of boring at 100.9 feet.
2) Depths measured below existing Pier 36 deck.
3) Boring backfilled with cement grout on 4/23/2009.
4) Hammer efficiency of automatic hammer assumed to be 75 percent (C_E=1.25).

LEGEND TO LOGS ON PLATE A-2

PLASTICITY CHART - Used for Classification of Fine Grained Soils



BLOW COUNT - The number of blows required to drive the sampler the last 12 inches of an 18-inch drive. When the sampler is not advanced the last 12 inches, i.e. 100 blows in 9 inches, the notation is 100/9.

ADDITIONAL TESTS -

C: Consolidation
CL: Chloride
CORR: Corrosion
CP: Compaction
DS: Direct Shear
EL: Elasticity Index
EX: Expansion
FC: Fines Content (#200 Sieve Wash)
GS: Grain Size Distribution
pH: Hydrogen Ion Concentration
PM: Permeability
R: R-Value
RS: Resistivity
S: Swell
SE: Sand Equivalent
SP: Specific Gravity
SU: Sulfate
TD: Triaxial Compression, Drained
TDy: Triaxial Compression, Dynamic
TCU: Triaxial Compression, Consolidated Undrained
TUU: Triaxial Compression, Unconsolidated Undrained

SAMPLE TYPES:

- MODIFIED CALIFORNIA SAMPLE
- DISTURBED SLEEVE
- UNSUCCESSFUL SLEEVE
- SHELBY TUBE
- STANDARD PENETRATION
- STANDARD PENETRATION NO RECOVERY
- ROCK or SOIL CORE
- BULK SAMPLE

CAVING:

- LIGHT CAVING
- HEAVY CAVING

WATER LEVEL:

- STABILIZED or PARTIALLY STABILIZED GROUNDWATER LEVEL
- UNSTABILIZED GROUNDWATER LEVEL
- SEEPAGE LEVEL

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISION			GROUP SYMBOL	DESCRIPTION	GRAPHIC LOG
COARSE GRAINED SOILS Over 50% By Weight Coarser Than No.200 Sieve Size	GRAVELLY SOILS OVER 50% OF COARSE FRACTION LARGER THAN NO.4 SIEVE SIZE	CLEAN GRAVELLY SOILS LITTLE OR NO FINES	GW	well graded gravels or gravel-sand mixtures	
			GP	poorly graded gravels or gravel-sand mixtures	
		GRAVELLY SOILS WITH FINES OVER 12% FINES	GM	silty gravels or gravel-sand-silt mixtures	
			GC	clayey gravels or gravel-sand-clay mixtures	
	SANDY SOILS OVER 50% OF COARSE FRACTION SMALLER THAN NO.4 SIEVE SIZE	CLEAN SANDY SOILS LITTLE OR NO FINES	SW	well graded sands or gravelly sands	
			SP	poorly graded sands or gravelly sands	
		SANDY SOILS WITH FINES OVER 12% FINES	SM	silty sands or sand-silt mixtures	
			SC	clayey sands or sand-clay mixtures	
FINE GRAINED SOILS Over 50% By Weight Finer Than No.200 Sieve Size	SILTY AND CLAYEY SOILS LIQUID LIMIT LESS THAN 50		ML	inorganic silts, very fine sands, silty fine sands, clayey silts with slight plasticity	
			CL	inorganic clays, gravelly, sandy, silty, or lean clays, of low to medium plasticity	
			OL	organic clays or organic silts of low plasticity	
	SILTY AND CLAYEY SOILS LIQUID LIMIT GREATER THAN 50		MH	inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
			CH	inorganic clays of high plasticity, fat clays	
			OH	organic clays or organic silts of medium to high plasticity	
HIGHLY ORGANIC SOILS			Pt	peat or other highly organic soil, organic content greater than 60%	
				trash fill-landfill refuse (not a part of unified soil classification system)	

GTC
GEOTECHNICAL CONSULTANTS, INC.
500 Sansome Street, Suite 402
San Francisco, CA 94111

LEGEND TO LOGS

PLATE A - 2

APPENDIX B
CONE PENETRATION TESTS

April 21, 2009

Geotechnical Consultants
Attn: Deron Van Heff
500 Sansome St., Suite 402
San Francisco, California 94111

Subject: CPT Site Investigation
Brannan St. Wharf Project
San Francisco, California
GREGG Project Number: 09-059MA

Dear Mr. Van Heff:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☐
3	Seismic Cone Penetration Tests	(SCPTU)	☐
4	Resistivity Cone Penetration Tests	(RCPTU)	☐
5	UVOST Laser Induced Fluorescence	(UVOST)	☐
6	Groundwater Sampling	(GWS)	☐
7	Soil Sampling	(SS)	☐
8	Vapor Sampling	(VS)	☐
9	Vane Shear Testing	(VST)	☐
10	SPT Energy Calibration	(SPTE)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (925) 313-5800.

Sincerely,
GREGG Drilling & Testing, Inc.

Mary Walden
Operations Manager

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (Feet)	Depth of Groundwater Samples (Feet)	Depth of Soil Samples (Feet)	Depth of Pore Pressure Dissipation Tests (Feet)
CPT-01	4/20/09	137	-	-	-
CPT-02	4/20/09	140	-	-	-

Bibliography

Lunne, T., Robertson, P.K. and Powell, J.J.M., "Cone Penetration Testing in Geotechnical Practice" E & FN Spon. ISBN 0 419 23750, 1997

Roberston, P.K., "Soil Classification using the Cone Penetration Test", Canadian Geotechnical Journal, Vol. 27, 1990 pp. 151-158.

Mayne, P.W., "NHI (2002) Manual on Subsurface Investigations: Geotechnical Site Characterization", available through www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html, Section 5.3, pp. 107-112.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8, 1986 pp. 791-803.

Robertson, P.K., Sully, J., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.J., "Guidelines for Estimating Consolidation Parameters in Soils from Piezocone Tests", Canadian Geotechnical Journal, Vol. 29, No. 4, August 1992, pp. 539-550.

Robertson, P.K., T. Lunne and J.J.M. Powell, "Geo-Environmental Application of Penetration Testing", Geotechnical Site Characterization, Robertson & Mayne (editors), 1998 Balkema, Rotterdam, ISBN 90 5410 939 4 pp 35-47.

Campanella, R.G. and I. Weemees, "Development and Use of An Electrical Resistivity Cone for Groundwater Contamination Studies", Canadian Geotechnical Journal, Vol. 27 No. 5, 1990 pp. 557-567.

DeGroot, D.J. and A.J. Lutenegger, "Reliability of Soil Gas Sampling and Characterization Techniques", International Site Characterization Conference - Atlanta, 1998.

Woeller, D.J., P.K. Robertson, T.J. Boyd and Dave Thomas, "Detection of Polyaromatic Hydrocarbon Contaminants Using the UVIF-CPT", 53rd Canadian Geotechnical Conference Montreal, QC October pp. 733-739, 2000.

Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, "Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers" BAT EnviroProbe and QED HydroPunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

Copies of ASTM Standards are available through www.astm.org

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*. The soundings were conducted using a 20 ton capacity cone with a tip area of 15 cm^2 and a friction sleeve area of 225 cm^2. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80.

The cone takes measurements of cone bearing (q_c), sleeve friction (f_s) and penetration pore water pressure (u_2) at 5-cm intervals during penetration to provide a nearly continuous hydrogeologic log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2002) ASTM standards (D 5778-95).

The cone also contains a porous filter element located directly behind the cone tip (u_2), *Figure CPT.* It consists of porous plastic and is 5.0mm thick. The filter element is used to obtain penetration pore pressure as the cone is advanced as well as Pore Pressure Dissipation Tests (PPDT's) during appropriate pauses in penetration. It should be noted that prior to penetration, the element is fully saturated with silicon oil under vacuum pressure to ensure accurate and fast dissipation.

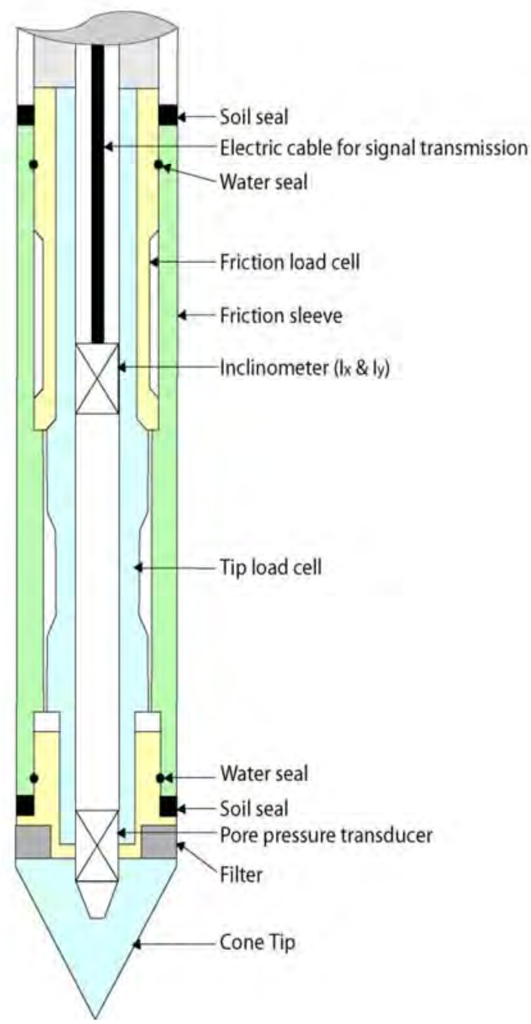


Figure CPT

When the soundings are complete, the test holes are grouted using a Gregg support rig. The grouting procedures generally consist of pushing a hollow CPT rod with a "knock out" plug to the termination depth of the test hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

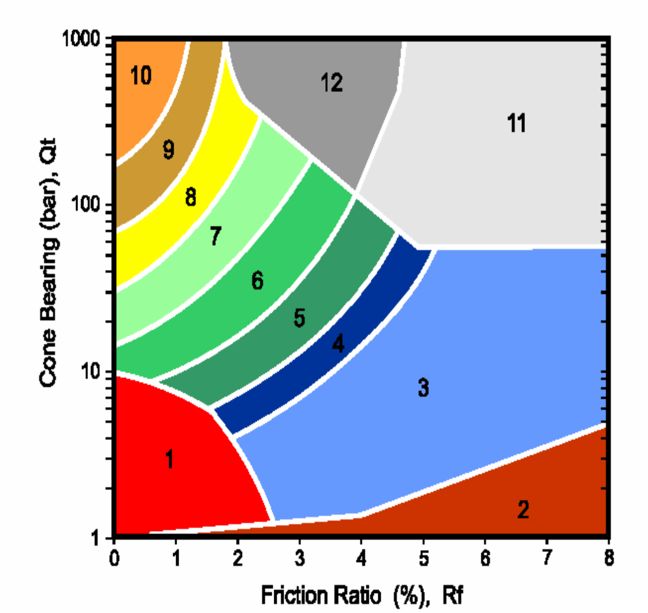
Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected from your site are presented in graphical form in the attached report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings extending greater than 50 feet, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. do not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and do not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



(After Robertson, et al., 1986)

ZONE		SBT
1		Sensitive, fine grained
2		Organic materials
3		Clay
4		Silty clay to clay
5		Clayey silt to silty clay
6		Sandy silt to clayey silt
7		Silty sand to sandy silt
8		Sand to silty sand
9		Sand
10		Gravely sand to sand
11		Very stiff fine grained*
12		Sand to clayey sand*

*over consolidated or cemented

Figure SBT

Cone Penetration Test (CPT) Interpretation

Gregg has recently updated their CPT interpretation and plotting software (2007). The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:

1 Units for display (Imperial or metric) (atm. pressure, pa = 0.96 tsf or 0.1 MPa)
2 Depth interval to average results,(ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
3 Elevation of ground surface (ft or m)
4 Depth to water table, z_w (ft or m) – input required
5 Net area ratio for cone, a (default to 0.80)
6 Relative Density constant, C_{Dr} (default to 350)
7 Young's modulus number for sands, α (default to 5)
8 Small strain shear modulus number
   a. for sands, S_G (default to 180 for SBT_n 5, 6, 7)
   b. for clays, C_G (default to 50 for SBT_n 1, 2, 3 & 4)
9 Undrained shear strength cone factor for clays, N_{kt} (default to 15)
10 Over Consolidation ratio number, k_{ocr} (default to 0.3)
11 Unit weight of water, (default to γ_w = 62.4 lb/ft^3 or 9.81 kN/m^3)

Column

1 Depth, z, (m) – CPT data is collected in meters
2 Depth (ft)
3 Cone resistance, q_c (tsf or MPa)
4 Sleeve friction, f_s (tsf or MPa)
5 Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u_2)
6 Other – any additional data, if collected, e.g. electrical resistivity or UVIF
7 Total cone resistance, q_t (tsf or MPa) $q_t = q_c + u\,(1\text{-}a)$

8	Friction Ratio, R_f (%)	$R_f = (f_s/q_t)$ x 100%
9	Soil Behavior Type (non-normalized), SBT	see note
10	Unit weight, γ (pcf or kN/m^3)	based on SBT, see note
11	Total overburden stress, σ_v (tsf)	$\sigma_{vo} = \gamma$ z
12	Insitu pore pressure, u_o (tsf)	$u_o = \gamma_w$ (z - z_w)
13	Effective overburden stress, σ'_{vo} (tsf)	$\sigma'_{vo} = \sigma_{vo} - u_o$
14	Normalized cone resistance, Q_{t1}	$Q_{t1} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo})$ x 100%
16	Normalized Pore Pressure ratio, B_q	$B_q = u – u_o / (q_t - \sigma_{vo})$
17	Soil Behavior Type (normalized), SBT_n	see note
18	SBT_n Index, I_c	see note
19	Normalized Cone resistance, Q_{tn} (n varies with Ic)	see note
20	Estimated permeability, k_{SBT} (cm/sec or ft/sec)	see note
21	Equivalent SPT N_{60}, blows/ft	see note
22	Equivalent SPT $(N_1)_{60}$ blows/ft	see note
23	Estimated Relative Density, D_r, (%)	see note
24	Estimated Friction Angle, ϕ', (degrees)	see note
25	Estimated Young’s modulus, E_s (tsf)	see note
26	Estimated small strain Shear modulus, Go (tsf)	see note
27	Estimated Undrained shear strength, s_u (tsf)	see note
28	Estimated Undrained strength ratio	s_u/σ_v’
29	Estimated Over Consolidation ratio, OCR	see note

Notes:

1 Soil Behavior Type (non-normalized), SBT Lunne et al. (1997) listed below

2 Unit weight, γ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)

3 Soil Behavior Type (Normalized), SBT_n Lunne et al. (1997)

4 SBT_n Index, I_c $I_c = ((3.47 – \log Q_{t1})^2 + (\log F_r + 1.22)^2)^{0.5}$

5 Normalized Cone resistance, Q_{tn} (n varies with Ic)

$Q_{tn} = ((q_t - \sigma_{vo})/pa) (pa/(\sigma'_{vo})^n$ and recalculate I_c, then iterate:

When $I_c < 1.64$, n = 0.5 (clean sand)
When $I_c > 3.30$, n = 1.0 (clays)
When $1.64 < I_c < 3.30$, $n = (I_c – 1.64)0.3 + 0.5$
Iterate until the change in n, $\Delta n < 0.01$

6 Estimated permeability, k_{SBT} (based on Normalized SBT_n)
(Lunne et al., 1997 and table below)

7 Equivalent SPT N_{60}, blows/ft Lunne et al. (1997)

$$\frac{(q_t/p_a)}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right)$$

8 Equivalent SPT $(N_1)_{60}$ blows/ft $(N_1)_{60} = N_{60} C_N$,
where $C_N = (pa/\sigma'_{vo})^{0.5}$

9 Relative Density, D_r, (%) $D_r^2 = Q_{tn} / C_{Dr}$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

10 Friction Angle, ϕ', (degrees) $\tan\phi' = \frac{1}{2.68}\left[\log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right]$

Only SBT_n 5, 6, 7 & 8 *Show'N/A' in zones 1, 2, 3, 4 & 9*

11 Young's modulus, E_s $E_s = \alpha\, q_t$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

12 Small strain shear modulus, Go
a. $G_o = S_G (q_t\ \sigma'_{vo}\ pa)^{1/3}$ *For SBT_n 5, 6, 7*
b. $G_o = C_G\, q_t$ *For SBT_n 1, 2, 3& 4*
Show 'N/A' in zones 8 & 9

13 Undrained shear strength, s_u $s_u = (q_t - \sigma_{vo}) / N_{kt}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

14 Over Consolidation ratio, OCR $OCR = k_{ocr} Q_{t1}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

SBT Zones **SBT_n Zones**

The following updated and simplified SBT descriptions have been used in the software:

1	sensitive fine grained	1	sensitive fine grained
2	organic soil	2	organic soil
3	clay	3	clay
4	clay & silty clay	4	clay & silty clay
5	clay & silty clay		
6	sandy silt & clayey silt		
7	silty sand & sandy silt	5	silty sand & sandy silt
8	sand & silty sand	6	sand & silty sand
9	sand		
10	sand	7	sand

11	very dense/stiff soil*	8	very dense/stiff soil*
12	very dense/stiff soil*	9	very dense/stiff soil*

*heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print ‘clays & silty clays’)

Estimated Permeability (see Lunne et al., 1997)

SBT_n	Permeability (ft/sec)	(m/sec)
1	$3x\ 10^{-8}$	$1x\ 10^{-8}$
2	$3x\ 10^{-7}$	$1x\ 10^{-7}$
3	$1x\ 10^{-9}$	$3x\ 10^{-10}$
4	$3x\ 10^{-8}$	$1x\ 10^{-8}$
5	$3x\ 10^{-6}$	$1x\ 10^{-6}$
6	$3x\ 10^{-4}$	$1x\ 10^{-4}$
7	$3x\ 10^{-2}$	$1x\ 10^{-2}$
8	$3x\ 10^{-6}$	$1x\ 10^{-6}$
9	$1x\ 10^{-8}$	$3x\ 10^{-9}$

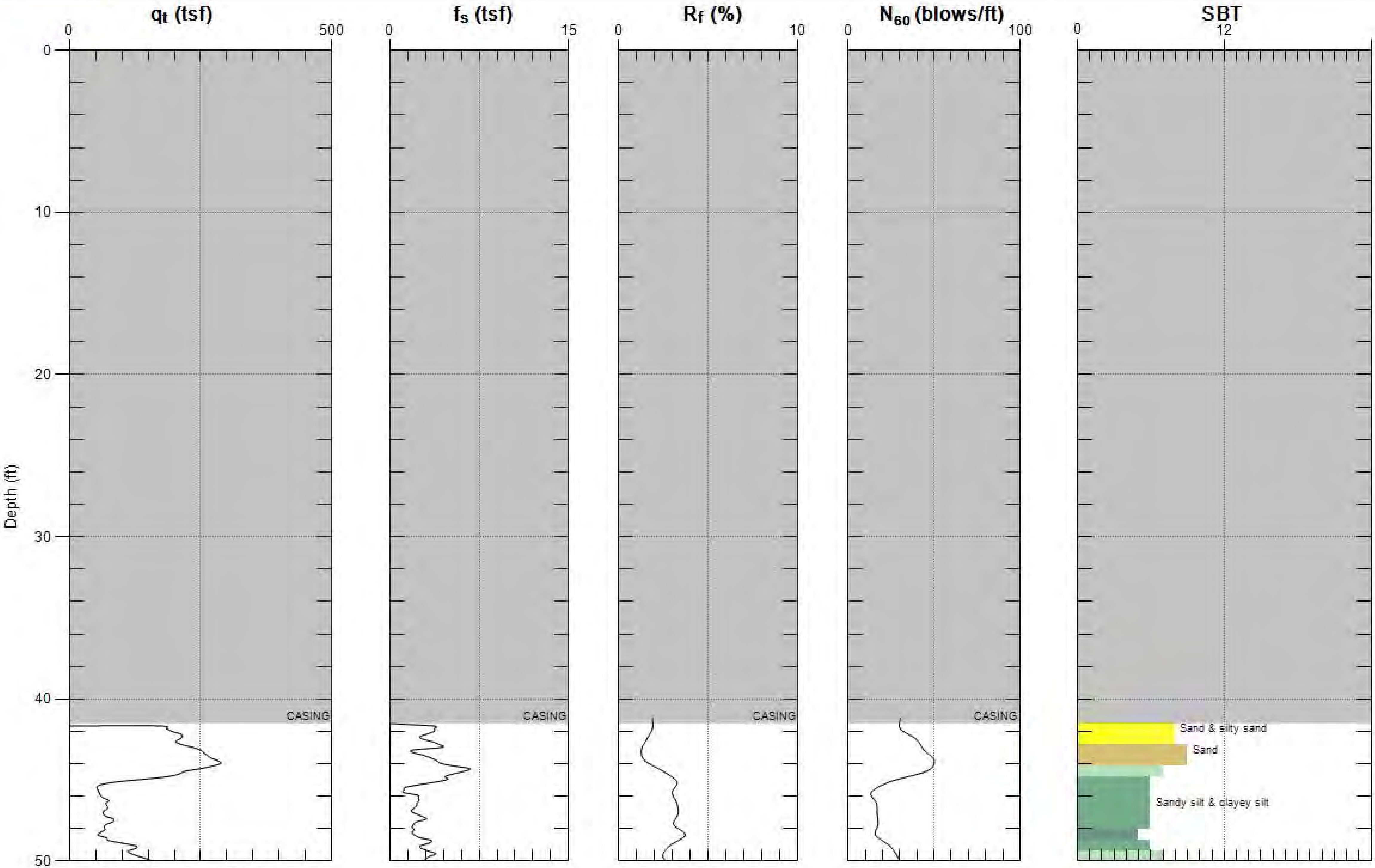
Estimated Unit Weight (see Lunne et al., 1997)

SBT	Approximate Unit Weight (lb/ft^3)	(kN/m^3)
1	111.4	17.5
2	79.6	12.5
3	111.4	17.5
4	114.6	18.0
5	114.6	18.0
6	114.6	18.0
7	117.8	18.5
8	120.9	19.0
9	124.1	19.5
10	127.3	20.0
11	130.5	20.5
12	120.9	19.0

GREGG GEOTECHNICAL CONSULTANTS

Site: BRANNAN ST.
Sounding: CPT-01

Engineer: D.VAN HOFF
Date: 4/20/2009 05:39

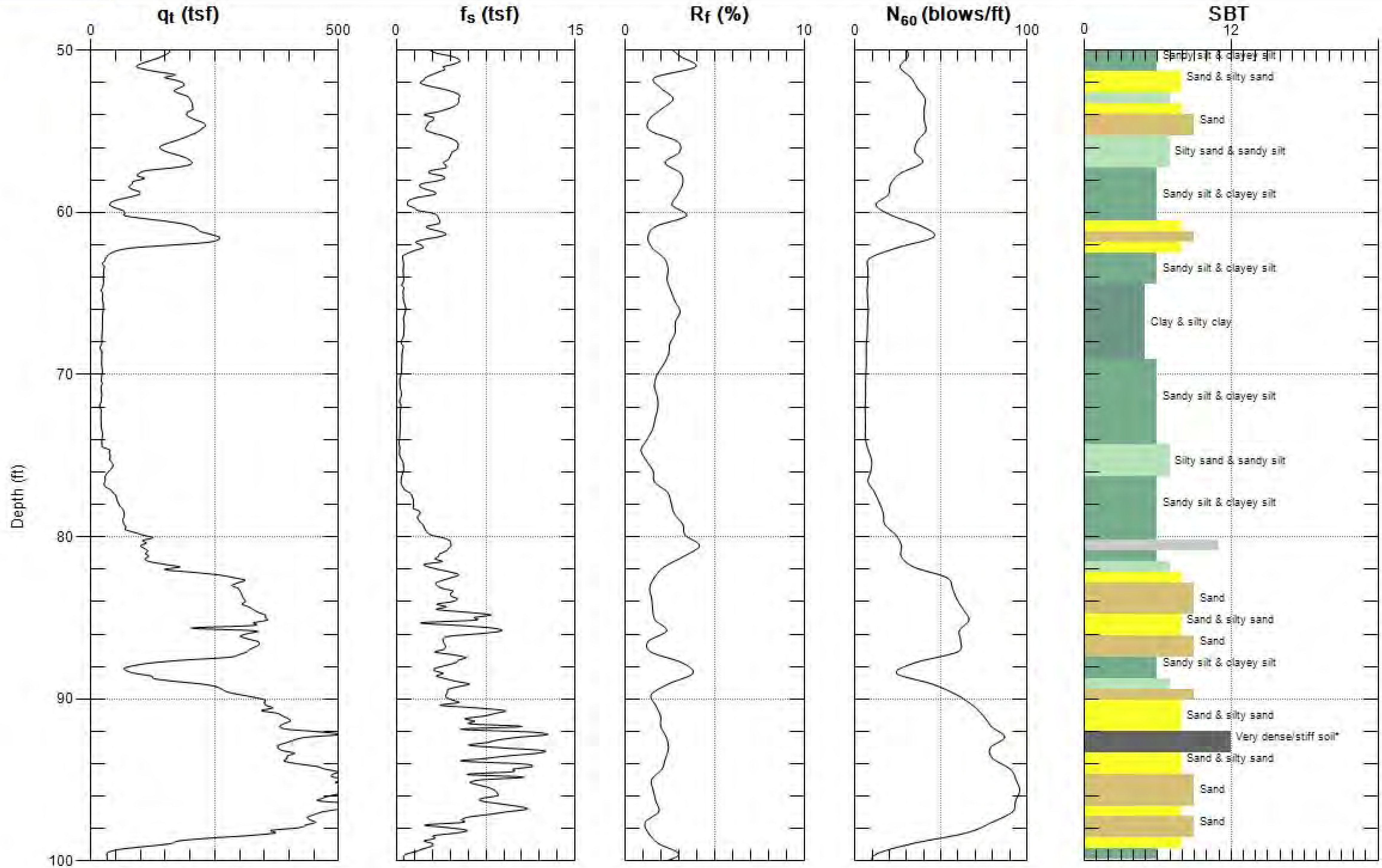


Max. Depth: 137.631 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG GEOTECHNICAL CONSULTANTS

Site: BRANNAN ST. | Engineer: D.VAN HOFF
Sounding: CPT-01 | Date: 4/20/2009 05:39



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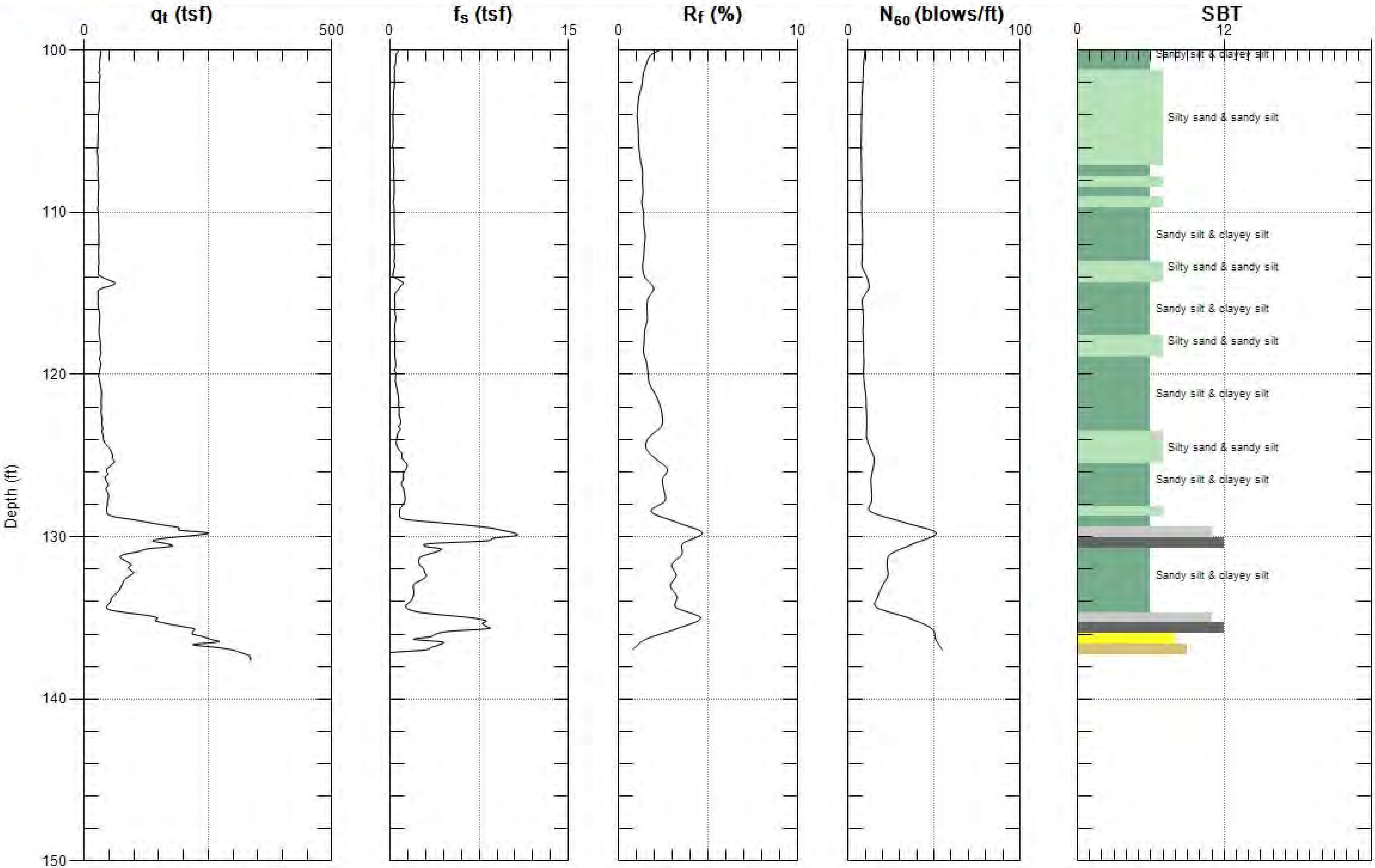
Site: BRANNAN ST.

Sounding: CPT-01

Engineer: D.VAN HOFF

Date: 4/20/2009 05:39



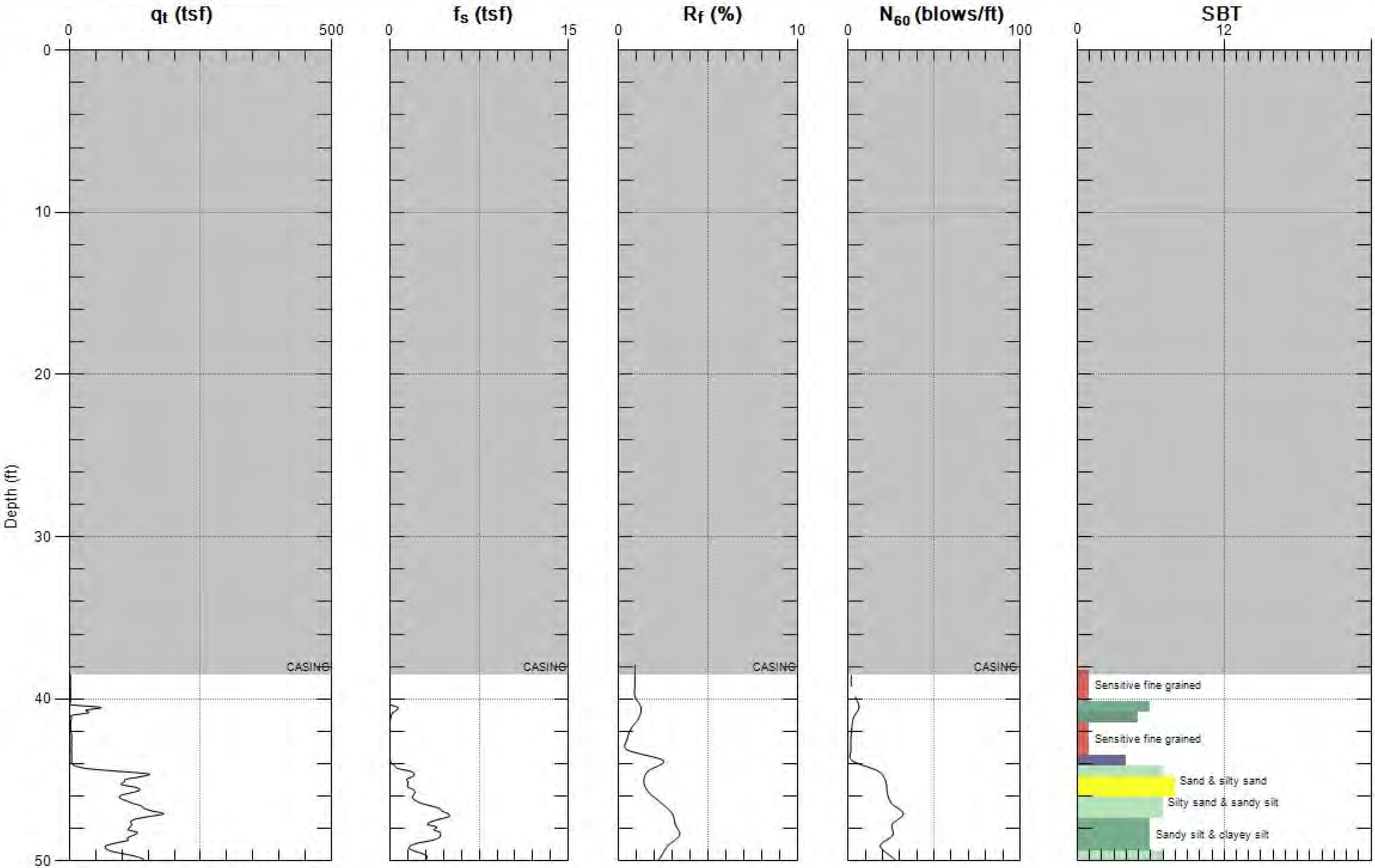
Max. Depth: 137.631 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG GEOTECHNICAL CONSULTANTS

Site: BRANNAN ST.
Sounding: CPT-02

Engineer: D.VAN HOFF
Date: 4/20/2009 03:23



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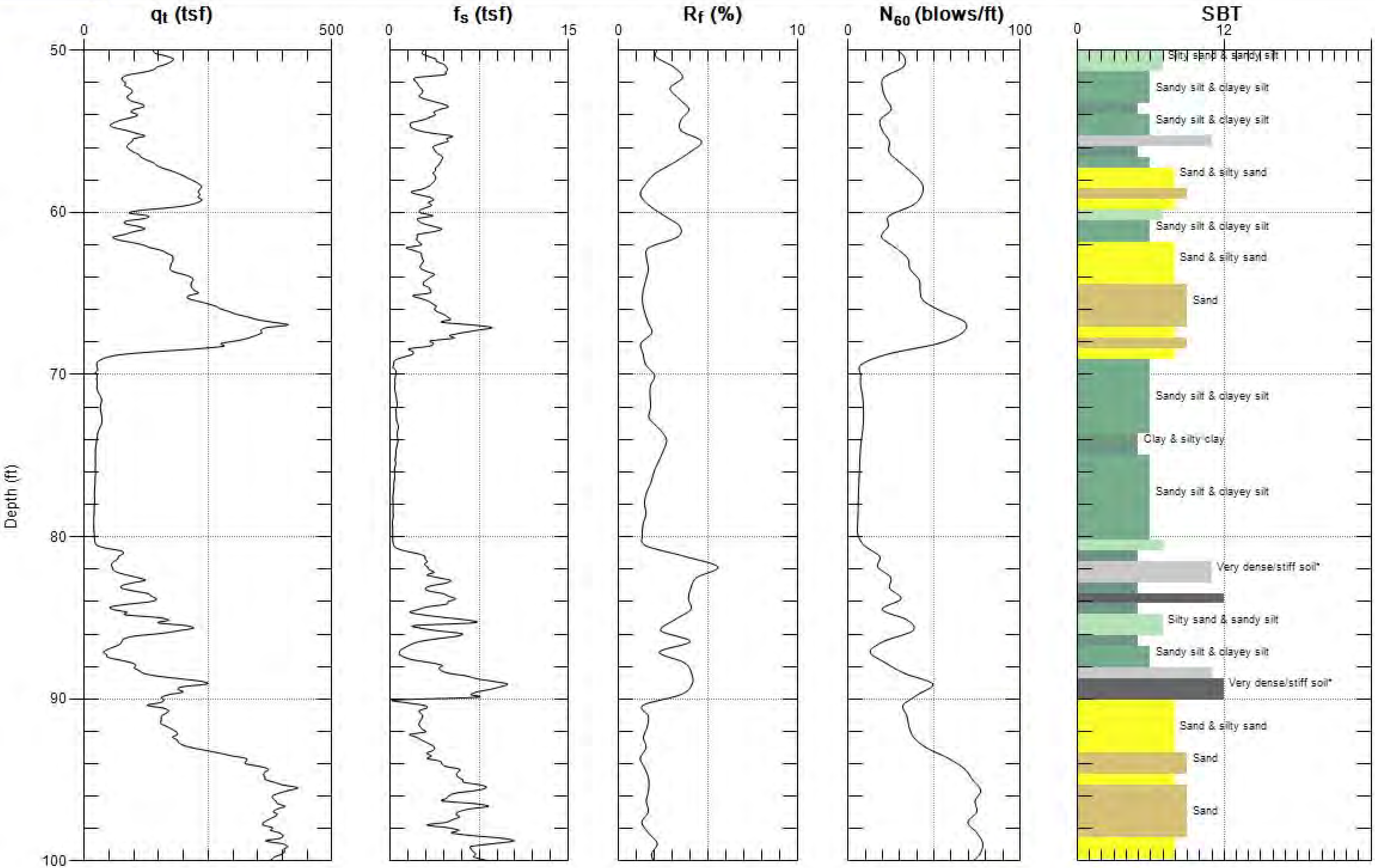
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG GEOTECHNICAL CONSULTANTS

Site: BRANNAN ST.
Sounding: CPT-02

Engineer: D.VAN HOFF
Date: 4/20/2009 03:23



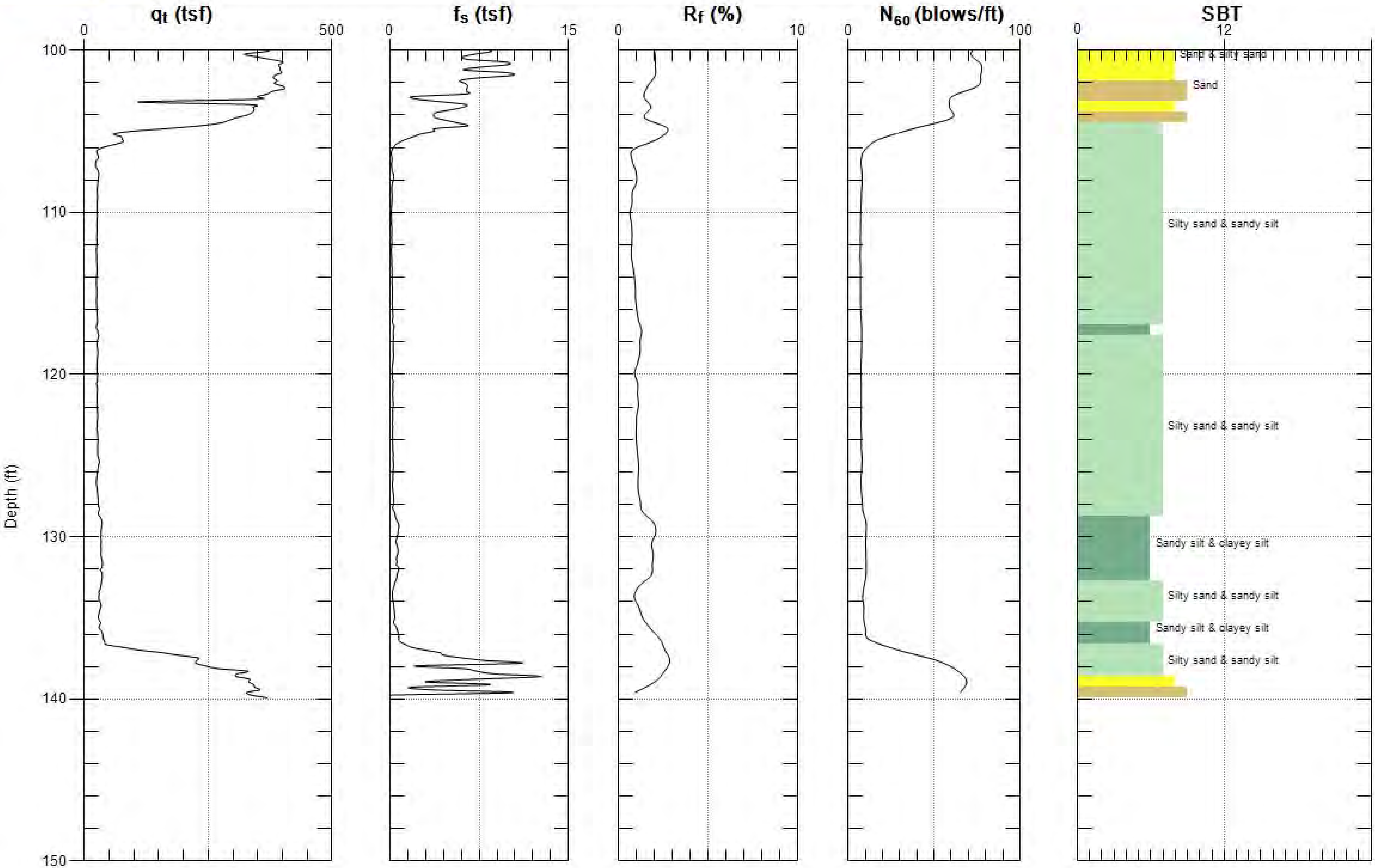
Max. Depth: 139.928 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **GEOTECHNICAL CONSULTANTS**

Site: BRANNAN ST. | Engineer: D.VAN HOFF

Sounding: CPT-02 | Date: 4/20/2009 03:23



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Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **GEOTECHNICAL CONSULTANTS**

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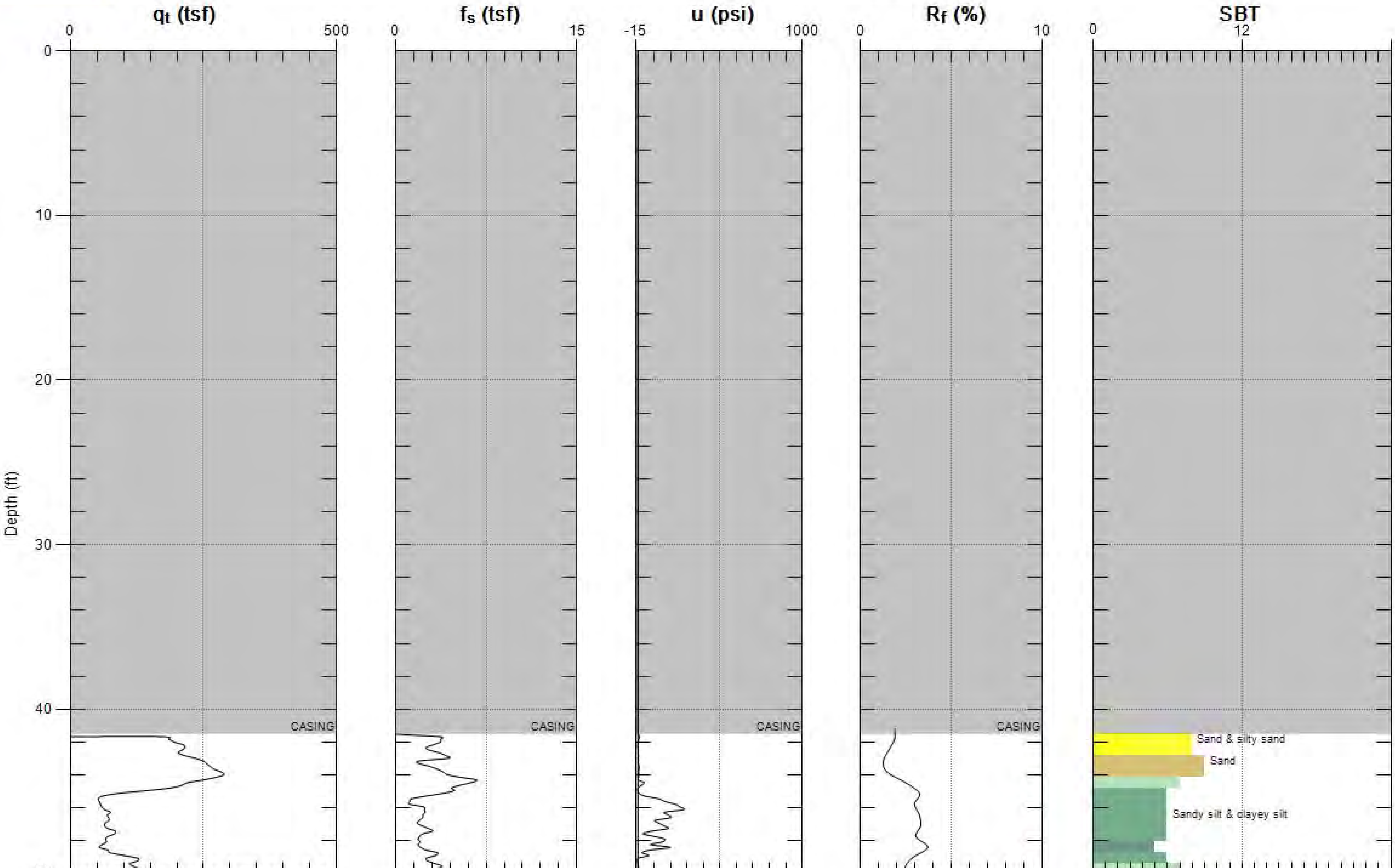
Sounding: CPT-01

Engineer: D.VAN HOFF
Date: 4/20/2009 05:39



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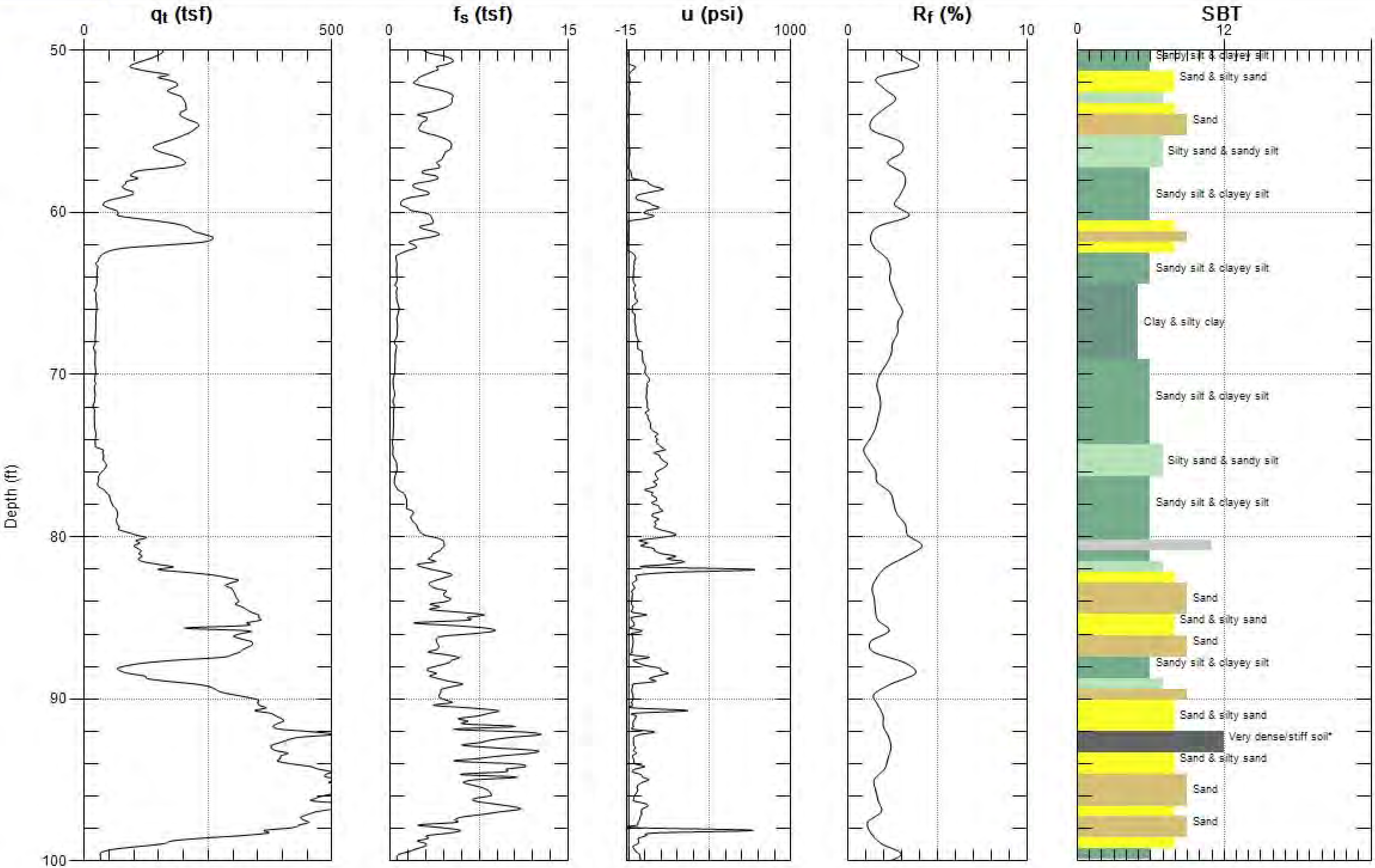

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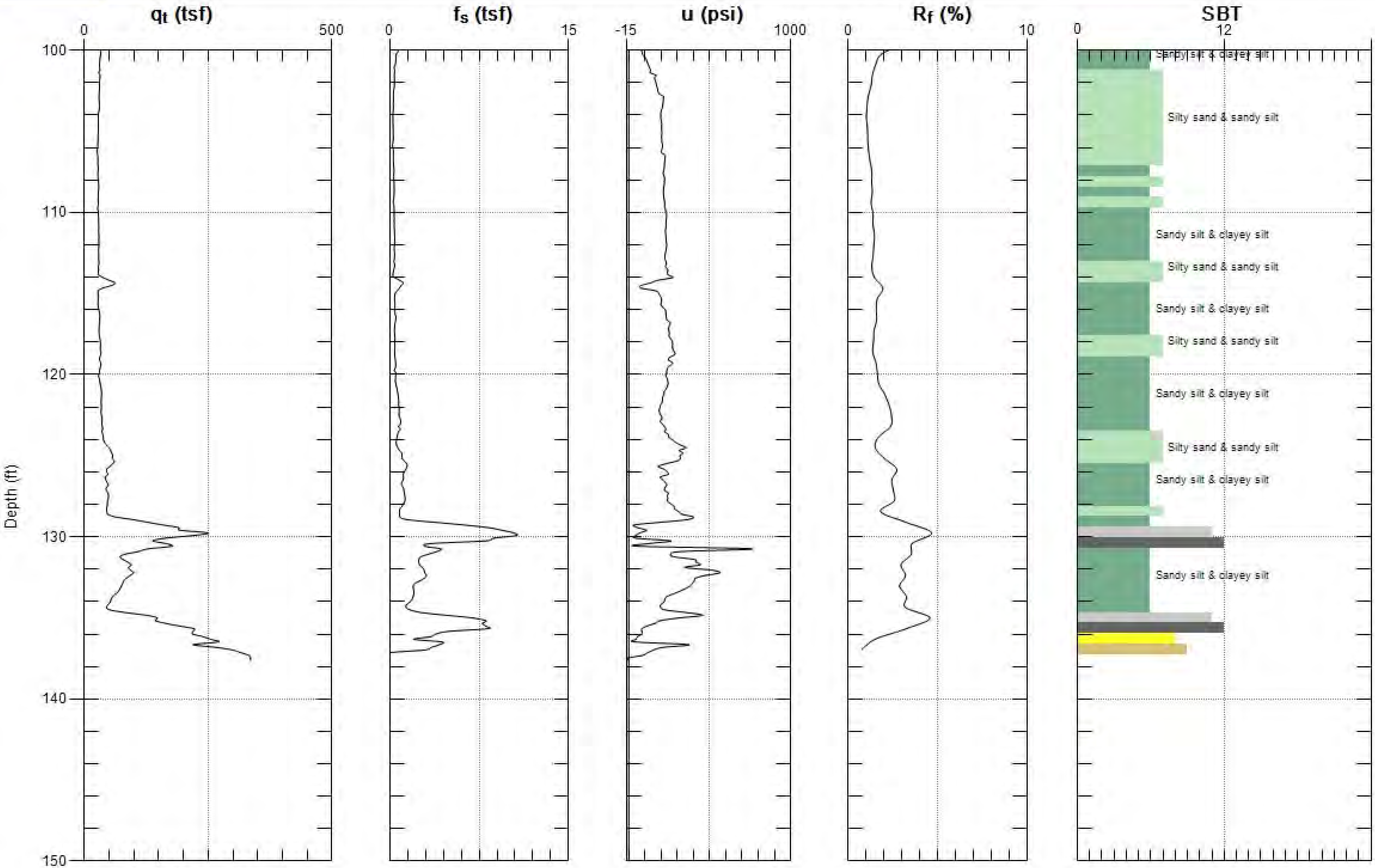
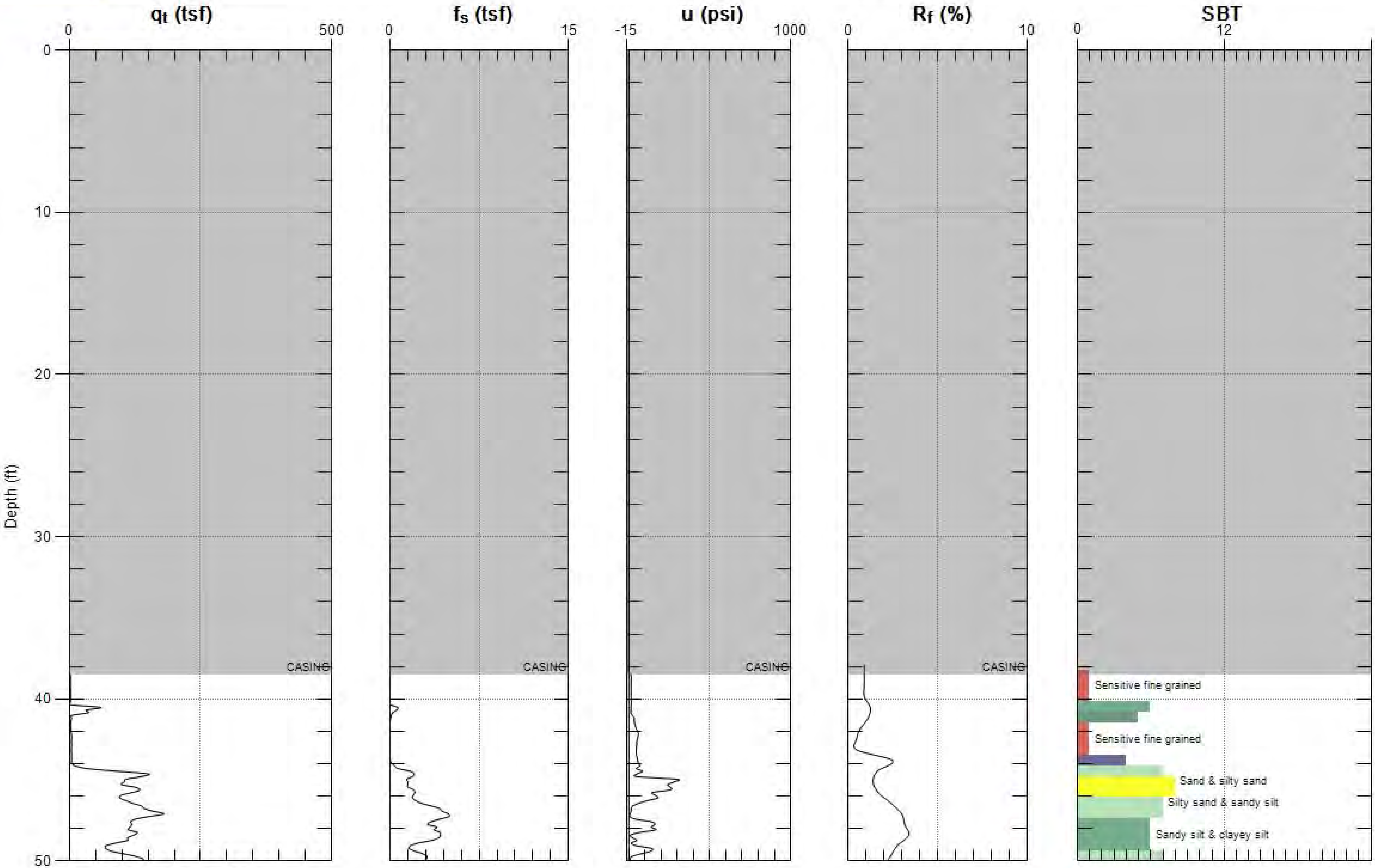


Max. Depth: 137.631 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GEOTECHNICAL CONSULTANTS

Site: BRANNAN ST. | Engineer: D.VAN HOFF
Sounding: CPT-02 | Date: 4/20/2009 03:23



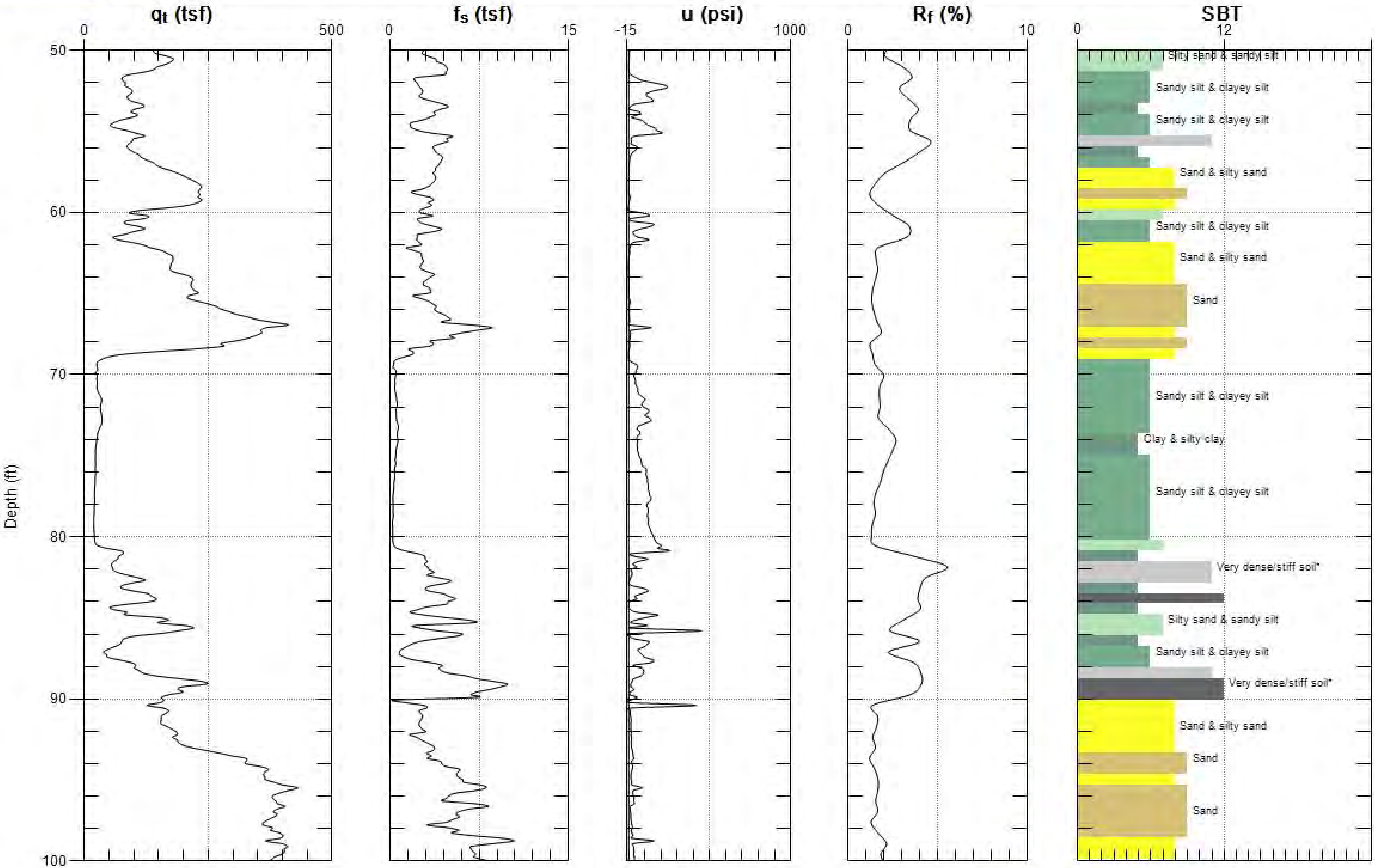
Max. Depth: 139.928 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG GEOTECHNICAL CONSULTANTS

Site: BRANNAN ST.
Sounding: CPT-02

Engineer: D.VAN HOFF
Date: 4/20/2009 03:23



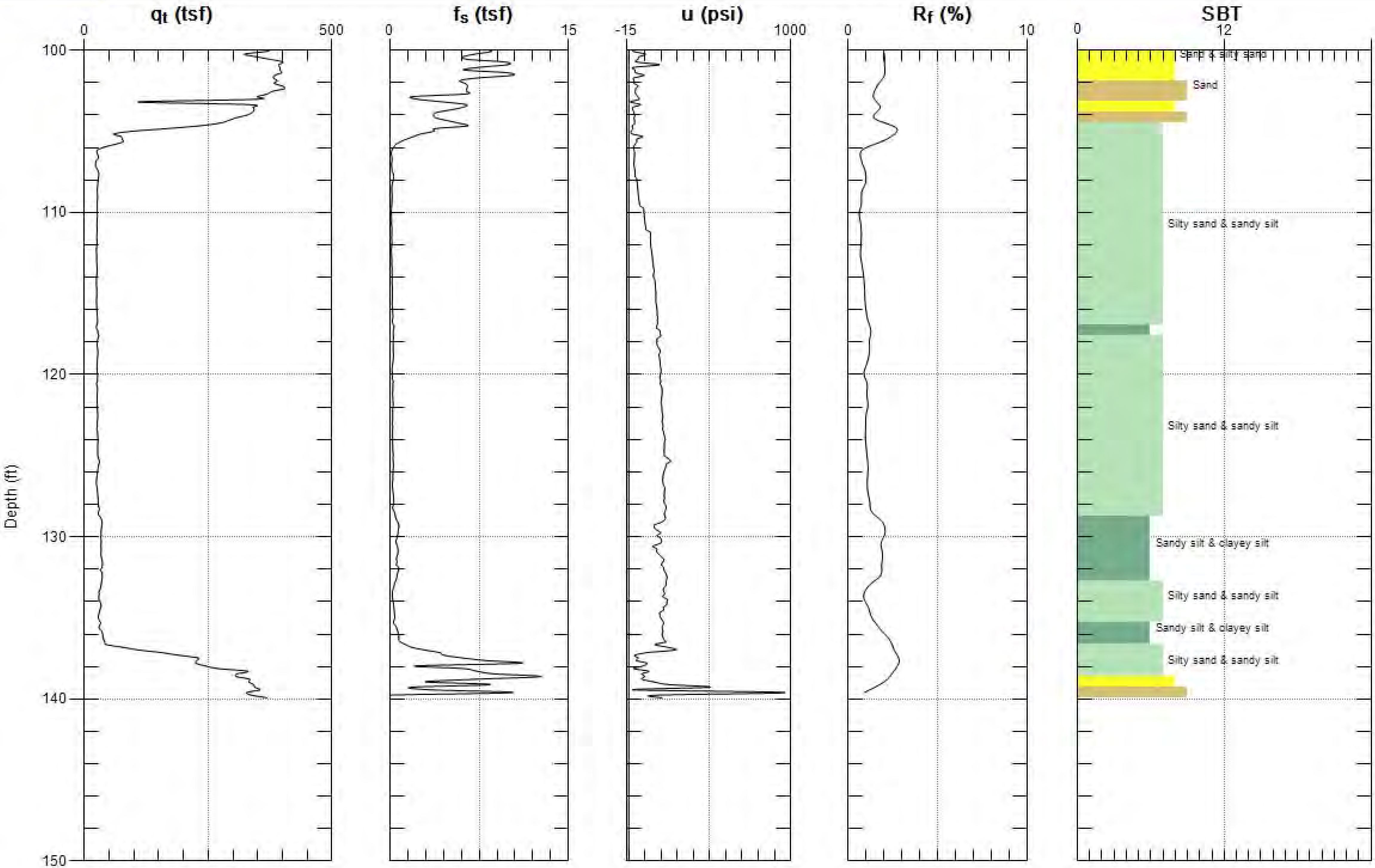
Max. Depth: 139.928 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG GEOTECHNICAL CONSULTANTS

Site: BRANNAN ST.
Sounding: CPT-02

Engineer: D.VAN HOFF
Date: 4/20/2009 03:23



Max. Depth: 139.928 (ft)
Avg. Interval: 0.656 (ft)

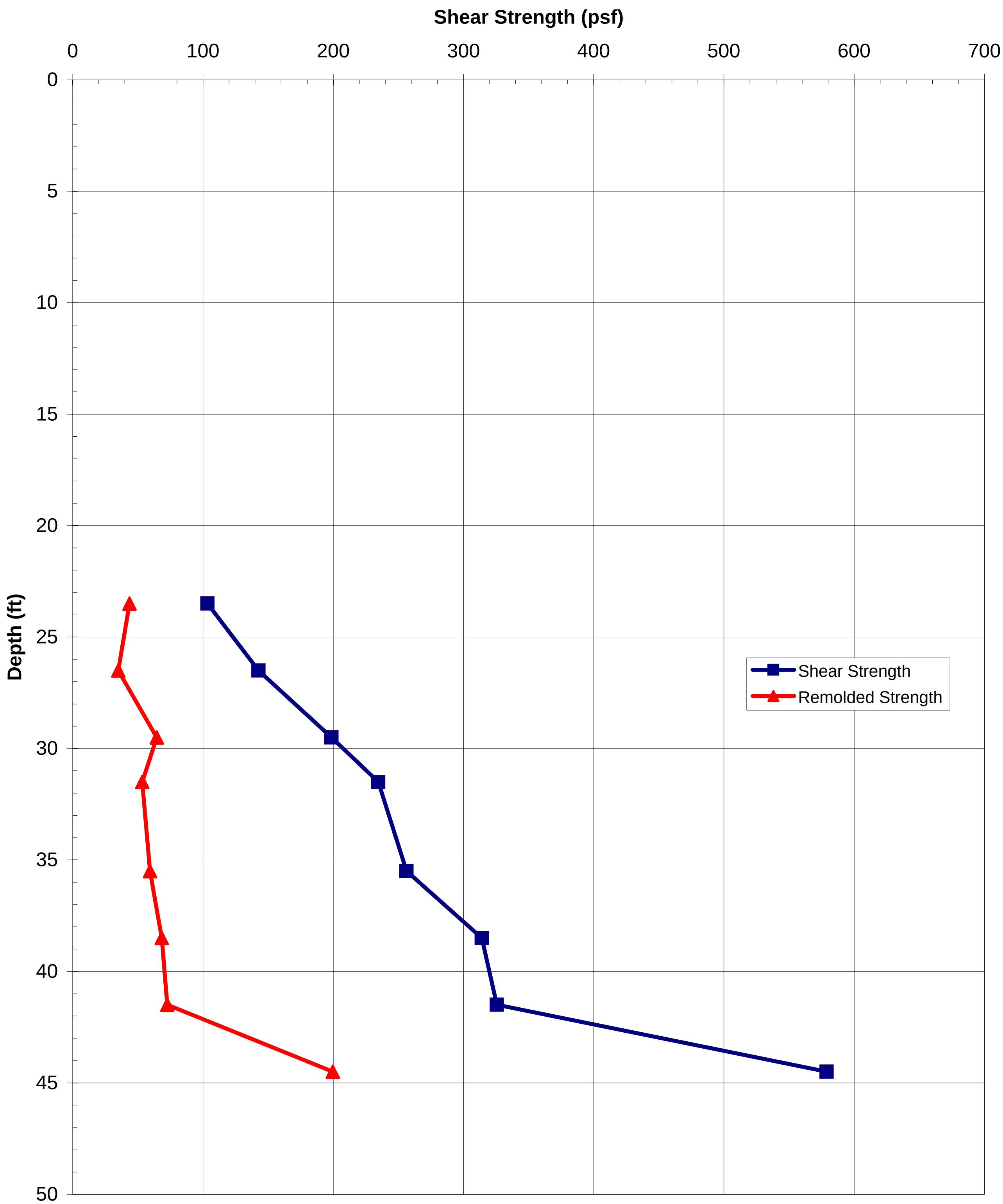
SBT: Soil Behavior Type (Robertson 1990)

APPENDIX C
VANE SHEAR TEST RESULTS

SITE NAME	Brannan St. Wharf	
LOCATION	B-3	
DATE	4/23/2009	
VANE TYPE	Geonor H-10	
VANE DIAMETER, d (mm)	65	Undrained Shear Strength, $c_u = 6M/7\pi D^3$
VANE LENGTH, l (mm)	130	Where M= Max. Recorded Torque, D=Vane Diameter

					REMOLDED			
DEPTH (m)	DEPTH (ft)	PEAK TORQUE READING (Nm)	SHEAR STRENGTH (kN/m^2)	SHEAR STRENGTH (psf)	PEAK DIAL READING (Nm)	SHEAR STRENGTH (kN/m^2)	SHEAR STRENGTH (psf)	SENSITIVITY
7.16	23.5	4.983	4.950	103.39	2.100	2.086	43.58	2.4
8.08	26.5	6.877	6.832	142.70	1.688	1.677	35.03	4.1
8.99	29.5	9.574	9.511	198.67	3.109	3.089	64.51	3.1
9.60	31.5	11.303	11.230	234.56	2.574	2.557	53.40	4.4
10.82	35.5	12.353	12.273	256.34	2.862	2.843	59.39	4.3
11.73	38.5	15.133	15.034	314.02	3.294	3.273	68.36	4.6
12.65	41.5	15.689	15.587	325.56	3.500	3.477	72.63	4.5
13.56	44.5	27.898	27.716	578.91	9.625	9.562	199.73	2.9

BRANNAN STREET WHARF: B-3 SUMMARY
SHEAR STRENGTH VS. DEPTH

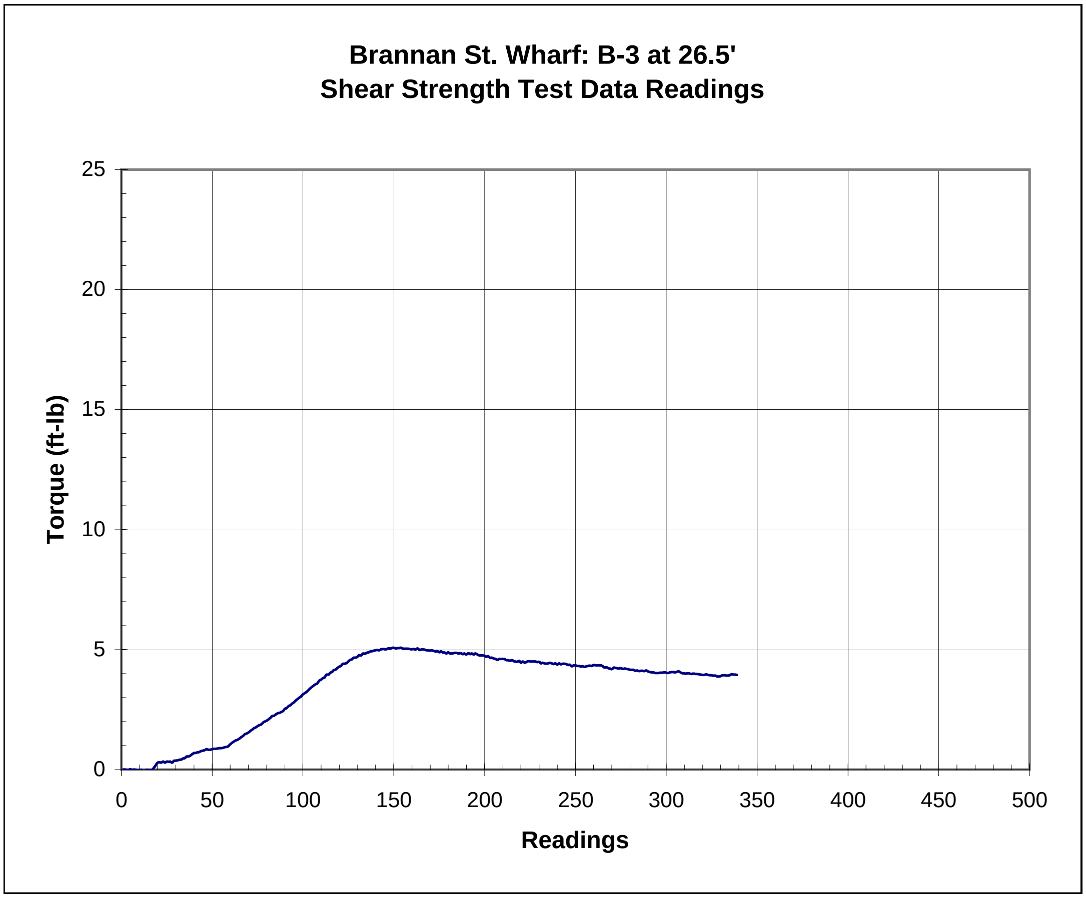


BRANNAN ST. WHARF: B-3 AT 23.5 FEET

Baseline mV	Peak mV	Difference mV	Calibration ft-lb/mV	Torque ft-lb	Torque Nm
-0.584	-0.342	0.242	15.18577	3.674956	4.982506

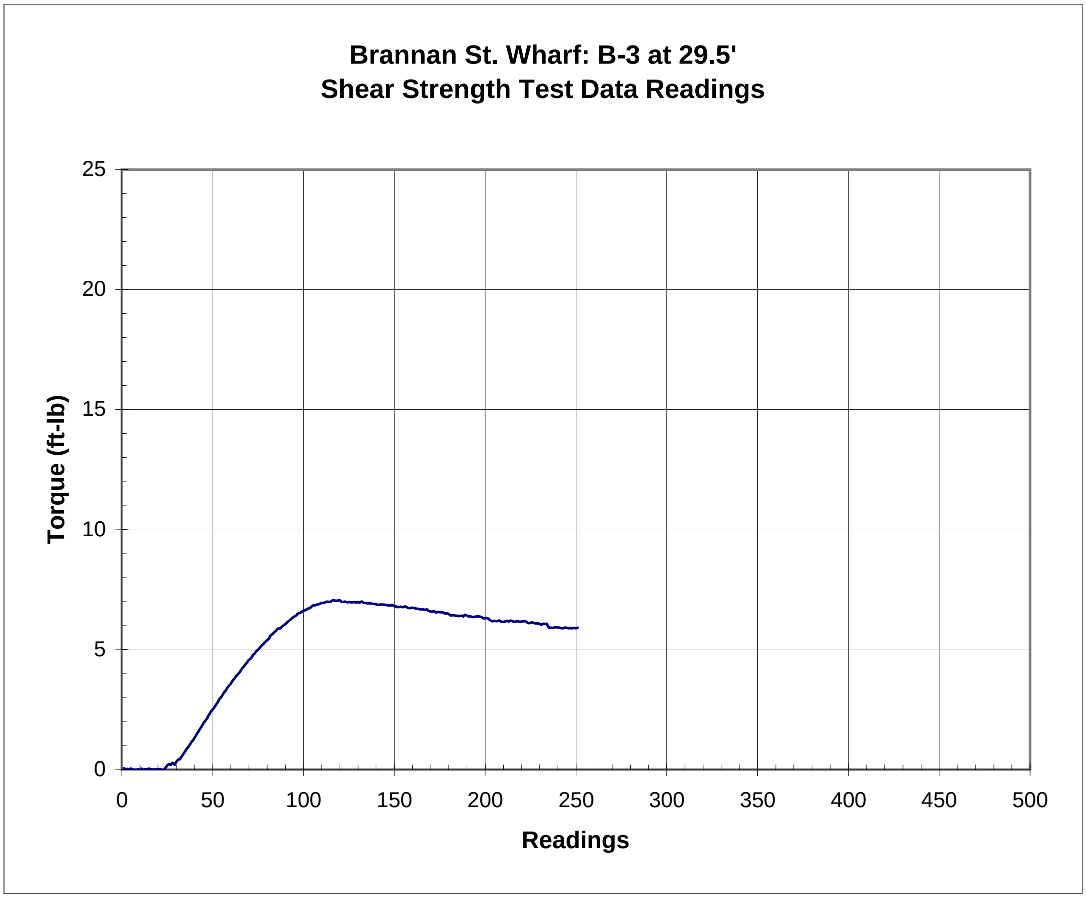
BRANNAN ST. WHARF: B-3 AT 26.5 FEET

Baseline	Peak	Difference	Calibration	Torque	Torque
mV	mV	mV	ft-lb/mV	ft-lb	Nm
-0.626	-0.292	0.334	15.18577	5.072047	6.876682



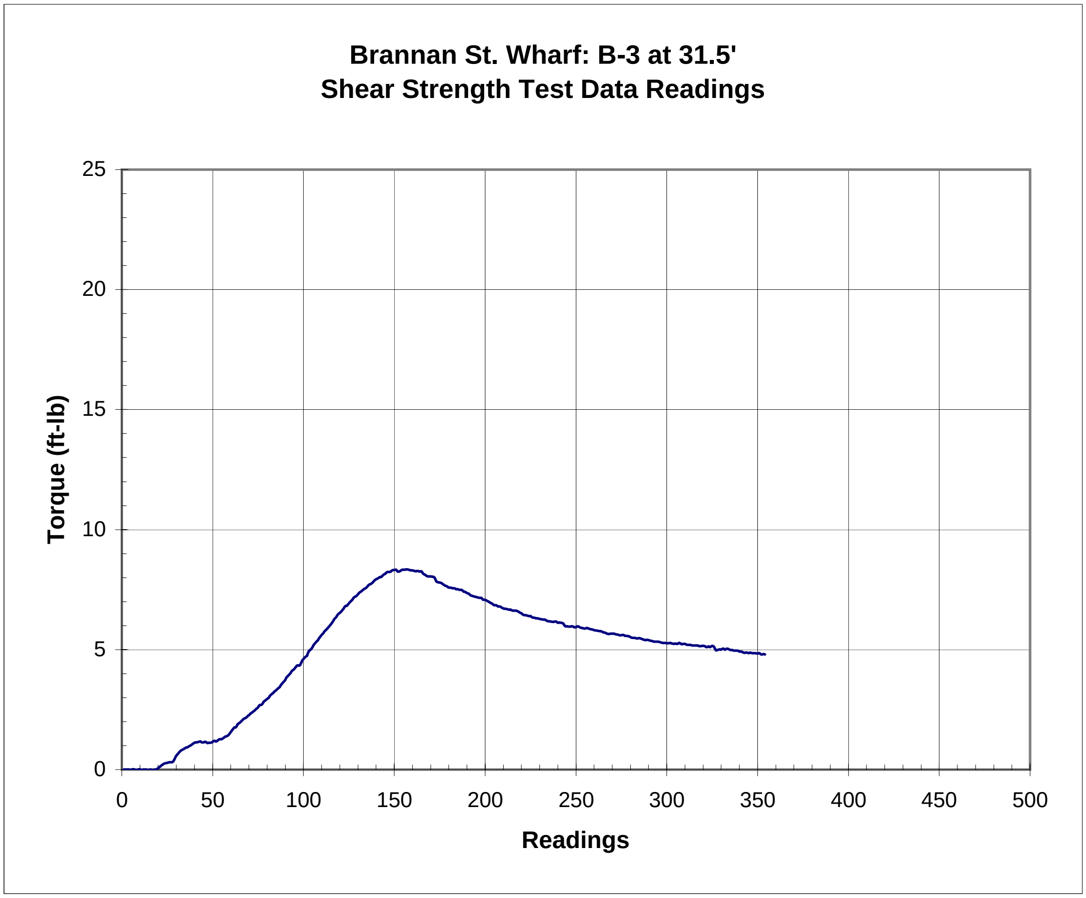
BRANNAN ST. WHARF: B-3 AT 29.5 FEET

Baseline	Peak	Difference	Calibration	Torque	Torque
mV	mV	mV	ft-lb/mV	ft-lb	Nm
-0.636	-0.171	0.465	15.18577	7.061383	9.573823



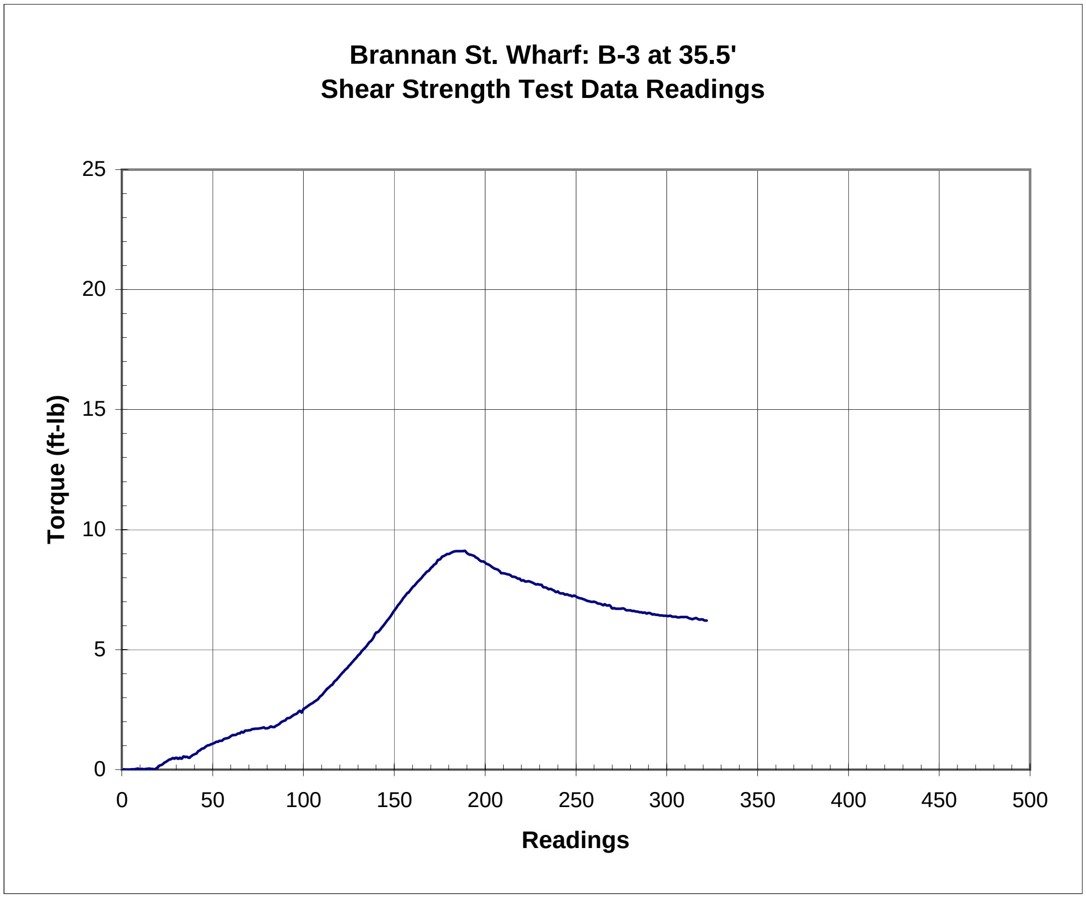
BRANNAN ST. WHARF: B-3 AT 31.5 FEET

Baseline	Peak	Difference	Calibration	Torque	Torque
mV	mV	mV	ft-lb/mV	ft-lb	Nm
-0.624	-0.075	0.549	15.18577	8.336988	11.30329

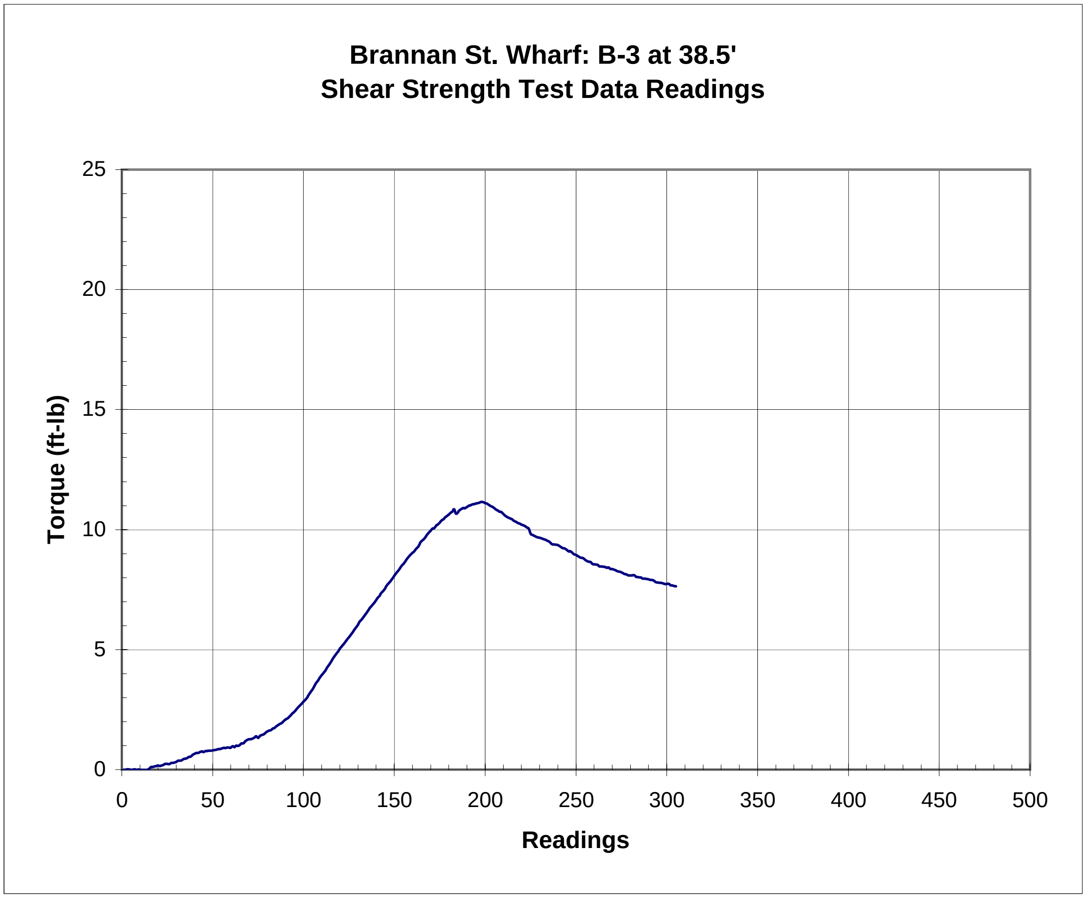


BRANNAN ST. WHARF: B-3 AT 35.5 FEET

Baseline	Peak	Difference	Calibration	Torque	Torque
mV	mV	mV	ft-lb/mV	ft-lb	Nm
-0.609	-0.009	0.6	15.18577	9.111462	12.35332

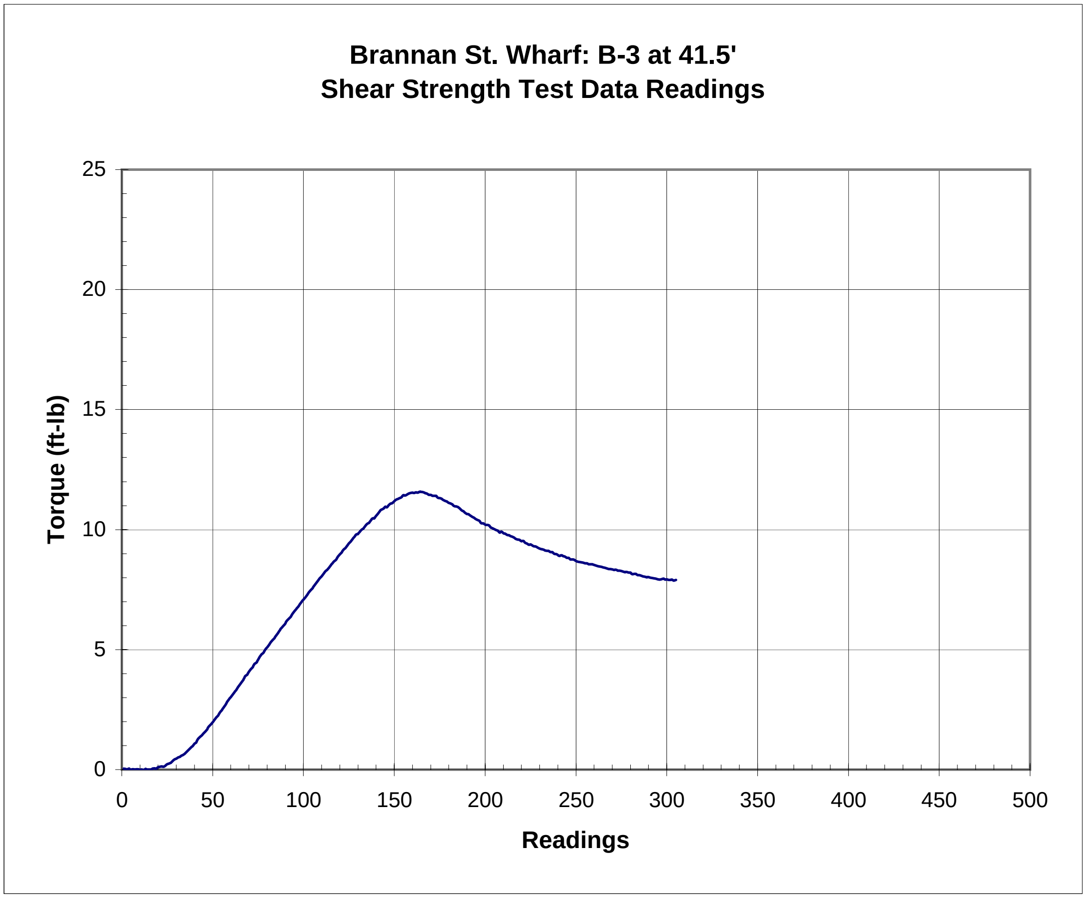


Baseline	Peak	Difference	Calibration	Torque	Torque
mV	mV	mV	ft-lb/mV	ft-lb	Nm
-0.615	0.12	0.735	15.18577	11.16154	15.13282



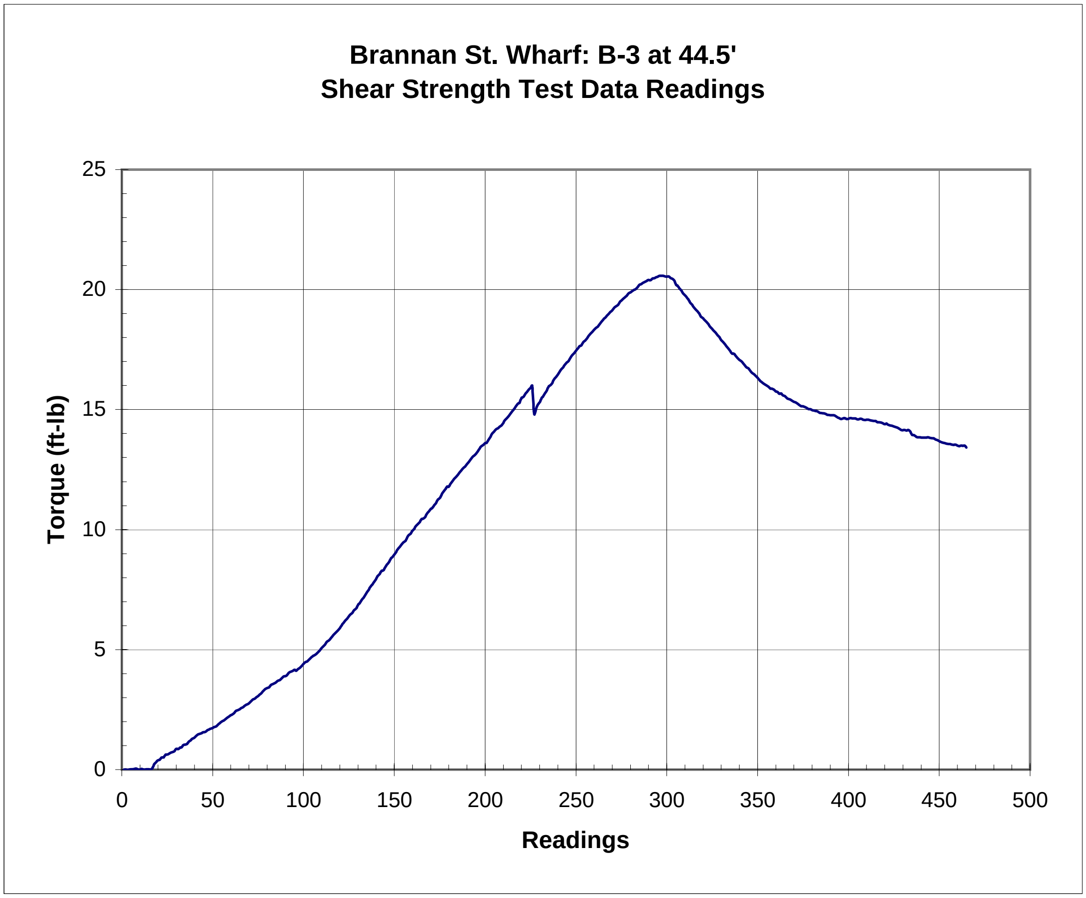
BRANNAN ST. WHARF: B-3 AT 41.5 FEET

Baseline mV	Peak mV	Difference mV	Calibration ft-lb/mV	Torque ft-lb	Torque Nm
-0.622	0.14	0.762	15.18577	11.57156	15.68872



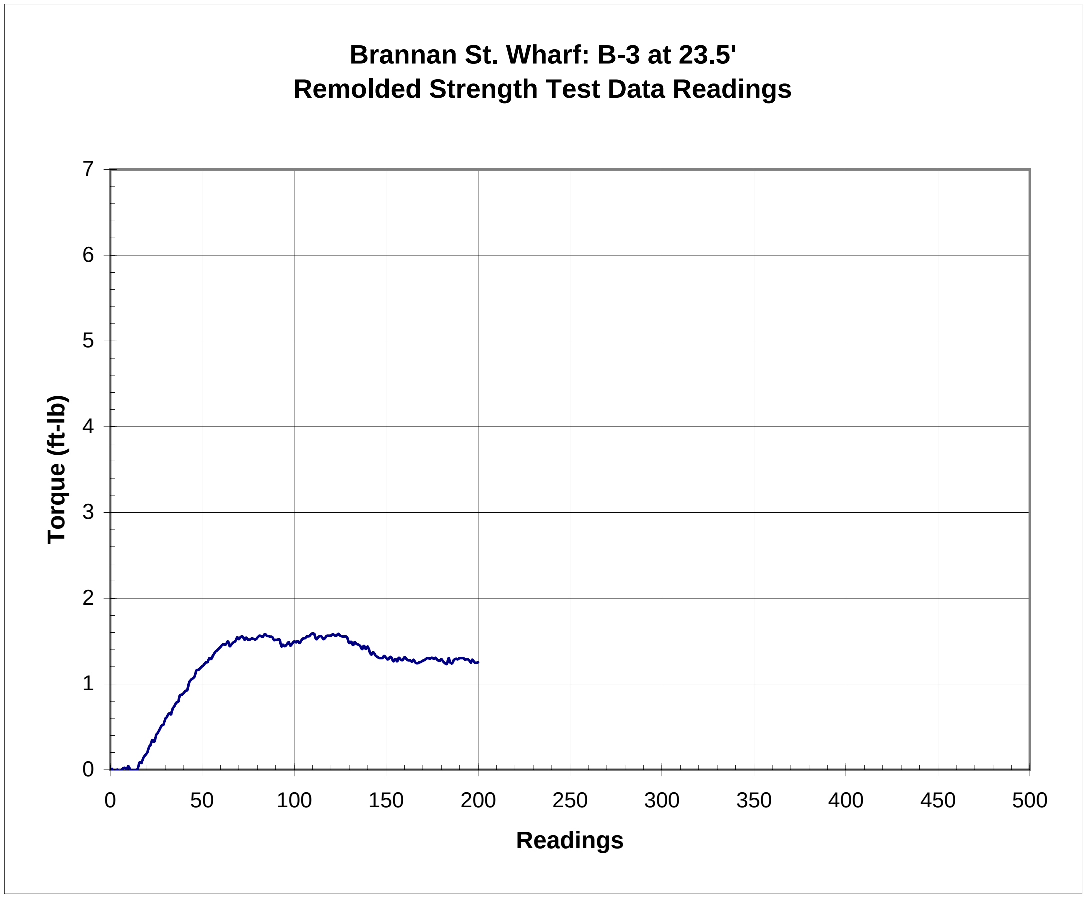
BRANNAN ST. WHARF: B-3 AT 44.5 FEET

Baseline mV	Peak mV	Difference mV	Calibration ft-lb/mV	Torque ft-lb	Torque Nm
-0.614	0.741	1.355	15.18577	20.57672	27.89791

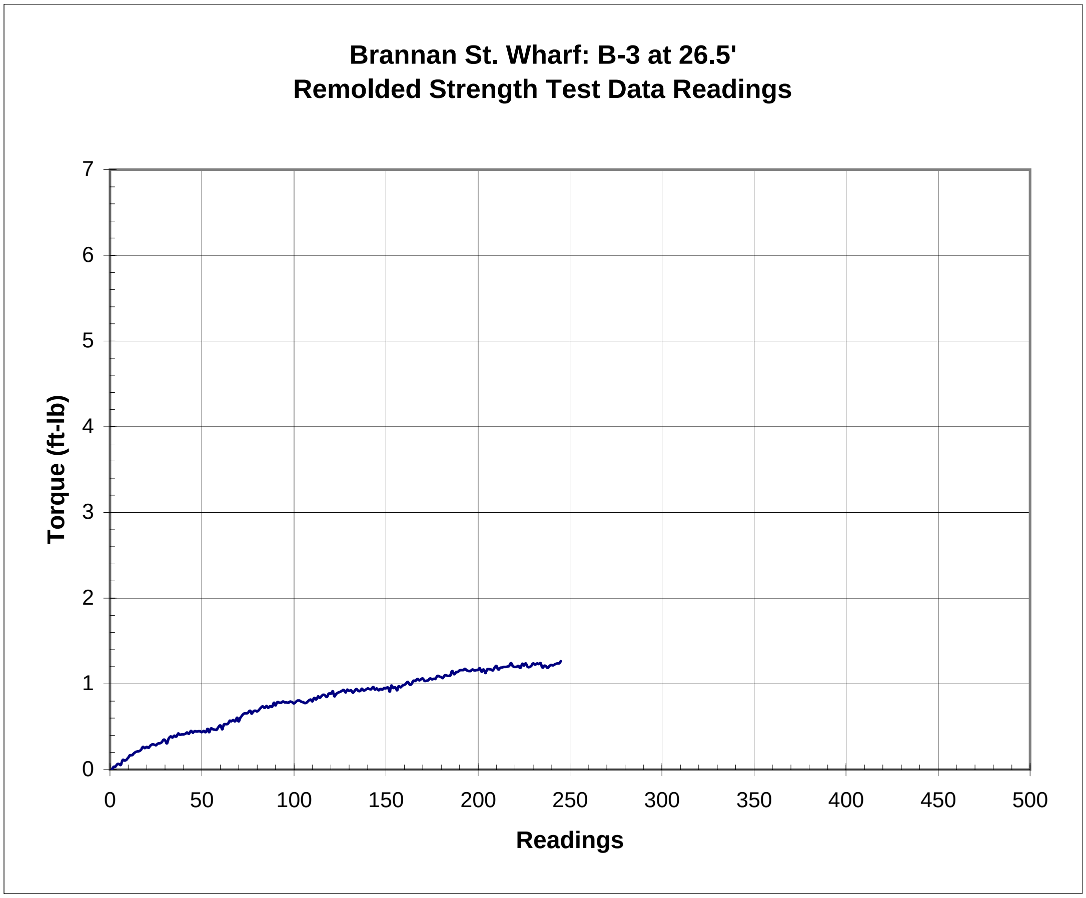


BRANNAN ST. WHARF: B-3 AT 23.5 FEET (Remolded)

Baseline mV	Peak mV	Difference mV	Calibration ft-lb/mV	Torque ft-lb	Torque Nm
-0.62	-0.518	0.102	15.18577	1.548949	2.100064

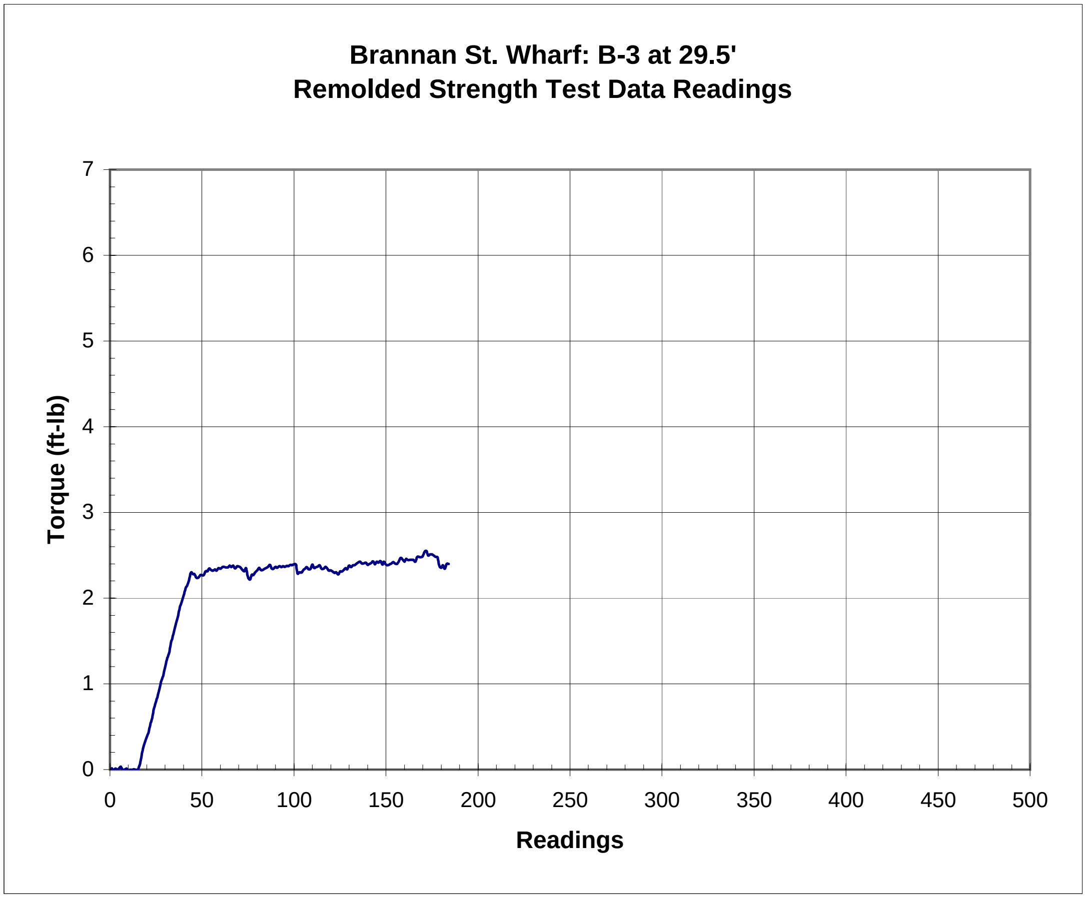


Baseline	Peak	Difference	Calibration	Torque	Torque
mV	mV	mV	ft-lb/mV	ft-lb	Nm
-0.601	-0.519	0.082	15.18577	1.245233	1.688287



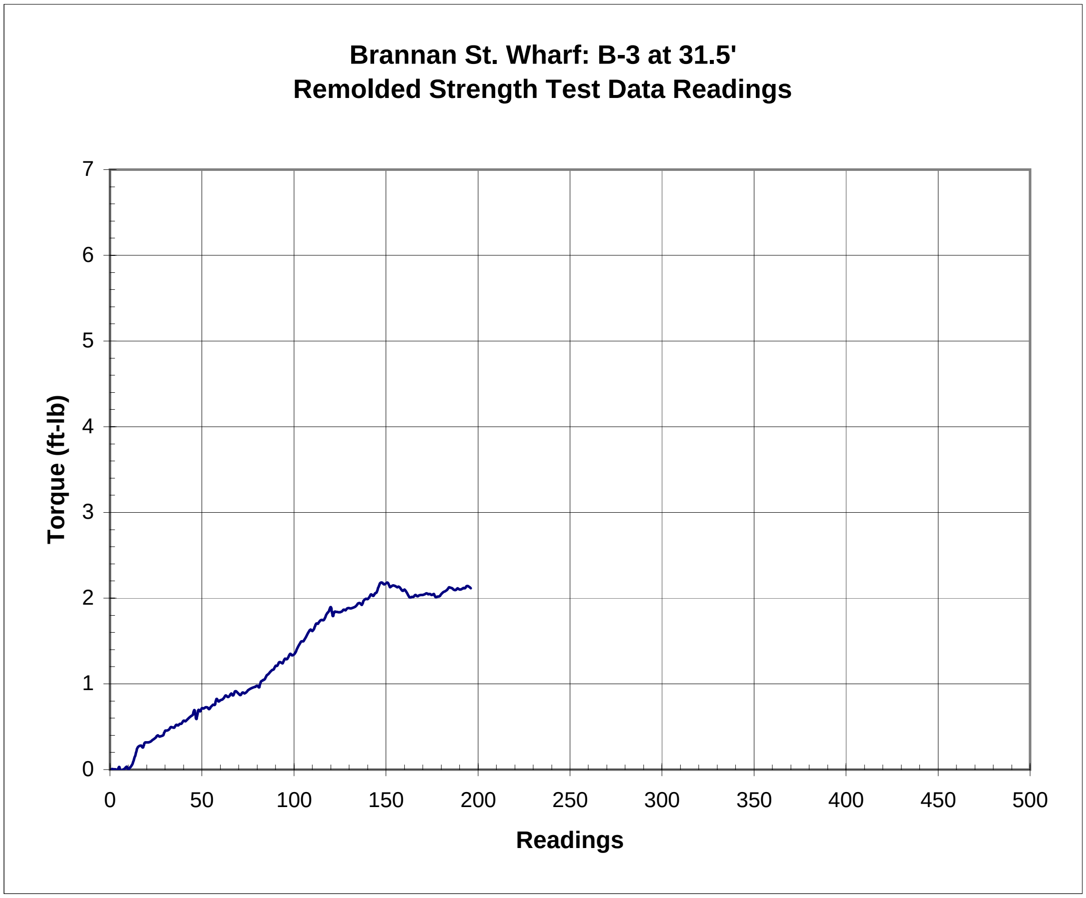
BRANNAN ST. WHARF: B-3 AT 29.5 FEET (Remolded)

Baseline	Peak	Difference	Calibration	Torque	Torque
mV	mV	mV	ft-lb/mV	ft-lb	Nm
-0.632	-0.481	0.151	15.18577	2.293051	3.108919



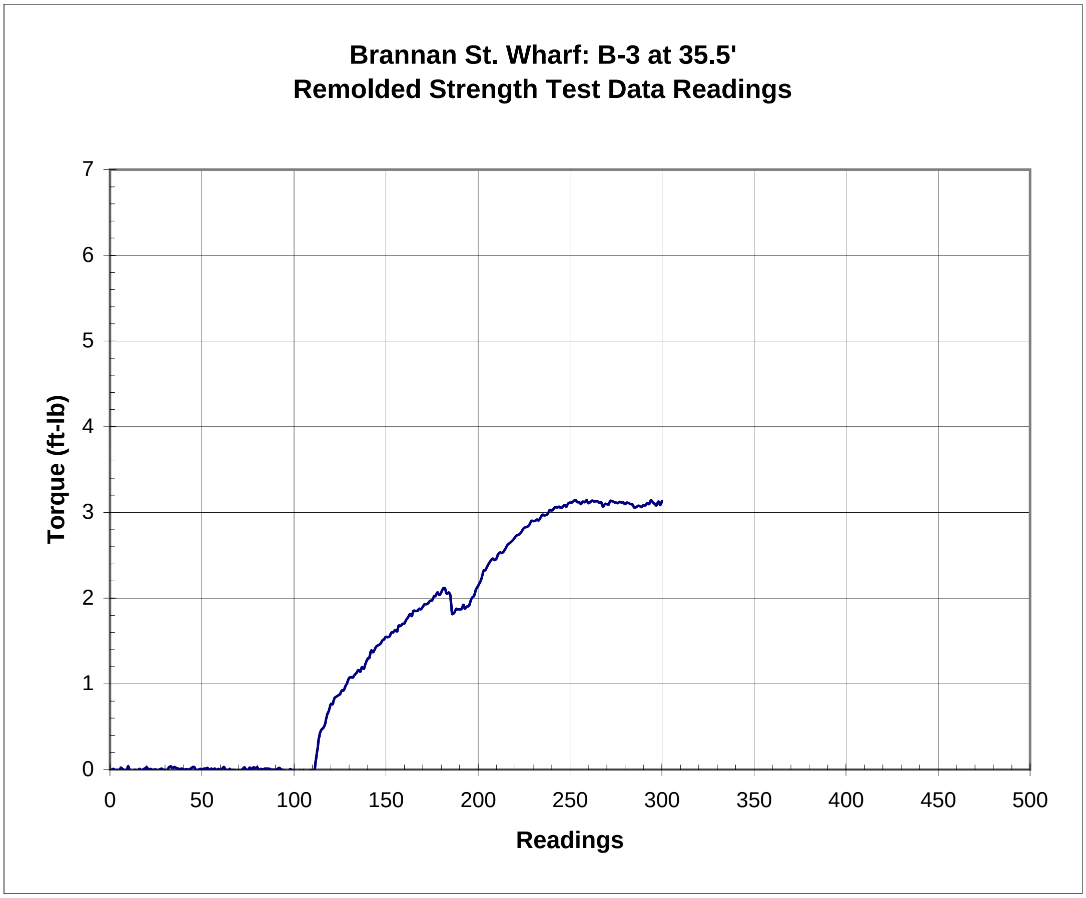
BRANNAN ST. WHARF: B-3 AT 31.5 FEET (Remolded)

Baseline	Peak	Difference	Calibration	Torque	Torque
mV	mV	mV	ft-lb/mV	ft-lb	Nm
-0.618	-0.493	0.125	15.18577	1.898221	2.573608



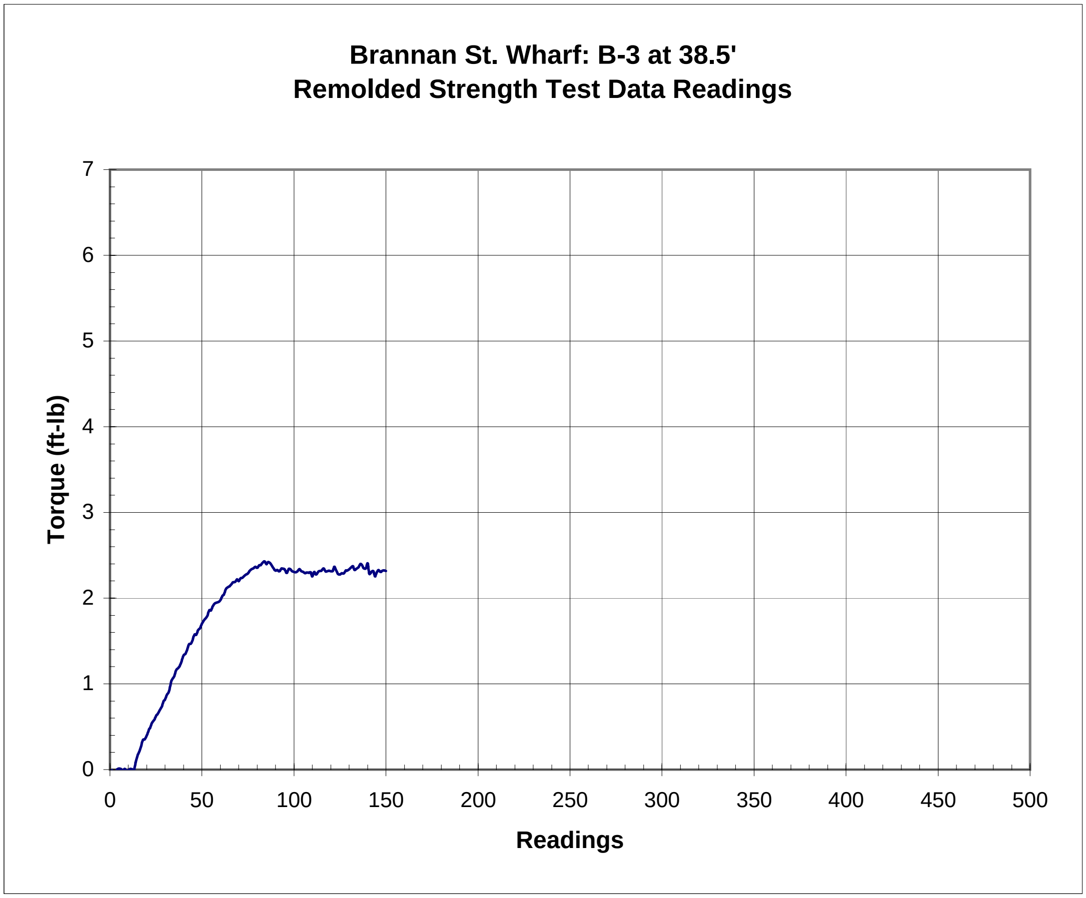
BRANNAN ST. WHARF: B-3 AT 35.5 FEET (Remolded)

Baseline mV	Peak mV	Difference mV	Calibration ft-lb/mV	Torque ft-lb	Torque Nm
-0.627	-0.488	0.139	15.18577	2.110822	2.861853



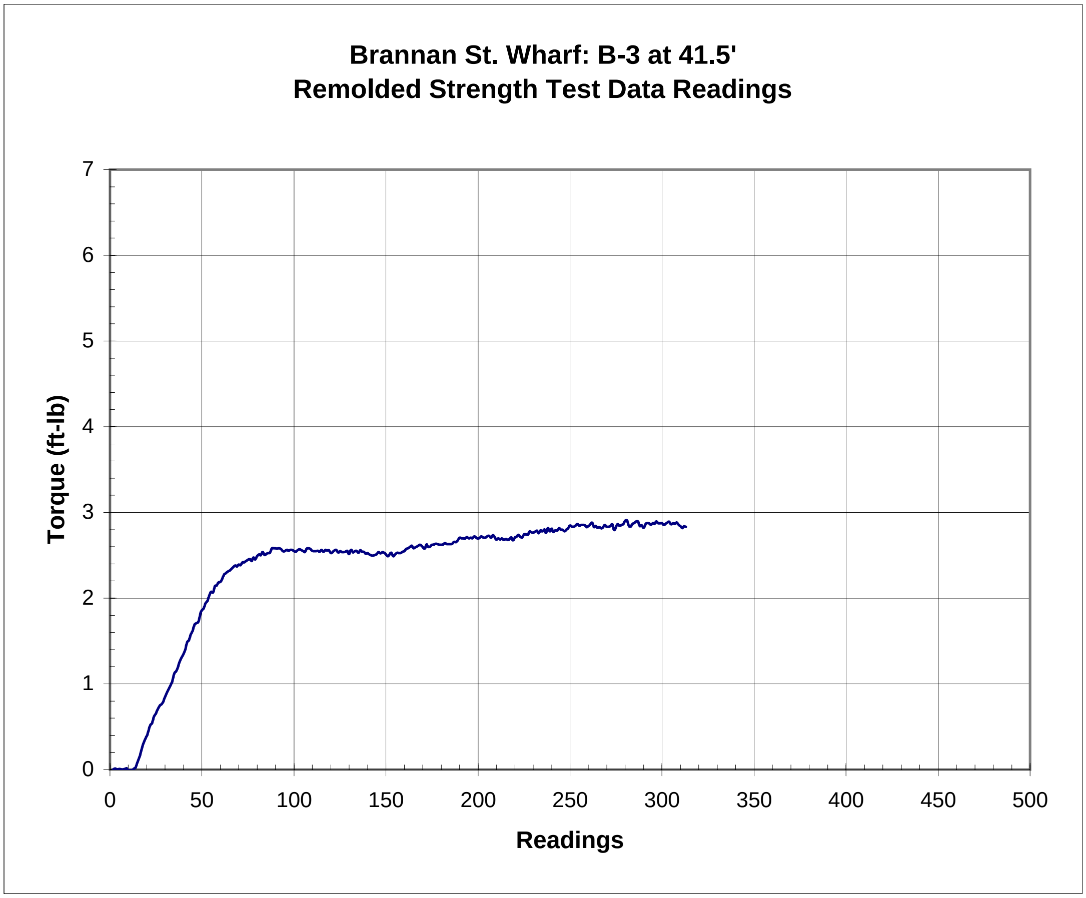
BRANNAN ST. WHARF: B-3 AT 38.5 FEET (Remolded)

Baseline	Peak	Difference	Calibration	Torque	Torque
mV	mV	mV	ft-lb/mV	ft-lb	Nm
-0.611	-0.451	0.16	15.18577	2.429723	3.294219



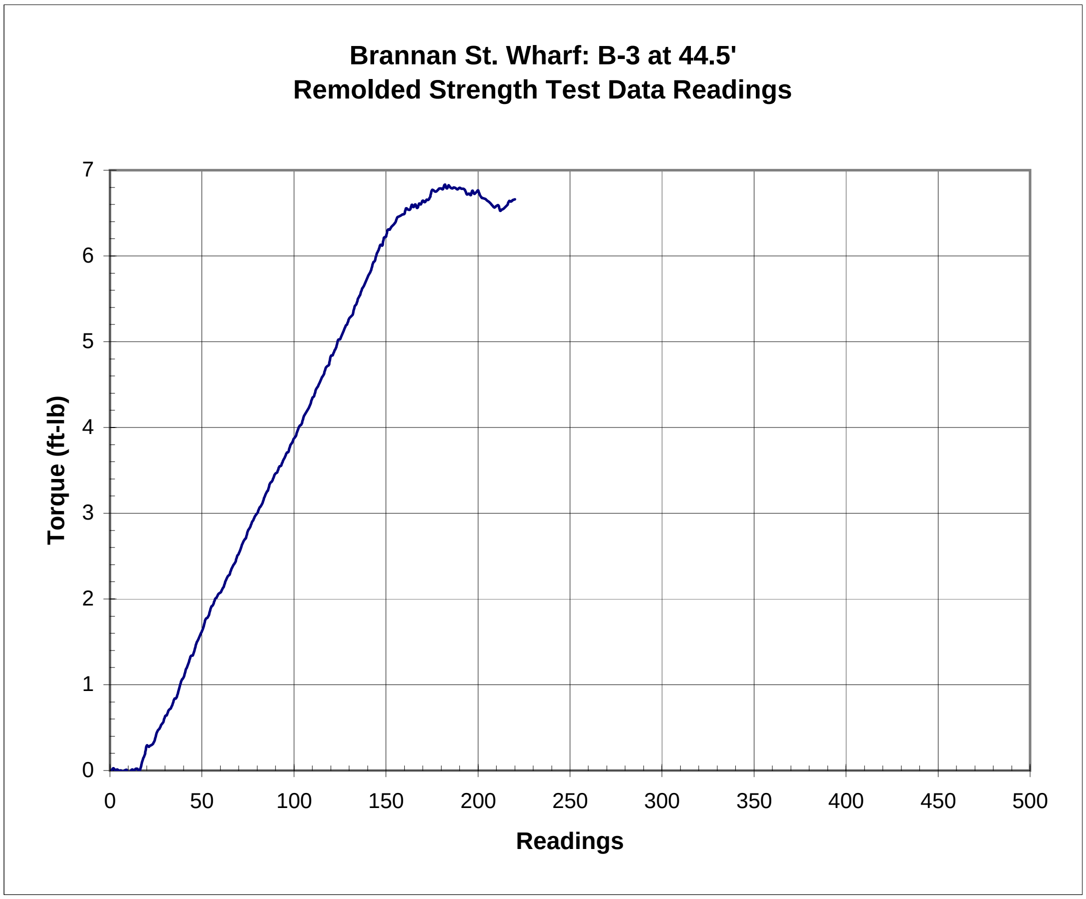
BRANNAN ST. WHARF: B-3 AT 41.5 FEET (Remolded)

Baseline	Peak	Difference	Calibration	Torque	Torque
mV	mV	mV	ft-lb/mV	ft-lb	Nm
-0.61	-0.44	0.17	15.18577	2.581581	3.500107



BRANNAN ST. WHARF: B-3 AT 44.5 FEET (Remolded)

Baseline	Peak	Difference	Calibration	Torque	Torque
mV	mV	mV	ft-lb/mV	ft-lb	Nm
-0.593	-0.143	0.45	15.18577	6.833597	9.26499



APPENDIX D
LABORATORY TEST RESULTS

APPENDIX D
LABORATORY TEST RESULTS

LABORATORY TESTING

Laboratory tests were performed on representative soil samples in order to define the engineering properties of the earth materials. Testing procedures followed accepted practice where possible. Where ASTM Standards were used, the latest edition or revision for each test procedure was employed.

MOISTURE AND DENSITY DETERMINATIONS

Moisture content and dry density determinations were performed on representative samples to evaluate the natural water content and dry density of the soils encountered. The results are presented on the boring logs.

GRAIN SIZE DISTRIBUTION DATA

Grain-size distribution tests were conducted on representative samples. The tests were performed in accordance with Standard Test Method ASTM D422 - Standard Method for Particle-Size Analysis of Soils. Results of these tests are included in this Appendix.

ATTERBERG LIMITS

Atterberg limits were performed on selected soil samples. Testing was performed in accordance with ASTM D4218 - Liquid Limit, Plastic Limit, and Plasticity Index of Soils. Results of these tests are presented on the boring logs, and included in this Appendix.

UNCONFINED COMPRESSION TESTS

Unconfined compression tests were performed on a total of six soil samples. Testing was performed in accordance with ASTM D2166 – Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. Results of these tests are presented on the boring logs, and included in this Appendix.

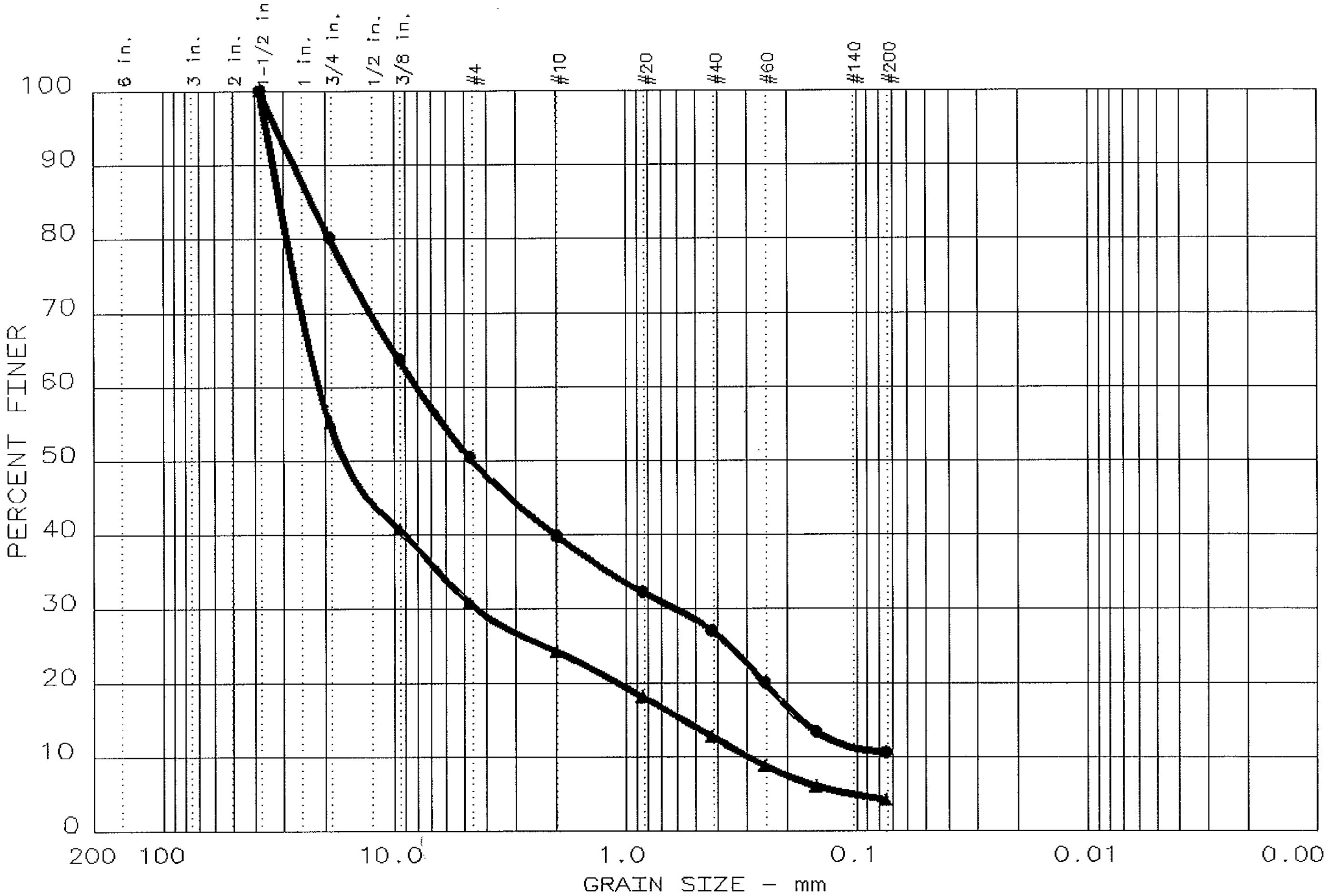
UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS (UU)

Unconsolidated undrained triaxial tests were performed on six soil samples. Testing was performed in accordance with Standard Test Method ASTM D2850 – Unconsolidated Undrained Triaxial Test on Cohesive Soils. The results of these tests are included in this Appendix.

CONSOLIDATION TESTS

A consolidation test was performed on one soil sample of young bay mud. Testing was performed in accordance with ASTM D2435 – Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading. The results of this test are included in this Appendix.

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	49.5	39.9	10.6		GP-GC		
▲	0.0	69.3	26.5	4.2		GP		

SIEVE inches size	PERCENT FINER ●	▲	
1.5	100.0	100.0	
0.75	80.1	55.2	
0.375	63.7	40.7	
	GRAIN SIZE		
D_{60}	7.94	21.09	
D_{30}	0.60	4.46	
D_{10}		0.29	
	COEFFICIENTS		
C_c		3.20	
C_u		71.6	

SIEVE number size	PERCENT FINER ●	▲	
4	50.5	30.7	
10	39.8	24.2	
20	32.2	18.2	
40	27.0	12.8	
60	20.1	8.8	
100	13.4	6.1	
200	10.6	4.2	

Sample information:

● B-1 10-11'
Dark brown sandy GRAVEL(Fill)

▲ B-1 20-21'
Dark brown sandy GRAVEL(Fill)

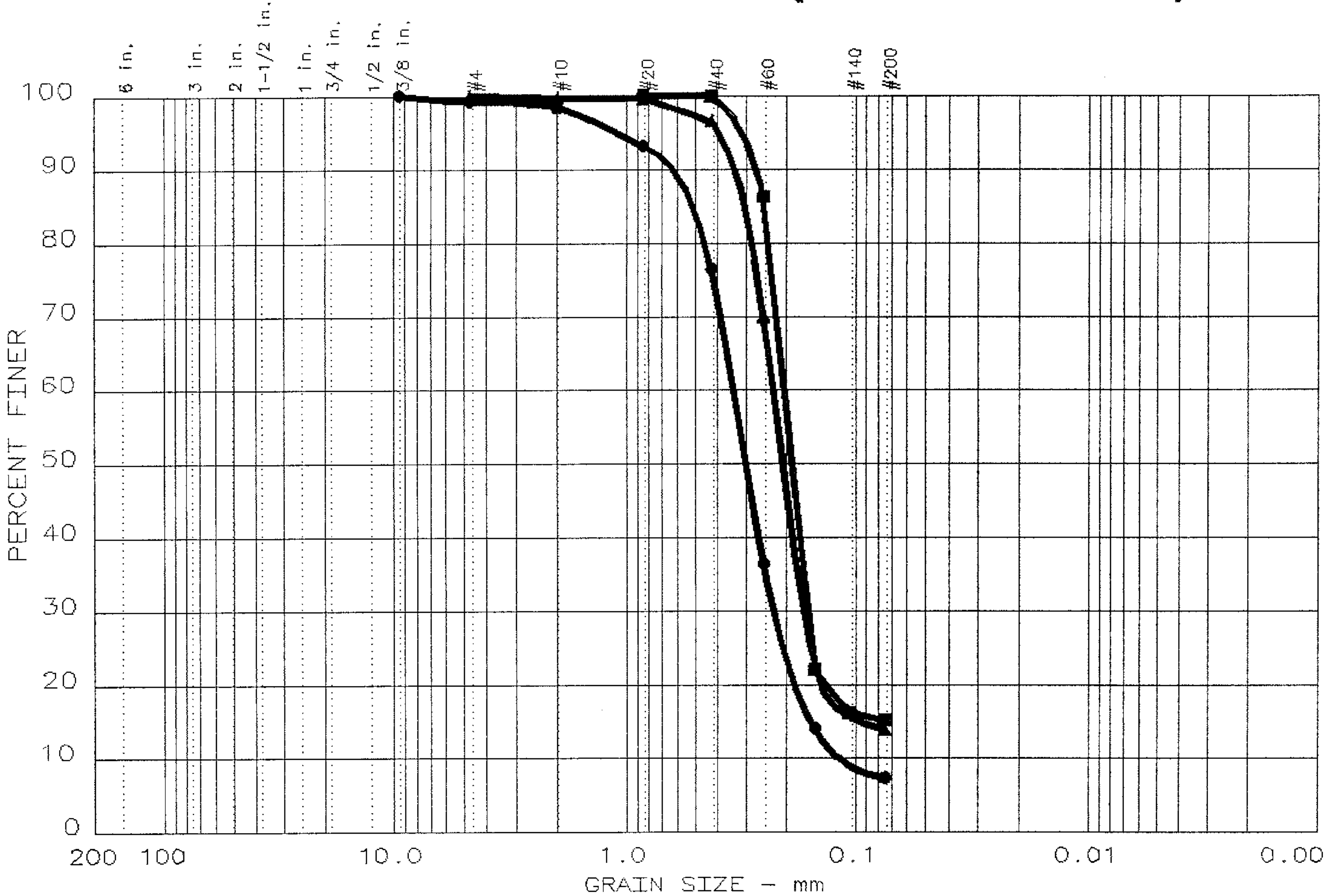
Remarks:

Soil Mechanics Lab

Project No.: SF09005
Project: Brannan St. Wharf

Date: 5-23-09 Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.8	91.9	7.3		SP-SM		
▲	0.0	0.0	86.2	13.8		SM		
■	0.0	0.0	84.8	15.2		SM		

SIEVE inches size	PERCENT FINER ●	▲	■
0.375	100.0		
	GRAIN SIZE		
D_{60}	0.34	0.23	0.20
D_{30}	0.23	0.17	0.16
D_{10}	0.11		
	COEFFICIENTS		
C_c	1.28		
C_u	2.9		

SIEVE number size	PERCENT FINER ●	▲	■
4	99.2	100.0	
10	98.3	99.7	
20	93.1	99.6	100.0
40	76.4	96.6	99.9
60	36.4	70.0	86.2
100	14.0	22.7	22.1
140			16.2
200	7.3	13.8	15.2

Sample information:

● B-1 36.5-37'
Dark gray fine SAND.

▲ B-1 61-61.5'
Gray/brown silty f-SAND

■ B-1 81-81.5'
Dk.olive brown silty f-SAND.

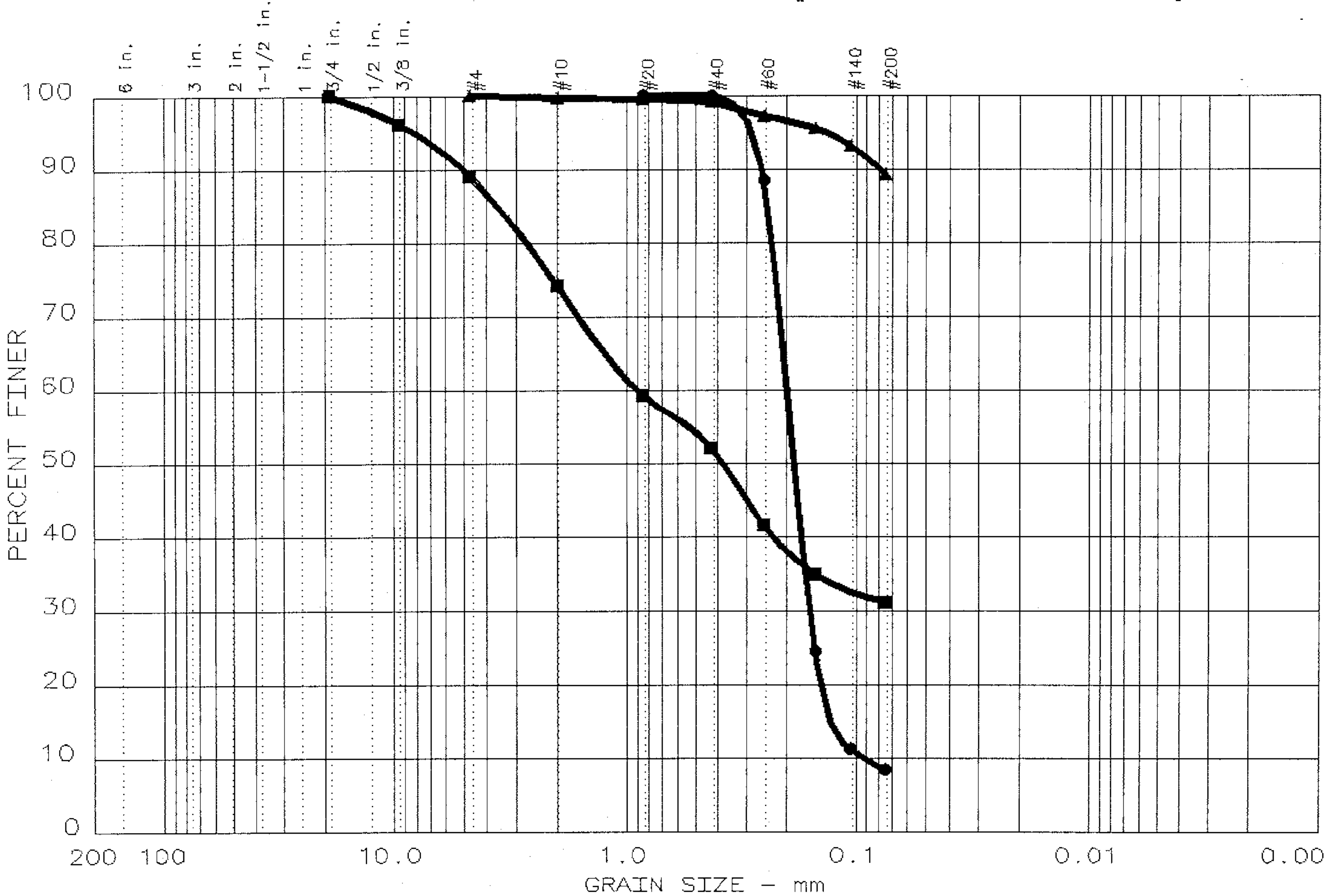
Remarks:

Soil Mechanics Lab

Project No.: SF09005
Project: Brannan St. Wharf

Date: 5-23-09 Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.0	91.5	8.5		SP-SM		
▲	0.0	0.0	10.8	89.2		CH		
■	0.0	10.8	58.1	31.1		SM/SC		

SIEVE inches size	PERCENT FINER ●	▲	■
0.75			100.0
0.375			96.2
	GRAIN SIZE		
D_{60}	0.20		0.89
D_{30}	0.16		
D_{10}	0.09		
	COEFFICIENTS		
C_c	1.38		
C_u	2.2		

SIEVE number size	PERCENT FINER ●	▲	■
4		100.0	89.2
10		99.7	74.2
20	100.0	99.5	59.3
40	99.9	99.1	52.1
60	88.4	97.3	41.6
100	24.6	95.5	34.9
140	11.3	93.2	
200	8.5	89.2	31.1

Sample information:

● B-1 90.5-91.5'
Dk.brown fine SAND.

▲ B-1 101-101.5'
Dk.gray FAT CLAY.

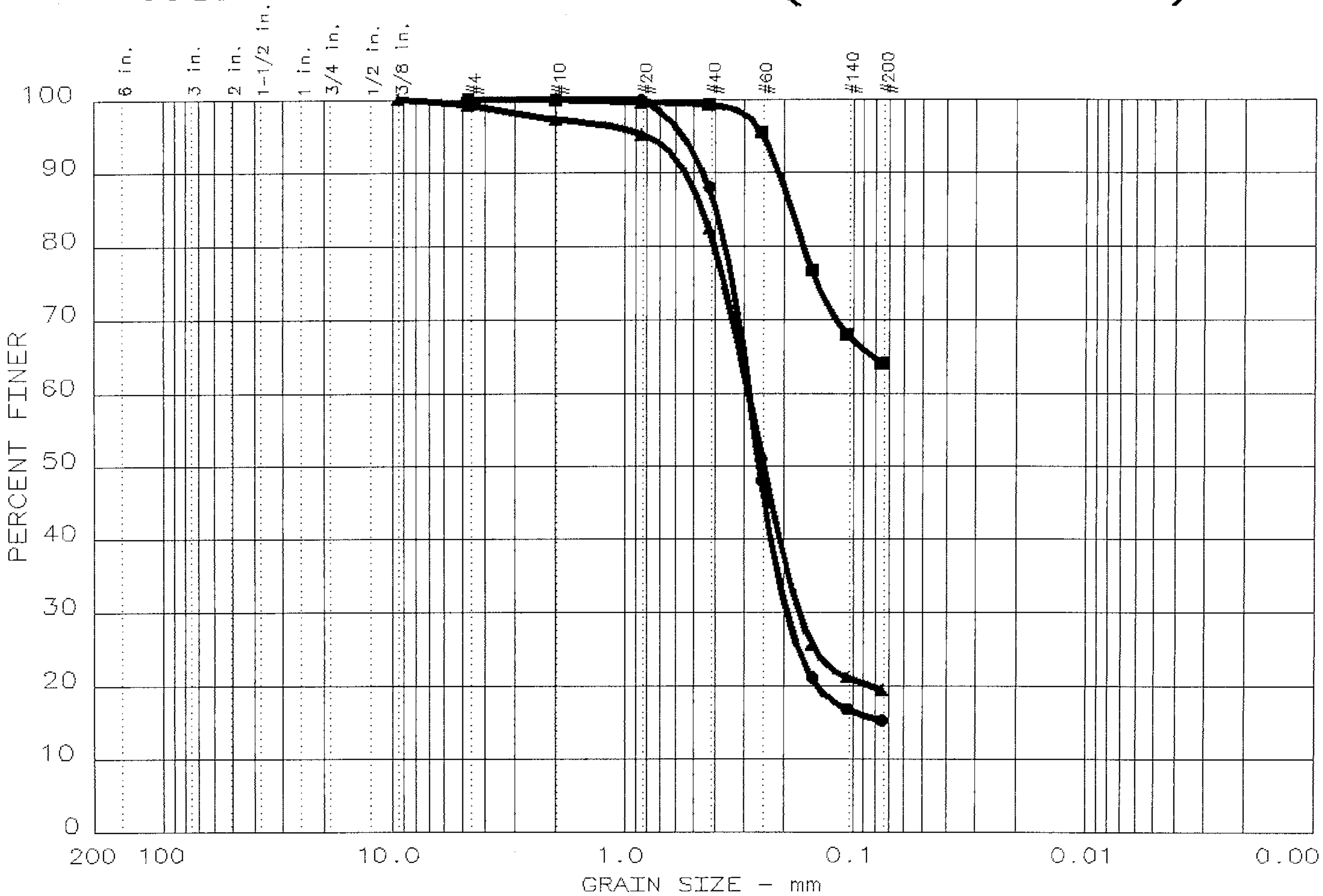
■ B-1 111-111.5'
Black silty,clayey f-c SAND portion of tube.

Remarks:

Soil Mechanics Lab

Project No.: SF09005
Project: Brannan St. Wharf
Date: 5-23-09
Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.0	84.8	15.2		SM		
▲	0.0	0.8	79.8	19.4		SM		
■	0.0	0.0	36.0	64.0		CL		

SIEVE inches size	PERCENT FINER ●	▲	■
0.375		100.0	
	GRAIN SIZE		
D_{60}	0.28	0.28	
D_{30}	0.19	0.17	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ●	▲	■
4		99.2	100.0
10	100.0	97.3	99.9
20	99.9	95.2	99.6
40	87.9	82.3	99.2
60	48.0	51.6	95.5
100	21.1	25.6	76.6
140	16.8	21.2	68.0
200	15.2	19.4	64.0

Sample information:

● B-3 46-46.5'
Dk.to very dk. gr/brn. silty f-SAND.

▲ B-3 50.5-51.5'
Dk.brown silty f-SAND.

■ B-3 76-76.5'
Gray sandy lean CLAY.

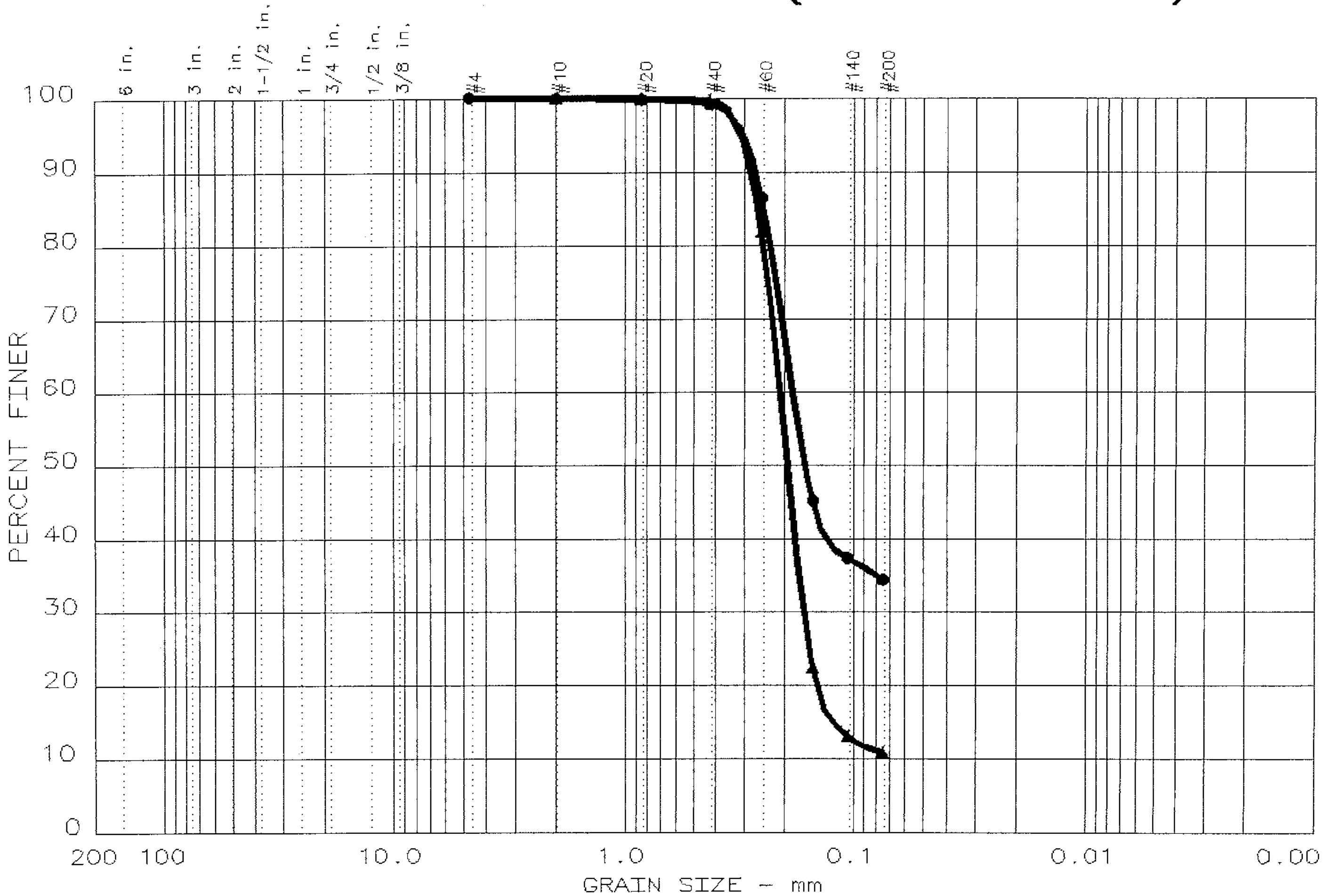
Remarks:

Soil Mechanics Lab

Project No.: SF09005
Project: Brannan St. Wharf

Date: 5-23-09 Data Sheet No. ______

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.0	65.6	34.4		SC		
▲	0.0	0.0	89.1	10.9		SP-SM		

SIEVE inches size	PERCENT FINER ●	PERCENT FINER ▲	
	GRAIN SIZE		
D_{60}	0.18	0.21	
D_{30}		0.16	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ●	PERCENT FINER ▲	
4	100.0		
10	99.9	100.0	
20	99.8	99.9	
40	99.2	99.7	
60	86.5	81.8	
100	45.2	22.4	
140	37.4	13.1	
200	34.4	10.9	

Sample information:

● B-3 86-86.5'
Olive gray clayey fine SAND.

▲ B-3 95.5-96.5'
Dark brown fine SAND.

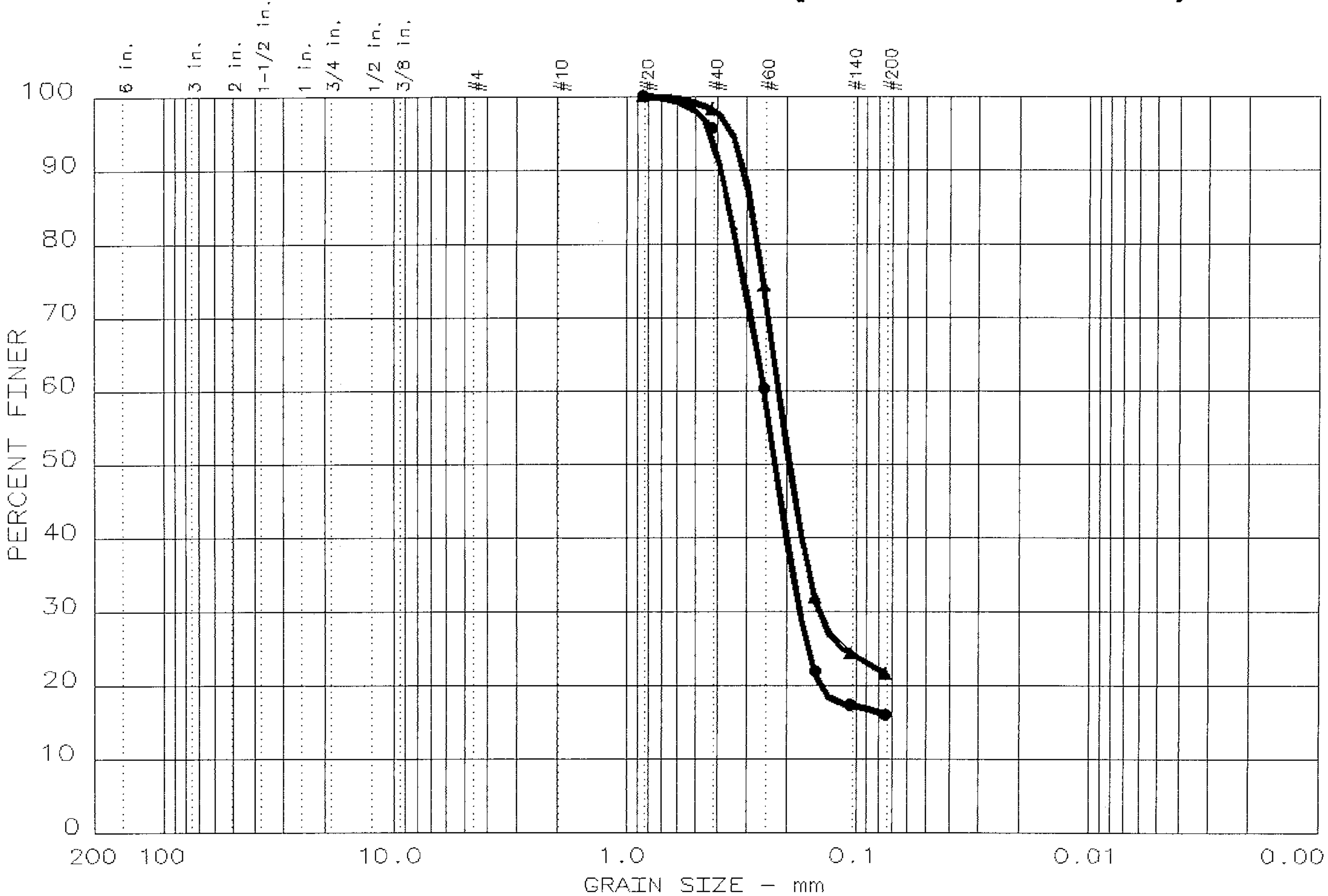
Remarks:

Soil Mechanics Lab

Project No.: SF09005
Project: Brannan St. Wharf

Date: 5-23-09 Data Sheet No. ____

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.0	84.0	16.0		SM		
▲	0.0	0.0	78.3	21.7		SM/SC		

SIEVE inches size	PERCENT FINER ●	▲	
	GRAIN SIZE		
D_{60}	0.25	0.21	
D_{30}	0.17	0.14	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ●	▲	
20	100.0	100.0	
40	95.7	98.2	
60	60.3	74.3	
100	21.9	32.0	
140	17.4	24.4	
200	16.0	21.7	

Sample information:

● B-4 51-51.5'
Dark brown silty f-SAND

▲ B-4 60.5-61.5'
Dark brown silty f-SAND

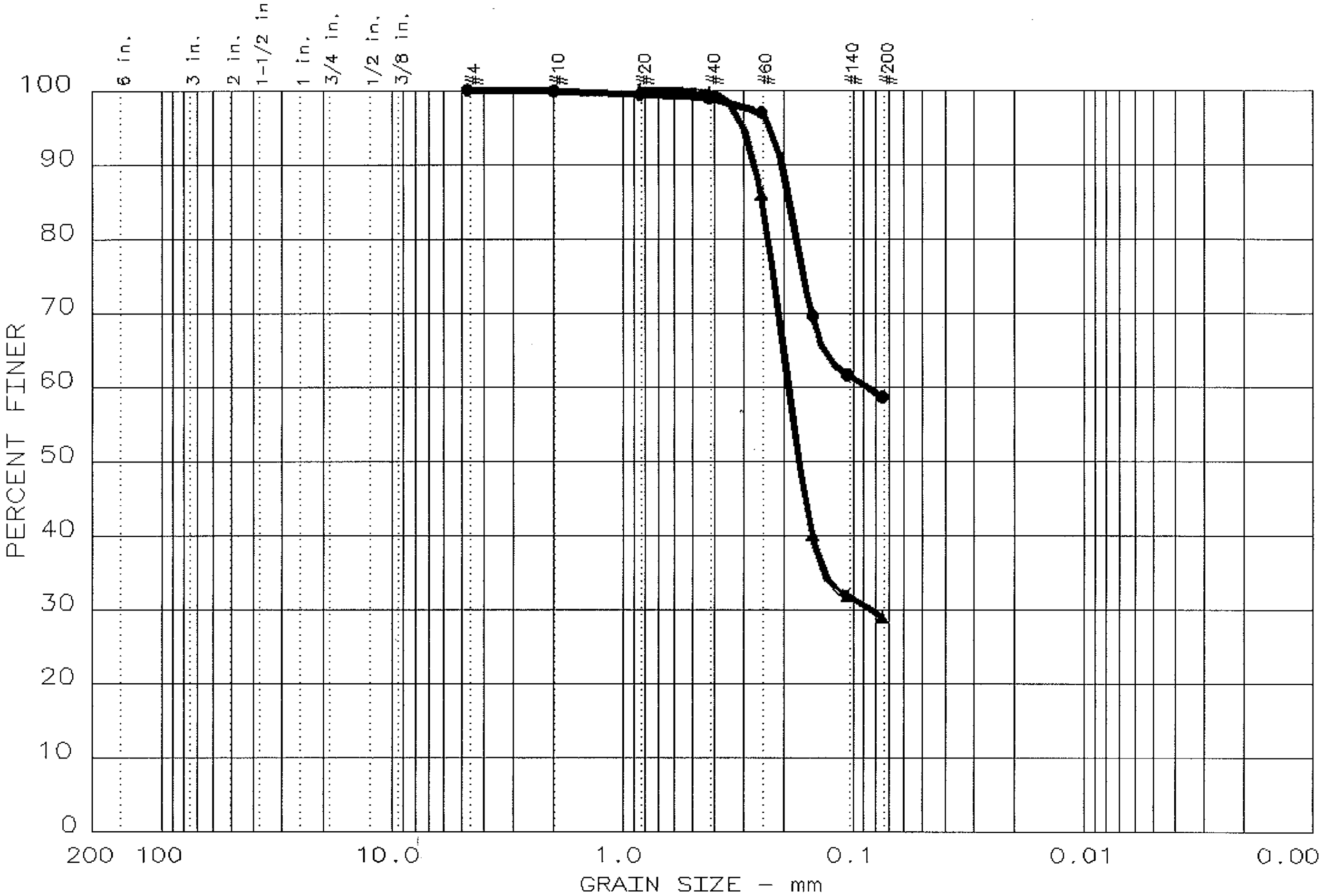
Remarks:

Soil Mechanics Lab

Project No.: SF09005
Project: Brannan St. Wharf

Date: 5-23-09 Data Sheet No. ______

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.0	41.4	58.6		CL		
▲	0.0	0.0	71.0	29.0		SM		

SIEVE inches size	PERCENT FINER ●	▲	
	GRAIN SIZE		
D_{60}	0.09	0.19	
D_{30}		0.08	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ●	▲	
4	100.0		
10	99.9		
20	99.4	100.0	
40	98.9	99.6	
60	97.0	85.9	
100	69.6	40.1	
140	61.6	31.9	
200	58.6	29.0	

Sample information:

● B-4 81-81.5'
Dark gray sandy CLAY.

▲ B-4 91-91.5'
Olive gray/brown silty SAND.

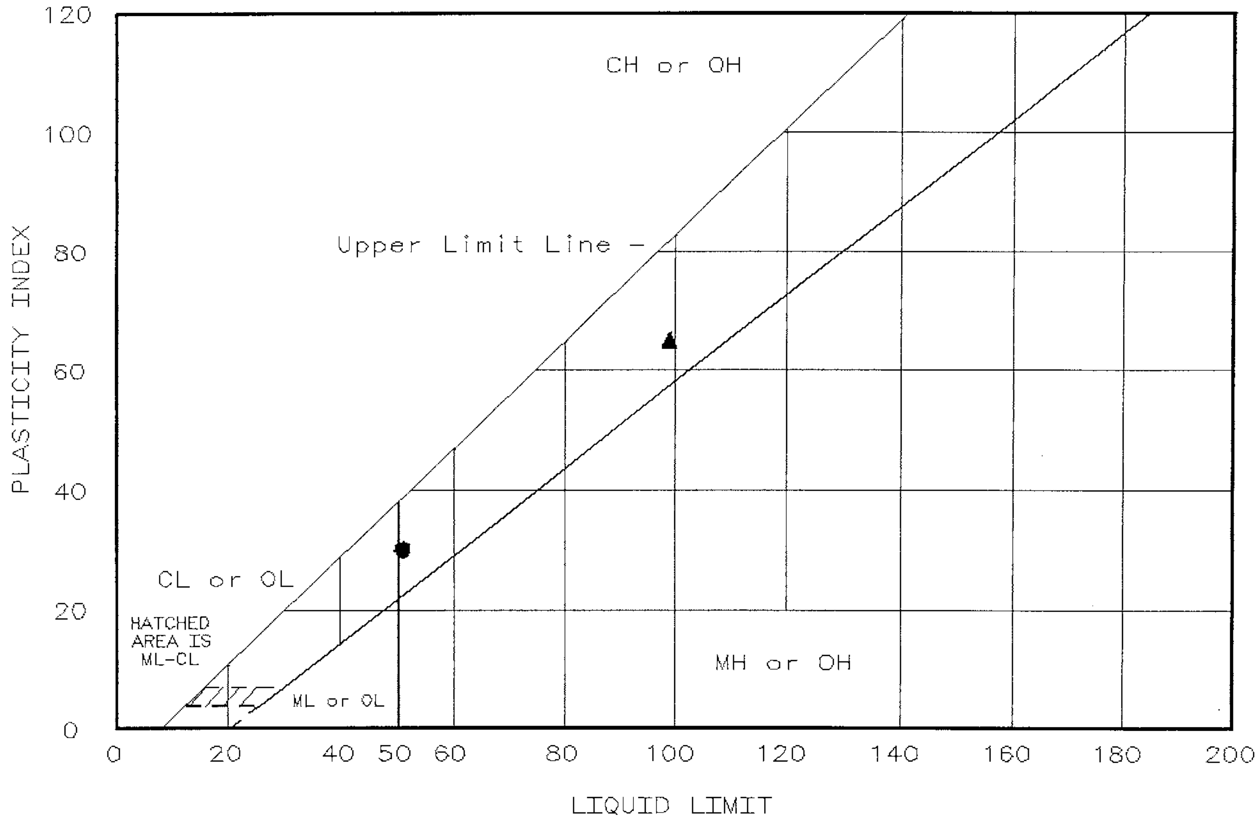
Remarks:

Soil Mechanics Lab

Project No.: SF09005
Project: Brannan St. Wharf

Date: 5-23-09 Data Sheet No. ____

LIQUID AND PLASTIC LIMITS TEST REPORT



	Location + Description	LL	PL	PI	-200	ASTM D 2487-90
●	B-1 @ 101':Dark groy	51	21	30	89.2	CH, Fat clay
▲	B-1 @ 111':Dark gray FAT CLAY(CH)portion of tube.	99	34	65		

Project No.: SF09005
Project: Brannan St. Wharf
Brannan St. and The Embarcadero
Client: Geotechnical Consultants,Inc.
Location:

Date: 5-23-09

LIQUID AND PLASTIC LIMITS TEST REPORT

Soil Mechanics Lab

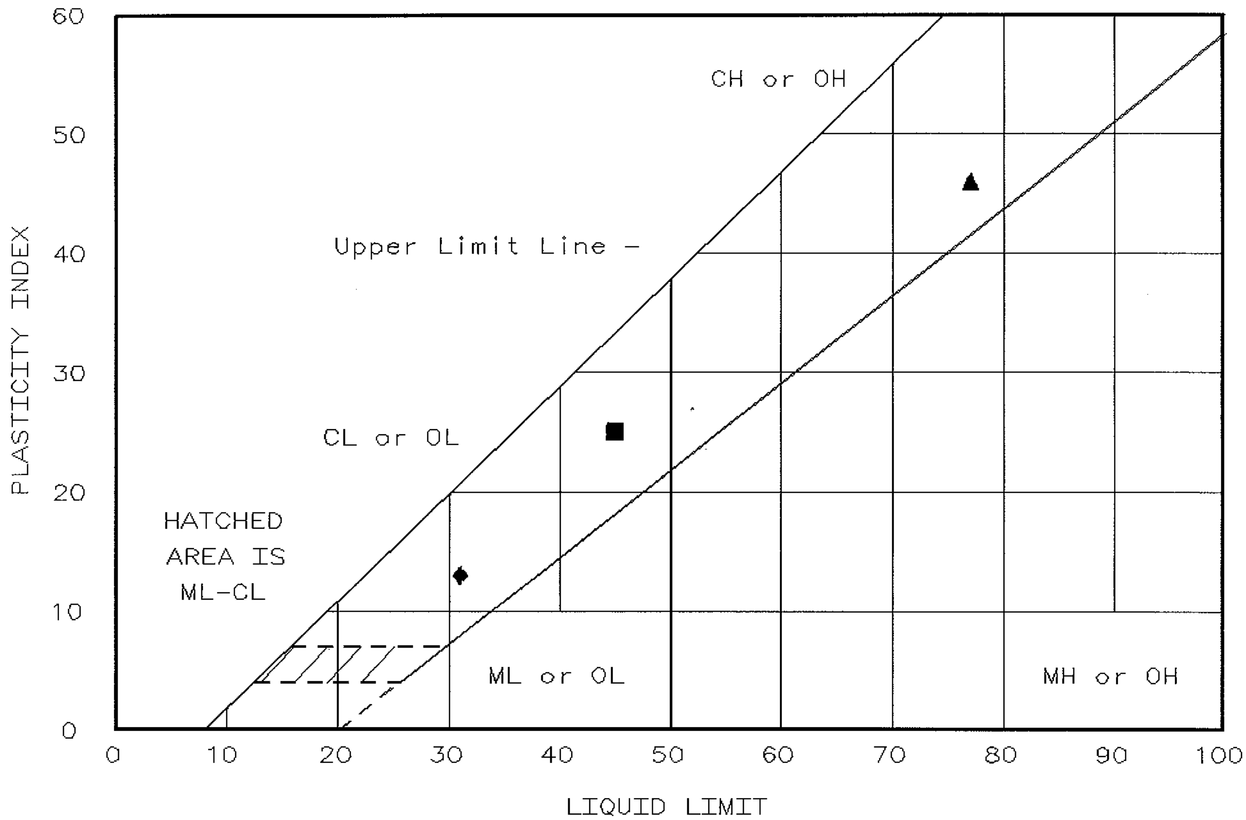
Remarks:

ASTM D 4318

Method B

Fig. No. ______

LIQUID AND PLASTIC LIMITS TEST REPORT



	Location + Description	LL	PL	PI	-200	ASTM D 2487-90
●	B-3 @ 50.5-51.5':Brown	NV	NP		19.4	SM, Silty sand
▲	B-3 @ 71':Dark olive gray FAT CLAY(CH)	77	31	46		
■	B-3 @ 76':Gray	45	20	25	64	CL, Sandy lean clay
◆	B-3 @ 86':Olive gray	31	18	13	34.4	SC, Clayey sand

NV - Non-Viscous NP - Non-Plastic

Project No.: SF09005
Project: Brannan St. Wharf
Brannan St. and The Embarcadero
Client: Geotechnical Consultants,Inc.
Location:

Date: 5-23-09

Remarks:

ASTM D 4318

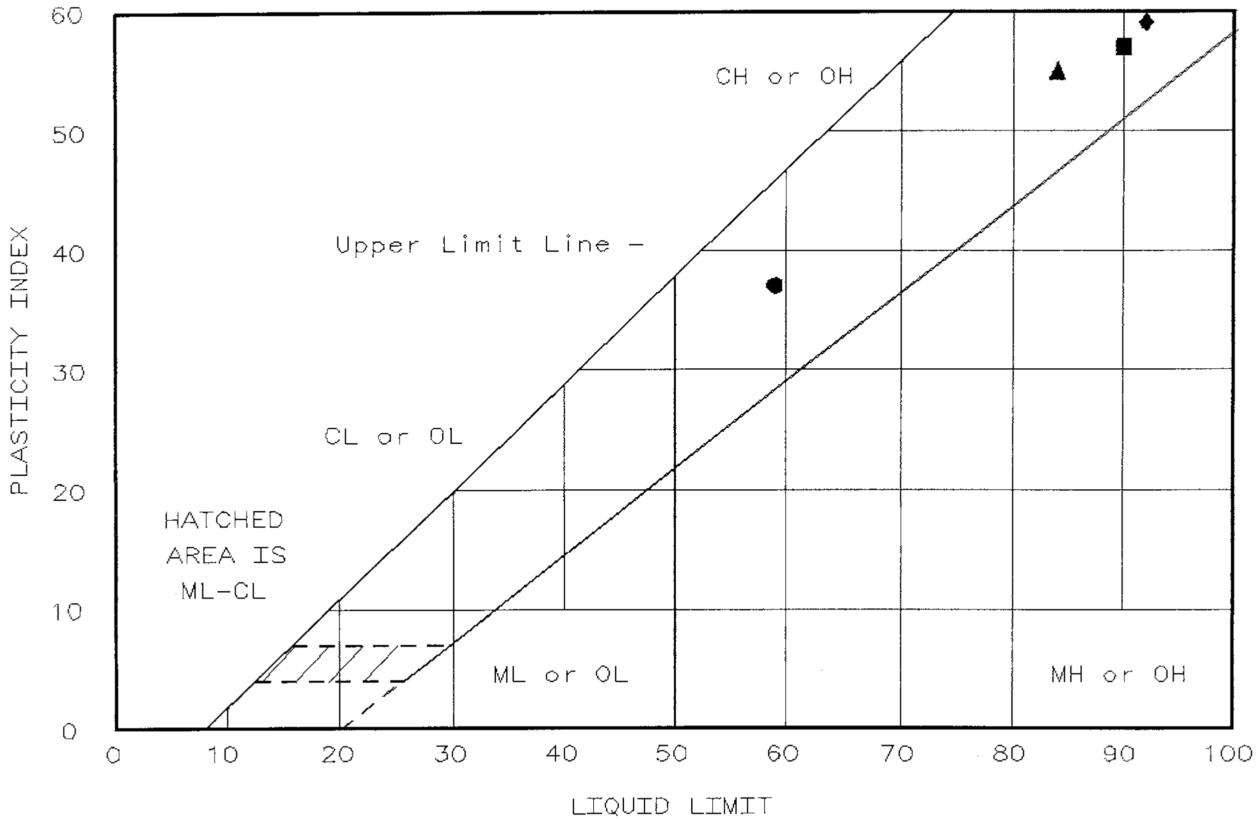
Method B

LIQUID AND PLASTIC LIMITS TEST REPORT

Soil Mechanics Lab

Fig. No. ______

LIQUID AND PLASTIC LIMITS TEST REPORT



	Location + Description	LL	PL	PI	-200	ASTM D 2487-90
●	B-3 @ 106':Dark,bluish gray FAT CLAY(CH)	59	22	37		
▲	B-3 @ 111':Dark gray FAT CLAY(CH)	84	29	55		
■	B-3 @ 115':Grayish black FAT CLAY(CH)	90	33	57		
◆	B-3 @ 121':Grayish black FAT CLAY(CH)	92	33	59		

Project No.: SF09005
Project: Brannan St. Wharf
Brannan St. and The Embarcadero
Client: Geotechnical Consultants,Inc.
Location:

Date: 5-23-09

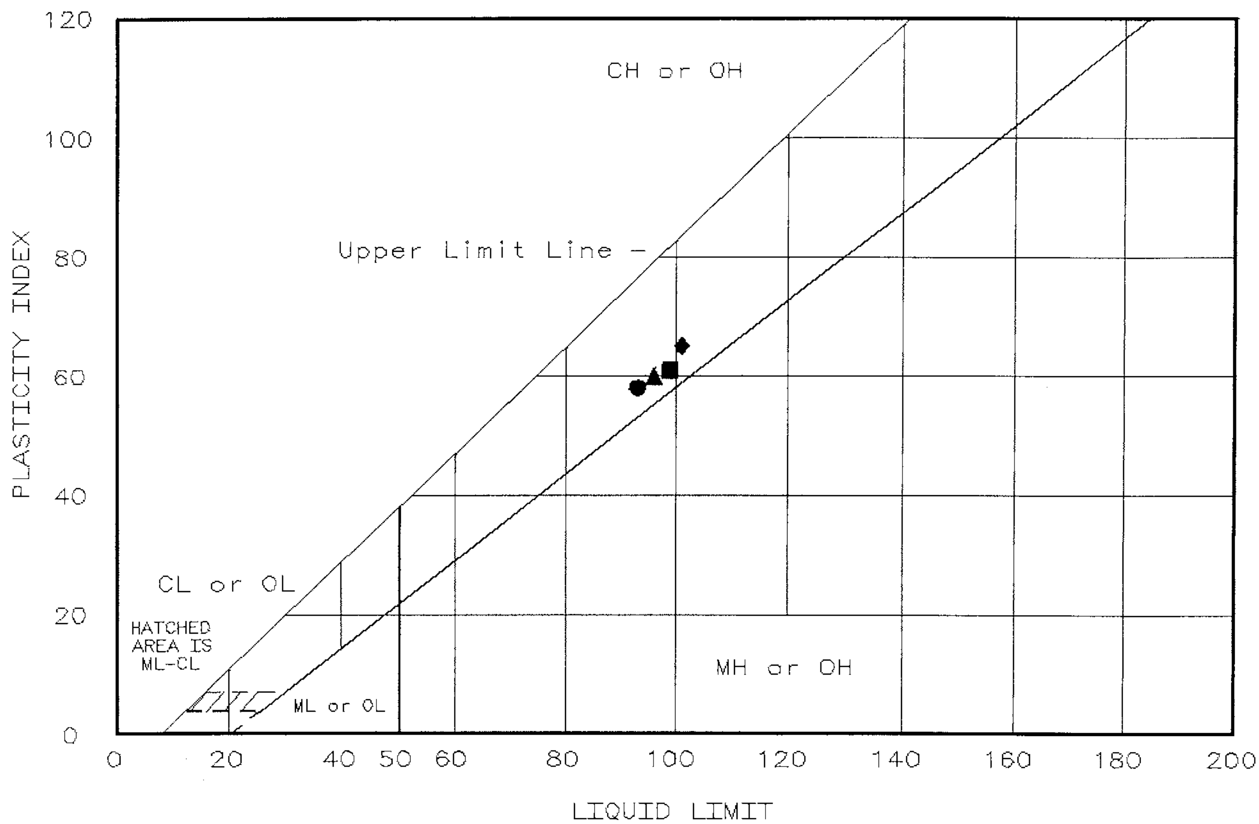
LIQUID AND PLASTIC LIMITS TEST REPORT

Soil Mechanics Lab

Remarks:
ASTM D 4318
Method B

Fig. No. ______

LIQUID AND PLASTIC LIMITS TEST REPORT



	Location + Description	LL	PL	PI	-200	ASTM D 2487-90
●	B-4 @ 26':Grayish black FAT CLAY(CH)	93	35	58		
▲	B-4 @ 31':Grayish black FAT CLAY(CH)	96	36	60		
■	B-4 @ 36':Grayish black FAT CLAY(CH)	99	38	61		
◆	B-4 @ 45':Very dk. gray to black FAT CLAY(CH)	101	36	65		

Project No.: SF09005
Project: Brannan St. Wharf
Brannan St. and The Embarcadero
Client: Geotechnical Consultants,Inc.
Location:

Date: 5-23-09

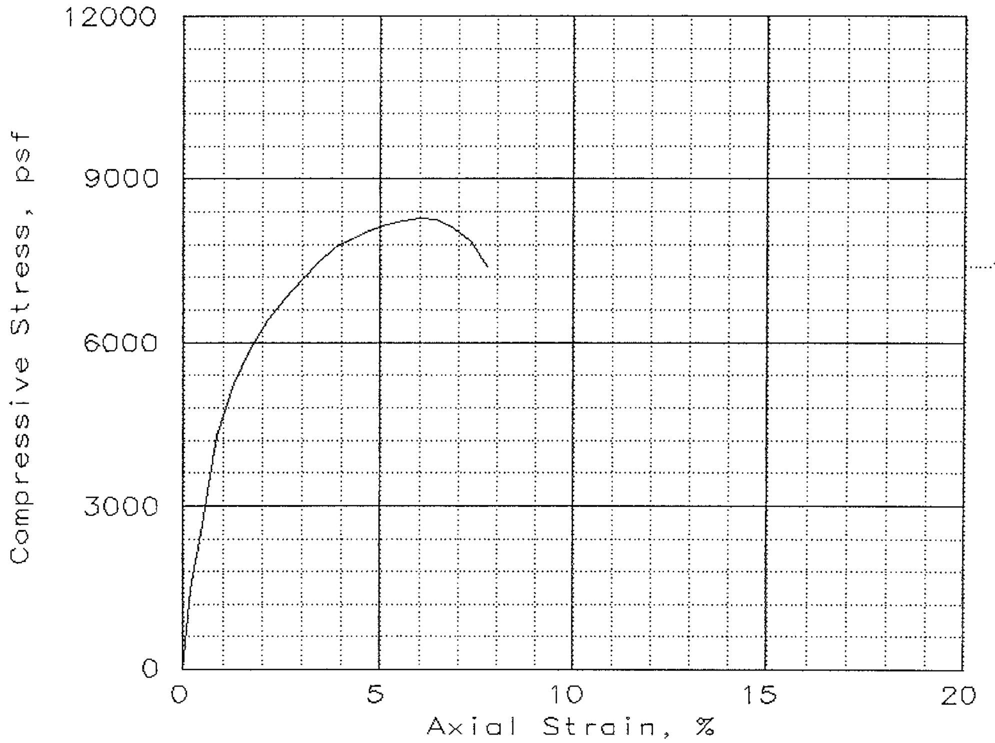
Remarks:
ASTM D 4318
Method B

LIQUID AND PLASTIC LIMITS TEST REPORT

Soil Mechanics Lab

Fig. No. ________

UNCONFINED COMPRESSION TEST



Specimen No.:	1			
Unconfined strength, psf	8285			
Undrained shear strength, psf	4142			
Failure strain, %	6.0			
Strain rate, in/min	0.0750			
Water content, % (cuttings before test)	15.4			
Wet density, pcf	139.9			
Dry density, pcf	121.2			
Saturation, %	106.4			
Void ratio	0.3906			
Specimen diameter, in	2.42			
Specimen height, in	4.64			
Height/diameter ratio	1.92			

1) Description: Hard,dark bluish gray sandy CLAY(CL)

2) Description:

3) Description:

4) Description:

GS= 2.7 Type: MC

Project No.: SF09005

Date: 5-22-09

Remarks:

Client: Geotechnical Consultants,Inc.

Project: Brannan St. Wharf
Brannan St. and The Embarcadero

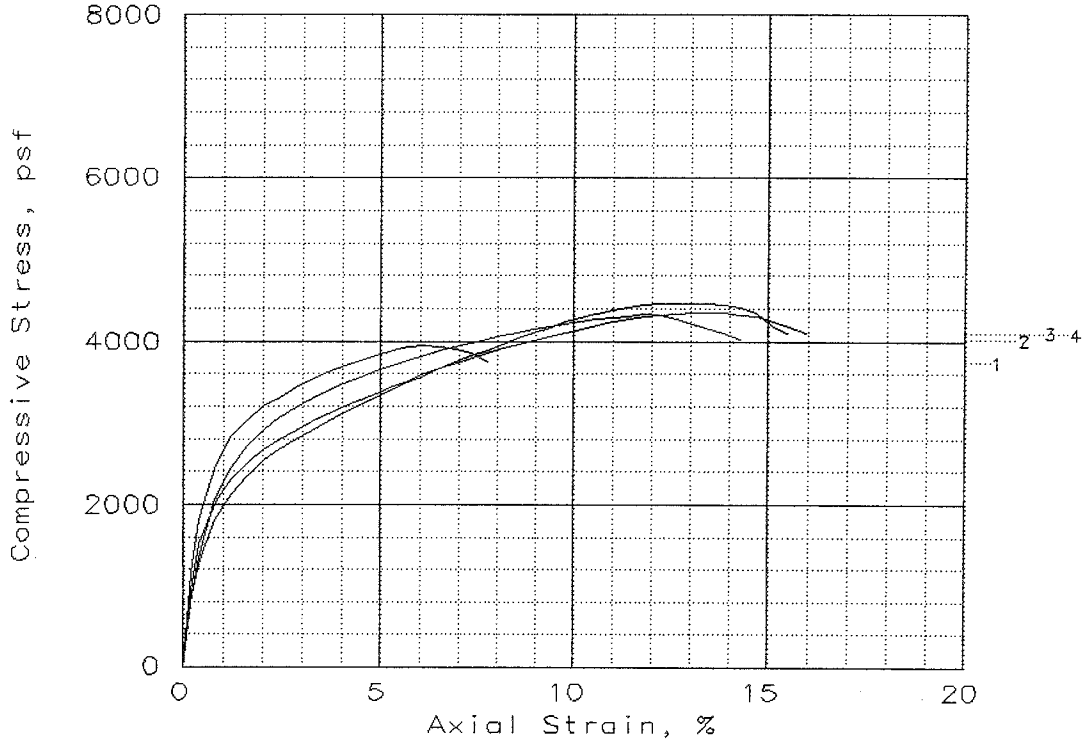
Location: B-1 121-121.5'

UNCONFINED COMPRESSION TEST

Soil Mechanics Lab

Fig. No.: ____

UNCONFINED COMPRESSION TEST



Specimen No.:	1	2	3	4
Unconfined strength, psf	3942	4337	4358	4469
Undrained shear strength, psf	1971	2168	2179	2235
Failure strain, %	6.1	11.8	13.5	12.7
Strain rate, in/min	0.0750	0.0750	0.0750	0.0750
Water content, % (cuttings before test)	45.4	35.0	44.8	49.3
Wet density, pcf	114.6	120.7	113.7	110.6
Dry density, pcf	78.9	89.5	78.6	74.1
Saturation, %	107.7	106.8	105.5	104.4
Void ratio	1.1377	0.8842	1.1458	1.2754
Specimen diameter, in	2.42	2.42	2.42	2.42
Specimen height, in	4.89	4.90	4.89	4.90
Height/diameter ratio	2.02	2.03	2.02	2.03

1) Description: Sa.1/71-71.5':Very stiff,dk. gray FAT CLAY(CH)

2) Description: Sa.2/106-106.5':-----same as above-----(CH)

3) Description: Sa.3/111-111.5':-----same as above-----(CH)

4) Description: Sa.4:121-121.5':-----same as above-----(CH)

GS= 2.7 | Type: MC

Project No.: SF09005

Date: 5-22-09

Remarks:

Client: Geotechnical Consultants,Inc.

Project: Brannan St. Wharf
Brannan St. and The Embarcadero

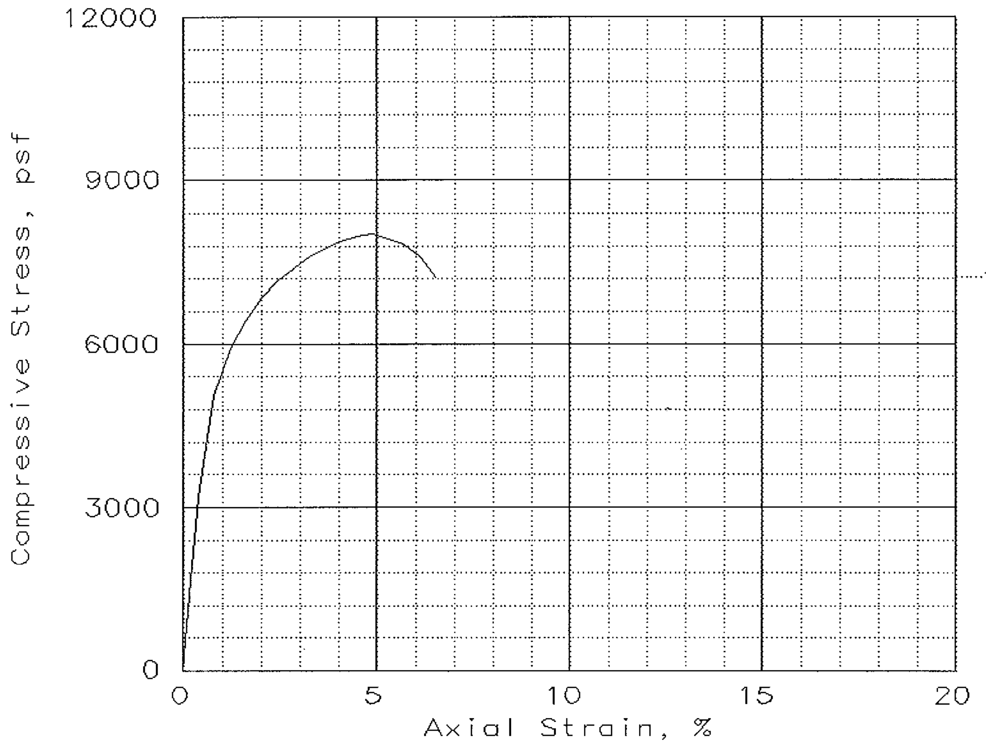
Location: B-3

UNCONFINED COMPRESSION TEST

Soil Mechanics Lab

Fig. No.: ______

UNCONFINED COMPRESSION TEST



Specimen No.:	1			
Unconfined strength, psf	8015			
Undrained shear strength, psf	4007			
Failure strain, %	4.9			
Strain rate, in/min	0.0750			
Water content, % (cuttings before test)	23.6			
Wet density, pcf	131.0			
Dry density, pcf	106.0			
Saturation, %	108.0			
Void ratio	0.5906			
Specimen diameter, in	2.42			
Specimen height, in	4.90			
Height/diameter ratio	2.03			

1) Description: Hard,dark olive gray CLAY(CL)

2) Description:

3) Description:

4) Description:

GS= 2.7 | Type: MC

Project No.: SF09005

Date: 5-22-09

Remarks:

Client: Geotechnical Consultants,Inc.

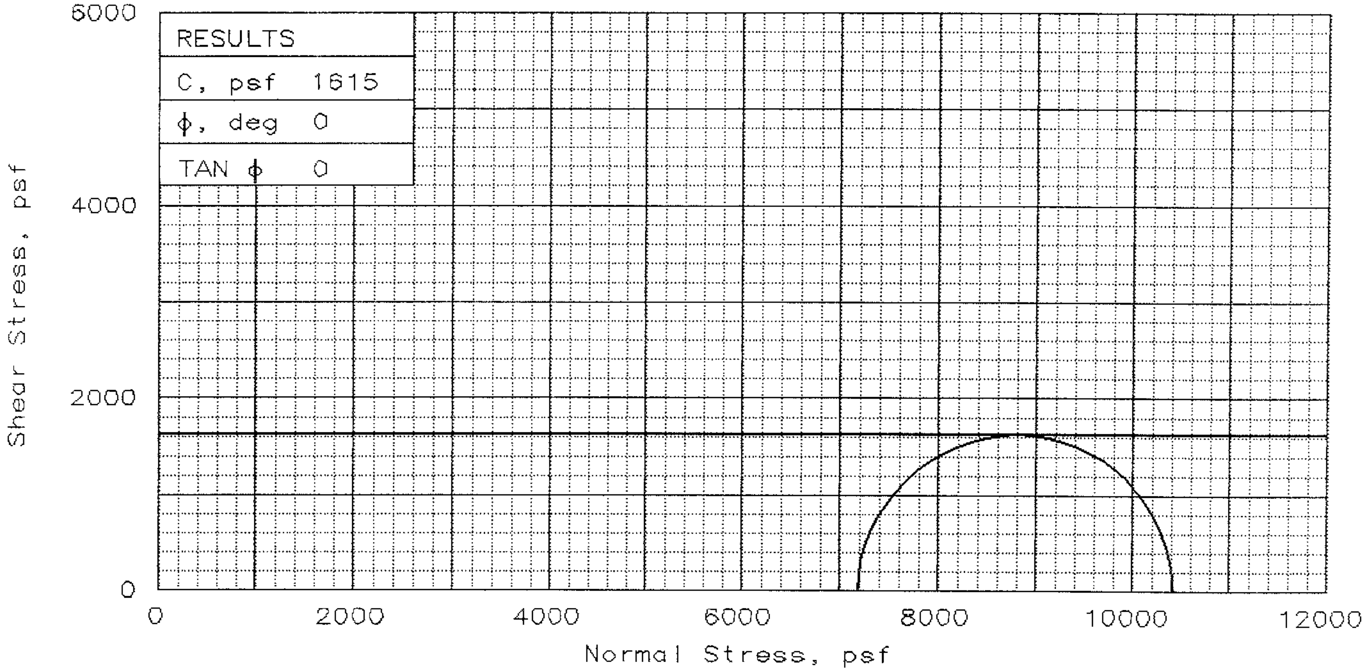
Project: Brannan St. Wharf
Brannan St. and The Embarcadero

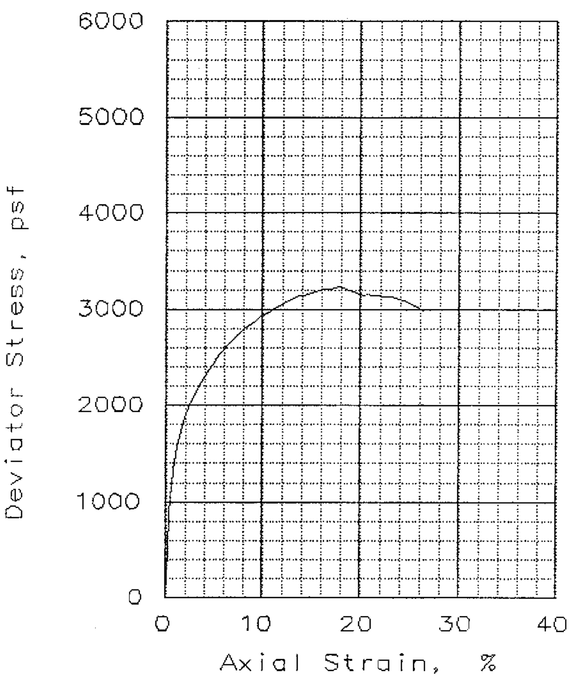
Location: B-4 71.71.5'

UNCONFINED COMPRESSION TEST

Soil Mechanics Lab

Fig. No.: ______





Specimen No.:		1
INITIAL	WATER CONTENT, %	31.8
	DRY DENSITY, pcf	92.3
	SATURATION, %	103.9
	VOID RATIO	0.827
	DIAMETER, in	2.42
	HEIGHT, in	4.92
AT TEST	WATER CONTENT, %	31.8
	DRY DENSITY, pcf	92.3
	SATURATION, %	103.9
	VOID RATIO	0.827
	DIAMETER, in	2.42
	HEIGHT, in	4.92
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		7200
FAIL. STRESS, psf		3229
STRAIN, %		17.9
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		10429
σ_3 FAILURE, psf		7200

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: MC

DESCRIPTION: Stiff, dark gray FAT CLAY(CH)

SPECIFIC GRAVITY= 2.7

REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants, Inc.

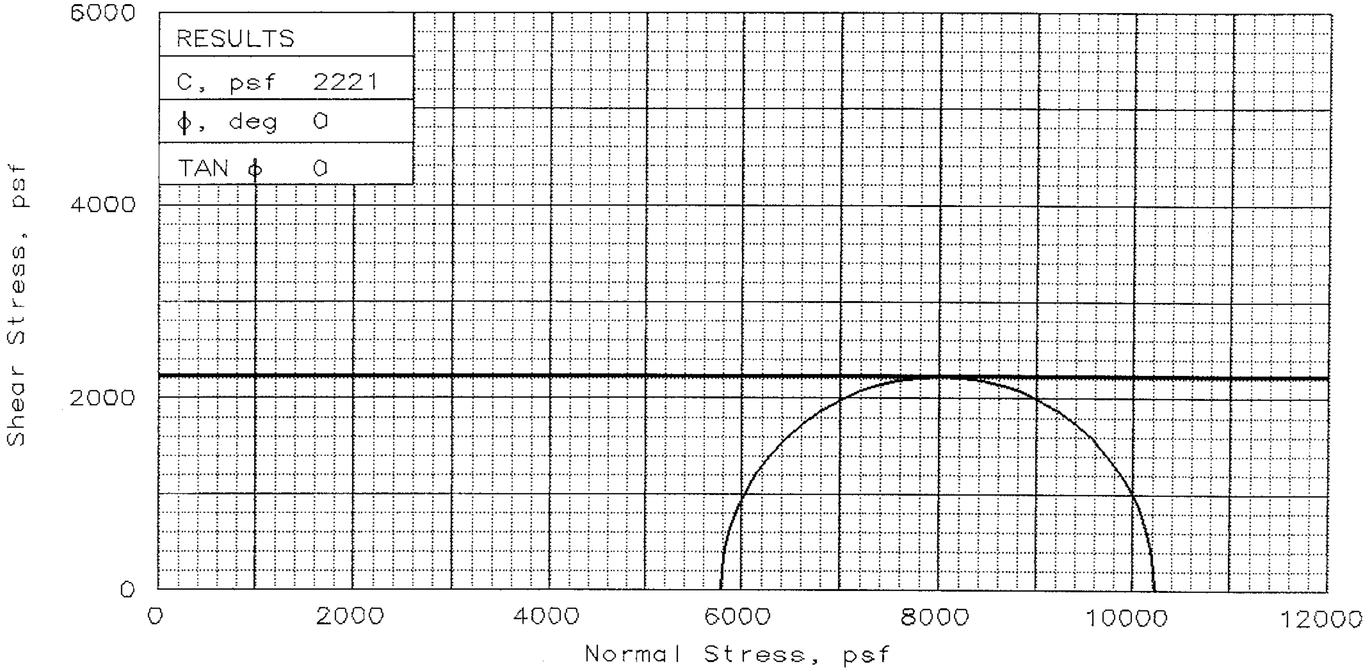
PROJECT: Brannan St. Wharf
Brannan St. and The Embarcadero

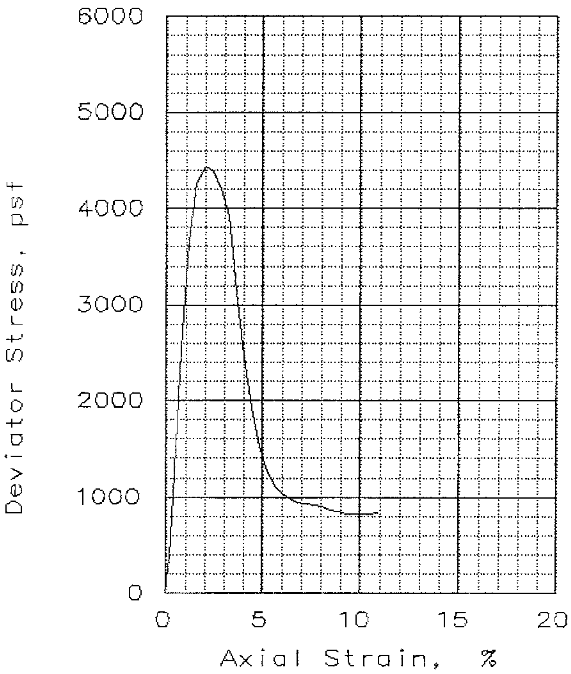
SAMPLE LOCATION: B-1 101-101.5'

PROJ. NO.: SF09005 DATE: 5-22-09

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





Specimen No.:		1
INITIAL	WATER CONTENT, %	51.2
	DRY DENSITY, pcf	72.6
	SATURATION, %	104.6
	VOID RATIO	1.322
	DIAMETER, in	2.88
	HEIGHT, in	4.94
AT TEST	WATER CONTENT, %	51.2
	DRY DENSITY, pcf	72.6
	SATURATION, %	104.6
	VOID RATIO	1.322
	DIAMETER, in	2.88
	HEIGHT, in	4.94
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		5800
FAIL. STRESS, psf		4442
STRAIN, %		2.0
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		10242
σ_3 FAILURE, psf		5800

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Very stiff, brittle dark gray FAT CLAY(CH)

SPECIFIC GRAVITY= 2.7

REMARKS: Slickenside along failure plane.

Fig. No.: ______

CLIENT: Geotechnical Consultants, Inc.

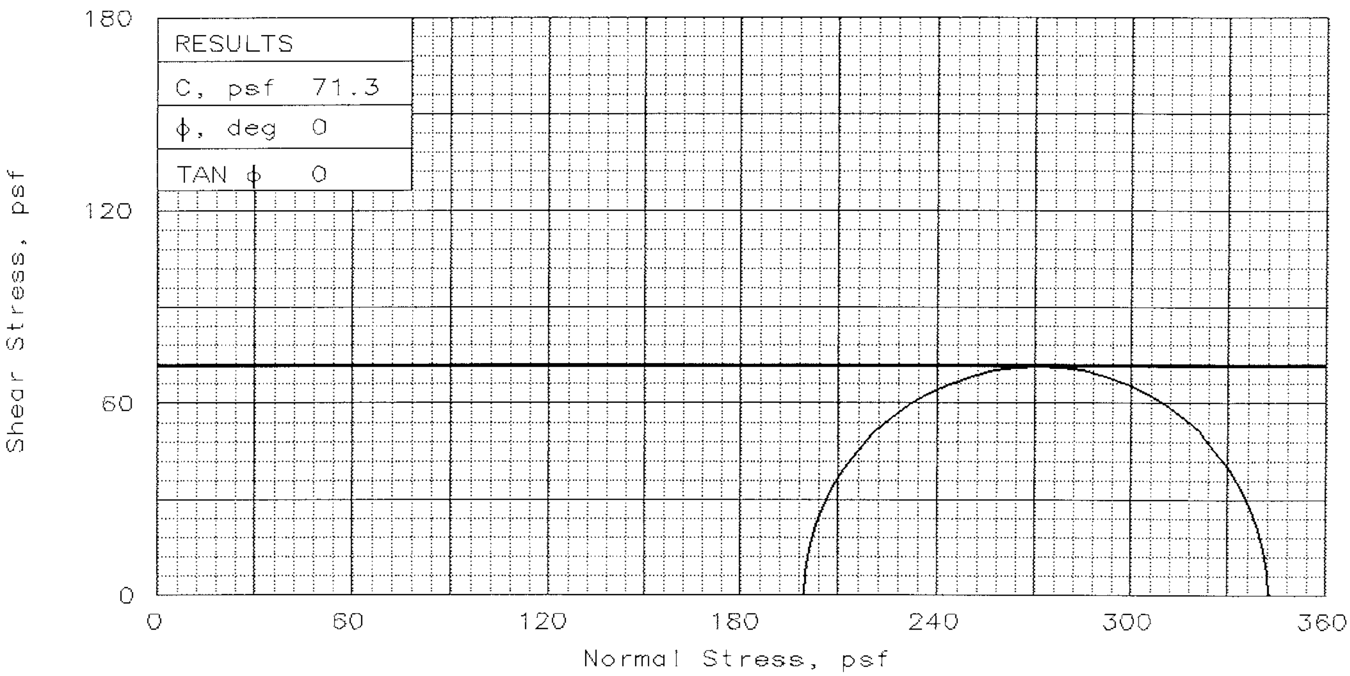
PROJECT: Brannan St. Wharf
Brannan St. and The Embarcadero

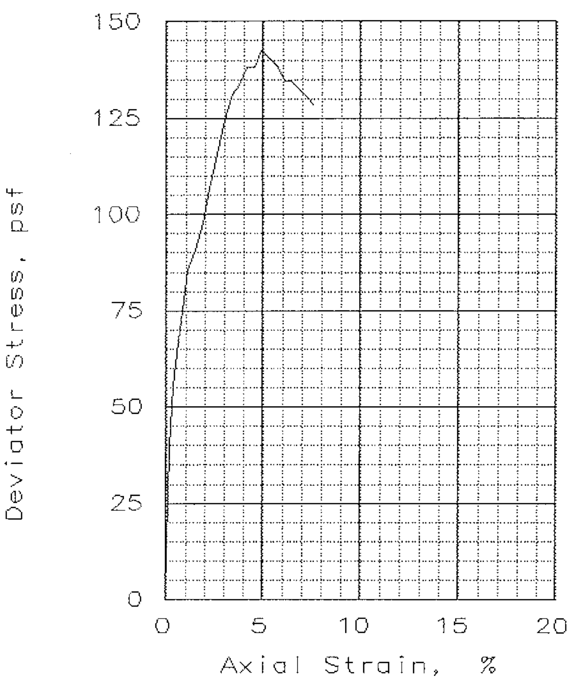
SAMPLE LOCATION: B-3 115-118'
Test @ 117.5-118'

PROJ. NO.: SF09005 DATE: 5-22-09

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





Specimen No.:		1
INITIAL	WATER CONTENT, %	107.0
	DRY DENSITY, pcf	42.4
	SATURATION, %	97.0
	VOID RATIO	2.978
	DIAMETER, in	2.88
	HEIGHT, in	5.25
AT TEST	WATER CONTENT, %	107.0
	DRY DENSITY, pcf	42.4
	SATURATION, %	97.0
	VOID RATIO	2.978
	DIAMETER, in	2.88
	HEIGHT, in	5.25
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		200
FAIL. STRESS, psf		143
STRAIN, %		5.0
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		343
σ_3 FAILURE, psf		200

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Very soft grayish black FAT CLAY(CH)

SPECIFIC GRAVITY= 2.7

REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants,Inc.

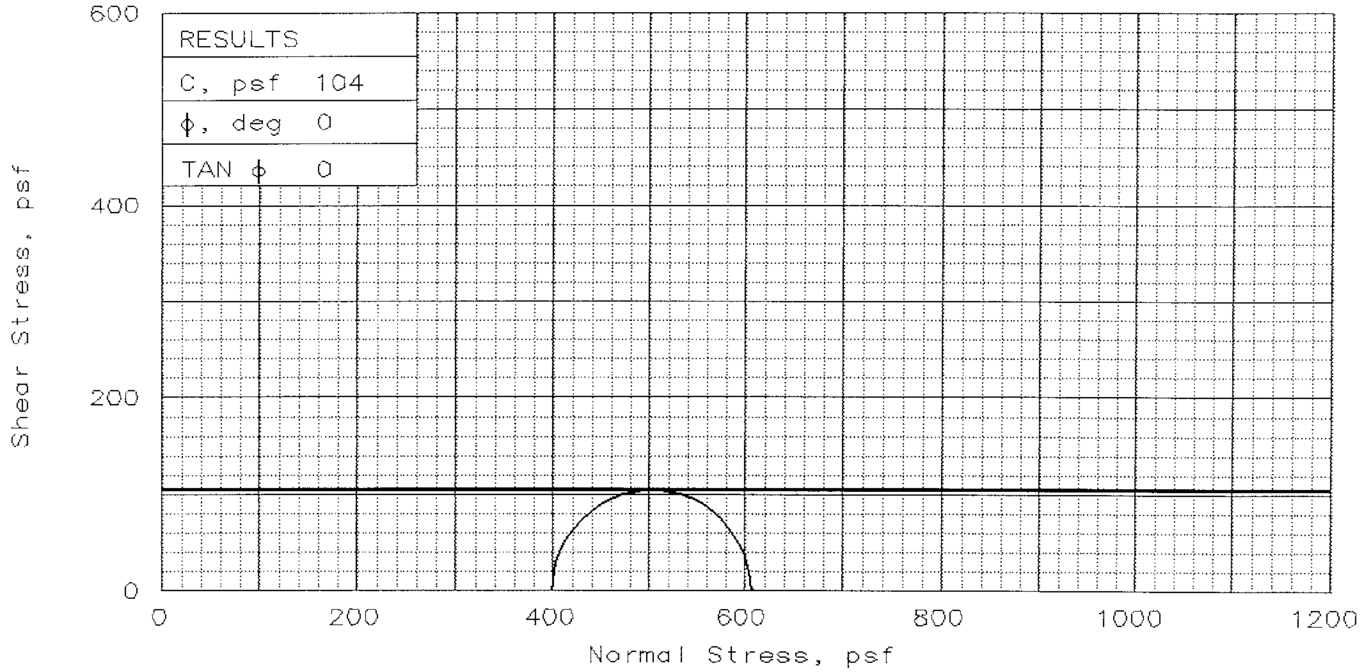
PROJECT: Brannan St. Wharf.
Brannan St. and The Embarcadero

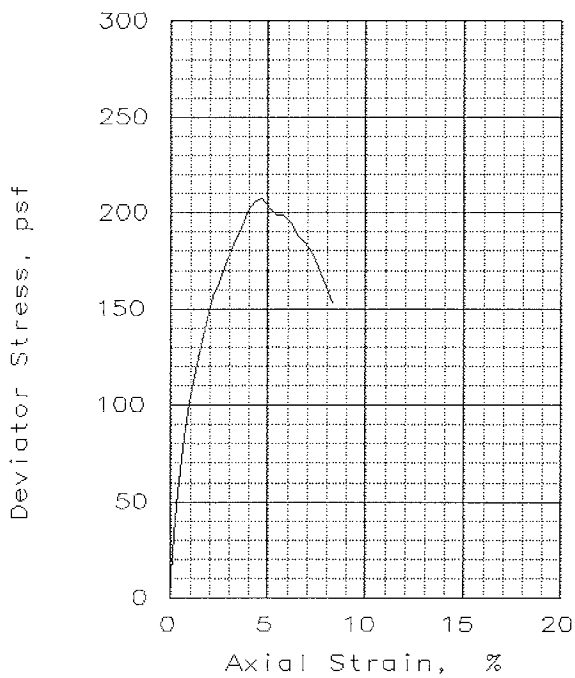
SAMPLE LOCATION: B-4 26-29'
Test @ 28.5-29'

PROJ. NO.: SF09005 DATE: 5-23-09

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





Specimen No.:		1
INITIAL	WATER CONTENT, %	88.4
	DRY DENSITY, pcf	47.4
	SATURATION, %	93.5
	VOID RATIO	2.553
	DIAMETER, in	2.88
	HEIGHT, in	5.51
AT TEST	WATER CONTENT, %	88.4
	DRY DENSITY, pcf	47.4
	SATURATION, %	93.5
	VOID RATIO	2.553
	DIAMETER, in	2.88
	HEIGHT, in	5.51
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		400
FAIL. STRESS, psf		207
STRAIN, %		4.7
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		607
σ_3 FAILURE, psf		400

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Very soft, grayish black FAT CLAY(CH)

SPECIFIC GRAVITY= 2.7

REMARKS:

CLIENT: Geotechnical Consultants, Inc.

PROJECT: Brannan St. Wharf
Brannan St. and The Embarcadero

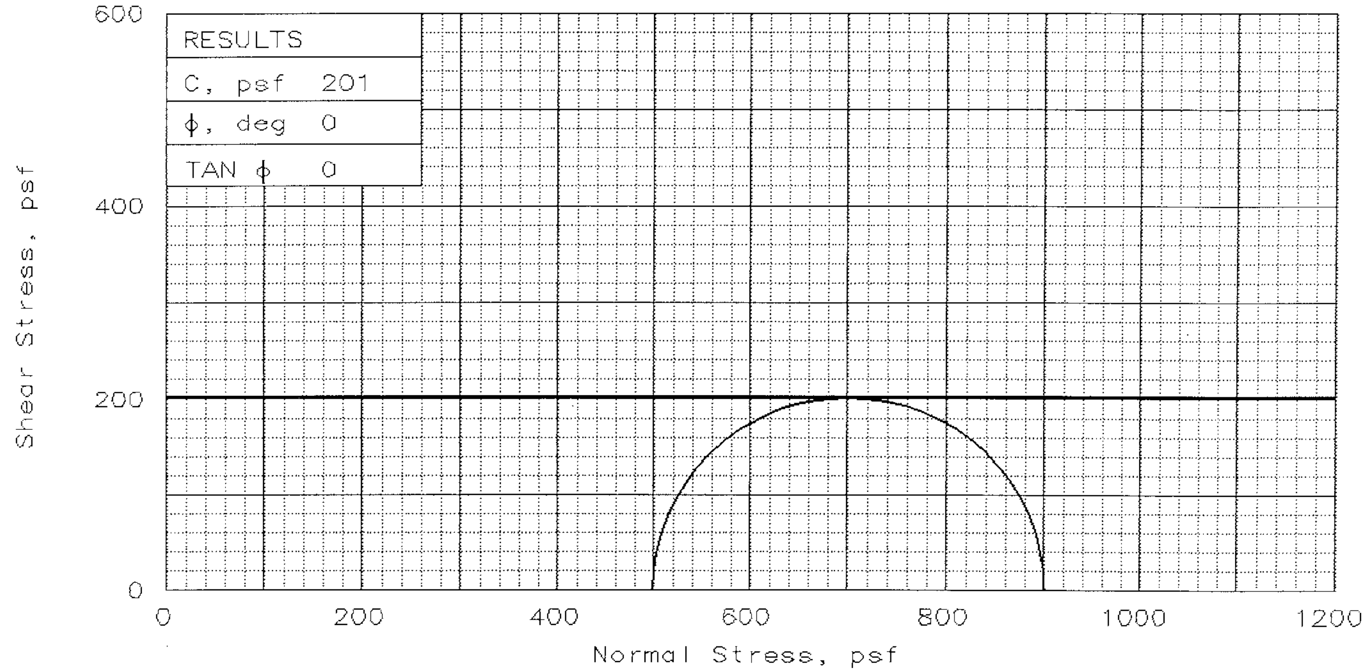
SAMPLE LOCATION: B-4 31-34'
Teast @ 33.5-34'

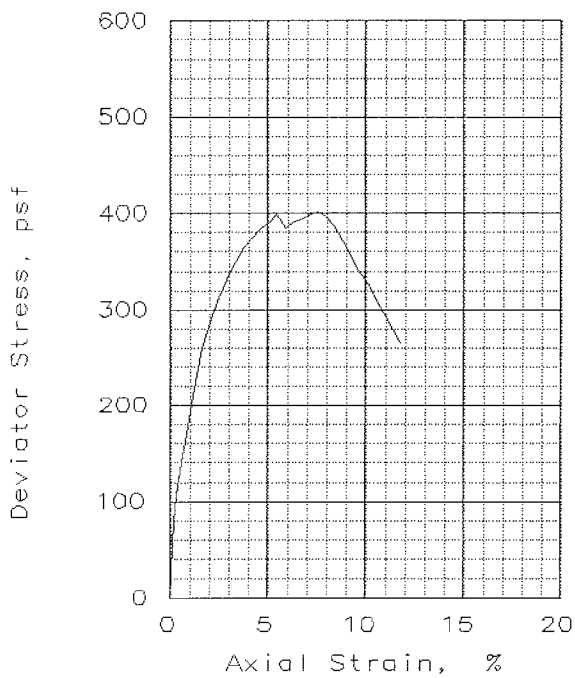
PROJ. NO.: SF09005 DATE: 5-23-09

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab

Fig. No.: ______





Specimen No.:		1
INITIAL	WATER CONTENT, %	87.4
	DRY DENSITY, pcf	47.9
	SATURATION, %	93.7
	VOID RATIO	2.518
	DIAMETER, in	2.88
	HEIGHT, in	4.75
AT TEST	WATER CONTENT, %	87.4
	DRY DENSITY, pcf	47.9
	SATURATION, %	93.7
	VOID RATIO	2.518
	DIAMETER, in	2.88
	HEIGHT, in	4.75
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		500
FAIL. STRESS, psf		402
STRAIN, %		7.6
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		902
σ_3 FAILURE, psf		500

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Very soft,grayish black FAT CLAY(CH)

SPECIFIC GRAVITY= 2.7

REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants,Inc.

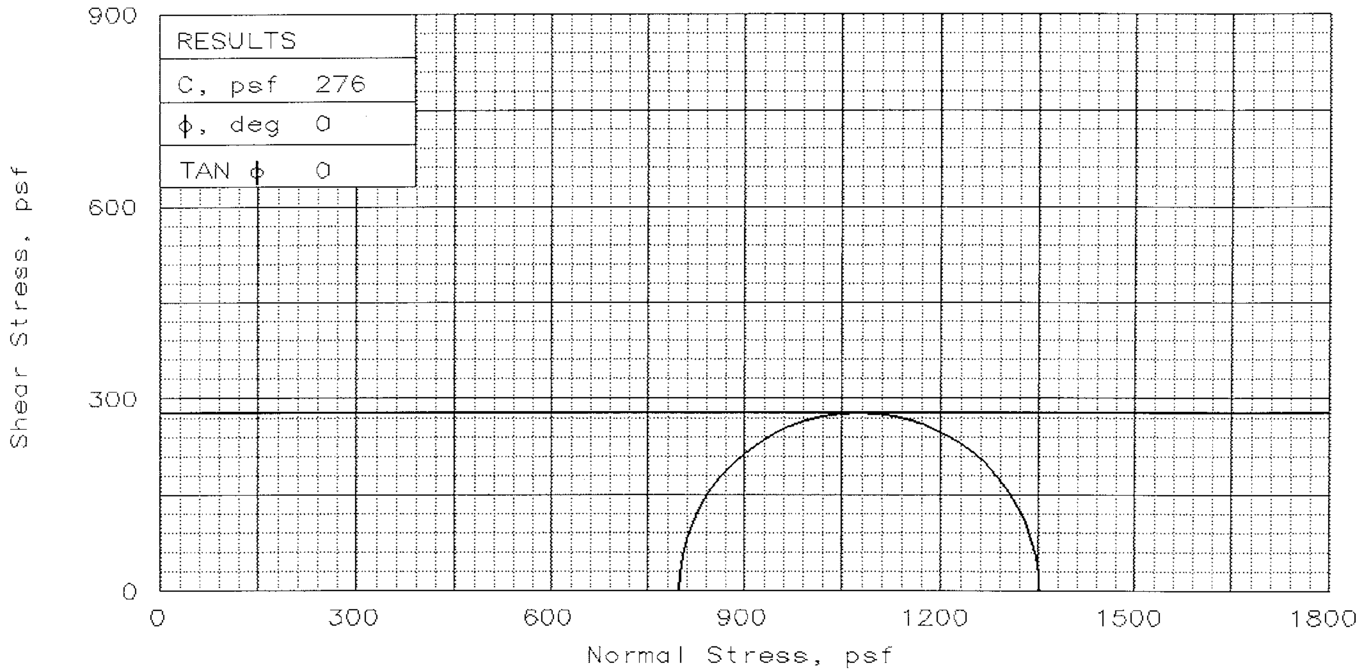
PROJECT: Brannan St. Wharf
Brannan St. and The Embarcadero

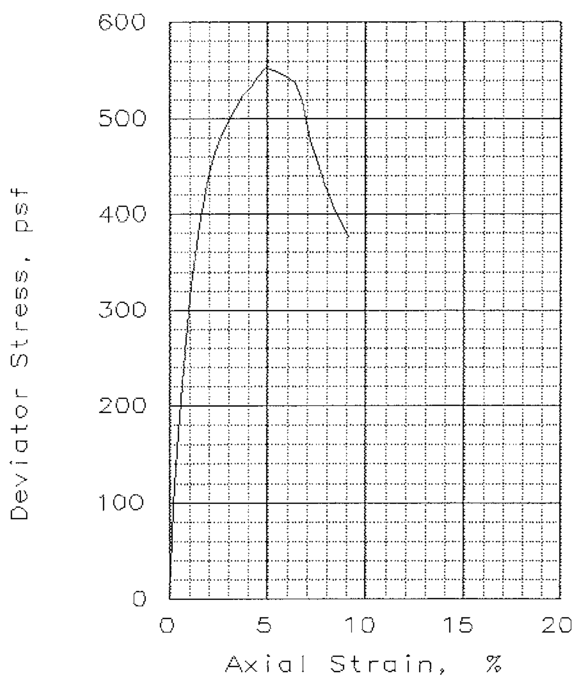
SAMPLE LOCATION: B-4 36-39'
Test @ 38-38.5'

PROJ. NO.: SF09+005 DATE: 5-23-09

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab





Specimen No.:		1
INITIAL	WATER CONTENT, %	77.6
	DRY DENSITY, pcf	52.4
	SATURATION, %	94.5
	VOID RATIO	2.219
	DIAMETER, in	2.88
	HEIGHT, in	5.28
AT TEST	WATER CONTENT, %	77.6
	DRY DENSITY, pcf	52.4
	SATURATION, %	94.5
	VOID RATIO	2.219
	DIAMETER, in	2.88
	HEIGHT, in	5.28
Strain rate, in/min		0.0750
BACK PRESSURE, psf		0
CELL PRESSURE, psf		800
FAIL. STRESS, psf		553
STRAIN, %		4.9
ULT. STRESS, psf		
STRAIN, %		
σ_1 FAILURE, psf		1353
σ_3 FAILURE, psf		800

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Shelby

DESCRIPTION: Very dark gray to black,FAT CLAY(CH)

SPECIFIC GRAVITY= 2.7

REMARKS:

Fig. No.: ______

CLIENT: Geotechnical Consultants,Inc.

PROJECT: Brannan St. Wharf
Brannan St. and The Embarcadero

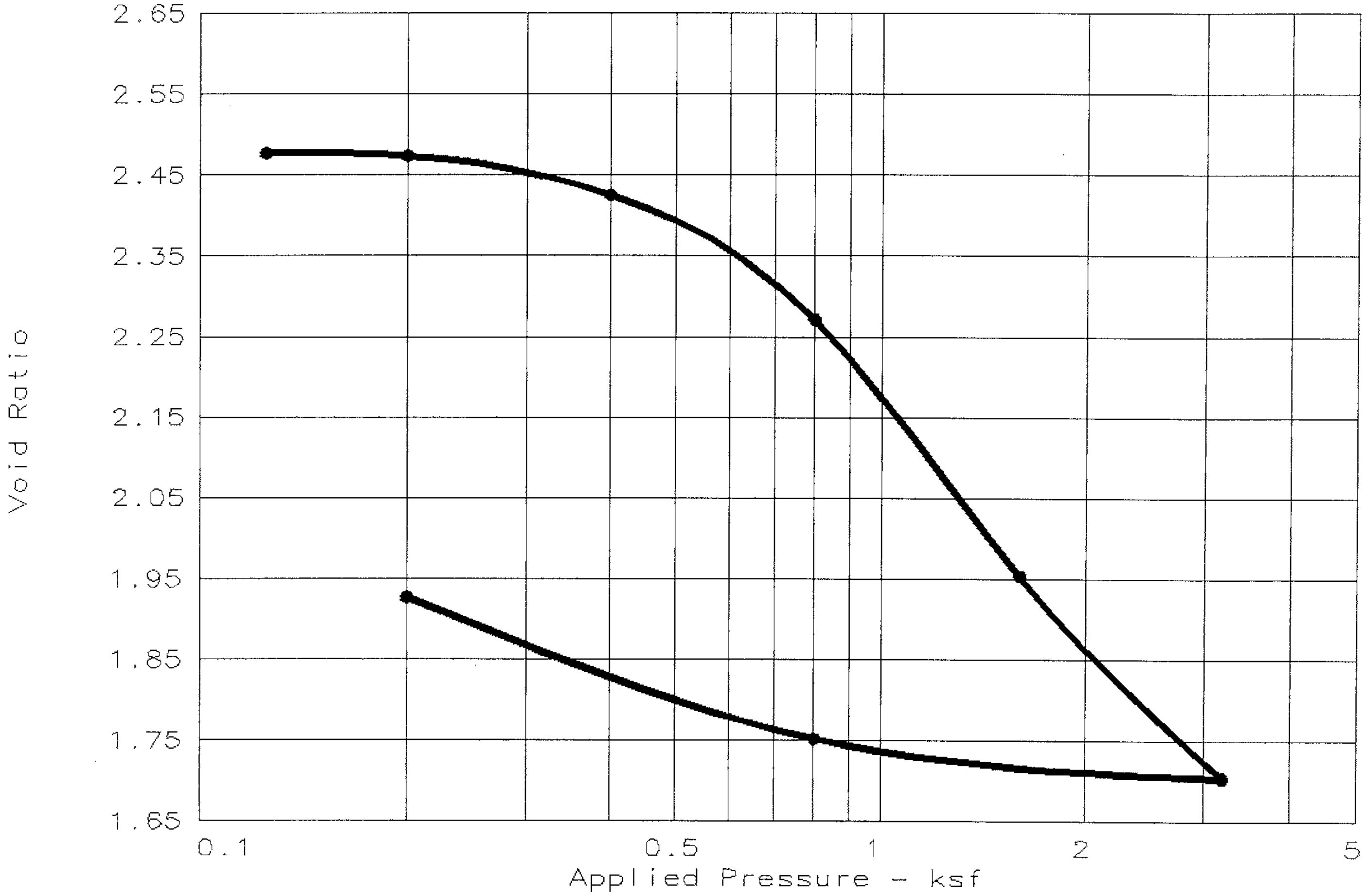
SAMPLE LOCATION: B-4 45-48'
Test @ 47-47.5'

PROJ. NO.: SF09005 DATE: 5-23-09

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab

CONSOLIDATION TEST REPORT ASTM D 2435



Coeffs. of Consolidation (sq. in./min.) & Secondary Consolidation											
No.	Load	Cv	Cα	No.	Load	Cv	Cα	No.	Load	Cv	Cα
4	0.80	0.003	0.026								
5	1.60	0.002	0.043								

Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Precons. (ksf)	C_c	e_o
96.3 %	88.3 %	48.5	99	61	2.700	0.72	1.11	2.4763

TEST RESULTS	MATERIAL DESCRIPTION
Compression Index = 1.11	Very soft, grayish black FAT CLAY w/organics. Class: CH

Project No.: SF09005
Project: Brannan Street Wharf
Location: B-4 36-39'
Test @ 38.5'
Date: 5-20-09

Remarks:
Cc btwn.0.9 & 1.8 ksf.

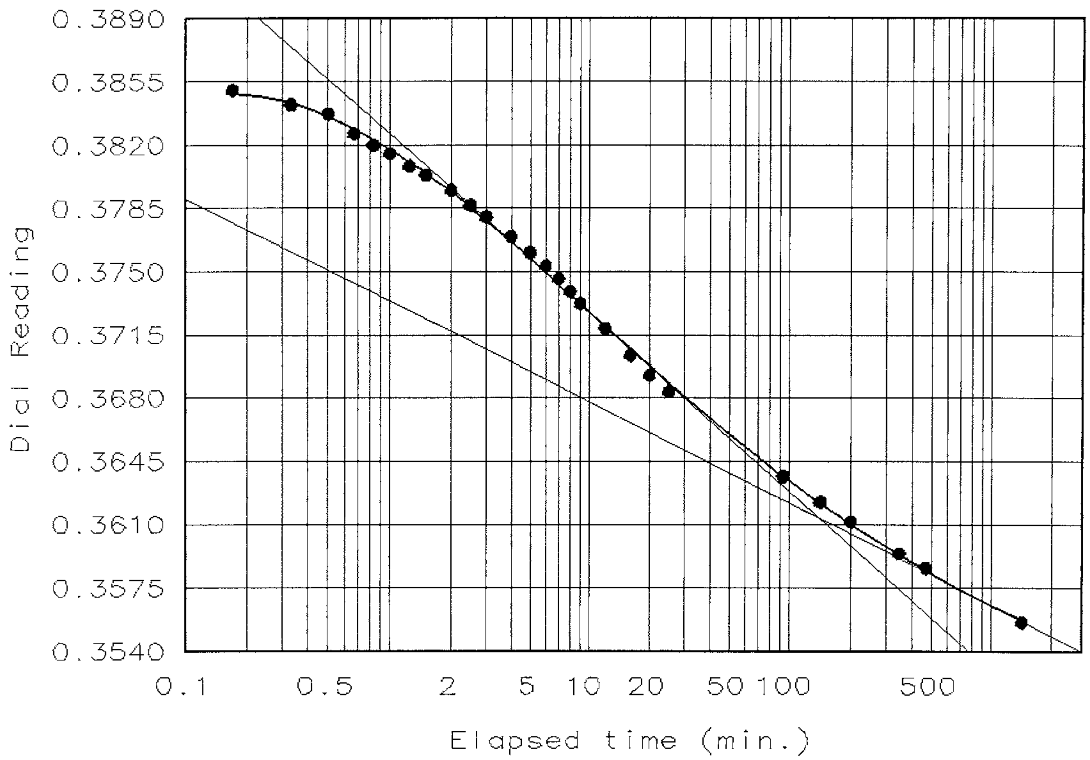
CONSOLIDATION TEST REPORT ASTM D 2435

Soil Mechanics Lab

Fig. No. ______

Dial Reading vs. Time

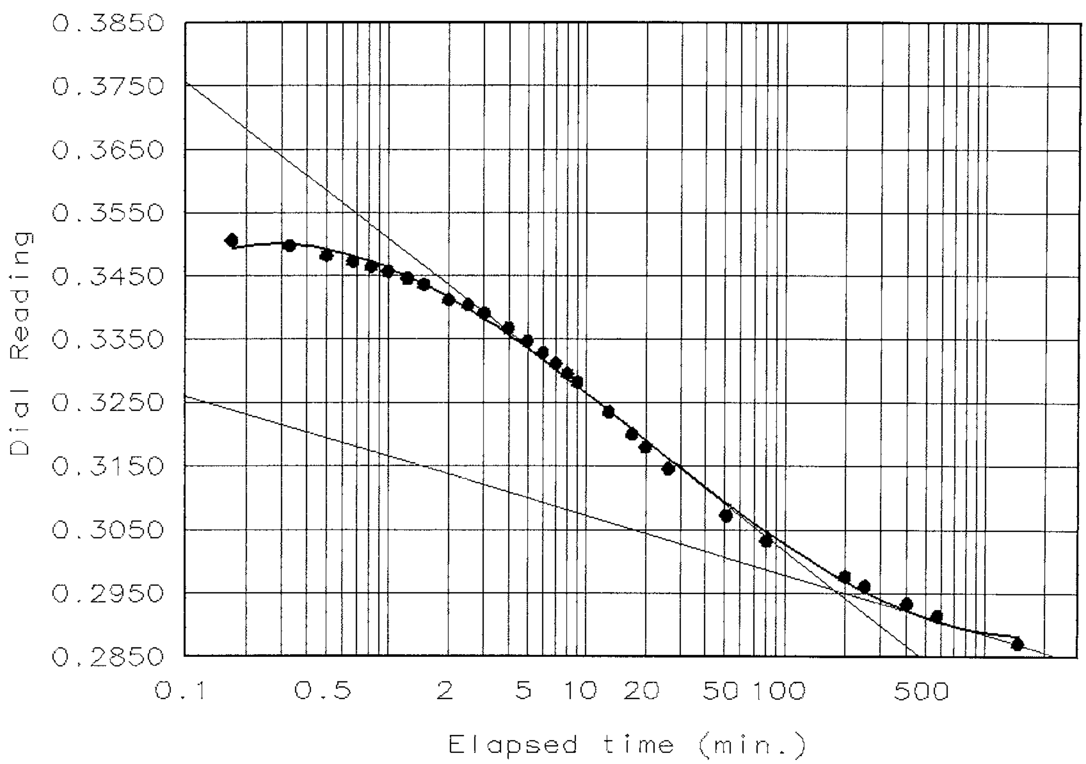
Project No.: SF09005
Project: Brannan Street Wharf
Location: B-4 36-39'
Test @ 38.5'
Date: 5-20-09



Load No.= 4
Load= 0.80 ksf
D_0 = 0.3868
D_{50} = 0.3740
D_{100}= 0.3613
T_{50}= 7.37 min.

C_v @ T_{50} = .003 in.^{2}/min.

C_α= 0.026



Load No.= 5
Load= 1.60 ksf
D_0 = 0.3506
D_{50} = 0.3231
D_{100}= 0.2956
T_{50}= 13.53 min.

C_v @ T_{50} = .002 in.^{2}/min.

C_α= 0.043

APPENDIX E
ACCELERATION RESPONSE SPECTRA

FIGURE E-1
2007 CBC DESIGN RESPONSE SPECTRUM
HORIZONTAL ACCELERATION RESPONSE SPECTRA
BRANNAN STREET WHARF

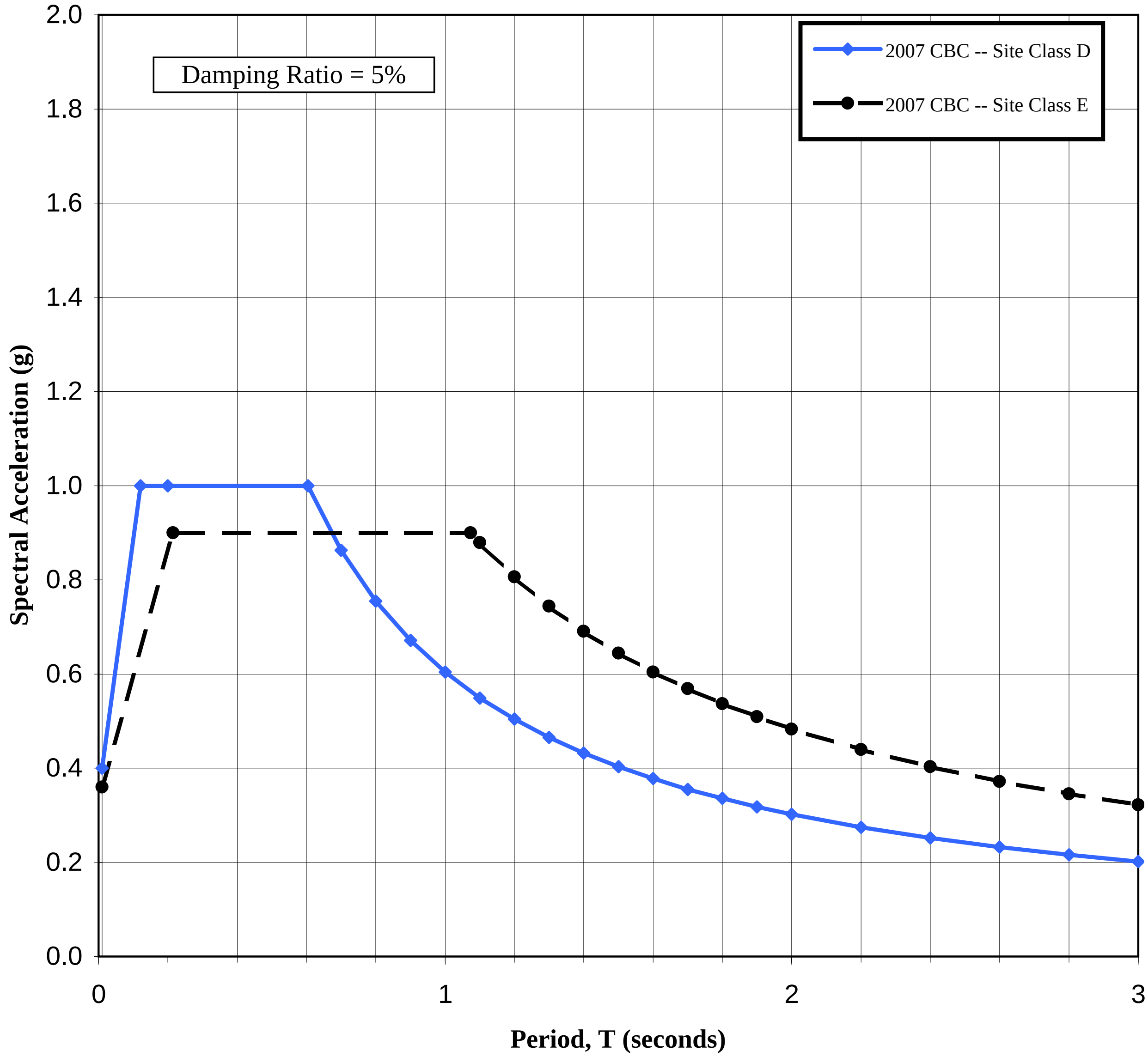


TABLE E-1
2007 CBC DESIGN RESPONSE SPECTRUM
HORIZONTAL ACCELERATION RESPONSE SPECTRA
BRANNAN STREET WHARF

2007 CBC Design Response Spectrum
Site Class D

Period (seconds)	Spectral Acceleration (g)
0.01	0.4
0.121	1
0.2	1
0.604	1
0.7	0.863
0.8	0.755
0.9	0.671
1	0.604
1.1	0.549
1.2	0.504
1.3	0.465
1.4	0.432
1.5	0.403
1.6	0.378
1.7	0.355
1.8	0.336
1.9	0.318
2	0.302
2.2	0.275
2.4	0.252
2.6	0.232
2.8	0.216
3	0.201

2007 CBC Design Response Spectrum
Site Class E

Period (seconds)	Spectral Acceleration (g)
0.01	0.36
0.215	0.9
1.074	0.9
1.1	0.879
1.2	0.806
1.3	0.744
1.4	0.691
1.5	0.644
1.6	0.604
1.7	0.569
1.8	0.537
1.9	0.509
2	0.483
2.2	0.440
2.4	0.403
2.6	0.372
2.8	0.345
3	0.322

FIGURE E-2
2007 CBC MCE RESPONSE SPECTRUM
HORIZONTAL ACCELERATION RESPONSE SPECTRA
BRANNAN STREET WHARF

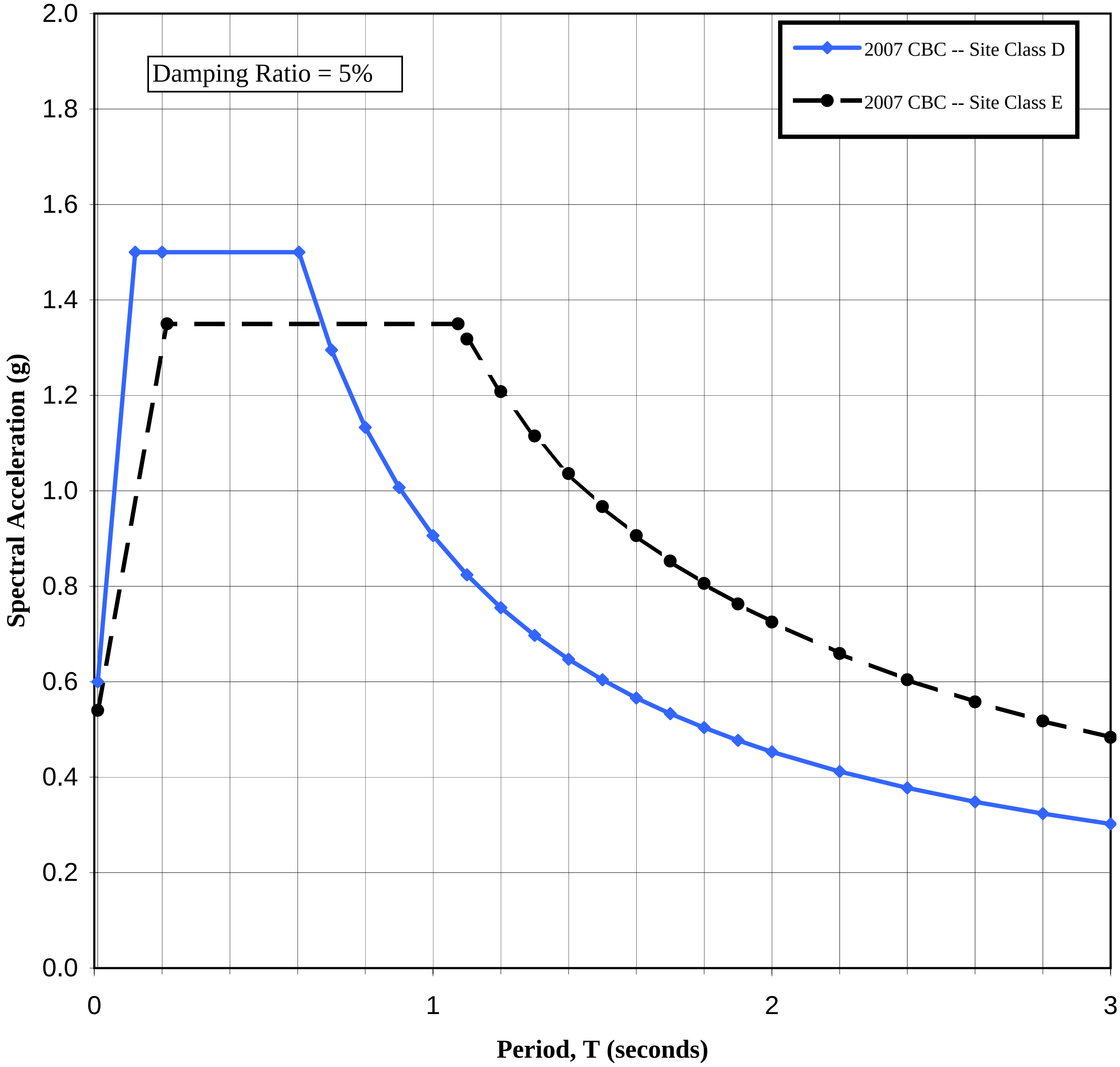


TABLE E-2
2007 CBC MCE RESPONSE SPECTRUM
HORIZONTAL ACCELERATION RESPONSE SPECTRA
BRANNAN STREET WHARF

2007 CBC Design Response Spectrum
Site Class D

Period (seconds)	Spectral Acceleration (g)
0.01	0.6
0.121	1.5
0.2	1.5
0.604	1.5
0.7	1.295
0.8	1.133
0.9	1.007
1	0.906
1.1	0.824
1.2	0.755
1.3	0.697
1.4	0.647
1.5	0.604
1.6	0.566
1.7	0.533
1.8	0.504
1.9	0.477
2	0.453
2.2	0.412
2.4	0.378
2.6	0.348
2.8	0.324
3	0.302

2007 CBC Design Response Spectrum
Site Class E

Period (seconds)	Spectral Acceleration (g)
0.01	0.54
0.215	1.35
1.074	1.35
1.1	1.318
1.2	1.208
1.3	1.115
1.4	1.036
1.5	0.967
1.6	0.906
1.7	0.853
1.8	0.806
1.9	0.763
2	0.725
2.2	0.659
2.4	0.604
2.6	0.558
2.8	0.518
3	0.483

FIGURE E-3
10% PROBABILITY OF EXCEEDANCE IN 50 YEARS
HORIZONTAL ACCELERATION RESPONSE SPECTRA
BRANNAN STREET WHARF

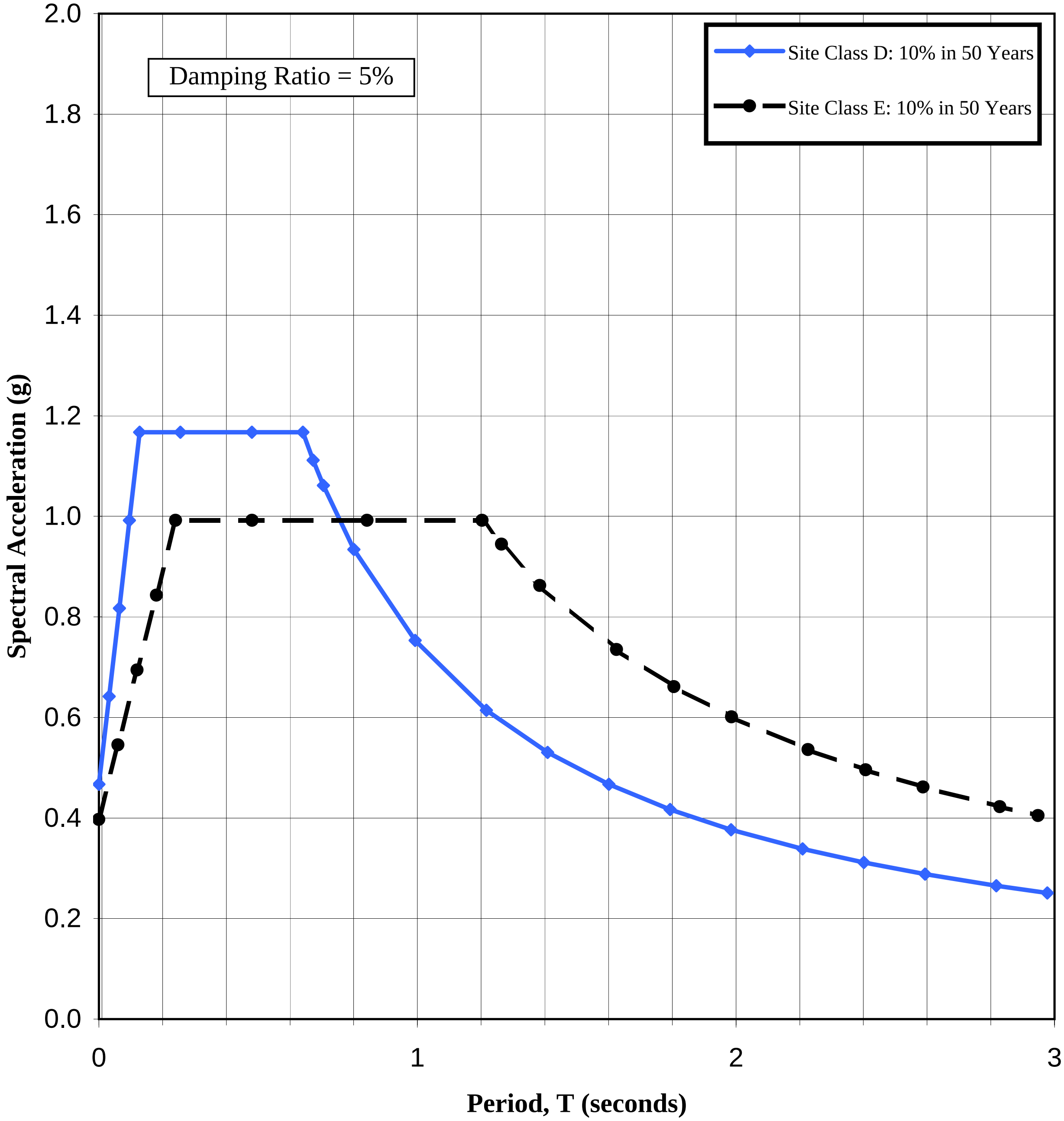


TABLE E-3
10% PROBABILITY OF EXCEEDANCE IN 50 YEARS
HORIZONTAL ACCELERATION RESPONSE SPECTRA
BRANNAN STREET WHARF

10% Probability of Exceedance in 50 yr
Site Class D

Period (seconds)	Spectral Acceleration (g)
0.00	0.467
0.03	0.642
0.06	0.817
0.10	0.992
0.13	1.167
0.26	1.167
0.48	1.167
0.64	1.167
0.67	1.112
0.70	1.061
0.80	0.934
0.99	0.753
1.22	0.614
1.41	0.531
1.60	0.467
1.79	0.417
1.99	0.377
2.21	0.338
2.40	0.311
2.59	0.288
2.82	0.265
2.98	0.251

10% Probability of Exceedance in 50 yr
Site Class E

Period (seconds)	Spectral Acceleration (g)
0.00	0.397
0.06	0.546
0.12	0.694
0.18	0.843
0.24	0.992
0.48	0.992
0.84	0.992
1.20	0.992
1.26	0.945
1.38	0.863
1.62	0.735
1.81	0.661
1.99	0.601
2.23	0.536
2.41	0.496
2.59	0.461
2.83	0.422
2.95	0.405

FIGURE E-4
50% PROBABILITY OF EXCEEDANCE IN 50 YEARS
HORIZONTAL ACCELERATION RESPONSE SPECTRA
BRANNAN STREET WHARF

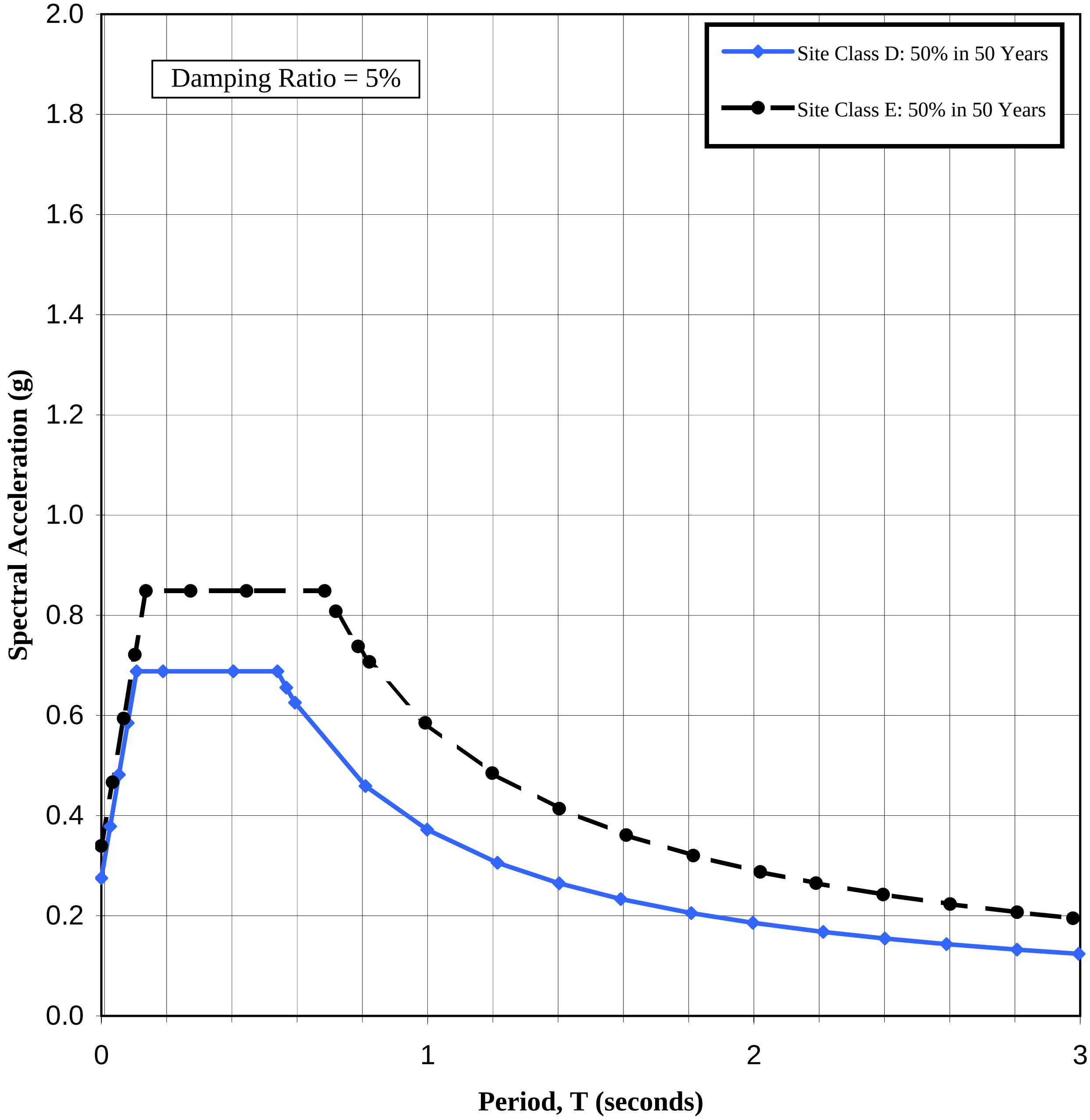


TABLE E-4
50% PROBABILITY OF EXCEEDANCE IN 50 YEARS
HORIZONTAL ACCELERATION RESPONSE SPECTRA
BRANNAN STREET WHARF

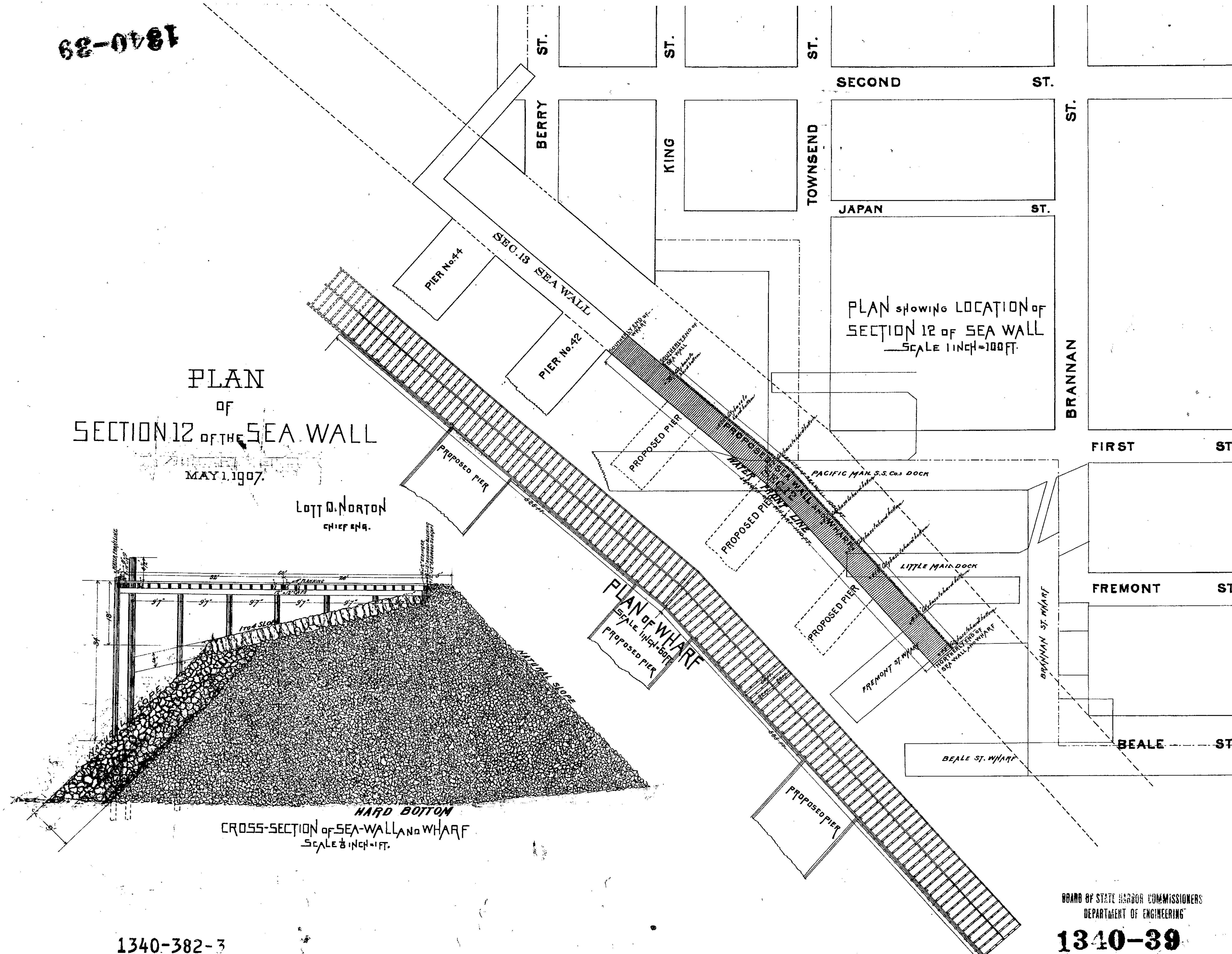
50% Probability of Exceedance in 50 yr
Site Class D

Period (seconds)	Spectral Acceleration (g)
0.00	0.275
0.03	0.379
0.05	0.482
0.08	0.585
0.11	0.688
0.19	0.688
0.40	0.688
0.54	0.688
0.57	0.655
0.59	0.626
0.81	0.459
1.00	0.372
1.21	0.306
1.40	0.265
1.59	0.233
1.81	0.205
2.00	0.186
2.21	0.168
2.40	0.155
2.59	0.143
2.81	0.132
3.00	0.124

50% Probability of Exceedance in 50 yr
Site Class E

Period (seconds)	Spectral Acceleration (g)
0.00	0.339
0.03	0.467
0.07	0.594
0.10	0.721
0.14	0.848
0.27	0.848
0.44	0.848
0.68	0.848
0.72	0.808
0.79	0.738
0.82	0.707
0.99	0.585
1.20	0.485
1.40	0.414
1.61	0.361
1.81	0.320
2.02	0.288
2.19	0.265
2.40	0.242
2.60	0.223
2.81	0.207
2.98	0.195

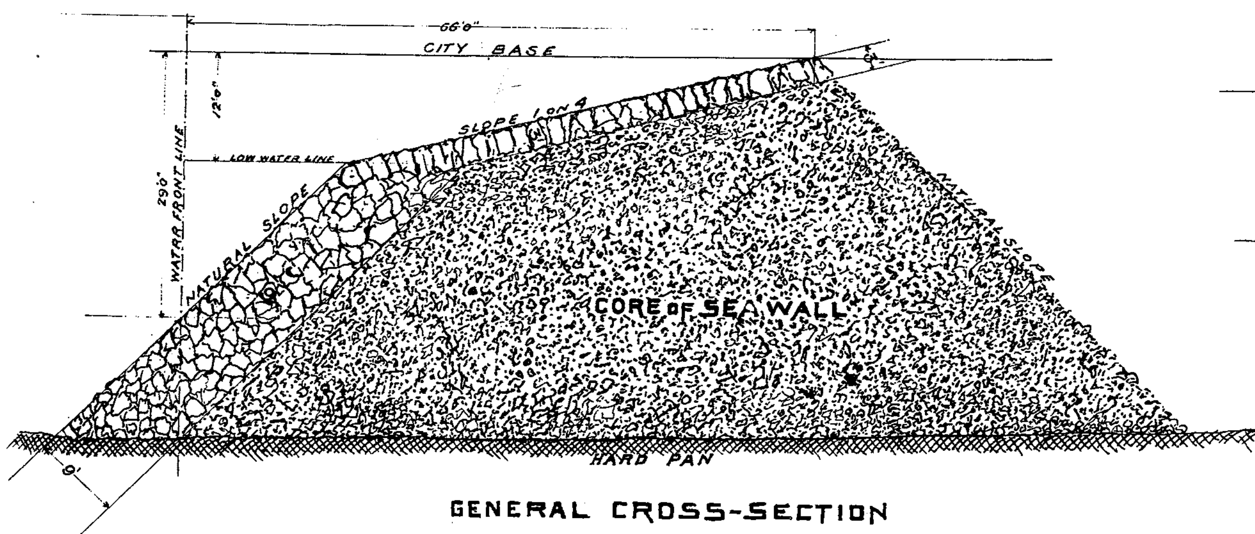
APPENDIX F
HISTORIC SEA WALL PLANS



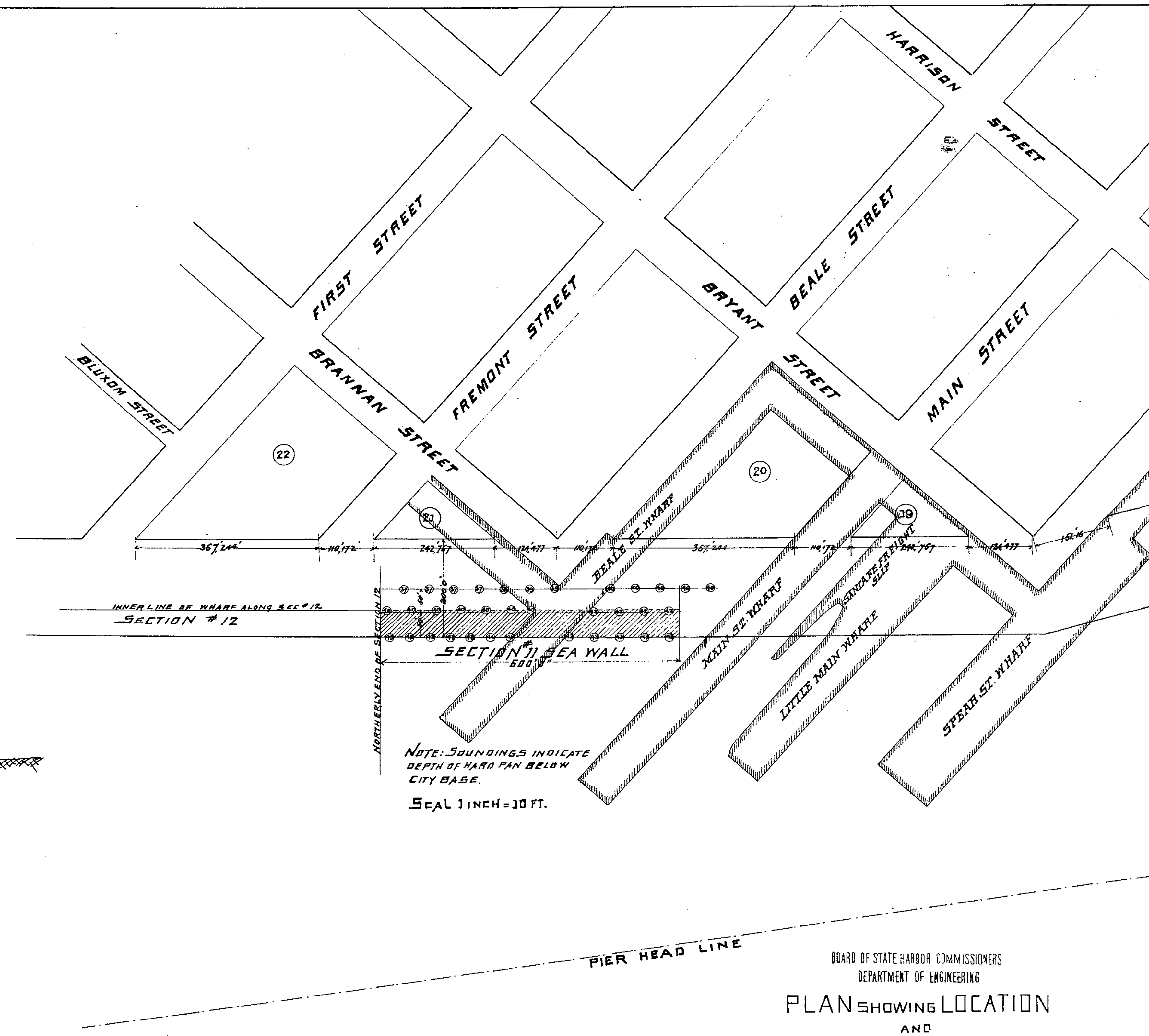
1340-382-3

BOARD OF STATE HARBOR COMMISSIONERS
DEPARTMENT OF ENGINEERING
1340-39
1340-382-3

1349-39



GENERAL CROSS-SECTION
OF
SEA WALL
SCALE $\frac{1}{8}$" = 1 FT.



BOARD OF STATE HARBOR COMMISSIONERS
DEPARTMENT OF ENGINEERING

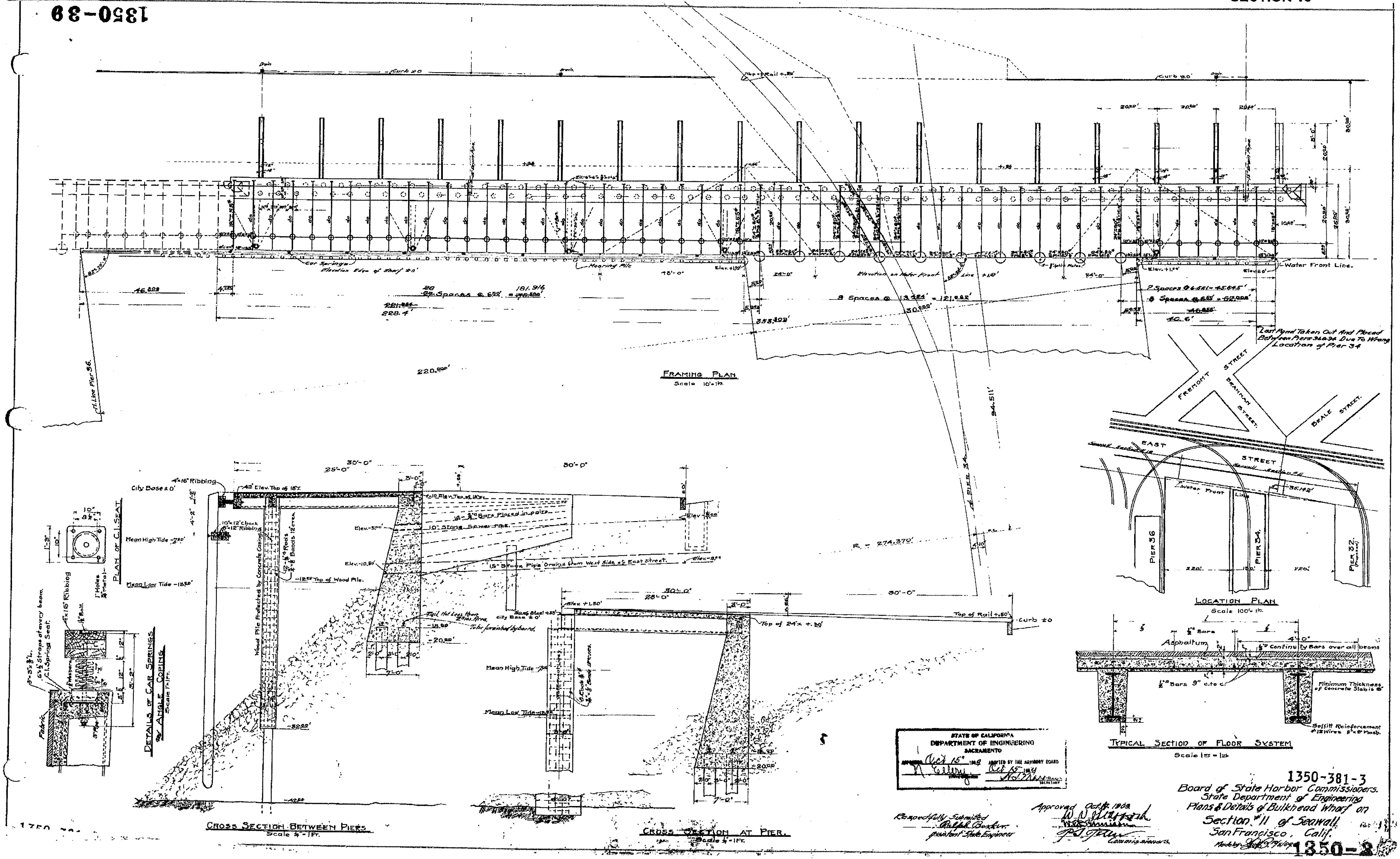
PLAN SHOWING LOCATION
AND
GENERAL CROSS-SECTION
OF
SEA WALL SECTION No. 11

OCTOBER 14, 1908.
Ralph Barker,
Asst State Engr.

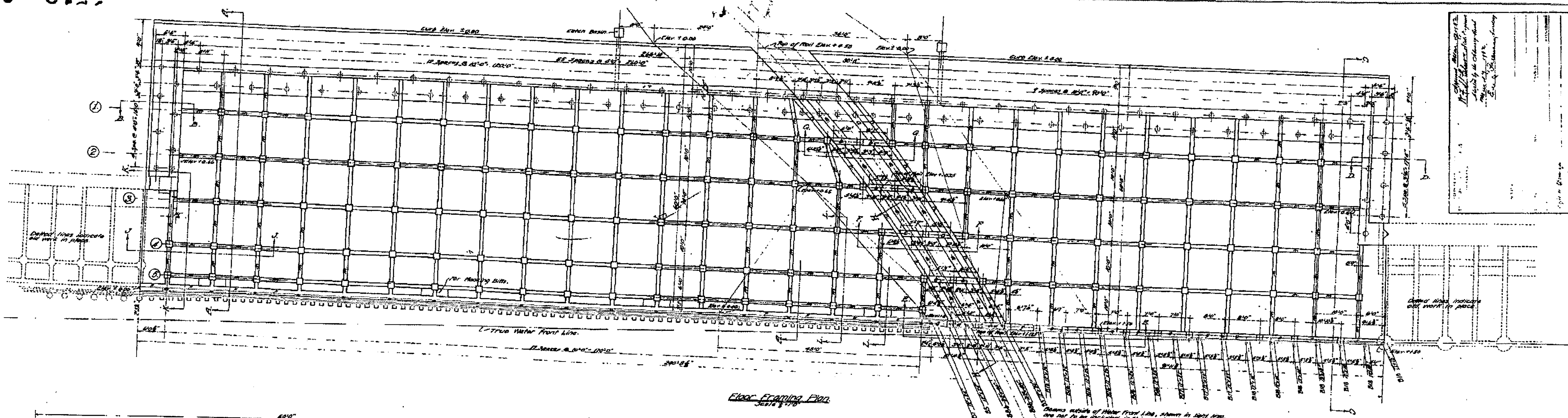
Approved 1 October 1908.
W. U. [illegible]
W. E. Dennison
Board State Harbor Commissioners

1349-381-3

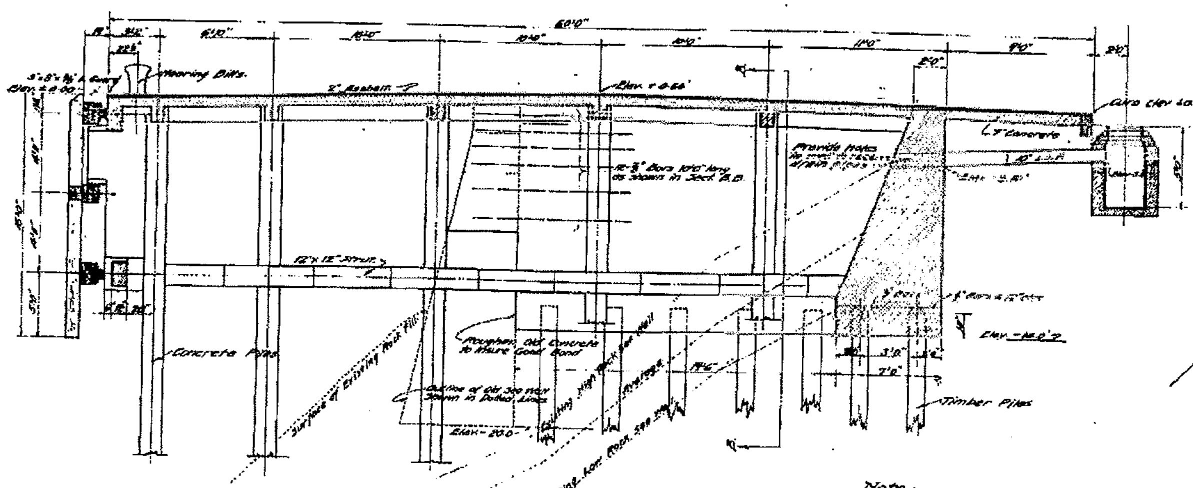
1349-381- 1349-39


1350-39
Framing Plan
Scale 10'=1in.
Water Front Line
Last Pond Taken Out And Placed Between Piers 36 & 34 Due To Wrong Location of Pier 34
Location Plan
Scale 100'=1in.
Fremont Street
Bryant Street
Beale Street
East Street
Pier 36
Pier 34
Pier 32
Typical Section of Floor System
Scale 1in=1in.
Details of Car Springs and Angle Coping
Scale 1"=1Ft.
Plan of C.I. Seat
Cross Section Between Piers
Scale 1/4"=1Ft.
Cross Section at Pier.
Scale 1/4"=1Ft.
State of California
Department of Engineering
Sacramento
Respectfully Submitted
Ralph Booker
Assistant State Engineer
Approved Oct. 1909
Commissioners
1350-381-3
Board of State Harbor Commissioners.
State Department of Engineering
Plans & Details of Bulkhead Wharf on
Section #11 of Seawall
San Francisco, Calif.
1350-2

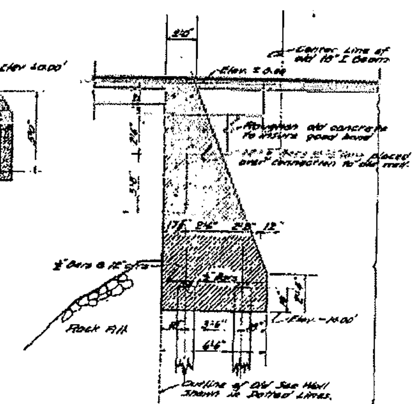
1518 - 39



Floor Framing Plan

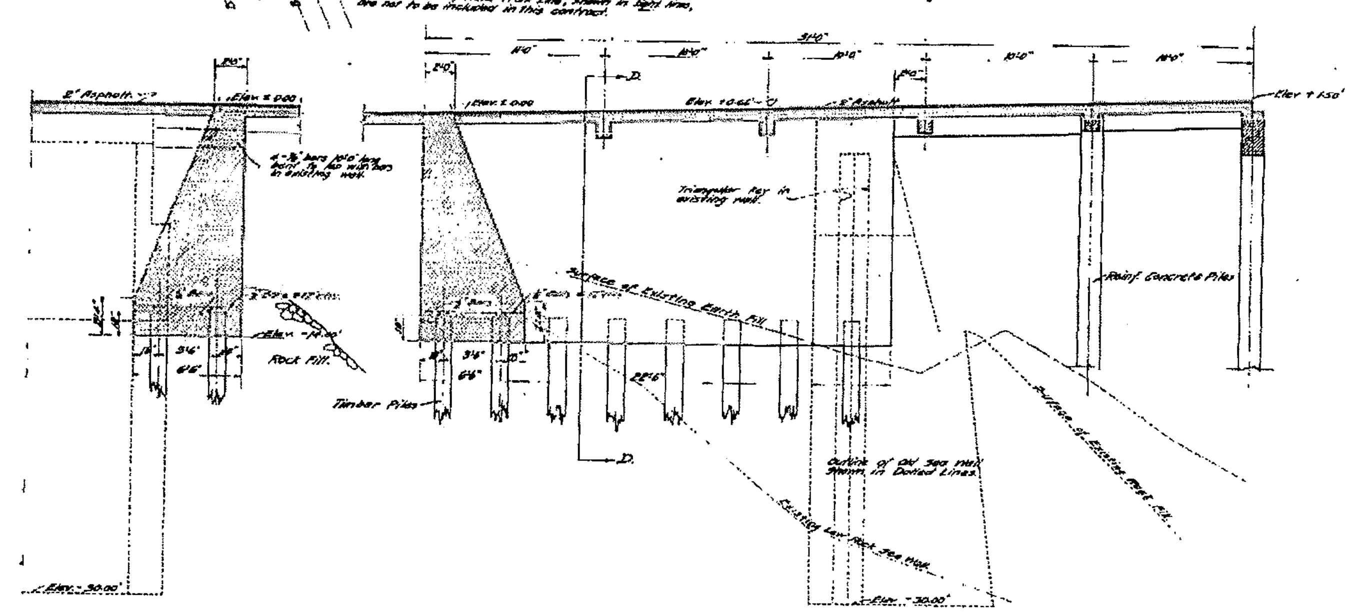


Section "A.A."



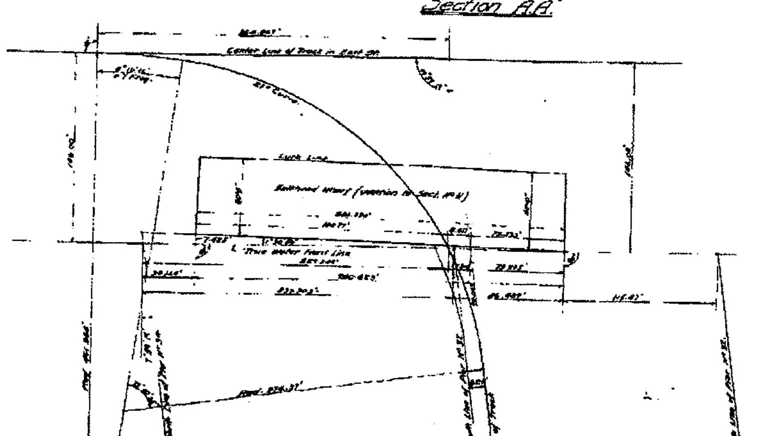
Section "B.B."

Note — Curb, Catch Basins, T.S. Pipe, Concrete Slab bet. Curb and Sea Wall, not to be included in this contract.
Elevations marked on Floor Framing Plan apply to finished work with respect to City Base. (Elev. = 0.00)
Floor Slabs to be 7" Reinf. Throughout.

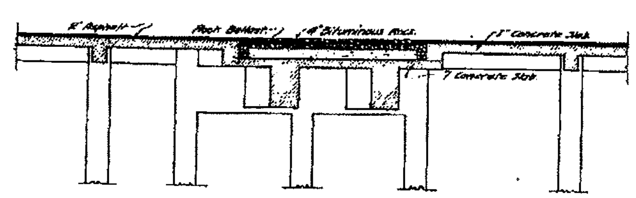


Section "D.D."

Section "C.C."



Location Plan



Cross Section Through Rail Road Track.

Schedule of Concrete Beams

Mark	Width	Depth	Main Reinf.	Stirrups	Remarks
a	20"	40"	7-¾" bars	26-⅜" bars.	See Sheets Nº 2 and 3 for details of beams
b	16"	40"	7-¾" and 2-½"	15-⅜"	
c	18"	43"	6-¾" and 2-½"	19-⅜"	
d	18"	60"	8-¾"	⅜" @ 4" spa.	
e	Varies		6-¾" and 2-½"	⅜" @ 6" spa.	
f	16"	38"	7-¾" and 2-½"	⅜" @ 8" spa.	
g	16"	38"	4-¾" and 2-½"	⅜" @ 8" spa.	
h	10"	20"	3-⅝"	15-⅜"	
k	12"	24"	4-¾"	12-⅜"	
m	10"	18"	3-⅝"	14-⅜"	
n	12"	23"	4-⅝"	20-⅜"	
p	12"	18"	4-¾"	13-⅜"	Under "k"
s	Varies	18"	4-¾"	11-⅜"	Under "e"
Struts	12"	12	4-¾"	#6 wire	See Details.

Elevations of Bottoms of Piles.

Reinforced Concrete Piles.		Timber Piles Under Retaining Wall.	
Location	Elevation	Location.	Elevation
Line 1	– 35.00'	End Walls	– 40.00'
" 2	– 40.00'	Main Wall	– 35.00'
" 3	– 45.00'		
" 4	– 50.00'		
" 5	– 50.00'		
Water Front Line	– 50.00'		

Respectfully Submitted
A. V. Saph
Assistant State Engineer

Approved March 1912
[signatures]
Thomas S. Williams
Commissioners

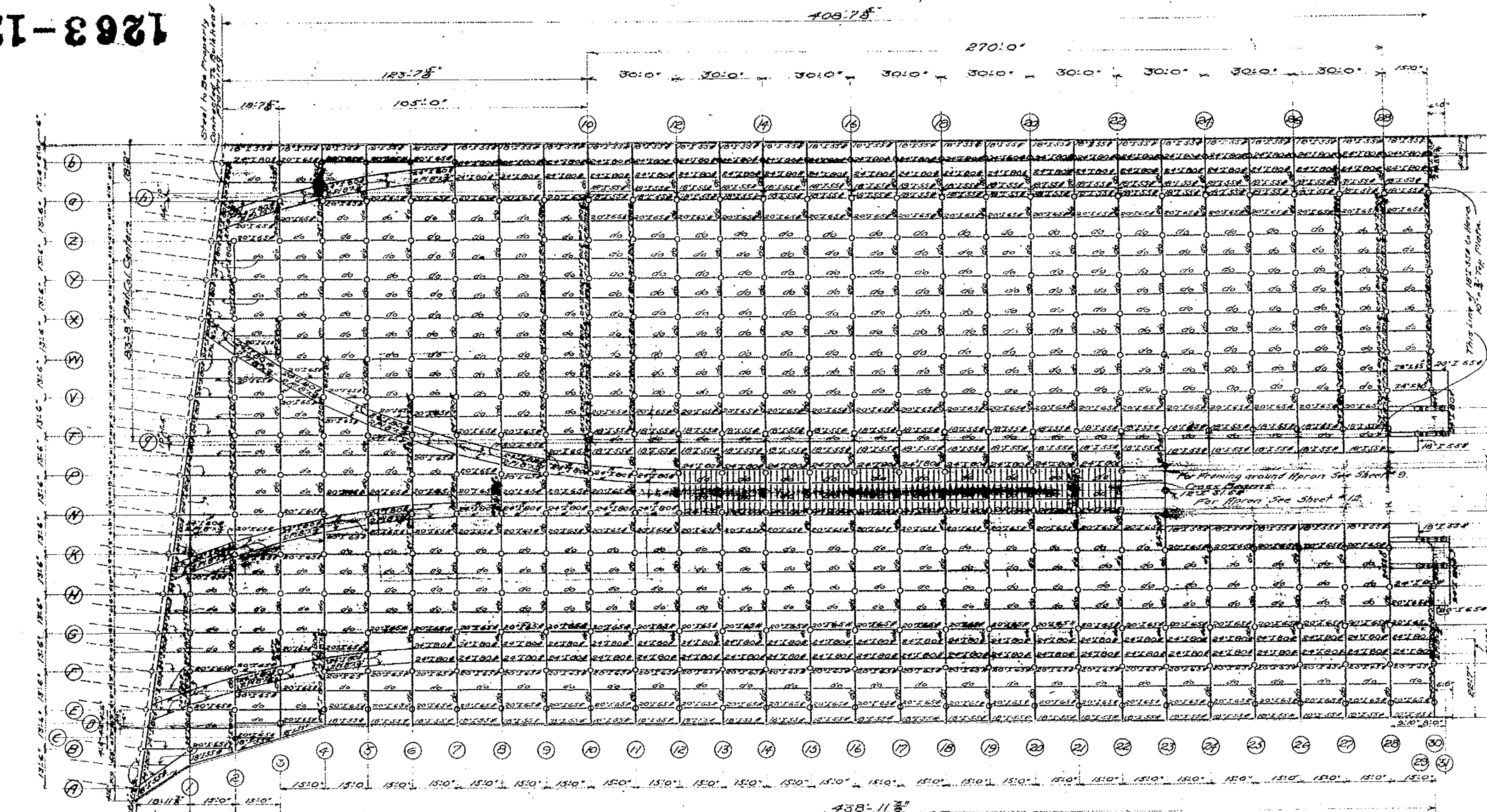
BOARD OF STATE HARBOR COMMISSIONERS
STATE DEPARTMENT OF ENGINEERING

BULKHEAD WHARF, RETAINING WALL & SECTION 11 a & SEAWALL
SAN FRANCISCO, CALIFORNIA.

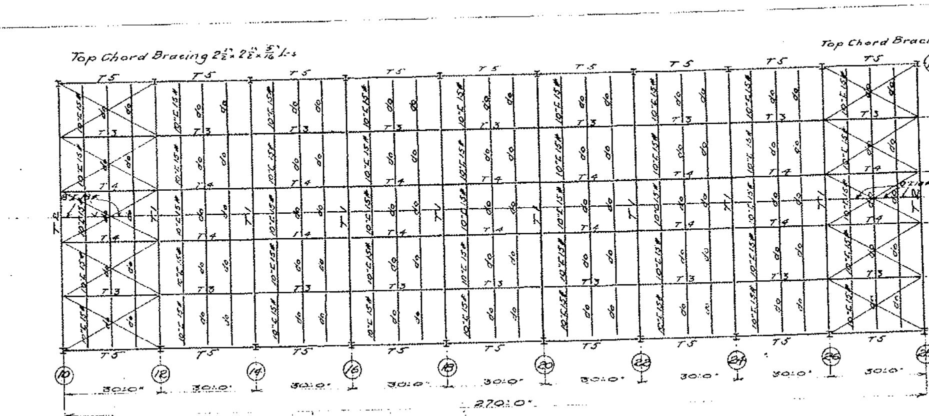
In charge of [signature] Date Jan. 17, 1912
Made by
Checked by
Drawing Number 1518 + 29 (32)

151.-381-3

APPENDIX G
HISTORIC PIER NO. 36 PLANS



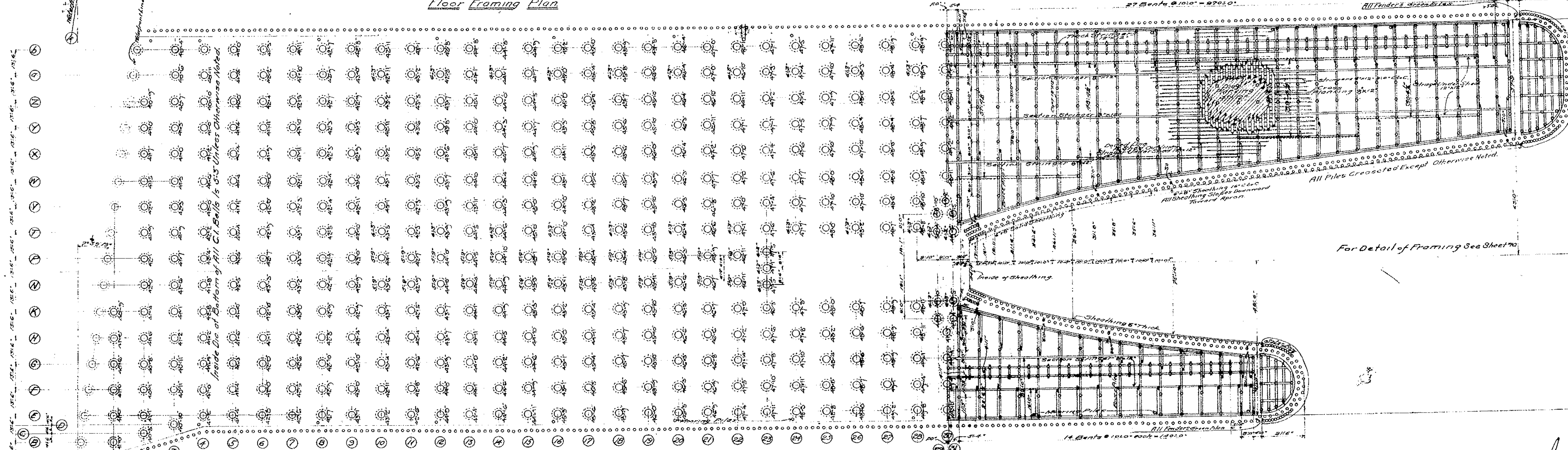
Floor Framing Plan



Roof Framing Plan

GEN. NOTES.

All 8"×¾" Top & Bottom Plates For 24"I Cross Girders to Be 9'-6" Long.
All 10"×¾" Top Plates On Beams Under Walls to Be Full Length of Beams.
All 8"×¾" Top & Bottom Plates on 24"I Track Stringers to Be Full Length of Girders.
Pier Construction to Be Properly Riveted to Beams in Bulkhead Framing
RIVETS ¾" DIAMETER.
All Beams Flush Tops 0'-9" Above City Base
Top of Track Rails 0'-9" Above Top of Beams.
All Concrete Cylinders to Be 3'-6" Dia.
For Inside Dia. of C.I. Bells See Figures on Plan of Cylinders (This Sheet)
In Connecting Bottom Plates to Is. Rivets Must Be Staggered.
No Holes To Be Punched In Bottom Flange of Purlins.
Support Bracing At Intermediate Points By Fastening
To Web of Purlins.



Plan of Cylinders.

NOTE: Red figures indicate Cylinder lengths as actually placed. Lengths measured from underside of 24" I.

STATE OF CALIFORNIA
DEPARTMENT OF ENGINEERING
SACRAMENTO
APP. Mch 9 1909 N. Ellery STATE ENGINEER
ADOPTED BY THE ADVISORY ... Mch 9 1909 ... SECRETARY

Board of State Harbor Commissioners
State Department of Engineering
Steel Framing, Cylinder
And Pile Plans

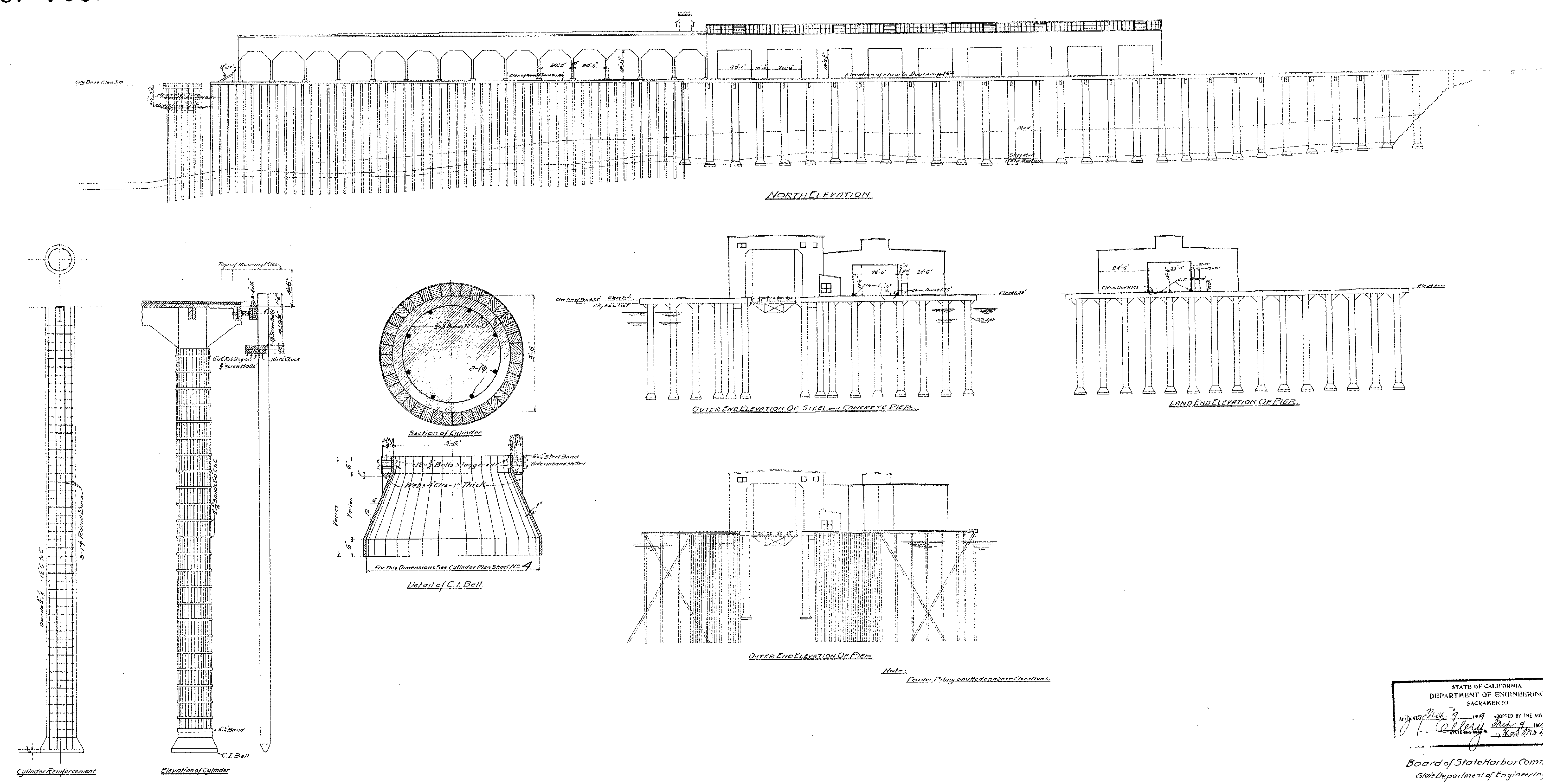
PIER No. 36.
San Francisco Cal.
1263-36-3

Approved Feb. 25, 1909.

Respectfully Submitted Ralph Barker
Assistant State Engineer

Commissioners

In Charge of ...
Made By—H.P.W.
Checked By ...



STATE OF CALIFORNIA
DEPARTMENT OF ENGINEERING
SACRAMENTO
Approved Nov. 9 1909 Adopted by the Advisory Board Nov. 9 1909
State Engineer — Secretary

Board of State Harbor Commissioners
State Department of Engineering

North and End Elevations, Cylinder Details.
Scale 1" = 20 Ft.

PIER No 36
San Francisco Cal.
1264-36-3

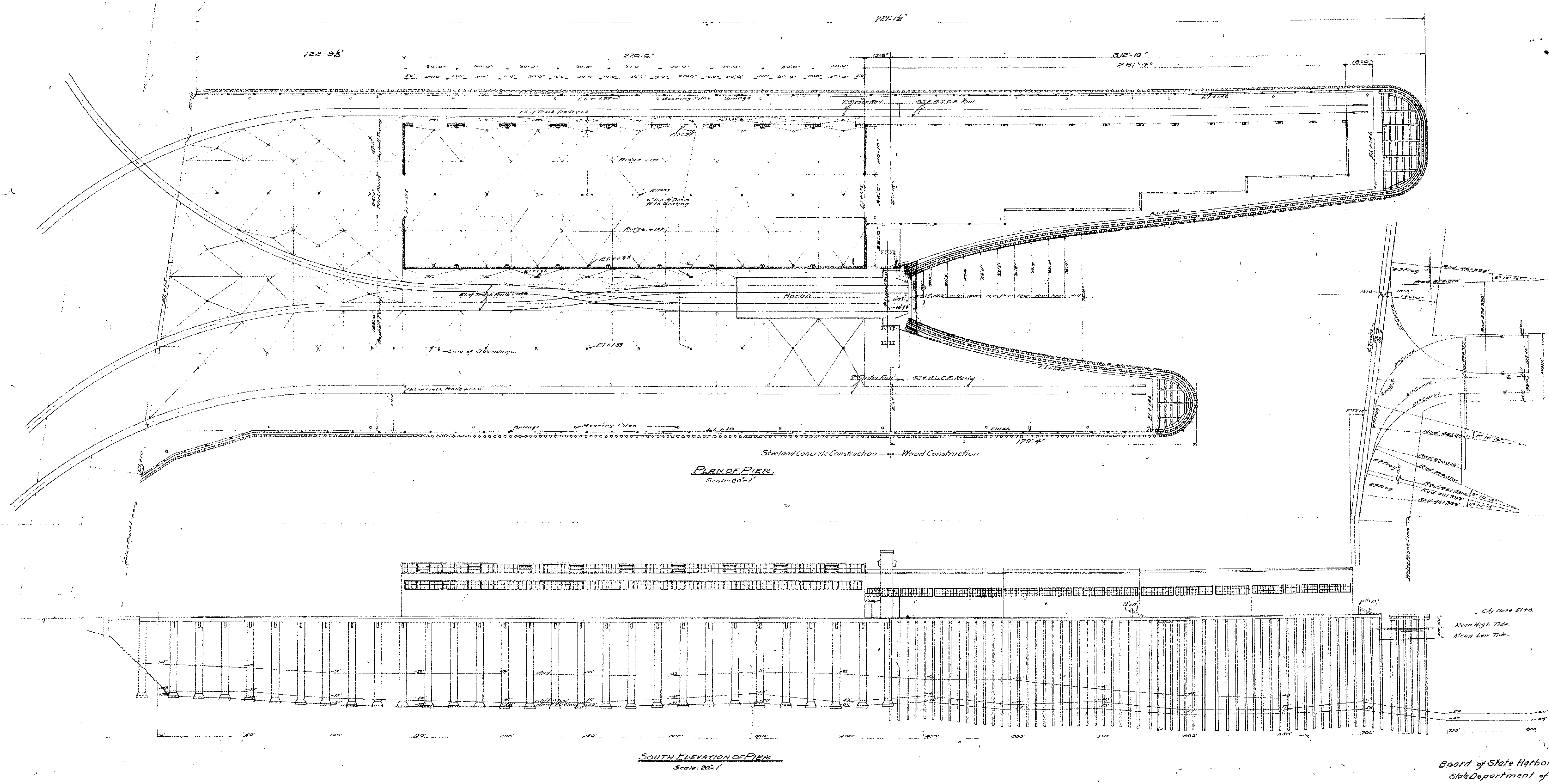
Respectfully submitted: Ralph Barker
Assistant State Engineer

Approved: Feb. 25, 1909
Commissioners

In Charge of
Made by
Checked by

1264-12 (3)

1264-36-3



STATE OF CALIFORNIA
DEPARTMENT OF ENGINEERING
SACRAMENTO
APPROVED Mch 9 1909 — STATE ENGINEER
ADOPTED BY THE ADVISORY BOARD Mch 9 1909 — SECRETARY

Respectfully Submitted Ralph R. Barker
Assistant State Engineer

Approved Feb 25, 1909
Commissioners

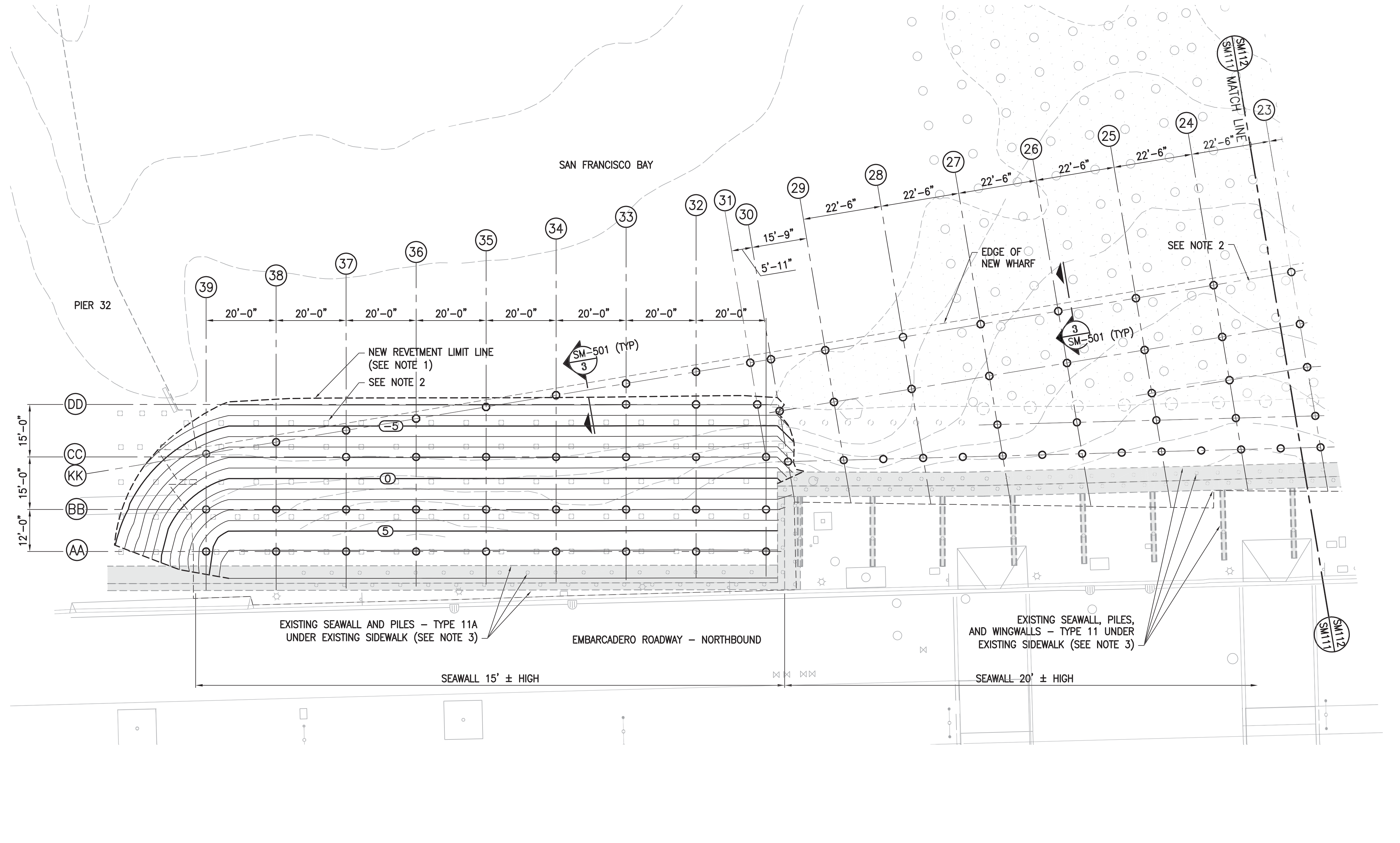
Board of State Harbor Commissioners
State Department of Engineering
Floor Plan and South Elevation
Scale 1" = 20 Ft.

PIER No. 36.
San Francisco Cal.
1265-36-3

In Charge
Made By
Checked By

1265-36

APPENDIX H
PRELIMINARY FOUNDATION PLANS FOR BRANNAN STREET WHARF



1 / SM-121 **STRUCTURE PLAN – WHARF**
SCALE: 1/16"=1'-0"

16' 8' 0 16' 32'
SCALE: 1/16" = 1'-0"

STRUCTURE SHEET NOTES:

1. NEW REVETMENT REQUIRED BETWEEN GRID LINES 31 AND 39. ESTIMATED NEW REVETMENT VOLUME IS 1800 CUBIC YARDS.
2. POTENTIAL EXISTING PILE AND CAISSON STRUCTURE APPROXIMATE LOCATIONS BELOW MUDLINE, SHOWN IN SHADOW LINE.
3. EXISTING SEAWALL LOCATION SHOWN PER PORT OF SAN FRANCISCO RECORD DRAWINGS, ACTUAL LOCATION MAY VARY.
4. PILE GRID LINES A–C, AA–DD TO BE LOCATED RELATIVE TO EXISTING SEAWALL.

PILE SCHEDULE:

SOUTH WHARF

GRID ROW	No. of PILES	PILE TYPE	PILE LENGTH (FEET)	DOWELS
A	59	24" x 0.625" Pipe	105	8-#7
B	10	24" x 0.625" Pipe	80	8-#7
C	17	24" x 0.625" Pipe	80	8-#7
J30**	1	24" x 0.625" Pipe	80	8-#7
D	5	24" PSC OCTAGONAL	100	8-#6
E	11	24" PSC OCTAGONAL	100	8-#6
F	16	24" PSC OCTAGONAL	100	8-#6
G	18	24" PSC OCTAGONAL	100	8-#6
H	23	24" PSC OCTAGONAL	100	8-#6
J	27	24" PSC OCTAGONAL	100	8-#6
K	28	24" PSC OCTAGONAL	100	8-#6
L	10	24" PSC OCTAGONAL	100	8-#6
SUBTOTAL	**225**			

NORTH WHARF

GRID ROW	No. of PILES	PILE TYPE	PILE LENGTH (FEET)	DOWELS
AA	9	24" x 0.625" Pipe	105	8-#7
BB	9	24" x 0.625" Pipe	95	8-#7
CC	7	24" x 0.625" Pipe	95	8-#7
DD	3	24" PSC OCTAGONAL	100	8-#6
KK	5	24" PSC OCTAGONAL	100	8-#6
KK36-KK39**	4	24" x 0.625" Pipe	95	8-#7
SUBTOTAL	**37**			

NO.	DATE	DESCRIPTION	BY	APP.
C	4.29.10	95% DESIGN SUBMITTAL		
B	12.23.09	60% DESIGN SUBMITTAL		
A	9.23.09	30% DESIGN SUBMITTAL		

TABLE OF REVISIONS
CHECK WITH TRACING TO SEE IF YOU HAVE LATEST REVISION

REFERENCE INFORMATION & FILE NO. OF SURVEYS

SAN FRANCISCO PORT COMMISSION
PORT OF SAN FRANCISCO
DEPARTMENT OF ENGINEERING
PORT OF SAN FRANCISCO

WINZLER & KELLY | Structus, Inc.
A Joint Venture

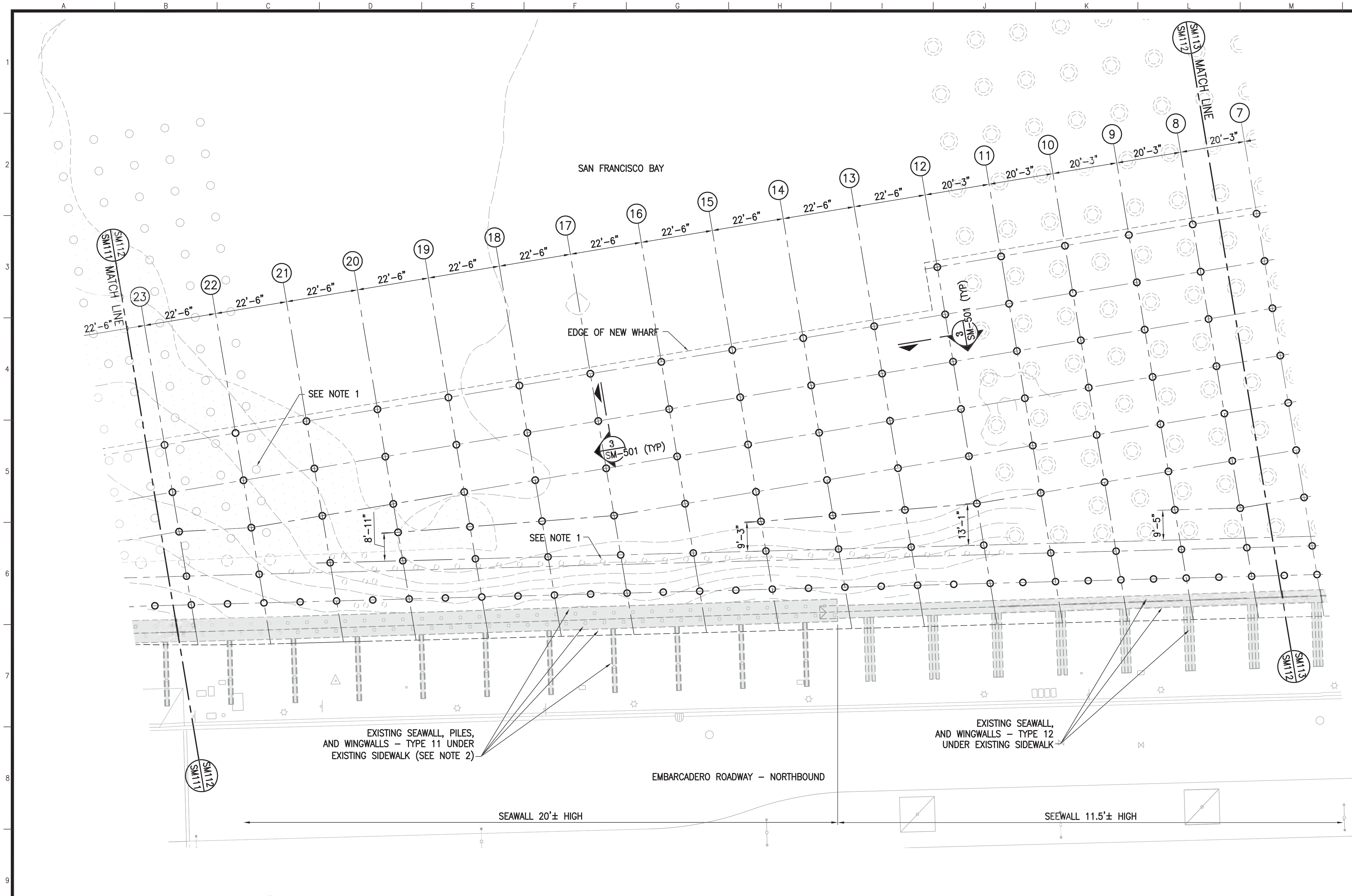
DESIGNED:	DATE:
CSS	08/28/09
DRAWN:	DATE:
PVB	08/28/09
CHECKED:	DATE:
CL	08/28/09

APPROVED BY
SAN FRANCISCO PORT COMMISSION
DATE: ________
CHIEF HARBOR ENGINEER

SCALE: AS NOTED
SHEET OF SHEETS: XX OF XX

BRANNAN STREET WHARF
FOUNDATION PLAN – WHARF 1

CONTRACT NO. 2726
DRAWING NO. SM-111
FILE NO. –
REV. NO. –



1
SM-122

STRUCTURE PLAN – WHARF

SCALE: 1/16"=1'-0"

16' 8' 0 16' 32'
SCALE: 1/16" = 1'-0"

STRUCTURE SHEET NOTES:

1. POTENTIAL (E) PILE AND CAISSON STRUCTURE APPROX LOCATIONS BELOW MUDLINE SHOWN IN SHADOW LINE.
2. (E) SEAWALL LOCATION SHOWN PER POSF TOPO DRAWING, ACTUAL LOCATION MAY VARY.
3. PILE GRID LINES A–C, AA–DD TO BE LOCATED RELATIVE TO EXISTING SEAWALL.

PILE SCHEDULE:

SOUTH WHARF

GRID ROW	No. of PILES	PILE TYPE	PILE LENGTH (FEET)	DOWELS
A	59	24" x 0.625" Pipe	105	8-#7
B	10	24" x 0.625" Pipe	80	8-#7
C	17	24" x 0.625" Pipe	80	8-#7
J30**	1	24" x 0.625" Pipe	80	8-#7
D	5	24" PSC OCTAGONAL	100	8-#6
E	11	24" PSC OCTAGONAL	100	8-#6
F	16	24" PSC OCTAGONAL	100	8-#6
G	18	24" PSC OCTAGONAL	100	8-#6
H	23	24" PSC OCTAGONAL	100	8-#6
J	27	24" PSC OCTAGONAL	100	8-#6
K	28	24" PSC OCTAGONAL	100	8-#6
L	10	24" PSC OCTAGONAL	100	8-#6
SUBTOTAL	**225**			

NORTH WHARF

GRID ROW	No. of PILES	PILE TYPE	PILE LENGTH (FEET)	DOWELS
AA	9	24" x 0.625" Pipe	105	8-#7
BB	9	24" x 0.625" Pipe	95	8-#7
CC	7	24" x 0.625" Pipe	95	8-#7
DD	3	24" PSC OCTAGONAL	100	8-#6
KK	5	24" PSC OCTAGONAL	100	8-#6
KK36-KK39**	4	24" x 0.625" Pipe	95	8-#7
SUBTOTAL	**37**			

NO.	DATE	DESCRIPTION	BY	APP.
C	4.29.10	95% DESIGN SUBMITTAL		
B	12.23.09	60% DESIGN SUBMITTAL		
A	9.23.09	30% DESIGN SUBMITTAL		

TABLE OF REVISIONS
CHECK WITH TRACING TO SEE IF YOU HAVE LATEST REVISION

REFERENCE INFORMATION & FILE NO. OF SURVEYS

SAN FRANCISCO PORT COMMISSION
PORT OF SAN FRANCISCO
DEPARTMENT OF ENGINEERING
PORT OF SAN FRANCISCO

WINZLER & KELLY | Structus, Inc.
A Joint Venture

DESIGNED:	DATE:
CSS	08/28/09
DRAWN:	DATE:
PVB	08/28/09
CHECKED:	DATE:
CL	08/28/09

APPROVED BY
SAN FRANCISCO PORT COMMISSION
DATE: ________
CHIEF HARBOR ENGINEER

SCALE: AS NOTED
SHEET OF SHEETS: XX OF XX

BRANNAN STREET WHARF
FOUNDATION PLAN – WHARF 2

CONTRACT NO. 2726
DRAWING NO. SM-112
FILE NO. –
REV. NO. –

Drawing Path: P:\Projects\11808 Port of SF Brannan St. Wharf (W&K-Structus JV)\1180809001 Port of San Francisco – Brannan Street Wharf\Phase 2\CAD\1180809001SM-111 THRU SM-113.dwg, Login: PScheetz
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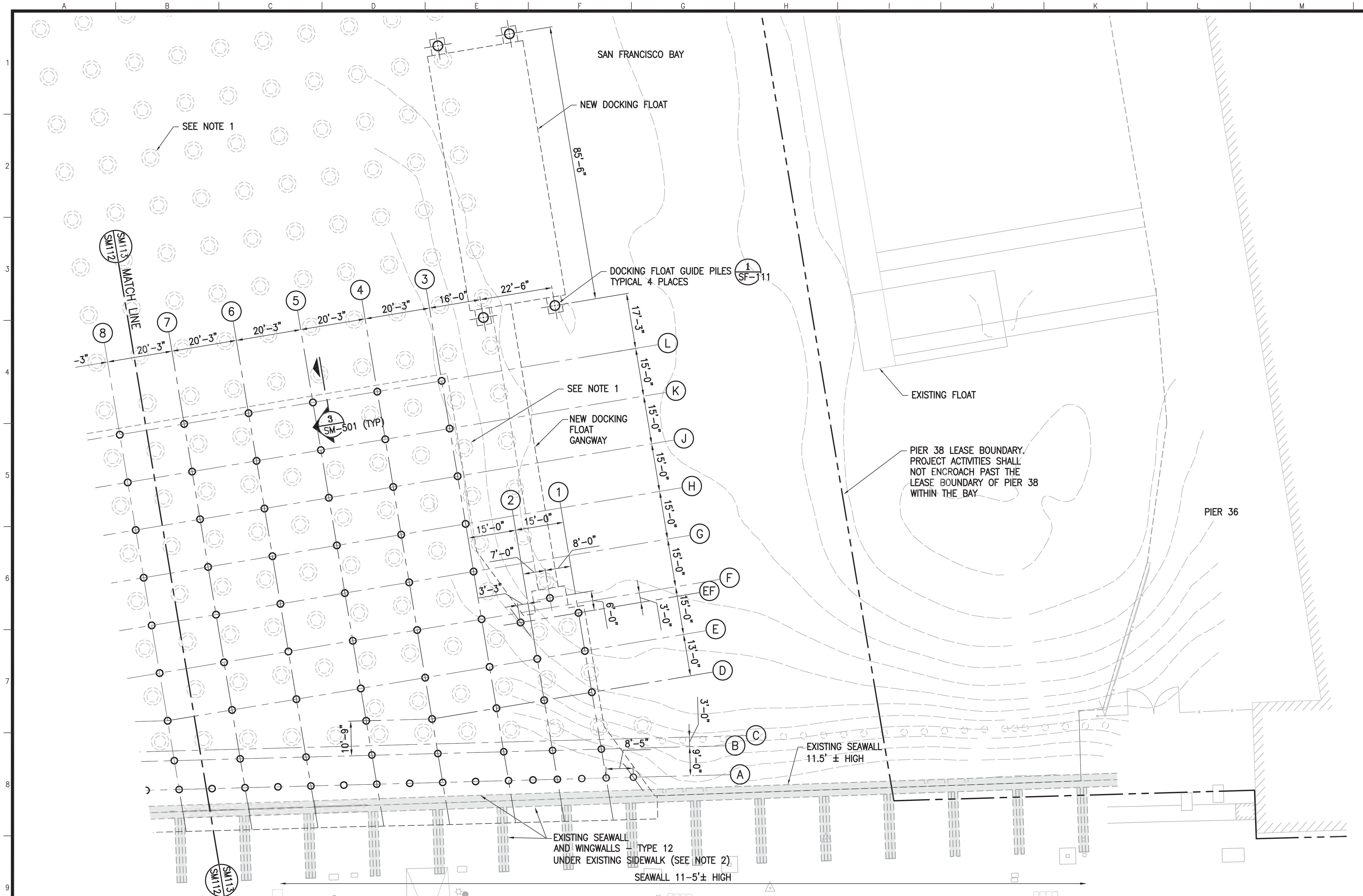
Dimension Scale: 192
Measurement Units are English
Model Units: Undefined

VIEW: PLOT1



1 STRUCTURE PLAN – WHARF

SM-123 SCALE: 1/16"=1'-0"

SCALE: 1/16" = 1'-0"

STRUCTURE SHEET NOTES:

1. PROTENTIAL (E) CAISSON STRUCTURE APPROX. LOCATIONS BELOW MUDLINE SHOWN IN SHADOW LINE.
2. (E) SEAWALL LOCATION SHOWN PER POSF TOPO DRAWING, ACTUAL LOCATION MAY VARY.
3. PILE GRID LINES A–C, AA–DD TO BE LOCATED RELATIVE TO EXISTING SEAWALL.

PILE SCHEDULE:

SOUTH WHARF				
GRID ROW	No. of PILES	PILE TYPE	PILE LENGTH (FEET)	DOWELS
A	59	24" x 0.625" Pipe	105	8-#7
B	10	24" x 0.625" Pipe	80	8-#7
C	17	24" x 0.625" Pipe	80	8-#7
J30**	1	24" x 0.625" Pipe	80	8-#7
D	5	24" PSC OCTAGONAL	100	8-#6
E	11	24" PSC OCTAGONAL	100	8-#6
F	16	24" PSC OCTAGONAL	100	8-#6
G	18	24" PSC OCTAGONAL	100	8-#6
H	23	24" PSC OCTAGONAL	100	8-#6
J	27	24" PSC OCTAGONAL	100	8-#6
K	28	24" PSC OCTAGONAL	100	8-#6
L	10	24" PSC OCTAGONAL	100	8-#6
SUBTOTAL	**225**			

NORTH WHARF				
GRID ROW	No. of PILES	PILE TYPE	PILE LENGTH (FEET)	DOWELS
AA	9	24" x 0.625" Pipe	105	8-#7
BB	9	24" x 0.625" Pipe	95	8-#7
CC	7	24" x 0.625" Pipe	95	8-#7
DD	3	24" PSC OCTAGONAL	100	8-#6
KK	5	24" PSC OCTAGONAL	100	8-#6
KK36-KK39**	4	24" x 0.625" Pipe	95	8-#7
SUBTOTAL	**37**			

NO.	DATE	DESCRIPTION	BY	APP.
C	4.29.10	95% DESIGN SUBMITTAL		
B	12.23.09	60% DESIGN SUBMITTAL		
A	9.23.09	30% DESIGN SUBMITTAL		

TABLE OF REVISIONS
CHECK WITH TRACING TO SEE IF YOU HAVE LATEST REVISION

REFERENCE INFORMATION & FILE NO. OF SURVEYS

SAN FRANCISCO PORT COMMISSION
PORT OF SAN FRANCISCO
DEPARTMENT OF ENGINEERING

PORT OF SAN FRANCISCO

WINZLER & KELLY | Structus, Inc.
A Joint Venture

DESIGNED: CSS	DATE: 08/28/09
DRAWN: PVB	DATE: 08/28/09
CHECKED: CL	DATE: 08/28/09

APPROVED BY SAN FRANCISCO PORT COMMISSION
DATE: ________
CHIEF HARBOR ENGINEER

SCALE: AS NOTED
SHEET OF SHEETS: XX OF XX

BRANNAN STREET WHARF
FOUNDATION PLAN – WHARF 3

CONTRACT NO. 2726
DRAWING NO. SM-113
FILE NO. –
REV. NO. –