REPORT

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED PARKING GARAGESTRUCTURE
1450 FASHION ISLAND BOULEVARD
SAN MATEO, CALIFORNIA

For Century Centre Investors

SEPTEMBER 2014

BAYGG ENGINEERS

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Century Centre Investors
C/o Harvest Properties
1400 Fashion Island Boulevard, Suite 305
San Mateo, California 94555

Attention: Mr. Tom Wagner

Geotechnical Engineering Investigation
Proposed Parking Garage Structure
1400 - 1450 Fashion Island Boulevard
San Mateo, California

Dear Mr. Wagner:

Transmitted herewith is our geotechnical engineering investigation report for the captioned project in San Mateo, California. The report presents information on the subsurface soil and groundwater conditions at the site, and the results of our laboratory testing and engineering analyses, which formed the basis of our conclusions and recommendations for the proposed parking garage structure.

Thank you for the opportunity to perform these services. Please do not hesitate to contact us, should you have any questions or comments.

Very truly yours,
BAGG Engineers

Ajay Singh
Senior Engineer

Ebbi Hamidieh
Principal

KO/AS
Distribution: 2 copies, addressee

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ASFE document titled "Important Information About Your Geotechnical Engineering Report"

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1.0 INTRODUCTION

This report presents the results of our geotechnical engineering investigation performed for the proposed parking structure at 1400-1450 Fashion Island Boulevard in San Mateo, California. The attached Plate 1, Vicinity Map, show the general location of the site, and Plate 2, Site Plan, shows the approximate footprint of the proposed parking garage, and the approximate locations of our exploratory borings drilled for this investigation. This report was prepared in accordance with the Agreement for Professional Services between BAGG Engineers and Harvest Properties, Inc., dated June 10, 2014.

2.0 SITE AND PROJECT DESCRIPTION

The subject 1400 and 1450 Fashion Island Boulevard property (approximately 10.5 acres) is bordered by Fashion Island Boulevard to the north, Mariner Island Boulevard to the east, State Route 92 to the south, and Seal Slough to the west. The proposed parking structure will be constructed in the existing parking area to the south of the two existing mid-rise structures at 1400 and 1450 Fashion Island Boulevard in Foster City, California. The site area is relatively flat with an approximate ground surface elevation of 7 feet and is currently covered with asphalt pavement.

The proposed parking structure, which will add about 305 parking spaces, will be constructed at the location shown on Plate 2, Site Plan. The three level parking structure will be 268.5 feet long and 121 feet wide and will have ground level parking at roughly the current site grades. The

column loads are anticipated to range from a dead load of 70 kips plus a live load of 10 kips to a dead load of 390 kips plus a live load of 115 kips.

3.0 PURPOSE AND SCOPE OF SERVICE

The purpose of this investigation was to develop foundation recommendations for the revised configuration of the parking garage and to update the seismic design parameters presented in our 2008 report. BAGG Engineers performed a geotechnical engineering investigation at the subject site in 2008 and the results of that investigation were summarized in our report, dated February 20, 2008.

As a part of our February 2008 investigation, BAGG conducted geologic/geotechnical research and reviews to develop an understanding of the geotechnical and geologic setting of the site vicinity, and performed site-specific subsurface exploration and laboratory testing to develop geotechnical engineering criteria for the design and support of the previously planned parking structure. Our subsurface exploration consisted of drilling, logging, and sampling three exploratory borings, and advancing three CPTs for the geotechnical engineering study. The soil borings were drilled to approximately 80 feet, whereas the CPTs were advanced to 100 feet each. We also performed laboratory tests on the samples collected from the site including triaxial strength tests, consolidation tests, Atterberg Limits tests, particle size distribution tests, and moisture-density measurements. The results of our subsurface exploration and laboratory testing were used to perform engineering analyses to form conclusions and develop recommendations for the support of the proposed parking structure.

4.0 FIELD EXPLORATION AND LABORATORY TESTING

BAGG did not perform any additional subsurface exploration or laboratory testing as a part of the current investigation; however, as a part of our 2008 investigation, BAGG drilled three borings to 80 feet each and advanced three CPTs to 100 feet each. The approximate locations of the exploratory borings drilled and CPTs advanced for this investigation are shown on the attached

Plate 2, Site Plan. The graphical representation of the materials encountered in the borings, and the results of laboratory tests as well as explanatory/illustrative data are attached, as follows:

- Plate 5, Unified Soil Classification System, illustrates the general features of the soil classification system used on the boring logs.
- Plate 6, Soil Terminology, lists and describes the soil engineering terms used on the boring logs.
- Plate 7, Key to Symbols, describes various symbols used on the boring logs and general and specific conditions that apply to the boring logs.
- Plates 8-A through 10-D, Boring Logs, describe the subsurface materials encountered, show the depths and blow counts for the samples, and show results of the strength tests, classification tests, and moisture-density data. Appendix A includes the results of CPTs performed at the site.
- Plate 11, Plasticity Data, graphs and presents the Atterberg Limits test results for a representative near-surface soil sample obtained from one of the borings.

Selected samples were tested in the laboratory to measure natural moisture content and dry unit weights, consolidation characteristics, Atterberg Limits, and strength values using unconsolidated undrained triaxial tests, and direct shear tests. Direct shear tests were performed at both natural (field) and artificially increased moisture contents and under various surcharge pressures. The moisture content and dry density of selected undisturbed samples were determined to aid in correlating their engineering properties. The results of our laboratory strength tests and moisture-density data are summarized on the boring logs. The remaining laboratory test results are shown on the plates described above, and/or tabulated in the following sections of this report.

5.0 GEOLOGY AND SEISMICITY

5.1 Regional Geology

A review of the Geologic Map of the Onshore Part of San Mateo County, California: A Digital Database by E.E. Brabb, R.W. Graymer, and D.L. Jones, Open – File Report 98-137 indicates the site is underlain by Artificial Fill (historic), which is described as:

"Loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man made debris in various combinations. Thickness is variable and may exceed 30 meters in places. Some is compacted and quite firm, but fill made before 1965 is nearly everywhere not compacted and consists simply of dumped materials."

The Regional Geologic Map, Plate 3 shows the mapped geology of the site vicinity.

Mariner's Island is a former tidal marshland, which was diked-off from tidal fluctuations approximately 100 years ago. In 1965, a system of interior lagoons was excavated, one of which traversed a portion of the site. This lagoon was located in the western portion of the site. Temporary State Route 92 was constructed in 1965-66. The construction of SR92 was inspected by Caltrans personnel. In the summer of 1979, the lagoon was backfilled with desiccated crust material scraped from the surface of the surrounding area to approximately 96.5 feet. In summer of 1980, the subject site was rough graded/prepared, which included dewatering, removing of deleterious material and any soft muck from surface depressions, and backfilling the depressions with reconditioned Bay Mud. Following site preparation, the site was filled to elevation 98 feet with compacted, reconditioned Bay Mud. Subsequently, compacted import material was placed at the site to bring the site elevations to 102 to 104 feet as reported by Cooper Engineers, 1983. The topographical map for the site area was not available at the time this report was prepared, however; the USGS maps indicate that the majority of the site has an elevation between 106 to 107 feet with a ground surface elevation of about 104 feet in the area near the slough.

5.2 Seismicity

The project site is located within the western portion of the seismically-active San Francisco Bay region (Plate 4). The state of California has not yet published seismic hazard maps for the subject portion of the San Francisco Bay area.

One of the nearest major active faults is the San Andreas Fault, located approximately 7.1 kilometers southwest of the project site. The San Andreas fault generated an earthquake of Magnitude 7.0 on the San Francisco peninsula in 1838, and the great San Francisco Earthquake

of 1906, with an estimated Moment Magnitude of 7.9 to 8.0. More recently, on October 17, 1989, the San Andreas Fault generated the Loma Prieta earthquake with a Magnitude of 6.9. Other faults in the vicinity include the Monte Vista – Shannon fault located 13.6 km from the site, the Hayward fault located 22.5 km northeast, and the San Gregorio fault located 19.0 km west, southwest of the site.

Table 1
Significant Earthquake Scenarios

Fault	Distance*/Direction	Moment Magnitude** (Mw)
Monte Vista-Shannon	13.6 km-SW	6.3–6.5
San Andreas (1906)	7.1 km-SW	7.9–8.0
Hayward	22.5 km-NE	7.2–7.3
San Gregorio	19.0 km-W	7.4–7.5

*Google Earth & Maps of Known Active Fault Near-Source Zone in California and adjacent Portions of Nevada, 1998.
**Working Group on California Earthquake Probabilities (2008)

Plate 4, Regional Fault Map, depicts the site location with respect to the nearby active faults.

6.0 SITE CONDITIONS

6.1 Subsurface Conditions

A review of the previous geotechnical engineering investigation reports at the subject site indicate that the site is blanketed with fills of varying ages, composition, and thickness. Underlying the fills is a thin desiccated crust of medium stiff-to-stiff silty clay (Bay Mud). The fill and crust are in-turn underlain by a substantial layer of soft Bay Mud. The soils below the soft mud generally consist of stiff to very stiff silty clays and clayey silts of low to moderate plasticity inter-layered with lenses and layers of dense granular soils.

Boring B-1 revealed the presence of fill with an estimated thickness of 8½ feet. The thickness of desiccated bay mud/bay mud mixed with fill was estimated to be about 3 feet, and the thickness of soft bay mud below this layer was estimated to be 37½ feet. Boring B-2 revealed 8 feet of fill

over 2½ feet of desiccated bay mud/bay Mud mixed with fill, over 36 feet of soft Bay Mud. Boring B-3 revealed 8 feet of fill over 3 feet of desiccated bay mud/mud mixed with fill over 32½ feet of soft bay mud. The thickness of soft bay mud was interpreted to be 35 feet in CPT-1, 28 feet in CPT-2 (with about 16 feet of fill overlying soft bay mud), and 35 feet in CPT-3. The consolidation test results for the soft bay mud layer samples and a sample of the underlying lean clay are listed below:

Consolidation Test Results

Boring No.	Depth (feet)	Description	Natural Moisture Content (%)	Dry Unit Weight (pcf)	Pre-consolidation pressure (ksf)	C_v @ T_{50} (ft^2/day)	C_c	Initial Void Ratio (e_0)	Over-consolidation Ratio
B-1	21	Soft, dark, gray Fat Clay	74.1	56.5	1.6	0.01	0.85	1.9842	2.5
B-2	29	Soft, dark gray Fat Clay	72.5	55.4	1.6	0.01	0.97	2.0408	1.9
B-2	75	Stiff, greenish gray Fat Clay	39.8	83.1	3.5	0.08	0.29	1.0279	1.7
B-3	39	Soft, dark gray Fat Clay	101.2	61.1	3.26	0.01	0.88	1.6309	3.1

The test results indicate that the majority of the soils present at the site are over-consolidated.

The results of the unconsolidated undrained triaxial tests are listed in the table below:

Triaxial Strength Testing Results

Boring No.	Depth (ft.)	Description	Undrained Strength (psf)
B-1	45	Medium stiff, dark gray Fat Clay	745
B-1	75	Stiff, dark greenish gray Fat Clay	1553
B-2	36	Very soft, dark gray Fat Clay	215
B-3	65	Stiff, dark gray Fat Clay	1653

The soils underlying the Bay Mud layer consisted of very stiff lean to fat clayey inter-layered with dense clayey sands. The average undrained shear strength of the soils underlying Bay Mud was estimated to be more than 3000 psf.

For more information regarding the subsurface materials, we refer you to Plates 9-A through 11-D, Boring Logs, and CPT data in Appendix A.

6.2 Groundwater

Free groundwater was initially encountered at about 9 feet in one of the borings and it stabilized to approximately 6 feet below ground surface. The CPT logs indicate the groundwater table to be as high as about 3 feet below the existing ground surface. The groundwater elevation will be influenced by the tidal elevations in the adjacent Seal Slough that connects with the San Francisco Bay.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 General

Based on the subsurface exploration conducted at the subject site and the results obtained from our laboratory testing program, it is our opinion that the proposed project is feasible from a geotechnical engineering standpoint provided the recommendations presented in this report are incorporated into the project design and construction. When the final development plans become available, they should be submitted to our office for review prior to construction to evaluate that the intent of our recommendations is reflected in the plans, and to assess that our recommendations properly address the proposed project in its final form. BAGG should be allowed to design and observe the indicator pile installation program prior to installation of production piles on the site.

A review of geologic maps, previous geotechnical engineering reports prepared by others, and the results of subsurface exploration and laboratory-testing performed as a part of this investigation, it is our opinion the proposed parking garage structure should be supported on a deep foundation system. If the proposed parking structure is supported on shallow foundations, it will undergo excessive post-construction settlements because the anticipated column loads are relatively high, and the underlying Bay Mud soils are soft and compressible. Because of the presence of groundwater at shallow depths, and the relatively thick layer of soft Bay Mud, it is our opinion that drilled, cast-in-place, reinforced concrete piers would be extremely cumbersome to construct. We recommend that the parking garage should be supported on pre-

cast, pre-stressed, reinforced concrete piles or auger-cast piles. Our recommendations for these foundation systems are presented in the following sections of the report.

The site could experience very strong ground shaking from future earthquakes during the anticipated lifetime of the project. The intensity of the ground shaking will depend on the magnitude of the earthquake, distance to the epicenter, and the response characteristics of the on-site soils and bedrock. While it is not possible to totally preclude damage to structures during major earthquakes, strict adherence to good engineering design and construction practices will help reduce the risk of damage. The latest Uniform Building Code defines the minimum acceptable engineering design and construction practices.

7.2 Site Grading

Site grading will consist of excavation for the parking garage floor slab and foundations, preparation of the lower slab subgrade area, placement of backfill behind the perimeter retaining walls, backfilling of utility trenches, and miscellaneous minor ground surface changes to improve surface drainage.

As used in this report, the term "compact" and its derivatives mean that all on-site soils or imported non-expansive soils should be compacted in lifts, not exceeding 8-inches in loose thickness, to a minimum of 90 percent of maximum dry density, as determined by ASTM Test Method D1557-07, while at moisture content that is slightly wet of optimum. Areas to receive fill should be scarified and moisture conditioned to a minimum depth of 8-inches prior to the placement of fill. The excavated fill material could be reused as engineered fill, provided it is broken into material with a maximum particle size of 3-inches and is moisture conditioned uniformly to a moisture content that is wet of optimum.

It is likely that groundwater will be encountered in the excavations exceeding 3 feet in depth. Therefore, the contractor should be prepared to install engineering measures necessary to keep the excavation area(s) relatively dry. Due to the presence of soft, compressible, and excessively wet soils at shallow depths, the bottom of the excavation may also need to be stabilized to

minimize ground deflections under the loads imposed by relatively heavy construction equipment.

All aspects of site grading including clearing/stripping, demolition, foundation excavation, and placement of fills or backfills should be performed under the observation of BAGG's field representatives.

It must be the Contractor's responsibility to select equipment and procedures that will accomplish the grading as described above. The Contractor must also organize his work in such a manner that our field representative(s) can observe and test the grading operations, including clearing, excavation, compaction of fill and backfill, and compaction of subgrades.

7.3 2013 California Building Code Site Characterization

The Structural Engineering Design Provisions of the California Building Code (CBC) have been revised in recent years to reflect the changing knowledge regarding shaking from major earthquakes. The new code uses mapped spectral acceleration values for periods of 0.2 and 1.0 seconds, to better represent the probabilistic shaking that can be expected for various types of structures at a given site. The "mapped" values generally represent "bedrock" shaking with a 2 percent probability of being exceeded in a 50-year period. The values are then modified for site-specific use based on a broad classification of the soil profile at the site. Using the site coordinates of 37.5564° North Latitude and 122.2876° West Longitude, and the U.S. Seismic Design Maps by USGS (http://geohazards.usgs.gov/designmaps/us/application.php), earthquake ground motion parameters were computed in accordance with the 2013 California Building Code and are listed in the following table.

TABLE 2

Parameters for Seismic Design

2013 CBC Site Parameter	Value
Site Latitude	37.5564° N
Site Longitude	122.2876° W
Site Class, Chapter 20 of ASCE 7	Class E
Risk Category	I/II/III

Mapped Spectral Acceleration for Short Period S_s	1.710g
Mapped Spectral Acceleration for 1-second Period S_1	0.789g
Site Coefficient F_a	0.90
Site Coefficient F_v	2.40
Site-Modified Spectral Acceleration for short Period S_{MS}	1.539g
Site-Modified Spectral Acceleration for 1-second Period S_{M1}	1.894g
Design Spectral Acceleration for short Period S_{DS}	1.026g
Design Spectral Acceleration for 1-second Period S_{D1}	1.263g

7.4 Foundations

As discussed above, the proposed garage structure should be supported on a driven pile foundation or auger-cast piles. Lightly loaded ancillary structures at the ground surface; however, can be supported on shallow footings. Recommendations for the foundation types are presented below.

7.4.1 Precast Concrete Piles

Because of the presence of over-consolidated bay mud, we anticipate that the piles will not experience any down drag due to long term settlement of the bay mud. However, because of the nature of the soft, compressible Bay Mud layer, piles will primarily derive their load bearing capacity through skin friction between the surface of the pile and the soils underlying the Bay Mud layer. The preliminary estimated axial load bearing capacity of a 14-inch square precast concrete pile with depth is shown on Plate 12. These load-bearing capacities shown on the chart are based on a safety factor of 2 for skin friction and a safety factor of 3 for tip resistance. Since the piles are likely going to be longer than 85 feet, the total estimated load bearing capacity (skin friction plus tip resistance) are only shown for pile lengths exceeding 85 feet. The preliminary estimated load bearing capacities should be confirmed through an indicator pile program as outlined in the following sections of the report. The uplift load bearing capacities of the piles may be assumed to be 80 percent of the downward load bearing capacity. The piles should be spaced a minimum of 3 pile diameters apart. The parking garage slab should be designed as a structural slab supported on the driven pile foundations.

Resistance to lateral loads would be provided by passive pressure against the sides of the foundation members (piles, pile caps, and grade beams). The deflection versus lateral load for a single 14-inch square concrete pile and 18-inch diameter APG piles, under free-head conditions, for various lateral loads is presented in Appendix B. The lateral loads were applied at the top of the pile, using the computer program LPILEP version 6.0. The charts presented herein are based on a safety factor of 1.0. The structural analysis will likely indicate that pile groups may be needed at column locations to support the anticipated loads. If it is desired to analyze the load bearing and/or lateral capacity of a pile group, please contact our office.

The pile foundation system will derive lateral resistance through passive pressures acting against the grade beams and the pile caps. The passive resistance against grade beam and pile caps may be calculated using an equivalent fluid pressure of 200 pcf.

Piles should be driven continuously, without interruption, to the maximum depth required using the pile hammer approved by the Geotechnical Engineer. The suitability of the pile-driving hammer will be evaluated during the indicator pile program.

7.4.2 Auger Pressure Grouted Piles

In order to avoid excessive noise and vibrations associated with driven piles, along with complications associated with transportation, storage and handling of long precast concrete piles, auger pressure grouted (APG) piles may be used at the site to provide the necessary foundation support. APG piles are generally designed by design-build contractors who generally require the geotechnical firms to provide them information about subsurface conditions and perform in-house design to calculate capacities. The geotechnical firms review their design, provide quality assurance services during the installation of piles and confirm the actual load bearing capacity of the piles through pile load tests performed by the pile contractor.

Based on a review of the subsurface soil conditions revealed by our borings and CPTs, and the results of our laboratory tests, BAGG performed engineering analysis to estimate vertical capacities of 18-inch diameter APG piles (most commonly used pile diameter in the San Francisco Bay Area) proposed to support the parking structure loads. The results of our analyses are shown on Plate 12. The capacities shown on Plate 12 are based on a safety factor of 2 for skin friction and a safety factor of 3 for tip resistance. Since the piles are likely going to be longer than 85 feet, the total estimated load bearing capacity (skin friction plus tip resistance) are only shown for pile lengths exceeding 85 feet.

The vertical capacities of the piles should be confirmed in the field by performing two pile load tests on piles installed using procedures very similar to those proposed for actual piles. One pile could be tested for the downward capacity and another could be tested for its capacity for uplift loads. In the San Francisco Bay Area, two specialty contractors (Malcolm Drilling and Berkel and Company) install APG piles using slightly different and patented installation procedures. We recommend that both of these firms should be contacted to provide cost estimates for installing piles for the pile load capacities, pile load tests to confirm the load bearing capacities, and mobilizing and demobilizing for indicator as well as the production piles. BAGG should be allowed an opportunity to review the bids and provide engineering input related to the bid evaluation process.

7.4.3 Indicator Pile Program

The purposes of driving/installing indicator piles are to: 1) assist the Geotechnical Engineer in the evaluation of the pile installation equipment, and 2) obtain better estimates of the actual load bearing capacities of the piles. In case of driven piles, it would be important to estimate the stresses in the foundation piles during the handling and driving process, and confirming the load bearing capacity of the pile. Therefore, the foundation contractor should drive the indicator piles with the same equipment as the one proposed to be used for the production piles. The final lengths of the production piles will be determined after the indicator piles are driven and the pile

driving records are evaluated. The indicator piles for the driven piles should be a minimum of 10 feet longer than the anticipated lengths of the production piles.

Two weeks prior to the installation of the indicator piles, the contractor should provide specifications on the equipment proposed to be used at the site. The Contractor's equipment must be capable of driving the piles to the specified ultimate capacity and tip elevation without overstressing the driven piles as determined by a wave equation analysis. Prior to commencing the production pile driving operations, it is recommended that a minimum of 5 indicator piles be installed. The indicator piles determined to be competent after the driving operation may be used as permanent structural piles.

The vertical load bearing capacity of the indicator piles may be estimated using dynamic monitoring or performing a pile load tests. Each indicator pile should be monitored with the pile driving analyzer (PDA) system, and the data should be further analyzed using the CAPWAP computer program. In case some indicator piles show lower bearing capacities than anticipated, we recommend that the piles be restruck 7 days after initial driving. If the indicator piles are to be used as permanent piles, the locations of the permanent piles should be pre-surveyed and marked clearly on the building pad. The indicator pile-driving program should be observed by the Geotechnical Engineer.

In case of APG piles, it would be important to install the piles using the same equipment, the same concrete mix, the same rebar cage configuration and size, and the same QA/QC procedures as the ones proposed for production pile installations. The actual load bearing capacity of the piles should be estimated by performing pile load tests. The contractor should submit pile load testing locations and procedures for BAGG's review and approval prior to performing the tests. BAGG should be allowed an opportunity to observe the installation of the piles and monitor the pile load test.

7.4.4 Conventional Footings

Shallow spread footings may be used to support minor compressive loads near the existing ground surface. These footings should be founded a minimum of 24-inches below the lowest adjacent soil grade. Spread footings founded less than 3 feet below the existing ground surface may be designed using an allowable bearing pressure of 1500 psf for dead plus live loads. These values may be increased by one-third to include short-term seismic and wind loads.

A friction value of 0.3 may be assumed between the bottom of the shallow concrete spread footings and the underlying soil in designing the footing against sliding. For design purposes the passive pressure may be assumed to be 250 pcf, equivalent fluid weight. The passive resistance from the top one-foot of embedment into the soils should be discounted when calculating the passive resistance, where no pavement abuts the foundations. If concrete or asphalt pavement abuts the foundation, no discount depth will be necessary when calculating the passive resistance. Both base friction and lateral passive resistance may be used without reduction for footings founded less than 3 feet below the existing ground surface.

7.5 Retaining Walls

Retaining walls should be designed to resist lateral earth pressures from adjoining natural materials and backfills. Freestanding walls supporting fill material should be designed to support an equivalent fluid pressure of 45 pcf. Restrained walls supporting native soils or compacted fill soils should be designed to resist active lateral pressures taken as an equivalent fluid pressure of 60 pounds per cubic foot (pcf) for level backfill. These pressures should be increased by 3 pcf for every 5 degree increase in backfill slope.

Retaining walls should be supported on foundations designed in accordance with the recommendations presented in the "Foundations" section of this report.

The above lateral pressures do not include any hydrostatic pressures resulting from groundwater, seepage water, or infiltration of natural rainfall and/or irrigation water behind the walls. Therefore, all walls over 3 feet in height, should be provided with a drainage blanket behind the wall. The drainage blanket should consist of a pre-manufactured drainage panel or a one-foot thick blanket of free-draining gravel or drain rock protected by a suitable filter fabric. A 12-inch cap of relatively impermeable soil should be compacted at the top of the drainage blanket to minimize infiltration of surface water. A perforated pipe should be installed at the base of the drainage blanket to conduct water away from the wall.

7.6 Parking Structure Floor Slab

The floor slab for the parking garage should be designed as a structural slab resting on grade beams spanning between the piles.

7.7 Slabs-on-Grade and Exterior Flatwork

It is recommended that the exterior slabs and flatwork be underlain by a 6-inch layer of Class II Aggregate Base (minimum R-value = 78) layer compacted to at least 95 percent of maximum dry density.

Where new driveways and exterior slabs are constructed adjacent to irrigated landscape areas, or where natural runoff drains toward the pavement areas, a vertical curb extending at least 6 inches below the aggregate base and into the subgrade soils would minimize water intrusion into the subgrade soils and maximize the serviceable life of the pavement or slabs.

7.8 Drainage

Drainage measures to control and collect surface run-off should be considered an integral part of the proposed development. The ground surface adjacent to all sides of the proposed garage structure should be sloped to drain away from the foundations. Unpaved and landscaped areas should slope at least 2 percent to a distance of 5 feet away from the face of the garage structure.

Any area where surface run-off becomes concentrated should be provided with a catch basin that drains to a suitable discharge point. The surface water flowing down the garage ramp should also be collected and drained/pumped away from the structure.

A solid pipe should be installed to carry runoff from the roof gutter away from the parking garage. Surface and subsurface drainage facilities and catchment areas should be checked frequently and cleaned or maintained throughout the project life, as necessary.

7.9 Utility Trench Backfill

Vertical trenches deeper than 4 feet will require temporary shoring. Where shoring is not used, the sides should be sloped or benched, with a maximum slope of 1:1 (horizontal: vertical). The trench spoils should not be placed closer than 3 feet (or one-half of the trench depth) from the trench sidewalls. All work associated with trenching must conform to the State of California, Division of Industrial Safety requirements. In our opinion, the soils at the upper 10 to 15 feet of the site should be classified as "Type B" Soil, except, below water it is Type "C" soil.

The utility lines should be properly bedded and shaded with granular material, such as, sand or pea gravel. As a general rule, the bedding layer should be at least 4 inches thick and the utility lines should be shaded with the granular materials to a minimum of 4 inches above the utility line. The bedding and shading layers should be compacted using a vibratory compactor before subsequent backfill is placed. The contractor should use extreme caution with the vibratory compactor on the shading layer because excessive vibrations and/or imbalanced shading materials could result in dislodging the pipe and loosening the joints.

Trench backfill materials and compaction should conform to the following:

- In general, soils used for trench backfill shall be free of debris, roots and other organic matter, debris, and rocks or lumps exceeding 3 inches in greatest dimension.
- Compaction shall be performed to a minimum of 90% relative compaction in accordance with ASTM D1557-02, at a moisture content recommended

previously. In pavement areas, the upper 24 inches of the backfill (below the pavement subgrade) shall be compacted to 95% of the maximum dry density.

- Alternatively, the utility trenches may be backfilled with flowable fill (a cementitious slurry consisting of a mixture of fine aggregate or filler, water, and cementitious material(s)) capable of filling all voids in irregular excavations and hard to reach places. The flowable fill is self- leveling material that hardens in a matter of a few hours without the need for compaction in layers. Flowable fill is sometimes referred to as controlled density fill (CDF), controlled low strength material (CLSM), and lean concrete slurry. A 1- to 2-sack flowable fill material is usually considered to be acceptable for the subject project.

7.10 Plan Review

It is recommended that the Geotechnical Engineer (BAGG Engineers) be retained to review the final grading, foundation, and drainage plans. This review is to assess general suitability of the earthwork, foundation, and drainage recommendations contained in this report and to verify the appropriate implementation of our recommendations into the project plans and specifications.

7.11 Observation and Testing

It is recommended that the Geotechnical Engineer (BAGG Engineers) be retained to provide observation and testing services during the grading, excavation, backfilling, and foundation construction phases of work. This is intended to verify that the work in the field is performed as recommended and in accordance with the approved plans and specifications, as well as verify that subsurface conditions encountered during construction are similar to those anticipated during the design phase. Changed or unanticipated conditions may warrant revised recommendations. Therefore, BAGG cannot accept responsibility or liability for the recommendations presented in this report if we are not given the opportunity to provide observation and testing services during construction.

8.0 CLOSURE

This report has been prepared in accordance with generally-accepted engineering practices for the strict use of Century Centre Investors, and other professionals associated with the specific project described in this report. The recommendations presented in this report are based on our understanding of the proposed construction as described herein, and upon the soil conditions encountered in our exploratory borings and CPTs advanced for our previous investigation.

The conclusions and recommendations contained in this report are based on subsurface conditions revealed by three (3) widely spaced borings and three (3) CPTs. It is not uncommon for unanticipated conditions to be encountered during site grading and/or foundation installation and it is not possible for all such variations to be found by a field exploration program appropriate for this type of project. The recommendations contained in this report are therefore contingent upon the review of the final grading, drainage, and foundation plans by this office, and upon geotechnical observation and testing by BAGG of all pertinent aspects of site grading including demolition, placement of fills and backfills, and foundation construction, including the indicator pile program and pile load tests.

Soil conditions and standards of practice change with time. Therefore, we should be consulted to update this report, if the construction does not commence within 18 months from the date this report is submitted. Additionally, the recommendations of this report are only valid for the proposed development as described herein. If the proposed project is modified, our recommendations should be reviewed and approved or modified by this office in writing.

The following references and plates are attached and complete this report:

Plate 1	Vicinity Map
Plate 2	Site Plan
Plate 3	Regional Geologic Map
Plate 4	Regional Fault Map
Plate 5	Unified Soil Classification System
Plate 6	Soil Terminology
Plate 7A&B	Key to Symbols

Plate 8-10	Boring Logs
Plate 11	Atterberg Limits
Plate 12	Vertical Load Bearing Capacity of Piles
Appendix A	Boring and CPT Logs
Appendix B	Lateral Loading Behavior of 14-inch Precast Concrete Piles and 18-inch Diameter APG Piles

ASFE document titled "Important Information About Your Geotechnical Engineering Report"

9.0 REFERENCES

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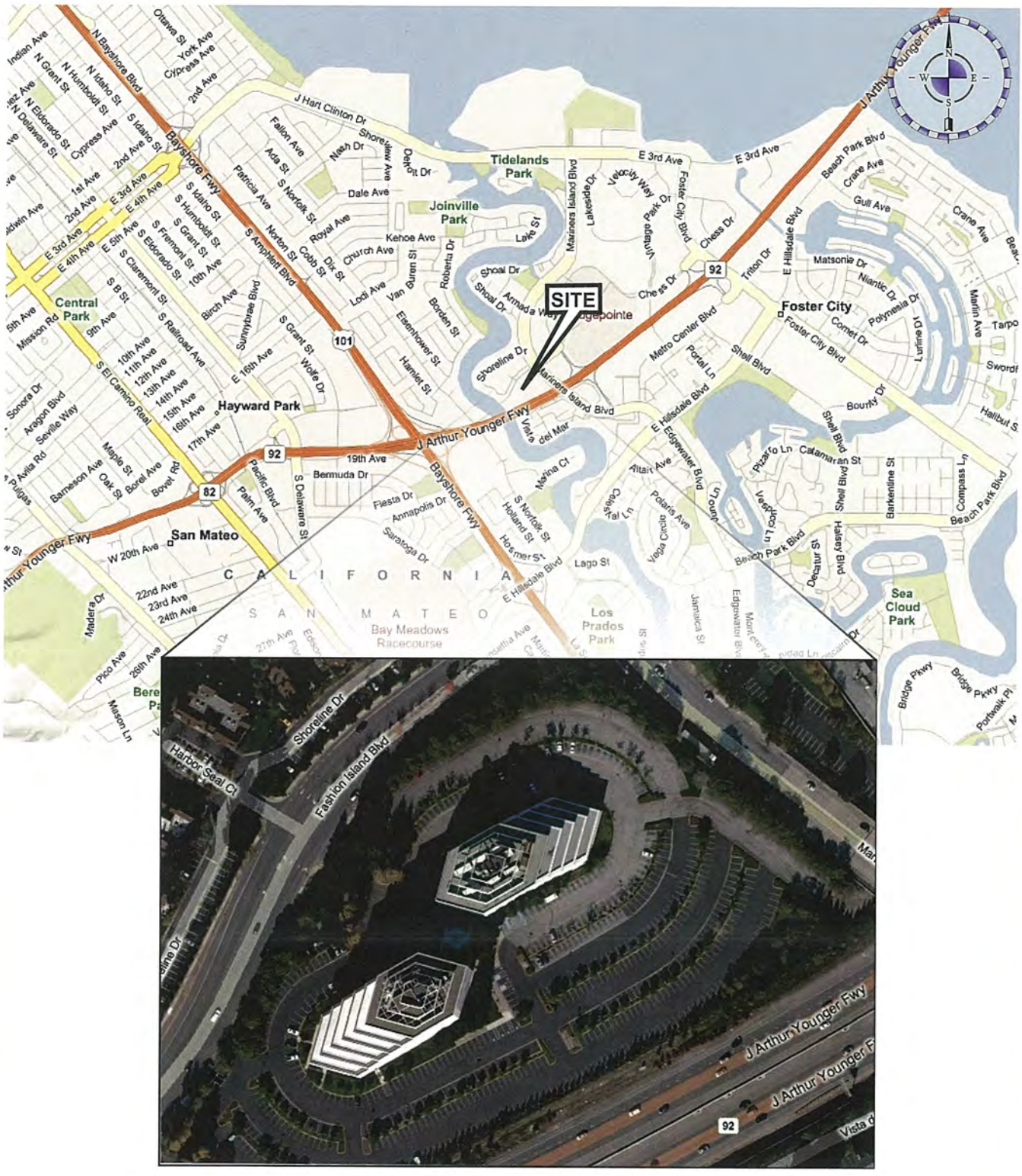
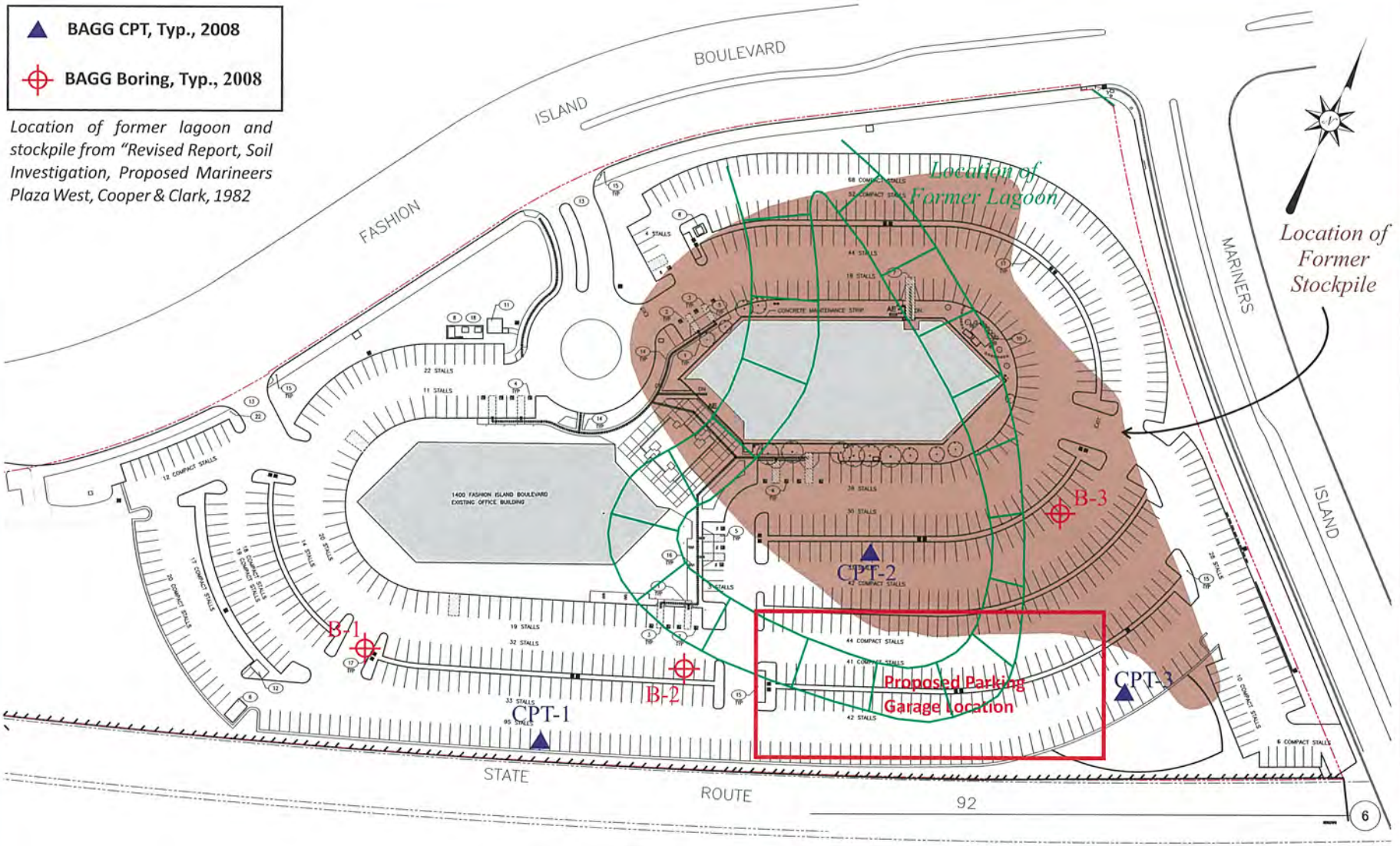


GEOTECHNICAL ENGINEERING STUDY
PROPOSED PARKING GARAGE
1450 FASHION ISLAND BOULEVARD
SAN MATEO, CALIFORNIA

VICINITY MAP

DATE:	JOB NUMBER:	PLATE
SEPT. 2014	FISHE-01-01	1

BAGG ENGINEERS

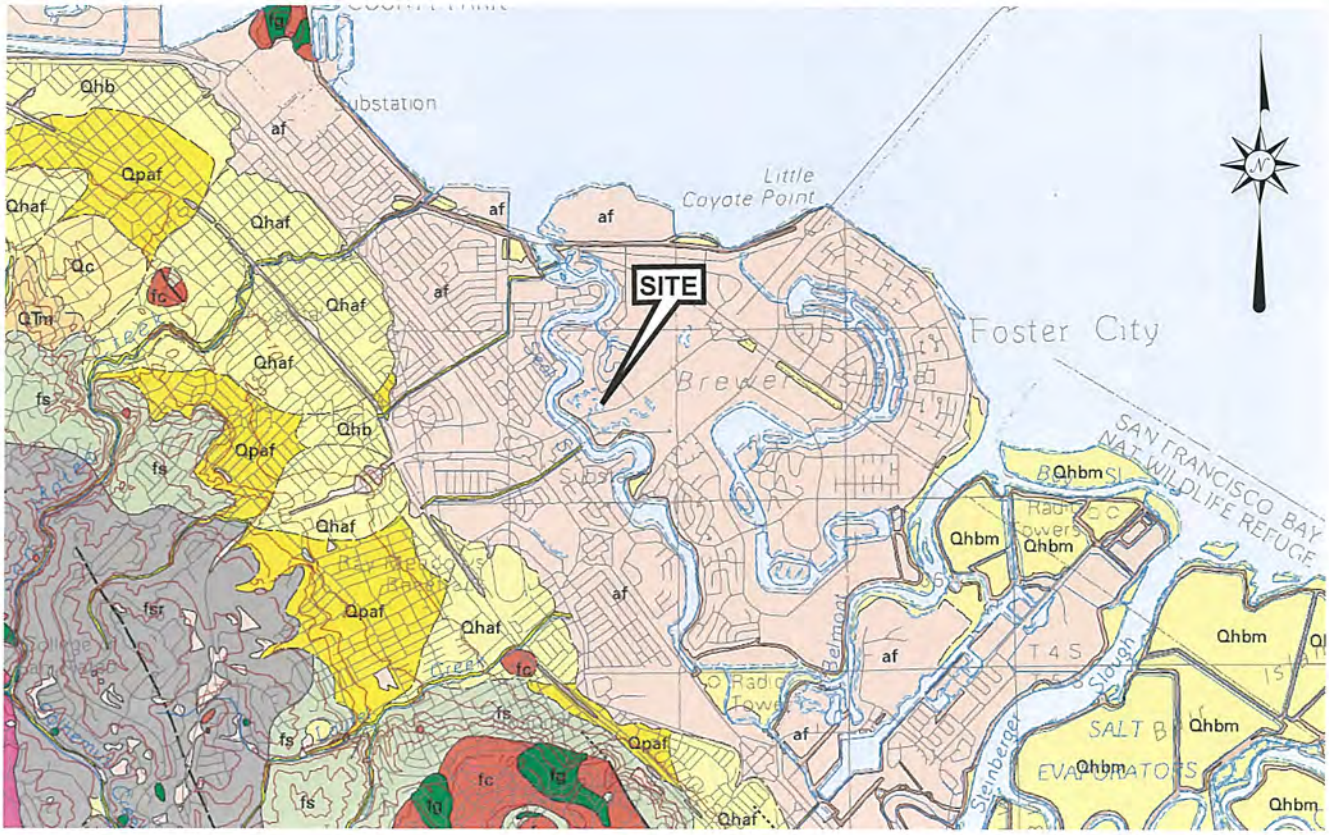


GEOTECHNICAL ENGINEERING STUDY
PROPOSED PARKING GARAGE
1450 FASHION ISLAND BOULEVARD
SAN MATEO, CALIFORNIA

BAGG ENGINEERS

SITE PLAN

DATE:	JOB NUMBER:	PLATE
Sept. 2014	FISHE-01-01	2



LEGEND

af **Artificial fill (Historic)**

Loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations. Thickness is variable and may exceed 30 m in places. Some is compacted and quite firm, but fill made before 1965 is nearly everywhere not compacted and consists simply of dumped materials

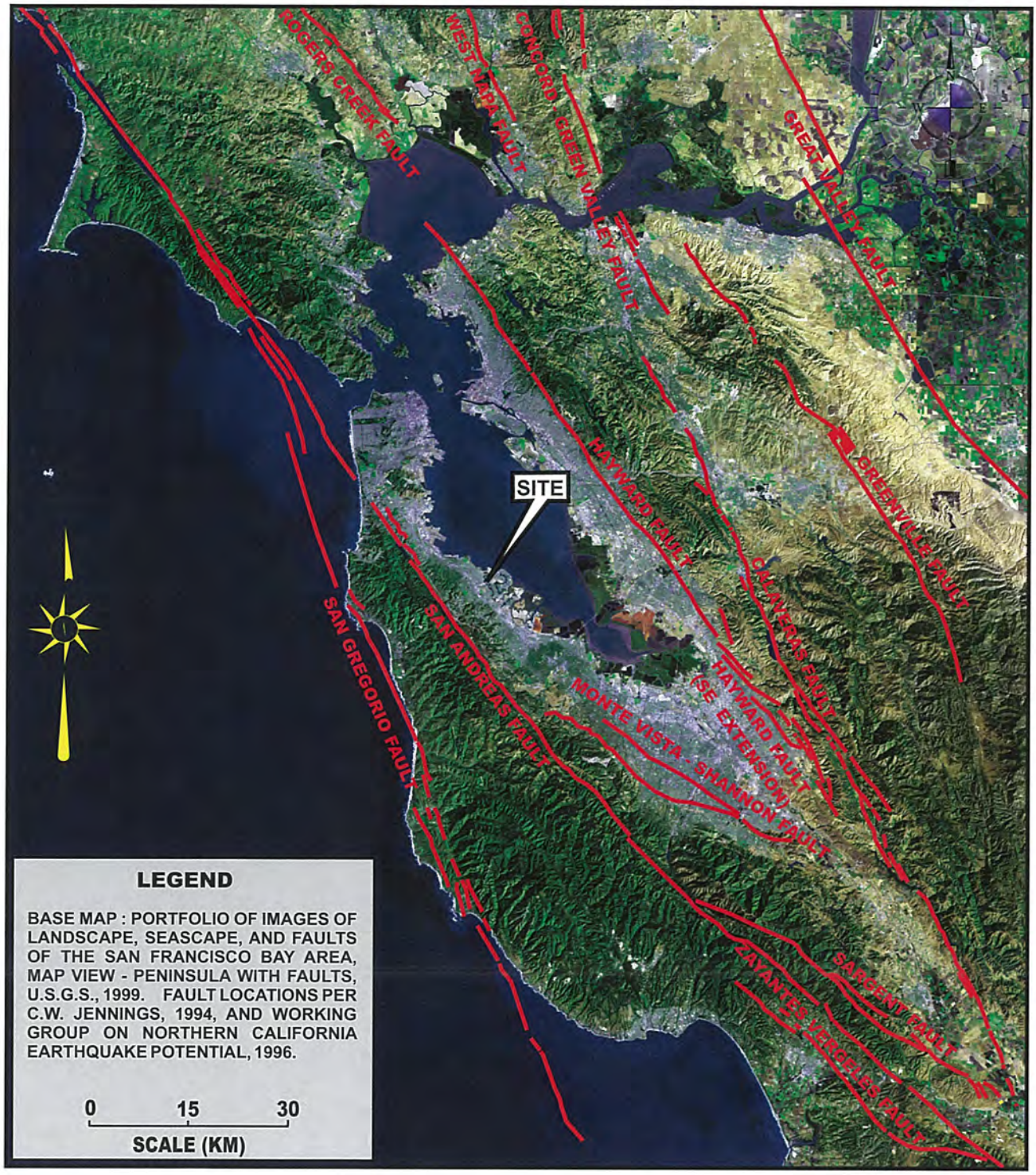
Qhaf Alluvial fan and fluvial deposits (Holocene)

Alluvial fan deposits are brown or tan, medium dense to dense, gravely sand or sandy gravel that generally grades upward to sandy or silty clay. Near the distal fan edges, the fluvial deposits are typically brown, never reddish, medium dense sand that fines upward to sandy or silty clay

Reference: *Brabb, E.E., and Pampeyan, E.H., 1972, Preliminary Geologic Map of San Mateo County, California: U.S. Geological Survey Miscellaneous Field Studies, Map MF-328.*

GEOTECHNICAL ENGINEERING STUDY PROPOSED PARKING GARAGE 1450 FASHION ISLAND BOULEVARD SAN MATEO, CALIFORNIA	REGIONAL GEOLOGIC MAP		
	DATE: Sept. 2014	JOB NUMBER: FISHE-01-01	PLATE 3

BAGG ENGINEERS



GEOTECHNICAL ENGINEERING STUDY
PROPOSED PARKING GARAGE
1450 FASHION ISLAND BOULEVARD
SAN MATEO, CALIFORNIA

REGIONAL FAULT MAP

DATE:	JOB NUMBER:	PLATE
SEPT. 2014	FISHE-01-01	4

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COARSE-GRAINED SOILS

LESS THAN 50% FINES*

GROUP SYMBOLS	ILLUSTRATIVE GROUP NAMES	MAJOR DIVISIONS
GW	Well graded gravel Well graded gravel with sand	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size
GP	Poorly graded gravel Poorly graded gravel with sand	
GM	Silty gravel Silty gravel with sand	
GC	Clayey gravel Clayey gravel with sand	
SW	Well graded sand Well graded sand with gravel	SANDS More than half of coarse fraction is smaller than No. 4 sieve size
SP	Poorly graded sand Poorly graded sand with gravel	
SM	Silty sand Silty sand with gravel	
SC	Clayey sand Clayey sand with gravel	

NOTE: Coarse-grained soils receive dual symbols if:
(1) their fines are CL-ML (e.g. SC-SM or GC-GM) or
(2) they contain 5-12% fines (e.g. SW-SM, GP-GC, etc.)

FINE-GRAINED SOILS

MORE THAN 50% FINES*

GROUP SYMBOLS	ILLUSTRATIVE GROUP NAMES	MAJOR DIVISIONS
CL	Lean clay Sandy lean clay with gravel	SILTS AND CLAYS liquid limit less than 50
ML	Silt Sandy silt with gravel	
OL	Organic clay Sandy organic clay with gravel	
CH	Fat clay Sandy fat clay with gravel	SILTS AND CLAYS liquid limit more than 50
MH	Elastic silt Sandy elastic silt with gravel	
OH	Organic clay Sandy organic clay with gravel	
PT	Peat Highly organic silt	HIGHLY ORGANIC SOIL

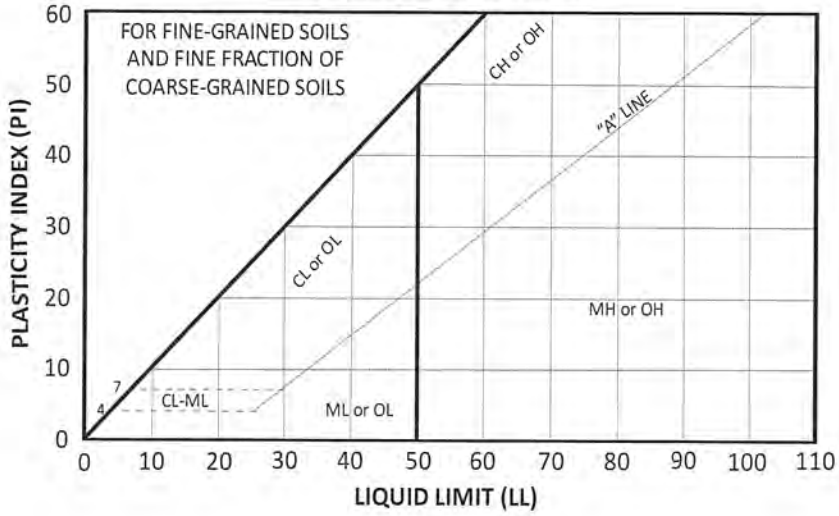
NOTE: Fine-grained soils receive dual symbols if their limits in the hatched zone on the Plasticity Chart(L-M)

SOIL SIZES

COMPONENT	SIZE RANGE
BOULDERS	ABOVE 12 in.
COBBLES	3 in. to 12 in.
GRAVEL	No. 4 to 3 in.
Coarse	¾ in to 3 in.
Fine	No. 4 to ¾ in.
SAND	No. 200 to No.4
Coarse	No. 10 to No. 4
Medium	No. 40 to No. 10
Fine	No. 200 to No. 40
*FINES:	BELOW No. 200

NOTE: Classification is based on the portion of a sample that passes the 3-inch sieve.

PLASTICITY CHART



Reference: ASTM D 2487-06, Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System).

GENERAL NOTES: The tables list 30 out of a possible 110 Group Names, all of which are assigned to unique proportions of constituent soils. Flow charts in ASTM D 2487-06 aid assignment of the Group Names. Some general rules for fine grained soils are: less than 15% sand or gravel is not mentioned; 15% to 25% sand or gravel is termed "with sand" or "with gravel", and 30% to 49% sand or gravel is termed "sandy" or "gravelly". Some general rules for coarse-grained soils are: uniformly-graded or gap-graded soils are "Poorly" graded (SP or GP); 15% or more sand or gravel is termed "with sand" or "with gravel", 15% to 25% clay and silt is termed clayey and silty and any cobbles or boulders are termed "with cobbles" or "with boulders".

UNIFIED SOIL CLASSIFICATION SYSTEM

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SOIL TYPES (Ref 1)

Boulders: particles of rock that will not pass a 12-inch screen.
Cobbles: particles of rock that will pass a 12-inch screen, but not a 3-inch sieve.
Gravel: particles of rock that will pass a 3-inch sieve, but not a #4 sieve.
Sand: particles of rock that will pass a #4 sieve, but not a #200 sieve.
Silt: soil that will pass a #200 sieve, that is non-plastic or very slightly plastic, and that exhibits little or no strength when dry.
Clay: soil that will pass a #200 sieve, that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when dry.

MOISTURE AND DENSITY

Moisture Condition: an observational term; dry, moist, wet, or saturated.
Moisture Content: the weight of water in a sample divided by the weight of dry soil in the soil sample, expressed as a percentage.
Dry Density: the pounds of dry soil in a cubic foot of soil.

DESCRIPTORS OF CONSISTENCY (Ref 3)

Liquid Limit: the water content at which a soil that will pass a #40 sieve is on the boundary between exhibiting liquid and plastic characteristics. The consistency feels like soft butter.
Plastic Limit: the water content at which a soil that will pass a #40 sieve is on the boundary between exhibiting plastic and semi-solid characteristics. The consistency feels like stiff putty.
Plasticity Index: the difference between the liquid limit and the plastic limit, i.e. the range in water contents over which the soil is in a plastic state.

MEASURES OF CONSISTENCY OF COHESIVE SOILS (CLAYS) (Ref's 2 & 3)

Very Soft	N=0-1*	C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by strong finger pressure
Very stiff	N=16-30	C=2000-4000 psf	Dented slightly by finger pressure
Hard	N>30	C>4000 psf	Dented slightly by a pencil point

*N=blows per foot in the Standard Penetration Test. In cohesive soils, with the 3-inch-diameter ring sampler, 140-pound weight, divide the blow count by 1.2 to get N (Ref 4).

MEASURES OF RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS, AND SILTS) (Ref's 2 & 3)

Very Loose	N=0-4**	RD=0-30	Easily push a ½-inch reinforcing rod by hand
Loose	N=5-10	RD=30-50	Push a ½-inch reinforcing rod by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a ½-inch reinforcing rod
Dense	N=31-50	RD=70-90	Drive a ½-inch reinforcing rod 1 foot
Very Dense	N>50	RD=90-100	Drive a ½-inch reinforcing rod a few inches

**N=Blows per foot in the Standard Penetration Test. In granular soils, with the 3-inch-diameter ring sampler, 140-pound weight, divide the blow count by 2 to get N (Ref 4).

xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx

Ref 1: ASTM Designation: D 2487-06, **Standard Classification of Soils for Engineering Purposes** (Unified Soil Classification System).

Ref 2: Terzaghi, Karl, and Peck, Ralph B., **Soil Mechanics in Engineering Practice,** John Wiley & Sons, New York, 2nd Ed., 1967, pp. 30, 341, and 347.

Ref 3: Sowers, George F., **Introductory Soil Mechanics and Foundations: Geotechnical Engineering,** Macmillan Publishing Company, New York, 4th Ed., 1979, pp. 80, 81, and 312.

Ref 4: Lowe, John III, and Zaccheo, Phillip F., **Subsurface Explorations and Sampling,** Chapter 1 in "Foundation Engineering Handbook," Hsai-Yang Fang, Editor, Van Nostrand Reinhold Company, New York, 2nd Ed, 1991, p. 39.

SOIL TERMINOLOGY

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KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Paving
	Clayey gravel
	Borderline gravelly clay to clayey gravel
	Moderate plasticity clay
	High plasticity organic clay
	Clayey sand
	Poorly graded sand with clay
	Description not given for: "OSG"
	Description not given for: "5S"
	Description not given for: "DO:"
	Sandy lean clay
	Clayey sand and gravel
Misc. Symbols	
	Boring continues
	Water first encountered during drilling
Soil Samplers	
	Modified California Sampler: 2.375" ID by 3" OD, split-barrel sampler driven w/ 140-pound hammer falling 30 inches

Notes:

1. The borings were drilled on May 10, 2014 with a portable minuteman drilling rig using 4.5-inch diameter continuous flight augers.

2. Groundwater was encountered in all three borings at the depths shown on the boring logs.

3. Following completion of drilling, the borings were backfilled with neat cement grout per standard protocol.

4. The borings were located by pacing distances from landmarks shown on attached Site Plan. The plotted locations are only approximate.

5. The soils were visually classified in the field in accordance with Plate 5, Unified Soil Classification System. Terms for soil consistency, and general strenght correlations with blow counts are presented on Plate 6, Soil Terminology.

6. The boring logs depict BAGG's interpretation of subsurface conditions at the locations shown on the Site Plan and on the date noted on the boring log. The log is intended for use only in conjuction with this report and only for the purposes outlined herein.

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KEY TO SYMBOLS

Symbol	Description
Soil Samplers	
	No recovery
	Description not given for: "S"
	Standard Penetration Test: 1 3/8" ID by 2" OD, split-spoon sampler driven with 140-pound hammer falling 30" (ASTM D 1586-99)
Line Types	
_____	Denotes a sudden, or well identified strata change
_ _ _	Denotes a gradual, or poorly identified strata change
Laboratory Data	
DS	Direct shear test performed on a sample at natural moisture content. (ASTM D2166)
DSX	Direct shear test performed on a sample at artificially increased moisture content.

BAGG ENGINEERS

BORING LOG

Boring No. B-1
Page 1 of 4

JOB NAME: Proposed Parking Garage
CLIENT: 1450 Mariners Island LLC
LOCATION: 1450 Fashion Island Blvd, Foster City
DRILLER: Pitcher Drilling
DRILL METHOD: 5" Rotary Wash, Failing

JOB NO.: FISHE-01-00
DATE DRILLED: 12/5/07
ELEVATION:
LOGGED BY: MRB
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0			3" AC over 4" AB	
				6.8	124	4	15 13 13 10 7 8	CL	LEAN CLAY WITH GRAVEL, dark gray and dark gray brown, moist, stiff, medium plasticity, little sand	Fill
				50.4		8	6 5 4	CL/ CH	LEAN to FAT CLAY WITH CLAY, medium gray, moist, medium to high plasticity, trace fine sand, trace organcis	Residual Bay Mud with Fill
						12		OH	ORGANIC CLAY, medium gray, moist, very soft, medium to high plasticity, trace organics	Young Bay Deposits
				78.1	53	16	0			
						20	0 psi			Consol
						24	50			

BYGG ENGINEERS

BORING LOG

Boring No. B-1
Page 2 of 4

JOB NAME: Proposed Parking Garage

JOB NO.: FISHE-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
							psi			
				73.9	53	28	0		...shell fragments	LL=102 PI=66
						32				
				72.8	59		75 psi			
						36				
				71.8	58		0			
						40				
									...increase in organic content	
Nat.	3000	57.3	745	57.3	66	44	0			
						48		SC	CLAYEY SAND, medium gray, medium grained, miost, dense, fragments of sandstone	
				12.3	127		9 23 39			
						52		CL/CH	LEAN to FAT CLAY, mottled medium gray and yellow brown,	

BVGG ENGINEERS

BORING LOG

Boring No. B-1
Page 3 of 4

JOB NAME: Proposed Parking Garage

JOB NO.: FISHE-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
				25.8	96		13 15		moist, very stiff, medium to high plasticity, little silt	
						56				
				22.9	107		9 16 22		...gravel gravel	
						60				
				15.2	116	64	20 28 28	SP-SC	POORLY LEAN CLAY WITH CLAY, yellow brown, medium grained, saturated, dense, trace gravel	
						68		CL/CH	LEAN to FAT CLAY, mottled medium gray and yellow brown, moist, very stiff, medium to high plasticity, little silt	
				25.6	101.6		8 11 13			
						72				
TXUU	5000	Nat.	1550	45.6	79		6 8 11		...medium gray, iron oxide staining	
						76				
				48.6	73					
						80				

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BORING LOG

JOB NAME: Proposed Parking Garage

JOB NO.: FISHE-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						84			Boring terminated at 80½ feet. Groundwater was not measured, backfilled with neat cement grout.	
						88				
						92				
						96				
						100				
						104				
						108				

BAGG ENGINEERS

BORING LOG

Boring No. B-2
Page 1 of 4

JOB NAME: Proposed Parking Garage
CLIENT: 1450 Mariners Island LLC
LOCATION: 1450 Fashion Island Blvd, Foster City
DRILLER: Pitcher Drilling
DRILL METHOD: 5" Rotary Wash, Failing

JOB NO.: FISHE-01-00
DATE DRILLED: 12/6/07
ELEVATION:
LOGGED BY: MRB
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0			3" AC over 4" AB	
				8.0	133		3 11 21	CL	SANDY LEAN CLAY WITH GRAVEL, dark gray, moist, stiff, medium plasticity	Fill
				8.0	129	4	8 13 24			
				70.7	58.3	8	5 4 4	CL/ CH	LEAN to FAT CLAY WITH CLAY, medium gray, moist, medium to high plasticity, trace fine sand, trace organcis	Residual Bay Mud with Fill
						12		OH	ORGANIC CLAY, medium gray, moist, very soft, medium to high plasticity, trace organics	Young Bay Deposits
				69.9	55		50 psi			
						16				
						20	0			
						24	0			

BYGG ENGINEERS

BORING LOG

Boring No. B-2
Page 2 of 4

JOB NAME: Proposed Parking Garage

JOB NO.: FISHE-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						28				Consol
							50 psi			
						32				
							100 psi			
TXUU	2500	Nat.	215	72.2	56	36				
				70.3	58		0 0 1		...trace shell fragments	
						40				
									ORGANIC FAT CLAY with SAND, medium gray, moist, high plasticity, trace organics	
				19.2	108	44	0 1 5			
								SC	CLAYEY SAND, medium gray, fine to medium grained, saturated, medium dense	
						48				
				18.1	116		8 9 13			
								SP-SC	POORLY GRADED SAND with CLAY, yellow brown, fine to medium grained, moist,	
						52				

BVGG ENGINEERS

BORING LOG

Boring No. B-2
Page 3 of 4

JOB NAME: Proposed Parking Garage

JOB NO.: FISHE-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
					14.0	56	13 15 25		medium dense	
				12.8	123	60	8 15 12	CL/ SC	Interbedded CLAYEY SAND, POORLY GRADED SAND, and LEAN TO FAT CLAY, medium gray, saturated, medium dense/stiff, trace organics	
				19.7	99	64	11 21 28	CL/ CH	LEAN to FAT CLAY, mottled medium gray and yellow brown, moist, very stiff, medium to high plasticity, little silt	
						68	7 14 15		...increase in sand content	
						72			...medium gray, little silt, iron oxide staining	
						76	3 8 11			Consol
						80	1 5 6			

BYGG ENGINEERS

BORING LOG

JOB NAME: Proposed Parking Garage

JOB NO.: FISHE-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						84			Boring terminated at 80½ feet. Groundwater was not measured, backfilled with neat cement grout.	
						88				
						92				
						96				
						100				
						104				
						108				

BAGG ENGINEERS

BORING LOG

Boring No. B-3
Page 1 of 4

JOB NAME: Proposed Parking Garage
CLIENT: 1450 Mariners Island LLC
LOCATION: 1450 Fashion Island Blvd, Foster City
DRILLER: Pitcher Drilling
DRILL METHOD: 5" Rotary Wash, Failing

JOB NO.: FISHE-01-00
DATE DRILLED: 12/5/07
ELEVATION:
LOGGED BY: MRB
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0			3" AC over 4" AB	
								CL	SANDY LEAN CLAY WITH GRAVEL, dark gray, moist, stiff, medium plasticity	Fill
DS	1000	12.4	1030	6.8	122	4	10 8 7			
DS	2000	12.2	1960	12.4	122		3 5 8			
						8		CL/ CH	LEAN to FAT CLAY WITH CLAY, medium gray, moist, medium to high plasticity, trace fine sand, trace organcis	Residual Bay Mud with Fill
				53.6	72		14 16 12			
						12		OH	ORGANIC CLAY, medium gray, moist, very soft, medium to high plasticity, trace organics	Young Bay Deposits
						16				
						20	50 psi			
				80.5	54	24	1 0			

BVGG ENGINEERS

BORING LOG

Boring No. B-3
Page 2 of 4

JOB NAME: Proposed Parking Garage

JOB NO.: FISHE-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						28	1			
				79.9	53		50 psi			
						32				
				72.2	52		0		...trace shells	
						36				
							50 psi			Consol
						40				
				17.5	113	44	1 6 9	CL/ SC	SANDY LEAN CLAY to CLAYEY SAND, medium gray, saturated, loose/stiff, medium plasticity	Stopped drilling 12/ 5/07
						48			...mottled medium gray and yellow brown	Resumed Drilling 12/6/07
				20.5	101		11 14 15			
						52				

BVGG ENGINEERS

BORING LOG

JOB NAME: Proposed Parking Garage

JOB NO.: FISHE-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
				12.0	129	56	20 50-5"	SC	CLAYEY SAND WITH GRAVEL, yellow brown, moist, very dense, moist	
						60		CL/CH	LEAN to FAT CLAY, mottled medium gray and yellow brown, moist, very stiff, medium to high plasticity, little silt	slough
						64	8 13 17			
				47.5	80.8	68	8 11 14		...medium gray, little silt, iron oxide staining	
						72	4 4 7			
						76				
						80	7 7 10			

BVGG ENGINEERS

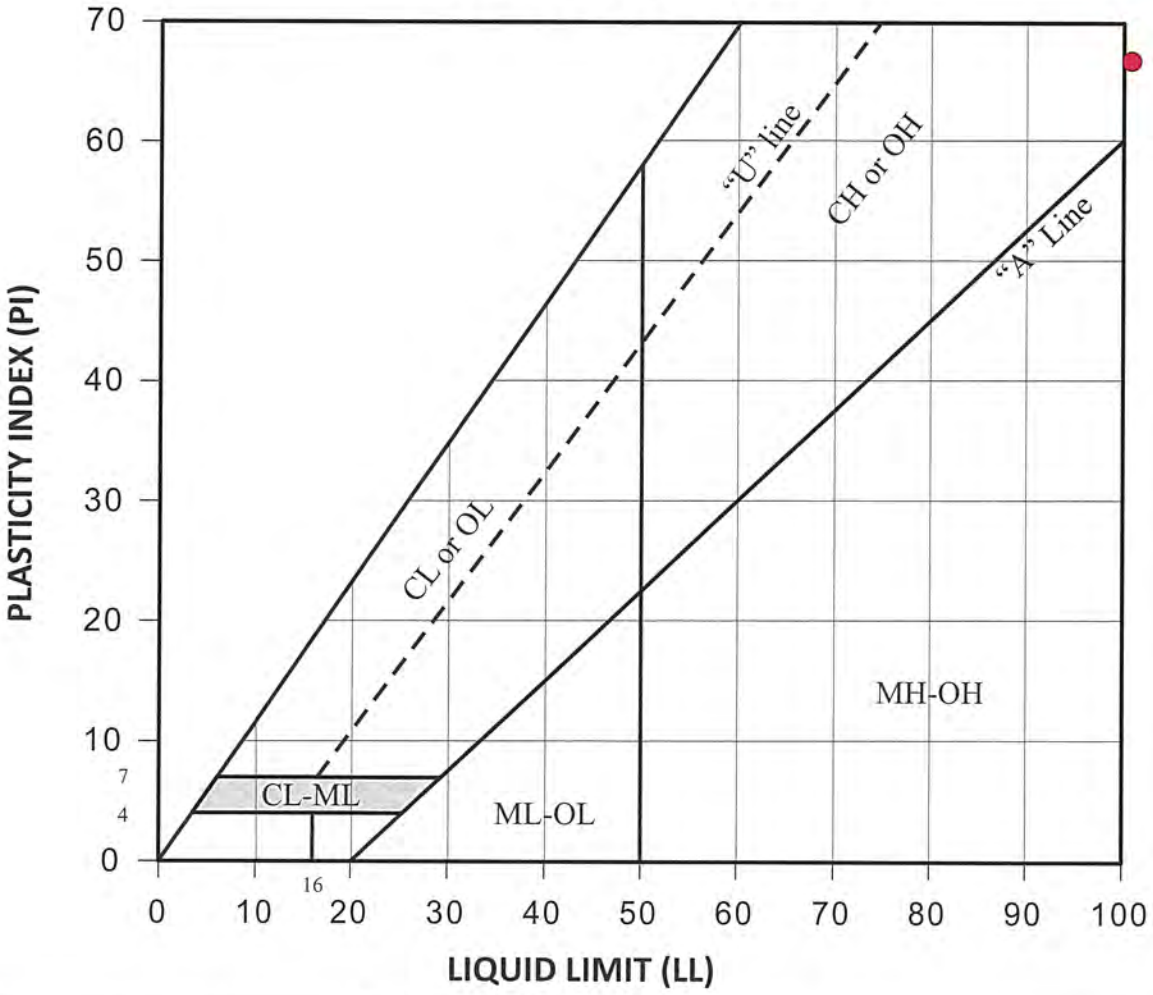
BORING LOG

JOB NAME: Proposed Parking Garage

JOB NO.: FISHE-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						84 88 92 96 100 104 108			Boring terminated at 80½ feet. Groundwater was measured at 6 feet, backfilled with neat cement grout.	

PLASTICITY CHART



SYMBOL	SAMPLE SOURCE	DEPTH (FEET)	NATURAL WATER CONTENT W[%]	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SAMPLE DESCRIPTION
●	B-1	30	74	102	36	66	Gray Brown Organic CLAY (OH) Bay Mud

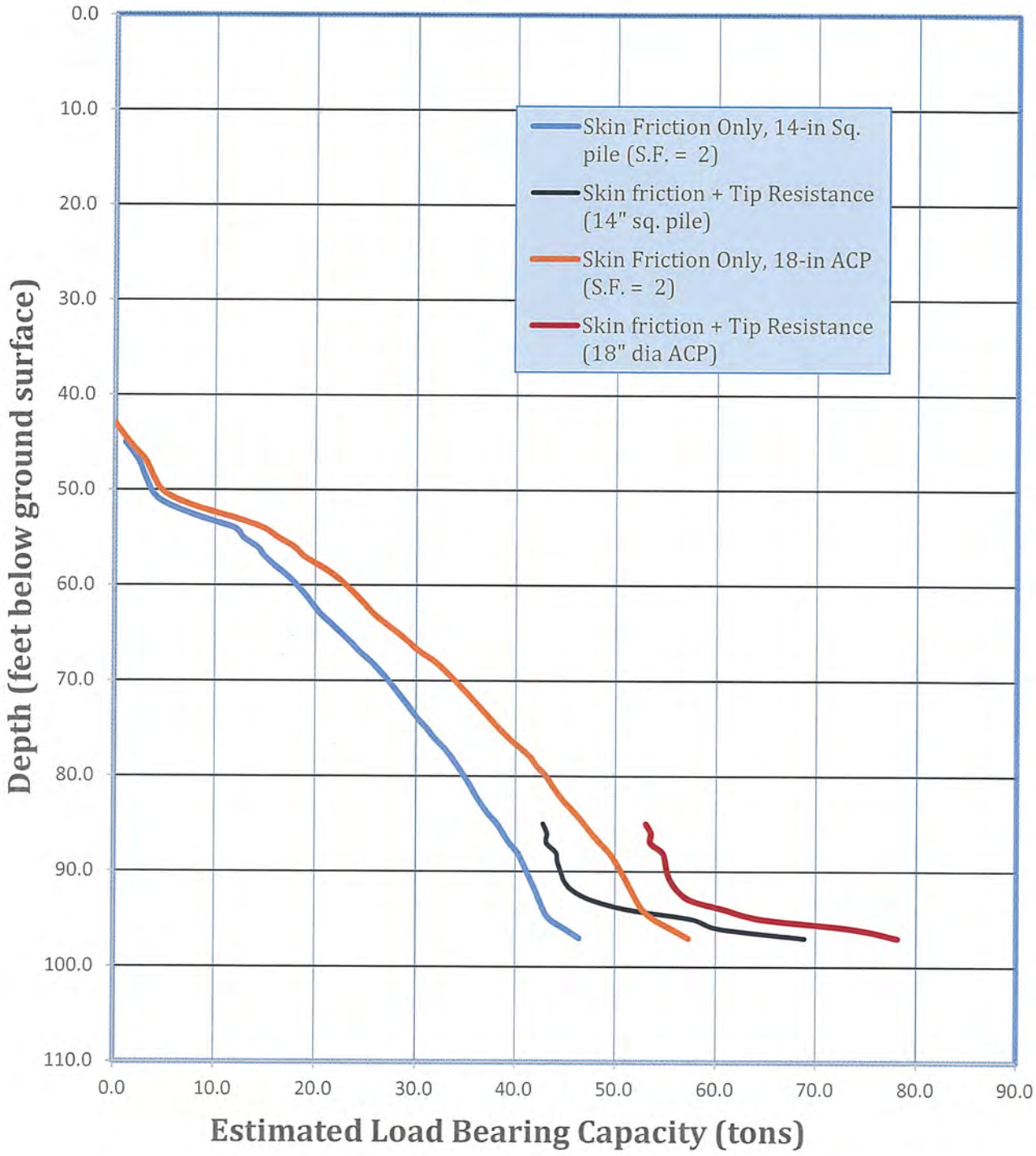
GEOTECHNICAL ENGINEERING STUDY
PROPOSED PARKING GARAGE
1450 FASHION ISLAND BOULEVARD
SAN MATEO, CALIFORNIA

PLASTICITY DATA

DATE:	JOB NUMBER:	PLATE
SEPT. 2014	FISHE-01-01	11

BAGG ENGINEERS

Load Bearing Capacities
14 inch Square Driven Pile and 18-Auger Cast Pile
Using CPT-3 Data



APPENDIX A

CPT LOGS

GREGG IN SITU, INC.

GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

December 12, 2007

Bay Area Geotechnical Group
Attn: Ajay Singh
847 W. Maude Ave.
Sunnyvale, California 94085

Subject: CPT Site Investigation
1450 Mariners Island Blvd.
San Mateo, California
GREGG Project Number: 07-372MA

Dear Mr. Singh:

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	UVIF Cone Penetration Tests	(UVIFCPTU)	☐
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

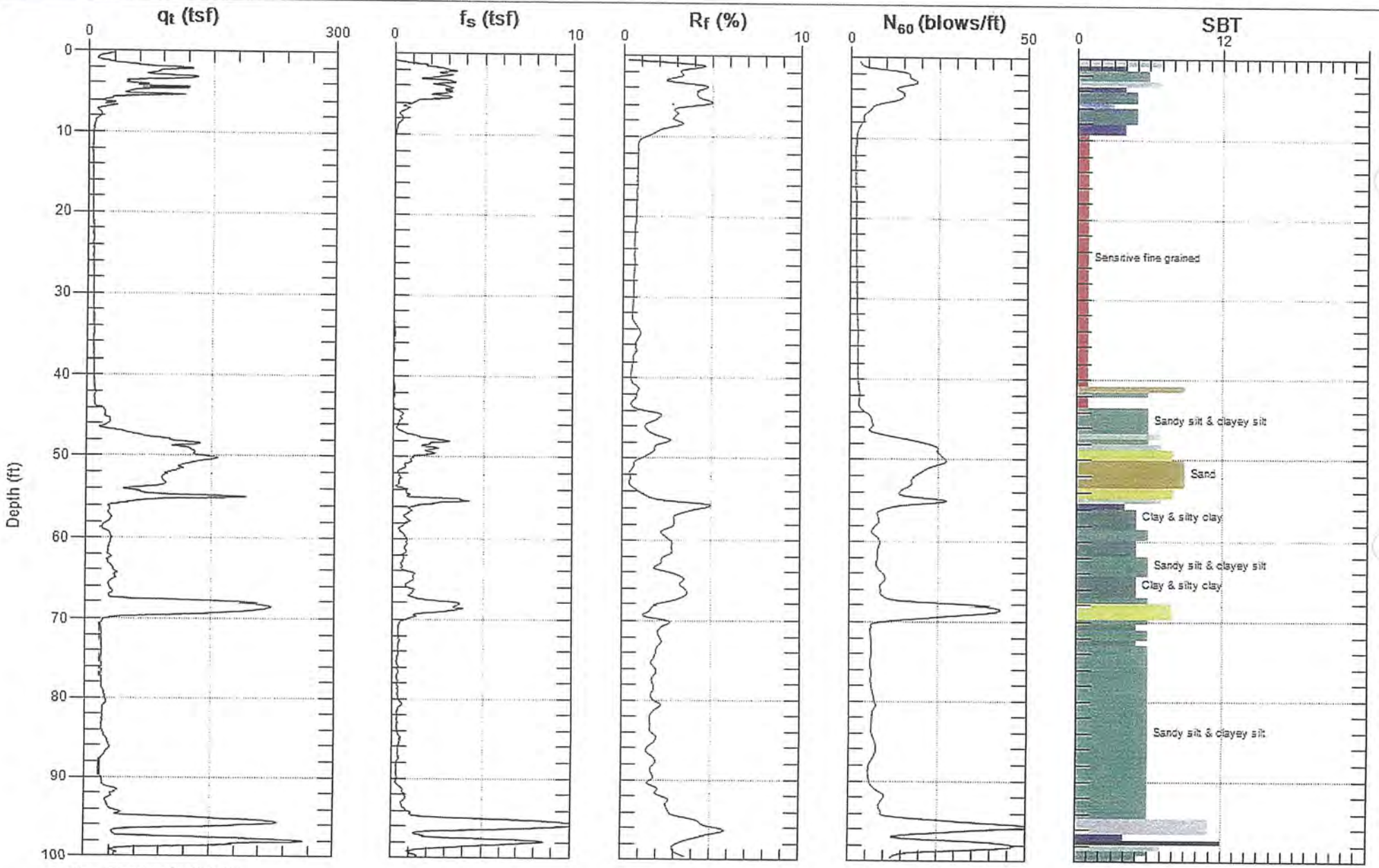
-Table 1-

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	12/11/07	100	-	-	48.7
CPT-02	12/11/07	100	-	-	90.7
CPT-03	12/11/07	100	-	-	53.1

BAY AREA GEOTECHNICAL

Site: 1450 MARINERS ISLAND Engineer: A.SINGH

Sounding: CPT-01 Date: 12/11/2007 08:32



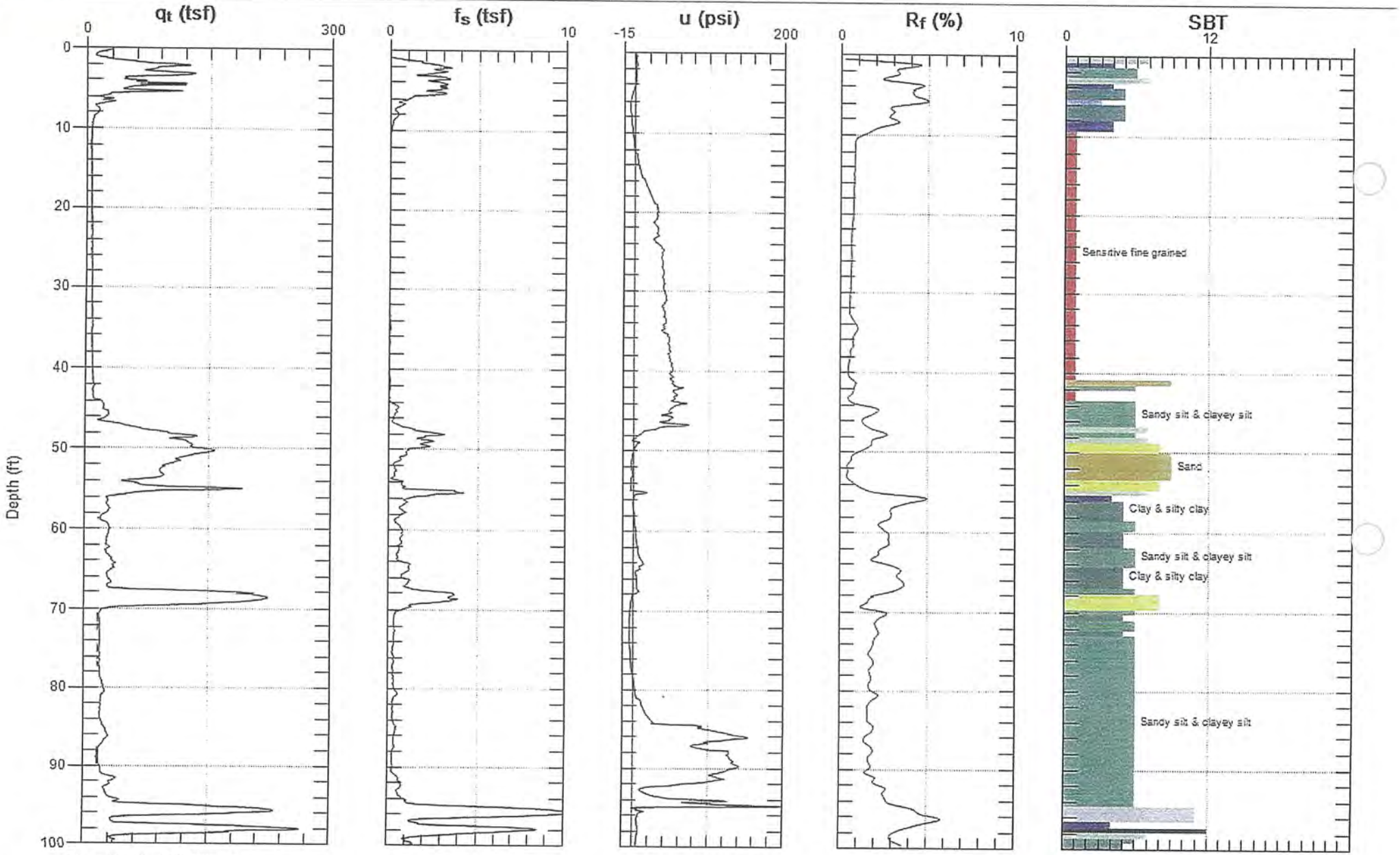
Max. Depth: 100.066 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BAY AREA GEOTECHNICAL

Site: 1450 MARINERS ISLAND Engineer: A.SINGH

Sounding: CPT-01 Date: 12/11/2007 08:32



Max. Depth: 100.066 (ft)
Avg. Interval: 0.656 (ft)

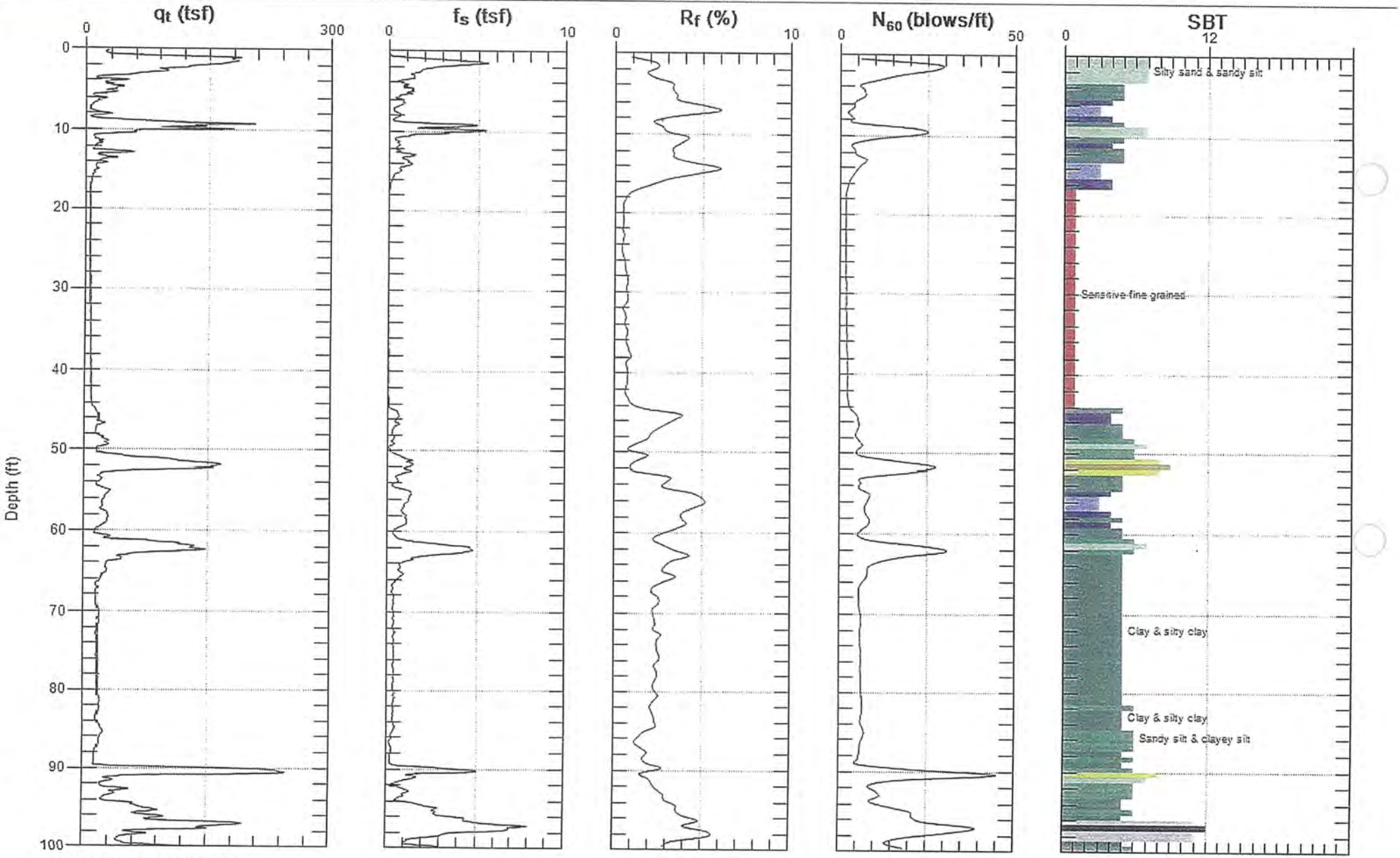
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BAY AREA GEOTECHNICAL

Site: 1450 MARINERS ISLAND Engineer: A.SINGH

Sounding: CPT-02 Date: 12/11/2007 10:10



Max. Depth: 100.066 (ft)

Avg. Interval: 0.656 (ft)

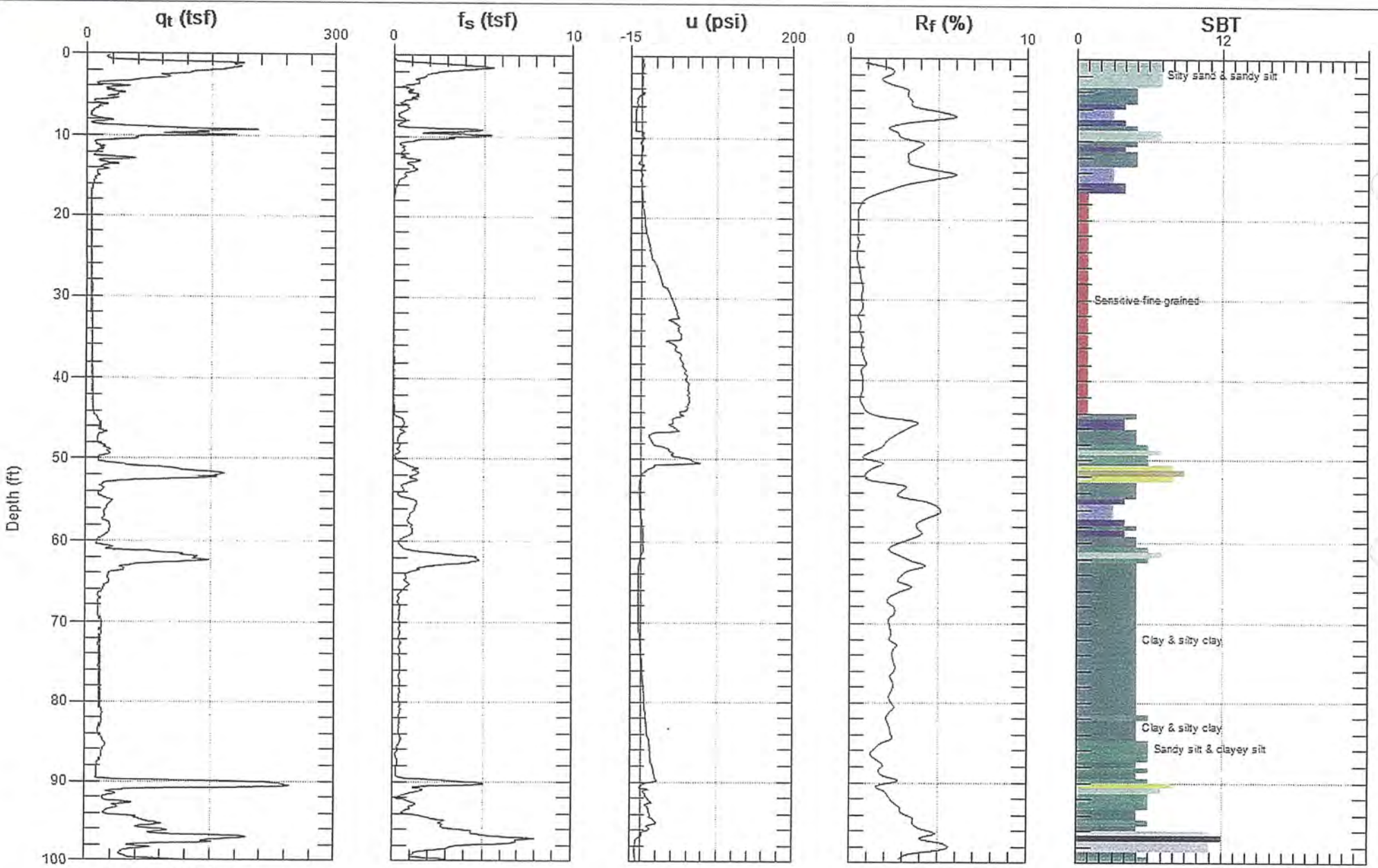
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BAY AREA GEOTECHNICAL

Site: 1450 MARINERS ISLAND Engineer: A.SINGH

Sounding: CPT-02 Date: 12/11/2007 10:10



Max. Depth: 100.066 (ft)
Avg. Interval: 0.656 (ft)

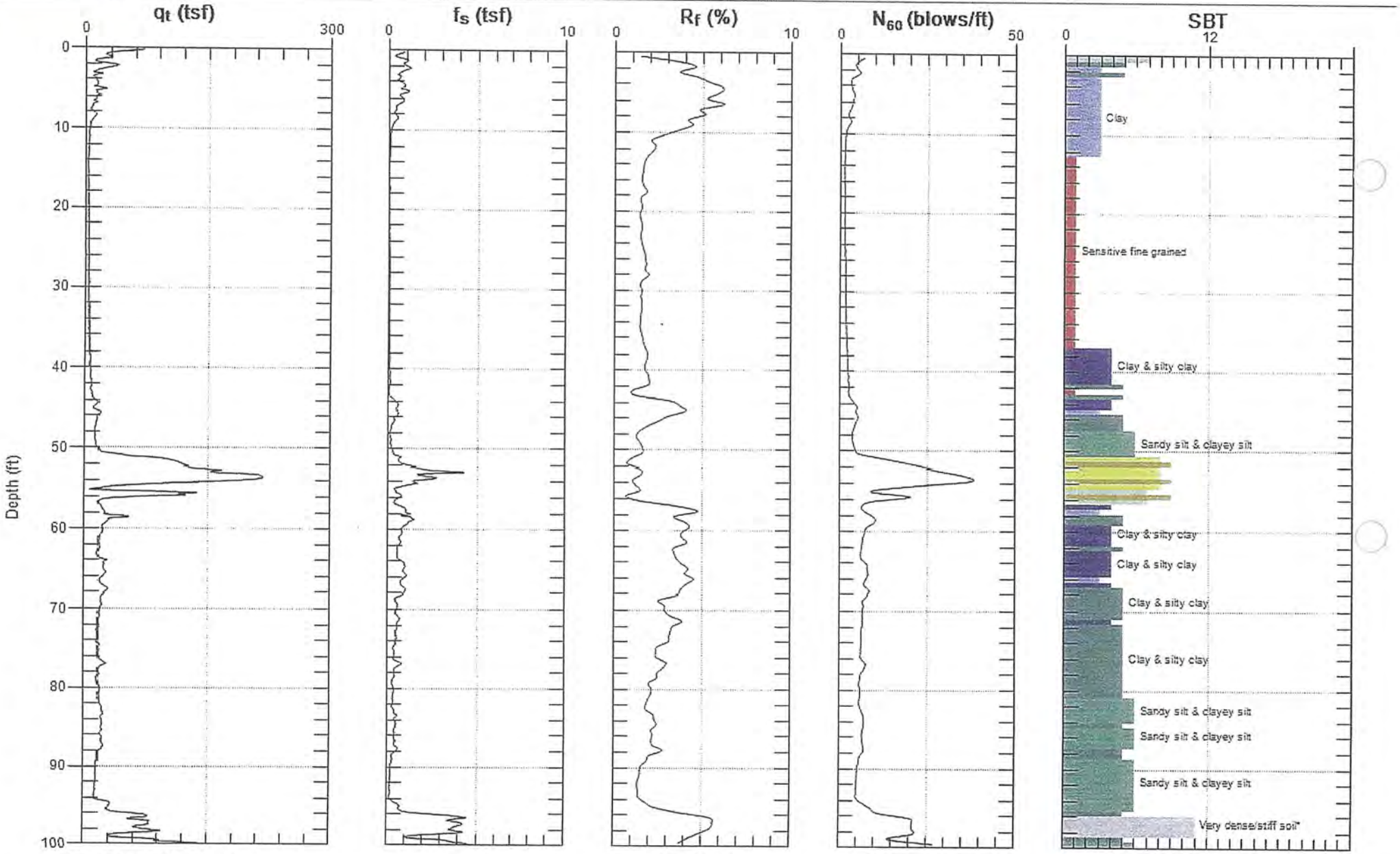
SBT: Soil Behavior Type (Robertson 1990)

GREGG

BAY AREA GEOTECHNICAL

Site: 1450 MARINERS ISLAND Engineer: A.SINGH

Sounding: CPT-03 Date: 12/11/2007 12:51



Max. Depth: 100.066 (ft)

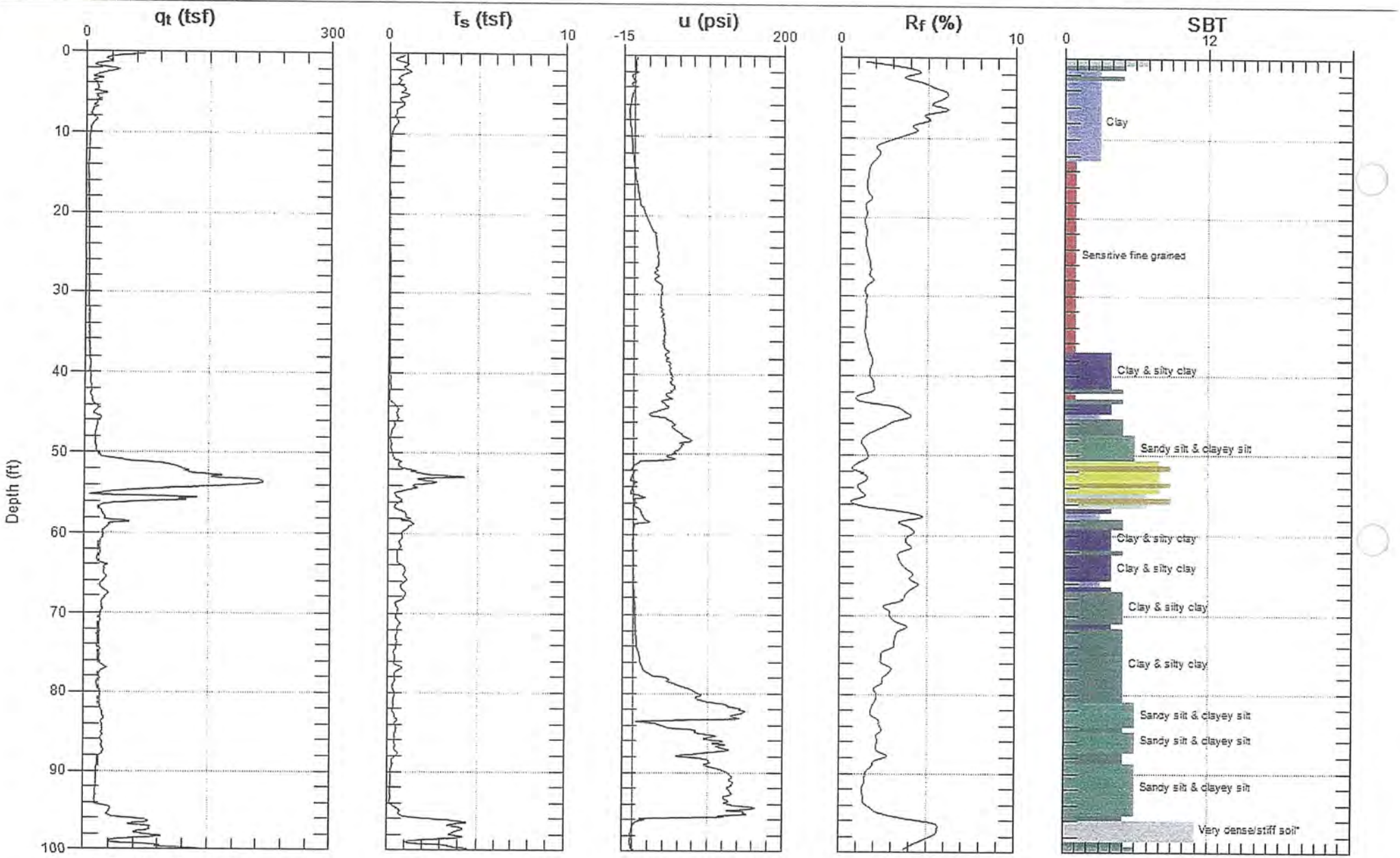
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

BAY AREA GEOTECHNICAL

Site: 1450 MARINERS ISLAND Engineer: A.SINGH

Sounding: CPT-03 Date: 12/11/2007 12:51



Max. Depth: 100.066 (ft)

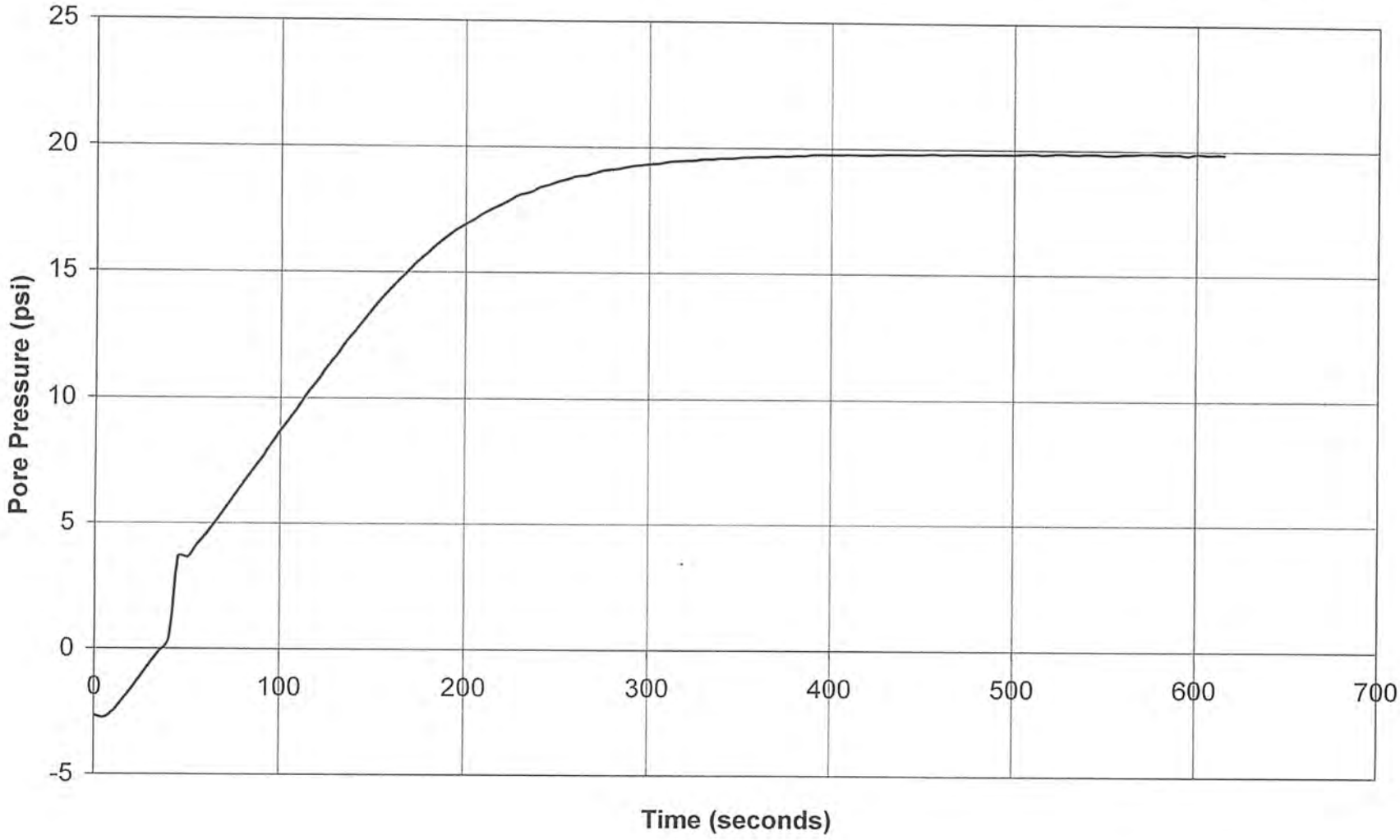
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

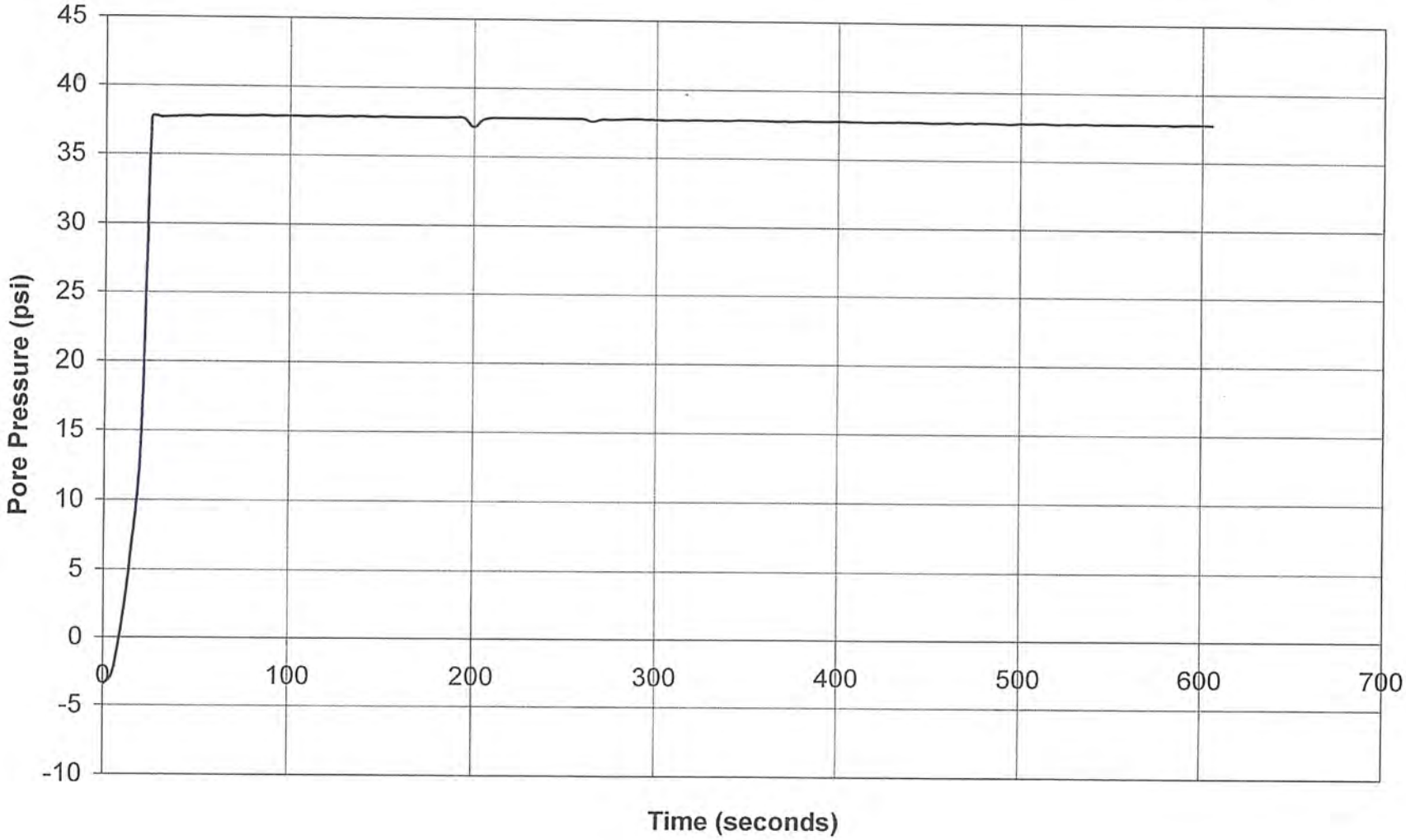
Sounding: CPT-01
Depth: 48.72
Site: 1450 MARINERS
Engineer: A. SINGH



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

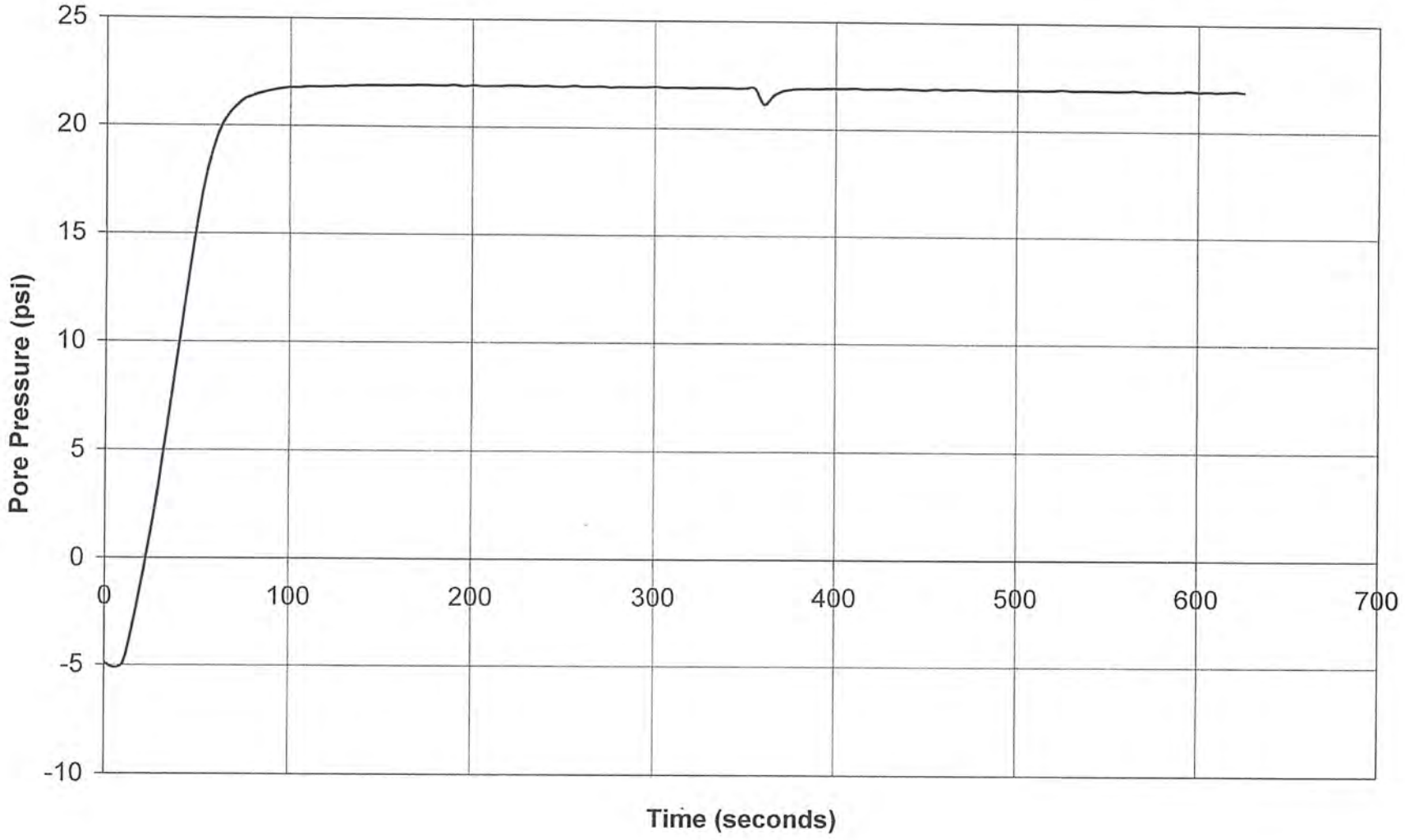
Sounding: CPT-02
Depth: 90.715
Site: 1450 MARINERS
Engineer: A. SINGH



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-03
Depth: 53.149
Site: 1450 MARINERS
Engineer: A. SINGH



Cone Penetration Testing Procedure (CPT)

Gregg In Situ, Inc. 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.

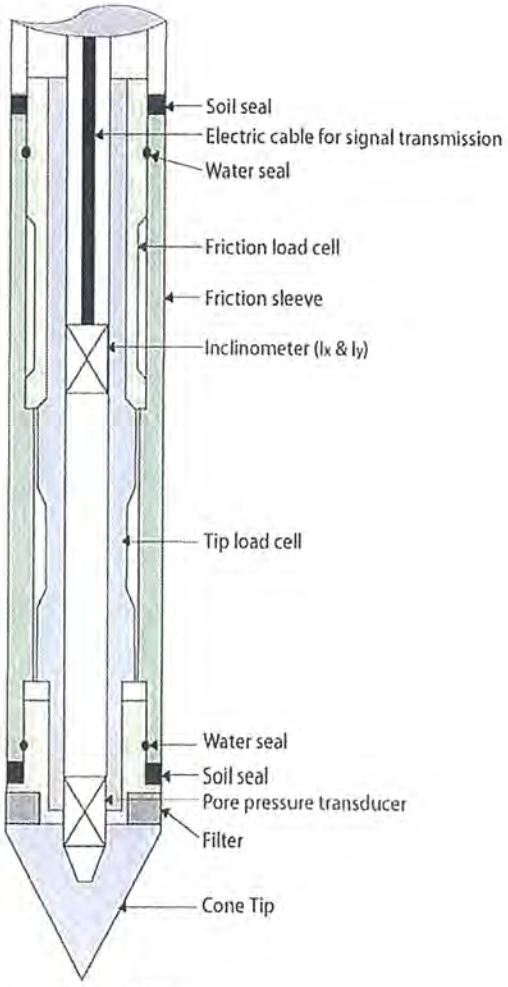


Figure CPT

When the soundings are complete, the test holes are grouted using a Gregg In Situ 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.

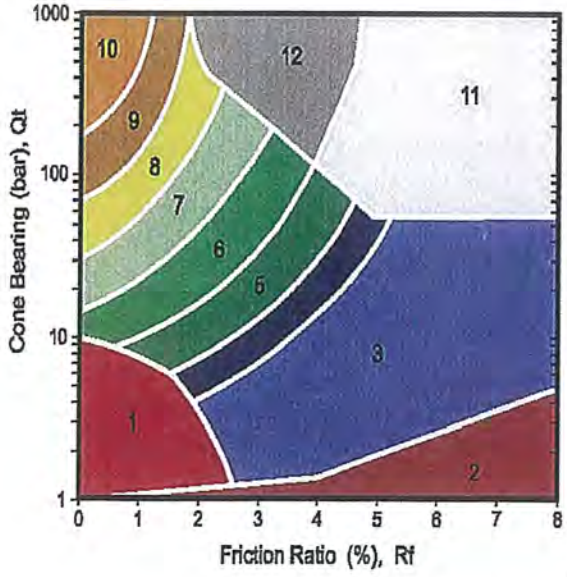
Cone Penetration Test Data & Interpretation

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 InSitu and 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 have 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 0.85)
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 $\gamma_w = 62.4$ 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) $q_t = q_c + u(1-a)$

8	Friction Ratio, R_f (%)	$R_f = (f_s/q_t) \times 100\%$
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_{tl}	$Q_{tl} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo}) \times 100\%$
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 — $I_c = ((3.47 - \log Q_{tl})^2 + (\log F_r + 1.22)^2)^{0.5}$

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) $\frac{(q_c/p_a)}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right)$
8	Equivalent SPT $(N_1)_{60}$ blows/ft where $C_N = (pa/\sigma'_{vo})^{0.5}$	$(N_1)_{60} = N_{60}\, C_N$,
9	Relative Density, D_r, (%) *Only SBT_n 5, 6, 7 & 8*	$D_r^2 = Q_{tn} / C_{Dr}$ *Show 'N/A' in zones 1, 2, 3, 4 & 9*
10	Friction Angle, ϕ', (degrees) *Only SBT_n 5, 6, 7 & 8*	$\tan\phi' = \frac{1}{2.68}\left[\log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right]$ *Show 'N/A' in zones 1, 2, 3, 4 & 9*
11	Young's modulus, E_s *Only SBT_n 5, 6, 7 & 8*	$E_s = \alpha\, q_t$ *Show 'N/A' in zones 1, 2, 3, 4 & 9*
12	Small strain shear modulus, Go a. $G_o = S_G (q_t\ \sigma'_{vo}\ pa)^{1/3}$ b. $G_o = C_G\, q_t$	*For SBT_n 5, 6, 7* *For SBT_n 1, 2, 3& 4* *Show 'N/A' in zones 8 & 9*
13	Undrained shear strength, s_u *Only SBT_n 1, 2, 3, 4 & 9*	$s_u = (q_t - \sigma_{vo}) / N_{kt}$ *Show 'N/A' in zones 5, 6, 7 & 8*
14	Over Consolidation ratio, OCR *Only SBT_n 1, 2, 3, 4 & 9*	$OCR = k_{ocr}\, Q_{tl}$ *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 soils	2	organic soils
3	clays	3	clays
4	clays & silty clays	4	clays & silty clays
5	clays & silty clays		
6	silty sands & sandy silts	5	silty sands & sandy silts
7	silty sands & sandy silts		
8	sands & silty sands	6	sands & silty sands
9	sands & silty sands		
10	sands	7	sands
11	very dense/stiff soils*	8	very dense/stiff soils*
12	very dense/stiff soils*	9	very dense/stiff soils*

* 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

APPENDIX PPD

Pore Pressure Dissipation Tests (PPDT)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals measured hydrostatic water pressures and determined the approximate depth of the ground water table. A PPaT is conducted when the cone is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (u) with time is measured behind the tip of the cone and recorded by a computer system.
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Purface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until such time as there is no variation in pore pressure with time, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992.

A summary of the pore pressure dissipation tests is summarized in Table 1. Pore pressure dissipation data is presented in graphical form in Appendix PPaT.

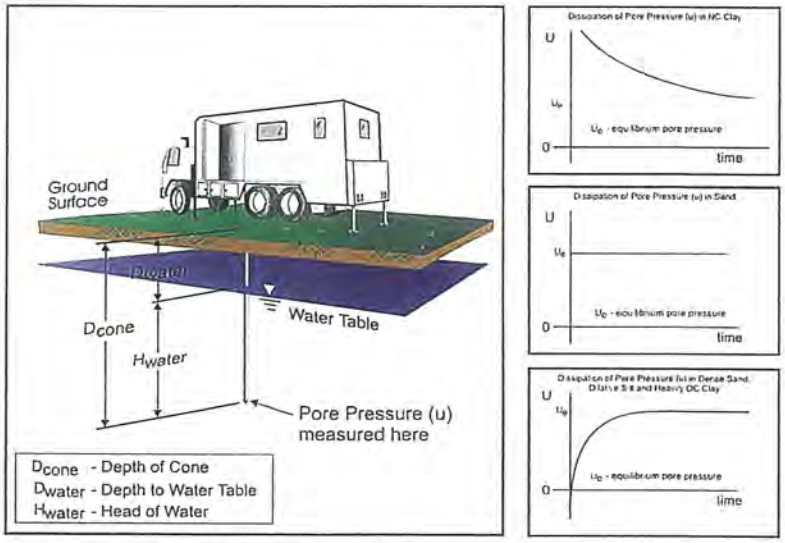


Figure PPDT

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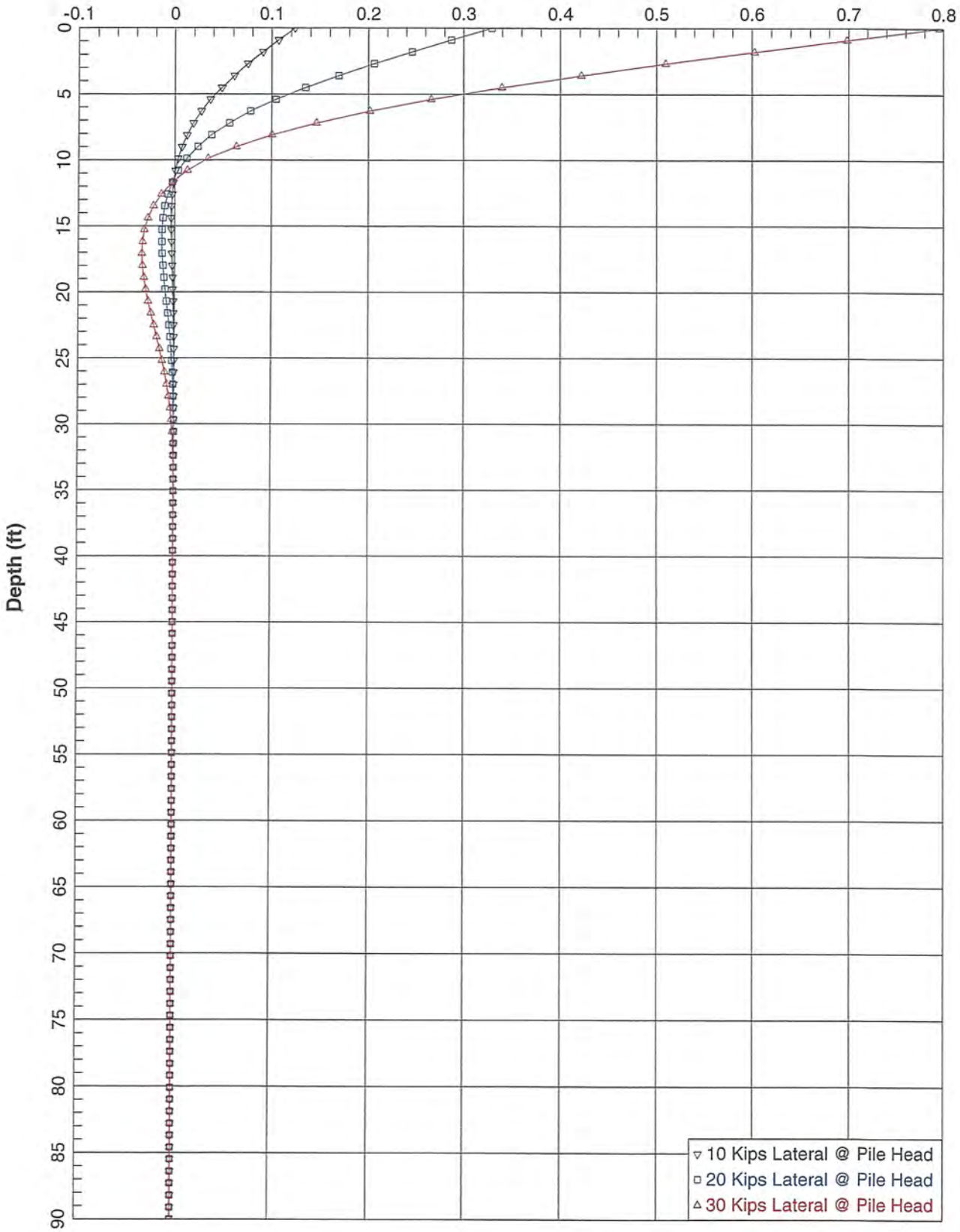
Copies of ASTM Standards are available through www.astm.org

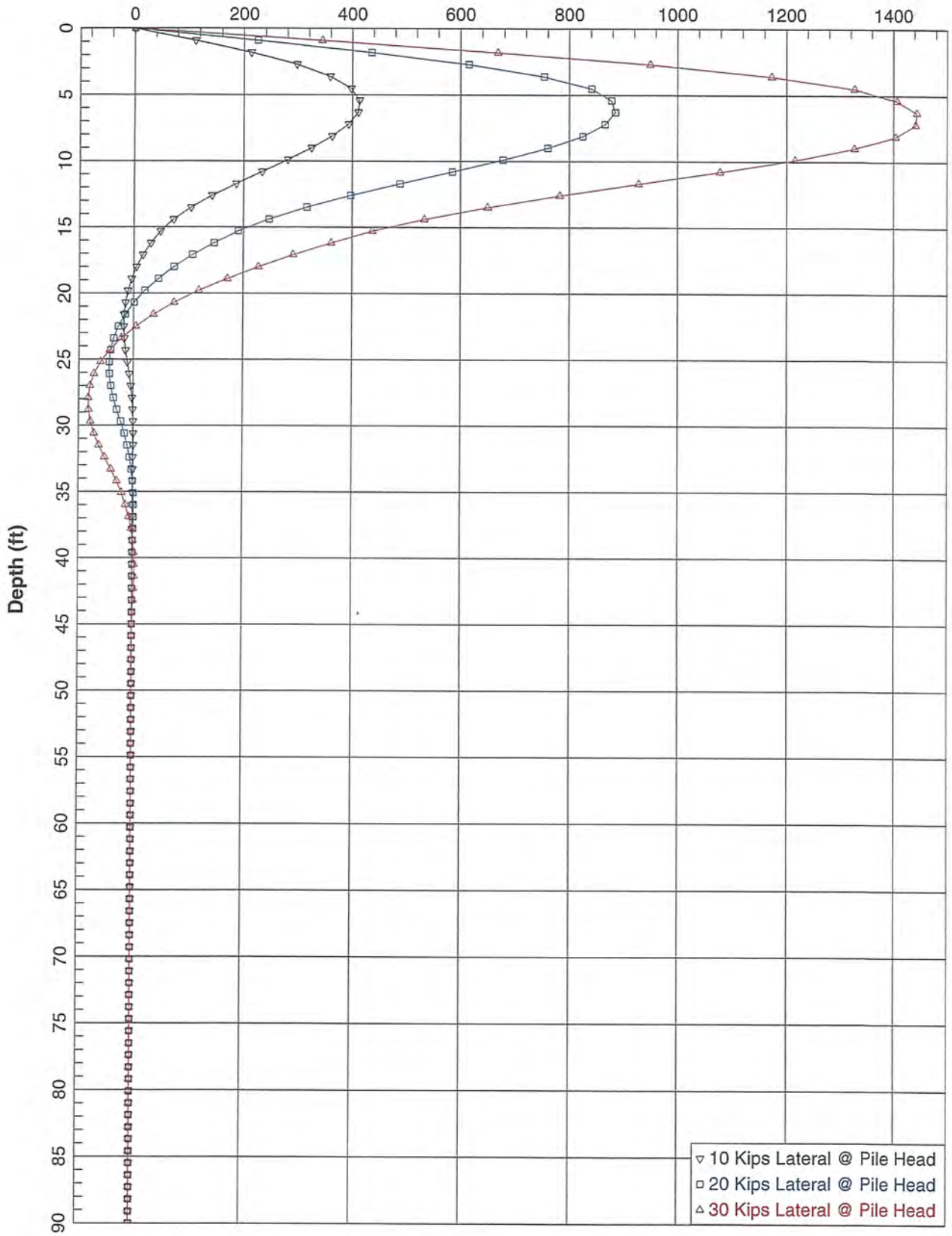
APPENDIX B

LATERAL LOADING BEHAVIOR OF 14-INCH PRECAST CONCRETE PILES AND 18-INCH DIAMETER APG PILES FOR LATERAL LOADS OF 10, 20, AND 30 KIPS

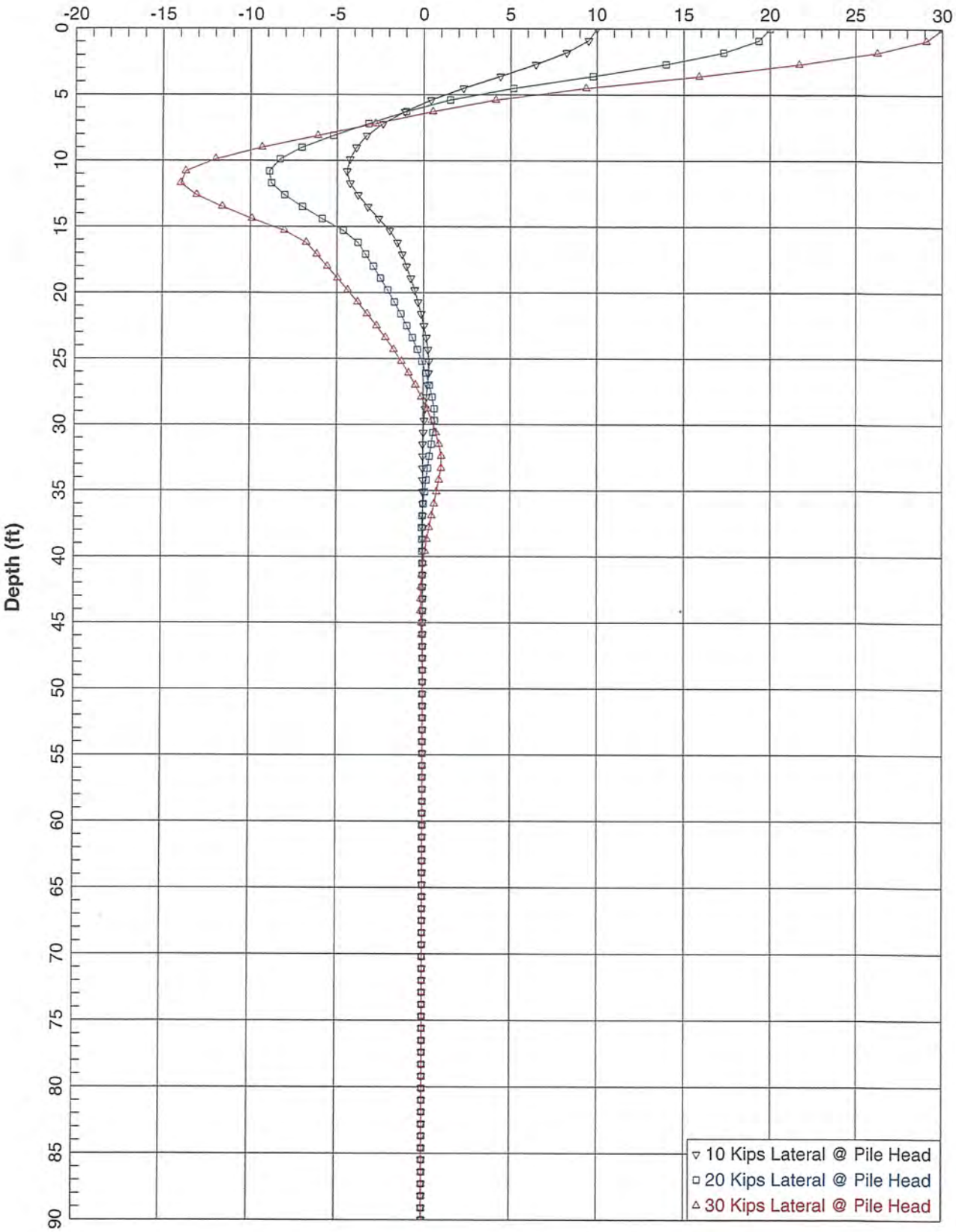
Deflection vs. Depth - 14-inch Square Concrete Pile



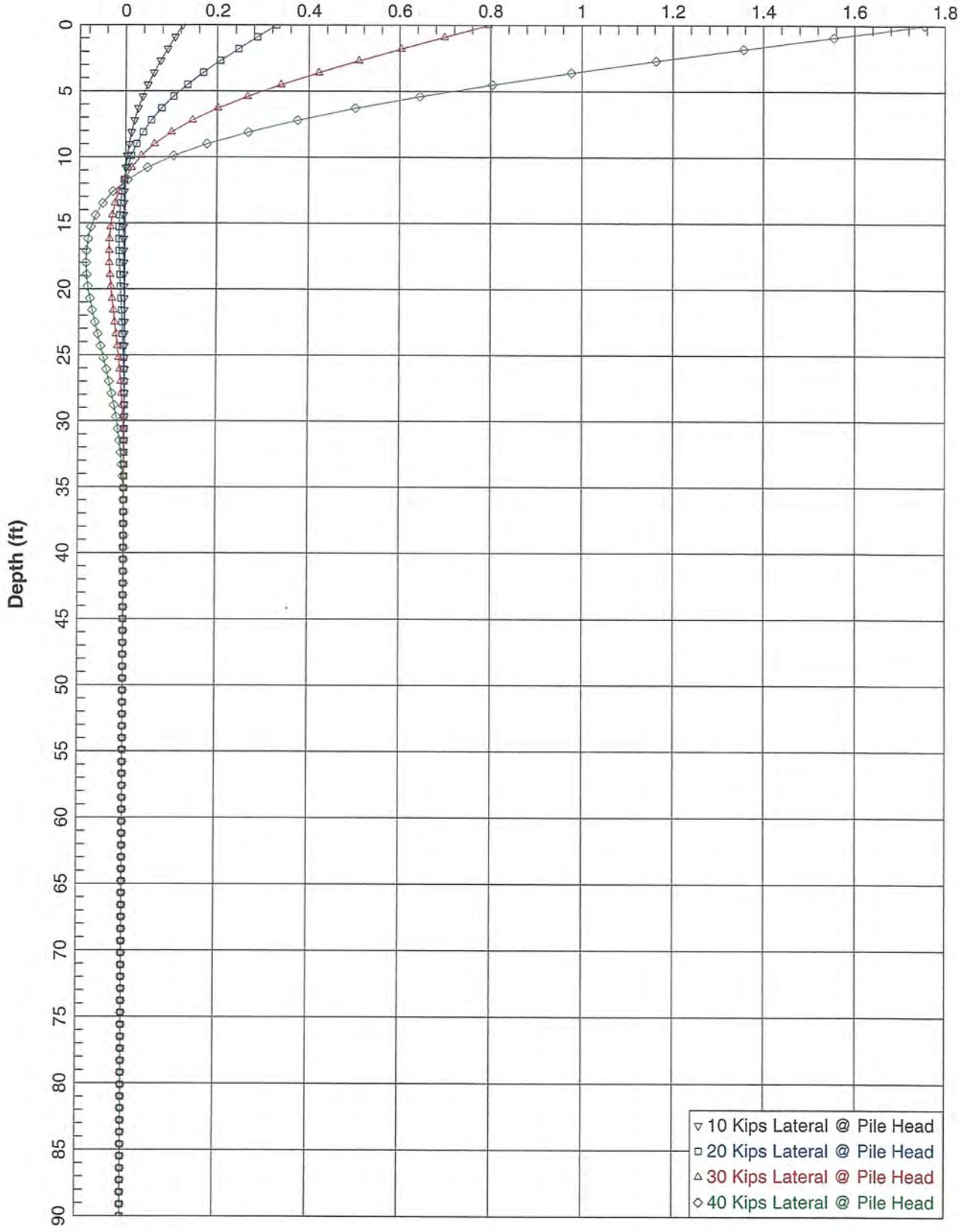
Bending Moment versus Depth, 14-inch Square Concrete Pile



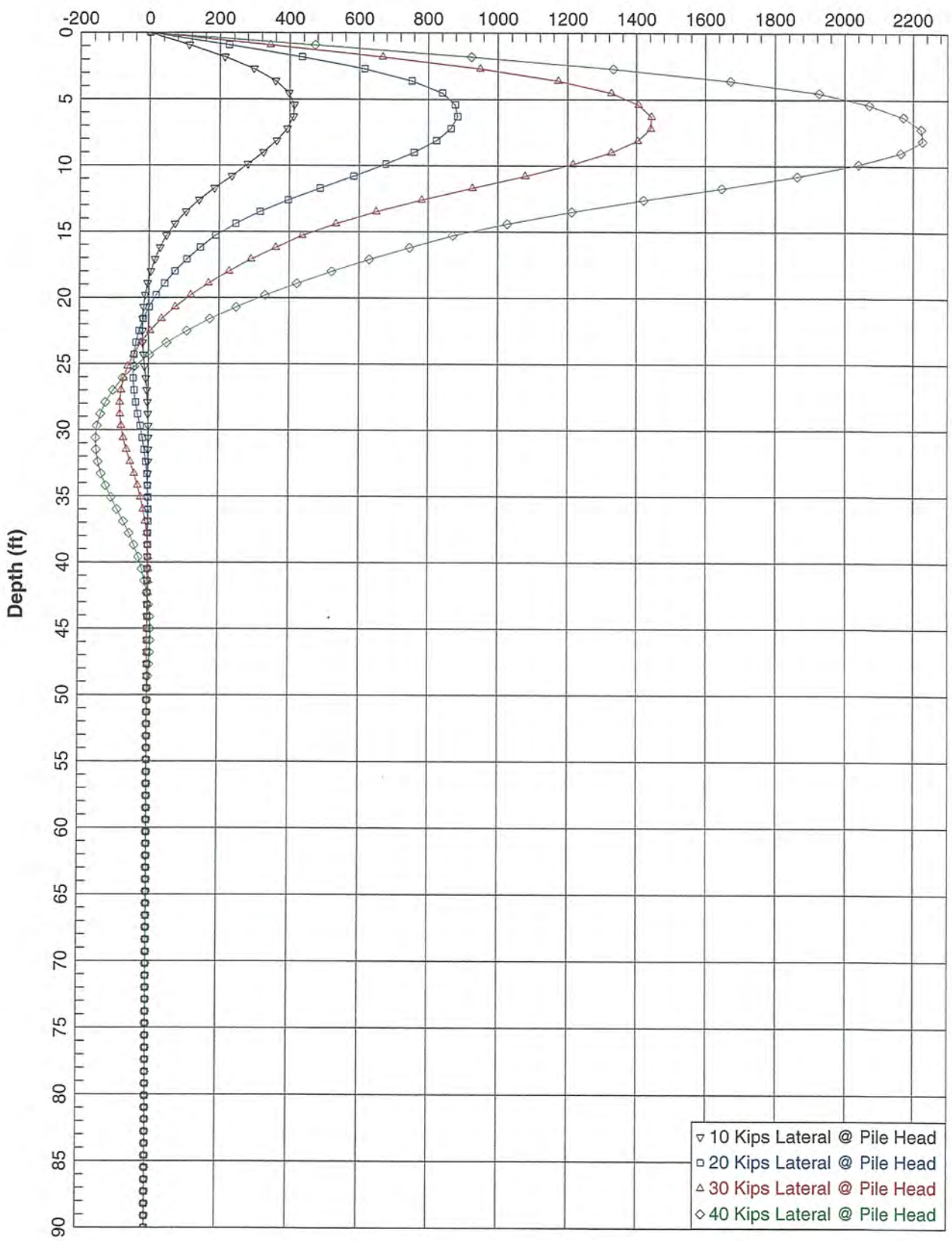
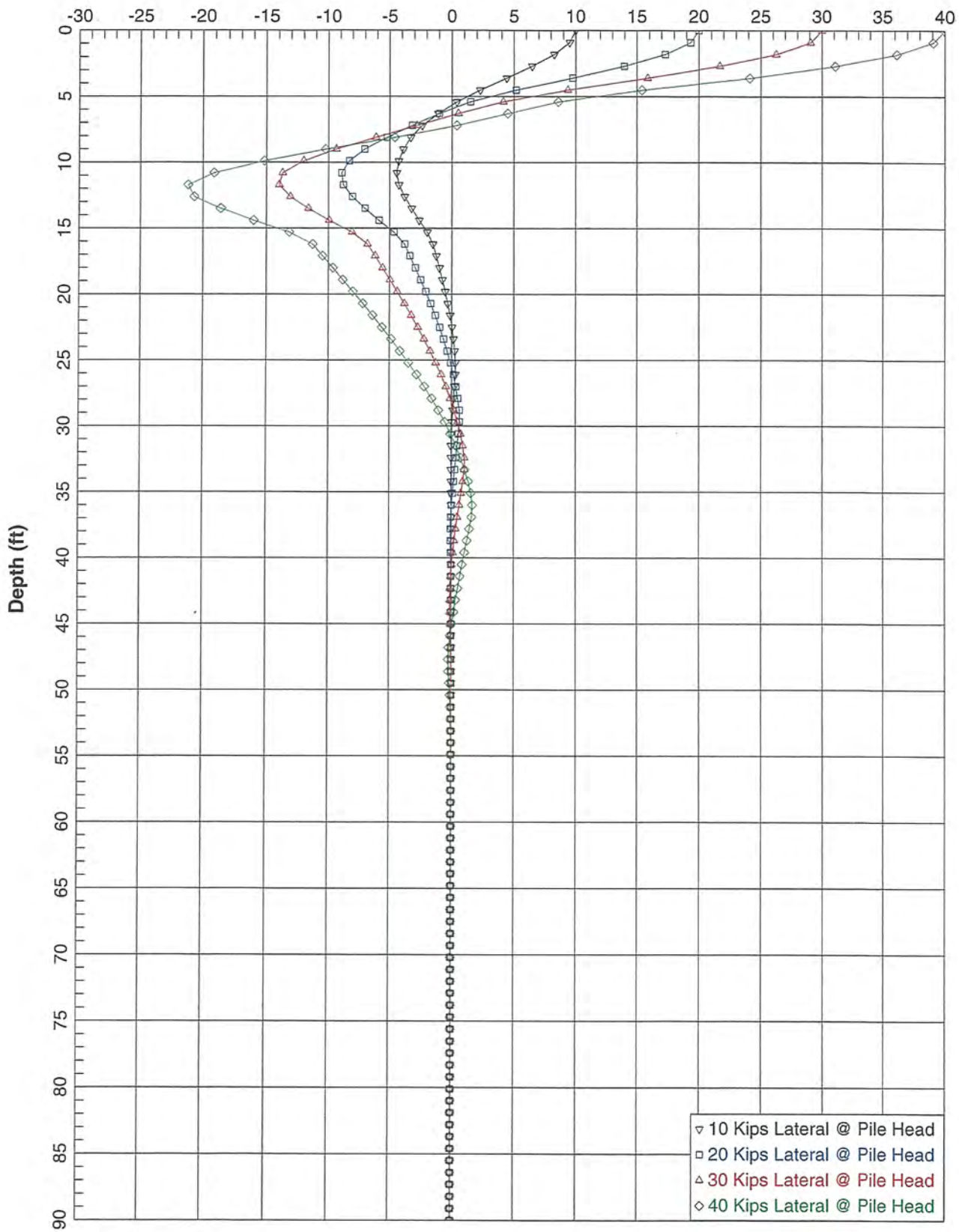
Shear Force vs. Depth, 14-inch Square Concrete Piles



Deflection vs. Depth - 18-inch APG Pile



Bending Moment versus Depth, 18-inch APG Pile



Shear Force vs. Depth, 18-inch APG Piles