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August 24, 2012

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Re: Geotechnical Engineering Services Report
Proposed Townhome / Condominium Development
NWC Via Lido & Via Malaga
Newport Beach, California
PSI Report No. 0559771

Dear Mr. Park:

Professional Service Industries, Inc. (PSI) is pleased to transmit our Geotechnical Engineering Services Report for the referenced project. This report includes the results of field and laboratory testing, and recommendations pertaining to site preparation, foundation design and construction for the proposed improvements.

We appreciate the opportunity to have performed this Geotechnical Study and look forward to our continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact our office.

Respectfully submitted,
PROFESSIONAL SERVICE INDUSTRIES, INC.

Zach McClellan, EIT
Project Engineer

Robert R. Russell, GE
Chief Engineer

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GEOTECHNICAL ENGINEERING SERVICES REPORT

PROPOSED TOWNHOME / CONDOMINIUM DEVELOPMENT
NWC VIA LIDO & VIA MALAGA
NEWPORT BEACH, CALIFORNIA

PSI REPORT NO. 0559771

PREPARED FOR

THE WIELAND-DAVCO CORP.
3355 VIA LIDO, SUITE A
NEWPORT BEACH, CA

ATTENTION: MR. JUSTIN PARK

AUGUST 24, 2012

BY

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PROJECT INFORMATION

PROJECT AUTHORIZATION

Professional Service Industries, Inc. (PSI) has completed a geotechnical exploration for the proposed townhome/condominium development to be located at the NWC of Via Lido and Via Malaga in Newport Beach, California. The Wieland-Davco Corp. authorized our services on August 7, 2012 by signing PSI proposal 0559-75353 dated August 1, 2012.

PROJECT DESCRIPTION

Mr. Justin Park of the Wieland-Davco Corp. provided the project information as described herein to PSI. Based on our discussions with Mr. Park and a review of the site plan prepared by Shusin + Donaldson Architects, Inc. for 3355 & 3388 Via Lido, we understand that new at-grade two to three story townhome/condominium buildings and associated improvements are planned to be constructed at the above mentioned addresses in Newport Beach, California. The site is presently occupied by commercial retail buildings along Via Lido (to the east) and a parking lot (to the west). The existing commercial/retail buildings and paving will be demolished to make way for the planned townhomes/condos to be constructed along the property lines and driveways and parking to be constructed between the proposed townhomes/condos. A Site Vicinity Map showing the site location is included as Figure 1 in the Appendix.

Detailed structural loading has not been provided to us, however we were informed that loads for a 3-story wood-framed residential structure on the order of 3 kips per foot for wall footings and 50 kips for columns would be reasonable assumptions. Detailed grading information has also not been provided, however, PSI has assumed that the site grading will consist of cuts and fills of less than 3 feet, not including any remedial grading.

The geotechnical recommendations presented in this report are based on the available project information, site location, laboratory testing, and the subsurface materials. If any of the noted information is incorrect, please inform PSI in writing so that we may amend the recommendations presented in this report if appropriate and if desired by the client. PSI will not be responsible for the implementation of its recommendations when it is not notified of changes in the project.

PURPOSE AND SCOPE OF SERVICES

The purpose of this geotechnical study was to explore the subsurface conditions and provide suitable foundation recommendations for the proposed construction. The geotechnical exploration for this project involved drilling four test borings, laboratory testing, and geotechnical analyses. This report briefly outlines the testing procedures, presents available project information, describes the site and subsurface conditions, and presents recommendations for the following:

- Site preparation and grading.
- Findings pertaining to potentially expansive, deleterious or corrosive materials.
- An assessment of the liquefaction potential and an estimate of seismic-induced settlements

- Recommendations pertaining to design and construction of foundations for support of the proposed construction, including allowable soil bearing pressure, anticipated bearing depths and estimated settlements.
- Pavement recommendations including subgrade preparation and construction control of groundwater.
- Comments regarding factors that may impact construction and performance of the proposed construction.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, surface water, groundwater, or air on or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes. PSI is concurrently performing a Phase I Environmental Site Assessment (ESA) at the subject site and a separate report will be issued to address environmental concerns.

A geologic fault study to evaluate the possibility of surface faulting at this site was beyond the scope of this investigation. Should you desire a detailed fault study, please contact us; however, active faults are not known to exist on or in the immediate vicinity of the site.

Services that investigate or detect the presence of moisture, mold, or other biological contaminants in or around any structure, or any service that was designed or intended to prevent or lower the risk of the occurrence of the amplification of the same, were not provided. Mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture. Site conditions are outside of PSI's control, and mold amplification will likely occur, or continue to occur, in the presence of moisture. As such, PSI cannot be held responsible for the occurrence or recurrence of mold amplification.

SITE AND SUBSURFACE CONDITIONS

SITE LOCATION AND DESCRIPTION

The project site is located adjacent to the NWC of Via Lido and Via Malaga in Newport Beach, California. Furnished information indicates the approximate site GPS coordinates are latitude: 34.6167°N and longitude: -117.9281°W. The subject site is currently developed with existing commercial/retail buildings along Via Lido (to the east) and a parking lot (to the west. The site is relatively level with a maximum elevation differential of about 2 feet, sloping down to the west (Google Earth, 2011). The site is triangular in shape and bounded by Via Lido to the northeast, Via Malaga to the south and Via Oporto to the west.

REGIONAL GEOLOGY

The subject site is located at elevations between approximately 6 to 8 feet above mean sea level (Google Earth). Based on a review of the *CGS Seismic Hazard Report, Newport Quadrangle*, the site is located within the Orange County coastal plain and underlain by Quaternary alluvial and fluvial sedimentary deposits.

REGIONAL SEISMICITY

The project site is located in Southern California, which has undergone a complex multiphase structural history and remains an active tectonic region with documented historic earthquakes. Generally, the seismicity within California can be attributed to faulting due to regional tectonic movement. This includes the San Andreas Fault and other sub-parallel strike-slip faults, as well as normal and thrust faulting within the State. The area of the subject site is considered seismically active. Seismic hazards within the site can be attributed to potential ground shaking resulting from earthquake events along nearby or more distant faulting.

The primary causes of damage in this general area during seismic events include ground shaking and liquefaction of the subsurface strata. Liquefaction occurs when loose granular and low plastic materials below the groundwater table are subjected to cyclic shear forces resulting from seismic events. During seismic shaking the porewater pressure increases with a corresponding decrease in the soils effective stress. Excess pore pressures ultimately dissipate and the soil consolidates, often resulting in significant total and differential settlement of the ground surface.

SUBSURFACE CONDITIONS

The boring locations were marked in the field by a PSI representative by referencing existing landmarks based on the information provided by the client. A truck-mounted CME-75 drill rig using mud rotary drilling methods was used to advance the borings. Soil samples were routinely obtained during the drilling process. Drilling and sampling techniques were accomplished general in accordance with ASTM procedures (ASTM D1586 and D3550).

The subsurface conditions were explored by drilling four soil borings at this site. Soil borings B1 through B4, were drilled within the existing parking lot to depths ranging from approximately 20 to 50-feet below the existing ground surface elevation. The locations of our test borings were restricted due to the existing on-site improvements. Figure 2 in the Appendix shows the approximate boring locations. The soil types encountered at the specific boring locations are presented on the attached Boring Logs in the Appendix.

As indicated on our boring logs, the existing pavement section generally consists of approximately 3 inches of asphalt underlain by a silty sand with gravel (apparent base course) that was estimated to be about 6 inches thick. The pavement section was underlain by native soil consisting of medium dense silty gravely sand with trace organics that extend to a depth of approximately 5-feet below existing grade, very soft to soft clayey silt that extends to a depth of approximately 7½-feet below existing grade, and loose to very dense slightly silty sand to the maximum depth explored of approximately 50-feet below the existing ground surface elevation.

During the sampling procedure, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586 and relatively undisturbed samples were obtained in general accordance with ASTM D3550. The SPT for soil borings is performed by driving a 2-inch diameter split-spoon sampler into the undisturbed formation located at the bottom of the advanced borehole with repeated blows of a 140-pound hammer falling a vertical distance of 30-inches. The number of blows required to drive the sampler the last 12-inches of an 18-inch penetration depth is a measure of the soil consistency. For ASTM D-3550 (California Modified

Sampler), the split barrel sampler possesses a 3-inch O.D. and is driven in the same manner as the SPT. The blow count obtained from the California Modified sampler should be reduced by approximately 40 percent to obtain a rough correlation to SPT blow counts (N-value). Samples were identified in the field, placed in sealed containers and transported to the laboratory for further classification and testing.

The stratification presented on the Boring Logs is based on a visual examination of the recovered soil samples and the interpretation of field logs by a geotechnical professional. Included on the Boring Logs are the standard penetration resistances (SPT N-values and California Modified sampler blows) recorded in the individual borings at standard testing intervals to the boring termination depths.

The above subsurface information is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The Boring Logs, included in the Appendix, should be reviewed for specific information at the boring locations. These records include soil descriptions, stratification, penetration resistance, locations of the samples and laboratory test data. The stratification shown on the logs represent the conditions only at the actual location at the time of our exploration. Variations may occur and should be expected between locations. The stratification that represents the approximate boundary between subsurface materials and the actual transition may be gradual. Lines of demarcation represent the approximate boundary between subsurface materials, and the transition may be gradual. It should be noted that, although the test borings are drilled and sampled by experienced professionals, it is sometimes difficult to record changes in stratification within narrow limits, especially at great depths. In the absence of foreign substances, it is also sometimes difficult to distinguish between discolored soils and clean fill soil.

GROUNDWATER INFORMATION

Groundwater was measured at approximately 5-feet below existing grade in all four borings at the time of drilling. Based on a review of the California Geological Survey (CGS) Seismic Hazard Zone Report for the Newport Quadrangle, the historic high groundwater depth for the site area is noted to be about 5 feet below grade.

It is possible that seasonal variations (temperature, rainfall, tide conditions etc) will cause fluctuations in the groundwater level. Additionally, perched water may be encountered in discontinuous zones within the overburden. The groundwater levels presented in this report are the levels that were measured at the time of our field activities. It is recommended that the contractor determine the actual groundwater levels at the site at the time of the construction activities to determine the impact, if any, on the construction procedures.

LABORATORY TESTING

The soil samples obtained during the field exploration were transported to our laboratory and selected soil samples were tested in the laboratory to determine the material properties for evaluation. Laboratory testing on selected samples included Moisture Content (ASTM D2216), Unit Weight, Sieve Analysis (ASTM D422 and D1140), Expansion Index testing (ASTM D4829), Corrosion testing (CTM 643, CTM 417 and CMT 422), Atterberg Limit testing (ASTM D4318), and Consolidation (ASTM D2435). Laboratory testing was performed in general accordance with ASTM and/or California Test procedures. Unless otherwise informed, the soil samples will be discarded 60 days from the issuance of the report.

Results of our laboratory testing indicate the tested materials have moisture contents between approximately 3 percent to 54 percent. Consolidation tests were performed on samples at depths of 7½ feet, 5 feet, and 10 feet below existing grade at Test Boring Nos. 1, 2, and 3, respectively, and these tests indicated the soils at 7½ feet and 10 feet in Test Borings Nos. 1 and 3, respectively possess a relatively low compressibility. However the consolidation test performed on the soil sample at 5-feet in Test Boring No. 2 indicated a high level of compressibility. The corrosion test results indicated the near surface soils are nearly neutral, have a low chloride content, possess a *negligible sulfate exposure* and resistivity results indicates the materials possess a *progressively less corrosive* environment for ferrous metals. The results of our Expansion Index testing indicate the near surface soils have a very low expansion potential (EI=0). Laboratory test data along with detailed descriptions of the soils can be found on the Boring Logs in the Appendix.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

The following geotechnical design recommendations have been developed on the basis of the previously described project characteristics and subsurface conditions encountered. If there are any changes in these project criteria, including building location on the site, PSI should be contacted to determine if modifications to the recommendations are warranted.

EARTHQUAKE AND SEISMIC DESIGN CONSIDERATIONS

The project site is located within a municipality that employs the 2010 California Building Code (CBC), the locally adopted version of the International Building Code, 2009 edition. As part of this code, the design of structures must consider dynamic forces resulting from seismic events. These forces are dependent upon the magnitude of the earthquake event as well as the properties of the soils that underlie the site. As part of the procedure to evaluate seismic forces, the code requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100 feet of the ground surface. To define the Site Class for this project, we have interpreted the results of soil test borings drilled within the project site and estimated appropriate soil properties below the base of the borings to a depth of 100 feet as permitted by the code. The estimated soil properties were based upon our experience with subsurface conditions in the general site area.

Based upon our evaluation, the subsurface conditions within the site are consistent with the characteristics of a Site Class "D" as defined in Table 1613.5.2 of the CBC. The associated USGS-NEHRP (2002) probabilistic ground acceleration values and site coefficients for the general site area were obtained from the USGS geohazards web page: http://earthquake.usgs.gov/research/hazmaps/design

Which is presented in Table 1.

Table 1: Ground Motion Values*

Period (sec)	Mapped MCE Spectral Response Acceleration** (g)		Site Coefficients		Adjusted MCE Spectral Response Acceleration (g)		Design Spectral Response Acceleration (g)	
0.2	S_s	1.847	F_a	1.0	S_{Ms}	1.847	S_{Ds}	1.231
1.0	S_1	0.695	F_v	1.5	S_{M1}	1.042	S_{D1}	0.695

**2% Probability of Exceedence in 50 years for Latitude 33.6167°N and Longitude -117.9281°W*
***At B-C interface (i.e. top of bedrock).*
MCE = Maximum Considered Earthquake

The Site Coefficients, F_a and F_v presented in the above table were also obtained from the noted USGS webpage, as a function of the site classification and mapped spectral response acceleration at the short (S_s) and 1-second (S_1) periods, but can also be interpolated from CBC Tables 1613.5.3(1) and 1613.5.3(2).

Hazard Assessment

Alquist-Priolo Fault Zone - The seismicity of the site was evaluated utilizing deterministic methods for active faults within the regional vicinity. According to the Alquist-Priolo Special Studies Zones Act of 1972 (revised 1994) faults have been classified as active faults which show apparent movement during the last 11,000 years (i.e., Holocene time). The site is not located within a currently designated Earthquake Fault Zone per the Alquist-Priolo Special Studies Zone Map produced by the California Geological Survey (CGS). The nearest zoned active fault is the Newport – Inglewood (L.A. Basin) Fault Zone, mapped 0.4 mile to the northwest of the subject site.

Lurching and Shallow Ground Rupture – Breaking of the ground because of active faulting is not likely due to the absence of known active fault traces within the project limits.

Liquefaction Induced Settlement - Liquefaction and seismically induced settlement typically occur in loose granular and low-plastic silt and clay soils with groundwater near the ground surface. During an earthquake, ground shaking causes the soil to consolidate and an increase in the pore pressures in saturated soils. After dissipation of the excess pore pressures, the saturated soils tend to settle. Fine-grained plastic soils are generally not susceptible to liquefaction or to short-term settlement due to seismic loads.

According to the California Geological Survey (CGS) Newport 7.5' Quadrangle hazard map, the subject site is located within an area that is classified as being susceptible to liquefaction and has a historic high groundwater depth of approximately 5 feet below existing ground surface elevation. Our borings indicate depth to groundwater was measured to be at an approximate depth of 5 feet after drilling was completed.

In order to evaluate the potential for soil liquefaction at this site, we performed an analysis utilizing the LIQUEFYPRO computer software program. For this analysis, we used a groundwater depth of

5 feet (historic high), the soil profile identified in Boring B-1 and a ground acceleration of 0.5g (S_{DS}/2.5, as per the CBC).The results of our analysis indicates that localized zones of the silty sand soils are potentially susceptible to liquefaction upon application of the design site acceleration. Our analysis indicates that the sandy soils between about 7½ to 10 feet, 26 to 28 feet, and 29 to 30 feet below grade are potentially susceptible to liquefaction upon application of the design earthquake. The most significant effect of soil liquefaction is expected to be ground surface settlement resulting from volumetric strain within the liquefiable soils. Based on our analysis, we estimate a maximum total seismic induced settlement of approximately 1-¼ inches with an estimated ⅔ inch of differential settlement across a 40 foot span. Based on this magnitude of estimated settlement, it is our opinion that mitigation of the liquefaction potential is not warranted. The output file from the analysis is provided within the Appendix.

Landsliding – Due to the generally flat nature of the site and surrounding properties, it is our opinion that the site has a low susceptibility to landslides.

Tsunamis and Seiches – Based on our review of the Tsunami Inundation Map for Emergency Planning, Newport Beach Quadrangle, dated March 15, 2009, issued by the State of California-Orange County, the site is located within a designated tsunami inundation area. As such the potential does exist for tsunami inundation to impact the site.

For Seismic Design Category designations of C, D, E or F, which are contingent on the structures "Seismic Use Group", the code requires an assessment of slope stability, liquefaction potential and surface rupture due to faulting or lateral spreading. Detailed evaluations of these factors were beyond the scope of this study. However, the following table presents a qualitative assessment of these issues considering the site class, the subsurface soil properties, the groundwater elevation and probabilistic ground motions.

Table 2: Qualitative Seismic Site Assessments

Hazard	Relative Risk	Comments
Liquefaction	Moderate	Differential seismic induced settlement of about ⅔ inch is estimated across a 40 foot span.
Slope Stability	Low	Based on the presumed grading plans, significant cut or fill slopes are not planned for construction.
Surface Rupture	Low	Active faults are not known to underlie the site.

SITE PREPARATION & GRADING

The current geotechnical issues at the site that will affect the construction of the proposed development include the following:

1. Surface and subsurface disturbance during clearing and demolition operations.
2. Shallow groundwater.
3. Potentially liquefiable soils.
4. Soft soil deposits which will require the use of a deep foundation system.

Site Preparation

Initial site preparation should include stripping of any vegetation, demolition of the existing buildings and removal of the existing pavement that is present within the planned new development areas. Demolition of the existing buildings should include removal of all shallow foundations, floor slabs and underground construction. Existing underground utilities should either be properly capped off at the property boundaries and removed or be re-routed around the new development. Utilities should be removed and properly abandoned in accordance with local regulatory requirements. All soils disturbed by the clearing and demolition operations should be removed, cleaned of deleterious materials and stockpiled on-site for future use as Engineered Fill. All debris and deleterious materials generated by the site stripping and demolition operations should be legally disposed off-site. If the existing buildings are supported by deep foundations, we recommend that the deep foundations be cut-off at least 3 feet below finished grade and to a depth where they will not impact construction of the new foundations.

If grading occurs in the winter rainy season, unstable subgrade conditions may be present. These conditions may require stabilizing the subgrade with admixtures, such as cement kiln dust or a coarse aggregate. Isolated areas may be stabilized using a geogrid, such as Tensar TX160 or equal, with one foot compacted Class II aggregate base over the geogrid. Additional recommendations can be provided, as required, during construction.

Remedial Grading

Following site clearing, demolition and lowering of site grades where needed, we recommend that the soils beneath the new buildings be over-excavated to a depth of at least two feet below existing or finished grade, whichever is deeper. The exposed soils should then be scarified to a depth of approximately 12 inches, be moisture conditioned to about 0 to 3 percent above the soil's optimum moisture content and then be compacted to at least 90 percent of the soil's maximum dry density, per ASTM D-1557.

The subgrade within all other development areas of site should be proof rolled with a heavy rubber-tired piece of construction equipment approved by and in the presence of the Geotechnical Engineer. Any soil that ruts or excessively deflects during proof rolling should be removed or stabilized as recommended by the Geotechnical Engineer. Due to the presence of shallow groundwater, some unstable soil requiring removal or stabilization should be expected. The soils exposed at the base of all excavations should be scarified to a depth of at least 12 inches, be moisture conditioned to about 0 to 3 percent above the soil's optimum moisture and compacted to at least 90 percent of the soil's maximum dry density, per ASTM D-1557. However, the top 12-inches of the pavement subgrade should be compacted to at least 95 percent of the modified Proctor value (ASTM D-1557).

Site grades may then be raised with low expansive Engineered Fill to achieve the design elevations at the site. A PSI representative should be on-site during site grading to evaluate the degree of compaction obtained by the contractor.

Engineered Fill

Engineered Fill material beneath the proposed exterior slabs to support the generators should not contain rocks greater than 3-inches in diameter or greater than 30 percent retained on the ¾-inch sieve, and should not contain more than 3 percent (by weight) of organic matter or other unsuitable material. The Expansion Index (EI) for the material should not exceed 40. Based on our subsurface investigation, existing on-site sandy soils are generally suitable for use as Engineered Fill; however, this should be confirmed by a PSI representative during grading. Import materials meeting the above requirements should be approved by the Geotechnical Engineer prior to use as Engineered Fill. The on-site clayey silt soils are not considered suitable for use as Engineered Fill beneath surface improvements.

Engineered Fill should be compacted to at least 90 percent of the maximum dry density as determined by the modified Proctor (ASTM D1557). The moisture content of Engineered Fill should be maintained at approximately 0 to 3 percent above the material's optimum moisture content as determined by the same index during compaction. If the Engineered Fill is too dry, water should be uniformly applied across the affected fill area. If the Engineered Fill is too wet, it must be dried. In either event, the Engineered Fill should be thoroughly mixed by disking to obtain relatively uniform moisture content throughout the lift immediately prior to compaction.

Engineered Fill should be placed in maximum lifts of 8-inches of loose material. Each lift of Engineered Fill should be tested by a PSI soils technician, working under the direction of our Project Geotechnical Engineer, prior to placement of subsequent lifts.

Compaction of the backfill should be checked with a sufficient number of density tests by a representative of the Geotechnical Engineer to determine if adequate compaction is being achieved by the contractor. The properly compacted Engineered Fill should extend horizontally outward beyond the exterior perimeter of the foundations a distance equal to the height of fill or 5-feet, whichever is greater, prior to significant sloping.

TEMPORARY EXCAVATION CONSIDERATIONS

In Federal Register Volume 54, No. 209 (October, 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P." This document was issued to insure better the safety of workers entering trenches or excavations. It is mandated by this federal regulation that all excavations, whether they be utility trenches, basement excavations, or footing excavations, be construction in accordance with the reviewed OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's responsible person, as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. PSI is not assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

DEEP FOUNDATIONS

Due to the presence of the soft and compressible soils at a depth of about 5 feet below grade and the shallow water table, it is our opinion that the proposed buildings should be supported by a deep foundation system that extends through the soft soil deposits and the potentially liquefiable soils and be supported within the underlying medium dense to dense sandy soil deposits. The deep foundation system may consist of cast in-place drilled piers (CIDH), driven pile, auger-cast piles or other propriety systems. We are providing recommendations for CIDH but other systems can be used and PSI can provide supplemental recommendations as needed. CIDH should possess a minimum diameter of 24 inches.

Our analysis included a factor of safety of 2 for skin friction and 3 for end bearing. In our analysis, we conservatively assumed that the soils to a depth of 10 feet had no load-carrying capacity (skin friction of zero). Additionally, drag load (negative skin friction) resulting from potential soil liquefaction has been included in the CIDH capacities provided below.

Based on our analysis, we anticipate that the drilled piers will have the allowable axial capacities as indicated on the following Table 3 for the various pile lengths noted.

Table 3: Allowable CIDH Axial Capacities Versus Depth*

CIDH Tip Depth	24 In. Diam.- CIDH
15 Feet	25 Kips
25 Feet	60 Kips
35 Feet	100 Kips

*Minimum depth of 15 feet recommended

CIDH may be installed on a spacing of 3 pier diameters (center to center) with no reduction in capacity for group effects. CIDH capacities for compressive and uplift loading may be increased by ⅓ for temporary wind and/or seismic loading conditions.

For uplift resistance, we recommend the capacity be based on an average allowable unit skin friction value of 500 psf within the soils below a depth of 10 feet. The upper 10 feet of soil should be neglected in calculating the uplift resistance.

We estimate settlement at the base of the CIDH for the design load will be less than ⅔ inch.

Detailed inspection of CIDH construction should be made to verify that the CIDHs are vertical and founded in the proper bearing stratum, and to verify that all loose materials have been removed prior to concrete placement. Due to the presence of shallow groundwater and sandy soils, temporary casing is recommended to limit sloughing of soil and groundwater intrusion into the drilled shafts. Any accumulated water must be removed prior to the placement of concrete. A hopper and tremie should be utilized during concrete placement to control the maximum free fall of the wet concrete to less than five feet unless the mix is designed so that it does not segregate during free fall and provided the pier excavation is dry. Temporary casing may be removed as the concrete is placed into the drilled shaft keeping a concrete head of at least two feet above the bottom of the casing as it is being removed.

Shafts should be clean and be free of all loose materials prior to placement of concrete. The drilled shafts should be installed in accordance with the guidelines provided in FHWA-IF-99-025. A PSI representative should verify the bearing stratum, bearing depth, bearing soil condition, and bearing area and that the pier installation procedures meet the specifications.

LATERAL CAPACITIES

To assess the deflection, moment and shear capacity of the CIDH piers, the computer software program L-Pile by Ensoft, Inc. was utilized. The analyses were performed for a 24 inch diameter CIDH pier for varying lengths. For the analyses, the bottom of the pile cap was assumed to be at finished grade (no pile cap). Lateral capacities were developed for both free and fixed-head pile conditions. In our analysis, we assumed the soil within the upper 10 feet of grade will have no lateral support capacity. The computer output files for those analyses are included within the Appendix. Once the specific foundation type, dimensions and structural detailing is known, the lateral pile capacities can be re-evaluated.

INTERIOR FLOOR SLABS

The proposed structures may incorporate a conventional slab-on-grade provided the subgrade is prepared as previously recommended. The on-grade floor slabs should be supported on Engineered Fill. Soft or otherwise unsuitable areas observed should be addressed on a case-by-case basis by our Geotechnical Engineer. Although the slab thickness and steel reinforcement should be determined by the structural engineer, we recommend the floor slab possess a minimum thickness of 5 inches.

Where concrete slabs are designed as beams on an elastic foundation, the subgrade should be assumed to have a modulus of subgrade reaction (k-value) of 150 pounds per cubic inch (pci), based on a one foot square plate bearing test. Dependent on how the floor slab load is applied, the above subgrade modulus value may need to be geometrically adjusted.

If reducing moisture vapor transmission is a design consideration, we would recommend a vapor retarding membrane be included in the design. Membrane specification should be provided by manufacturer. Vapor retarders should be installed in accordance with ACI 302.1, Chapter 3. A capillary break material (sand) should be provided beneath the vapor retarder.

The precautions listed below should be followed closely for construction of all slabs-on-grade. These details will not reduce the amount of movement, but are intended to reduce potential damage should some settlement of the supporting subgrade take place.

- Cracking of slabs-on-grade is normal and should be expected. Cracking can occur not only because of heaving or compression of the supporting soil, but also because of concrete curing stresses. The occurrence of concrete shrinkage cracks, and problems associated with concrete curing may be reduced and/or controlled by limiting the water/cement ratio of the concrete, proper concrete placement, finishing, and curing, and by the placement of crack control joints at frequent intervals, particularly, where re-entrant slab corners occur. The American Concrete Institute (ACI) recommends a maximum panel size (in feet) equal to approximately three times the thickness of the slab (in inches) in both directions. For example, joints are recommended at a maximum spacing of 12 feet, assuming a four-inch thick slab. We recommend also that control joints be scored three feet in from, and parallel to, the foundation walls. Using fiber reinforcement in the concrete can also control shrinkage cracking.
- Some increase in moisture content is inevitable because of development and associated landscaping. However, extreme moisture content increases can be largely controlled by proper and responsible site drainage, building maintenance and irrigation practices.
- Exterior slabs should be isolated from the building. These slabs should be reinforced to function as independent units. Movement of these slabs should not be transmitted to the building foundation or superstructure.

PAVEMENT DESIGN

The recommended thicknesses presented below are considered typical and minimum for the assumed parameters. We understand that budgetary considerations sometimes warrant thinner pavement sections than those presented. However, the client, the owner, and the project principals should be aware that thinner pavement sections might result in increased maintenance costs and lower than anticipated pavement life.

In designing the proposed paved areas, the existing subgrade conditions must be considered together with the expected traffic use and loading conditions.

The conditions that will influence the pavement design can be summarized as follows:

1) Subgrade support characteristics of the subgrade. This is typically represented by a R-Value for the design of flexible pavements in this region.

2) Vehicular traffic, in terms of the number and frequency of vehicles and their range of axle loads.

3) Probable increase in vehicular use over the life of the pavement.

We recommend that the exposed subgrade be prepared in accordance with the site preparation requirements specified previously in this report. The upper one foot of pavement subgrade should be compacted to at least 95 percent of the maximum dry density as determined by the modified Proctor (ASTM D1557). The fill moisture content at the time of compaction should be within 1 to 3 percent above the optimum moisture content value. Undercut soil should be replaced by Engineered Fill.

The appropriate pavement section depends primarily upon the type of subgrade soil, shear strength, traffic load, and planned pavement life. For preliminary purposes, we have assumed Traffic Indices of TI=5.0 for parking areas and TI=6.5 for those driveway and truck lanes subject to relatively heavy traffic. These assumed traffic indices should be verified by the project civil engineer prior to construction. Based on the soils encountered within our test borings, we have assumed an R-value of 30 for the near-surface soils within pavement areas. Since an evaluation of the characteristics of the actual soils at pavement subgrade can only be provided at the completion of grading, the following pavement sections should be used for planning purposes only. Final pavement designs should be evaluated after R-value tests have been performed on the actual subgrade material.

It should be noted that additional earthwork and/or ground improvement efforts may be required during grading on the actual subgrade material, in order to achieve the aforementioned design parameters and assumptions. These design thicknesses assume that a properly prepared subgrade has been achieved.

Table 4: Flexible Pavement Recommendations

Pavement Loading Conditions	Assumed Traffic Index	Recommended Pavement Section
Standard Duty (Parking Areas)	5.0	3 inches AC over 6-inches Class II Aggregate Base
Heavy Duty (Drive Aisles)	6.5	4 inches AC over 8-inches Class II Aggregate Base

Concrete pavement is recommended in areas that receive continuous repetitive traffic such as loading areas and parking lot entrances. Due to heavy wheel loads and impact loads, concrete approach aprons and dumpster pads, should have a minimum thickness of 6 inches, with an underlying 4-inch thick section of Class II Aggregate Base (AB). Portland Cement Concrete pavement sections should incorporate appropriate steel reinforcement and crack control joints as designed by the project structural engineer. We recommend that sections be as nearly squared as possible and no more than 15-feet on a side. A minimum 3,500 psi mix is recommended. The actual design should also be in accordance with design criteria specified by the governing jurisdiction.

Asphalt Concrete (AC), Portland Cement Concrete, and Class II aggregate base should conform to and be placed in accordance with the latest revision of the California Department of Transportation Standard Specifications and American Concrete Institute (ACI) codes. Aggregate base should be compacted to a minimum of 95 percent of the maximum dry density as determined by the modified Proctor (ASTM D1557) prior to placement of AC. Subgrade preparation for pavement areas is included in the Site Preparation section of this report.

CONSTRUCTION CONSIDERATIONS

Moisture Sensitive Soils/Weather Related Concerns

Note that the upper soils are sensitive to disturbances caused by construction traffic and to changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. Furthermore, perched groundwater conditions can develop during periods of heavy rainfall as a result of less permeable layers impeding infiltration. In these instances, overlying subgrade soils may become unstable and require remedial measures. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather.

Groundwater was measured to be approximately 5-feet below existing ground surface elevation. It should be noted, however, that variations in the groundwater table may result from fluctuation in the ground surface topography, subsurface stratification, precipitation, irrigation, and other factors that may not have evident at the time of our exploration. This sometimes occurs where relatively impermeable and/or cemented materials are overlain by fill soils. We recommend that a representative of PSI be present during grading operations to evaluate areas of seepage. Drainage devices for reduction of water accumulation can be recommended if these conditions occur.

Water should not be allowed to collect in the foundation excavation, on floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff. Positive site drainage should be provided to reduce infiltration of surface water around the perimeter of the building and beneath the floor slabs. The grades should be sloped away from the building and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill and floor slab areas of the building.

Corrosive Soil Concerns

The corrosive testing on a representative sample of the site soils indicates that the soils possess a negligible sulfate exposure. Based on this result, it is our opinion that special sulfate-resistant concrete mix designs are not warranted and Type II cement may be used. Additional testing should be performed during site grading to assess the sulfate content of the as-graded soils.

The resistivity results indicate a progressively less corrosive environment for metal pipes. We suggest that a corrosion engineer be consulted to determine what corrosion protection may be warranted at this site.

PLAN REVIEW

Once final design plans and specifications are available, a review of grading and foundation plans by PSI is recommended as a means to check that the evaluations made in preparation of this report are correct and that earthwork and foundation recommendations have been properly interpreted and implemented.

OBSERVATION AND TESTING DURING CONSTRUCTION

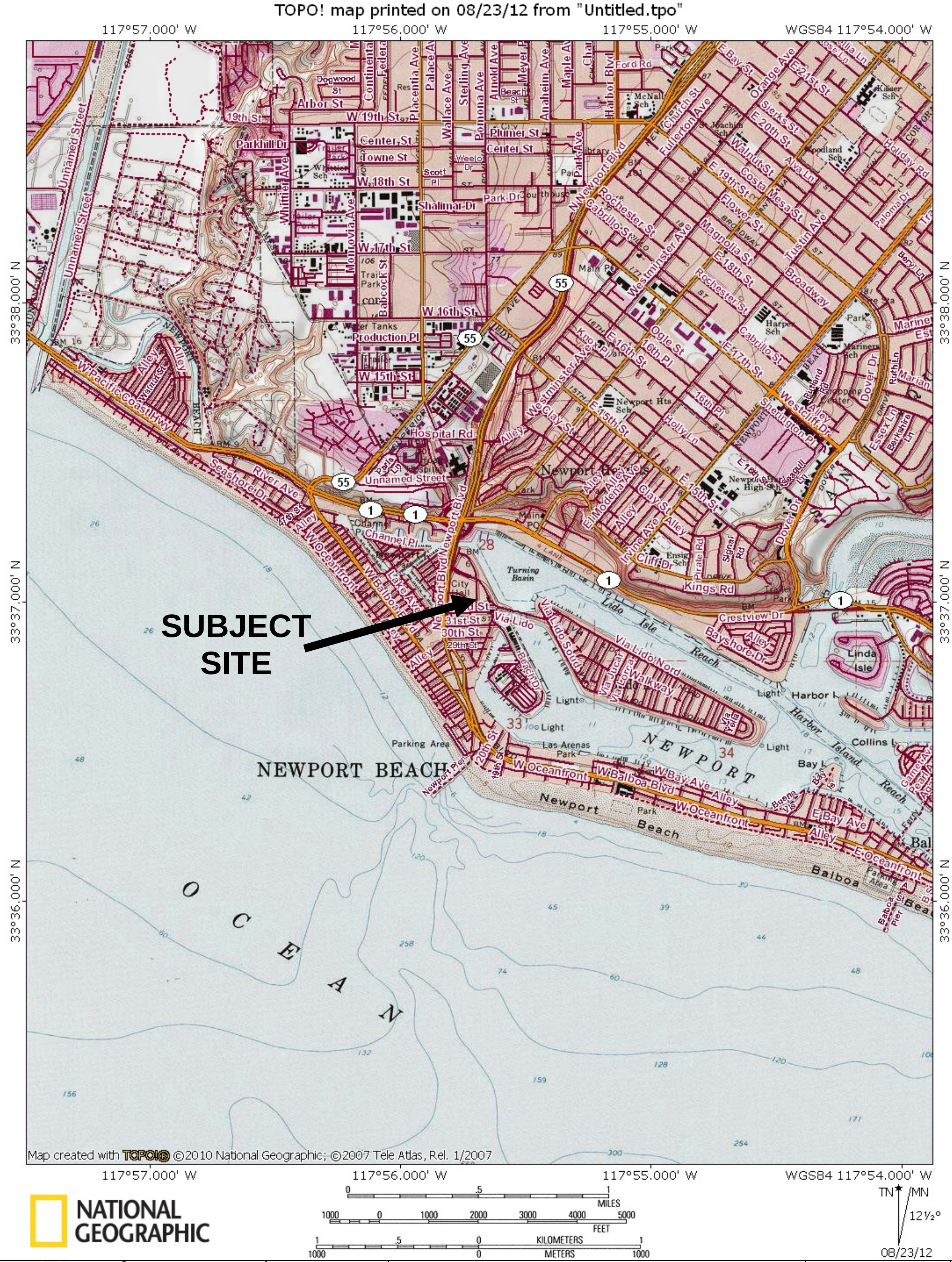
It is recommended that PSI be retained to provide observation and testing services during for newly proposed construction. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

REPORT LIMITATIONS

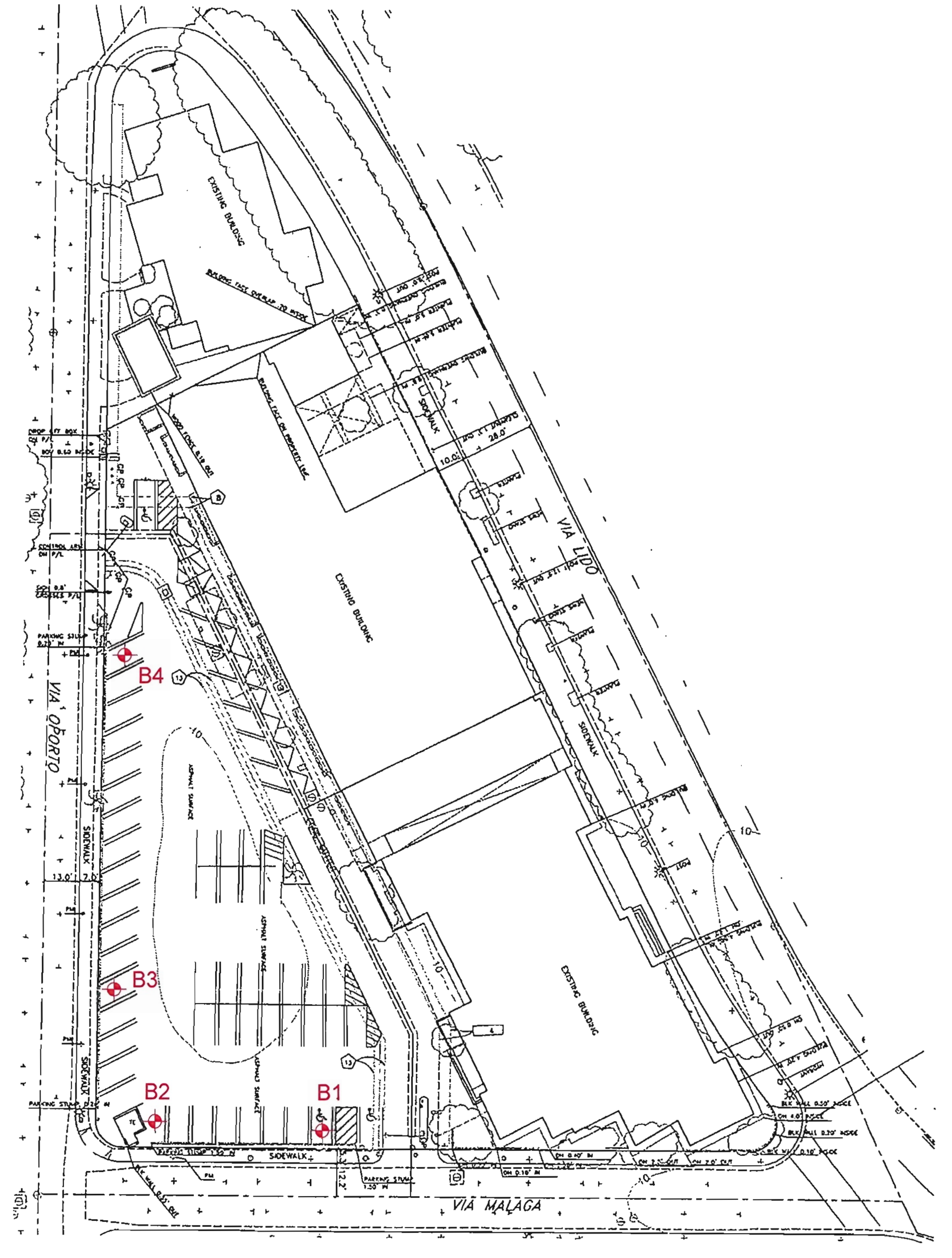
The proposed professional services have been performed, findings obtained, and recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices at the time of this report. PSI is not responsible for the conclusions, opinions, or recommendations made by others based on this data. No other warranties are implied or expressed. The Wieland-Davco Corp., its subsidiaries and affiliates can rely upon the report under the same terms as if it was originally prepared for them.

The scope of exploration was intended to evaluate soil conditions within the influence of the proposed foundations. The analyses and recommendations submitted in this report are based upon the data obtained from the soil borings performed at the locations indicated. If any subsoil variations become evident during the course of this project, a re-evaluation of the recommendations contained in this report will be necessary after we have had an opportunity to observe the characteristics of the conditions encountered. The applicability of the report should also be reviewed in the event significant changes occur in the design, nature, or location of the proposed improvements.

APPENDIX



psi Information To Build On Engineering • Consulting • Testing	DATE: 8/23/12	PROPOSED TOWNHOMES/CONDOS NWC OF VIA LIDO & VIA MALAGA IRVINE, CA	PSI PROJECT NUMBER: 0559771
PSI, INC. 6330 GATEWAY DRIVE, SUITE B CYPRESS, CALIFORNIA	DRAWN BY: ZM	SITE VICINITY MAP	FIGURE 1



LEGEND:

B4 APPROXIMATE BORING LOCATION

0 60 120

APPROXIMATE SCALE IN FEET

psi Information To Build On

Engineering · Consulting · Testing

6330 Gateway Drive, Suite B
Cypress, California 90630
(714) 484-8600

DATE:	PROPOSED NEWPORT BEACH TOWNHOMES 3303-3315 VIA LIDO NEWPORT BEACH, CALIFORNIA	PROJECT No.:
8/13/2012		0559771
DRAWN BY:	BORING LOCATION MAP	FIGURE:
EFD		2

N

Professional Service Industries, Inc.
6330 Gateway Drive, Suite B
Cypress, CA 90630
Telephone: (714) 484-8600
Fax: (714) 484-8601

LOG OF BORING B1

PSI Job No.:	0559771	Drilling Method:	Mud Rotary
Project:	Proposed Townhome/Condo Development	Sampling Method:	2-in SS
Location:	NWC Via Malaga & Via Lido	Hammer Type:	Automatic
	Newport Beach, CA	Boring Location:	

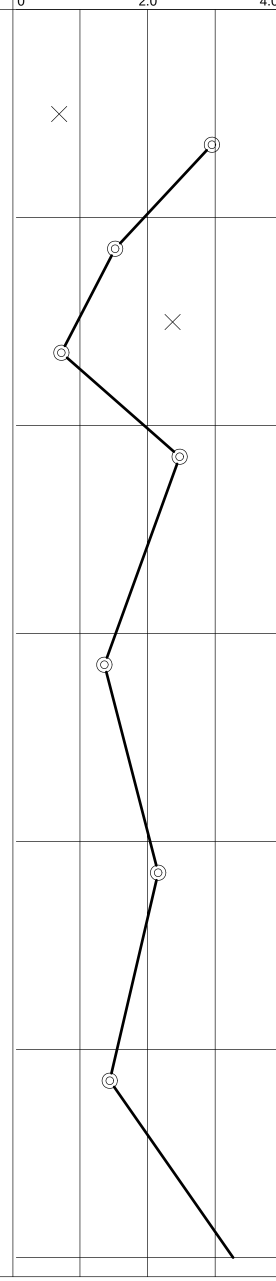
WATER LEVELS

While Drilling	feet
Upon Completion	5 feet
Delay	N/A

Station: N/A
Offset: N/A

Elevation (feet)	Depth, (feet)	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	0				**ASPHALT**, 3 inches	AC			
					Silty SAND, light brown, damp Apparent Base	SM			
					Silty SAND with Gravel, trace organics, light to dark brown, wet, medium dense	SM		9	DD = 128 pcf
		Calif. Sampler	1	18			14/15/22 N=37		
	5	Calif. Sampler	2	6	**Sandy GRAVEL**, light to dark brown, subangular gravel sand mixture, trace organics	GW	5/7/12 N=19		
					Silty SAND, trace gravel, trace organics, grey-brown, wet, loose			30	DD = 93 pcf
		Calif. Sampler	3	18			1/3/6 N=9		
	10	Calif. Sampler	4	18	medium dense		7/13/18 N=31		
	15	Split-Spoon	5	18			6/8/9 N=17		Fines=6.0%
	20	Split-Spoon	6	18			8/12/15 N=27		
	25	Split-Spoon	7	18			7/7/11 N=18		Fines=5.1%
	30					SM/SP			

STANDARD PENETRATION TEST DATA
N in blows/ft ◎
× Moisture ◪ PL ✚ LL
0 25 50
STRENGTH, tsf
▲ Qu ✳ Qp
0 2.0 4.0



Continued Next Page

Completion Depth:	51.5 ft
Date Boring Started:	8/13/12
Date Boring Completed:	8/13/12
Logged By:	ZM
Drilling Contractor:	Cal-Pac

Sample Types: Auger Cutting, Split-Spoon, Rock Core, Shelby Tube, Hand Auger, Calif. Sampler, Texas Cone

Latitude: 33.616706°
Longitude: -117.928053°
Drill Rig: CME 75
Remarks:

The stratification lines represent approximate boundaries. The transition may be gradual.

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Cypress, CA 90630
Telephone: (714) 484-8600
Fax: (714) 484-8601

LOG OF BORING B1

PSI Job No.: 0559771
Project: Proposed Townhome/Condo Development
Location: NWC Via Malaga & Via Lido
Newport Beach, CA

Drilling Method: Mud Rotary
Sampling Method: 2-in SS
Hammer Type: Automatic
Boring Location:

WATER LEVELS

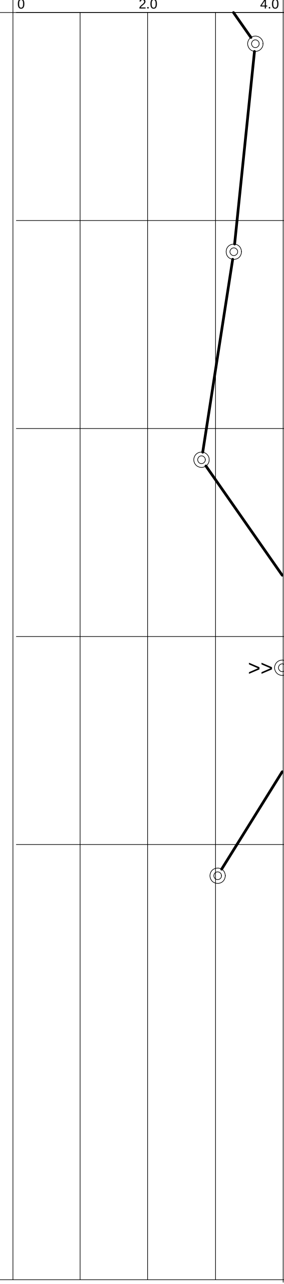
While Drilling	feet
Upon Completion	5 feet
Delay	N/A

Station: N/A
Offset: N/A

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	30		Split-Spoon	8	18	**Silty SAND**, trace gravel, trace organics, grey-brown, wet, loose dense		13/19/26 N=45		
	35		Split-Spoon	9	18			14/18/23 N=41		
	40		Split-Spoon	10	18			10/15/20 N=35		Fines=5.0%
	45		Split-Spoon	11	18	very dense		17/26/36 N=62		
	50		Split-Spoon	12	18	dense		16/22/16 N=38		

Boring Terminated at 51.5 feet
Groundwatermeasured at approximately 5 ft after drilling
Boring backfilled with bentonite and capped with asphalt

STANDARD PENETRATION TEST DATA
N in blows/ft
Moisture · PL · LL
0 · 25 · 50
STRENGTH, tsf
Qu · Qp
0 · 2.0 · 4.0



Completion Depth: 51.5 ft
Date Boring Started: 8/13/12
Date Boring Completed: 8/13/12
Logged By: ZM
Drilling Contractor: Cal-Pac

Sample Types: Auger Cutting, Split-Spoon, Rock Core, Shelby Tube, Hand Auger, Calif. Sampler, Texas Cone

Latitude: 33.616706°
Longitude: -117.928053°
Drill Rig: CME 75
Remarks:

The stratification lines represent approximate boundaries. The transition may be gradual.

Professional Service Industries, Inc.
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LOG OF BORING B2

Sheet 1 of 2

PSI Job No.:	0559771	Drilling Method:	Mud Rotary
Project:	Proposed Townhome/Condo Development	Sampling Method:	2-in SS
Location:	NWC Via Malaga & Via Lido	Hammer Type:	Automatic
	Newport Beach, CA	Boring Location:	

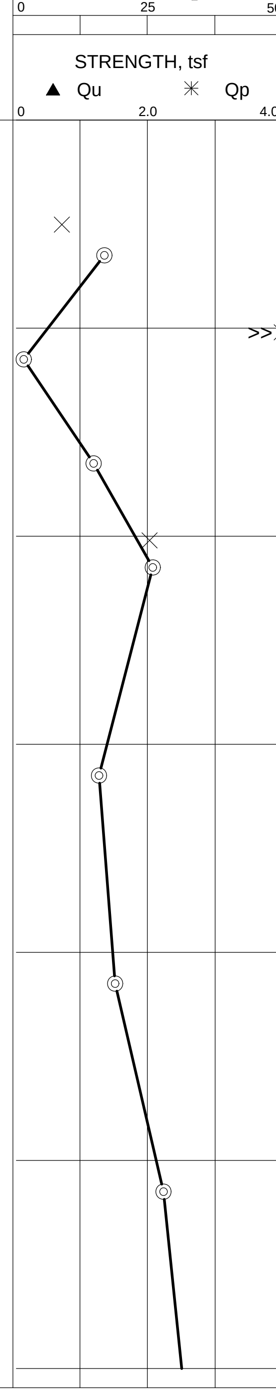
WATER LEVELS

While Drilling	feet
Upon Completion	5 feet
Delay	N/A

Station: N/A
Offset: N/A

Elevation (feet)	Depth, (feet)	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	0				**ASPHALT**, 3 inches	AC			
					Silty SAND, trace gravels, light brown, moist Apparent Base	SM			
					Silty SAND, trace organics, light to dark brown, wet, medium dense	SM		9	DD = 99 pcf
			1	18			7/8/9 N=17		
	5		2	18	**Clayey SILT**, trace organics, grey-brown, wet, very soft	MH	2/1/1 N=2	54	DD = 61 pcf
					Silty SAND, trace organics, grey-brown, wet, medium dense	SM/SP			Fines=8.8%
			3	18			3/5/10 N=15		
	10		4	18			5/10/16 N=26	25	DD = 102 pcf Fines=10.1%
	15		5	18			6/7/9 N=16		
	20		6	18			8/8/11 N=19		
	25		7	18			7/12/16 N=28		
	30								

STANDARD PENETRATION TEST DATA
N in blows/ft
Moisture PL LL
STRENGTH, tsf
Qu Qp



Continued Next Page

Completion Depth:	31.5 ft
Date Boring Started:	8/13/12
Date Boring Completed:	8/13/12
Logged By:	ZM
Drilling Contractor:	Cal-Pac

Sample Types: Auger Cutting, Split-Spoon, Rock Core, Shelby Tube, Hand Auger, Calif. Sampler, Texas Cone

Latitude: 33.616706°
Longitude: -117.928053°
Drill Rig: CME 75
Remarks:

The stratification lines represent approximate boundaries. The transition may be gradual.

Professional Service Industries, Inc.
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LOG OF BORING B2

PSI Job No.: 0559771
Project: Proposed Townhome/Condo Development
Location: NWC Via Malaga & Via Lido, Newport Beach, CA

Drilling Method: Mud Rotary
Sampling Method: 2-in SS
Hammer Type: Automatic
Boring Location:

WATER LEVELS	
While Drilling	feet
Upon Completion	5 feet
Delay	N/A

Station: N/A
Offset: N/A

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	STANDARD PENETRATION TEST DATA (N in blows/ft; Moisture; PL; LL; 0 – 25 – 50) / STRENGTH, tsf (Qu; Qp; 0 – 2.0 – 4.0)	Additional Remarks
	30		Split-Spoon	8	18	**Silty SAND**, trace organics, grey-brown, wet, medium dense dense		13/16/16 N=32			
						Boring Terminated at 31.5 ft Groundwater measured at approximately 5 ft after drilling Boring backfilled with bentonite and capped with ashpalt					

Completion Depth: 31.5 ft
Date Boring Started: 8/13/12
Date Boring Completed: 8/13/12
Logged By: ZM
Drilling Contractor: Cal-Pac

Sample Types: Auger Cutting, Split-Spoon, Rock Core, Shelby Tube, Hand Auger, Calif. Sampler, Texas Cone

Latitude: 33.616706°
Longitude: -117.928053°
Drill Rig: CME 75
Remarks:

The stratification lines represent approximate boundaries. The transition may be gradual.

Professional Service Industries, Inc.
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LOG OF BORING B3

PSI Job No.: 0559771
Project: Proposed Townhome/Condo Development
Location: NWC Via Malaga & Via Lido
Newport Beach, CA

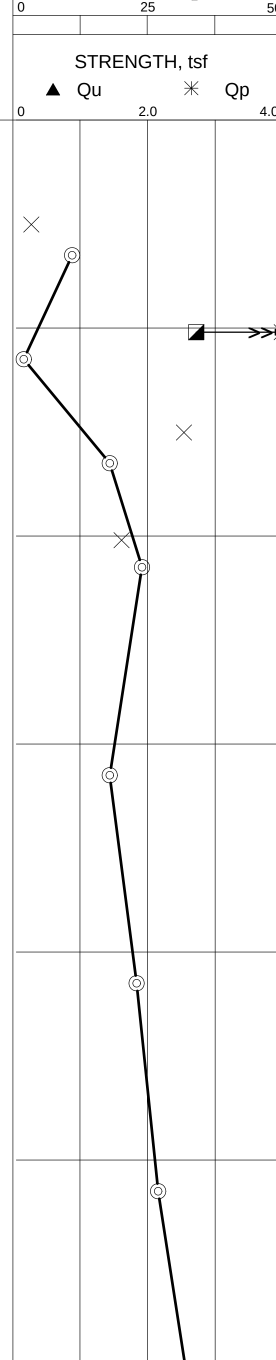
Drilling Method: Mud Rotary
Sampling Method: 2-in SS
Hammer Type: Automatic
Boring Location:

WATER LEVELS	
While Drilling	feet
Upon Completion	5 feet
Delay	N/A

Station: N/A
Offset: N/A

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	0			**ASPHALT**, 3 inches	AC			
				Silty SAND, trace gravels, light brown, moist Apparent Base	SM			
				Silty SAND, trace organics, light to dark brown, wet, loose	SM		3	DD = 105 pcf
		1	18			5/5/6 N=11		
	5			**Clayey SILT**, trace organics, grey-brown, very soft	MH		85	DD = 68 pcf LL = 66 PL = 34
		2	18			1/1/1 N=2		
				Silty SAND, trace organics, grey-brown, wet, medium dense	SM/SP		32	DD = 93 pcf
		3	18			3/8/10 N=18		
	10						20	DD = 109 pcf
		4	18			4/9/15 N=24		
	15	5	18			6/8/10 N=18		
	20	6	18			7/8/15 N=23		
	25	7	18			8/13/14 N=27		
	30							

STANDARD PENETRATION TEST DATA
N in blows/ft
Moisture
PL
LL
STRENGTH, tsf
Qu
Qp



Continued Next Page

Completion Depth: 31.5 ft
Date Boring Started: 8/13/12
Date Boring Completed: 8/13/12
Logged By: ZM
Drilling Contractor: Cal-Pac

Sample Types: Auger Cutting, Split-Spoon, Rock Core, Shelby Tube, Hand Auger, Calif. Sampler, Texas Cone

Latitude: 33.616706°
Longitude: -117.928053°
Drill Rig: CME 75
Remarks:

The stratification lines represent approximate boundaries. The transition may be gradual.

Professional Service Industries, Inc.
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Cypress, CA 90630
Telephone: (714) 484-8600
Fax: (714) 484-8601

LOG OF BORING B3

PSI Job No.:	0559771	Drilling Method:	Mud Rotary
Project:	Proposed Townhome/Condo Development	Sampling Method:	2-in SS
Location:	NWC Via Malaga & Via Lido	Hammer Type:	Automatic
	Newport Beach, CA	Boring Location:	

WATER LEVELS

While Drilling	feet
Upon Completion	5 feet
Delay	N/A

Station: N/A
Offset: N/A

STANDARD PENETRATION TEST DATA: N in blows/ft ◎; × Moisture; PL; LL (0 – 25 – 50). STRENGTH, tsf: ▲ Qu; ✳ Qp (0 – 2.0 – 4.0)

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	30		Split-Spoon	8	18	**Silty SAND**, trace organics, grey-brown, wet, medium dense dense		11/16/17 N=33		
						Boring Terminated at 31.5 ft Groundwater measured at approximately 5 ft after drilling Boring backfilled with bentonite and capped with asphalt				

Completion Depth:	31.5 ft
Date Boring Started:	8/13/12
Date Boring Completed:	8/13/12
Logged By:	ZM
Drilling Contractor:	Cal-Pac

Sample Types: Auger Cutting; Split-Spoon; Rock Core; Shelby Tube; Hand Auger; Calif. Sampler; Texas Cone

Latitude: 33.616706°
Longitude: -117.928053°
Drill Rig: CME 75
Remarks:

The stratification lines represent approximate boundaries. The transition may be gradual.

Professional Service Industries, Inc.
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Cypress, CA 90630
Telephone: (714) 484-8600
Fax: (714) 484-8601

LOG OF BORING B4

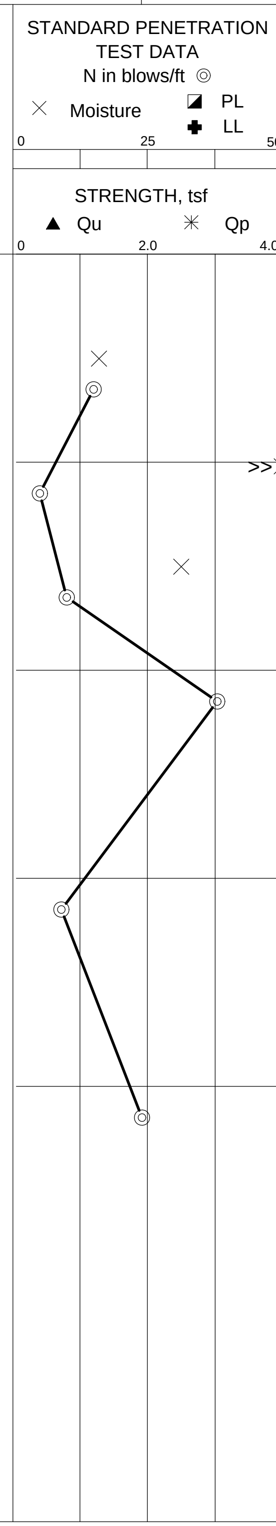
PSI Job No.:	0559771	Drilling Method:	Mud Rotary
Project:	Proposed Townhome/Condo Development	Sampling Method:	2-in SS
Location:	NWC Via Malaga & Via Lido, Newport Beach, CA	Hammer Type:	Automatic
		Boring Location:	

WATER LEVELS

While Drilling	feet
Upon Completion	5 feet
Delay	N/A

Station: N/A
Offset: N/A

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	0			**ASPHALT**, 3 inches	AC			
				Silty SAND, trace gravels, light brown, moist Apparent Base	SM			
				Silty SAND, trace organics, light to dark brown, wet, loose	SM		16	DD = 93 pcf
		1	18			6/7/8 N=15		
	5	2	18	**Clayey SILT**, trace organics, grey-brown, wet, soft	MH	1/2/3 N=5	87	DD = 59 pcf
		3	18	**Silty SAND**, trace organics, grey-brown, wet, loose	SM/SP	4/5/5 N=10	31	DD = 92 pcf
	10	4	18	medium dense		11/18/20 N=38		
	15	5	18	loose		4/4/5 N=9		
	20	6	18	medium dense		6/11/13 N=24		
				Boring Terminated at 21.5 ft Groundwater measured at approximately 5 ft after drilling Boring backfilled with bentonite and capped with asphalt				



Completion Depth:	21.5 ft	Latitude:	33.616706°
Date Boring Started:	8/13/12	Longitude:	-117.928053°
Date Boring Completed:	8/13/12	Drill Rig:	CME 75
Logged By:	ZM	Remarks:	
Drilling Contractor:	Cal-Pac		

Sample Types: Auger Cutting, Split-Spoon, Rock Core, Shelby Tube, Hand Auger, Calif. Sampler, Texas Cone

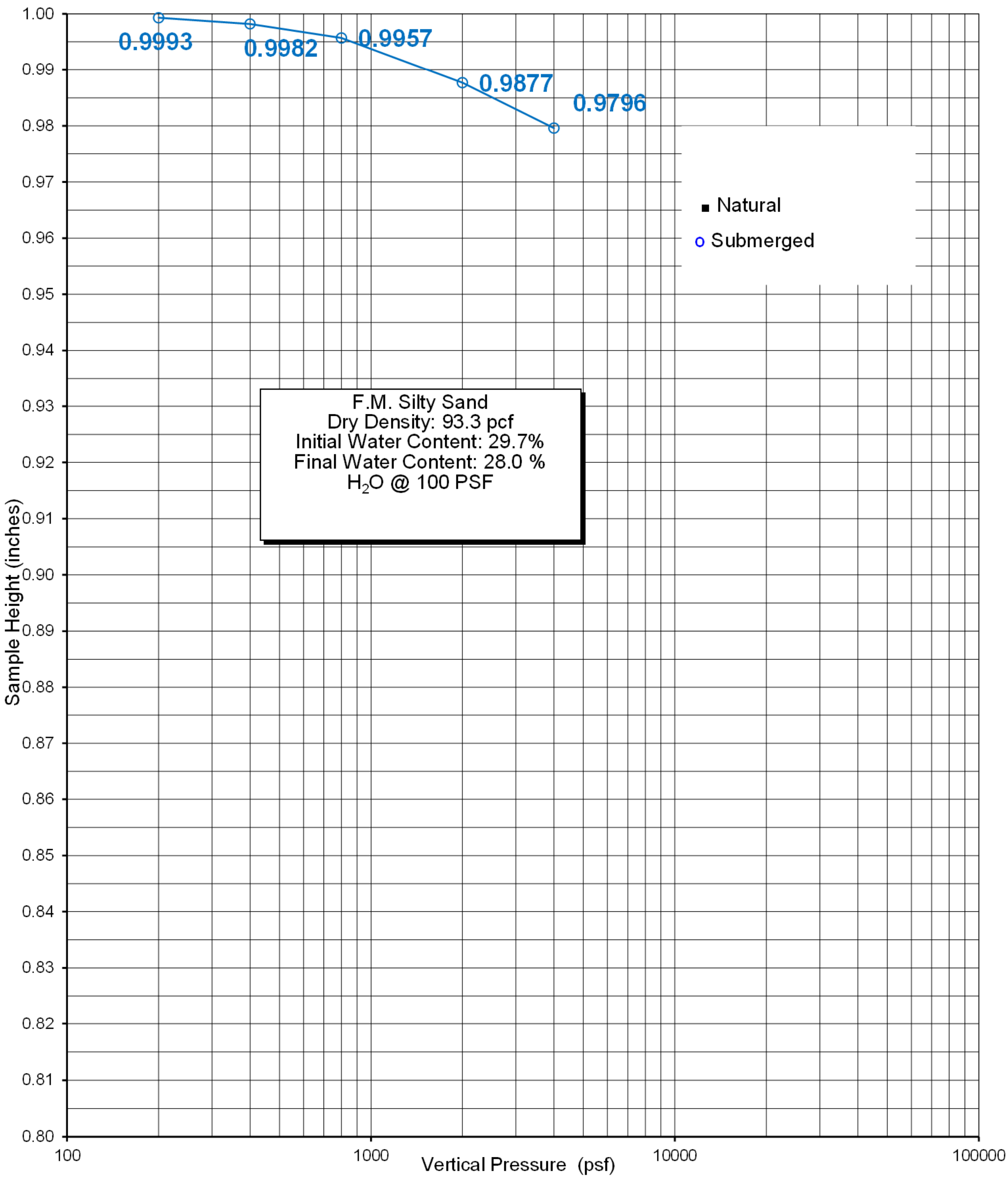
The stratification lines represent approximate boundaries. The transition may be gradual.

PSI # 559-771

CONSOLIDATION TEST - ASTM D2435

Job No. 2008-026

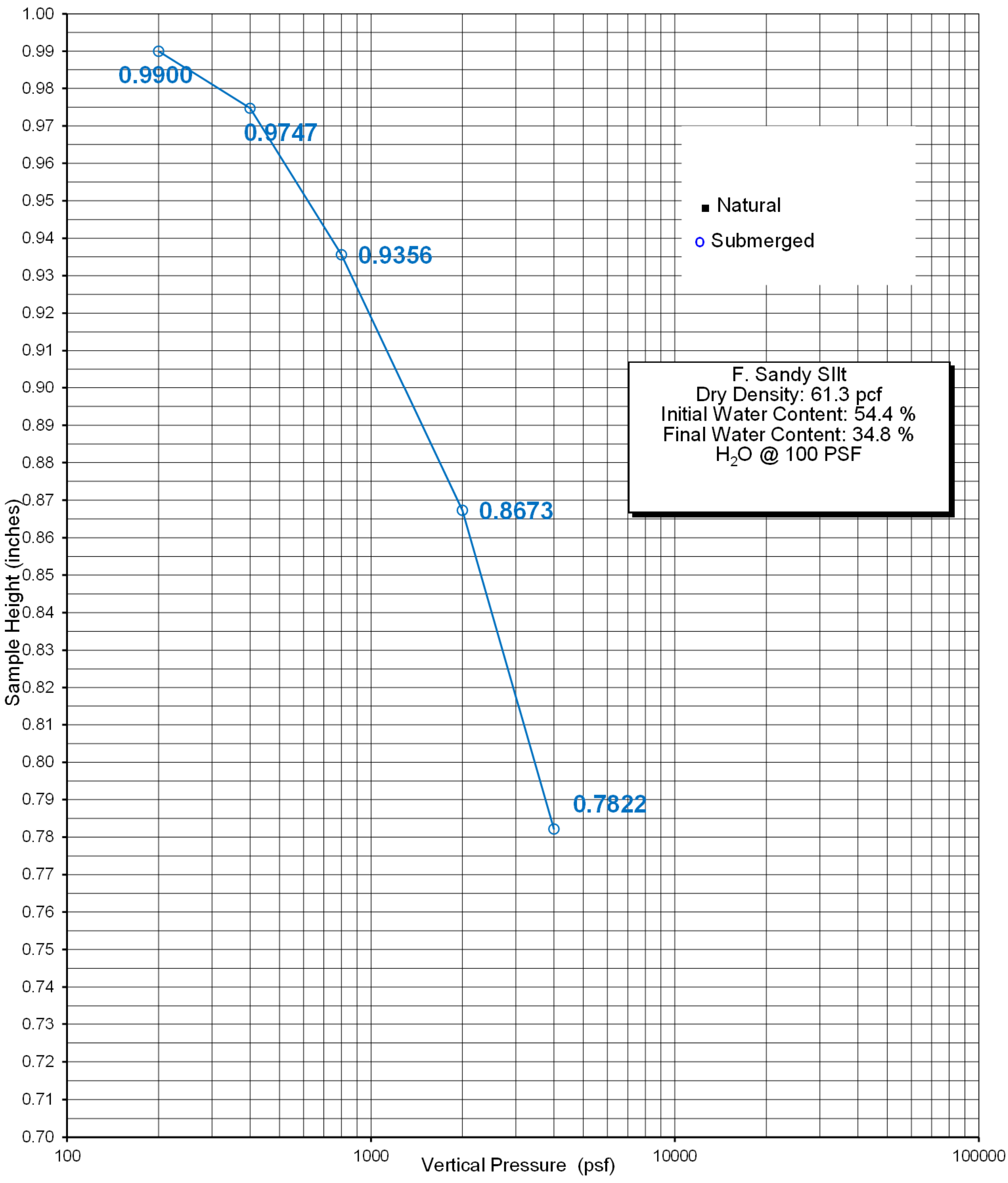
Boring / Sample No. B-1	Depth: 7.5'	Date 08-15-12



Geo-Logic ASSOCIATES

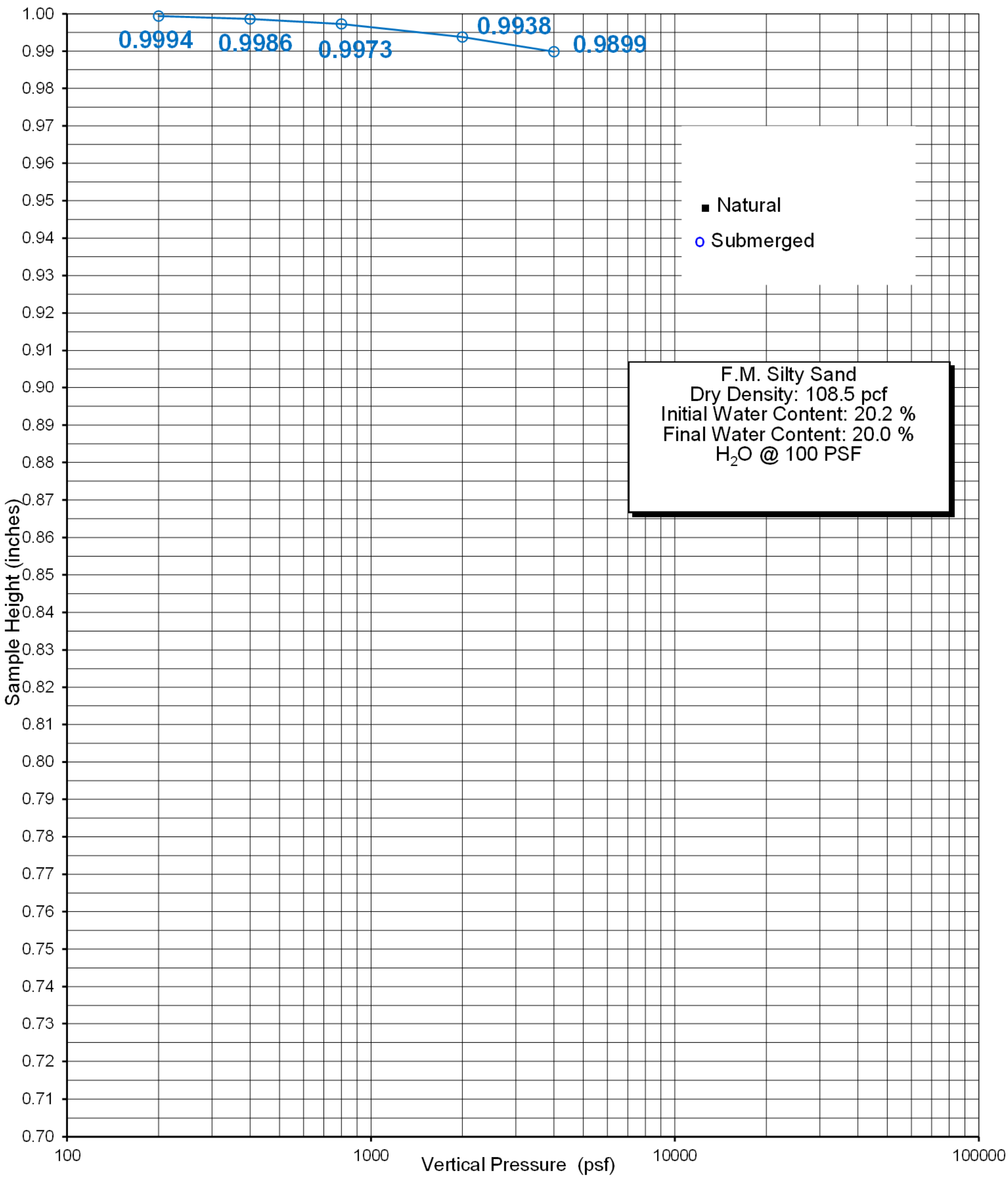
CONSOLIDATION TEST - ASTM D2435

Boring / Sample No. B-2	Depth: 5.0'	Date 08-15-12



CONSOLIDATION TEST - ASTM D2435

Boring / Sample No.	B-3	Depth:	10.0'	Date	08-15-12



SAMPLE NO.:	`	B-4 @ 0-3'			
DESCRIPTION		F.M. Sand			
DIRECT SHEAR TEST (type)					
Initial Moisture Content %					
Dry Density (pcf)					
Normal Stress (psf)					
Peak Shear Stress (psf)					
Ultimate Shear Stress (psf)					
Cohesion (psf)					
Internal Friction Angle (degrees)					
EXPANSION TEST UBC STD 18-2					
Initial Dry Density (pcf)					
Initial Moisture Content %					
Final Moisture Content %					
Pressure (psf)					
Expansion Index / Swell %					
CORROSIVITY TEST					
Resistivity (CTM643) (ohm-cm)		11150			
pH (CTM643)		7.8			
CHEMICAL TESTS					
Soluble Sulfate (CTM 417) (ppm)		156			
Chloride Content (CTM 422) (ppm)		99			
Wash #200 Sieve (ASTM-1140) %					
Sand Equivalent (ASTM D2419)					

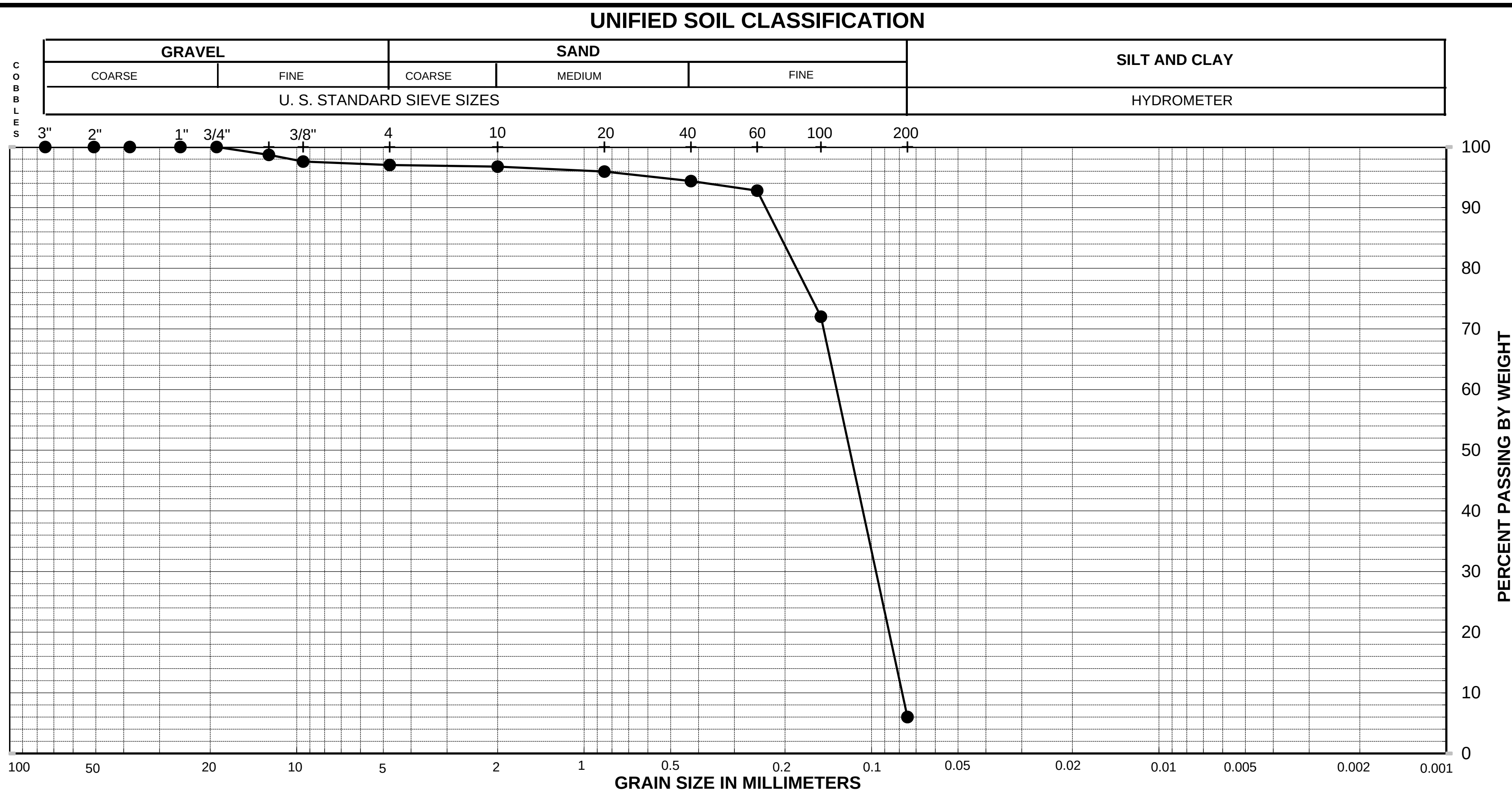
Geo-Logic ASSOCIATES

EXPANSION INDEX - UBC 18-2 & ASTM D 4829-88

PROJECT PSI # 559-771 JOB NO. 2008-026

Sample	B-4 @ 0-3.0'		By	LD	Sample			By	
Sta. No.					Sta. No.				
Soil Type	Brown, F.M. Sand				Soil Type				
Date	Time	Dial Reading	Wet+Tare	595.4	Date		Dial Reading	Wet+Tare	
8/20/2012	15:00	0.1253	Tare	219.6				Tare	
		H2O	Net Weight	375.8				Net Weight	
8/21/2012	10:30	0.1259	% Water	12.5				% Water	
				101.2				Dry Dens.	
			% Max					% Max	
			Wet+Tare	619.5				Wet+Tare	
			Tare	219.6				Tare	
			Net Weight	399.9				Net Weight	
INDEX	-1	-0.1%	% Water	19.7	**INDEX**			% Water	

Sample			By		Sample			By	
Sta. No.					Sta. No.				
Soil Type					Soil Type				
Date		Dial Reading	Wet+Tare		Date		Dial Reading	Wet+Tare	
			Tare					Tare	
			Net Weight					Net Weight	
			% Water					% Water	
			Dry Dens.					Dry Dens.	
			% Max					% Max	
			Wet+Tare					Wet+Tare	
			Tare					Tare	
			Net Weight					Net Weight	
INDEX			% Water		**INDEX**			% Water	



Exploration No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% Clay	Description and Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
B1		15.0	●					**Silty Sand with trace organics**	0.14	0.09	0.08	1.8	0.7
			◆										
			■										
			❍										
			□										

PROJECT NAME: **Newport Beach, CA Townhomes/Condos**
PROJECT NUMBER: **0559771**

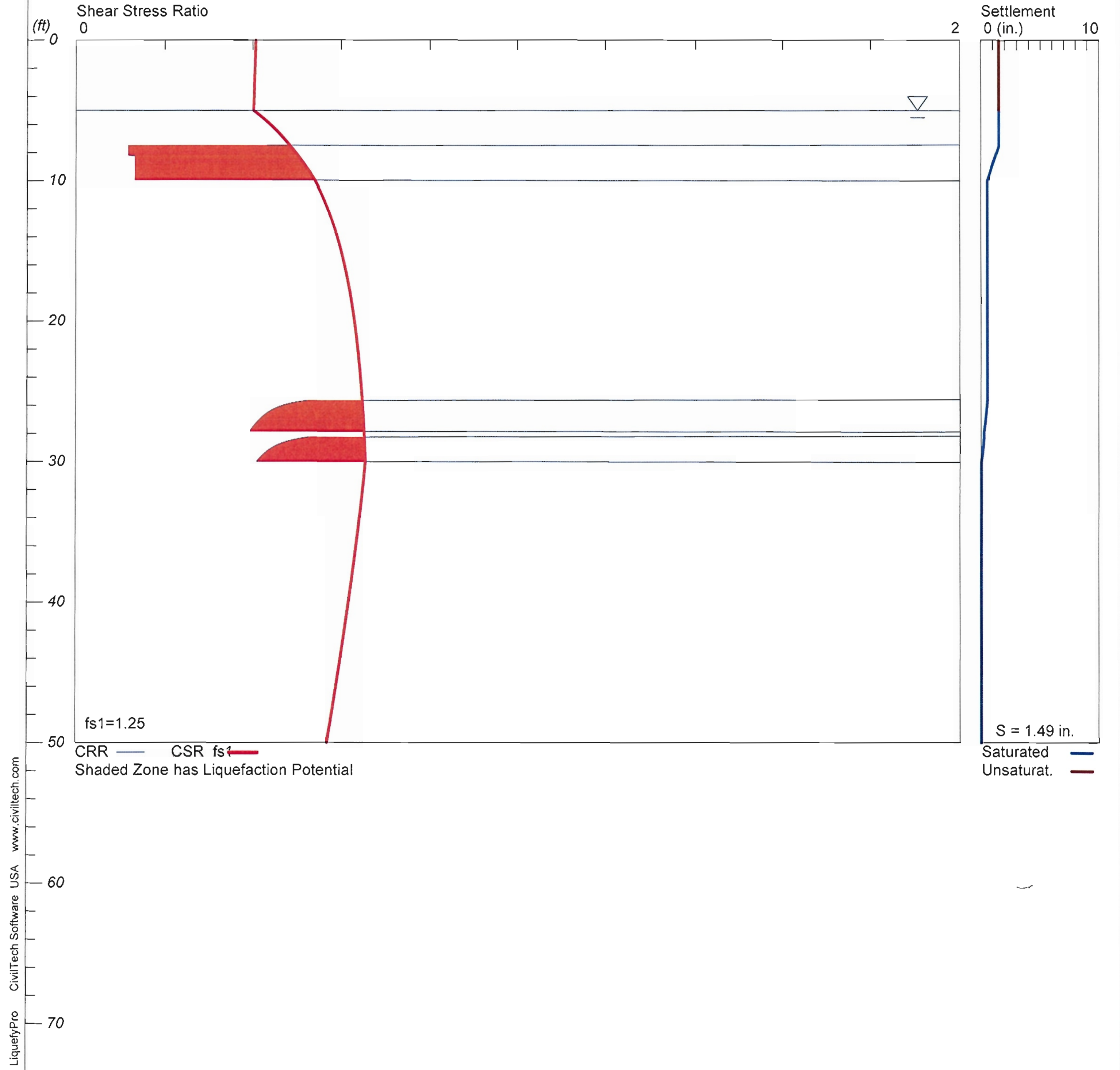
PARTICLE-SIZE DISTRIBUTION CURVES

LIQUEFACTION ANALYSIS

Newport Beach - Townhomes/Condos

Hole No.= Water Depth=5 ft

Magnitude=7.3
Acceleration=0.5g



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LIQUEFACTION ANALYSIS SUMMARY

****

Font: Courier New, Regular, Size 8 is recommended for this report.
Licensed to , 8/23/2012 4:16:10 PM

Input File Name: P:\056\2012 Projects\0559771 Newport Beach Townhomes\liquefy.liq
Title: Newport Beach - Townhomes/Condos
Subtitle: 0559771

Surface Elev.=
Hole No.=
Depth of Hole= 50.00 ft
Water Table during Earthquake= 5.00 ft
Water Table during In-Situ Testing= 0.00 ft
Max. Acceleration= 0.5 g
Earthquake Magnitude= 7.30

Input Data:
Surface Elev.=
Hole No.=
Depth of Hole=50.00 ft
Water Table during Earthquake= 5.00 ft
Water Table during In-Situ Testing= 0.00 ft
Max. Acceleration=0.5 g
Earthquake Magnitude=7.30
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.25
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: No
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	20.00	116.00	NoLiq
5.00	19.00	126.00	5.00
7.50	5.00	110.00	8.80
10.00	19.00	126.00	10.10
15.00	17.00	126.00	6.00
20.00	27.00	126.00	6.00
25.00	18.00	126.00	5.10
30.00	45.00	126.00	5.10
35.00	41.00	126.00	5.10

40.00	35.00	126.00	5.00
45.00	62.00	126.00	5.00
50.00	38.00	126.00	5.00

Output Results:
Settlement of Saturated Sands=1.49 in.
Settlement of Unsaturated Sands=0.00 in.
Total Settlement of Saturated and Unsaturated Sands=1.49 in.
Differential Settlement=0.746 to 0.985 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.41	5.00	1.49	0.00	1.49
1.00	2.00	0.41	5.00	1.49	0.00	1.49
2.00	2.00	0.40	5.00	1.49	0.00	1.49
3.00	2.00	0.40	5.00	1.49	0.00	1.49
4.00	2.00	0.40	5.00	1.49	0.00	1.49
5.00	2.14	0.40	5.00	1.49	0.00	1.49
6.00	2.14	0.44	4.88	1.49	0.00	1.49
7.00	2.14	0.47	4.56	1.49	0.00	1.49
8.00	0.12	0.50	0.24*	1.29	0.00	1.29
9.00	0.13	0.52	0.26*	0.89	0.00	0.89
10.00	2.14	0.54	3.96	0.50	0.00	0.50
11.00	2.14	0.56	3.85	0.50	0.00	0.50
12.00	2.14	0.57	3.76	0.50	0.00	0.50
13.00	2.14	0.58	3.68	0.50	0.00	0.50
14.00	2.14	0.59	3.62	0.50	0.00	0.50
15.00	2.14	0.60	3.57	0.50	0.00	0.50
16.00	2.14	0.61	3.53	0.50	0.00	0.50
17.00	2.14	0.61	3.49	0.50	0.00	0.50
18.00	2.14	0.62	3.45	0.50	0.00	0.50
19.00	2.14	0.63	3.43	0.50	0.00	0.50
20.00	2.14	0.63	3.40	0.50	0.00	0.50
21.00	2.14	0.63	3.38	0.50	0.00	0.50
22.00	2.14	0.64	3.36	0.50	0.00	0.50
23.00	2.14	0.64	3.34	0.50	0.00	0.50
24.00	2.14	0.64	3.33	0.50	0.00	0.50
25.00	2.14	0.65	3.32	0.50	0.00	0.50
26.00	0.47	0.65	0.72*	0.47	0.00	0.47
27.00	0.42	0.65	0.64*	0.35	0.00	0.35
28.00	2.14	0.65	3.29	0.22	0.00	0.22
29.00	0.45	0.65	0.68*	0.14	0.00	0.14
30.00	0.41	0.65	0.63*	0.01	0.00	0.01
31.00	2.14	0.65	3.29	0.00	0.00	0.00
32.00	2.14	0.65	3.31	0.00	0.00	0.00
33.00	2.14	0.64	3.32	0.00	0.00	0.00
34.00	2.14	0.64	3.34	0.00	0.00	0.00
35.00	2.14	0.64	3.36	0.00	0.00	0.00
36.00	2.14	0.63	3.38	0.00	0.00	0.00
37.00	2.14	0.63	3.40	0.00	0.00	0.00
38.00	2.14	0.63	3.43	0.00	0.00	0.00
39.00	2.14	0.62	3.45	0.00	0.00	0.00
40.00	2.14	0.62	3.47	0.00	0.00	0.00
41.00	2.14	0.61	3.50	0.00	0.00	0.00
42.00	2.14	0.61	3.52	0.00	0.00	0.00
43.00	2.14	0.60	3.55	0.00	0.00	0.00
44.00	2.14	0.60	3.58	0.00	0.00	0.00
45.00	2.14	0.59	3.61	0.00	0.00	0.00
46.00	2.14	0.59	3.64	0.00	0.00	0.00
47.00	2.14	0.58	3.67	0.00	0.00	0.00
48.00	2.14	0.58	3.70	0.00	0.00	0.00

49.00	2.14	0.57	3.73	0.00	0.00	0.00
50.00	2.14	0.57	3.77	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

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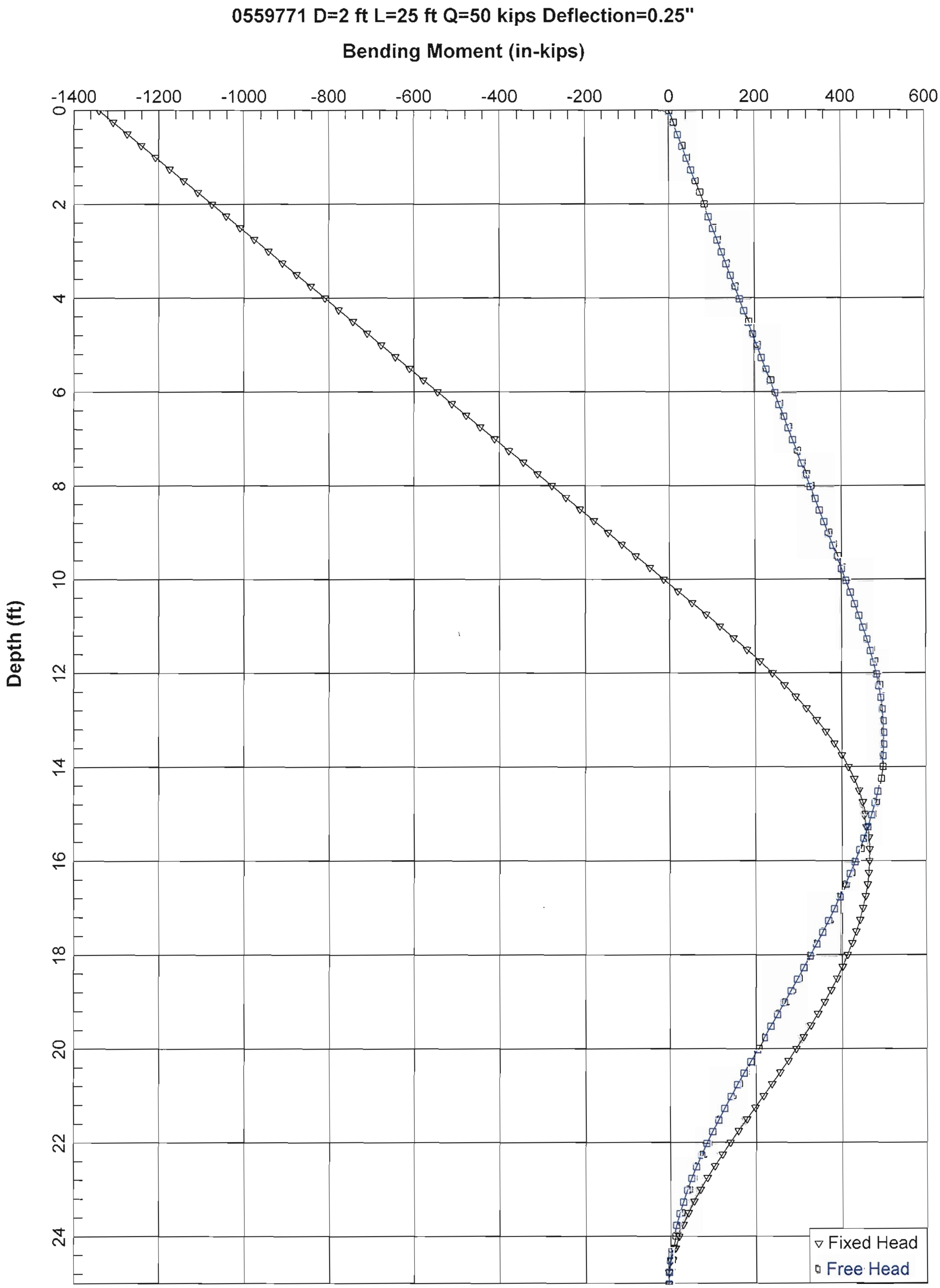
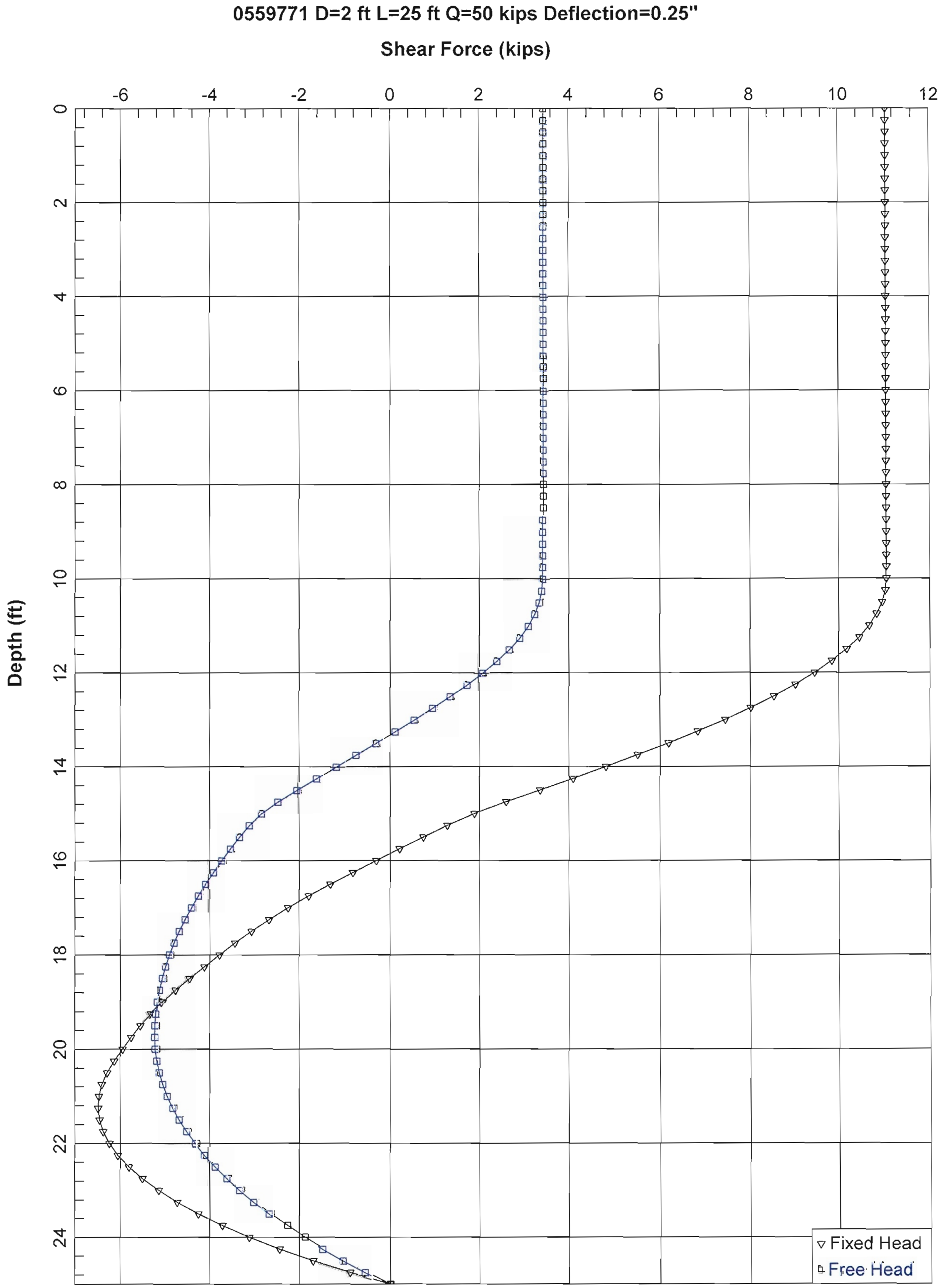
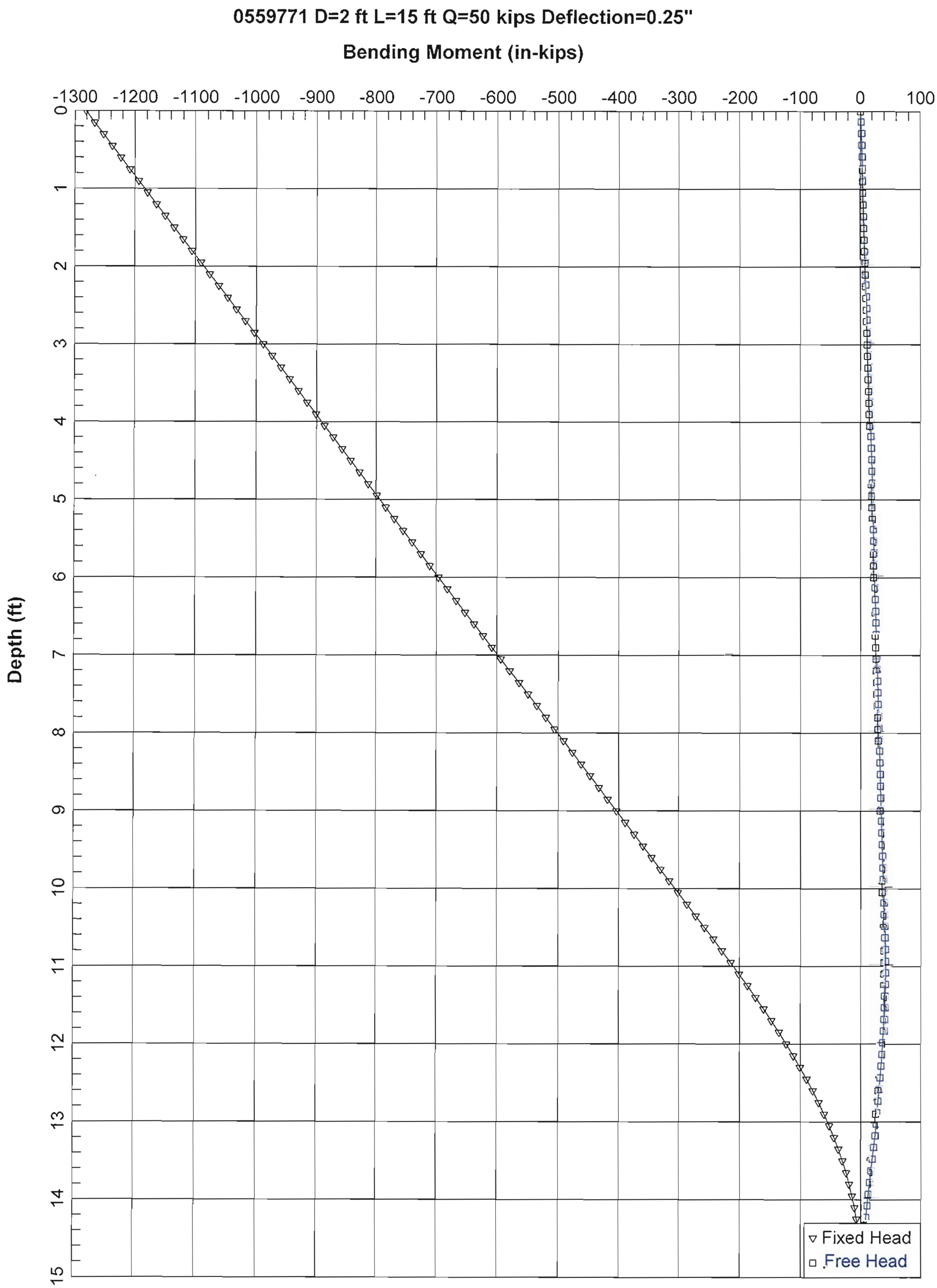
1 atm (atmosphere) = 1 tsf (ton/ft2)

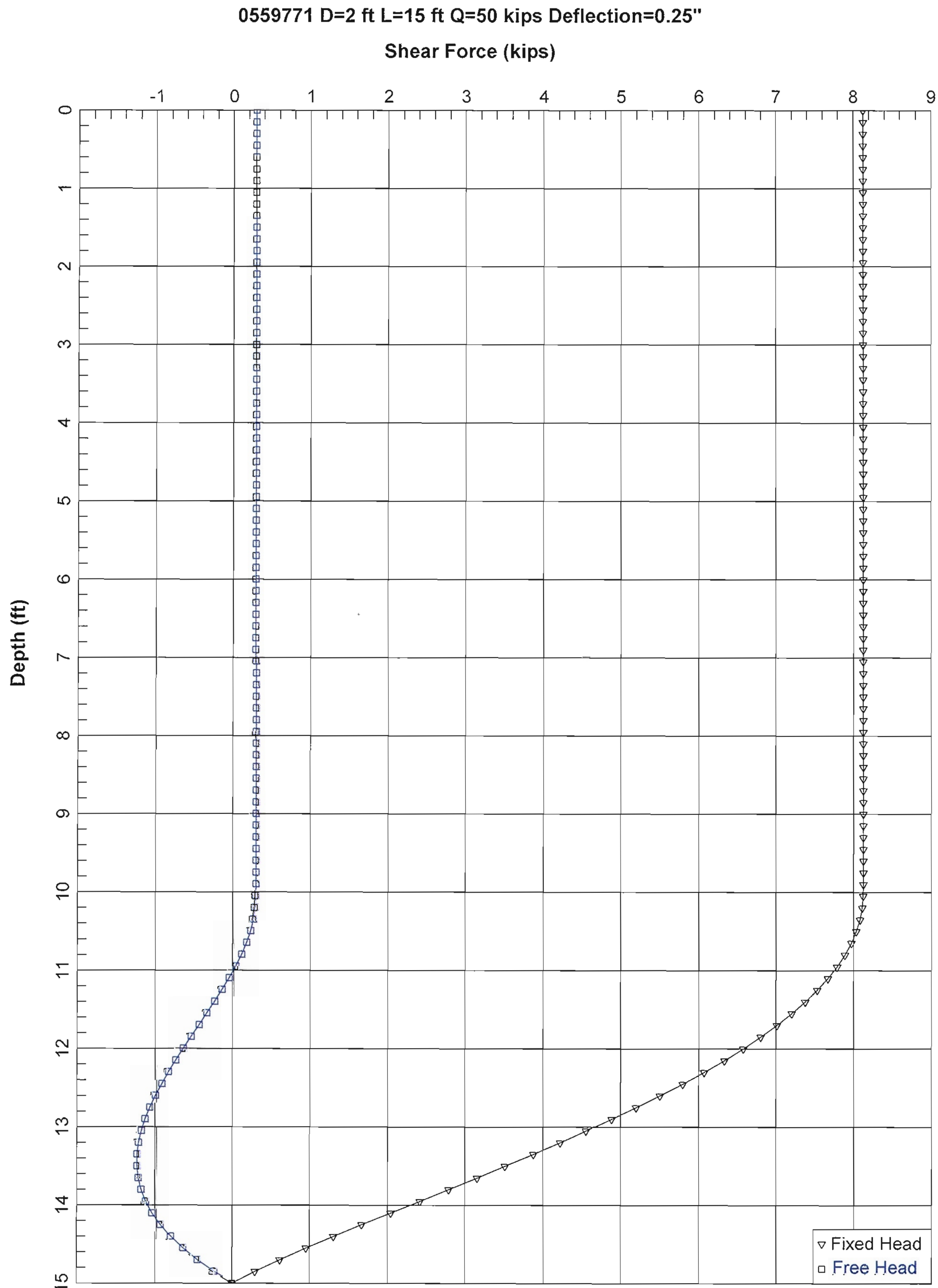
CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

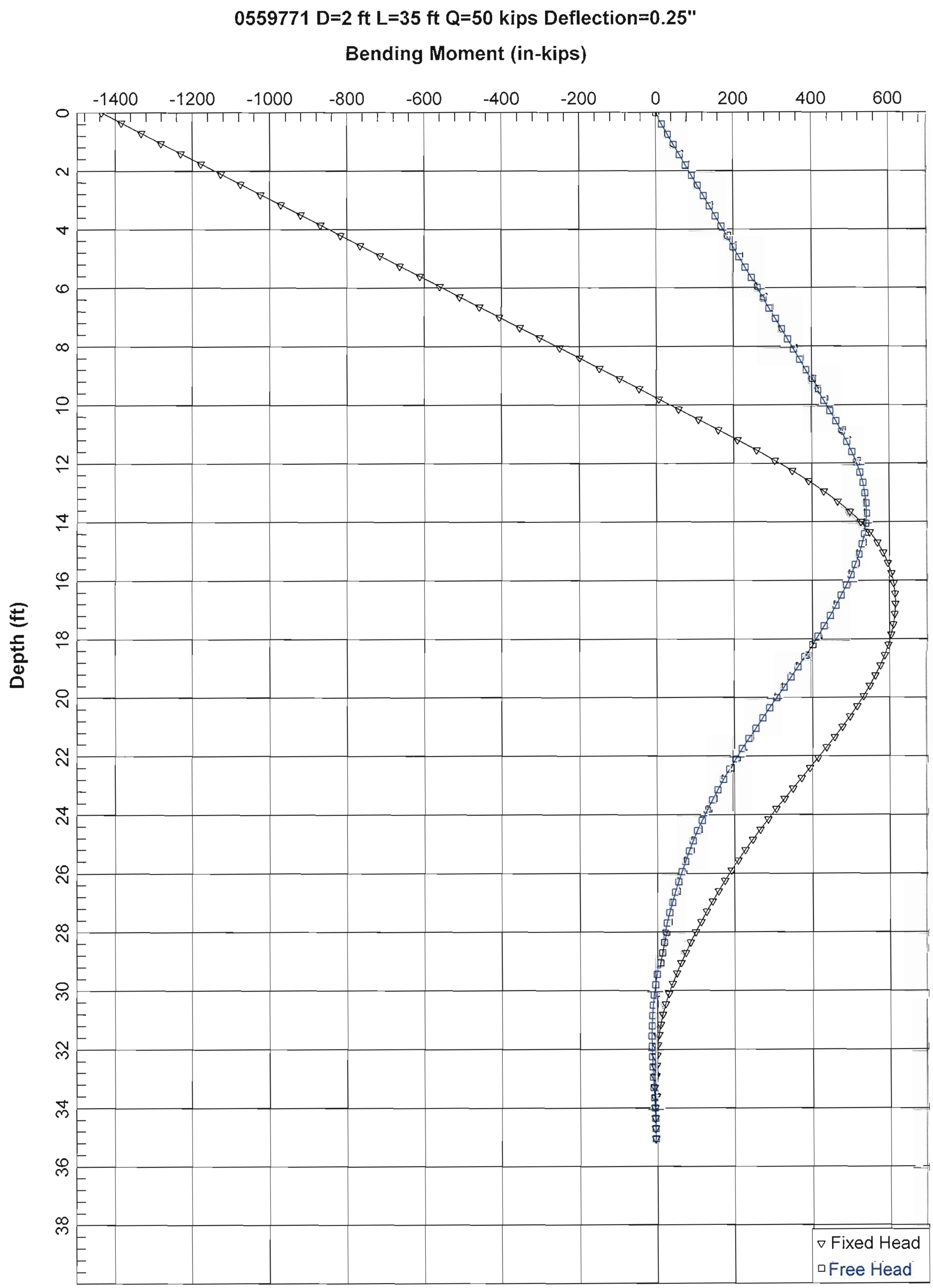

0559771 D=2 ft L=25 ft Q=50 kips Deflection=0.25"
Bending Moment (in-kips)
-1400
-1200
-1000
-800
-600
-400
-200
0
200
400
600
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
Fixed Head
Free Head



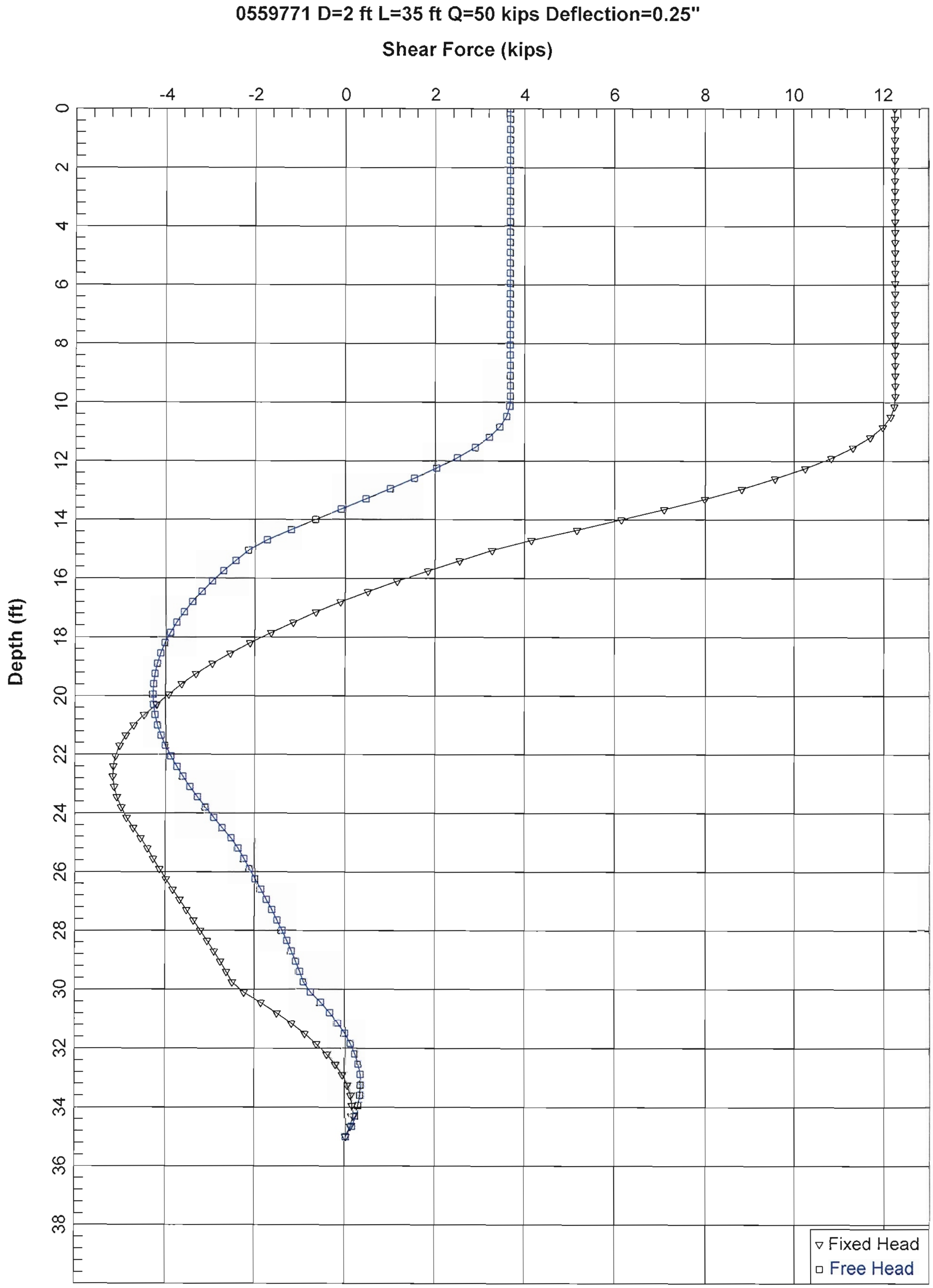
0559771 D=2 ft L=25 ft Q=50 kips Deflection=0.25"
Shear Force (kips)
Depth (ft)
-6
-4
-2
0
2
4
6
8
10
12
0
2
4
6
8
10
12
14
16
18
20
22
24
Fixed Head
Free Head


0559771 D=2 ft L=15 ft Q=50 kips Deflection=0.25"
Bending Moment (in-kips)
-1300 -1200 -1100 -1000 -900 -800 -700 -600 -500 -400 -300 -200 -100 0 100
Depth (ft)
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
Fixed Head
Free Head


0559771 D=2 ft L=15 ft Q=50 kips Deflection=0.25"
Shear Force (kips)
-1 0 1 2 3 4 5 6 7 8 9
Depth (ft)
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
Fixed Head
Free Head



0559771 D=2 ft L=35 ft Q=50 kips Deflection=0.25"
Bending Moment (in-kips)
-1400
-1200
-1000
-800
-600
-400
-200
0
200
400
600
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
Fixed Head
Free Head


0559771 D=2 ft L=35 ft Q=50 kips Deflection=0.25"
Shear Force (kips)
Depth (ft)
-4
-2
0
2
4
6
8
10
12
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
Fixed Head
Free Head