GEOTECHNICAL INVESTIGATION REPORT

AutoNation – Newport Porsche
600 West Coast Highway
Newport Beach, California

Prepared for:
AutoNation
200 SW 1st Street, Suite 1400
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Prepared by:
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April 20, 2015

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April 20, 2015

Mr. Axay Patel
AutoNation
200 SW 1st Street, Suite 1400
Ft. Lauderdale, Florida 33301

RE: GEOTECHNICAL INVESTIGATION REPORT
AutoNation – Newport Porsche
600 West Coast Highway
Newport Beach, California

Dear Mr. Patel,

Pursuant to the request of the AutoNation, Stantec Consulting Services Inc. (Stantec) has prepared the attached Geotechnical Investigation report for the proposed Newport Porsche auto dealership facility, located at 600 West Coast Highway, in the City of Newport Beach, California.

This investigation was performed in general accordance with Stantec's standard protocol for geotechnical investigations. The objective of the geotechnical investigation was to assess the soil conditions underlying the Site and make geotechnical recommendations for design and construction of the proposed development, which includes a 31,290-square-foot (sf) showroom, service, and parts building, a retaining wall north of the building, and associated landscaping and parking.

The findings of this investigation are presented in the attached report. It is our pleasure to be of service to you and we look forward to providing AutoNation with future engineering services. Should you have any questions regarding the information contained in the attached report, please contact the undersigned at your convenience.

Respectfully submitted,
Stantec Consulting Services Inc.

Jaret Fischer, P.E.
Senior Engineer

Enclosure: Geotechnical Investigation Report

cc: Ms. Vandana Kelkar
Stantec Architecture Inc.
38 Technology Drive, Suite 100
Irvine, California 92618

Facility: AutoNation – Newport Porsche
Location: 600 West Coast Highway
Newport Beach, California

Consultant: Stantec
Stantec JN: 2007105003

REPORT SUMMARY

Footing Bearing Pressures	- Building Foundations	2,000 psf
(See Section 7 for alternative building foundation recommendations)		
Coefficient of Friction	- Building Foundations	0.30
Expansive Soils		o Yes x No
Expansion Potential	o V. Low x Low o Medium o High o V. High	
R-Value		20 (estimated)
Automobile Traffic (TI = 5)		4.0" AC / 4.0" AB
Automobile and Truck Traffic (TI = 7)		4.0" AC / 10.0" AB
Artificial Fill		x Yes o No
Relatively Loose Near-Surface Soils		x Yes o No
Groundwater Within 20 Feet of Surface		x Yes o No
Monitoring Well Installed		o Yes x No
Hydrocarbons Detected		o Yes x No
Existing Underground Tanks		o Yes x No
Existing Structures		x Yes o No

Special Considerations:

- To provide uniform and firm support for the proposed building conventional foundation, the existing soils should be stabilized to a minimum depth of 7 feet below the ground surface (bgs) in accordance with recommendations provided in Section 7.3.1.
- In lieu of subgrade stabilization, the building may be supported on grade beams and drilled piers that extend to a minimum depth of 10 feet bgs.
- To provide uniform and firm support for the proposed pavement area, the existing soils should be removed to a minimum depth of one foot below the bottom of the structural pavement section and replaced with compacted fill.

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1.0 INTRODUCTION

1.1 AUTHORIZATION AND LIMITATIONS

This report presents the results of a geotechnical investigation performed at the request of the AutoNation, by Stantec Consulting Services Inc. (Stantec), for the proposed Newport Porsche automobile dealership facility, located at 600 West Coast Highway, in the City of Newport Beach, California. This report has been prepared for AutoNation and their project design consultants to be used solely in the design of the proposed project, as described herein. This report may not contain sufficient information for other uses or the purposes of other parties.

1.2 PURPOSE AND SCOPE OF WORK

The objective of this investigation was to assess the nature and engineering properties of the encountered subsurface soils and to provide geotechnical design recommendations for Site development. The scope of work was performed in general accordance with Stantec's standard protocol for geotechnical assessments, and consisted of the following tasks:

- Review available subsurface information for the Site,
- Drill, log and sample 4 borings,
- Advance 3 cone penetrometer test (CPT) soundings,
- Perform soil mechanics laboratory testing on select soil samples,
- Evaluate geotechnical properties of soils pertinent to the design and construction of the proposed project, and
- Develop conclusions and recommendations regarding:
    - Minimum building foundation recommendations for the new showroom, service, and parts building,
    - Minimum foundation recommendations and lateral earth pressures for the proposed retaining wall,
    - Subgrade preparation beneath new foundations, pavements, and sidewalks,
    - Fill and backfill materials along with fill and backfill placement and compaction criteria,
    - Appropriate foundation type(s) for support of new structures along with geotechnical criteria for foundation design,
    - New flexible pavement structural sections for driveway and parking areas,
    - Corrosivity of Site soils with respect to steel and concrete.

1.3 SITE LOCATION

The Site is located approximately 800 feet west of Dover Drive on the north side of West Coast Highway (CA Highway 1), at 600 West Coast Highway, in the City of Newport Beach, California (referred herein as the Site). The Site is bounded by West Coast Highway followed by the single family residential homes to the south, commercial businesses to the east and west, and single family residential to the north above a 40-foot high slope.

1.4 SITE DESCRIPTION

The Site is rectangular in shape, approximately 1.8 acres in size, and occupied by several existing retail businesses. The retail businesses include a small classic automobile dealership (European Collections), a dog food store (Just Food for Dogs), a two story former motel converted to small retail shops (Shops at the Cove), a linen shop (La Tavola Fine Linens), and a consignment store (Find Consignments).

An existing retaining wall, ranging from approximately 2 to 12 feet tall, is located along the northern portion of the property.

2.0 PROJECT DESCRIPTION

Stantec Architecture (Stantec), of Irvine, California provided the preliminary development layout for the proposed project. The proposed development will consist of a 31,290-square-foot (sf) showroom, service, and parts building, a retaining wall north of the building, and associated landscaping and parking. The retaining wall will be located approximately 3 feet south of the northern property line. The Site location is shown on Figure 1 and the layout of the proposed structure is shown on Figure 2.

There were no building and grading plans or design loads available at the time of this report. Foundation loads for the proposed structures were estimated for the purpose of this report at less than 2.0 kips per linear foot (klf) for walls and 50 kips for columns. If actual design loading conditions differ from those indicated above, the recommendations of this report may have to be re-evaluated and are subject to change.

Based upon Stantec's review of the existing Site topography, it is assumed that the final surface elevations away from the slope will not vary more than 0.5 to 1.0 foot from existing grades and that minor grade changes will be made for the purpose of establishing Site drainage. Stantec recommends that the final grading plan be provided to the Project Geotechnical Engineer for review. The recommendations in this report are subject to change based upon review of the final grading plan.

3.0 SUBSURFACE INVESTIGATION

3.1 PRE-DRILLING PROCEDURES

Underground Service Alert (USA) was notified several days prior to commencing drilling activities to identify public utilities that may conflict with the proposed boring locations. In addition, potential conflict with underground utilities was minimized by manually augering the upper five feet of soil at each proposed geotechnical sol boring location prior to drilling.

3.2 CONE PENETRATION TEST SOUNDINGS

Three (3) cone penetration test (CPT) soundings (CPT-1 through CPT-3) were completed on March 20, 2015, by Gregg Drilling and Testing, Inc. (Gregg) under the direction of a Stantec engineer or geologist. All CPT soundings were performed under the general guidance of ASTM D 6441 (Standard Test Method for Mechanical Cone Penetration Tests of Soils).

The CPT soundings completed for this geotechnical investigation were advanced using a truck mounted CPT rig, to a maximum depth of approximately 55 feet below the ground surface (bgs), at the locations shown on Figure 2. The soundings were distributed throughout the area of the proposed building to assess underlying subsurface conditions, including skin friction.

Piezo-cone penetrometers were advanced using a push rod equipped with a telescoping penetrometer tip. Continuous tip and side friction data was collected for each sounding. Following completion of the CPT soundings, the holes were abandoned by removing the CPT equipment from the hole and subsequently backfilling with native soil. CPT data is included in Appendix B.

3.3 HOLLOW STEM AUGER DRILLING

Four hollow stem auger (HSA) borings were drilled on March 20, 2015, by California Pacific Drilling (CalPac) under the direction of a Stantec representative. CalPac drilled the soil borings using a Mobile B-61 HSA drill rig. Drilling and soil sampling were performed under the general guidance of ASTM D6151 (Standard Practice for Using Hollow-Stem Augers (HSA) for Geotechnical Exploration and Soil Sampling).

The HSA soil borings drilled for this geotechnical investigation were advanced using six-inch outside diameter auger, to a maximum depth of approximately 36.5 feet below the ground surface (bgs), at the locations shown on Figure 2.

At each boring location, drilling was initiated by pushing the lead HSA auger below the ground surface and rotating it at a low velocity. Firm downward pressure and low rotation velocity were maintained in the beginning to produce a straight borehole. Once a straight hole was initiated and the HSA auger appeared clear of potential underground utilities, rotation velocity and downward pressure were increased. The rotation velocity and downward pressures were adjusted during drilling to optimize penetration rates with appropriate drill cutting return up the HSA auger flight. Additional five-foot sections of HSA auger flight were attached to the drill column to achieve the desired drilling/sampling depths.

When the desired sampling depth was achieved, the bottom of the borehole was cleaned by slowly rotating the auger with minimal downward pressure. When the borehole was sufficiently clean, soil samples were collected as described in the section below.

Following completion of drilling and soil sampling, the borings were abandoned by removing the auger and/or sampling equipment from the borehole and subsequently backfilling with native soil.

3.4 SPLIT SPOON SOIL SAMPLING

A Stantec representative, under the direct supervision of a licensed engineer, was onsite to supervise field operations, log subsurface soil conditions, and to collect soil samples for physical and chemical analysis. Soil samples were collected using a California Modified (CM) and Standard Penetration Test (SPT) split-spoon samplers, under the general guidance of ASTM D1586 (Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils), D3550 (Standard Practice for Ring-Lined Barrel Sampling of Soils) and D6066 (Standard Practice for Determining the Normalized Penetration Resistance of Sands for Evaluation of Liquefaction Potential). The CM sampler is approximately 18 inches long by 2.5 inches inside diameter (ID). The SPT sampler is approximately 18 inches long by 1.5 inches ID. The samplers were driven at approximately 5 foot intervals with a 140 pound hammer, free-falling 30 inches. Unless otherwise indicated on the boring logs, the samplers were advanced 18 inches at each sample interval and the blow counts required to advance the sampler each six-inch drive length were recorded on the boring logs. The blow counts are used in the evaluation of the consistency of the soils and are correlated to various engineering properties. The observed soils were classified in accordance with the Unified Soil Classification System, under the guidance of ASTM D2488 (Standard Practice for Description and Identification of Soils [Visual-Manual Method]).

Geotechnical samples were collected from the CM and SPT samplers. Six relatively undisturbed brass rings were carefully removed from the CM sampler, placed in a plastic sleeve and sealed with plastic end caps. Electrical tape was used to secure the end caps to the plastic sleeve to preserve natural moisture content. Disturbed samples were also collected from the lowermost brass tube of the SPT sampler. The soil was extruded from the brass tube and placed in a sealed plastic bag. Geotechnical ring and bulk samples were labeled and transported to a soil mechanics laboratory for physical testing. The CM soil samples were securely packed with foam or other shipping materials to minimize sample disturbance, under the guidance of ASTM D4220 (Standard Practice for Preserving and Transporting Soil Samples).

3.5 LABORATORY SOIL TESTING

The following laboratory tests were performed on samples collected at the Site either in general accordance with the American Society for Testing and Materials (ASTM) or contemporary practices of the soil engineering profession:

- In-Situ Moisture and Density (ASTM D2216): In-situ moisture and density are calculated by weighing and measuring the drive samples obtained from the borings to determine their in-place moisture and density. These results are used to analyze the density or consistency of the subsurface soils.
- Direct Shear Test (ASTM D3080): The tests were performed on an undisturbed sandy soil sample in order to obtain the soil shear strength values, which are among the basic soil parameters that are used to estimate soil bearing capacity and lateral earth pressures.
- Consolidation Tests (ASTM D2435): One-dimensional consolidation tests were conducted to evaluate soil compressibility and estimate the potential settlement of the structures. A one-inch thick sample contained in a 2.5-inch diameter ring was subjected to various load increments. The compression under each load increment was recorded and plotted against the logarithm of applied effective stress.

- Sieve Analysis (ASTM D422 and ASTM C136): This test is used to evaluate the distribution of soil grain sizes, which constitute the soil fabric and is used in soil classification and assessment of soil engineering behavior.
- No. 200 Sieve Wash (ASTM D1140): This test is used to evaluate the amount of soil grain sizes finer than the 0.075 mm (No. 200 sieve) and is used in soil classification and assessment of soil engineering behavior.
- Hydrometer Analysis (ASTM D422 and ASTM C 136): This test is used to evaluate the distribution of soil grain sizes, which constitute the soil fabric and is used in soil classification and assessment of soil engineering behavior.
- Expansion Index (ASTM D4829 and UBC Standard 18-2): This test is performed on a near surface bulk sample, remolded to approximately 50 percent saturation, to determine the expansion potential of the soil when fully saturated.
- Atterberg Limits (ASTM D 4318): The Atterberg Limits are utilized to classify fine-grained soils and correlate them to specific engineering properties. The Atterberg limits are composed of the liquid limit, and the plastic limit. The liquid limit is the moisture where the soil changes from a plastic to a liquid state and the plastic limit is the moisture content where the soil changes from a semi-solid state to a plastic state.
- Maximum Dry Density and Optimum Moisture Content (ASTM D1557): The compaction curve defines the relationship between water content and dry unit weight of soils compacted soils effort. The maximum dry density and optimum water content are used to determine the relative density of existing soils and to determine the level of compaction during grading activities.
- Chemical Tests for Corrosion Potential (Applicable EPA, ASTM or local test methods): The pH, resistivity, and the quantity of various chemical components useful in the assessment of corrosion potential were evaluated in a near surface soil sample.

The laboratory test results are presented in Appendix C.

4.0 REGIONAL GEOLOGIC CONDITIONS

4.1 REGIONAL PHYSIOGRAPHIC CONDITIONS

The Site is located in the northwestern portion of the Peninsular Range Geomorphic Province in southwestern California. The region is separated by northwest trending valleys, subparallel to faults branching from the San Andreas Fault. The Site resides in the portion of the Province drained by surface runoff into Newport Bay.

Newport Bay is located approximately 1,100 feet southwest of the Site, the California State Highway 1 (Pacific Coast Highway) is located adjacent to the south of the Site, and the Balboa Peninsula is located approximately 3,700 feet southwest of the Site. Based on interpretation of the ground surface elevation contour lines drawn on the topographic map, the Site is located at an elevation of approximately 12 to 61 feet above mean sea level (msl). The regional topography consists of northwest trending mountain ranges and valleys. The topography in the on the majority of the Site is relatively flat, with a slope to the southwest toward Newport Bay. The western portion of the Site slopes from approximately 14 to 61 feet above msl (USGS, 1965).

4.2 REGIONAL GEOLOGY

The regional surficial geology is described as late Holocene deposits consisting of unconsolidated sand, silt, and clay. The sloped northern portion of the Site is underlain by middle Miocene age siltstone facies consisting of massive to crudely bedded and friable white to pale gray siltstone and mudstone (USGS, 2004).

The Site is located in Southern California, a seismically active area. The nearest recently active fault includes the Newport Inglewood (LA Basin) Fault located approximately 1.2 southwest of the Site. The Site is not located within an Alquist-Priolo Earthquake Fault Zone (CDMG, 2000).

4.3 REGIONAL HYDROGEOLOGY

According to the California Department of Water Resources (CDWR) Bulletin 118 report, the Site is located within the Coastal Plain of Orange Groundwater Basin, which underlies approximately 350 square miles of Orange County. This subbasin is bounded on the north by unconsolidated rocks exposed on the Puente and Chino Hills, on the east by the Santa Ana Mountains, on the south by the San Joaquin Hills, and on the west by the Pacific Ocean (DWR, 2004).

Based on documented data in the site vicinity, the regional depth to first groundwater is approximately 5 to 10 feet below the ground surface (bgs) (CDMG, 1997). Groundwater in the site vicinity generally flows to the southwest toward Newport Bay.

5.0 SUBSURFACE SOIL AND GROUNDWATER CONDITIONS

5.1 STANTEC FIELD INVESTIGATION

The subsurface soils encountered are composed of various mixtures and combinations of interbedded layers of sand (SP, SW, SP-SM, and SM USCS soil type), clay and clay with sand (CL and CH USCS soil type), and silt (MH USCS soil type) from the ground surface to the maximum depth of exploration. The sands were fine to coarse grained and generally, moist to wet and very loose to loose in density. The clays exhibited low to high plasticity and were moist to wet and very soft to hard in consistency. The highly plastic silts were moist and very stiff to hard in consistency.

The subsurface soils were difficult to penetrate past at a depth of approximately 18 to 35 feet where drilling refusal was encountered in borings B1 and B2. Groundwater was encountered at depths of approximately 6 to 7 feet bgs during this investigation.

A more detailed description of the interpreted soil profile in each borehole is presented on boring logs in Appendix A. The groupings represent the predominant materials encountered in soil samples. Also, stratification lines indicate the approximate boundary between the major material types. The actual transition may be gradual.

6.0 REGIONAL SEISMIC CONDITIONS

6.1 REGIONAL SEISMICITY

The Site, as is most of California, is located in a seismically active area. The estimated distance of the Site to the nearest expected surface expression of nearby faults is presented in the table below.

Fault	Distance (miles) [1]	Maximum Moment Magnitude [1]
Newport – Inglewood (L.A. Basin)	1.2	7.5
Newport – Inglewood (Offshore)	1.8	7.0
San Joaquin Hills	5.3	7.1
Palos Verdes	13.1	7.7
Puente Hills (Coyote Hills)	17.6	6.9
Elsinore - Whittier	20.8	7.0
Elsinore – Glen Ivy	20.8	7.3
Coronado Bank	23.2	7.4
Chino – Central Avenue (Elsinore)	24.2	6.7

1. Measured from 2008 National Seismic Hazard Maps - USGS (USGS, 2008).

6.2 CALIFORNIA BUILDING CODE SEISMIC CRITERIA

Based on the specified design criteria of the 2013 California Building Code, the following Site seismic information may be considered for earthquake design.

Design Criteria	Design Value
Site Class	D
Mapped Spectral Response Acceleration for Short Periods S_S (g)	1.855
Mapped Spectral Response Acceleration for 1-second Period S_1 (g)	0.696
Maximum Considered Earthquake Spectral Acceleration for Short Periods S_{MS} (g)	1.855
Maximum Considered Earthquake Spectral Response Acceleration for 1-second Periods S_{M1} (g)	1.045
5-percent Design Spectral Response Acceleration for Short Periods S_{DS} (g)	1.236
5-percent Design Spectral Response Acceleration for 1-second Periods S_{D1} (g)	0.696
Site Coefficient F_a	1.0
Site Coefficient F_v	1.5

6.3 REGIONAL SEISMIC HAZARDS

6.3.1 Fault Rupture Hazard

The Site is not located within a currently mapped California Earthquake Fault Zone. As described above, the nearest fault is the Newport Inglewood (LA Basin) Fault, located approximately 1.2 miles southwest of the Site. Based on available geologic data, there is low potential for surface fault rupture from the Newport Inglewood (LA Basin) Fault and other nearby active faults propagating to the surface of the Site during the design life of the proposed development.

6.3.2 Liquefaction Hazard

Liquefaction Background

Liquefaction of saturated sandy soils is generally results in a sudden decrease in soil shear strength due to vibration. During cyclic shaking, typically caused by an earthquake, the soil mass is distorted, and interparticulate stresses are transferred from the soil particles to the pore water. As pore pressure increases the bearing capacity decreases and the soil may behave temporarily as a viscous fluid (liquefaction) and, consequently, lose its capacity to support the structures founded thereon.

Engineering research of soil liquefaction potential (Seed, et. al., 1982 and 1985) indicates that generally three basic factors must exist concurrently in order for liquefaction to occur, namely:

- A source of ground shaking, such as an earthquake, capable of generating soil mass distortions.
- A relatively loose, clean sandy soil fabric exhibiting a potential for volume reduction.
- A relative shallow groundwater table (within approximately 50 feet below ground surface) or completely saturated soil conditions that will allow positive pore pressure generation.

Screening Investigation for Liquefaction Potential

The Site is located within a current, mapped California Liquefaction Hazard Zone. A liquefaction evaluation for the Site was completed under the guidance of Special Publication 117a: Guidelines for Evaluating and Mitigating Seismic Hazards in California," published by the California Department of Conservation, California Geologic Survey, dated 2008 and based on empirical procedures described in summarized by Martin and Lew et al. (1999). The in-situ characteristics of the subsurface soils were analyzed, and similarities and dissimilarities of the subsurface conditions were compared with those sites where the subsurface soils are known to have liquefied.

The general Site characteristics, such as potential for seismic shaking, soil type, soil density, depth to groundwater, etc., were evaluated in an initial Screening Investigation (CGS, 2008). These characteristics were compared to conditions of known liquefaction susceptibility.

- *Potential for Strong Seismic Activity*—The Site is located within 1.2 miles of active faults capable of generating a magnitude 7.5 earthquake, respectively.
- *Shallow Groundwater within 50 feet* — Groundwater was encountered at a depth of approximately 6 to 7 feet in the borings drilled during this investigation. However, historic high depth to groundwater is reported at approximately 2 feet bgs (CDMG, 1997).

- *Relatively Loose Soils*—Blowcounts were recorded at less than 30 blows per foot in subsurface soils in the upper 50 feet bgs.
- *Cohesionless Soils*—Boring logs indicate that subsurface soils consist of interbedded layers of relatively clean sands and silty sands along with silt and clay soils in the upper 50 feet bgs.
- Potentially liquefiable soils with $(N_1)_{60}$ values less than 15, are generally considered potentially susceptible to lateral displacement (Youd et al., 2002). Various layers below the site exhibit $(N_1)_{60}$ values less than 15.

The data indicate conditions at the Site may be susceptible to seismically induced liquefaction. Consequently, a quantitative evaluation of liquefaction potential was conducted.

Quantitative Evaluation of Liquefaction Resistance

In accordance with protocols outlined in SP 117a (CDMG, 2008), a Quantitative Evaluation of Liquefaction Resistance was performed on soil layers in the upper 40 feet bgs. The assumed or estimated soil conditions used in the analysis are based on the boring logs, laboratory data, and applicable references, as discussed below.

The soil conditions used in the liquefaction model are based on conditions represented in the boring logs. Where blow counts were recorded using a 2.5-inch inside diameter California Modified sampler, the representative California Modified sampler blow counts were converted to equivalent SPT blow counts following the Lowe and Zaccheo Sampler Hammer Ratio (Winterkorn and Fang, 1975).

A probabilistic seismic hazard analysis (PSHA) was conducted to estimate ground motion accelerations corresponding to an earthquake having a 10 percent probability of exceedance over a 50-year time period. The design peak ground acceleration (PGA) was determined using the computer program FRISKSP, Version 4.00. The faults used in the PSHA were based upon a CGS fault catalog.

The site specific PGA was developed using Campbell and Bozorgonia's 1997 revised ground motion attenuation relation for alluvium. Dispersion in the Campbell and Bozorgonia ground motion attenuation relationship was considered by inclusion of the standard deviation of the ground motion data in the attenuation relationship used in the PSHA. For liquefaction analysis, the DBE induced peak ground acceleration (PGA) was scaled to an earthquake magnitude 7.5 using the NCEER 1997 magnitude scaling factor (Youd and Idriss, 2001). The 7.5 earthquake magnitude scaled site specific PGA is 0.8g (where "g" is the acceleration due to gravity).

The liquefaction probabilities were calculated using guidance developed by Seed and others (2003), and represent the corrected cyclic stress ratio (CSR*) required to cause liquefaction in a given soil layer divided by the overburden correction value. For the purpose of this evaluation, a layer was considered to be susceptible to liquefaction if the probability of liquefaction was greater than 20 percent. A liquefaction hazard analysis was conducted for the liquefaction susceptible soils in the depth interval of 2 to 7 feet bgs.

Effect of Potential Soil Liquefaction

Based on a quantitative evaluation, the loose saturated sand and silty sand appear to be susceptible to liquefaction in the event of a major earthquake. Soil liquefaction alone does not pose a risk to site development, but the effects of soil liquefaction on a site typically do. Such risks may include sand boils, lateral spreading, foundation bearing failure, and ground settlement.

Based on the available data for the site, the potentially liquefiable soils are overlain by approximately 5 feet of relatively loose soils in the unsaturated zone. The potential for surface manifestation of sand boils and lateral spreading are considered to be moderate.

The potential ground settlement resulting from seismic induced settlement was evaluated for the site based on the empirical procedures developed by Seed and others (2003) which compare the volumetric strain in the soil with the induced cyclic stress ratios. Assuming that the epicenter of the design earthquake occurs at the closest proximal distance from the fault to the site, the anticipated settlement in the potentially liquefiable layers between 2 to 7 feet bgs is expected to be approximately 2.6 inches, with differential settlements on the order of 1.3 to 1.7 inches.

6.3.3 Seismic Induced Settlement in Unsaturated Zone

Near surface soils in the unsaturated zone consist of relatively loose sands and silts. These sediments may be prone to significant volumetric strain as a result of cyclic loading from seismic activity. Although difficult to predict, surface settlements in the unsaturated zone were estimated to be approximately 1.6 inches, with differential settlements on the order of 0.8 inches, following methods promulgated by Tokimatsu and Seed (1987).

7.0 ENGINEERING RECOMMENDATIONS

Based upon the results of the investigation and previous geotechnical documentation, development of the Site is geotechnically feasible provided that the recommendations presented herein are implemented in the design and construction of the project. Soil stabilization followed by removal and recompaction of the near surface soils is recommended in the building area to provide a relatively uniform and firm engineered soil blanket for support of the proposed development and reduce the potential for differential settlement.

7.1 EXPANSIVE SOIL POTENTIAL

The near-surface soils consist of silt and silty sand. Expansion index testing in the area of the proposed development area indicate near surface soils exhibit low expansion potential, as defined by the 2013 California Building Code (CBC, 2013). Design for expansive soils is not required.

If imported soils are used for earthwork, Stantec recommends that the proposed soils be tested for expansion potential prior to import. Imported soils should be pre-approved by the Project Geotechnical Engineer prior to utilization.

7.2 CORROSIVE SOIL POTENTIAL

Chemical tests to evaluate corrosive soil potential of near surface soils were performed by Converse Consultants. The test results indicated pH of 7.7, water soluble sulfate = 1,060 ppm, soluble chlorides = 531 ppm, and saturated resistivity = 480 ohm-cm.

Based on the test results, the near surface soils are expected to have a moderate corrosion potential for concrete (Caltrans, 2014) and a very severe corrosion potential for steel (Romanoff, 1989). As such, special design considerations for concrete and steel are required.

Material Type	Degree of Corrosivity	Recommendation
Concrete	Moderate	Type II Modified Portland Cement
Steel	Very Severe	Corrosion Resistant Piping and Adequate Concrete Cover Over Reinforcing Steel

If imported soil is utilized for earthwork at the site, Stantec recommends that the proposed soils be tested for corrosion potential prior to import.

7.3 FOUNDATION DESIGN

7.3.1 Building Foundations

The shallow spread footings may provide adequate support under static conditions, but likely will not provide adequate support for the proposed structure in the event of seismic induced settlement should the design earthquake occur.

Engineered measures will be required to mitigate potential hazards from liquefaction-induced total. Such measures may include:

- Grade beam or mat foundation system supported by
  - Drilled in place concrete piers,
  - Helical piles,
  - Geopiers
- Conventional foundation with soil stabilization such as
  - Vibrocompaction,
  - Soil mixing,
  - Pressure grouting

It is recommended that a contractor specializing in ground improvement techniques be consulted if soil stabilization techniques are considered.

The potentially liquefiable soils extend to a depth of approximately 7 feet. Consequently, foundations should extend below this depth. The following recommendations may be used in design. If alternative mitigation measures are selected, additional investigations or recommendations may be required to evaluate foundation capacity at depth.

Vertical loads will be resisted by end bearing and skin friction in the silt and clay soils below about 10 feet bgs. Stantec estimates that additional vertical loading will occur due to seismic settlement of the loose to medium dense sand in the upper 7 feet. As a result, the piers will be required to support structural load as well as temporary negative skin friction. The allowable loads below include considerations of negative skin friction.

The lateral capacity of the piles will depend on the permissible deflection and on the degree of fixity at the top of the pile. The vertical and lateral capacities presented below are based upon soil characteristics described in the boring logs in Appendix A, a minimum 28-day grout or concrete compressive strength of 4,000 pounds per square inch (psi), fixed head condition, and a maximum deflection of 0.5 inches. The structural engineer should be consulted for actual design specifications and reinforcement.

Pile Design Parameters

Pile Diameter (inches)	**Design Parameters**	
	Allowable Vertical Capacity (kips)**	Allowable Lateral Capacity (kips)**
24	75	10

**Allowable vertical capacity is based on a minimum embedment depth of 10 feet bgs.

Design parameters for alternative pile diameters can be developed during the detailed design.

Design Data

The computer algorithm L-Pile or ALL PILE may be used to model the lateral behavior of a drilled shaft using estimated non-linear response of the soil. For a given pile loading, an iterative solution is performed to evaluate the deflection of the pile vs. depth.

Recommended soil parameters for use in L-Pile or ALL PILE (or similar) to analyze lateral soil interaction are presented in the table below. The native site soils in the proposed service canopy area are primarily silts and clays with variable amounts of sand with generally moderate moisture contents (moderate degree of saturation).

Soil Parameters Recommended for L-Pile or ALL PILE Lateral Drilled Shaft Analysis

Soil Boring Location	Recommended Soil Type to Model	Elevation Range (ft) (below existing grade)	Effective Unit Weight (pcf)	Angle of Internal Friction, phi (degrees)	Undrained Shear Strength, Cohesion, c (psf)	Lateral Soil Parameter, K (pci)	Soil Strain Ratio, e50	Relative Density (Dr) (%)
B3	Silty SAND (SM)	0-10	105	30	0	12	--	20
B3	Hard SILT (MH)	10-30	75	35	1,000	200	0.4	--

- pcf = pounds per cubic foot, psf = pounds per square foot, pci = pounds per square inch
- Neglect Lateral Resistance in the Upper 7 Feet

7.3.2 Showroom Building Foundation with Mitigation

Shallow foundations are expected to provide adequate support for the proposed building between 0 and 10 feet following stabilization utilizing one of the options in Section 7.3.1. An allowable bearing pressure of 2,000 pounds per square foot (psf) may be incorporated in the design. The footings should be at least 12 inches in width and founded a minimum of 12 inches below the lowest adjacent grade. For resistance to transient lateral loads, such as earthquake and wind loads, the allowable bearing capacity may be increased by one-third.

Design for resistance to lateral forces may be based upon a passive lateral earth pressure/resistance (equivalent fluid pressure) of 300D psf/ft and a coefficient of friction between the concrete footing and subsurface soils equal to 0.30, where D corresponds to the embedment depth of the footing in feet. For lateral bearing capacity analysis and design, the passive earth pressure and frictional resistance may be combined without reduction.

7.3.3 Retaining Wall Foundations

Cantilever Wall:

For cantilever retaining wall foundations, bearing on small diameter drilled piers or micro piles may be incorporated in the design with an average allowable bearing capacity of 3,000 psf embedded a minimum of 10 feet below the ground surface (bgs). To mitigate against potential detrimental effects of loose potentially liquefiable soils the following recommendations should be incorporated into the foundation design:

1. Minimum cantilever foundation embedment depth of 12 inches.
2. Micropiles (3” to 10” diameter) or small diameter drilled piers installed to a minimum depth of 10 feet below the wall foundation to mitigate potential liquefaction settlement.
3. Design and construction of features that prevent surface water from infiltrating around the foundation including construction of hardscape extending out from the footing at

least 6 feet, sloping surface and providing drainage away from the footings, and no planters or irrigation within 6 feet of the footing.
4. Weep holes or a back drain should be installed to provide positive drainage from behind wall.
5. All fill soil behind retaining wall should be non-expansive, and extend at least one foot beyond the back of the wall (presuming competent bedrock is encountered).
6. Compaction moisture content in clay should be 2 percentage points over optimum and maintained through construction.
7. Alternatively, depending on the height of the wall, the upper 7 feet of soil could be improved using techniques described in Section 7.3.1 and the wall supported on conventional spread footings.

The following lateral earth pressures (equivalent fluid pressures with a triangular pressure distribution) may be used in the design of the cantilever retaining wall foundations, up to a wall height of 20 feet.

Static Passive: 400D psf/ft, where the resultant force acts at 0.33D from the base of the wall
Static Active: 35H psf/ft, where the resultant force acts at 0.33H from the base of the wall.
Dynamic (Earthquake) Active: 24H psf/ft, where the resultant force acts at 0.65H measured from the base of the retaining structure

where H is the vertical height of the wall measured from the ground surface to the heel of the footing (or base of keyway) and D is the embedment depth of the footing measured from the ground surface to the bottom of the toe in front of the retaining wall, and a coefficient of friction between the concrete footing and subsurface soils equal to 0.30. The equivalent fluid pressures should be applied as a triangular pressure distribution and assume level backfill behind and in front of retaining wall, with the exception of the dynamic (earthquake) active, which should be considered an inverted triangular pressure distribution.

The earth pressures are based on drained conditions (no hydrostatic or buoyant conditions) and the assumption that the retaining wall is vertical (no batter). For different wall geometries or loading conditions, the above lateral earth pressures will need to be reevaluated. The earth pressures indicated above do not include a safety factor, therefore the retaining wall design should include an appropriate safety factor.

Retaining Structures:

Retaining structures can increase the stability of slopes by:

- Retaining fills that add weight to the resisting part of the landslide block.
- Retain part of the driving forces.
- Transfer driving forces into stable ground.
- Increase the resisting forces of the soil along the failure surface.

In-situ walls are structures that are built in place, without removing large volumes of soil to form a footing. They are well suited to retain the soil where access limitations or stability concerns prevent excavations needed to construct other wall types or to place earth buttress fills.

Several types of retaining structures were evaluated to create an integral stabilized ground reinforcement system capable of resisting the driving forces in the slope. These alternative retaining/stabilization systems included the following: (1) secant pile wall and/or tangent pile wall; (2) soldier pile wall; and (3) soil nail wall. As noted above, the minimum retaining structure depth was determined to be 35 feet bgs, which includes a minimum of 10 feet of embedment below the slide plane. A brief description of the slope stabilizing retaining structure options are provided below.

Secant Pile or Tangent Pile Wall:

Secant piles are drilled shafts that interlock to form a continuous reinforced concrete wall. The wall is constructed by drilling alternate shafts and then "back stepping" to drill the intervening shafts in order to interlock the two adjacent shafts. Every second shaft is reinforced usually with a wide flanged steel section or reinforcing steel cage. The reinforced shafts are called "primary". The alternate shafts, which are not reinforced, are called "secondary". The drilling sequence typically calls for the secondary piles to be drilled first, so the reinforcing of the primary piles will not be compromised by subsequent drilling. The concrete used for the secondary piles is usually lean concrete; to remain soft enough for the drilling and interlocking of the primary shafts. The primary piles are usually poured with structural concrete. In a secant pile wall, overlap is typically in the order of 3 inches (8 cm). In a tangent pile wall, there is no pile overlap as the piles are constructed flush to each other.

It may to necessary to install tieback anchors to further resist sliding forces. A tieback anchor consists of single or multiple steel wires, strands, or bars that are installed at a shallow inclination from the face of the pile, through the landslide mass, and into the underlying undisturbed soil. The tieback is anchored into stable, dense or hard soils with a cement grout or a mechanical end such as a helical plate or a swivel plate that expands into the soil when pulled. Typically the tieback anchor is post-tensioned and load tested. These anchors transmit sliding forces exerted by the landslide mass into the underlying stable soil. Secant pile walls and tieback installations require a specialty contractor with equipment capable of driving deep shafts and installing tiebacks into a hillside.

One of the drawbacks for these walls is that the amount of material and, consequently, the number of truck trips required to construct the wall, is generally much greater than for other retaining wall options. However, the secant wall offers the greatest protection against continued erosion of soil from behind the wall and the least amount of post slide maintenance.

Soldier Pile and Lagging or Soil Nail Wall:

A soldier pile and lagging wall or a soil nail wall may be technically practical. However, depending on the proximity to the property line, soil nails may not be feasible unless an agreement is made with the adjacent property owner(s). The slope in front of the wall face would need to be near vertical, thus these wall types may be desirable. As with the previously discussed wall type, these structures are permanent installations with little need for maintenance or repair. Typical maintenance would include visual inspection of weep holes and cleaning of weep holes if they become clogged.

The soldier pile and lagging wall would involve boring holes on the order of 18 to 24 inches in diameter through the soil overburden and creating a rock socket into the bedrock. Steel H-piles would be placed into the boreholes and the portion of the borehole in the bedrock grouted or filled with concrete. Spacing of the piles would be between 6 feet to 10 feet on center. The face of the wall would be constructed of wood timbers or precast concrete panels. The borehole construction would require a track mounted machine. A crane capable of lifting steel H-piles on the order of 30 to 40 feet long would be required to install the piles. Depending of the forces, tieback anchors may be required.

Soil nailing would involve the excavation of a vertical face through the soil at the top of the slope in increments of 5 vertical feet and installing rows of soil nails in inclined boreholes. The soil nails consist of steel bars grouted into the boreholes anchored to the wall face. A permanent concrete face is established by applying shotcrete to the soil face. The construction would be performed using a relatively small track-mounted drill rig and an on-site grout plant.

7.3.4 Foundation Construction

The Project Geotechnical Engineer should review and approve the foundation plans and observe foundation excavations prior concrete placement to check that foundation excavations extent into suitable material. The bottom of the foundation excavations should be drilled or excavated in such a way as to minimize slough, debris and unsuitable material from collecting at the bottom of the excavation.

7.3.5 Estimated Foundation Settlement

Static foundation settlement for the above described foundations is estimated to be less than one-inch total and less than one-half inch differential over a lateral distance of 50 feet, between similarly loaded footings of the same size.

Seismically induced settlements in the event of the design earthquake are calculated to be on the order of 4.2 inches total and approximately 2.1 to 2.7 inches of differential settlement over a lateral distance of 50 feet, between similarly loaded footings of the same size. Incorporating sufficient stiffness into the foundation design for the building expansion (i.e. footings tied together with grade beams), will minimize the differential movement in the event of a significant earthquake. Nevertheless, some damage to the building requiring subsequent repair should be anticipated in the event of a major earthquake in conjunction with a historic high groundwater level.

7.4 CONCRETE FLOOR

If soil stabilization and conventional foundations are incorporated into the design, concrete slab-on-grade floors can be used following site preparation as described in Section 7.8.2. Concrete slabs-on-grade should have a thickness of at least 5 inches and be reinforced with at least No. 4 reinforcing bars placed at 18 inches on-center each way. Slab reinforcement should be placed approximately at mid-height of the slab and extend at least 6 inches down into the footings.

Slabs-on-grade should be underlain by a 4-inch thick blanket of clean, poorly graded, coarse sand or crushed rock. A moisture vapor retarder/barrier should be placed beneath slabs where floor coverings will be installed. Typically, plastic is used as a vapor retarder/barrier. If plastic is used, a minimum 10 mil is recommended. The plastic should comply with ASTM E1745. Plastic installation should comply with ASTM E1643.

Current construction practice typically includes placement of a two-inch thick sand cushion between the bottom of the concrete slab and the moisture vapor retarder/barrier. This cushion can provide some protection to the vapor retarder/barrier during construction, and may assist in reducing the potential for edge curling in the slab during curing. However, the sand layer also provides a source of moisture vapor to the underside of the slab that can increase the time required to reduce moisture vapor emissions to limits acceptable for the type of floor covering placed on top of the slab. The floor covering manufacturer should be contacted to determine the volume of moisture vapor allowable and any treatment needed to reduce moisture vapor emissions to acceptable limits for the particular type of floor covering installed.

7.5 SLOPES

Although pertinent grading information is currently unavailable, permanent slopes are anticipated in the northern portion of the project. The stability of slopes should be evaluated when design-grading information becomes available.

7.6 TEMPORARY EXCAVATIONS

Temporary excavations should be shored or excavated with a slope not steeper than 1:1 (horizontal to vertical) in accordance with OSHA requirements. The excavations should be inspected by the contractor's competent person daily before personnel are allowed to enter the excavation. Surcharges from soil stockpiles, structures, vehicles, etc., should not be positioned a horizontal distance from the top of the excavation equal to the excavation depth.

Where cantilevered shoring is used in lieu of sloping the temporary excavation sidewalls, the shoring design may be tentatively based upon the following lateral earth pressures (equivalent fluid pressures with a triangular pressure distribution), up to an excavation depth of 16 feet bgs.

Active:	35H psf/ft,
At-rest:	55H psf/ft,
Passive:	370D psf/ft,

where H is the length of the sheet pile below the ground surface and D is the embedment depth of the shoring measured from the bottom of the excavation (unless pavement or hardscape are present, exclude the upper foot when calculating passive resistance to account for erosion). These equivalent fluid pressures should be applied as a triangular pressure distribution behind the shoring and assume level backfill behind and in front of shoring.

For braced shoring, a uniform rectangular pressure distribution should be used from top to bottom of the shoring equivalent to the following,

Bracing:	25H psf/ft

where H is the depth of the excavation.

The earth pressures are based on drained conditions (no hydrostatic or buoyant conditions) and the assumption that the shoring is vertical (no batter), and the ground surface in front and behind the shoring is level. For different geometries or conditions, the above lateral earth pressures should be reevaluated. The earth pressures indicated above do not include a safety factor; therefore, the shoring design should include an appropriate safety factor for the overall performance of the system.

7.7 PRELIMINARY PAVEMENT DESIGN

Preliminary flexible pavement structural sections were developed based on the visual onsite soil classifications, a presumed subgrade resistance R-Value of 20, an equivalent single axle load (ESAL) value comparable to the referenced traffic index (TI) value below, and an AASHTO Reliability Factor of 75%.

7.7.1 Asphalt Concrete Pavement

Traffic Type	Auto Traffic TI = 5.0	Auto and Truck Traffic TI = 7.0
Asphalt Concrete (AC) Thickness	4.0"	4.0"
Class 2 Aggregate Base (AB) Thickness	5.0"	10.0"

*AASHTO Highway Design Manual

7.7.2 Portland Cement Concrete Pavement

Proposed Portland cement concrete pavement areas that are subject to vehicle traffic loads, should have a minimum thickness of six inches overlying a minimum of six inches of Class II Aggregate Base.

The concrete should exhibit a minimum 28-day compressive strength of 2,500 psi and approximate three-inch slump (± one inch). Minimum reinforcement for concrete pavement in vehicle traffic areas should include #3 bars on 18-inch centers. Additional reinforcement and/or slab thickness may be appropriate as structural conditions dictate, as determined by the project structural or civil engineer. Other design and construction criteria for concrete floor slabs, such as mix design, strength, durability, reinforcement, joint spacing, etc., should conform to current specifications promulgated by the American Concrete Institute (ACI).

7.7.3 Subgrade and Aggregate Base Recommendations

The above pavement sections are based upon the assumption that the subgrade is uniformly compacted to at least 90 percent relative compaction with uniform moisture content within 2 percentage points above or below the optimum moisture content, as determined by ASTM Standard D1557, to a depth of 12 inches at the time of base course placement. Final geotechnical observation and testing of subgrade should be performed just prior to the placement of aggregate base or asphalt concrete.

The aggregate base for asphalt concrete pavement sections should meet Caltrans specifications for Class 2 base or the specifications for Processed Miscellaneous Base (PMB), as contained in the Standard Specifications for Public Works Construction. Aggregate base should be compacted to at least 95 percent relative compaction with uniform moisture content near the optimum percent, as determined by ASTM Standard D1557. Final geotechnical observation and testing of aggregate base should be performed just prior to the placement of asphalt concrete.

It is possible that Site grading, use of import fill soils, utility line backfilling, and/or underground storage tank installation could alter the distribution of near-surface materials, thus requiring re-evaluation of the recommended pavement structural sections. Stantec recommends that at least one near surface soil sample be tested to evaluate the subgrade R-value, following rough grading of the pavement areas.

7.8 SITE GRADING

Site grading will be required to achieve plan grades and to provide uniform support for foundations, slabs-on-grade and pavement. Recommendations for Site grading are presented in the following subsections, while general guide specifications for earthwork and grading are presented in Appendix D. The following grading recommendations are subject to change, depending on the actual earthwork required for the project and the subsurface conditions encountered during grading.

7.8.1 Clearing and Grubbing

The ground surface of the Site should be cleared and grubbed all of vegetation and deleterious materials, prior to grading. Clearing and grubbing is considered complete when soil supporting structural fill material or soil to be excavated as reused as structural fill materials contains less than five percent organic materials (by volume). Excavations created by removing underground structures, construction debris, vegetation roots, contaminated soils, and any other unsuitable materials should be backfilled with clean fill soil and should be compacted in accordance with the recommendations presented below.

7.8.2 Site Preparation

Pavement Areas:

Removal of the existing soils to a minimum depth of one foot below the subgrade elevation is recommended. The removed soils may be placed back in the excavation as compacted fill, in accordance with the recommendations in Section 7.8.3. Removal, replacement, and compaction beneath pavement areas should extend horizontally at least 3 feet beyond the rear curb face or as property line constraints dictate.

Unsuitable areas, as determined by the geotechnical engineer, should be removed to a minimum depth of one foot. Depending on the condition of the subexcavation bottom, additional removal depth may be recommended. Once a suitable excavation bottom is achieved, the exposed surface at the bottom of the excavation should be scarified to a depth of 6 inches, moisture conditioned, and surface compacted to the specified density. The removed soils can be placed back in the excavation as compacted fill, in accordance with the recommendations of Section 7.8.3.

Required Inspection of Subexcavation:

The project geotechnical engineer should check the bottoms of all subexcavations. Should unsuitable materials be encountered, the depth of removal may be extended.

7.8.3 Placement of Compacted Fill

General guide specifications for placement of fill and backfill are provided in Appendix D. The bottom of excavations and areas to receive fill should be scarified to a depth of six inches, moisture conditioned to 0 to 2 percentage points over optimum moisture content and then surface compacted to the relative compaction specified below.

Placement of compacted fill should be performed in thin lifts at two percentage points over optimum moisture content using mechanical compaction equipment and maintained at this moisture content until after pavement, slabs, or foundations are constructed. Unless specified otherwise, fill should be compacted to a minimum of 90 percent relative compaction based upon the maximum density obtained in accordance with ASTM Standard D1557. Gravel should not be used to backfill excavations onsite without the approval of the project geotechnical engineer.

During grading, frequent density testing should be performed by a representative of the geotechnical engineer to evaluate compliance with grading specifications. Where testing indicates insufficient relative compaction, additional compactive effort should be applied, with the adjustment of moisture content where necessary, until the required relative compaction is obtained.

7.9 POST INVESTIGATION SERVICES

Post investigation services are an important and necessary continuation of this investigation, and it is recommended that Stantec be retained as the Project Geotechnical Engineer to perform such services to assure adherence with the intent of the geotechnical recommendations presented herein.

Final project grading and foundation plans, foundation details and specifications should be reviewed by Stantec, prior to construction, to confirm that the intent of the recommendations presented herein have been applied to the designs. Following review of plans and specifications, sufficient and timely observation during construction should be performed to correlate the findings of this investigation with the actual subsurface conditions exposed during construction.

The following should be observed and tested by the Project Geotechnical Engineer:

- Rough Site grading, including the bottom of subexcavations.
- Footing excavations to confirm that the foundation elements are founded in the recommended materials.
- Utility trench backfill.
- Subgrade preparation, base placement and compaction.
- All other items of work requiring an opinion of adequacy from the Project Geotechnical Engineer to be included in a final geotechnical report.

During construction, the Project Geotechnical Engineer and/or authorized representatives should be present to observe the geotechnical aspects of the project and to test the earthwork. It is the sole responsibility of the contractor performing the work to confirm that the work complies with federal, state, and local safety procedures/regulations and with all applicable plans, specifications, and ordinances.

8.0 CLOSURE

Our conclusions, recommendations and discussions are (1) based upon an evaluation and interpretation of the findings of the field and laboratory programs, (2) based upon an interpolation of subsurface conditions between and beyond the exploration locations, (3) subject to confirmation of the actual conditions encountered during construction, and (4) based upon the assumption that sufficient observation and testing will be provided by Stantec during construction.

Any person using this report for bidding or construction purposes should perform such independent investigations deemed necessary to be satisfied as to the surface and subsurface conditions to be encountered and the procedures to be used in the performance of work on this project.

This report contains information which is valid as of this date. However, conditions that are beyond our control or that may occur with the passage of time may invalidate, either partially or wholly, the conclusions and recommendations presented herein.

The conclusions in this report are based on an interpolation and extrapolation of subsurface conditions encountered at the boring locations. The actual subsurface conditions at unexplored locations may be different. Consequently, the findings and recommendations in this report will require re-evaluation if subsurface conditions different than stated herein are encountered.

Inherent in most projects performed in the heterogeneous subsurface environment, additional subsurface investigations and analyses may reveal findings that are different than those presented herein. This facet of the geotechnical profession should be considered when formulating professional opinions on the limited data collected on this project.

The findings and recommendations contained in this report were developed in accordance with generally accepted current professional principles and practice ordinarily exercised, under similar circumstances, by geotechnical engineers and geologists practicing in this locality. No other warranty, express or implied, is made.

9.0 REFERENCES

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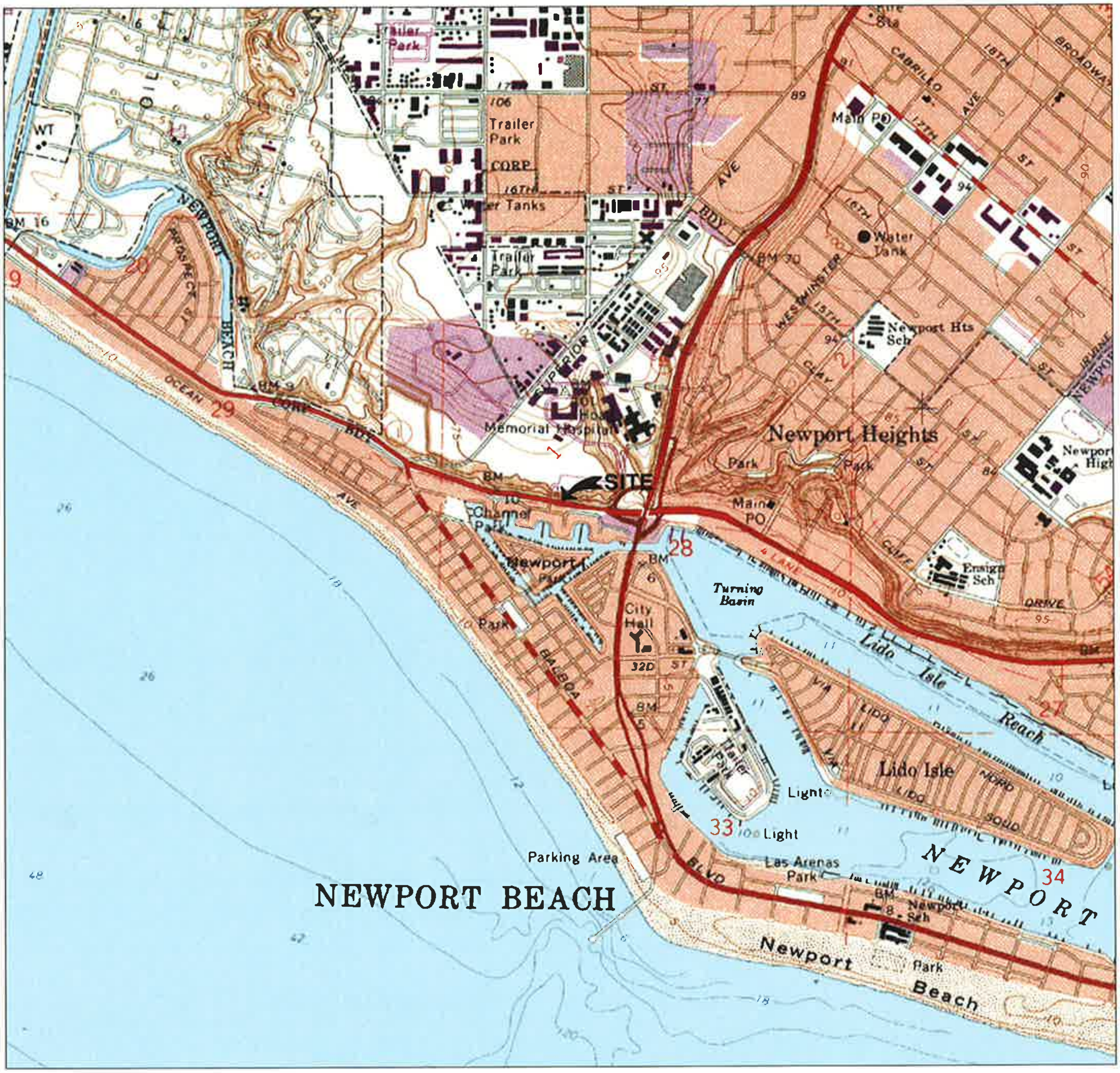
FIGURES



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SCALE IN MILE

1000 0 1000 2000 3000 4000 5000 6000 7000

SCALE IN FEET

REFERENCE: USGS 7.5 X 15 MINUTE QUADRANGLE; NEWPORT BEACH, 1965, PHOTOREVISED 1981.

Stantec 25864-F BUSINESS CENTER DRIVE REDLANDS, CA 92374 PHONE: (909) 335-6116 FAX: (909) 335-6120	FOR: AUTO NATION - NEWPORT PORSCHE 600 WEST COAST HIGHWAY (CA HWY 1) NEWPORT BEACH, CALIFORNIA		SITE LOCATION MAP	FIGURE: 1
	JOB NUMBER: 2007105003	DRAWN BY: JEF	CHECKED BY: JEF	APPROVED BY: JEF	DATE: 4/15/15

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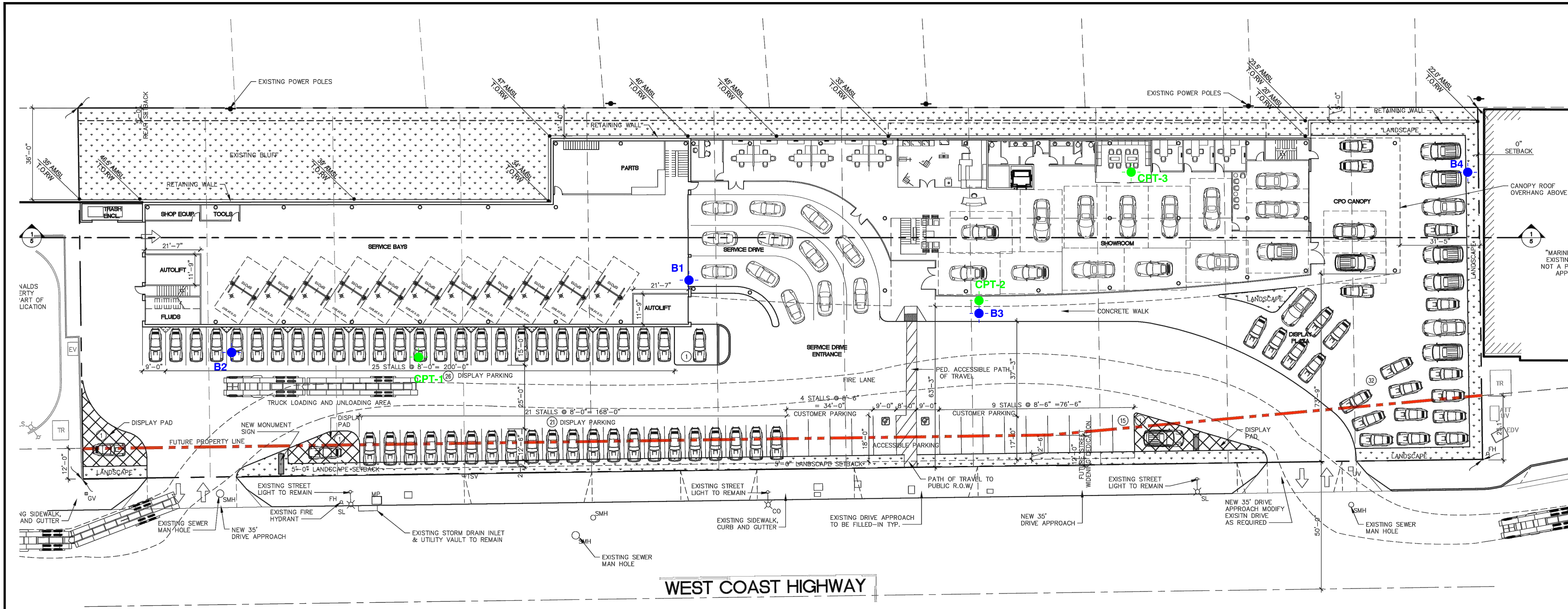

EXPLANATION

- ☐ APPROXIMATE LOCATION OF PROPOSED FEATURES
- B4 APPROXIMATE SOIL BORING LOCATION
- CPT-3 APPROXIMATE CONE PENETRATION TEST (CPT) LOCATION

N

0 40 80

APPROXIMATE SCALE (FEET)

Stantec

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REDLANDS, CA 92374
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FOR:

AUTO NATION - NEWPORT PORSCHE
600 WEST COAST HIGHWAY (CA HWY 1)
NEWPORT BEACH, CALIFORNIA

SITE PLAN

FIGURE: 2

JOB NUMBER:	DRAWN BY:	CHECKED BY:	APPROVED BY:	DATE:
2007105003	JEF	JEF	JEF	12/9/15

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APPENDIX A
BORING LOGS

PROJECT: **Geotechnical Legend**
LOCATION: **123 Main St. Anywhere USA**
PROJECT NUMBER: **00AB.12345.00**

WELL / PROBEHOLE / BOREHOLE NO:

Legend

PAGE 1 OF 1

DRILLING: STARTED **1/1/06**	COMPLETED: **1/1/06**	NORTHING (ft):	EASTING (ft):
INSTALLATION: STARTED **1/1/06**	COMPLETED: **1/1/06**	LATITUDE:	LONGITUDE:
DRILLING COMPANY: **Driling Sub-contractor**		GROUND ELEV (ft):	TOC ELEV (ft):
DRILLING EQUIPMENT: **Drilling Equipment**		INITIAL DTW (ft): **NE**	BOREHOLE DEPTH (ft): **25.0**
DRILLING METHOD: **Drilling Method**		STATIC DTW (ft): **NE**	WELL DEPTH (ft): **25.0**
SAMPLING EQUIPMENT: **Sampling Equipment**		WELL CASING DIAMETER (in): **NA**	BOREHOLE DIAMETER (in):
		LOGGED BY: **Onsite Technician**	CHECKED BY: **Project Eng.**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Geotechnical Lab Testing	Environmental Lab Testing	Blow Count	Headspace PID (units)	Depth (feet)	Well Construction
			Geotechnical Lab Testing							Surface Completion
			CNSL - Consolidation		CNSL					
			CRSN - Corrosion		CRSN					
			EI - Expansion Index		EI					
			HA - Hydrometer Analysis		HA					
5			MD - Moisture Density		MD				5	
			M - Moisture		M					
			R-Val - R-Value		R-Val			As Shown		
			SA - Sieve Analysis		SA					
			DS - Direct Shear		DS					
			UC - Unconfined Compression		UC					
			AL - Atterberg Limits		AL					
10			#200 - #200 Sieve Wash		#200				10	
			MP - Modified Proctor		MP					
			Environmental Lab Testing							
			8015M - Volatile and/or Extractable Petroleum Hydrocarbons			8015M				Backfill Description
			8260 - Halogenated Volatile Organic Compounds with Oxygenates			8260				
15			8270 - Semi-Volatile Organic Compounds			8270			15	
			8081 - Organochlorine Pesticides			8081				
			Hand Auger Sample							
20			Driven Sample, Blows Per 6 Inches, 2.5 Inch ID California Modified Sample Interval				10 11 15		20	
			Driven Sample, Blows Per 6 Inches, 1.5 Inch ID SPT Sample Interval				20 22 23			
25			Hole terminated at 25 feet.						25	
30									30	
35									35	

GEO FORM 304 GEOTECHNICAL LEGEND.GPJ SECOR INTL.GDT 12/5/06

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488). The classification excludes particles larger than 76 mm (3 inches). The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test N-Value (also known as N-Index). A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests.

Consistency	Undrained Shear Strength	
	kips/sq.ft.	kPa
Very Soft	<0.25	<12.5
Soft	0.25 - 0.5	12.5 - 25
Firm	0.5 - 1.0	25 - 50
Stiff	1.0 - 2.0	50 – 100
Very Stiff	2.0 - 4.0	100 - 200
Hard	>4.0	>200

ROCK DESCRIPTION

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor*
25-50	*Poor*
50-75	*Fair*
75-90	*Good*
90-100	*Excellent*

Rock quality classification is based on a modified core recovery percentage (RQD) in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures. The terminology describing rock mass quality based on RQD is subjective and is underlain by the presumption that sound strong rock is of higher engineering value than fractured weak rock.

Terminology describing rock mass:

Spacing (mm)	Joint Classification	Bedding, Laminations, Bands
> 6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

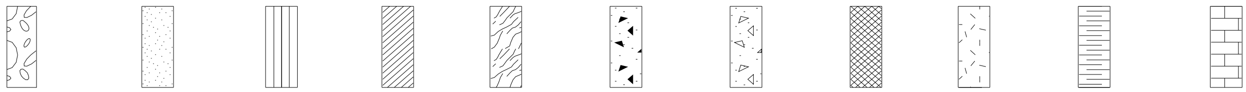
Strength Classification	Unconfined Compressive Strength (MPa)
Extremely Weak	< 1
Very Weak	1 – 5
Weak	5 – 25
Medium Strong	25 – 50
Strong	50 – 100
Very Strong	100 – 250
Extremely Strong	> 250

Terminology describing rock weathering:

Term	Description
Fresh	No visible signs of rock weathering. Slight discolouration along major discontinuities
Slightly Weathered	Discolouration indicates weathering of rock on discontinuity surfaces. All the rock material may be discoloured.
Moderately Weathered	Less than half the rock is decomposed and/or disintegrated into soil.
Highly Weathered	More than half the rock is decomposed and/or disintegrated into soil.
Completely Weathered	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Igneous Bedrock	Metamorphic Bedrock	Sedimentary Bedrock

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

▼ measured in standpipe, piezometer, or well

▽ inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and N-values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N value corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to A size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (305 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

PROJECT: **Newport Porsche**
LOCATION: **600 West Coast Highway, Newport Beach, CA**
PROJECT NUMBER: **2007105003**

WELL / PROBEHOLE / BOREHOLE NO: **B1** PAGE 1 OF 1

Stantec

DRILLING: STARTED **3/20/15** COMPLETED: **3/20/15**
INSTALLATION: STARTED **3/20/15** COMPLETED: **3/20/15**
DRILLING COMPANY: **Calpac Drilling**
DRILLING EQUIPMENT: **Mobile B-61**
DRILLING METHOD: **Hollowstem Auger**
SAMPLING EQUIPMENT: **Split Spoon**

NORTHING (ft):
LATITUDE:
GROUND ELEV (ft):
INITIAL DTW (ft): **7.0 3/20/15**
STATIC DTW (ft): **NE**
WELL CASING DIAMETER (in): **-**
LOGGED BY: **JS**

EASTING (ft):
LONGITUDE:
TOC ELEV (ft):
BOREHOLE DEPTH (ft): **18.0**
WELL DEPTH (ft): **18.0**
BOREHOLE DIAMETER (in): **6**
CHECKED BY: **JF**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Well Construction
			2" AC							
1050		SM	**SILTY SAND** ; SM; 10YR 5/6 yellowish brown; 82.4% fine to medium grained sand; 17.4% fines; 0.2% fine gravel; loose; moist; no staining; no odors.		1050 B1-2	SA, MD	3 4 5			
1055 5			...same as above; 10YR 4/4; dark yellowish brown; 69.0% fine to medium grained sand; 30.8% fines; 0.2% fine gravel; very loose; moist; no staining; no odor.		1055 B1-5	SA	1 1 2		5	
1100			...same as above; 10YR 5/4; yellowish brown; 55% fine to medium grained sand; 45% fines; loose; wet; no staining; no odors		1100 B1-7	DS	2 2 2			Native soil backfill
1108 10		MH	**SILT** ; MH; 10YR 4/3 brown; 95% fines; 5% fine grained sand; high plasticity; stiff; moist; no staining; slight organic odor.		1108 B1-10		1 5 5		10	
1115 15			... same as above; dark grayish brown (10YR 3/2); hard, laminations present.		1115 B1-15	MD	14 22 33		15	
			Refusal at 18 feet. Hole terminated at 18 feet.							

PROJECT: **Newport Porsche**	WELL / PROBEHOLE / BOREHOLE NO:
LOCATION: **600 West Coast Highway, Newport Beach, CA**	**B2** PAGE 1 OF 2
PROJECT NUMBER: **2007105003**	Stantec

DRILLING: STARTED **3/20/15**	COMPLETED: **3/20/15**	NORTHING (ft):	EASTING (ft):
INSTALLATION: STARTED **3/20/15**	COMPLETED: **3/20/15**	LATITUDE:	LONGITUDE:
DRILLING COMPANY: **Calpac Drilling**		GROUND ELEV (ft):	TOC ELEV (ft):
DRILLING EQUIPMENT: **Mobile B-61**		INITIAL DTW (ft): **7.0 3/20/15**	BOREHOLE DEPTH (ft): **36.5**
DRILLING METHOD: **Hollowstem Auger**		STATIC DTW (ft): **NE**	WELL DEPTH (ft): **36.5**
SAMPLING EQUIPMENT: **Split Spoon**		WELL CASING DIAMETER (in): **-**	BOREHOLE DIAMETER (in): **6**
		LOGGED BY: **JS**	CHECKED BY: **JF**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Well Construction
			4" AC							
1233		SM	**SILTY SAND** ; SM; 10YR 4/4 dark yellowish brown; 87.5% fine to medium grained sand; 12.5% fines; minor shell fragments; very loose; moist; no staining; no odor.		1233 B2-2	SA	2 1 1			
1240 5			...same as above; 79.5% fine to medium grained sand; 19.8% fines; 0.6% fine gravel; loose; moist; no staining; no odor.		1240 B2-5	SA, MD	3 5 6		5	
1250		SW	**SAND** ; SW; 10YR 5/6 yellowish brown; poorly graded; 90% fine to medium grained sand; 10% fines; loose; wet; no staining; no odors.		1250 B2-7		16 7 13			
		MH	**SILT** ; MH; 10YR 3/2 very dark grayish brown; 95% fines; <5% fine sand; very stiff; high plasticity; moist; no staining; slight organic odor.							
1300 10			... same as above; hard.		1300 B2-10	CNSL	9 26 34		10	
1310 15			... same as above; 85.5% fines; 14.5% fine to medium grained sand.		1310 B2-15	HA, AL, %M	5 13 20		15	Native soil backfill

GEO FORM 304 NEWPORT_PORCHE.GPJ SECOR INTL.GDT 4/21/15

PROJECT: **Newport Porsche**
LOCATION: **600 West Coast Highway, Newport Beach, CA**
PROJECT NUMBER: **2007105003**

WELL / PROBEHOLE / BOREHOLE NO: **B2** PAGE 2 OF 2

Stantec

DRILLING: STARTED **3/20/15**	COMPLETED: **3/20/15**	NORTHING (ft):	EASTING (ft):
INSTALLATION: STARTED **3/20/15**	COMPLETED: **3/20/15**	LATITUDE:	LONGITUDE:
DRILLING COMPANY: **Calpac Drilling**		GROUND ELEV (ft):	TOC ELEV (ft):
DRILLING EQUIPMENT: **Mobile B-61**		INITIAL DTW (ft): **7.0 3/20/15**	BOREHOLE DEPTH (ft): **36.5**
DRILLING METHOD: **Hollowstem Auger**		STATIC DTW (ft): **NE**	WELL DEPTH (ft): **36.5**
SAMPLING EQUIPMENT: **Split Spoon**		WELL CASING DIAMETER (in): **-**	BOREHOLE DIAMETER (in): **6**
		LOGGED BY: **JS**	CHECKED BY: **JF**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Well Construction
1315		MH	... same as above; 83.9% fines; 16.1% fine to medium grained sand.		1315 B2-20	HA, AL, MD	14 50-6"			
1325 25		SP-SM	**SAND WITH SILT** ; SP-SM; 5Y 3/2 dark olive gray; 89.5% medium sand; 10.5% fines; wet; dense; no staining; no odor.		1325 B2-25	SA	6 15 21		25	
1330 30		MH	**SILT** ; MH; 10YR 3/2 very dark grayish brown; 79.3% fines; 20.7% fine sand; hard; high plasticity; moist; no staining; slight organic odor.		1330 B2-30	HA, AL, %M	6 14 50-3"		30	
1335 35			... same as above; angular 1-2 cm rock chips		1335 B2-35		25 50-6"		35	
			Refusal at 35 feet. Hole terminated at 36.5 feet.							

PROJECT: **Newport Porsche**
LOCATION: **600 West Coast Highway, Newport Beach, CA**
PROJECT NUMBER: **2007105003**

WELL / PROBEHOLE / BOREHOLE NO: **B3** PAGE 1 OF 2

Stantec

DRILLING: STARTED **3/20/15** COMPLETED: **3/20/15**
INSTALLATION: STARTED **3/20/15** COMPLETED: **3/20/15**
DRILLING COMPANY: **Calpac Drilling**
DRILLING EQUIPMENT: **Mobile B-61**
DRILLING METHOD: **Hollowstem Auger**
SAMPLING EQUIPMENT: **Split Spoon**

NORTHING (ft):
EASTING (ft):
LATITUDE:
LONGITUDE:
GROUND ELEV (ft):
TOC ELEV (ft):
INITIAL DTW (ft): **6.0 3/20/15**
BOREHOLE DEPTH (ft): **31.5**
STATIC DTW (ft): **NE**
WELL DEPTH (ft): **31.5**
WELL CASING DIAMETER (in): **-**
BOREHOLE DIAMETER (in): **6**
LOGGED BY: **JS**
CHECKED BY: **JF**

Time & Depth (feet)	USCS	Description	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Well Construction
		2" AC						
0831	SP-SM	**SAND WITH SILT** ; SP-SM; 10YR 4/4 dark yellowish brown; 90.9% fine to medium grained sand; 8.2% fines; loose; moist; no staining; no odors.	0831 B3-2	SA, MD	2 3 4			
0838 5	SM	**SILTY SAND** ; SM; 10YR 3/2 very dark grayish brown; 71.0% fine to medium grained sand; 29.0% fines; very loose to loose; moist; no staining; no odor.	0838 B3-5	#200	1 2 2		5	
0845		...same as above; 2.5YR 4/3; dark olive brown; 82.7% fine to medium grained sand; 17.3% fines; loose; wet; no staining; no odor.	0845 B3-7	SA, MD	1 2 9			
0852 10	MH	**SILT** ; MH; 2.5Y 3/2 very dark grayish brown; 95% fines; 5% fine sand; hard; high plasticity; moist; no staining; no odor.	0852 B3-10		5 11 20		10	
0900 15		... same as above.	0900 B3-15		6 25 50-6"		15	Native soil backfill

PROJECT: **Newport Porsche**	WELL / PROBEHOLE / BOREHOLE NO:
LOCATION: **600 West Coast Highway, Newport Beach, CA**	**B3** PAGE 2 OF 2
PROJECT NUMBER: **2007105003**	Stantec

DRILLING: STARTED **3/20/15**	COMPLETED: **3/20/15**	NORTHING (ft):		EASTING (ft):
INSTALLATION: STARTED **3/20/15**	COMPLETED: **3/20/15**	LATITUDE:		LONGITUDE:
DRILLING COMPANY: **Calpac Drilling**		GROUND ELEV (ft):		TOC ELEV (ft):
DRILLING EQUIPMENT: **Mobile B-61**		INITIAL DTW (ft): **6.0**	**3/20/15**	BOREHOLE DEPTH (ft): **31.5**
DRILLING METHOD: **Hollowstem Auger**		STATIC DTW (ft): **NE**		WELL DEPTH (ft): **31.5**
SAMPLING EQUIPMENT: **Split Spoon**		WELL CASING DIAMETER (in): **-**		BOREHOLE DIAMETER (in): **6**
		LOGGED BY: **JS**		CHECKED BY: **JF**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Well Construction
0908			... same as above.		0908 B3-20		6 13 21			
0915 25			... same as above.		0915 B3-25		4 12 19		25	
0923 30			... same as above.		0923 B3-30		6 16 24		30	
			Hole terminated at 31.5 feet.							
35									35	

GEO FORM 304 NEWPORT_PORCHE.GPJ SECOR INTL.GDT 4/21/15

PROJECT: **Newport Porsche**
LOCATION: **600 West Coast Highway, Newport Beach, CA**
PROJECT NUMBER: **2007105003**

WELL / PROBEHOLE / BOREHOLE NO: **B4** PAGE 1 OF 2

Stantec

DRILLING: STARTED **3/20/15**	COMPLETED: **3/20/15**	NORTHING (ft):	EASTING (ft):
INSTALLATION: STARTED **3/20/15**	COMPLETED: **3/20/15**	LATITUDE:	LONGITUDE:
DRILLING COMPANY: **Calpac Drilling**		GROUND ELEV (ft):	TOC ELEV (ft):
DRILLING EQUIPMENT: **Mobile B-61**		INITIAL DTW (ft): **7.0 3/20/15**	BOREHOLE DEPTH (ft): **31.5**
DRILLING METHOD: **Hollowstem Auger**		STATIC DTW (ft): **NE**	WELL DEPTH (ft): **31.5**
		WELL CASING DIAMETER (in): **-**	BOREHOLE DIAMETER (in): **6**
SAMPLING EQUIPMENT: **Split Spoon**		LOGGED BY: **JS**	CHECKED BY: **JF**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Well Construction
1530		SM	**SILTY SAND** ; SM; 10YR 4/3 brown; 57.7% fine grained sand; 42.3% fines; trace shell fragments; moist; very loose; no staining; no odor.		1530 B4-2	#200	1 1 2			
1535 5			...same as above; 10YR 4/4; dark yellowish brown; 58.9% fine to medium grained sand; 38.1% fines; 3.0% fine gravel; loose; moist; no staining; no odor.		1535 B4-5	SA, DS	3 3 4		5	
1540		CL	**CLAY** ; CL; 10YR 4/4 dark yellowish brown; 75% fines; 25% fine to medium grained sand; low plasticity; very soft; wet; no staining; no odor.		1540 B4-7		1 1 1			
1545 10		SP	**SAND** ; SP; 10YR 3/4 dark yellowish brown; poorly graded; 95% fine to medium grained sand; 5% fines; trace shell fragments; loose; wet; no staining; no odor.		1545 B4-10	DS	1 2 3		10	
1550 15		CH	**CLAY** ; CH; 10YR 3/2 very dark grayish brown; 95% fines; 5% fine sand; high plasticity; very stiff; moist; no staining; slight organic odor; minor laminations present.		1550 B4-15		5 9 16		15	Native soil backfill

PROJECT: **Newport Porsche**	WELL / PROBEHOLE / BOREHOLE NO: **B4** PAGE 2 OF 2
LOCATION: **600 West Coast Highway, Newport Beach, CA**	
PROJECT NUMBER: **2007105003**	

DRILLING: STARTED **3/20/15**	COMPLETED: **3/20/15**	NORTHING (ft):	EASTING (ft):
INSTALLATION: STARTED **3/20/15**	COMPLETED: **3/20/15**	LATITUDE:	LONGITUDE:
DRILLING COMPANY: **Calpac Drilling**		GROUND ELEV (ft):	TOC ELEV (ft):
DRILLING EQUIPMENT: **Mobile B-61**		INITIAL DTW (ft): **7.0 3/20/15**	BOREHOLE DEPTH (ft): **31.5**
DRILLING METHOD: **Hollowstem Auger**		STATIC DTW (ft): **NE**	WELL DEPTH (ft): **31.5**
SAMPLING EQUIPMENT: **Split Spoon**		WELL CASING DIAMETER (in): **-**	BOREHOLE DIAMETER (in): **6**
		LOGGED BY: **JS**	CHECKED BY: **JF**

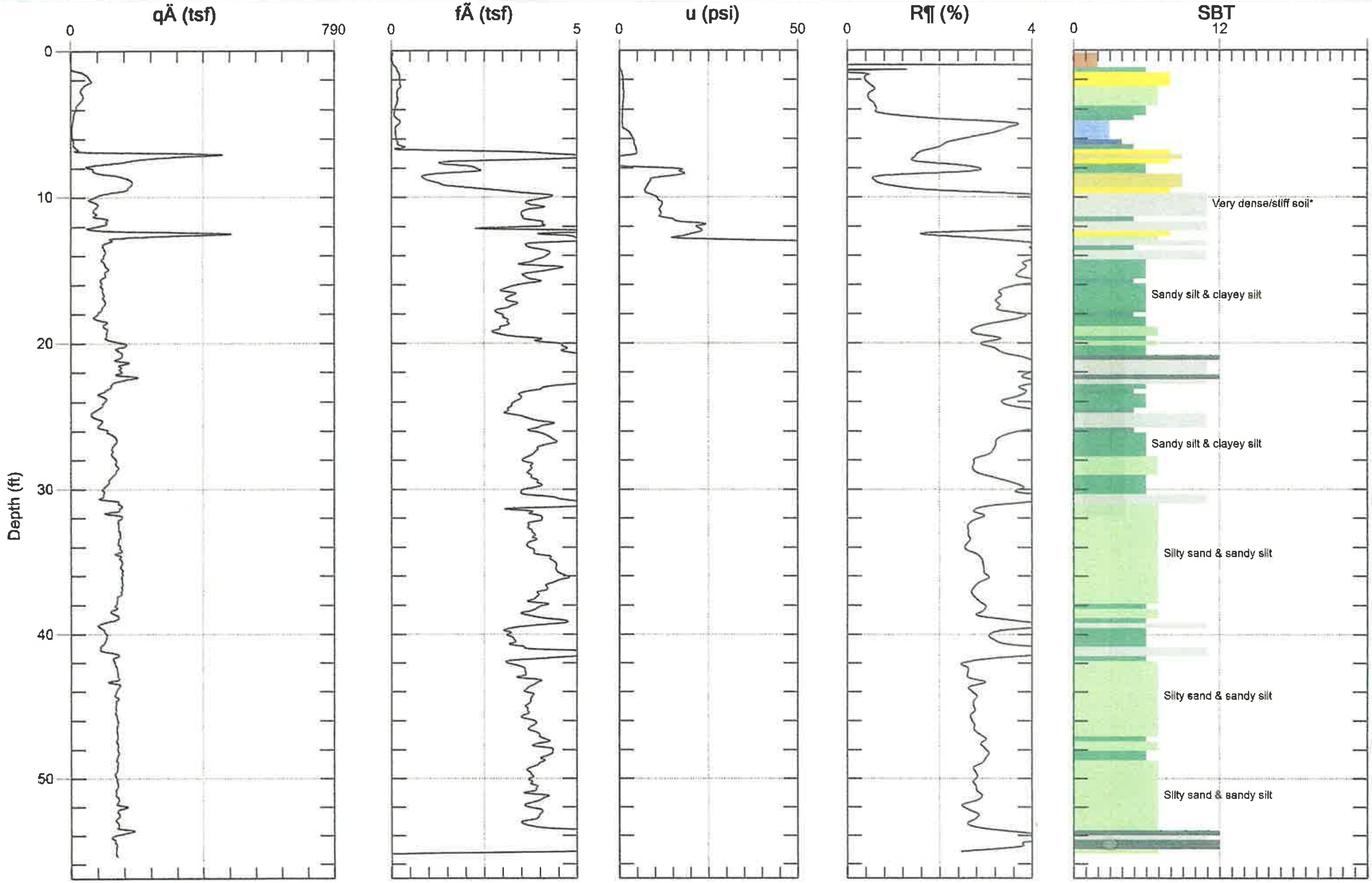
Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Well Construction
1600			... same as above; hard.		1600 B4-20	MD	7 23 32			
1605 25			... same as above; 2" angular rock @ 25.5'; trace shell fragments.		1605 B4-25		17 32 22		25	
1610 30			... same as above; no laminations present.		1610 B4-30		7 14 26		30	
			Hole terminated at 31.5 feet.							
35									35	

APPENDIX B
CONE PENETROMETER TEST RESULTS

Stantec Consulting Services

Site: Future Auto Nation
Sounding: CPT-1

Engineer: Josh Sargent
Date: 3/20/2015 11:06



Max. Depth: 55.446 (ft)
Avg. Interval: 0.328 (ft)

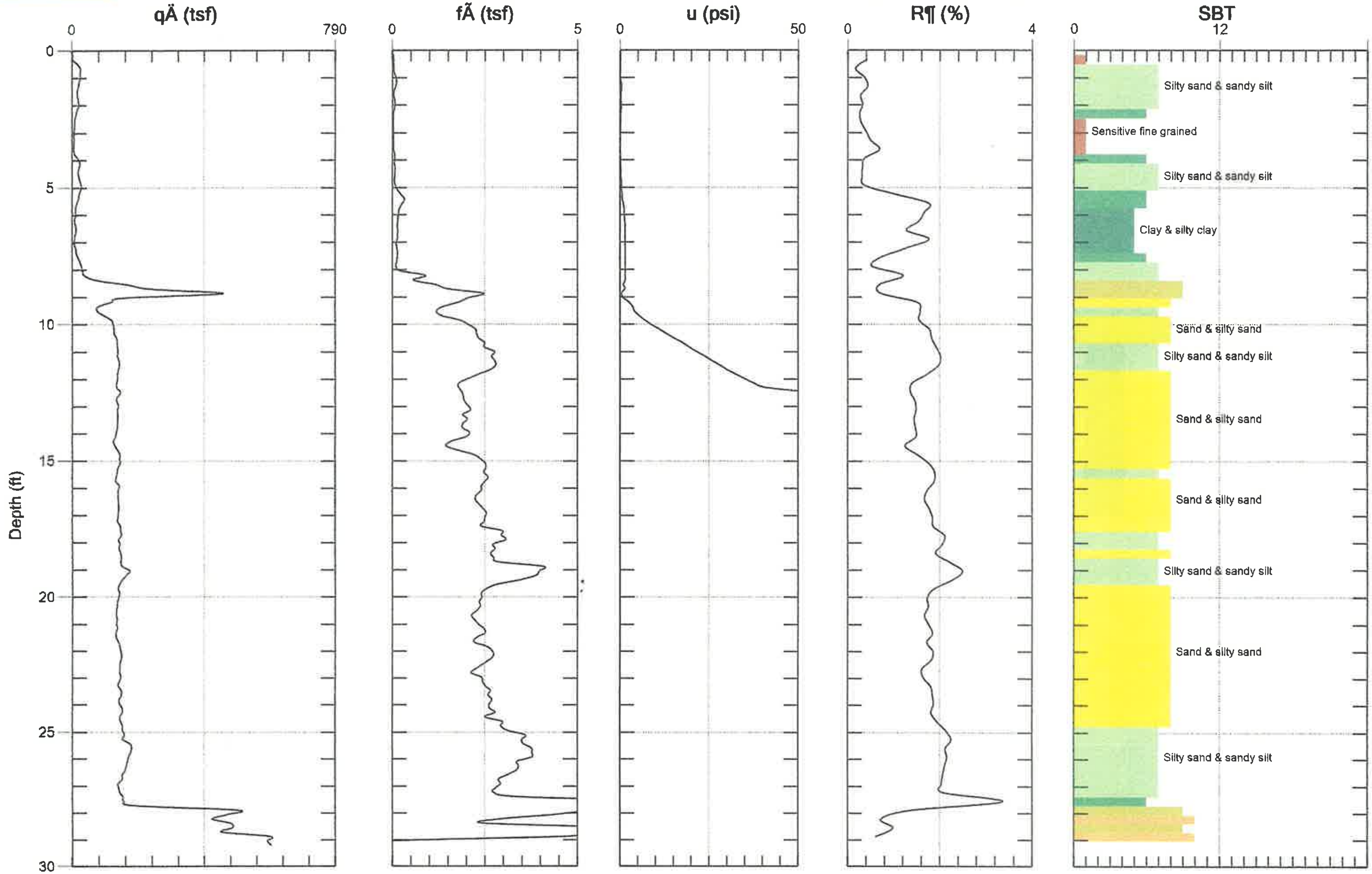
SBT: Soil Behavior Type (Robertson 1990)

GREGG

Stantec Consulting Services

Site: Future Auto Nation
Sounding: CPT-2

Engineer: Josh Sargent
Date: 3/20/2015 07:20



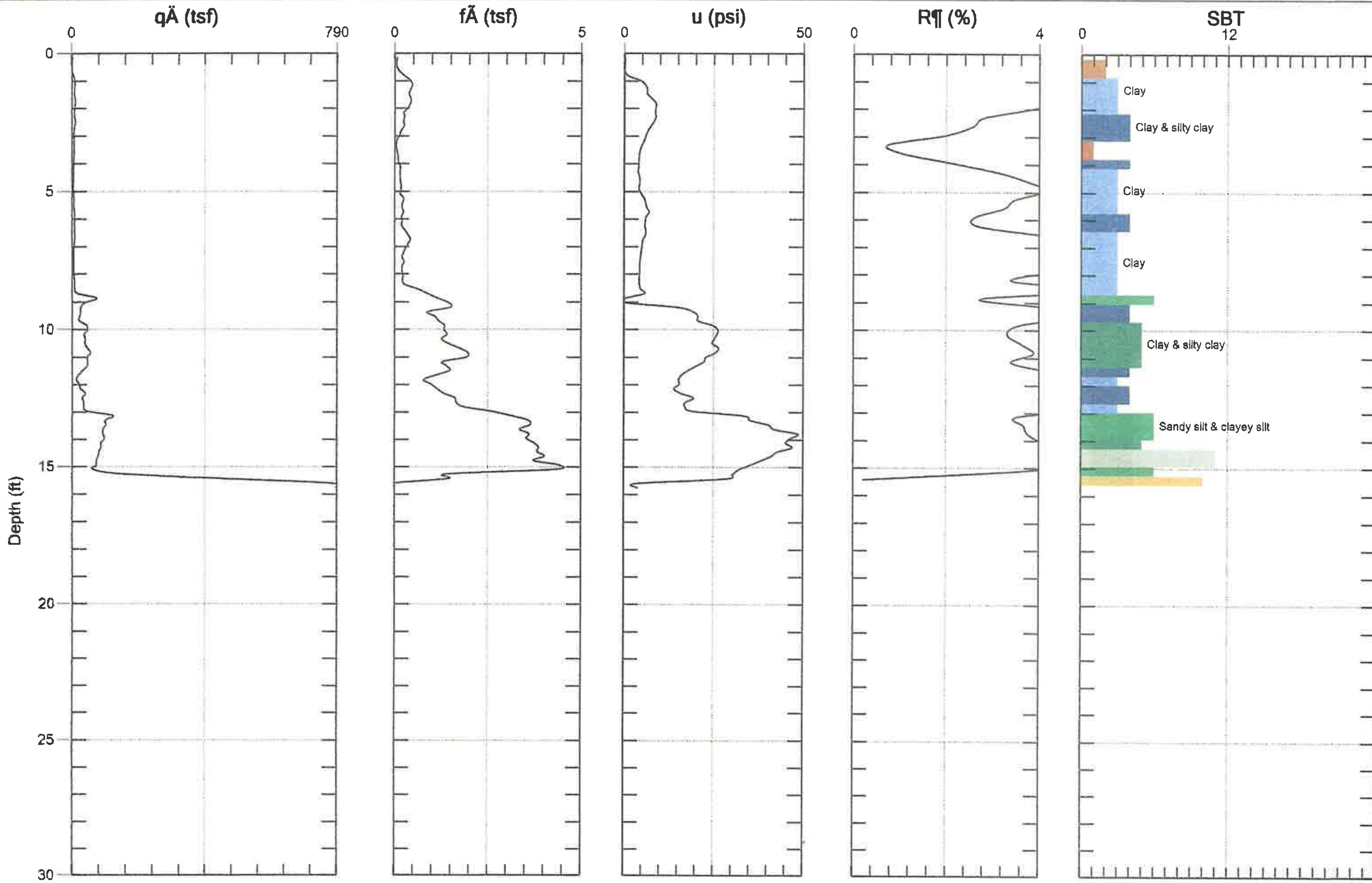
Max. Depth: 29.199 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Stantec Consulting Services

Site: Future Auto Nation
Sounding: CPT-3

Engineer: Josh Sargent
Date: 3/20/2015 08:35



Max. Depth: 15.748 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

APPENDIX C
LABORATORY TEST RESULTS

SUMMARY OF MOISTURE DENSITY TEST RESULTS
ASTM D 2216

Boring Location	Sample Depth (ft)	Wet Density (lb/ft^3)	Dry Density (lb/ft^3)	Moisture Content (percent)
B1-2	2	125.6	116.0	8.3
B1-15	15	116.5	90.0	29.4
B2-5	5	106.3	89.5	18.8
B2-20	20	110.4	81.8	34.9
B3-2	2	107.6	100.3	7.3
B3-7	7	108.6	92.4	17.5
B4-20	20	111.9	84.8	32.0

Stantec

Gradation Analysis
ASTM D 422

Project Name AutoNation - Newport
Source B3-2

Preparation Method ASTM D 1140 Method A
Particle Shape Angular
Particle Hardness Hard and Durable
Sample Dry Mass (g) 360.20
Moisture Content (%) 3.8

Project Number 2007105003
Lab ID B3-2
Date Received 03-23-2015
Preparation Date 03-25-2015
Test Date 03-26-2015

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/4"	0.00	0.0	100.0
3/8"	0.00	0.0	100.0
No. 4	3.20	0.9	99.1
No. 8	3.00	0.8	98.3
No. 10	1.10	0.3	98.0
No. 16	9.70	2.7	95.3
No. 30	36.60	10.2	85.1
No. 40	43.20	12.0	73.1
No. 50	73.00	20.3	52.9
No. 80	110.10	30.6	22.3
No. 100	20.60	5.7	16.6
No. 200	30.20	8.4	8.2
Pan	29.50	8.2	---

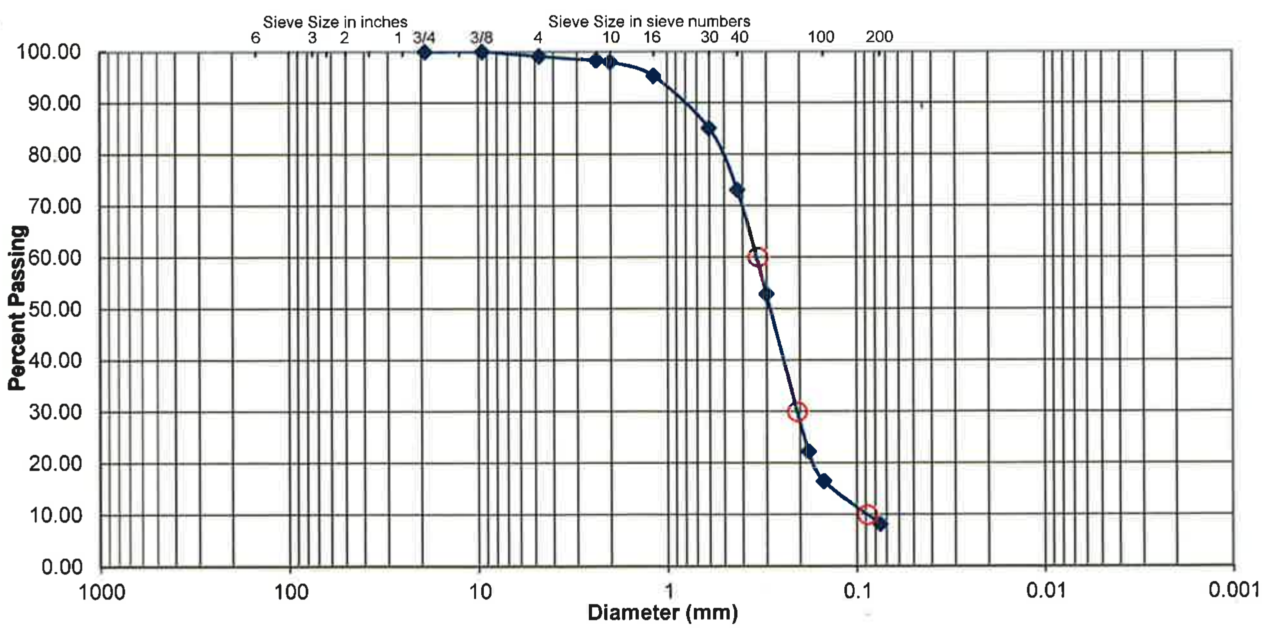
% Gravel	0.9
% Sand	90.9
% Fines	8.2
Fines Classification	ML
D_{10} (mm)	0.0871
D_{30} (mm)	0.2047
D_{60} (mm)	0.3292
Cu	3.78
Cc	1.46

Classification

Poorly Graded Sand (SP-SM) with Silt

Classification determined by ASTM D 2487. -200 material classification determined by visual assessment, ASTM D 2488.

Particle Size Distribution



Comments ______

Reviewed By ______

Stantec

Gradation Analysis
ASTM D 422

Project Name	AutoNation - Newport
Source	B3-7
Preparation Method	ASTM D 1140 Method A
Particle Shape	Angular
Particle Hardness	Hard and Durable
Sample Dry Mass (g)	132.00
Moisture Content (%)	23.3

Project Number	2007105003
Lab ID	B3-7
Date Received	03-23-2015
Preparation Date	03-25-2015
Test Date	03-26-2015

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/4"	0.00	0.0	100.0
3/8"	0.00	0.0	100.0
No. 4	0.00	0.0	100.0
No. 8	0.40	0.3	99.7
No. 10	0.40	0.3	99.4
No. 16	2.80	2.1	97.3
No. 30	12.40	9.4	87.9
No. 40	10.70	8.1	79.8
No. 50	16.50	12.5	67.3
No. 80	40.10	30.4	36.9
No. 100	8.80	6.7	30.2
No. 200	17.00	12.9	17.3
Pan	22.90	17.3	---

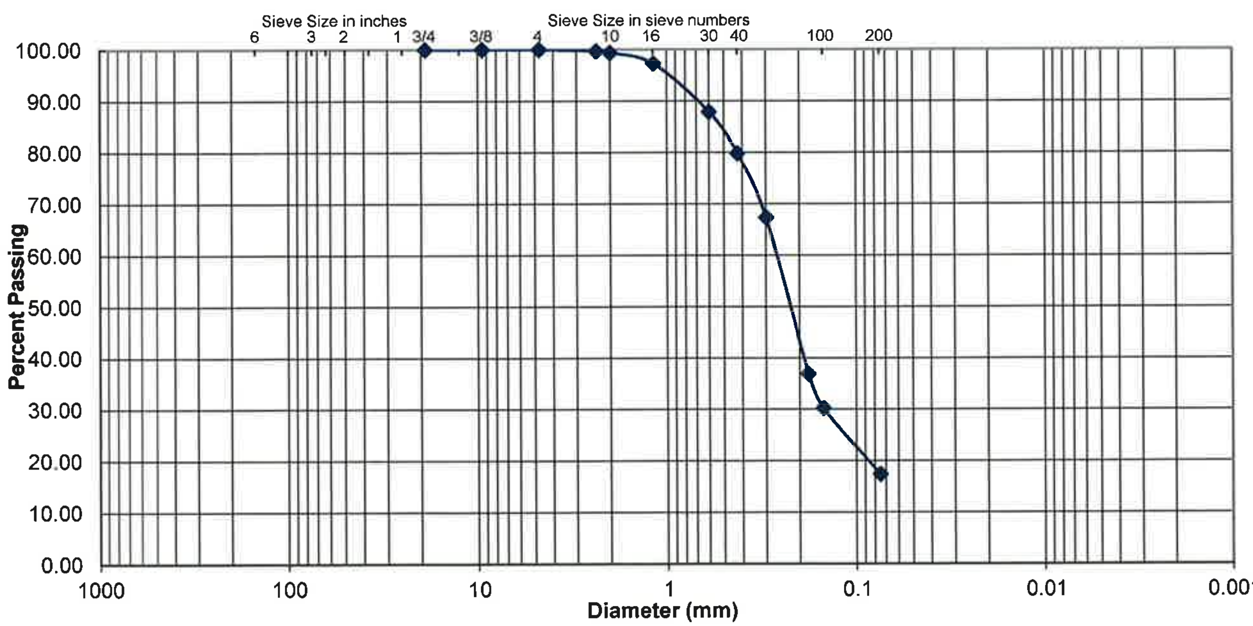
% Gravel	0.0
% Sand	82.7
% Fines	17.3
Fines Classification	ML
D_{10} (mm)	N/A
D_{30} (mm)	N/A
D_{60} (mm)	N/A
Cu	N/A
Cc	N/A

Classification

Silty Sand (SM)

Classification determined by ASTM D 2487. -200 material classification determined by visual assessment, ASTM D 2488.

Particle Size Distribution



Comments ____________________

Reviewed By ____________________

File: newport_porsche_b3-7_sieve.xlsm Sheet: Report
Preparation Date: 1-2008
Revision Date: 4-2008

Stantec Consulting Services Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Stantec

Gradation Analysis
ASTM D 422

Project Name AutoNation - Newport
Source B1-2

Project Number 2007105003
Lab ID B1-2
Date Received 03-23-2015
Preparation Date 03-25-2015
Test Date 03-26-2015

Preparation Method ASTM D 1140 Method A
Particle Shape Angular
Particle Hardness Hard and Durable
Sample Dry Mass (g) 309.20
Moisture Content (%) 9.7

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/4"	0.00	0.0	100.0
3/8"	0.00	0.0	100.0
No. 4	0.60	0.2	99.8
No. 8	0.40	0.1	99.7
No. 10	0.20	0.1	99.6
No. 16	2.40	0.8	98.8
No. 30	24.50	7.9	90.9
No. 40	45.30	14.7	76.3
No. 50	67.20	21.7	54.5
No. 80	79.00	25.5	29.0
No. 100	12.40	4.0	25.0
No. 200	23.50	7.6	17.4
Pan	53.70	17.4	---

% Gravel 0.2
% Sand 82.4
% Fines 17.4
Fines Classification ML

D_{10} (mm) N/A
D_{30} (mm) N/A
D_{60} (mm) N/A

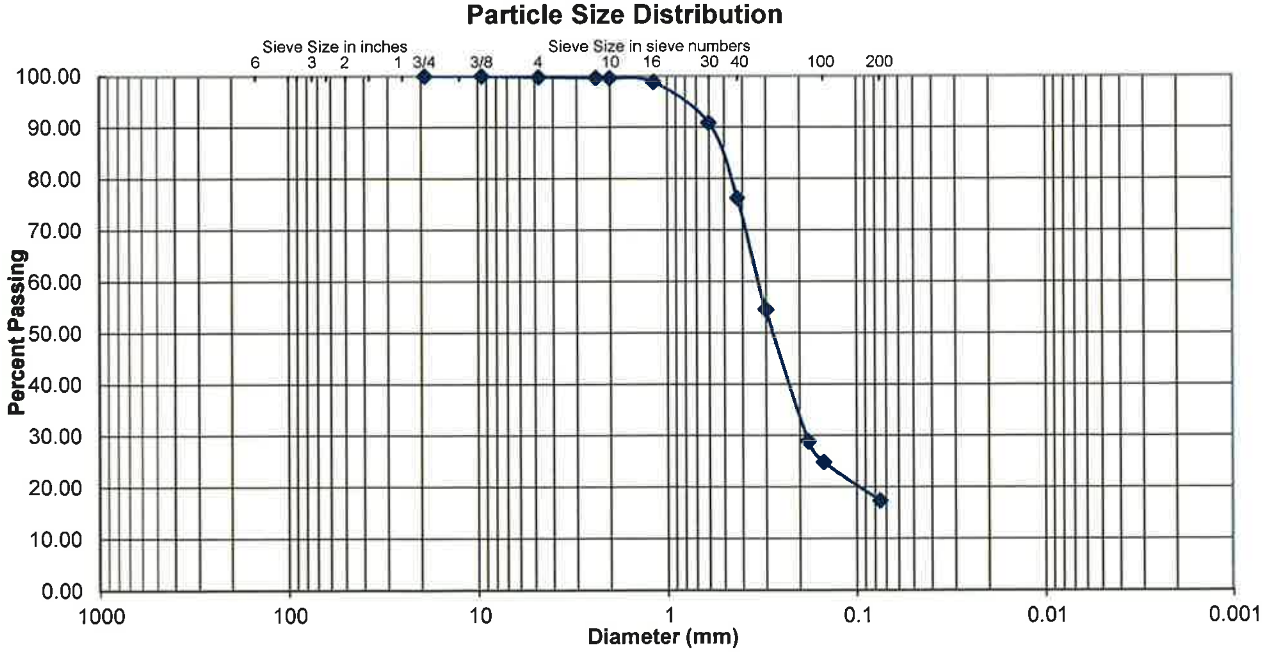
Cu N/A
Cc N/A

Classification

Silty Sand (SM)

Classification determined by ASTM D 2487. -200 material classification determined by visual assessment, ASTM D 2488.

Particle Size Distribution



Comments

Reviewed By

Stantec

Gradation Analysis
ASTM D 422

Project Name AutoNation - Newport
Source B1-5

Project Number 2007105003
Lab ID B1-5
Date Received 03-23-2015
Preparation Date 03-25-2015
Test Date 03-26-2015

Preparation Method ASTM D 1140 Method A
Particle Shape Angular
Particle Hardness Hard and Durable
Sample Dry Mass (g) 310.60
Moisture Content (%) 18.1

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/4"	0.00	0.0	100.0
3/8"	0.00	0.0	100.0
No. 4	0.50	0.2	99.8
No. 8	1.50	0.5	99.4
No. 10	0.30	0.1	99.3
No. 16	3.00	1.0	98.3
No. 30	21.10	6.8	91.5
No. 40	32.20	10.4	81.1
No. 50	48.40	15.6	65.6
No. 80	62.20	20.0	45.5
No. 100	12.00	3.9	41.7
No. 200	33.60	10.8	30.8
Pan	95.80	30.8	---

% Gravel 0.2
% Sand 69.0
% Fines 30.8
Fines Classification ML

D_{10} (mm) N/A
D_{30} (mm) N/A
D_{60} (mm) N/A

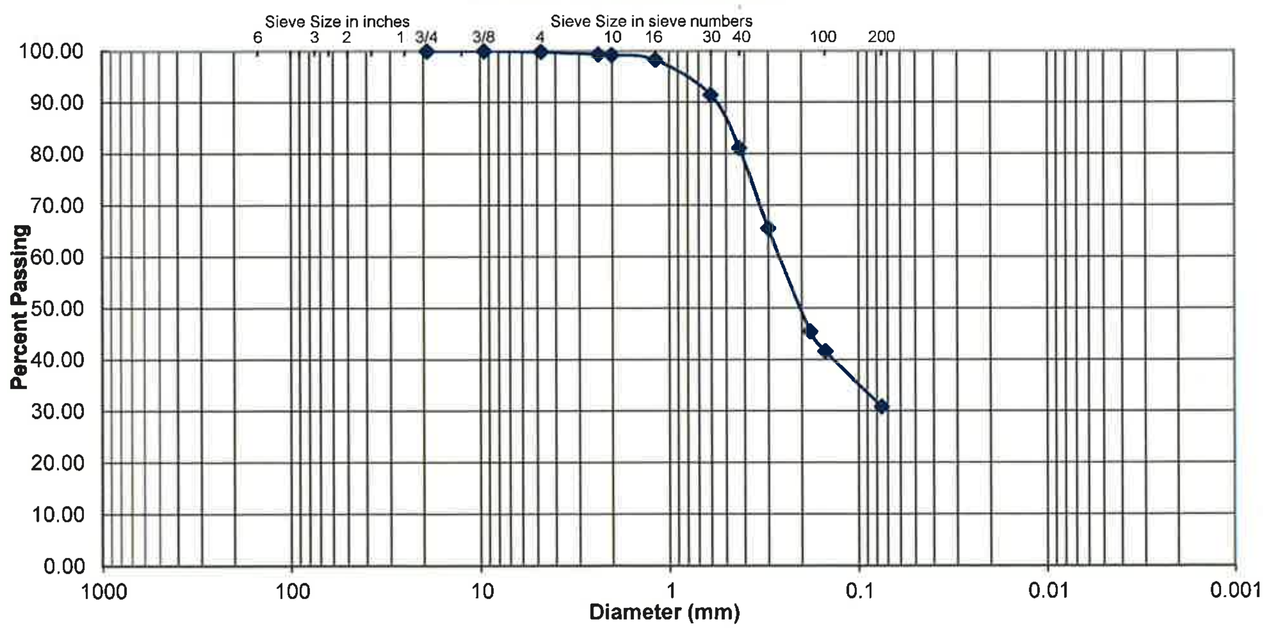
Cu N/A
Cc N/A

Classification

Silty Sand (SM)

Classification determined by ASTM D 2487. -200 material classification determined by visual assessment, ASTM D 2488.

Particle Size Distribution



Comments

Reviewed By

Gradation Analysis
ASTM D 422

Project Name AutoNation - Newport
Source B2-2

Project Number 2007105003
Lab ID B2-2
Date Received 03-23-2015
Preparation Date 03-25-2015
Test Date 03-26-2015

Preparation Method ASTM D 1140 Method A
Particle Shape Angular
Particle Hardness Hard and Durable
Sample Dry Mass (g) 255.80
Moisture Content (%) 11.4

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/4"	0.00	0.0	100.0
3/8"	0.00	0.0	100.0
No. 4	0.00	0.0	100.0
No. 8	0.50	0.2	99.8
No. 10	0.20	0.1	99.7
No. 16	1.40	0.5	99.2
No. 30	8.10	3.2	96.0
No. 40	14.20	5.6	90.5
No. 50	30.20	11.8	78.7
No. 80	70.10	27.4	51.3
No. 100	30.60	12.0	39.3
No. 200	68.50	26.8	12.5
Pan	32.00	12.5	---

% Gravel 0.0
% Sand 87.5
% Fines 12.5
Fines Classification ML

D_{10} (mm) N/A
D_{30} (mm) N/A
D_{60} (mm) N/A

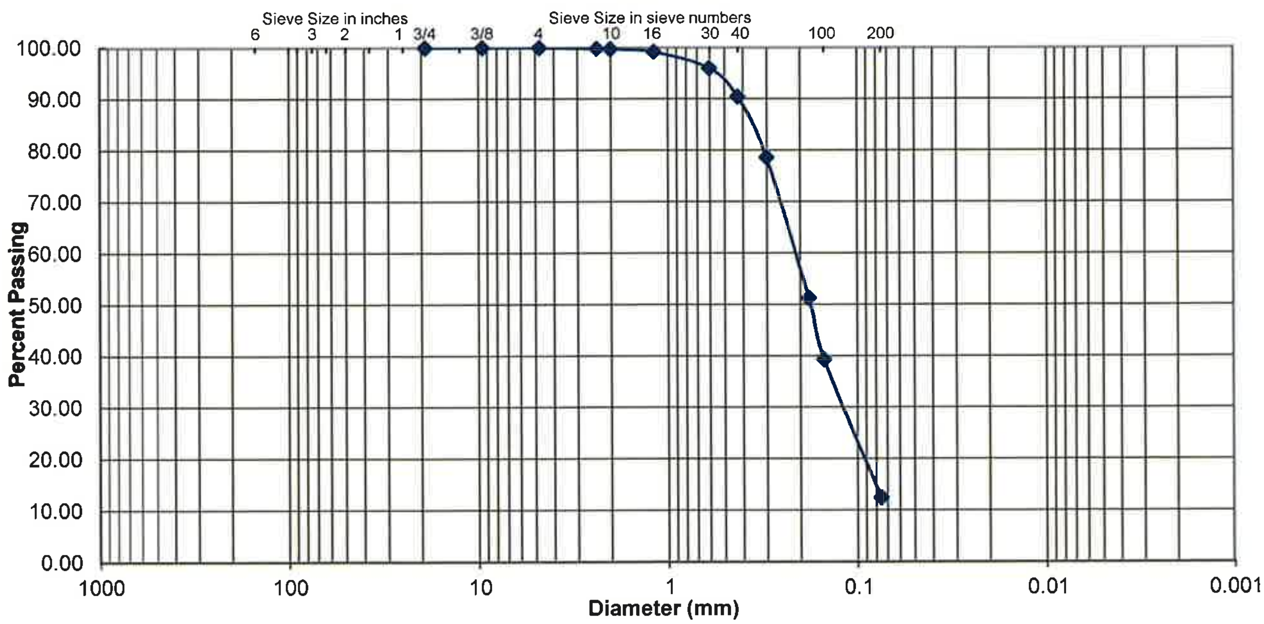
Cu N/A
Cc N/A

Classification

Silty Sand (SM)

Classification determined by ASTM D 2487. -200 material classification determined by visual assessment, ASTM D 2488.

Particle Size Distribution



Comments

Reviewed By

Stantec

Gradation Analysis

ASTM D 422

Project Name	AutoNation - Newport	Project Number	2007105003
Source	B2-5	Lab ID	B2-5
		Date Received	03-23-2015
Preparation Method	ASTM D 1140 Method A	Preparation Date	03-25-2015
Particle Shape	Angular	Test Date	03-26-2015
Particle Hardness	Hard and Durable		
Sample Dry Mass (g)	345.60		
Moisture Content (%)	14.8		

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/4"	0.00	0.0	100.0
3/8"	0.00	0.0	100.0
No. 4	2.20	0.6	99.4
No. 8	0.80	0.2	99.1
No. 10	0.50	0.1	99.0
No. 16	2.00	0.6	98.4
No. 30	12.10	3.5	94.9
No. 40	21.30	6.2	88.7
No. 50	44.20	12.8	76.0
No. 80	91.10	26.4	49.6
No. 100	33.80	9.8	39.8
No. 200	69.10	20.0	19.8
Pan	68.50	19.8	---

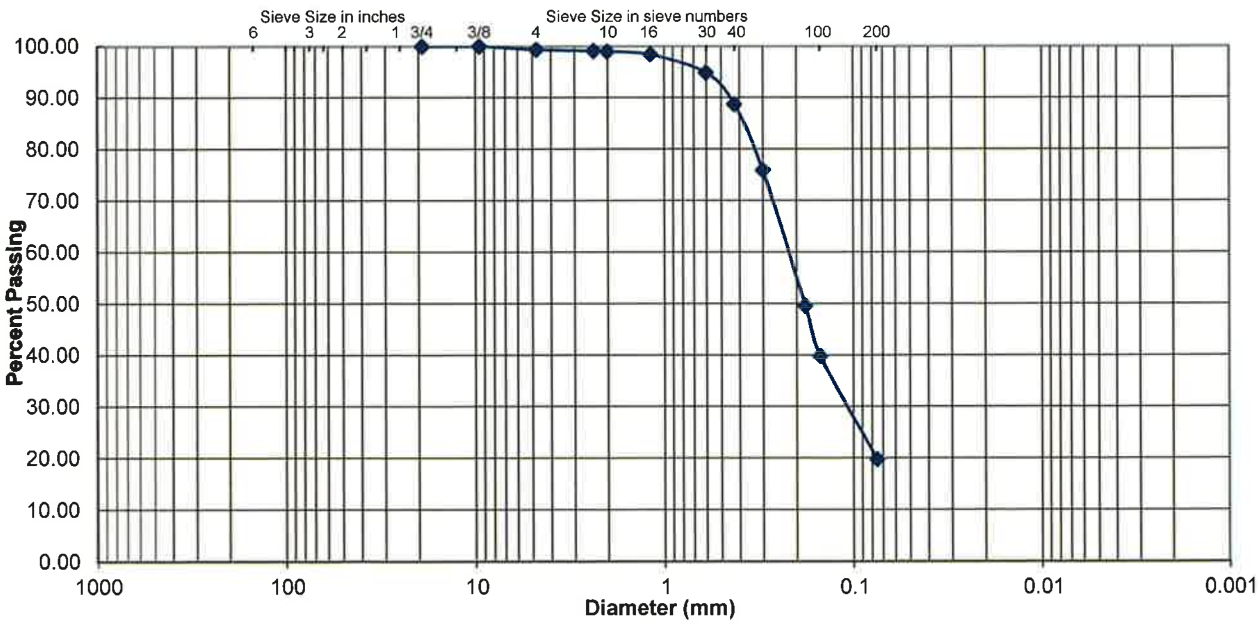
% Gravel	0.6
% Sand	79.5
% Fines	19.8
Fines Classification	ML
D_{10} (mm)	N/A
D_{30} (mm)	N/A
D_{60} (mm)	N/A
Cu	N/A
Cc	N/A

Classification

Silty Sand (SM)

Classification determined by ASTM D 2487. -200 material classification determined by visual assessment, ASTM D 2488.

Particle Size Distribution



Comments ______

Reviewed By ______

Stantec

Gradation Analysis
ASTM D 422

Project Name	AutoNation - Newport
Source	B2-25
Preparation Method	ASTM D 1140 Method A
Particle Shape	Angular
Particle Hardness	Hard and Durable
Sample Dry Mass (g)	143.10
Moisture Content (%)	17.3

Project Number	2007105003
Lab ID	B2-25
Date Received	03-23-2015
Preparation Date	03-25-2015
Test Date	03-26-2015

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/4"	0.00	0.0	100.0
3/8"	0.00	0.0	100.0
No. 4	0.00	0.0	100.0
No. 8	3.50	2.4	97.6
No. 10	2.40	1.7	95.9
No. 16	16.00	11.2	84.7
No. 30	31.30	21.9	62.8
No. 40	15.90	11.1	51.7
No. 50	14.20	9.9	41.8
No. 80	21.40	15.0	26.8
No. 100	6.60	4.6	22.2
No. 200	16.80	11.7	10.5
Pan	15.00	10.5	---

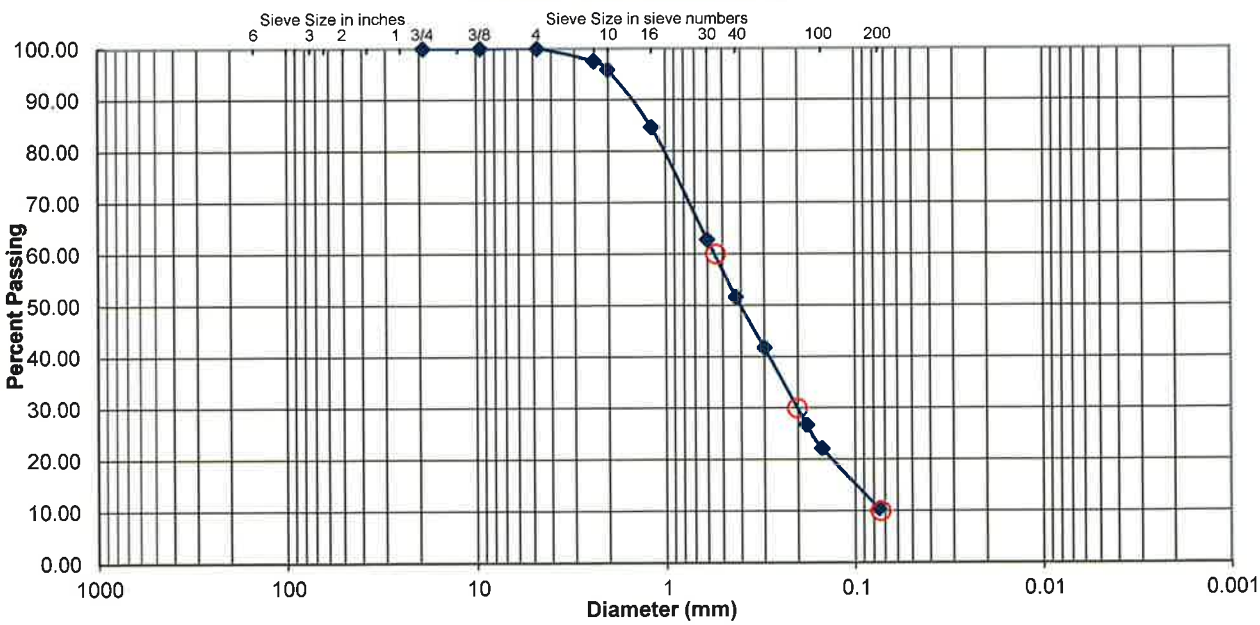
% Gravel	0.0
% Sand	89.5
% Fines	10.5
Fines Classification	ML
D_{10} (mm)	0.0729
D_{30} (mm)	0.2006
D_{60} (mm)	0.5397
Cu	7.40
Cc	1.02

Classification

Poorly Graded Sand (SP-SM) with Silt

Classification determined by ASTM D 2487. -200 material classification determined by visual assessment, ASTM D 2488.

Particle Size Distribution



Comments ____________________

Reviewed By ____________________

Stantec

Gradation Analysis
ASTM D 422

Project Name AutoNation - Newport
Source B4-5

Project Number 2007105003
Lab ID B4-5
Date Received 03-23-2015
Preparation Date 03-25-2015
Test Date 03-26-2015

Preparation Method ASTM D 1140 Method A
Particle Shape Angular
Particle Hardness Hard and Durable
Sample Dry Mass (g) 308.20
Moisture Content (%) 17.3

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
3/4"	0.00	0.0	100.0
3/8"	6.20	2.0	98.0
No. 4	3.10	1.0	97.0
No. 8	5.60	1.8	95.2
No. 10	2.20	0.7	94.5
No. 16	11.80	3.8	90.6
No. 30	27.90	9.1	81.6
No. 40	17.30	5.6	76.0
No. 50	16.80	5.5	70.5
No. 80	30.50	9.9	60.6
No. 100	14.80	4.8	55.8
No. 200	54.50	17.7	38.1
Pan	117.50	38.1	---

% Gravel 3.0
% Sand 58.9
% Fines 38.1
Fines Classification ML

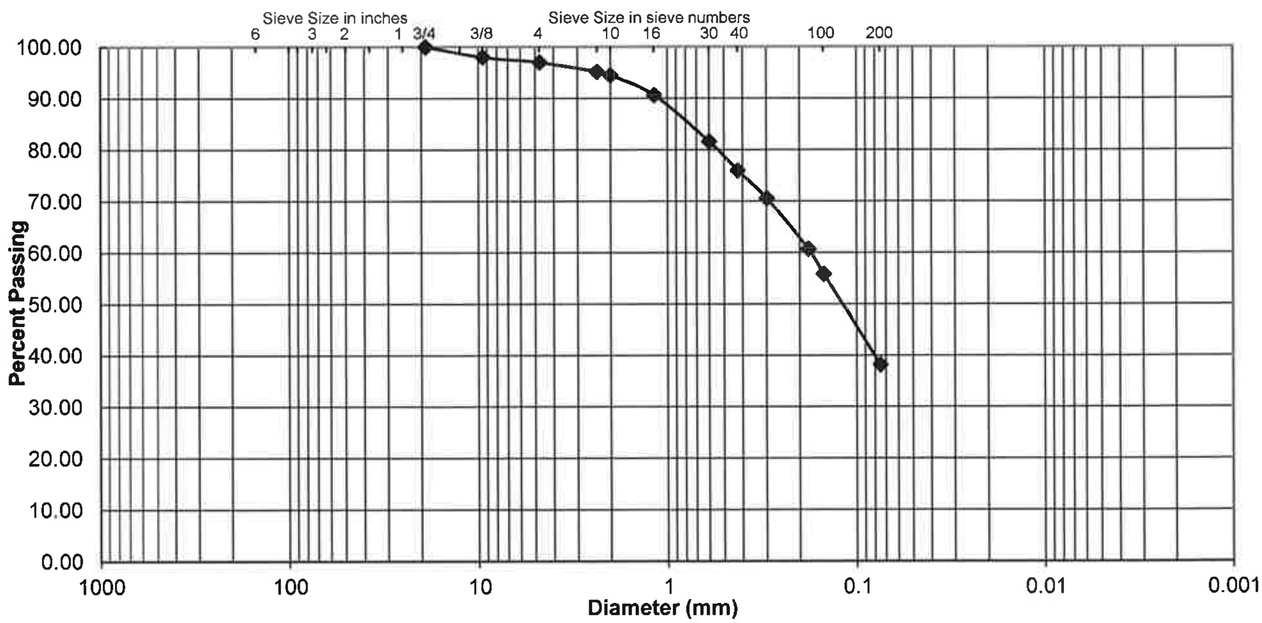
D_{10} (mm) N/A
D_{30} (mm) N/A
D_{60} (mm) N/A

Cu N/A
Cc N/A

Classification
Silty Sand (SM)

Classification determined by ASTM D 2487. -200 material classification determined by visual assessment, ASTM D 2488.

Particle Size Distribution



Comments

Reviewed By

Stantec

Materials Finer Than 75µm (No. 200) Sieve
ASTM D 1140

Project Name	AutoNation - Newport	Project Number	2007105003
Source	B3-5	Lab ID	B3-5
		Date Received	03-23-2015
Preparation Method	ASTM D 1140 Method A	Test Date	03-25-2015

Initial Sample Wet Mass (g)	410.90	Moisture Content (%)	25.3
Initial Oven Dry Sample Mass (g)	327.90		
Final Oven Dry Sample Mass (g)	232.90		
Materials Finer Than 75µm (No. 200) Sieve (g)	95.00		
Percent Finer Than 75µm (No. 200) Sieve (%)	29.0		

Comments ______________________

Reviewed By ______________

Stantec

Materials Finer Than 75µm (No. 200) Sieve

ASTM D 1140

Project Name	AutoNation - Newport	Project Number	2007105003
Source	B4-2	Lab ID	B4-2
		Date Received	03-23-2015
Preparation Method	ASTM D 1140 Method A	Test Date	03-25-2015

Initial Sample Wet Mass (g)	252.10	Moisture Content (%)	20.9
Initial Oven Dry Sample Mass (g)	208.50		
Final Oven Dry Sample Mass (g)	120.40		
Materials Finer Than 75µm (No. 200) Sieve (g)	88.10		
Percent Finer Than 75µm (No. 200) Sieve (%)	42.3		

Comments

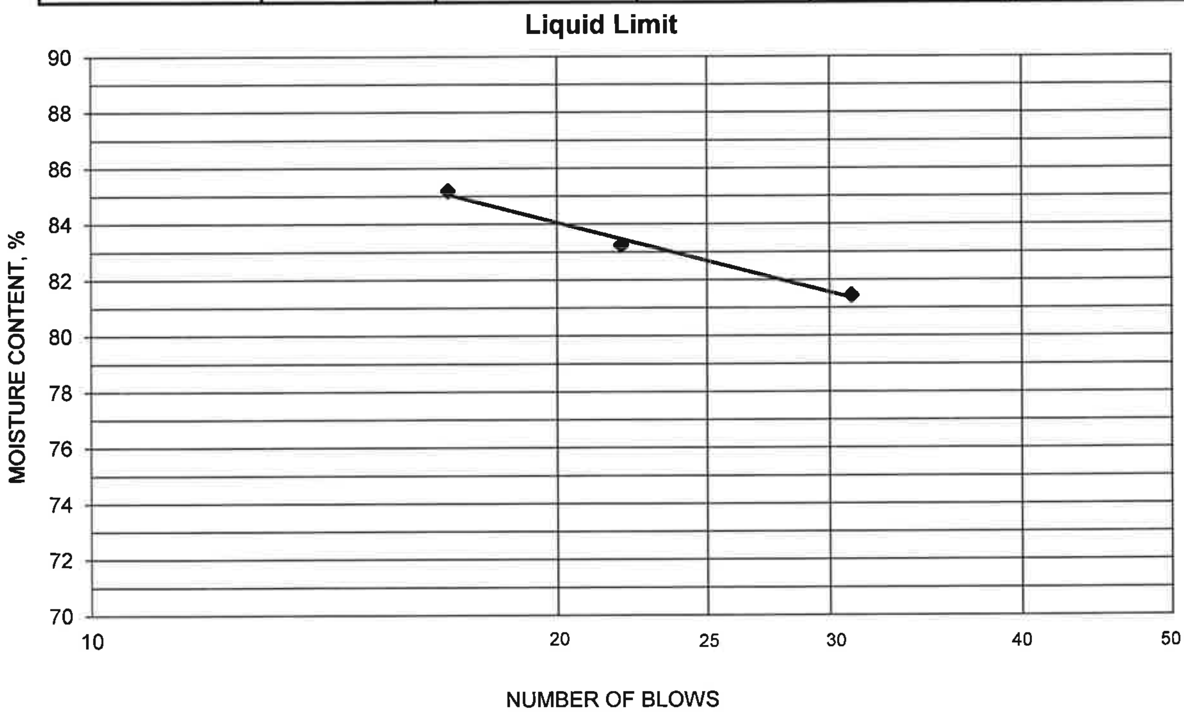
Reviewed By

Stantec

ATTERBERG LIMITS

Project	Newport Porsche	Project No.	2007105003
Source	B2-15	Lab ID	B2-15
Tested By	JP	Test Method	ASTM D 4318
% + No. 40	70		
Test Date	03-24-2015	Prepared	Dry
Date Received	03-23-2015		

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Number of Blows	Water Content (%)	Liquid Limit
14.88	13.08	10.87	31	81.4	
13.85	12.41	10.68	22	83.2	
15.41	13.34	10.91	17	85.2	83



PLASTIC LIMIT AND PLASTICITY INDEX

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Water Content (%)	Plastic Limit	Plasticity Index
26.87	22.00	12.13	49.3	49	34

Remarks: ______________________

Reviewed By ______________

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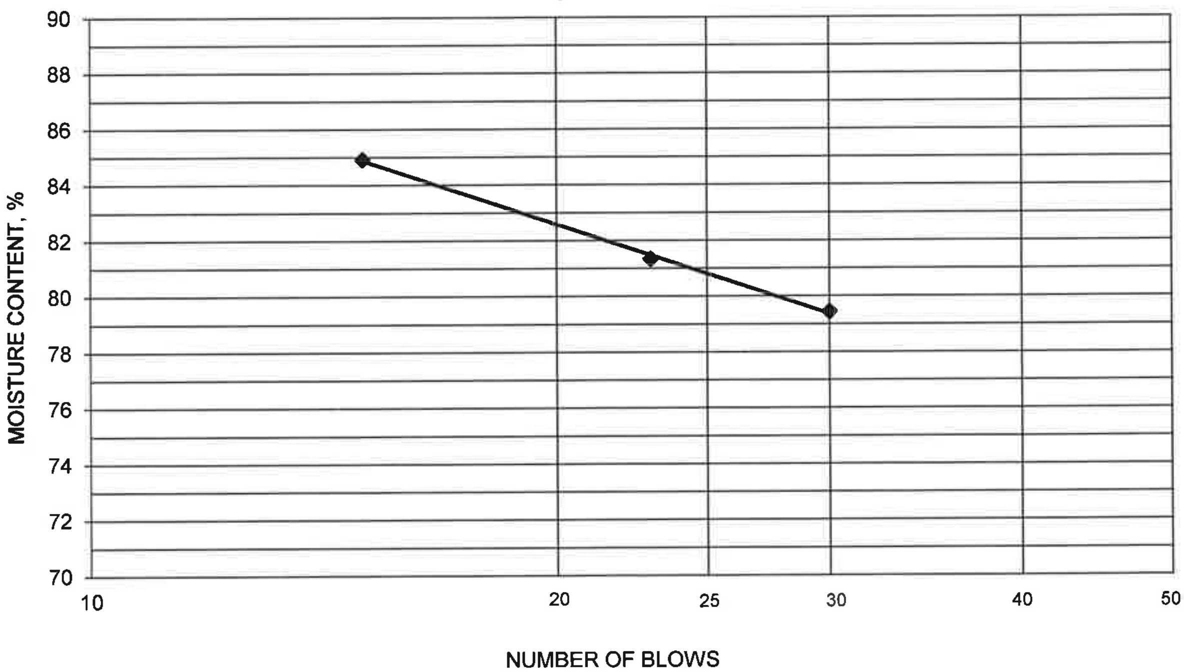
ATTERBERG LIMITS

Project	Newport Porsche	Project No.	2007105003
Source	B2-20	Lab ID	B2-20
Tested By	JP	Test Method	ASTM D 4318
% + No. 40	60		
Test Date	03-24-2015	Prepared	Dry
Date Received	03-23-2015		

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Number of Blows	Water Content (%)	Liquid Limit
12.69	10.99	8.85	30	79.4	
15.13	13.30	11.05	23	81.3	81
11.63	10.11	8.32	15	84.9	

Liquid Limit



PLASTIC LIMIT AND PLASTICITY INDEX

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Water Content (%)	Plastic Limit	Plasticity Index
25.01	20.88	12.01	46.6	47	34

Remarks: ______________________

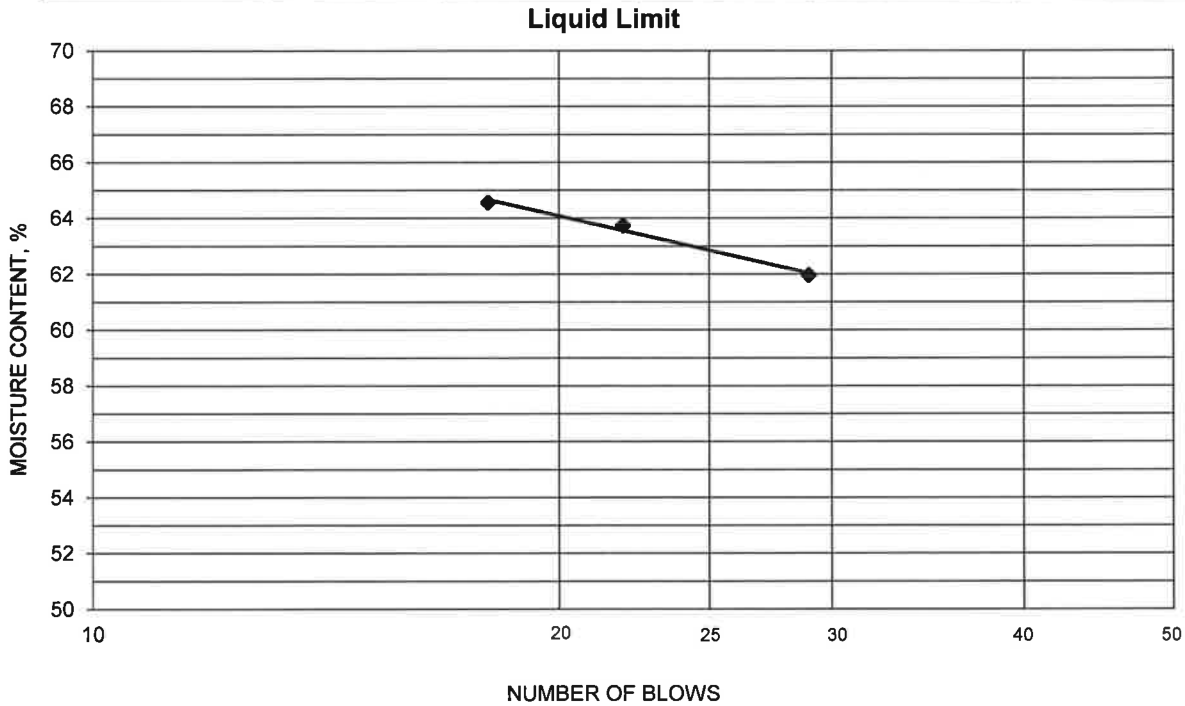
Reviewed By ______________

Stantec

ATTERBERG LIMITS

Project	Newport Porsche	Project No.	2007105003
Source	B2-30	Lab ID	B2-30
Tested By	JP	Test Method	ASTM D 4318
% + No. 40	50		
Test Date	03-24-2015	Prepared	Dry
Date Received	03-23-2015		

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Number of Blows	Water Content (%)	Liquid Limit
15.16	13.58	11.03	29	62.0	63
15.88	14.00	11.05	22	63.7	
14.04	12.71	10.65	18	64.6	



PLASTIC LIMIT AND PLASTICITY INDEX

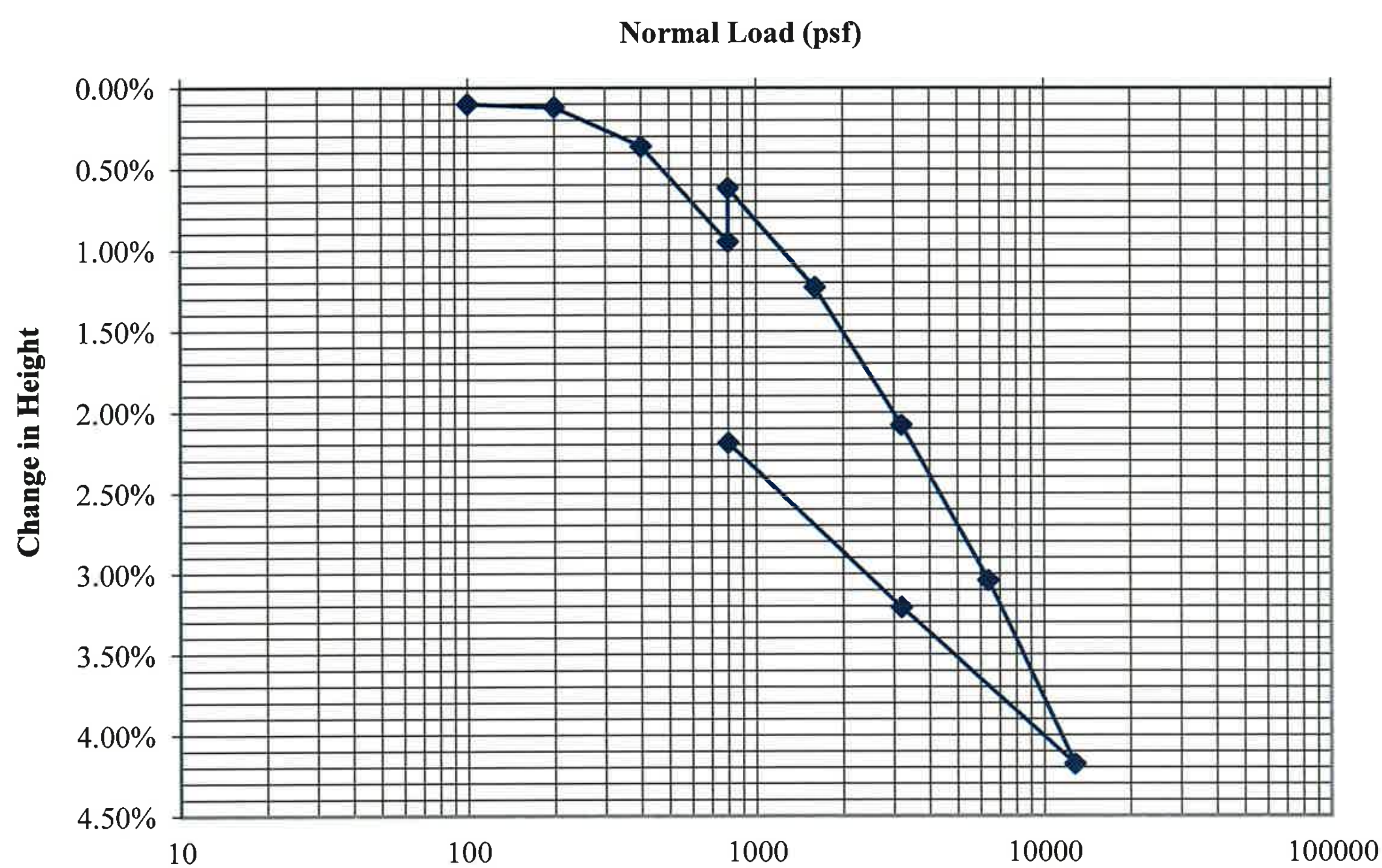
Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Water Content (%)	Plastic Limit	Plasticity Index
24.04	20.89	12.09	35.8	36	27

Remarks: ______________________

Reviewed By ______________

CONSOLIDATION TEST

AUTONATION - NEWPORT PORSCHE
B2-10



Note: Water was added at 800 psf normal load.

γ_w=105.4 pcf
m=46.5%

Figure B-1

Stantec

Compaction Characteristics of Soil Using Modified Effort

ASTM D 1557 - Method A

Project: AutoNation Newport
Source:
Description: Dark Yellow Brown SAND with Silt
Visual Notes:

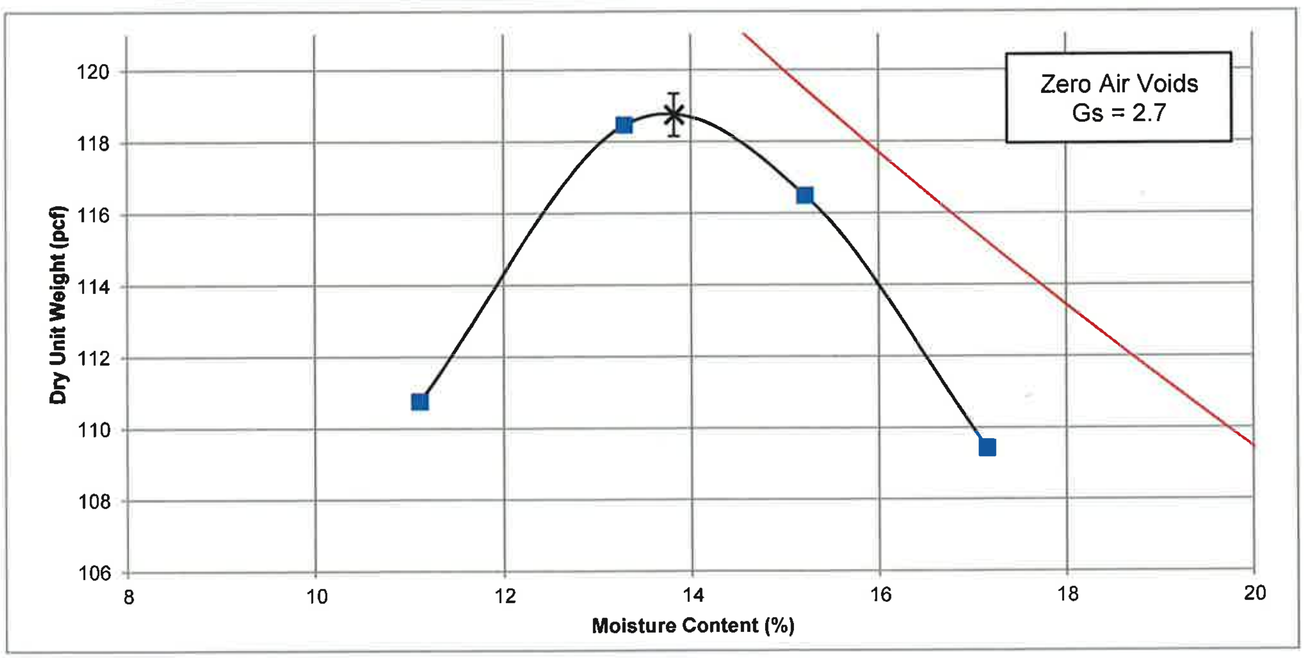
Project No.: 2007105003
Sample ID: B4 Bulk
Date Received: 03/23/2015
Date Tested: 03/19/2015

Test Fraction (%):
Gs of Test Fraction: 2.7 Estimated
Oversized Fraction Sieve: 3/4"

Oversized Fraction (%):
Gs of Oversized Fraction: 2.7 ASTM C 127
MC of Oversized Fraction (%): 11.1

Mold Weight (g): 4227
Preparation Method: Moist
Rammer Type: Manual

Wet Soil & Mold Weight (g)	Wet Soil Weight (g)	Moisture Content Determination: Wet Soil & Tare (g)	Dry Soil & Tare (g)	Tare (g)	Water Content (%)	Dry Unit Weight (pcf)
6069	1842	340.00	306.00	0.00	11.1	110.8
6236	2009	409.00	361.00	0.00	13.3	118.5
6236	2009	371.00	322.00	0.00	15.2	116.5
6146	1919	362.00	309.00	0.00	17.2	109.4



Maximun Dry Unit Weight (pcf): **118.7**
Optimum Moisture Content (%): **13.8**

Corrected Maximun Dry Unit Weight (pcf): **N/A**
Corrected Optimum Moisture Content (%): **N/A**

Comments:

Converse Consultants

Geotechnical Engineering, Environmental and Groundwater Science, Inspection and Testing Services

April 8, 2015

Mr. Jaret Fischer
Stantec Consulting Inc.
25864-F Business Center Drive
Redlands, CA 92374

Subject: **LABORATORY TEST RESULTS**
2007105003 – Auto Nation Porsche
Converse Project No. 15-81-104-08

Dear Mr. Fischer:

Presented below are the results of the laboratory tests that you requested for the above-referenced project. We received the samples from your office on March 24, 2015. The following tests were performed in accordance with the relevant standard:

- Three (3) Direct Shear Tests (ASTM D3080)
- Three (3) Hydrometer Tests (ASTM D422)
- One (1) Expansion Index Test (ASTM D4829)
- One (1) Soil Corrosivity Test (Caltrans 643, 422, 417, and 532)

We appreciate the opportunity to be of continued service to Stantec Consulting Inc. If you should have any questions or need additional information, please feel free to contact us at (909) 796-0544.

CONVERSE CONSULTANTS

Jordan Roper, E.I.T.
Staff Engineer

KVG/JR

Encl: Table No. 1, *Direct Shear Test Results*
Table No. 2, *Hydrometer Test Results*
Table No. 3, *Expansion Index Test Results*
Table No. 4, *Corrosivity Test Results*
Drawing No. 1 - 3, *Direct Shear Test Results*
Drawing No. 4, *Grain Sized Distribution Results*

Table No. 1, Direct Shear Test Results

Sample ID	Depth (feet)	Soil Description	Cohesion	Friction Angle
B-4 @ 5'	5.0	Silty Sand (SM), Fine to Medium Grained, Olive-Brown	10	33
B-1 @ 7'	7.0	Silty Sand (SM), Fine to Medium Grained, Yellow-Brown	40	32
B-4 @ 10'	10.0	Sandy Clay (CL), Fine to Medium Grained Sand, Olive-Yellow	440	27

Table No. 2, Hydrometer Test Results

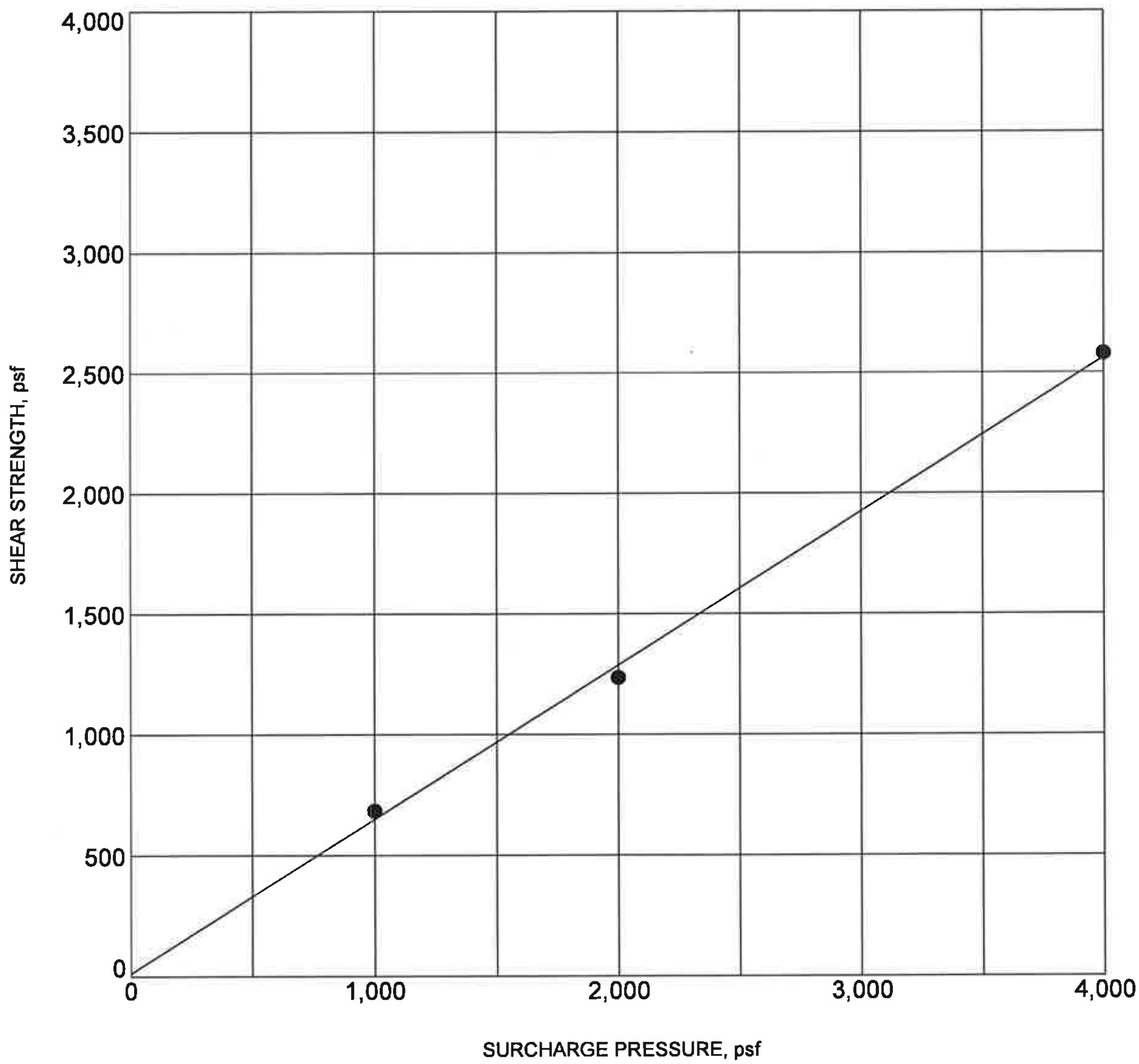
Sample ID	Depth (feet)	Percent Finer (%)			Silt (%)	Clay (%)
		#10	#50	#200		
B-2 @ 15'	15.0	100.00	85.48	85.5	64.84	20.64
B-2 @ 20'	20.0	99.40	93.03	83.87	59.66	24.21
B-2 @ 30'	30.0	100.00	90.70	79.27	61.98	17.29

Table No. 3, Expansion Index Test Results

Sample ID	Soil Description	Expansion Index	Expansion Potential
B-4 Bulk	Clayey Sand (SC), Fine to Medium Grained, Olive	30	Low

Table No. 4, Corrosivity Test Results

Sample Location	pH	Soluble Sulfate (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Saturated Resistivity (CA 643) Ohm-cm
B-1 @ 0-5'	7.7	1060	531	480



BORING NO. :	B-4 @ 5'	DEPTH (ft) :	5.0
DESCRIPTION :	Silty Sand (SM), Fine to Medium Grained, Olive-Brown		
COHESION (psf) :	10	FRICTION ANGLE (degrees):	33
MOISTURE CONTENT (%) :	24.5	DRY DENSITY (pcf) :	85.7

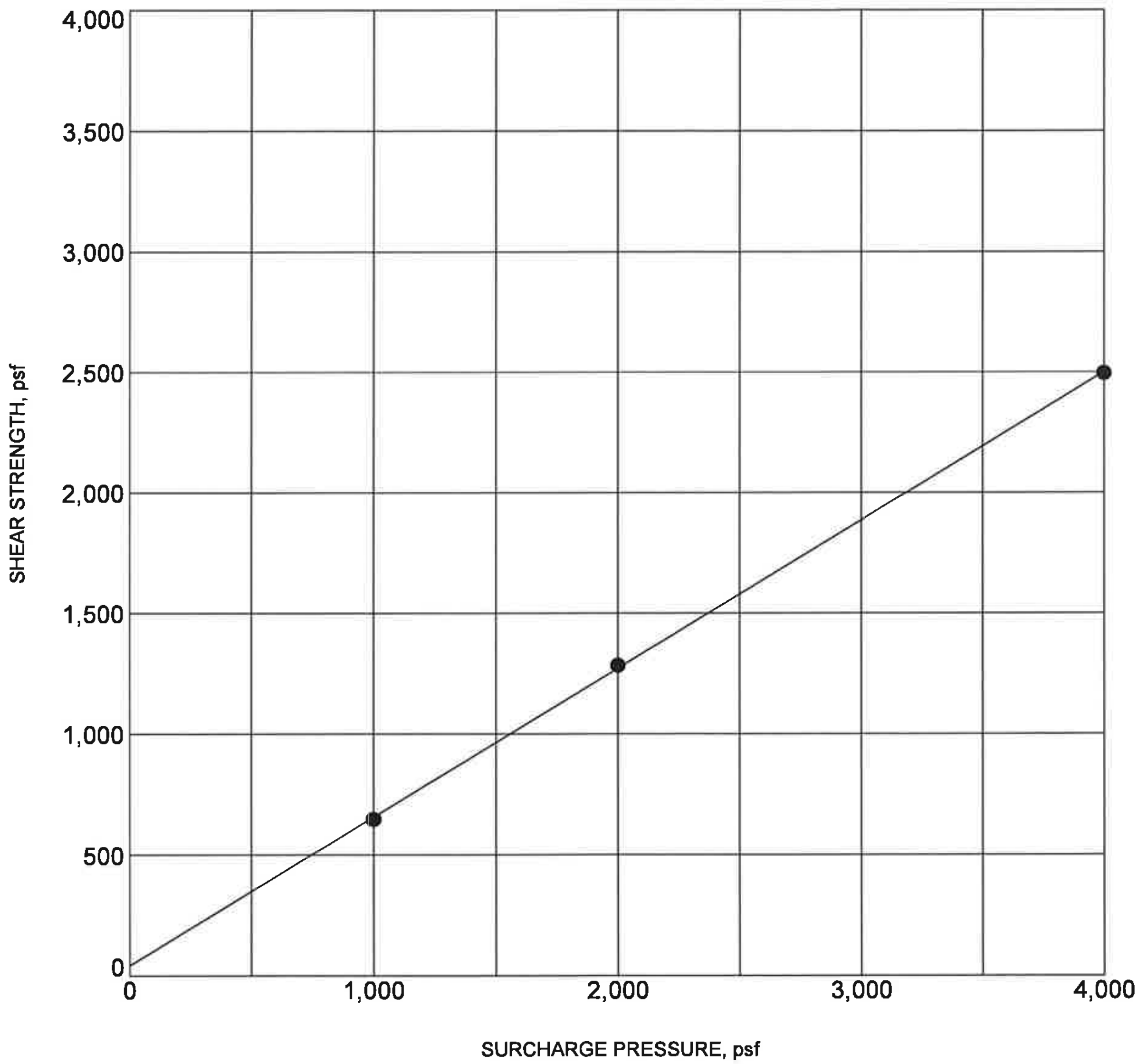
NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Auto Nation Porsche
Job No: 2007105003
For: Stantec

Project No.	Drawing No.
15-81-104-08	1



BORING NO. :	**B-1 @ 7'**	DEPTH (ft) :	**7.0**
DESCRIPTION :	**Silty Sand (SM), Fine to Medium Grained, Yellow-Brown**		
COHESION (psf) :	**40**	FRICTION ANGLE (degrees):	**32**
MOISTURE CONTENT (%) :	**26.3**	DRY DENSITY (pcf) :	**97.2**

NOTE: Ultimate Strength.

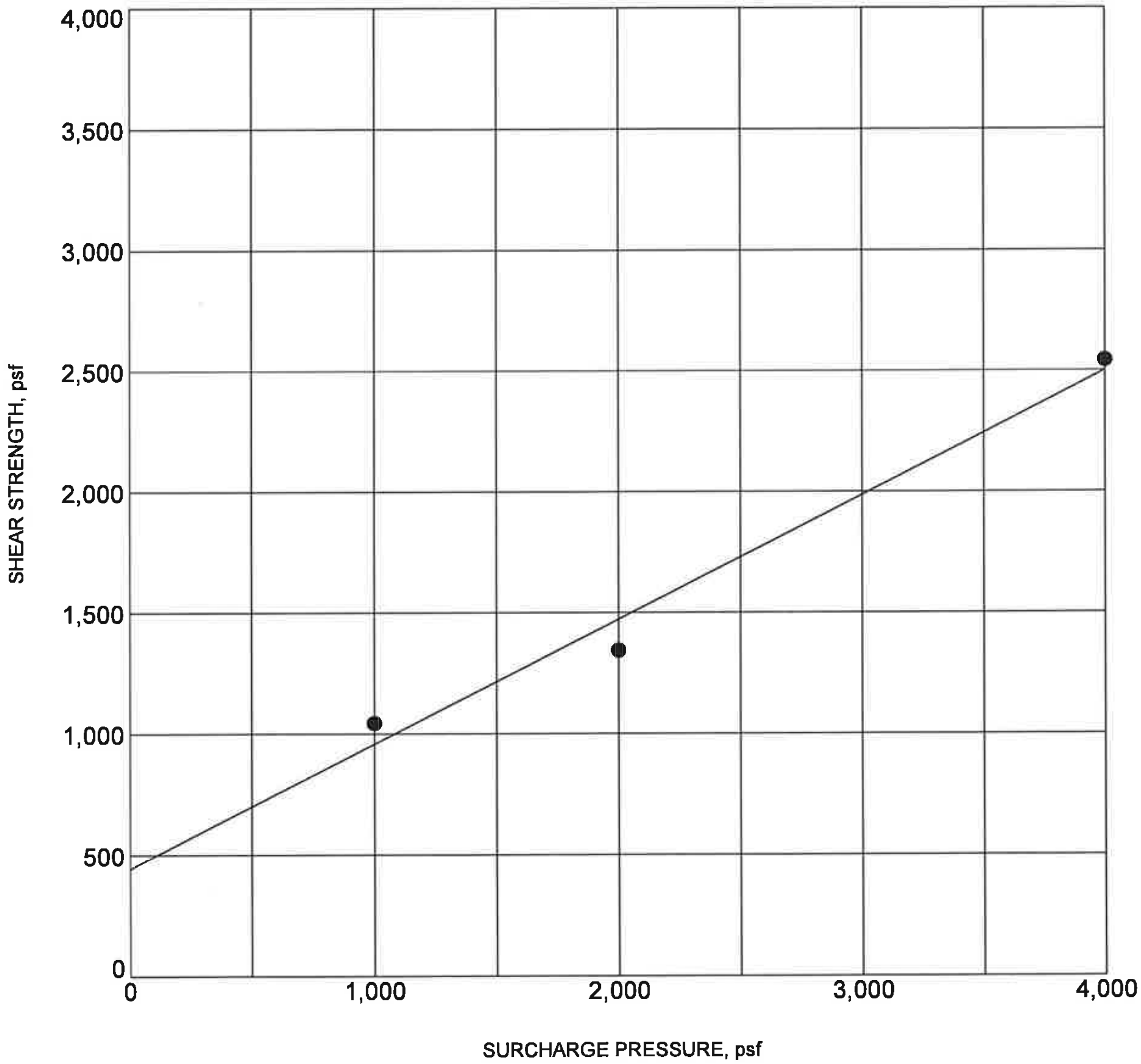
DIRECT SHEAR TEST RESULTS

Converse Consultants

Auto Nation Porsche
Job No: 2007105003
For: Stantec

Project No. **15-81-104-08**

Drawing No. **2**



BORING NO. :	B-4 @ 10'	DEPTH (ft) :	10.0
DESCRIPTION :	Sandy Clay (CL), Fine to Medium Grained, Olive-Yellow		
COHESION (psf) :	440	FRICTION ANGLE (degrees):	27
MOISTURE CONTENT (%) :	53.0	DRY DENSITY (pcf) :	69.3

NOTE: Ultimate Strength.

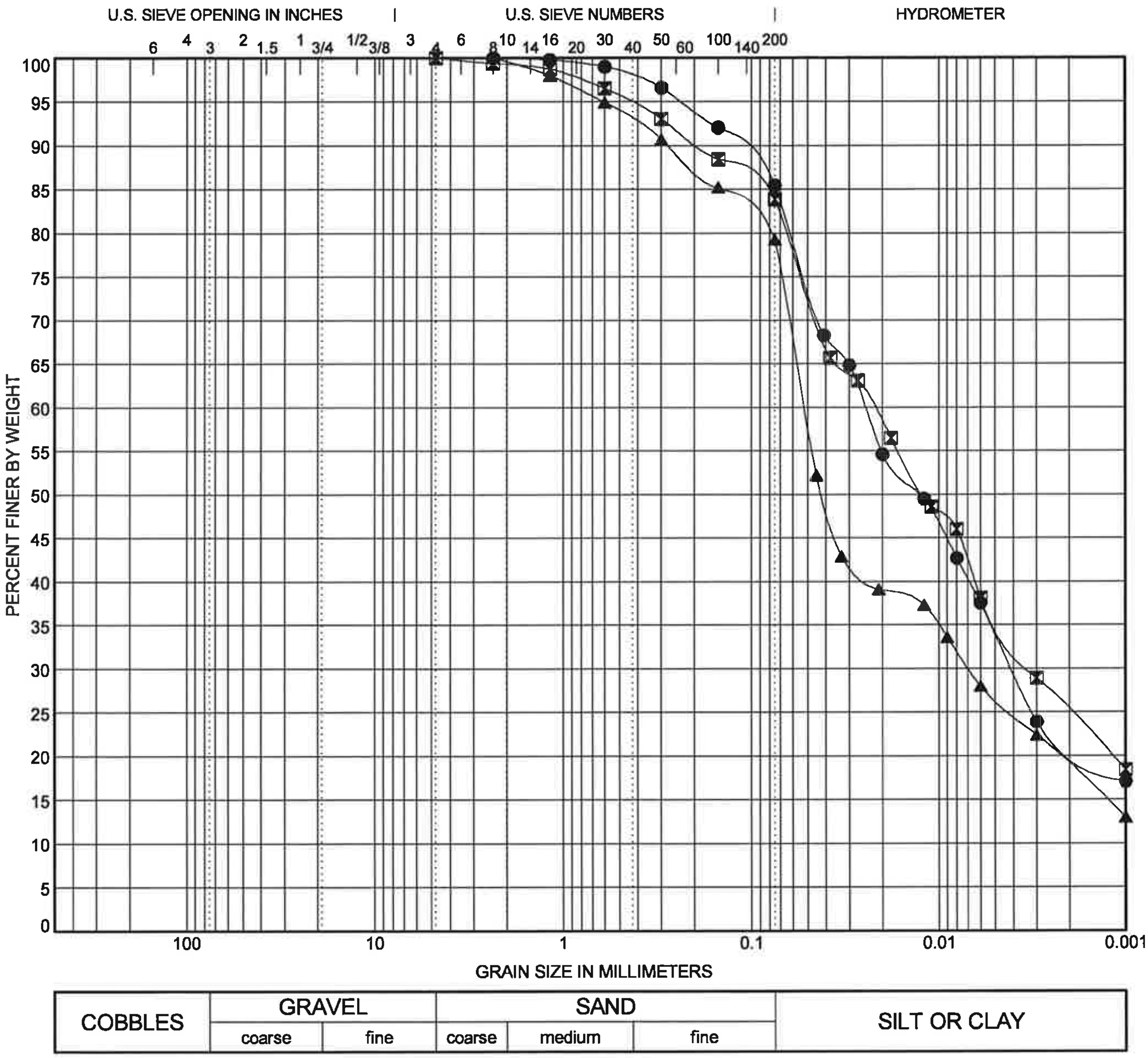
DIRECT SHEAR TEST RESULTS

Converse Consultants

Auto Nation Porsche
Job No: 2007105003
For: Stantec

Project No. 15-81-104-08

Drawing No. 3



	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-2 @ 15'	15	Clayey Silt (ML)					
⊠	B-2 @ 20'	20	Clayey Silt (ML), with Sand					
▲	B-2 @ 30'	30	Clayey Silt (ML), with Sand					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-2 @ 15'	15	2.36	0.025	0.004		0.0	14.5	85.5	
⊠	B-2 @ 20'	20	4.75	0.022	0.003		0.0	16.1	83.9	
▲	B-2 @ 30'	30	2.36	0.052	0.007		0.0	20.7	79.3	

GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Auto Nation Porsche
Job No: 2007105003
For: Stantec

Project No. 15-81-104-08

Drawing No. 4

HDR

Table 1 - Laboratory Tests on Soil Samples

Converse Consultants
Auto Nation
Your #15-81-104-08, HDR Lab #15-0248LAB
31-Mar-15

Sample ID			
			B-1 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	2,000
saturated		ohm-cm	480
pH			7.7
Electrical Conductivity		mS/cm	0.84
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	316
magnesium	Mg^{2+}	mg/kg	59
sodium	Na^{1+}	mg/kg	419
potassium	K^{1+}	mg/kg	45
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	131
fluoride	F^{1-}	mg/kg	1.0
chloride	Cl^{1-}	mg/kg	531
sulfate	SO_4^{2-}	mg/kg	1,060
phosphate	PO_4^{3-}	mg/kg	3.0
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	15
sulfide	S^{2-}	qual	na
Redox		mV	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

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APPENDIX D
GENERAL EARTHWORK AND GRADING SPECIFICATIONS

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GENERAL EARTHWORK AND GRADING SPECIFICATIONS

These general earthwork and grading specifications are for the grading and earthwork shown on the approved grading plan(s) and/or as indicated in this geotechnical report(s). These specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these general specifications. However, observations of the earthwork by the Project Geotechnical Engineer during the course of grading could result in new or revised recommendations that could supersede these specifications or the recommendations of the geotechnical report(s).

PROJECT GEOTECHNICAL ENGINEER

The owner shall contract with the Project Geotechnical Engineer of Record. The Project Geotechnical Engineer shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of grading. During the grading and earthwork operations, the Project Geotechnical Engineer shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Project Geotechnical Engineer shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of overexcavation areas, all key bottoms, and benches made on sloping ground to receive fill.

The Project Geotechnical Engineer shall observe the moisture conditioning and processing of the areas to receive fill materials and the fill materials themselves, and perform compaction testing of fill to determine the level of compaction. The responsibility of achieving soil compaction is that of the Contractor. The Project Geotechnical Engineer shall provide the test results to the owner and the Contractor on a routine and frequent basis to assist the Contractor in determining the best means to achieve the required soil compaction. The Project Geotechnical Engineer shall schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing as informed by Contractor of the anticipated schedule. The purpose of these specifications, the term Project Geotechnical Engineer includes workman working under the authority of the Project Geotechnical Engineer.

EARTHWORK CONTRACTOR

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture conditioning, processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

If requested by the Owner, the Contractor shall prepare and submit to the owner and the Project Geotechnical Engineer a work plan that indicates the sequence of earthwork grading and the estimated quantities of daily earthwork contemplated for the Site prior to commencement of grading. The Contractor shall inform the Owner and the Project

Geotechnical Engineer of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Project Geotechnical Engineer is aware of all grading operations. The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Project Geotechnical Engineer, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Contractor shall rectify the unsatisfactory conditions to the satisfaction of the Project Geotechnical Engineer. If the unsatisfactory conditions cannot be rectified to the satisfaction of the Project Geotechnical Engineer, the Owner should stop construction until an adequate plan to remedy the conditions can be established.

GUIDE SPECIFICATIONS

The following items of these guide specifications should be regarded as the minimum requirements for general earthwork and grading operations. On a Site specific basis, local governmental agencies may have more stringent requirements than specified herein.

1. All filling and backfilling operations should conform with applicable local building and safety codes and to the rules and regulations of those governmental agencies having jurisdiction over the subject construction. The earthworks contractor is responsible to notify governmental agencies, as required, and the Project Geotechnical Engineer at the initiation of grading, and when grading operations are resumed after an interruption. Each step of the grading should be approved in a specific area by the Project Geotechnical Engineer and, where required, by the applicable governmental agencies before proceeding with subsequent work.
2. Prior to the start of grading, the Site shall be cleared and grubbed of all debris, vegetation, deleterious materials, surface obstructions and loose unapproved fill shall be removed and disposed offsite. Existing irrigation, drainage or utility lines, or other abandoned subsurface structures shall be removed, destroyed or abandoned in compliance with specifications and recommendations from the Project Geotechnical Engineer, owner or local governing agencies. The Project Geotechnical Engineer shall evaluate the extent of these removals depending on Site specific conditions. No fill material or soil supporting structural fill material shall contain more than five percent organic materials (by volume). As allowed by the Owner, unsuitable materials may potentially by utilized in non-structural fill areas.
3. Existing ground that has been declared satisfactory to support fill by the Project Geotechnical Engineer shall be scarified a minimum depth of six inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.
4. In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, uncontrolled artificial fill, soft, loose, dry, saturated, spongy, organic-rich, highly fractured, porous, collapsible or otherwise unsuitable ground shall be overexcavated to competent ground, as evaluated by the Project Geotechnical Engineer during grading. Competent ground may include dense, non-porous natural deposits of soil.

5. If potentially hazardous materials are encountered, the Contractor shall stop work in the area and the Project Environmental Engineer or Project Geotechnical Engineer shall be informed immediately for proper evaluation and handling of these materials prior to continuing work in that area.
6. Where fill is placed on a sloping ground that is steeper than 20 percent, the ground to receive fill shall be prepared by proper keying and benching. The Project Geotechnical Engineer shall determine the vertical and horizontal sizes of the keys and benches. In general, the lowest keyway shall be constructed under the toe of the fill at least 15 feet in width and at least two feet deep, into competent material, as evaluated by the Project Geotechnical Engineer. Subsequent benches shall be excavated a minimum height of four feet into competent material or as otherwise recommended by the Project Geotechnical Engineer. Fill placed on sloping ground that is flatter than 20 percent shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.
7. All areas to receive fill, including processed areas, overexcavation bottoms, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested to evaluate if geotechnically suitable materials have been exposed.
8. Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan or as recommended by the Project Geotechnical engineer. The Project Geotechnical engineer may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on the actual subsurface conditions encountered during grading. A registered land surveyor/civil engineer shall survey all subdrains after installation and prior to burial for line and grade.
9. Material to be used as fill shall be approved by the Project Geotechnical engineer and shall be essentially free of organic matter and other deleterious substances. Soils of poor quality, such as those with unacceptable gradation, expansive potential (import soils with an expansion index greater than 20), or low strength shall be placed in areas acceptable to the Project Geotechnical engineer and/or mixed with other soils to achieve satisfactory fill material.
10. Oversize material defined as rock, or other irreducible material with a maximum dimension greater than three inches, shall not be buried or incorporated in the fill unless the Project Geotechnical Engineer specifically accepts the placement methods. If approved by the Project Geotechnical Engineer, placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill.
11. If importing of fill material is required for grading, proposed import material shall meet the requirements specified herein. The potential import source shall be given to the Project Geotechnical Engineer at least two working days before importing begins so that its suitability can be determined and appropriate tests can be performed.
12. Approved fill material shall be placed in areas prepared to receive fill in near-horizontal layers not exceeding eight inches in loose thickness. The Project Geotechnical Engineer may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture content throughout. Thinner layers of soil may be necessary if the Contractor is unable to achieve the required compaction.
13. Fill soils shall be moisture conditioned (e.g. watered, dried back, blended, and/or mixed, as necessary) to attain a relatively uniform moisture content near the optimum. The maximum dry density and optimum soil moisture content of fill materials shall be performed in accordance with ASTM Test Method D 1557.
14. After each layer has been moisture-conditioned, mixed, and evenly placed, the soil shall be uniformly compacted to not less than 90 percent of maximum dry density, unless otherwise specified in the approved geotechnical report(s). The contractor shall utilize equipment that is sized to efficiently achieve the specified level of compaction in a

uniform manner. The contractor's earthwork operations should not result in movement or damage to completed work.

15. Field tests for moisture content and relative compaction of the fill soils shall be performed by the Project Geotechnical Engineer in accordance with ASTM standards or as required by local governmental agencies. The location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Tests shall be taken at intervals not exceeding two feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. The Contractor shall allow the Project Geotechnical Engineer a safe means to adequately test fill construction. If the Contractor achieves substandard compaction, the contractor shall adjust the earthwork operations (which may include additional compactive energy, adjustment of moisture content, thinner soil lifts, uniform soil placement, etc.) to meet the project specifications.
16. Wherever, in the opinion of the Project Geotechnical Engineer or Owner, an unstable condition is being created by cutting or filling, the work shall not proceed in that area until an investigation has been made and the grading recommendations revised, if necessary.