Proposed Commercial Structure Preliminary Geotechnical Investigation Report

Prepared for:
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Project No. 185804814

June 27, 2020

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June 27, 2020

Mr. Zach Vela
L&R ZAV 650 Sepulveda, LLC
8445 Santa Monica Boulevard, Suite 5
West Hollywood, California 90069

RE: **PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT**
Proposed Commercial Building
650-700 North Pacific Coast Highway
El Segundo, California 90245

Dear Mr. Vela:

This letter transmits Stantec Consulting Services Inc.'s (Stantec's) preliminary geotechnical investigation report for the proposed commercial office and parking structure building located at 650-700 North Pacific Coast Highway, in the City of El Segundo, California. The purpose of this report is to evaluate the subsurface conditions and provide preliminary geotechnical recommendations for the proposed development. Final geotechnical recommendations will be necessary once the final layout of the facility and final structural loads have been established.

We appreciate the opportunity to work with you on this project. If you have any questions, please call us at the numbers below.

Respectfully submitted,

STANTEC CONSULTING SERVICES INC.

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1. INTRODUCTION

This report presents the results of Stantec's preliminary geotechnical investigation for the proposed office and parking structure building at 650 -700 North Pacific Coast Highway, in El Segundo, California (the Site). The project location is shown on the Site Location Map, Figure 1 and the approximate area of the proposed development is shown on the Site Vicinity Map, Figure 2.

1.1 PROPOSED DEVELOPMENT

We understand that the proposed office and parking structure building will include construction of a 62,426 square feet (sf) seven-story building and associated pavement and landscape areas in the proposed development. The existing eight-story 120,713 sf steel building and combined single story and two-story brick buildings totaling 73,517 sf will remain. The Site is approximately 7.2 acres in size and is currently at the proposed rough grade elevation. The approximate area of the proposed structural improvements is included on the Subsurface Exploration Map, Figure 3.

1.2 PURPOSE AND SCOPE OF WORK

1.2.1 Purpose

The purpose of this report is to evaluate the subsurface conditions at the Site and provide geotechnical recommendations for design and construction of the proposed project. This report has been prepared in general accordance with accepted geotechnical engineering principles and in general conformance with the approved proposal.

1.2.2 Scope of Work

Our scope of work consisted of the following:

- Review available subsurface information for the Site and nearby locations,
- Perform a site reconnaissance to evaluate general geotechnical and site conditions,
- Perform a field subsurface exploration program consisting of drilling six hollow stem auger (HSA) borings, and converting three of the HSA borings into deep percolation wells,
- Perform geotechnical laboratory tests on selected samples,
- Perform geotechnical engineering analyses, and
- Preparation of this geotechnical investigation report for the proposed project.

2. FIELD INVESTIGATION

2.1 PRE-DRILLING PROCEDURES

DigAlert (Underground Service Alert of Southern California) was notified before commencing subsurface exploration activities to identify underground utilities that could conflict with the proposed borings. In addition, a private utility locator was retained, and the upper five feet were hand augered to clear the boring locations for potential conflicts with underground utilities.

2.2 DRILLING OPERATIONS

Six test borings (B1 through B3 and P1 through P3) were drilled using a CME-85 drill rig equipped with hollow-stem augers between May 28 and May 29, 2020 by ABC Liovin Drilling Co. (ABC). Soil borings were advanced to depths ranging from approximately 21.5 and 51.5 feet below the existing ground surface (bgs), and their approximate locations are shown on the Subsurface Exploration Map, Figure 3. Specifically, B1 through B3 were advanced to depths ranging from 21.5 to 31.5 feet bgs and P1 through P3 were advanced to 51.5 feet bgs. The borings were logged by a Stantec field geologist, who also collected samples of the materials encountered for examination and laboratory testing.

2.3 PERCOLATION TESTING

Three of the borings were converted to deep percolation wells (P1 through P3) and percolation testing was performed in the wells by a Stantec field geologist on June 1, 2020 in general accordance with Los Angeles County percolation testing guidelines.

2.4 SAMPLING

Relatively undisturbed samples were obtained using a modified California (CAL) sampler, which is a ring-lined split tube sampler with a 3-inch outer diameter and 2½-inch inner diameter. CAL sampling followed ASTM D3550 (Standard Practice for Ring-Lined Barrel Sampling of Soils) procedures. Disturbed samples were obtained using a Standard Penetration Test (SPT) sampler, which is a split tube sampler with a 2-inch outer diameter and 1⅜-inch inner diameter. SPTs were performed in general accordance with ASTM D1586 (Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils), and D6066 (Standard Practice for Determining the Normalized Penetration Resistance of Sands for Evaluation of Liquefaction Potential). Disturbed bulk samples were also obtained from the drill cuttings.

The CAL and SPT samplers were driven with a 140-pound weight dropping 30 inches. The number of blows per 6-inch increment is noted on the boring logs. ABC provided a report (Earthspectives, 2019) which indicates the average hammer energy efficiency on the drill rig used at the project was 80%.

Samples were classified in the field using the Unified Soil Classification System (USCS), in accordance with ASTM D2488 (Standard Practice for Description and Identification of Soils [Visual-Manual Method]) procedures. The laboratory testing confirmed or modified field classifications as necessary for presentation on the boring logs. Soil samples were removed from the samplers, placed in appropriate containers, and transported in accordance with ASTM D4220 (Standard Practice for Preserving and Transporting Soil Samples). Upon completion, borings were backfilled with grout. The boring logs are included in Appendix A.

3. LABORATORY TESTING

The following laboratory tests were performed in general accordance with ASTM and California Test procedures:

Table 1. Summary of Laboratory Tests

Type of Test	ASTM Designation	Number Performed
Materials Finer than No. 200 Sieve	ASTM D1140	1
Gradation Analysis	ASTM D422 and ASTM C136	11
Atterberg Limits	ASTM D4318	1
Direct Shear	ASTM D3080	5
Chemical Tests for Corrosion Potential	CA DOT test methods	2

The laboratory test results are presented in Appendix B.

4. GEOLOGIC SETTING AND SITE CONDITIONS

4.1 REGIONAL GEOLOGY

The site is located in the northwestern portion of the Peninsular Range geologic province which consists of a series of ranges separated by northwest trending valleys, subparallel to faults branching from the San Andreas Fault. The Peninsular Ranges extend into lower California and are bound on the east by the Colorado Desert. The Site resides in the portion of the Province drained by surface runoff toward San Pedro Bay.

Geologic mapping presented in the California Geological Survey (CGS), Geologic Map of the Long Beach 30x60' Quadrangle (CGS, 2003) indicates the site is underlain by Quaternary Old Eolian Basin deposits. Literature from the CGS indicates the site is underlain by artificial fill in the western portion of the site and old eolian deposits consisting dense to very dense sand of the late to middle Pleistocene era.

4.2 SURFACE CONDITIONS

The project Site is approximately 7.2 acres in size and is currently occupied by an eight-story 128,544 sf building, one single story and one 2-story brick building totaling 78,512 sf, paved parking, and landscape areas. The existing buildings will be repurposed as office space. The project Site is bound by North Pacific Coast Highway (PCH) (aka North Sepulveda Boulevard) followed by restaurants and paved parking lots to the west, and a mix of light industrial and commercial buildings to the north, east, and south.

The Site is generally flat and slopes to the northwest to southeast. Based on Google Earth®, the ground surface of the Site is at an approximate elevation of 110 to 130 feet (WGS84 Datum).

4.3 SUBSURFACE CONDITIONS

The materials encountered in our borings consist of Old Eolian deposits (Qoe). A brief description of the subsurface conditions is provided in this section. Detailed descriptions of the subsurface conditions are provided in the boring logs included in Appendix A.

Old Eolian Deposits (Qoe) – Old Eolian deposits were encountered below the site and may extend to depths greater than 51.5 feet bgs. The Old Eolian deposits encountered at this location primarily consist of sand with variable amounts of silt and clay (SP, SW, SP-SM, SW-SC, and SM USCS soil type) and clay with variable amounts of sand (CL USCS soil type) to the maximum depth of exploration. The sandy deposits encountered were loose to very dense and generally dry to moist. The low plasticity clays were stiff and generally moist.

Groundwater - Groundwater was not encountered during this investigation. Based on groundwater data collected at an offsite location approximately 250 feet north of the site, groundwater is expected to be encountered at a depth of approximately 121 feet below the ground surface (bgs) (CRA, 2010). Groundwater levels may fluctuate in the future due to rainfall, irrigation, broken pipes, or changes in site drainage.

5. GEOLOGIC HAZARDS

5.1 FAULTING AND SURFACE FAULT RUPTURE

As in most parts of California, the Site is located in a seismically active area. The estimated closest distance from the Site to major nearby mapped active faults is presented in the table below.

Table 2. Faults in Site Vicinity

Fault	Distance (miles) (1)	Maximum Moment Magnitude (1)
Newport – Inglewood	3.5	7.5
Palos Verdes	4.9	7.7
Puente Hills (LA)	8.1	7.0
Santa Monica	8.8	7.4
Malibu Coast	10.4	7.0
Hollywood	11.0	6.7
Anacapa - Dume	11.2	7.2
Elysian Park (Upper)	13.4	6.7
Puente Hills (Santa Fe Springs)	14.7	6.7
Raymond	16.7	6.8
Verdugo	18.5	6.9
Puente Hills (Coyote Hills)	20.3	6.9
Elsinore	20.5	7.9
Sierra Madre	24.5	7.3
Northridge	26.5	6.9

[1]Measured from 2008 National Seismic Hazard Maps – Source Parameters Website - USGS (USGS, 2008).

As noted above, the closest known active fault is the Newport-Inglewood Fault, located approximately 2.7 miles northeast of the Site. No active faults are known to underlie or project toward the Site. Therefore, the probability of surface fault rupture at the Site from a known active fault is considered low.

5.2 CALIFORNIA BUILDING CODE SEISMIC CRITERIA

A geologic hazard likely to affect the project is ground-shaking as a result of movement along an active fault zone in the vicinity of the Site. The seismic parameters in accordance with the 2019 California Building Code (CBC) are presented below:

Table 3. 2019 CBC Seismic Parameters and Peak Ground Acceleration

Parameter	Value
Site Coordinates	Latitude : 33.925021° Longitude : -118.396152°

Table 3. 2019 CBC Seismic Parameters and Peak Ground Acceleration

Parameter	Value
Mapped Spectral Acceleration Value at Short Period: $\mathbf{S_s}$	1.848g
Mapped Spectral Acceleration Value at 1-Second Period: $\mathbf{S_1}$	0.65g
Seismic Site Classification	D
Short Period Site Coefficient: $\mathbf{F_a}$	1.0
1-Second Period Site Coefficient: $\mathbf{F_v}$	2.5
Site Class Adjusted Acceleration Value at Short Period: $\mathbf{S_{MS}}$	1.848g
Site Class Adjusted Acceleration Value at 1-Second Period: $\mathbf{S_{M1}}$	1.625g
Design Spectral Response Acceleration at Short Periods: $\mathbf{S_{DS}}$	1.232g
Design Spectral Response Acceleration at 1-Second Period: $\mathbf{S_{D1}}$	1.089g
Peak Ground Acceleration adjusted for Site Class Effects: $\mathbf{PGA_M}$	0.875g

ASCE 7-16 – Report generated through ASCE 7 Hazards Report website (ASCE, 2020) – accessed June 22, 2020.

A site-specific ground motion hazard analysis is recommended as part of the Final Geotechnical Investigation.

5.3 LIQUEFACTION AND DYNAMIC SETTLEMENT

Liquefaction is the transformation of a deposit of soil from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress. Often, this transformation results from the cyclic loading of an earthquake and the soil acquires “mobility” sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, saturated (below groundwater), and uniformly graded sands. The vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Cohesive soils with a plasticity index (PI) greater than 7 are generally not considered susceptible to soil liquefaction, although they can be subject to cyclic softening if they are soft enough, and if the seismic demand is relatively high.

The Site is not located in a California Geological Survey Liquefaction Hazard Zone. This zone is defined as areas where historical occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation would be required.

Given the depth to groundwater, the site is not subject to liquefaction induced settlement. However, the dry sand settlement potential was evaluated with the LiqSVs v1.3.3.1 computer program (Geologismiki, 2018) using the SPT data from soil borings P1 through P3. Liquefaction triggering methods developed by Idriss and Boulanger (2014) were used in our liquefaction evaluation. Our evaluation was based on the site class adjusted peak ground acceleration of 0.88g, as presented in Table 2, and an earthquake magnitude of 6.7, the modal earthquake magnitude from the 2014 USGS deaggregation website. The in-situ groundwater level of 121 feet bgs was used to evaluate the cyclic resistance ratio of the on-site soil, and the historical high groundwater depth of approximately 121 feet was used to evaluate the cyclic stress ratio for the design earthquake.

Loose to very dense granular soil is generally present from the ground surface to a depth of at least 26 feet at the site. Based on the denseness, most of this granular soil is generally not susceptible to seismically induced settlement. However, some of the unsaturated, loose to medium dense sand in the upper 50 feet may densify as a result of earthquake shaking, causing ground surface settlement. Ground surface total settlements due to compression in the unsaturated zone are estimated to be on the order of 1.72-inches. Differential settlement over a span of approximately 30 feet is estimated to be approximately half of the total settlement. The results of the liquefaction analyses using the LiqSVs software are provided in Appendix C - Liquefaction Analyses.

If the fundamental period (T) for the structure is greater than 0.5 seconds as determined by the project structural engineer, a site-specific ground motion hazard analysis or site response analysis may be required to determine ground motions for the proposed structures.

5.4 LIQUEFACTION-INDUCED LATERAL SPREADING

Liquefaction induced lateral spreading can occur in areas of sloping ground, or towards a free face. Given the relatively flat topography, distance to a free face, and near surface soil conditions above the water table, the potential for liquefaction-induced lateral spreading is considered low.

5.5 FLOODING, TSUNAMIS AND SEICHES

The Site is located within a FEMA flood Zone X, which is an area of minimal flood hazard (FEMA, 2008). Therefore, damage due to flooding is considered low.

The Site is not located within a Tsunami Inundation Area; therefore, damage due to tsunamis is considered low.

5.6 EXPANSIVE SOILS

The near-surface soils (upper approximate 10 feet) have a low expansion potential. Our soil classifications and laboratory test results show that the near surface (upper 10 feet) samples tested are granular with low-plasticity fines. Accordingly, mitigation for expansive soils is not considered necessary for onsite soils at this site.

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 approved by the Geotechnical Engineer before being imported.

6. CONCLUSIONS

Based on our field exploration, laboratory testing and engineering and geologic analyses, it is our opinion that the Site is suitable for construction of the proposed commercial building improvements from a geotechnical engineering and engineering geology viewpoint; however, there are existing geotechnical conditions associated with the Site that warrant consideration during the planning stages. The main geotechnical conclusions for the project are presented in the following paragraphs.

- No active faults are known to underlie or project toward the Site. Therefore, the probability of surface fault rupture occurring at the Site from a known active fault is considered low.

- Old Eolian deposits were encountered below the site and may extend to depths greater than 51.5 feet bgs. The Old Eolian deposits encountered at this location primarily consist of sand with variable amounts of silt and clay (SP, SW, SP-SM, SW-SC, and SM USCS soil type) and clay with variable amounts of sand (CL USCS soil type) to the maximum depth of exploration. The sandy deposits encountered were loose to very dense and generally dry to moist. The low plasticity clays were stiff and generally moist.

- Groundwater was not encountered during this investigation. Based on groundwater data collected at an offsite location approximately 250 feet north of the site, groundwater is expected to be encountered at a depth of approximately 121 feet below the ground surface (bgs) (CRA, 2010). Groundwater levels may fluctuate in the future due to rainfall, irrigation, broken pipes, or changes in site drainage.

- Loose to very dense granular soil is generally present from the ground surface to a depth of at least 26 feet at the site. Based on the denseness, most of this granular soil is generally not susceptible to seismically induced settlement. However, some of the unsaturated, loose to medium dense sand in the upper 50 feet may densify as a result of earthquake shaking, causing ground surface settlement. Ground surface total settlements due to compression in the unsaturated zone are estimated to be on the order of 1.72-inches. Differential settlement over a span of approximately 30 feet is estimated to be approximately half of the total settlement. The results of the liquefaction analyses using the LiqSVs software are provided in Appendix C - Liquefaction Analyses.

7. RECOMMENDATIONS

7.1 EARTHWORK

The following recommendations are provided regarding specific aspects of the proposed earthwork construction. These preliminary recommendations should be considered subject to revision based on Site layout, foundation loads, and additional geotechnical evaluation of the conditions observed by the Geotechnical Engineer during grading operations.

7.1.1 Site Preparation

Site preparation should begin with the removal of existing buried slabs and foundations, vegetation, highly organic soil, leach lines, septic tanks, and any other unsuitable materials, as applicable. Existing underground utilities within the proposed construction areas should be completely removed and/or rerouted. Grading should conform to the guidelines presented in the 2019 California Building Code (CBC, 2019), as well as the pertinent requirements of the City of El Segundo and Los Angeles County.

7.1.2 Remedial Grading

Building Foundations:

To provide uniform support for the proposed retaining wall, removal of the existing soils to a minimum depth of 2 feet below the bottom of the footings is recommended. The removed soils may be placed back in the excavation as compacted fill, in accordance with the recommendations of Section 7.1.3. Removal, replacement, and compaction should be completed laterally at least five feet beyond the outside edge of the footings unless constrained by existing structures.

The continuous footing along subexcavations that do not extend at least five feet beyond the outside edge of the footing should be designed as a grade beam or with sufficient rebar reinforcement to reduce the potential for differential settlement.

Retaining Wall Foundations:

To provide uniform support for the proposed retaining wall, removal of the existing soils to a minimum depth of 2 feet below the bottom of the footings is recommended. The removed soils may be placed back in the excavation as compacted fill, in accordance with the recommendations of Section 7.1.3. Removal, replacement, and compaction should be completed laterally at least five feet beyond the outside edge of the footings unless constrained by existing structures.

The continuous footing along subexcavations that do not extend at least five feet beyond the outside edge of the footing should be designed as a grade beam or with sufficient rebar reinforcement to reduce the potential for differential settlement.

Concrete Pavement and Hardscape:

Remedial grading for pavement and hardscape areas should include removal of the existing soils to a depth of at least 18 inches below the existing ground surface or subgrade elevation, whichever is deeper. Subgrade elevation is defined as the top of soil elevation provided in the grading plan. The soil exposed at the base of the excavation should be scarified to a depth of 8 inches, and moisture conditioned to within 2 percentage points of the optimum moisture content. Hardscape subgrade should be compacted to at least 90% relative compaction. Pavement subgrade should be compacted to at least 90% relative compaction.

Field Observations:

The Geotechnical Engineer should check the bottom of excavations. If soft, loose, or otherwise unsuitable soils are encountered, the depth of removal may need to be extended.

7.1.3 Fill Placement and Compaction

Excavated materials determined by the Geotechnical Engineer to be satisfactory can be reused as compacted fill. We anticipate that the majority of the excavated materials can be re-used as compacted fill soils. The Geotechnical Engineer should approve the fill material before placement.

Where large compaction equipment is used, such as sheep's foot or smooth drum compactors, fill should be placed in 6- to 8-inch thick loose, horizontal lifts, moisture conditioned to within 2 percentage points of the optimum moisture content and compacted to at least 90% relative compaction. Thinner lifts will be required for smaller compaction equipment. The maximum dry density and optimum moisture content for the evaluation of relative compaction should be determined in accordance with ASTM D1557.

7.1.4 Yielding Subgrade Conditions

The soil encountered at the bottom of the remedial grading excavations can exhibit "pumping" or yielding if they become saturated in response to periods of significant precipitation, such as during the winter rainy season. If this occurs, corrective measures should be performed with oversight from the Geotechnical Engineer.

In order to help stabilize the yielding subgrade soils within the bottom of the removal areas, the contractor can consider the placement of stabilization fabric or geo-grid over the yielding areas, depending on the relative severity of the yielding.

Mirafi 600X (or approved equivalent) stabilization fabric may be used for areas with low to moderate yielding conditions. Geo-grid such as Tensar TX-5 may be used for areas with moderate to severe yielding conditions. Uniform sized, ¾- to 2-inch crushed rock should be placed over the stabilization fabric or geo-grid. A 6- to 12-inch thick section of crushed rock will typically be necessary to stabilize yielding ground.

If significant voids are present in the crushed gravel, a filter fabric should be placed over the crushed gravel to prevent migration of fines into the gravel and thus potential settlement of the overlying fill. Fill soils, which should be placed and compacted in accordance with the recommendations presented herein, should then be placed over the fabric or geo-grid until design grades are reached. The crushed gravel and stabilization fabric or geo-grid should extend at least 5 feet laterally beyond the limits of the yielding areas.

7.1.5 Dewatering

Groundwater was not encountered during our investigation to a maximum depth of 51.5 feet bgs. Accordingly, we do not anticipate that groundwater will be a significant consideration for this project.

7.1.6 Expansive Soil

The near-surface soils (approximately upper 10 feet) have a low expansion potential. Our soil classifications and laboratory test results show that the near surface (upper 10 feet) samples tested are granular with low-plasticity fines. Accordingly, mitigation for expansive soils is not considered necessary at this site. The grading and foundation recommendations presented in this report reflect a low expansion potential

7.1.7 Imported Material

Imported materials, if used for fill, should be predominately granular, contain no rocks or lumps greater than 3 inches in maximum dimension, and have an Expansion Index less than 20, and a Plasticity Index less than 15. Imported materials should be reviewed and approved by the Geotechnical Engineer before being brought to the Site.

7.1.8 Site Excavation Characteristics

During the recent geotechnical investigation, the soil boreholes were drilled using a truck-mounted, hollow stem auger drill rig. As the drilling was completed with moderate effort, conventional earth moving equipment should be capable of performing the excavations required for site development.

7.1.9 Oversized Material

Excavations may generate oversized material. Oversized material is defined as rocks or cemented clasts greater than 3 inches in largest dimension. Oversized material should be broken down to no greater than 3 inches in largest dimension for use in fill or be removed from the Site.

7.1.10 Temporary Excavations

The existing native soils can be considered Type C for excavation in accordance with OSHA and Cal-OSHA requirements. Temporary excavations should be shored or excavated with a slope not steeper than 1:1 (horizontal to vertical) in accordance with OSHA and Cal-OSHA requirements.

The excavations should be inspected daily by the contractor's Competent Person before personnel are allowed to enter the excavation. Any zones of potential instability, sloughing or raveling should be brought to the attention of the Geotechnical Engineer and corrective action implemented before personnel begin working in the excavation. Excavated soils should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation.

The project geotechnical engineer should be notified if other surcharge loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during the rainy season, berms are recommended near the tops of slopes to prevent runoff water from entering the excavation and eroding the slope faces.

7.1.11 Temporary Cantilever Shoring

Temporary excavations to depths up to approximately 10 feet bgs are anticipated for construction of the foundations. Where cantilevered shoring is used in lieu of sloping the temporary excavation sidewalls, the shoring design may be tentatively based upon an active earth pressure equal to a fluid weighing 40 pounds per cubic foot (pcf). The passive pressures above the groundwater level may be based on a fluid weighing 340 pcf.

These pressures are based on level ground conditions in front and behind the wall with no surcharge loads within 10 feet of the excavation. 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.

7.1.12 Braced Shoring System

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

Bracing: 30H psf/ft

where H is the depth of the excavation, in feet.

These pressures are based on level ground conditions in front and behind the wall with no surcharge loads within 10 feet of the excavation. 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.

7.1.13 Pipelines

Typical pipe bedding as specified in the Standard Specifications for Public Works Construction (GREENBOOK) may be used. As a minimum, it is recommended that pipe be supported on at least 4 inches of granular bedding material, such as 3/4-inch rock or clean coarse sand with less than 5 percent fines and a sand equivalent of 40 or more as evaluated by ASTM D2419.

The bedding should extend from the bottom of the trench to at least 1 foot above the top of the pipe. Sand bedding should be mechanically compacted to at least 90 percent relative compaction. Jetting of sand bedding should not be permitted.

Onsite material, imported select material, or 2-sack cement/sand slurry may be used as backfill in trenches above the pipe bedding. The material selected should match the engineering characteristics of the soils adjacent to the trench. Utility trench backfill beneath structures and hardscape should be compacted to at least 90% relative compaction.

The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed along the sides of buried flexible pipelines. For the purpose of evaluating deflection due to the load associated with trench backfill over the pipe, a value of 1,500 pounds per square inch (lbs/in^2) is recommended for the general site conditions assuming granular bedding material (sand or gravel) is placed around the pipe.

7.1.14 Surface Drainage

Final surface grades around structures should be designed to collect and direct surface water away from the structure and toward appropriate drainage facilities. The ground around the structure should be graded so that surface water flows rapidly away from the structure without ponding. In general, we recommend that the ground adjacent to the structure slope away at a gradient of at least 2%. Densely vegetated areas where runoff can be impaired should have a minimum gradient of at least 5% within the first 5 feet from the structure. Roof gutters with downspouts that discharge directly into a closed drainage system are recommended on structures. Drainage patterns established at the time of fine grading should be maintained throughout the life of the proposed structures. Site irrigation should be limited to the minimum necessary to sustain landscape growth. Should excessive irrigation, impaired drainage, or unusually high rainfall occur, saturated zones of perched groundwater can develop. Saturated soil zones may result in increased maintenance and could impact structure stability.

7.1.15 Grading Plan Review

Stantec should review the grading plans and earthwork specifications to ascertain whether the intent of the recommendations contained in this report have been implemented, and that no revised recommendations are needed due to changes in the development scheme.

7.2 FOUNDATIONS

7.2.1 Shallow Building Foundations

Conventional shallow foundations (spread footings/strip footings) are considered suitable for support of the proposed building provided the recommendations in this report are incorporated into the design. Ground improvement may be required and should be included if the project structural engineer determines that the settlements provided below are outside the tolerable limits for the proposed structure.

The following foundation recommendations are minimum criteria based on geotechnical considerations. They should not be considered a structural design, nor should they be considered to preclude more restrictive criteria by governing agencies or the structural engineer. The design of the foundation system should be performed by the project structural engineer.

Conventional Shallow Foundations:

An allowable bearing pressure of 6,300 pounds per square foot (psf) may be used for conventional square or rectangular shallow foundations founded in properly compacted fill prepared in accordance with the recommendations of this report. The bearing capacity can be increased by one third for transient loading conditions such as earthquake and wind.

Additional parameters for shallow foundations are provided below.

Minimum Footing Width:	24 inches for continuous footings
Maximum Footing Width:	8 feet for square/rectangular footings
Minimum Footing Depth:	7 feet below lowest adjacent soil grade
Minimum Reinforcement:	Two No. 5 bars at both top and bottom in continuous and rectangular footings

7.2.2 Mat Foundations

To accommodate loose to medium dense near surface soil conditions, we recommend the isolated building expansion areas be constructed with a stiffened foundation system in combination with a recompacted fill mat. The stiffened foundation system should consist of a rigid mat, post-tensioned slab, or a waffle foundation system tied together with grade beams.

The following foundation recommendations are minimum criteria based on geotechnical considerations. They should not be considered a structural design, nor should they be considered to preclude more restrictive criteria by governing agencies or the structural engineer. The design of the foundation system should be performed by the project structural engineer.

If a rigid mat foundation is used, the mat slab should extend at least 12 inches below the adjacent ground surface. The mat thickness should be determined by the structural engineer. The mat can be designed assuming an allowable bearing pressure of 1,500 pounds per square foot for dead plus live loads, with a one-third increase for all loads including wind or seismic. This allowable bearing pressure is a net value; therefore, the weight of the mat can be neglected for design purposes. The mat should be integrally connected to all portions of the structure, so the entire foundation system moves as a unit. The mat should be reinforced with top and bottom steel in both directions to allow the foundation to span local irregularities that may result from potential differential settlement. As a minimum, we recommend that the mat be reinforced with sufficient top and bottom steel to span as a simple beam an unsupported distance of at least 10 feet. The mat can be designed using a modulus of subgrade reaction, Kv1, of 200 pounds per cubic inch. The actual modulus of subgrade reaction would need to be adjusted for the plan dimensions of the mat.

If a waffle foundation system is used, an allowable bearing pressure of 1,500 pounds per square foot (psf) may be used for shallow spread footings with grade beams founded in properly compacted fill prepared in accordance with the recommendations of this report. The bearing capacity can be increased by one third for transient loading conditions such as earthquake and wind.

Additional parameters for shallow foundations are provided below.

Minimum Footing Width:	18 inches for continuous footings 24 inches for square/rectangular footings
Minimum Footing Depth:	18 inches below lowest adjacent soil grade
Minimum Reinforcement:	Two No. 5 bars at both top and bottom in continuous footings.

We recommend incorporating flexible utility connections to the building to accommodate the estimated total and differential settlements.

7.2.3 Foundation Settlement

The following static and seismic foundation settlements are estimated.

Static Settlement:	Less than 1-inch total settlement ½ inch differential settlement over 30 feet
Seismic Settlement:	Less than 1.7 inches total settlement 0.85 inches differential settlement over 30 feet

7.2.4 Lateral Resistance

Lateral loads will be resisted by friction between the bottoms of footings and passive pressure on the faces of footings and other structural elements below grade. An allowable coefficient of friction of 0.30 can be used.

Passive pressure can be computed using an equivalent fluid pressure of 340 lbs/ft^3 for level ground conditions. Reductions for sloping ground should be made. The upper 1 foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs. The earth pressure indicated above does not include a safety factor; therefore, the foundation design should include an appropriate safety factor for the overall performance of the system.

7.2.5 Retaining Wall Foundations

For cantilever retaining wall foundations, bearing entirely in compacted fill soils, an average allowable bearing pressure of 1,500 psf (i.e. 0 psf at the heal and 3,000 psf at the toe) may be incorporated in the design. To mitigate against potential detrimental effects of loose near surface soils the following recommendations should be incorporated into the foundation design:

1. Subgrade preparation in accordance with the recommendations in Section 7.1.3.
2. Minimum foundation embedment depth of 12 inches.
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 three feet beyond the back of the wall.
6. Compaction moisture content in clay should be 2 percentage points over optimum and maintained through construction.

If the existing wall is to remain, the contractor should consider methods of soil removal that will not undermine existing footings or result in an unstable situation. The contractor may consider bracing or slot cutting for excavation and compaction near existing 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 10 feet.

Active:	40H psf/ft,
Passive:	340D psf/ft,

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.

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.

For resistance to transient lateral loads, such as earthquake loads, the aforementioned allowable bearing capacity may be increased by one-third.

7.2.6 Foundation Plan Review

Stantec should review the foundation plans to ascertain that the intent of the recommendations in this report has been implemented and that revised recommendations are not necessary as a result of changes after this report was completed.

7.2.7 Foundation Excavation Observations

A representative working under direct supervision of the Geotechnical Engineer should observe the foundation excavations prior to forming or placing reinforcing steel.

7.3 SLABS-ON-GRADE

If a deep foundation system with slab-on-grade is included in the design, the top 24 inches of material below interior concrete slabs-on-grade should have an expansion index of 20 or less. The project structural engineer should design the interior concrete slabs-on-grade floor. However, we recommend a minimum thickness of 5 inches.

A vapor barrier should be placed beneath slabs where moisture sensitive floor coverings will be installed. If plastic is used, a minimum 10-mil is recommended. The plastic should comply with ASTM E1745. Installation should comply with ASTM E1643. Current construction practice typically includes placement of a 2-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 to the underside of the slab that can increase the time required to reduce vapor emissions to limits acceptable for the type of floor covering placed on top of the slab. The slab can be placed directly on the vapor retarder/barrier. 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. The project team should determine the appropriate treatment for the specific application.

In addition to the moisture vapor barrier, a capillary moisture break can be constructed below the slab to further reduce moisture transmission from the subgrade soil, if desired. The capillary moisture break should consist of at least 4-inches of clean, free-draining gravel or crushed rock placed below the moisture vapor retarder/barrier. The components of the capillary moisture break should meet the particle-size gradation presented in Table 5.

Table 4. Gradation for Capillary Moisture Break

Gradation for Capillary Moisture Break	
Sieve Size	**Percentage Passing Sieve**
1 inch	100
3/4 inch	30-75
1/2 inch	5–10
3/8 inch	0-2

7.3.1 Exterior Slabs on Grade (Sidewalks)

Exterior slabs not subject to vehicular traffic should have a minimum thickness of 4 inches and be reinforced with at least No. 3 bars at 18 inches on center each way. Slabs should be provided with crack control joints placed in accordance with the American Concrete Institute (ACI) guidelines. The project architect or civil engineer should select the final joint patterns.

7.4 SOIL CORROSIVITY

Two samples of the onsite soils were tested to provide a preliminary indication of the corrosion potential of the onsite soils. The test results are presented in Appendix B. A brief discussion of the corrosion test results is provided in the following text.

- The samples tested had soluble sulfate concentrations between 44 and 49 parts per million (ppm), which indicates the samples have a low sulfate corrosion potential relative to concrete. It should be noted that soluble sulfate in the irrigation water supply, and/or the use of fertilizer may cause the sulfate content in the surficial soils to increase with time. This may result in a higher sulfate exposure than that indicated by the test results reported herein. Studies have shown that the use of improved cements in the concrete, and a low water-cement ratio will improve the resistance of the concrete to sulfate exposure.

- The samples tested had chloride concentrations between 33 and 36 ppm, which indicates the sample has a low chloride corrosion potential relative to metal.

- The samples tested had a minimum resistivity of 9,399 to 10,101 ohm-cm, which indicates the samples are moderately corrosive.

- The samples tested had a pH ranging from 8.2 to 8.4, which indicates the samples are slightly alkaline.

Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 1,500 ppm, or the pH is 5.5 or less (Caltrans, 2012).

Based on Caltrans criteria, the test results indicate the site is not considered to be a corrosive environment for concrete and steel. Other samples at the Site could yield significantly different concentrations to those described above. Therefore, additional testing may be performed to further evaluate corrosion during the planning or construction stages and to evaluate the as-graded corrosion potential of the onsite soils after site grading. We recommend evaluation by a corrosion engineer should be performed if deemed necessary.

7.5 PAVEMENT

7.5.1 Asphalt Concrete Pavement

An R-value of 25 has been assumed for preliminary design of pavement sections based on the soil classification and gradation of the on-site material in the upper 5 feet. The actual R-value of the subgrade soils should be determined after grading to provide final pavement design. Flexible pavement sections have been calculated in general conformance with Caltrans guidelines. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. Based on an assumed R-value of 25, the following pavement structural sections have been calculated.

Table 5. Flexible Pavement Sections

Traffic Type	Traffic Index	Asphalt Concrete (inches)	Aggregate Base* (inches)
Automobile Parking	5.0	4	5
Automobile Drive Lanes	5.5	4	6
Medium Truck Traffic	6.0	4	7
Heavy Truck Traffic	7.0	5	9

*Aggregate Base should conform to Class 2 Aggregate Base in accordance with the Caltrans Standard Specifications or Crushed Miscellaneous Base in accordance with the Standard Specifications for Public Works Construction.

Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned to slightly above the optimum moisture content, and recompacted to a dry density of at least 95% of the laboratory maximum. The base material should also be compacted to slightly above the optimum moisture content and a dry density of at least 95% of the laboratory maximum. Asphalt concrete should be compacted to at least 95% of the laboratory Hveem density in accordance with ASTM D2726.

Rigid concrete pavement (described below) should be placed in driveway entrance aprons and trash bin loading/storage areas. Concrete pavement design is provided in the following section.

7.5.2 Concrete Pavement

Concrete pavements have been calculated in general conformance with the procedure recommended by the American Concrete Institute (ACI 330R-08) using the parameters presented in Table 6 and assuming a 20-year design life. The following design parameters were used in our analyses.

Table 6. Concrete Pavement Parameters

Design Parameter	Value
Modulus of Subgrade Reaction (k)	50 pci
Modulus of Concrete Rupture (M_R)	550 psi
Concrete Compressive Strength	3,700 psi
Traffic Categories (TC)	A and C
Average Daily Truck Traffic (ADTT)	10 and 100

Based on the parameters above, we recommend the following minimum concrete pavement thickness (Table 7).

Table 7. Recommended Concrete Pavement Sections

Traffic Type	Pavement Thickness (inches)	Aggregate Base (inches)
Automobile Parking and Driveways (TC = A)	6	6
Heavy Truck Traffic and Fire Lane Areas (TC = C)	8	6

The project civil engineer should confirm whether the assumed ADTT is appropriate for the anticipated traffic level. Concrete compressive strength for pavement should be at least 3,700 psi. Minimum reinforcement should consist of #3 bars on 24-inch centers. Crack control joints should be placed in accordance with the American Concrete Institute (ACI) guidelines.

Prior to placing concrete, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned to slightly above the optimum moisture content, and recompacted to a dry density of at least 90% of the laboratory maximum.

7.6 PERCOLATION TESTING

Percolation testing was performed in four borings (P1 through P3) in general accordance with the guidelines described in Los Angeles County Administrative Manual (LACAM) (LA,2014).

Based on the laboratory sieve test results, the natural soils located at the bottom of the percolation well soil borings consist of sand (USCS: SP, SP-SM, SM) with variable amounts of silt. The sands were brown, dry to moist, and dense to very dense.

The percolation tests were performed in eight-inch diameter, 50-feet deep borings. Pre-soaking was performed the day of percolation testing. The stabilized percolation rate from the final tests was measured as 186 to 522 inches per hour or less than 0.3 minutes per inch which corresponds to a high percolation rate (un-factored).

Los Angeles County requirements for infiltration include converting the percolation rate to an infiltration rate (I_t) and applying a safety factor. Once the infiltration rate was calculated and a factor of safety of 4 is applied, the design infiltration rate is 12 inches per hour. Note that this infiltration rate is applicable at the locations tested. Different locations and depths may have different rates and soil compaction will reduce the infiltration rate. An appropriate factor of safety, if necessary, should be applied to the overall system design in accordance with the LA County Administrative Manual.

Soil percolation rates from in situ tests can vary significantly from one location to another due to heterogeneous characteristics of subsurface conditions. The test results from these borings should be considered a screening level value and additional testing should be performed if an on-site disposal system is to be constructed for the project. Soil compaction can decrease infiltration rates significantly. Final percolation testing should be performed in as graded conditions so that effects from soil compaction are incorporated in the test results.

7.7 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 Geotechnical Engineer to perform such services. Final project grading and foundation plans, foundation details and specifications should be reviewed by Stantec prior to construction to ascertain that the intent of the recommendations presented herein have been applied to the design. Following review of plans and specifications, observation during construction should be performed to correlate the findings of this exploration with the actual subsurface conditions exposed.

8. CLOSURE

Our conclusions, recommendations, and discussions presented herein are based upon an evaluation and interpretation of the findings from the field and laboratory programs, with interpolation and extrapolation of subsurface conditions between and beyond the exploration locations. This report contains information that is valid as of the report's date and to the extent directly known to Stantec. However, conditions can change with the passage of time or construction subsequent to this report's preparation that may invalidate, either partially or wholly, the conclusions and recommendations presented herein.

Inherent in most projects performed in the heterogeneous subsurface environment, continuing subsurface explorations and analyses may reveal conditions that are different than those described in this report. 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 engineering geologists practicing in this locality. No other warranty, express or implied, is made.

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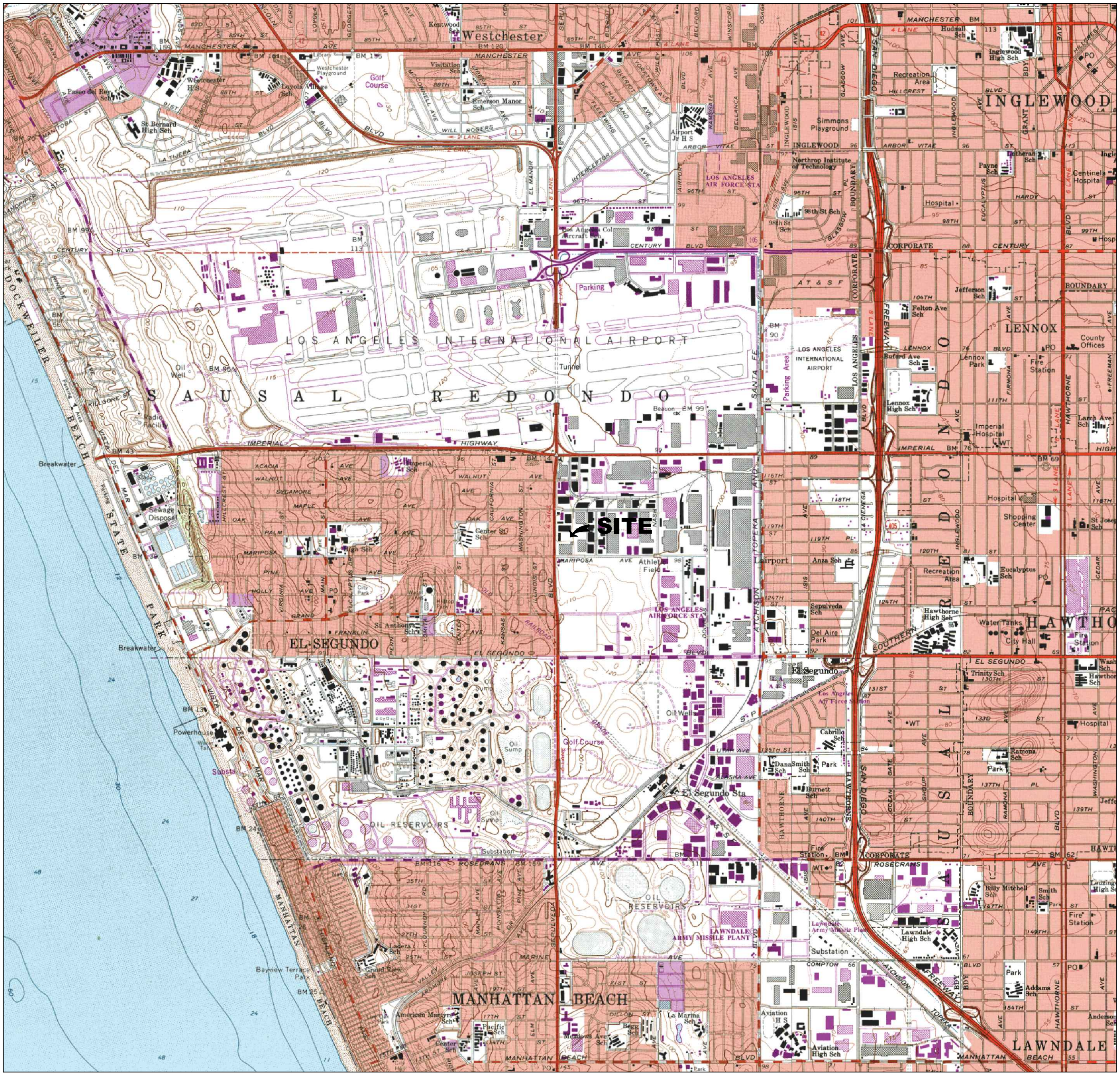
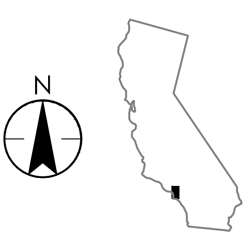
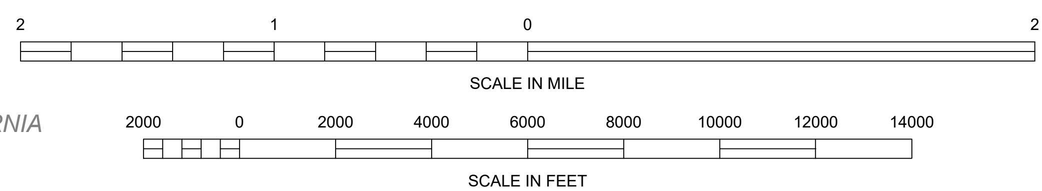
FIGURES







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Stantec 735 EAST CARNEGIE DRIVE, SUITE 280 SAN BERNARDINO, CA 92408 PHONE: (909) 335-6116 FAX: (909) 335-6120	FOR: L&R ZAV 650 SEPULVEDA LLC 650 - 700 NORTH PACIFIC COAST HIGHWAY EL SEGUNDO, CALIFORNIA 90245		SITE LOCATION MAP		FIGURE: 1
	JOB NUMBER: 185804814	DRAWN BY: JEF	CHECKED BY: JEF	APPROVED BY: JEF	DATE: 6/15/20

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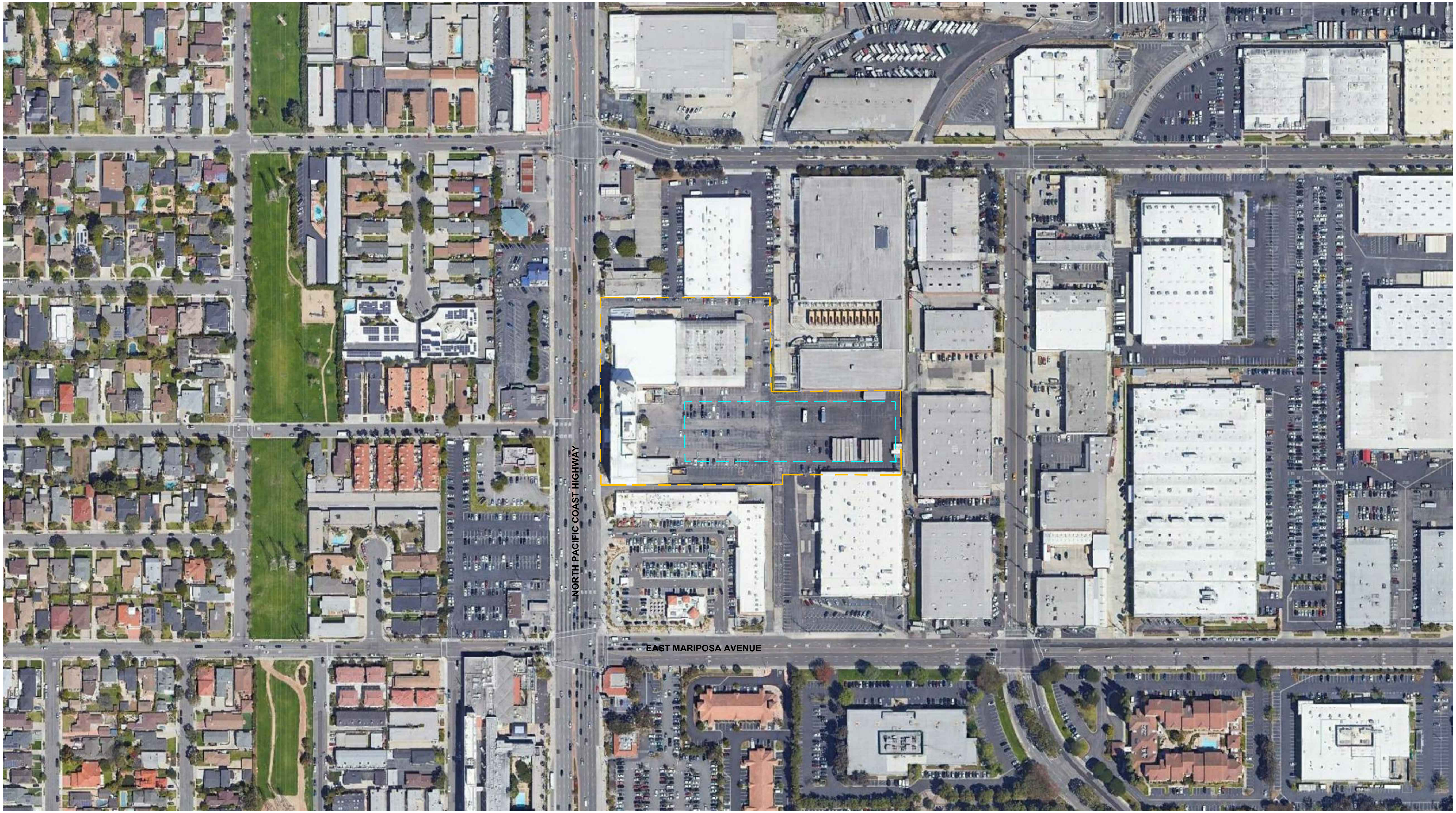

LEGEND

APPROXIMATE LIMITS OF PROPERTY

APPROXIMATE LIMITS OF PROPOSED BUILDING

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SCALE (FEET)

Stantec

735 EAST CARNEGIE DRIVE, SUITE 280
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PHONE:(909) 335-6116 FAX:(909) 335-6120

FOR:

L&R ZAV 650 SEPULVEDA LLC
650 - 700 NORTH PACIFIC COAST HIGHWAY
EL SEGUNDO, CALIFORNIA 90245

SITE VICINITY MAP

FIGURE: 2

JOB NUMBER:	DRAWN BY:	CHECKED BY:	APPROVED BY:	DATE:
185804814	JEF	JEF	JEF	6/15/20

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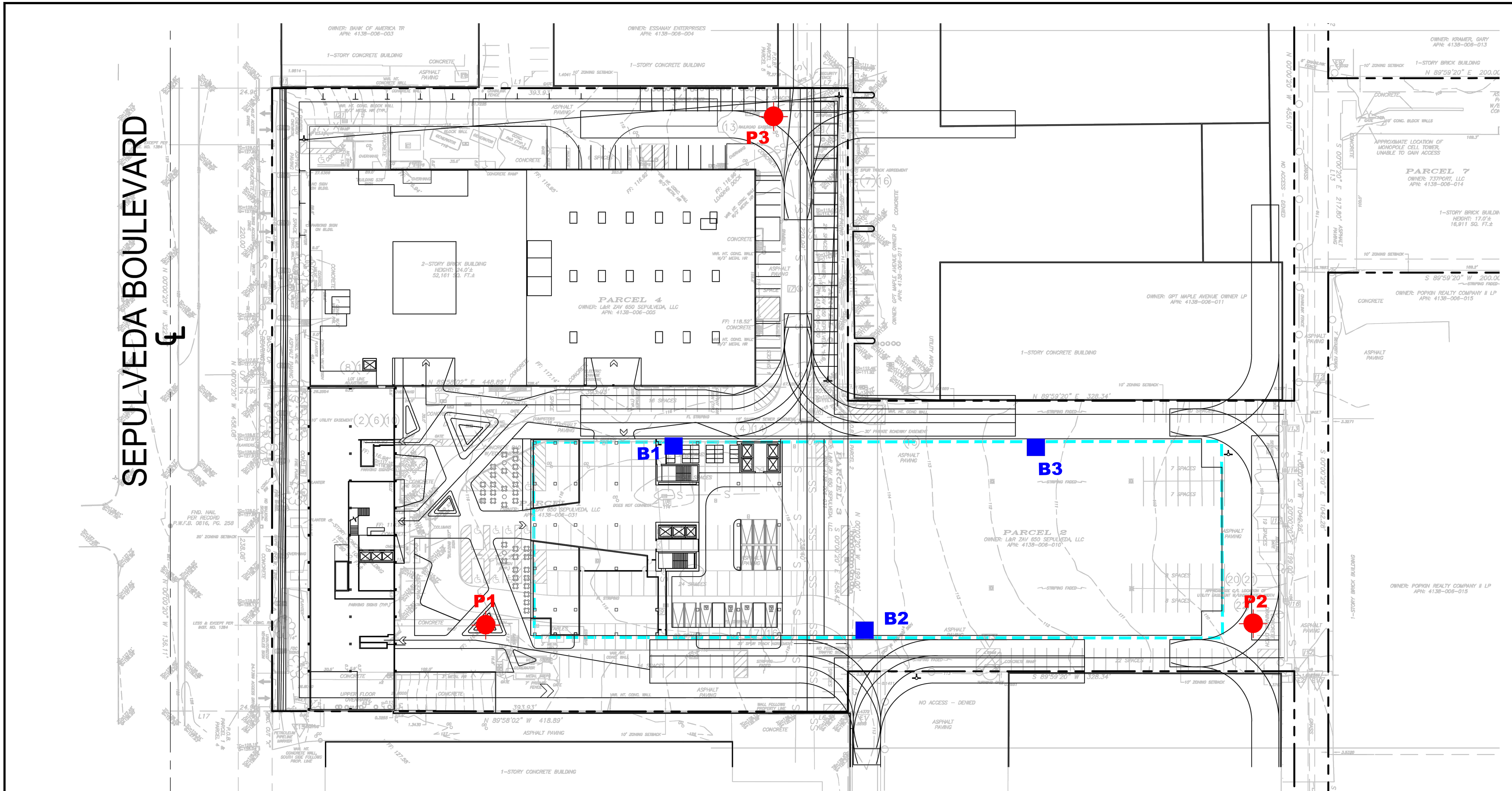

LEGEND

B3 ■ APPROXIMATE LOCATION OF SOIL BORING

P3 ● APPROXIMATE LOCATION OF PERCOLATION TEST

APPROXIMATE LOCATION OF PROPOSED BUILDING

APPROXIMATE LOCATION OF EXISTING IMPROVEMENTS

0 70 140

SCALE (FEET)

Stantec
735 EAST CARNEGIE DRIVE, SUITE 280
SAN BERNARDINO, CA 92408
PHONE:(909) 335-6116 FAX:(909) 335-6120

FOR:		SUBSURFACE LOCATION MAP		FIGURE:
L&R ZAV 650 SEPULVEDA LLC 650 - 700 NORTH PACIFIC COAST HIGHWAY EL SEGUNDO, CALIFORNIA 90245				3
JOB NUMBER: 185804814	DRAWN BY: JEF	CHECKED BY: JEF	APPROVED BY: JEF	DATE: 6/15/20

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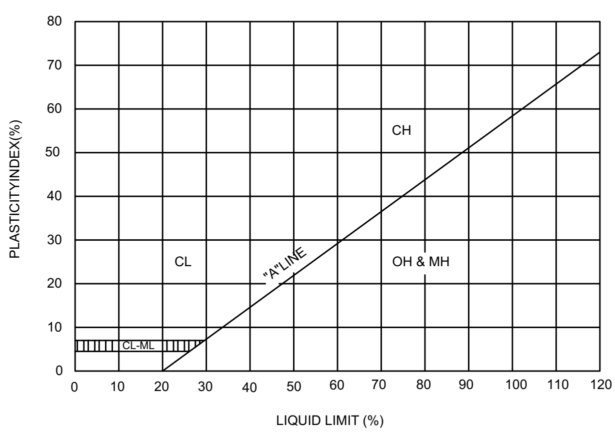
APPENDIX A
BORING LOGS

UNIFIED SOIL CLASSIFICATION (ASTM D-2487)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	*CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		*GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	*CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		*SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

* Dual symbols required for fines content between 5% and 12%

PLASTICITY CHART



SAMPLER TYPES

SPT

Modified California (2.5" I.D.)

Rock Core

Shelby Tube

No Recovery

Grab Sample

ADDITIONAL TESTS

COR - CHEMICAL ANALYSIS (CORROSIVITY)
CD - CONSOLIDATED DRAINED TRIAXIAL
CN - CONSOLIDATION
CU - CONSOLIDATED UNDRAINED TRIAXIAL
DS - DIRECT SHEAR
PP - POCKET PENETROMETER (TSF)
#200 - Percent Passing #200 SIEVE
RV - R-VALUE
SA - SIEVE ANALYSIS: % PASSING
- WATER LEVEL

PI - PLASTICITY INDEX
EI - EXPANSION INDEX
TC - CYCLIC TRIAXIAL
TV - TORVANE SHEAR
UC - UNCONFINED COMPRESSION
(1.5) - (WITH SHEAR STRENGTH IN KSF)
UU - UNCONSOLIDATED UNDRAINED TRIAXIAL

PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5-1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

* * UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

LEGEND TO BORING LOGS AND SOIL DESCRIPTIONS

Stantec

PROJECT: **650-700 PCH**	WELL / TEST PIT / BOREHOLE NO: **B1** PAGE 1 OF 2
LOCATION: **El Segundo, CA**	NORTHING (ft):
PROJECT NUMBER: **185804814**	EASTING (ft):
DRILLING: STARTED **5/28/20** COMPLETED: **5/28/20**	LATITUDE: **33° 55' 30.72"**
INSTALLATION: STARTED **5/28/20** COMPLETED: **5/28/20**	LONGITUDE: **118° 23' 42"**
DRILLING COMPANY: **ABC Liovin Drilling**	GROUND ELEV (ft): **115.0**
DRILLING EQUIPMENT: **CME 85**	TOC ELEV (ft):
DRILLING METHOD: **HSA**	INITIAL DTW (ft): **NE**
SAMPLING EQUIPMENT: **Split Spoon**	BOREHOLE DEPTH (ft): **31.5**
	STATIC DTW (ft): **NE**
	WELL DEPTH (ft): **---**
	WELL CASING DIAMETER (in): **---**
	BOREHOLE DIAMETER (in): **8**
	LOGGED BY: **R. Woodford**
	CHECKED BY: **J. Fischer**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)
			3" ASPHALT (AC)						
			3" AGGREGATE BASE (AB)						
			OLD EOLIAN DEPOSITS (Qoe)						
		SM	**SILTY SAND** ; SM; 7.5YR 4/4 brownish brown; 74% fine to medium grained sand; 26% fines; medium dense, moist.		B1-2'	SA			
5			7.5YR 5/8 dark brownish brown		B1-5'	DS	7 12 17		5
10			Dense below 10 feet.		B1-10'		6 11 20		10
15			Very dense below 15 feet.		B1-15'		18 27 29		15
20			Medium dense below 20 feet.		B1-20'		12 16 31		20

PROJECT: **650-700 PCH**	WELL / TEST PIT / BOREHOLE NO: **B1** PAGE 2 OF 2
LOCATION: **El Segundo, CA**	NORTHING (ft):
PROJECT NUMBER: **185804814**	EASTING (ft):
DRILLING: STARTED **5/28/20** COMPLETED: **5/28/20**	LATITUDE: **33° 55' 30.72"**
INSTALLATION: STARTED **5/28/20** COMPLETED: **5/28/20**	LONGITUDE: **118° 23' 42"**
DRILLING COMPANY: **ABC Liovin Drilling**	GROUND ELEV (ft): **115.0**
DRILLING EQUIPMENT: **CME 85**	TOC ELEV (ft):
DRILLING METHOD: **HSA**	INITIAL DTW (ft): **NE**
SAMPLING EQUIPMENT: **Split Spoon**	BOREHOLE DEPTH (ft): **31.5**
	STATIC DTW (ft): **NE**
	WELL DEPTH (ft): **---**
	WELL CASING DIAMETER (in): **---**
	BOREHOLE DIAMETER (in): **8**
	LOGGED BY: **R. Woodford**
	CHECKED BY: **J. Fischer**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)
25			Dense below 25 feet.		B1-25'		8 13 19		25
30					B1-30'		8 11 19		30
			Hole terminated at 31.5 feet.						
35									35
40									40

PROJECT: **650-700 PCH**
LOCATION: **El Segundo, CA**
PROJECT NUMBER: **185804814**
DRILLING: STARTED **5/28/20** COMPLETED: **5/28/20**
INSTALLATION: STARTED **5/28/20** COMPLETED: **5/28/20**
DRILLING COMPANY: **ABC Liovin Drilling**
DRILLING EQUIPMENT: **CME 85**
DRILLING METHOD: **HSA**
SAMPLING EQUIPMENT: **Split Spoon**

WELL / TEST PIT / BOREHOLE NO:
B2 PAGE 1 OF 2

NORTHING (ft):
EASTING (ft):
LATITUDE: **33° 55' 29.28"**
LONGITUDE: **118° 23' 40.2"**
GROUND ELEV (ft): **113.0**
TOC ELEV (ft):
INITIAL DTW (ft): **NE**
BOREHOLE DEPTH (ft): **31.5**
STATIC DTW (ft): **NE**
WELL DEPTH (ft): **---**
WELL CASING DIAMETER (in): **---**
BOREHOLE DIAMETER (in): **8**
LOGGED BY: **R. Woodford**
CHECKED BY: **J. Fischer**

Time & Depth (feet)	USCS	Description	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)
		1" ASPHALT					
		1" AB					
		OLD EOLIAN DEPOSITS (Qoe)					
	SM	**SAND WITH SILT** ; SM; 7.5YR 3/4 dark brown; fine-grained; 76% fine grained sand; 24% fines; medium dense, moist.	B2-2'	SA			
5			B2-5'		3 1 1		5
10		7.5YR 5/6 dark brown; medium dense below 10 feet.	B2-10'	DS	14 15 31		10
15	CL	**LEAN CLAY WITH FINE SAND** ; CL; 10YR 5/4 yellowish brown; fine-grained; 95% fines; 5% fine grained sand; very stiff, moist, mottled oxidation staining, trace root holes.	B2-15'		4 7 12		15
20	SW	**WELL GRADED SAND** ; SW; 7.5YR 4/6 dark brown; fine to medium-grained; 95% fine to medium grained sand; 5% fines; medium dense, moist	B2-20'		3 10 16		20

PROJECT: **650-700 PCH**
LOCATION: **El Segundo, CA**
PROJECT NUMBER: **185804814**
DRILLING: STARTED **5/28/20** COMPLETED: **5/28/20**
INSTALLATION: STARTED **5/28/20** COMPLETED: **5/28/20**
DRILLING COMPANY: **ABC Liovin Drilling**
DRILLING EQUIPMENT: **CME 85**
DRILLING METHOD: **HSA**
SAMPLING EQUIPMENT: **Split Spoon**

WELL / TEST PIT / BOREHOLE NO:

B2

Stantec

NORTHING (ft):
EASTING (ft):
LATITUDE: **33° 55' 29.28"**
LONGITUDE: **118° 23' 40.2"**
GROUND ELEV (ft): **113.0**
TOC ELEV (ft):
INITIAL DTW (ft): **NE**
BOREHOLE DEPTH (ft): **31.5**
STATIC DTW (ft): **NE**
WELL DEPTH (ft): **---**
WELL CASING DIAMETER (in): **---**
BOREHOLE DIAMETER (in): **8**
LOGGED BY: **R. Woodford**
CHECKED BY: **J. Fischer**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)
25					B2-25'		8 8 14		25
30					B2-30'		8 13 16		30
			Hole terminated at 31.5 feet.						
35									35
40									40

PROJECT: **650-700 PCH**		WELL / TEST PIT / BOREHOLE NO: **B3** PAGE 1 OF 1
LOCATION: **El Segundo, CA**		
PROJECT NUMBER: **185804814**		
DRILLING: STARTED **5/28/20** COMPLETED: **5/28/20**	NORTHING (ft):	EASTING (ft):
INSTALLATION: STARTED **5/28/20** COMPLETED: **5/28/20**	LATITUDE: **33° 55' 30.72"**	LONGITUDE: **118° 23' 39.12"**
DRILLING COMPANY: **ABC Liovin Drilling**	GROUND ELEV (ft): **111.0**	TOC ELEV (ft):
DRILLING EQUIPMENT: **CME 85**	INITIAL DTW (ft): **NE**	BOREHOLE DEPTH (ft): **21.5**
DRILLING METHOD: **HSA**	STATIC DTW (ft): **NE**	WELL DEPTH (ft): **---**
	WELL CASING DIAMETER (in): **---**	BOREHOLE DIAMETER (in): **8**
SAMPLING EQUIPMENT: **Split Spoon**	LOGGED BY: **R. Woodford**	CHECKED BY: **J. Fischer**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)
			2" ASPHALT						
			1" AB						
			OLD EOLIAN DEPOSITS (Qoe)		B3-Bulk	CORR			
		SM	**SILTY SAND** ; SM; 7.5YR 3/3 dark brown; fine-grained; 73% fine grained sand; 26% fines; dense, moist.		B3-2'	SA			
5					B3-5'	DS	7 6 5		5
10			10YR 5/3 brown; medium dense below 10 feet.		B3-10'		3 9 16		10
15			59% fine grained sand; 41% fines below 15 feet.		B3-15'	SA	3 11 15		15
20			86% fine grained sand; 14% fines below 20 feet.		B3-20'	SA	9 14 14		20
			Hole terminated at 21.5 feet.						

PROJECT: **650-700 PCH**	WELL / TEST PIT / BOREHOLE NO: **P1**	PAGE 1 OF 3
LOCATION: **El Segundo, CA**	NORTHING (ft):	EASTING (ft):
PROJECT NUMBER: **185804814**	LATITUDE: **33° 55' 29.28"**	LONGITUDE: **118° 23' 44.16"**
DRILLING: STARTED **5/28/20** COMPLETED: **5/28/20**	GROUND ELEV (ft): **120.0**	TOC ELEV (ft):
INSTALLATION: STARTED **5/28/20** COMPLETED: **5/28/20**	INITIAL DTW (ft): **NE**	BOREHOLE DEPTH (ft): **51.5**
DRILLING COMPANY: **ABC Liovin Drilling**	STATIC DTW (ft): **NE**	WELL DEPTH (ft): **---**
DRILLING EQUIPMENT: **CME 85**	WELL CASING DIAMETER (in): **---**	BOREHOLE DIAMETER (in): **8**
DRILLING METHOD: **HSA**	LOGGED BY: **R. Woodford**	CHECKED BY: **J. Fischer**
SAMPLING EQUIPMENT: **Split Spoon**		

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Borehole Backfill
			3" ASPHALT							
			3" AB							
			OLD EOLIAN DEPOSITS (Qoe)		P1-Bulk	CORR				Backfilled with bentonite.
		SM	**SILTY SAND** ; SM; brown (7.5 YR 4/4) to strong brown (7.5YR 5/8), 90% fine to coarse grained sand; 10% fines; medium dense, moist.		P1-2'					
5			Dense below 5 feet.		P1-5'		10 18 25		5	
10			Very dense below 10 feet		P1-10'	DS	16 32 50-5"		10	
15			Dense below 15 feet.		P1-15'		9 18 25		15	
20			Medium dense below 20 feet.		P1-20'		7 12 19		20	Backfilled with bentonite

GEO FORM 304 650_700_PCH.GPJ SECOR INTL.GDT 6/28/20

PROJECT: **650-700 PCH**		WELL / TEST PIT / BOREHOLE NO: **P1** PAGE 2 OF 3	
LOCATION: **El Segundo, CA**		NORTHING (ft):	EASTING (ft):
PROJECT NUMBER: **185804814**		LATITUDE: **33° 55' 29.28"**	LONGITUDE: **118° 23' 44.16"**
DRILLING: STARTED **5/28/20**	COMPLETED: **5/28/20**	GROUND ELEV (ft): **120.0**	TOC ELEV (ft):
INSTALLATION: STARTED **5/28/20**	COMPLETED: **5/28/20**	INITIAL DTW (ft): **NE**	BOREHOLE DEPTH (ft): **51.5**
DRILLING COMPANY: **ABC Liovin Drilling**		STATIC DTW (ft): **NE**	WELL DEPTH (ft): **---**
DRILLING EQUIPMENT: **CME 85**		WELL CASING DIAMETER (in): **---**	BOREHOLE DIAMETER (in): **8**
DRILLING METHOD: **HSA**		LOGGED BY: **R. Woodford**	CHECKED BY: **J. Fischer**
SAMPLING EQUIPMENT: **Split Spoon**			

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Borehole Backfill
25			Dense below 25 feet.		P1-25'		10 25 22		25	
30			Very dense below 30 feet.		P1-30'		10 30 50-5"		30	
35					P1-35'		23 39 46		35	
40					P1-40'		20 31 50-6"		40	

PROJECT: **650-700 PCH**
LOCATION: **El Segundo, CA**
PROJECT NUMBER: **185804814**
DRILLING: STARTED **5/28/20** COMPLETED: **5/28/20**
INSTALLATION: STARTED **5/28/20** COMPLETED: **5/28/20**
DRILLING COMPANY: **ABC Liovin Drilling**
DRILLING EQUIPMENT: **CME 85**
DRILLING METHOD: **HSA**
SAMPLING EQUIPMENT: **Split Spoon**

WELL / TEST PIT / BOREHOLE NO:
P1

Stantec

NORTHING (ft):
EASTING (ft):
LATITUDE: **33° 55' 29.28"**
LONGITUDE: **118° 23' 44.16"**
GROUND ELEV (ft): **120.0**
TOC ELEV (ft):
INITIAL DTW (ft): **NE**
BOREHOLE DEPTH (ft): **51.5**
STATIC DTW (ft): **NE**
WELL DEPTH (ft): **---**
WELL CASING DIAMETER (in): **---**
BOREHOLE DIAMETER (in): **8**
LOGGED BY: **R. Woodford**
CHECKED BY: **J. Fischer**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Borehole Backfill
45					P1-45'		21 29 41		45	Backfilled with gravel pack.
		SP-SM	**SILTY SAND** ; SP-SM; 7.5YR 4/4 brown; 92% fine grained sand; 8% fines; very dense.							
50					P1-50'	SA	15 24 33		50	Backfilled with bentonite.
			Hole terminated at 51.5 feet.							
55									55	
60									60	
65									65	

PROJECT: **650-700 PCH**		WELL / TEST PIT / BOREHOLE NO: **P2**	PAGE 1 OF 3
LOCATION: **El Segundo, CA**			
PROJECT NUMBER: **185804814**			
DRILLING: STARTED **5/29/20**	COMPLETED: **5/29/20**	NORTHING (ft):	EASTING (ft):
INSTALLATION: STARTED **5/29/20**	COMPLETED: **5/29/20**	LATITUDE: **33° 55' 29.64"**	LONGITUDE: **118° 23' 37.32"**
DRILLING COMPANY: **ABC Liovin Drilling**		GROUND ELEV (ft): **111.0**	TOC ELEV (ft):
DRILLING EQUIPMENT: **CME 85**		INITIAL DTW (ft): **NE**	BOREHOLE DEPTH (ft): **51.5**
DRILLING METHOD: **HSA**		STATIC DTW (ft): **NE**	WELL DEPTH (ft): **---**
		WELL CASING DIAMETER (in): **---**	BOREHOLE DIAMETER (in): **8**
SAMPLING EQUIPMENT: **Split Spoon**		LOGGED BY: **R. Woodford**	CHECKED BY: **J. Fischer**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Borehole Backfill
			3" ASPHALT							
			3" AB							
			OLD EOLIAN DEPOSITS (Qoe)							Backfilled with bentonite.
		SM	**SILTY SAND** ; SM; 7.5YR 3/2 dark brown; fine to medium-grained; 62% fine to medium grained sand; 38% fines; moist.		P2-2'	SA				
5			Trace coarse grained sand; loose below 5 feet.		P2-5'		7 7 7		5	
					P2-7'					
10		CL	**SANDY LEAN CLAY** ; CL; 10YR3/2 very dark grayish brown; fine-grained; 90% fines; 10% fine grained sand; stiff, moist.		P2-10'	#200, AL	4 8 13		10	
15			Sorted below 15 feet.		P2-15'		2 2 3		15	
20		SP	**POORLY GRADED SAND** ; SP; 5YR 5/6 yellowish red; fine-grained; 95% fien grained sand; 5% fines; very dense, moist.		P2-20'		18 20 31		20	Backfilled with bentonite

GEO FORM 304 650_700_PCH.GPJ SECOR INTL.GDT 6/28/20

PROJECT: **650-700 PCH**	WELL / TEST PIT / BOREHOLE NO: **P2**	
LOCATION: **El Segundo, CA**	NORTHING (ft):	EASTING (ft):
PROJECT NUMBER: **185804814**	LATITUDE: **33° 55' 29.64"**	LONGITUDE: **118° 23' 37.32"**
DRILLING: STARTED **5/29/20** COMPLETED: **5/29/20**	GROUND ELEV (ft): **111.0**	TOC ELEV (ft):
INSTALLATION: STARTED **5/29/20** COMPLETED: **5/29/20**	INITIAL DTW (ft): **NE**	BOREHOLE DEPTH (ft): **51.5**
DRILLING COMPANY: **ABC Liovin Drilling**	STATIC DTW (ft): **NE**	WELL DEPTH (ft): **---**
DRILLING EQUIPMENT: **CME 85**	WELL CASING DIAMETER (in): **---**	BOREHOLE DIAMETER (in): **8**
DRILLING METHOD: **HSA**	LOGGED BY: **R. Woodford**	CHECKED BY: **J. Fischer**
SAMPLING EQUIPMENT: **Split Spoon**		

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Borehole Backfill
25			7.5YR 5/6 dark brown; fine to medium-grained; dense below 25 feet.		P2-25'		8 14 18		25	
30			Very dense below 30 feet.		P2-30'		11 24 34		30	
35			Dense below 35 feet.		P2-35'		10 16 20		35	
40			Very dense below 40 feet.		P2-40'	SA	16 26 26		40	

PROJECT: **650-700 PCH**
LOCATION: **El Segundo, CA**
PROJECT NUMBER: **185804814**
DRILLING: STARTED **5/29/20** COMPLETED: **5/29/20**
INSTALLATION: STARTED **5/29/20** COMPLETED: **5/29/20**
DRILLING COMPANY: **ABC Liovin Drilling**
DRILLING EQUIPMENT: **CME 85**
DRILLING METHOD: **HSA**
SAMPLING EQUIPMENT: **Split Spoon**

WELL / TEST PIT / BOREHOLE NO:
P2

Stantec

NORTHING (ft):
EASTING (ft):
LATITUDE: **33° 55' 29.64"**
LONGITUDE: **118° 23' 37.32"**
GROUND ELEV (ft): **111.0**
TOC ELEV (ft):
INITIAL DTW (ft): **NE**
BOREHOLE DEPTH (ft): **51.5**
STATIC DTW (ft): **NE**
WELL DEPTH (ft): **---**
WELL CASING DIAMETER (in): **---**
BOREHOLE DIAMETER (in): **8**
LOGGED BY: **R. Woodford**
CHECKED BY: **J. Fischer**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Borehole Backfill
45			Trace coarse rounded sand; dense below 45 feet.		P2-45'		12 17 21		45	Backfilled with gravel pack.
50		SP-SM	**POORLY GRADED SAND WITH SILT** ; SP-SM; dark brown; 93% fien grained sand; 7% fines; dry, dense.		P2-50'	SA	11 16 22		50	Backfilled with bentonite.
			Hole terminated at 51.5 feet.							
55									55	
60									60	
65									65	

PROJECT: **650-700 PCH**
LOCATION: **El Segundo, CA**
PROJECT NUMBER: **185804814**
DRILLING: STARTED **5/29/20** COMPLETED: **5/29/20**
INSTALLATION: STARTED **5/29/20** COMPLETED: **5/29/20**
DRILLING COMPANY: **ABC Liovin Drilling**
DRILLING EQUIPMENT: **CME 85**
DRILLING METHOD: **HSA**
SAMPLING EQUIPMENT: **Split Spoon**

WELL / TEST PIT / BOREHOLE NO:
P3 PAGE 1 OF 3

Stantec

NORTHING (ft):
LATITUDE: **33° 55' 33.24"**
GROUND ELEV (ft): **113.0**
INITIAL DTW (ft): **NE**
STATIC DTW (ft): **NE**
WELL CASING DIAMETER (in): **---**
LOGGED BY: **R. Woodford**

EASTING (ft):
LONGITUDE: **118° 23' 40.92"**
TOC ELEV (ft):
BOREHOLE DEPTH (ft): **51.5**
WELL DEPTH (ft): **---**
BOREHOLE DIAMETER (in): **8**
CHECKED BY: **J. Fischer**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Borehole Backfill
			2" ASPHALT							
			1" AB							
			OLD EOLIAN DEPOSITS (Qoe)							Backfilled with bentonite.
		SM	**SILTY SAND** ; SM; 7.5YR 3/4 dark brown; fine to medium-grained; 90% fine to medium grained sand;10% fines; moist.		P3-2'					
5			Medium dense below 5 feet.		P3-5'		3 6 6		5	
10		SC	**WELL GRADED SAND WITH CLAY** ; SC; 7.5YR 4/6 dark brown; fine to medium-grained; 90% fien to medium grained sand; 10% fines; medium dense, moist.		P3-10'		8 11 22		10	
15					P3-15'		8 10 23		15	
20		SW	**WELL GRADED SAND** ; SW; 7.5YR 5/8 dark brown; fine to medium-grained; 95% fine to medium grained sand; <5% fines; medium dense, moist.		P3-20'		11 26 36		20	Backfilled with bentonite

PROJECT: **650-700 PCH**
LOCATION: **El Segundo, CA**
PROJECT NUMBER: **185804814**
DRILLING: STARTED **5/29/20** COMPLETED: **5/29/20**
INSTALLATION: STARTED **5/29/20** COMPLETED: **5/29/20**
DRILLING COMPANY: **ABC Liovin Drilling**
DRILLING EQUIPMENT: **CME 85**
DRILLING METHOD: **HSA**
SAMPLING EQUIPMENT: **Split Spoon**

WELL / TEST PIT / BOREHOLE NO:
P3 PAGE 2 OF 3

Stantec

NORTHING (ft):
LATITUDE: **33° 55' 33.24"**
GROUND ELEV (ft): **113.0**
INITIAL DTW (ft): **NE**
STATIC DTW (ft): **NE**
WELL CASING DIAMETER (in): **---**
LOGGED BY: **R. Woodford**

EASTING (ft):
LONGITUDE: **118° 23' 40.92"**
TOC ELEV (ft):
BOREHOLE DEPTH (ft): **51.5**
WELL DEPTH (ft): **---**
BOREHOLE DIAMETER (in): **8**
CHECKED BY: **J. Fischer**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Borehole Backfill
25		SW-SC	**WELL GRADED SAND WITH CLAY** ; SW-SC; 7.5YR 4/6 dark brown; fine to medium-grained; 90% fine to medium grained sand; 10% fines; medium dense, moist.		P3-25'		11 18 20		25	
30					P3-30'		14 25 28		30	
35					P3-35'		17 26 36		35	
40		SP	**POORLY GRADED SAND** ; SP; 7.5YR 4/6 dark brown; fine to medium-grained; 95% fine to medium grained sand; 5% fines; very dense; dry.		P3-40'	SA	11 22 30		40	

PROJECT: **650-700 PCH**	WELL / TEST PIT / BOREHOLE NO: **P3** PAGE 3 OF 3
LOCATION: **El Segundo, CA**	NORTHING (ft):
PROJECT NUMBER: **185804814**	EASTING (ft):
DRILLING: STARTED **5/29/20** COMPLETED: **5/29/20**	LATITUDE: **33° 55' 33.24"**
INSTALLATION: STARTED **5/29/20** COMPLETED: **5/29/20**	LONGITUDE: **118° 23' 40.92"**
DRILLING COMPANY: **ABC Liovin Drilling**	GROUND ELEV (ft): **113.0**
DRILLING EQUIPMENT: **CME 85**	TOC ELEV (ft):
DRILLING METHOD: **HSA**	INITIAL DTW (ft): **NE**
SAMPLING EQUIPMENT: **Split Spoon**	BOREHOLE DEPTH (ft): **51.5**
	STATIC DTW (ft): **NE**
	WELL DEPTH (ft): **---**
	WELL CASING DIAMETER (in): **---**
	BOREHOLE DIAMETER (in): **8**
	LOGGED BY: **R. Woodford**
	CHECKED BY: **J. Fischer**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Geotechnical Lab Testing	Blow Count	PID Reading (ppmv)	Depth (feet)	Borehole Backfill
45					P3-45'		13 20 31		45	Backfilled with gravel pack.
50		SM	**SILTY SAND** ; SM; 7.5YR 4/6 dark brown; fine-grained; 88% fine grained sand; 12% fines; very dense; dry.		P3-50'	SA	16 24 28		50	Backfilled with bentonite.
			Hole terminated at 51.5 feet.							
55									55	
60									60	
65									65	

APPENDIX B
LABORATORY TEST RESULTS

Stantec

Gradation Analysis

ASTM D 422

Project Name	650-700 PCH	Project Number	185804814
Source	Grab	Lab ID	B1-2'
		Date Received	06-15-2020
Preparation Method	ASTM D 1140 Method A	Preparation Date	06-19-2020
Particle Shape		Test Date	06-20-2020
Particle Hardness			
Sample Dry Mass (g)	323.10		
Moisture Content (%)	6.5		

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 8	1.00	0.3	99.7
No. 16	0.50	0.2	99.5
No. 30	15.90	4.9	94.6
No. 50	144.10	44.6	50.0
No. 100	69.10	21.4	28.6
No. 200	8.30	2.6	26.1
Pan	84.20	26.1	---

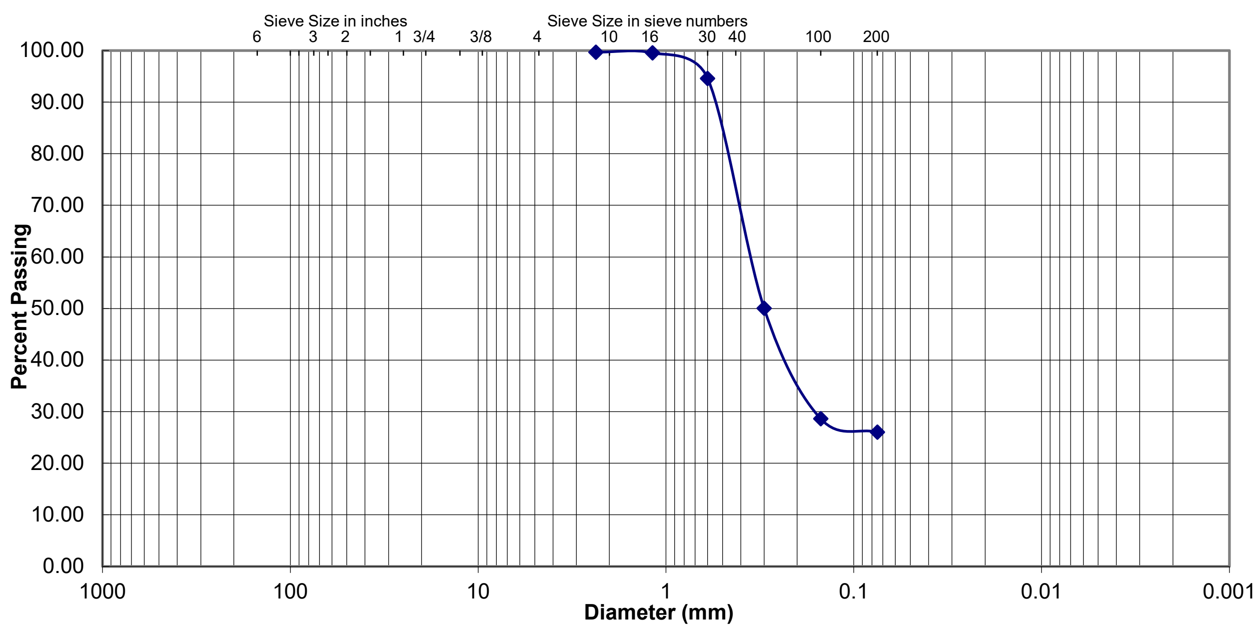
% Gravel	0.0
% Sand	73.9
% Fines	26.1
Fines Classification	ML
D_{10} (mm)	N/A
D_{30} (mm)	N/A
D_{60} (mm)	N/A
Cu	N/A
Cc	N/A

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 JF

File: 650-700_PCH_B1-2_Sieve.xlsm Sheet: Report
Preparation Date: 1-2008
Revision Date: 4-2008

Stantec Consulting Services Inc.

Laboratory Document
Prepared By: JW
Approved By: TLK

Gradation Analysis
ASTM D 422

Project Name	650-700 PCH	Project Number	185804814
Source	Grab	Lab ID	B2-2'
		Date Received	06-15-2020
Preparation Method	ASTM D 1140 Method A	Preparation Date	06-19-2020
Particle Shape	N/A	Test Date	06-20-2020
Particle Hardness	N/A		
Sample Dry Mass (g)	289.30		
Moisture Content (%)	6.9		

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 30	11.80	4.1	95.9
No. 50	131.40	45.4	50.5
No. 100	63.70	22.0	28.5
No. 200	11.50	4.0	24.5
Pan	70.90	24.5	---

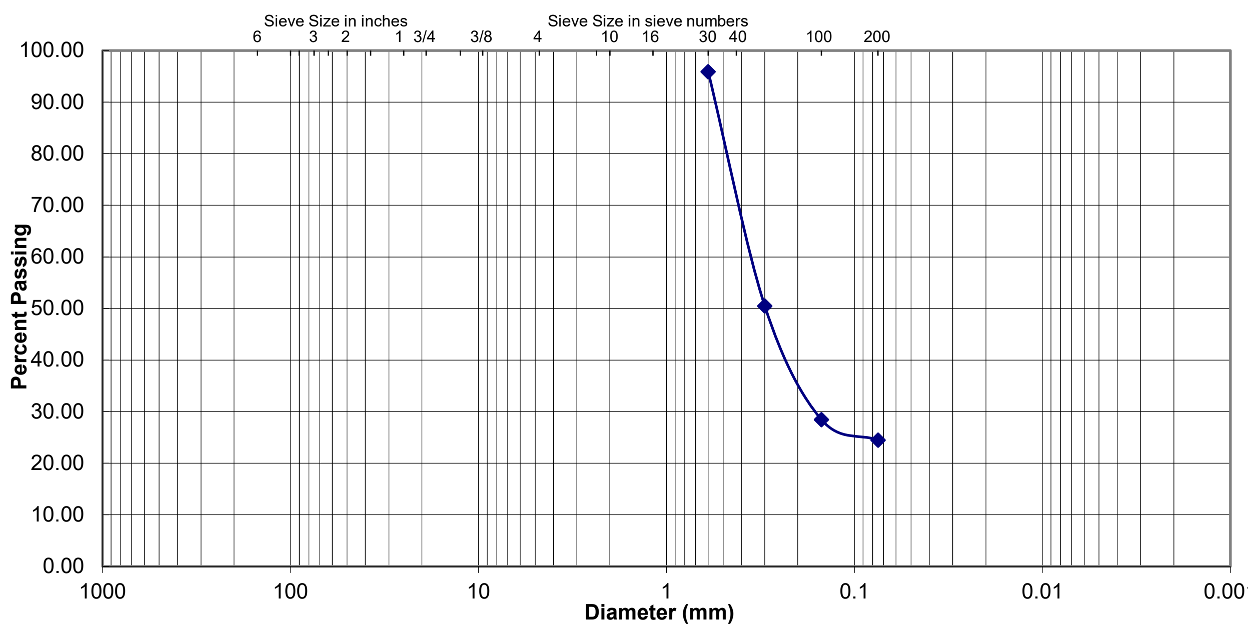
% Gravel	0.0
% Sand	75.5
% Fines	24.5
Fines Classification	ML
D_{10} (mm)	N/A
D_{30} (mm)	N/A
D_{60} (mm)	N/A
Cu	N/A
Cc	N/A

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 JF

Stantec

Gradation Analysis
ASTM D 422

Project Name	650-700 PCH	Project Number	185804814
Source	Grab	Lab ID	B3-2'
		Date Received	06-15-2020
Preparation Method	ASTM D 1140 Method A	Preparation Date	06-19-2020
Particle Shape		Test Date	06-20-2020
Particle Hardness			
Sample Dry Mass (g)	245.50		
Moisture Content (%)	9.3		

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 4	2.90	1.2	98.8
No. 8	2.70	1.1	97.7
No. 16	2.90	1.2	96.5
No. 30	12.50	5.1	91.4
No. 50	93.60	38.1	53.3
No. 100	55.80	22.7	30.6
No. 200	10.50	4.3	26.3
Pan	64.60	26.3	---

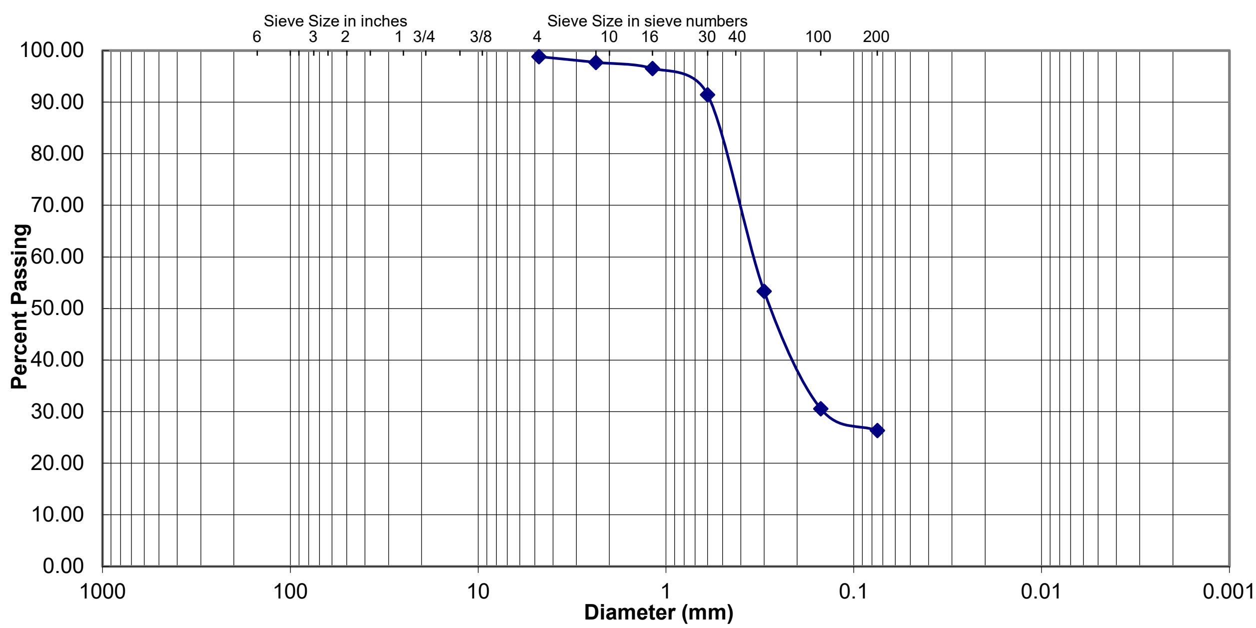
% Gravel	1.2
% Sand	72.5
% Fines	26.3
Fines Classification	ML
D_{10} (mm)	N/A
D_{30} (mm)	N/A
D_{60} (mm)	N/A
Cu	N/A
Cc	N/A

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 JF

Gradation Analysis

ASTM D 422

Project Name 650-700 PCH
Source CM

Project Number 185804814
Lab ID B3-15'
Date Received 06-15-2020
Preparation Date 06-19-2020
Test Date 06-20-2020

Preparation Method ASTM D 1140 Method A
Particle Shape N/A
Particle Hardness N/A
Sample Dry Mass (g) 261.00
Moisture Content (%) 18.0

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 30	1.40	0.5	99.5
No. 50	19.40	7.4	92.0
No. 100	64.80	24.8	67.2
No. 200	69.10	26.5	40.7
Pan	106.30	40.7	---

% Gravel 0.0
% Sand 59.3
% Fines 40.7
Fines Classification ML

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

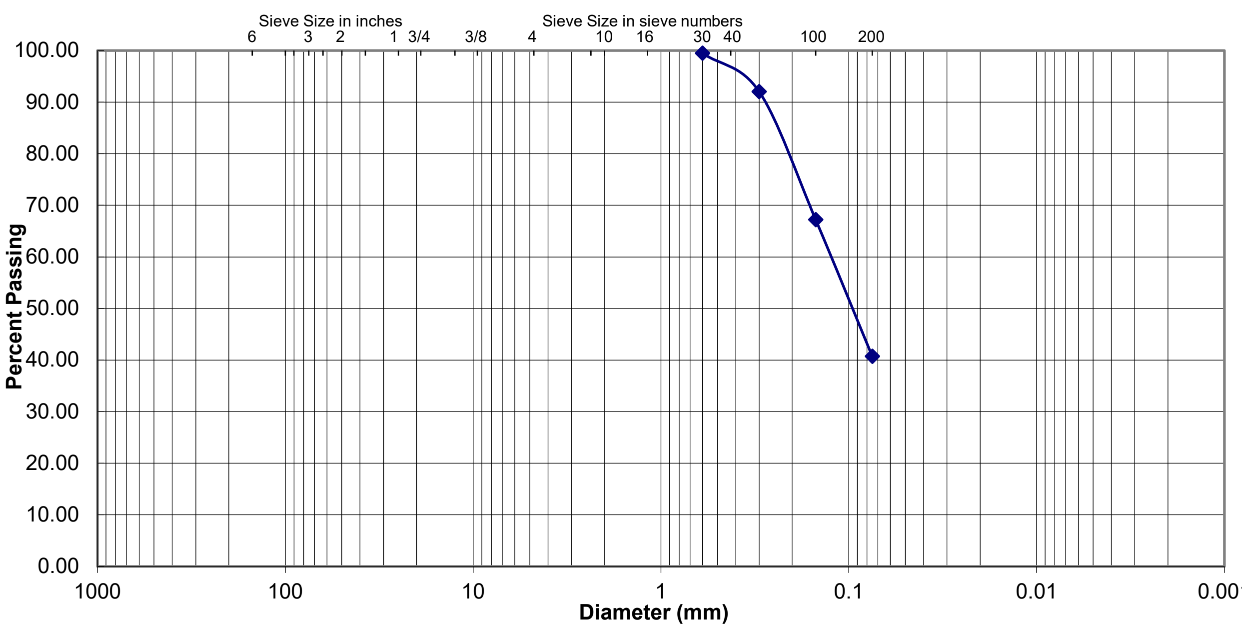
Cu N/A
Cc N/A

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 JF

Gradation Analysis
ASTM D 422

Project Name 650-700 PCH
Source SPT

Project Number 185804814
Lab ID B3-20'
Date Received 06-15-2020
Preparation Date 06-19-2020
Test Date 06-20-2020

Preparation Method ASTM D 1140 Method A
Particle Shape N/A
Particle Hardness N/A
Sample Dry Mass (g) 258.60
Moisture Content (%) 6.7

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 30	8.90	3.4	96.6
No. 50	128.70	49.8	46.8
No. 100	78.60	30.4	16.4
No. 200	6.80	2.6	13.8
Pan	35.60	13.8	---

% Gravel 0.0
% Sand 86.2
% Fines 13.8
Fines Classification ML

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

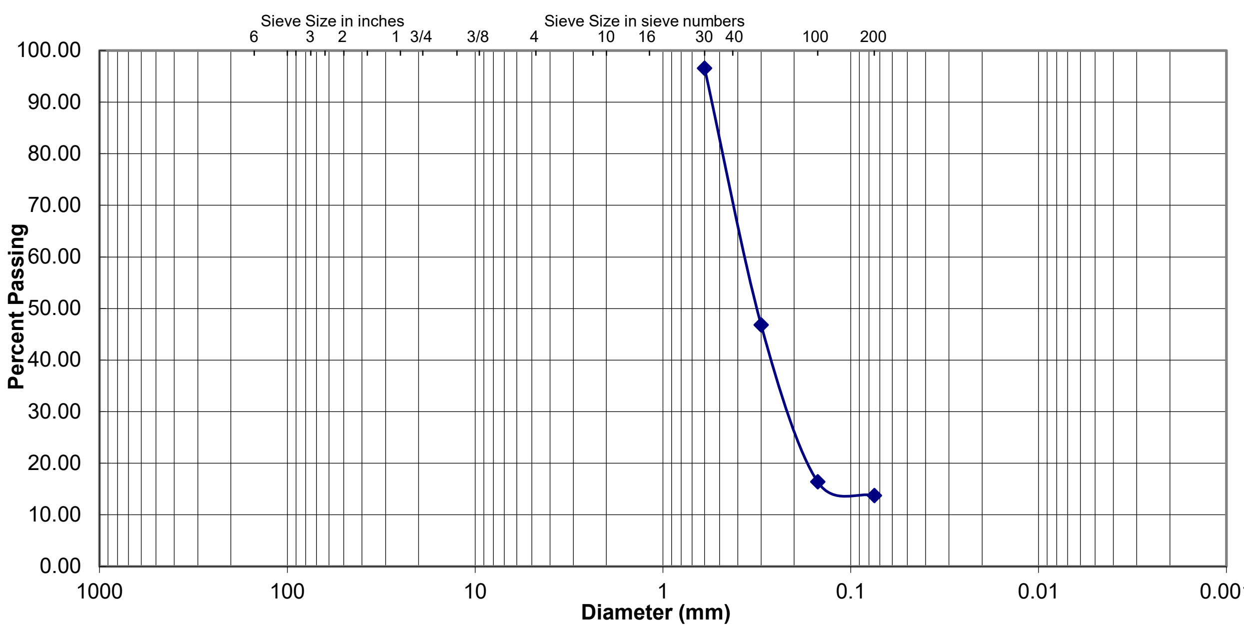
Cu N/A
Cc N/A

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 JF

Stantec

Gradation Analysis
ASTM D 422

Project Name	650-700 PCH	Project Number	185804814
Source	SPT	Lab ID	P1-50'
		Date Received	06-15-2020
Preparation Method	ASTM D 1140 Method A	Preparation Date	06-19-2020
Particle Shape	N/A	Test Date	06-20-2020
Particle Hardness	N/A		
Sample Dry Mass (g)	220.30		
Moisture Content (%)	5.4		

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 30	2.50	1.1	98.9
No. 50	14.90	6.8	92.1
No. 100	117.00	53.1	39.0
No. 200	67.40	30.6	8.4
Pan	18.50	8.4	---

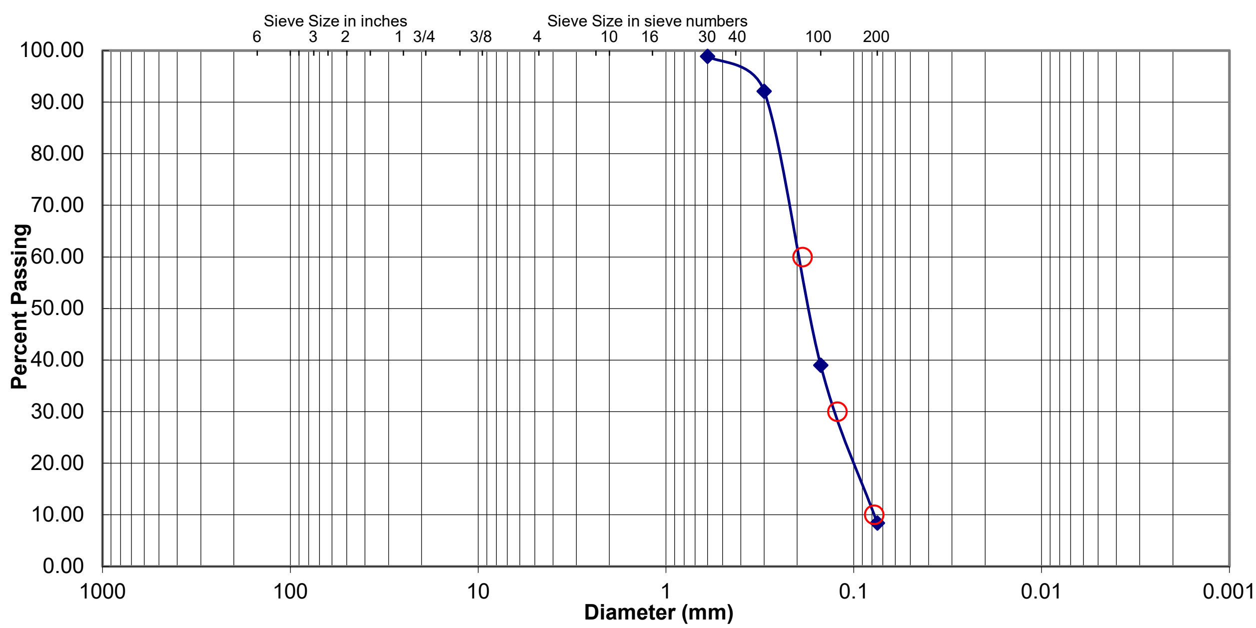
% Gravel	0.0
% Sand	91.6
% Fines	8.4
Fines Classification	ML
D_{10} (mm)	0.0778
D_{30} (mm)	0.1224
D_{60} (mm)	0.1873
Cu	2.41
Cc	1.03

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 JF

Stantec

Gradation Analysis
ASTM D 422

Project Name 650-700 PCH
Source Grab

Preparation Method ASTM D 1140 Method A
Particle Shape
Particle Hardness
Sample Dry Mass (g) 310.40
Moisture Content (%) 11.8

Project Number 185804814
Lab ID P2-2'
Date Received 06-15-2020
Preparation Date 06-19-2020
Test Date 06-20-2020

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 4	1.30	0.4	99.6
No. 8	0.40	0.1	99.5
No. 16	1.40	0.5	99.0
No. 30	9.30	3.0	96.0
No. 50	100.40	32.3	63.7
No. 100	56.90	18.3	45.3
No. 200	22.70	7.3	38.0
Pan	118.00	38.0	---

% Gravel 0.4
% Sand 61.6
% Fines 38.0
Fines Classification ML

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

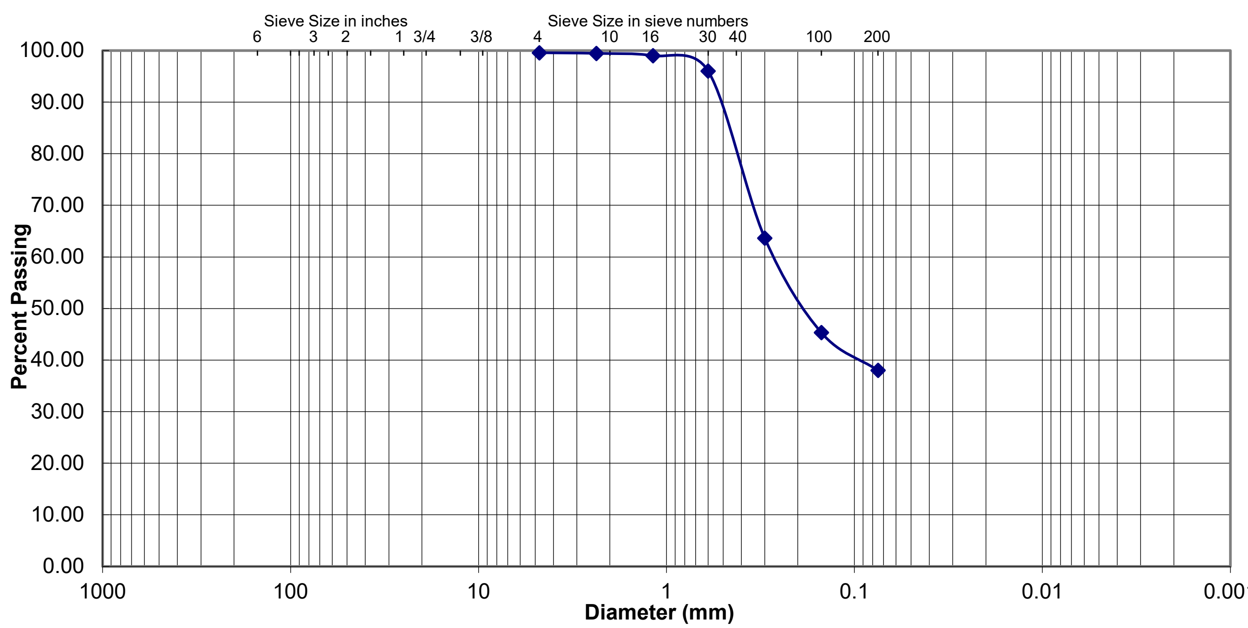
Cu N/A
Cc N/A

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 JF

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

Project Name	650-700 PCH	Project Number	185804814
Source	P2-10'	Lab ID	P2-10'
		Date Received	06-15-2020
Preparation Method	ASTM D 1140 Method A	Test Date	06-19-2020

Initial Sample Wet Mass (g)	236.60	Moisture Content (%)	17.9
Initial Oven Dry Sample Mass (g)	200.60		
Final Oven Dry Sample Mass (g)	97.40		
Materials Finer Than 75µm (No. 200) Sieve (g)	103.20		
Percent Finer Than 75µm (No. 200) Sieve (%)	51.4		

Comments

Reviewed By JF

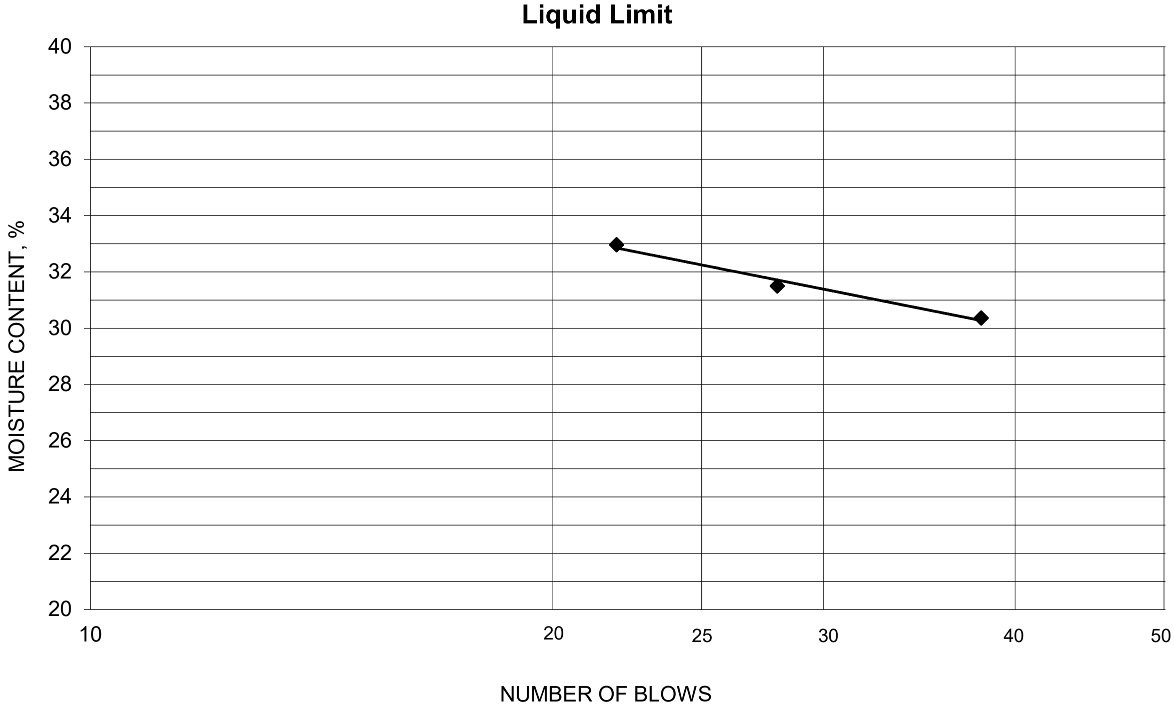
Stantec

ATTERBERG LIMITS

Project	650-700 PCH	Project No.	185804814
Source	SPT	Lab ID	P2-10'
Tested By	M.P. Test Method ASTM D 4318	% + No. 40	10
Test Date	06-20-2020 Prepared Dry	Date Received	06-15-2020

Wet Soil and Tare Mass (g)	Dry Soil and Tare Mass (g)	Tare Mass (g)	Number of Blows	Water Content (%)	Liquid Limit
21.84	19.93	13.64	38	30.4	
25.94	23.02	13.75	28	31.5	
25.68	22.67	13.54	22	33.0	32

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
21.53	20.41	13.72	16.7	17	15

Remarks: USCS Soil Type = Clay (CL)

Reviewed By JF

Stantec

Gradation Analysis
ASTM D 422

Project Name	650-700 PCH	Project Number	185804814
Source	SPT	Lab ID	P2-40'
		Date Received	06-15-2020
Preparation Method	ASTM D 1140 Method A	Preparation Date	06-19-2020
Particle Shape		Test Date	06-20-2020
Particle Hardness			
Sample Dry Mass (g)	256.80		
Moisture Content (%)	11.2		

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 16	0.50	0.2	99.8
No. 30	12.00	4.7	95.1
No. 50	63.00	24.5	70.6
No. 100	115.20	44.9	25.7
No. 200	53.50	20.8	4.9
Pan	12.60	4.9	---

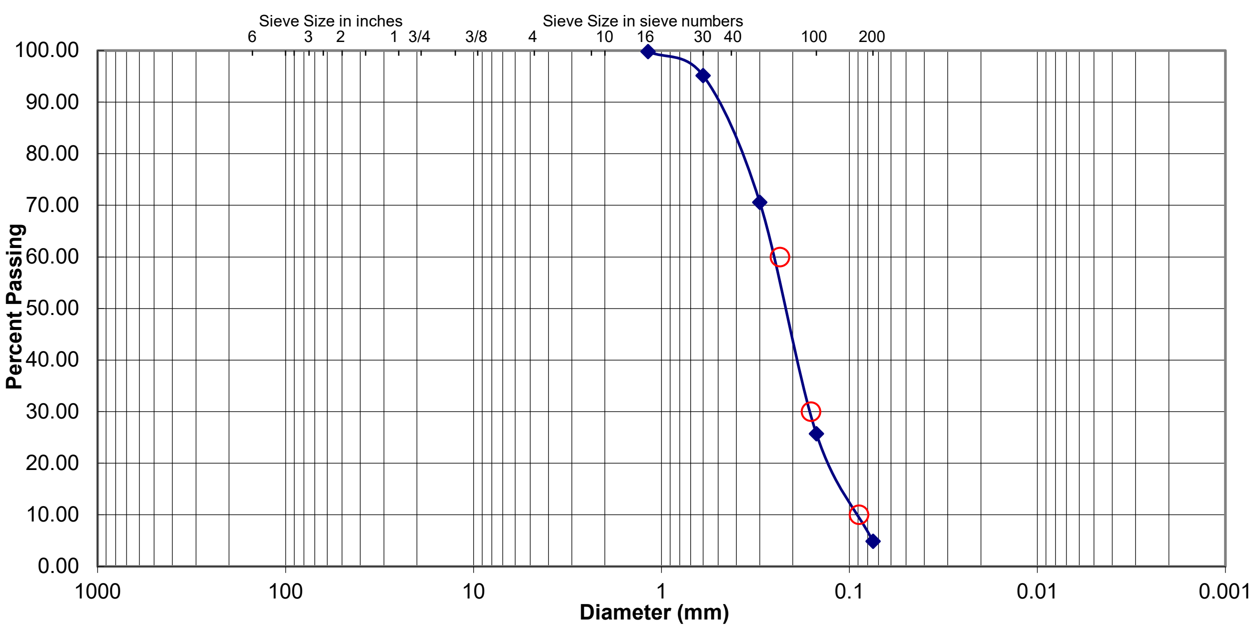
% Gravel	0.0
% Sand	95.1
% Fines	4.9
Fines Classification	ML
D_{10} (mm)	0.0889
D_{30} (mm)	0.1602
D_{60} (mm)	0.2347
Cu	2.64
Cc	1.23

Classification

Poorly Graded Sand (SP)

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

Particle Size Distribution



Comments

Reviewed By JF

Stantec

Gradation Analysis
ASTM D 422

Project Name: 650-700 PCH
Source: SPT

Project Number: 185804814
Lab ID: P2-50'
Date Received: 06-15-2020
Preparation Date: 06-19-2020
Test Date: 06-20-2020

Preparation Method: ASTM D 1140 Method A
Particle Shape:
Particle Hardness:
Sample Dry Mass (g): 248.00
Moisture Content (%): 8.0

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 8	0.60	0.2	99.8
No. 16	0.00	0.0	99.8
No. 30	0.00	0.0	99.8
No. 50	1.10	0.4	99.3
No. 100	112.40	45.3	54.0
No. 200	115.60	46.6	7.4
Pan	18.30	7.4	---

% Gravel: 0.0
% Sand: 92.6
% Fines: 7.4
Fines Classification: ML

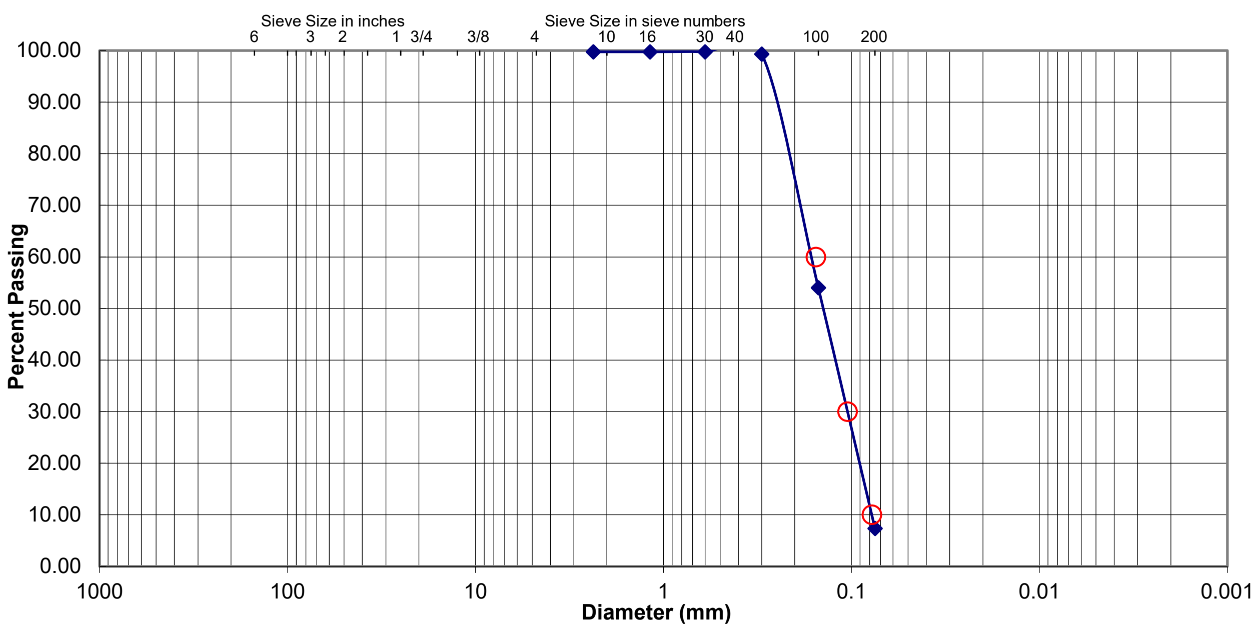
D_{10} (mm): 0.0780
D_{30} (mm): 0.1050
D_{60} (mm): 0.1544

Cu: 1.98
Cc: 0.92

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: JF

Gradation Analysis
ASTM D 422

Project Name	650-700 PCH	Project Number	185804814
Source	SPT	Lab ID	P3-40'
		Date Received	06-15-2020
Preparation Method	ASTM D 1140 Method A	Preparation Date	06-19-2020
Particle Shape	N/A	Test Date	06-20-2020
Particle Hardness	N/A		
Sample Dry Mass (g)	213.80		
Moisture Content (%)	7.6		

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 30	2.80	1.3	98.7
No. 50	16.40	7.7	91.0
No. 100	151.40	70.8	20.2
No. 200	32.70	15.3	4.9
Pan	10.50	4.9	---

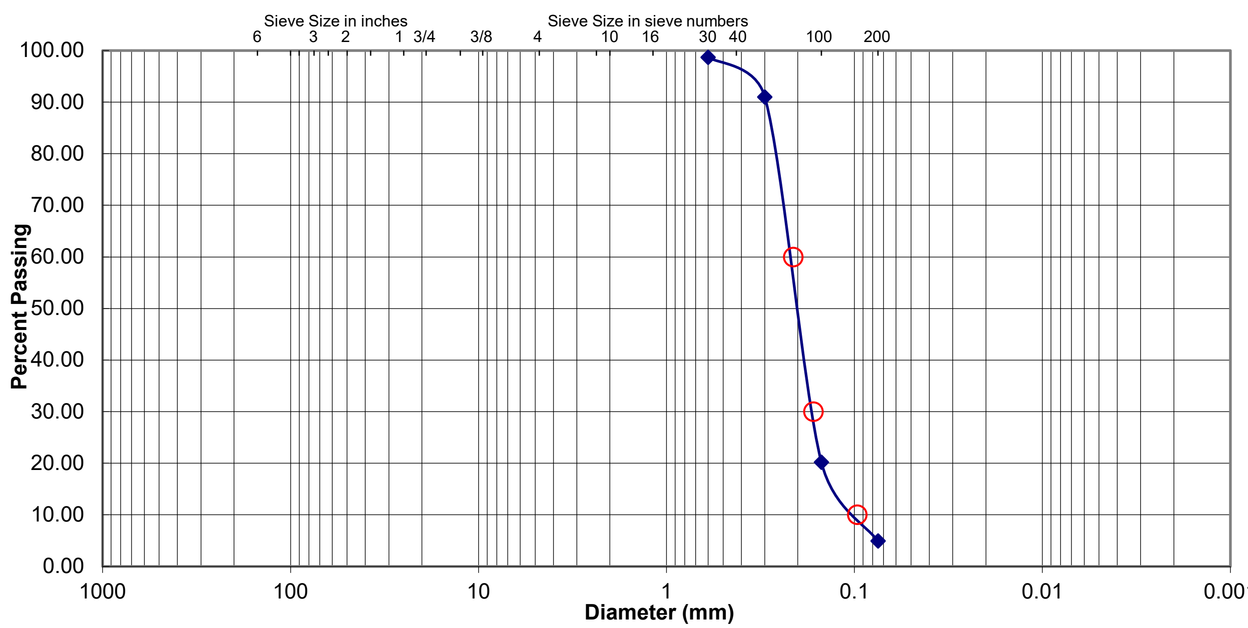
% Gravel	0.0
% Sand	95.1
% Fines	4.9
Fines Classification	ML
D_{10} (mm)	0.0965
D_{30} (mm)	0.1651
D_{60} (mm)	0.2114
Cu	2.19
Cc	1.34

Classification

Poorly Graded Sand (SP)

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

Particle Size Distribution



Comments

Reviewed By JF

Stantec

Gradation Analysis
ASTM D 422

Project Name 650-700 PCH
Source SPT

Project Number 185804814
Lab ID P3-50'
Date Received 06-15-2020
Preparation Date 06-19-2020
Test Date 06-20-2020

Preparation Method ASTM D 1140 Method A
Particle Shape
Particle Hardness
Sample Dry Mass (g) 308.80
Moisture Content (%) 9.3

Analysis based on total sample.

Sieve Size	Grams Retained	% Retained	% Passing
No. 8	1.90	0.6	99.4
No. 16	0.00	0.0	99.4
No. 30	0.00	0.0	99.4
No. 50	1.00	0.3	99.1
No. 100	72.70	23.5	75.5
No. 200	195.30	63.2	12.3
Pan	37.90	12.3	---

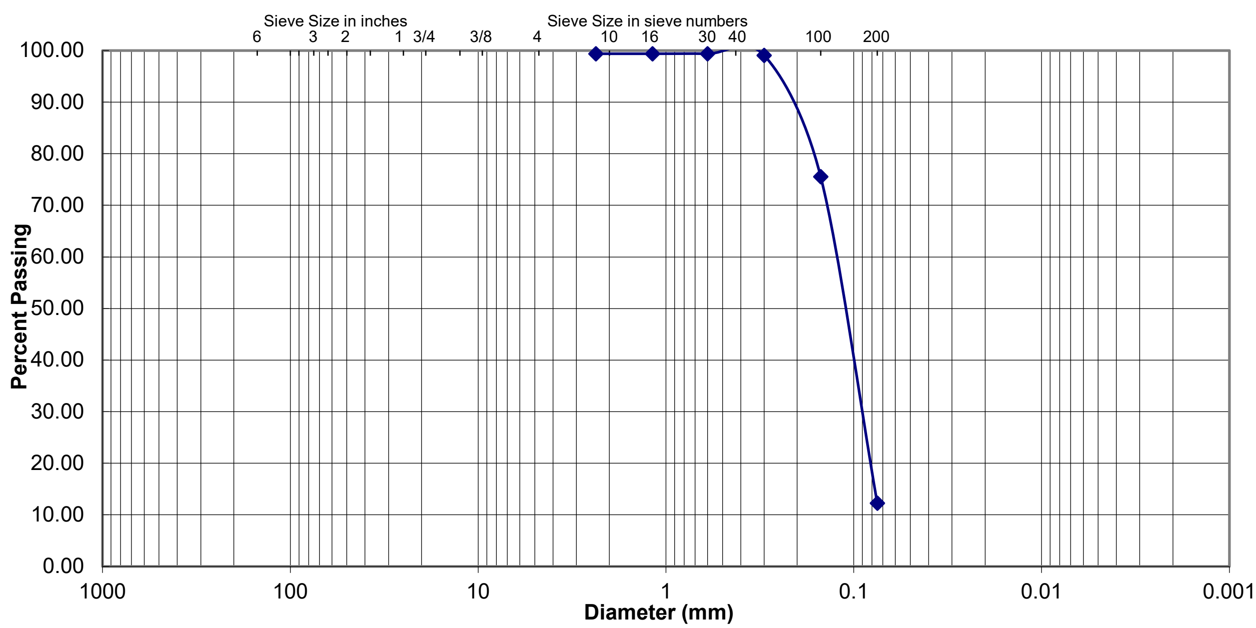
% Gravel	0.0
% Sand	87.7
% Fines	12.3
Fines Classification	ML
D_{10} (mm)	N/A
D_{30} (mm)	N/A
D_{60} (mm)	N/A
Cu	N/A
Cc	N/A

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 JF

JD & S Testing

Direct Shear Moisture Content & Density

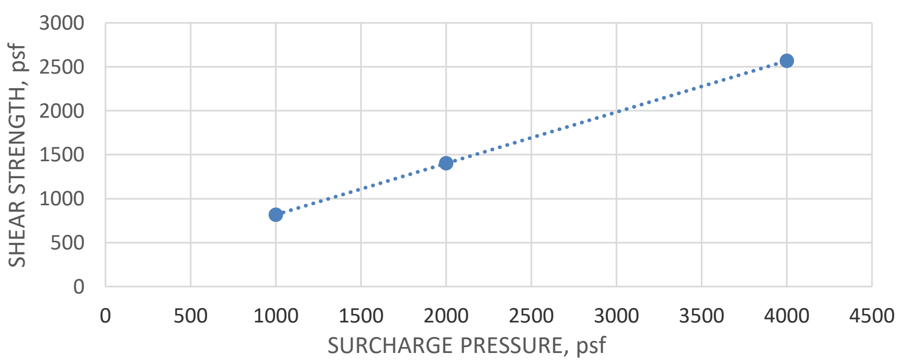
Project Name: 650-700 PCH   Project Number: 185804814

Sampled By: Client   Sample Date: 5/29/2020   Lab #:

Source/Location: B1-5'   Tested By: M.P.

Description: Test Date: 6/20/2020

	Shearing Rate (in./min.)	0.040		
	Normal Pressure (psf)	1000	2000	4000
[A]	Initial Weight of Wet Soil + Ring (0.1g)	176.2	168.9	177.3
[B]	Weight of Ring (0.1g)	43.5	45.1	42.3
[C] = [A] - [B]	Initial Weight of Wet Soil (0.1g)	132.7	123.8	135.0
[D]	Initial Reading / Time	0.0000	0.0000	0.0000
	Time			
	Time			
	Time			
[E]	Final Reading / Time			
[F] = [E] - [D]	Height Change	0.0008	0.0031	0.0024
[G] = 1 - [F]	Final Height	0.9992	0.9969	0.9976
	Shear Strength (psf)	816	1404	2568
[H]	Final Weight of Wet Soil (0.1g)	141.5	134.4	146.6
[J]	Weight of Dry Soil (0.1g)	120.6	113.7	124.6
[K]	Specific Gravity	2.7		
[L]	Average Maximum Dry Density (0.1pcf)	99.5		
[M]	Initial Volume (0.01 cubic inch)	4.60	4.60	4.60
[P] = ([C] - [J]) / [J]	Initial Moisture Content (0.1%)	10.0%	8.9%	8.3%
[Q] = [C] / [M] * 3.81	Initial Wet Density (0.1pcf)	109.9	102.5	111.8
[R] = [Q] / (1 + [P])	Initial Dry Density (0.1pcf)	99.9	94.2	103.2
[S]=[P]x[K]x[R]/([K]x62.3)-[R]	Initial Saturation (0.1%)	43.6%	33.2%	38.8%
[T] = [R] / [L]	Initial Relative Compaction (0.1%)	100.4%	94.7%	103.7%
[U] = [M] X [G]	Final Volume (0.01 cubic inch)	4.60	4.59	4.59
[V] = ([H]-[J]) / [J]	Final Moisture Content (0.1%)	17.3%	18.2%	17.7%
[W] = [H] / [U] * 3.81	Final Wet Density (0.1pcf)	117.3	111.7	121.7
[X] = [W] / (1 + [V])	Final Dry Density (0.1pcf)	100.0	94.5	103.4
[Y]= [V]x[K]x[X]/([K]x62.3)-[X]	Final Saturation (0.1%)	68.5%	63.0%	76.2%
[Z] = [X] / [L]	Final Relative Compaction (0.1%)	100.5%	95.0%	104.0%



Cohesion (psf)	240
Friction Angle (degrees)	30

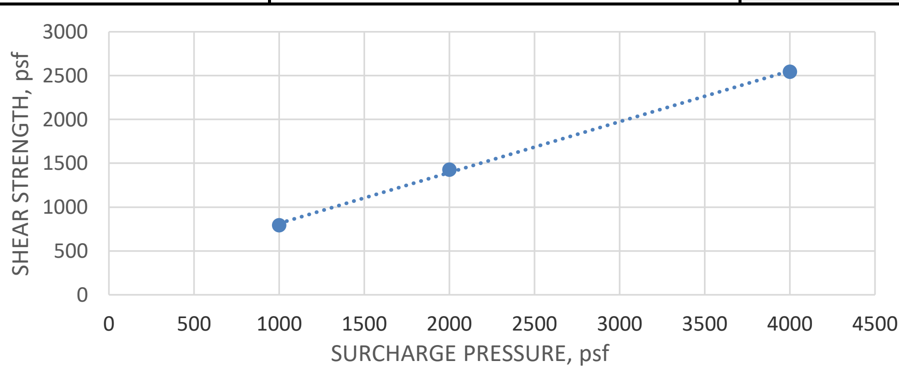
Remarks: Reviewed by: JF

JD & S Testing

Direct Shear Moisture Content & Density

Project Name:	650-700 PCH	Project Number:	185804814
Sampled By:	Client	Sample Date: 5/29/2020	Lab #:
Source/Location:	B2-10'	Tested By:	M.P.
Description:		Test Date:	6/20/2020

	Shearing Rate (in./min.)		0.040		
	Normal Pressure (psf)		1000	2000	4000
[A]	Initial Weight of Wet Soil + Ring (0.1g)		177.7	177.4	178.2
[B]	Weight of Ring (0.1g)		46.1	45.9	46.0
[C] = [A] - [B]	Initial Weight of Wet Soil (0.1g)		131.6	131.5	132.2
[D]	Initial Reading	Time	0.0000	0.0000	0.0000
		Time			
		Time			
		Time			
[E]	Final Reading	Time			
[F] = [E] - [D]	Height Change		0.0052	0.0010	0.0032
[G] = 1 - [F]	Final Height		0.9948	0.9990	0.9968
	Shear Strength (psf)		792	1428	2544
[H]	Final Weight of Wet Soil (0.1g)		138.8	140.4	142.8
[J]	Weight of Dry Soil (0.1g)		116.3	118.5	120.3
[K]	Specific Gravity			2.7	
[L]	Average Maximum Dry Density (0.1pcf)			98.4	
[M]	Initial Volume (0.01 cubic inch)		4.60	4.60	4.60
[P] = ([C] - [J]) / [J]	Initial Moisture Content (0.1%)		13.2%	11.0%	9.9%
[Q] = [C] / [M] * 3.81	Initial Wet Density (0.1pcf)		109.0	108.9	109.5
[R] = [Q] / (1 + [P])	Initial Dry Density (0.1pcf)		96.3	98.1	99.6
[S]=[P]x[K]x[R]/([K]x62.3)-[R]	Initial Saturation (0.1%)		53.9%	46.0%	42.6%
[T] = [R] / [L]	Initial Relative Compaction (0.1%)		97.9%	99.7%	101.2%
[U] = [M] X [G]	Final Volume (0.01 cubic inch)		4.58	4.60	4.59
[V] = ([H]-[J]) / [J]	Final Moisture Content (0.1%)		19.3%	18.5%	18.7%
[W] = [H] / [U] * 3.81	Final Wet Density (0.1pcf)		115.6	116.4	118.7
[X] = [W] / (1 + [V])	Final Dry Density (0.1pcf)		96.8	98.2	100.0
[Y]= [V]x[K]x[X]/([K]x62.3)-[X]	Final Saturation (0.1%)		70.9%	70.1%	74.0%
[Z] = [X] / [L]	Final Relative Compaction (0.1%)		98.4%	99.8%	101.6%



Cohesion (psf)	240
Friction Angle (degrees)	30

Remarks:

Reviewed by: JF

JD & S Testing

Direct Shear Moisture Content & Density

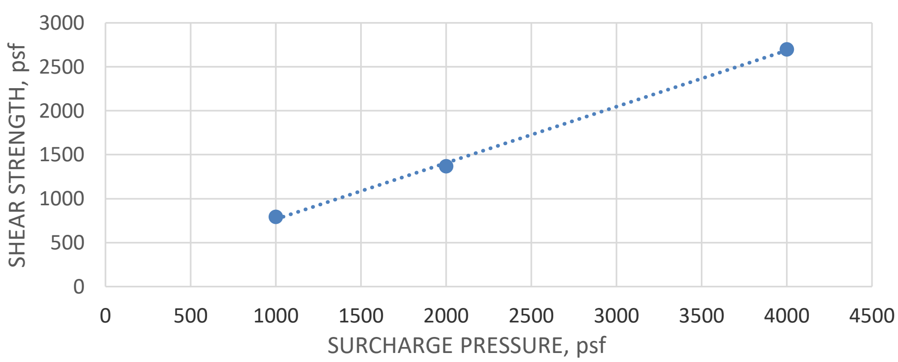
Project Name: 650-700 PCH Project Number: 185804814

Sampled By: Client Sample Date: 5/29/2020 Lab #:

Source/Location: B3-5' Tested By: M.P.

Description: Test Date: 6/20/2020

	Shearing Rate (in./min.)		0.040		
	Normal Pressure (psf)		1000	2000	4000
[A]	Initial Weight of Wet Soil + Ring (0.1g)		190.5	187.0	194.0
[B]	Weight of Ring (0.1g)		47.2	45.6	44.7
[C] = [A] - [B]	Initial Weight of Wet Soil (0.1g)		143.3	141.4	149.3
[D]	Initial Reading	Time	0.0000	0.0000	0.0000
		Time			
		Time			
		Time			
[E]	Final Reading	Time			
[F] = [E] - [D]	Height Change		0.0012	0.0005	0.0086
[G] = 1 - [F]	Final Height		0.9988	0.9995	0.9914
	Shear Strength (psf)		792	1368	2700
[H]	Final Weight of Wet Soil (0.1g)		151.2	147.3	150.4
[J]	Weight of Dry Soil (0.1g)		129.2	125.2	129.3
[K]	Specific Gravity			2.7	
[L]	Average Maximum Dry Density (0.1pcf)			106.4	
[M]	Initial Volume (0.01 cubic inch)		4.60	4.60	4.60
[P] = ([C] - [J]) / [J]	Initial Moisture Content (0.1%)		10.9%	12.9%	15.5%
[Q] = [C] / [M] * 3.81	Initial Wet Density (0.1pcf)		118.7	117.1	123.7
[R] = [Q] / (1 + [P])	Initial Dry Density (0.1pcf)		107.0	103.7	107.1
[S]=[P]x[K]x[R]/([K]x62.3)-[R]	Initial Saturation (0.1%)		57.1%	63.4%	84.5%
[T] = [R] / [L]	Initial Relative Compaction (0.1%)		100.6%	97.5%	100.7%
[U] = [M] X [G]	Final Volume (0.01 cubic inch)		4.59	4.60	4.56
[V] = ([H]-[J]) / [J]	Final Moisture Content (0.1%)		17.0%	17.7%	16.3%
[W] = [H] / [U] * 3.81	Final Wet Density (0.1pcf)		125.4	122.1	125.7
[X] = [W] / (1 + [V])	Final Dry Density (0.1pcf)		107.1	103.8	108.0
[Y]= [V]x[K]x[X]/([K]x62.3)-[X]	Final Saturation (0.1%)		80.7%	76.7%	79.1%
[Z] = [X] / [L]	Final Relative Compaction (0.1%)		100.7%	97.5%	101.6%



Cohesion (psf)	130
Friction Angle (degrees)	33

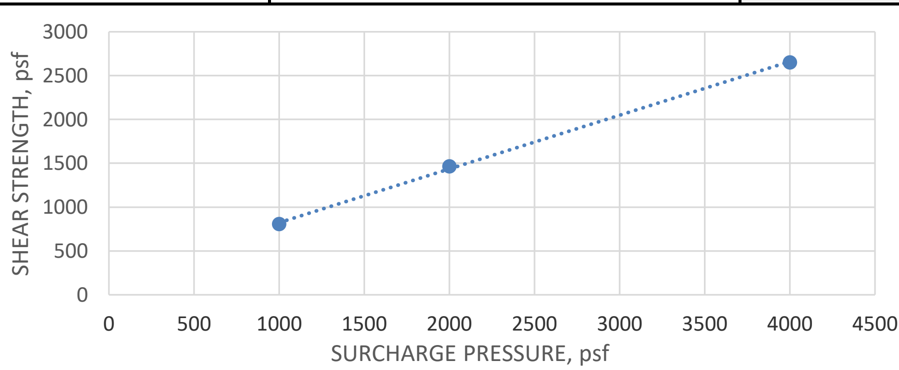
Remarks: Reviewed by: JF

JD & S Testing

Direct Shear Moisture Content & Density

Project Name: 650-700 PCH
Project Number: 185804814
Sampled By: Client
Sample Date: 5/29/2020
Lab #:
Source/Location: B3-15'
Tested By: M.P.
Description:
Test Date: 6/21/2020

	Shearing Rate (in./min.)	0.040		
	Normal Pressure (psf)	1000	2000	4000
[A]	Initial Weight of Wet Soil + Ring (0.1g)	196.4	195.1	195.6
[B]	Weight of Ring (0.1g)	45.3	42.3	45.1
[C] = [A] - [B]	Initial Weight of Wet Soil (0.1g)	151.1	152.8	150.5
[D]	Initial Reading Time	0.0000	0.0000	0.0000
	Time			
	Time			
	Time			
[E]	Final Reading Time			
[F] = [E] - [D]	Height Change	0.0116	0.0039	0.0027
[G] = 1 - [F]	Final Height	0.9884	0.9961	0.9973
	Shear Strength (psf)	804	1464	2652
[H]	Final Weight of Wet Soil (0.1g)	152.7	152.9	151.1
[J]	Weight of Dry Soil (0.1g)	125.3	125.8	124.5
[K]	Specific Gravity	2.7		
[L]	Average Maximum Dry Density (0.1pcf)	104.1		
[M]	Initial Volume (0.01 cubic inch)	4.60	4.60	4.60
[P] = ([C] - [J]) / [J]	Initial Moisture Content (0.1%)	20.6%	21.5%	20.9%
[Q] = [C] / [M] * 3.81	Initial Wet Density (0.1pcf)	125.2	126.6	124.7
[R] = [Q] / (1 + [P])	Initial Dry Density (0.1pcf)	103.8	104.2	103.1
[S]=[P]x[K]x[R]/([K]x62.3)-[R]	Initial Saturation (0.1%)	108.0%	114.6%	108.0%
[T] = [R] / [L]	Initial Relative Compaction (0.1%)	99.7%	100.1%	99.0%
[U] = [M] X [G]	Final Volume (0.01 cubic inch)	4.55	4.58	4.59
[V] = ([H]-[J]) / [J]	Final Moisture Content (0.1%)	21.9%	21.5%	21.4%
[W] = [H] / [U] * 3.81	Final Wet Density (0.1pcf)	128.0	127.1	125.5
[X] = [W] / (1 + [V])	Final Dry Density (0.1pcf)	105.0	104.6	103.4
[Y]= [V]x[K]x[X]/([K]x62.3)-[X]	Final Saturation (0.1%)	98.1%	95.7%	92.0%
[Z] = [X] / [L]	Final Relative Compaction (0.1%)	100.8%	100.5%	99.3%



Cohesion (psf)	210
Friction Angle (degrees)	32

Remarks:
Reviewed by: JF

JD & S Testing

Direct Shear Moisture Content & Density

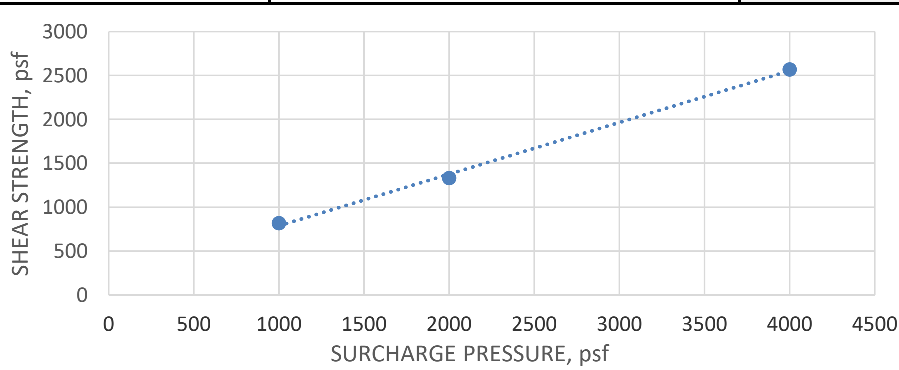
Project Name: 650-700 PCH   Project Number: 185804814

Sampled By: Client   Sample Date: 5/29/2020   Lab #:

Source/Location: P1-10'   Tested By: M.P.

Description: Test Date: 6/21/2020

	Shearing Rate (in./min.)		0.040		
	Normal Pressure (psf)		1000	2000	4000
[A]	Initial Weight of Wet Soil + Ring (0.1g)		196.2	190.4	186.1
[B]	Weight of Ring (0.1g)		45.5	45.2	45.0
[C] = [A] - [B]	Initial Weight of Wet Soil (0.1g)		150.7	145.2	141.1
[D]	Initial Reading	Time	0.0000	0.0000	0.0000
		Time			
		Time			
		Time			
[E]	Final Reading	Time			
[F] = [E] - [D]	Height Change		0.0158	0.0192	0.0143
[G] = 1 - [F]	Final Height		0.9842	0.9808	0.9857
	Shear Strength (psf)		816	1332	2568
[H]	Final Weight of Wet Soil (0.1g)		156.0	152.8	147.0
[J]	Weight of Dry Soil (0.1g)		134.8	131.9	127.1
[K]	Specific Gravity			2.7	
[L]	Average Maximum Dry Density (0.1pcf)			109.2	
[M]	Initial Volume (0.01 cubic inch)		4.60	4.60	4.60
[P] = ([C] - [J]) / [J]	Initial Moisture Content (0.1%)		11.8%	10.1%	11.0%
[Q] = [C] / [M] * 3.81	Initial Wet Density (0.1pcf)		124.8	120.3	116.9
[R] = [Q] / (1 + [P])	Initial Dry Density (0.1pcf)		111.6	109.2	105.3
[S]=[P]x[K]x[R]/([K]x62.3)-[R]	Initial Saturation (0.1%)		70.3%	55.5%	55.2%
[T] = [R] / [L]	Initial Relative Compaction (0.1%)		102.3%	100.1%	96.4%
[U] = [M] X [G]	Final Volume (0.01 cubic inch)		4.53	4.51	4.53
[V] = ([H]-[J]) / [J]	Final Moisture Content (0.1%)		15.7%	15.8%	15.7%
[W] = [H] / [U] * 3.81	Final Wet Density (0.1pcf)		131.3	129.0	123.5
[X] = [W] / (1 + [V])	Final Dry Density (0.1pcf)		113.4	111.4	106.8
[Y]= [V]x[K]x[X]/([K]x62.3)-[X]	Final Saturation (0.1%)		88.0%	83.9%	73.5%
[Z] = [X] / [L]	Final Relative Compaction (0.1%)		103.9%	102.0%	97.8%



Cohesion (psf)	200
Friction Angle (degrees)	30

Remarks: Reviewed by: JF

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

CORROSION TEST RESULTS

Client Name: Stantec Consulting, Inc.
Project Name: 650-700 PCH
Project No.: 185804814

AP Job No.: 20-0619
Date: 06/10/20

Boring No.	Sample Type	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B3	Bulk	-	Silty Sand	9399	8.4	44	33
P1	Bulk	-	Silty Sand	10101	8.2	49	36

NOTES:
Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

APPENDIX C
LIQUEFACTION ANALYSIS

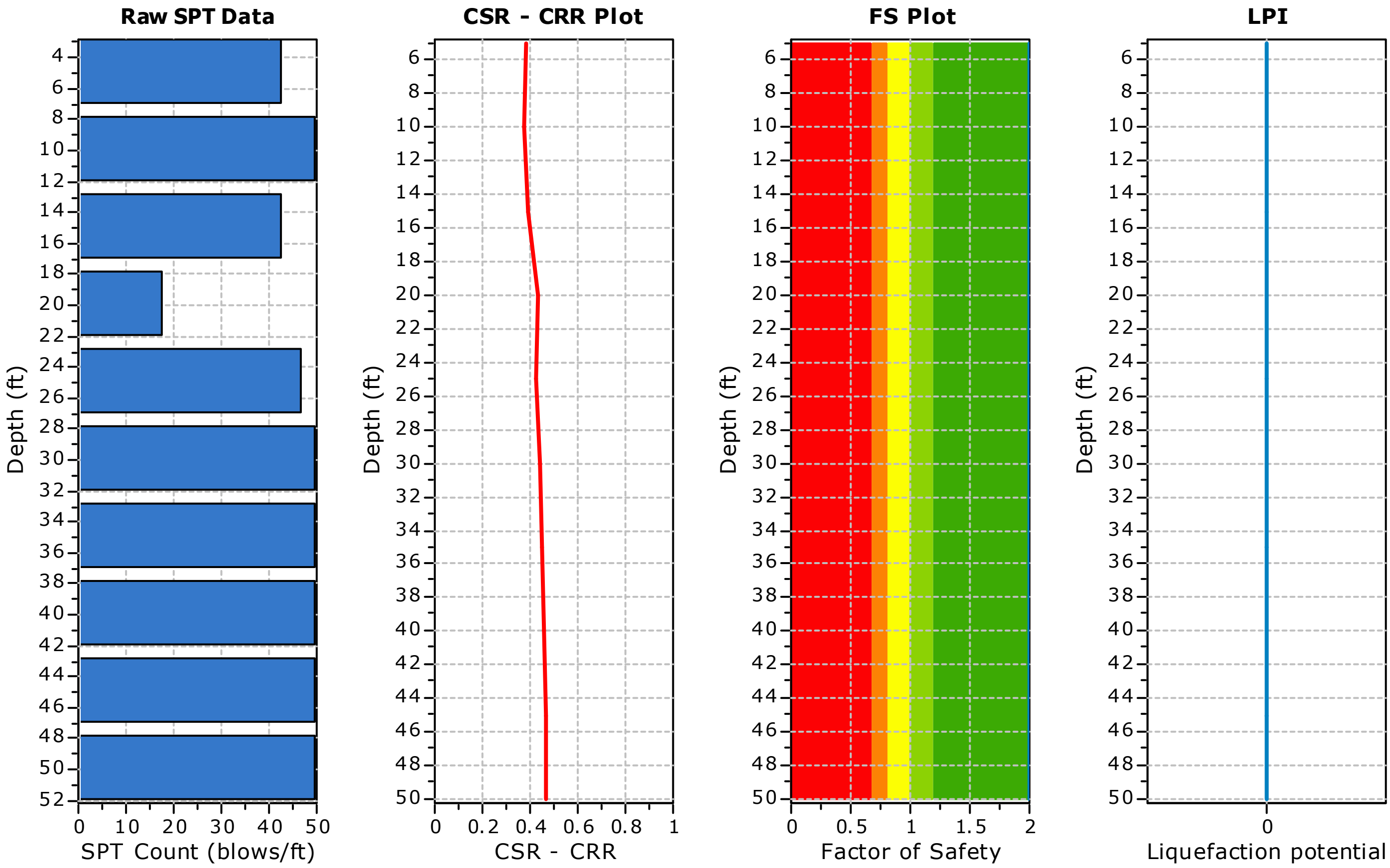
SPT BASED LIQUEFACTION ANALYSIS REPORT

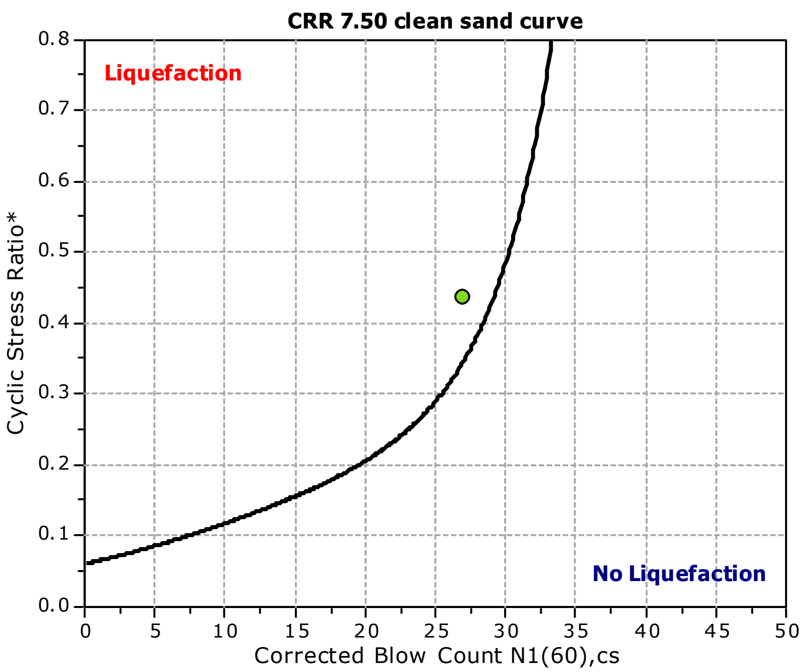
Project title : **SPT Name: P1**

Location :

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	121.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	121.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	6.69
Borehole diameter:	200mm	Peak ground acceleration:	0.88 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.20		





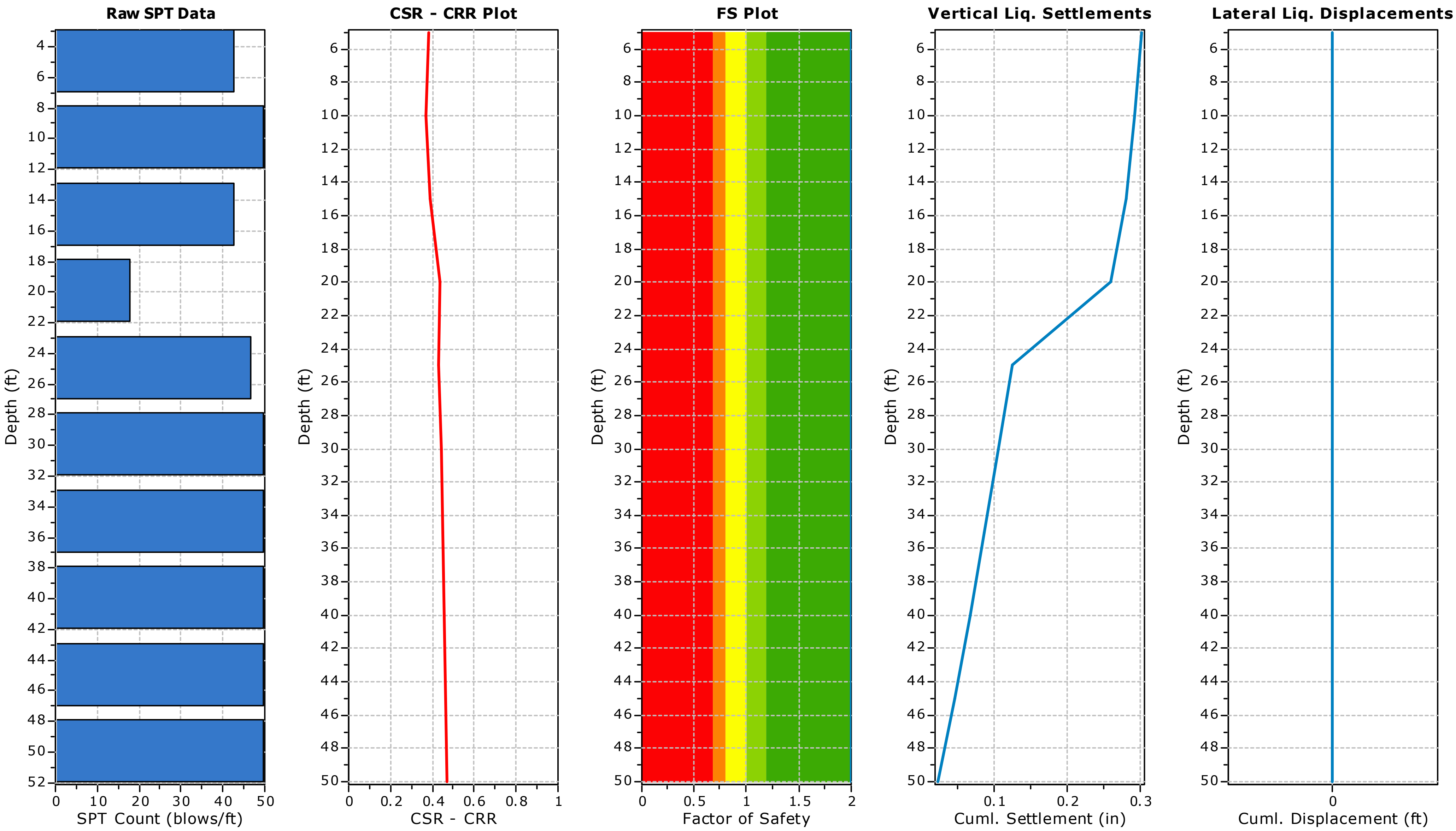
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	43	10.00	130.00	5.00	No
10.00	50	10.00	135.00	5.00	No
15.00	43	10.00	130.00	5.00	No
20.00	18	10.00	120.00	5.00	No
25.00	47	10.00	130.00	5.00	No
30.00	50	10.00	135.00	5.00	No
35.00	50	10.00	135.00	5.00	No
40.00	50	10.00	135.00	5.00	Yes
45.00	50	10.00	135.00	5.00	Yes
50.00	50	8.00	135.00	5.00	Yes

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	43	130.00	0.33	0.00	0.33	0.26	1.36	1.20	1.15	0.75	1.20	73	10.00	1.15	74	4.000
10.00	50	135.00	0.66	0.00	0.66	0.26	1.13	1.20	1.15	0.85	1.20	80	10.00	1.15	81	4.000
15.00	43	130.00	0.99	0.00	0.99	0.26	1.02	1.20	1.15	0.85	1.20	62	10.00	1.15	63	4.000
20.00	18	120.00	1.29	0.00	1.29	0.38	0.93	1.20	1.15	0.95	1.20	26	10.00	1.15	27	4.000
25.00	47	130.00	1.61	0.00	1.61	0.26	0.90	1.20	1.15	0.95	1.20	66	10.00	1.15	67	4.000
30.00	50	135.00	1.95	0.00	1.95	0.26	0.85	1.20	1.15	1.00	1.20	70	10.00	1.15	71	4.000
35.00	50	135.00	2.29	0.00	2.29	0.26	0.82	1.20	1.15	1.00	1.20	68	10.00	1.15	69	4.000
40.00	50	135.00	2.63	0.00	2.63	0.26	0.79	1.20	1.15	1.00	1.20	65	10.00	1.15	66	4.000
45.00	50	135.00	2.96	0.00	2.96	0.26	0.76	1.20	1.15	1.00	1.20	63	10.00	1.15	64	4.000
50.00	50	135.00	3.30	0.00	3.30	0.26	0.74	1.20	1.15	1.00	1.20	61	8.00	0.37	61	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cydic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
5.00	130.00	0.33	0.00	0.33	0.99	1.00	0.566	2.20	74	1.36	0.417	1.10	0.379	2.000 ●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
10.00	135.00	0.66	0.00	0.66	0.97	1.00	0.554	2.20	81	1.36	0.408	1.10	0.371	2.000
15.00	130.00	0.99	0.00	0.99	0.94	1.00	0.540	2.20	63	1.36	0.398	1.02	0.390	2.000
20.00	120.00	1.29	0.00	1.29	0.92	1.00	0.525	1.82	27	1.25	0.422	0.97	0.437	2.000
25.00	130.00	1.61	0.00	1.61	0.89	1.00	0.509	2.20	67	1.36	0.375	0.88	0.428	2.000
30.00	135.00	1.95	0.00	1.95	0.86	1.00	0.492	2.20	71	1.36	0.362	0.82	0.442	2.000
35.00	135.00	2.29	0.00	2.29	0.83	1.00	0.474	2.20	69	1.36	0.349	0.77	0.452	2.000
40.00	135.00	2.63	0.00	2.63	0.80	1.00	0.456	2.20	66	1.36	0.336	0.73	0.459	2.000
45.00	135.00	2.96	0.00	2.96	0.77	1.00	0.439	2.20	64	1.36	0.323	0.70	0.464	2.000
50.00	135.00	3.30	0.00	3.30	0.74	1.00	0.421	2.20	61	1.36	0.311	0.66	0.467	2.000

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d:	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cydic Stress Ratio
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
5.00	73	0.18	0.22	0.88	0.14	12561.74	0.00	0.00	8.56	0.01	5.00	0.010
10.00	80	0.37	0.44	1.29	0.15	8193.48	0.00	0.00	8.56	0.01	5.00	0.011
15.00	62	0.53	0.66	1.45	0.16	6448.49	0.00	0.00	8.56	0.02	5.00	0.021
20.00	26	0.68	0.86	1.25	0.17	5499.61	0.00	0.00	8.56	0.11	5.00	0.134

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
25.00	66	0.82	1.08	1.89	0.19	4804.85	0.00	0.00	8.56	0.02	5.00	0.020
30.00	70	0.96	1.31	2.12	0.20	4287.06	0.00	0.00	8.56	0.02	5.00	0.018
35.00	68	1.08	1.53	2.27	0.21	3895.50	0.00	0.00	8.56	0.02	5.00	0.020
40.00	65	1.20	1.76	2.40	0.23	3586.76	0.00	0.00	8.56	0.02	5.00	0.021
45.00	63	1.30	1.98	2.52	0.24	3335.68	0.00	0.00	8.56	0.02	5.00	0.022
50.00	61	1.39	2.21	2.62	0.25	3126.59	0.00	0.00	8.56	0.02	5.00	0.024

Cumulative settlemetns: 0.302

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

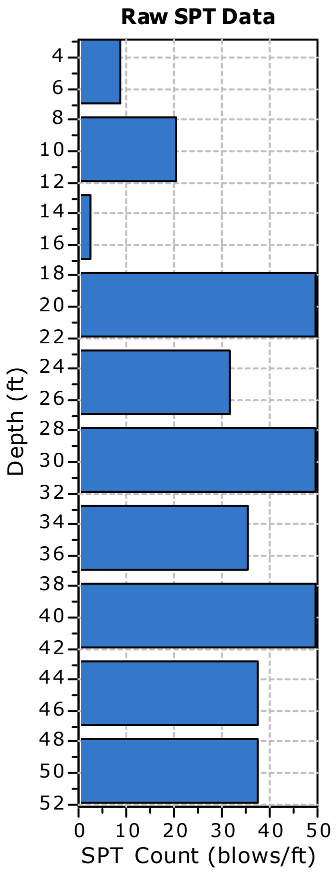
SPT BASED LIQUEFACTION ANALYSIS REPORT

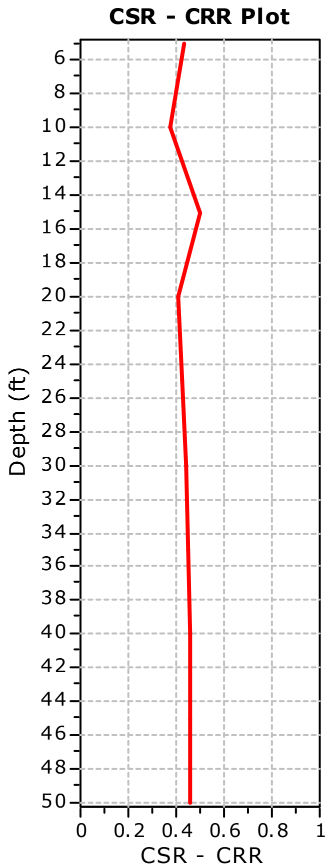
Project title : **SPT Name: P2**

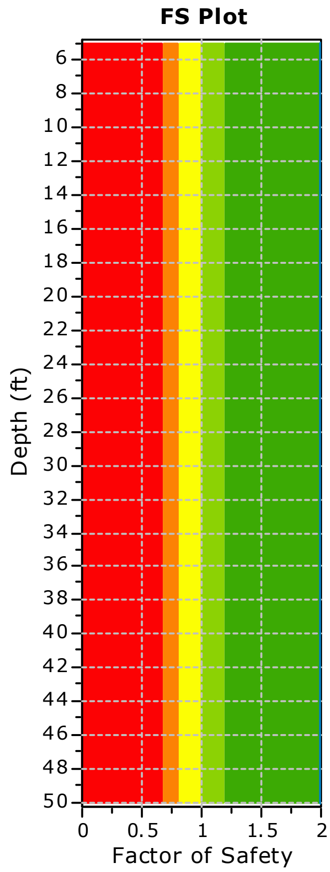
Location :

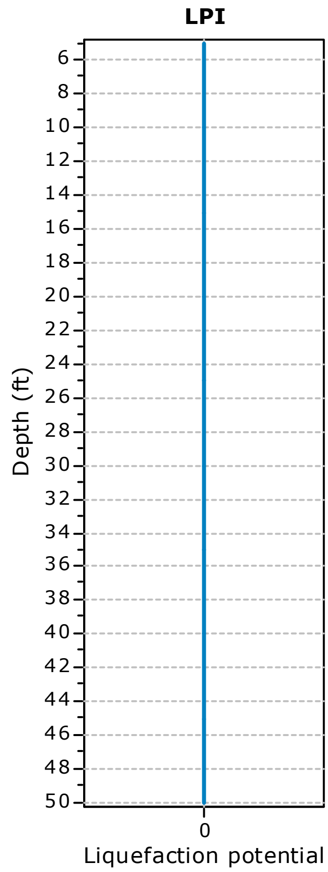
:: Input parameters and analysis properties ::

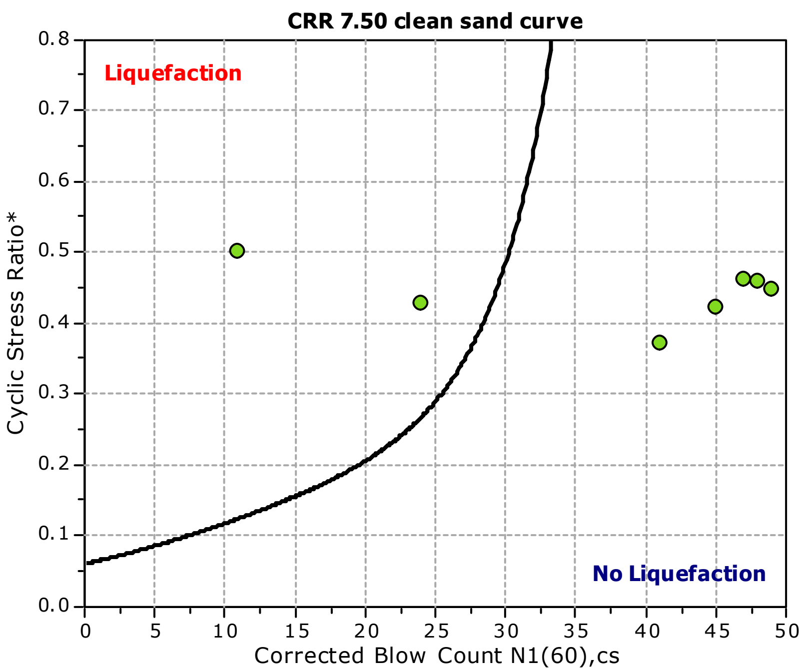
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	121.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	121.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	6.69
Borehole diameter:	200mm	Peak ground acceleration:	0.88 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.20		











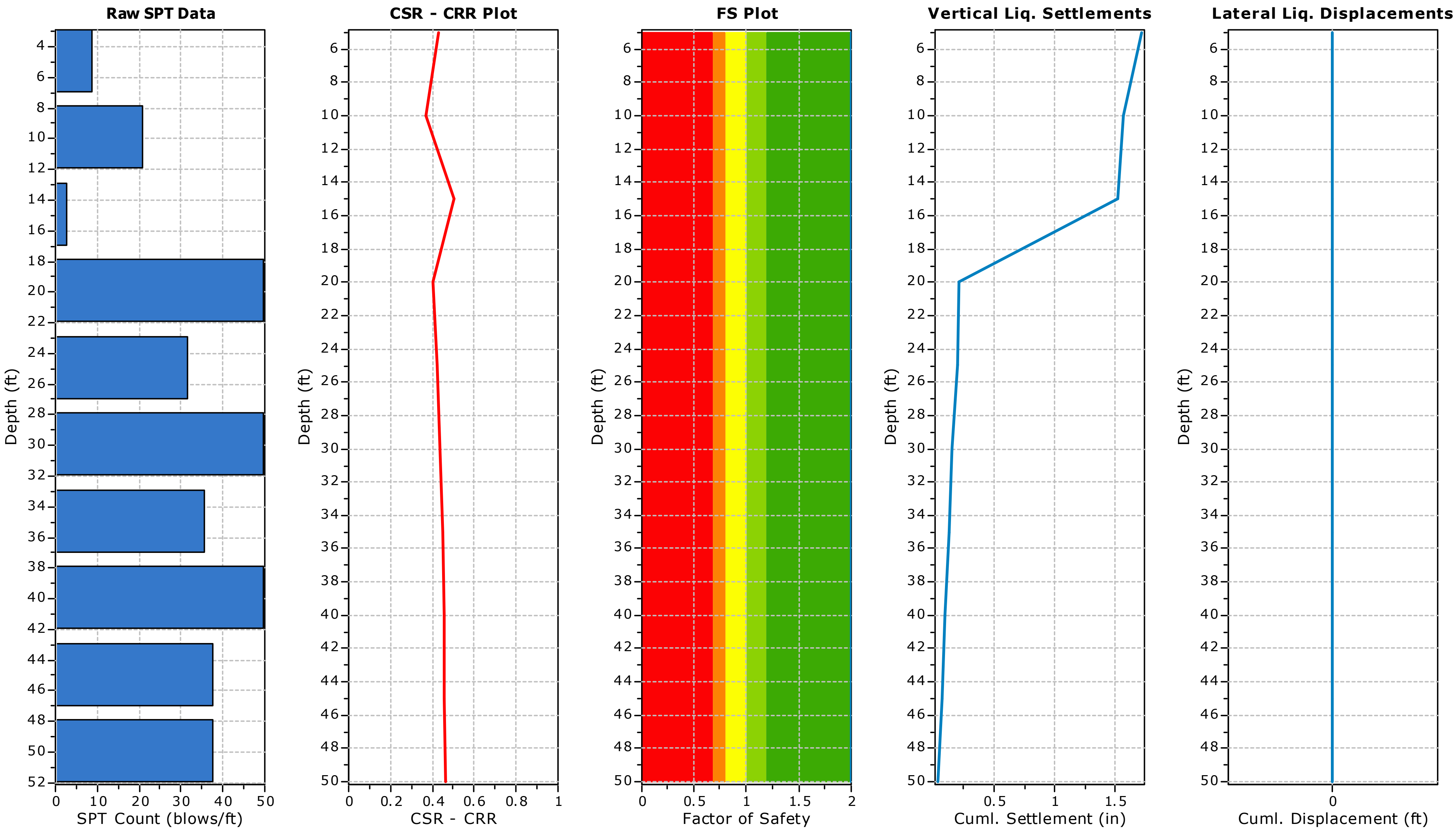
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	9	38.00	120.00	5.00	No
10.00	21	40.00	125.00	5.00	No
15.00	3	51.00	115.00	5.00	No
20.00	50	51.00	135.00	5.00	No
25.00	32	5.00	130.00	5.00	No
30.00	50	5.00	135.00	5.00	No
35.00	36	5.00	130.00	5.00	No
40.00	50	5.00	135.00	5.00	Yes
45.00	38	5.00	130.00	5.00	Yes
50.00	38	7.00	130.00	5.00	Yes

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	9	120.00	0.30	0.00	0.30	0.38	1.62	1.20	1.15	0.75	1.20	18	38.00	5.55	24	4.000
10.00	21	125.00	0.61	0.00	0.61	0.29	1.17	1.20	1.15	0.85	1.20	35	40.00	5.58	41	4.000
15.00	3	115.00	0.90	0.00	0.90	0.53	1.09	1.20	1.15	0.85	1.20	5	51.00	5.61	11	4.000
20.00	50	135.00	1.24	0.00	1.24	0.26	0.96	1.20	1.15	0.95	1.20	75	51.00	5.61	81	4.000
25.00	32	130.00	1.56	0.00	1.56	0.27	0.90	1.20	1.15	0.95	1.20	45	5.00	0.00	45	4.000
30.00	50	135.00	1.90	0.00	1.90	0.26	0.86	1.20	1.15	1.00	1.20	71	5.00	0.00	71	4.000
35.00	36	130.00	2.23	0.00	2.23	0.26	0.82	1.20	1.15	1.00	1.20	49	5.00	0.00	49	4.000
40.00	50	135.00	2.56	0.00	2.56	0.26	0.79	1.20	1.15	1.00	1.20	66	5.00	0.00	66	4.000
45.00	38	130.00	2.89	0.00	2.89	0.26	0.77	1.20	1.15	1.00	1.20	48	5.00	0.00	48	4.000
50.00	38	130.00	3.21	0.00	3.21	0.26	0.75	1.20	1.15	1.00	1.20	47	7.00	0.14	47	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cydic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
5.00	120.00	0.30	0.00	0.30	0.99	1.00	0.566	1.67	24	1.20	0.472	1.10	0.429	2.000 ●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
10.00	125.00	0.61	0.00	0.61	0.97	1.00	0.554	2.20	41	1.36	0.408	1.10	0.371	2.000
15.00	115.00	0.90	0.00	0.90	0.94	1.00	0.540	1.21	11	1.06	0.508	1.02	0.501	2.000
20.00	135.00	1.24	0.00	1.24	0.92	1.00	0.525	2.20	81	1.36	0.387	0.95	0.406	2.000
25.00	130.00	1.56	0.00	1.56	0.89	1.00	0.509	2.20	45	1.36	0.375	0.88	0.424	2.000
30.00	135.00	1.90	0.00	1.90	0.86	1.00	0.492	2.20	71	1.36	0.362	0.83	0.438	2.000
35.00	130.00	2.23	0.00	2.23	0.83	1.00	0.474	2.20	49	1.36	0.349	0.78	0.448	2.000
40.00	135.00	2.56	0.00	2.56	0.80	1.00	0.456	2.20	66	1.36	0.336	0.74	0.455	2.000
45.00	130.00	2.89	0.00	2.89	0.77	1.00	0.439	2.20	48	1.36	0.323	0.70	0.459	2.000
50.00	130.00	3.21	0.00	3.21	0.74	1.00	0.421	2.20	47	1.36	0.311	0.67	0.462	2.000

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d:	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
5.00	18	0.17	0.20	0.58	0.14	13179.75	0.00	0.00	8.56	0.12	5.00	0.146
10.00	35	0.34	0.41	0.99	0.15	8588.48	0.00	0.00	8.56	0.04	5.00	0.045
15.00	5	0.49	0.60	0.77	0.16	6817.65	0.01	0.01	8.56	1.10	5.00	1.316
20.00	75	0.65	0.83	1.76	0.17	5631.87	0.00	0.00	8.56	0.01	5.00	0.013

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
25.00	45	0.80	1.05	1.63	0.18	4896.52	0.00	0.00	8.56	0.04	5.00	0.044
30.00	71	0.93	1.27	2.09	0.20	4354.39	0.00	0.00	8.56	0.02	5.00	0.018
35.00	49	1.05	1.49	2.00	0.21	3960.79	0.00	0.00	8.56	0.03	5.00	0.038
40.00	66	1.17	1.72	2.37	0.22	3639.00	0.00	0.00	8.56	0.02	5.00	0.021
45.00	48	1.27	1.93	2.26	0.24	3387.40	0.00	0.00	8.56	0.03	5.00	0.038
50.00	47	1.35	2.15	2.37	0.25	3177.42	0.00	0.00	8.56	0.03	5.00	0.039

Cumulative settlemetns: 1.718

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

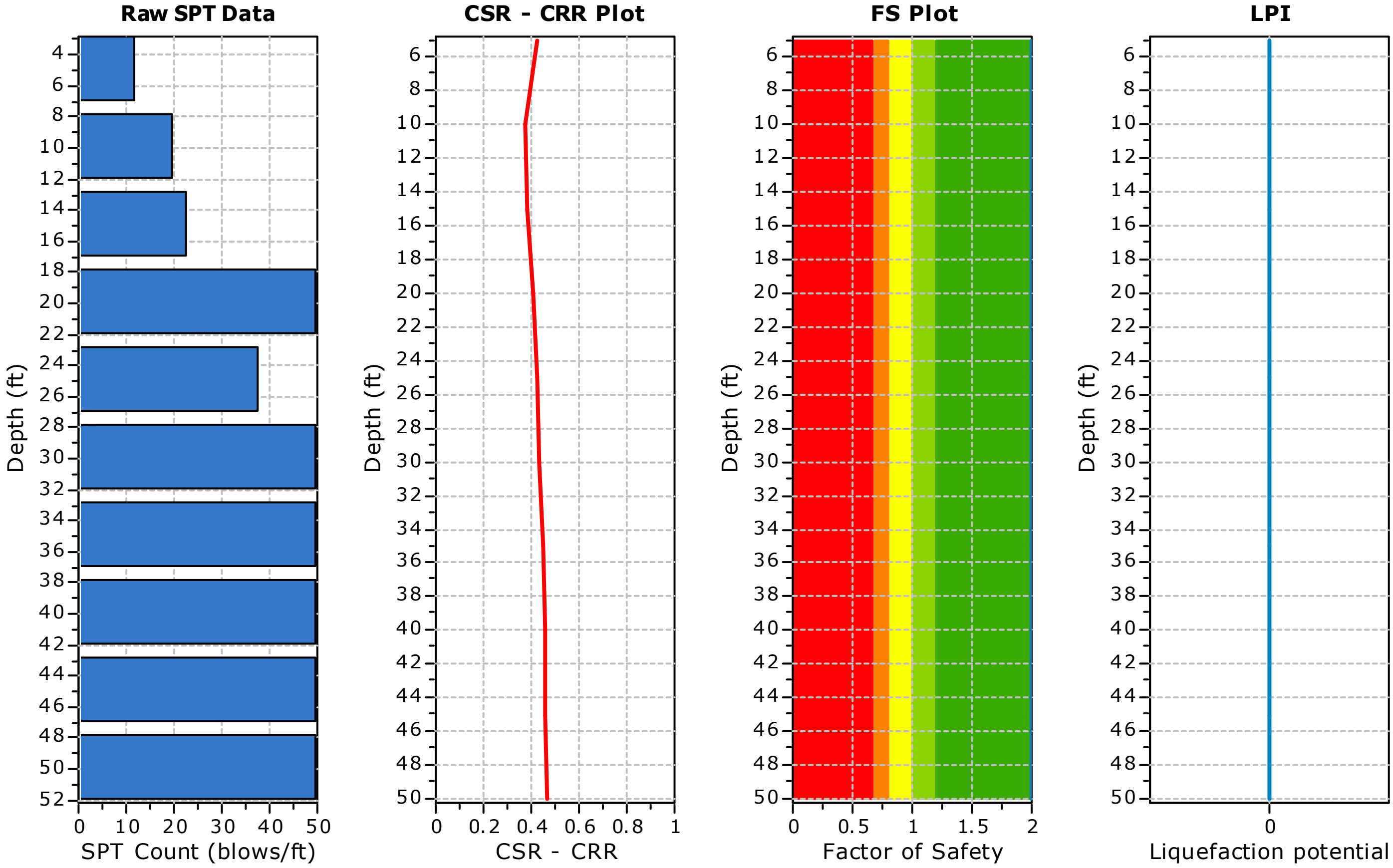
SPT BASED LIQUEFACTION ANALYSIS REPORT

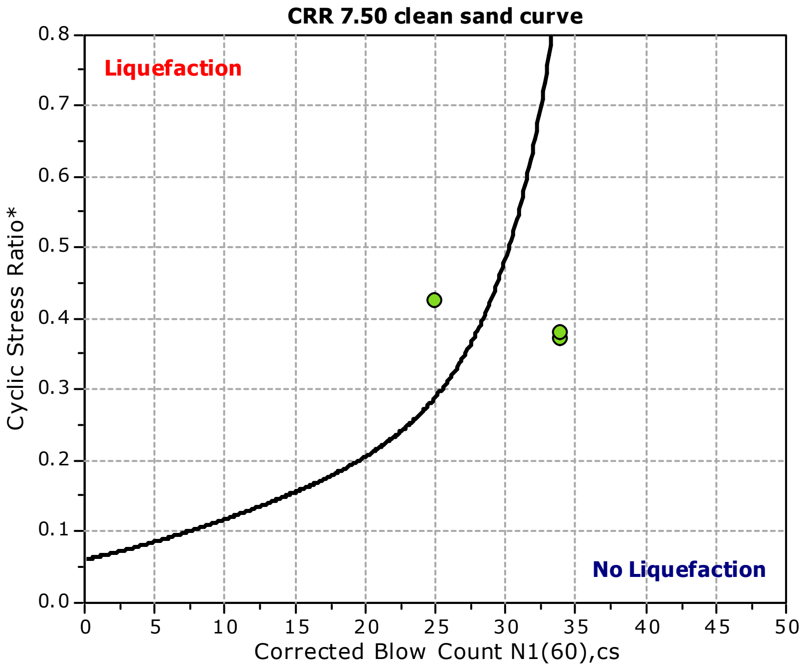
Project title : **SPT Name: P3**

Location :

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	121.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	121.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	6.69
Borehole diameter:	200mm	Peak ground acceleration:	0.88 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.20		





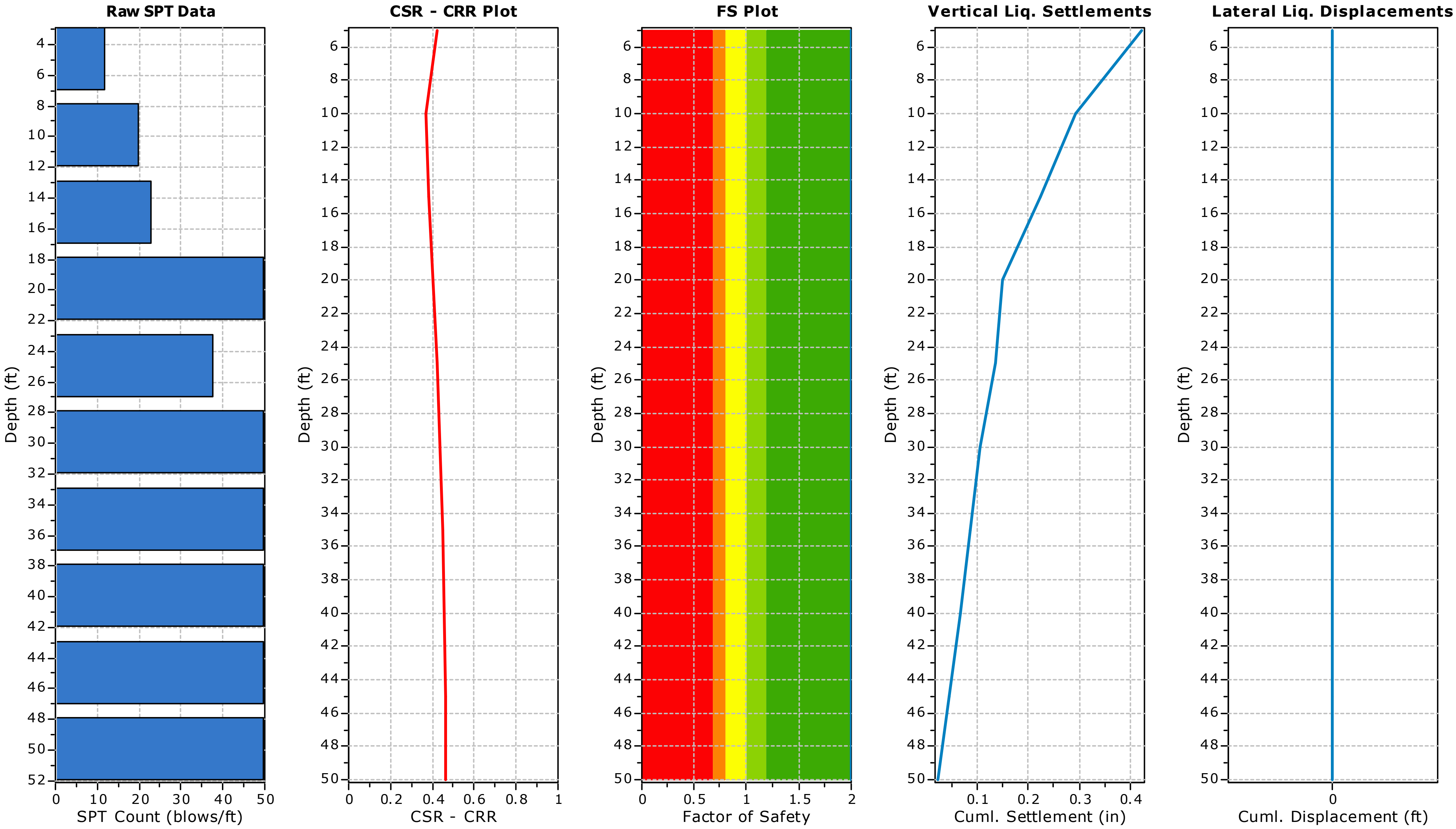
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	12	0.00	115.00	5.00	No
10.00	20	0.00	120.00	5.00	No
15.00	23	0.00	120.00	5.00	No
20.00	50	0.00	135.00	5.00	No
25.00	38	0.00	130.00	5.00	No
30.00	50	0.00	135.00	5.00	No
35.00	50	0.00	135.00	5.00	No
40.00	50	0.00	135.00	5.00	No
45.00	50	0.00	135.00	5.00	No
50.00	50	0.00	135.00	5.00	No

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	12	115.00	0.29	0.00	0.29	0.40	1.68	1.20	1.15	0.75	1.20	25	0.00	0.00	25	4.000
10.00	20	120.00	0.59	0.00	0.59	0.33	1.22	1.20	1.15	0.85	1.20	34	0.00	0.00	34	4.000
15.00	23	120.00	0.89	0.00	0.89	0.33	1.06	1.20	1.15	0.85	1.20	34	0.00	0.00	34	4.000
20.00	50	135.00	1.23	0.00	1.23	0.26	0.96	1.20	1.15	0.95	1.20	76	0.00	0.00	76	4.000
25.00	38	130.00	1.55	0.00	1.55	0.26	0.90	1.20	1.15	0.95	1.20	54	0.00	0.00	54	4.000
30.00	50	135.00	1.89	0.00	1.89	0.26	0.86	1.20	1.15	1.00	1.20	71	0.00	0.00	71	4.000
35.00	50	135.00	2.23	0.00	2.23	0.26	0.82	1.20	1.15	1.00	1.20	68	0.00	0.00	68	4.000
40.00	50	135.00	2.56	0.00	2.56	0.26	0.79	1.20	1.15	1.00	1.20	66	0.00	0.00	66	4.000
45.00	50	135.00	2.90	0.00	2.90	0.26	0.77	1.20	1.15	1.00	1.20	64	0.00	0.00	64	4.000
50.00	50	135.00	3.24	0.00	3.24	0.26	0.75	1.20	1.15	1.00	1.20	62	0.00	0.00	62	4.000

Abbreviations

σ_v:	Total stress during SPT test (tsf)
u_o:	Water pore pressure during SPT test (tsf)
σ'_{vo}:	Effective overburden pressure during SPT test (tsf)
m:	Stress exponent normalization factor
C_N:	Overburden corretion factor
C_E:	Energy correction factor
C_B:	Borehole diameter correction factor
C_R:	Rod length correction factor
C_S:	Liner correction factor
$N_{1(60)}$:	Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$	Equivalent clean sand adjustment
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cydic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
5.00	115.00	0.29	0.00	0.29	0.99	1.00	0.566	1.72	25	1.21	0.466	1.10	0.424	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
10.00	120.00	0.59	0.00	0.59	0.97	1.00	0.554	2.20	34	1.36	0.408	1.10	0.371	2.000	●
15.00	120.00	0.89	0.00	0.89	0.94	1.00	0.540	2.20	34	1.36	0.398	1.04	0.382	2.000	●
20.00	135.00	1.23	0.00	1.23	0.92	1.00	0.525	2.20	76	1.36	0.387	0.96	0.405	2.000	●
25.00	130.00	1.55	0.00	1.55	0.89	1.00	0.509	2.20	54	1.36	0.375	0.89	0.423	2.000	●
30.00	135.00	1.89	0.00	1.89	0.86	1.00	0.492	2.20	71	1.36	0.362	0.83	0.437	2.000	●
35.00	135.00	2.23	0.00	2.23	0.83	1.00	0.474	2.20	68	1.36	0.349	0.78	0.448	2.000	●
40.00	135.00	2.56	0.00	2.56	0.80	1.00	0.456	2.20	66	1.36	0.336	0.74	0.455	2.000	●
45.00	135.00	2.90	0.00	2.90	0.77	1.00	0.439	2.20	64	1.36	0.323	0.70	0.460	2.000	●
50.00	135.00	3.24	0.00	3.24	0.74	1.00	0.421	2.20	62	1.36	0.311	0.67	0.464	2.000	●

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d:	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cydic Stress Ratio
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
5.00	25	0.16	0.19	0.57	0.14	13520.64	0.00	0.00	8.56	0.11	5.00	0.130
10.00	34	0.33	0.39	0.91	0.15	8805.93	0.00	0.00	8.56	0.06	5.00	0.069
15.00	34	0.48	0.59	1.12	0.16	6875.10	0.00	0.00	8.56	0.06	5.00	0.074
20.00	76	0.64	0.82	1.72	0.17	5666.28	0.00	0.00	8.56	0.01	5.00	0.015

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
25.00	54	0.79	1.04	1.72	0.18	4920.18	0.00	0.00	8.56	0.03	5.00	0.030
30.00	71	0.93	1.26	2.08	0.20	4371.67	0.00	0.00	8.56	0.02	5.00	0.018
35.00	68	1.05	1.49	2.23	0.21	3960.79	0.00	0.00	8.56	0.02	5.00	0.020
40.00	66	1.17	1.72	2.37	0.22	3639.00	0.00	0.00	8.56	0.02	5.00	0.021
45.00	64	1.27	1.94	2.49	0.24	3378.63	0.00	0.00	8.56	0.02	5.00	0.022
50.00	62	1.36	2.17	2.61	0.25	3162.67	0.00	0.00	8.56	0.02	5.00	0.023

Cumulative settlemetns: 0.421

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

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APPENDIX D
PERCOLATION TEST DATA

PERCOLATION TEST DATA SHEET

Project:	650 PCH	Project No.	185804814	Date:	6/1/2020
Test Hole No.	P1	Tested By:	RW		
Depth of Test Hole, D_T:	50 ft	USCS Soil Classification	SM		
Test Hole Dimensions (inches)			Length	Width	
Diameter (if round)	8	Sides (if rectangular)			
Sandy Soil Test Criteria*					

Trial No.	Water Run time	Settle time	Initial Depth of Water (ft)	Final Depth of Water (ft)
1	30 min	10 min	3.2	0.0
2	30 min	10 min	3.2	0.0

*If two consecutive measurements show that twelve inches of water seeps away in less than 30 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) for a minimum of four hours before running test. Obtain at least eight measurements per hole over at least four hours (approximately 30 minute intervals) with a precision of at least 1/8" until the last 3 consecutive readings are within 10% of each other.

Trial No.	Δt, Time Interval, (min)	D_o, Initial Depth of Water (in)	D_f, Final Depth of Water (in)	ΔD, Change in Water Level (in.)	Percolation Rate (in/hr)
1	10	37.4	0.0	37.4	224.4
2	10	40.8	0.0	40.8	244.8
3	10	40.8	0.0	40.8	244.8
4	10	40.5	0.00	40.5	243.0
5	10	40.8	0.00	40.8	244.8
6	10	40.5	0.00	40.5	243.0
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8					
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12					
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Comments: I_t = 40.1 inches/hr (see attached spreadsheet) => Assume FS = 3, I_{tall} = 13.4 inches/hour:

Porchet Method - Conversion of Percolation Rate to Infiltration Rate	Perc Test No. P1	**Legend:**	Required Entries / Calculated Cells

Company Name: Stantec Consulting Services Inc — Date: 6/20/2020

Designed by: J. Fischer — County/City Case No: Los Angeles

Percolation Conversion to Infiltration Rate

The conversion equation is used:

$$I_t\,(in/hr) = \frac{\Delta H(in) \times 60\,(min/hr) \times r(in)}{\Delta t(min) \times [r(in) + 2H_{avg}(in)]}$$

If test hole is round - Enter radius here → $r =$ 8.00 inches

If test hole is square - Enter average side width below

$w =$ 0.00 inches — $r_{eq} =$ 0.00 inches

Time interval	$\Delta t =$	10.0	minutes
Initial height of water during selected time interval	$H_o =$	40.50	inches
Final height of water during selected time interval	$H_f =$	0.00	inches
Change in height of water during selected time interval	$\Delta H =$	40.50	inches
Average head height over the selected time interval	$H_{avg} =$	20.25	inches
Converted infiltration rate per test data	$I_t =$	40.08	inches/hour

Comments

PERCOLATION TEST DATA SHEET

Project:	650 PCH	Project No.	185804814	Date:	6/1/2020
Test Hole No.	P2	Tested By:	RW		
Depth of Test Hole, D_T:	51 ft	USCS Soil Classification	SW		

Test Hole Dimensions (inches)		Length	Width		
Diameter (if round)	8	Sides (if rectangular)			

Sandy Soil Test Criteria*

Trial No.	Water Run time	Settle time	Initial Depth of Water (ft)	Final Depth of Water (ft)
1	30 min	10 min	2.7	0.0
2	30 min	10 min	2.5	0.0

*If two consecutive measurements show that twelve inches of water seeps away in less than 30 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) for a minimum of four hours before running test. Obtain at least eight measurements per hole over at least four hours (approximately 30 minute intervals) with a precision of at least 1/8" until the last 3 consecutive readings are within 10% of each other.

Trial No.	Δt, Time Interval, (min)	D_o, Initial Depth of Water (in)	D_f, Final Depth of Water (in)	ΔD, Change in Water Level (in.)	Percolation Rate (in/hr)
1	10	30.0	0.0	30	180.0
2	10	30.0	0.0	30	180.0
3	10	26.4	0.0	26.4	158.4
4	10	30.3	0.00	30.3	181.8
5	10	30.0	0.00	30	180.0
6	10	31	0.00	31	186.0
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15					

Comments: I_t = 38.2 inches/hour (see attached spreadsheet) => Assume FS = 3, I_{tall} = 12.7 inches/hour

Porchet Method - Conversion of Percolation Rate to Infiltration Rate	Perc Test No. P2	**Legend:**	Required Entries / Calculated Cells

Company Name: Stantec Consulting Services Inc — Date: 6/20/2020

Designed by: J. Fischer — County/City Case No: Los Angeles

Percolation Conversion to Infiltration Rate

The conversion equation is used:

$$I_t\,(\text{in/hr}) = \frac{\Delta H(\text{in}) \times 60\ (\text{min/hr}) \times r(\text{in})}{\Delta t(\text{min}) \times [r(\text{in}) + 2H_{avg}(\text{in})]}$$

If test hole is round - Enter radius here →	$r =$	8.00	inches
If test hole is square - Enter average side width below			
$w =$ 0.00 inches	$r_{eq} =$	0.00	inches
Time interval	$\Delta t =$	10.0	minutes
Initial height of water during selected time interval	$H_o =$	31.00	inches
Final height of water during selected time interval	$H_f =$	0.00	inches
Change in height of water during selected time interval	$\Delta H =$	31.00	inches
Average head height over the selected time interval	$H_{avg} =$	15.50	inches
Converted infiltration rate per test data	$I_t =$	38.15	inches/hour

Comments

PERCOLATION TEST DATA SHEET						
Project:	650 PCH	Project No.	185804814	Date:	6/1/2020	
Test Hole No.	P3	Tested By:	RW			
Depth of Test Hole, D_T:	50 ft	USCS Soil Classification	SC			
Test Hole Dimensions (inches)		Length	Width			
Diameter (if round)	8	Sides (if rectangular)				

Sandy Soil Test Criteria*

Trial No.	Water Run time	Settle time	Initial Depth of Water (ft)	Final Depth of Water (ft)
1	30 min	10 min	5.6	0.0
2	30 min	10 min	5.6	0.0

*If two consecutive measurements show that twelve inches of water seeps away in less than 30 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) for a minimum of four hours before running test. Obtain at least eight measurements per hole over at least four hours (approximately 30 minute intervals) with a precision of at least 1/8" until the last 3 consecutive readings are within 10% of each other.

Trial No.	Δt, Time Interval, (min)	D_o, Initial Depth of Water (in)	D_f, Final Depth of Water (in)	ΔD, Change in Water Level (in.)	Percolation Rate (in/hr)
1	10	67.2	0.0	67.2	403.2
2	10	76.8	0.0	76.8	460.8
3	10	86.4	0.0	86.4	518.4
4	10	87	0.00	87	522.0
5	10	86.8	0.00	86.8	520.8
6	10	87	0.00	87	522.0
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12					
13					
14					
15					

Comments: It = 44 inches/hour (see attached spreadsheet) => Assume FS = 3, Itall = 14.7 inches/hour

Porchet Method - Conversion of Percolation Rate to Infiltration Rate

Perc Test No.	P3	**Legend:**	Required Entries / Calculated Cells
Company Name:	Stantec Consulting Services Inc	Date:	6/20/2020
Designed by:	J. Fischer	County/City Case No:	Los Angeles

Percolation Conversion to Infiltration Rate

The conversion equation is used:

$$I_t\,(\text{in/hr}) = \frac{\Delta H(\text{in}) \times 60\,(\text{min/hr}) \times r(\text{in})}{\Delta t(\text{min}) \times [r(\text{in}) + 2H_{avg}(\text{in})]}$$

If test hole is round - Enter radius here →	$r =$	8.00	inches
If test hole is square - Enter average side width below			
$w =$ 0.00 inches	$r_{eq} =$	0.00	inches
Time interval	$\Delta t =$	10.0	minutes
Initial height of water during selected time interval	$H_o =$	87.00	inches
Final height of water during selected time interval	$H_f =$	0.00	inches
Change in height of water during selected time interval	$\Delta H =$	87.00	inches
Average head height over the selected time interval	$H_{avg} =$	43.50	inches
Converted infiltration rate per test data	$I_t =$	43.96	inches/hour

Comments