Appendix GEO

Geotechnical Evaluation

Geotechnical Investigation Report

BlueWave Car Wash
929 Oakland Road
San Jose, California 95661

Prepared for:
Stantec
7660 Woodway Drive, Suite 400
Houston, Texas

Prepared by:
Stantec Consulting, Inc.
Rocklin, California

August 15, 2017

Stantec Consulting Inc.
3875 Atherton Road, Rocklin CA 95765

August 15, 2017

Attention: Ms. Amanda Mullen
Stantec Consulting Services Inc.
7660 Woodway Drive, Suite 400
Houston, Texas 77063

Reference: Geotechnical Investigation Report
BlueWave Carwash
929 Oakland Road
San Jose, California

Dear Ms. Mullen,

This letter transmits Stantec's geotechnical investigation report for the proposed BlueWave Carwash located in San Jose, California. The purpose of this report is to evaluate the subsurface conditions and provide geotechnical recommendations for design and construction of the project.

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

Respectfully,

STANTEC CONSULTING SERVICES INC.

REGISTERED PROFESSIONAL ENGINEER
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1.0 INTRODUCTION

This report presents the results of Stantec's geotechnical investigation for the proposed Blue Wave Carwash facility located in San Jose, California. The project location is shown on the USGS Quadrangle Map, Figure 1.

1.1 PROPOSED DEVELOPMENT

The proposed carwash facility is shown on the Subsurface Exploration Map, Figure 2. The property has an approximate triangular shape, with a long dimension parallel to Oakland Road of approximately 300 feet (north-south), and a short dimension parallel to Horning Street of approximately 160 feet (east-west).

The site plan provided to us indicates the carwash building will have plan dimensions of approximately 40 feet by 80 feet, and will be located on the east-central portion of the property adjacent to Oakland Road. The grading plan for the project has not yet been developed. The project team anticipates grading will be relatively minor with cuts and fills less than approximately 2 feet.

Based on our discussions with the project structural engineer (Stantec, Sacramento), we understand the carwash building will be a single-story concrete structure, and that the office portion of the carwash building will be constructed with a slab on grade. Perimeter wall footings for the building will have dead plus live loads of approximately 2 kips per foot.

1.2 PURPOSE AND SCOPE OF WORK

1.2.1 Purpose

The purpose of this report is to evaluate the subsurface conditions and provide geotechnical recommendations for design and construction of the carwash facility. 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 four hollow stem auger borings and advancing one dynamic cone penetration (DCP) test,
- Perform geotechnical laboratory tests on selected samples,
- Perform geotechnical engineering analyses, and
- Preparation of this geotechnical investigation report for the proposed project.

2.0 FIELD INVESTIGATION

2.1 PRE-DRILLING PROCEDURES

Underground Service Alert (USA) was notified before commencing drilling activities to identify underground utilities that could conflict with the proposed borings and DCP. In addition, a private utility locator was retained to clear each of the boring locations for potential conflicts with underground utilities. Prior to drilling, the top 5 feet at each boring location was investigated in the field for potential conflicts with marked or observed utility lines and other obstructions.

2.2 HOLLOW STEM AUGER DRILLING

Four test borings (B-1 through B-4) were drilled using a truck mounted, CME 75 drill rig equipped with a hollow-stem auger on July 13, 2017, by Cascade Drilling, L.P. (Cascade) to a maximum depth of 44 feet. The boring locations are shown on the Subsurface Exploration Map, Figure 2, and the Subsurface Exploration Image, Figure 3. The borings were logged by a Stantec field engineer who also collected samples of the materials encountered for examination and laboratory testing.

2.3 PERCOLATION TESTING

One of the borings was converted to a shallow percolation well (P-1) and percolation testing was performed in the well by a Stantec field engineer on July 14, 2017 in general accordance with Santa Clara County percolation testing guidelines.

2.4 DYNAMIC CONE PENETRATION TESTING

One DCP (DCP-1) was performed in general accordance with ASTM D6951M – 09(2015) - Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications. The DCP test was performed at boring location B-4 as shown on Figure 2. The DCP test results are presented in Appendix C.

2.5 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 borings logs. Cascade Drilling informed us that the average hammer energy efficiency on the drill rig used at the project was 80% based on the latest results (June 15, 2017).

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). The boring logs are presented in Appendix A.

2.6 BORING BACKFILL

Upon completion, borings were backfilled with cement grout using the tremmie method in accordance with applicable regulatory guidelines.

3.0 LABORATORY TESTING

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

- In-Situ Moisture and Density (ASTM D2216)
- Sieve Analysis (ASTM D422 and ASTM C136)
- Percent Passing No. 200 Sieve (ASTM D1140)
- Atterberg Limits (ASTM D 4318)
- R Value (California Test 301)
- Chemical Tests for Corrosion Potential (CA DOT test methods)

The laboratory test results are presented in Appendix B.

4.0 GEOLOGIC SETTING AND SITE CONDITIONS

4.1 REGIONAL GEOLOGY

The site is located in the northern portion of the Great Valley Geomorphic Province in the Northern California. This region is characterized as a 50-mile-wide and 400-mile-long sediment filled trough in which the sediments have been deposited, almost continuously, since the Jurassic period. The site resides in the portion of the Province drained by the Sacramento River.

Regional geologic maps (Dibblee, 2007) indicate the site is underlain by Quaternary age Holocene alluvial deposits. Geologic mapping presented in the California Geological Survey Seismic Hazard Zone Report (Knudson et al.) indicate the site is underlain by Holocene alluvial fan deposits.

4.2 SURFACE CONDITIONS

The site is located at 949 Oakland Road, just north of Madera Boulevard, in San Jose, California. The site is bound by Madera Avenue to the south, Oakland Road to the east, a paved parking lot and residential properties to the west, and a paved commercial storage yard to the north. The site has an approximate triangular shape, with the long dimension parallel to Oakland Road (approximately 300 feet in a north-south direction), and a short dimension parallel to Madera Avenue (approximately 160 feet in an east-west direction). Site photographs are presented on Figure 4.

Several single-story buildings exist on the site. One of the existing buildings is partially located within the footprint of the proposed carwash building. Concrete pavement, and unpaved ground exists on the remainder of the site. The site is generally flat. Based on Google Earth®, the ground surface at the site is at an approximate elevation of 61 feet to 62 feet (WGS84 Datum).

4.3 SUBSURFACE CONDITIONS

The materials encountered in our borings consist of undocumented fill and Quaternary age Holocene alluvial deposits. A brief description of the subsurface conditions is provided in this section. Detailed descriptions of the subsurface conditions are provided on the boring logs included in Appendix A.

Undocumented Fill - Undocumented fill was encountered in all of the borings completed at the site, and extends to depths of approximately 1 to 1.5 feet below ground surface. The fill consists of dense, dry, silty gravel with sand (Unified Soil Classification Symbol: GM). This fill material appears to be imported, base course. Concrete debris was encountered at the base of the fill in boring B-2. The DCP test results indicate this fill material is dense.

Alluvial Deposits - Quaternary age (Holocene) alluvial deposits underlies the undocumented fill in all of our borings. The upper portion (approximately 10 to 11 feet deep) of the alluvial deposits consists of very loose to medium dense, dark yellowish brown to olive, moist, silty sand (SM), clayey sand (SC), poorly graded sand (SP), and sandy silt (ML). The DCP test results indicate the alluvium in the upper 3 feet at boring B-4 is loose to medium dense.

The lower portion (below 11-foot depth) of the alluvium generally consists of soft to stiff, brown, dark yellowish brown to olive, moist to wet, lean clay (CL), sandy lean clay (CL), and lean clay with sand (CL). An approximate 3-foot thick layer of loose, non-plastic silt (ML) was encountered in one boring between depths of approximately 13 feet and 16 feet below ground surface (bgs).

Groundwater - Groundwater was encountered at a depth of approximately 19 feet bgs in the borings performed for this investigation. The historic high groundwater level based on the California Geological Survey Seismic Hazard Zone Report is approximately 10 feet. Groundwater levels may fluctuate in the future due to rainfall, irrigation, broken pipes, or changes in site drainage.

5.0 GEOLOGIC HAZARDS

5.1 FAULTING AND SURFACE FAULT RUPTURE

The site is located in a seismically active area. The estimated distance from the site to nearby mapped active faults is presented in the table below.

Table 1. Faults in Site Vicinity

Fault	Distance (miles) [1]	Maximum Moment Magnitude [1]
Hayward-Rodgers Creek	6.8	6.8
Calaveras	7.2	6.5
Monte Vista-Shannon	11.9	6.5
North San Andreas	24.7	7.2

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

As noted above, the closest known active fault is the Hayward-Rodgers Creek Fault, located approximately 6.8 miles northeast of the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. No active faults are known to underlie or project toward the site. Therefore, the probability of surface fault rupture 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 subject site. The seismic parameters in accordance with the 2016 California Building Code (CBC) are presented below:

Table 2. 2016 CBC Seismic Parameters and Peak Ground Acceleration

Parameter	Value
Site Coordinates	Latitude 37.36043° Longitude -121.89137°
Mapped Spectral Acceleration Value at Short Period: S_s	1.5g
Mapped Spectral Acceleration Value at 1-Second Period: S_1	0.6g
Site Classification	D
Short Period Site Coefficient: F_a	1.000
1-Second Period Site Coefficient: F_v	1.500
Site Class Adjusted Acceleration Value at Short Period: S_{MS}	1.5g
Site Class Adjusted Acceleration Value at 1-Second Period: S_{M1}	0.9g
Design Spectral Response Acceleration at Short Periods: S_{DS}	1.0g
Design Spectral Response Acceleration at 1-Second Period: S_{D1}	0.6g
Peak Ground Acceleration adjusted for Site Class Effects: PGA_M	0.5g

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 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 located in a California Geological Survey Liquefaction Hazard Zone (CGS, 2002). 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.

Medium stiff to stiff cohesive soil is generally present below the historic high groundwater level at the site. However, two relatively thin, granular layers (10 feet to 11 feet bgs in borings B-1 and B-2, and 13 feet to 16 feet bgs in boring B-2) are present below the historic high groundwater level which are potentially susceptible to liquefaction. We estimate total liquefaction induced settlements from these thin layers and from dry sand settlement above the historic high groundwater table would be in the order of approximately 2 inches if these layers were to liquefy.

5.4 LIQUEFACTION INDUCED LATERAL SPREADING

Liquefaction induced lateral spreading can occur in areas of sloping ground, or towards a free face. Since the topography at the site is relatively flat, the potential for liquefaction induced lateral spreading is considered low.

5.5 LANDSLIDES AND SLOPE STABILITY

The site is located in a relatively flat valley and there are no historic landslides mapped at the site. Evidence of landslides or slope instabilities was not observed within the immediate vicinity during our site activities. Accordingly, the potential for landslides or slope instabilities to occur at the site is considered low.

5.6 FLOODING, TSUNAMIS AND SEICHES

The site is not located within a flood zone or a mapped area on the State of California Tsunami Inundation Maps (Cal EMA, 2009); therefore, damage due to flooding or tsunamis is considered low. Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The site is not located immediately adjacent to any lakes or confined bodies of water; therefore, the potential for a seiche to affect the site is considered low.

5.7 EXPANSIVE SOILS

The near-surface soils (upper approximate 10 feet) consist of non-expansive granular soil. Accordingly, mitigation for expansive soils is not considered necessary at this site.

6.0 CONCLUSIONS

Based on our field exploration, laboratory testing and engineering and geologic analyses, it is our opinion that the subject property is suitable for the proposed development from a geotechnical engineering and engineering geology viewpoint; however, there are existing geotechnical conditions associated with the site that will warrant mitigation and/or consideration during the planning stages. The main geotechnical conclusions for the project are presented in the following paragraphs.

- The site is underlain by shallow undocumented granular fill (approximately 1.5 feet thick) and Quaternary age Holocene alluvial deposits (alluvium). Concrete debris was encountered within the fill in one of the borings (B-4). The Alluvium that underlies the fill generally consists of very loose to medium dense granular soil in the upper 11 feet, and soft to stiff cohesive soil to a depth of at least 44 feet below ground surface.

- Groundwater was encountered at a depth of approximately 19 feet bgs in the borings performed for this investigation. The historic high groundwater level based on the California Geological Survey Seismic Hazard Zone Report is approximately 10 feet.

- Several layers of the alluvium have a high potential for liquefaction if subjected to the design seismic event based on the historic high groundwater level. Total seismic settlement of the liquefiable layers and the loose non-liquefiable layers are estimated to be approximately 2 inches. Differential seismic settlement is estimated to be approximately 1 inch over a distance of 30 feet. This magnitude of seismic settlement combined with static settlement can typically not be accommodated by a conventional shallow foundation system. Accordingly, we recommend a stiffened foundation system combined with removal and re-compaction of soil within the upper 5 feet below finished grade. Other mitigation options such as deep foundations or deeper ground improvement are also available. However, these options will likely not be economically feasible for this project.

- It is anticipated that the existing soil may be reused as compacted fill provided construction debris and oversized material is removed.

7.0 RECOMMENDATIONS

7.1 EARTHWORK

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

Should Stantec not be retained as the geotechnical consultant during construction, Stantec would cease to be the Geotechnical Engineer of Record (Geotechnical Engineer) for this project. In accordance with the industry standard of practice, the consultant selected to perform the observation and testing during construction would assume the responsibility of Geotechnical Engineer and would be required to accept the recommendations of the approved geotechnical report or provide a new report as the controlling document for the earthwork construction. In addition, the new consultant should formally accept the responsibility as the Geotechnical Engineer in a written letter to the owner.

7.1.1 Site Preparation

Site preparation should begin with the removal of existing buildings, pavement, buried slabs and foundations, vegetation, highly organic soil, leach lines, septic tanks, and any other unsuitable materials. Existing underground utilities within the proposed construction areas should be completely removed and/or rerouted. Grading should conform to the guidelines presented in the 2016 California Building Code (CBC, 2016), as well as the requirements of the city of San Jose and Santa Clara County.

7.1.2 Remedial Grading

Building Area:

The upper portion of the alluvium is generally loose to medium dense, considered moderately compressible, and not suitable for support of foundation loading in its present condition.

In order to provide adequate and uniform support for the proposed carwash building, removal of the existing soils to a minimum depth of 2 feet below the bottom of the proposed footings, or 5 feet below existing site grade, whichever is deeper, is recommended. Removal, replacement, and compaction should be completed below the entire building footprint, and laterally at least five feet beyond the outside edge of the building perimeter unless constrained by existing structures or improvements.

The bottom of the excavation should be scarified to a depth of 8-inches, moisture conditioned to within 0 to 2 percentage points above the optimum moisture content and compacted to at least 90% relative compaction based on the ASTM D1557 laboratory test procedure. All references to optimum moisture content and relative compaction in this report are based on this test method.

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 18 inches below final subgrade elevation, whichever is lower. The soil exposed at the base of the excavation should be scarified to a depth of 8 inches, and moisture conditioned to within 0 to 2 percentage points above the optimum moisture content. Hardscape subgrade should be compacted to at least 90% relative compaction. Pavement subgrade should be compacted to at least 95% relative compaction.

Field Observations:

The Geotechnical Engineer should check the bottom of excavations. If soft, loose, wet, or otherwise unsuitable soils are encountered, the depth of removal may be extended, or additional recommendations can be provided.

7.1.3 Fill Placement and Compaction

Excavated materials determined by the Geotechnical Engineer to be satisfactory can be reused as compacted fill. It is anticipated that the majority of the excavated materials can be re-used as compacted fill soils. The fill material in the upper 1.5 feet appears to be base course. Consideration should be given to stockpiling this material and reusing it for support of pavements if it meets the project criteria for base course. The existing concrete pavement on the site may be reused within the compacted fill provided all debris such as rebar is removed and the concrete is broken down or crushed to a maximum 3-inch size. The concrete pieces should be thoroughly mixed with finer soil such that nesting of coarse fragments which could result in voids does not occur. The Geotechnical Engineer should approve the fill material before placement.

Fill should be placed in 6- to 8-inch thick loose, horizontal lifts, moisture conditioned to within 0 to 2 percentage points above the optimum moisture content, and compacted to at least 90% relative compaction. 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 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 adjacent to the pipe.

7.1.5 Expansive Soil

The onsite materials have a low expansion potential. The grading and foundation recommendations presented in this report reflect a low expansion potential.

7.1.6 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 of less than 20 or a Plasticity Index less than 15. Imported materials should be reviewed and approved by the Geotechnical Engineer before being brought to the site.

7.1.7 Site Excavation Characteristics

It is anticipated that excavations can be achieved with conventional earthwork equipment in good working order.

7.1.8 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.

7.1.9 Temporary Excavations

The existing fill 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 1½:1 (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.

Stantec 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.

7.1.10 Dewatering

Groundwater seepage may occur locally due to broken pipes, local irrigation or following heavy rain. Dewatering can be accomplished by sloping the excavation bottom to a sump and pumping from the sump. A layer of gravel about 6 inches thick placed in the bottom of the excavation will facilitate groundwater flow and can be used as a working platform.

7.1.11 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.

7.1.12 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 Foundations

Due to the combination of static and seismic settlement, we recommend the new building should be constructed with a stiffened foundation system in combination with a recompacted fill pad

below the structure. Other mitigation options such as deep foundations or deeper ground improvement are also available. However, these other options will likely not be economically feasible for this project. A stiffened foundation system may consist of a mat foundation, or conventional shallow foundations connected with grade beams in both horizontal directions. The intent of the stiffened foundation system is to reduce the risk of collapse. Significant structural building damage may still occur as the result of a significant seismic event.

The following foundation recommendations are minimum criteria based on geotechnical concerns. 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 2,500 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:	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.

Reinforced grade beams should connect the footings in both horizontal directions throughout all portions of the structure so that the entire foundation system moves as a unit.

Mat Foundations:

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. 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 vertical modulus of subgrade reaction, K_{v1}, of 100 pounds per cubic inch. The vertical modulus of subgrade reaction should be adjusted for the plan dimensions of the mat.

7.2.1.1 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: 2-inches total settlement
1 inch differential settlement over 30 feet

Seismically induced settlements are typically erratic, and difficult to predict. The seismic settlement estimates provided above are rough estimates based on commonly accepted analytical methods.

7.2.2 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 0.35 can be used. Passive pressure can be computed using an allowable lateral pressure of 300 psf per foot of depth below the ground surface for level ground conditions. Reductions for sloping ground should be made. The passive pressure can be increased by one-third when considering the total of all loads, including wind or seismic forces. The upper 1 foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs.

7.2.3 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.

7.2.4 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

7.3.1 Interior Slabs on Grade

The project structural engineer should design the interior concrete slabs-on-grade floor. However, we recommend a minimum thickness of 5 inches and minimum reinforcement of #3 bars on 18 inch centers, each way.

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 architect and structural engineer 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 3.

Table 3. 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

7.3.2 Exterior Slabs on Grade

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 should select the final joint patterns.

7.4 CORROSIVITY

One sample of the onsite soils was 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 sample tested had a soluble sulfate concentration of 0.028 percent, which indicates the sample has a negligible 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 sample tested had a chloride concentration of 0.024 percent, which indicates the sample has a negligible chloride corrosion potential relative to metal.
- The sample tested had a minimum resistivity of 400 ohm-cm, which indicates the sample is extremely corrosive.
- The sample tested had a pH of 7.34, which indicates the sample is slightly alkaline.

Other samples at the site could yield significantly different concentrations than those described above. Therefore, additional testing may be performed to further evaluate corrosion during the planning stages and to evaluate the as-graded corrosion potential of the onsite soils after site grading. We do not practice in the field of corrosion engineering. Evaluation by a corrosion engineer should be performed if deemed necessary.

7.5 PAVEMENT

7.5.1 Asphalt Concrete Pavement

An R-value of 40 has been assumed for preliminary design of pavement sections based on one laboratory test 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 40, the following pavement structural sections have been calculated.

Table 4. Flexible Pavement Sections

Traffic Type	Traffic Index	Asphalt Concrete (inches)	Aggregate Base* (inches)
Automobile Parking	5.0	3	4
Automobile Drive Lanes	5.5	3	6
Medium Truck Traffic	6.0	4	5
Heavy Truck Traffic	7.0	4	7

*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 5. The following design parameters were used in our analyses.

Table 5. Concrete Pavement Parameters

Design Parameter	Value
Modulus of Subgrade Reaction (k)	150 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 6. Recommended Concrete Pavement Sections

Traffic Type	Pavement Thickness (inches)
Automobile Parking and Driveways (TC = A)	6.0
Heavy Truck Traffic and Fire Lane Areas (TC = C)	6.5

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. Rigid concrete pavement should be placed in driveway entrance aprons, trash bin loading/storage areas, or other areas of the project site as desired. 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 95% of the laboratory maximum.

7.6 PERCOLATION TESTING

Percolation testing was performed in one boring (B-3) in general accordance with the County of Santa Clara Department of Environmental Health guidelines as described in their On-Site Systems Manual (Santa Clara County, 2014).

The percolation test was performed in a 12-inch diameter, 5 foot deep boring. Pre-soaking was performed the day prior to percolation testing. The stabilized percolation rate from the three final consecutive tests was measured as 1.25 inches per hour or 48 minutes per inch which corresponds to a slow to moderate percolation rate (un-factored) based on Santa Clara County guidelines. The percolation test data is presented in Appendix C.

Santa Clara County requirements for infiltration include applying a safety factor to the percolation rate, provision of a minimum separation between groundwater and the percolation device, and testing of percolation rate during the wet season in some circumstances. We note the following relative to these requirements.

- A safety factor has not been applied to the percolation rates provided in this report. The project civil engineer should determine the appropriate safety factor to apply to the test results.
- The percolation test was not performed during the wet weather season. The percolation rate may decrease in the wet season.
- The separation from the bottom of the percolation test to the groundwater level at the time of testing was approximately 14 feet. This separation will likely decrease during the wet season.
- Santa Clara County provides additional guidelines and requirements in their On-Site Systems Manual including setbacks from various features.

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 this boring 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 any 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 check 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.

8.0 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.

9.0 REFERENCES

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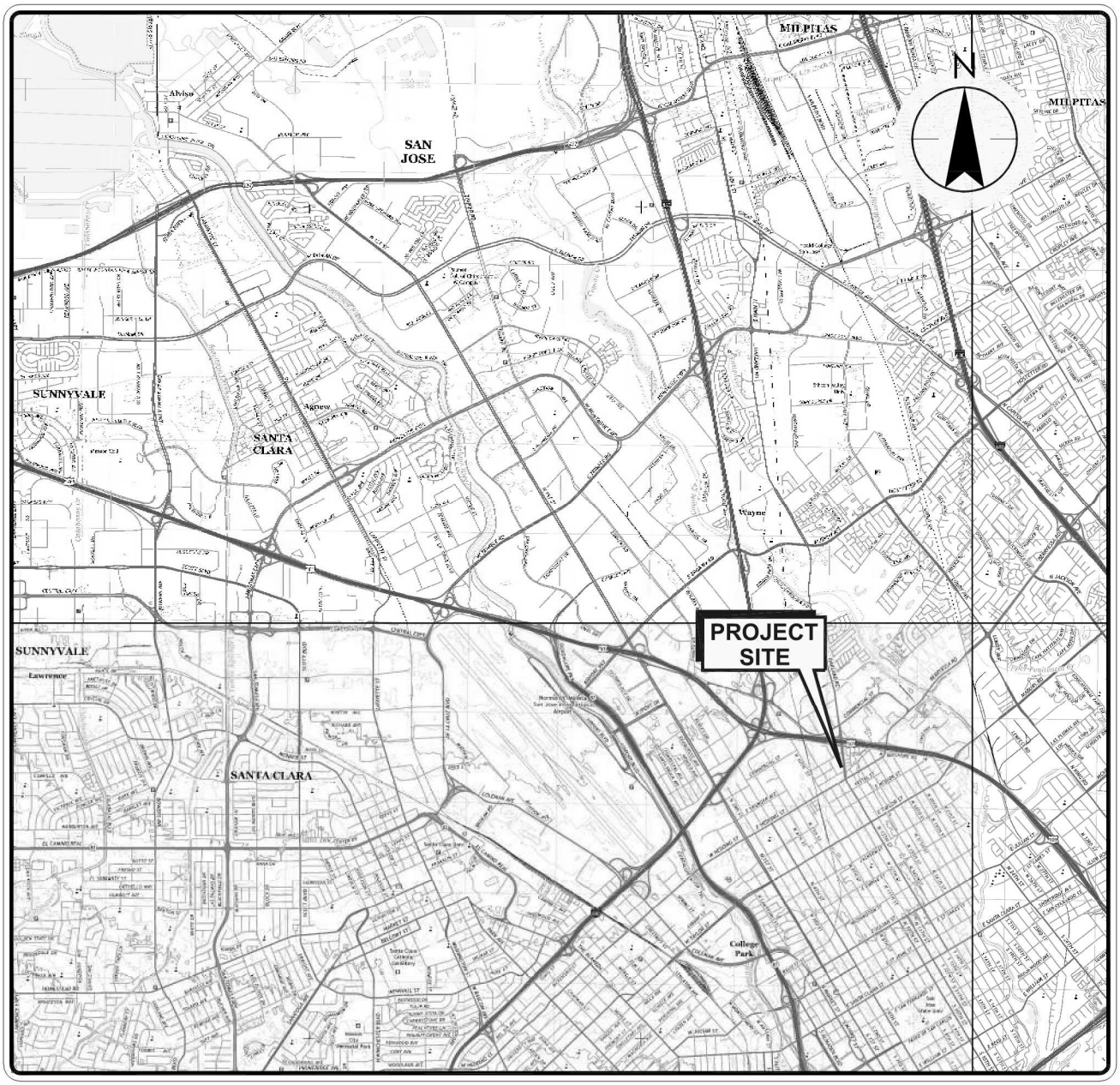
FIGURES



QUADRANGLE LOCATION

1	2	3
4		5
6	7	8

ADJOINING QUADRANGLES

1 Mountain View
2 Milpitas
3 Calaveras Reservoir
4 Cupertino
5 San Jose East
6 Castle Rock Ridge
7 Los Gatos
8 Santa Teresa Hills

0 6000 12000

1 INCH = 6000 FEET

Reference: U.S.G.S., 2015 San Jose West, 2015 San Jose East, 2015 Milpitas, 2015 Calaveras Reservoir, California Quadrangles. 7.5–Minute Topographic Map.

Note: Coordinate system; NAD 83 California State Planes, Zone III (Ft).

ORIGINAL SHEET - ANSI A

July, 2017
222310633

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Client/Project

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929 Oakland Road
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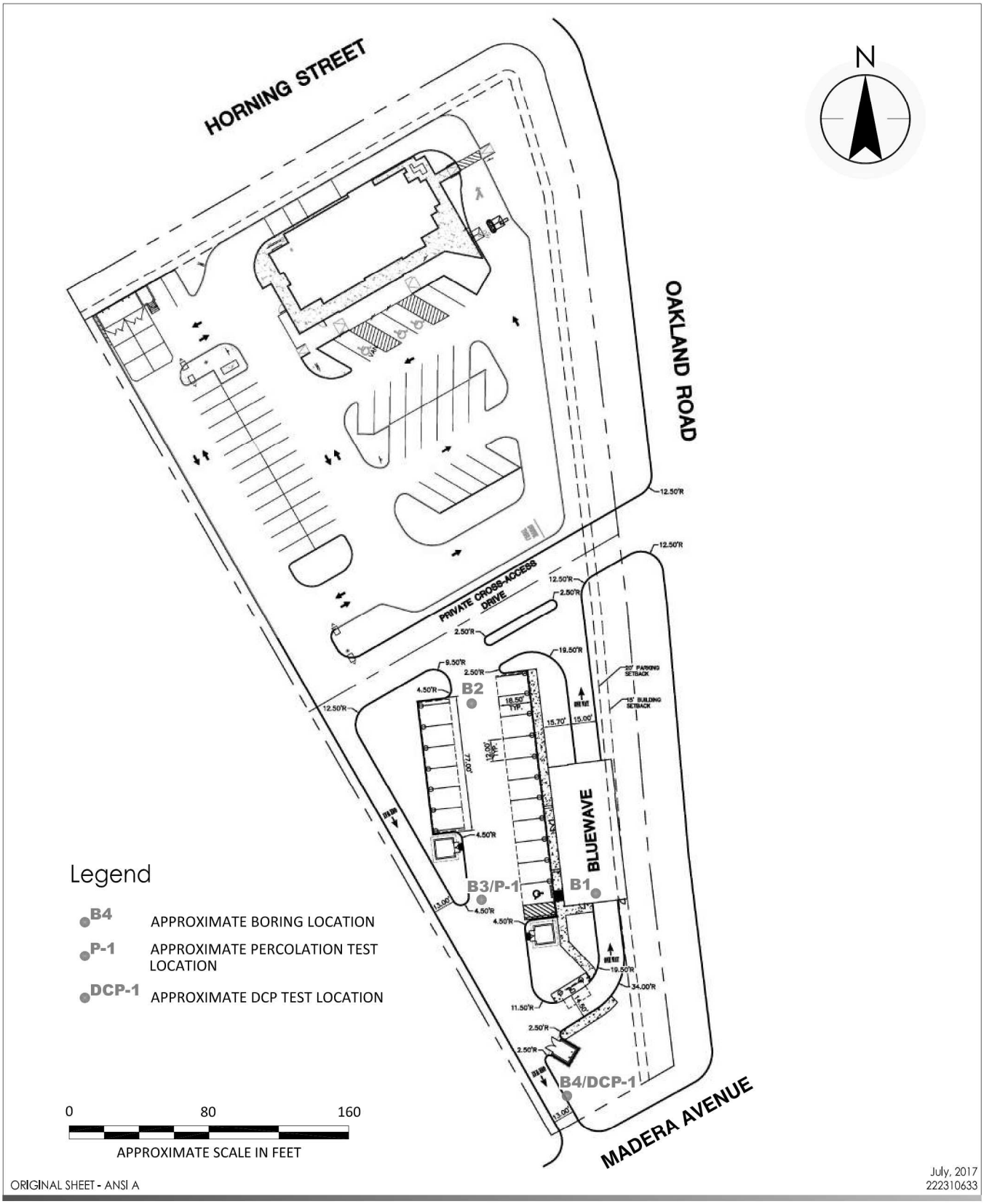
Figure No.

1

Title

USGS QUADRANGLE MAP



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SUBSURFACE EXPLORATION MAP

Legend

- B4 APPROXIMATE BORING LOCATION
- P-1 APPROXIMATE PERCOLATION TEST LOCATION
- DCP-1 APPROXIMATE DCP TEST LOCATION

N

0 80 160

APPROXIMATE SCALE IN FEET

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July, 2017
222310633

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Figure No.

3

Title

SUBSURFACE EXPLORATION IMAGE

Photograph A - Looking north-east towards project site.

Photograph B - Looking north towards project site.

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Figure No.

4

Title

SITE PHOTOGRAPHS

APPENDIX A
BORIING LOGS AND DCP DATA

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

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

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DRILLING: STARTED **1/1/06** COMPLETED: **1/1/06**
INSTALLATION: STARTED **1/1/06** COMPLETED: **1/1/06**
DRILLING COMPANY: **Drilling Sub-contractor**
DRILLING EQUIPMENT: **Drilling Equipment**
DRILLING METHOD: **Drilling Method**
SAMPLING EQUIPMENT: **Sampling Equipment**

NORTHING (ft):
EASTING (ft):
LATITUDE:
LONGITUDE:
GROUND ELEV (ft):
TOC ELEV (ft):
INITIAL DTW (ft): **NE**
BOREHOLE DEPTH (ft): **25.0**
STATIC DTW (ft): **NE**
WELL DEPTH (ft): **25.0**
WELL CASING DIAMETER (in): **NA**
BOREHOLE DIAMETER (in):
LOGGED BY: **Onsite Technician** CHECKED BY: **Project Eng.**

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

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

Terminology describing soil structure:

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

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

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

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

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

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

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

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

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ROCK DESCRIPTION

Terminology describing rock quality:

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

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

Terminology describing rock mass:

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

Terminology describing rock strength:

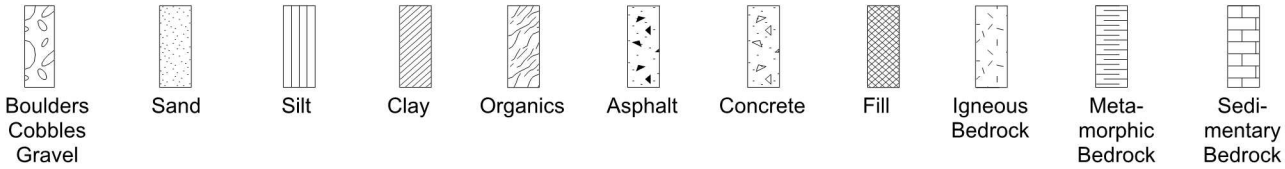
Strength Classification	Unconfined Compressive Strength (MPa)
Extremely Weak	< 1
Very Weak	1 – 5
Weak	5 – 25
Medium Strong	25 – 50
Strong	50 – 100
Very Strong	100 – 250
Extremely Strong	> 250

Terminology describing rock weathering:

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

STRATA PLOT

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



SAMPLE TYPE

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

WATER LEVEL MEASUREMENT

measured in standpipe, piezometer, or well

inferred

RECOVERY

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

N-VALUE

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

DYNAMIC CONE PENETRATION TEST (DCPT)

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

OTHER TESTS

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

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

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PROJECT: **Blue Wave - San Jose** LOCATION: **949 Oakland Road, San Jose, California** PROJECT NUMBER: **222310633**	WELL / PROBEHOLE / BOREHOLE NO: **B1** PAGE 1 OF 2
DRILLING: STARTED **7/13/17** COMPLETED: **7/13/17** INSTALLATION: STARTED COMPLETED: DRILLING COMPANY: **Cascade Drilling** DRILLING EQUIPMENT: **CME 75** DRILLING METHOD: **Hollow Stem Auger** SAMPLING EQUIPMENT: **Split Spoon**	NORTHING (ft): **1528890.330** EASTING (ft): **6593252.670** LATITUDE: **36° 11' 37.2437"** LONGITUDE: **-120° 23' 34.7708"** GROUND ELEV (ft): **62** TOC ELEV (ft): INITIAL DTW (ft): **NE** BOREHOLE DEPTH (ft): **44** STATIC DTW (ft): **19.5 7/13/17** WELL DEPTH (ft): **---** WELL CASING DIAMETER (in): **---** BOREHOLE DIAMETER (in): **6** LOGGED BY: **M. Sapp** CHECKED BY: **M. Amendolagine**

Time & Depth (feet)	USCS	Description	Sample	Time Sample ID	Measured Recov. (feet)	Blow Count (per foot)	Headspace PID (units)	Depth (feet)	Borehole Backfill
		4" Concrete.							Capped with 4" Concrete.
	GM	**FILL:** **SILTY GRAVEL WITH SAND** ; GM; 50% fine gravel; 35% fine to coarse grained sand; 15% non-plastic fines; dry; dense							
	SM	**ALLUVIUM:** **SILTY SAND** ; SM; 10YR 3/4 dark yellowish brown; 8% fine to coarse gravel; 68% fine to coarse grained sand; 24% non-plastic fines; moist; loose to medium dense		B1-2/5 Bulk					
5				14:28 B1-5		2 2 3		5	
	ML	**SANDY SILT** ; ML; 10YR 3/4 dark yellowish brown; 30% very fine to fine grained sand; 70% non-plastic fines; moist; medium stiff							
	SM	**SILTY SAND** ; SM; 10YR 3/4 dark yellowish brown; 70% very fine to fine grained sand; 30% non-plastic fines; moist; loose		14:38 B1-7		9 6 6			
	SC	**CLAYEY SAND** ; SC; 10YR 3/4 dark yellowish brown; 14% fine to coarse gravel; 69% fine to coarse grained sand; 17% medium plasticity fines; moist, loose							
10	SP-SM	**POORLY GRADED SAND WITH SILT** ; SP-SM; 10YR 4/4 brown; 88% very fine to medium grained sand; 12% non-plastic fines; moist; loose		14:42 B1-10		4 4 2		10	
	CL	**SANDY LEAN CLAY** ; CL; 10YR 3/4 dark yellowish brown; 40% very fine to fine grained sand; 60% medium plasticity fines; moist; medium stiff							Backfilled with hydrated bentonite.
15	CL	**LEAN CLAY WITH SAND** ; CL; 10YR 3/4 dark yellowish brown; 20% very fine to fine grained sand; 80% medium plasticity fines; moist; medium stiff		14:48 B1-15		6 4 5		15	
20	CL	**LEAN CLAY** ; CL; 10YR 3/3 dark brown; 10% very fine grained sand; 90% medium plasticity fines; wet; stiff		14:53 B1-20		3 5 9		20	

PROJECT: **Blue Wave - San Jose** LOCATION: **949 Oakland Road, San Jose, California** PROJECT NUMBER: **222310633**	WELL / PROBEHOLE / BOREHOLE NO: **B1** PAGE 2 OF 2
DRILLING: STARTED **7/13/17** COMPLETED: **7/13/17** INSTALLATION: STARTED COMPLETED: DRILLING COMPANY: **Cascade Drilling** DRILLING EQUIPMENT: **CME 75** DRILLING METHOD: **Hollow Stem Auger** SAMPLING EQUIPMENT: **Split Spoon**	NORTHING (ft): **1528890.330** EASTING (ft): **6593252.670** LATITUDE: **36° 11' 37.2437"** LONGITUDE: **-120° 23' 34.7708"** GROUND ELEV (ft): **62** TOC ELEV (ft): INITIAL DTW (ft): **NE** BOREHOLE DEPTH (ft): **44** STATIC DTW (ft): **19.5 7/13/17** WELL DEPTH (ft): **---** WELL CASING DIAMETER (in): **---** BOREHOLE DIAMETER (in): **6** LOGGED BY: **M. Sapp** CHECKED BY: **M. Amendolagine**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Measured Recov. (feet)	Blow Count (per foot)	Headspace PID (units)	Depth (feet)	Borehole Backfill
25			No Recovery at 25' bgs.		15:01 B1-25		2 4 6		25	
30		CL	**LEAN CLAY WITH SAND** ; CL; 2.5Y 3/3 dark olive brown; 20% very fine to coarse grained sand; 80% medium plasticity fines; moist; stiff		15:10 B1-30		3 7 8		30	
35		CL	**LEAN CLAY** ; CL; 2.5Y 3/3 dark olive brown; 3% very fine to fine grained sand; 97% medium plasticity fines; moist; stiff		15:21 B1-35		6 10 12		35	Backfilled with hydrated bentonite.
40			5Y 4/1 dark gray		15:41 B1-42.5		5 7 9		40	
			Borehole terminated at 44 feet bgs.							

PROJECT: **Blue Wave - San Jose** LOCATION: **949 Oakland Road, San Jose, California** PROJECT NUMBER: **222310633**	WELL / PROBEHOLE / BOREHOLE NO: **B2** PAGE 1 OF 1

Stantec

DRILLING: STARTED **7/13/17** COMPLETED: **7/13/17**	NORTHING (ft): **1528992.259** EASTING (ft): **6593150.371**
INSTALLATION: STARTED COMPLETED:	LATITUDE: **36° 11' 38.2526"** LONGITUDE: **-120° 23' 36.017"**
DRILLING COMPANY: **Cascade Drilling**	GROUND ELEV (ft): **61** TOC ELEV (ft):
DRILLING EQUIPMENT: **CME 75**	INITIAL DTW (ft): **NE** BOREHOLE DEPTH (ft): **22**
DRILLING METHOD: **Hollow Stem Auger**	STATIC DTW (ft): **19 7/13/17** WELL DEPTH (ft): **---**
SAMPLING EQUIPMENT: **Split Spoon**	WELL CASING DIAMETER (in): **---** BOREHOLE DIAMETER (in): **6**
	LOGGED BY: **M. Sapp** CHECKED BY: **M. Amendolagine**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Measured Recov. (feet)	Blow Count (per foot)	Headspace PID (units)	Depth (feet)	Borehole Backfill
			2" Gravel.							Capped with 4" Concrete.
		GM	**FILL:** **SILTY GRAVEL WITH SAND** ; GM; 50% fine gravel; 35% fine to coarse grained sand; 15% non-plastic fines; dry							
			4" Layer of Concrete.							
		SM	**ALLUVIUM:** **SILTY SAND** ; SM; 10YR 3/4 dark yellowish brown; 5% coarse gravel; 70% very fine to coarse grained sand; 25% non-plastic fines; moist; loose to medium dense							
5		SM	**SILTY SAND** ; SM; 10YR 3/4 dark yellowish brown; 3% coarse gravel; 75% fine to coarse grained sand; 22% non-plastic fines; moist; loose		12:13 B2-5		2 2 3		5	
			80% fine to medium grained sand; 20% non-plastic fines below 7'.		12:24 B2-7		9 5 4			
10		SM	**SILTY SAND** ; SM; 10YR 4/4 brown; 53% very fine to fine grained sand; 47% non-plastic fines; moist; very loose		12:33 B2-10		3 2 2		10	Backfilled with hydrated bentonite.
		SC	**SANDY LEAN CLAY** ; SC; 2.5Y 3/3 dark olive brown; 40% very fine to fine grained sand; 60% medium plasticity fines; moist; soft							
15		ML	**SILT** ; ML; 10YR 4/4 brown; 10% very fine to medium grained sand; 90% non-plastic fines; moist; medium stiff		12:39 B2-15		2 4 5		15	
20		CL	**LEAN CLAY** ; CL; 2.5Y 3/3 dark olive brown; 5% very fine grained sand; 95% medium plasticity fines; wet; medium stiff		12:47 B2-20		3 3 3		20	
			Borehole terminated at 21.5 feet bgs.							

PROJECT: **Blue Wave - San Jose** LOCATION: **949 Oakland Road, San Jose, California** PROJECT NUMBER: **222310633**	WELL / PROBEHOLE / BOREHOLE NO: **B3/P-1** PAGE 1 OF 1

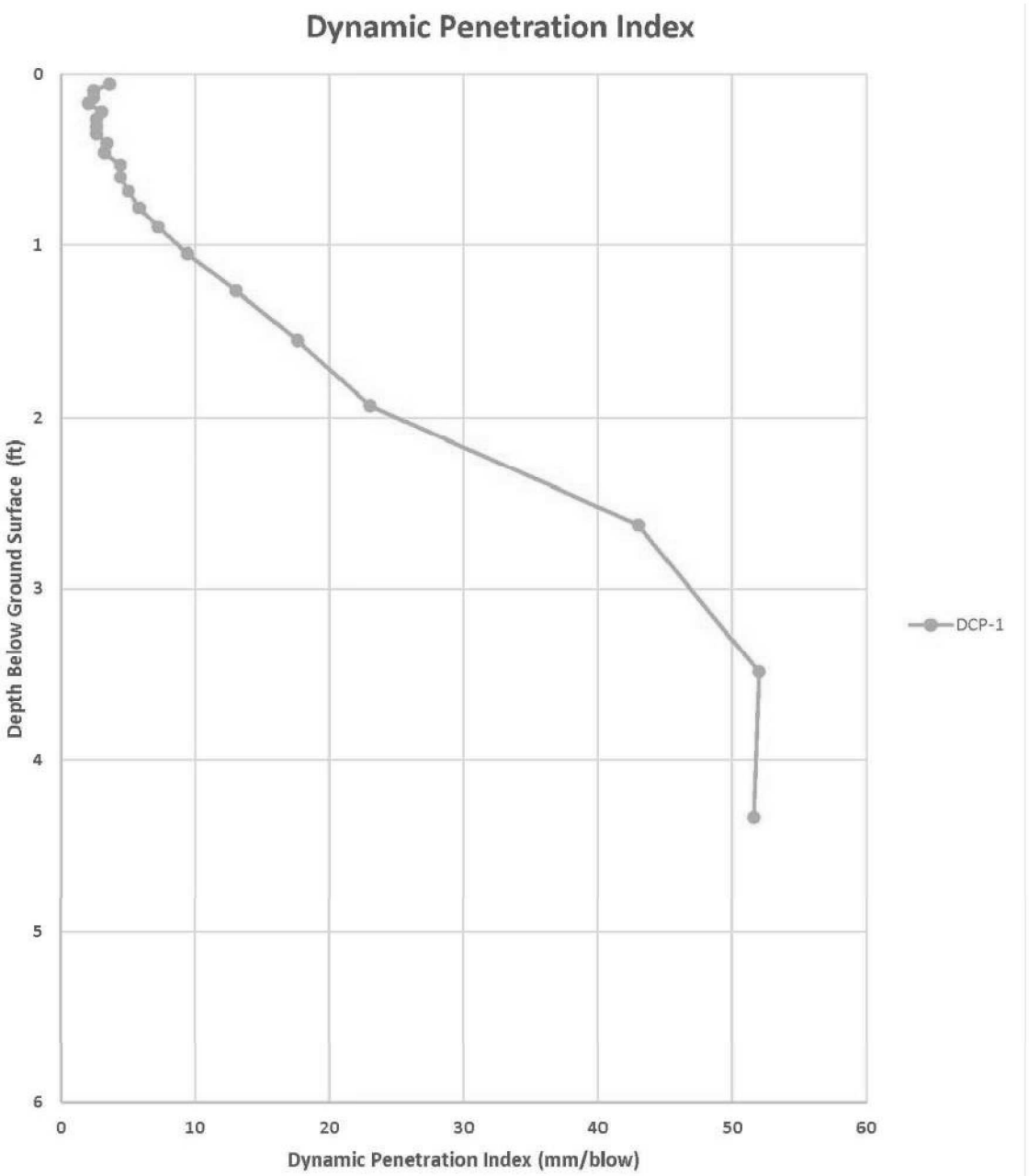
Stantec

DRILLING: STARTED **7/13/17** COMPLETED: **7/13/17**	NORTHING (ft): **1528870.876** EASTING (ft): **6593187.891**
INSTALLATION: STARTED COMPLETED:	LATITUDE: **36° 11' 37.0522"** LONGITUDE: **-120° 23' 35.5611"**
DRILLING COMPANY: **Cascade Drilling**	GROUND ELEV (ft): **61** TOC ELEV (ft):
DRILLING EQUIPMENT: **CME 75**	INITIAL DTW (ft): **NE** BOREHOLE DEPTH (ft): **5**
DRILLING METHOD: **Hollow Stem Auger**	STATIC DTW (ft): **NE** WELL DEPTH (ft): **---**
SAMPLING EQUIPMENT: **Split Spoon**	WELL CASING DIAMETER (in): **---** BOREHOLE DIAMETER (in): **6**
	LOGGED BY: **M. Sapp** CHECKED BY: **M. Amendolagine**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Measured Recov. (feet)	Blow Count (per foot)	Headspace PID (units)	Depth (feet)	Borehole Backfill
		GM	**FILL:** **SILTY GRAVEL WITH SAND** ; GM; 50% fine gravel; 35% fine to coarse grained sand; 15% non-plastic fines; dry							
		SM	**ALLUVIUM:** **SILTY SAND WITH GRAVEL** ; SM; 10YR 3/4 dark yellowish brown; 15% fine to coarse gravel; 55% fine to coarse gravel; 55% fine to coarse grained sand; 30% non-plastic fines; moist; loose to medium dense 70% fine to coarse grained sand; 15% non-plastic fines below 4'.							
5			Borehole terminated at 5 feet bgs.						5	Backfilled with hydrated bentonite once percolation well was removed. Backfilled with gravel for percolation testing.
10									10	
15									15	
20									20	

PROJECT: **Blue Wave - San Jose** LOCATION: **949 Oakland Road, San Jose, California** PROJECT NUMBER: **222310633**	WELL / PROBEHOLE / BOREHOLE NO: **B4/DCP-1** PAGE 1 OF 1
DRILLING: STARTED **7/13/17** COMPLETED: **7/13/17** INSTALLATION: STARTED COMPLETED: DRILLING COMPANY: **Cascade Drilling** DRILLING EQUIPMENT: **CME 75** DRILLING METHOD: **Hollow Stem Auger** SAMPLING EQUIPMENT: **Split Spoon**	NORTHING (ft): **1528786.337** EASTING (ft): **6593212.681** LATITUDE: **36° 11' 36.2161"** LONGITUDE: **-120° 23' 35.2599"** GROUND ELEV (ft): **61** TOC ELEV (ft): INITIAL DTW (ft): **NE** BOREHOLE DEPTH (ft): **5** STATIC DTW (ft): **NE** WELL DEPTH (ft): **---** WELL CASING DIAMETER (in): **---** BOREHOLE DIAMETER (in): **6** LOGGED BY: **M. Sapp** CHECKED BY: **M. Amendolagine**

Time & Depth (feet)	Graphic Log	USCS	Description	Sample	Time Sample ID	Measured Recov. (feet)	Blow Count (per foot)	Headspace PID (units)	Depth (feet)	Borehole Backfill
		GM	**FILL:** **SILTY GRAVEL WITH SAND** ; GM; 50% fine gravel; 35% fine to coarse grained sand; 15% non-plastic fines; dry							Backfilled with soil cuttings.
		GM	**ALLUVIUM:** **SILTY GRAVEL WITH SAND** ; GM; 10Y 4/2 dark grayish olive; 60% fine to coarse gravel; 25% fine to coarse grained sand; 15% non-plastic fines; moist; loose to medium dense		B4-1/5 Bulk					Backfilled with hydrated bentonite.
5			Refusal at 5'. Stepped out twice. Borehole terminated at 5 feet bgs.						5	
10									10	
15									15	
20									20	



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ORIGINAL SHEET - ANSI A

July, 2017
222310633

Stantec

3875 Atherton Road
Rocklin, California
www.stantec.com

Client/Project

Bluewave Car Wash
929 Oakland Road
San Jose, California 95661

Title

DYNAMIC CONE PENETRATION TEST RESULTS

APPENDIX B
LABORATORY TEST RESULTS

Sierra Geotech DVBE Inc.
4470 Yankee Hill Rd, Ste 110
(916)990-1884
info@sierrageotech.com
www.sierrageotech.com
A disabled veteran business enterprise and small business (micro)

Project: Blue Wave - San Jose

Client: Stantec Consulting

Project No: RN2017101

Performed by: MB Reviewed by: RN/SJ

Amount of Material Finer than 75-µm (No.200) Sieve- ASTM D1140

Sample Number	B1-10	B1-35	B2-10	B2-15		
Depth, ft	10	35	10.0	15		
Tare ID	202	205	201	204		
Dry Sample Weight+ Tare, Pre- Wash, g	680.3	755.1	875.2	849.2		
Tare Weight, g	177.3	177.3	177.3	177.3		
Dry Sample Weight+ Tare, Post-Wash, g	619	193	545	247		
Percent Finer than No.200 Sieve	12%	97%	47%	90%		
Soil Description	Brown Sand with Silt	Dark brown Sandy Clay	Brown Sand with Silt	Brown Sandy Silt with Clay		

Notes:	Stantec Project Number - 222310633
Sample No./Depth:	Various
Date Tested	7/20/2017 - 7/21/2017

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A disabled veteran business enterprise and small business (micro)

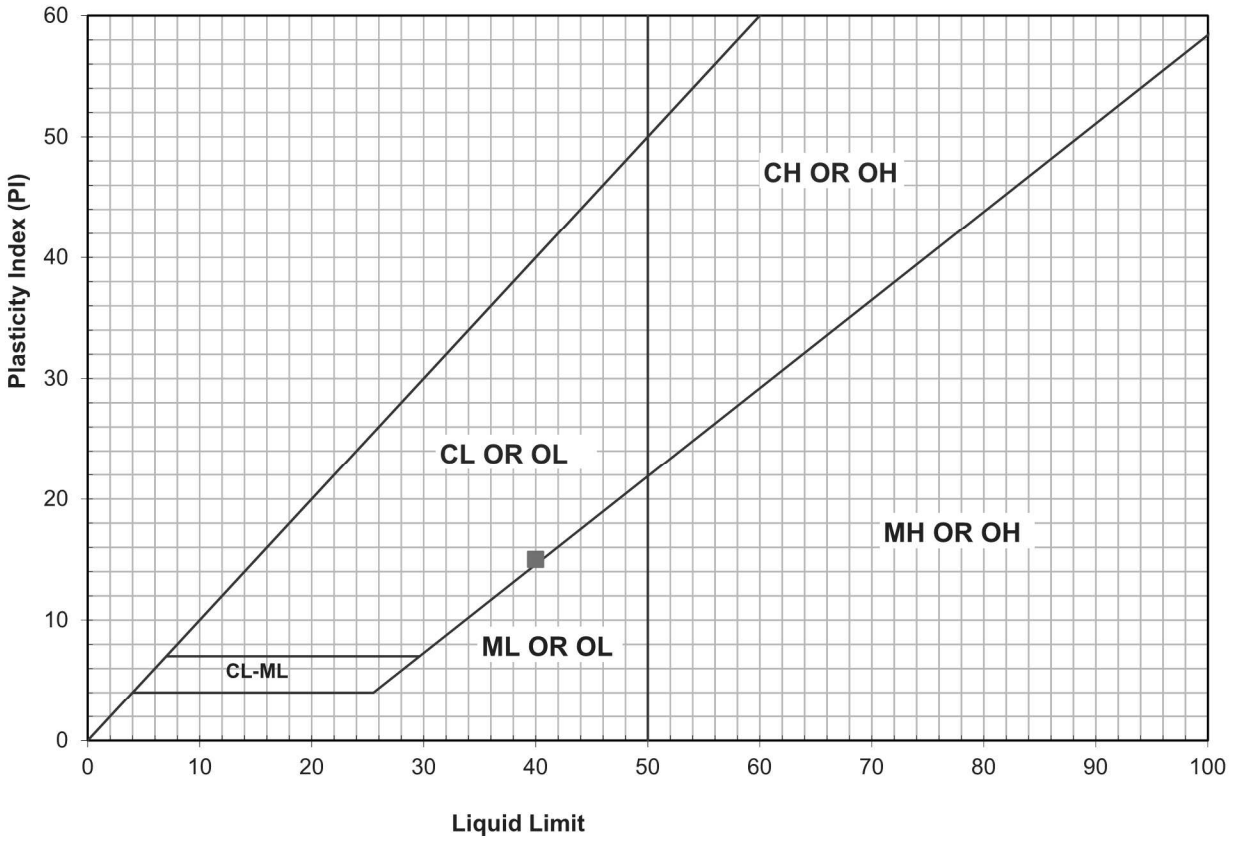
Project: Blue Wave - San Jose

Client: Stantec Consulting

Project No: RN2017101

Performed by: MB Reviewed by: RN/SJ Date: 7/20/2017

Liquid Limit, Plastic Limit, and Plasticity Index of Soils Using ASTM D4318, Method A



Liquid Limit	Plastic Limit	Plasticity Index	Unified Soil Classification, ASTM D2487
40	**25**	**15**	CL

Material Description:	Dark brown Sandy CLAY
Notes:	Stantec Project Number - 222310633
Sample No./Depth:	B1-11
Date Tested	7/20/2017 - 7/21/2017

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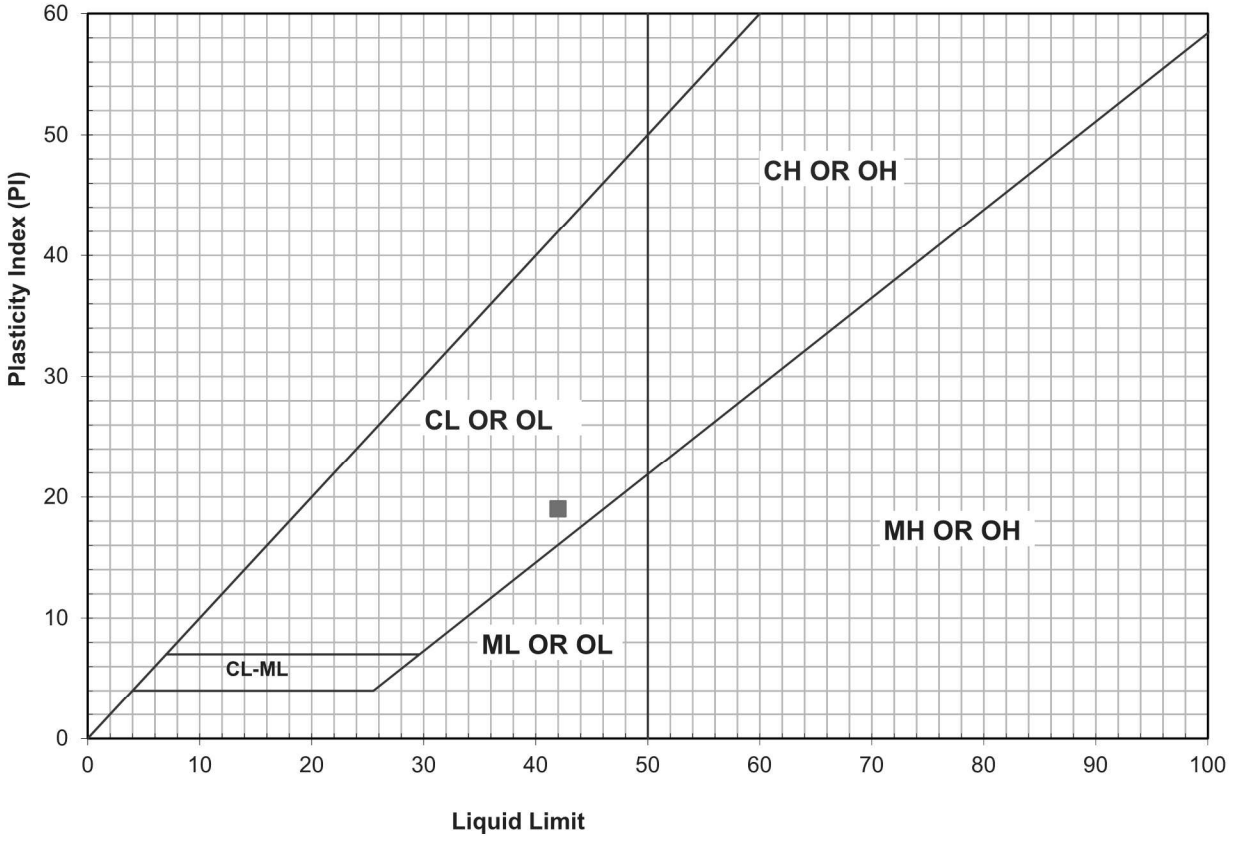
Project:Blue Wave - San Jose

Client: Stantec Consulting

Project No: RN2017101

Performed by: MB Reviewed by: RN/SJ Date: 7/20/2017

Liquid Limit, Plastic Limit, and Plasticity Index of Soils Using ASTM D4318, Method A



Liquid Limit	Plastic Limit	Plasticity Index	Unified Soil Classification, ASTM D2487
42	23	19	CL

Material Description:	Dark brown Sandy CLAY
Notes:	Stantec Project Number - 222310633
Sample No./Depth:	B1-30
Date Tested	7/20/2017 - 7/21/2017

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Project: Blue Wave - San Jose

Client: Stantec Consulting

Project No: RN2017101

Performed by: MB Reviewed by: RN/SJ Date: 7/20/2017

Moisture Content and Dry Density of Soil, ASTM D2216, Method B and D7263, Method B

Sample	B1-7	B1-15	B1-42.5	B2-20		
Depth (feet)	7	15	42.5	20		
Diameter of sample (in)	2.41	2.39				
Height of sample (in)	5.75	5.25				
Tare ID	101	102	103	104		
Empty Tare Weight	86	86	51.69	51.19		
Wet weight with Tare	870	818	154.31	157.86		
Dry weight with Tare	737	662	126.24	134.41		
Weight of Water	133	155	28	23		
Weight of Dry Soil	651	577	75	83		
Moisture Content, %	20.4%	26.9%	37.7%	28.2%		
Dry Density, pcf	94.6	93.2				
Visual Soil Description	Yellow brown Clayey Sand	Dark brown Sandy Clay	Dark grey Sandy Clay	Dark brown Sandy Clay		

(all weights in grams)

Notes:	Stantec Project Number - 222310633
Sample No./Depth:	Various
Date Tested	7/20/2017 - 7/21/2017

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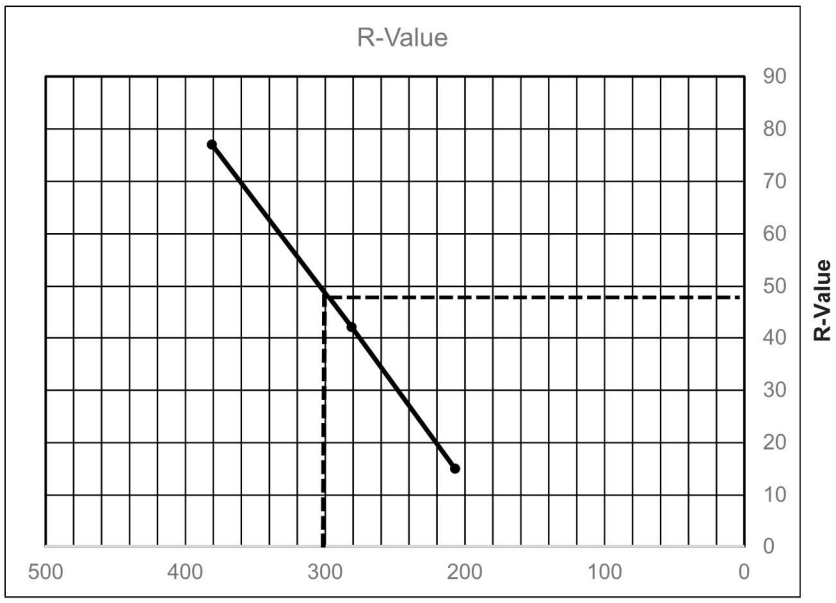
Project: Blue Wave - San Jose

Client: Stantec Consulting

Project No: RN2017101

Performed by: JC Reviewed by: RN/SJ Date: 7/20/2017



Sample #	MC %	Dry Density in pcf	Pressure (Expansion) in psf	Pressure (Exudation) in psi	R-Value
1	11.5	136.3	0	207	15
2	10.4	134.5	43	281	42
3	9.7	132.5	87	381	77

R Value at 300 psi Exudation Pressure is 48

Material Description:	Dark brown Silty SAND
Notes:	Stantec Project Number - 222310633
Sample No./Depth:	B4-1/5-Bulk
Date Tested	7/20/2017 - 7/21/2017

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A disabled veteran business enterprise and small business (micro)

Project:Blue Wave - San Jose

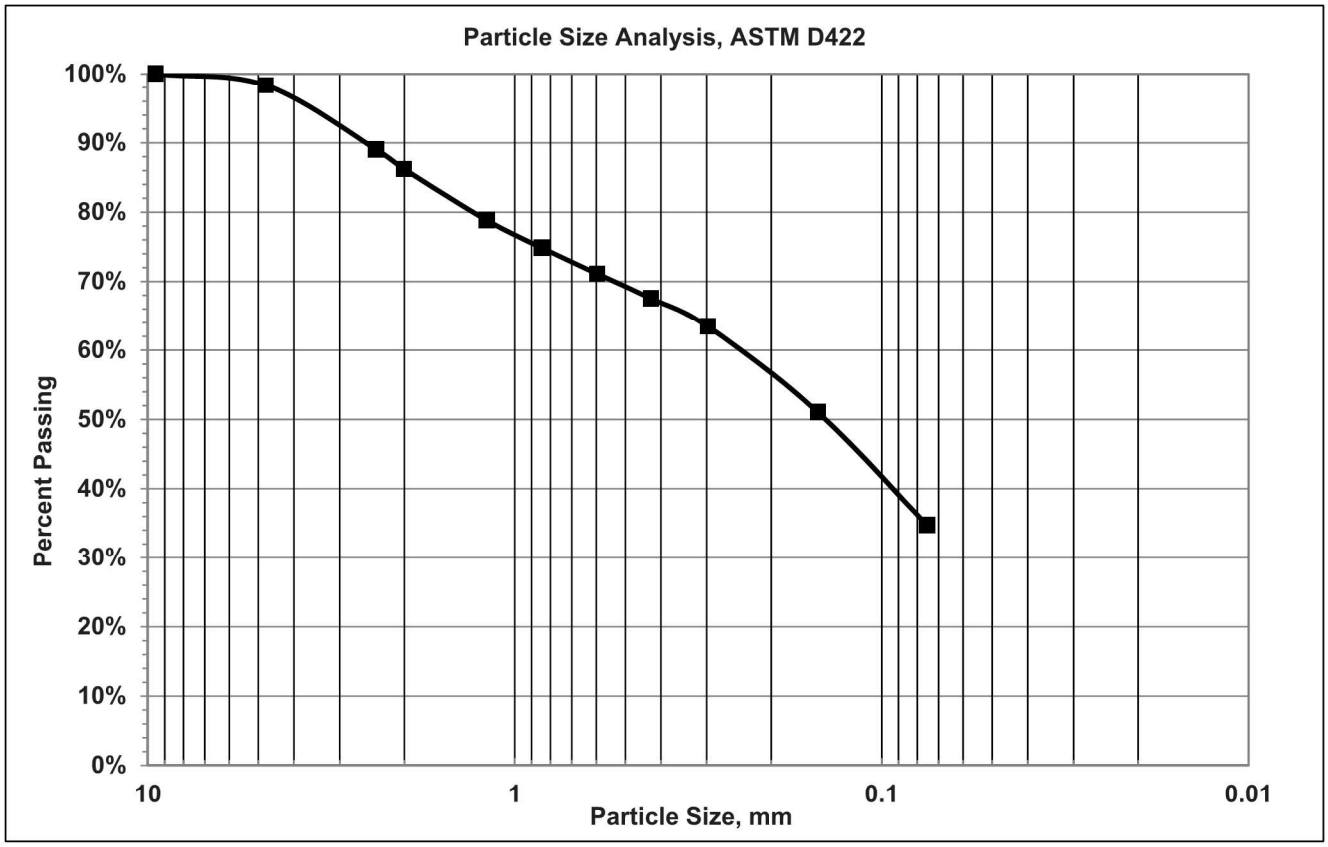
Client: Stantec Consulting

Project No: RN2017101

Performed by: MB Reviewed by: RN/SJ Date: 7/21/2017

US Standard Sieve Size	% Mass Retained	% Finer
3/8" (9.5 mm)	0%	100%
No.4 (4.75 mm)	2%	98%
No.8 (2.4 mm)	9%	89%
No. 10 (2 mm)	3%	86%
No. 16 (1.2 mm)	7%	79%
No. 20 (850 μm)	4%	75%

US Standard Sieve Size	% Mass Retained	% Finer
No.30(600 μm)	4%	71%
No. 40 (425 μm)	4%	68%
No. 50 (300 μm)	4%	63%
No. 100 (150 μm)	12%	51%
No. 200 (75 μm)	16%	34.7%
PAN	34.7%	



Material Description:	Dark brown Silty SAND			
Notes:	Stantec Project Number - 222310633			
Sample No./Depth:	B1-5	USCS Classification	Liquid Limit	Plasticity Index
Date Tested	7/20/2017 - 7/21/2017	SM	-	-

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Project: Blue Wave - San Jose

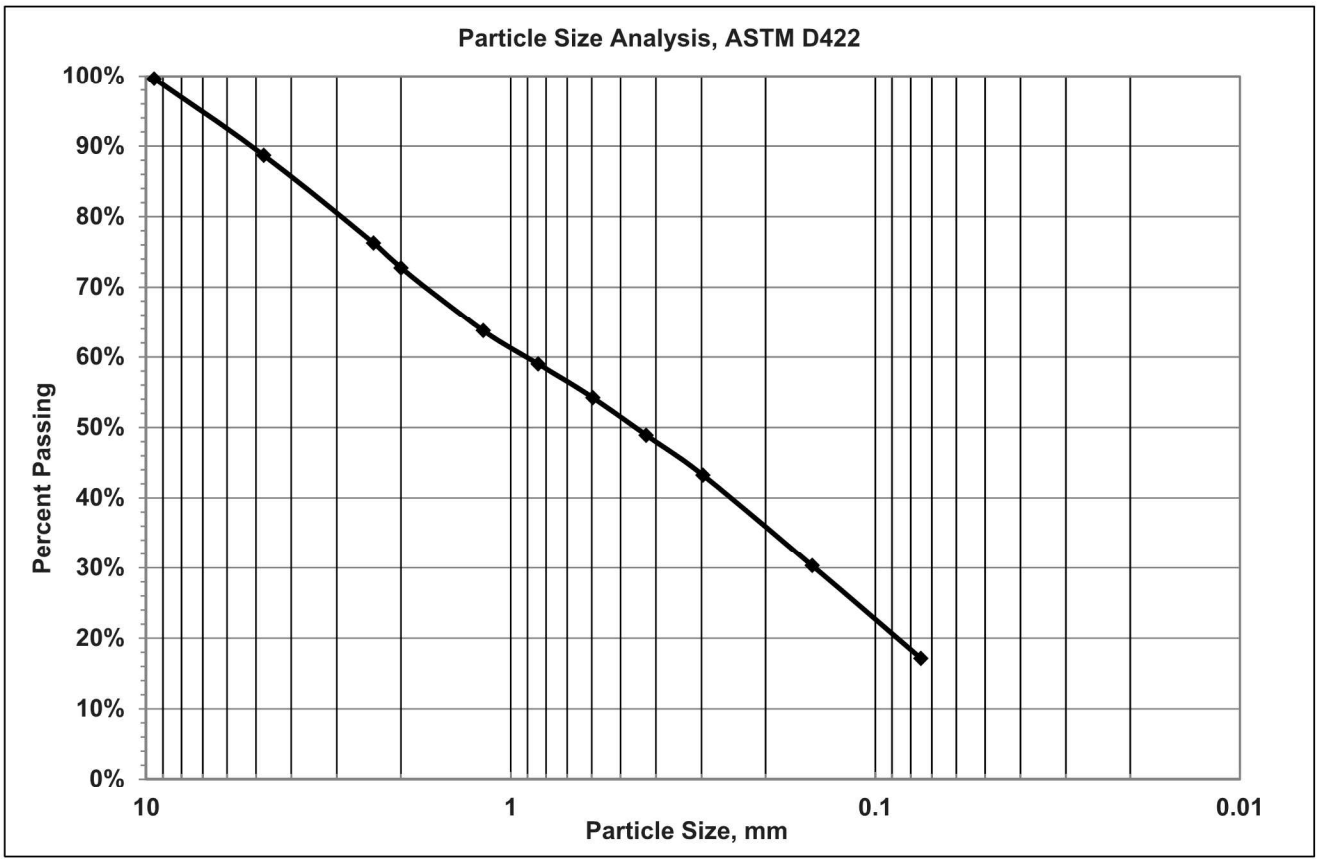
Client: Stantec Consulting

Project No: RN2017101

Performed by: MB Reviewed by: RN/SJ Date: 7/21/2017

US Standard Sieve Size	% Mass Retained	% Finer
3/8" (9.5 mm)	0.4%	99.6%
No.4 (4.75 mm)	11%	89%
No.8 (2.4 mm)	12%	76%
No. 10 (2 mm)	4%	73%
No. 16 (1.2 mm)	9%	64%
No. 20 (850 μm)	5%	59%

US Standard Sieve Size	% Mass Retained	% Finer
No.30(600 μm)	5%	54%
No. 40 (425 μm)	5%	49%
No. 50 (300 μm)	6%	43%
No. 100 (150 μm)	13%	30%
No. 200 (75 μm)	13%	17.1%
PAN	17.1%	



Material Description:	Yellow brown Clayey SAND			
Notes:	Stantec Project Number - 222310633			
Sample No./Depth:	B1-7	USCS Classification	Liquid Limit	Plasticity Index
Date Tested	7/20/2017 - 7/21/2017	SC	-	-

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A disabled veteran business enterprise and small business (micro)

Project: Blue Wave - San Jose

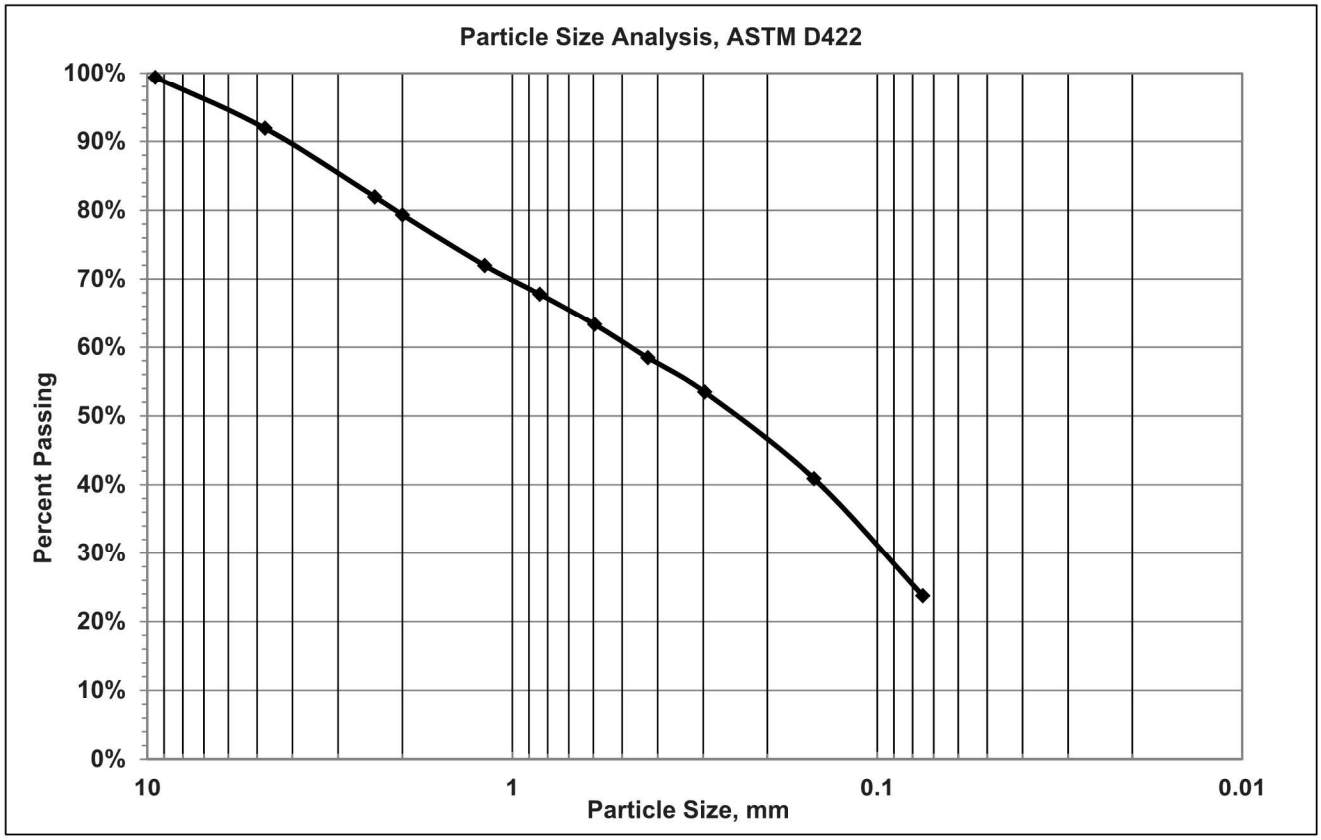
Client: Stantec Consulting

Project No: RN2017101

Performed by: MB Reviewed by: RN/SJ Date: 7/21/2017

US Standard Sieve Size	% Mass Retained	% Finer
3/8" (9.5 mm)	1%	99%
No.4 (4.75 mm)	7%	92%
No.8 (2.4 mm)	10%	82%
No. 10 (2 mm)	3%	79%
No. 16 (1.2 mm)	7%	72%
No. 20 (850 μm)	4%	68%

US Standard Sieve Size	% Mass Retained	% Finer
No.30(600 μm)	4%	63%
No. 40 (425 μm)	5%	58%
No. 50 (300 μm)	5%	53%
No. 100 (150 μm)	13%	41%
No. 200 (75 μm)	17%	23.7%
PAN	23.7%	



Material Description:	Dark brown Silty SAND			
Notes:	Stantec Project Number - 222310633			
Sample No./Depth:	B1-2/5-Bulk	USCS Classification	Liquid Limit	Plasticity Index
Date Tested	7/20/2017 - 7/21/2017	SM	-	-

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A disabled veteran business enterprise and small business (micro)

Project: Blue Wave - San Jose

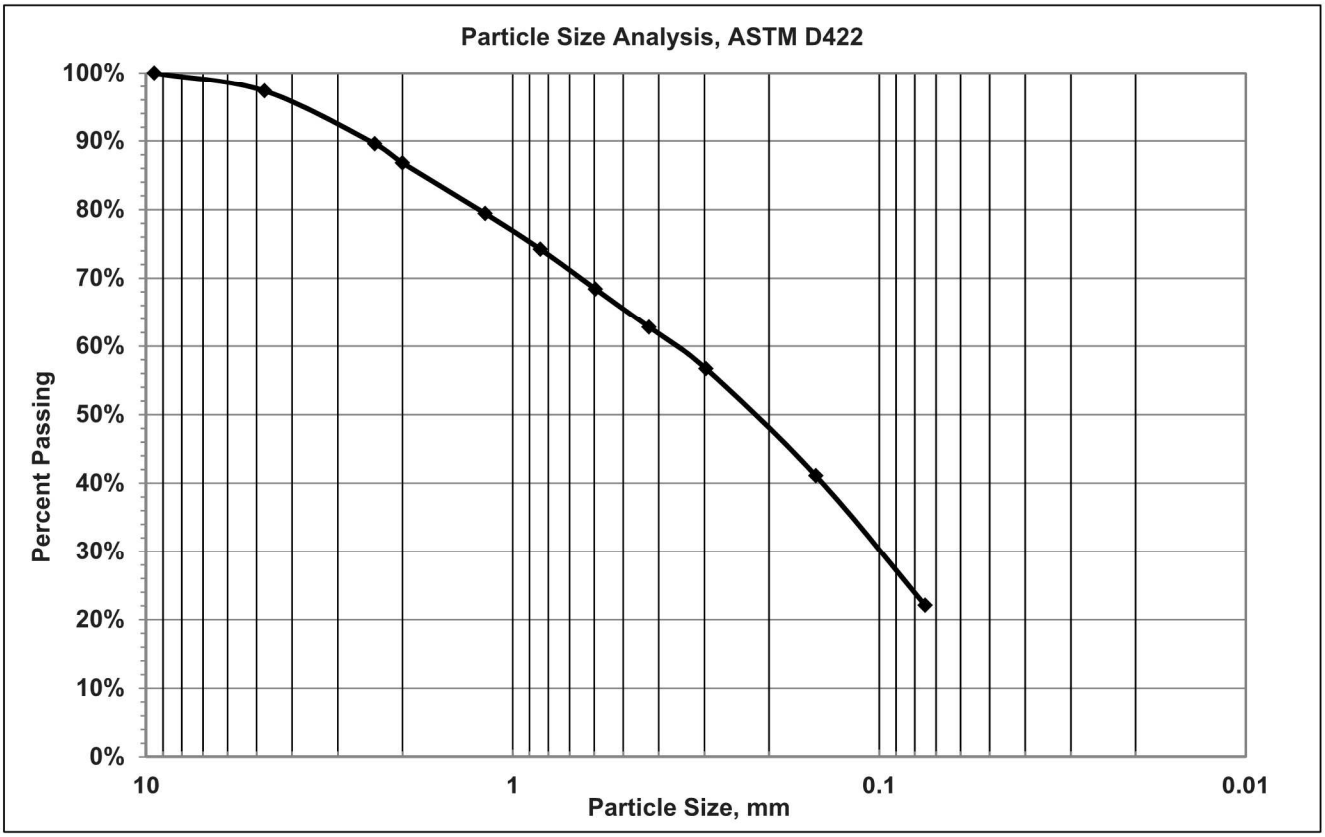
Client: Stantec Consulting

Project No: RN2017101

Performed by: MB Reviewed by: RN/SJ Date: 7/21/2017

US Standard Sieve Size	% Mass Retained	% Finer
3/8" (9.5 mm)	0%	100%
No.4 (4.75 mm)	3%	97%
No.8 (2.4 mm)	8%	90%
No. 10 (2 mm)	3%	87%
No. 16 (1.2 mm)	7%	79%
No. 20 (850 μm)	5%	74%

US Standard Sieve Size	% Mass Retained	% Finer
No.30(600 μm)	6%	68%
No. 40 (425 μm)	6%	63%
No. 50 (300 μm)	6%	57%
No. 100 (150 μm)	16%	41%
No. 200 (75 μm)	19%	22.1%
PAN	22.1%	



Material Description:	Dark brown Silty SAND			
Notes:	Stantec Project Number - 222310633			
Sample No./Depth:	B2-5	USCS Classification	Liquid Limit	Plasticity Index
Date Tested	7/20/2017 - 7/21/2017	SM	-	-

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 07/26/2017
Date Submitted 07/20/2017

To: Seth Jayne
Sierra Geotech DVBE, Inc.
4470 Yankee Hill Rd, Ste 110
Rocklin, CA 95765

From: Gene Oliphant, Ph.D. \ Randy Horney RA
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : SAN JOSE OAKLAND RD. Site ID : B1-2/5-BULK.
Thank you for your business.

* For future reference to this analysis please use SUN # 74830-156220.

EVALUATION FOR SOIL CORROSION

Soil pH	7.34	
Minimum Resistivity	0.40 ohm-cm (x1000)	
Chloride	246.6 ppm	00.0.2466 %
Sulfate	279.6 ppm	00.02796 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

APPENDIX C
PERCOLATION TEST RESULTS

PERCOLATION TEST DATA SHEET							
Project:	Blue Wave - SJ		Project No.	222310633		Date:	7/14/2017
Test Hole No.	B3		Tested By:	M. Sapp			
Depth of Test Hole, D_T:	5' 0"		USCS Soil Classification	SM			
Test Hole Dimensions (inches)				Length	Width		
Diameter (if round)	12"	Sides (if rectangular)					
Presoak 7/13/17*							
Trial No. (7/13/17)	Start Time	Stop Time	Time Interval, (min)	Initial Depth of Water (in)	Final Depth of Water (in)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	10:00am	11:00am	60.0	48.0	NA	NA	NA
2	11:00am	12:00pm	60.0	48.0	NA	NA	NA
3	12:00pm	1:00pm	60.0	48.0	NA	NA	NA
4	1:00pm	2:00pm	60.0	48.0	NA	NA	NA
* Pre-soaking is to consist of filling each percolation hole to at least 12 inches above the gravel bottom, for a 4-hr period, refilling approximately once per hour. Alternatively, presoaking can be divided into 2-hr morning, 2-hr afternoon period or other schedule to achieve 4-hr total presoak period. Presoak passed; no water in boring on 7/14/17.							
Trial No. (7/14/17)	Start Time	Stop Time	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	9:15am	9:45am	30	56.0	56.5	0.5	1.0
2	9:50am	10:50am	30	56.0	56.625	0.625	1.25
3	10:25am	10:55am	30	56.0	56.625	0.625	1.25
4	11:00am	11:30am	30	56.0	56.625	0.625	1.25
5							
6							
7							
8							
9							
10							
Comments: Un-factored infiltration rate = 1.25 inches/hour or 48 minutes/inch							